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PROBABILISTIC PERFORCEMANCE-BASED SEISMIC RISK ASSESSMENT OF CANADIAN BRIDGES – A PILOT STUDY

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ABSTRACT

In Canada and other parts of the world, it is recognized that there are seismic deficiencies in existing aging bridge infrastructure. To help improve public safety and emergency preparedness, it is important to have a methodology to evaluate and quantify the seismic risk and vulnerability of the existing bridge systems. Due to the large number of bridges in existing infrastructure inventories and the limitation of resources available, there is a need to prioritize which bridges should be selected to undergo structural assessments by detailed analysis and possible rehabilitations. The current practice, based on physical site inspections and assessment by computer structural analysis, does not give high level assessment information on the risk and vulnerability of the entire bridge infrastructure from a system perspective. A probabilistic performance-based approach has been developed to assess the systemic risk and vulnerabilities of bridges in a city, regional or national network. This framework involves four interim steps of site hazard analysis, structural response analysis, damage analysis and estimation of economic loss. In this framework, the calculated responses of the evaluated facility subject to the seismic hazard at a site are linked to the probability of damage occurrence by using fragility curves. The expected economic losses are then evaluated at estimated damage levels. Utilizing available bridge performance databases the probabilistic seismic risk assessment methodology can be applied to inventories of bridges. This paper presents a pilot study of a sampled database of bridges in the City of Ottawa in Canada to show how this approach can be implemented to obtain seismic risk and vulnerability information of existing bridges from a system perspective. The system performance information will help with better decision making on resource allocation for bridge infrastructure maintenance and management.

Introduction

Current seismic bridge design follows the principles of collapse prevention and life

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safety. However, a limitation of this approach is that it provides little indication as to how well the design bridges will stand up to earthquakes in terms of damage and losses that may result as a consequence of the damage experienced. This limitation has led to the development of the performance-based design (PBD) concept in earthquake engineering which is a design methodology to meet specific performance objectives, such as those defined in terms of displacement, drift, ductility, and material behavior under specified design earthquake events.

In performance-based earthquake engineering (PBEE), the methodology encompasses four standard phases. The first of which is the seismic hazard analysis of the site. This is followed by structural analysis of the design structure to determine its responses to a range of seismic loading as representative to the seismic hazard of the site. The third phase is damage analysis in which the probability of occurrence of a particular damage level is assessed. The final phase of the methodology is review of potential economic losses as a result of the expected damage level.

Probabilistic Performance-Based Seismic Assessment Methodology

In earthquake engineering, performance-based design methodologies have been developed by several research groups of which their applications have mainly focused on the design or assessment of buildings. On the other hand, the Pacific Earthquake Engineering Research Center (PEER), in California, has initiated to develop a probabilistic performance-based approach for the design and assessment of bridges. The goal of the PEER methodology is to realize a full probabilistic design and evaluation of earthquake resistance of bridge structures and systems by decoupling the problem into smaller and more clearly defined steps (hazard analysis, demand analysis, damage analysis and loss analysis) (Mackie and Stojadinovic, 2003).

Hazard Analysis

In probabilistic performance-based design or assessment methodologies the problem is divided into individual steps represented by interim probabilistic models. The first of these models is the seismic hazard model which identifies the probability or frequency of occurrence of different seismic events of varying intensity at the site of the design or evaluated structure. In the determination of appropriate seismic hazards for structural design or assessment it is important to select a representative intensity measure (IM) for the site's seismic risks that minimize uncertainty in the probability analysis. For bridge structures, the first mode 5% damped elastic spectral acceleration of the structure ($Sa(T_1)$), the Peak Ground Acceleration (PGA) and the Peak Ground Velocity (PGV) are commonly selected as IMs for probabilistic performance-based evaluations (Mackie et al., 2008).

Studies have shown that the seismic hazard of a site can be approximated as a linear function on a log-log scale (Sewell et al., 1991; Kennedy et al., 1994; Cornell, 1996). The median hazard curve is 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) where λ_{IM} is the mean annual frequency of occurrence.

$$\lambda_{\mathrm{IM}} = k_0[\mathrm{IM}]^{-k} \tag{1}$$

To generate the probabilistic hazard model it is necessary to consider different probable hazard events for design of structures to meet different performance objectives. Typically, events with a high, moderate and low probability of occurrence are considered, which correspond to seismic events with the probability of occurrence of 50%, 10% and 2% in 50-years, respectively (Kunnath, 2007). Numerous factors are considered in the selection of representative site-specific ground motions for probabilistic design of structures. The selected or simulated ground motions should have similar characteristics to events that could be developed by any predominant faults (magnitude (M), distance (R), local soil type characteristics, and faulting mechanisms) of the area (Mackie et al., 2008).

The majority of the work required for this hazard analysis stage is carried out by engineering seismologists. In Canada, the probable seismicity of regions across Canada is tabulated in the NBCC (2005) and can be used in probabilistic performance based seismic design or assessment of bridge structures.

Demand Analysis

Following the determination of probable seismic hazard at the bridge site, the next step 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). In demand analysis, EDPs represent the structure's response to a particular loading and typically are presented in the form of drift ratio, displacement ductility, plastic rotation or compressive strain (Berry and Eberhard, 2003). 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 selected site-specific ground motion suite. Some studies have shown that, of the *EDP*s considered, the most efficient and practical demand model is the relationship between $Sa(T_1)$ and drift ratio (Mackie and Stojadinovic (2005)).

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) (Kunnath, 2007). Based of this relation, which is referred to as the interim demand model, the probability of occurrence for the representative EDP of the design or evaluated bridge for a given intensity level can be written in the form shown in equation (3) (Mackie et al., 2008)

$$\ln\left(\widehat{\mathrm{EDP}}\right) = A + Bln(IM) \tag{2}$$

where $\widehat{EDP}$ is the median EDP

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

where Φ is the standard normal distribution function

Damage Analysis

As part of a performance-based analysis, it is important to relate the response predicted by analytical demand models to physical descriptions of damage. 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 between the probability of different damage states occurring, such as spalling of concrete or longitudinal bar buckling, under different structural response levels is established. This is referred to as the interim damage model and is accomplished through incorporating observed, experimental, or analytical estimates of damage into the performance-based formulation. 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.

Over the past decade, various forms of damage fragility curves have been developed relating damage to demand. The objectives of these 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 (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 (Mackie et al., 2008). Depending on the relationship used, examples of damage states of reinforced concrete columns include cracking, spalling, longitudinal bar buckling and transverse reinforcement fracture, or the less definitive description of almost no damage, minor, moderate, or major damage or collapse. In the case of continuous damage measures, such as loss of lateral load carrying capacity, the median relationship between EDP and DM and the associated dispersion ($\sigma_{DM|EDP}$) is defined by the relationship shown in equation (4). Due to the discrete nature of most DMs, it is often difficult to formulate a damage model in terms of a median DM value conditioned on EDP because the cumulative distribution function describing the observed discrete DMs is a step rather than a continuous function. These discrete damage models can be simplified to act as continuous functions when the coefficients of variation (c.o.v.) for each of the discrete damage states are approximately equal. In this case, the regression constants in equation (4) are assumed to be C = 0, D = l, and $\sigma_{DM|EDP}$ = c.o.v. Such an approximation is consistent with assuming that damage limit states can be defined at discrete (median) levels of demand; however, it increases the overall uncertainty by the uncertainty from the damage model. Once an interim damage model has been developed, a probability relationship between the damage state and seismic hazard can be calculated as shown in equation (5). (Mackie et al., 2008)

$$\ln\left(\widehat{\mathrm{DM}}\right) = C + Dln(EDP) \tag{4}$$

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

The difficulty in the damage analysis stage lies in obtaining the interim damage model. All bridges have unique structural characteristics and so ideally each should have unique damage models. However; it is not economically realistic to carry out individual investigations or tests for every bridge or component in typical bridge infrastructure inventories under investigation. As suggested by Shinozuka et al., (2007), it is envisioned that a class of bridges of similar configuration, materials, and size will have correspondingly similar failure mechanism that apply to that class. Databases of collected experimental results (SPD, 2003) are available and have been used to develop general damage models. Several damage models developed from experiments for reinforced concrete columns already exist (Berry and Eberhard, 2003; Panagiotakos and Fardis, 2001).

Similar to the experimental models, damage models have been developed from observed damage states after major earthquakes. One such approach was developed by the Multidisciplinary Centre for Earthquake Engineering Research (MCEER).This approach has been described by Shinozuka et al., (2007).

The damage model incorporated in this study is the mathematical model developed by Berry and Eberhard (2003) which is based on the seismic performance database (SPD) of over 400 reinforced concrete column tests of varying material and structural properties. The database includes data from tests carried out in the United States, Canada, Japan and New Zealand (SPD, 2003). The relations developed by Berry and Eberhard (2003) estimate the response at which different damage levels (concrete spalling or longitudinal bar buckling) will occur based on different structural characteristics. These equations combine plastic-hinge analysis with approximations for the column yield displacement, plastic curvature, buckling strain, and plastic hinge length to develop relationships between column damage and commonly used engineering demand parameters (plastic rotation, drift ratio, and displacement ductility). Equation (6) shows the relationship developed for drift ratio at the initiation of concrete spalling for both rectangular and spiral reinforced columns. Equation (7) shows a similar relation developed for estimating the drift ratio at the initiation of longitudinal bar buckling where the constant $k_{e\,bb}$ is 50 for rectangular-reinforced columns and 150 for spiral-reinforced columns (Berry and Eberhard, 2003).

$$\frac{\Delta_{spall\,calc}}{L}(\%) = 1.6\left(1-\frac{P}{A_g f_c'}\right)\left(1+\frac{L}{10D}\right) \tag{6}$$

$$\frac{\Delta_{bb\,calc}}{L}(\%) = 3.25\left(1+k_{e\,bb}\rho_{eff}\frac{d_b}{D}\right)\left(1-\frac{P}{A_g f_c'}\right)\left(1+\frac{L}{10D}\right) \tag{7}$$

Berry and Eberhard (2003) have compared estimated values from equation (6) and (7) at which damage is expected to occur to the actual damage occurrence demand of a large group of columns in the SPD. Based on the comparison study, they have developed general fragility curves that can be easily converted to fragility curves for specific columns. Figures 1 to 4 show the general fragility curves for rectangular and circular column concrete spalling and longitudinal bar buckling. These curves provide the probability of damage occurring given the ratio of actual demand to the demand at which damage is estimated to occur ($\Delta_{demand}/\Delta_{damage\,calc}$), based on equations (6) and (7). Based on the assumption that the database is representative of the general population of rectangular- and spiral-reinforced columns, Figure 1 to 4 can be used to evaluate individual rectangular columns and/or develop the fragility curves for these columns.

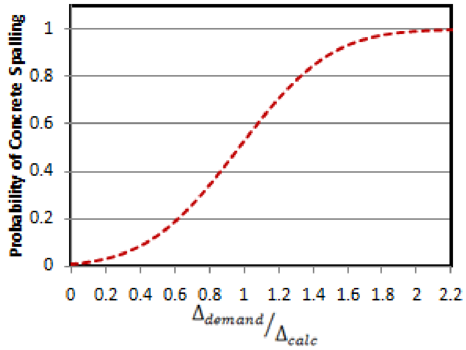

Figure 1: General fragility curve for concrete cover spalling of rectangular columns

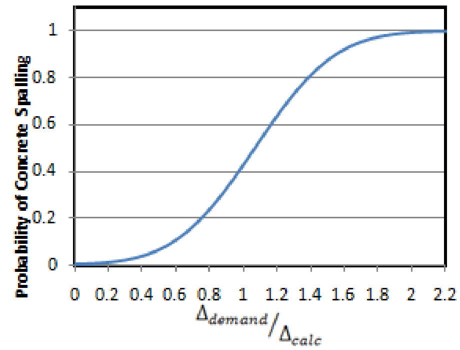

Figure 2: General fragility curve for concrete cover spalling of circular columns

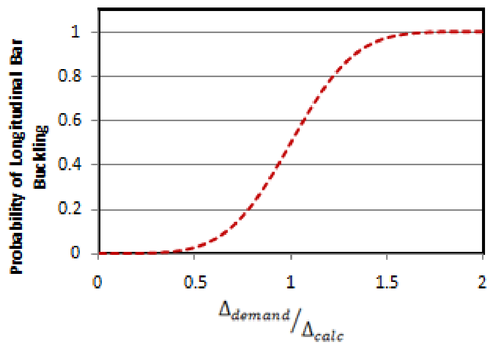

Figure 3: General fragility curve for longitudinal bar buckling of rectangular columns

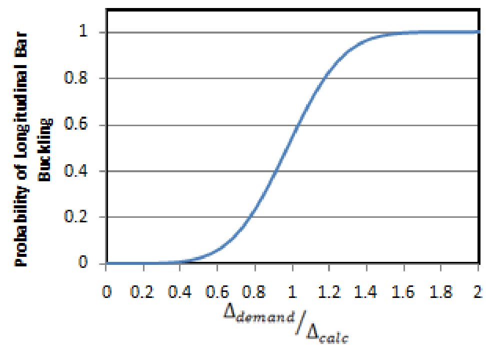

Figure 4: General fragility curve for longitudinal bar buckling of circular columns

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. (Mackie et al., 2008) 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. Some of the key concerns are what is the safe load a damaged bridge can support, what are the repair cost of the damages, and the duration of interruption to service. Today, answers to these questions are based on experience and engineering judgement rather than quantitative analyses and engineering evaluations. After an earthquake, quick decisions for timely emergency response are necessary. It is important to have a probability framework relating the decision variables to hazard intensity measures (Mackie and Stojadinovic, 2005).

Bridge decision variables can be separated into two classes: 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 included 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 (8) 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 relation between DV and IM as shown in equation (9).

$$\ln\left(\widehat{\mathrm{DV}}\right) = E + F\ln(DM) \tag{8}$$

$$P(DV|IM) = 1 - \Phi\left[\frac{\ln(dv^{LS}) - (E + FC + FDA + FDB\ln(im))}{\sqrt{DF^2\sigma^2_{EDP|IM} + F^2\sigma^2_{DM|EDP} + \sigma^2_{DV|DM}}}\right] \tag{9}$$

Application to Canadian Bridges

The following section shows the application of the above described methodology to the Heron Road Bridge in Ottawa. The bridge selected is a seven span structure with an overall length of approximately 275m. The superstructure is a prestressed concrete box girder with an overall width of approximately 15.5 m and height of 1.3 m. The general profile of the bridge is shown in figure 5 and a typical cross section is shown in figure 6.

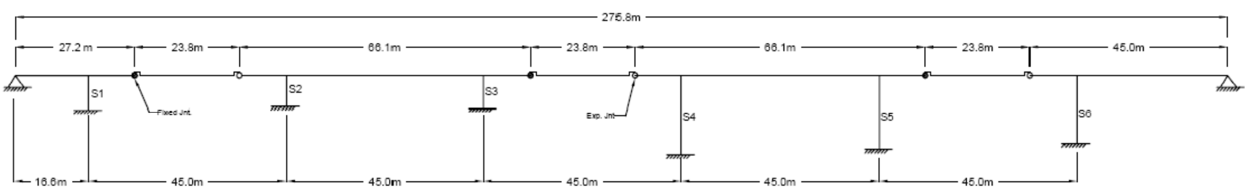

Figure 5. Heron Road Bridge profile

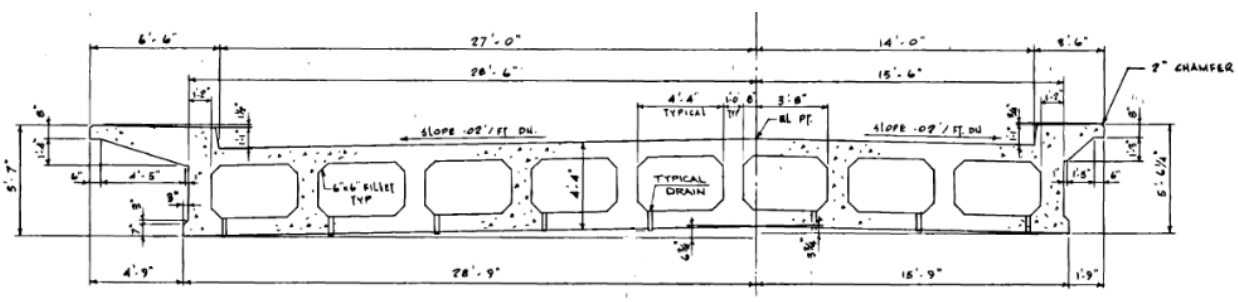

Figure 6. Sample cross-section of Heron Road Bridge superstructure

Hazard Analysis

As previously discussed, the first stage in a PBEE assessment is the Hazard Analysis of the site. For the Heron Road Bridge, the uniform hazard spectrum (UHS) curves at three hazard levels were obtained for the Ottawa area from the Geological Survey of Canada (GSC) as shown in Figure 7. Due to the fact that the UHS represents a composite of different types of earthquakes and not any single event, no recorded earthquake event will have a complete matching response spectrum. For this research artificial time histories are generated to closely fit the UHS. Numerous studies have shown that simulated records and real records are functionally equivalent, from both linear and non-linear point of view (Atkinson and Beresnev, 1998).

From the response spectra shown in Figure 7 and using the first modal period of the structural model generated of the bridge, a seismic hazard curve is derived by plotting the return periods against the magnitude of the spectral acceleration of the first structural mode shape as shown in figure 8. The data follows the characteristic fit as represented by equation (1). The resulting coefficients were determined to be $k_0 = 1.00x10^{-5}$ and $k = 1.36$.

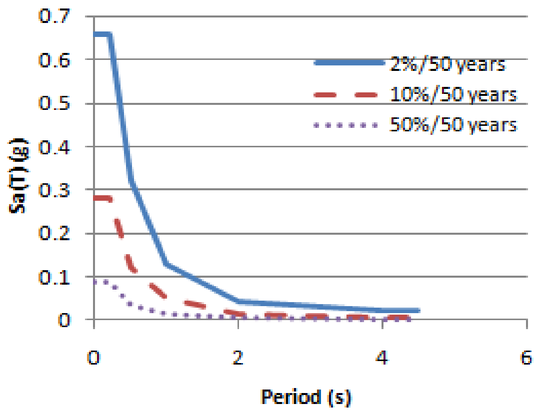

Figure 7. Ottawa area UHS response spectrum

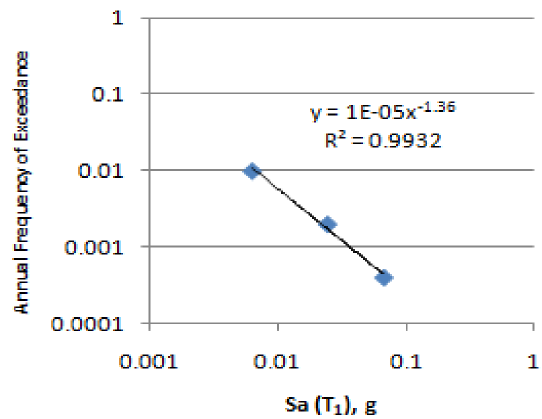

Figure 8. Hazard curve for Ottawa, ON

Demand Analysis

Using 30 different simulated time histories generated to fit the three response spectrum levels (10 per hazard level) a series of non-linear time history analyses have been performed. Pier drift ratio is selected as the demand parameter observed. Using the drift ratios of the piers in response to each time history case, a demand model is developed as shown in figure 9. Using a lognormal distribution and least square fit as discussed earlier, a best fit line gives an approximate relationship between the resulting drift ratio verses $S_a(T_1)$ data. The resulting regression coefficients A and B are determined to be -3.71 and 0.9402 respectively for the expression given in equation (2). The resulting probability of a particular drift ratio occurring is calculated using equation (3) and shown in figure 10.

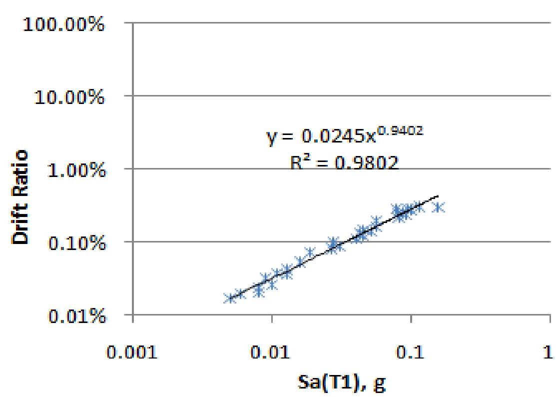

Figure 9. Regression relation of drift ratio given hazard intensity

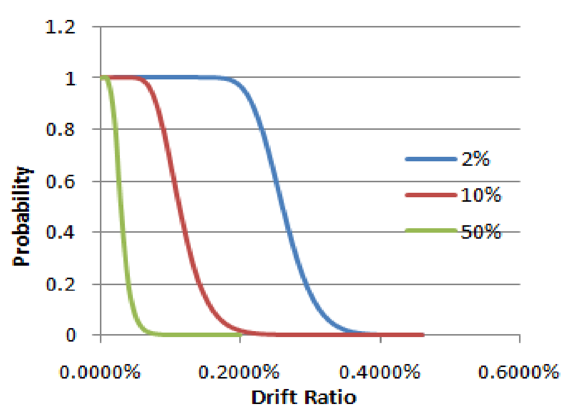

Figure 10. Probability of exceeding drift ratio

Damage Analysis

Equation (6) and (7) are used to estimate the drift ratio at which concrete spalling and longitudinal bar buckling are expected to occur for pier S5 of the bridge being investigated. Based on this estimated value, the general fragility curves shown in figures (1) and (3) are adjusted to be representative of the behavior performance of pier S5. The resulting fragility curves (Figure 11) show that the probability of concrete spalling is very low at even a 1% drift ratio and virtually zero for longitudinal bar buckling. Figure 10 developed in the demand analysis phase shows that the bridge will not reach the 1% drift ratio under any of the hazard events investigated. Following equation (5) a probabilistic damage model for the example bridge could be developed however may be inaccurate due to the observation that the size of earthquake that has been shown as having the probability of causing damage is outside the boundary intensity measure investigated.

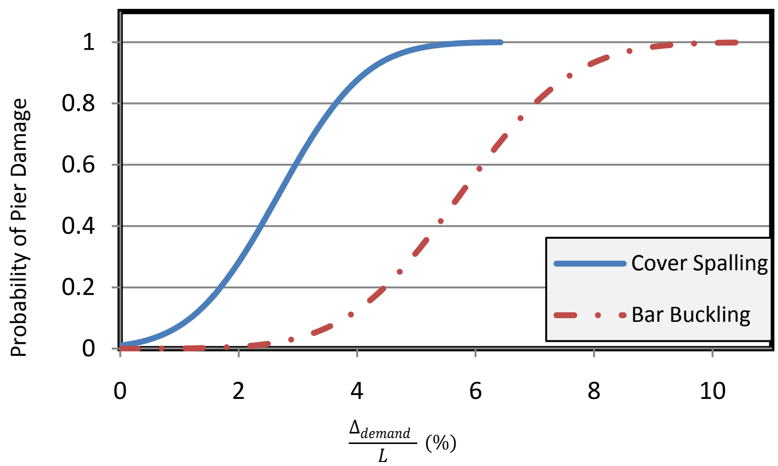

Figure 11. Damage fragility curve for pier S5 of Heron Bridge

Systematic Application

It is the objective of this ongoing research to perform numerous probabilistic earthquake assessments on a sample of bridges that are representative of bridges in a region. Once a representative sample has been investigated, key risk features can be identified to better allocate resources

Conclusions

A probabilistic performance based earthquake assessment was performed on an existing bridge in the City of Ottawa. The assessment results show that the example bridge will not suffer damage to the extent as specified by the selected performance parameters. The probability of either of the two damage states investigated occurring was zero and therefore a loss analysis could not be performed. The objective of this ongoing research is to develop and implement a probabilistic performance based seismic risk assessment methodology of bridges and by using this methodology develop a system performance database of bridges in Canadian cities which associates key features of bridges with probability of damage and losses. A system performance database such as this would help with decision making on resource allocation for bridge infrastructure maintenance and management.

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