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EFFECTS OF SITE AMPLIFICATION ON THE SEISMIC VULNERABILITY OF TYPICAL QUEBEC CITY'S BRIDGES

B. Galy[1], A. Khaled[2] and M.J. Nollet[2]

ABSTRACT

The site amplification of the ground motion has been found to be a fundamental factor to consider in the seismic evaluation of structures. In this paper, the effects of site amplification on the seismic vulnerability of bridge structures is assessed throughout linear and non linear analyses carried out for a typical Québec City R/C generic bridge model subjected to historical earthquakes representative of eastern Canada. A total of 48 linear modal time history analyses and 24 non-linear time history analyses were performed. It was found that the model shows a linear-elastic behavior for site classes A and B. For site classes C, D and E, plastic hinges form at the base of the columns, leading to permanent displacements of the bridge deck. Results also show that the energy dissipation is a better indicator of damages than the permanent displacement.

Introduction

Most of Quebec City bridges were built between 1960 and 1980. During this period, most codes and regulations did not include detailed seismic prescriptions, specifically in consideration with local site effects. Recent work done on the microzonation for the territory of the City of Quebec has established the predominance of soils considered to represent an amplification risk during an earthquake (Leboeuf and Nollet 2006). Therefore the seismic vulnerability of these bridges raises questions, regarding to the seismic site conditions on which they are built.

Site properties can modify the input motion and make it more harmful by amplification of the seismic motion or liquefaction. The structural engineering community has acknowledged the significance of seismic site effects. In most building codes, seismic site effects are taken into consideration through amplification factors applied to the spectrum and defined according to a seismic site categorization. Such categorization offers a simplified approach to consider amplification effect in a context of seismic evaluation of existing structures such as bridges. This has been shown in the project on the seismic vulnerability evaluation of essential infrastructures of Quebec City (Nollet and al. 2008). The procedure introduced by the NEHRP (NEHRP 1994) and adapted in the National Building Code of Canada (NBCC) (NRC 2005), modifies the linear-elastic design spectrum, which implies that the structure is analyzed in the linear-elastic domain. However, under moderate to high seismic motion earthquakes, structures may exhibit a non-

[1]Ph.D. student, Dept. of Civil Engineering, École de Technologie Supérieure, Montréal, QC H3C 1K3
[2]Professor, Dept. of Civil Engineering, École de Technologie Supérieure, Montréal, QC H3C 1K3

linear behavior. This raises questions on the significance of site amplification upon non-linear response of bridges. The study presented in this paper aims to evaluate the seismic site amplification effects for typical Quebec City's sites, and their influence on the response of bridges in a context of seismic vulnerability assessment. More specifically, the objectives are to: (i) consider the transformation of the seismic motion through real stratigraphies of Quebec City soils and, (ii) consider the non-linear response of typical 1970's Quebec City bridges.

The methodology used to achieve the objectives of this study includes the following steps:

- Modeling of real stratigraphies corresponding to the five site categories of 2005 NBCC.
- Selection of historical earthquake records representative of Eastern Canada.
- Transformation of the selected accelerograms through the soil columns models.
- Selection of a generic bridge model representative of Quebec City bridge structures.
- Non-linear time-history analyses of the selected bridge model.

This paper presents the main elements of this study and the analysis of the results.

Ground motion time-histories

Selection of ground motion earthquake records

Generally, smooth hazard spectrum are used to describe the seismic excitation in order to evaluate the response of structures (Léger and al. 1993). But for some critical structures, such as nuclear power plants, and in some high seismic activity regions, time-history analyses are preferred to spectral analyses because they yield more accurate results (Priestley and al. 1996; Singh and Mittal 2005). Furthermore, for the study of non-linear behavior of critical structures, advanced analyses such as dynamic time-history analyses with appropriate site specific earthquake records are usually required (Léger and al. 1993). The input motions used in these studies are either historical records scaled to match a smooth hazard spectrum, or synthetic accelerograms matching this spectrum. For Eastern Canada, scaled historical records present the advantages of preserving the characteristics of real earthquakes such as the time evolution of the frequency content (Léger and Leclerc 1996). Furthermore, the use of scaled historical records that match codes response spectra is common because they yield sparsely scattered results which allows finding the median with less accelerograms (Carballo Arevalo 2000).

In this study four ground motion records from historical earthquakes were used. The selected ground motion records have been recorded during the 1988 Saguenay and the 1985 Nahanni events. The characteristics of these records are presented in Table 1. Because of Nahanni earthquake characteristics, inner a continental plate and rich in high frequency motion, it is reasonable to consider its records as suitable for the analysis of structures located in Quebec City (Adams and Basham 1989; Lam and al. 1996; Léger and al. 1993).

Table 1. Characteristics of the unscaled ground motions.

N°	Date, Event	M_w	R (km)	Component	PGA (g)	PGV (m/s)
1	25 Nov. 1988, Saguenay	5,7	43	Chicoutimi Nord, N124	0,131	0,025
2	25 Nov. 1988, Saguenay	5,7	90	Les Éboulements, EW 270°	0,102	0,027
3	25 Nov. 1988, Saguenay	5,7	64	Saint – André, EW 270°	0,091	0,009
4	23 Dec. 1985, Nahanni	6,5	24	Battlement Creek-S3, N270°	0,186	0,063

Transformation of the ground motion earthquake records

The seismic site categorization defined by the S6 bridge design code (CSA 2006) is less detailed than the one adopted by the 2005 NBCC presented in Table 2. This simplicity may be an advantage for structural engineers who can determine the site class by using a simple borehole. Nevertheless, in this study, the classification of the 2005 NBCC has been chosen because site classes are defined through soil properties such as the average shear-wave velocity and average standard penetration resistance. There is another advantage to the use of this classification: microzonation maps based on the 2005 NBCC site classes have been developed for Quebec City, and new ones are in development.

In order to represent the amplification effect of site classes as defined in the 2005 NBCC on non-linear response of Quebec City's bridges, the selected ground motion earthquake records were first transformed through soil columns representing real stratigraphies of Quebec City soils. This transformation was done with Shake 2000 software (Ordoñez 2005). Shear modulo and damping variations due to soil deformations are characteristics describing the non-linear soil behavior under cyclic loading (Llambias and al. 1993). These variations are described by experimentally plotted curves. The choice of these curves is extremely important in order to obtain realistic results for rock motion transformation through a soil column. Shake 2000 software uses these curves in order to represent soil non-linearity by the linear equivalent method. There are two options to model the different 2005 NBCC site classes. The first one is to represent the soil by a homogeneous 30 m column subdivided in 10 layers of 3 m each. The second one is to model heterogeneous soils with a variable number of soil layers. The access to a Quebec City's borehole database enabled the modeling of realistic site models; therefore the second option was adopted in this study. The database includes the following informations: soil nature, precise stratigraphy, and results of Standard Penetration Tests.

The selected accelerograms (Table 1) were recorded on hard rock sites. Therefore no transformation of the seismic motion was necessary for site class A. For site class B, a 30 m homogeneous soil column subdivided in ten layers was modeled. Soil columns for site classes C, D and E modeled real stratigraphies obtained from boreholes selected in the Quebec City's database. A 30 m homogeneous soil column was also modeled for site class C in order to compare it to the realistic model.

Table 2. Simplified 2005 NBCC site categorization.

Site Class	Ground Profile Name	Average Shear Wave Velocity, $\overline{V}_s$ (m/s) in Top 30 m as per Appendix A
A	Hard *Rock*	$\overline{V}_s > 1500$
B	*Rock*	$760 < \overline{V}_s \leq 1500$
C	Very Dense *Soil* and Soft *Rock*	$360 < \overline{V}_s < 760$
D	Stiff *Soil*	$180 < \overline{V}_s < 360$
E	Soft *Soil*	$\overline{V}_s < 180$ or PI > 20, w ≥ 40%, and $s_u < 25$ kPa
F	Specific evaluation required	a. Liquefiable *soils*, quick and highly sensitive clays, collapsible weakly cemented *soils*, and other *soils* susceptible to failure or collapse under seismic loading. b. Peat and/or highly organic clays greater than 3 m in thickness. c. Highly plastic clays (PI > 75) with thickness greater than 8 m. d. Soft to medium stiff clays with thickness greater than 30 m.

The selected accelerograms were transformed through this site models and then scaled to match the 2005 NBCC uniform hazard spectra. The spectrum matching was made in the frequency domain, where the original signal is scaled by an iterative procedure for each frequency. Among its advantages, this method offers a fast convergence (time-efficient computation) and the preservation of the non stationary characteristics of the original accelerogram (Carballo Arevalo 2000). It tends however to reduce the number of impulses for Eastern earthquakes but gives good results for low magnitude events such as the Saguenay 1988 earthquake (Léger and al. 1993). The scaling was limited to two iterations in order to preserve the characteristics of the recorded motions (Léger and Leclerc 1996).

Bridge model representative of Quebec City's structures

Selection and description of the bridge model

The selection of the bridge model used in this study was based on a statistical analysis on 119 Quebec City's bridges, most of which were built around 40 years ago. The most common ones are bridge girder type: 71% of them are straight bridge and 91% have four or more girders to support the slab.

The generic bridge model (Fig. 1) consists of a two 40 m span straight bridge with a central four-columns bent having circular cross-section. The bridge deck consists of concrete slab and type VI AASTHO precast prestressed concrete girders. All the columns are assumed to be fixed at the base. The Quebec Ministry of Transportation (QMT) identifies the selected bridge model as bridge-type 42.

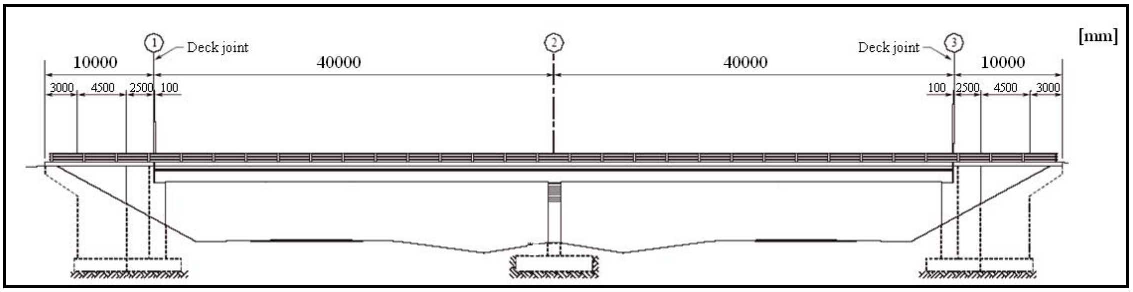

Figure 1. Longitudinal view of the bridge model.

Modeling of the selected bridge

In order to represent a bridge built in the 1970's, the generic bridge model was designed according to the S6-74 rules and regulations. This building code specifies that seismic forces must be considered in regions were seismic risk is significant. However, no particular seismic specifications (PGA or hazard spectra) are given to evaluate such a risk: the first proposed hazard spectra for design was proposed in the 1985 NBCC Code. That spectrum was used to represent the seismic forces that might have been considered in the 70's.

The purpose of this study is to evaluate the overall response of the bridge model. For that purpose, a concentrated mass modeling was adopted. In modeling the structure, un-cracked in-plane flexural and shear stiffness properties were specified for the bridge deck The deck is represented by an elastic beam element, a common practice since the bridge deck rarely yields because they are designed to support heavy loads (Jeremic and al. 2004). A vertical diaphragm has been included for the deck and the crosshead in order to simulate a very large rigidity. The abutments have only one translational degree of freedom: along the longitudinal axis. All their rotational degrees of freedom are set free. The pile has been modeled more precisely in order to implement plastic hinges and evaluate the non-linear response of the structure. 95% of the participating modal mass has been considered for the modal analysis.

The characteristics of the plastic hinges have been automatically determined by SAP2000 (Computers and Structures 2007) software, according to FEMA 356 rules. The stress-strain analysis results have been compared to a sectional analysis conducted with Response 2000 (Bentz and Collins 2000). The two analyses showed a good correlation.

Non-linear analyses of a typical bridge under the transformed accelerograms

Seventy-three analyses were conducted in order to assess the seismic vulnerability of the selected bridge model regarding to the site class. A linear-elastic modal analysis was first conducted to determine the periods and vibration modes of the bridge. Then forty-eight modal time-history analyses were performed (24 for each direction). Those analyses, using the elastic modes found in the previous step, aimed at evaluating the yielding probability for the input ground motion in the longitudinal and transversal principal directions. It appeared that only the input ground motions in the longitudinal direction could lead to yielding, consequently the input ground motions in the transversal direction were not considered in non-linear analyses. Finally, twenty-four non-linear direct integration time-history analyses were conducted on the bridge model. Integration of the accelerograms was done with the Newmark-β time-stepping method with constant average acceleration (parameters γ and β taken as 0.5 and 0.25 respectively). This method eliminates numerical damping and is unconditionally stable. It is perhaps the most popular method for earthquake response analysis because of its superior accuracy (Chopra 2007).

Non-linear displacement analyses results

Non-linear displacements of the bridges were considered for the input ground motions in the longitudinal direction. In order to enlighten the yielding phenomenon, maximum displacements at the top of the columns have been normalized with respect to the yielding displacement Δ_y. When ductility factor μ_Δ is higher than 1, plastic hinges appear at the base of the columns. Those cases are highlighted in bold in Table 3.

Table 3. Maximum displacements at column top for a longitudinal solicitation (in mm).

Site class (2005 NBCC)	Saguenay						Nahanni	
	Chicoutimi Nord		Les Éboulements		Saint - André			
	Δ_{U1} max	μ_Δ	Δ_{U1} max	μ_Δ	Δ_{U1} max	μ_Δ	Δ_{U1} max	μ_Δ
A Rock	8,00	0,55	9,42	0,65	9,75	0,67	9,96	0,69
B Homogenous	9,44	0,65	11,4	0,79	11,3	0,78	10,6	0,73
C REF : bore hole 545	13,2	0,92	15,8	**1,09**	15,2	**1,05**	15,4	**1,06**
C Homogenous	13,3	0,92	16,8	**1,16**	14,9	**1,03**	14,8	**1,02**
D REF : bore hole 2528	21,0	**1,45**	18	**1,24**	22,4	**1,55**	17,1	**1,18**
E REF : bore hole 2365	32,3	**2,23**	23,5	**1,62**	25,9	**1,79**	21,1	**1,45**

It is found that for bridge models subjected to seismic motions with no transformation (i.e., site class A or hard rock) or with transformation through site class B (rock), maximum displacements stay in the linear elastic domain (60 to 80% of Δ_y), as can be seen in Table 3. Yielding occurs for bridge models subjected to seismic motions with transformation through site classes C, D and E (i.e., very dense soil to soil profile with soft clay), except for site class C, Saguenay, Chicoutimi Nord. Fig. 2 shows that yielding initiates for bridge models resting on site class C and that yielding is more important for site classes D and E. This was expected since spectral pseudo-accelerations from uniform hazard spectra are amplified as soil quality degrades. However, it can be seen from Fig. 2 and Table 4 that in the case of the Nahanni earthquake, the permanent displacement of bridge model on site class E is smaller than on site class D. This can be explained by considering the hysteretic behavior of the columns (Fig. 3). For the bridge model on site class E, several positive displacements in the plastic domain (ductility factor μ_Δ higher than 1) are recorded before the column reaches its maximum peak displacement on the negative side. This results in a smaller negative permanent displacement than for the bridge model on site class D. However, the energy dissipation in the plastic hinge is larger for the bridge model on site class E (more hysteretic loops as shown on Fig. 3) than for the bridge model on site class D.

Table 4. Permanent displacement (in mm) at column tops

Site class (2005 NBCC)	Saguenay						Nahanni	
	Chicoutimi Nord		Les Eboulements		Saint - André			
	Δ_{U1P}	% Δy	Δ_{U1P}	% Δy	Δ_{U1P}	% Δy	Δ_{U1P}	% Δy
A Rock	0	0,0	0	0,0	0	0,0	0	0,0
B Homogenous	0	0,0	0	0,0	0	0,0	0	0,0
C REF : bore hole 545	0	0,0	1	7,2	0,1	0,7	0,8	5,8
C Homogenous	0	0,0	2,3	16,7	0,5	3,6	0	0,0
D REF : bore hole 2528	6,2	44,9	3,2	23,2	4,6	**33,3**	2,3	**16,7**
E REF : bore hole 2365	17	123,2	6,4	46,4	4	**29,0**	1,6	**11,6**

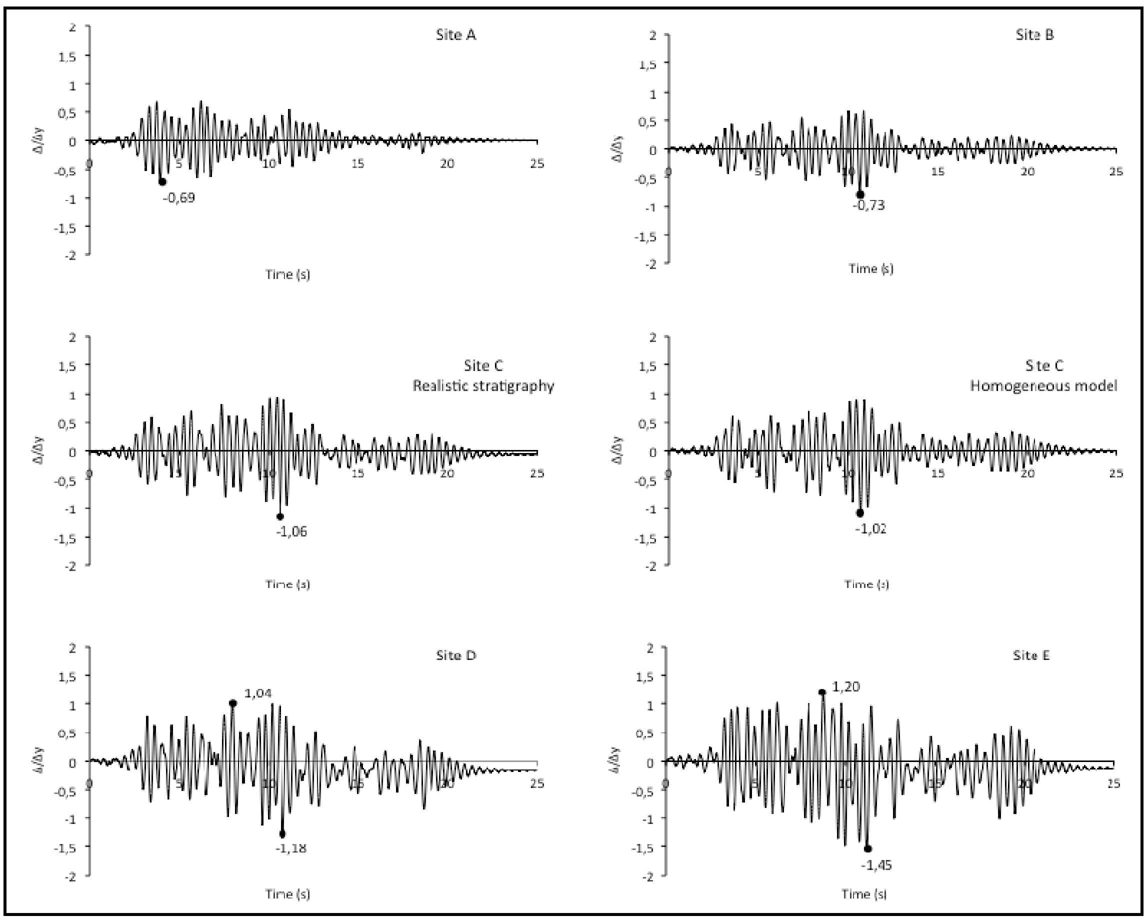

Figure 2. Normalized displacements at the top of columns for each site class, recorded during longitudinal non-linear analysis (Nahanni, Battlement Creek).

Ductility and security levels

Table 5 presents curvature and displacement ductility demand. The curvature ductility is related to an individual section response, while displacement ductility is related to the overall structure and its geometry (Priestley and al. 1996). Therefore, the relationship between displacement and curvature ductility, in the plastic domain, depends on the structural geometry.

The ductility demands imposed by eastern earthquake ground motions are relatively high and the maximum values can be used to determine security levels as defined by FEMA 356 (Table 6). FEMA 356 security levels are considered here since moment-curvature diagram was defined according to its recommendations.

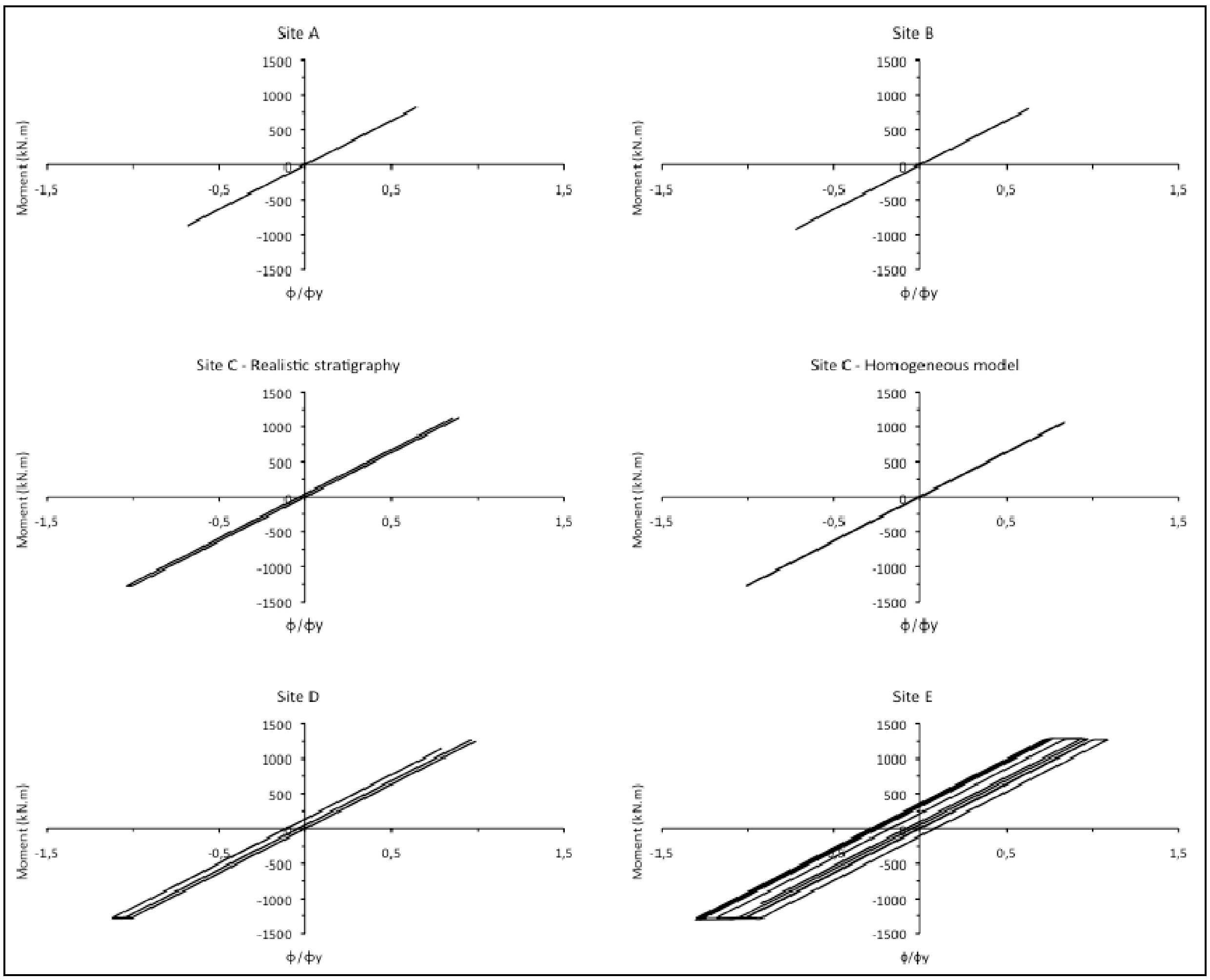

Figure 3. Normalized moment-curvature diagrams for each site class (Nahanni, Battlement Creek).

Table 5. Curvature and displacement ductility demands.

Site class (2005 NBCC)	Saguenay						Nahanni	
	Chicoutimi Nord		Les Eboulements		Saint-André			
	μ_Δ	μ_Φ	μ_Δ	μ_Φ	μ_Δ	μ_Φ	μ_Δ	μ_Φ
A Rock	0,55	0,55	0,65	0,64	0,67	0,67	0,69	0,68
B Homogenous	0,65	0,65	0,79	0,78	0,78	0,77	0,73	0,72
C REF : bore hole 545	0,92	0,91	1,09	1,06	1,05	1,03	1,06	1,04
C Homogenous	0,92	0,91	1,16	1,11	1,03	1,02	1,02	1,01
D REF : bore hole 2528	1,45	1,30	1,24	1,16	1,55	1,37	1,18	1,12
E REF : bore hole 2365	**2,23**	1,83	1,62	1,40	**1,79**	1,53	1,45	1,31

Table 6. FEMA 356 security levels.

Security Level		μ_Δ	μ_Φ
Elastic	%	1	1
Immediate Occupancy (IO)	20	1,79	2,13
Life Safety (LS)	80	4,14	5,53
Collapse Prevention (CP)	100	4,93	6,66

FEMA 356 defines Collapse Prevention level (CP) as the stage beyond which the bridge collapses. According to the results presented in Table 4, the bridge stays way beneath Life Safety (LS) and CP levels. In two cases, Immediate Occupancy (IO) level is reached: for Chicoutimi Nord and Saint-André records. From these results we can conclude that the bridge safety would not be endangered by seismic events with similar characteristics as those of the selected accelerogramms, but that some repairs may be necessary after such events.

Conclusions

In this study, the non-linear response of a bridge model representative of a typical Quebec City 1970's R/C bridge was analyzed under different soil conditions. The five site classes defined in the NBCC 2005 (A, B, C, D and E) were considered and real soil stratigraphies from Quebec City area were modeled as soil columns to transform the accelerogram recorded on the rock. Analysis of the results leads to the following conclusions:

- The bridge structure is more vulnerable on soft soils, particularly for site classes D and E.
- The maximum ductility demand reached is below the collapse prevention security level. Nevertheless, the structure reaches an Immediate Occupancy level in two cases, suggesting that an evaluation of the structural capacity should be performed after strong earthquakes for bridges built on sites of classes D or E.

- In two cases, permanent displacement is greater for bridge model on site class D than on site class E. However, the plastic energy dissipation is greater for site class E. This suggests that a simple measure of permanent displacement or curvature after a seismic event is a poor indicator of the damage level experienced by the structure.
- There is no significant difference between models simulating a homogeneous soil column and a realistic non-homogeneous soil column of a site class. The results are very close and none of the two models yields unfavorable behavior. A homogeneous model of the soil column could be sufficient to represent site class C for Quebec City.

References

Adams, J., and P. Basham. 1989. « Seismicity and seismotectonics of Canada east of the Cordillera ». vol. 16, p. 3-16.

Bentz, E.C., and M. P. Collins. 2000. *Response 2000*. Université de Toronto

Carballo Arevalo, Jorge Eduardo. 2000. « Probabilistic seismic demand analysis: Spectrum matching and design ». Ph.D. Thesis , Stanford University.

Chopra, Anil K. 2007. *Dynamics of structures : theory and applications to earthquake engineering*, Upper Saddle River, N. J.: Pearson/Prentice-Hall,

Computers and Structures, Inc. 2007. *CSI Analysis reference manual. For SAP2000, ETABS, and SAFE.* Berkeley, California, 467 p.

CSA. 2006. *Canadian highway bridge design code*, 10th éd. Mississauga, Ont. Canadian Standards Association.

Jeremic, Boris, Sashi Kunnath and Feng Xiong. 2004. Influence of soil-foundation-structure interaction on seismic response of the I-880 viaduct. *Engineering Structures,* vol. 26, nº 3, p. 391-402.

Lam, N., J. Wilson and G. Hutchinson. 1996. Building ductility demand : interplate versus intraplate earthquakes. *Earthquake Engineering & Structural Dynamics,* vol. 25, nº 9, p. 965-985.

Leboeuf, Denis, and Marie-José Nollet. 2006. *Microzonage et vulnérabilité sismique de la ville de Québec 2005-2006* Rapport final soumis à la ville de Québec, 17 Juillet, 108 p.

Léger, P., and M. Leclerc. 1996. Evaluation of earthquake ground motions to predict cracking response of gravity dams. *Engineering Structures,* vol. 18, nº 3, p. 227-239.

Léger, P., A. K. Tayebi and P. Paultre. 1993. Spectrum-compatible accelerograms for inelastic seismic analysis of short-period structures located in eastern Canada. *Canadian Journal of Civil Engineering,* vol. 20, p. 951-968.

Llambias, J.M., D.J. Shepherd and M.D. Rodwell. 1993. Sensivity of seismic structural response to intrepretation of soils data. In *Soil Dynamics and Earthquake Engineering VI*. p. 101-113. Southampton Boston: Computational Mechanics Publications.

NEHRP. 1994. *Recommended provisions for seismic regulations of new buildings: Part 1, provisions. FEMA 222A*. Washington, D.C: National Earthquake Hazard Reduction Program, Federal Emergency Management Agency, .

Nollet, Marie-José, Denis LeBoeuf and Amar Khaled. 2008. *Vulnérabilité sismique des infrastructures essentielles de la ville de Québec*. Rapport final soumis à la ville de Québec, 115 p.

NRC. 2005. *National Building Code of Canada*. Ottawa, Ontario: National Reasearch Counsel of Canada.

Ordoñez, G. A. 2005. *Shake 2000*, version. 2.00. University of California

Priestley, M. J. N., Frieder Seible and Gian Michele Calvi. 1996. *Seismic design and retrofit of bridges*. New York, N.Y.: J. Wiley and Sons.

Singh, V. N. , and A. Mittal. 2005. Synthetic accelerograms for two Himalayan earthquakes using convolution. *Current Science,* vol. 88, nº 8 (Avril), p. 1289-1297.