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Performance based Seismic Evaluation of the J.C Van Horne Bridge

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ABSTRACT:

The J.C. Van Horne Bridge represents a major transportation link between Quebec and New Brunswick over the Restigouche River. It was constructed between 1958 and 1961 to replace the pre-existing ferry service. It serves as a vital interprovincial link for the communities of Campbellton, NB and Listuguj/Point-à-la Croix, QC in the eastern Canada.

The structure consists of 13 spans including cantilevered and suspended trusses, south approach and north approach deck-truss spans. The 805.5 m long bridge comprises of the 419.1 m long cantilevered and suspended trusses (Main Truss) and the 193.2 m long deck trusses each for the south and north approaches. The travelled roadway over the bridge deck is approximately 8.00 m wide and carries one lane in each direction. The piers consist of a concrete stem wall supported on a large concrete caisson-type footing set below the riverbed. The concrete caisson-type footings are 9 m to 11 m in depth through soft sediment and placed into a layer of dense sands and gravels. At the ends of the approach deck-trusses there are short end spans, 1X and 10X, tapered towards the abutments.

The seismic evaluation was performed for two hazard levels, 475-year and 975-year earthquakes, in accordance with the current Canadian Highway Bridge Design Code [1]. Both hazard levels are based on the 6th generation seismic hazard maps of 2020 National Building Code of Canada (NBCC 2020). The bridge is classified as a Major route bridge. Two seismic performance objectives were defined for this evaluation including (a) limited service/repairable damage for the 475-year hazard and (b) life safety/probable replacement for the 975-year hazard.

The seismic evaluation indicated that the deformation of all piers will remain essentially elastic and no plastic hinging of the pier walls is expected to occur during either the 475-year or 975-year events. The truss members of the main truss and approaches were found to have adequate capacity to resist the seismic loads. Expansion joints located between the main truss and the approaches do not have sufficient gap to accommodate the anticipated seismic movements. Pounding is expected to occur among the truss segments. The anchors at the existing rocker and fixed bearings were found deficient in shear/tension and as a result, needed to be retrofitted or replaced to transfer the seismic loads to the substructure components.

Keywords: Truss Bridge, Seismic Evaluation, Performance Based Design, lightly reinforced pier walls.

STRUCTURE DESCRIPTION

The JC Van Horne Bridge, which is a cantilevered steel truss bridge, spanning the Restigouche River between Quebec and New Brunswick, was constructed between 1958 and 1961 to replace the pre-existing ferry service. It serves as a vital interprovincial link for the communities of Campbellton, NB and Listuguj/Point-à-la Croix, QC in the eastern Canada.

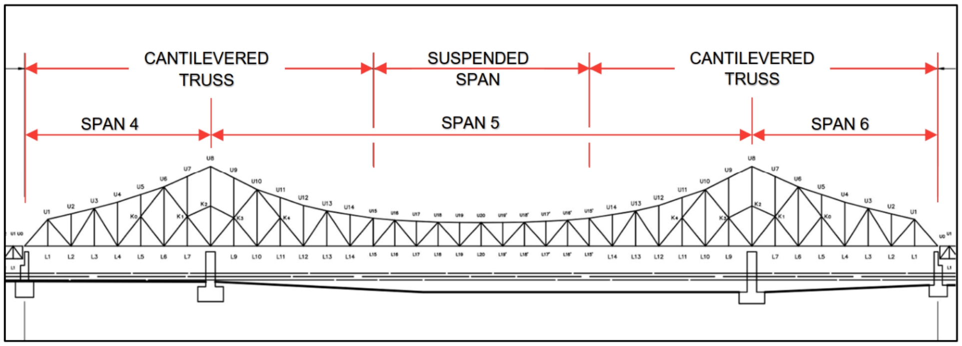

Figure 1. JC Van Horne Main Bridge

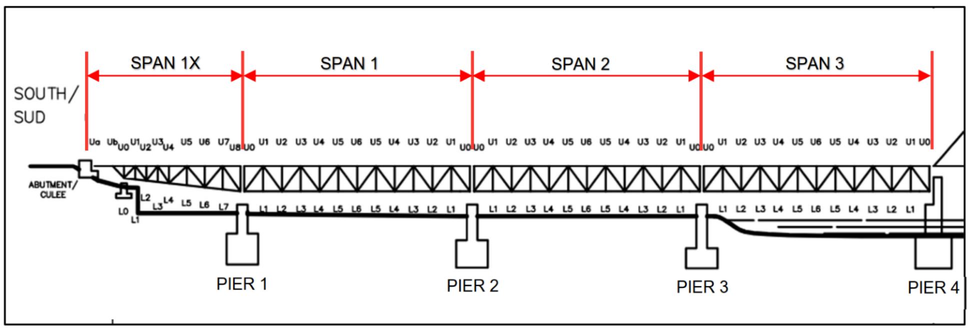

Figure 2. JC Van Horne Bridge South Approach Spans (North Approach Spans Similar)

The JC Van Horne Bridge is composed of 13 spans, with an overall length of approximately 805.5 m, which is made of the 419.1 m cantilevered and suspended trusses (Figure 1 - Spans 4 to 6), the 193.2 m south approach (Figure 2 - Spans 1X & 1 to 3) and the 193.2 m north approach spans (Spans 7 to 10 & 10X). Each cantilevered truss is approximately 160 m long, the central suspended span is approximately 99.1 m long, and the rise of the truss at the supporting towers is about 36.6 m. The width of the truss is around 12.2 m, and the deck over the Main Truss is made of a steel grating filled with lightweight concrete, paved with asphalt, and with lightweight concrete sidewalks.

The members of the Main Truss, including the top chord, bottom chord, verticals, and diagonals, are built-up steel members assembled by rivets, while the secondary members, including the top and bottom lateral bracing, portal, and sway frame, are lattice-type steel members. The Main Truss members use perforated plate elements and are connected at the joints with steel gusset plates by bolts. At Piers 4 and 7 the Main Truss is tied to the substructure via articulated steel plate anchors embedded into the piers, and wind shoes are provided to restrain the truss laterally. At Piers 5 and 6, the Main Truss is supported on pin bearings.

The approach spans consist of steel deck trusses, with the end spans (1X and 10X) tapered towards the abutments. The approach spans are simply supported on rocker and pin bearings, having span lengths varying between 52.7m and 53.2m, and the tapered end span is also simply supported with a span length of 27.3 m. The approach structures have a reinforced lightweight concrete deck, with the roadway deck paved with asphalt overlay.

The main truss and approach span piers all consist of a concrete stem wall supported on a deep concrete footing, penetrating 9 m to 11 m and 4 m to 8m, respectively, through soft sediment into a layer of dense sands and gravels.

GEOTECHNICAL

The subsurface conditions, in general, consist of fill or loose silts which are underlain by a compact to dense deposits of sands and gravels with some boulders over bedrock. The average V_{s30} for the piers 6 to 10 on the QC side is about 325 m/s. A Site Class D designation is considered appropriate for the site. The seismic ground spectra for Site Class D were generated using the 6th generation seismic hazard maps as recently published in 2020 NBCC and adjusted using the Cl. 4.4.3.3 site coefficients (Figure 3).

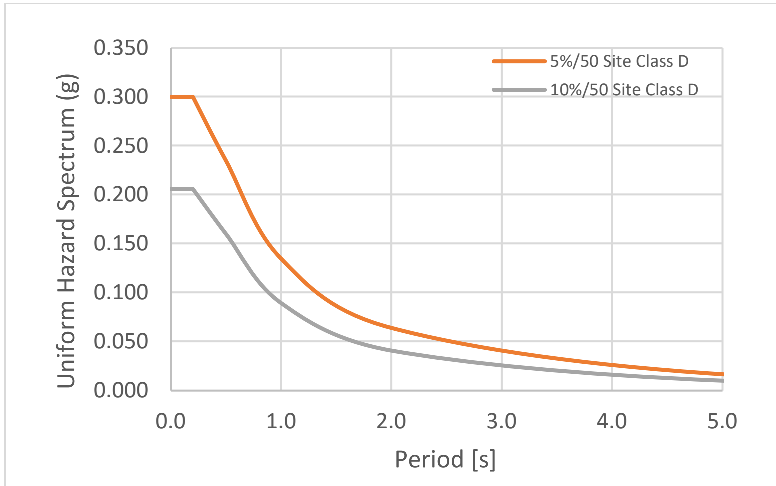

Figure 3. Uniform Hazard Spectra for Site Class D at the Bridge Site for the Two Hazard Levels of 5%/50 and 10%/50

The result of the liquefaction assessment indicates that the very loose to loose silt deposits (that are located below the riverbed and above the sand and gravel deposit) are potentially liquefiable during the 975-year return period earthquake (i.e., probability of 5% in 50 years). Therefore, additional downdrag loads were considered for all the pier footings for the seismic evaluation. Investigation regarding potential for lateral spreading was not part of this project.

SEISMIC DESIGN APPROACH AND CRITERIA

The J.C. Van Horne Bridge is classified as a major-route bridge. The seismic evaluation of the structure was carried out using a performance-based approach in accordance with CHBDC S6-19.

In consideration of the remaining service life of the bridge, a risk-based approach was used to select appropriate seismic hazard levels. Consequently, two lower hazard levels: 5% and 10% probabilities of exceedance in 50 years, corresponding to return periods of 975 and 475 years, respectively, were selected for the seismic evaluation. The seismic performance objectives for the selected seismic hazard levels are: (a) limited service/repairable damage for the 475-year hazard level and (b) life safety/probable replacement for the 975-year hazard level.

For the older highway bridges such as the J.C. Van Horne Bridge where substructure components do not meet the CHBDC seismic detailing requirements and do not have the required ductility, concrete and steel strain limits must be adjusted to account for the earlier onset of capacity degradation of the substructure components. Hence, the concrete strain limit for pier walls with base lap-splices to remain essentially elastic with minimum damage was reduced from 0.006 to 0.002.

SEISMIC PERFORMANCE CATEGORY

Based on the fundamental periods of vibration for the Main Truss of 0.97 s and 2.44 s, respectively, in the longitudinal and transverse direction and the bridge importance category, it was determined that Seismic Performance Category 2 shall be used for the seismic evaluation.

ANALYSIS REQUIREMENTS

As per CHBDC S6-19 Table 4.12, an EDA and ISPA analyses shall be carried out for the 5% in 50 years earthquake. An EDA analysis shall be carried out for the 10% in 50 years earthquake.

Given the complexity of the structure, the appropriate EDA analysis for this bridge was a multi-modal elastic response spectral analysis. This analysis was used to obtain the seismic force and displacement demands in the Main Truss members, bearings, substructure, and foundation elements. An ISPA was performed for pier 5 where the inertia demand causes the pier wall to reach to the yield moment in the longitudinal direction.

SUPPORT ARTICULATIONS

The Main Truss is anchored to Piers 4 and 7 by a pendulum-type anchorage system which prevent vertical uplift but still allow for longitudinal translation and rotation. Transversely, the Main Truss is further restrained by a wind shoe (steel shear key) anchored into concrete pedestals on the top pf Piers 4 and 7, respectively. The pinned bearings at Piers 5 and 6 restrain both longitudinal and transverse movements of the Main Truss. The suspended span is pinned to the cantilevered truss at the south end and free to move in the longitudinal direction at the north end. At the north end, the suspended truss is picked up by a vertical hanger pinned to the top chord of the cantilever truss.

At the approach spans, each of deck truss spans is simply supported. Steel pin bearings are provided at one end and steel rocker bearings at another end.

EFFECTIVE SECTION PROPERTIES

Pier Walls

The effective stiffness of stem section of pier walls (from top of the piers to top of the footings) in the weak direction was calculated as columns. The lower portion of the stem walls was encased by a steel jacketing with infill concrete for Piers 2, 3, 4, 6, 8 and 9 extending from top of footing to 0.762 m above the mean water level (Figure 4). The lower portion of Piers 5, 7 and 10 extending from top of footing to 0.8 m above mean water level was also encased during 2005 repairs (Figure 5). The thickness of the steel jacket is 12.7mm.

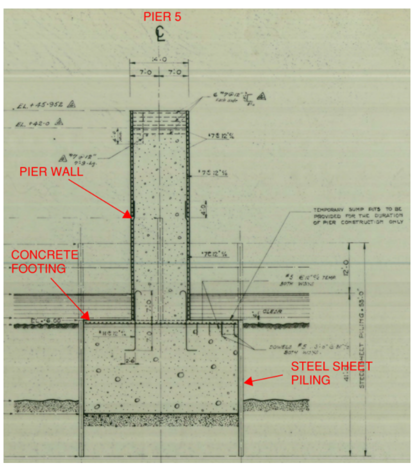

Figure 4. Typical Pier (Pier 5 shown), Ref. existing drawings

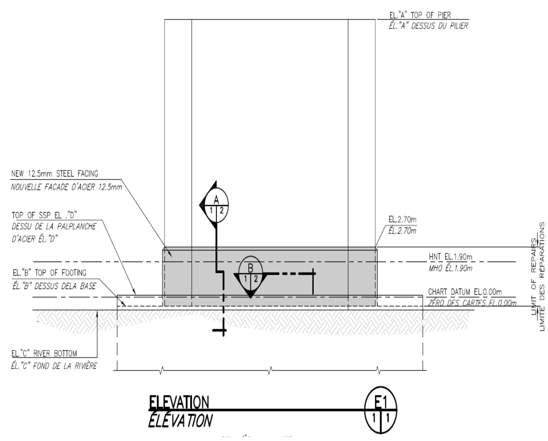

Figure 5. Typical Pier Repaired (2005)

The steel jackets were not considered in the capacity calculations, as indicated by Priestley et al. [4] that the steel jackets for rectangular sections will provide little restraint against the lateral dilation of the core as the jacket will deform and release confinement. Confinement for enhanced compression strain or improved lap-splice performance is unlikely to be effective.

The behaviour of the pier walls in the strong direction is different from the weak direction and is dominated by the shear deformation rather than the flexural deformation. The stiffness of the pier walls in the strong direction was based on uncracked section properties.

Footings

The pier footings are essentially non-ductile concrete elements encased by sheet piling. The effect of sheet piling was ignored from the footing section properties.

CSiBRIDGE MODEL

A 3D Finite Element (FE) model of the Main Truss and the Approach Spans (Figure 6) was created using CSiBridge v23.3.1. The bridge was modeled based on the configuration and member geometry obtained from the original as-built drawings. The truss members were modelled using frame elements with the appropriate releases of bending connectivity at each end of the frame elements.

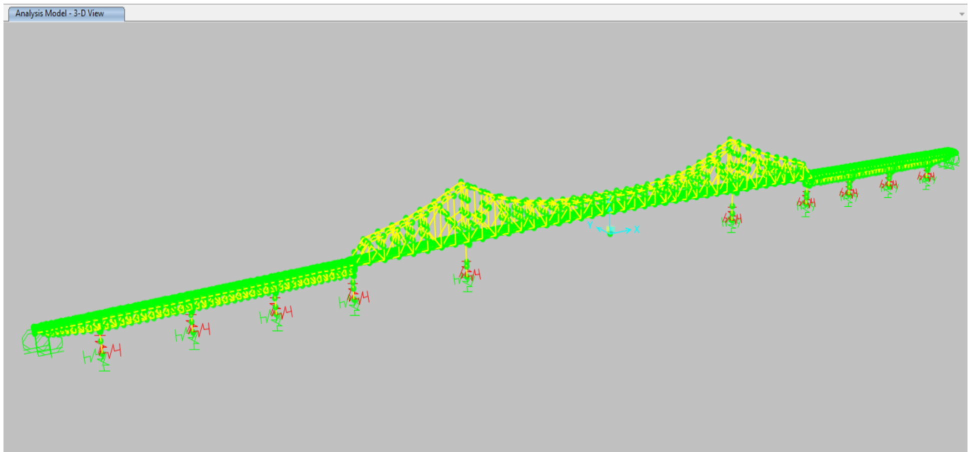

Figure 6. CSiBridge Model of the JC Van Horne Bridge

The bearings were modeled using linear springs located between the bottom chords and the top of the piers. The suspended span is connected to the two cantilever trusses at axes 15 and 15' by a pinned and moveable connection, respectively. For the Main Truss, the concrete deck is supported on a floor beam connected to the bottom chord of the truss. Longitudinal stringers span between the floor beams, and the top of the deck and stringers are rigidly connected to eliminate any differential movement between the centroid of the composite member (stringer and deck) and the floor beam.

There are expansion joints in the Main Truss spans and the approach spans, and between the Main Truss spans and the approach spans. The opening and closing of expansion joints introduce discontinuities that affect the load path and hence the dynamic response of bridges. Such effects were properly addressed in the FE models using a bounded approach as per in Caltrans Bridge Design Practice Manual[3], that is, the nonlinear response of the expansion joints is bounded by two linear models (tension and compression model). The tension model is used to capture out-of-phase motions and the compression model is used to capture in-phase motions of structures adjacent to expansion joints.

For the JCVH bridge, a total of five models were utilized to capture the nonlinear responses of the bridge superstructure due to expansion joints using tension and compression models and to account for the worst scenarios of foundation soil-structural interactions using the upper bound and lower bound soil-structure interaction foundation springs. The key attributes of those models are described below:

Model 1: tension model for all expansion joints and lower bound foundations springs for all piers.

Model 2: compression model for all expansion joints except at Axis 5B where the finger expansion joint is permitted to move freely in the longitudinal direction, and lower bound foundation springs for all piers.

Model 3: tension model for all expansion joints and upper bound foundation springs for all piers.

Model 4: compression model for all expansion joints except at Axis 5B where the finger expansion joint is permitted to move freely in the longitudinal direction, and upper bound foundation springs for all piers.

Model 5: gross section properties for all pier walls in longitudinal and transverse directions, and rigid foundations for all piers. The purpose of this model is to capture the maximum inertia force effects on the primary truss members.

PIER WALL ASSESSMENT

The pier walls consist of a stem wall connected to a thick concrete spread footing via vertical reinforcement dowels (Figure 7). The vertical dowels consist of # 11 bars that are embedded 2,133 mm in the pier wall and footing with hooks at each end. The lap-splice length of the dowel generally meets the full development length of # 11 bars. However, lap splices are located in the potential plastic hinge regions which are no longer permitted by CSA S6-19.

Lap splices in such regions will likely concentrate inelastic deformations near the base and then potentially shorten the plastic hinge length. The pier walls are lightly reinforced in both directions with no lateral ties confining the concrete core at the potential plastic hinge region in the weak direction. Therefore, the pier walls are not considered as a ductile substructure component.

All pier footings were encased by steel sheet piling. In 1977 the pier walls at Piers 2, 3, 4, 6, and 8 were encased by 12.5mm thick steel jackets and 165mm thick of reinforced concrete infill (Figure 8). In 2005 a similar repair was undertaken for Piers 5, 7, and 10. The complex geometry of all pier walls was captured in the CSiBridge to assess the seismic performance of the pier walls.

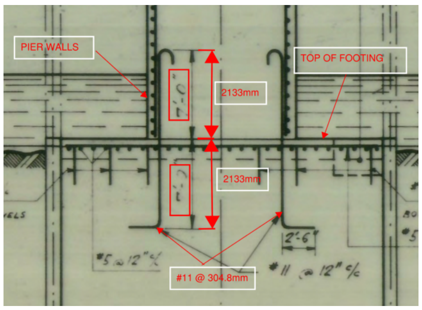

Figure 7. Typical Dowel Connection of Pier Walls, ref. existing drawings

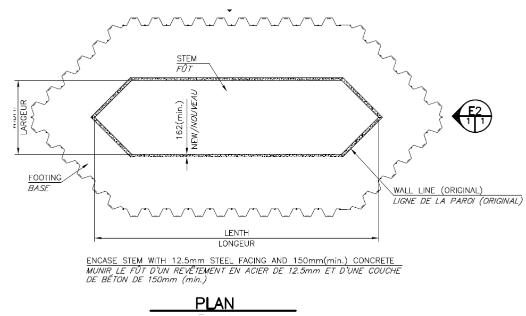

Figure 8. Typical Repairs of Pier Walls

The moment curvature diagram was utilized to determine the flexural resistance of the pier walls and the resulting steel and concrete strains, while accounting for the applied axial load. For this assessment, the strength contribution of the steel jackets and infill concrete in the repaired sections of the pier walls were not accounted for. This assumption will reduce the moment resistance of the pier walls but increase the concrete and steel strains. Table 1 summarizes the maximum concrete and steel strains obtained for Pier 5 and 6.

Table 1. Concrete and Steel Strains at 5%/50 Performance Level

	STRAIN AT 5%/50 PERFORMANCE LEVEL			
PIER NO	**CONCRETE STRAIN**	**CONCRETE STRAIN LIMIT**	**STEEL STRAIN**	**STEEL STRAIN LIMIT**
Pier 5 & 6	0.00115	0.002	0.0168	0.075

The shear resistance of the pier walls sections in the longitudinal direction was calculated using the general method and the initial and final shear resistance calculated in accordance with the FHWA Seismic Retrofitting Manual for Highway Structures [2]. The shear resistance of the pier walls was found adequate.

FOUNDATIONS ASSESSMENT

The stability of the footings at Piers 1 to 10 was evaluated for overturning of the footings and for the maximum bearing pressure under footing using the uniform pressure distribution.

In accordance with CSA S6-19 for seismic load cases, both geotechnical resistance and consequence factors were assumed to be unity when calculating the bearing capacity of the footings. The sheet piling surrounding the concrete footings was ignored. The maximum bearing pressure under the pier footings was calculated using a uniform pressure distribution. The reduced width and length were used to determine the equivalent footing area, taking into account the eccentrical loading in both directions as per CSA S6-19.

The pier footings were found to have adequate bearing capacity and overturning resistance for the lateral seismic loads.

SEAT LENGTH ASSESSMENT

The required and provided seat lengths were calculated in accordance with CSA S6-19 for a Seismic Performance Category 2. The available seat length was found to be adequate at abutments and piers.

BEARING ASSESSMENT

Pendulum Anchorage at Pier 4 and 7: The cantilever truss span that supports the suspended truss span between joints U15 and U15' is connected to Pier 4 using a pendulum-type anchorage system (Figure 9). This system prevents vertical uplift while allowing for longitudinal rotation and movement of the cantilever truss at the support. The gusset plates, anchor plates, and pins were evaluated as per CSA S6-19. The top and bottom pins and anchor bolt connections were also assessed and were deemed capable of resisting the axial uplift force in the pendulum at both hazard levels.

Figure 9. Pendulum anchorage at Pier 4 & 7

Figure 10. Wind shoe at Pier 4 & 7

Wind shoe at Pier 4 and 7: The cantilever truss is laterally restrained by a steel wind shoe anchored into concrete pedestals at the top of Pier 4 and 7 (Figure 10). The wind shoe was evaluated for shear demands from the 100% transverse earthquake and checked for the interaction between shear and tension. The shear demands were increased by a factor of 1.25, as required by CSA S6-19. The connection between the steel bracket and the floor beam was checked for the ultimate shear resistance of the bolts and found to be sufficient to withstand the shear demands under both hazard levels.

Main Shoe at Pier 5 and 6: Lateral fixities for the truss superstructure spanning from Span 4 to 6 are achieved by the pin bearings at Pier 5 and 6 (Figure 11). These bearings provide the required longitudinal and transverse restraints for translational movements while allowing for longitudinal rotation. A steel shoe, known as the pin stand, supports the 254mm diameter pin with a 305mm x 63mm pin head, and vertically stiffened plates welded to a base plate anchored into the top of the pier wall using eight 50.8mm diameter anchor rods, with an average embedment length of 914mm. The sole plate, similar in construction to the base plate, bears on the pin and is anchored to the bottom chord of the truss. In the transverse direction, the resistance of the bearing shoe is achieved by the pin bearing against the pin stand, with the pin cap providing resistance against the pin stand plate. The seismic lateral force could cause the pin cap to fail, potentially resulting in loss of truss supports at Pier 5 and 6. Although the pin has enough capacity to withstand the transverse seismic load, it is recommended to strengthen the pinned bearings to ensure reliable load paths to the substructure. In addition, the bearing anchor rods do not have sufficient shear resistance to resist the seismic loads under the 5%/50 hazard level.

Rocker and Fixed Pin Bearings at Approaches: The approach truss spans are supported by a fixed pin bearing at one end and a rocker bearing (Figure 12) at the other end. The base plate at the rocker and fixed bearings is attached to the piers with four anchor bolts, which are 32mm in diameter with an embedment of 711mm. The rocker bearings rockers pins and anchors at the base plate were assessed according to CSA S9-19. The anchor bolts are found to be inadequate to withstand the shear demand at the base plate of the fixed bearings. Stability of rocker bearings was evaluated based on FHWA Seismic Retrofitting Manual for Bridges[2], and it is found that while the longitudinal displacement capacity of the rocker bearings is sufficient, steel rocker bearings are highly susceptible to damage during an earthquake. This rocker bearing is considered the most vulnerable to large seismic events due to its high aspect ratio, difficulty to restrain, and potential for instability and overturning after experiencing only limited movement. Therefore, it is recommended to replace or stabilize all the rocker bearings at the approaches.

Figure 11. Pin Bearing at Pier 5 & 6

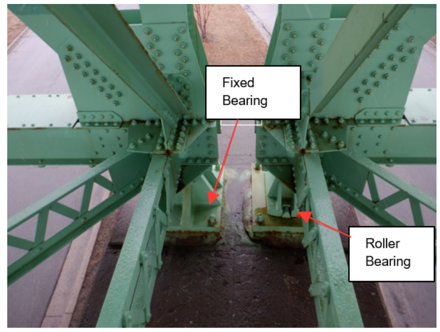

Figure 12. Rocker Bearings at Approach Span

MAIN TRUSS AND APPROACH TRUSS MEMBER ASSESSMENT

The structural resistances of all truss members were calculated in accordance with Section 10 of CSA S6-19. The tensile resistances of the primary truss members were assessed based on the gross sectional area and the net sectional area as appropriate.

The unbraced lengths of the compression members were determined from centroid to centroid of riveted or bolted connections. The compression resistance of the primary truss members was assessed based on the member section classes to ensure they meet the requirements of a Class 3 section. However, it was found that some of elements (angles/channels and plates) of the built-up members forming a closed section are class 4 sections. For class 4 sections, an effective area was calculated using reduced element widths meeting the maximum width-to-thickness ratios indicated in Table 10.3 of CSA S6-19. This effective area was then used to calculate the compressive resistance of the Class 4 members.

The capacity of the lattice was also assessed. The lattice elements are required to resist shear normal to the longitudinal axis of the lattice, equal to 2.5% of the axial compression in the member.

It was found that all truss members of the Main Truss and approaches have sufficient capacity to withstand the seismic loads, and no buckling of primary members is expected to occur.

EXPANSION JOINT ASSESSMENT

As per CSA S6-19, the seismic design displacement (d_s) is taken as 1.25 times the seismic displacement in the longitudinal direction. The total design displacement (d_t) is the total displacement across the expansion joint caused by the seismic and thermal displacements. For the 5%/50 hazard level, 100% of the thermal displacements were combined with the factored seismic displacement. The seismic displacement at the expansion joints was obtained from the relative displacements of the truss spans at either side of the expansion joint. The maximum joint displacement would occur when the two adjacent truss spans were moving completely out-of-phase. Based on the results of this evaluation, the maximum opening capacities of the strip seal expansion joints at Pier 1 to 4 and 7 to 10 were not sufficient to accommodate the total displacement demand of the 5%/50 seismic displacement plus 100% of the thermal displacement. Consequently, some degree of pounding between truss spans at the concrete deck level is expected.

CONCLUSIONS

The seismic evaluation results indicate that all piers will remain essentially elastic, and no plastic hinging of the pier walls would be expected during the 5%/50 and 10%/50 events. The strains in the concrete and steel reinforcement of the pier walls remain below the strain limits prescribed for the life safety and limited-service performance levels. The main truss and approach truss members have adequate capacity to withstand the seismic loads. While the finger joint between the suspended span and the north cantilever truss has the adequate joint movement capacity to accommodate the total predicted thermal and seismic movement, the strip seal expansion joints elsewhere do not have the adequate joint movement capacity. Some degree of pounding would likely occur between the cantilevered trusses and approach spans, as well as between the approach truss spans. The anchors of the fixed and rocker bearings at the approach truss spans are anticipated to fail in shear at both hazard levels and could cause excessive displacement of the truss segments. The shear capacity of the anchors at the fixed bearing (Pier 5 and 6) is insufficient under the 5%/50 hazard level and it will require the strengthening of the fixed bearings to ensure reliable load paths to transfer of lateral forces from the truss superstructure to the substructure. The pin caps of the fixed pin bearings at Piers 5 and 6 are vulnerable to failure, particularly considering that the transverse seismic load may not distribute equally between the fixed pin bearings due to slight variation in gap clearance at the end of the pin. As a result, the pin caps shall be strengthened to ensure equal seismic participation by the two fixed pin bearings.

To conclude, the J.C. Van Horne bridge, including its superstructure, substructure and foundations, is expected to meet the CSA S6-19 performance-based design requirements except for the rocker and fixed bearings, which will require reinforcement or replacement.

REFERENCES

[1] Canadian Highway Bridge Design Code, CAN/CSA S6-19, 2019.
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