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# TransCanada Highway 1 – Upper Levels Bridge Seismic Retrofits

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## ABSTRACT

The British Columbia Ministry of Transportation and Infrastructure (BC MoTI) identified three underpass bridges along the TransCanada Highway 1 – Upper Levels for potential seismic retrofit as part of its Bridge Seismic Retrofit Program. The bridges are located in West Vancouver and were constructed circa 1973 using steel rigid-frame configurations with inclined pier legs founded on soils ranging from Site Class B to D. The BC MoTI retained T.Y. Lin International Canada Inc. (TYLin) to assess the vulnerability of the existing structures using Canada's new 6th Generation Seismic Hazard Model demands, which were found to be significantly higher than the previous 5th Generation demands. TYLin developed detailed finite element method models to analyze the demand effects and propose retrofit designs that addressed the particular vulnerabilities of each bridge. Key criteria for the retrofit designs included minimizing pier foundation modifications and maintaining at least one lane of traffic throughout the construction work. The improvements for two of the bridges were isolated to their abutment and pier bearing connections, whereas the largest and most complicated bridge required more extensive retrofit improvements, including deadman anchors, link slabs, connection improvements, and the installation of a fluid viscous damper system to provide supplementary energy dissipation. The proposed retrofits for all the bridges were designed to provide a life-safety service level and a probable-replacement damage level for the 975-year return period design earthquake. This seismic performance level permits safe highway traffic flow below all the bridges while retaining limited live load capacity for traffic traversing the bridges. Construction is set to begin on two of the bridges in the summer of 2023. This paper provides a discussion of the seismic hazards and vulnerabilities affecting each of the bridges as well as the details of the seismic retrofit designs.

Keywords: Bridge Seismic Retrofit Program, 6th Generation Seismic Hazard, Rigid-Frame Bridge, Viscous Damper System

## INTRODUCTION

T.Y. Lin International Canada Inc. (TYLin) was contracted by the British Columbia Ministry of Transportation and Infrastructure (BC MoTI) to provide consulting engineering services for the seismic evaluation and seismic retrofit design of three bridges, Horseshoe Bay Drive (Horseshoe Bay), Eagleridge Drive (Eagleridge), and Caulfeild Drive (Caulfeild) Underpasses, crossing the TransCanada Highway 1 – Upper Levels in West Vancouver, BC. All three brides have steel girder-concrete deck composite superstructures, with girder lines that are rigidly framed to inclined steel pier legs. The locations where the girders frame into the pier legs are here referred to as the "knee joints". The inclined pier legs sit on pin bearings on reinforced concrete pedestals that are founded on a reinforced concrete spread footing. Abutments, which are also supported on reinforced concrete spread footings, support the girder ends via elastomeric bearings. Two of the bridges, Eagleridge and Caulfeild, have similar structural configurations, whereas Horseshoe Bay has a significantly more complex structure (including plan curvature, skew angles greater than 45 degrees, in-span half-joints, and a central rigid frame "delta" pier). Collectively these three bridges, which are shown in Figure 1, are hereafter referred to as the Upper Levels Underpasses.

TYLin began the seismic evaluation with a structural and geotechnical site investigation to assess the condition of the existing bridge structure and foundation. The structural geometry was determined using record drawings made available by the BC MoTI, shop drawings acquired from the original precast concrete supplier for the decks of the two similar bridges, and previous retrofit drawings in the case of the Eagleridge bridge. Global analysis models for the bridges were produced using this available structural and geotechnical information, and local structural and geotechnical models were developed to efficiently capture the complex seismic behaviour at the knee joints and foundation regions within the global analysis models.

Due to the timing of the project, TYLin consulted a seismic hazard specialist firm to produce Uniform Hazard Spectra (UHS) appropriate to the location and soil conditions for the three bridge sites. The UHS calculations were performed using the software OpenQuake [1], and the associated input files were obtained from the Geological Survey of Canada Open File 8630. The UHS were generated at three return periods (probabilities of exceedance): 2475 years, 975 years, and 475 years, representing a 2%, 5%, and 10% chance of exceedance in 50 years, respectively. The UHS developed by the OpenQuake [1] software was later compared to the Seismic Hazard Tool provided by Natural Resources Canada (NRC) [2] for use in conjunction with the 2020 edition of the National Building Code of Canada (NBCC 2020) [3], CSA S6:14 [4], and the BC MoTI Supplement to CSA S6:14 [5]. The difference between the UHS from the OpenQuake [1] software and the NRC Seismic Hazard Tool was found to be on the order of only +/- 2%, which was judged to represent a negligible difference in seismic demands.

*Figure 1. General Arrangement Views (clockwise from top left to bottom): Eagleridge, Caulfeild, Horseshoe Bay*

TYLin's retrofit design was guided by the BC MoTI's seismic performance criteria requirements for the project, which are shown in Figure 2. Based on discussion with the BC MoTI, the 975-yr Hazard Level retrofit was the preferred retrofit option; however, if the required retrofits were significantly more expensive and/or invasive than the 475-yr Hazard Level retrofit, the 475-yr Hazard Level retrofit would be selected. The intent of the 2475-yr Hazard Level retrofit is to offer an even less expensive and/or invasive retrofit option than the 475-yr Hazard Level retrofit. This was because the "Possible Loss of Span Prevention" Damage Level did not require detailed evaluation or retrofit of the substructure or foundations, only the superstructure and superstructure-to-substructure connections. However, this retrofit option was believed to be intended for conventional girder bridges that generally do not require retrofit to the superstructure for seismic demands, meaning that only bearing or restrainer retrofit would be required. The rigid framing of the superstructure to pier legs on the Upper Levels Underpasses exhibited different behaviour since the "superstructure" (consisting of the girders, deck, and pier legs) could be subject to large seismic demands. The 2475-yr Hazard Level retrofit was therefore not necessarily expected to yield a lower cost or less invasive alternative to the 475-yr Hazard Level retrofit for the Upper Levels Underpasses; however, it was included in the evaluation for completeness.

| Seismic Hazard Level | Service Level | Damage Level | Description |
|---|---|---|---|
| 975-yr | Life-Safety | Probable Replacement | Baseline Retrofit Option |
| 475-yr | Life-Safety | Probable Replacement | Comparison to Retrofit Option 1 |
| 2475-yr | Possible Loss of Service | Loss of Span Prevention | Superstructure-Only Retrofit Option |

*Figure 2. Seismic Performance Objectives Summary*

This paper discusses the seismic hazards affecting these bridges and the vulnerabilities of their structural configurations. Commentary is presented on the seismic provisions contained in CSA S6:14 [4], the BC MoTI Supplement to CSA S6:14 [5], and other seismic design guidelines used in the evaluation [6,7,8,9]. Discussion of the details of the seismic retrofit designs is also provided, including the specifics of the seismic analyses and the rationale behind each design.

## SEISMIC ANALYSIS

### Numerical Modelling

Finite element method (FEM) numerical analysis models were developed in the CSiBridge v22.2.0 software platform [10]. The global analysis models used frame elements to represent the pier legs and girders and shell elements to model the concrete deck, as shown in Figure 3. Composite action between the girders and deck was created via rigid link elements spaced along the girders. For the non-composite negative moment regions on the Eagleridge and Caulfeild bridges, shear and flexure releases were applied to the rigid link elements to account for the non-composite action in these locations. The pier and abutment bearings were modelled as links with shear and flexure releases and stiffnesses to reflect the true behaviour of the respective bearings. Frame elements were also used to model the substructure. The weight and mass of the wearing surface and railing loads were accounted for through material modifiers and discrete loads/masses, respectively. Concrete elastic stiffnesses on the gross section were reduced over areas where high tensile stresses were identified to have developed in the deck, and in the case of Eagleridge and Caulfeild, where the deck panel joints were not filled with concrete.

For computational efficiency, the geometry of the knee joints was not explicitly modelled in the global analysis models. Instead, girder and pier leg frame elements were extended to the knee joint control node, which represented the centroid intersection of the frame elements. Due to overlapping frame element effects, a strong axis moment-of-inertia property modifier was applied to the pier leg frame elements within the knee joint. This moment-of-inertia modifier required calibration with a more detailed local analysis model that used highly discretized shell elements to define the knee joints, as shown in Figure 4. By the end of the calibration process, the global analysis model produced key outputs that were within approximately 5% of the local analysis model.

All three bridges have two distinct seismic load paths for longitudinal and transverse seismic demands. The abutments provide no longitudinal fixity to the bridge superstructure, and therefore all longitudinal demands have a load path through the inclined pier legs and into the pier foundations. Conversely, the unbraced pier legs are very slender in the transverse direction, and therefore nearly all transverse demands are transferred through the deck diaphragm to the abutment foundations via shear keys. On all three bridges, the existing transverse shear connection between the superstructure and abutment is deficient for seismic loads and is considered to represent a vulnerability. In the case of Horseshoe Bay, the in-span half joints is also considered to be a seismically vulnerable element, as it breaks the continuity of the deck diaphragm and lacks the transverse shear capacity to transfer the diaphragm loads.

In the analysis models, both the abutment transverse connections and the half joints connections were considered to be able to rigidly transfer shear forces. This is not entirely representative of the original condition, as neither of these components have the capacity to reliably transfer seismic shears. However, if the limited capacity of these components was modelled, the dynamic behaviour of the structure would change substantially, as transverse inertial loads from the deck would have been required to be carried by the pier legs down to the pier foundations. A preliminary investigation found that this is not an acceptable load path for any of the hazard levels considered. Furthermore, reliable gravity support to the side spans would be lost if shear transfer at the Horseshoe Bay abutment and half joints were to be lost. Therefore, in these two aspects the analysis models better represents the as-retrofit structure than the current condition, as retrofits to increase the shear transfer capacity of these components is required at all Seismic Hazard Levels.

Deterioration observed during the inspection is generally not considered explicitly in this seismic evaluation as it is not believed to significantly affect the seismic performance of the bridge. However, the relatively poor condition of the deck was taken into consideration when assuming effective deck stiffnesses in the model.

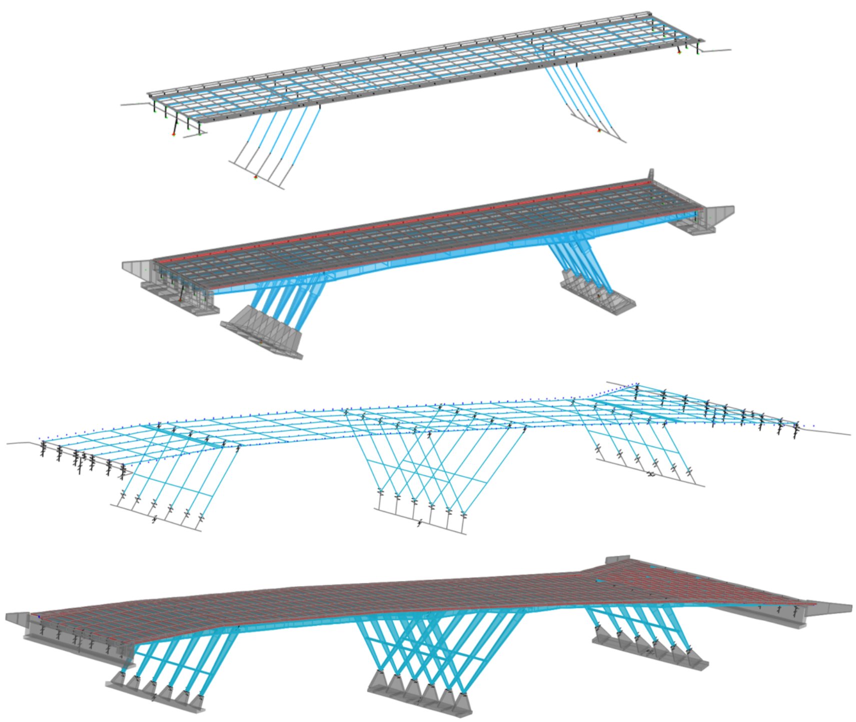


*Figure 3. Global Analysis Models (from top to bottom): Wireframe View for Eagleridge (Caulfeild Similar), Complete View for Eagleridge (Caulfeild Similar), Wireframe View for Horseshoe Bay, Complete View for Horseshoe Bay*

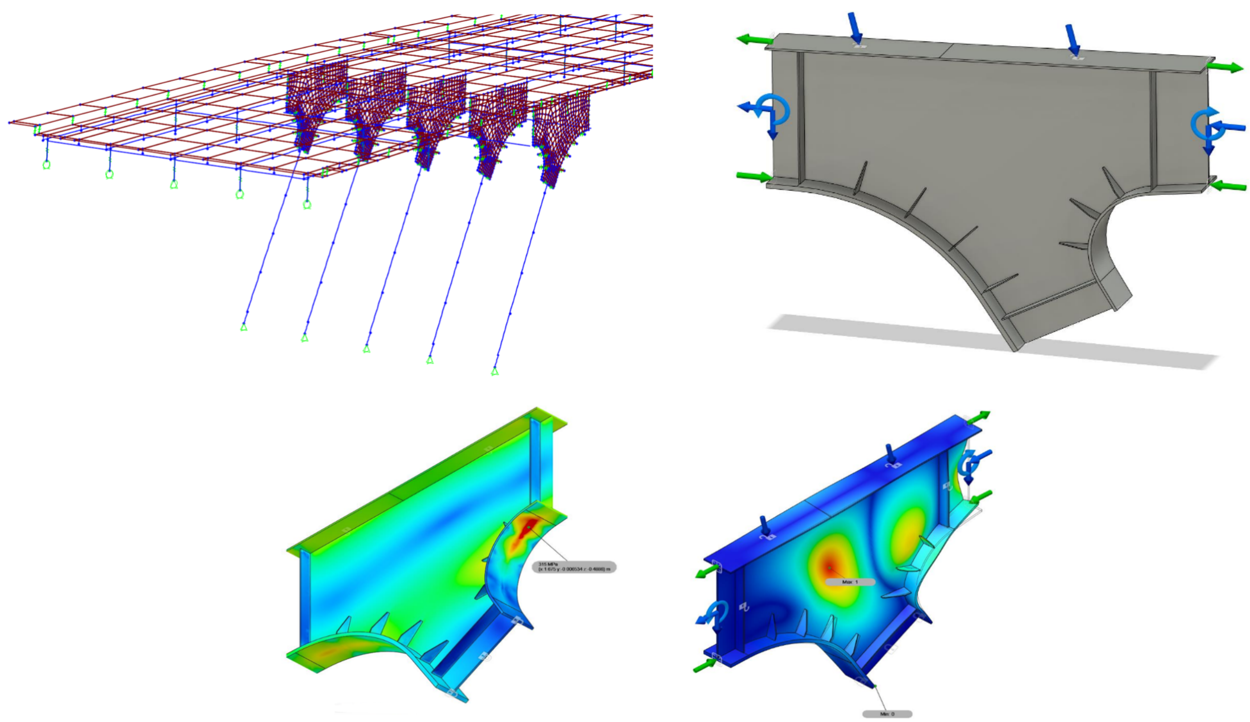


*Figure 4: Local Analysis Model of Knee Joint using Shell Elements*

**Soil-Structure Interaction**

Soil-structure interaction was considered via an arrangement of equivalent linearized soil springs. Six degree-of-freedom (6-DOF) soil springs, calculated in accordance with the commentary to S6-14 §4.6.4, were positioned at the center of the area of the underside of each pier and abutment slab footing. The stiffnesses of the slab footing frame elements were correspondingly increased to create effectively rigid elements in order to avoid artificial deformations associated with the single lumped foundation spring. Distributed 1-DOF soil springs were used to model the interaction between the abutment stem, ballast, and wing walls and the retained earth.

The soil springs were calculated for both upper- and lower-bound estimates of $G_{max}$. Two versions of the analysis model were developed, one with upper-bound springs and one with lower-bound springs. The nonlinear nature of the 6-DOF soil springs required that a set of initially-assumed trial spring stiffnesses be used in the first analysis iteration to produce estimated ULS5 975-year Hazard reactions and associated longitudinal load eccentricity ratios for the footings. The spring stiffnesses decrease with increasing longitudinal load eccentricity ratios for all DOF springs. The longitudinal load eccentricity ratio results from the assumed spring stiffness were then used to update the next set of trial spring stiffnesses. This iterative process led to convergence at spring stiffnesses which were then used for the determination of all other structural and geotechnical demands for that particular model, e.g. upper-bound or lower-bound model.

On Horseshoe Bay, longitudinal seismic movements were significant enough to cause expansion joint gap closing and subsequent pounding between the superstructure and the abutment back walls. This was accounted for in the analysis using an equivalent linearized stiffness based on the force-displacement response of the combined back wall hinging and soil wedge failure. This force-displacement response was determined via a 2D non-linear pushover finite element analysis that explicitly modelled the back wall and soil behind the abutment.

**Demands**

- Seismic Loads: The OpenQuake [1] software requires site-specific geotechnical properties to be explicitly input, rather than using “Site Class” designations, which are used in CSA S6:14 [4]. As only visual geotechnical investigations were conducted as part of this project, the analyses were run multiple times with geotechnical parameters that bounded the CSA S6:14 Site Class for each bridge. The larger spectral accelerations from the two bounding analyses were used in all cases. As part of the project, TYLin also evaluated the differences in design spectral accelerations between the new 6th Generation Seismic Hazard Model used with CSA S6:19 [6] and the 5th Generation Seismic Hazard Model used with CSA S6:14 [4]. The evaluation found that the 6th Generation design spectral accelerations exceeded the 5th Generation design spectral accelerations by 10-31% and 13-59% for Site Class B and D soil conditions, respectively, as shown in Figure 5.

  Per the Seismic Retrofit Design Criteria (BC MoTI, 2005), a damping value of 5% of critical was assumed at all Seismic Hazard Levels. This damping value is in line with the recommendations from well-recognized literature for welded structures experiencing stress levels near the yield point (Newmark and Hall [8]; Chopra [9]). This 5% damping value was considered conservative as it essentially neglects soil damping provided by rocking and sliding of the spread footing foundations, which can significantly supplement structural damping, as outlined in FEMA 440 [11]. Taking advantage of soil damping was not deemed appropriate for these bridges due to the limited geotechnical investigations performed.

- Load Combinations: The evaluation considered the CSA S6:14 Ultimate Limit States load combination 5 (ULS 5: 1.0D+1.0EQ), where D = dead load and EQ = earthquake load) [4]. Note that the maximum and minimum dead load factors specified in CSA S6:14 [4] Table 3.1 and Table 3.3 were not used, as these are intended to indirectly represent vertical seismic effects, which are not required to be considered for seismic evaluations as described in CSA S6:14 §C4.11.6 [4].

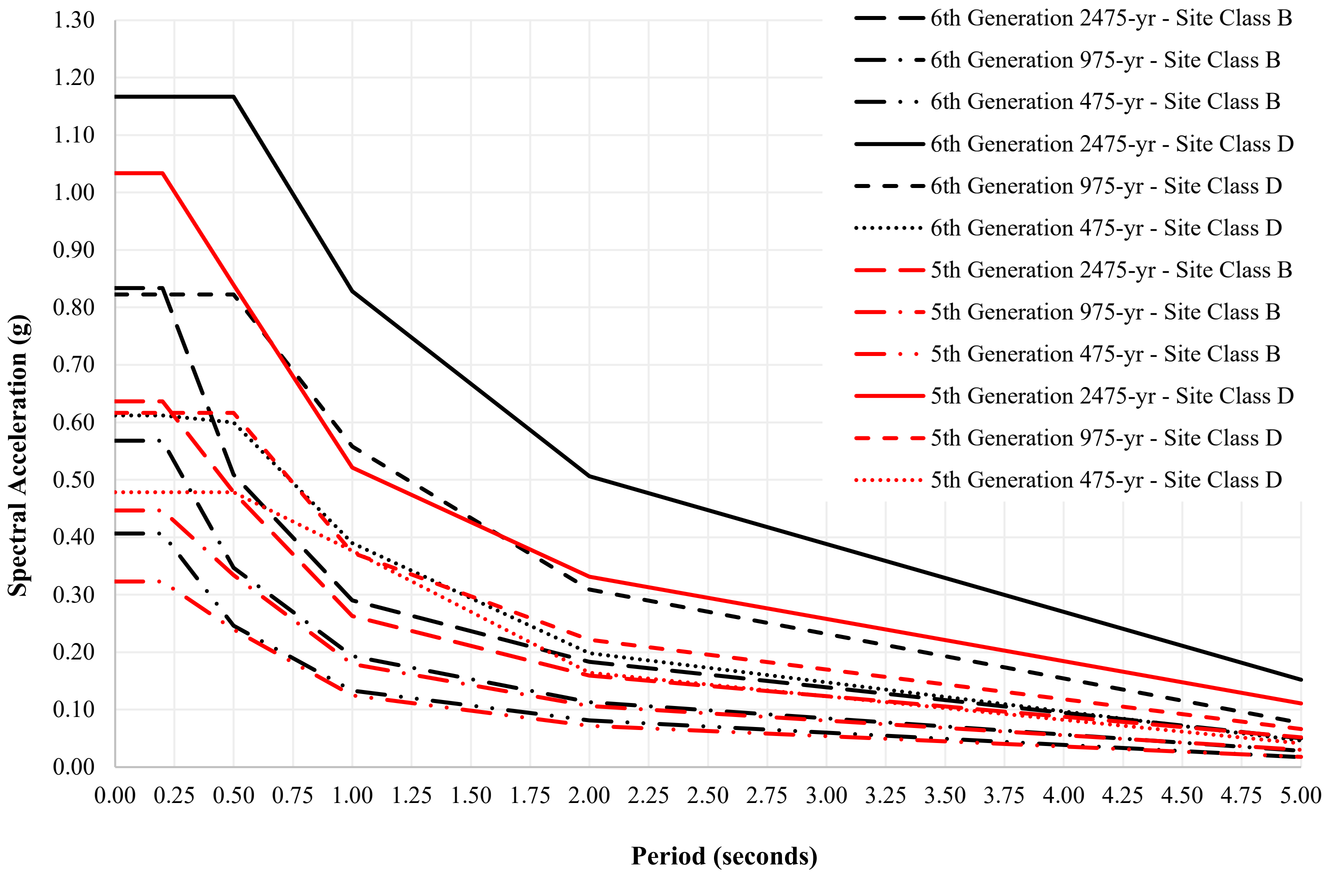



*Figure 5. 6th Generation Seismic Hazard Model Site Class B and D Design Hazard Spectra (5% Damping), where the properties of Site Class B and D are defined in CSA S6:14*

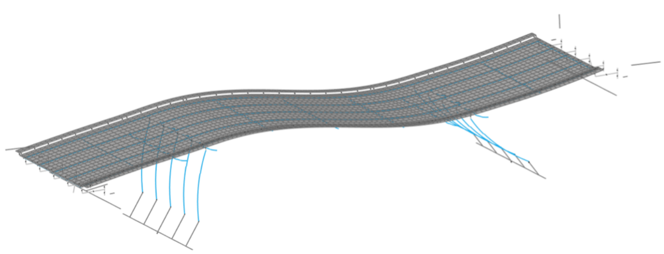
**Resistances**

Member capacities were determined in accordance with CSA S6:14 §4.11.8 [4]. Nominal resistances, calculated using the material properties provided below and a resistance factor φ = 1.0, were used in all cases.

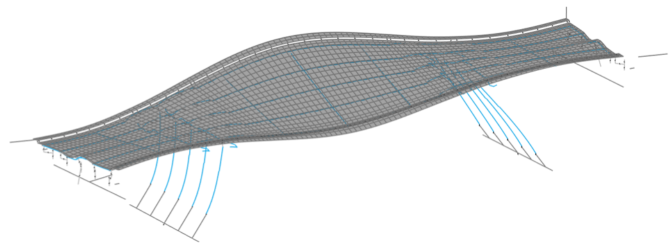
- Structural Steel: The record drawings noted that the structural steel as conforming to the "current" CSA G40.8 specification [12]. As the record drawings are dated circa 1975, it was assumed that the 1971 CSA G40.8 specification [12] as applicable, with minimum yield and tensile strengths ranging from 36-40 ksi (248-276 MPa) and 65 ksi (448 MPa), respectively, depending on the steel plate thicknesses, with thinner plates having higher yield strengths.
- Concrete: According to the record drawings, all concrete had a minimum 28-day compressive strength of 3000 psi (20.7 MPa), except for the deck slab, which has a minimum 28-day compressive strength of 4000 psi (27.6 MPa).
- Reinforcing Steel: The record drawings noted that all reinforcing steel as conforming to CSA G30.6 specification [13]. Since the grade of reinforcement is not indicated on the drawings, the lowest strength 40 ksi (276 MPa) grade from the 1967 CSA G30.6 specification [13] is assumed.

**Response Spectrum Analyses**

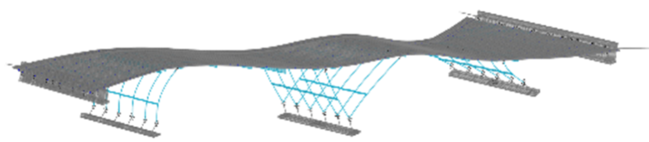
Response spectrum analyses were carried out using the 6th Generation Seismic Hazard Model demands converted to a design spectrum in accordance with S6-14 §4.4.3.4. The multi-mode spectral method was used to capture at least 90% participation of the contributing inertial mass. The modal member forces and displacements were combined using the complete quadratic combination method, while the horizontal elastic seismic effects were directionally combined in accordance with S6-14 §4.4.9.2 (100%-30% method). As discussed earlier, vertical seismic effects were not considered for this evaluation per S6-14 §C4.11.6. The multi-mode spectral method was used, with 100 modes included in order to capture at least 90% participation of the contributing inertial mass in all three orthogonal directions. Select fundamental mode shapes from the Upper Levels Underpasses are provided in Figure 6.



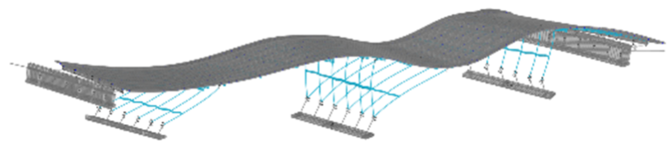
*Mode 1 – Primary Fundamental Longitudinal Mode Eagleridge, 0.49 seconds (Caulfeild Similar)*



*Mode 2 – Primary Fundamental Transverse Mode Eagleridge, 0.25 seconds (Caulfeild Similar)*



*Mode 1 – Primary Fundamental Longitudinal Mode Horseshoe Bay, 0.54 seconds*



*Mode 2 – Primary Fundamental Transverse Mode Horseshoe Bay, 0.54 seconds*

*Figure 6. Fundamental Mode Shapes for the Upper Levels Underpasses*

**Pushover Analyses**

For the Horseshoe Bay Underpass, a 2D frame pushover analysis model was developed to better understand (i) the strength hierarchy between the longitudinal elements, (ii) the expected degree of inelastic behaviour, and (iii) whether geometric nonlinearity had any significant effect in the longitudinal direction. Material nonlinearity was incorporated via the fiber hinge method available in CSiBridge [10]. Each cross-sectional model consisted of a series of discretized fibers which correspond to nonlinear uniaxial material models. The moment-curvature response of each hinge at each step of the analysis is then directly obtained via iterative integration of the cross-section fibers. This method enables hinge P-M interaction to be considered explicitly in the analysis.

Key results from the pushover analysis are shown in Figure 7. The response at the 475-year and 975-year hazard level is entirely elastic, with some limited yielding at the 2475-year hazard level (displacements were taken from the response spectrum analysis). The pushover response with or without consideration of geometric nonlinearity was essentially identical.

Limitations to the pushover analysis include no consideration of out-of-plane behaviour or brittle failure modes. However, the results do indicate that the structure behaves essentially elastically and that geometric nonlinearity has a negligible effect. Therefore, seismic assessment based on response spectrum analysis results is considered acceptable for this structure.

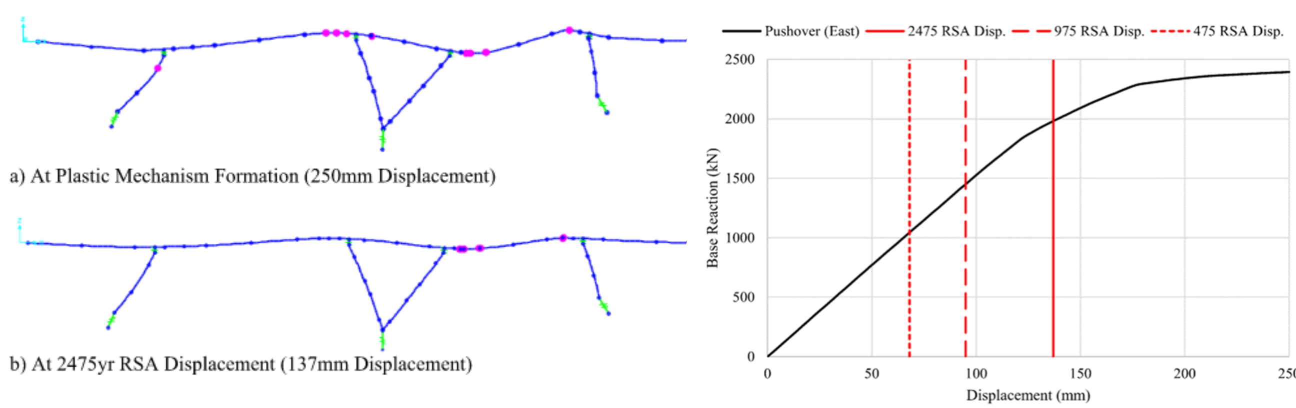



*Figure 7. Horseshoe Bay Pushover Analysis Key Findings*

## SEISMIC RETROFIT DESIGN

A series of seismic retrofit design strategies were employed to protect structural vulnerabilities identified during the seismic analyses. To maximize cost-effectiveness on the project, several of these seismic retrofit strategies were employed on multiple bridges, as described in this section.

### Abutment Shear Keys

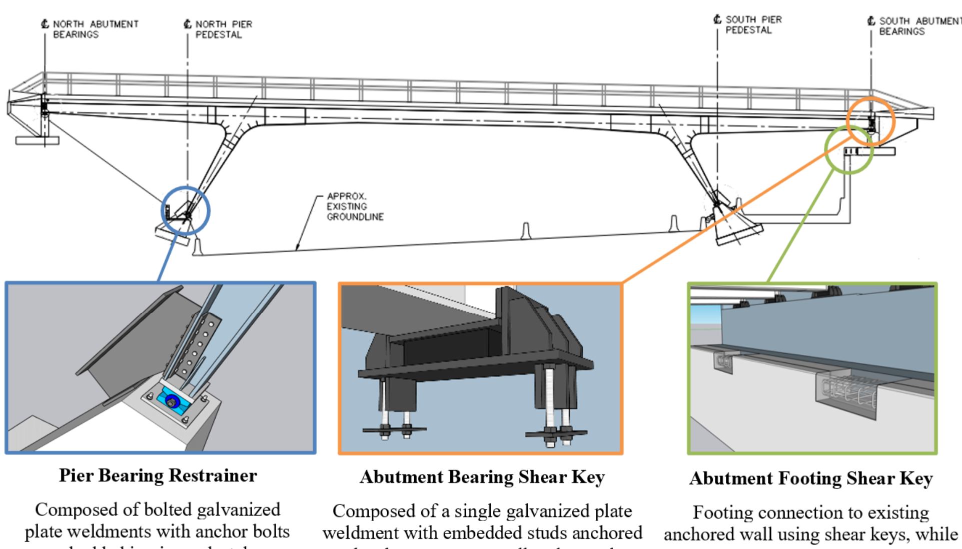
Individual shear keys composed of galvanized steel plate weldments with cast-in embedded shear studs were determined to be an economical and effective option to address the existing shear key vulnerabilities of both Caulfeild and Eagleridge, as shown in Figure 8. The individual shear keys proposed include a space for new elastomeric bearing pads that will replace the existing bearings. The weldments also provide uplift restraint by engaging the existing bearing sole plates. On Horseshoe Bay, the transverse seismic demands were higher and therefore new concrete shear keys are required. The concrete shear keys are fixed to the existing abutment via concrete breakout, dowelling in new reinforcement, and reinstating the concrete.

### Pier Leg Bearing Restraints

For all the Upper Levels Underpasses, since the shear, bending, and axial capacities of the existing bearings could not be reliably determined, they could not be safely relied upon without unloading the bearings and replacing them. Replacing the bearings would require as much installation and design effort as a seismic retrofit. Additionally, the pier anchor bolts did not possess sufficient shear concrete breakout strength to accommodate the 975-yr seismic event. Therefore, shear restrainers were designed to carry the seismic loads imposed on the bearings. The restrainers are built-up galvanized steel weldments that are bolted to the top flange of each pier leg, which are also shown in Figure 8. Under seismic loading, the weldment bears on the side of the bearing pedestal and prevents the pier bearing from unseating by providing an alternate shear load path should the bearing components or the anchor bolts fail. When longitudinal shear loads act towards the bridge midspan, the bearing components are at their highest risk of failure, as these loads occur concurrently with the lowest bearing compression. Under these loads, the weldment will engage and prevent unseating. When longitudinal shear loads act in the opposite direction towards the abutment, they occur concurrently with the highest bearing compression. This magnifies the interface shear capacity of the bearing, which provides an alternate load path to the bearing components and anchor bolts. Lateral shear demands can be carried by the existing washer plates on either side of the bearings in the case of Caulfeild or retrofit lateral shear tabs in the case of Eagleridge and Horseshoe Bay.

### Abutment Footing Shear Keys

While the Caulfeild abutments were found not to be vulnerable to sliding displacements due to their Site Class B soil conditions, the transverse sliding displacements at the Site Class D Eagleridge south abutment were determined to be excessive. To prevent these sliding displacements, the south abutment footing was connected to the existing retaining wall using shear keys, also shown in Figure 8. A minimum 50 mm gap between the south abutment footing and the retaining wall was maintained along the full length of the interface to prevent longitudinal movements from affecting the wall.




**Pier Bearing Restrainer**

Composed of bolted galvanized plate weldments with anchor bolts embedded in pier pedestals – typical at all pier leg bearings

**Abutment Bearing Shear Key**

Composed of a single galvanized plate weldment with embedded studs anchored to the abutment stem wall and complete with new elastomeric bearing pad – typica at all abutment bearing locations

**Abutment Footing Shear Key**

Footing connection to existing anchored wall using shear keys, while leaving a gap between the footing and the anchored wall to prevent additional loading on the wall and the anchors

*Figure 9. Eagleridge Seismic Retrofit Strategies (Caulfeild similar except for Abutment Footing Shear Keys)*

### Link Slab

The existing half-joints on Horseshoe Bay were found to be vulnerable elements as they disrupt the deck diaphragm, preventing it from transferring shear forces away from the slender pier legs. The proposed retrofit concept to address this vulnerability involves demolishing a ~2.0m length of the bridge deck on either side of each of the west and east half-joints and replacing it with a continuous concrete link slab, as shown in Figure 9. A number of the existing shear studs would be removed to create a "debond layer", increasing the flexibility of the link slab to reduce the potential for cracking. New link slab reinforcement would be placed and the slab would be reinstated with new fiber-reinforced concrete, eliminating the existing deck joint. The retrofit results in a deck that acts as a continuous diaphragm, carrying seismic demands to the new abutment shear keys. The link slab also has the advantage of providing protection to the half-joints from moisture and debris ingress, which may improve the long-term durability of the area.

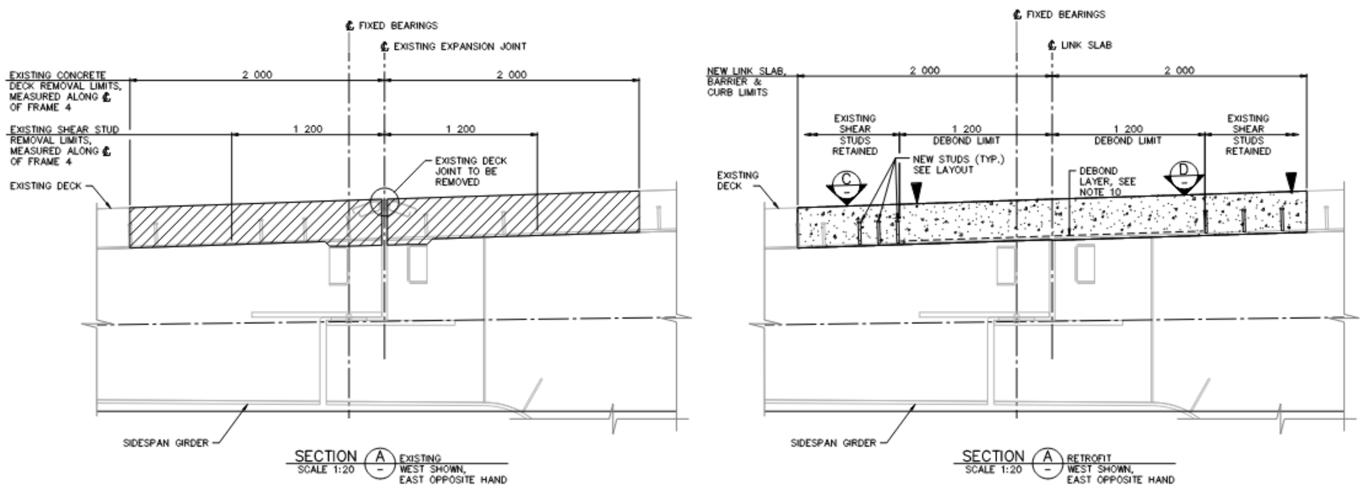



*Figure 9. Horseshoe Bay Link Slab Retrofit (section view at half-joint locations)*

### Viscous Dampers and Anchor Slabs (Horseshoe Bay)

Both the abutments (due to transverse sliding under transverse shear demands) and pier leg foundations (due to overturning failure under longitudinal shear demands) were found to be vulnerable elements on Horseshoe Bay. The proposed retrofit concept for addressing these vulnerable foundation elements at the 975-year hazard level involved (i) preventing abutment transverse sliding via the installation of new "anchor slabs", and (ii) reducing seismic demands on the pier foundations via the installation of new viscous dampers at the girder-to-abutment connections (both shown in Figure 10). These two systems also work together, with the anchor slab (and associated deadman anchor) resisting the force in the dampers when they go into tension. The viscous dampers have the added benefit of reducing seismic demands on the pier legs to the extent that they can still be considered acceptable at the 975-year hazard level, even in the event that the potentially vulnerable pier leg bracing connections fail. The viscous dampers also generally reduce the seismic demands on all elements on the bridge, simplifying the retrofit works required at the shear keys and pier leg bearing restraints and increasing the overall robustness of the bridge to withstand seismic events.

The viscous damper system was designed by first determining the required level of supplementary damping to achieve the desired performance objectives, then designing the damper units in accordance with ASCE 41 Chapter 15 linear dynamic procedure provisions [14]. The number of dampers, damping constant, and damping exponent were designed to achieve the required damping level while minimizing unwanted force effects in the surrounding structure and the overall cost of the retrofit design.

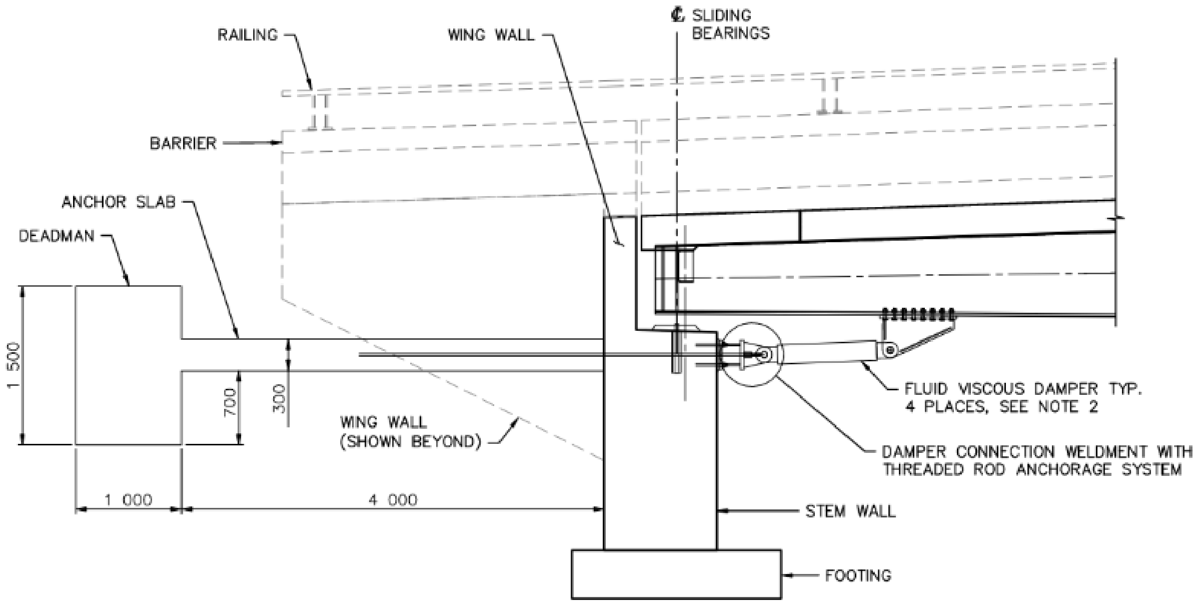



*Figure 10. Horseshoe Bay Viscous Damper and Anchor Slab Retrofits*

## CONCLUSIONS

This paper provided a discussion of the seismic hazards and vulnerabilities affecting each of the three Upper Levels Underpass bridges as well as the details of the seismic retrofit designs. Once implemented, these seismic retrofit designs will significantly improve the seismic reliability of these bridges, permitting safe highway traffic flow below all the bridges while retaining limited live load capacity for traffic traversing the bridges following a significant seismic event. Construction is set to begin on the Caulfeild and Eagleridge bridges in the summer of 2023, while construction work on Horseshoe Bay is expected in 2024.

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