



UTILIZING UNIFORM HAZARD SPECTRA FOR SEISMIC PERFORMANCE EVALUATION OF HIGHWAY BRIDGES IN CANADA

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SUMMARY

The effects and the consequences of adopting the Uniform Hazard Spectra approach in the seismic design of bridges in Canada are investigated. Seismic resistance demands of two typical bridges that are commonly found as highway overpasses in Canada are considered. Eight simulated ground motion time-histories compatible with the Uniform Hazard Spectra specified for Ottawa in east Canada and Vancouver in west Canada and three recorded earthquake time histories are chosen as the input excitations for nonlinear time history analyses. To attain insight on the typical seismic response behavior of the example bridges structural and local deformation demands of the two typical bridges are evaluated. It is shown that the generated time-histories compatible with the UHS spectrum for the probability level of 2% in 50 years can be scaled to estimate other uniform probability levels. The effects of different intensity criteria to scale these accelerograms on the responses of bridges are assessed accordingly. The seismic behavior of typical bridges in east and west Canada is assessed based on the results of these nonlinear analyses. It is expected that the results of this study will help to achieve a better understanding of bridge behavior subjected to earthquake ground motions in Canada.

INTRODUCTION

A bridge structure should be designed and detailed in such a way that the ductile components, such as bridge piers and columns, during a severe earthquake will have sufficient reserved ductility capacity to accommodate the expected ductility demand of the system. The National Building Code of Canada [1] (NBCC) is a national standard of safety for building design. In Canada, building design is governed by regulations of the local (municipal) governments. Usually the local governments incorporate the NBCC in their regulations; the code then becomes a legally binding document. The current seismic zoning map for Canada, first adopted in the NBCC of 1985, are based on results derived from statistical analyses of recent and past earthquakes in Canada. In these maps, Canada is divided into seven acceleration-related seismic zones Z_a , and seven velocity-related seismic zones Z_v . The level of seismic risk is considered to be similar within each zone and specified in terms of both peak horizontal ground acceleration and peak horizontal

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ground velocity, each with a probability of exceedence of 10% in 50 years. The use of two ground motion parameters to characterize seismicity is a significant improvement toward a more rational quantification of seismic risk. The ratio Z_a/Z_v gives an indication of the characteristics of the ground motion expected at a particular site. For sites with a low zonal ratio Z_a/Z_v , the expected ground motion is velocity dominated (typical of distant large earthquakes); whereas at sites with a high zonal ratio, acceleration dominates the ground motion (typical of earthquakes originating at a nearby source). The separate acceleration and velocity seismic maps also provide independent ground motion reference levels for small rigid structures having short fundamental periods and taller, more flexible structures with longer fundamental periods (Paz [2]).

Since 1996, new national seismic hazard maps have been developed by the Geological Survey of Canada. The maps present 5% damped uniform-hazard response spectra (UHS) for firm ground sites for a number of vibrational periods, at a probability level of 1/500 per annum. The uniform hazard spectra are currently being evaluated as the standard means for specifying the seismic hazard for earthquake resistant design of structures in Canada. The uniform hazard spectra (UHS) are derived from a probabilistic hazard analysis. The basic steps of the analysis are as follows. First, seismotectonic information is used to define seismic source zones. Generally, a number of alternative hypotheses regarding the configuration of these seismic zones are formulated. For each source zone, the earthquake catalogue is used to define the magnitude-recurrence relation and its uncertainty, which provides the description of the frequency of occurrence of events within the zone, as a function of earthquake magnitude. Ground motion relations are then defined to provide the link between the occurrence of earthquakes within the zones, and the resulting ground motions at a specified location. Ground motion relations can be given in terms of peak ground acceleration or velocity or in terms of response spectral ordinates of specific periods of vibration. The final step of the hazard analysis is integration over all earthquake magnitudes and distances, of the contributions to the probability of exceeding specified ground motion levels at the site of interest. Repeating this process for a number of vibration periods defines the uniform hazard spectrum, which is a response spectrum having a specified probability of exceedence at the particular site. The uniform hazard spectrum can be thought of as a composite of the types of earthquakes that contribute most strongly to the hazard at the specified probability level. The shape of a ground motion spectrum and therefore, the response spectrum is strongly dependent upon magnitude and distance. In general, the dominant contributor to the short-period ground motion hazard comes from small-to-moderate earthquakes at close distance, whereas larger earthquakes at greater distance contribute most strongly to the long-period ground motion hazard.

In some cases that engineer may wish to perform a dynamic analysis based on a time history method, time histories that are equivalent to the UHS are required. These ground motion records are referred to as UHS compatible time histories. To be compatible with the UHS, separate time histories should be simulated for each of the earthquake magnitude and distance combinations which dominate the seismic hazard at the specified probability level. The spectra of the simulated records, when taken as a suite, should approximately match the target UHS response spectrum. These considerations indicate that more than one type of earthquake are required to match the target spectrum over the entire period range of interest (Atkinson and Beresnev [3]).

The Canadian Highway Bridge Design Code (CHBDC) (CSA 2000) is the first national bridge design code in Canada. In CHBDC, the seismic loading effects at a site are quantified by an elastic seismic response coefficient given in the seismic zoning map of the NBCC 1985. This coefficient represents the elastic design spectrum which is specified in terms of the peak ground acceleration of the site with the same prescribed spectral shape considered throughout Canada. Both these parameters are defined in the same way as in the 1994 AASHTO LRFD code [4]. The peak ground acceleration corresponds to “firm ground” conditions and a probability of exceedence of 10% in 50 years. Mitchell et al. [5] reported that other approaches had been considered by CHBDC for defining the seismic effects on bridges before

deciding to adopt the correct elastic seismic response coefficient approach. In anticipation of the future direction of adopting the UHS methodology as the standard basis in specifying the seismic hazards in Canada for building design, the main subjective of this study is to evaluate the significance and impact of adopting the UHS approach on seismic design of bridges in Canada.

MODELING DESCRIPTION

Two typical bridges are investigated which are typical of highway bridges in many parts of Canada. Vancouver and Ottawa are selected as the sites of the bridges in the study to represent the significant seismic regions in western and eastern Canada, respectively. The bridges considered in this study are two and three-span bridges commonly found as highway overpasses in Canada. The three-span bridge (Bridge-1) has two reinforced concrete pier walls, and the other (Bridge-2) has a single reinforced circular concrete column. The structural elevation of the bridges is shown in Figure (1). For the analysis model, the pier end on the foundation at the bent is assumed fixed. At the abutment supports, translation is allowed only in the longitudinal and transverse directions. Rotation is allowed about the transverse and vertical axes at the abutments. No energy-absorbing device is installed at the bridge. Proportional damping matrix is defined as a linear combination of the mass and initial stiffness matrices, in order to have 5% damping for the first and the last mode contributing significantly to the response. The period of the fundamental vibration modes of these bridges are calculated as 0.45 and 0.76 second, respectively.

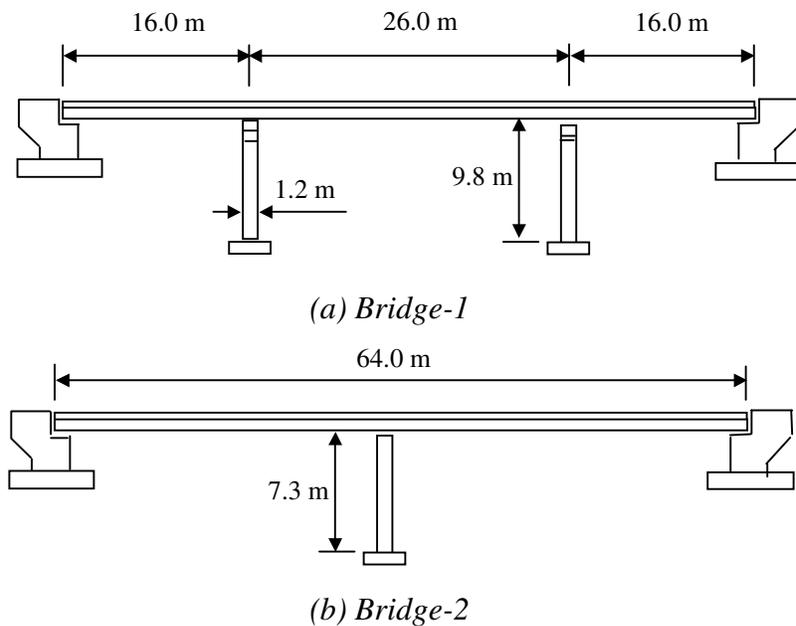


Figure 1. Elevation of deck and piers of bridges

Considering the bridges are located in Ottawa in the east and Vancouver in the west, the ground motions used as input excitations in the analyses are eight simulated (four short and four long period) time-histories compatible with the median values of the Uniform Hazard Spectra for each of Ottawa and Vancouver developed by the Geological Survey of Canada with 2% probability of exceedence in 50 years (Adams et al. [6]). The numerical procedure for deriving the simulated UHS compatible time-histories has been described by Atkinson and Beresnev [3]. To supplement the results obtained using simulated time-histories, three actual recorded earthquake ground motions appropriate to the seismic hazard expected at the site are also used in the analyses.

The nonlinear dynamic analyses are carried out using the NEABS bridge computer analysis program (Penzien et al. [7]). This program has the capability to carry out the dynamic time-history nonlinear analysis of discrete parameter systems subjected to the applied dynamic loading and/or prescribed support motions. Bridge systems can be modeled by a combination of linear and nonlinear elements in this program.

ANALYSES RESULTS

Each of the example bridges has been analyzed with the ground motion time-histories stated above applying (a) in the longitudinal direction (Case-I); and (b) in the transverse direction (Case-II). Each ground motion time-history is applied separately in the two perpendicular directions in the analysis of each bridge considered in this study. To gain insight on the typical seismic response behavior of the example bridges, the displacement, Δ_{max} , displacement ductility, μ_{Δ} , member end rotation ductility, μ_{θ} , and curvature ductility, μ_{ϕ} , demands of the two typical bridges are evaluated.

Eastern Canada (Ottawa)

Table 1 shows the responses of two bridges when they are subjected to the earthquake ground motions in the longitudinal direction (Case-I). The displacement ductility and rotational ductility responses show that the Bridge-1 has sufficient capacity to remain elastic with no plastic hinge formed in the piers. On the other hand, Bridge-2 reaches its elastic limit and is on the threshold of inelastic behavior when subjected to the simulated UHS compatible ground motion time-histories. However, the bridge responses remain elastic when subjected to the actual earthquake ground motions. The behavior in Case-II (transverse direction) is similar as in Case-I.

Table 1. Maximum Response of bridges (Case-I)

Ottawa								
Earthquake	Δ_{max} (mm)		μ_{Δ}		μ_{ϕ}		μ_{θ}	
	Brdg 1	Brdg 2	Brdg 1	Brdg 2	Brdg 1	Brdg 2	Brdg 1	Brdg 2
Avg. for Short Period Records	17.00	24.07	0.54	0.96	0.90	0.99	1.00	1.07
Avg. for Long Period Records	18.37	23.90	0.53	0.95	0.90	0.99	1.00	1.12
Northern N.Y., US 2002	7.90	16.61	0.25	0.65	0.84	0.91	1.00	1.00
Sagunay 1998 (H1-Long. Comp.)	7.10	4.43	0.21	0.19	0.83	0.80	1.00	1.00
Sagunay 1998 (H2-Trns. Comp.)	7.08	12.75	0.22	0.51	0.84	0.88	1.00	1.00
Vancouver								
Avg. for Short Period Records	131.7	941.0	2.71	30.82	1.37	8.36	6.71	36.30
Avg. for Long Period Records	15.58	29.39	0.47	1.18	0.88	1.05	1.00	1.48

Western Canada (Vancouver)

The behavior of the two bridges is totally different in the case of western Canada. As shown in Table 1, Bridge-1 in western Canada will experience large structural and local deformations when it is subjected to short period time-histories. However, this bridge remains elastic under the effect of long period time-histories. On the other hand, Bridge-2 experiences very large structural and local deformations for short period time-histories, and in addition its behavior is nonlinear for long period time-histories too. Therefore, the results indicate that both of these bridges need to adopt special seismic design considerations in order to resist the seismic effects in western Canada. Figure (2) shows the response time-histories of Bridge-2 under a long period time-history compatible with the UHS in the longitudinal direction.

OTHER PROBABILITY LEVELS

In the past three decades, seismic hazard is specified in the seismic design provisions of the National Building Code of Canada (NBCC) in terms of peak ground motions. In the 1970 edition of NBCC the seismic zoning maps are expressed in terms of peak ground acceleration at an annual probability of exceedence of 0.01. In the 1985 and subsequent editions of NBCC [1], peak ground velocity and peak ground acceleration are specified at a 10% probability of exceedence in 50 years. The Geological Survey of Canada has now quantified seismic hazard by means of Uniform Hazard Spectra (UHS) instead of peak ground motions. Adams et. al. [6] gives UHS values calculated at probability levels of 2% in 50 years and 10% in 50 years, as well as a description of the methodology.

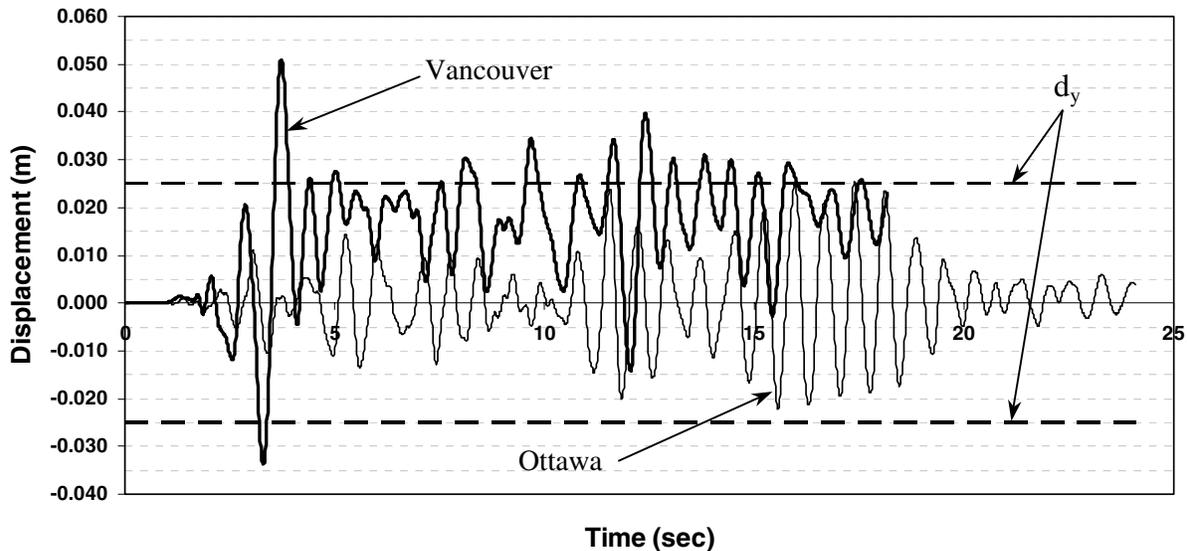


Figure 2. Time histories of displacement at top of the column of Bridge-2 subjected to a long-period UHS compatible ground motion.

A question arises whether the UHS compatible simulated time-histories can be scaled with parameters of the peak ground motions to obtain ground motions corresponding to different probability levels. To answer this question, the UHS spectrum in Victoria for the probability level of 2% in 50 years, as shown in Figure 3, is scaled down through consideration of the peak ground acceleration of the simulated motion to the probability level of 10% in 50 years. As shown in this figure, the resultant spectrum is very close to the UHS spectrum for the probability level of 10% in 50 years. The differences are less than 6.8% for the period range up to 2.0 sec. Although not indicated here, the scaled spectra of all other cities are also approximately matched to the UHS spectra for the probability level of 10% in 50 years, especially for the period range of interest for bridges. This may be concluded then that the generated time-histories compatible with the UHS spectrum for the probability level of 2% in 50 years (Atkinson and Beresnev [3]) can be scaled to obtain the UHS at other probability levels. The reason for this scaling behavior is the linear relationship between time-history accelerogram values and its response spectrum in the elastic range of behavior. This fact justifies scaling the simulated time-histories in order to evaluate the seismic response of bridges at a different level of seismic intensity.

Because of the random vibration generation process in the UHS methodology, each UHS compatible simulated time-histories has slightly different peak ground acceleration (and velocity). Adams et al. [6] have also presented specific values of PGA and PGV for each city in Canada beside the UHS spectrum (no PGV has been proposed for western cities) as part of the quantification of seismic hazard in Canada.

These peak ground motion parameters may be considered as the equivalent peak ground acceleration and peak ground velocity of the simulated time-histories and used for scaling them to other peak ground motions. The proposed peak ground acceleration and peak ground velocity for Ottawa are 0.42g and 0.18 m/s, respectively. For Vancouver, the PGA value of 0.48g has been proposed in Adams et al. [6], where g is the gravity acceleration.

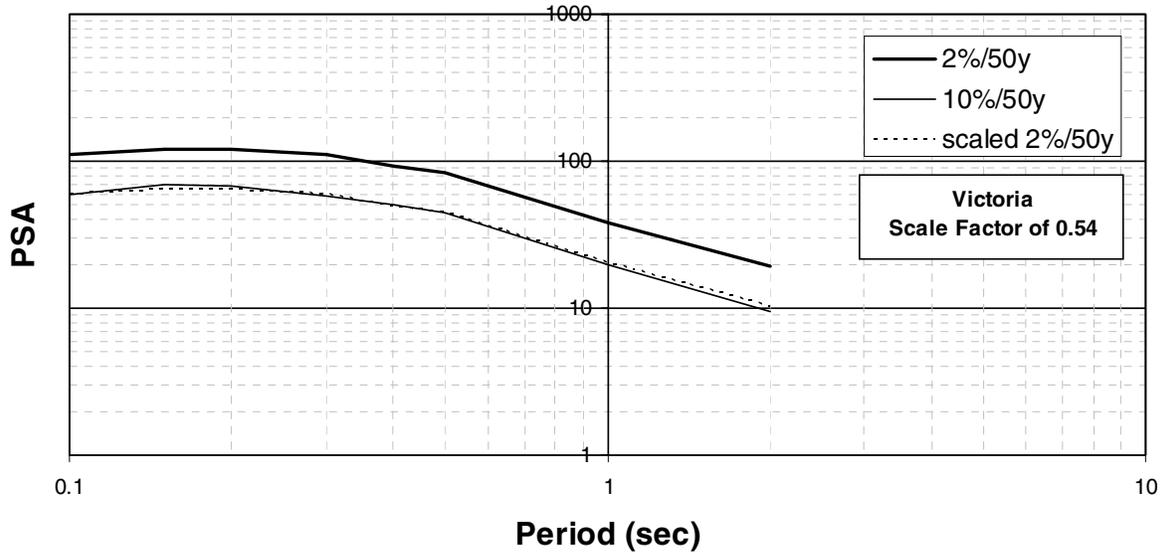


Figure 3. Comparison of scaled UHS spectra for probability of 2%/50yr with UHS spectra for probability of 10%/50yr.

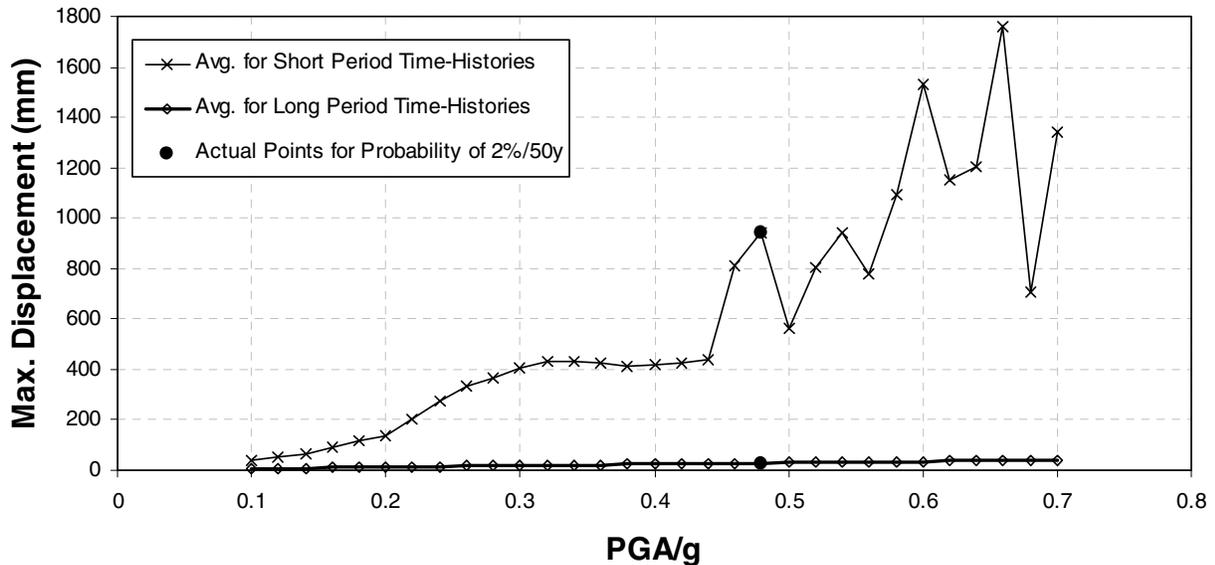


Figure 4. Effects of peak ground acceleration on maximum displacement of the column of Bridge-2 subjected to different ground motions (Vancouver).

PGA and PGV as the scaling criteria

Peak ground acceleration (PGA) and peak ground velocity (PGV) are traditional scaling criteria. Figure (4) shows the effect of using PGA as the scaling criterion on the maximum displacement of the column of

Bridge-2 in Vancouver. In the analyses the accelerograms are scaled down or up to arrive at different severity levels. The actual points in the figure show the maximum displacement resulting from the earthquake ground motions without any scaling (probability level of 2%/50yr). These points are the average of the maximum displacement computed for each individual short or long period time-history.

Eastern Canada

For Bridge-1 the variation of displacement is linear with respect to PGA (all probability levels) for Long-period simulated time histories. For short-period time histories this is true until the PGA value of about 0.7g (which corresponds to an unknown specific level of probability that is significantly lower than 2%/50yr). This linear behavior shows that the responses at the probability level of 2% in 50 years are in the elastic range with a sufficient reserved capacity before inelastic behavior. The same result could be concluded of this bridge when subjected to PGV-based scaled ground motions. Different results may be concluded for Bridge-2. For this bridge, actual responses are on the threshold of inelastic behavior.

Western Canada

The response of the bridges to long period time-histories scaled to various PGAs is almost linear even for large values of PGA (or very small probability levels). Although, both of the bridges experience nonlinear deformations when they are subjected to short period time-histories scaled to even very small values of PGA. This nonlinear behavior can be observed in Figure (4) for small PGA values of 0.2g or less. It can be concluded that the bridges with small fundamental periods are in a bigger risk of having large nonlinear deformations and severe damages in western Canada. Therefore, design of more flexible bridges may be a good recommendation for this part of the country.

OTHER INTENSITY MEASURES

The philosophy of modern earthquake engineering requires different design criteria for minor, moderate, and major earthquakes. Hence, a major problem encountered is the identification of earthquake ground motions with respect to their severity, or damage potential. It is especially important in the development of seismic provisions to provide consistent level of safety against seismic hazards. Although acceleration response spectra provide a very convenient tool for specifying earthquake inputs, recent developments in earthquake engineering clearly indicate that they are deficient and inadequate in representing the damage potential of earthquake ground motions. There are several parameters contributing to the damage potential of earthquake ground motions. Seismic energy input, or seismic energy dissipated during an earthquake, represents the damage potential of an earthquake ground motion more accurately than spectral acceleration, because they better reflect the effects of significant excitation and system characteristics contributing to the seismic response and consequence structural damage. Various studies in earthquake engineering literature have focused on the quantification of the damage potential of earthquake ground motions. Although all of the proposed quantities are either identified as intensity or damage potential, they describe different physical values in terms of different physical units. Uang and Bertero [8, 9] give a conscientious evaluation of these quantities, and the most prominent ones are discussed in the Sucuoglu and Nurtug [10].

According to the discussion in the previous section, the effects of different scaling criteria applied to the accelerograms on the responses of bridges are assessed here. Typical scale approach adopts a constant scaling factor for scaling the entire accelerogram. There are several methods proposed in the literature for quantifying the severity, intensity or damage potential of earthquake ground motions. The intensity measures discussed here are evaluated with respect to their ability in considering the most important seismic parameters, such as the peak acceleration, strong motion duration, duration of dominant acceleration pulse, and frequency content of ground motions.

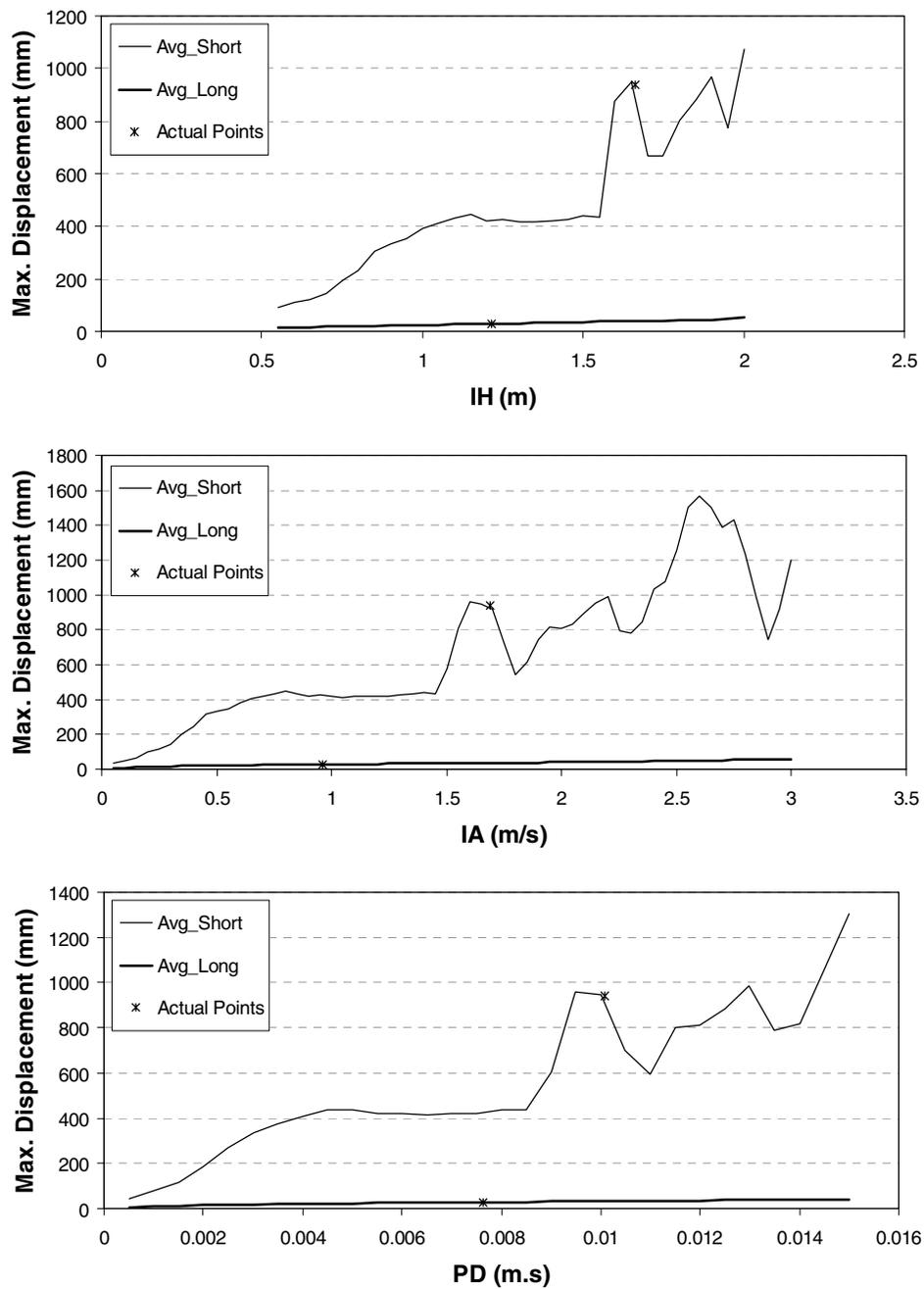


Figure 5. Effects of different scaling criteria on maximum displacement of the column of Bridge-2 in Vancouver.

The effect of intensity indices

Figure (5) shows the effect of the following intensity indices on the maximum displacement responses of Bridge-2 in Vancouver: (a) Housner intensity index (IH); (b) Arias index (IA); and (c) Araya and Saragony index (PD) (Sucuoglu and Nurtug [10]). The previous observations about the behavior of Bridge-1 and

Bridge-2 are confirmed here based on the Housner, Arias, and Araya indices again. This study shows a good safety margin for Bridge-1 and a more susceptible situation of the Bridge-2 in Ottawa, and the significant effect of short period time-histories on the bridges in Vancouver. All the above results are also valid for the responses of two bridges in the transverse direction (Case-II), which are not shown here because of the limited space.

CONCLUSION

Seismic behavior of two typical (a two and a three-span) bridges in Canada have been evaluated. Using UHS compatible and actual ground motions for nonlinear time history analyses of the bridges; the following conclusion has been drawn:

It is shown that the simulated UHS compatible time-histories can be normalized with respect to their peak ground motions to estimate the effect of earthquakes at different severity and probability levels. For the bridges in eastern Canada, while the three-span bridge experiences no structural and local inelastic responses and has a safe margin from inelastic behavior, the two-span bridge is on the threshold of inelastic behavior and needs more careful considerations in the design for adequate seismic behavior, especially for sufficient ductility capacity at the bottom of its column. In the west, both the bridges are expected to experience severe structural and local damage when subjected to short period time-histories. However, the level of nonlinear deformations for long period time-histories is significantly smaller. Design of more flexible bridges is thus a good suggestion for seismic design of bridges in this part of country.

The behavior of typical bridges has been evaluated for different earthquakes at different severity and probability levels using the scaling criteria of PGA and PGV. The effects of using the Housner, Arias, and Araya intensity indices on the behavior of bridges have also been evaluated and compared using the short and long period UHS compatible simulated time-histories. The results of this part of study confirmed the previous conclusions.

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