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SEISMIC VULNERABILITY EVALUATION OF HIGHWAY BRIDGES IN QUEBEC USING FRAGILITY CURVES.

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ABSTRACT

Fragility curves are used to evaluate the seismic vulnerability of the multi span simply supported concrete girder bridges in the Province of Quebec. Fragility curves are a probabilistic tool that is used to evaluate the vulnerability of a structure. They express the probability of a bridge reaching a certain damage state for a given seismic event. Due to their probabilistic aspect, fragility curves enable to account for uncertainties in some properties of the bridges or in the seismic excitation. The seismic historical activity in the Province of Quebec demonstrates the need to consider seismic effects in the evaluation and retrofit of existing bridges. The bridge network in Quebec, like many all over the world, dates back more than 30 years. At the time it was designed and built, the technology and knowledge in this domain were far from their current state. Also, monitoring and maintenance of these existing structures have been a great challenge in view of the severe weather condition they are exposed to. From the Transports Quebec (TQ, 2005) database, a total of 2592 multi-span bridges compose this network, thus, it would be unfeasible to develop a fragility curve for each bridge. Therefore, the bridge network is divided in bridge portfolios and for each of those, fragility curves are developed. The portfolios or classes are defined according to the construction material and construction system type. A three dimensional nonlinear finite element numerical model is developed for each class and these models are submitted to a series of events. The responses of some bridge components are analyzed and a linear regression is performed to develop probabilistic demand models (PDMs). The ensemble of these PDMs is compared to predefined limit states to develop the bridge system fragility curves.

Introduction

Fragility curves are an emerging tool in the development of a probabilistic risk assessment evaluation in a bridge network. They can also be used to optimize bridge retrofit methods and in the development of a post-event action plan. Fragility curves describe the probability of a structure being damaged beyond a specific damage state for various levels of ground shaking. This is

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particularly useful in regions of moderate seismicity, such as Eastern Canada and more specifically, Quebec, where bridge officials are beginning to consider retrofit programs, and pre-earthquake planning. In addition, some bridge design codes begin to recommend the use of a screening process based on the expected damage (or vulnerability) estimation (MCEER–06–SP10, 2006 and FIB CEB-FIP Bulletin 39, 2007). According to TQ (2005), 75% of Quebec's bridges have more than thirty years. Since their construction, there have been significant improvements in bridge design code requirements, particularly in earthquake bridge design and analysis, in the past few years. The design for earthquake and the technology has improved, the Seismic Hazard in most of the regions in Quebec has changed and new codes and design procedures have been developed. The damage to bridges observed in recent earthquakes (Chang 2000 and JSCE 1999) highlights the need to perform adequate assessment of the vulnerability of bridges and bridge networks prior to seismic events, especially when they were not designed to resist to these events.

Comparisons of empirical and analytical fragility curves have shown good agreement between theory and field observation for the 1994 Northridge, and 1989 Loma Prieta earthquakes (Basöz 1999, Mander 1999). Therefore, analytical fragility curves are suitable to be used in a region where empirical data is not available. The fragility analysis generally includes three major parts: (a) the simulation of bridges in the network to account for uncertainty in bridge properties, (b) the simulation of ground motions, and (c) the generation of fragility curves from the seismic response data of the bridges related to the structural demand and the predetermined limit states related to the structural capacity. Thus an appropriate model for assessing the fragility of a structural system, such as a bridge, is able to determine the probability that the structural demand exceeds the structural capacity. The seismic demand can be determined through nonlinear static methods often referred to as capacity spectrum methods (Mander 1999) elastic spectral analysis (Hwang 2000) or nonlinear time history analysis (Choi 2004). The most rigorous method for developing analytical fragility curves for bridges is through the utilization of non-linear time history analysis (THA) to define the seismic demand and it is the one used in this study.

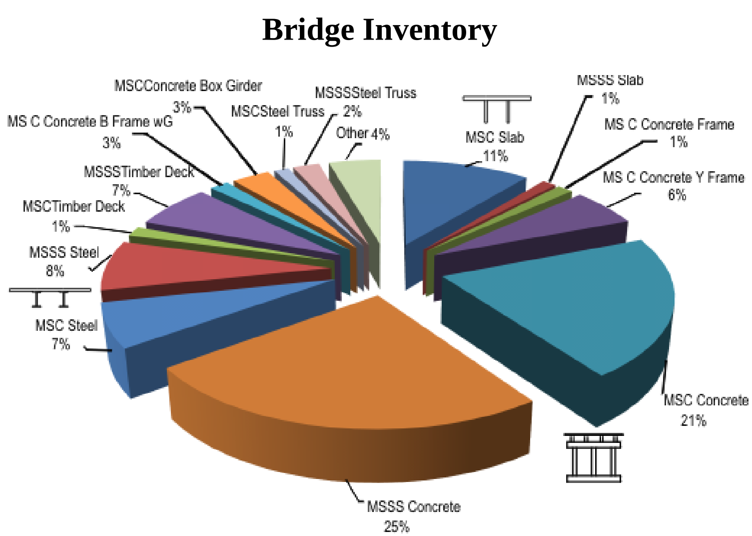

Figure 1. Bridge Classes Distribution in Quebec

Since there are 2592 multi-span bridges in Quebec's network (TQ 2005), it would be difficult, costly and time consuming to produce fragility curves for each bridge. The solution is to group similar bridges into portfolios and produce fragility curves for each portfolio, thus instead of

2592 series of fragility curves, one would have 5 to 15 series of curves depending in the number of bridge portfolios or classes considered. This method effectiveness depends on a reliable description of each class or portfolio though a limited number of parameters. Figure 1 shows the multi-span bridge classes and their distribution in Quebec's bridge network. The bridges were classified according to the construction material, construction system type and number of spans (Nielson 2005). In Quebec's network 7 of the 12 bridge classes represent 83% of the network and the bridges with concrete girders represent 46% of the bridges. The class used to illustrate the development of fragility curves in this paper is the Multi Span Simply Supported Concrete Girder – MSSS Concrete which represents 25% of all bridges in Quebec.

The multi-span simply supported concrete girder bridge (MSSS Concrete) consists of three simply supported spans supported by multi-column bents. There are three columns in each bent and 4 girders supporting the deck. The girders are seating in elastomeric bearing pads resting on the bent caps. The bridges are on average 68 meters long and 13 meters wide with a vertical under clearance of 7 meters and a ratio between the main span and end spans of 0.4 (Figure 2a).

Analytical Model of MSSS Concrete Bridges

The MSSS Concrete Bridge Class consists of elements that may exhibit highly nonlinear behavior, such as elastomeric bearings, columns, abutments and the impact between the decks. These nonlinearities are incorporated into three dimensional nonlinear analytical models developed using OpenSees (Manzoni 2001). The superstructure is represented by a single element in the center of the deck cross section and the transverse properties are represented by rigid elements to enable the distribution of the forces to the rest of the structure (Figure 2c). There are 10 main superstructures elements in the end spans and 20 in the main span. Their properties include the transverse sectional area, A, the elastic modulus, E, the shear modulus, G, and the moments of inertia the three main directions: torsion, J, I_z and I_y.

Pounding between the decks is accounted for using the contact element approach including the effects of hysteretic energy loss which is represented by a bilinear model with a gap as it was presented in the work of Muthukumar (2003). Damping is accounted for in the model using Rayleigh damping but is treated as a random variable (Table 1). The connection between the superstructure and bents and abutments are made with elastomeric bearings. The behavior of the elastomeric bearings was represented in OpenSees using a zero-length element with a bilinear model material behavior in both horizontal directions. The material used to define the spring behavior was *Steel01*. This bilinear model is defined by the elastic stiffness K_1, the post-elastic stiffness K_2, the yielding displacement D_y and the final displacement D. These constants were calculated based on AASHTO (2007).

The behavior of the seat type abutments is incorporated in the model through the use of translational and rotational springs based on a linear elastic half-space theory. The longitudinal stiffness of the abutment springs are a function of the spread footing stiffness, resistance of the backfill soil and stem wall. The transverse stiffnesses are a function of the resistance of the spread footing stiffness, the embankment and wing wall. Spring constants for this element behavior are calculated based on the recommendation of Wilson (1988). The mass of the abutments are calculated including a participation of the embankment as recommended by Wilson and Tan (1990).

The bent beam and columns are represented using non-linear beam column elements (figure 2b). The elements used the materials *Concrete02* and *Steel02*. The sections were divided in fibers (Figure 2d and 2e). Each fiber element has its own stress–strain relationship, and can be used to model the cross-section of the column with its confined and unconfined concrete regions as well as the longitudinal steel reinforcement. The elements can be defined as forced based elements with spread plasticity as presented in Neuenhofer (1998). The foundation is represented by rigid beams to account for its geometry. The foundation mass is applied in the center of the footing height and they are distributed for each column in the bent. The springs are represented by an elastic material defined by a constant stiffness and dampers by a material called *Viscous* defined with the damping constant C (Clough and Penzien, 1975).

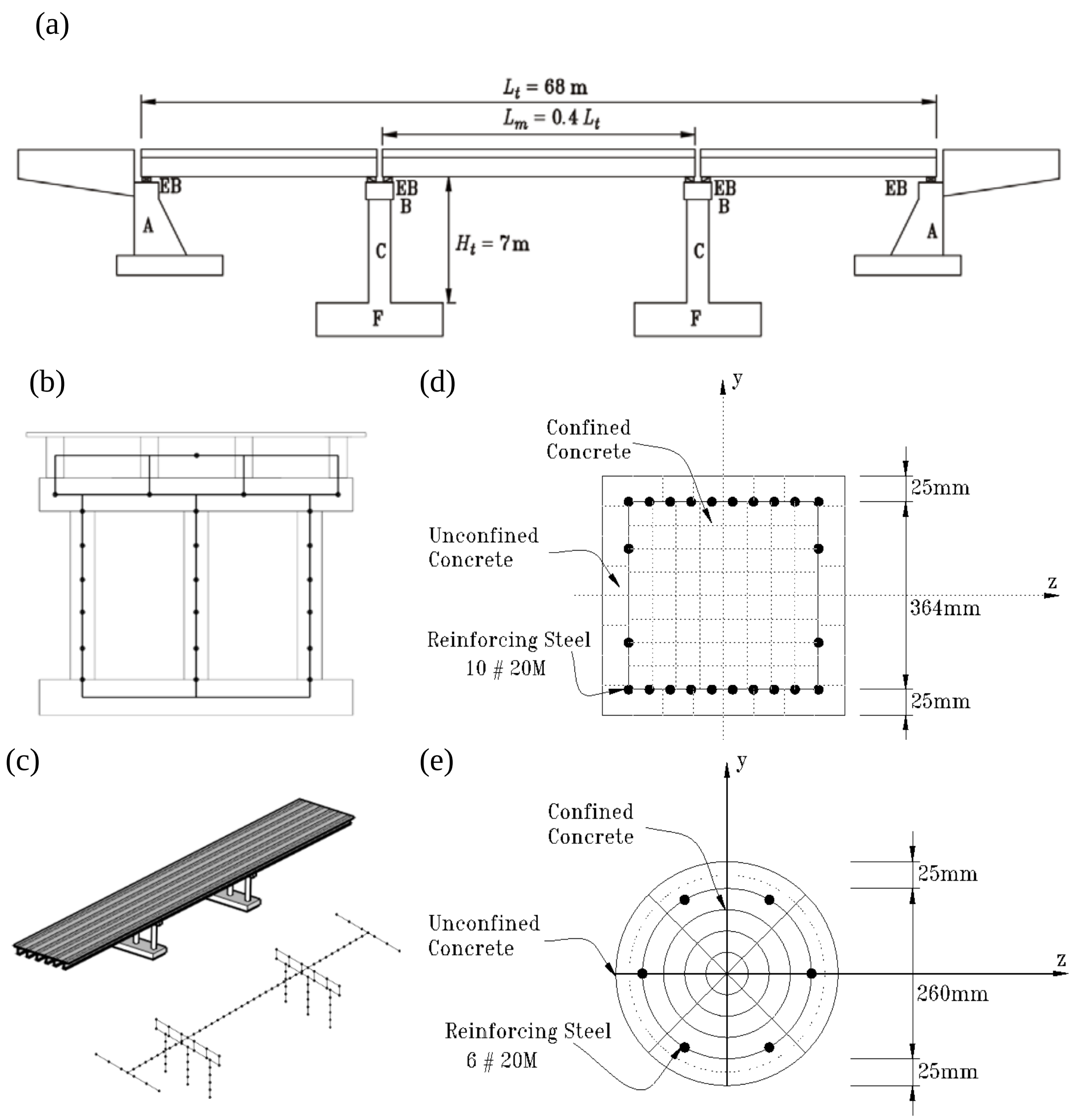

Figure 2. MSSS Concrete Geometric Description. (a) Mean Bridge Elevation, where A = abutments, EB = elastomeric bearings, B = transverse beams, C = columns, F foundations. (b) Bridge Bent Cross Section and Model Representation, (c) 3D FE Model, (d) Transverse Beam Fiber Section and (e) Column Fiber Section.

Seismic Hazard Simulation

To evaluate the seismic hazard, ground motion time histories (GMTHs) representing the uncertainties related to the seismicity are necessary. Since the available records of real ground motion in Quebec are not sufficient, synthetic acceleration time histories were used. The GMTHs developed by Atkinson (2009) using the stochastic finite-fault method to match the NBCC (2005) for a range of Canadian sites for eastern Canada and soil site Class C were chosen. The ground motions were simulated for moment magnitudes $M = 6$ at fault distances from 10 to 30 km, and for $M = 7$ at 15 to 100 km. The $M = 6$ events match the short period end of the UHS, while the $M = 7$ events match the long-period end of the UHS. For each magnitude there were two fault distance ranges: $M = 6$ at 10 to 15 km (M6 set 1) and 20 to 30 km (M6 set 2); and $M = 7$ at 15 to 25 km (M7 set 1) and 50 to 100 km (M7 set 2). These records were used combining two horizontal components applied in the longitudinal and transverse directions of the bridge. The direction of the component can be either transverse or longitudinal, no angle of approach is considered, and each component can be applied in the longitudinal and transverse direction. Thus, there are $2 \times 4 \times 15$ bidirectional earthquakes for a total of 120 GMTHs. Figure 4 shows the GMTHs distribution of the peak ground acceleration (PGA) and their mean response spectrum.

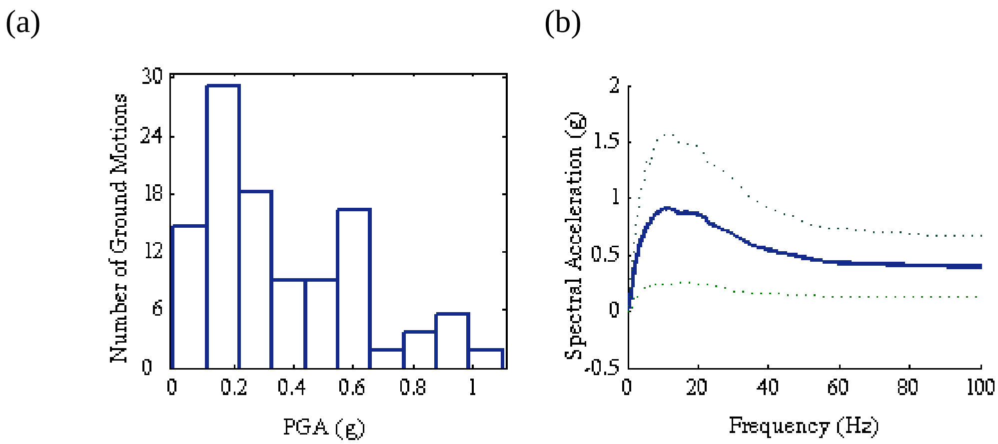

Figure 3. GMTHs Series. (a) PGA distribution of ground motions and (b) Mean and Standard Deviation Response Spectrum.

Bridges Simulation

To enable the development of probabilistic bridge models the uncertainties due to geometry, material and other variations have to be considered. The parameters involved in the geometry uncertainties are the deck length and width, the ratio between the larger span length and total length and the vertical under-clearance or the height of the columns. The material uncertainties include the concrete and steel yielding strengths and the stiffness of the abutments, bearings and foundations. Other uncertainties include the variability in the mass of the bridge, damping, the gaps between decks and decks and abutments. Due to the number of variables involved in the problem, it is logical to question if the variations of all parameters have any significant impact upon the response of the structure. Answering this question for each parameter will dictate whether its inherent variation must be explicitly considered or if it may be neglected in the probabilistic bridge model. To evaluate the significance of each parameter involved in the problem an analysis of variation (ANOVA) or a sensitivity analysis was performed. The main parameters effects were considered neglecting the effects of their combination. Due to their importance and sensitivity, the parameters involved in the geometric uncertainties were treated as macro variables using statistic blocks, due to the nature of a fractional factorial analysis applied for the ANOVA, the number of different blocks had to be a 2^x, in the case of the MSSS Concrete bridge class the number of blocks was defined as $2^3 = 8$ blocks.

The parameters were represented in a probability density function (PDF). Depending on the nature of the parameter, they were represented with normal, log-normal or uniform distributions. The sampling method for all the variables was the Latin hypercube sampling (LHS). For the geometry parameters, 8 samples were combined in blocks, assuring the representation of real bridges. For the rest of the parameters, the number of samples was defined by the series of GMTHs, therefore there were 120 samples for each parameter. The Latin hypercube sampling (LHS) is a stratified-random procedure that provides an efficient way of sampling variables from their distributions (MacKay 1979). Unlike simple random sampling, this method ensures a full coverage of the range of each variable by maximally stratifying each marginal distribution. Thus, the sampling results in 120 earthquake-bridge samples for the MSSS Concrete bridge class. The probability density function (PDF) and their properties for the parameters having a significant influence on the response of this bridge class are shown in Table 1.

Table 1. MSSS Concrete Significant Parameters

MSSS Concrete Parameter	**Type of PDF**	**Median or Upper**	**Std. Dev or Bottom**	**Units**
Total Length	Lognormal	4.13	0.53	m
Total Width	Lognormal	2.52	0.30	m
Column Height	Lognormal	1.62	0.44	m
L_m/L_t	Normal	0.40	0.10	-
Concrete Strength	Normal	33.8	4.3	MPa
Steel Strength	Lognormal	463	6.13	MPa
Bearing Stiffness	Uniform	50	150	%
Abutment Passive Stiffness	Uniform	50	150	%
Abutment Active Stiffness	Uniform	50	150	%
Deck Gap	Normal	0	50.8	mm
Damping	Normal	1.3	0.00607	%
Mass Variability	Uniform	90	110	%

Damage Simulation

The capacities of the bridge components are defined in terms of limit state models. Traditionally, these limit states for bridge components have been defined by qualitative damage states such as slight, moderate, extensive and complete presented in HAZUS (2003). With the definition of qualitative damage states, quantitative limit states must be derived. These limit states should use the same metrics and components as the ones that will be used for the probabilistic seismic demand models PSDMs. A general method to define limit states is a mechanics-based approach. The median values of the prescriptive limit states previously used in the work by Choi (2004) are used in this study for the definition of the limit states for the elastomeric bearings and abutments. The evaluation of the columns damage is in terms of the column ductility ratio using displacement ductility as a limit state (Hwang, 2001). The limit states for all considered components are given in Table 2.

Table 2. Quantitative Limit States

Component	Slight	Moderate	Severe	Complete
column (disp. ductility)	1.00	1.58	3.22	6.84
abutment_passive (mm)	13	50	76	152
abutment_active (mm)	13	50	76	152
abutment_transverse (mm)	13	50	76	152
elastomeric_bearing_long (mm)	24.5	75	200	250
elastomeric_bearing_trans (mm)	24.5	75	200	250

Fragility Curves

(a) Column (b) Elast. Bearing (c) Abutment

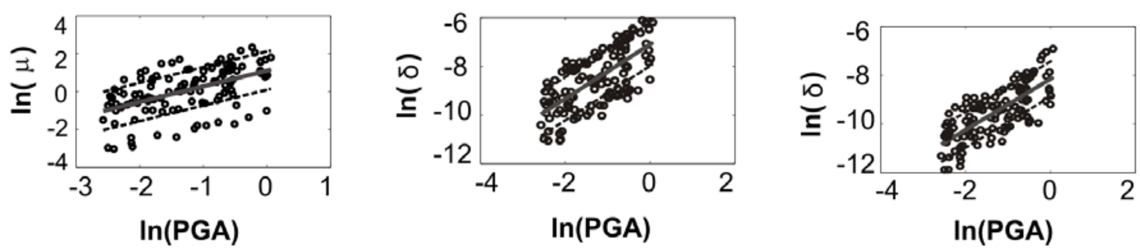

Figure 4. Regression of the probabilistic seismic demand models of the MSSS Concrete Girder for: (a) Columns, (b) Elastomeric Bearing and (c) Abutment.

The fragility curves developed in this study were based on nonlinear response history analyses. The method includes several major steps. First, a bridge is represented by an analytical model, which includes the inelastic behavior of the appropriate components (i.e. bearings, columns and abutments). Second, earthquake input motions for various characteristic magnitudes and epicenter distances were chosen. Third, uncertainties in the modeling of seismic source, path attenuation and bridge components are quantified to establish a set of earthquake-bridge samples. Fourth, for each earthquake-bridge sample, a nonlinear time history analysis is performed with OpenSees. Using predetermined damage indices, a damage state is assigned to each component of the bridge. Finally, using a probabilistic seismic demand model obtained by regression analysis on the simulated damage results in OpenSees, the component fragility curves can be developed. The seismic demand is expressed as in Equation 1, where S_D is the mean for the demand, a and b are unknown regression coefficients, and x is the ground motion intensity parameter (typically PGA or S_a). Figure 4 shows the results of the probabilistic seismic demand model for the components of the MSSS concrete girder bridge in terms of PGA having R^2 and the mean values in the range of 0.27-0.71 and 0.49-1.01, respectively.

$$\ell n(S_D) = a \cdot \ell n(x) + b \tag{1}$$

A fragility curve describes the probability of reaching or exceeding a damage state as a function of a chosen ground motion intensity parameter (PGA or S_a). The peak ground acceleration is proven to be a good intensity measure (Padgett, 2008). In this study, four damage states were quantified in terms of the column displacements ductility and the deformations of the elastomeric bearings and abutments. The probability that the demand on the structure exceeds the structural capacity can be computed as shown in Equation 2.

$$P[LS|IM] = P[D \geq C|IM] = \Phi\left(\frac{\ell n(S_D / S_C)}{\sqrt{\beta_{D|IM}^2 + \beta_C^2}}\right) > 1.0 \tag{2}$$

Where S_C is the mean and β_C is the logarithmic standard deviation for the capacity, S_D is the mean and β_D is the logarithmic standard deviation for the demand, and Φ is the standard normal distribution function. It should be noted that even though the probabilistic seismic demand model is performed for a PGA range of 0.08-0.97 g, assuming a log-normal fit for the fragility curves allows extrapolation beyond this range within reason. However, this results in fragility curves for the component level. The fragility of the complete bridge is performed through a crude Monte-Carlo simulation using joint probabilistic demand models and the limit state models. This simulation does not include variance reduction sampling. It is intended to integrate the joint probabilistic seismic demand models (JPSDM) which is the combination of the PSDMs using their mean, standard deviation and a correlation matrix, over all possible failure domains. This is done by selecting a value of the intensity measure (IM), which in this case is PGA. At this level of IM, 10^5 samples are taken from both the demand and capacity sides. Next, an estimate of the probability that the demand exceeds the capacity at that IM level is obtained. This step is repeated for increasing levels of the IM until a curve is defined (Figure 5).

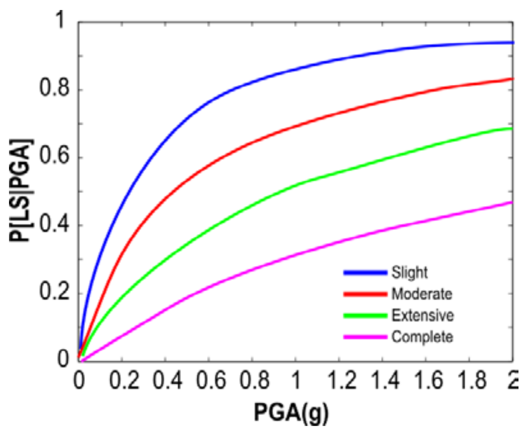

Figure 5. Fragility Curve for the MSSS Concrete Girder.

This figure shows the fragility curves defined for the MSSS Concrete bridges using PGA as an intensity measure and 4 damage states defined as slight, moderate, severe and complete. It can be seen that the results were extrapolated for PGA up until 2 g. For a PGA of 1 g these bridges have almost 90% of probability of exceeding the slight limit state, 70% of probability of exceeding the moderate limit state, 50 % of probability of exceeding the extensive limit state and less than 30% of probability of collapse. Since, ground motions of this intensity are probable to occur in Quebec, the fragility curve shows that almost one half of the MSSS Concrete Girder Bridges will not probably be safe to be used after such an event.

Conclusions

This study presents the development of fragility curves for a class of bridges commonly found in Quebec. The fragility curves that are presented were generated analytically using the Monte Carlo Method. The components included are the columns, bearings abutments and the foundations. These curves maybe improved when more information is collected on the individual responses of the various bridge components in both the longitudinal and transverse directions. These fragility curves can be used in determining the potential losses resulting from earthquakes and can be used to assign prioritization for retrofitting. The median peak ground acceleration for slight, moderate, extensive and complete damage for the MSSS concrete girder bridge is approximately 0.213, 0.433, 0.950 and 2.296 g, respectively. These results show that the multi-span simply supported concrete girder bridges are vulnerable bridge types for about 1.0 g peak ground acceleration for the various damage states, however, a large number of collapse is not to be expected.

Acknowledgments

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