Performance of Long-span Bridges under Seismic Loading - a Case Study

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SUMMARY:
Long-span bridges are special structures and advanced analysis methods are required for the investigation of their performance under seismic loading. This paper describes results from a comprehensive study on the dynamic behavior of the Confederation Bridge when subjected to seismic ground motions. The 12.9 km Confederation Bridge in eastern Canada, is one of the longest reinforced concrete bridges in the world. For the purpose of the seismic analysis of the bridge, a finite element model was developed using 3-D beam elements. The model was calibrated using measured data of the bridge vibrations during a dynamic load test. Dynamic analyses including response-spectrum analyses and time-history analyses were conducted. The bending moments and the displacements obtained from the seismic analysis were compared with the design values. It was found that the seismic effects used in the design are quite representative of the seismic hazard of the bridge location.

Keywords: bridge, dynamic analysis, seismic hazard, design spectrum, response.

1. INTRODUCTION

The 12.9 km long Confederation Bridge, which was opened to traffic in June 1997, is one of the longest reinforced concrete bridges in the world. The bridge crosses the Northumberland Strait in eastern Canada and connects the province of Prince Edward Island and the province of New Brunswick. In addition to approach spans, the bridge has 45 main spans, each with a length of 250 m. The cross section of the bridge is a trapezoidal box with a depth that varies from 14.5 m at the piers to 4.5 m at mid-span. The width of the deck is 11.0 m. The bridge structure consists of a series of rigid portal frames connected by simply supported girders, which are referred to as drop-in girders (Fig. 1.1). Each of these frames has a span of 250 m, and two 95 m overhangs. The drop-in girders are 60 m long. Every second span is constructed as a rigid portal frame, and all other spans are constructed using drop-in girders. There are 22 rigid portal frames, and 23 spans with drop-in girders.

Since the Confederation Bridge represents an essential transportation link, a comprehensive research program was undertaken to monitor and study the behaviour of the bridge. As part of this program, a study was conducted to investigate the dynamic performance of the bridge under seismic loads. The objective of the study was to compare the responses of the bridge for seismic actions representative of the seismic hazard at the bridge location with those used in the design.

This paper describes the main findings from the study and includes: (i) an overview of the seismic parameters used in the design of the bridge, (ii) development of a finite element model of the bridge for use in seismic analysis, (iii) selection of seismic ground motions representative of the seismic hazard at the bridge location, and (iv) dynamic analysis of the bridge model and comparison of the analytical results with the design values.

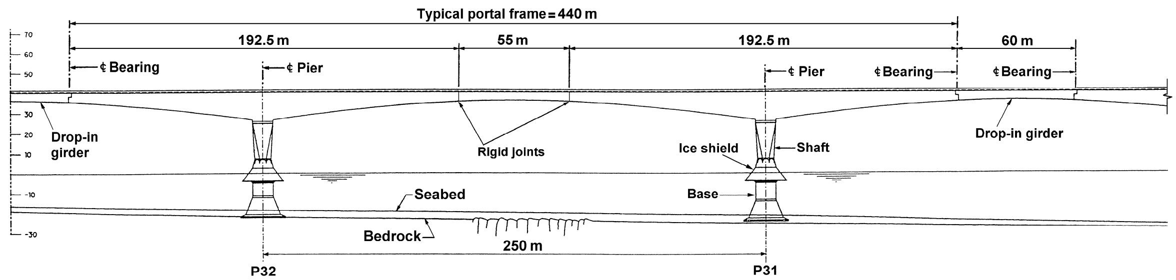

Figure 1.1. Typical portal frame

2. SEISMIC DESIGN PARAMETERS AND SEISMIC HAZARD FOR THE BRIDGE

2.1 Seismic Design Parameters

The seismic ground motion parameters used in the design of the Confederation Bridge are given in the design specifications for the bridge (JMS 1996). These include the peak ground acceleration, the peak ground velocity, the peak ground displacement, and the seismic design spectrum for the bridge location. Two methods have been used for the estimation of the peak ground acceleration of the expected seismic motions at the bridge location (Jaeger et al. 1997). The first method is based entirely on probabilistic considerations. According to this method, the peak ground acceleration for the design service life of 100 years and the design safety index of 4.0, corresponds to an annual probability of exceedance of 0.00027. The value of the peak ground acceleration for this probability of exceedance is found to be A = 0.136 g. The second approach is primarily based on engineering considerations. In this approach, the peak ground acceleration is determined for a probability of exceedance of 10% during the design service life of 100 years, and that value is increased by applying an importance factor of 1.43. The resulting peak ground acceleration is 0.12 g, and this value is adopted for the design. Using the same approach, the peak ground velocity is found to be 10.8 cm/s. Having the values for the peak ground acceleration (A) and the peak ground velocity (V), a value for the peak ground displacement (D) of 5.9 cm is obtained using an empirical relationship (Newmark and Hall 1982) between A, V, and D. The elastic seismic design spectrum for horizontal seismic motions is developed using the foregoing values for the peak ground acceleration, velocity and displacement, and applying the corresponding spectral amplification factors (Newmark and Hall 1982) for the mean plus one standard deviation level. This level corresponds to a probability of 84% that the spectral amplification factors will not be exceeded. The design spectrum for vertical seismic motions is taken as 2/3 of the spectrum for horizontal motions (JMS 1996).

2.2 Seismic Hazard

Since the design of the bridge in the early 1990s, significant improvements have been made in the estimation of the seismic hazard in eastern Canada (Adams and Halchuk 2003). The seismic hazard models (i.e., the source zones and their seismic activities) have been updated based on new findings gathered during the last two decades. Also, attenuation relations for spectral accelerations have been developed for use in the seismic hazard analyses (Atkinson 1995). This enabled the use of a uniform hazard spectrum (UHS) as a hazard parameter, rather than peak ground acceleration and peak ground velocity as used in the early 1990s.

For the purpose of this study, Geological Survey of Canada computed the horizontal uniform hazard spectrum for the bridge location for annual probability of exceedance of 0.00027 and confidence level of 84% (Adams and Halchuk, personal communication 2004). Spectral values were computed for eight periods between 0.1 s and 2.0 s. For periods between 2.0 s and 4.0 s, the spectrum was extended assuming a constant spectral velocity with the same value as that at 2.0 s. It can be seen in Fig. 2.1 that the uniform hazard spectrum is somewhat higher than the design spectrum for periods below 1.5 s.

As discussed later, this difference does not have significant effects on the seismic response of the bridge.

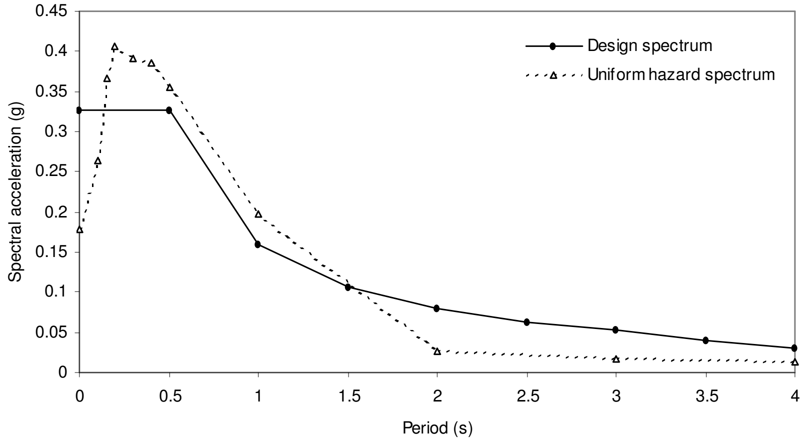

Figure 2.1. Design and uniform hazard spectra; 5% damping

3. DYNAMIC MODELLING OF THE BRIDGE

For the purpose of dynamic analysis of the bridge, a finite element model was developed using 3-D beam elements. The computer program SAP 2000 (CSI 2000) was used in the modelling. The model represents the bridge segment between piers P29 and P32, which consists of two rigid portal frames (P29–P30 and P31–P32), and one drop-in span (P30–P31). This segment was modelled since it is the instrumented portion of the bridge, and recorded data is available for use in the calibration of the model. The heights of the piers of this segment are considered as typical of the main bridge, with the exception of the few spans in the vicinity of the navigation span. Note that this segment is normally used as a typical segment in studies on the behaviour of the Confederation Bridge (e.g., Lau et al. 2004; Ghali et al. 2000; Cheung and Li 2003).

Figure 3.1 shows schematically the finite element model of the bridge. It consists of 179 beam elements and 180 joints. The bridge girder is modeled by 123 elements, and each pier is modeled by 14 elements. The geometrical properties of the end sections of the elements, such as cross-sectional area, moments of inertia, and torsional constant, were determined from the dimensions in the design drawings. The interaction with the adjacent drop-in girders (left of P32, and right of P29) was modeled by adding masses at the ends of the overhangs, as shown in Fig. 3.1. A half the mass of each drop-in girder was added at the end of the supporting overhang in transverse and vertical directions, full mass was added in the longitudinal direction for a hinge connection, and no mass was added in the longitudinal direction for a sliding connection.

Measured data for tilts at two locations of pier P31 resulting from an application of a horizontal force to the pier, and recorded acceleration time-histories of free vibrations of the bridge caused by a sudden release of the force were used in the calibration of the finite element model. A modulus of elasticity of the concrete of 40,000 MPa was used in the analysis. This value was based on experimental data for the bridge (Ghali et al. 2000), and is representative of the modulus of elasticity at the time when the tilts and the free vibrations were measured. Lin (2005) reported that the modulus of elasticity of 40,000 MPa is also appropriate for the investigation of the dynamic behaviour of the bridge in general.

Since the foundation conditions can have significant effects on the dynamic behaviour of the bridge, the model was calibrated to take into account these effects. Rotational springs at the bases of the piers in the longitudinal and transverse directions were introduced in the model to represent the foundation

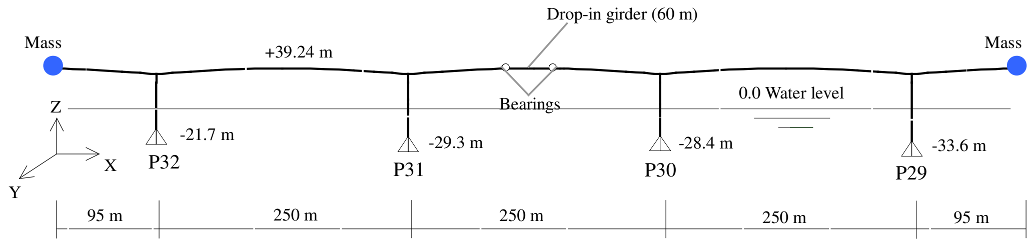

Figure 3.1. Model of two portal frames and one drop-in span using 3-D beam elements

stiffness. A number of static and dynamic analyses were conducted in order to determine the stiffness of the springs that provides a close match between the computed and the measured tilts and free vibrations of the bridge. For each selected stiffness of the springs, the tilts and the response were computed by applying the same loads as those that caused the measured tilts and free vibrations. It was found that the model with a rotational stiffness of 3.35×10^9 kN·m/rad provided the best matching of the computed and measured responses.

Table 3.1 shows the natural periods of the first ten modes of the evaluation model. The types of the modes are also shown in the table. It can be seen that the longest-period modes are the transverse modes. The periods of the first 6 transverse modes (modes 1 to 5, and mode 7) range between 2.08 s and 3.13 s, the periods of the longitudinal modes 6 and 8 are 2.13 s and 2.01 s respectively, and those of the vertical modes 9 and 10 are 1.54 s and 1.43 s respectively.

Table 3.1. Natural periods of the first ten modes of the bridge model

Mode No.	Period (s)	Mode type
1	3.13	Transverse
2	2.99	Transverse
3	2.72	Transverse
4	2.48	Transverse
5	2.22	Transverse
6	2.13	Longitudinal
7	2.08	Transverse
8	2.01	Longitudinal
9	1.54	Vertical
10	1.43	Vertical

4. SEISMIC EXCITATIONS FOR TIME-HISTORY ANALYSIS

It was of special importance to select seismic excitations representative of the seismic hazard for the bridge location. To cover all possible seismic motions that might occur at the bridge location, records of seismic motions obtained in other countries (referred to as the world-wide seismic records), records from earthquakes in eastern Canada, and simulated seismic motions for eastern Canada were considered in the study.

4.1 World-wide Seismic Records

Two sets of ground motion records obtained during strong earthquakes around the world were selected. One of the sets is representative of short-period hazard motions (i.e., ground motions with predominant periods shorter than about 0.5 s) and is referred to as the short-period set. It consists of 13 pairs of horizontal and companion vertical records. The records are obtained at distances below 40 km, from 10 earthquakes with magnitudes between 5.25 and 6.9. The other set is representative of intermediate- and long-period motions (i.e., motions with predominant periods longer than about 0.5 s) and is referred to as the intermediate/long-period set. It consists of 15 pairs of horizontal and companion vertical records obtained at distances between 40 km and 470 km, from 7 earthquakes with magnitudes between 6.3 and 8.1. The records of both sets are recorded on rock or stiff soil sites. The earthquake magnitudes and the distances of the records in the sets cover the magnitude and distance ranges of the scenario earthquakes for the bridge location (Lin 2005).

The excitation motions for the time-history analysis were obtained by scaling the records to the peak ground velocity for the bridge location of 7.1 cm/s, computed by Geological Survey of Canada for annual probability of exceedance of 0.00027. For each pair of horizontal and vertical records, a scaling factor was determined using the peak ground velocity of the horizontal record, and that factor was used to scale both the horizontal and the vertical records. Scaling to peak ground velocity is appropriate for this study because the predominant periods of the bridge are in the velocity-controlled region, i.e., approximately between 0.5 s and 3.5 s.

Figure 4.1 shows the response spectra of the scaled horizontal records of the ensemble. For comparison, the design spectrum is superimposed on the figure. It can be seen that the spectra of the records exceed significantly the design spectrum for periods shorter than approximately 0.5 s, and the spectra are well below the design spectrum for longer periods.

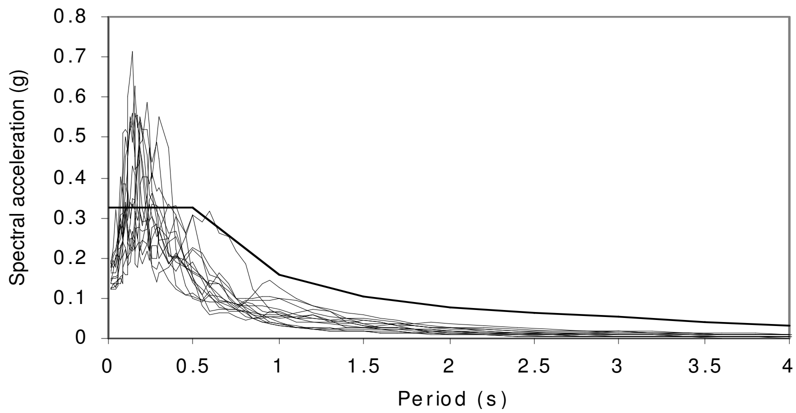

Figure 4.1. Design spectrum and response spectra of scaled world-wide set records; 5% damping

4.2 Eastern Canadian Seismic Records

Since the bridge is in eastern Canada, ground motion records were selected from two major earthquakes in eastern Canada, i.e., 1988 Saguenay, Quebec earthquake, and 1982 Miramichi, New Brunswick earthquake. The magnitudes of the two earthquakes were 5.7 and 5.0 respectively (Munro and Weichert 1989, Weichert et al. 1982). Records at 16 locations were obtained during the Saguenay earthquake and at five locations during the Miramichi earthquake. Considering the response spectra, the strongest five pairs of horizontal and companion vertical records of the Saguenay earthquake, and the strongest three pairs of the Miramichi earthquake were selected for the analysis.

Figures 4.2(a) and 4.2(b) show the acceleration response spectra of the selected horizontal records of the Saguenay and the Miramichi earthquakes, respectively, scaled to peak ground velocity of 7.1 cm/s. It can be seen that the spectra of both earthquakes are much higher than the design spectrum for very short periods, i.e., below 0.25 s for the Saguenay earthquake, and below 0.15 s for the Miramichi earthquake. This was expected because the records are obtained close to the epicentres and are characterized by very short predominant periods.

4.3 Simulated Seismic Motions

In addition to the "real" records of seismic ground motions discussed above, "simulated" acceleration time histories (i.e., accelerograms) were also used as excitation motions. Tremblay and Atkinson (2001) reported that the seismic hazard for eastern Canadian sites can be approximated using a

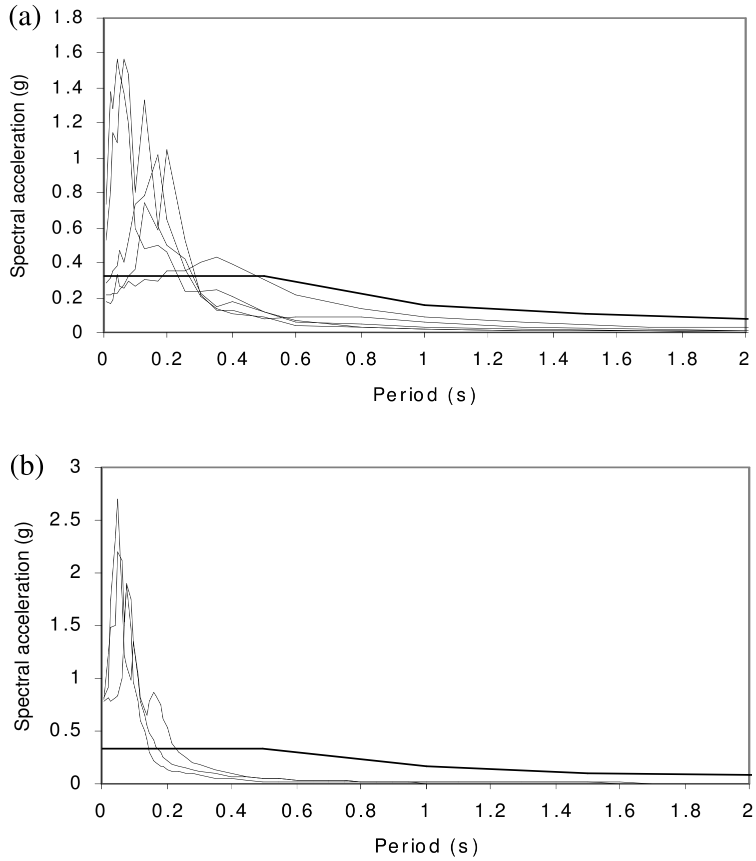

Figure 4.2. Design spectrum and response spectra of scaled records; 5% damping
(a) Saguenay records, (b) Miramichi records

magnitude M = 6.0 event to represent the short-period hazard, and M = 7.0 event to represent the long-period hazard. They simulated ground motion accelerograms for eastern Canada for M = 6.0 and M = 7.0, and for different distances. For each distance, four accelerograms were simulated for a probability of exceedance of 2% in 50 years (i.e., annual probability of exceedance of 0.0004).

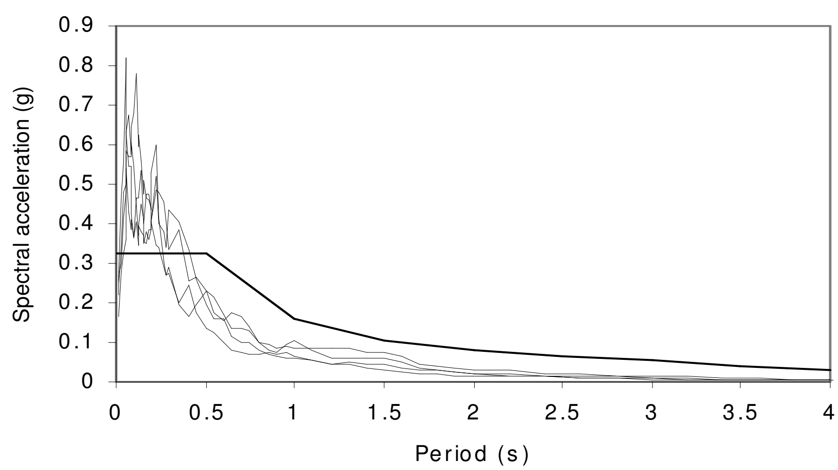

Figure 4.3. Design spectrum and scaled response spectra of simulated records; 5% damping

Since the seismic hazard at the bridge location corresponds to an annual probability of exceedance of 0.00027, it was necessary to scale the simulated accelerograms to be consistent with the UHS for a probability of exceedance of 0.00027 (Fig. 2.1). To determine the short-period hazard motions for the bridge, the simulated accelerograms for the M = 6.0 event were scaled to have the same spectral values at the period of 0.2 s as that of the UHS for the bridge location. Similarly, the long-period hazard motions were obtained by scaling the simulated accelerograms for the M = 7.0 event to have the same spectral values as that of the UHS at the period of 2.0 s. Based on the preliminary study, it was found that the largest responses of the bridge model are associated with the scaled accelerograms corresponding to the epicentral distances of R = 50 km for the M = 6.0 event and R = 100 km for the M = 7.0 event, and therefore, only these accelerograms were considered. The scaled response spectra of the selected simulated records are shown in Fig. 4.3.

5. DYNAMIC ANALYSIS AND RESULTS

For the purpose of the seismic evaluation of the bridge, dynamic analyses were conducted on the bridge model to determine the responses due to seismic actions represented by the uniform hazard spectrum and the selected sets of records. Elastic material properties of the model were assumed in the analyses. The dynamic analyses included both the response-spectrum analyses and time-history analyses.

5.1 Response-spectrum Analyses

Response-spectrum analyses were performed for seismic actions represented by the uniform hazard spectrum. Separate response-spectrum analyses were carried out for the following two cases of seismic actions: (i) seismic actions in the longitudinal and vertical directions of the model, and (ii) seismic actions in the transverse and vertical directions. The horizontal and the vertical actions were applied simultaneously at the bases of the piers. The horizontal seismic actions were represented by the horizontal uniform hazard spectrum (UHS) (Fig. 2.1), and the vertical actions were represented by a spectrum obtained by multiplying the horizontal UHS by 2/3. The factor of 2/3 is commonly used for defining vertical design spectra relative to horizontal spectra (Newmark et al. 1973).

The analyses included the first 100 modes, which covered all natural periods above 0.02 s. It is necessary to mention that the mass participation of the modes considered in the analysis is larger than 90% in each of the three principal directions of the model. A modal damping of 5% was used for all the modes. The response maxima at each joint of the models were computed by combining the modal responses using the complete quadratic combination (CQC) rule.

5.2 Time-history Analyses

Time-history analyses were conducted to determine the responses of the model subjected to the records of the selected sets. As in the response-spectrum analysis, simultaneous seismic excitations in the longitudinal and vertical directions, and in the transverse and vertical directions of the model were used in the time-history analysis. In each analysis, the seismic excitations consisted of a pair of scaled horizontal and vertical acceleration time histories, applied at the bases of the piers.

5.3 Discussion of Results

The response quantities obtained from both the response-spectrum analysis and the time-history analysis included bending moments, shear forces, axial forces, and displacements. A detailed review of the response results showed that the observations from the shear forces and the axial forces were the same as those from the bending moments. Given this, only the bending moments and the displacements were used for the evaluation of the seismic performance of the bridge.

The moments and the displacements at the joints of the model resulting from the response-spectrum analysis represent the maximum absolute values and are positive. The time-history analysis provided a comprehensive set of results for each excitation motion. Time histories and maximum positive and negative values for the moments and displacements were obtained for the joints of the model. Moment and displacement envelopes for both the girder and the piers were determined using the largest *absolute* values of the computed maxima for each of the selected sets of ground motions.

The computed responses were compared with those used in the design. The design moments and displacements were calculated by Jaeger et al. (1997), and are discussed in detail by Lin (2005). The comparisons of bending moments are shown in Figs. 5.1 and 5.2. Figure 5.1(a) shows the envelopes of the longitudinal moments in the bridge girder for seismic actions in the longitudinal direction, and Fig. 5.1(b) shows the envelopes of the transverse moments for seismic actions in the transverse direction. The moment envelopes are plotted using the corresponding values at selected sections along the bridge girder. Similarly, Figs. 5.2(a) and 5.2(b) present the moment envelopes for pier P31 for excitations in the longitudinal and transverse directions respectively. The moment envelopes for the other piers are similar to those for pier P31, and they are not shown here. The values of the displacement envelopes at selected sections of the bridge girder are listed in Table 5.1. The designation "Design" in Figs. 5.1 and 5.2, and in Table 5.1 is for the design responses, and "UHS" is for the responses due to seismic actions represented by the uniform hazard spectrum.

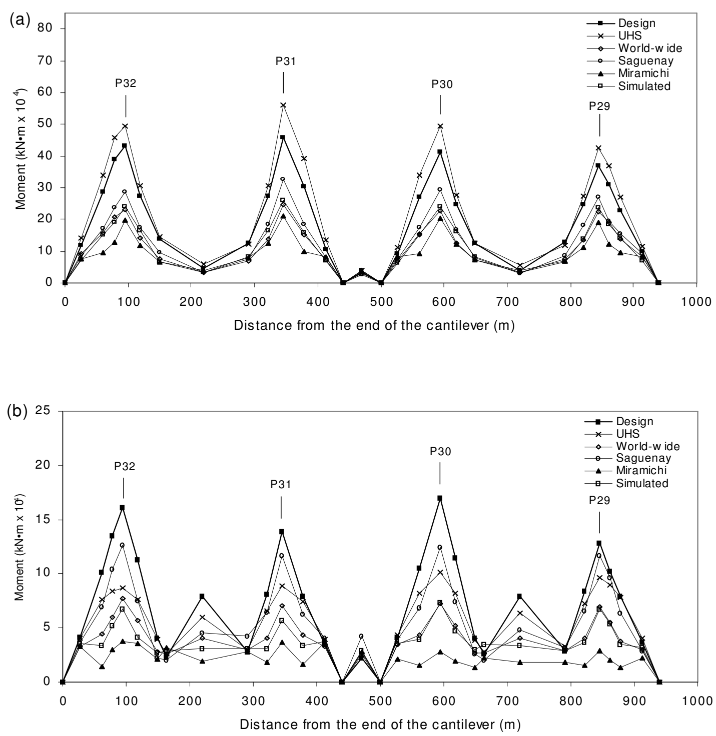

Figure 5.1. Moment envelopes for the bridge girder: (a) longitudinal moments and (b) transverse moments
Note: Piers P29 to P32 are indicated in the figures to identify the sections of the girder at the piers

For the purpose of clarity, the results from the response-spectrum analysis (i.e., the "Design" and the "UHS" results) are discussed first. It can be seen in Fig. 5.1(a) that for the seismic actions in the longitudinal direction, the UHS envelope of the moments in the bridge girder is somewhat higher than the design envelope. Also, the values of the UHS envelope for the pier (Fig. 5.2(a)) resulting from the longitudinal seismic actions are larger than those of the design envelope in the upper 25 m of the pier. The largest differences are approximately 20%. Table 5.1 also shows that the vertical displacements at the mid-spans of the portal frames (P29-P30 and P31-P32) obtained from the longitudinal seismic actions represented by the UHS are about 20% larger than the design values. The UHS vertical displacements at all other sections are smaller than the design values. These observations for the longitudinal seismic actions were expected because the periods of the predominant longitudinal and vertical modes of the bridge are shorter than 1.5 s, i.e., these are within the range in which the uniform hazard spectrum is higher than the design spectrum (Fig. 2.1). For seismic actions in the transverse direction, the UHS envelopes of the moments in the bridge girder and in the pier (Figs. 5.1(b) and 5.2(b) respectively), as well as the maximum transverse displacements (Table 5.1) are all smaller than the design values. This is because the uniform hazard spectrum is lower than the design spectrum for the periods of the predominant transverse modes, i.e., periods longer than approximately 2.0 s (Fig. 2.1).

With regard to the response results obtained from the time-history analysis of the model subjected to the selected sets of excitations, it can be seen in Figs. 5.1 and 5.2, and Table 5.1 that the maximum moments and displacements are all smaller than the design responses for both the longitudinal and transverse excitations. This was expected since the response spectra of the excitation motions used in the analysis are lower than the design spectrum for periods longer than approximately 0.5 s (see Figs. 4.1 to 4.3), i.e., within the period range of the longitudinal and transverse modes that produce almost the entire response. The contributions of the modes with periods below 0.5 s, where the spectra of the excitation motions exceed the design spectrum, are very small.

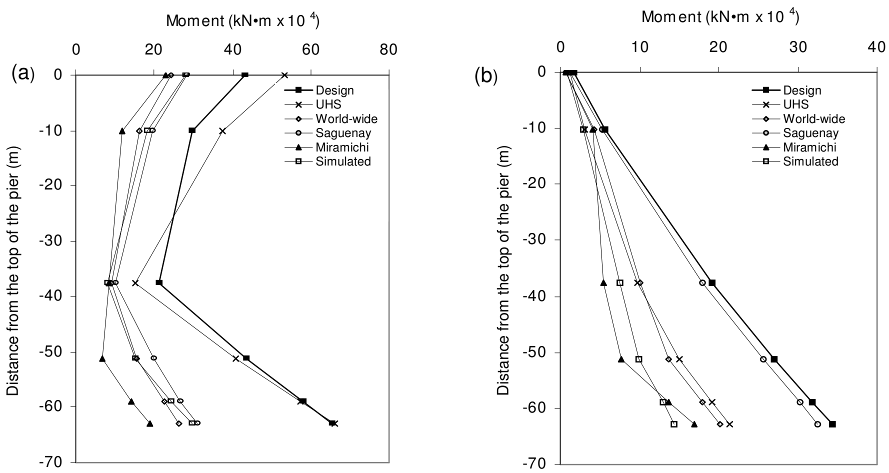

Figure 5.2. Moment envelopes for pier P31: (a) in longitudinal direction and (b) in transverse direction

Table 5.1. Maximum displacements of the bridge girder

Location	Vertical displacement (mm)						Transverse displacement (mm)					
	Design	UHS	World-wide	Sagu-enay	Mira-michi	Simu-lated	Design	UHS	World-wide	Sagu-enay	Mira-michi	Simu-lated
End of cantilever, left of P32*	171	123	65	68	20	60	129	50	41	80	24	34
Pier 32	0	0	0	0	0	0	43	28	18	27	8	14
Mid-span (P31-P32)	93	108	39	40	11	45	153	59	54	74	22	48
Pier 31	0	0	0	0	0	0	50	27	20	39	12	19
Bearing (left)	156	156	65	79	22	64	107	46	25	98	30	30
Middle of drop-in girder	78	74	32	38	10	39	105	43	37	95	29	34
Bearing (right)	163	148	73	88	27	79	164	62	46	94	29	42
Pier 30	0	0	0	0	0	0	59	31	18	39	12	15
Mid-span (P29-P30)	86	101	36	39	11	47	167	65	56	85	26	55
Pier 29	0	0	0	0	0	0	62	30	22	49	15	18
End of cantilever, right of P29	153	111	62	63	23	60	103	45	37	108	33	36

*Location, see Figure 3.1.

6. CONCLUSIONS

The Confederation Bridge, with its length of 12.9 km, is one of the longest reinforced concrete bridges built over water in the world. It crosses the Northumberland Strait in eastern Canada and connects the province of Prince Edward Island and the province of New Brunswick. The bridge was designed for a service life of 100 years, which is twice the service life of typical highway bridges. The objective of this study was to investigate the performance of the Confederation Bridge due to seismic excitations expected at the bridge location. A finite element model of a typical segment of the bridge was subjected to selected seismic motions representative of the seismic hazard for the bridge location. The response results obtained from the dynamic analysis of the model were compared with the seismic design parameters.

In this study, it was found out that a finite element model consisting of 3-D beam elements is suitable for the Confederation Bridge, provided that the foundation flexibility is taken into account in the modelling. Given the characteristics of the Confederation Bridge, the modelling method used in this study is considered to be applicable to long-span single-box girder bridges in general. The responses of the bridge for the excitation motions considered in this study were smaller than the estimated design values, with the exception of the responses from the seismic actions represented by the uniform hazard spectrum, which were somewhat larger (maximum difference is 20%) than the design values. However, considering the conservative assumptions involved in the design through the use of factored material strengths and specified safety factors, as well as the characteristics of the uniform hazard spectrum as discussed earlier, the exceedance of the design responses by 20% does not represent any concern regarding the seismic safety of the bridge.

REFERENCES

Adams, J., and Halchuk, S. (2003). Forth generation seismic hazard maps of Canada: Values for over 650 Canadian localities intended for the 2005 National Building Code of Canada. Open File 4459, Geological Survey of Canada, Ottawa, Ontario, Canada.

Atkinson, G. (1995). Ground motion relations for use in eastern hazard analysis. *Proceedings of the 7th Canadian Conference on Earthquake Engineering*, pp. 1001-1008.

Cheung, M.S., and Li, W.C. (2003). Static and dynamic modelling of the Confederation Bridge. *Proceedings of the Annual Conference of the Canadian Society for Civil Engineering*: 10 pp.

CSI. (2000). SAP 2000 integrated software for structural analysis and design. Computers and Structures Inc., Berkeley, Calif.

Ghali, A., Elbadry, M., and Megally, S. (2000). Two-year deflections of the Confederation Bridge. *Canadian Journal of Civil Engineering*, **27(6):** 1139-1149.

Jaeger, L.G., Mufti, A.A., Tadros, G., and Wong, P. (1997). Seismic design for the Confederation Bridge. *Canadian Journal of Civil Engineering,* **24(6):** 922-933.

JMS. (1996). Design criteria – Northumberland Strait Crossing project. Revision 7.2. J. Muller International – Stanley Joint Venture Inc., San Diego, Calif.

Lau, D.T., Brown, T., Cheung, M.S., and Li., W.C. (2004). Dynamic Modelling and behavior of the Confederation Bridge. *Canadian Journal of Civil Engineering*, **31(2):** 379-390.

Lin, L. (2005). Seismic evaluation of the Confederation Bridge. M.A.Sc. thesis, Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, Canada.

Munro, P.S., and Weichert, D.H. (1989). The Saguenay earthquake of November 25, 1988 – Processed strong motion records. Open File Report No. 1966, Geological Survey of Canada, Energy, Mines and Resources, Ottawa, Ontario, Canada.

Newmark, N.M., Blume, J.A., and Kapur, K.K. (1973). Seismic design spectra for nuclear power plants. *Journal of the Power Division*, ASCE, **99(PO2):** 287-303.

Newmark, N.M., and Hall, W.J. (1982). Earthquake spectra and design. Monograph, earthquake Engineering Research Institute, Berkeley, Calif.

Tremblay, R., and Atkinson, G.M. (2001). Comparative study of the inelastic seismic demand of eastern and western Canadian sites. *Earthquake Spectra*, **17(2):** 333-358.

Weichert, D.H., Pomeroy, P.W., Munro, P.S., and Mork, P.N. (1982). Strong motion records from Miramichi, New Brunswick, 1982 aftershocks. Open File Report 82-31, Energy, Mines and Resources Canada, Ottawa, Ontario, Canada.