Performance based Seismic Evaluation of the Jacques-Cartier Bridge Part 2 : Engineer's perspective

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ABSTRACT

The Jacques-Cartier Bridge is a 3.4 km long, 5-lane, truss bridge connecting Montréal to the south shore. The bridge was opened in 1930 and was extensively modified in 1955 to accommodate construction of the St. Lawrence Seaway – this required raising piers of the south approach by as much as 25 m. Most of the bridge is supported on massive concrete or concrete and stone masonry piers, all of which are either only lightly reinforced or completely unreinforced. This means the piers have essentially no ductility and must resist seismic demands elastically. Because of the size, age and configuration of the piers, there is significant uncertainty regarding their capacity.

The seismic evaluation was performed for three levels of ground motions, with 475, 975 and 2475-year return periods, in accordance with the performance-based principles defined in the current Canadian Highway Bridge Design Code, CAN/CSA-S6-14. For this evaluation, the expected performance of the bridge was compared to the specified performance objectives for a new Lifeline bridge. The large number of different structural systems, including a building supporting part of the bridge, meant the evaluation had to be very focused and efficient to meet the project schedule. Because of the overall lack of ductility in the structure, the evaluation was based primarily on linear-elastic analyses (response spectra), using simplified, partial and representative models.

Not surprisingly, the majority of the issues with the seismic performance are due to the lack of reinforcement, and thus lack of reliable capacity and ductility, in the massive concrete piers. Spherical bearings which date from the initial construction are another element of concern. The remainder of the bridge is expected to have some amount of damage for the different ground motions considered; however, the issues are not as significant as the piers and bearings.

Keywords: Bridge, Seismic Evaluation, Performance Based Design, Unreinforced Concrete, Masonry Piers

INTRODUCTION

The Jacques-Cartier Bridge in Montréal, Québec, is one of 5 major crossings connecting the island of Montréal to the south shore (the others being the Honoré Mercier, Champlain, and Victoria bridges and the Louis-Hippolyte Lafontaine tunnel and bridge). Built from 1925 to 1930, it is the second-oldest of the crossings. It carries approximately 115,000 vehicles a day, making it the third-busiest crossing in Canada.

The bridge is owned and operated by the Jacques-Cartier and Champlain Bridges Incorporated (JCCBI) corporation. In 2017, JJCBI issued a mandate to perform a seismic evaluation of the bridge in accordance with the performance-based seismic provisions of the current Canadian Highway Bridge Design Code, CAN/CSA-S6-14 [1]. The first part of the mandate, which is the object of this paper, was to evaluate the seismic performance of the existing structure and identify the deficiencies relative to the performance criteria for a new Lifeline bridge. The next step will be to develop appropriate performance objectives for the seismic retrofit considering use, costs, remaining service life, and public safety. Further details of the mandate and the owner's objectives are presented in a companion paper by Loubar et al. (JCCBI) [2].

THE BRIDGE

The Jacques-Cartier bridge carries five lanes of traffic and connects Montréal to Longueuil, with entrance and exit ramps to Île Sainte-Hélène at the mid-point. The bridge is 3.4 km long and is divided in 9 sections, numbered from south to north as shown in Figure 1, each with its own structural system:

- Section 1: roadway interchange on the south shore, on fill (excluded from mandate);
- Section 2: 420 m, 9 simply supported deck-truss spans on lightly/unreinforced concrete and masonry piers; piers raised in 1955 to accommodate the Seaway;
- Section 3: 76 m simply-supported through-truss span over the St Lawrence Seaway on lightly/unreinforced concrete and masonry piers; piers raised in 1955;
- Section 4: 673 m, 9 simply-supported deck-truss spans; last span connected to building in Section 5; half the piers raised in 1955; piers founded on rock except for two founded on timber piles;
- Section 5: 3-story pavilion supporting roadway at roof level; open floors; lightly reinforced walls; founded on rock;
- Section 6: 256 m, 3 simply-supported deck-truss spans; half-span connecting to building; piers founded on rock;
- Section 7: 590 m mainspan; cantilevered through-trusses with suspended section mid-span; on four piers – three on rock, one on soil;
- Section 8: 592 m, 16 simply-supported deck-truss spans; supported on pairs of steel towers except for one span on four lightly reinforced concrete columns with masonry cladding; towers are on unreinforced concrete pedestals, some on spread footings, some on timber piles;
- Section 9: 62 m, 6 spans, concrete portals (excluded from mandate - under consideration for replacement given severe deterioration).
- The bridge also includes two access ramps connected to the Pavilion; upstream ramp was evaluated; downstream ramp was excluded as it was recently rehabilitated and retrofitted.

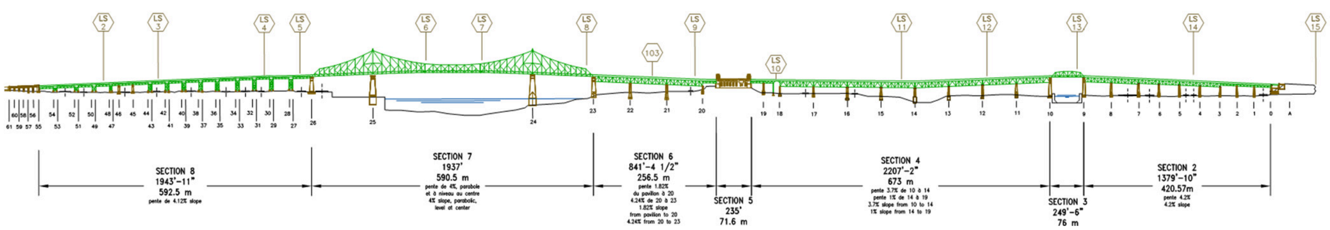


Figure 1. Jacques-Cartier Bridge elevation view

The massive piers supporting sections 2, 3, 4, 6 and 7 consist of essentially unreinforced concrete and masonry and thus have little to no ductility. To complicate matters, in 1955, the piers in sections 2, 3 and 4 were raised up to 25 m to make way for the St. Lawrence Seaway, introducing potential stability problems under seismic loads. The other significant modification to the bridge was a complete replacement of the concrete deck in 2000 to 2002. Many of the bearings were replaced; however, a number of original bearings remain in service.

METHODOLOGY

The seismic evaluation was done following the performance-based approach defined in the CAN/CSA-S6-14 code. Unlike previous versions of the code that used a force-based approach where seismic demand is checked against capacity for individual elements, the performance-based approach aims to evaluate the seismic response of the structure in terms of damage levels and service level. This requires a more holistic approach to evaluating the seismic performance that examines how the damage in different elements impacts the overall functionality of the bridge post-earthquake. For this evaluation, the expected response of the structure is compared against the performance objectives for a new Lifeline bridge defined in the code:

- For 475 yr return period ground motions: immediate service / no damage
- For 975 yr return period ground motions: immediate service / minimal damage
- For 2475 yr return period ground motions: limited service / repairable damage

The evaluation used the design response spectra defined in S6-14. These spectra are defined assuming a 5% global damping ratio, which is appropriate for typical concrete highway bridges. For modern steel bridges, a lower damping ratio around 2% is sometimes used, although it should be noted that this lower value is largely based on in-service ambient vibration testing of bridges and thus implies small displacements and no damage. A useful reference for this project was a vibration study by Lau and Humar [3] of the Alexandra Bridge, which is a similar large truss bridge built in 1900. They found that a damping ratio of 2% to 3% was representative for the in-service condition. For the Jacques-Cartier bridge there are many sources of uncertainty regarding the damping ratio: the steel trusses have many more members and connections than newer steel bridges, the level of damage in the concrete piers will significantly influence energy dissipation in the structure, different bearing types will have different levels of damping, and any damping from the soil will differ between shallow, deep and piled foundations and whether they are on rock or soil. Overall, the damping will likely be different for each level of earthquake ground motion. Rather than try to predict the appropriate value for each case, values of 2% and 5% were used throughout the evaluation. This allows the impact of the damping ratio on the expected performance of the structure to be evaluated. The damage levels and displacements estimated during the evaluation can then be used to inform the choice of appropriate damping ratio to use in designing the seismic retrofits.

The large number of different structural systems, particularly in the substructure, meant the evaluation had to be very focused and efficient to meet the project schedule. Because of the overall lack of ductility in the structure, the evaluation could be based primarily on linear-elastic analyses (response spectra). Outside of the main span (section 7), all the spans are simply supported: this meant they could be considered individually rather than having to consider the bridge all at once. Using this approach, a few representative spans could be evaluated and the results applied to large sections of the bridge: this was true for sections 2, 4, 6, and 8. Sections 7 and 3 were considered individually given the unique nature of the trusses in those sections.

A complete model of section 7 was prepared (Fig. 2). It was used primarily to evaluate the superstructure, but also to calibrate a simplified spine model which could then be used to focus on the response of the piers. Similarly, for the south approach one-span and two-span detailed models (Fig. 3) were used to evaluate the superstructure but also to calibrate simplified pier models. Using simplified models of the piers, with the superstructure represented as a point mass, allowed for the rapid evaluation of various assumptions and scenarios (e.g., varying damping ratios, concrete properties, soil stiffnesses, bearing stiffnesses). It is important to note that the concrete piers have a significant mass of their own, representing 3/4 of the total pier and superstructure mass, and are much stiffer than the superstructure. The seismic demands in the piers are thus dominated by the behaviour of the piers themselves. Simplifying the superstructure to a spine beam or a point mass allows the analyses to efficiently focus on the response of the piers.

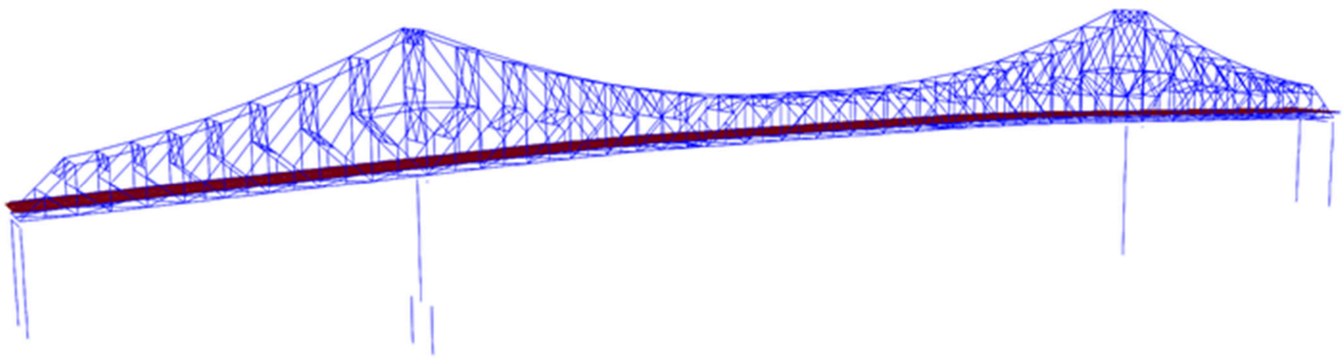
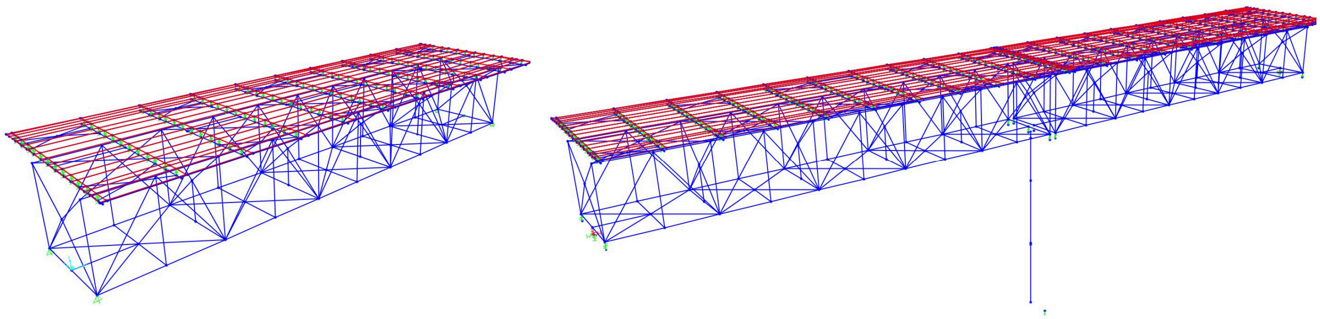


Figure 2. 3D model used for analysis of Section 7.

Complete three-dimensional models of the steel superstructure and deck of representative spans of sections 2, 4, 6 and 8 were created. A multimodal spectral analysis was performed to determine the forces and displacements in the different elements of the bridge. Frame and shell elements are used in the model, as well as linear springs to model bearings. More complex models joining two spans and a pier were assembled in order to validate modelling assumptions and behavior for the simpler models used for the analysis of the piers themselves and models of the frame and deck only to evaluate the superstructure.



a) 3D model of one 245ft span. *b) 3D model of two 245ft spans and a pier.*

Figure 3. 3D models used for analyses of the south approach superstructure.

It further makes sense to use simplified models and analyses for the seismic evaluation when considering that the objective is to identify deficiencies with sufficient certainty to know they need to be addressed. A higher degree of precision on damage levels or extent of damage that would come from more detailed (e.g, non-linear) models and analyses is not necessary at this stage; it is more useful when designing the seismic retrofits, to optimize their design and confirm their effectiveness. This is particularly true in this case, where the existing structure has very little inelastic capability.

CONCRETE PIERS

To evaluate the seismic performance of the concrete piers, specific methods and criteria had to be established to account for the lack of reinforcement in the concrete and the presence of unreinforced stone masonry, neither of which are covered in current codes. The size of the piers, with cross sections up to 29 m x 14 m, also puts them outside the realm of current codes or experimental evidence.

Concrete tensile strength is typically discounted when evaluating the flexural response under seismic loads, since extensive flexural cracking is expected; however, for unreinforced concrete, the tensile strength plays a more significant role in the flexural response. To determine how important tensile strength might be to the response of the piers, the evaluation was performed assuming both no tensile strength and full tensile strength, as specified in the code. Particularly with such large sections, neglecting tensile strength could be overly conservative since even a very small tensile stress can provide significant resistance; on the other hand, it is conceptually not recommended to have the adequate performance of a concrete section depend solely on the tensile resistance of concrete, particularly when considering the age of this concrete and the potential for deterioration over the years.

To aid in assessing the vulnerability of the piers in flexure, and because of the significant uncertainty when considering such large and varying sections with limited to no reinforcement and uncertain concrete properties, a static stability, or equilibrium, check was introduced as a simple way to guard against instability due to the peak forces at any section should it rupture fully.

The shear capacity of the piers is even more uncertain than the flexural capacity since there is no shear reinforcement and size effects are known to affect the shear response. To address this uncertainty, different methods to assess shear capacity were examined:

- CAN-CSA S6-14 [1] – Cl. 8.9.3.7: Based on Modified Compression Field Theory, not really applicable to unreinforced concrete;
- CAN-CSA A23.3-14 [4] – Cl. 22.6.6.1.2: Applies to unreinforced concrete, shear stress from elastic theory, not strictly applicable to dynamic loads or this size of section;
- Eurocode 2 [5] – Cl. 12.16.3: Based on principal stresses, independent of reinforcement, not strictly applicable to dynamic loads or this size of section;
- Principal stresses: First principles approach, calculating principal stresses at multiple points in the section assuming elastic theory, not strictly applicable to dynamic loads, doesn’t fully account for size effects.

Of all the methods, the principal stresses approach was preferred since it best accounted for the dimensions and unconventional geometry of the piers.

Piers of the Main Span (Section 7)

The main span (Section 7) is supported on four massive piers, numbered 23 to 26. The piers have some reinforcement but the quantity is too small relative to the concrete section to provide any reliable ductility; below ground, the foundations are unreinforced. As shown in Figures 4 and 5, each pier has a distinct set of characteristics that influences its seismic performance: pier 23 is an arch-type pier with tie-down anchors to hold down the south end of the main span truss, and wind shoes for lateral restraint; pier 24 is a wall-type pier with longitudinal and lateral restraint through the bearings and significant axial load from supporting one-half of the truss; pier 25 is similar to pier 24 but takes additional longitudinal forces from the suspended span which is attached to that side of the truss. Piers 23 and 24 are founded on shallow rock while pier 25 is founded on rock 20 m deep. Pier 26 is another arch with tie-downs, similar to pier 23, but is founded on soil rather than rock.

For piers 23, 25 and 26, flexure-shear strength is the most critical issue because of the lack of reinforcement provided in the concrete. Pier 24 fares better because of the high axial compression and lower longitudinal force than pier 25. Piers 23 and 26 also have potential sliding and overturning issues because the tension in the tie-downs reduces the axial compression at the base. The deep foundations of pier 25 are subject to possible shear and flexural failure. Piers 23, 25 and 26 clearly do not meet the performance objectives. Although pier 24 appears to meet the objectives, the lack of ductility and uncertainty with the shear resistance are issues that need to be addressed.

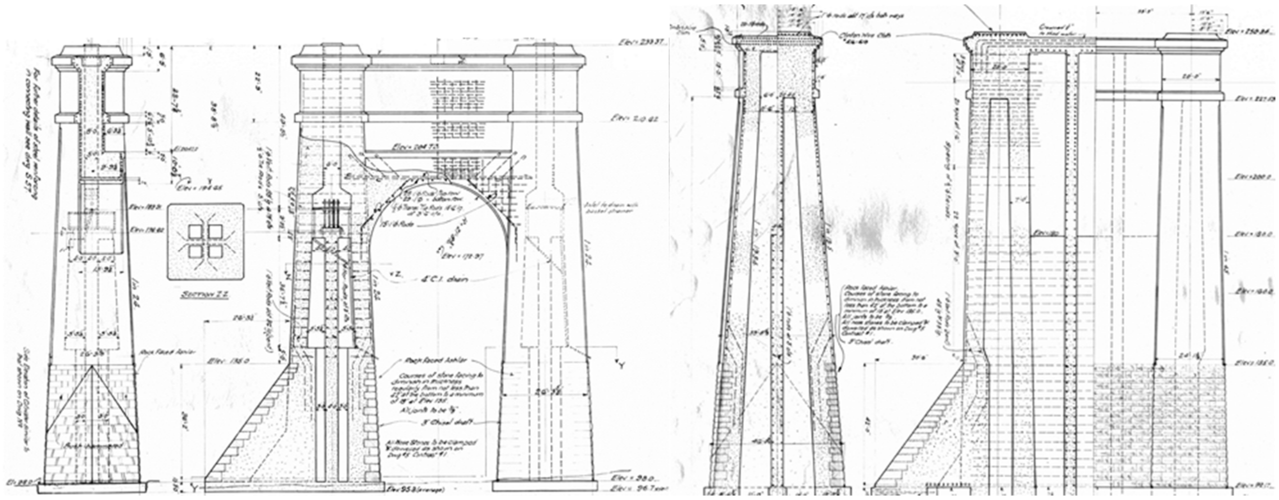


Figure 4. Piers 23 (left, 26 similar) and 24 (right, 25 similar) elevations showing hollow section.

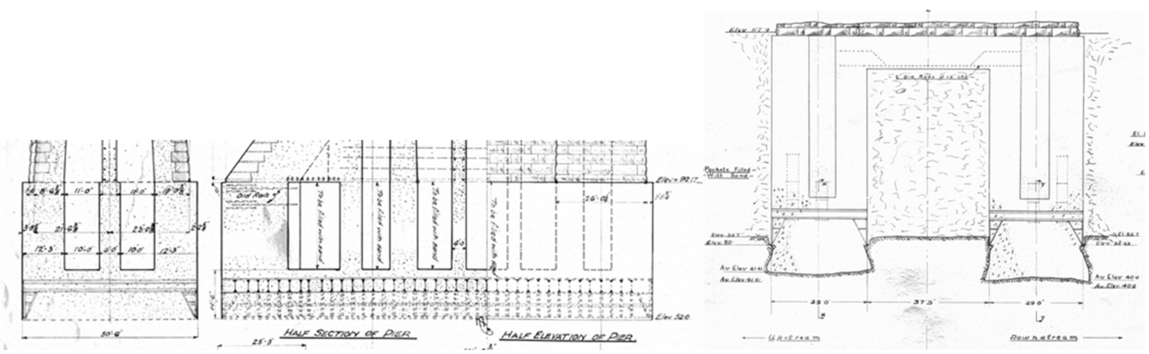


Figure 5. Foundations for Piers 24 (left) and 25 (right).

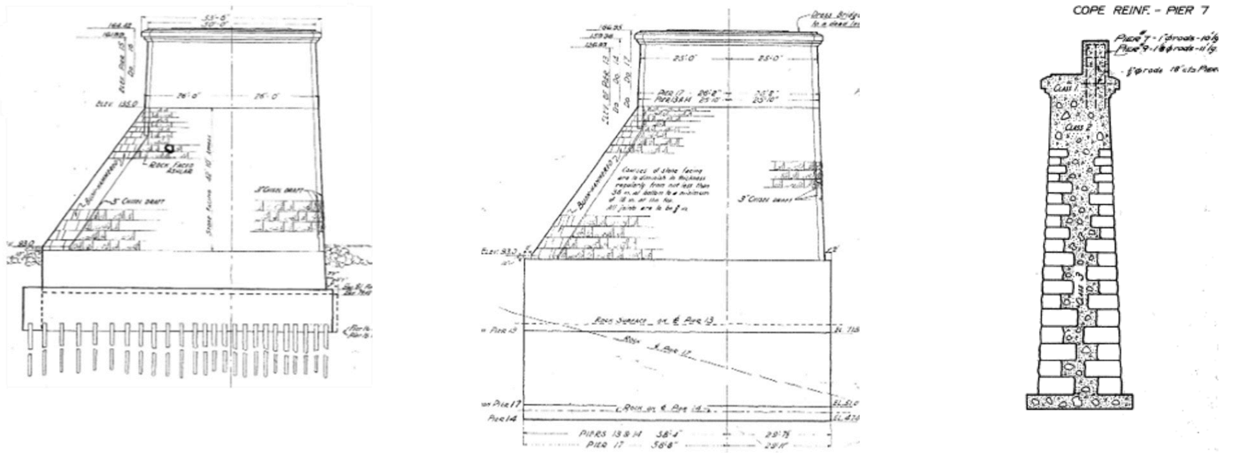
Piers of the South Approach (Sections 2, 3, 4, 6)

Piers on the south approach (Sections 2, 3, 4, 6) were built in two distinct sections (Fig. 6): the lower portion, with the tapered breakwater, consists of limestone masonry with a lean concrete infill while the upper portion consists of unreinforced concrete. Most of the south approach piers rest directly on bedrock; however, two piers (15 and 16) are located over a low spot in the rock profile and thus are supported on wooden piles.

Piers 1 to 13 were raised and enlarged for the construction of the St. Lawrence Seaway channel that passes between piers 9 and 10. To maintain traffic during the modifications, the superstructure was slowly jacked in four increments of 150 mm with concrete blocks inserted under the bearings after each increment (Fig. 7). After each 600 mm jacking step, the blocks were encased in a layer of concrete. This created a succession of unreinforced cold joints in the upper portion of the piers.

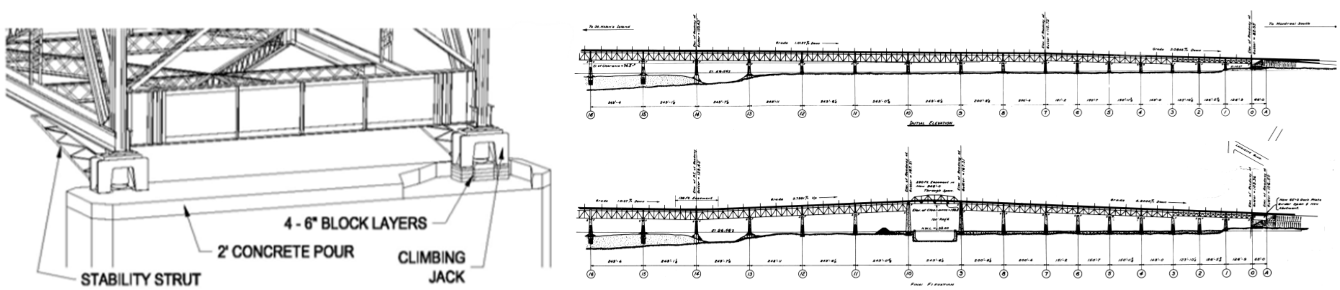
The south approach piers were analyzed individually using linear elastic response spectrum analysis. A point mass and linear spring are used to capture the influence of the superstructure on the response of the piers. The significant mass and high rigidity of the piers (natural period around 0.25 sec. transversally and 0.6 to 0.7 sec. longitudinally) means they have their own response separate from the response of the superstructure (natural period around 0.8 sec. transversally): the result is a two-degree-of-freedom system rather than the single-degree-of-freedom system typically representative of new bridges.

Unreinforced stone masonry is not covered in current seismic design codes, either for bridges or buildings, thus, an extensive literature review and rational assumptions were used to identify appropriate material properties and estimate the seismic performance of the masonry sections. One key assumption was to assign no tensile capacity to the interface between the mortar and the stone. When the eccentricity of the axial load caused by the bending moment is large enough to create traction at an extreme fiber of the section, cracking reduces the effective area and the inertia of the section. The stress distribution on the reduced pier section is analogous to the stress distribution under a rigid rectangular footing subject to a biaxial moment with uplift, which is a well-studied problem. For this evaluation we used an analytical solution developed by Bellos and Bakas [6]. The same approach was applied to the upper portion of piers 1 to 13, which have the numerous cold joints.



a) Pier 15 and 16 on timber piles *b) Pier 17 on bedrock* *c) Typical Cross section*

Figure 6. Details of typical south approach piers.



a) Details of construction method *b) Before/After elevation view of south approach*

Figure 7. Modifications to the south approach for construction of the Seaway channel.

The shear capacity of the stone masonry comes primarily from the friction that develops in the stone-mortar interface. A coefficient of friction μ of 0.6 to 0.8 is suggested by Chidiac and Foo [7] for stone block masonry structures. A value of 0.6 is consistent with the coefficient of friction in Clause 8.9.5.2.1(a) of CAN/CSA S6-14 for the calculation of interface shear for a hardened joint interface with no cohesion. Thus, a value of $\mu = 0.6$ was used to estimate the shear strength of the masonry sections.

As expected, the masonry sections fare poorly and significant damage is expected for all earthquake levels. Similar results were obtained for the concrete upper portion of the piers. Overall, the south approach piers, like the main span piers, do not meet the performance objectives.

SUPERSTRUCTURE

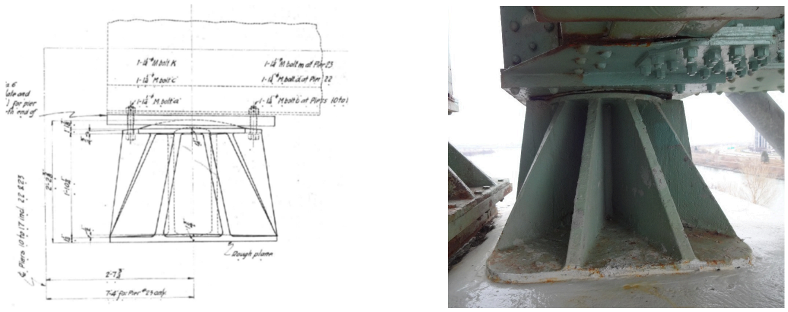
Steel Truss

The biggest challenge with evaluating the seismic performance of a 3.4 km long truss bridge is evaluating the performance of the very large number of individual truss members. Luckily, a number of live-load capacity studies had been performed in recent years. This enabled us to make a rapid first screening by comparing the seismic loads to the live loads and to the member capacities calculated for those studies. Additional seismic-specific checks were required, for example, for members where stresses reverse under seismic loads but not under live loads, but these checks only needed to be performed on a relatively small number of members.

In general, members that were overstressed under live loads were also overstressed under seismic loads. In the main span, the main truss chords fared well; however, some main diagonal members were found to be overstressed due to out-of-plane bending (a seismic-only load case). On the approach spans, the main truss members are expected to perform adequately for all return period earthquakes. Some lateral and sway bracing members are expected to buckle under compression, but when accounting for redistribution to the tension counterpart of the bracings, these are expected to perform acceptably, with damage that exceeds the criteria occurring in only a few of the members. Overall, the steel trusses are expected to perform well: they do not quite meet the performance objectives, but are close, with the number of deficient members increasing with increasing earthquake level.

Bearings

The bridge currently has quite a collection of different bearings, including original spherical bearings, original roller bearings, replacement lubrite sliding bearings that have exceeded their intended service life and are now largely seized, and newer elastomeric and pot bearings. The spherical bearings (Fig. 8) are the most concerning for the seismic performance of the structure: they have no tie-downs, anchors or guides to resist lateral or uplift forces; they rely on the vertical reaction to provide lateral resistance. High lateral forces from seismic effects, combined with reduced axial load when taking into consideration vertical accelerations, can cause unseating of the spherical articulation. Given the height of these bearings, this would lead to a significant drop for the superstructure truss, which could lead to overstressing and instability of truss members. Also, some piers do not have sufficient surface area around the bearing to reliably catch the superstructure. Unseating of the spherical bearings is expected at the fixed end of the spans for all return period earthquakes.



a) Details from shop drawings *b) Photo of actual bearing*

Figure 8. – Spherical bearings on Jacques-Cartier bridge.

PAVILION

Mid-way along the bridge, the pavilion (Fig. 9) is a 3-story Art Deco building which supports the deck and two ramps providing access to and from Île Sainte-Hélène. Not only is the building style a rarity in Montréal, but the building-as-a-bridge-pier concept is pretty unique. This required the building to be evaluated as a bridge component, using the bridge code and the same performance objectives as for other piers, and as a building in accordance with the National Building Code [8]. The latter requires the building to provide life-safety performance for the 2475-year return period ground motions.

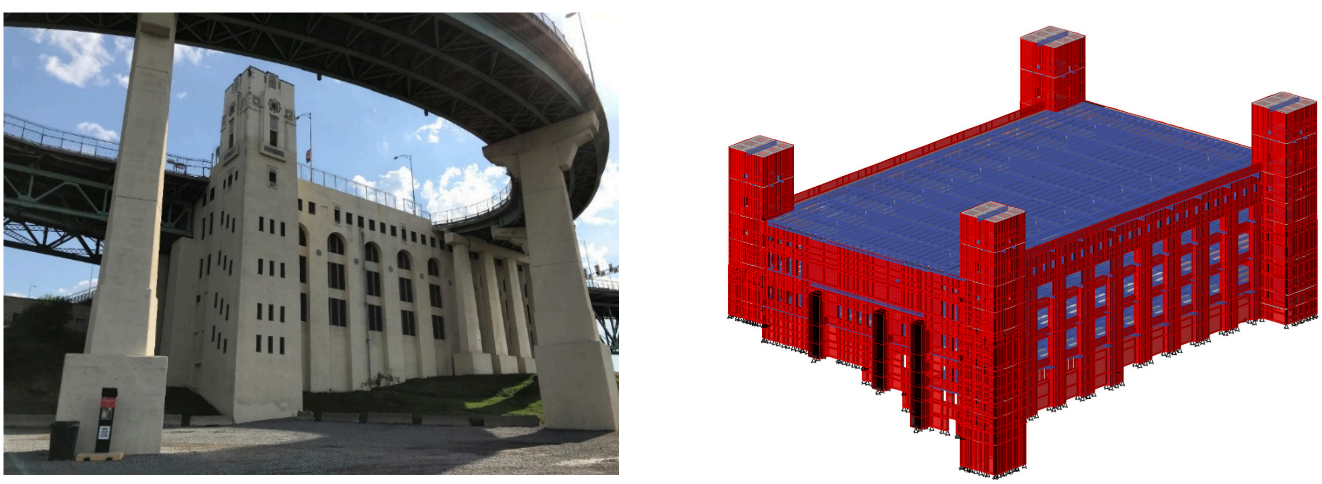


Figure 9. – Pavilion (a) Photo and (b) 3D model.

The building has few interior columns in order to create a large event space and uses the perimeter walls to carry some of the vertical load from the bridge. All resistance to lateral loads is provided by the perimeter walls. The building was fully modelled in 3D (Fig. 9) and was analyzed for the three earthquake levels using linear elastic (response spectrum) analysis.

The building meets the requirements of the building code; however, it does not meet the performance objectives of the bridge code. The main issue is that the perimeter walls do not have sufficient reinforcement, particularly around the numerous openings (where stress concentrations occur), to resist tension stresses that develop. Some zones are also overstressed in compression. For all earthquake levels, the expected damage exceeds the criteria.

CONCLUSIONS

The seismic performance of the Jacques-Cartier Bridge was evaluated in accordance with the performance-based requirements of CAN/CSA S6-14. The structure was evaluated for earthquake ground motions with return periods of 475 years, 975 years and 2475 years and the expected performance was compared to the performance objectives for a new Lifeline bridge (475 yr return period: immediate service/no damage; 975 yr return period: immediate service/minimal damage; 2475 yr return period: limited service/repairable damage).

Since the bridge has essentially no reliable ductility, linear-elastic analysis (response spectrum) was the primary tool used to evaluate seismic demands. The judicious use of simplified and partial models enabled the evaluation of this 3.4 km long structure with multiple structural systems to be completed in an effective and timely manner.

The main conclusion is that the concrete piers do not have satisfactory behaviour under any of the earthquake ground motions. The Section 7 piers are mainly vulnerable in flexure and shear, which is understandable given the concrete is significantly under-reinforced or completely unreinforced. Along the south approach, the stone masonry and the pier extensions with cold joints and unreinforced concrete are particularly vulnerable to weak axis bending causing decompression and stability issues.

In the end, the two key uncertainties – global damping ratio and concrete tensile capacity – are not critical to the evaluation conclusions: in particular, the concrete piers have deficiencies that need to be addressed regardless of the assumptions made. The assumptions do influence the level of damage in some elements, some of the stability issues in the piers, and in particular the number of truss members that fail; thus, they do need to be considered carefully for the design of the seismic retrofit.

Not unexpectedly, the bridge in its present state does not meet the performance objectives for a new Lifeline bridge that were the target for this evaluation; however, that was only meant to establish a baseline against which deficiencies in the expected performance can be measured. The next step is to define performance objectives for the retrofitted bridge, taking into account factors such as cost, public safety, emergency response, and remaining life of the structure.

ACKNOWLEDGEMENTS

Golder Associés Ltée (Montréal) provided geotechnical characterization of the soils and estimates of bearing capacities and lateral resistances and evaluated the potential for liquefaction around the foundations.

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