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Seismic Retrofit of a 900 m Long Twin Bridge using Two Types of Isolation Devices

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Abstract: This paper examines the seismic rehabilitation of a twin bridge on Highway 10 crossing the Richelieu River near Montreal (Canada). Each viaduct consists of 19 simply-supported spans and a three-span continuous section, for a total length of 895 m. It was constructed in 1963 with separate deck for each direction. The piers are supported by shallow foundations, some common for both viaducts at certain locations. The bridge was evaluated and its seismic resistance was found to be inadequate, not even being able to sustain a moderate earthquake. The structural deficiencies observed were mainly non-ductile columns and seismically deficient footings. Based on the evaluation, it was decided to perform a seismic retrofit. Taking advantage of the separated decks, two different performance levels were chosen by the client: Importance Category "Other" for the South Viaduct and "Lifeline" for the North Viaduct. Also, two types of isolators were used: elastomeric bearings for the South Viaduct while friction pendulum bearings were employed on the North Viaduct. The analyses demonstrated that the isolation significantly reduces the forces transferred to the foundations and hence no additional retrofit is necessary for the piers and foundations. This presentation will also present the results of different analysis techniques: from simplified analysis using target characteristics of the isolation system, to non-linear time-history analysis with characteristics determined by large-scale testing. The presentation highlights the differences between the analysis techniques as well as those between both isolation systems. The detailing, durability and the design strategy for the common foundations are discussed as well.

1 Introduction

The seismic isolation controls the movement of the structure under earthquake, minimizes the seismic forces and provides a post-seismic functionality according to the needs. Among the techniques for seismic isolation, the deck separation from the foundation using isolation bearings is particularly effective. It lengthens the natural period, increases the damping and one of its main advantages is that it applies very well to the seismic rehabilitation of existing bridges.

The basic concept behind seismic isolation is to uncouple the deck movement from those of the piers and the ground. In practice, flexible bearings are used to increase the vibration period of the bridge in order to reduce the design spectral acceleration. The isolation can reduce the elastic forces by 75%, which translates into direct savings in the construction costs of new bridges. Moreover, in the case of a major earthquake, an isolated bridge performs better than one with a conventional seismic design with plastic hinges. In the first case, the piers are designed to remain elastic whereas for the conventional design, the structure dissipates energy through localised damages, generally at the piers' base. For existing bridges, the deck isolation can improve the seismic performance up to a certain level without any strengthening of the foundations units.

2 Case study: Twin Bridge on Highway 10 crossing the Richelieu River

Both viaducts have 22 spans and two hollow abutments for a total length of 895 meters. They cross the Richelieu River, the Chambly Canal and two major roadways. The twin bridge was built in 1963 and has two types of decks. First, 19 simply supported spans consisting of four prestressed concrete girders with a length of 37.9 meters supporting a 200 mm thick concrete slab. Also, three continuous spans consisting of two steel box girders supporting the same 200 mm thick concrete slab cross the Chambly Canal. The box girders have a central span of 56.4 meters and two end spans of 45.5 meters. An elevation view of the bridge and a cross section of the twin bridge with both types of decks are shown on Figures 1 and 2.

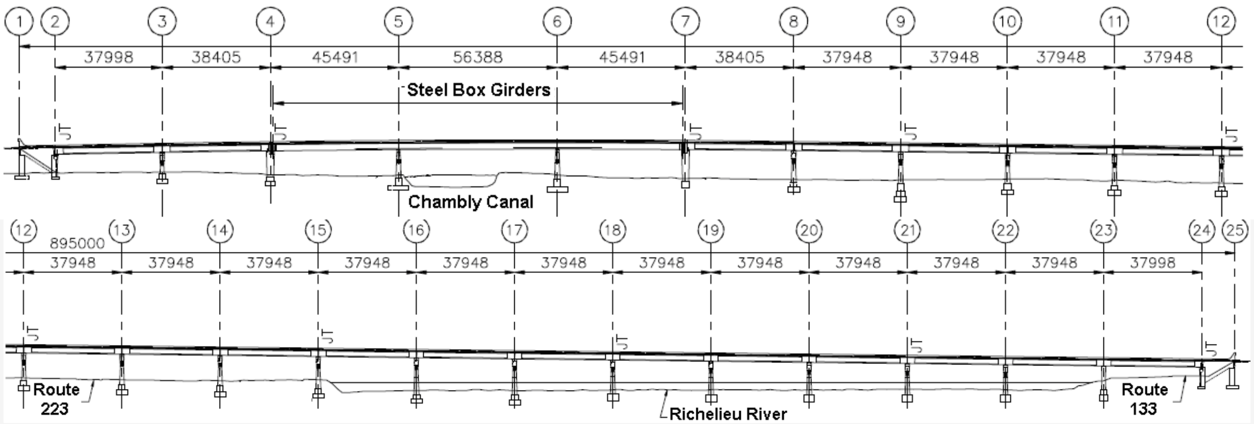


Figure 1: Elevation view of the bridge (dimensions in mm)

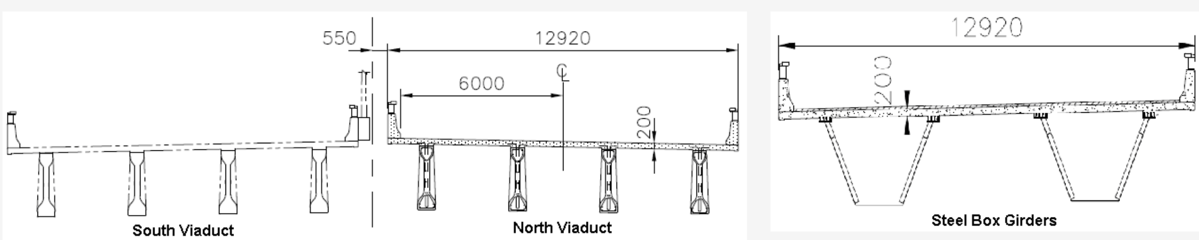


Figure 2: Cross section of the twin bridge (dimensions in mm)

With the exception of three piers that are multi-bent columns, the substructures consist of a hammerhead pier for each viaduct supported by a shallow foundation. Most footings lay directly on the bedrock while some are founded on a dense to very dense till. At some locations, piers 4, 7 and 16 to 23, both viaducts share a common footing (see figure 3). The bridge was evaluated and its seismic resistance was found to be inadequate, only being able to sustain a 90-year return period earthquake. The main structural deficiencies were related to the overturning capacity of the footings as well as the piers' flexural capacity. Based on this evaluation, it was decided to perform a seismic retrofit as part of repairs to the structure. Those repairs consisted of deck slab replacement, bearings replacement and minor repairs on some of the prestressed concrete beams damaged by leaking joints. To improve the durability of the structure, two thirds of the deck joints have been eliminated by providing a continuous deck slab between the adjacent spans. The deck isolation solution has been preferred to a conventional strengthening solution since it reduces sufficiently the demand on the structure and was three times less expensive. For the South Viaduct, the owner (*Ministère des Transports du Québec*) chose the seismic importance category as "Other bridges" and it was decided to improve as much as possible the seismic performance of the bridge using conventional rubber bearings. For the North Viaduct, the target performance chosen by the owner was the "Lifeline" Importance Category. In this case, isolation devices with a large energy-dissipation capacity were required to achieve this seismic performance.

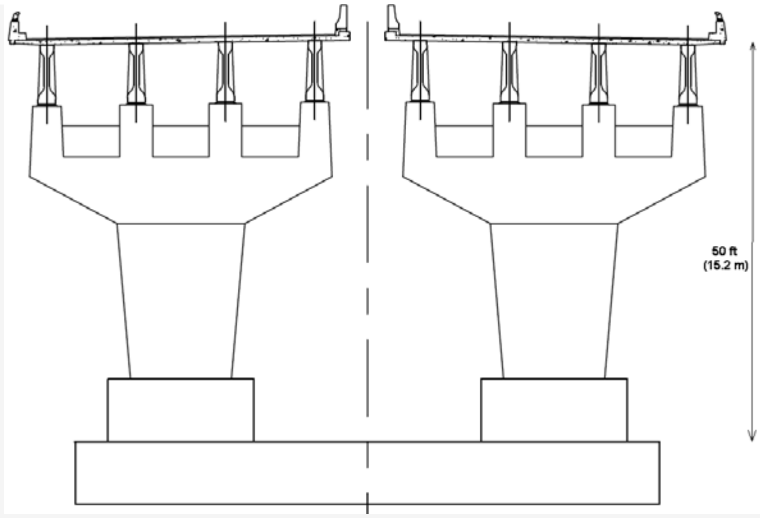


Figure 3: Typical Hammerhead Pier with a Common Footing

3 Seismic performance evaluation

The use of base isolation for seismic retrofit of existing bridge is more challenging than its use for new bridges. First, engineers need to design the seismic isolators according to the structural capacity of the existing structure. Special care is needed while performing the dynamic analyses and determining the isolation device specifications. Second, the displacements of the decks need to be controlled to respect the existing clearances and avoid pounding between adjacent superstructures. Therefore, a compromise between forces and displacements needs to be achieved by modifying the isolator's stiffness, a property which also changes with temperature, aging, wear and contamination. Moreover, considering that each viaduct has a different seismic performance target and that some footings are common for both viaducts, additional checks were needed to make sure that in case of a major earthquake, the overloaded South Viaduct will not cause the collapse of the North Viaduct. The next paragraphs describe the methodology used to evaluate the seismic performance of both bridges.

The first step is to carefully evaluate the resistance of existing structural components based on the bridge construction drawings by taking into account each cross-section and detailing. Considering the importance of the bridge, a site investigation was carried out to sample the concrete and steel reinforcement. Moreover, the geotechnical capacity was determined with SPT by drilling through the existing footings. By doing so, an important improvement of the geotechnical capacity was achieved by accounting for the soil compaction from the bridge utilization for over almost 50 years. It was also possible to verify the elevation of the footing and its concrete quality. For the piers, the bending and shear strength were the governing factors whereas for the footings, the overturning moment and the soil capacity were.

The next step was to conduct dynamic analyzes to evaluate the seismic demand on the structure. For a preliminary design of the isolation devices, a single-mode spectral method could be used. However, it neglects the pier acceleration and can significantly underestimate the demand for heavy piers, particularly for the ones located in the Eastern North America. With a three-dimensional modeling of the structure, the multi-mode spectral method (MM) can provide an excellent evaluation of displacements and forces. Considering the important bridge length, the bridge was split in three models and a boundary frame or an abutment was used to take into account the influence of the adjacent frames. A spine model was used to represent the superstructure while each pier was modeled with several frame elements to represent its mass distribution. Spring foundations were also used to represent the soil-structure interaction whereas an equivalent linear stiffness and an equivalent viscous damping were used to represent the isolator's behaviour. Figure 4 shows a view of the model used for a section of the twin bridge.

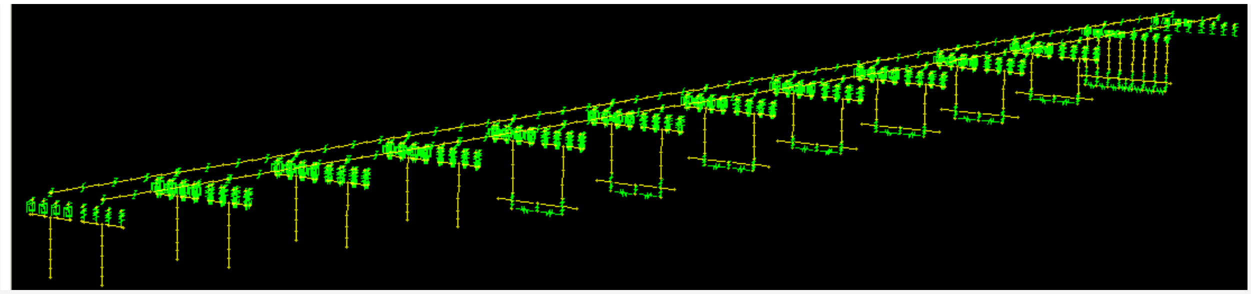


Figure 4: Spine Model of the Twin Bridge from Axis 12 to 24.

Nevertheless, the analysis of the two bridges together was very complicated. First of all, both bearing types have different damping properties: the rubber isolators on the South Viaduct provide a damping of 5 to 10% while 25% to 35% is reached with the friction pendulums on the North Viaduct. Since the natural periods of both viaducts are close to each other, this causes difficulties in the design spectra definition, especially in the long period zone. Indeed, with the MM analysis technique, the accelerations of the periods larger than 0,8Teff are reduced by a damping coefficient (BL) to account for the equivalent viscous damping. Even if one succeeds to set up a user defined spectra that applies the correct acceleration at each isolated modes, it was observed that the modal mass distribution was not correct. Since the modes are close to each other, part of the South Viaduct deck modal mass is excited by the vibration modes of the North Viaduct and vice versa. Consequently, part of the bridge masses is not correctly accelerated and thus, the forces determined are not accurate. The solution was to analyse them separately. With the MM dynamic analysis, the maximum response for each viaduct is obtained using a combination method, such as CQC (Complete Quadratic Combination). However, to assess the twin bridge stability against the overturning moment of a common footing, no result was directly available. Considering it is generally not recommended to use the results of a combination method to determine another parameter, the maximum responses were simply added up at this stage. Thereafter, non-linear time history analyses were performed, using direct integration, to correctly assess the demand of the twin bridge. Seven pairs of earthquake horizontal ground-motion were scaled while the hysteresis curves of the seismic isolators have been taken into account. The earthquake scaling was done without any modification in the frequency content and by limiting respectively to 0.25 and 4 the minimum and maximum scaling factor. For the range of periods under consideration, which was assumed to be 0.5Teff and 1.5Teff, each pair of motions was scaled such that the SSRS spectra does not fall by more than 10%, at any period, below 1.3 times the design spectra. The PEER Strong Motion database was used to select the earthquakes and seven events have been chosen to only consider the average value of the response parameter of interest, not the maximum. The thin curves on figure 5 illustrate the SSRS spectra of each earthquake selected whereas the blue and red curves represent, respectively, the average SSRS spectra and the design spectra.

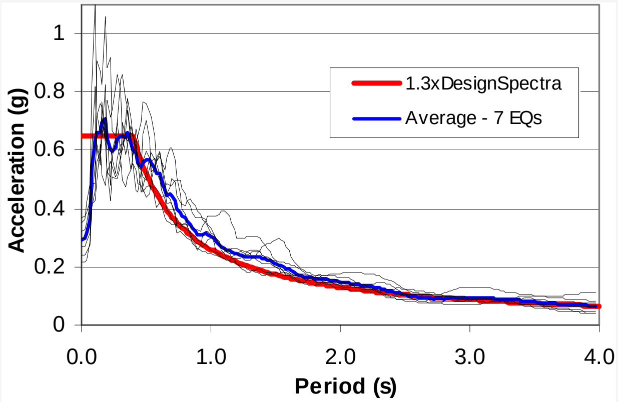


Figure 5: SSRS of the Earthquake Spectral Horizontal Components

It is interesting to examine the results obtained with different dynamic analysis methods. In this case, the objective was not only to compare the three most common methods, which are the single-mode spectral, multi-mode spectral and time history, but to see the differences when the bridges are modeled as a whole and separately. This exercise was conducted at axis 16, which is the most critical pier for this section of the bridge and where two hammerhead piers shared a common foundation in the Richelieu River (as shown on Figure 3). The results are presented in Table 1 for the maximum force in the seismic isolation, the maximum bending moment in the pier, the maximum pier base shear and the maximum overturning moment. In all cases, the isolators properties used are the ones determined during the prototype tests at a temperature of -30°C. Moreover, the results are only presented for the longitudinal direction.

For the single-spectral method (MU), considering seismic isolator devices are used at each pier, the effective seismic weight was assumed to be equal to the superstructure loads it supports. Considering that, in this section of the bridge, the substructure stiffnesses are similar, the spans are identical and the isolators have the same properties, it is an appropriate assumption.

For the multimode-spectral method (MM), the three-dimensional modeling of this section of each bridge was used. An iterative procedure was used to converge to a solution where the values obtained for the isolator displacement and the equivalent viscous damping are equal to the ones assumed. For this method as well as for the MU method, a maximum damping of 30% was considered to reduce the spectral acceleration, as recommended by most Standards.

For the time-history method (TH), each pair of motions was applied simultaneously. Two analyses were required for each earthquake since each horizontal component needed to be applied in longitudinal and transversal directions. The conventional rubber bearings were modeled as a linear element while the friction pendulums were set-up with their hysteresis curve.

Table 1: Results from the Different Dynamic Analysis Methods at Pier 16

	South Viaduct		**North Viaduct**		**Twin Bridge**
Dynamic analysis method	**MU**	**MM**	**MU**	**MM**	**TH (South/North)**
Maximum force in the isolation device (kN)	172	169	99	100	215/112
Pier base shear (kN)	1387	1913	795	1870	2550/1864
Bending moment at pier base (kNm)	19693	21790	11285	17953	27133/17613
Overturning moment at footing (kNm)	24546	28315	14066	24368	54483

The first observation we can note in Table 1 is that the spectral methods give almost identical results in term of the maximum seismic isolator force. This is not surprising, since the main difference between the two methods is the consideration of the non-isolated modes, which generate very little movement at the pier top. However, the differences are large for the other parameters because the pier itself represents approximately 40% of the total mass. From these results, it is clear that single-spectral method gives non-conservative results and should not be used for a seismic isolated bridge with heavy piers. For a more conventional seismic design, as when the bridge deck is pinned to the pier, it is generally recommended to include a part of the pier mass in the total mass considered for the deck. However, for an isolated structure, it may correctly adjust the forces in the foundations but it will overestimate the ones in the seismic isolators as well as the displacements. Therefore, this correction is not appropriate for seismic isolated bridge.

An interesting observation based on the results of the MM method is the very small difference in the substructure forces between the South and North Viaducts. Even if the probable force in the seismic isolator is more than 50% larger for the South Viaduct (169 kN vs 100 kN), the difference drops to 15-20% for the pier bending and the foundation overturning moment. The reduction is even more significant for the probable pier shear force, where both viaducts end up with similar forces. However, it must be remembered that these are not exact results, but most probable values obtained with a combination method. In this case, the CQC was used but the results from the TH method clearly show that the difference between each viaduct is quite larger.

From the results of the MM and TH methods, it can be said that the scaled earthquakes are conservative considering the larger forces in the isolators. Indeed, for both methods, the rubber isolators were modeled exactly in the same way but the seven-motion-average force is 25% larger. For the friction pendulum, the difference is smaller because it was modeled with the hysteresis curve instead of an equivalent stiffness and the benefit effect of damping was not limited to 30%. This conservatism comes from the scaling method used and described previously in the text. By scaling both horizontal motions to 1.3 times the design spectra, we can end up with one of the components being 20-30% larger than the design spectra, while the MM method only applies the design spectra as the maximum in one direction. In the end, the overall overturning moment is very similar, but the distribution between each bridge is quite different, which was mandatory in order to obtain a different performance level for each bridge.

At pier 16, if the vertical accelerations are considered by including only 80% of the dead load, the pier bending capacity is 18000 kNm. Without it, the capacity reaches 20500 kNm while the shear capacity is around 4000 kN in both cases. These capacities were calculated with a conservative reinforcement yield strength of 275 MPa whereas the average found during site investigations was 325 MPa. All the reinforcement detailing was checked for this second larger value. Moreover, the overturning capacity of pier 16 is respectively around 50000 kNm and 60000 kNm whether or not the vertical accelerations are considered. Based on these capacities and the results presented above, the bending capacity of the South Viaduct pier is reached well before the overturning of the common footing. Therefore, in case of a major earthquake, the South Viaduct will not control the seismic performance of the North Viaduct, since it will not be able to sustain such high loads, even if the steel reinforcements perform better than anticipated. The seismic performance evaluation of each bridge is based on the earthquake return period it can sustain. Figure 6 shows the longitudinal response of pier 16 during an earthquake.

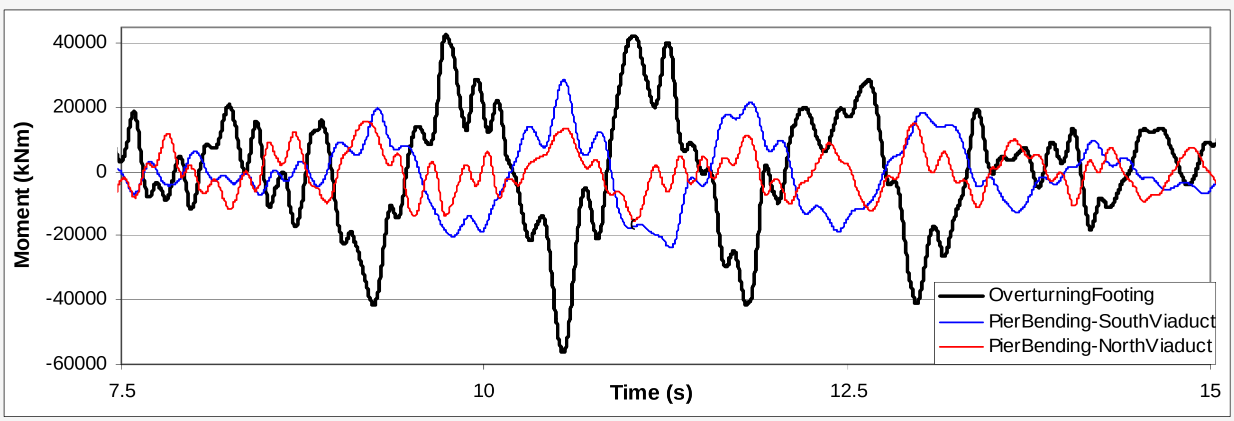


Figure 6: Twin Bridge Response during an Earthquake at Pier 16

The seismic retrofit design of the North Viaduct was done during the construction works of the South Viaduct and therefore, the rubber isolator properties were known from the large-scale testing. They were used during the design optimization of the North Viaduct to determine the isolator properties required to achieve the target performance. The final specifications were the result of a long process where, at each step, dynamic analyses were performed for the three bridge sections to evaluate the forces and the displacements. For the North Viaduct, it was decided to use a lower stiffness to reduce the probability of having both viaducts' maximum overturning moment occurring simultaneously. The South Viaduct's concrete girder rubber isolators have a constant stiffness of 3.0 kN/mm at 20°C while the North Viaduct friction pendulums have a 1.5 kN/mm at the design displacement. At low temperatures, which are the governing case for the seismic forces, the stiffnesses respectively increase to 4.4 and 3.0 kN/mm. Some of the existing clearances were smaller than 100 mm and thus, the deck seismic displacement was limited to around 50 mm. In the end, it was found that seismic isolators with a large-energy dissipation capacity were necessary for the North Viaduct to sustain a 1000-year return period earthquake.

4 Large-scale testing

4.1 Test Procedure

This section summarizes the results of the qualification tests performed on isolation bearings on both bridges. In accordance with Canadian Bridge Code (S6-06), two full-size samples of each bearings type must be tested to validate the dynamic forces and displacements under thermal, service and ultimate loads. They are also used to verify the dynamic response under different displacements, the survivability after a major earthquake, and the bearing stability under 1.5 times the maximum design displacement, all these requirements being very similar to the ones suggested by the previous edition of the Guide Specifications for Seismic Isolation (AASHTO, 1999). Additional tests at low temperature (-30°C) were also conducted, as required by the owner. In addition to the qualification tests, all bearings were subjected to quality tests, but only qualification tests are discussed in the paper.

Two bearings, identical to those to be installed under the concrete beams for each bridge, laminated bearings for the South Viaduct and double pendulum for the North Viaduct, were tested in the Structural laboratory of Sherbrooke University. These tests were carried out and supervised by the staff of the Civil Engineering Department. They submitted a compliance report at the end of the qualification tests. The test system, shown in Figure 7, includes hydraulic and mechanical elements that embed the vertical load applied simultaneously, representing the gravity loads, with a horizontal movement, representing the relative movement of the deck relative to the top pier.

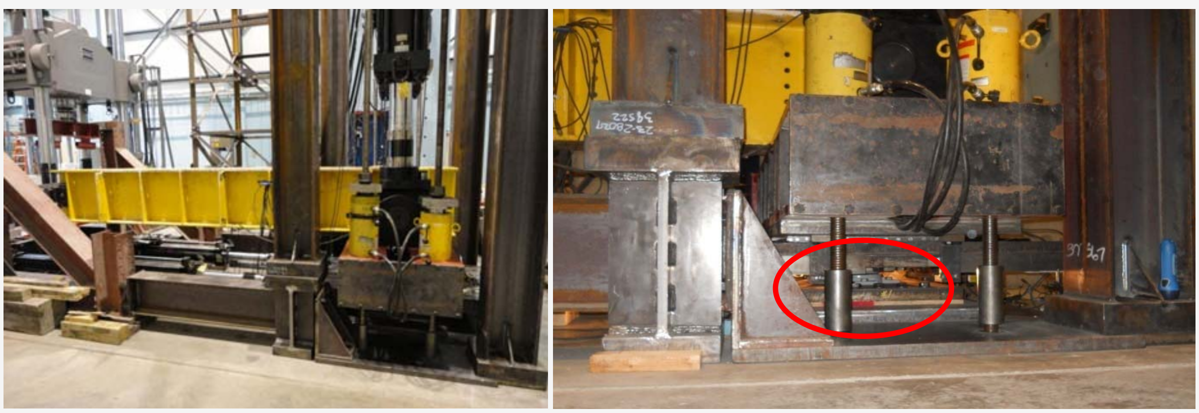


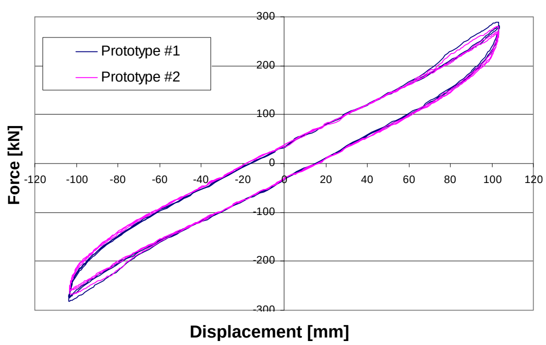
Figure 7: Test Set-Up (courtesy of Sherbrooke University)

The vertical force is applied by several actuators connected in parallel to a hydraulic pump. A pair of actuators installed symmetrically about the longitudinal axis of the bearing applies the horizontal loads on one side of the bearing top plate. The horizontal displacement is applied by a MTS controller. The horizontal force necessary to impose the displacement is measured by a load cell connected to the control system. Measurements from the load cell are stored in the data acquisition system. The displacements are measured by two displacement transducers (LVDT), one mounted on the top plate and the other on the bottom plate of the specimen. The horizontal displacement is obtained by subtracting the two readings and the displacement control is based on this value. Due to some set-up limitations and the fact that the rubber properties are not less dependant on the displacement velocity, the tests on the laminated bearings were performed with relatively slow frequency. However, for the pendulum isolators, the energy dissipation depends on the loading velocity. Thus, the dynamic tests on the pendulum isolators were carried out at a frequency approximately equal to the natural frequency of the isolators. With the forces and movements involved, this requires equipment with very high dynamic capacity.

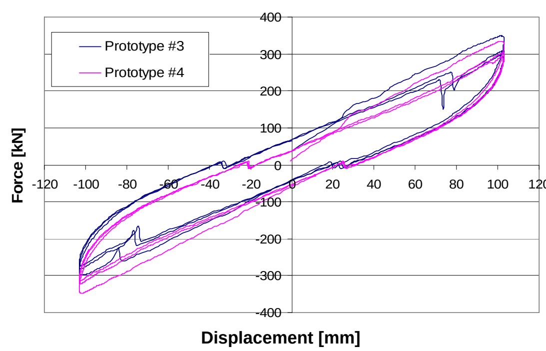
4.2 Test Results – South Viaduct

To accelerate the tests and reduce the waiting time required for cooling the bearings for the low temperature tests, four prototypes were manufactured. Prototypes #1 and #2 were used for ambient temperature tests and prototypes #3 and #4 for low temperature tests. At ambient temperature, both prototypes, # 1 and #2, have shown a very similar behaviour. Some results are shown in Figure 8. The characteristics of isolators are determined using Clause 4.10.11.3 of the S6-06. The measured force-displacement response shows a hardening post peak behaviour. During the three cycle tests at design displacement, the maximum variation of the effective stiffness for each prototype was less than 5% respecting the 10% limit of the Code. Moreover, the average stiffness during this test was similar for both prototypes, 2.6 kN/mm specimen 1 and 2.5 kN/mm for specimen 2. During the twenty cycles test at the design displacement, the stiffness variation was one percent (1%), which is well below the maximum limit of 20% permitted by the Code. During this test, dissipated energy was decreasing with the increase of the number of cycles. However, the maximum observed inter-cycle reduction was 5%, or four times lower than the 20% limit set by the code. For this test, the average stiffness's were respectively 2.33 kN/mm and 2.26 kN/mm for specimens 1 and 2 and with an effective damping of 5.1% and 5.7% respectively.

At low temperatures, for specimens 3 and 4 under a displacement of 25 mm, the calculated effective stiffness was 4.4 kN/mm. however, under the design displacement, the effective stiffness decreased to 3.0 kN mm due to warming of the samples. Also, an interesting phenomenon was observed: the effective damping of the isolator increased at low temperature. An average damping of 10% was found for both specimens. The average horizontal forces were respectively 320 kN and 302 kN with an average horizontal force of 311 kN, significantly lower than the design value of 386 kN. This decrease can be explained by the fact that the design was based on higher stiffness and a lower effective damping of 5%.



a) Three cycle test – rubber bearings



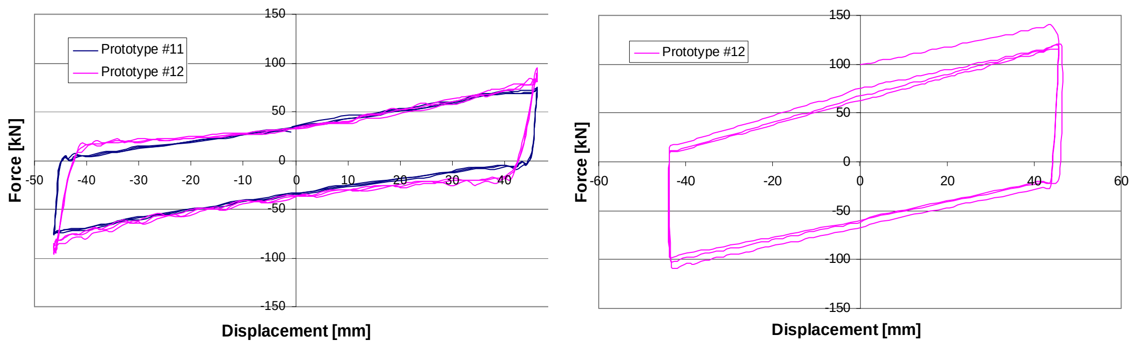
b) Low temperature test – rubber bearings

Figure 8: Qualification Tests Results at Design Displacement – South Viaduct

4.3 Test Results – North Viaduct

Qualification tests at ambient temperatures were performed on two specimens, prototypes 11 and 12. Some results are shown in Figure 9. Under service loads, the isolators have a high stiffness of about 190 kN/mm, and can be considered fix for all practical purposes, thus no longitudinal restrainers, initially required by the design, were needed. Similarly to the South Viaduct, specimens 11 and 12 have shown a hardening post yielding stiffness the 3-cycle tests at the design displacement and at ambient temperature. The maximum variation in the average effective stiffness during testing was less than 2% (respecting the tolerance of 10%), and with an average stiffness of 1.61 kN/mm. During the test with twenty cycles, the stiffness variation was 19%, which is below the 20% limit set by the S6-06. The bearing experienced also a decrease in the ability to dissipate energy with increasing number of cycles. With the exception of the first cycle, the maximum reduction in inter-cycle damping observed was 19% (20% limit). For this test, the observed average stiffness was 1.61 kN / mm with an average damping of 33%. Moreover, the damping of the first cycle is estimated equal to 39%, which is higher than the subsequent cycles and the minimum damping specified in the specifications (30%). To verify the isolator's behaviour under thermal loading, an additional test of three cycles (AASHTO, 2010) was also added to verify the bearing forces that isolator develops during thermal movements.

At low temperatures, only specimen 12 was tested. Under a displacement of 45 mm, the calculated effective stiffness was 2.5 kN/mm. However, under the design displacement of 30 mm, the effective stiffness was increased to 3.0 kN/mm but considering the effective damping increases to about 40%, it helps mitigate the effects of the increased isolator stiffness. The average horizontal force was 95 kN at 30 mm displacement - lower than the design value. This decrease can be explained by the fact that the design optimization was based on a maximum damping of 30% whereas the values measured during the tests were much more favourable. Considering that the design optimization was made before the qualification tests, it was decided to use a conservative assumption for damping value (30%).



a) Three cycle test – pendulum bearings

b) Low temperature test– pendulum bearings

Figure 9: Qualification Tests Results at Design Displacement - North Viaduct

5 Conclusion

The seismic rehabilitation of a twin bridge on Highway 10 using seismic isolators has significantly improved its seismic performance. Considering that two different performance levels were targeted, two types of isolators were used: elastomeric bearings for the South Viaduct and friction pendulum bearings for the North Viaduct. For the latter, the use of seismic isolators with a large-energy dissipation capacity allows the structure to sustain a 1000-year return period earthquake without any strengthening to the existing substructures. Considering both viaducts share a common footing at some locations, non-linear time history analyzes were conducted to assess the dynamic response. It was shown that the bending capacity of the South Viaduct is reached well before the overall overturning moment and therefore, the South Viaduct will not control the seismic performance of the North Viaduct. Finally, to verify the isolator characteristics used in the analyses, large-scale testing were performed on two prototypes of each isolator type.

Acknowledgement

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