Influence of soil-structure interaction on the seismic response of highway bridges

Mélissa Barrière[1], François Dallaire[2], Nathalie Roy[3], Jean Proulx[3]

[1] Jr. Eng., Les services EXP Inc, Drummondville, QC, Canada.
[2] P. Eng., M.Sc.A.,WSP Canada Inc, Montréal, QC, Canada.
[3] Professor, Department of Civil and Building Engineering, University of Sherbrooke - Sherbrooke, QC, Canada.

ABSTRACT

Damage to a bridge after a large earthquake can result in high repair costs and problems with traffic disruption. To reduce the risk of collapse and ensure the safety of residents, seismic safety assessments are carried out by municipalities and transport authorities. During these analyses, bridge structures are often modeled considering rigid foundations, thus neglecting the soil-structure interaction, which can have a significant impact on the evaluation of the base shear forces and superstructure displacements. This paper presents a parametric study on the influence of soil-structure interaction, using 3D models calibrated with ambient and forced-vibration tests, with different approaches for the energy dissipation and stiffness of the foundation soil. Superstructure displacements and base shears obtained with the rigid foundation model are compared with those obtained using linear soil-structure interaction models: (i) the simplified method proposed in the latest edition of the Canadian Highway Bridge Design Code (CSA S6-14); and (ii) the method proposed in the US National Earthquake Hazards Reduction Program (NEHRP-2012). The latter represents the foundation-structure interaction using a parallel damper and spring system, while the CSA S6-14 uses only a spring, without added damping. The earthquake responses are calculated by time-history analyses for a case study bridge, using artificial accelerograms for different soil types. The influence of soil-structure interaction on displacements and shear forces is quantified, as well as the performance of the different foundation models with respect to the rigid foundation. The decision to include soil-structure interaction into structural models, with various levels of complexity, can then be evaluated in the context of seismic safety evaluation of highway bridges.

Keywords: Soil-structure interaction; Rigid foundation; 3D numerical models; Artificial accelerogram; Rigid foundation.

INTRODUCTION

Bridges must be designed according to the codes in effect for new construction or rehabilitation work. Models used during the design process, are often based on rigid foundations, neglecting soil-structure interaction (foundations, abutments). The latest edition of the Canadian Highway Bridge Design Standard (CSA S6-14) details a simplified model to represent the linear behaviour of foundations using a spring system without the addition of damping. The absence of damping in this foundation model neglects the dissipation of energy in the ground (i.e. radial and material damping). The main objective of the work presented in this paper is to evaluate the influence of soil-structure interaction on the seismic responses (i.e. deck displacements, shear at the base of the piers), using different soil-structure models. The secondary objectives are to compare the performance of foundation models and to assess the relevance of using foundation models with different levels of complexity.

CASE STUDY

Different highway bridges were tested and modelled in this project. This paper will focus on a typical bridge as case study. Built in 2009, the *Chemin Roy* Bridge (Figure 1) crosses Highway 10 near Magog in the province of Quebec, Canada. It is 59.4 m long and 11.4 m wide. It consists of two spans and has a six-degree bias. The superstructure consists of a 0.2 m thick concrete deck resting on four 1.2 m high steel beams. The bridge bent of the central support consists of a 1.2 m high cap-beam, as well as three RC columns with a width of 0.76 m, a depth of 1.66 m and a height of 4.43 m. This bridge bent is based on a rectangular surface foundation. The foundation soil is type "C" according to CSA S6-14.

EXPERIMENTAL INVESTIGATION

Previous work [1][2] included both experimental and numerical investigations, where dynamic in-situ tests under ambient and forced vibration were carried out on three bridges to identify the vibration modes and their associated elastic viscous damping rates, in order to calibrate 3D finite element models. Results of this process are presented here for the *Chemin Roy* Bridge.

Ambient and forced-vibration dynamic tests

During ambient vibration tests, vehicles travelling on the bridge randomly excite the structure. The data was recorded using Kinemetrics FBA ES-T triaxial accelerometers. Six roving sensor configurations were used to properly capture the mode shapes of the structure. Experimental frequencies, damping and mode shapes were obtained with the Enhanced Frequency Domain Decomposition method. Forced vibration tests were subsequently carried out using a linear mass shaker that was attached to the bridge deck and provided excitation in the vertical and transverse (horizontal) directions. The same accelerometers were used, but unlike ambient vibration tests, traffic on the structure was stopped for data acquisition in order to isolate the responses due to the sinusoidal force induced by the shaker. Frequency sweeps from 0 to 20 Hz were carried out for each setup. Frequency response curves for acceleration (normalized by the calculated input force) were obtained for each recording station, for amplitude and phase with respect to the driving force. These curves were then used to extract frequencies and mode shapes. The responses also lead to a better evaluation of equivalent viscous damping ratios, using the half-power bandwidth method (complete details of the experimental setup and data processing can be found in ref [2]).

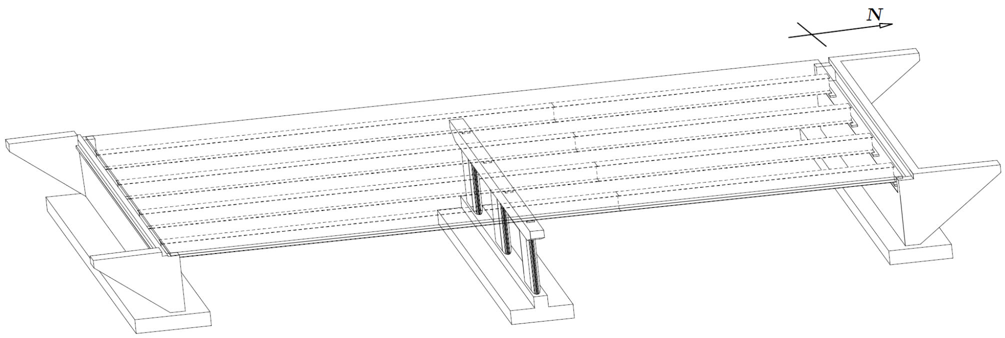


Figure 1. 3D view of the Chemin Roy *Bridge model (Magog, Quebec).*

Calibration of 3D model on rigid foundations

The information collected during the dynamic tests allowed the calibration of 3D numerical models for the bridges. Results from both types of tests were used. Ambient vibration tests are easier and quicker to complete, and more recording stations can be used (different positions on the bridge and different recording axes). When available, forced vibration results provide better definitions of the response curves and hence frequencies, mode shapes and damping. The experimental results and the numerical calculations were compared for the various modes, based on the frequency deviation and the modal assurance criterion (MAC) [3], which is a measure of the degree of coherence between the experimental and numerical mode shape vectors. Assuming a rigid foundation (without soil-structure interaction), the model was calibrated by adjusting the rigidity of the bearings, beams and diaphragms [2].

Table 1 presents the experimental frequencies and damping ratios obtained from both series of tests as well as the calculated frequencies. MAC values are also presented, and values above 90 % were observed for all vibration modes, which confirms the accuracy of the numerical model to represent mode shapes. Modal damping values obtained during the tests were mostly close to 1%, i.e. below the 2 % value proposed by CSA Standard S6-14 [4] for that bridge type. This proposed rate of 2 % is much closer to what is observed in situ than the 5 % value that has long been used. These experimental values are obtained, however, for a structure that is behaving linearly and a higher damping value could account for non-linear behaviour under earthquake loading.

Table 1. Experimental frequencies and damping, and numerical frequencies for Chemin Roy Bridge.

Vibration mode	Ambient Vibrations Test		Forced Vibrations Test		MAC
	Frequencies (Hz)	Damping (%)	Frequencies (Hz)	Damping (%)	
Bending – 1	3.17	1.01	3.19	0.80	0.97
Torsion – 1	3.55	0.89	3.56	0.79	0.99
Horiz./Transv. -1	N/A	N/A	4.25	1.80	N/A
Bending – 2	4.60	1.22	4.59	0.88	0.99
Torsion – 2	4.86	0.96	4.89	0.61	0.98
Bending – 3	7.73	1.18	7.91	1.14	0.97
Horiz./Long. – 2	N/A	N/A	7.91	1.24	N/A
Bending – 4	8.33	1.02	8.47	1.09	0.97
Horiz./Transv. – 3	N/A	N/A	8.72	2.05	N/A

NUMERICAL MODELLING

The finite element numerical studies were carried out using the OpenSees (Open System for Earthquake Engineering Simulation) [5] open source software developed and maintained by the Pacific Earthquake Engineering Research Center (PEER). The following subsections present the modelling approach used to represent the behaviour of the superstructure (i.e. deck, intermediate support, and elastomeric bearings), foundations and abutments. The parametric study was carried out with the model resulting from the experimental calibration.

Superstructure

The superstructure was modelled using a grid of nodes that were connected by linear elements of the *ElasticBeamColumn* type. This type of element allows a more realistic distribution of the rigidity of the beams and diaphragms that are combined with the slab. The nodes are positioned vertically at the centroid of the composite elements (i.e. main beam, slab, bitumen coat), which implies that the masses associated with them include these elements.

Bridge bent

The bridge bent includes the cap-beam, columns and shallow foundation. Figure 2 shows the elements of the intermediate support, as well as a typical cross-section of a column with the fibre sections used in the numerical model.

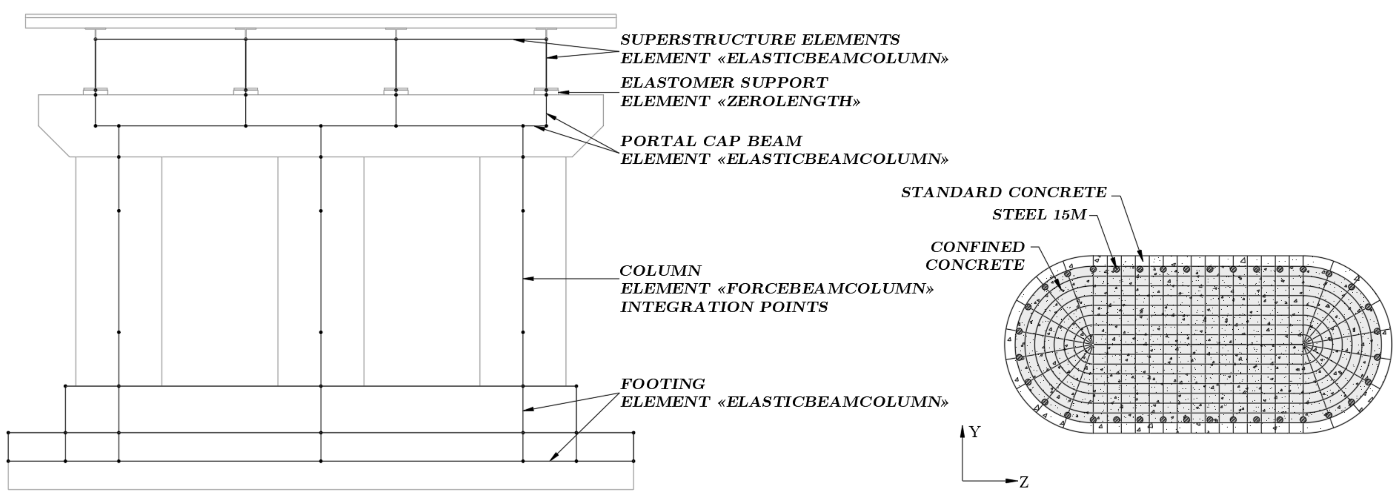


Figure 2. Discretization of the elements of the bridge bent and transversal section of a column.

Cap-beam

The cap-beam was modelled by linear elements of the *ElasticBeamColumn* type. The nodes, which discretize the elements of the beam, were located in the centres of three columns and four main beams. To provide some rigidity to the model, rigid extensions were added to the nodes positioned at the centroid of the columns to represent their half lengths, as well as to the nodes of the beams and outer columns. Knowing that the columns are dimensioned so that the plastic hinge will develop at their bases and that, consequently, no damage is expected in the cap-beam, an elastic behaviour was assumed for this part of the bridge bent.

Columns

In the model used for this research, energy dissipation takes place mainly in the columns where a plastic hinge is formed at the base of the columns. In order to model this behaviour, the columns were modelled using non-linear *ForceBeamColumn* elements. The beam-column elements consider only axial and rotational flexibility, which implies that it is necessary to manually include shear stiffness and torsional stiffness.

The exact distribution of the curvature was approximated by using integration points located at 8/3 of the length of the plastic hinge region and at the ends of the element, which were inserted using the modified Gauss-Radau method [1]. The length of the plastic hinge region was defined by: $L_p = 0.08L + 0.022 f_y d_{bd} \geq 0.044 f_y d_{bd}$, where L is the free height of the column, f_y is the yield strength of the longitudinal steel bars and d_{bd} is the diameter of these bars. In addition, these elements were discretized into fiber sections created using the *Fibers* function to include concrete "patch" surfaces and "layers" points representing steel bars. By dividing the cross-section into small concrete and steel elements, as well as by assigning different materials to each fibre, an uneven distribution of stresses across the cross-section was considered. This configuration simulates concrete cracking, reinforcement steel yielding, spalling and concrete crushing [6].

The *Concrete06* material was used in the model, since its tensile and compressive behaviour is user-defined. This approach uses linear behaviour during the unloading process in compression and tension. On the compression side, the unloading stiffness is 7.1% of its initial stiffness E_c as proposed by Palermo and Vecchio [7] The *Steel04* material was used, which includes the Baushinger effect, isotropic hardening and a memory of previously applied stresses.

Elastomeric bearings

In the case of the *Chemin Roy* Bridge, the central support is made of confined elastomers while the abutment supports are multi-layered. The multi-layered elastomer bearings of abutment 1 and the unidirectional confined elastomers of the central bearing are fixed in both directions (i.e. longitudinal direction, transverse direction), which implies that non-linear behaviour can be observed. For multi-layered elastomers in abutment 3, movements are constrained in the transverse direction and are allowed in the longitudinal direction. This particular pattern implies that non-linear behaviour is observed in the transverse direction while linear behaviour is observed in the longitudinal direction.

These bearings were modelled using *ZeroLength* elements to connect two nodes located at the same location. In the numerical model, these elements are considered not to deform under gravity loads (i.e. vertical direction). Indeed, the frets (i.e. steel plates) segmenting the elastomer offer a better resistance to vertical load and limit its lateral expansion when the latter is compressed vertically. In horizontal directions, the *Steel01* material were used to simulate the hysteretic damping of this material. In accordance with previous work in this research project [1][2], assumptions that the secondary slope begins at 4 % of the service axial load and the shear modulus is 1 GPa were retained. Linear behaviour was assumed for the three degrees of freedom in rotation.

SOIL-STRUCTURE INTERACTION

As shown in Figure 1, the bridge is built on shallow foundations (i.e. footing, no piles), resting on a type "C" soil (very dense soil or dense rock) according to CSA S6-14 standard [4]. For comparison purposes, a "D" type soil (solid soil) was also used for the analyses. Short and medium span bridges are often modelled by considering a rigid foundation, thereby neglecting soil-structure interaction, and the structure is then fixed at the nodes under the columns and under the elastomeric bearings of the abutments. In addition to this rigid foundation model, two soil-structure modelling approaches were investigated in this study: (i) the simplified method proposed in the latest edition of the Canadian Highway Bridge Design Code (CSA S6-14) [4], which includes the foundation stiffness but no damping; and (ii) the method proposed in the US National Earthquake Hazards Reduction Program (NEHRP-2012) [8], where both stiffness and damping are considered.

Canadian Highway Bridge Design Code (CSA S6-14) Methodology

The CHBDC approach for soil-structure interaction uses a linear model for the foundation, which consists of a multiple degrees of freedom spring system without accounting for damping. The footing is modelled as a rigid element and the ground under this element deforms under seismic loading. Figure 3 shows the spring model distribution used for the central bridge bent and abutment footings (dampers are not present in this model but are shown in the figure for the NEHRP model discussed below).^

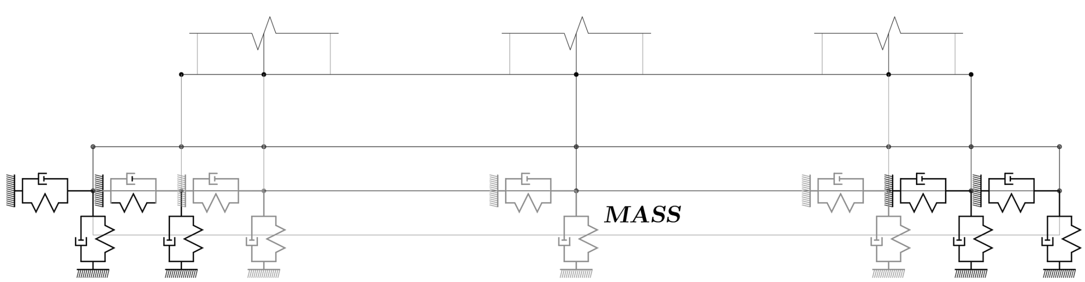


Figure 3. Foundation model for the bridge bent fo Chemin Roy *Bridge (dampers are present only in the NEHRP model).*

Depending on the depth of the footing beneath ground level, the standard proposes factors for correcting the foundation stiffness. These corrected stiffnesses $K_{corr} = \beta K_{on}$ are introduced to the model using *ZeroLength* elements. One of the two nodes near this element is fixed while the other is attached to the foundation and is free to move according to the mechanical properties of the soil. K_{corr} is the effective stiffness as a function of the depth of the foundation, K_{on} the stiffness of the foundation when the footing is above ground and β the correction factor. These parameters, proposed by FEMA Guide 356 [9] and used by CSA Standard S6-14 [4], partially describe the flexible behaviour of geotechnical soil components, since radial and hysteretic soil damping is omitted.

When using multiple vertical springs, spaced along the base of the foundation in the transverse direction, as shown in Figure 3, and adding rotation stiffness at all nodes, the overall rotational stiffness is overestimated. The NEHRP guide [8] proposes to remove added rotational stiffnesses at the nodes and provide modified equivalent vertical stiffness factors to properly model the rocking motion of the foundation. These added vertical stiffnesses (in bold in Figure 3) should be provided in the extremities of the footing in outer areas defined by: $R_e = L_{end} / L$, where R_e is the dimension ratio of the outer zones, L_{end} the length of the increased stiffness zone and L the total length of the footing. The NEHRP guide recommends the use of a length ratio (R_e) between 0.3 and 0.5. Thus, the vertical springs must be distributed in three zones (i.e. two outer zones, one central zone) to avoid overestimation of rotational stiffness.

The individual stiffness of the vertical springs in the inner portion is the product of the stiffness coefficient of the foundation ground and the effective spring area $K_z{}^i = K_z / 4BL$, Where $K_z{}^i$ is the stiffness coefficient of the foundation soil, K_z the vertical stiffness, B the width of the footing and L the depth of the footing. In outer areas, stiffness is increased by multiplying the stiffness coefficient of the foundation ground, the effective spring area and the ratio of the stiffness of the springs in the outer areas:

$$R_k = \frac{\left(\frac{3K_{xx}}{4K_z{}^i LB^3}\right) - (1 - R_e)^3}{1 - (1 - R_e)^3} \tag{2}$$

where $K_z{}^i$ is the stiffness coefficient of the foundation soil, K_{xx} the rotational stiffness about the X axis, B the width of the footing, L the depth of the footing and R_e is the dimension ratio of the outer zones.

NEHRP Guide Methodology

The NEHRP approach for soil-structure interaction also uses a multiple degrees of freedom linear model for the foundation, and also accounts for energy dissipation by adding dampers in the system. The footing is also modelled as a rigid element, and connected to spring/damper elements in the vertical and transverse directions. As in the case of the CHBDC method, the footing is modelled with a rigid element, and the ground deformation is allowed for by the springs. Figure 3 shows the node distribution and damper-spring model used for the central bridge bent.

The NEHRP guide proposes correction factors as a function of footing depth and the dimensionless frequency, given by $a_o = \omega B / V_s$, where a_o is the dimensionless frequency, ω the frequency corresponding to the fundamental period in the studied direction, V_s the average shear wave velocity and B the width of the footing. These corrected stiffnesses $K_{corr} = \alpha_i K_{on} \eta_i$ are introduced in the model, along with the radial damping, acting in parallel, using *ZeroLength* elements. K_{corr} is the effective stiffness as a function of the depth of the foundation, K_{on} the stiffness of the foundation when the footing is above ground, α_i a stiffness modification factor and η_i the correction factor as a function of the depth of the footing. One of the two nodes near this element is fixed while the other is attached to the foundation and is free to move according to the idealised mechanical properties of the soil.

The individual damping of the vertical damper models is the product of the damping coefficient of the foundation soil and the effective area of the damper model. The hysteretic damping used is that given by ASCE 7-16 [10]. The damping values introduced to the foundation models are a combination of radial and hysteresis damping : $c_z{}^i = c_z / 4BL$, where $c_z{}^i$ is the damping coefficient of the foundation soil, c_z is the vertical damping, B is the width of the footing and L is the depth of the footing. In external areas, the damping is increased by multiplying the damping coefficient of the foundation ground, the effective area of the damper model and the damping ratio of the damper models in the external areas.

$$R_c = \frac{\left(\frac{3c_{xx}}{4c_z{}^i LB^3}\right)}{R_k(1 - (1 - R_e)^3) + (1 - R_e)^3} \tag{3}$$

where c_{xx} is the rotational damping about the X and R_e is the dimension ratio of the external zones.

Modelling of abutments

The displacement response of a bridge deck can be greatly affected by the ability of abutments to resist longitudinal and transverse movements. The longitudinal capacity depends on the type of abutment and the characteristics of the backfill. Seat-type abutment are used for the bridge under study, where the elastomeric bearings support the superstructure. This configuration allows the deck to move independently of the abutments until a maximum displacement corresponding to the spacing between the deck and the wall is reached. When this spacing is filled, the deck applies a compressive force that mobilizes the passive strength of the backfill and the wall [11].

Longitudinal direction: CSA Standard S6-14 [4], based on the principle of the Caltrans guide [12], specifies that the force exerted by the active pressure of a backfill can be represented by using a spring with a bi-linear behaviour. For an abutment consisting of a wall less than 1.7 m high and a compact backfill that is not susceptible to loss of capacity, which is the case for the bridge under investigation, the lateral stiffness representing the near field is calculated by : $K_x = K_i w[h_w / 1.7]$, where K_x is the stiffness in the longitudinal direction, K_i is the initial stiffness of the backfill, w is the width of the wall in metres and h_w is the height of the wall in metres. The Caltrans guide [3] states that an initial stiffness of 28.7 kN/mm/l.m. can be used for a standard backfill.

The maximum passive force that can be developed is described by : $P_{bw} = h_{bw} w_{bw}[h_w / 1.7](239\ kPa)$, where w_{bw} is the effective depth of the abutment in metres and h_{bw} is the height of the wall in metres. Using the equations 10 and 11, the bi-linear behaviour of the spring is introduced using the unidirectional material *ElasticPPGap*. Figure 5 shows the abutment model in the longitudinal direction.

Transverse direction The Caltrans guide states that the transverse capacity of seat-type abutments should not be considered effective for seismic design unless the designer can demonstrate the rigidity of the elements that can contribute to the transverse strength. In this project, the transverse capacity is neglected in order to consider the most critical case.

INPUT GROUND MOTION

Non-linear dynamic analyses were carried out first with the rigid foundation model, and then with the spring systems added to represent both soil-structure interaction approaches. The soil-structure models are linear (spring and dampers) and the nonlinear elements, as explained above, are the columns and elastomeric bearings. Two types of soil were considered in the analyses. The following describes the impact of soil type on the spectral accelerations and the procedure used to select the input ground motion. Theses motions were applied horizontally, in the transverse and longitudinal directions, and the maximum displacements and column base shears were obtained from the resulting time history responses.

Soil types

In CSA S6-14 standard, soil classes are defined as a function of the average shear wave velocity and penetration resistance over a depth of 30 m. The *Chemin Roy* Bridge is located on soil type "C" (very dense soil and dense rock). Soil type "D" (solid soil), was also used in this parametric analysis. The type of soil affects the computational spectrum and parameters of the soil-structure interaction models. Figure 6 shows the target spectra for the locality of Magog for both soil types. For a low intensity seismic zone, as is the case for the bridge under investigation, the acceleration response for type "C" soil is lower than that for type "D". Parameters also having an influence on foundation models are the shear modulus, Poisson's ratio and the soil density. The shear modulus is obtained by : $G_0 = \rho V_s^2$, where G_0 is the initial shear modulus, ρ the soil density and V_s the velocity of the shear wave in the ground. The secant module (G) is used in the models using the relations G/G_0 of the ASCE standard [10].

Accelerograms

In the absence of actual recordings for Eastern Canada, synthetic accelerograms were used in this study. These accelerograms were selected according to the NBCC 2015 procedure [13], as presented by Atkinson *et al.* [14] Response spectra resulting from the accelerograms were calibrated on the target spectra for a given range of periods. This range covers the periods of vibration modes that significantly contribute to the dynamic response of the structure. NBCC 2015 requires that the selected accelerograms be compatible with the design spectrum, i.e., that their response spectra be equal to or greater than the target spectrum established for a probability of 1/2475 years.

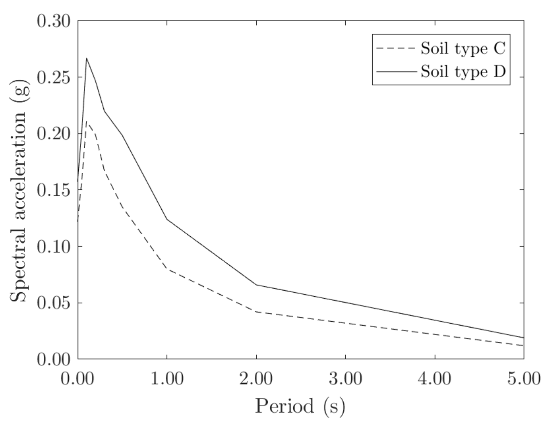


Figure 4. Target spectra for the chemin Roy Bridge.

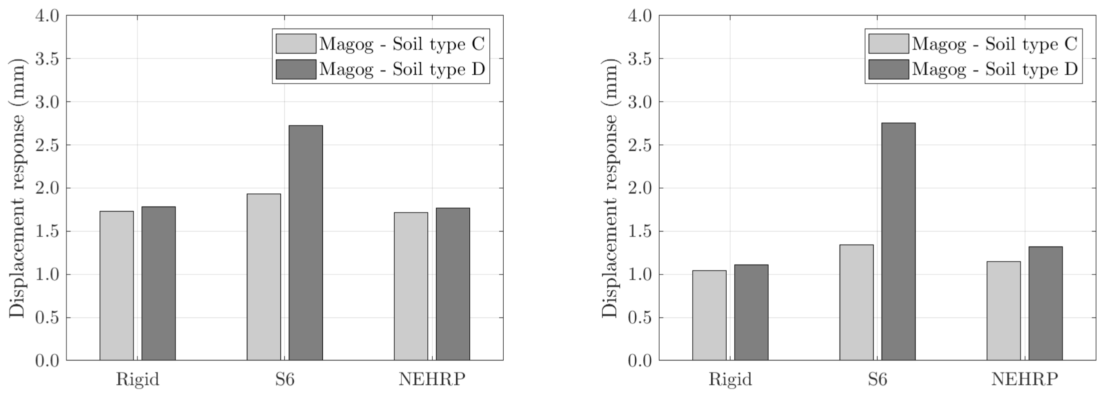
ANALYSES RESULTS

Superstructure displacement responses

Figure 7 presents the maximum displacement responses computed from time history analysis, for the rigid foundation approach (fixed model, no SSI), as well as for the two soil-structure models. Results are shown for both soil types, in the longitudinal and transverse directions. Although in some cases, the soil-structure approach leads to larger responses, the differences are not always significant, and this is due to the bridge being located in a low to moderate seismic zone, and the type of bridge. The inclusion of the soil-structure models induces more flexibility into the system and the displacement responses increase for both types of soil. When energy dissipation is introduced into the model (NEHRP), the displacements are reduced with respect to the stiffness-only model, and this effect is more pronounced for soil type D.

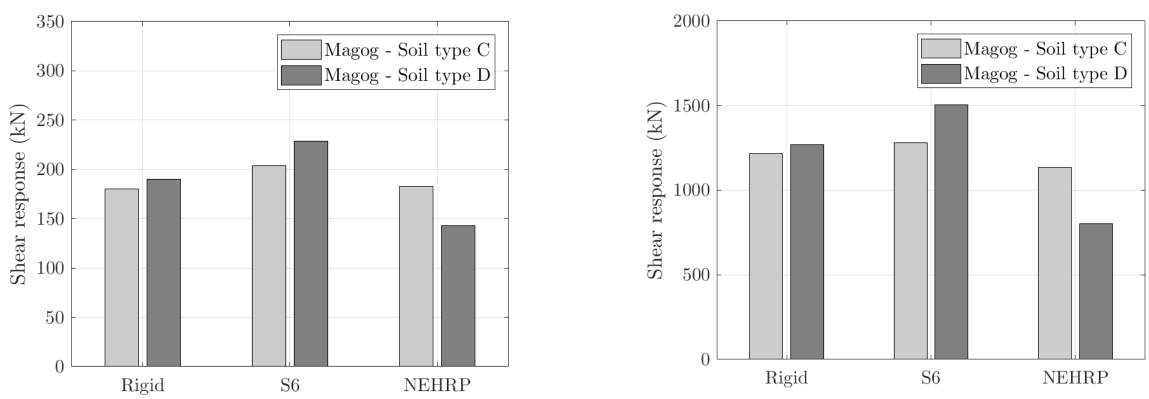
Shear responses at the base of the columns

Figure 8 also shows the base shear results from the time history analysis for the rigid foundation model, along with those obtained with the two soil-structure interaction models. Again, shears computed for both types of soil are shown, and in both directions. Vibration periods are typically short for this type of structure, in both directions, due to its relatively high stiffness. These periods are increased when the foundation models are introduced. For the bridge under investigation, the measured vibration periods are located along the upward slope of the design spectrum, and it is normal to see an increase in the shear response when adding flexibility with the spring models at the base. Introducing damping into these models then lowers these responses, as is expected.



(a) (b)

Figure 5. Superstructure displacement responses: (a) longitudinal direction and (b) transverse direction.



(a) (b)

Figure 6. Shear responses at the base of the bridge bent: (a) longitudinal direction and (b) transverse direction.

CONCLUSIONS

This paper presented a numerical investigation of the effect of two different soil-structure interaction models (with and without damping), applied to an existing 60-m long highway bridge. The models were first calibrated using ambient and forced vibration tests and were then used to carry out a parametric study, focusing on the foundation models. Including the stiffness properties of the soil resulted in increased displacement and base shears. The model that accounted for energy dissipation in the foundation (both material and radial damping) produced lower displacements and base shears with respect to the stiffness-only model, and in some cases lower than the rigid foundation model.

Including a model without damping, such as the one proposed by the CSA S6-14 standard, results in higher displacements on the superstructure, especially for soil type D. The bridge used in this case study is located in an area of low seismicity, and the overall impact of including soil-structure interaction, including damping, did not result in larger displacements or base shears. However, given the relatively low amount of additional effort in this type of dynamic analysis to include a model that accounts for the foundation flexibility and energy dissipation (spring-damper system), the authors recommend that such a model be used, especially for moderate to high seismicity areas.

ACKNOWLEDGEMENTS

The authors would like to thank NSERC, FRQNT and CEISCE for their financial support for this project. The authors also acknowledge the collaboration of Sylvain Bouchard of Transport Québec for the access to the bridge site and bridge drawings. The authors thank Xavier Robert-Veillette for his work on dynamic testing. Finally, the authors recognize the essential assistance of Jeason Desmarais and Claude Aubé, technicians at Université de Sherbrooke.

REFERENCES

[1] Dallaire, F. (2016). "Quantification de l'amortissement visqueux élastique des piles en béton armé des ponts routiers québecois". Master thesis, Université de Sherbrooke, Canada

[2] Robert-Veillette, X. (2013). "Impact du taux d'amortissement visqueux élastique des ponts routiers sur les résultats d'analyses sismiques linéaires et non linéaires". Master thesis, Université de Sherbrooke, Canada

[3] Allemang, R. J. and Brown, D. L. (1982). "A Correlation Coefficient for Modal Vector Analysis." Proceedings of the 1st international modal analysis conference, Vol 1, p.110-116.

[4] Canadian Standards Association – *CSA (2014). CAN/CSA S6-14 : Commentary on CSA S6-14.* Prepared by Canadian Highway Design Code, Mississauga, Canada

[5] University of Berkeley (2006). OpenSees, http://opensees.berkeley.edu/

[6] Pinard, M. (2014). "Méthode de modélisation numérique et étude paramétrique de la performance sismique des piliers de ponts typiques du Canada". Master thesis, Université de Sherbrooke, Canada

[7] Palermo, D. and Vecchio, F. (2003). "Compression field modeling of reinforced concrete subjected to reversed loading: Verification". *ACI-Structural Journal*, 101(2), 155-164.

[8] NEHRP (2012). *Soil-structure Interaction for Building Structures*, Prepared by NEHRP Consultants Joint Venture for the National Institute of Standards and Technology, Gaithersburg, MD, USA

[9] Federal Emergency Management Agency – FEMA (2000). *FEMA 356 : Prestandard and commentary for the seismic rehabilitation of buildings.* Prepared by American Society of civil Engineers, Washington, D.C. USA.

[10] American Society of Civil Engineers – ASCE (2016). *ASCE/SEI 7-16* : *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. Prepared by the ASCE, Virginia, USA.

[11] Suescun, J.R. (2010). "Courbes de fragilité pour les ponts au Québec tenant compte du sol de fondation". Master thesis, Université de Sherbrooke, Canada

[12] Caltrans (2013). *Seismic Design Criteria (v.1.7).* Prepared by Caltrans, USA.

[13] National Building Code of Canada – NBCC (2015). *National building code- Part 4*. Prepared by National Building Code of Canada, Ottawa, Canada.

[14] Atkinson, G. *et al.*. (2015). "Selection and scaling of ground motion time histories for seismic analysis using NBCC 2015" In *11th Canadian Conference on Earthquake Engineering*, Victoria, BC, Canada.