Canadian Journal of Civil Engineering

Seismic Reliability Assessment of Base-Isolated Bridges in Quebec

Journal:	<i>Canadian Journal of Civil Engineering</i>
Manuscript ID	cjce-2020-0036.R2
Manuscript Type:	Article
Date Submitted by the Author:	09-Mar-2021
Complete List of Authors:	Nassar, Mohamad; École de technologie supérieure, École de technologie supérieure- Département de génie de la construction Guizani, Lotfi; École de technologie supérieure, Construction Engineering Nollet, Marie-José; École de technologie supérieure, Genie de la construction Tahan, Antoine; École de technologie supérieure, Département de génie mécanique
Keyword:	Limit state design, Seismic base isolation, low temperature, reliability analyses < Struct.Eng. & Constr.Mate, bridges < Struct.Eng. & Constr.Mate
Is the invited manuscript for consideration in a Special Issue? :	Not applicable (regular submission)

SCHOLARONE™
Manuscripts

Title: Seismic Reliability Assessment of Base-Isolated Bridges in Quebec

M. Nassar, L. Guizani, M.-J. Nollet and A. Tahan

Authors:

Mohamad Nassar, Ph.D. Candidate (**corresponding author**)

Address: École de technologie supérieure: 1100 rue Notre-Dame Ouest, Montréal, QC H3C 1k1, Dept. of Construction Engineering

E-mail: Mohamad.Nassar.1@ens.etsmtl.ca

Phone number: +1 514 396 8800

Lotfi Guizani, Professor,

Address: École de technologie supérieure: 1100 rue Notre-Dame Ouest, Montréal, QC H3C 1k1, Dept. of Construction Engineering

E-mail: Lotfi.Guizani@etsmtl.ca

Phone number: +1 514 396 8800

Marie-José Nollet, Professor,

Address: École de technologie supérieure: 1100 rue Notre-Dame Ouest, Montréal, QC H3C 1k1, Dept. of Construction Engineering

E-mail: Marie-Jose.Nollet@etsmtl.ca

Phone number: +1 514 396 8800

Antoine Tahan, Professor,

Address: École de technologie supérieure: 1100 rue Notre-Dame Ouest, Montréal, QC H3C 1k1, Dept. of Mechanical Engineering

E-mail: Antoine.Tahan@etsmtl.ca

Phone number: +1 514 396 8800

Word Count: 134 words

Abstract

The aim of this work is to estimate the seismic reliability of a simple typical two span lifeline base-isolated bridge designed to behave essentially elastic or as per the Canadian Highway Bridge Design Code, for seven localities in Quebec. Two limit states are considered for possible failure due to unacceptable damage: by flexure at pier-base and by displacement within the SIS. Main problem random variables (RVs) considered and modeled are: Seismic hazard, temperature, pier base dimensions and material mechanical properties. The Monte-Carlo method is used to evaluate each limit state reliability and probability of failure. Preliminary results reveal that notwithstanding the large temperature and seismic hazard variabilities between the seven sites in Quebec, the global reliability indices are almost uniform, around 3.45 ± 0.02 . Furthermore, security factor (i.e. 1.25) on SIS displacement capacity results in reliability indices for SIS displacement are not levelled with the flexural reliability indices and needs further exam and consideration.

Keywords: Limit state design, seismic base isolation, low temperature, reliability-based approach, bridges.

1. Introduction

The seismic isolation (SI) of bridges is an earthquake protection method that aims to reduce seismic forces by decoupling the structures of the bridge from ground accelerations during earthquakes to control damage (Guizani 2003, 2007; Guizani and Chaallal 2010; Makris 2019). Many studies (Ghobarah and Ali 1988, 1990; Jangid 2004) have investigated the efficiency of SI devices to reduce the seismic force effects transmitted to the bridge deck/superstructure and found that the structural response of the base-isolated bridge is mainly affected by the SISs' hysteretic properties. Generally, the hysteretic behaviour of base isolation systems is well represented by the bilinear model shown in Figure 1, with two main hysteretic features: the post-elastic stiffness K_d and the characteristic strength Q_d (Buckle et al. 2006).

Many conditions, notably low temperatures, affect the hysteretic behaviour of almost all available seismic isolation systems (SISs), and therefore, the seismic response for base-isolated bridges in cold regions (Yakut 2000; Yakut and Yura 2002; Constantinou et al. 2007). Recent research in Quebec proved that, depending on its intensity and frequency, the temperature may lead to significant instantaneous stiffening of natural rubber used for SI (Ankik 2019; Guay and Bouaanani 2016). These researches also showed that, for elastomeric bearings, such changes are amplified by the crystallisation phenomenon which is function of the duration of exposure to low temperature. The effect of changes in SIS properties on the seismic response of base-isolated bridges in Quebec was studied in (Beji 2015).

Many reliability-based structural analysis methods aim at deriving load and resistance factors (Allen 1992; García-Soto et al. 2015; Kennedy et al. 1992). Limit states design in Canada and the load and resistance factor design (LRFD) in USA are analogous design approaches that were introduced to replace the allowable stress design (ASD) for which all inherent uncertainties are incorporated in one global security factor (FS), chosen on an engineering judgment basis. In 2002, AASHTO (2002) included the load and resistance factor design (LRFD), introduced in Canada and

in many other countries in the 70s (Chen and Duan 2014). Nevertheless, in these approaches, the seismic design load combinations are still not based on a target reliability (Nowak 1993; Szerszen and Nowak 2003).

The current Canadian Highway Bridge Design Code (CAN/CSA-S6:19) proposes the use of a bounding analyses approach to consider the effect of different conditions affecting the SIS properties, such as temperature, aging, scragging etc. In this approach, property modification factors are applied on nominal property values of the SIS in order to calculate their maximal and minimal values, as well as the corresponding base-isolated bridge responses, such as the base shear force and displacement, establishing lower and upper bounds of seismic demand for design purposes (Constantinou 1999; Constantinou et al. 2007; Thompson et al. 2000). Concomitant low temperatures with the design earthquake are specified for each location. Nevertheless, these specified concomitant temperatures, as well as the bounding analyses approach, were not developed on a probability approach basis, and thus, the resulting reliability of the base-isolated bridges designed in this way remains unknown. Further, earlier work showed that the probability of exceedance of the design earthquake concomitant low temperature, specified by the code, varies significantly with location in Quebec, questioning the resulting reliability uniformity (Guay and Bouaanani 2016).

The aim of this study is to estimate the reliability of base-isolated bridges in Quebec, as designed by the current code specifications, based on the bounding analyses, with regard to the possibility that such bridges be damaged by earthquakes.

Many stochastic studies have been performed on sliding SISs and frictional base isolators (Constantinou and Papageorgiou 1990; Constantinou et al. 2007; Kitayama and Constantinou 2019) to understand the behaviour and reliability of such SISs. These studies consider random variables (RVs) in order to account for uncertainties in structure and in ground motion that has a stochastic nature. For these studies, the Monte-Carlo method (MCM) is used because of the

complexity of the problem and the difficulty of analytically modelling the whole process. However, these earlier studies were aimed at specific structures, SISs and locations' seismic hazards.

2. Limit States in Seismic Performance-Based and Probability-Based Design Approaches

The performance-based design (PBD), which originated in New Zealand in the 1970s, and was developed in USA by the 1980s (Rojahn and Sharpe 1985; Hamburger 2004) has now been adopted in an explicit form by many bridge design codes (CSA 2019; NZT 2016). PBD is a design methodology that encompasses all types of engineering structural designs and activities, in order to create structures with a predictable seismic performance that relies on an established risk level (Bathurst et al. 2019; Bertero and Bertero 2002; SEAOC - Vision 2000 1995; Priestley 2000). The performance level defines performance objectives and needs in turn to be associated with quantitative performance criteria (indicators) in order to measure the damage level.

To illustrate the above, Table 1, for example, gives the minimum performance levels required by the CAN/CSA-S6:19, for lifeline bridges, and the corresponding salient damage quantitative criteria expressed in terms of the maximum allowed deformations of concrete (ϵ_c), reinforcing (ϵ_s), and SIS displacement.

The design codes, notably with PBD, use many return periods (RPs) with different seismic performance criteria. The CAN/CSA-S6:19, uses RPs of 475 years, 975 years and 2475 years (i.e. Table 1). The AASHTO uses comparable RPs. Eurocode 8 part 1 (CEN 2004) defines two fundamental requirements: the no-collapse and the minimisation of damage, generally associated to seismic hazard RPs of 975 years and 475 years, respectively.

Both PBD and Force Based Design (FBD) approaches use the limit states to define load combinations, considering security and functionality performance objectives.

However, in order to include the intrinsic uncertainties in resistance, solicitation, modelling and analysis method many authors introduced the probability-based limit state design to quantify the performance of the structures and process inherent uncertainties in a more rational way (Allen 1975; Ellingwood and Galambos 1980; Ellingwood and Galambos 1982). This was translated into load and resistance factors for each type of load and/or resistance, excluding the earthquake load effect (AASHTO-LRFD 2014).

Further, increasingly, the probability associated with the different parameters and the target performance are explicitly used in earthquake engineering for buildings and highway bridges (Ghobarah 2001; Sheikh and Legeron 2014). Nevertheless, for base-isolated bridges, significant research efforts are still needed to develop and apply generally applicable probability-based design approaches, notably considering the effect of different conditions on SISs' properties. To this end, Nassar et al. (2019) presented a reliability assessment method, based on MCM, for base-isolated bridges in cold climate.

3. The Adopted Reliability Assessment Methodology

This paper applies the reliability assessment method proposed by Nassar et al. (2019) to estimate the seismic reliability indices for typical base-isolated bridges in seven different locations in Quebec. For the present study, the temperature, the horizontal seismic hazard and the main parameters affecting the resistance of the bridge components, such as dimensions and material properties are modelled as RVs. Based on literature review and proprietary testing data, continuous probability distributions are affected to these RVs and their relations to SIS hysteretic properties are determined. The seismic performance objectives described in the modern codes are first translated into a vector of m limit state functions $\mathbf{f} = \{f_1, \dots, f_m\}$ that represents the quantitative seismic performances of the studied base-isolated bridge. Therefore, the vector $\mathbf{X} = \{X_1, \dots, X_n\}^T$ of the problem independent RVs is created. A generic structural model of the base-isolated bridge is

created as well. The MCM is then used to generate a large number of realisations of the problems' RVs, with respect to their probabilistic distributions. Each realisation is a vector of sampled simultaneously occurring RVs' values. A unique sample model is created with each realisation values and analysed. Results of the seismic analyses are then used to evaluate the limit state functions, f . The probability of failure, for each limit state function, f_i , is evaluated based on the number of failures to the total generated number ratio, which allows computing the reliability indices for each performance criterion¹.

4. Evaluation of the Reliability of Base-Isolated Bridge

4.1. Base-Isolated Bridge Case Study Design

The above methodology is applied to evaluate the reliability of a reinforced concrete two-span base-isolated bridge, designed as per CSA-S6:19, for seven (7) different localities in Quebec, Canada: Montreal, Gatineau, Quebec City, Roberval, Rivière-du-Loup (RDL), Baie-Comeau and Gaspé. These sites are representative of the different seismic hazard and temperature conditions across the province of Quebec.

The studied bridge is a conventional box-girder bridge, supported by two abutments and a two-column central pier. A header beam couples the two columns of the pier. The total deck's weight is 30 000 kN. The height and the seismic tributary weight of each of the 1.2 m × 1.2 m columns are $H = 5.00\text{ m}$ and $DL = 7500\text{ kN}$, respectively². The bridge is built on a class C site category. The fundamental periods of vibration of the conventional bridge model (fixed-base), with bearings being longitudinally guided at the abutments and fixed in both directions at the central pier, are: $T_{lf} = 0.67\text{ sec}$ and $T_{tf} = 0.21\text{ sec}$ in the longitudinal and transverse directions, respectively.

¹ See Figure S1, in supplementary material

² See Figure S2, in supplementary material

The generic bridge is designed for each site according to CSA-S6:19 specifications. A bilinear model of the base isolation system is assumed, with nominal properties set to achieve the following target nominal properties of the isolated bridge, within elastic range:

1. Fundamental vibration period around 2.0 sec ($T_{li} \approx T_{ti} \approx 2.0$ sec),
2. Equivalent viscous damping ratio around 25 %.

Given every bridge site, the obtained maximum and minimum values of the property modification factors $(\lambda_{max,Q_d})_i$ and $(\lambda_{max,K_d})_i$ specified by both AASHTO (AASHTO 2014) and CAN/CSA S6:19, for effects of temperature on LRB properties, are evaluated at the specified concomitant temperature and used to calculate the hysteretic design properties of the SIS.

Table 2 shows the resulting nominal design values of Q_d and K_d along with the concomitant temperature (CT) and the values of $(\lambda_{max,Q_d})_i$ and $(\lambda_{max,K_d})_i$ according to the CAN/CSA-S6:19 code for each site. The computed lower and upper values of Q_d and K_d are in turn used to obtain bounding values of the seismic demand for each studied site. To compute the seismic demand, the base-isolated bridge is modelled as an equivalent elastic single degree of freedom (SDOF), with the superstructure mass and including the pier flexibility, and a spectral analysis is used with an iterative method (Buckle et al. 2006). The effectiveness of the SDOF model of the bridge is checked against more refined models and the fundamental mode mobilises 95% of the superstructure mass in both horizontal directions. Furthermore, peak seismic demands of the base-isolated bridge model in both horizontal directions are equal (confirmed by a 3D model). This allows reducing the base-isolated bridge model to a unique SDOF, representing both directions.

The central pier is then designed to ensure that its seismic behaviour is governed by bending and brittle failure modes, such as shear and anchorage failures, are discarded by assigning enough resistance reserves to them. The superstructure is assumed to remain elastic.

The SIS is composed of two LRBs at the pier top. Geometrical details, such as rubber outer diameter, rubber effective thickness and lead plug diameter, with nominal properties $(K_d)_i$, $(Q_d)_i$ are illustrated in Figure S3 and listed in Table 2.

4.2. Seismic Performance Criteria and Associated Limit State Functions

According to the Canadian code, a lifeline bridge shall satisfy minimum performance levels for different seismic hazard return periods. Repairable damage and limited service under an earthquake with a RP of 2475 years, are set as a minimal performance. The province of Quebec is a moderate seismicity area where, because of the shape of the design spectrum which vanishes rapidly with period elongation, the seismic base isolation is very efficient. As a result, the elastic seismic force demand for base isolated bridges is almost always more than covered by the minimum reinforcing steel ratio of 0.8% specified by the code. Consequently, base-isolated bridges are designed to respond essentially in the elastic domain, under the 2475 years RP earthquake. In this study, for the main studied cases, called elastic designs, the same philosophy and design performance criterion are used: failure is reached when the seismic demand exceeds the elastic capacity.

However, to illustrate the applicability of the proposed results and methodology for base-isolated bridges meeting the minimal performance specified by the CAN/CSA-S6:19 code, an additional case study, with inelastic behaviour is carried out, for Rivière-du-Loup site. The design of the pier-bridge is revised (i.e. by decreasing steel ratio), allowing inelastic response and deformations, to meet the different criteria described in Table 1. Axial load of the bridge pier column is also decreased by half to obtain a sufficiently high reinforcing steel demand to achieve the target performance. Despite this, a lower than the minimum steel ratio specified by the code is obtained. For the purpose of this study, soil-structure interaction is neglected. Abutments and foundation failures are discarded so that only the central pier can fail. In the latter, it is assumed that only flexural failure at the base can occur, as damage related to any brittle failure mode is discarded.

Further, the displacement capacity of the SIS is a concern, as demonstrated by Kitayama and Constantinou (2019) and Roussis et al. (2003).

Consequently, two limit states related to the pier base damage by bending inelastic deformation and the SIS damage by exceeding its displacement capacity are considered. These are formulated as follows:

- $f_m = R - S$, where R is the flexural resistance at the pier base and S is the maximal seismic demand in terms of flexure at the pier base for elastic designs or the top pier displacement threshold corresponding to the damage level (three damage levels) for the inelastic design.
- $f_D = D_{isol,cap} - D_i$, where $D_{isol,cap}$ is the isolation system displacement capacity and D_i is the displacement demand on the isolation system. Note that displacement resulting from temperature and creep at the central pier vanish, because of the symmetry.

4.3. Bridge Capacity Evaluation

The flexural pier base resistance and the isolation displacement capacity, identified in the limit states above, are obtained through the design values of the bridge, as per CAN/CSA-S6:19.

The displacement capacity of the seismic isolator $D_{isol,cap}$ is taken equal to the seismic maximal displacement demand at the isolation system, $D_{isol,E}$, multiplied by a security factor of 1.25.

Since the responses in the two directions are equal (see 4.1.) only one spectral analysis is performed, allowing computation of the shear forces in the two horizontal directions $V_{E,x}$ and $V_{E,y}$. These shear

forces, acting at the superstructure centre of mass, are translated in terms of bending moments and axial forces, for both directions³.

For the transverse direction, the effect of the vertical eccentricity of the seismic force action line relative to the isolation units (hinge), e_v , is taken into account by an adjustment of the axial force at the pier columns to maintain equilibrium. For the longitudinal direction, this effect is negligible because of the very large ratio between the bridge length and the vertical eccentricity, e_v . Thus, the shear force is applied at the isolation bearing (hinge), which is taken at the top of the pier.

The shear forces, and consequently, the resulting forces at the pier base, in the two horizontal directions are then combined according to the 100% - 30% rule recommended by the CAN/CSA-S6:19: 100% in one direction occurs simultaneously with 30% in the other direction, and vice versa, to obtain seismic forces demand. The seismic forces in the vertical direction are included in the load factors applied to dead load. Consequently, the bending moment capacity at the column base in the transverse direction is designed for the moment demand at the following two levels of axial forces: $N_1 = 1.25 \text{ DL} + \Delta N$ and $N_2 = 0.8 \text{ DL} - \Delta N$.

For simplicity and given the magnitude of ΔN relative to DL ($\frac{\Delta N}{\text{DL}} \cong 1\%$), the same axial levels are also used for the longitudinal direction to get a symmetrical section design. SP-column (STRUCTUREPOINT 2017) was used to this end. The obtained interaction diagrams (at the two levels of axial load) for this practical design are typically higher than the seismic demand because of practical considerations (availability of reinforcement at discrete diameters only). However, in order to investigate the reliability of an isolated bridge as warranted by the design code, a theoretical design is used, where the bending resistances $M_{rx} = 1.0 M_{Ex}$ and $M_{ry} = 1.0 M_{Ey}$. Therefore, the diagram obtained for the practical design are scaled to fit the exact nominal demand and is used as

³ See Figure S4, in supplementary material

the theoretical capacity. Linear interpolation is used to build a 3D interaction diagram: $M_x; M_y; N$, for axial loads between and outside the pair N_1 and N_2 .

For the PBD inelastic design, the nominal capacity curve, base shear-top of pier displacement (i.e. $V-D_p$ curve) shown in Figure 2 is obtained through integration of sectional response along the pier height, using Response 2000 software with tension stiffening option (Bentz 2000). The concrete Popovics/Thorenfeldt/Collins base curve (Mitchel and Collins 1991) and Vecchio-Collins (1986) concrete compression softening model are used and the CSA G30.12 400 MPa stress strain curve is used for steel (Bentz 2000).

Furthermore, the sectional response at the pier base relates the shear base and corresponding bending moment to the steel and concrete deformations at this critical section, which are damage indicators. Considering the superposed steel deformations at corner rebars, rising from 100%-30% simultaneous shear forces acting in both directions, three pier top displacements, D_p , under unidirectional loading, corresponding to the three damage thresholds (elastic limit, minimal damage and repairable damage) are identified. The displacements from results obtained for MC realisations are therefore compared to these three thresholds to identify the damage level for each realisation and thereby compute the probability of failure by bending at the pier base.

To solve the nonlinear problem with the spectral analysis, the $V-D_p$ pier response curve is linearized, in a similar way to the response of the isolation system, after application of the uncertainty and its idealisation by a bilinear curve. Thus, the obtained two components series system that behaves in the inelastic range (i.e. SIS and pier system), can be solved by an interactive nested loop scheme. Damping from the pier inelastic response is included in the analysis.

4.4. Problem RVs and their Probabilistic Distributions

The RVs of the problem are:

- The geometric and material properties of the central pier, such as the concrete compressive strength f'_c , the steel yield stress f_y , the depth of the pier section h , and the tributary seismic weight DL ;
- The effective temperature inside the seismic isolator T_{eff} ;
- The seismic hazard through the seismic spectral acceleration value S_a .

Their probability distribution functions are hereafter described.

4.4.1. Flexural Resistance Model, Including Geometric and Material Properties Variability

The uncertainties of the geometric and material properties are indirectly considered in this study⁴. The used probabilistic distributions of these RVs are based on statistical data collected in the USA (Wisniewski et al. 2009), assumed to be representative of the current scatter and tolerances of the reinforced concrete construction industry in Canada.

Using the distributions for geometric and material properties of the above RVs and MCM simulations results combined to complementary information about source errors originating from modelling, Wisniewski et al. (2009) proposed a resultant probabilistic bending resistance (yield moment) model as follows:

⁴ See Table S1 in supplementary material

$$R = R_n M F P \quad (1)$$

where R_n is the nominal cross section resistance/response. The terms M , F consider the uncertainties associated to material properties and geometrical properties, including fabrication tolerances, P represents uncertainties arising from the professional treatment, related to the material and structural behaviour modeling and used equations to predict the section resistance. Terms M and F are combined in their product $M F$.

This study uses this resulting probabilistic resistance model for reinforced concrete section and for moment limit state. R follows a Normal random variable with a Bias = 1.19 and Coefficient of variation of 0.09⁵. Applying these resistance model parameters to the adjusted interaction diagram of the elastic designs or to the ordinates of the nominal V - D_p response curve of the inelastic design gives the statistical distribution of the resistance.

The application of this model, originally developed for beams, to the studied bridge piers is justified by the low axial load level.

4.4.2. Probabilistic Temperature Model

The instantaneous air temperature, T_{amb} , at the time of an earthquake is, indirectly, considered as an RV of the problem. Because the material properties and consequently SI properties are related to the effective internal temperature of the isolator, T_{eff} . The hourly ambient temperature distributions are therefore transformed into hourly effective temperature distributions by Laplace Transform, calibrated on results of transient heat transfer analyses on refined finite element (FE) models of the isolation units along the days preceding the earthquake (Niemann and Miklos, 2014), as detailed in (Nassar et al. 2019).

⁵ See Table S2 in supplementary material, for statistical parameters.

Figure 3 shows the input (T_{amb}) and output temperature (T_{eff}) probability distributions for Montreal and Rivière du Loup sites, after fitting nonparametric probabilistic distributions (Everitt 1985)⁶.

4.4.3. Probabilistic Seismic Hazard Model

The seismic hazard RV values and parameters are based on the 5th generation seismic hazard model for Canada (Halchuk et al. 2014). The seismic hazard, for each site, is represented in terms of spectral acceleration, S_a , as a continuous function of the structure period of vibration (T) and return period (RP). Assuming linear variation of the $S_a(T)(g)$ between the discrete RP values, the RP of the earthquake can be assigned to any sampled spectral acceleration value. Linear extrapolation is used to calculate the hazard values for RPs higher than 2475 years. It should be mentioned that this extrapolation of the model is uncertain and may be unreliable (e.g. due to the crudeness of the seismic source zones) (Halchuk et al. 2015).

For a given RP, the spectral acceleration value at any vibration period is obtained by linear interpolation between values at discrete periods given by the uniform hazard spectrum (UHS) of Canada. For periods higher than 2.0 sec, the spectral displacement is first interpolated and the spectral acceleration is then deduced (CAN/CSA-S6:19).

Furthermore, an initial estimate of the epistemic uncertainty is included in the Ground Motion Prediction Equations (GMPE) for Eastern Canada, as proposed by Atkinson and Adams (2013). To include this epistemic uncertainty, in the current study, the value of the spectral acceleration $S_a(T)$ is considered as a normal RV with mean value μ_{S_a} equal to the mean hazard value of $S_a(T)$, and a standard deviation (SD) of $\sigma_{S_a} = 0.5 \mu_{S_a}$. Such normal distribution of $S_a(T)$ generates less than

⁶ See Figure S5, in supplementary material, for other sites.

2.3% negative values which are set to zero. A uniform hazard spectrum is derived for the sampled $S_a(T)$ value as detailed by Nassar et al. (2019).

4.5. Structural Model and Relations with RVs

4.5.1. Structural Model

The nonlinear base-isolated bridge behaviour in both the longitudinal and transversal directions is modelled by an equivalent linear SDOF model shown in Figure 4.

4.5.2. Relations Between Temperature and Structural Model Properties

The effect of temperature on the seismic response of the base-isolated bridges is considered through the relation between the temperature and the structural model properties, as follows.

a) Reinforced Concrete

The effect of temperature variation, within the studied range, on the mechanical properties of concrete and reinforcing steel is not significant, and is neglected (El-Tayeb et al. 2017).

b) Lead

Earlier studies showed that the lead plug has an almost perfect elasto-plastic behaviour with a yield limit, σ_{yl} , significantly affected by temperature (M. Constantinou et al. 2007). This study adopts the relation between lead initial effective temperature, T_{eff} , and its yield limit, σ_{yl} , proposed by Kalpakidis and Constantinou (2008):

$$\sigma_{yl}(T_{eff}) = \sigma_{yl_0} e^{-0.0069(T_{eff} - 20)} \quad (2)$$

where σ_{yl_0} is the initial yield limit at $T = 20^\circ\text{C}$.

Consequently, the characteristic strength (Q_d) is related to lead temperature by:

$$Q_d(T_{eff}) = \left(1 - \frac{K_d}{K_u}\right) A_l \sigma_{yl_0} e^{-(0.0069(T_{eff} - 20))} \quad (3)$$

Where A_l is the lead core section area, $\frac{K_d}{K_u}$ is the tangent to initial stiffness ratio of the isolation unit, set to the typical value of 0.1 for LRBs. The heating of lead core under cycling is not considered in this study (Ozdemir 2014).

c) Rubber

The lateral stiffness of an LRB is proportional to the rubber shear modulus, which in turn varies with the temperature. The relations proposed by Cardone and Gesualdi (2012) are extended by the authors to cover a larger range, and are used to relate the shear modulus of rubber to its effective temperature. The lateral stiffness of an LRB unit is therefore obtained by (Buckle et al. 2006):

$$K_d(T_{eff}) = 1.1 \frac{G_0 A_b}{t_r} C(T_{eff}) \quad (4)$$

Where, A_b is the rubber net bonded area, t_r is the rubber thickness, G_0 is the rubber shear modulus at $T = 20^\circ\text{C}$, and $C(T_{eff})$ is an adjustment factor considering the rubber shear modulus variation with temperature, given by:

$$C(T_{eff}) = \begin{cases} 0.0005 T_{eff} - 0.03 T_{eff} + 1.4 & T_{eff} \leq +30^\circ\text{C} \\ -0.00058 (T_{eff} - 30) + 0.95 & T_{eff} > +30^\circ\text{C} \end{cases} \quad (5)$$

4.6. Generation of Realisations and Computation of Probability of Failure

Using Matlab® (Statistics and Machine Learning Toolbox), $N \geq 1 \times 10^6$ MCM realisations are generated following the above-defined RVs distribution functions. Then, for each realisation, single mode spectral analysis (ESA) is carried out to compute the specific SDOF peak response in terms of displacement and bending moment/shear at the pier base.

For the elastic designs, after the above demands are computed in the two horizontal directions, the pier base bending moment limit state relative to this demand point ($M_{E,x}; M_{E,y}$) where $M_{E,y} = 0.3 M_{E,x}$ is evaluated using the resistance M_x, M_y capacity interaction diagram for the axial load level, as shown in Figure 5. If $M_{R,y}$ is less than $M_{E,y}$, the pier base bending capacity has been exceeded, and a FAILURE event (damage) is considered; otherwise, a SAFE event (no damage) is considered.

For the inelastic design, failure is obtained if the top-pier displacement exceeds the displacement threshold corresponding to the considered damage level.

For the SIS displacement limit state, a failure event is considered when the SIS displacement demand exceeds the displacement capacity of the seismic isolator.

Therefore, for a given limit state, the total number of failed cases divided by the total number of realisations represents the annual probability of failure P_f .

Figure 6 shows the 3D illustrations of the demand output for Montreal and Rivière-du-Loup sites⁷ elastic designs, in terms of moment $S = M_{E,y} = 0.3 M_{E,x}$ and the resistance R that corresponds to $M_{R,y}$.

The global reliability refers to the overall reliability of the base-isolated bridge system and is computed by considering the two limit states (f_m : flexural; f_D : displacement) as two non-mutually exclusive events (i.e. a two components series system). Consequently, a global bridge system failure event is obtained when one or the other limit states is at failure. Global reliability index is then deduced from the probability of such system failure.

5. Results and Discussion for Elastic Designs

Table 3 presents the computed probabilities of failure for bending (P_{fM}) and displacement (P_{fD}) limit states as well as their reliability indices β_M and β_D for all the elastic designs, for $N = 10^6$ and $N = 10^7$ realisations. The reliability indices computed for the $N = 10^7$ are used in the following discussion. Table 3 reports the bending capacity of the pier column bases.

It should be noted that all conclusions are made by comparing the analysis results indicators to the design criteria used (i.e. no damage and an elastic response).

⁷ See Figure S6, in supplementary material, for other sites

5.1. Seismic Reliability of the Studied Cases

As shown in Table 3, the reliability indices for the two limit states β_M and β_D of the case study, and therefore the global reliability indices, are relatively uniform. This is in spite of the results of a recent study showing that the probabilities of temperature being less than concomitant temperature, prescribed by the code, are non-uniform, and vary significantly between sites in the province (Guay and Bouaanani 2016).

In addition, we note that the reliability index for the SIS displacement limit state, β_D , is always lower than that for the pier base bending limit state, β_M , for all studied cases. This indicates that there would be a higher margin of safety for bending resistance, due notably to the bias applied to the bending resistance model, than for SIS displacement capacity, despite the application of a multiplier security factor of 1.25 for the displacement capacity. A similar result was obtained by (Nassar et al. 2019), pointing to the need to revisit the factor of 1.25, which, in turn, affects the estimated global reliability of the base-isolated bridge system.

Nevertheless, the currently provided SIS displacement capacity would have a variability and a bias relative to their nominal design value that should be considered as an RV. However, because of lack of data, such bias and variability are not included in this study and consequently any conclusions should be interpreted with caution to this regard.

As shown in Table 3, the global reliability index is lower than the reliability index for any limit state, with the exception of the Roberval site. In the latter, the global reliability index equals the lowest reliability index of all limit states (displacement limit state, in this case), which is explained by the fact that most of bending failures happened simultaneously with displacement failures. Nevertheless, the greater the number of limit states and possibilities of failures included in the analysis, the lower the expected global reliability index. This means that the results obtained from

this simple case study are an upper bound of a warranted theoretical reliability index. Then, for more complex bridge systems, lower global reliabilities would be expected.

However, applying a flexural capacity reserve (as per engineering practice) should increase the reliability index of the flexural limit state for all cities. In our case, applying a multiplication factor of 1.2 to the bending capacity results only in an increase of between 2% and 5% of the bending limit state reliability index, despite a decrease of between 32% and 38% of this limit state probability of failure. Moreover, the global reliability index remains practically constant (less than 0.2% increase). This is because the reliability index of the displacement limit state, which governs the global reliability index, remains constant and the fact that reliability index is more robust and varies less than the probability of failure (Nowak and Collins 2000).

The above results indicate the need to adopt a target global seismic reliability index for the seismic design of bridges. Further, in the opinion of the authors, the different limit states should be calibrated for the same or comparable target reliability index with regard to consequences of failure, complexity of the bridge and its importance. It should be noted that a too low target reliability for any limit state will lower the global reliability of the bridge. Brittle failure modes can be discarded with the use of higher reliability indices.

Besides, in order to show the impact of the concomitant temperature used for the design and the computation of the nominal value of the resistance moment, Nassar et al. (2019) included three case studies for Montreal site. The first without considering the minimal concomitant temperature ($T_1 = +20^\circ\text{C}$), the second where the concomitant temperature was considered as prescribed by the code (i.e. equal to the one used for Montreal site in this study) ($T_2 = -9.5^\circ\text{C}$), and the third where concomitant temperature is set equal to the minimum mean daily temperature ($T_3 = -34^\circ\text{C}$) (CAN/CSA, S6:19). The results show that flexural reliability indices increase from 3.74 to 3.89 to 4.11 for concomitant temperatures T_1 , T_2 and T_3 , respectively.

5.2. Effect of Temperature Distribution and Uncertainties on Bridge Seismic Reliability

To further investigate the effect of the hourly temperature probabilistic distribution on the probability of failure and/or the reliability index of the bridge, Table 4 shows the probabilities (%) of the temperature T_{amb} and T_{eff} being less than the concomitant temperature CT for the studied sites. As indicated, the average probability of T_{amb} and T_{eff} being less than CT is around 14.1% and 17.6%, respectively.

Table 4 also compares the probability of the effective temperature being less than the CT at failure for the seven studied cases. Averagely, this probability is estimated to 20% for the seven sites. This means that the temperature has a non-negligible effect on the response of the studied bridges in the seven different sites. Therefore, the effect of the low temperature on the reliability of the studied bridges is developed in the following.

First, results show that the linear correlation coefficient (Weisstein 2006) between the global probability of failure and the probability of the effective temperature being less than the CT for the studied sites, is around 7%. Then, it can be seen that the temperature decrease does not have a considerable effect on the seismic base-isolated bridge probability of failure.

Second, despite the fact that the City of Roberval site has the highest probability of $T_{eff} < CT$, it does not have the highest probability of $T_{eff} < CT$ at failure. Moreover, although the ambient temperature is lower than CT in 30% of the failure cases for Quebec City, and almost half of this percentage is estimated for the City of Roberval (18%), the displacement reliability index β_D is higher in the case of Quebec City (3.52 vs. 3.44). This seemingly unexpected result is explained by the fact the more the temperature decreases, the more the main hysteretic property values of the SIS increases (stiffening the isolation system), which results in a lower displacement demand on the seismic isolator units at the pier, and consequently a higher reliability index β_D is obtained.

Further, despite the reliability β_D being higher in the case of Quebec City as compared to the case of Roberval, the global reliability indices are almost similar for both sites. This is due to the flexural limit state effect on the global reliability and therefore the code's recommendations regarding temperature, that mainly levelled the global reliability of the base-isolated bridge for the studied cases.

5.3. Effect of the Earthquake RP on the Seismic Reliability of the Bridge

The estimated mean value, for all sites, of the earthquake RP causing failure is 3838 years for SIS displacement and 2290 years for flexural limit state⁸. Although the design RP is set equal to 2475 years for all sites, seismic hazard RP that causes displacement limit state failure exceeds this design RP for all generated realisations.

As shown in Figure 7.a. for the case of Montreal site, flexural failures of the base-isolated bridge for RP between 1380 years and 2475 years correspond to low temperatures at failure respectively between [-38°C ; -10°C]. This is due to the low temperature effect which increases the force demand.

In fact, the SIS is designed to supply the displacement demand associated to the RP of 2475 years. Low temperature makes the isolation system stiffer and thus reduces the displacement demand. Consequently, as shown in Figure 7.b SIS displacement failure occurs only when the input seismic hazard is higher than the design value. However, for the flexural moment limit state, low

⁸ See Table S3, in supplementary material for more details.

temperature makes the SIS stiffer but, at the same time, increases the force demand causing failure by bending moment at the pier base for lower seismic hazard values than the design.

This observation allows defining and limiting the domain where displacement failure and/or flexural failure can occur. It is therefore very useful to the application of the used methodology to complex bridges, where more refined analyses on refined models are required to evaluate the reliability of the bridge relative to different performance criteria which may require non-linear time history analysis, because it allows reducing the number of sampled realisations to analyse. It should be noted that the results vary depending on the model linking the seismic isolation properties to temperature.

5.4. Effect of the Earthquake Seismic Hazard Uncertainty Probabilistic Model on the Seismic Reliability

Total uncertainty on seismic hazard values originates from two sources: epistemic uncertainty on GMPE models and aleatory uncertainty. The latter is related to the error on earthquake rate prediction/assumption. Although the fact that epistemic error is the main source of uncertainty in moderate seismic areas (Adams and Halchuk 2019), aleatory uncertainty cannot be neglected.

In order to include the aleatory source of uncertainty, the total uncertainty is modelled based on Adams and Halchuk (2019) estimate of its spread around the mean seismic hazard value for only few ENA localities (e.g. Montreal, La Malbaie and La Pocatière sites). To evaluate the effect of the modelling of seismic hazard uncertainty on results, analyses with a lognormal distribution model of the total uncertainty with the following parameters is carried out for two sites: Montreal and Rivière-du-loup: For each location, the mean value of the seismic hazard is set to code mean value and the SD of the distribution is given by following equation:

$$\frac{1}{4}(\log(S_a(0.2 \text{ sec}) \text{ at } 98^{th} \text{ percentile}) - \log(S_a(0.2 \text{ sec}) \text{ at } 2^{th} \text{ percentile})) \quad (6)$$

The same lognormal distribution SD is applied to seismic hazard associated to other period values $S_a(T)$ (e.g. $S_a(2 \text{ sec})$) as per Adams and Halchuk (2019) recommendations.

As a result, the reliability indices of the flexural limit state increase by 1.1% from $\beta_{M_norm} = 3.56$ when using a normal distribution to $\beta_{M_logn} = 3.60$ when using a lognormal distribution (i.e. 13.5% decrease in the probability of failure) for Montreal site and increase by 2.5% from $\beta_{M_norm} = 3.52$ to $\beta_{M_logn} = 3.61$ (i.e. 29% decrease in the probability of failure) for Rivière-du-Loup site as shown in Table 5. For seismic reliability indices of the displacement limit state, similar increases are observed, that is 1.5% for Montreal site (i.e. from $\beta_{D_norm} = 3.50$ to $\beta_{D_logn} = 3.55$) and of 2% for Rivière-du-Loup site (from $\beta_{D_norm} = 3.50$ to $\beta_{D_logn} = 3.57$). The global reliability indices increase therefore from $\beta_{norm} = 3.47$ to $\beta_{logn} = 3.52$ (i.e. 16% decrease in the probability of failure) for Montreal and from $\beta_{norm} = 3.44$ to $\beta_{logn} = 3.50$ in Rivière-du-Loup (i.e. 20% decrease in the probability of failure). This increase in reliability indices for both flexure and displacement is due to the decrease in the number of occurrences of seismic hazard values higher than the mean value used for the design of the bridges as explained in the following.

Figure 8. compares the spread of the $S_a(0.2 \text{ sec})$ for Montreal site using two models: 1. the lognormal distribution model of the total uncertainty with mean value $= 0.587 \text{ (g)}$ and average SD (in $\log_{10}$ units) equal to 0.300, which is equivalent to $SD = 2.000$ (Adams and Halchuk 2019) and ; 2. the normal distribution model of the epistemic uncertainty based on Atkinson and Adams (2013), with mean value $= 0.587 \text{ g}$ and $SD = 0.293 \text{ g}$.

For Montreal, it is shown that the probability that $S_a(0.2 \text{ sec})$ does not exceed 1.5 g is 99.96% for the normal distribution and 95% for the lognormal distribution. Also, for the lognormal distribution, the probability that the seismic hazard exceeds its mean value is equal to 35% while it equals 50% for the normal distribution. For Rivière-du-Loup site, which is comparable to the La

Pocatière site (Adams and Halchuk 2019), this latter probability is around 29% when the distribution of the $S_a(0.2 \text{ sec})$ follows a lognormal probabilistic distribution.

It should be mentioned that code's percentile for mean $S_a(0.2 \text{ sec})$ value in Montreal is equal to 71.5% (Adams and Hachuck 2019). Therefore, the probability of exceeding the value of $S_a(0.2 \text{ sec}) = 0.587 g$ is equal to 28.5% compared to 35% and 50% in our lognormal and normal models respectively. Consequently, as the probabilities of exceeding the mean value are higher in these models, it is expected that resulting reliability indices are overestimated for the studied sites, especially for the normal distribution which results in a higher probability of exceeding the average value, used in design.

However, the obtained results cannot be generalized to any performance criteria because despite the fact that the normal distribution results in more frequently exceeding the design value, the lognormal distribution generates more extreme values (Figure 8, e.g. probability of $S_a(0.2 \text{ sec})$ exceeding $1.5 g$ is about 5% while it is only 1% for normal distribution). Hence, for severe and more advanced damage levels, the reliability indices should be lower with the lognormal distribution.

6. Results and Discussion for the Inelastic Design

For the inelastic design, probabilities of failure and reliability indices for the different damage levels at the pier base and SIS displacement limit state are computed and compared to the elastic design results for the RDL site (for the lognormal seismic hazard uncertainty model).

A Probability of exceedance of the elastic nominal capacity of the pier base of 1.101×10^{-3} and a reliability index of 3.06 are obtained. As expected, because this inelastic design has a lower force capacity than the elastic designs, a much higher probability of damage and a much lower reliability index are obtained, compared to values shown in Table 5 (i.e. $P_{fM_{logn}} = 0.153 \times 10^{-3}$ and $\beta_{M_{logn}} = 3.61$, respectively). As the damage level increases, the probability of failure and the reliability

of the pier base increase so that a probability of exceedance of the minimal damage of 0.343×10^{-3} and a reliability index of 3.39 are obtained while for a repairable damage level, these values are 0.126×10^{-3} and 3.66, respectively. These latter values are much more comparable to those associated to the elastic design for exceeding of its elastic capacity ($P_f = 0.126 \times 10^{-3}$ vs $P_{fM_logn} = 0.153 \times 10^{-3}$ and $\beta_M = 3.66$ vs $\beta_{M_logn} = 3.61$). This is because both of these designs used these two damage limits under the same design earthquake RP.

Finally, the SIS displacement limit state shows a probability of failure $P_f = 0.206 \times 10^{-3}$ and a reliability index of $\beta = 3.53$, which are comparable, but slightly higher probability and lower reliability than the values obtained for the elastic design (respectively $P_{fD_logn} = 0.178 \times 10^{-3}$ and $\beta_{D_logn} = 3.57$). Further, in a similar way to the elastic design cases, the SIS displacement limit state governs the global reliability of the bridge for the repairable damage level limit state and the global reliability of the bridge system is lowered to 3.45 which is comparable to results obtained for the elastic designs.

Note that the performance criterion regarding the residual displacement is not directly evaluated. However, Nguyen and Guizani (2020) showed that residual displacement for SISs conforming to the minimum restoring capacity should not exceed 6 to 10 mm in Quebec, which is manageable without the need of an intervention to re-centre the bridge.

7. Conclusion

This paper presents the seismic reliability of LRB base-isolated bridges through the application of a probability-based assessment method on a two-span case study base-isolated bridge, designed according to the Canadian CAN/CSA-S6:19 code, for seven different sites in Quebec, Canada. The adopted methodology relies on MCM simulations. Temperature, seismic hazard and bending resistance of the central pier are the main problem RVs considered. Their probabilistic distributions

and their relations to the bridge model properties, notably isolation system properties are defined. An epistemic uncertainty is applied on the seismic hazard model for each city. Uncertainties associated with material properties and pier dimensions are also indirectly modelled. The Laplace transform is used to estimate the effective temperature of the LRB unit, calibrated by FE heat transfer results. The flexural and displacement limit states are considered, and their reliabilities are calculated for the seven studied base-isolated bridges. A large number of $N = 10^7$ sampled realisations are analysed to deduce the failure probability and the associated reliability index for each of the two studied limit states.

The studied bridges are designed to respond essentially in the elastic range under a 2475 years RP earthquake. However, an inelastic design case, tuned to meet the minimal performance of lifeline bridges is also carried out and investigated.

Results show that:

- The seismic global reliability indices for the studied bridges, designed to respond essentially elastic, associated to no damage state, are almost uniform, and vary between 3.44 and 3.47. In fact, the global reliability is governed by the SIS displacement limit state reliability, which is more uniform for all sites (Table 3). This uniformity is because the SIS displacement capacity ($D_{isol, cap}$) and maximum demands from MC realisations analyses (D_i) are obtained under warm temperatures and high RP earthquakes. Low temperatures are associated to low displacement demands and high force demands. Thus, any discrepancy in the concomitant low temperature or low temperature distributions from one site to the other does not affect the reliability of the SIS displacement limit state and thereby affects less the global reliability.
- Comparatively, inelastic design results show the decrease of the reliability related to bending damage for low damage levels but a similar reliability index for repairable damage level. They also show that the SIS displacement limit state have a very comparable

reliability index and still governs the global reliability of the bridge system, in a similar way to elastic designs. Consequently, authors believe that uniform overall reliability indices should be obtained for inelastic designs.

- The multiplier security factor of 1.25 recommended by the current CAN/CSA-S6:19 (CSA 2019) code does not level the reliabilities of the SIS displacement and flexural limit states for both elastic and inelastic designs. To level the reliabilities of the two studied limit states, this multiplier factor needs further examination and possible review.
- Averagely, the seismic hazard RP at flexural failure is less than the RP design value of 2475 years. For these failures it was found that low temperature has a significant effect and damage can occur at earthquake RP as low as 1300 years simultaneous with a very low temperature. For SIS displacement failure, seismic hazard is the main cause of failure.
- For Montreal and Rivière-du-Loup sites, the total seismic hazard value spread, including epistemic and aleatory uncertainties is modelled as lognormal probabilistic distribution and calibrated to match available data from Adams and Halchuk (2019). It is found that the model of the seismic hazard uncertainty has a significant effect on the probability of failure and much less effect on the reliability indices.

Finally, a sensibility analysis must be carried out in future work to investigate the influence of variation models and the analysis method approximations on the estimated seismic reliability indices of base-isolated bridges.

Acknowledgements

The financial support from the École de technologie supérieure (ETS), the Natural Sciences and Engineering Research Council (NSERC) and the Fond Québécois de Recherche sur la Nature et les Technologies (FRQNT) are gratefully acknowledged.

Draft

References

- Adams, J., Halchuk, S., Allen, T., and Rogers, G. 2015. Canada's 5th generation seismic hazard model, as prepared for the 2015 National Building Code of Canada". Dans *Proc., 11th Canadian Conference on Earthquake Engineering, Victoria, BC*.
- Adams, J., and Halchuk, S. 2019. Uncertainty spread in the 5th Generation seismic hazard results used in NBCC2015. In *Proceedings of 12th Canadian Conference on Earthquake Engineering, Quebec City, June 2019*. pp. 17-20. 2019.
- Alipour, A., Shafei, B., and Shinozuka, M. 2012. Reliability-based calibration of load and resistance factors for design of RC bridges under multiple extreme events: Scour and earthquake. *Journal of Bridge Engineering*, **18**(5), 362-371.
- Allen, D. 1992. Canadian highway bridge evaluation: reliability index. *Canadian Journal of Civil Engineering*, **19**(6), 987-991.
- Allen, D. E. 1975. Limit states design-A probabilistic study. *Canadian Journal of Civil Engineering*, **2**(1), 36-49.
- American Association of State Highway and Transportation Officials (AASHTO) 2002. *Standard specifications for highway bridges*. Washington, D.C, USA: American Association of State Highway and Transportation Officials.
- American Association of State Highway and Transportation Officials (AASHTO) 2014. *Guide Specifications for Seismic Isolation Design*. GSSID 4th Edition. Washington, D.C, USA: American Association of State Highway and Transportation Officials.
- American Association of State Highway and Transportation Officials (AASHTO) 2017. *AASHTO LRFD bridge design specifications*. AASHTO LRFD 8th Edition. Washington, D.C, USA: American Association of State Highway and Transportation Officials.
- Ankik, D. 2019. *Comportement sous basses températures des composés en élastomère utilisés dans les isolateurs sismiques de ponts*. (Mémoire de maîtrise électronique, Montréal, École de technologie supérieure). Available from: <https://espace.etsmtl.ca/1772/>
- Atkinson, G. M., and Adams, J. 2013. "Ground motion prediction equations for application to the 2015 Canadian national seismic hazard maps". *Canadian Journal of Civil Engineering*, **40**(10), 988-998.
- Bathurst, R. J., Allen, T. M., Miyata, Y., Javankhoshdel, S., and Bozorgzadeh, N. 2019. Performance-based analysis and design for internal stability of MSE walls. *Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards*, 1-12.
- Beji, A. 2015. *Effet de la variation des paramètres caractéristiques des isolateurs sismiques affectés par la température sur le comportement des ponts isolés*. (M.Sc. thesis, École de technologie supérieure, Montréal, QC).
- Bentz, E. C. 2000. Sectional analysis of reinforced concrete members (p. 310). University of Toronto.
- Bertero, R. D., and Bertero, V. V. 2002. Performance-based seismic engineering: the need for a reliable conceptual comprehensive approach. *Earthquake Engineering and Structural Dynamics*, **31**(3), 627-652.
- Buckle, I. G., Constantinou, M. C., Diceli, M., and Ghasemi, H. 2006. *Seismic isolation of highway bridges*, MCEER, University at Buffalo, USA.
- Canadian Standard Association 2019. *Canadian Highway Bridge Design Code*. Norme CSA S6:19. Mississauga, ON: Canadian Standards Association.
- Cardone, D., and Gesualdi, G. 2012. Experimental evaluation of the mechanical behavior of elastomeric materials for seismic applications at different air temperatures. *International Journal of Mechanical Sciences*, **64**(1), 127-143.
- CEN 2004. European standard EN 1998-1:2004. Eurocode 8: design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings. Comité Européen de Normalisation, Brussels.

- Chen, W.-F., and Duan, L. 2014. *Bridge Engineering Handbook: Construction and Maintenance* (2e éd.). Florida, USA: CRC press.
- Collins, M.P. and Mitchell, D. 1991. *Prestressed Concrete Structures*, Prentice-Hall.
- Constantinou, M., and Papageorgiou, A. 1990. Stochastic response of practical sliding isolation systems. *Probabilistic Engineering Mechanics*, **5**(1), 27-34.
- Constantinou, M. C., Tsopelas, P., Kasalanati, A., and Wolff, E. D. 1999. *Property modification factors for seismic isolation bearings*. Buffalo, NY: MCEER, University at Buffalo, the State University of New York.
- Constantinou, M. C., Whittaker, A., Kalpakidis, Y., Fenz, D., and Warn, G. P. 2007. *Performance of seismic isolation hardware under service and seismic loading*. Buffalo, NY: MCEER, University at Buffalo, the State University of New York.
- El-Tayeb, E. H., El-Metwally, S. E., Askar, H. S., and Yousef, A. M. 2017. Thermal analysis of reinforced concrete beams and frames. *HBRC Journal*, **13**(1), 8-24.
- Ellingwood, B., and Galambos, T.V. 1982. Probability-based criteria for structural design. *Structural safety*, **1**(1), 15-26.
- Ellingwood, B., and Galambos, T. 1980. *Development of a Probability Based Load Criteria for American National Standard A58, Building Code Requirements for Minimum Design Loads in Buildings and Other Structures*. National Bureau of Standards.
- Everitt, B. S. 1985. *Mixture Distributions—I, Encyclopedia of statistical sciences*, 7.
- García-Soto, A., Hernández-Martínez, A., and Valdés-Vázquez, J. G. 2015. Probabilistic assessment of a design truck model and live load factor from weigh-in-motion data for Mexican Highway bridge design. *Canadian Journal of Civil Engineering*, **42**(11), 970-974.
- Ghobarah, A., and Ali, H. 1988. Seismic performance of highway bridges. *Engineering Structures*, **10**(3), 157-166.
- Ghobarah, A., and Ali, H. 1990. Seismic design of base-isolated highway bridges utilizing lead-rubber bearings. *Canadian Journal of Civil Engineering*, **17**(3), 413-422.
- Ghobarah, A. 2001. Performance-based design in earthquake engineering: state of development. *Engineering Structures*, **23**(8), 878-884.
- Guay, L.-P., and Bouaanani, N. 2016. Assessment of low temperature exposure for design and evaluation of elastomeric bridge bearings and seismic isolators in Canada. *Canadian Journal of Civil Engineering*, **43**(9), 851-863.
- Guizani, L. 2003. Sur l'isolation sismique des ponts au Canada. *10e Colloque sur la progression de la recherche québécoise sur les ouvrages d'art*, Quebec, Canada.
- Guizani, L. 2007. Isolation sismique et technologies parasismiques pour les ponts au Québec: Mise au point. In *Proceedings of the 14e Colloque sur la progression de la recherche québécoise sur les ouvrages d'art*, Quebec City, May 2007.
- Guizani, L., and Chaallal, O. 2010. Mise en conformité sismique des ponts par isolation de la base—Application au pont Madrid au Québec. *Canadian Journal of Civil Engineering*, **38**(1), 1-10.
- Halchuk, S., Allen, T., Adams, J., and Rogers, G. 2014. Fifth generation seismic hazard model input files as proposed to produce values for the 2015 National Building Code of Canada. *Geol. Surv. Canada Open File*, 7576, 15.
- Halchuk, S., Adams, J., and Allen, T. I. 2015. Fifth generation seismic hazard model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada. *Geol. Surv. Canada Open-File Rept.* 7893.
- Hamburger, R., Rojahn, C., Moehle, J., Bachman, R., Comartin, C., and Whittaker, A. 2004. The ATC-58 project: development of next-generation performance-based earthquake engineering design criteria for buildings. Paper presented at the 13th World Conference on Earthquake Engineering, Vancouver, BC, Canada.
- Imai, K., and Frangopol, D. M. 2002. System reliability of suspension bridges. *Structural safety*, **24**(2-4), 219-259.

- Jangid, R. 2004. Seismic response of isolated bridges. *Journal of Bridge Engineering*, **9**(2), 156-166.
- Kalpakidis, I. V., and Constantinou, M. C. 2008. *Effects of heating and load history on the behavior of lead-rubber bearings*, MCEER, University at Buffalo, State University of New York.
- Kennedy, D. L., Gagnon, D. P., Allen, D. E., and MacGregor, J. G. 1992. Canadian highway bridge evaluation: load and resistance factors. *Canadian Journal of Civil Engineering*, **19**(6), 992-1006.
- Kitayama, S., and Constantinou, M. C. 2019. Probabilistic seismic performance assessment of seismically isolated buildings designed by the procedures of ASCE/SEI 7 and other enhanced criteria. *Engineering Structures*, **179**, 566-582.
- Makris, N. 2019. Seismic isolation: Early history. *Earthquake Engineering and Structural Dynamics*, **48**(2), 269-283.
- Nassar, M., Guizani, L., Nollet, M., and Tahan, A. 2018. Approche probabiliste pour évaluer la fiabilité des ponts isolés à la base, sous l'effet simultané des séismes et de la température en régions froides. In *Proceedings of the 25 ième Colloque sur la progression de la recherche québécoise sur les ouvrages d'art, Quebec City, May 2018*.
- Nassar, M., Guizani, L., Nollet, M., and Tahan, A. 2019. A probability-based reliability assessment approach of seismic base-isolated bridges in cold regions. *Engineering Structures*, **197**, 109353.
- Nguyen, X.D. and Guizani, L. 2020. Optimal seismic isolation characteristics for bridges in moderate and high seismicity areas. *Canadian Journal of Civil Engineering*. **Just-IN** <https://doi.org/10.1139/cjce-2020-0058>
- Niemann, H., and Miklos, R. 2014. *A simple method for estimation of parameters in first order systems*, *Journal of Physics: Conference Series*, IOP Publishing (n° 1).
- Nowak, A. 1993. Calibration of LRFD bridge design code, Final Report, NCHRP 12-33. *Transportation Research Board, Washington, DC*.
- Nowak, A. S., and Collins, K. R. 2000. *Reliability of structures* McGraw-Hill, New York.
- Nowak, A. and Szerszen, M. 2003. Calibration of design code for buildings (ACI 318): Part 1 - Statistical models for resistance. *ACI Structural Journal*. **100**. 377-382.
- NZ Transport Agency 2016. *Highway Structures Design Guide*. Wellington, New Zealand: NZ Transport Agency.
- Ozdemir, G. 2014. Lead core heating in lead rubber bearings subjected to bidirectional ground motion excitations in various soil types. *Earthquake Engineering and Structural Dynamics*, **43**(2), 267-285.
- Priestley, M. 2000. Performance based seismic design. *Bulletin of the New Zealand society for earthquake engineering*, **33**(3), 325-346.
- Priestley, M. N., Seible, F., and Calvi, G. M. 1996. *Seismic design and retrofit of bridges*. John Wiley and Sons. New York.
- Rojahn, C., and Sharpe, R. L. 1985. Earthquake damage evaluation data for California. Applied technology council.
- Roussis, P. C., Constantinou, M. C., Erdik, M., Durukal, E., and Dicleli, M. 2003. Assessment of performance of seismic isolation system of Bolu Viaduct. *Journal of Bridge Engineering*, **8**(4), 182-190.
- Sheikh, M. N., and Legeron, F. 2014. Performance based seismic assessment of bridges designed according to Canadian Highway Bridge Design Code. *Canadian Journal of Civil Engineering*, **41**(9), 777-787.
- Strauss, A. 2003. *Stochastische Modellierung und Zuverlässigkeit von Betonkonstruktionen*. PhD thesis, Institute für Konstruktion und Ingenieurbau, Universität für Bodenkultur, Vienna.
- SEAOC - Vision 2000 Committee 1995. Performance based seismic engineering of buildings. Structural Engineers Association of California (SEAOC)

- STRUCTUREPOINT 2017. *spColumn User Manual, v6, the Engineering Software Group of the Portland Cement Association (PCA), Skokie, IL.*
- Thompson, A. C., Whittaker, A. S., Fenves, G. L., and Mahin, S. A. 2000. Property modification factors for elastomeric seismic isolation bearings. Dans *Proceedings of the 12th World Conference on Earthquake Engineering*. New Zealand Society for Earthquake Engineering Upper Hutt, NZ.
- Vecchio, F.J. and Collins, M.P. 1986. The Modified Compression Field Theory for Reinforced Concrete Elements Subjected to Shear, *ACI Journal*, **83**(2):219-231.
- Weisstein, Eric W. 2006. Correlation coefficient. Available from *mathworld.wolfram.com*
- Wisniewski, D. F. 2007. Safety formats for the assessment of concrete bridges: with special focus on precast concrete
- Wisniewski, D., Casas, J. R., Henriques, A. A., and Cruz, P. J. 2009. Probability-Based Assessment of Existing Concrete Bridges—Stochastic Resistance Models and Applications. *Structural Engineering International*, **19**(2), 203-210.
- Yakut, A. 2000. *Performance of elastomeric bridge bearings at low temperatures* (Citeseer).
- Yakut, A., and Yura, J. A. 2002. Parameters influencing performance of elastomeric bearings at low temperatures. *Journal of Structural Engineering*, **128**(8), 986-994.
- Zhang J., and Foschi, R.O. 2004. Performance-based design and seismic reliability analysis using designed experiments and neural networks. *Probabilistic Engineering Mechanics*. **19**(3): p. 259-267.

Draft

List of Figure Captions

Figure 1. Bilinear behaviour idealisation of the seismic isolation system

Figure 2. Nominal bridge capacity curve (unidirectional seismic force, V , vs pier-top displacement, D_p) and thresholds for damage levels under bidirectional loading

Figure 3. Ambient temperature and effective temperature data and distributions for Montreal and Rivière-du-Loup sites

Figure 4. Base-isolated SDOF bridge model

Figure 5. Graphical illustration of failure and safe events for flexural behaviour at pier column base for elastic designs.

Figure 6. Probabilistic distributions of the variables R , S and D for Montreal and Rivière-de-Loup sites (elastic designs)

Figure 7. Distribution of the generated RP values of the seismic hazard for (a) flexural failure cases (b) displacement failure cases

Figure 8. $S_a(0.2 \text{ sec})$ distribution data and the normal and lognormal fit distributions for Montreal site.

Figure 1. Bilinear behaviour idealisation of the seismic isolation system

Figure 2. Nominal bridge capacity curve (unidirectional seismic force, V , vs pier-top displacement, D_p) and thresholds for damage levels under bidirectional loading

Figure 3. Ambient temperature and effective temperature data and distributions for Montreal and Rivière-du-Loup sites

Figure 4. Base-isolated SDOF bridge model

Figure 5. Graphical illustration of failure and safe events for flexural behaviour at pier column base for elastic designs.

Figure 6. Probabilistic distributions of the variables R , S and D for Montreal and Rivière-de-Loup sites (elastic designs)

Figure 7. Distribution of the generated RP values of the seismic hazard for (a) flexural failure cases (b) displacement failure cases

Figure 8. $S_a(0.2 \text{ sec})$ distribution data and the normal and lognormal fit distributions for Montreal site.

Table 1. Minimal performance and salient deformations thresholds specified for concrete, steel and seismic isolation bearings by the CSA-S6:19 for base isolated reinforced concrete lifeline bridges

Seismic probability of exceedance in 50 years (Return Period)	Service performance level	Damage performance level	CSA-S6-19 Quantitative criteria
10% (475 years)	Immediate	–	–
5% (975 years)	Immediate Service	Minimal damage	Concrete: : $\epsilon_c \geq -0.006$ Steel : $0.01 \leq \epsilon_s \leq 0.01$ SI: No damage & no post-event centring required.
2% (2475 years)	Service Limited	Repairable damage	Steel : $\epsilon_s < 0.025$ SI : $D_r < 0.5 D_{iso, cap}$ & $D_i \leq D_{isol, cap}$

SI: Seismic Isolation, ϵ_s : Reinforcing steel deformation; ϵ_c : concrete deformation; D_r : residual displacement, D_i : maximum seismic isolation bearing displacement, $D_{isol, cap}$: Displacement capacity of seismic isolation bearing.

Table 2. Nominal properties of the base isolation system for each site

City/Town	$\left(\frac{kN}{(K_d)_i(mm)}\right)$	$(Q_d)_i (kN)$	Concomitant temperature (CT) (°C)	$(\lambda_{max}, Q_d)_i$	$(\lambda_{max}, \kappa_d)_i$	$R_1 (mm)$	$R_2 (mm)$	$R_3 (mm)$	$\tau(h)$
Montreal	5	160	−9.50	1.1	1.4	565	70	575	6.80
Gatineau	5	135	−9.50	1.1	1.4	570	66	580	6.90
Quebec City	5	135	−7.50	1.1	1.35	570	66	580	6.90
Roberval	5	160	−9.50	1.1	1.4	570	72	580	6.74
Rivière DL	4.7	300	−7.50	1.1	1.37	555	98	565	5.98
Baie-Comeau	4.7	95	−9.50	1.1	1.4	550	56	560	7.05
Gaspé	5.0	72	−7.00	1.1	1.36	510	48	520	7.46

Table 3. Reliability indices for the different studied sites for the two limit states

City	Maximal demand point coordinates		$N = 10^6$		$N = 10^7$			
	M_{Ex} (kN.m)	M_{Ey} (kN.m)	Probability of failure ($\times 10^{-6}$)	Reliability indices	Probability of failure ($\times 10^{-6}$)	Reliability indices	Global probability of failure P_f ($\times 10^{-6}$)	Global reliability index β
Montreal	2250	675	$P_{fM} = 200$	$\beta_M = 3.54$	$P_{fM} = 185$	$\beta_M = 3.56$	260	3.47
			$P_{fD} = 250$	$\beta_D = 3.46$	$P_{fD} = 230$	$\beta_D = 3.50$		
Gatineau	1965	590	$P_{fM} = 172$	$\beta_M = 3.58$	$P_{fM} = 185$	$\beta_M = 3.56$	290	3.44
			$P_{fD} = 290$	$\beta_D = 3.44$	$P_{fD} = 250$	$\beta_D = 3.48$		
Quebec City	1910	573	$P_{fM} = 200$	$\beta_M = 3.54$	$P_{fM} = 165$	$\beta_M = 3.59$	260	3.47
			$P_{fD} = 228$	$\beta_D = 3.50$	$P_{fD} = 215$	$\beta_D = 3.52$		
Roberval	2415	725	$P_{fM} = 187$	$\beta_M = 3.56$	$P_{fM} = 159$	$\beta_M = 3.60$	290	3.44
			$P_{fD} = 318$	$\beta_D = 3.41$	$P_{fD} = 290$	$\beta_D = 3.44$		
Rivière-Du-Loup	4355	1306	$P_{fM} = 185$	$\beta_M = 3.56$	$P_{fM} = 215$	$\beta_M = 3.52$	260	3.44
			$P_{fD} = 266$	$\beta_D = 3.46$	$P_{fD} = 230$	$\beta_D = 3.50$		
Baie-Comeau	1370	411	$P_{fM} = 155$	$\beta_M = 3.60$	$P_{fM} = 131$	$\beta_M = 3.65$	280	3.45
			$P_{fD} = 280$	$\beta_D = 3.45$	$P_{fD} = 245$	$\beta_D = 3.49$		
Gaspé	875	263	$P_{fM} = 155$	$\beta_M = 3.56$	$P_{fM} = 131$	$\beta_M = 3.65$	260	3.47
			$P_{fD} = 241$	$\beta_D = 3.49$	$P_{fD} = 215$	$\beta_D = 3.52$		

Table 4. Hourly temperature percentages for temperature profiles of the seven sites

City	Hourly temperature		$P(T_{eff} < CT)$ (%) (at failure)	Probability of failure ($\times 10^{-6}$) ($N = 10^7$)
	$P(T_{amb} < CT)$ (%)	$P(T_{eff} < CT)$ (%)		
Montreal	13.0	13.8	11.8	$P_{fM} = 160$
				$P_{fD} = 230$
Gatineau	12.0	11.3	12.8	$P_{fM} = 185$
				$P_{fD} = 250$
Quebec City	22.0	21.9	30.0	$P_{fM} = 165$
				$P_{fD} = 215$
Roberval	23.0	23.3	18.0	$P_{fM} = 159$
				$P_{fD} = 290$
Rivière-du-Loup	19.0	18.5	15.0	$P_{fM} = 215$
				$P_{fD} = 230$
Baie-Comeau	17.0	17.6	21.0	$P_{fM} = 131$
				$P_{fD} = 245$
Gaspé	16.0	17.0	29.0	$P_{fM} = 131$
				$P_{fD} = 215$
Mean values	14.1	17.6	20.0	

Table 5. Reliability indices and probabilities of failure for normal and lognormal distribution models

Site	β_{M_norm} ($P_{fM_norm} \times 10^{-6}$)	β_{M_logn} ($P_{fM_logn} \times 10^{-6}$)	β_{D_norm} ($P_{fD_norm} \times 10^{-6}$)	β_{D_logn} ($P_{fD_logn} \times 10^{-6}$)	β_{norm} ($P_{norm} \times 10^{-6}$)	β_{logn} ($P_{logn} \times 10^{-6}$)
Montreal	3.56 (185)	3.60 (160)	3.50 (233)	3.55 (193)	3.47 (260)	3.52 (216)
Rivière-du-Loup	3.52 (216)	3.61 (153)	3.50 (233)	3.57 (178)	3.44 (290)	3.50 (233)