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# Seismic fragility analysis of a highway bridge in Quebec via metamodelling

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## ABSTRACT

Highway bridges play a crucial role in the infrastructure and in the economy of a country, and past earthquakes have exposed their vulnerability as the weak links of a transportation network. In Quebec, several highway bridges were built without any seismic design criteria; therefore, they do not comply with the current seismic detailing provisions. To mitigate potential economic losses and avoid casualties during an earthquake, performance assessment of existing structures is crucial for stakeholders. Due to the random nature of earthquakes and material uncertainties, probabilistic methods have been shown to be more suitable for engineering analysis, and fragility assessment has emerged as a promising tool to evaluate the seismic performance of highway bridges. Additionally, fragility functions are extremely valuable in regions of moderate seismicity, such as Eastern Canada, where the scarcity of empirical data of earthquake damage requires risk evaluation to be based on analytical methods explicitly accounting for the uncertainty inherent to the structural response. Nevertheless, the definition of analytical fragility functions may demand a substantial amount of nonlinear time history analyses on rather complex finite element models, as a manner of accounting for the variability not only in structural properties but also in seismic excitation. To overcome this computational burden, this study leverages a metamodel-based approach for the construction of seismic fragility functions for highway bridges. Metamodels are tools used to model the outcome of experiments, both physical and computational, by replacing, in this case, the costly finite element simulations. More precisely, the adopted methodology employs polynomial response surface metamodels to predict the column lateral displacement and deformation on abutments and bearings for a bridge subjected to seismic loading. For this purpose, a characterization of the seismic hazard at the site of interest is initially performed, and samples of the bridge configuration are generated to cover the variability of the parameters by employing a suitable technique for the design of experiments. The training data are then produced from nonlinear time history analyses on the generated samples, and the predictive capacity of the metamodel is assessed through cross-validation. An application of the explored methodology is presented for a case-study concrete girder bridge in Quebec, in which the influence of the ratio of transverse reinforcement in the pier columns and the uncertainty regarding other structural parameters are considered. The transverse reinforcement ratio is an indicator of the level of required ductility on the columns, which progressed according to the evolution of bridge design guidelines. Thus, the enhancement of the seismic performance over the design requirements on column ductility is herein investigated.

Keywords: bridges, fragility, earthquake, metamodels.

## INTRODUCTION

The damage and losses caused by the disruption of transportation networks worldwide after recent earthquakes have emphasized the need for risk assessment and retrofit prioritization plans for existing bridge inventories. The province of Quebec in eastern Canada contains approximately 8500 bridges in its road network, and a significant part of its production is transported along highways [1]. Furthermore, 75% of the bridges in Quebec are more than 35 years old and were designed without modern seismic conception and detailing methods [2]. Therefore, these structures do not comply with current guideline requirements, and their seismic performance must be assessed. A recent study in the region verified the seismic vulnerability of the most typical bridge classes in Quebec for their as-built configuration [3]. Since then, new ground motion models have been defined for eastern and western Canada, resulting in the 5$^{th}$ generation of hazard maps adopted by the latest review of the National Building Code of Canada (NBCC) 2015 [4]. Additionally, new ground motion selection approaches were developed for the replacement of more traditional methods that adopt the uniform hazard spectrum (UHS) as the target spectrum [5-6]. Instead of matching the selected set of records to a UHS, which is built upon the assumption of equal probability of exceedance over all periods, these recent approaches target the selection on a conditional distribution of seismic intensity measures ($IM$) given the occurrence of an expected seismic scenario. Such recent advances in seismic hazard analysis and ground motion selection should be incorporated into the assessment of seismic vulnerability of highway bridges. Analytical fragility analysis is a valuable tool for such assessment, although the number of required dynamic analyses can grow fast once more parameters are introduced into the evaluation, e.g., the influence of recent design criteria, the improvement of different retrofitting measures, or the consideration of additional seismic intensity measures. To avoid such a computational burden, supervised learning techniques known as metamodels (or surrogate models) can be used to replace numerous finite element simulations [7-8]. This study presents the development of surrogate models for a case study bridge in Quebec to investigate the selection of optimal model and intensity measures. The models were trained using results from dynamic analyses that employed ground motions that match a conditional distribution of intensity measures consistent with the region's seismic hazard. The chosen metamodel was then employed on the development of seismic fragility surfaces to assess the impact of the transverse reinforcement ratio on the bridge's seismic vulnerability.

## CASE STUDY BRIDGE

This study focuses on a single bridge, the *Chemin des Dalles* overpass, located over highway 55 near Trois-Rivières in Quebec. This bridge was designed in 1975 and is not in accordance with current bridge design standards for seismic events [2] and the design event level adopted in Canada [4]. The *Chemin des Dalles* bridge is a symmetric continuous concrete girder bridge with a length of 106.5 m divided into three equally spaced spans and a 13.2 m wide deck. The bridge pier bents are composed of three circular columns and a transverse beam joining column tops. Each column has a 0.914 m diameter and vertical clearance of 6.2 m. The bents and abutments are supported by shallow foundations. In addition, wing walls compose the support system in seat-type abutments. The superstructure is composed of a 0.165 m depth deck and six prestressed concrete AASHTO type V girders directly connected at the bents and supported by elastomeric bearings at the abutments. Bent columns are rigidly connected to the shallow foundations in the west bent and free for rotation in the east bent. With such characteristics, one can identify several similarities with the average bridge for Quebec [1].

The three-dimensional (3D) finite element model originally built by Tavares *et al.* [9] is revisited in this work. The model was created on OpenSees (Open System for Earthquake Engineering) [10], and it uses beam-column elements and zero-length elements to represent the behaviour of this structural system and capture the nonlinear behaviour of critical structural components. Bent columns were modelled using force-based beam-column elements with their cross-sections discretized in fibres. Soil-structure interaction was incorporated by adding spring-dashpot systems using zero-length elements and mass to the footing nodes. Elastomeric bearings were modelled to behave as an elastic-perfectly plastic material. An overview of the bridge model as well as some details on bents, columns, and abutments are illustrated in Figure 1. More details on the numerical model are found elsewhere [9].

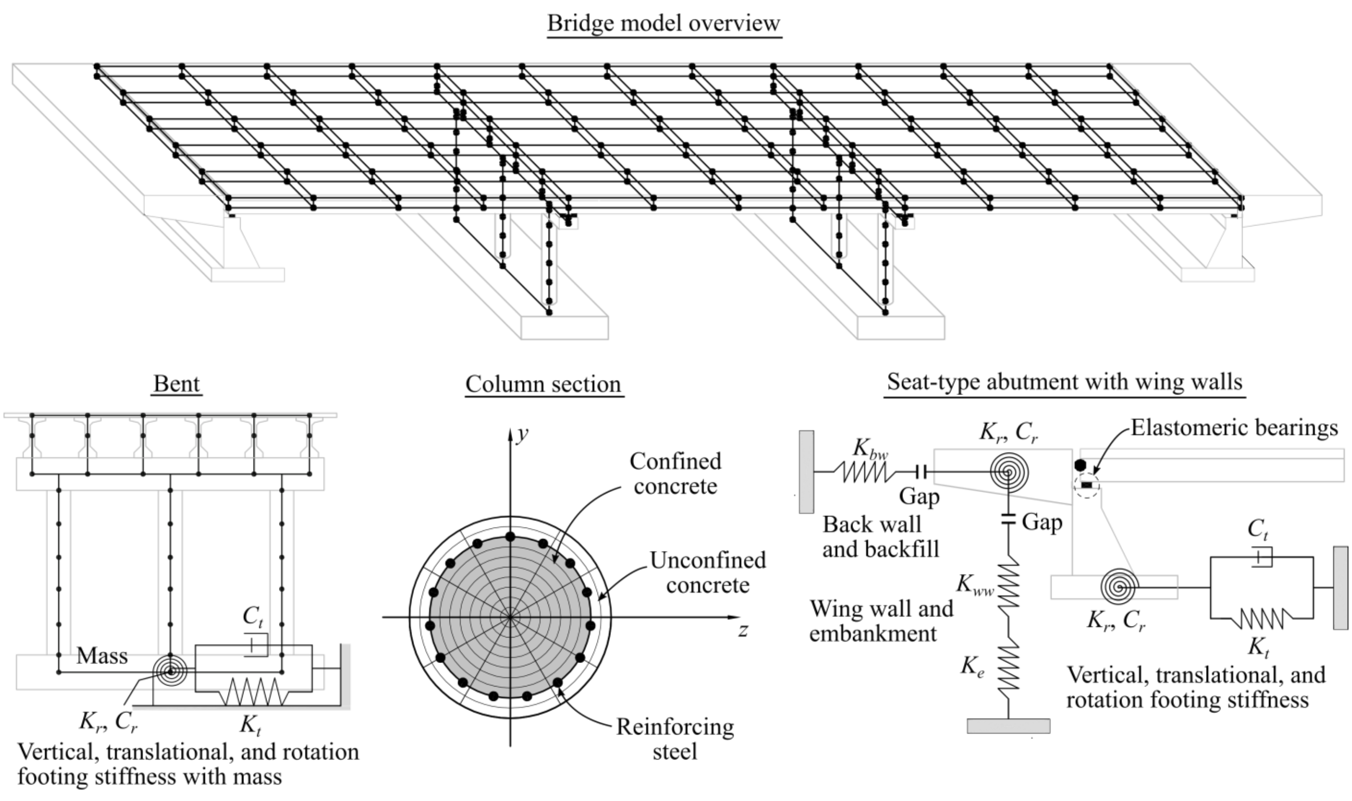


*Figure 1. Case study bridge model (adapted from Ref. [9])*

## SEISMIC HAZARD AND RECORD SELECTION

Assessing the seismic performance of engineered systems via dynamic analyses requires an appropriate representation of the seismic hazard at the site by selecting hazard-consistent ground motion records. The seismic hazard maps of Canada have been recently updated to their 5th generation and are in force for the design of structures according to the NBCC 2015 [4] and the Canadian Highway Bridge Design Code (CHBDC) CSA S6-14 [2]. The open-source software OpenQuake [11] was employed to perform the probabilistic seismic hazard analysis (PSHA) at the bridge location for the determination of the hazard curve for (pseudo) spectral acceleration $S_a$ at the bridge's fundamental period ($T_1 = 0.38$ s). Seven levels of spectral acceleration were defined, ranging from 0.2 g to 1.5 g. This interval was chosen for conformity with a previous study [9]. Then, seismic disaggregation was performed to determine the expected earthquake scenarios for each $S_a(T_1)$ level of interest.

The next step was to select ground motions consistent with the seismic hazard determined. The generalized conditional intensity measure (GCIM) approach [6, 12] was chosen for selecting the records that suit the region of interest. For such, the latest ground motion models (GMM) determined by Atkinson and Adams [13] for Eastern Canada were employed to define the values of mean and standard deviation for spectral accelerations, peak ground acceleration ($PGA$) and peak ground velocity ($PGV$). The spectral acceleration at the bridge's fundamental period $S_a(T_1)$ was selected as the conditioning intensity measure, and the seven levels previously used in the PSHA were employed to build the conditional target distributions of $IM$s. The Next Generation Attenuation – West 2 (NGA-West2) database for shallow crustal earthquakes [14] was used for the selection. In total, 280 ground motions were selected (7 $S_a(T_1)$ levels and 40 records per level).

## METAMODELLING

A metamodel is a probabilistic model that estimates the relationship between covariates (or predictors) $x$ and system response $y$ and can be generalized by the following expression:

$$\hat{y} = g(x) + \varepsilon \tag{1}$$

where $\hat{y}$ is the approximative response, $g(x)$ represents the expected response, and $\varepsilon$ is an error often assumed to be normally distributed with zero mean and non-null standard deviation [15].

Polynomial response surface models (PRSMs) have been explored as surrogate models for bridges [7-8] and other structures [16] and were chosen in the present work for their simplicity. A PRSM is defined by the following equation:

$$g(x) = \beta_0 + \sum_{i=1}^{p} \beta_i \phi_i(x) \tag{2}$$

where $\phi_i(x)$ are polynomial basis functions and $\beta_j$, $j = 0,1,\ldots,p$ are the model hyperparameters, which are calibrated by training the metamodel.

For the case study, a PRSM is used to build a probabilistic seismic demand model that can predict the bridge seismic response given parameters related to ground motion intensity. A total of 12 structural parameters were chosen as predictors and are presented in Table 1 along with their ranges of variation [3]. These parameters were chosen to incorporate uncertainty about material properties and to allow the assessment of different design eras in terms of ductility requirements for reinforced concrete bridge bent columns. These categories are also indicated in the table.

*Table 1. Ranges for structural parameters.*

| $x$ | Parameter | Lower bound | Upper bound | Category |
|---|---|---|---|---|
| $f_c'$ | Concrete strength (MPa) | 22 | 38 | Critical bridge modelling parameter |
| $f_y$ | Steel strength (MPa) | 400 | 525 | Critical bridge modelling parameter |
| $K_{brg}$ | Initial stiffness of elastomeric bearings | 0.5 | 1.5 | Critical bridge modelling parameter |
| $K_{ww}$ | Abutment wing wall stiffness | 0.5 | 1.5 | Critical bridge modelling parameter |
| $K_e$ | Abutment embankment stiffness | 0.5 | 1.5 | Critical bridge modelling parameter |
| $K_{bw}$ | Abutment back wall stiffness | 0.5 | 1.5 | Critical bridge modelling parameter |
| $K_{r,f}$ | Footing rotational stiffness | 0.5 | 1.5 | Critical bridge modelling parameter |
| $K_{t,f}$ | Footing translational stiffness | 0.5 | 1.5 | Critical bridge modelling parameter |
| $\Delta m$ | Mass variability (superstructure) | 0.9 | 1.1 | Critical bridge modelling parameter |
| $\xi$ | Damping ratio (%) | 0.4 | 3 | Critical bridge modelling parameter |
| $\ell_{gap}$ | Abutment gap (mm) | 20 | 80 | Critical bridge modelling parameter |
| $s$ | Transverse reinforcement spacing (mm) | 50 | 350 | Design era |

Different polynomial orders were investigated for the definition of an optimal surrogate model for each critical structural component. The polynomial orders varied from 1 to 4, and cases considering interaction among predictors were also investigated. Table 2 summarizes the models that were assessed, where $p$ is the number of predictors (i.e., 13 in total: 12 structural parameters and one intensity measure).

Apart from the selection of the optimal surrogate model, this study also investigated the choice of the seismic intensity measure that optimizes the model's predictability. For such, traditional intensity measures (e.g., spectral acceleration at the fundamental period, peak ground acceleration, and peak ground velocity) and other $IM$s (e.g., Arias intensity $AI$ and cumulative absolute velocity $CAV$) were calculated from the selected records. Moreover, the average spectral acceleration $S_{a,Avg}$ over a period range of interest was also considered. The average spectral acceleration is a promising $IM$ because it incorporates the effects of higher vibration modes into the response and the effects of period elongation caused by nonlinearity. Some studies examined the efficiency and proficiency of this measure in comparison with those of simple spectral accelerations at a single period and showed better performance in the prediction of building response [17]. In this case, periods ranged from 0.1 to 1.0 s to consider the effects of the second and third transverse vibration modes and the period elongation caused by nonlinear behaviour (usually taken as $2T_1$).

*Table 2. Polynomial response surface models assessed for surrogate modelling.*

| **Model** | $\boldsymbol{g}(\boldsymbol{x})$ |
|---|---|
| M1 | $\beta_0 + \sum_{i=1}^{p} \beta_i x_i$ |
| M2 | $\beta_0 + \sum_{i=1}^{p} \beta_i x_i + \sum_{i=1}^{p} \sum_{j=2,j>i}^{p} \beta_{ij} x_i x_j$ |
| M3 | $\beta_0 + \sum_{i=1}^{p} \beta_i x_i + \sum_{i=1}^{p} \beta_{ii} x_i^2$ |
| M4 | $\beta_0 + \sum_{i=1}^{p} \beta_i x_i + \sum_{i=1}^{p} \sum_{j=1,j\geq i}^{p} \beta_{ij} x_i x_j$ |
| M5 | $\beta_0 + \sum_{i=1}^{p} \beta_i x_i + \sum_{i=1}^{p} \beta_{ii} x_i^2 + \sum_{i=1}^{p} \beta_{iii} x_i^3$ |
| M6 | $\beta_0 + \sum_{i=1}^{p} \beta_i x_i + \sum_{i=1}^{p} \beta_{ii} x_i^2 + \sum_{i=1}^{p} \beta_{iii} x_i^3 + \sum_{i=1}^{p} \beta_{iiii} x_i^4$ |

For the training, the uniform design method was adopted to define statistically significant samples of the 12 model predictors within their ranges (Table 1). These values were then used for more than 800 nonlinear time history analyses performed on OpenSees. The adopted engineering demand parameters (EDPs) were column drift and deformations on abutment wing walls and elastomeric bearings installed on abutments, considered only in the bridge's transverse direction [18]. Given the differences in the covariates in terms of order of magnitude, responses and covariates were transformed into the log-space for better performance of the regression models.

Following the construction of the dataset, a 10-fold cross-validation approach was used to assess model performance and to select the optimal one based on the lowest root mean squared error (RMSE). Figure 2 presents the results of the 10-fold cross-validation in terms of RMSE for all the assessed models and all the intensity measures considered as predictors, where the dashed line crossing the bars indicates the lowest value of RMSE. The optimal model for deformation on abutment wing walls and deformation on elastomeric bearings is M2 (first-order polynomial with interactions), while M1 (first-order polynomial) generated the lowest prediction error amongst the assessed models for column drift.

One can easily verify that spectral acceleration at the fundamental period had the best performance amongst all the intensity measures in study and for all the assessed models. That measure was followed by the average spectral acceleration, whereas the cumulative absolute velocity was the worst seismic $IM$ predictor for the case study bridge. It is worth noting that the spectral acceleration at the fundamental period was the conditioning $IM$ for building the target conditional distribution of intensity measures used for record selection.

Once the optimal model was selected, a stepwise regression approach was performed in which predictors were added to the model sequentially, and then, the method could also remove any covariate that no longer provided an improvement to the model performance. For such, the surrogate models were trained using the complete dataset, and for the sake of consistency, the upper model allowed during the stepwise regression was the optimal model obtained from the 10-fold cross-validation. The Bayesian information criterion (BIC) was used for the selection of predictors.

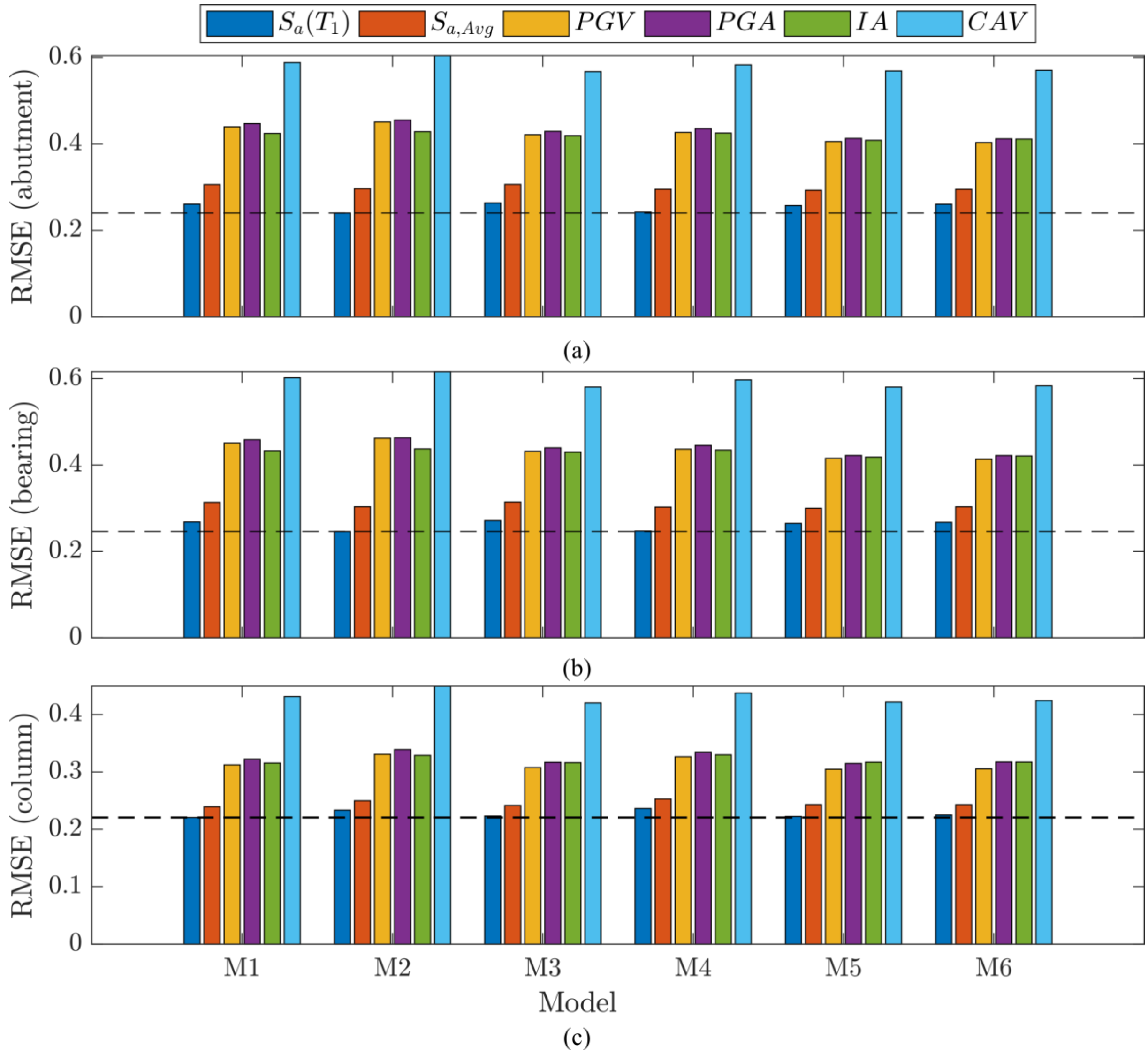


*Figure 2. Comparison of performance of metamodels with different intensity measures from 10-fold cross-validation for: (a) deformation on abutment wing walls, (b) deformation on elastomeric bearings, and (c) column drift. Dashed lines indicate lowest root mean squared errors.*

Table 3 presents the final surrogate models and reports the value of the adjusted $R^2$ metric. The observed data used for training $y_{obs}$ were then compared to the mean predicted responses generated from each surrogate model $\bar{y}_{pred}$ and are plotted in Figure 3. Although all surrogate models present relatively high values of adjusted $R^2$, the model for column drift tends to underestimate the component response.

*Table 3. Final metamodels after stepwise regression for each EDP.*

| EDP | Metamodel (log-space) | Adjusted $R^2$ |
|---|---|---|
| Abutment wing wall deformation | $4.623 + 0.105\ln(K_{r,f}) + 0.878\ln(\Delta m) + \ln(S_a)\left[0.640\ln(K_{brg}) + 0.315\ln(\ell_{gap})\right]$ | 0.922 |
| Elastomeric bearing deformation | $4.614 + 0.105\ln(K_{r,f}) + 0.891\ln(\Delta m) + \ln(S_a)\left[0.675\ln(K_{brg}) + 0.315\ln(\ell_{gap})\right]$ | 0.921 |
| Column drift | $-0.624 + 0.977\ln(S_a) - 0.311\ln(K_{brg}) + 0.709\ln(\Delta m)$ | 0.859 |

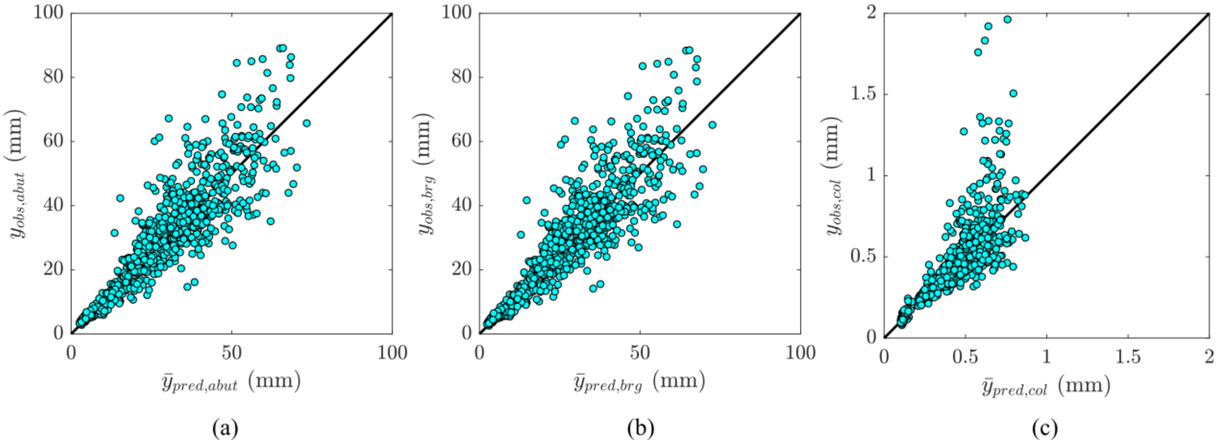


*Figure 3. Comparison of observed and predicted responses for (a) abutments, (b) bearings, and (c) columns.*

## FRAGILITY ANALYSIS

Seismic fragility functions traditionally represent the conditional probability of achieving a given damage state on the occurrence of a certain seismic intensity measure $IM$. In the effort of comprehending the impact of structural parameters on bridge seismic vulnerability, fragility can also be conditioned on a structural feature $x$:

$$\text{Fragility} = P(\text{damage state}|IM, x) \tag{3}$$

The damage states adopted in this study are in accordance with the performance criteria defined on the CHBDC [2] (i.e., slight, moderate, extensive, and complete damage). Although the case study bridge is a multi-component structure, the columns govern its fragility for it as-built configuration as demonstrated in previous studies [9, 18] and confirmed in the present work. Hence, only the fragility for the bent columns are presented herein, which fairly approximates the system fragility. The impact of different reinforcement configurations on the columns of the case study bridge was investigated experimentally by Le Tartesse *et al*. [19], where the damage states were defined in terms of drift ratios. The results obtained in laboratory tests are in good agreement with the analytical models defined by Stefanidou and Kappos [20] for circular columns, which were adopted herein to consider the influence of the transverse reinforcement ratio.

For the construction of the fragility functions, logistic regression was employed following the recommendations of Rokneddin *et al.* [21]. For the training of the fragility functions, 10,000 samples were drawn from the surrogate model for column drift. Given its tendency to underestimate larger responses, the samples were drawn for spectral accelerations up to 2.0 g (i.e., the samples extrapolate the domain used for training). This approach was adopted for presentation of the fitted fragility surfaces. The parameters of interest were the spectral acceleration at the fundamental period and the spacing of transverse reinforcement, which were uniformly sampled while all the other parameters were taken as their mean values. Although fragility surfaces are generated for deterministic values in this study, the developed surrogate models can also be employed to build fragility functions based on probability distributions of the predictors if available.

The fragility surfaces generated for the four damage states of interest are presented in Figure 4 and incorporate the impact of both spectral acceleration and the transverse reinforcement ratio $\rho_t$, which was calculated as [22]:

$$\rho_t = \frac{2A_{st}}{c\ s} \tag{4}$$

where $A_{st}$ is the area of transverse reinforcement and $c$ is the diameter of the confined concrete core.

Two values of the transverse reinforcement ratio are highlighted in Figure 4: the red line corresponds to the as-built bridge with a spacing of 300 mm between hoops, and the blue line indicates the fragility if the columns were designed according to the latest version of the CHBDC with $s = 110$ mm. The impact of the transverse reinforcement ratio is largely appreciated in reducing the seismic fragility for extensive and complete damage states, whereas its effect for slight and moderate damage states is less significant.

## CONCLUSIONS

Leverages of a metamodel-based approach for construction of seismic fragility functions of highway bridges were investigated in this study. For this purpose, surrogate models were built for critical components of a case study bridge in Quebec subject earthquake loading and used for the construction of seismic fragility surfaces to investigate the impact of the transverse reinforcement ratio on the bridge's seismic vulnerability. The resulting models presented rather high metrics for goodness-of-fit and low errors, although the resulting model for column drift showed a tendency to underestimate larger displacements, and thus, further enhancement should be investigated for this critical bridge component.

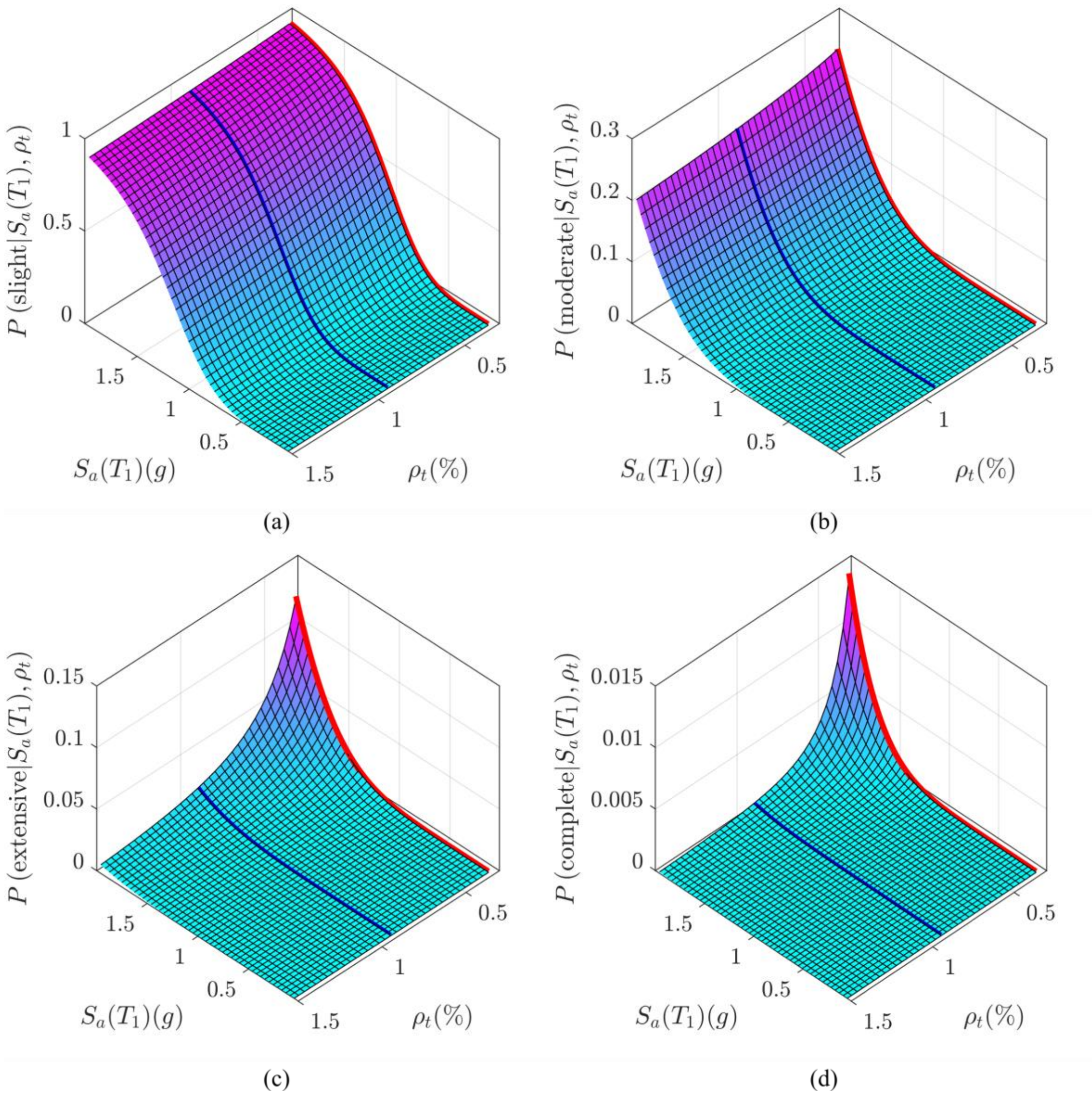


*Figure 4. Fragility surfaces for columns conditioned on spectral acceleration and transverse reinforcement ratio for (a) slight, (b) moderate, (c) extensive, and (d) complete damage states. Red and blue lines indicate transverse reinforcement ratios for as-built and according to current Canadian design requirements, respectively.*

Although the metamodels demonstrated that the column transverse reinforcement ratio did not significantly influence the seismic response of columns, this structural parameter played an important role in the bridge's seismic performance. As expected, the impact of the parameter is not as significant on lower damage states (slight and moderate), for which capacity is slightly affected by ductility. However, for extensive and complete damage states, the confinement of the concrete core provided by the transverse reinforcement considerably increases the column ductility and, thus, its capacity. The enhancement of the bridge's seismic performance due to ductility provisions could be appreciated on the fragility surfaces presented here, in which the probability of achieving higher damage states is extremely low for columns designed according to recent guideline requirements.

The seismic demands on the studied components are clearly greater than the elastic limit, for which the spectral acceleration at the fundamental period is expected to demonstrate good predictability. Previous studies have demonstrated the superior capability of the average spectral acceleration to incorporate the effects of higher modes and nonlinear behaviour. Nevertheless, spectral acceleration at the fundamental period as the chosen intensity measure performed better than other studied I$M$s throughout all assessed metamodels. Such superior performance might have been influenced by the record selection approach due to low dispersion of the conditioning intensity measure in comparison to the other $IM$s, and this feature should be addressed in future research.

## ACKNOWLEDGMENTS

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