SEISMIC RETROFIT OF THE MACDONAL CARTIER BRIDGE USING BASE-ISOLATION

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ABSTRACT: The Macdonald Cartier Bridge is a 650 m long, five span continuous twin steel box girder structure supported on mass reinforced concrete piers founded on bedrock. This major interprovincial link links Ottawa, ON and Gatineau, QC. The Owner wanted to seismically retrofit the bridge to the current code lifeline requirements while improving functionality and roadway geometrics. Multi-mode spectral analysis revealed unacceptable performance for the piers under the original articulation. Conventional retrofit comprising seismic load sharing among various piers was considered costly due to associated in-river works at Pier 2. A subsequent non-linear time-history analysis indicated that seismic isolation would resolve the deficient substructure behaviour. The increased lateral displacement demands at the approach span to main span interface were accommodated by incorporating new specialty seismic deck joints effectively disconnecting the main bridge and the approaches from each other. The Gatineau Approach abutment founded in soft soils on piles showed resonant response leading to very high demands corresponding to the site-specific time histories. This would lead to a costly and disruptive retrofit decision of the Gatineau Approach abutment. In lieu of a costly FLAC analysis, a code based assessment of the Gatineau Approach Span structure was carried out to ascertain the abutment performance corresponding to the current design code level seismicity. This approach was adopted since the approach span can be considered as an independent structure undergoing no interaction with the main span. The construction of both bridge widening and seismic retrofit works is currently under way. This paper presents the seismic analyses and retrofit strategy adopted for this lifeline bridge to achieve the prescribed seismic performance by the Owner.

1. Introduction

The Macdonald-Cartier Interprovincial Bridge represents one of the most important transportation links between the cities of Ottawa, Ontario and Gatineau, Quebec by providing 6 traffic lanes (and two sidewalks) with an annual average daily traffic (AADT) or nearly 70,000 vehicles (Fig. 1). The bridge spans the Ottawa River from King Edward Avenue, just east of Ottawa's Byward Market to connect with AutoRoute 5 through Gatineau and western Quebec. The structure is the principal interprovincial structure in the National Capital Region for the commuters and the transportation of goods. The structure plays a vital role in business and tourism in the National Capital Region.

Constructed between 1964 and 1966, the bridge is a non-prismatic five span continuous, built-up steel plate box girder, with transverse floor beam and stringer system supporting a reinforced concrete deck (Fig. 2). The overall length of the main structure is 618.75 m, consisting of five symmetrical spans of 88.4 m, 141.4 m, 159.1 m, 141.4 m, and 88.4 m. The overall bridge deck width is 30.07 m. In its present configuration, the bridge supports three lanes of traffic and one sidewalk in each direction. The northbound and southbound lanes are separated by a 1.2 m wide concrete median with a concrete curb and a steel railing. The sidewalks are 2.24 m wide with a 200 mm curb on the traffic side and an open steel railing on the exterior for a clear width of 1.83 m.
The structure is supported on two reinforced concrete abutments and four large reinforced concrete piers. The Ottawa main span abutment and the piers are founded on spread footings. The Gatineau main span abutment is founded on two cast-in-place concrete caissons. Each main span abutment contains a mechanical room chamber with a simply supported approach span overhead consisting of concrete T-beams and concrete box girders. The Gatineau approach span abutment is founded on piles, while the Ottawa approach span abutment is founded on a spread footing with wing walls.

Fig. 1 – Aerial View of Macdonald Cartier Bridge

Fig. 2 – Steel Superstructure and Deck Soffit

In 2009, a structural evaluation of the structure for dead, live and seismic loads was undertaken by MMM Group (McCormick Rankin) at Public Works and Government Services Canada's (PWGSC) request. The results for the live load evaluation demonstrated that the structural components, with the exception of the sidewalks and railings which had both been observed to be in an advanced state of deterioration, had sufficient capacity with respect to the prescribed CL-625-ONT live load. The seismic evaluation was completed by means of a response spectrum analysis in general conformance with Section 4.11 of CSA S6-06.

The results of the response spectrum analysis demonstrated that for both an emergency route or lifeline classification the structure required the implementation of seismic retrofit measures to meet CHBDC requirements. The seismic deficiencies included the following:

- Bearings did not have sufficient capacity to transmit anticipated seismic shear forces to the substructure.
- Piers did not have adequate ductility to resist anticipated longitudinal seismic force demands assuming fixed base conditions (Pier 2 for emergency route classification and Piers 2, 3 and 4 for lifeline classification).
- Pier footing geometry could lead to uplift and the initiation of rocking behaviour during a significant seismic event. Although this behaviour would lengthen the period of response of the structure and reduce force effects, very large displacements (i.e. approximately 1.5 m) would have to be accommodated by the structure.

2. Feasibility of a Conventional Retrofit Strategy

The Owner wanted to seismically retrofit the bridge to CHBDC lifeline requirements while improving functionality and geometrics with a multi-use pathway and sidewalk and improved roadway geometrics

requiring a 4.0 m deck widening. The existing main-span superstructure is supported on rocker bearings on all piers except Pier 2, where fixed bearings are used to transfer longitudinal shear forces from the superstructure to substructure. A conventional retrofit strategy was first considered; this comprised replacement of the existing rocker bearings on Piers 3 and 4 and the fixed bearings on Pier 2 with conventional pot bearings that would essentially “lock up” during a seismic event allowing the load to be shared among Piers 2, 3, and 4. Pier 1 was not included as a possible load sharing mechanism, since the geometry and detailing of the pier section and foundation did not provide a reliable lateral load resisting system. Sharing loads between all 4 piers would not have significant reduction to the loads in Piers 2, 3, and 4 and would have likely required retrofit to the piers in order to provide sufficient capacity.
To assess the feasibility of this option a preliminary response spectrum analysis was performed on the proposed strategy and demand-capacity ratios were calculated. It was found that this strategy did not reduce the demands on Pier 2 to within its elastic capacity. For this retrofit option to work, expensive pot bearings would have to be used in addition to carrying out costly in-river retrofits to Pier 2. Such in-river works would entail, at a minimum, pier base widening or anchor installation to prevent foundation overturning, pier wall strengthening using fiber wrapping or steel beam assemblies, and coffer dam installation to facilitate these activities. These activities were deemed complex and the preliminary associated cost estimate was in excess of $8M (2013 dollars) assuming all went well. This retrofit option was not considered cost-effective and practical, and was therefore explored any further.

3. Base Isolation as a Potential Retrofit Strategy

Base isolation can be employed as an effective retrofit strategy if certain conditions are present. In particular, the following three conditions, alone or together, can warrant base isolation as an effective retrofit as discussed in Priestly et.al (1996).

1. Bridge has stiff piers, with a short period of vibration.
2. Bridge is highly non-regular, for example, with piers of significantly different heights or stiffness, and therefore prone to concentration of ductility demands on stiffer piers.
3. Nature of expected ground motion is well characterized with high dominant frequencies and low energy at large periods of vibration. This usually means shallow earthquakes and foundations on rock.

The MacDonald Cartier Bridge exhibits each of these conditions. The bridge has stiff wall piers, with the fundamental longitudinal and transverse periods of the existing bridge at 2.25 and 0.75 seconds respectively. Although the piers are not of considerably different cross sectional dimensions, the piers have varying heights. In addition, the superstructure is only pinned to Pier 2 longitudinally, thus presenting the possibility of high force demands at this support. This was shown to be the case during the as-built structure assessment in the first phase. In addition, the source earthquakes are expected to be a result of shallow crustal events, while all foundations except the Gatineau abutment are founded on rock.

Base isolation can expect to address these issues efficiently in the following manner:

(a) Base isolation can impart flexibility to the system by shifting the fundamental period of the structure thus reducing seismic demands on the structural elements. However, this has to be accompanied by an effective energy dissipation mechanism to control structural displacements. Energy dissipation increases effective equivalent damping of the structure resulting in the reduction of force and displacement demands.

(b) Base isolation can modify the structural response such that it will not only reduce the overall seismic demand but also cause piers of different stiffnesses to attract more uniform force demands.

Given the bridge vintage and era, a critical issue for the MacDonald Cartier Bridge was the absence of appropriate transverse reinforcement quantities in the piers along with poor seismic detailing. This meant small ductility capacities and the tendency of plastic hinge locations to exhibit brittle failure under cyclic loading. Base isolation was therefore deemed to be an efficient strategy for this structure as it would lengthen the natural period reducing the overall global seismic demands in addition to making the structural response more uniform. This strategy targeted a significant amount of reduction in pier demands so that these would not exceed their elastic limits and ideally have some reserve capacity keeping in line with capacity-protected design principles.

In addition, the bridge structure has rocker bearings that have been shown to be vulnerable under seismic loading conditions. As described in the subsequent sections of this paper, all of the above-described objectives were achieved by using friction pendulum bearings as the proposed base isolation devices.

3.1. Friction Pendulum Bearings for Base-Isolation

As discussed earlier, increased flexibility can reduce seismic demands but it may lead to unacceptably large displacements. The performance requirements for the MacDonald Cartier Bridge stipulated that the bridge must remain in full operation after the 1:475 year design earthquake event. In addition, the structure needed to accommodate emergency vehicles after the 1:975 year design earthquake event. Friction pendulum bearings have adequate energy dissipation along with self-centering characteristics. In addition, they exhibit consistent performance over large temperature ranges, which is critical given the geographical setting of this bridge. The consistent performance over a large temperature range made the friction pendulum isolators the preferred isolation option as other products (such as Lead Rubber Bearings and Elastomeric Bearings) perform inconsistently over large temperature ranges. As these devices have energy dissipation properties that depend on the vertical load exerted on them by the structure, two designs of friction pendulum isolators were used for this project, namely, Pier and Abutment isolators. The Abutment isolators have different properties than the Pier isolators in order to provide the desired performance with different axial loads. Friction pendulum bearings were therefore employed as base isolation devices for the MacDonald Cartier Bridge Analysis.

3.2. Base Isolation Analysis

A set of five spectrally matched time histories were used to carry out the non-linear time history analysis for this structure. The lifeline bridge performance requirements per the Canadian Highway Bridge Design code CAN/CSA-S6-06 (2006) require the bridge to be open to all traffic after a 1 in 475 year earthquake event, while ensuring emergency vehicle access corresponding to the 1 in 975 year earthquake event. The site-specific response spectrum corresponding to the 1 in 975 year earthquake event was found to be smaller than the 80% code spectrum cut-off (0.8 Csm) corresponding to the 1 in 475 year earthquake event. The site-specific spectrum corresponding to 1 in 975 year event was therefore ignored and the target spectrum values were taken as 0.8Csm for the 1 in 475 year event, whereby the Csm values were calculated using the code approach employing both the Soil Coefficient (S) and the Importance Factor of 1. The design response spectrum and the seed records employed for producing spectrally matched time histories are shows in Fig.3 and Table 1, respectively.

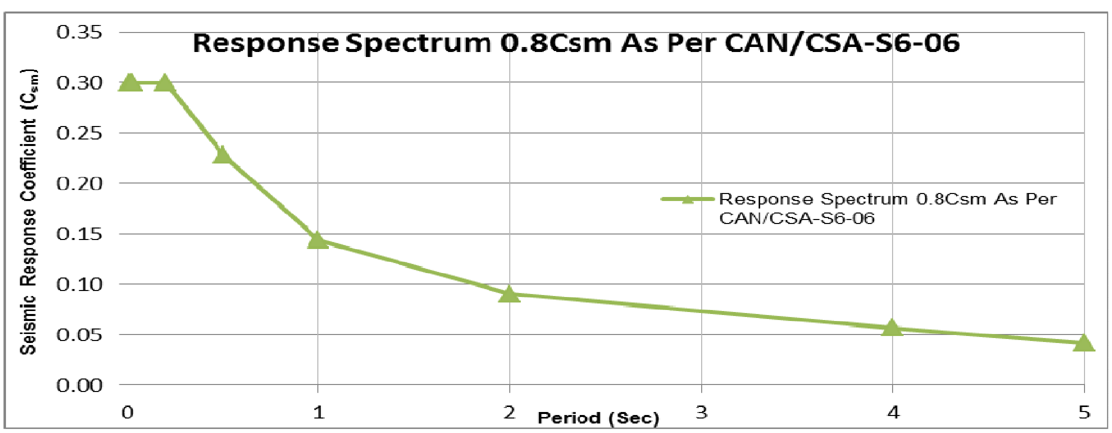

Figure 3 – Site Specific Response Spectrum

Table 1 – Input Earthquake Seed Time Histories

Set	Earthquake	Magnitude	Stations
1	1971 San Fernando	6.6	Lake Hughes #4
2	1986 N. Palm Springs	6.1	Winchester Bergman
3	1979 Coyote Lake	5.7	Gilroy #1
4	1994 Northridge	6.7	LA - Wonderland
5	1985 Nahanni	6.8	Site 3

The non-linear modal analysis or the Fast Non-linear Analysis using Ritz Vectors implemented in CSI Bridge is particularly suitable for structures with non-linearities occurring in a few, preselected elements/locations. Since the base-isolation retrofit scheme required the induction of friction pendulum bearings between the superstructure and substructure, with the anticipation that the piers would behave elastically, this method was determined to be most suitable and efficient. This method was not only efficient in terms of the computational effort and time but also helped achieve a constant 5% damping for all structural modes. A lower damping ratio of 2% was used for the primary isolation modes. Additional damping due to bearing hysteresis was explicitly accounted for in the Friction Pendulum Bearing elements.

As FNA uses nonlinear modal superposition to carry out the analysis, solution accuracy and duration are dependent on the number of Ritz modes that are used. For the purposes of obtaining the realistic behaviour of the bridge superstructure, the isolators, and the piers in the transverse and longitudinal direction, 306 modes were selected as the solution basis; this was used as an efficiency measure as the analysis (and processing of results) became much more time consuming as the number of modes was increased. It was found that 306 modes were sufficient to determine bending and shear demands but insufficient to capture high frequency behavior, such as torsion of the wall piers indicating unrealistically high demands. To verify whether these high torsion values were caused due to numerical instabilities in the analysis procedure and could therefore be ignored, 1200 Ritz modes were run resulting in the elimination of high torsion values. This verified that the high torsion loads obtained from the 306 Ritz mode analysis could be ignored.

The existing finger joints were likely to be replaced as part of the structural rehabilitation and prone to damage due to large in-plane movements caused by the design earthquake. Specialty seismic joints that can accommodate large displacements were considered a likely retrofit. The use of such joints would effectively disconnect the main structure from the approach spans. The base isolation analysis and the resulting assessment were therefore based on assuming no interaction between the approach and main spans.

4. Gatineau Approach Span Abutment and Piles

The Gatineau approach span abutment founded on 66 H-piles driven to bedrock had extremely high demands when analyzed using the free-field input time history provided by the geotechnical consultant, Golder Associates. It was found that the peak spectral acceleration of the free-field input time history was approximately five times the peak ground acceleration corresponding to rock conditions. This occurred at the fundamental period (0.2 sec) of the Gatineau approach span thus resulting in resonant response (Fig.4). The abutment walls were noted to be highly overstressed and would experience failure under the free field input time history. A D/C value of 4.3 for the wall also indicated a likelihood of high overstress in piles, which meant that a complete replacement or extensive retrofit of the Gatineau Abutment would be required resulting in significant direct retrofit costs in addition to indirect costs due to traffic disruption.

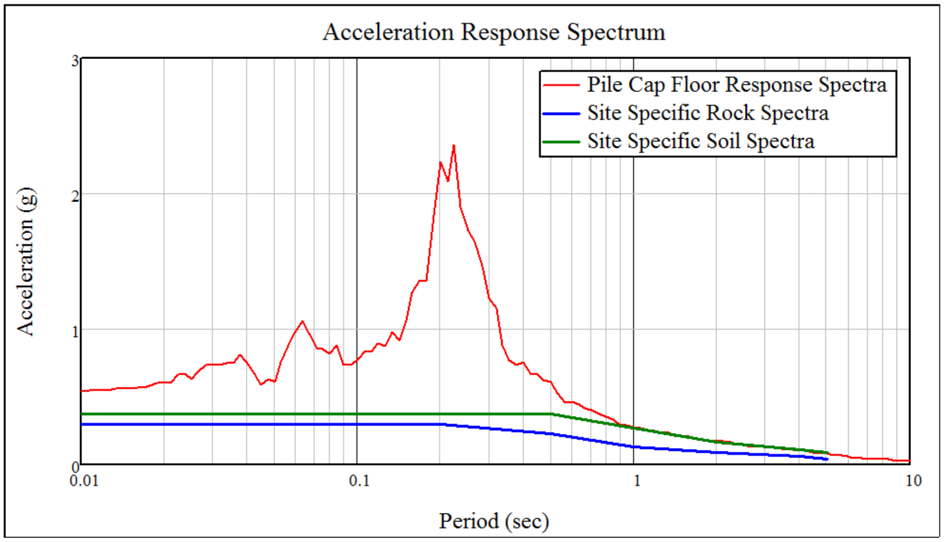

Fig. 4 - Input Pile Time History Response Spectrum

The geotechnical consultant was however of the view that the free field response based time histories would be modified due to the stiffening effects of the piles not accounted for originally. This however required sophisticated and time consuming FLAC analysis accounting for the soil-structure interaction at the abutment location. Such sophisticated analysis would likely show a shift in the resonant frequency of the response spectrum to a lower period reducing the seismic demands on the approach span. In lieu of carrying out such refined and costly FLAC analysis incorporating soil structure interaction effects, a further code based assessment of the Gatineau Approach Span structure was performed to determine and compare the abutment performance corresponding to the current design code level seismicity (CSA-S6-06 Clause 4.4.7.1). This approach was adopted since the approach span could be considered as an independent structure having no interaction with the main span due to the incorporation of specialty seismic deck joints. The methodology entailed employing dynamic analysis using time histories matched to the code design response spectrum for the local site soil class to ascertain the Gatineau Approach Span performance. A soil type III was determined for this location based on the shear wave velocities through the abutment soil profile provided by Golder Associates. This methodology was much more detailed than the current state of practice of designing or assessing such short, single span structures based on the uniform or single mode spectral methods. Spectral matching was performed using the input (rock) time histories for the site as seed events. This process involved using the spectral matching program RSPMatch to adjust the input time histories at the pile cap. Three spectral matching “passes” were conducted in order to find the best fit to the design spectrum; it was found that the second pass had the best fit to the spectral periods of interest.

Figures 5 and 6 show the seed and matched spectra for the Gatineau abutment location.

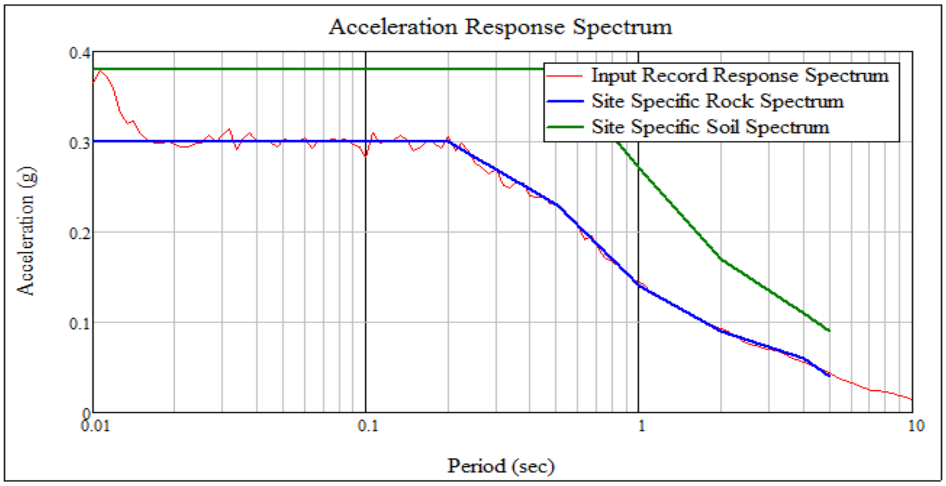

Fig.5 – Input Rock Time History Response Spectrum

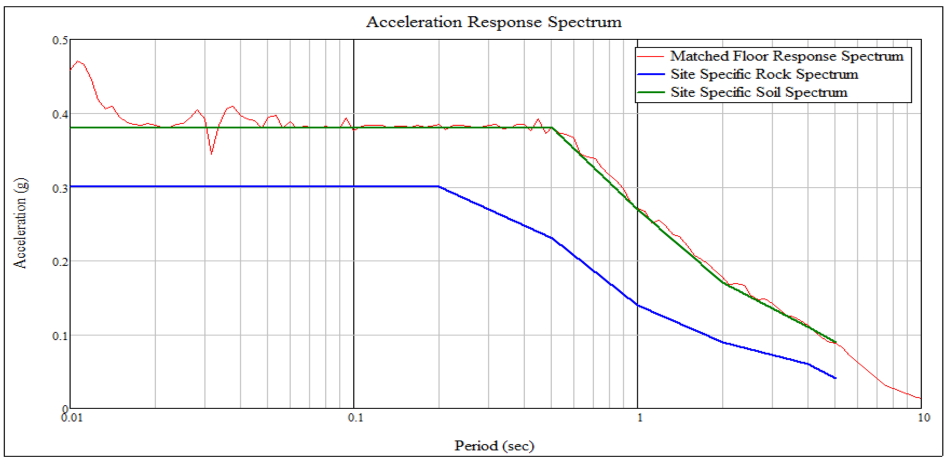

Fig.6 – Matched Time History Response Spectrum at the Pile Cap

It should be noted that only the behaviour of the Gatineau Approach Span was impacted by the adjustment of the pile input time history. All other components were assessed based on the originally provided site specific time histories and found to be adequate for the imposed demands. The results of the assessment are further described in the next section.

5. Results Discussion

Based on the original articulation, the superstructure was only able to transfer longitudinal seismic loads to Pier 2 via a fixed bearing. This location showed failure under this loading scenario. Base-isolation resulted in a lowering of demand at this critical location. D/C values for longitudinal and transverse pier wall shear and flexure are summarized in Tables 2 and 3 respectively, as follows. As shown, all D/C values are below 1.

Table 2 – Pier Wall Shear D/C Values

	Longitudinal Shear		Transverse Shear	
	After Base Isolation D/C	Before Base Isolation D/C	After Base Isolation D/C	Before Base Isolation D/C
Pier 1	0.2	0.38	0.14	0.15
Pier 2	0.14	1.47	0.15	0.19
Pier 3	0.12	0.46	0.18	0.2
Pier 4	0.12	0.45	0.18	0.17

Table 3 – Pier Wall Bending D/C Values

	Longitudinal Bending		Transverse Bending	
	After Base Isolation D/C	Before Base Isolation D/C	After Base Isolation D/C	Before Base Isolation D/C
Pier 1	0.36	0.26	0.08	0.16
Pier 2	0.42	1.46	0.09	0.22
Pier 3	0.49	0.41	0.17	0.33
Pier 4	0.55	0.5	0.20	0.29

Although the main span abutment demands in the existing arrangement did not exceed their capacities, seismic isolation of the entire main span was implemented in order to ensure uniform response and stability of piers during the 1:475 year design earthquake event. Table 4 compares overall structural D/C ratios before and after base isolation.

Table 4 – Main Span Abutment D/C Values

	Longitudinal Shear		Transverse Shear		Longitudinal Bending		Transverse Bending	
	D/C After Isolation	D/C Before Isolation	D/C After Isolation	D/C Before Isolation	D/C After Isolation	D/C Before Isolation	D/C After Isolation	D/C Before Isolation
Gatineau Abutment Base	0.10	0.05	0.07	0.14	0.19	0.03	0.01	0.01
Ottawa Abutment Base	0.10	0.08	0.15	0.14	0.19	0.50	0.04	0.02

Before isolation, the Pier 2 foundation overturning capacity was found to be of concern since it was marginally exceeded (D/C = 1.02). The corresponding D/C value after base-isolation was found to be 0.48, thereby showing a much improved performance. All other pier foundations were also found to have D/C values below 1 (with a maximum value of 0.6 for Pier 1) based on overturning and sliding. The

Ottawa main span abutment foundation showed a D/C value of 0.72. The Gatineau main span abutment caissons showed a maximum D/C value of 0.67 based on flexure and shear.
For the Ottawa approach span abutment, the wall longitudinal flexure and overturning D/C values were found to be 1.02 and 1.05 for the isolated case. The shear and sliding D/C values were shown to be adequate. Given the very slight overstress and the fact that the weight and restraining effects of the winwalls were ignored for the longitudinal direction, the abutment performance was considered acceptable.
As mentioned earlier, the Gatineau approach span abutment founded on 66 steel H-piles driven to bedrock had exceptionally high demands when analyzed using the free-field input time history provided by Golder. The abutment wall showed D/C values of 1.9 and 4.3 for longitudinal shear and flexure, respectively for the free-field input motions. A D/C value of 4.3 for the wall also indicated a likelihood of high overstress in piles, which meant that a complete replacement or extensive retrofit of the Gatineau Abutment would be required resulting in significant direct retrofit costs in addition to indirect costs due to traffic disruption. However, as shown below in Table 5, the demands based on time histories spectrally matched to the code design spectrum showed a significant reduction. All abutment wall D/Cs shown in Table 5 are either below or just below 1.0 thus circumventing the need for any retrofits. The maximum pile D/C was found to be 1.2. Given that these are steel H-piles, this slight overstress was deemed acceptable.

Table 5 - Gatineau Approach Span Abutment Wall D/C Values – Code Spectrum Matched Time Histories

	Longitudinal Shear			Transverse Shear			Longitudinal Bending		
	$D_{isolated}$ (MN)	C (MN)	$D_{isolated}$/C	$D_{isolated}$ (MN)	C (MN)	$D_{isolated}$/C	$D_{isolated}$ (MN)	C (MN)	$D_{isolated}$/C
Gatineau	6.3	6.8	0.92	4	22	0.17	21.9	22	0.99

The superstructure (girders and cross bracing) demands were also extracted from the seismic model; all D/C's were found to be below 1.

6. Retrofit Design

The seismic isolation strategy involved replacement of all existing main span girder bearings at piers and main span abutments with friction pendulum type bearings. The friction pendulum bearings provide consistent performance under large temperature fluctuations as is the case for the site. Two different types of friction pendulum bearings were designed, one for piers and one for the main span abutment locations.
As the bridge main span was to be isolated in both the longitudinal and transverse direction, the expansion joints at the interface between main and approach spans need to be able to handle the increased in-plane displacements resulting from seismic isolation. Two scenarios were considered to accommodate the differential movement between the approach span and the main span during a seismic event, namely, (a) no immediate retrofit and accepting the risk of damage during the design earthquake, and (b) replacing the existing with new seismic expansion joints. No immediate retrofit would likely result in significant damage to the existing finger joints during a seismic event. Following a seismic event, steel plates would have to be placed over the damaged joints to allow for immediate use of the bridge, following which the damaged expansion joints would have to be replaced. This behaviour not recommended as it did not ensure independent vibration of the main and approach spans, in addition to not meeting the prescribed "Immediate Use" performance objective following a 1:475 design earthquake. The final retrofit solution was to replacing the existing finger joints with robust new seismic expansion joints which would allow for differential displacements in both longitudinal and transverse directions (+/-300mm longitudinally, +/-250mm transversely). These new joints would allow the approach and main span to move freely of one another in each direction.
In addition to the above, new elastomeric bearings were also incorporated for retrofit/rehabilitation of the approach span girders.

7. Conclusions

The Owner wanted to seismically retrofit the MacDonald Cartier Bridge to CHBDC lifeline requirements while improving functionality and geometrics. An initial response spectrum based assessment revealed several vulnerabilities including deficient bearings, inadequate pier longitudinal performance and likelihood of pier footing rocking and uplift. A conventional retrofit strategy comprising steel rocker and pin bearings with pot bearings thus effecting seismic load sharing between piers proved ineffective. The remaining effort was devoted to the base isolation retrofit strategy using friction pendulum bearings. Non-linear time history analysis was employed per the Canadian Highway Bridge Design Code (CHBDC) to determine seismic demands for the base-isolated structure. The time-histories were matched to the site-specific response spectrum limited to a minimum value of 80% of the code spectrum at all periods (CHBDC clause 4.4.7.3), as directed by the PWGSC. The use of friction pendulum bearings reduced the seismic demands on all main span piers considerably and brought them to within the elastic capacities. This, however, increased the lateral displacement demands at the approach span to main span interface and specialty seismic deck joints were therefore considered for accommodating seismic displacements at the interface locations. The incorporation of these specialty joints serves to disconnect the main bridge and the approaches from each other, leading to an independent response of each structure. The Ottawa

The Ottawa Approach abutment is founded on a spread footing in rock and did not require any retrofit as assessed by using the site specific time histories. The Gatineau Approach abutment is founded in soft soils on piles and the abutment showed resonant response leading to very high demands and failure when assessed based on the site-specific time histories derived from a simplified free-field assumption. This would lead to a costly and disruptive retrofit decision of the Gatineau Approach abutment. As the response of the free-field analysis was conservative and neglected the beneficial effects of soil-structure interaction, it was not considered to be realistically representative of the site conditions. In lieu of carrying out a much more refined and costly FLAC analysis incorporating soil structure interaction effects, a further code based assessment of the Gatineau Approach Span structure was carried out to determine and compare the abutment performance corresponding to the current design code level seismicity. This approach was adopted since the approach span could be considered as an independent structure undergoing no interaction with the main span due to the incorporation of specialty seismic deck joints. The methodology entailed the use of dynamic analysis employing time histories matched to the code design response spectrum for the local site soil class (III) to ascertain the Gatineau Approach abutment performance. This methodology is much more detailed than the current state of practice of designing or assessing such short, single spans based on the uniform load or single mode spectral methods. The assessment showed a significant demand reduction and no need for retrofit of the Gatineau Approach abutment wall. A slight overstress of the abutment piles was noted but it was deemed that given the large number of piles and the inherent ductility of steel, no pile retrofit was needed.

8. References

Canadian Highway Bridge Design Code, CAN/CSA-S6-06, 2006

Priestly, M.J.N., Seible, F., and Calvi, G.M., “Seismic Design and Retrofit of Bridges”, 1996.