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Generalized Fragility Relationships with Local Site Conditions for Probabilistic Performance-based Seismic Risk Assessment of Bridge Inventories

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Abstract: The current practice of detailed seismic risk assessment cannot be easily applied to all the bridges in a large transportation networks due to limited resources. This paper presents a new approach for seismic risk assessment of large bridge inventories in a city or national bridge network based on the framework of probabilistic performance based seismic risk assessment. To account for the influences of local site effects, a procedure to generate site-specific hazard curves that includes seismic hazard microzonation information has been developed for seismic risk assessment of bridge inventories. Simulated ground motions compatible with the site specific seismic hazard are used as input excitations in nonlinear time history analysis of representative bridges for calibration. A normalizing procedure to obtain generalized fragility relationships in terms of structural characteristic parameters of bridge span and size and longitudinal and transverse reinforcement ratios is presented. The seismic risk of bridges in a large inventory can then be easily evaluated using the normalized fragility relationships without the requirement of carrying out detailed nonlinear time history analysis.

Keywords: Bridges, concrete structures, fragility relationships, performance-based earthquake engineering, seismic risk and vulnerability.

Introduction

Evolving challenges in earthquake engineering have motivated earthquake engineers and researchers to improve existing practices and develop new approaches and methodologies for better design, more accurate risk assessmentand more effective retrofit of structures. In current practice of seismic risk assessment, there are limitations in the accuracy of predicting performance particularly in associating seismic response behaviour with seismic performance and losses that may result as a consequence of the damage sustained. Recognition of these limitations has led to the development of the concept of performance-based design (PBD) in earthquake engineering. As opposed to prescriptive requirements in conventional design standards, the goal of the performance-based earthquake engineering (PBEE) is to meet specific performance objectives, such as those defined in terms of displacement, drift, ductility, and material behavior under specified design earthquake events, by allowing the engineers the flexibility to consider various design options and creative solutions.

The practice of performance-based earthquake engineering involves the prediction of damage states and calculation of the probability of reaching a given damage state under particular seismic events [1]. In PBEE, the methodology encompasses four standard phases [2]. The first phase, hazard analysis, is seismic hazard analysis of the site. The second phase, demand analysis, is to determine its responses to a range of seismic loading as representative of the seismic hazard at the site. The third phase is damage analysis, in which the probability of occurrence of a particular damage level is assessed. The final phase, loss analysis, is a review of potential economic losses as a result of the expected damage levels.

The objectives of the present study are: (1) To develop a new approach for seismic risk assessment of large bridge inventories in a city or region or national bridge network based on the framework of probabilistic performance-based seismic risk assessment, and (2) To investigate the influence of the local site conditions on seismic vulnerability of bridges incorporation of microzonation.

Probabilistic Performance-Based Seismic Assessment Methodology

Performance-based design methodologies in earthquake engineering have been developed by several research groups. The document FEMA-356 is the first publication that describes the approach and formulation of probabilistic performance-based earth-

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quake engineering (PBEE). Researchers at the Pacific Earthquake Engineering Research Center (PEER) extend the procedure developed in FEMA-356 to a performance-based framework for the seismic design and assessment of buildings [3]. Recently, researchers have initiated the development of a probabilistic performance-based approach for the design and assessment of bridges [3-6].

Hazard Analysis

The first step of probabilistic performance-based seismic risk assessment is the seismic hazard analysis, which identifies the probability of occurrence of different seismic events of varying intensity at the site of the evaluated structure. In the determination of appropriate seismic hazards for structural design or assessment, it is important to select a representtative intensity measure (IM) for the site's seismic risks that minimizes uncertainty in the probability analysis. For bridge structures, the first mode 5% damped elastic spectral acceleration of the structure (S_a (T_1)), the Peak Ground Acceleration (PGA) and the Peak Ground Velocity (PGV) are commonly selected as IMs for probabilistic performance-based evaluations [7]. In the PBEE methodology, the seismic hazard analysis is identified as development of a site seismic hazard curve that relates the mean annual frequency of occurrence (λ_{IM}) to intensity measures [3]. From past studies [8,9,10], the equation of the hazard curve is commonly assumed to have a power-law form with two unknown parameters (k and k_0) in the range of the ground motions investigated as shown in Equation 1.

$$\lambda_{IM} = k_0[IM]^{-k} \quad (1)$$

Demand Analysis

The second step of performance-based analysis is to relate this hazard to structural response in the form of a demand model. The objective of a demand model is to describe the probable effect of site-specific ground motions on a structure in terms of engineering demand parameters (EDPs) such as drift ratio, displacement ductility, and plastic rotation [1]. A relation between IMs and EDPs can be derived by using the structural responses obtained from structural analysis of the design structures subject to the earthquake loadings of the site specific ground motion suite. Some studies have shown that the most efficient and practical demand model is the relationship between first mode spectral acceleration S_a (T_1) and drift ratio [3,11]. Similar to the seismic hazard model, the distribution of EDPs conditioned on IMs is assumed to have a lognormal distribution of the form shown in Equation 2 [3]. Based on this relation, which is referred to as the interim demand model, the probability of occurrence for the representative EDP of the evaluated bridge for a given Intensity Level of seismic hazard can be written in the form shown in Equation 3 [7].

$$\ln(\widehat{EDP}) = A + B\ln(IM) \text{ where } \widehat{EDP} \text{ is the median EDP} \quad (2)$$

$$P\,(EDP/IM) = 1 - \Phi\left[\frac{\ln(EDP) - A - B\ln(IM)}{\sigma_{\ln(EDP|IM)}}\right] \quad (3)$$

Damage Analysis

In the damage analysis phase, the structural response associated with different hazard levels is linked with the probable damage induced. To establish this link, first a relationship is estabilished between the probability of different damage states occurring and different structural response levels. This relationship, the interim damage model, and is derived through observed, experimental, or analytical estimates of damage. Once this relationship is obtained, it can be combined with the demand model developed in the preceding step to form the damage model which gives the probability of damage of a given earthquake event. The objectives of interim damage models are to estimate the probable damage state of a structure in terms of damage measures (DMs), under a given level of structural response described in EDP. In performance-based design methodology, DMs are usually taken as discrete, rather than continuous quantities, defined as observations of the onset of certain damage states [7]. Depending on the relationship used, examples of damage states of reinforced concrete columns include cracking, spalling, longitudinal bar buckling and transverse reinforcement fracture.

In the case of continuous damage measures, such as loss of lateral load carrying capacity, the median relationship between EDP and DM Equation 4 and the associated dispersion given by the standard deviation of the model error ($\sigma_{DM/EDP}$) completely define the continuous damage model. Discrete damage models can be simplified to act as continuous damage models when the coefficients of variation for each of the discrete damage states are approximately equal. Once an interim damage model has been developed, the probability of occurrence of damage state for a given level of seismic hazard can be calculated as shown in Equation 5 [7].

$$\ln(\widehat{DM}) = C + D\ln(EDP) \quad (4)$$

$$P(DM/IM) = 1 - \Phi\left[\frac{\ln(DM) - (C + DA + DB\ln(IM))}{\sqrt{D^2\sigma^2_{\ln(EDP|IM)} + \sigma^2_{\ln(DM|EDP)}}}\right] \quad (5)$$

Loss Analysis

The final stage of a PBEE assessment is the loss analysis stage. This stage is where, based on the preceding models, the probable losses are evaluated in terms of decision variables (DVs). Typical DVs include: repair cost, downtime, repair time, and loss of life [7]. The objective of loss analysis is to provide information on impact or consequence of potential earthquake damage which are of immediate concerns to emergency managers, recovery planners, and structural engineers after an earthquake.

The decision variables (DVs) for bridges can be separated into two categories: functional DVs and repair DVs. Functional DVs describe the post-earthquake operational state of the bridge such as required lane closures, reduction in traffic volume, or complete bridge closure. The repair DVs include time and cost of bridge repair and restoration. Following the same relationships discussed in the earlier sections, an interim loss model, relating DV to DM, can be developed with the form shown in Equation 6, where $\widehat{DV}$ represents the median DV. Once this interim model is developed, it can be combined with the hazard, demand and damage models to determine the probability of occurrence of decision variables at a given level of intensity level of seismic hazard as shown in Equation 7.

$$\ln(\widehat{DV}) = E + F\ln(DM) \tag{6}$$

$$P(DV|IM) = 1 - \Phi\left[\frac{\ln(dv^{LS}) - (E + FC + FDA + FDB\ln(IM))}{\sqrt{DF^2\sigma^2_{EDP|IM} + F^2\sigma^2_{DM|EDP} + \sigma^2_{DV|DM}}}\right] \tag{7}$$

Application to Canadian Bridge Inventory

Studies by Waller [5] have shown that bridges with similar characteristics and structural properties, such as degree of skew, span length, continuity, reinforcement ratio, and other structural configuretions and design details, can be expected to respond similarly during seismic events and have similar vulnerability to earthquake damage. Bridges constructed during a particular period of time typically have similar design details and thus similar structural properties because their design and construction are based on similar design codes and standards.

In collaboration with the City of Ottawa, ten bridges are selected as the sample bridge inventory in the present study. This sample inventory includes bridges constructed between 1966 and 2005 of different geometric layouts. Two bridges are selected as representative of the bridge inventory for the derivation of the fragility relationships between structural responses and damage states in the new probabilistic performance-based seismic evaluation methodology by detailed analysis. The first selected bridge on Blair Road, as shown in Figures 1 to 2, is a continuous four span concrete bridge with a prestressed hollow core deck. It crosses Highway 417 in Ottawa. The Blair Road Bridge is straight in alignment and has four columns in each bent. The deck is supported on fixed bearings at the middlebent and on expansion bearings at the other bents and abutments. To account for the influence of the field operational conditions of the bridge on its seismic behaviour, two boundary condition scenarios are considered for the Blair Road Bridge assuming the expansion bearings are totally free to move and another case the expansion bearings become totally fixed due to restriction by friction and road debris. The second selected bridge on Terminal Avenue crossing Alta Vista Drive, as shown in Figures 3 to 4, is a continuous two span concrete bridge with a prestressed hollow core deck similar to the first bridge. The deck is supported on fixed bearings at the bent and on expansion bearings at the abutments. The Terminal Avenue Bridge is straight in alignment and supported by a two-column bent.

Hazard Analysis

As indicated earlier, the primary task of seismic hazard analysis in the PBEE methodology is development of a seismic hazard curve at the site.

Selection of ground motions. From past earthquake events, seismic hazards in eastern Canada are characterized by infrequent but damaging earthquakes. Although there was a recent earthquake near Ottawa (Val-des-bois, 2010) with magnitude estimated at M_w 5.5 [12], there is still not enough ground motion data in regions of eastern Canada around the Ottawa areas. Therefore, artificially simulated ground motion records are used as input excitations for time history analysis of the selected representative bridges. Numerous studies have shown that simulated records and actual earthquake records are functionally equivalent, from both linear and nonlinear perspectives [13]. For this study, the artificial time histories are generated to match the uniform hazard spectrum (UHS) of Ottawa as specified by the Geological Survey of Canada (GSC) [14].

Probabilistic seismic hazard curves. To generate the probabilistic hazard model, it is necessary to consider different probable hazard events for design of structures to meet different performance objecttives. In the formulation of the new probabilistic performance-based seismic risk assessment methodology, events of high, moderate and low probability are considered, which correspond to the probability of occurrence of 40%, 10% and 2% in 50 years, respectively [3, 14].

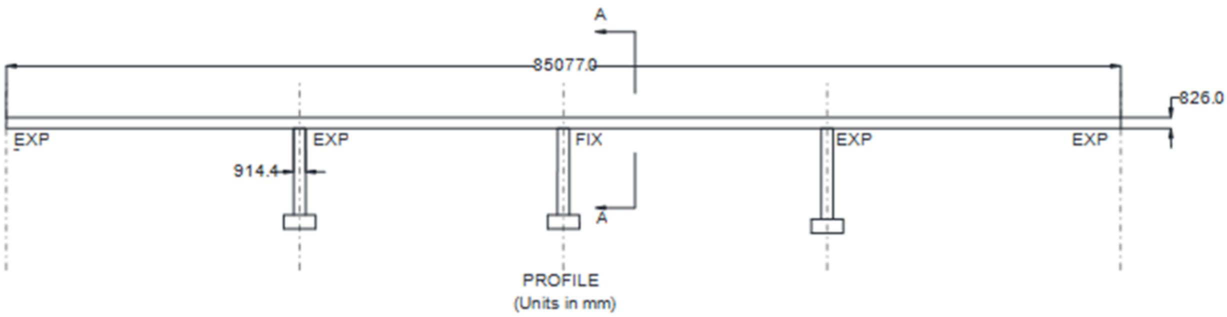
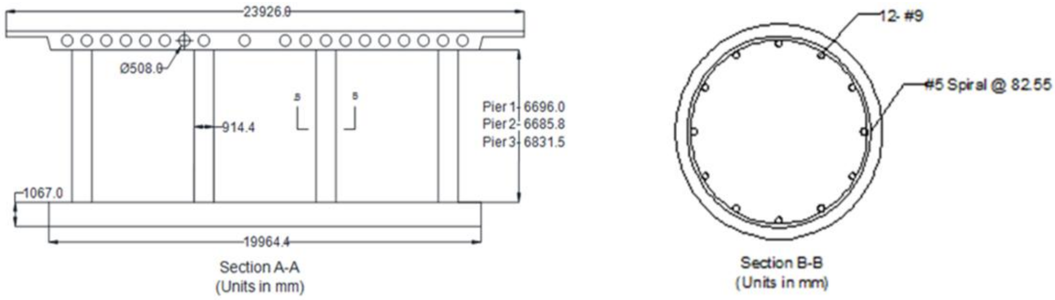


Figure 1. Blair Road Bridge Profile



(a) (b)

Figure 2. (a) Cross Section of Blair Road Bridge Super Structure; (b) Cross Section of Blair Road Bridge Column

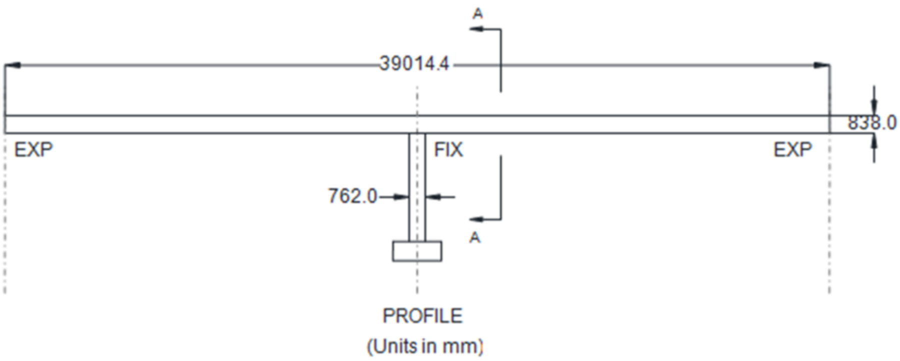
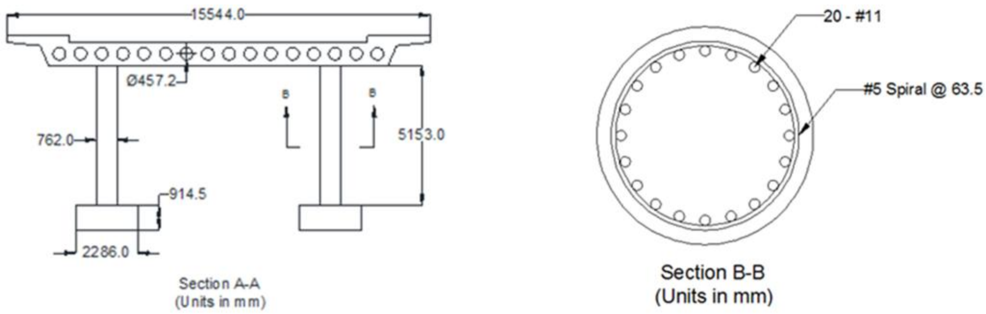


Figure 3. Terminal Avenue Bridge Profile



(a) (b)

Figure 4. (a) Cross Section of Terminal Avenue Bridge Super Structure; (b) Cross Section of Terminal Avenue Bridge Column

The uniform hazard spectra (UHS) on firm ground condition (Site Class C) [15] at three hazard levels are obtained from the Geological Survey of Canada [14]. From the UHS on firm ground condition, the UHS curves on other ground conditions (Site Classes A, B, D and E) [15] at the probability level of 2% exceedance in 50 years, are obtained by using the site factors F_a and F_v reported in the National Building Code of Canada [NBCC 2010] [15]. The resulting 2% in 50 years UHS curves for the different site conditions are shown in Figure 5. Since the values of the site factors F_a and F_v are not available for other hazard levels (10% and 40% in 50 years), the UHS for all site conditions at the moderate and low hazard levels are derived by soil amplification analysis using the computer program ProShake [16]. Figures 6(a) and 6(b) show two approaches for the derivation of the spectra of 10% exceedance in 50 years from soil amplification analysis results. In the first approach, the spectral acceleration value in the period range of 0.04 to 0.2 sec is assumed to be constant, taken as the spectral acceleration value at 0.04 sec from the soil amplification analysis results. In the second approach, the constant spectral acceleration value over the same period range is taken as the average spectral acceleration value calculated from the maximum and minimum values obtained in the soil amplification analysis. The maximum envelope from the two approaches is used in the construction of the UHS curves for 10% and 40% in 50 years as shown Figures 7(a) and 7(b). These modeled curves follow the same trend as the 2% in 50 years UHS curves derived based on the site factors F_a and F_v.

Using the UHS curves shown in Figures 5, 7(a) and 7(b), the first mode spectral acceleration values of the representative bridges can be obtained by considering the fundamental period (T_1) of the representative bridge. To relate the intensity measure as defined by the first mode spectral acceleration of the representative bridge with the likelihood or probability of occurrence of the seismic hazard, site specific hazard curves are derived by plotting the mean annual frequencies as a function of the estimated first mode spectral acceleration values. Figure 8 shows the hazard curves obtained for the Blair Road Bridge with free expansion bearing case using UHS curves on different ground conditions. The hazard curves in Figure 8 follow the form given by Equation 1.

Demand Analysis

The next step in the new PBEE seismic assessment methodology is to determine the impact of the seismic hazard on the behaviour and performance of the representative bridges. Nonlinear time history analysis [17] of the representative bridges has been carried out to evaluate the engineering demand parameters (EDPs) by using 30 different simulated time histories that match the three target response spectra (10 per hazard level) of each site class. The Newmark's Average Acceleration method (γ = 0.5 and β = 0.25) is employed in time step integration. The drift ratio of the bridge pier is selected as the demand parameter.

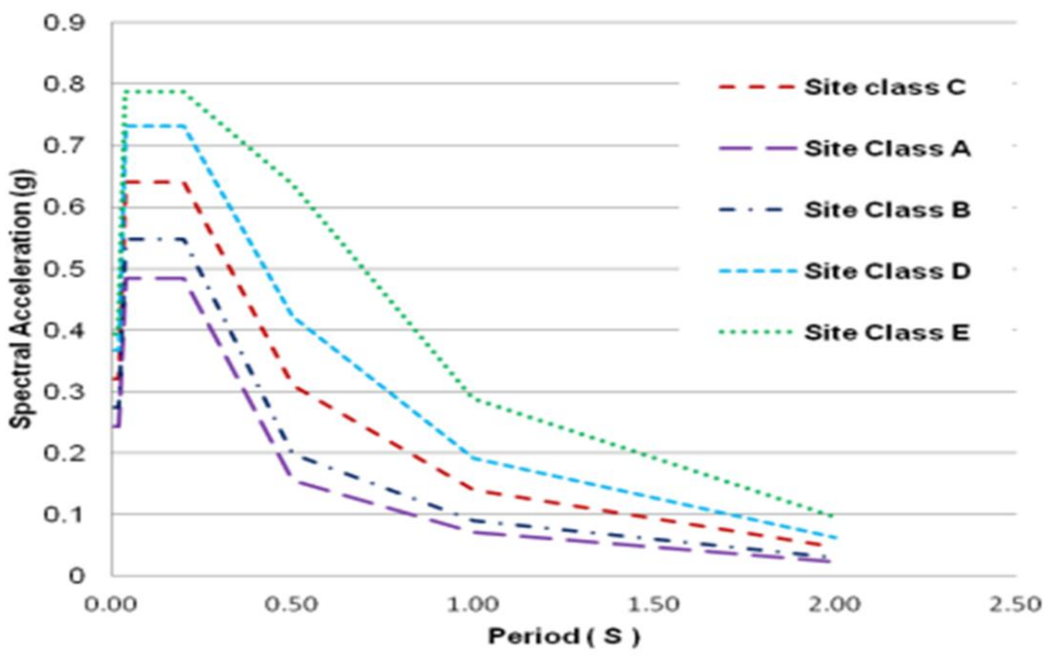
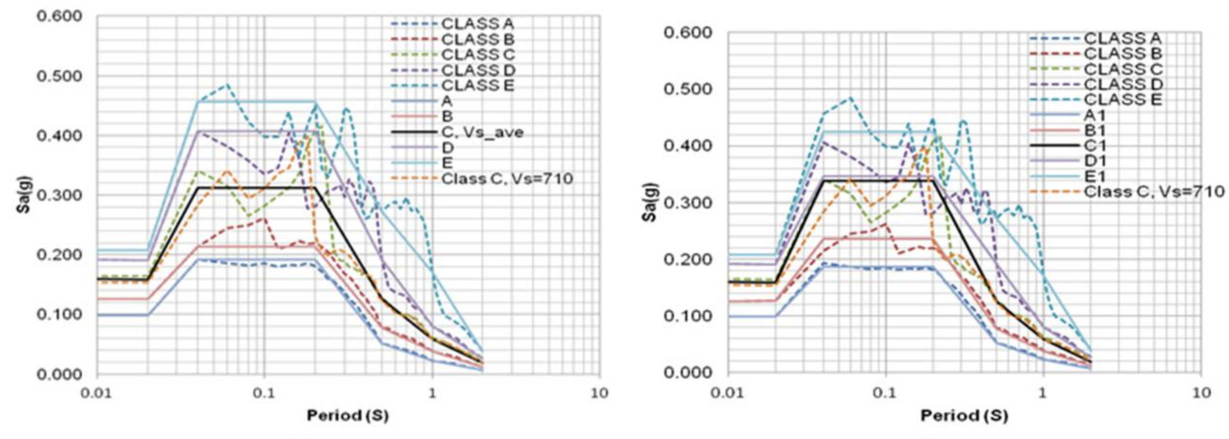
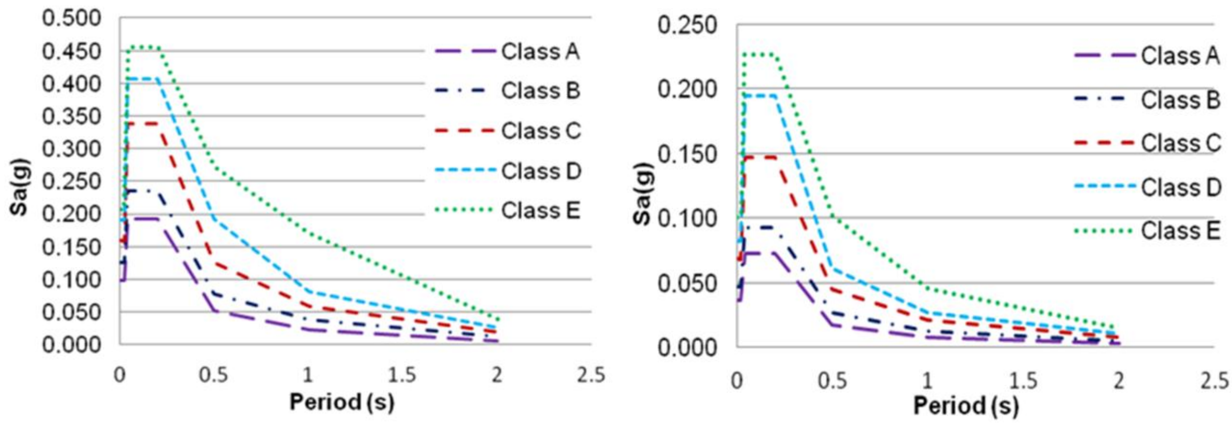


Figure 5. UHS Curves on Different Site Conditions for Ottawa at 2% Exceedance in 50 Years



(a) (b)

Figure 6. (a); and (b) Two Scenarios of UHS Curves on Different Site Conditions at 10% Exceedance in 50 Years from Soil Amplification Analysis



(a) (b)

Figure 7. (a) UHS Curves on Different Site Conditions for Ottawa at 10% Exceedance in 50 Years from Soil Amplification Analysis, and (b) UHS Curves on Different Site Conditions for Ottawa at 40% Exceedance in 50 Years from Soil Amplification Analysis

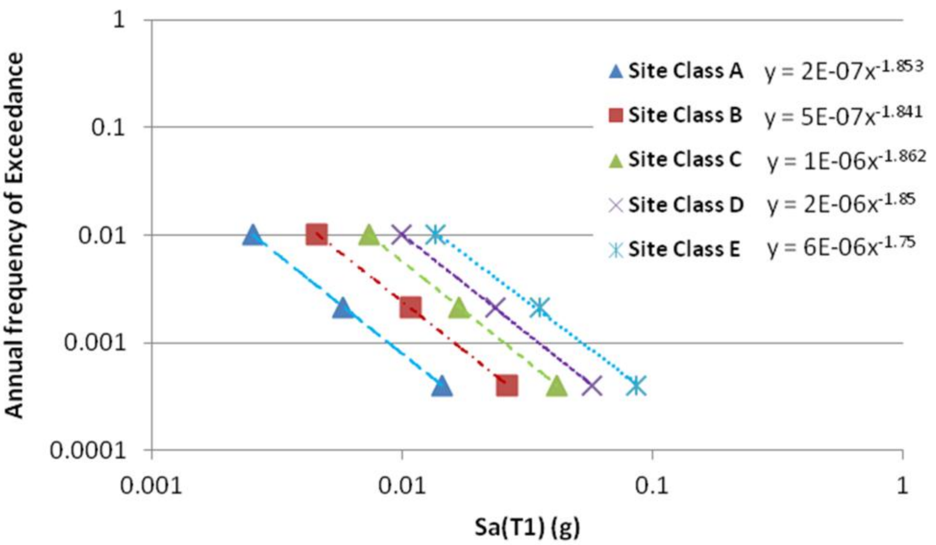


Figure 8. Hazard Curves for Blair Road Bridge with Free Expansion Bearing Case on Different Site Conditions

The most critical drift ratio is calculated from the maximum lateral displacement at the top of the shortest pier of the bridge. Using the calculated drift ratios of the bridge piers, demand models in terms of drift ratio can be developed for each bridge. Figure 9(a) shows the demand curves developed for the Blair Road Bridge with free expansion bearing case for all site classes. Assuming a lognormal distribution and applying least square fit to the results obtained as discussed earlier, a best fit line, as shown in Figure 9(a), relates the resulting drift ratio as a function of the first mode spectral acceleration $Sa(T_1)$. The resulting regression coefficients A and B can be determined from the demand models given in Equation 2. The resulting probability of occurrence of drift ratio of the representative bridge due to a specific seismic hazard level at the bridge site can then be calculated using Equation 3. The variation of probability of occurrence with drift ratio for the Blair Road Bridge with free expansion bearing case for Site Class C is shown in Figure 9(b). Results for other site conditions are similar. They are not presented here due to space limitation.

Damage Analysis

Damage fragility curves can be developed using experiment-based damage models [1, 18] or damage models developed from observation information on bridge damage states from major earthquakes [19]. The damage model incorporated in this study is the mathematical model developed by Berry and Eberhard [1] which is based on the seismic performance database (SPD) of over 400 reinforced concrete column tests of varying material and structural properties worldwide [20]. Several failure mechanisms, such as concrete crushing, spalling, longitudinal bar buckling, longitudinal bar fracture, spiral fracture, and loss of axial load capacity, can be considered as damage states for the development of the fragility relationships. In this study, concrete spalling and longitudinal bar buckling are considered as the damage states. The damage state of concrete cover spalling represents the initiation of failure, whereas the failure modeof the longitudinal bar buckling is considered to represent the start of more substantial damage with serious consequent effect on the seismic load resistant capacity of the bridge. Based on observations on test results of cover spalling and bar buckling during cyclic lateral load tests of reinforced concrete columns, Berry and Eberhard [1] have developed a structural response model that links the structural characteristic parameters to engineering demand parameters. Equations 8 and 9 show the relationships developed for drift ratio at the initiation of concrete spalling and longitudinal bar buckling for both rectangular and spiral reinforced concrete columns,

$$\frac{\Delta_{\text{spall calc}}}{L}(\%) = 1.6\left(1-\frac{P}{A_g f'_c}\right)\left(1+\frac{L}{10D}\right) \tag{8}$$

$$\frac{\Delta_{\text{bb calc}}}{L}(\%) = 3.25\left(1+k_{e\,bb}\rho_{eff}\frac{d_b}{D}\right)\left(1-\frac{P}{A_g f'_c}\right)\left(1+\frac{L}{10D}\right) \tag{9}$$

Where P is the axial load, Ag is the gross section area, f'_c is the concrete compressive strength, L is the distance from point of fixity to point of inflection, D is the column diameter, $k_{e\,bb}$ is taken as a constant value 150 for spiral reinforced concrete column, $\rho_{eff} = \frac{\rho_s f_{ys}}{f'_c}$ is the volumetric transverse reinforcement ratio, ρ_s is the transverse reinforcement ratio, f_{ys} is the yield strength of transverse reinforcement, and d_b is the longitudinal bar diameter.

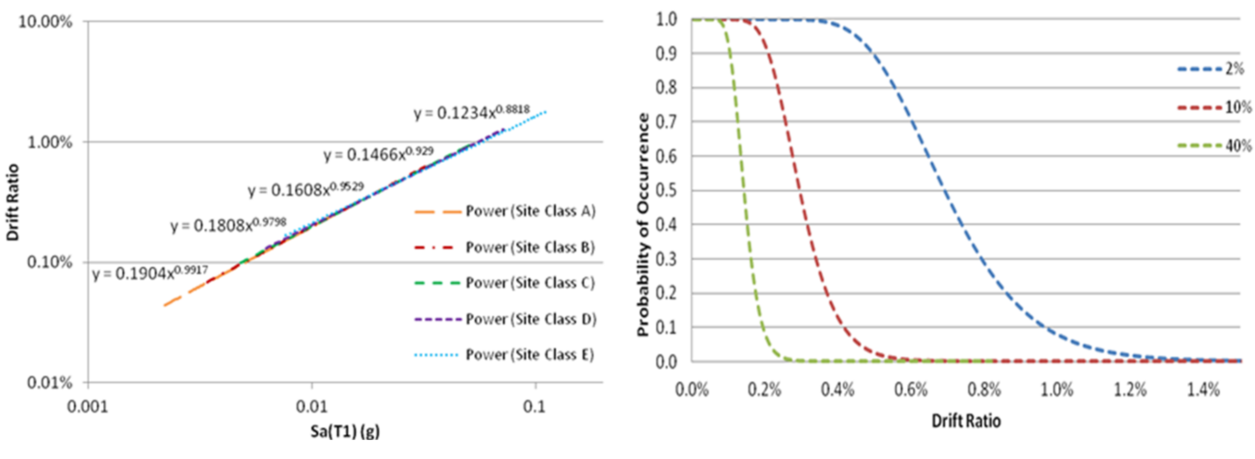


Figure 9. (a) Demand Curves for Blair Road Bridge with Free Expansion Bearing Case on Different Site Conditions; (b) Probable Drift Ratio of the Blair Road Bridge with Free Expansion Bearing Case for Site Class C

In addition, Berry and Eberhard [1] have compared estimated values from Equations 8 and 9, at which damage is expected to occur to the actual damage occurrence demand of a large group of columns in the Structural Performance Database (SPD). Based on the comparison study, they have developed general fragility curves (cumulative probability of cover spalling and bar buckling as function of $\Delta_{\text{damage}} / \Delta_{\text{damage-cal}}$) that can be easily converted to fragility curves for specific columns.

To estimate the probability of concrete cover spalling and bar buckling for the piers of the representative bridge investigated, the drift ratio at the onset of cover spalling and onset of bar buckling are estimated from Equations 8 and 9. The general fragility curves are adjusted by multiplying the estimated drift ratios. The fragility functions for cover spalling and bar buckling for the typical Blair Road Bridge on firm ground condition are shown in Figure 10(a). The relationships of the probability of demand for a given earthquake event and probability of damage for a given demand are combined to obtain relationships for the probability of damage for a given earthquake event by substituting the results from the demand and damage analysis in Equation 5. The resulting probability of damage for a given seismic hazard level for the Blair Road Bridge with free expansion bearing case for Site Class C is shown in Figure 10(b). Results for other site conditions are similar. They are not presented here due to space limitation.

Fragility Evaluation of Sample Bridge Inventory

As discussed earlier, the bridges constructed decades ago tend to be vulnerable during earthquakes. Therefore, it is important to carry out seismic risk assessment of bridges that were constructed using obsolete design standards. However, it is not realistic to carry out nonlinear time history analysis of all the bridges in a large bridge transportation network inventory due to the requirement of vast amount of engineering efforts on modelling and analysis. In the new probabilistic performance-based methodology, the developed fragility relationships of representtative bridges are used through a normalization process to generate generalized fragility relationships that can be used for evaluating the seismic vulnerability and risk of other bridges with similar structural characteristics. The advantage of this new assessment methodology is that evaluation of the seismic vulnerability and risk of large number of bridges with similar characteristics does not require detailed structural modelling and nonlinear time history analysis.

The basic premise of the assessment methodology developed in the present work is that structural performance of bridges is related to structural characteristic parameters [5]. For this work, three structural characteristic parameters are considered: (1) Pier longitudinal reinforcement ratio (ρ_L), (2) Pier transverse reinforcement ratio (ρ_S), and (3) Span over pier height ratio (Span/L). These structural characteristic parameters of bridges in the sample inventory are presented in Table 1. In order to develop a relationship between damage probabilities with the corresponding structural charactерristic of bridge columns to account for the differences in size and configuration of different bridge structures, effective or normalized structural characterristic parameters are obtained based on modification using the tributary lateral load resisted by the bridge column.

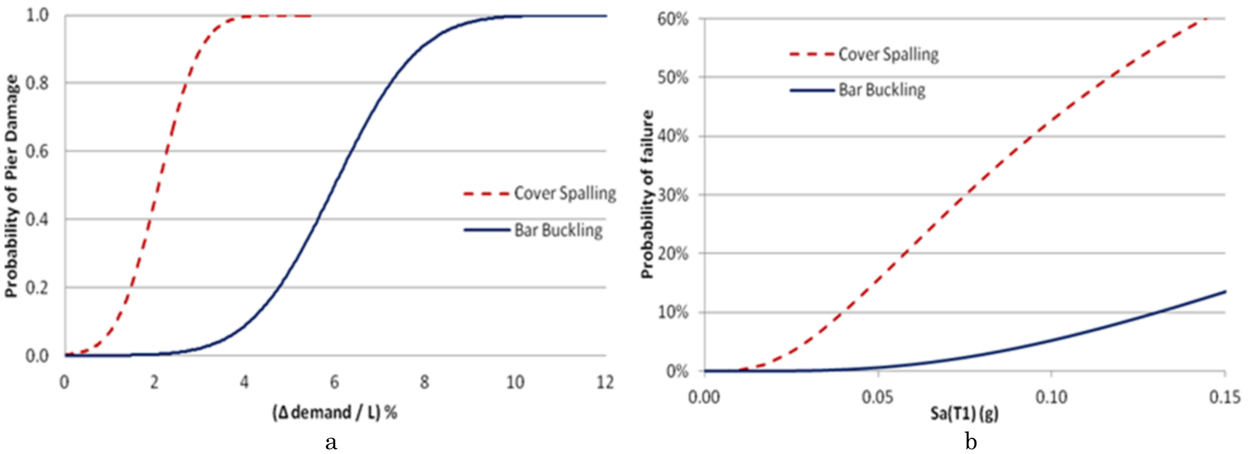


Figure 10. (a) Damage Fragility Curve Based on Material and Structural Properties of the Typical Blair Road Bridge Pier; (b) Probability of Failure Given a Level of Seismic Event for Blair Road Bridge with Free Expansion Bearing Case (Site Class C)

In this investigation, the representative bridge inventory includes a variety of bridges of different geometric layout. Therefore, effective structural characteristics parameters ($\rho_L{}^*, \rho_S{}^*$ and $\frac{\text{span}^*}{L}$) are determined for comparing the seismic load experienced by each pier. Equations 10 to 13 are used to evaluate effective structural characteristic parameters of bridges in the sample inventory. The actual and evaluated effective characteristic parameters of the sample bridge inventory are presented in Table 1. The probability of failure of the representtative bridges at different hazard levels on various ground conditions are normalized by the effective structural characteristic parameters. The normalized fragility relationships of the representative bridges at 2% exceedance in 50 years on various ground conditions are shown in Figures 11 to 13. Results for other hazard levels are similar. They are not presented here due to space limitation. These figures show the probability of occurrence of the damage states such as cover spalling and bar buckling decreases with increasing longitudinal and transverse reinforcement ratios for all site classes. However, the slopes of the curves become steeper from Site Classes A to E. In contrast, the variation of the probability of cover spalling and bar buckling with Span over pier height ratio follows the opposite trend.

$$\text{Effective no. of spans} = \text{Actual no. of spans} - \text{Piers with Exp. Bearing} - \frac{\text{Abutmnets with Exp. Bearings}}{2} \quad (10)$$

$$\text{Effective Tributary Span Area} = \frac{\text{Bridge Length} \times \text{Bridge Width}}{\text{Effective no.of Spans} \times \text{No.of col.per Pier}} \quad (11)$$

$$\text{Effective} \begin{Bmatrix} \rho_L \\ \rho_S \end{Bmatrix} \text{ or } \begin{Bmatrix} \rho_L{}^* \\ \rho_S{}^* \end{Bmatrix} = \frac{\text{Tributary Span Area}}{\text{Reference Span Area}} \times \begin{Bmatrix} \rho_L \\ \rho_S \end{Bmatrix} \quad (12)$$

$$\text{Effective} \begin{Bmatrix} \frac{\text{span}}{L} \end{Bmatrix} \text{ or } \begin{Bmatrix} \frac{\text{span}^*}{L} \end{Bmatrix} = \frac{\text{Span}}{\text{Effective no.of Spans} \times L} \quad (13)$$

Using the normalized fragility relationships of each damage measure with respect to the effective structural characteristic parameters, the probability of failure of other bridges in the inventory can be easily estimated. The correct estimate of the probability of failure of the bridges in the inventory including the representative bridges should be based on the actual site conditions of the individual bridges by utilizing seismic microzonation information of the City of Ottawa [21]. For demonstration purposes here, the probability of failure of allthe bridges in the sample

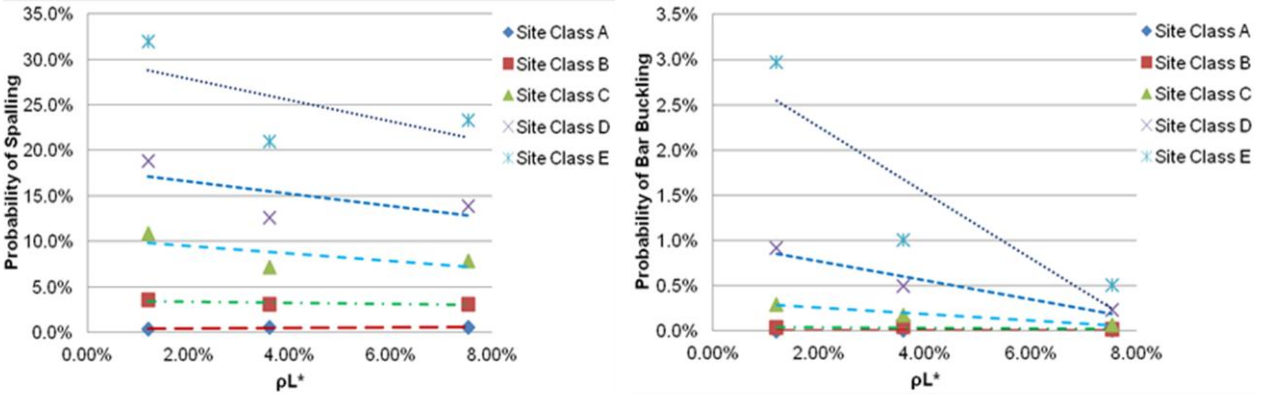


Figure 11. Fragility Relationships Based on Pier Longitudinal Reinforcement Ratio, Cover Spalling and Bar Buckling at 2% Exceedance in 50 Years for Site Class C

Figure 12. Fragility Relationships Based on Pier Transverse Reinforcement Ratio, Cover Spalling and Bar Buckling at 2% Exceedance in 50 Years for Site Class C

Figure 13. Fragility Relationships Based on Span over Pier Height ratio, CoverSpalling and Bar Buckling at 2% exceedance in 50 years for Different Site Conditions

Table 1. Effective Structural Characteristics Parameters of Bridges in the Sample Bridge Inventory

	Overall Span	Deck Width	Span Count	Column Height	Pier Deck Supports with Expansion Brgs	Abutment Deck Supports with Expansion Brgs	Revised Span Count (based on lateral load)	Column per bent	Tributary Span Area per	ρL	ρs	ρL*	ρs*	(Span/L)*
1A	85.1	23.9	4	6.7	2	2	1	4	508.5#	1.19%	1.21%	1.19%	1.21%	12.70
1B	85.1	23.9	4	6.7	0	2	3	4	169.5	1.19%	1.21%	3.56%	3.63%	4.23
2	38.0	15.5	2	5.2	0	2	1	2	303.1	4.44%	1.94%	7.45%	3.26%	7.57
3	61.0	20.0	2	5.0	0	2	1	3	405.7	2.22%	1.46%	2.78%	1.83%	12.13
4	83.0	13.0	3	9.3	0	2	2	2	268.9	2.05%	0.67%	3.88%	1.27%	4.48
5	57.2	19.7	3	9.2	0	2	2	3	187.4	1.84%	1.32%	4.99%	3.58%	3.11
6	74.0	8.7	3	5.9	0	2	2	1	320.1	5.15%	0.42%	8.18%	0.67%	6.27
7	112.0	10.3	4	4.7	2	2	1	1	1154.7	1.13%	1.21%	0.50%	0.53%	23.83
8	44.0	6.3	3	7.8	1	2	1	2	137.5	1.77%	1.64%	6.55%	6.07%	5.64
9	46.6	26.2	3	5.9	0	2	2	5	122.2	4.46%	1.25%	18.56%	5.20%	3.99
10	65.6	13.7	3	5.8	0	2	2	2	225.0	4.62%	0.83%	10.44%	1.88%	5.66

inventory is estimated assuming Site Class C. The estimated probability of failure of the damage states of concrete cover spalling and longitudinal bar buckling for low (40% in 50 years), moderate (10% in 50 years), and high (2% in 50 years) seismic events are tabulated in Tables 2 to 4. From the results, a priority list of vulnerable bridges can then be readily established to support decision making in retrofit planning and resource allocation.

Table 2. Estimated Probabilities of Cover Spalling and Bar Buckling Based on Effective Longitudinal Reinforcement Ratios

	ρL*	Spalling			Bar Buckling		
		2%/50yr	10%/50yr	40%/50yr	2%/50yr	10%/50yr	40%/50yr
1a	1.19%	9.828%	0.950%	0.035%	0.278%	0.004%	0.000%
1b	3.56%	8.843%	0.871%	0.026%	0.194%	0.003%	0.000%
2	7.45%	7.229%	0.740%	0.010%	0.056%	0.001%	0.000%
3	2.78%	9.168%	0.897%	0.029%	0.222%	0.003%	0.000%
4	3.88%	8.712%	0.860%	0.024%	0.183%	0.003%	0.000%
5	4.99%	8.249%	0.823%	0.020%	0.143%	0.002%	0.000%
6	8.18%	6.925%	0.716%	0.007%	0.030%	0.000%	0.000%
7	0.50%	10.114%	0.973%	0.038%	0.302%	0.005%	0.000%
8	6.55%	7.604%	0.771%	0.014%	0.088%	0.001%	0.000%
9	18.56%	2.621%	0.368%	0.000%	0.000%	0.000%	0.000%
10	10.44%	5.988%	0.640%	0.000%	0.000%	0.000%	0.000%

Table 3. Estimated Probabilities of Cover Spalling and Bar Buckling Based on Effective Transverse Reinforcement Ratios

	ρS*	Spalling			Bar Buckling		
		2%/50yr	10%/50yr	40%/50yr	2%/50yr	10%/50yr	40%/50yr
1a	1.21%	10.855%	1.053%	0.044%	0.285%	0.005%	0.000%
1b	3.63%	7.285%	0.738%	0.013%	0.114%	0.003%	0.000%
2	3.26%	7.824%	0.785%	0.018%	0.140%	0.003%	0.000%
3	1.83%	9.943%	0.972%	0.036%	0.241%	0.004%	0.000%
4	1.27%	10.766%	1.045%	0.044%	0.280%	0.005%	0.000%
5	3.58%	7.354%	0.744%	0.014%	0.117%	0.003%	0.000%
6	0.67%	11.655%	1.123%	0.051%	0.323%	0.005%	0.000%
7	0.53%	10.114%	1.141%	0.053%	0.332%	0.006%	0.000%
8	6.07%	3.689%	0.420%	0.000%	0.000%	0.001%	0.000%
9	5.20%	4.964%	0.533%	0.000%	0.003%	0.001%	0.000%
10	1.88%	9.872%	0.966%	0.036%	0.238%	0.004%	0.000%

Conclusions

This paper presents the formulation of a new probabilistic performance-based seismic risk assessment methodology suitable for quick and reliable assessment of large bridge inventories in a city, regional or national bridge network.

Table 4. Estimated Probabilities of Cover Spalling and Bar Buckling Based on Span over Height Ratios

	(SPAN/L)*	Spalling			Bar Buckling		
		2%/50yr	10%/50yr	40%/50yr	2%/50yr	10%/50yr	40%/50yr
1a	12.70	10.799%	1.058%	0.049%	0.284%	0.004%	0.000%
1b	4.23	7.073%	0.719%	0.015%	0.115%	0.002%	0.000%
2	7.57	8.543%	0.853%	0.028%	0.181%	0.003%	0.000%
3	12.13	10.546%	1.035%	0.047%	0.273%	0.003%	0.000%
4	4.48	7.182%	0.729%	0.016%	0.120%	0.002%	0.000%
5	3.11	6.578%	0.674%	0.010%	0.092%	0.002%	0.000%
6	6.27	7.969%	0.801%	0.023%	0.155%	0.002%	0.000%
7	23.83	15.695%	1.503%	0.093%	0.507%	0.006%	0.000%
8	5.64	7.692%	0.776%	0.021%	0.143%	0.002%	0.000%
9	3.99	6.964%	0.709%	0.014%	0.110%	0.002%	0.000%
10	5.66	7.703%	0.777%	0.021%	0.143%	0.002%	0.000%

The new methodology based on the use of generalized fragility relationships of concrete bridges requires only minimal engineering effort in determining simple structural characteristics parameters of the evaluated structures without the need of detailed nonlinear time history analysis, thus allowing relatively simple and fast evaluation of large bridge inventtories. The generalized fragility relationships are derived and calibrated from detailed structural modelling and nonlinear time history analysis of only a few selected representative bridges in the inventory. The new approach is efficient and yet can provide accurate detailed assessment information for large number of bridges in a network inventory that is more reliable than typical quick assessment check-list type of approach. Using this new approach, high level assessment information on the vulnerability and risk of the entire bridge infrastructure can be developed from a limited amount of structural details. Based on this methodology, bridges most at risk can be identified and prioritized for detailed engineer evaluations. The assessment results obtained using the proposed new evaluation approach for bridge inventory can provide critically needed information for better decision making on resource allocation by bridge engineers, owners, and bridge authorities for more efficient and effective seismic risk mitigation and management of bridge infrastructure.

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