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Highway 1 Burnside Bridges Widening and Seismic Retrofit

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ABSTRACT

Located in Saanich, British Columbia (BC), Canada, the bridges, locally known as the Burnside Bridges, carry Hwy 1 across Colquitz Creek and Interurban Road. The 01378 Colquitz River No. 1 Bridge, constructed in 1954, is an 82.6m long five-span steel I girder structure carrying two traffic lanes over the river. The 02655 Colquitz River No. 2 Bridge, constructed in 1977, is a 75.5 m long two-span steel I girder structure carrying two traffic lanes over the river.

Both bridges underwent a Seismic Safety Retrofit in 1994 with a stated performance goal to "prevent collapse and reduce damage" under a design earthquake with a 10% chance of occurrence in 50 years (1/475 year event). Both bridges are categorized as a Major-route bridge by the BC Ministry of Transportation and Infrastructure (BC MOTI) and have been assessed for two seismic events based on the Canadian Highway Bridge Design Code S6-19 (S6-19) and the BC MoTI's Supplement to S6-19. A detailed geotechnical and seismic assessment of both bridges was completed, including evaluating adequacy and performance of the bridge foundations under static and seismic loads. This included computation of lateral soil movements at each abutment and permitted evaluation of the lateral performance of the piled abutment foundations in response to the computed static and seismic soil movement. A nonlinear static analysis ("pushover analysis") and linear dynamic analysis ("response spectrum analysis") were performed. This paper will discuss the expected seismic performance of both structures, outlining the structural and geotechnical vulnerabilities and the retrofit strategies used to meet the performance requirements of S6-19 and the BC MOTI's supplement.

INTRODUCTION

Background

Located in Saanich, British Columbia (BC), Canada, the bridges, locally known as the Burnside Bridges, carry Hwy 1 across Colquitz Creek and Interurban Road. BC MOTI engaged Mott MacDonald Canada Limited (MMCL) to perform a scoping study which included widening of both bridges to accommodate a new bus lane. In addition to, a live load assessment of the bridges for existing and widened case and seismic evaluation and retrofit were also completed.

Bridge Description

01378 Colquitz River No. 1

The 01378 Colquitz River No. 1 Bridge, constructed in 1954, has a total length of 82.6 m and carries two lanes of Highway 1 westbound towards Nanaimo over Colquitz River and Interurban Road (north of Highway 1). It is located approximately 1 km east of the Admiral McKenzie Interchange in Saanich, BC near Victoria. The 02655 Colquitz River No. 2 Bridge carries eastbound traffic immediately south of the structure.

It is a five-span structure with six lines of continuous steel I-girders supporting a non-composite reinforced concrete deck (Figure 1).

The abutments consist of a cast-in-place concrete ballast wall and abutment seat supported on buried concrete columns and spread footings at the West Abutment and steel H-piles at the East Abutment. Each of the two-column bents is comprised of reinforced concrete and supported on a spread footing with the exception of Bent #1 which is supported on steel H-piles. Concrete wingwalls at either side of the abutments retain the adjacent embankment fill.

Six steel rocker bearings are located at both abutments and at Bent #1 and Bent #4. As part of the 1994 Seismic Retrofit, the segmental roller of the rocker bearings at Bent #1 and Bent #4 were encased with reinforced concrete seats. There are six fixed steel bearings at Bent #2 and Bent #3.

There are 1.83 m sidewalk with concrete posts and steel railings on the north side of the bridge and concrete barriers with steel railings on the south side of the bridge. Armored compression seal joints allow for longitudinal movement at each abutment.

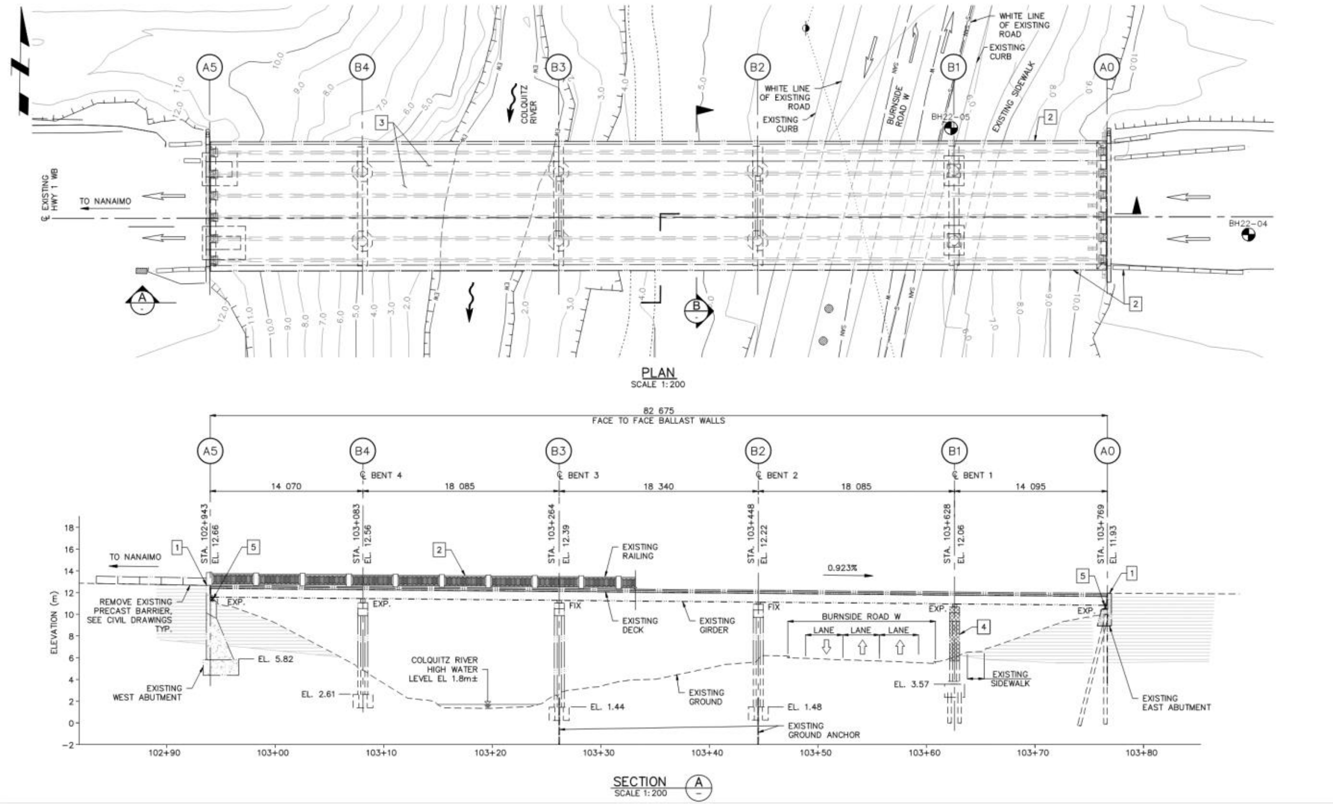


Figure 1. Details of the Colquitz River Bridge No. 1

<u>02655 Colquitz River No. 2</u>

The 02655 Colquitz River No. 2 Bridge, constructed in 1977, has a total length of 75.5 m and carries two lanes of Highway 1 eastbound towards Victoria over Colquitz River and Burnside Road West (known as Interurban Road north of Highway 1). It is located approximately 1 km east of the Admiral McKenzie Interchange in Saanich BC near Victoria. The 01378 Colquitz River No. 1 Bridge carries westbound traffic immediately north of the structure.

It is a two-span semi-integral structure with five lines of continuous steel I-girders supporting a composite reinforced concrete deck over the positive moment regions and a non-composite deck over the negative moment regions. There are concrete diaphragms at the girder ends spanning between the wingwalls (Figure 2).

The abutments consist of a cast-in-place concrete abutment seat, with wingwalls at either side of to retain the adjacent embankment fill. The abutments are supported by concrete pile caps bearing on steel pipe piles, which are battered and driven to bedrock.

There are five thermally fixed and five steel expansion bearings located at the pier and at each abutment, respectively.

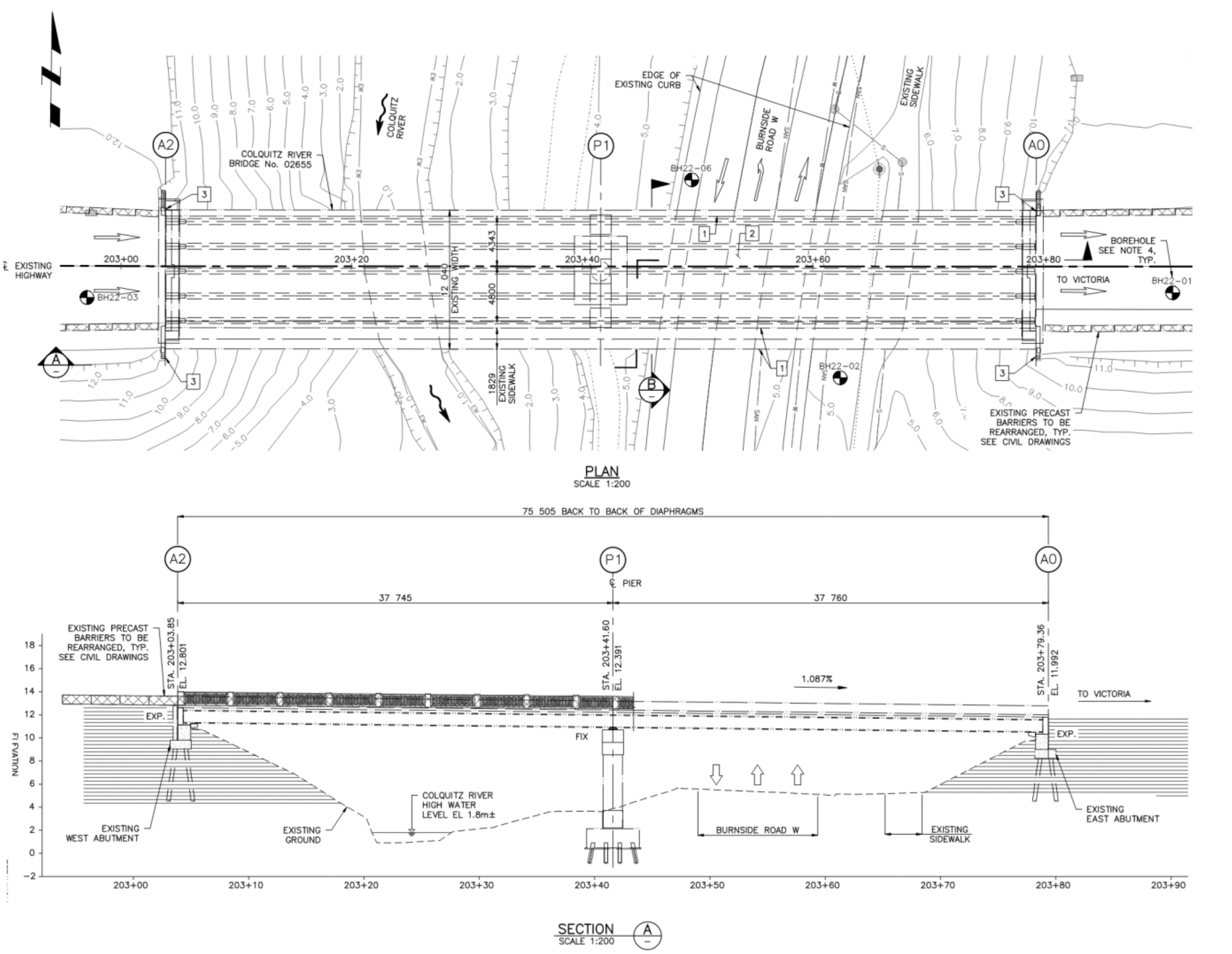


Figure 2. Details of the Colquitz River Bridge No. 2

WIDENING

Scope of Work

The key objective of the project was to extend shoulder bus lane between Tillicum Road and Admirals-McKenzie Interchange in each direction. Each widened bridge achieves a total of 12.4 m clear roadway width, providing a 1 m inside shoulder, 2 – 3.70 m lanes, and a 4.0 m outside shoulder to accommodate shoulder bus lane operations. The existing sidewalks, not connected on either end and unused accordingly, are eliminated on both bridges; the nearby Galloping Goose Trail is available for active modes of transport. The widening will also achieve a continuity of shoulder bus lanes between Tillicum Road and Admirals-McKenzie Interchange. The longitudinal profile was designed so that elevations of the finished deck surface and the finished road surface beyond the bridge limits remain close to existing levels. The proposed finished deck surface elevations are adjusted to accommodate a cross fall of 2%. The crown along the 01378 Colquitz River Bridge No. 1 was shifted 430 mm to ensure it is located between the proposed lanes, allowing for equal widening on both sides, and preventing the need for another girder line. The crown along the 02655 Colquitz River Bridge No. 2 was shifted 500 mm to eliminate the need for a retaining wall on the south side of the embankment at the east abutment. An additional girder line was proposed on the south end of the structure.

Renewal items include mill and fill concrete deck overlays, joint elimination, miscellaneous concrete patch repairs, and substructure upgrades to address existing geotechnical capacity deficiency for the piles. Functional upgrades, in addition to the widening, include barrier upgrades to current standards.

The existing and widened bridges were assessed for CL-625 loading as per S6-19. With the release of the new BC MOTI Supplement to S6-19 in July 2022 requiring a CL-800 loading, the bridges were assessed for the increased loading. The live load evaluations were carried out for ultimate limit states for all superstructure and substructure components.

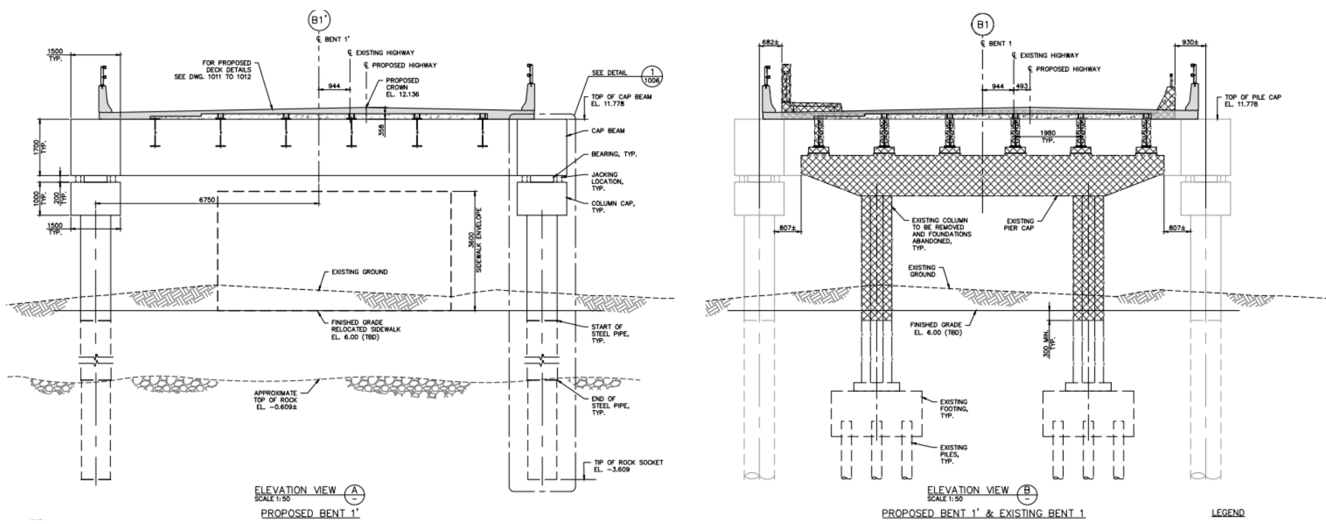
01378 Colquitz River No. 1

The proposed widening includes widening the bridge symmetrically about each side by approximately 650 mm. This approach eliminates the need to provide additional girder lines or widen the substructure. The geotechnical capacity for the piles at the Bent #1, indicate additional retrofit measures are needed in-order to satisfy the static demands.

The proposed summary of work is as follows:

- Construct a new Bent #1 with a new concrete cap supported on new column and steel pipe piles. The existing Bent #1 to be demolished once the new cap and column supports are installed.
- Remove existing raised sidewalk and railing on the north side, extend deck by approximately 650 mm.
- Remove existing barrier and railing on the south side, extend deck by approximately 650 mm.
- Remove existing concrete overlay and additional reinforcement which was placed in 1980 as a replacement to the original asphalt overlay. The existing concrete overlay tapers from 50 mm to 100 mm in height; the underlying structural deck is 165 mm in thickness.
- Add new concrete overlay and modify the existing grade profile to achieve a new 2% cross fall, providing a minimum cover of 60 mm on the deck.
- The provision of additional concrete overlay will include the addition of a new top mat of reinforcement.
- The finished crown will be shifted approximately 430 mm to the south to align with the lane line.
- Eliminate the existing joints at each abutment end by removing the top of the ballast wall and extending the deck over top of it.
- Adding bumpers to work as restrainers engaging Bent#2 in longitudinal direction.
- Adding approach slab and a deadman at the end of the approach slab to work as a second backwall.
- Removing fill around Bent #2 to allow for rocking prior to failure of the wall.
- Replacing the upper portion of the backfill underside of the approach slab area.

Figure 3 illustrates the proposed widening at the new Bent #1.



(a) *(b)*

Figure 3. Details of the widening at Bent #1: (a) elevation view of proposed bent, (b) elevation view of existing/proposed bent

02655 Colquitz River No. 2

The proposed widening includes widening the bridge on both sides, including the provision of an additional girder line on the south side of the bridge. The analysis concluded that a new girder was required to accommodate the deck widening at the south edge of the bridge. This subsequently required an extension to the pier cap beam. The existing pier cap, column, and piles did not meet the strength requirements for both CL-625 and CL-800 loading, and as such a new column was proposed underneath the new girder to alleviate load from the existing column. The existing deck and girders were found to be sufficient for CL-625 and CL-800.

The proposed summary of work is as follows:

- Install 350 mm thick topping to the existing pier foundation.
- Remove existing barrier and railing on the north side, extend deck by approximately 715 mm.

- Remove existing sidewalk and railing on the south side.
- Install new 762 mm diameter steel pipe pile with a 6m drilled rock socket at the south side of the pier. Install new 762 mm concrete column.
- Extend the concrete pier cap at the south side of the pier 1600 mm, tie into the new column and existing cap.
- Install new bearings and steel girder at the south side of the pier and abutments, offset by 1600 mm from existing exterior girder.
- Add new steel bracing to connect the new girder with existing girder.
- Extend deck by approximately 430 mm on the south side.
- Undertake a mill and fill concrete deck renewal for the remaining deck between the overhangs.
- Maintain 70 mm cover to deck reinforcing.
- Extend the deck at each abutment end.

Figure 4 illustrates the proposed widening at the pier.

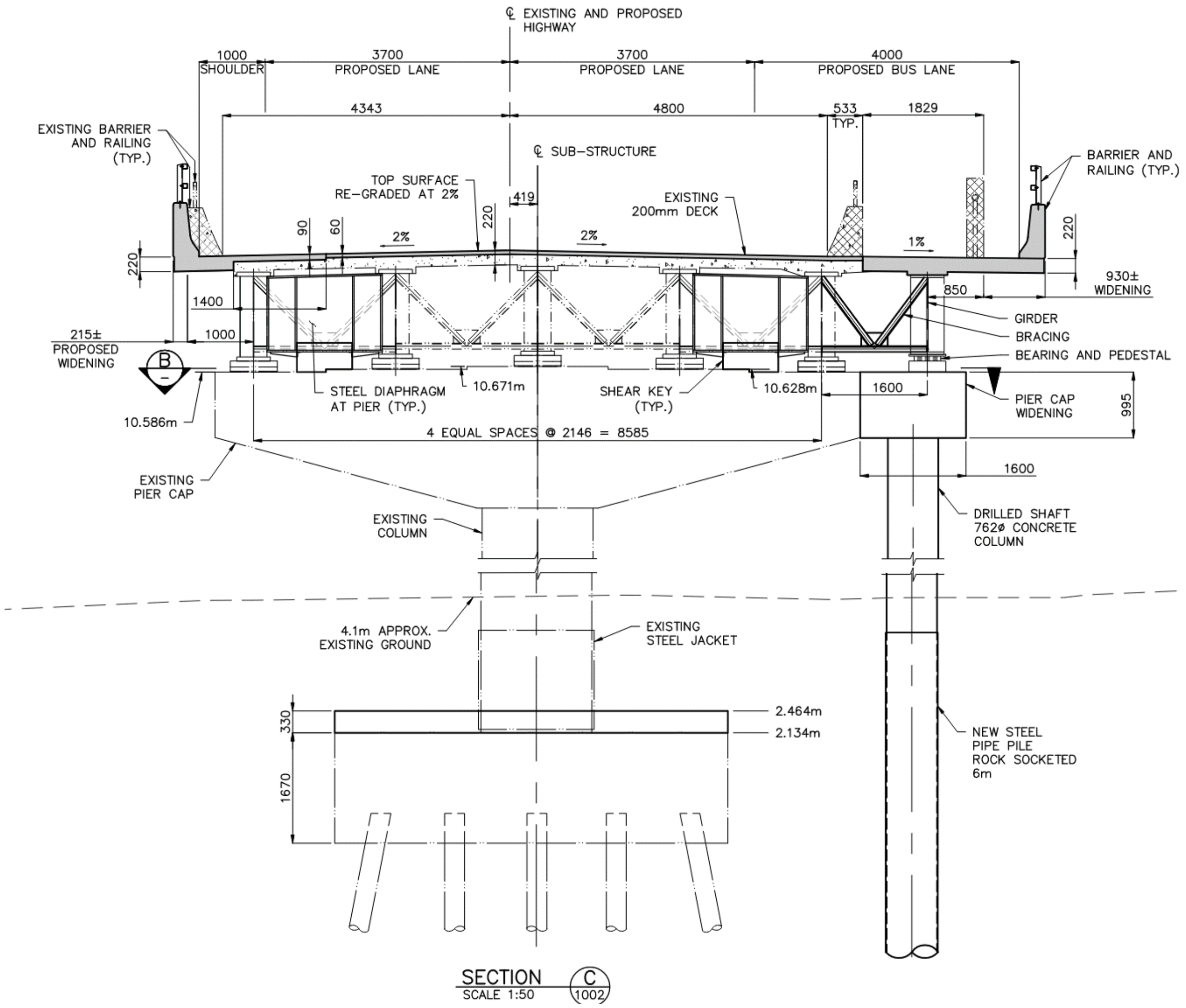


Figure 4. Details of the widening at the pier

SEISMIC STUDY & RETROFIT

Scope of Work

A Structural Seismic Evaluation (SSE) were completed to confirm compliance with performance requirements outlined in the S6-19 and BC MOTI Supplement to S6-19. BC MOTI engaged WSP (formally Wood Group) to provide input on the geotechnical assumptions used as part of the SSE. This involved geotechnical seismic hazard assessment of the bridges and evaluating adequacy and performance of the foundations under static and seismic loads to support the proposed widening.

A preliminary assessment was completed at the Functional Design stage to propose retrofit strategies with a more in-depth assessment completed during the Detailed Design stage of the project.

Seismic Design Criteria

The analysis was conducted for two levels of seismic events, 475-year and 2,475-year return periods with a probability of exceedance of 10% and 2% in 50 years, based on the S6-19 and BC MOTI Supplement to S6-19 for a major-route bridge, in Seismic Performance Category 3. The S6-19 requires major-route bridges to be fully serviceable following a 475-year return period earthquake and usable for emergency traffic following a 2,475-year return period earthquake. For the 475-year case, S6-19 requires limited lateral and vertical movements of bridge foundations such that slight misalignment of bridge spans and settlement of piers do not interfere with normal traffic. For the 2,475-year case, the code allows residual foundation movements such that the repairs can bring the structure back to the original operation capacity.

The structures were assessed for Site Class E at Functional Design stage based on a desktop study of existing geotechnical information for the site. In Detailed Design the design spectra were revised based on ground investigations which exceed Site Class E acceleration as per 5th Generation earthquake model. Figure 5 outlines the design spectrum for the 2,475-year return period for 02655 Colquitz River Bridge No. 2.

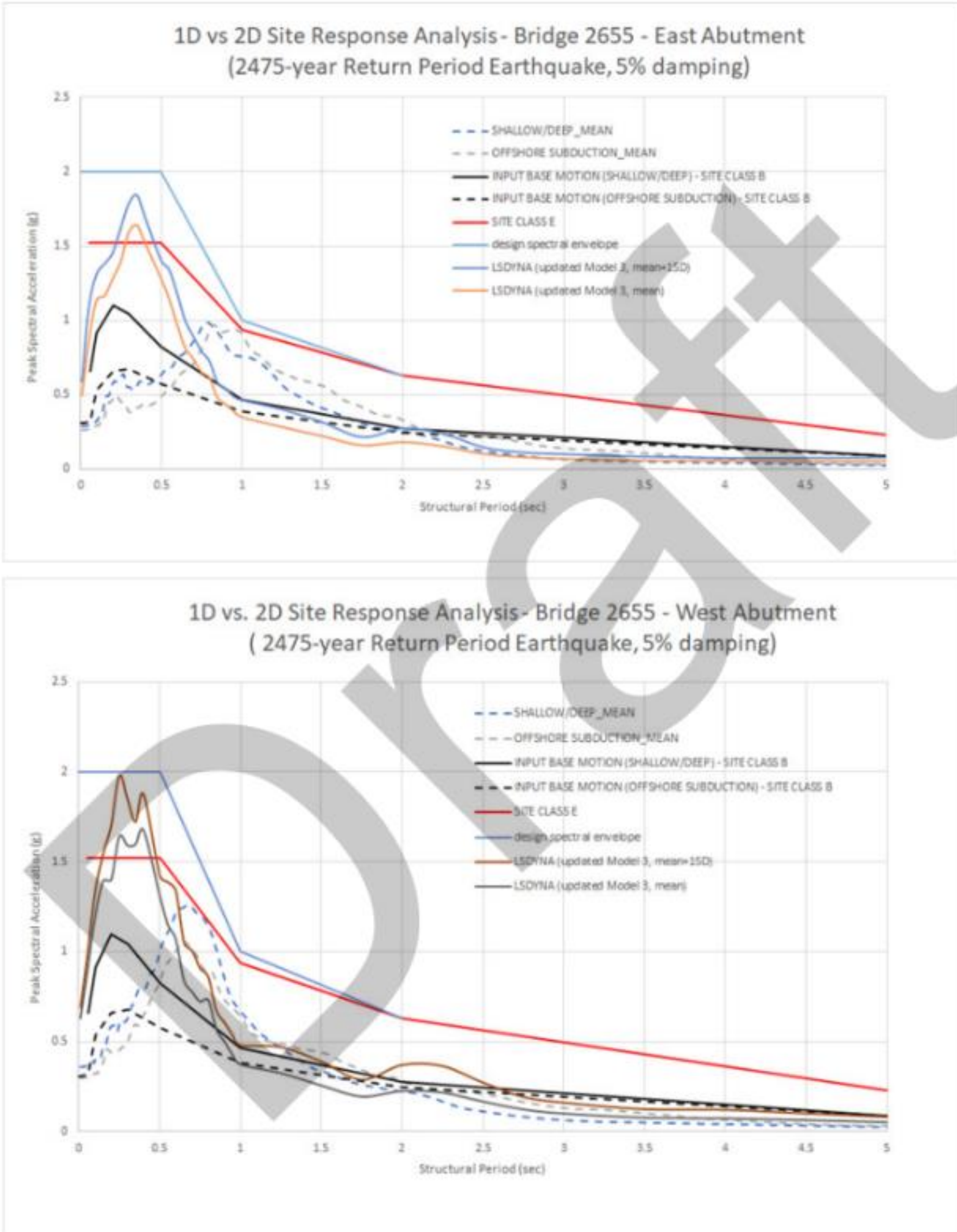


Figure 5. Design Spectrum for 02655 (01378 Similar)

Background and Retrofit History

The bridge underwent a Seismic Safety Retrofit in 1994 with a stated performance goal to "prevent collapse and reduce damage" under a design earthquake with a 10% chance of occurrence in 50 years (475-year event). The retrofit work for 01378 Colquitz River No. 1 included the addition/replacement of steel diaphragms at each abutment as well as Bent #2 and Bent #3; new concrete walls, rock anchors, and shear keys at Bent #2 and Bent #3; new raised concrete seats at each abutment as well as Bent #1 and Bent #4; and, new concrete shear keys at each abutment. The retrofit work for 02655 Colquitz River No. 2

included shear keys and steel brackets to the underside of the girders at the abutments as well as an extension of the abutment seats. In addition to, a concrete steel jacketing was provided to the bottom of the column.

Evaluation

A nonlinear static analysis (“pushover analysis”) and linear dynamic analysis (“response spectrum analysis”) were performed on the FE model. Two levels of seismic events (475-year and 2475-year) were considered in the response spectrum (RS) analysis. In-order to carry out the RS analysis, a modal analysis of the structure in the FE model is performed to obtain the relative mode shapes, frequencies and modal participation factors. The acceleration RS is used to derive an equivalent static load, which will provide the same maximum response as that obtained in each mode of vibration. The maximum modal responses are then combined to find the total maximum response of the structure. The RS will provide the elastic load and displacement demand at each member.

The following assumptions were made in-order to proceed with the SSE:

- New deck and overlay, steel girder, barrier and railing dead loads were accounted for in this evaluation as part of the widening work. In addition, dead loads for the new components including the Bent #1 at structure 01378 Colquitz River Bridge No. 1, and concrete cap extension, concrete column and steel pipe pile for structure 02655 Colquitz River Bridge No. 2 were also captured.
- 50% of the longitudinal stiffness associated with the fill engaged by each abutment backwall has been accounted for. Soil stiffness values have been calculated based on provided capacity of the passive pressure by the geotechnical engineer.
- Cracked section properties were used for the pier for the RS. The cracked section property is extrapolated for columns and beams as per ACI 318. This represents behavior of the bridge during an intense seismic event.
- Member capacities and D/C ratios do not consider section loss due to deterioration in the components which is consistent with the 2018 Detailed Condition Assessment.

<u>01378 Colquitz River No. 1</u>

The primary periods of the structure shifts from short to long in both directions due to yielding of the embankment in the longitudinal direction and rocking of the bents and sliding of the abutment in transverse direction.

A multi modal RS analysis is carried out to capture the elastic loads and displacement demands at each member. The capacities of the structural members were calculated using the applicable clauses of the S6-19 and compared with the 2475-year and 475-year RS demands. If the demand-to-capacity (D/C) ratios identified inelastic behavior, damage levels were verified in ductile components with non-linear pushover analysis. The pushover analysis considers the individual bents and West Abutment separately. The bents and the West Abutment are pushed in both the transverse and longitudinal direction (independently) to establish the pushover curves (load vs displacement) for the two directions.

Longitudinally, it is expected that the gap between the backwall and the steel girders (50 mm) will close during the 475-year and 2475-year events. As such, damage is expected at the girder and deck ends through pounding into the abutment backwall. The abutment backwall fails for both the 475-year and 2475-year longitudinal demands. However, it should be noted that a longitudinal load path remains through engaging the backfill at each abutment end. The failure of the backwall in the longitudinal direction limits the imposed demand on the West Abutment columns. Therefore, the columns remain elastic in the longitudinal direction.

The backfill resistance is reported as around 1000 kN (for the first 1.2m) by WSP. The root cause of low capacity of the backfill is reported to be due to the sub-standard soil and inadequate compaction. This leads to a large displacement (around 900 mm) in the longitudinal direction which leads to unseating of the girders at the abutments.

In the transverse direction, the demands at the West Abutment columns are limited to the interface shear capacity of the shear key (1500 kN). However, in this direction the North Column (Short Column) flexural capacity is exceeded at its base before reaching the capacity of the shear key.

The factored capacity for the rock anchors at Bent #2 and Bent #3 is 850 kN per anchor. To assess the order of failure modes, the anchors are modelled as elastic perfectly plastic elements. First mode of failure on Bent#3 was rocking due to the anchors yielding. However, as the Bent#2 is embedded into soil for its first 4 to 6 meters (varies along the length of the bent wall), rocking is not expected at the bent and thus, the shear key will become the first mode of failure.

The shear capacity of the wall is 4300 kN is larger than the shear capacity of the two shear keys at each bent (1500 kN shear capacity for each shear key).

<u>02655 Colquitz River No. 2</u>

When experiencing seismic acceleration, demands are transferred from the concrete deck through shear studs to the girders. Diaphragms transfer transverse loads from girders into shear keys at the abutments and anchor bolts at the piers. Longitudinally the loads are transferred into the embankment soil through the abutment diaphragms, to the abutment foundation through steel bumper brackets. At the pier, the anchor bolts that pin the bearings to the pier cap fail during both 475-year and 2,475-year seismic events, and thus alleviate load (from the superstructure mass) to the columns and foundations. As such, the longitudinal seismic loads are resisted by the abutments only, with load transfer through the existing bumper brackets into the abutment. Transversely, loads in the abutments are transferred to the soil through the pipe piles. Loads in the pier are transferred from the single column, into the pile cap, piles and soil below.

The fundamental period of the structure in the longitudinal and transverse directions are 0.48 and 0.61 seconds, respectively, indicating that the structure is predominantly in the short-period range. The longitudinal displacement of the deck is approximately 95 mm. Transversely, the displacements at the top of the east and west abutments, and central pier are 33 mm, 42 mm, and 104 mm, respectively, for the 2,475-year event.

The transverse pushover curves for the central pier in the south to north and north to south direction are illustrated in Figure 6 and Figure 7, respectively. Comparing the displacement demands obtained from the 2,475-year RSA vs the pushover analysis indicate that the existing column does not hinge. However, the elastic seismic demands at the base of the existing column from the RSA model indicate D/C ratios of 1.25 considering the expected capacity of the column. This discrepancy is due to the RS analysis model not capturing the non-linear behavior of the piles and is more restrained than the Pushover Analysis where the tension capacity of the piles is nil. For the new column, under 2,475-year seismic demands, hinging occurs at the top of the column. The existing pier cap is within the elastic limits under 2,475-year event. The existing pile cap is designed for the overstrength capacity of the column in-order to allow the system to behave elastically. The central pier piles remain elastic under the 2,475-year event considering inertial demands.

The existing piles at the pier exceed the recommended geotechnical capacity for seismic conditions for only the 2,475-year event.

At the abutments, the results conclude that the existing piles exceed the recommended vertical geotechnical capacity for seismic conditions based on the geotechnical assessment completed by WSP. Furthermore, the piles exhibit inelastic behavior for flexure demands under the 2,475-year event but remain elastic under 475-year event. Adopting a force-based design approach for comparison, the response modification factor, R, as per Table 4.17 of S6-19, for vertical and batter steel piles is 5 and 3, respectively. The associated R required for the piles under flexure is well below 3. As such, the piles are expected to have the ability to develop an appropriate level of ductility.

The existing abutment shear keys installed as part of the 1994 seismic retrofit are overstressed by a factor of 1.6 under 2,475-year seismic demands. The longitudinal bumper brackets have sufficient capacity for the 2,475-year event.

Transversely, the base shear demands at the 475-year and 2,475-year displacements exceed the factored capacity of the anchor bolts at the pier, as such, new shear keys would be required at the pier.

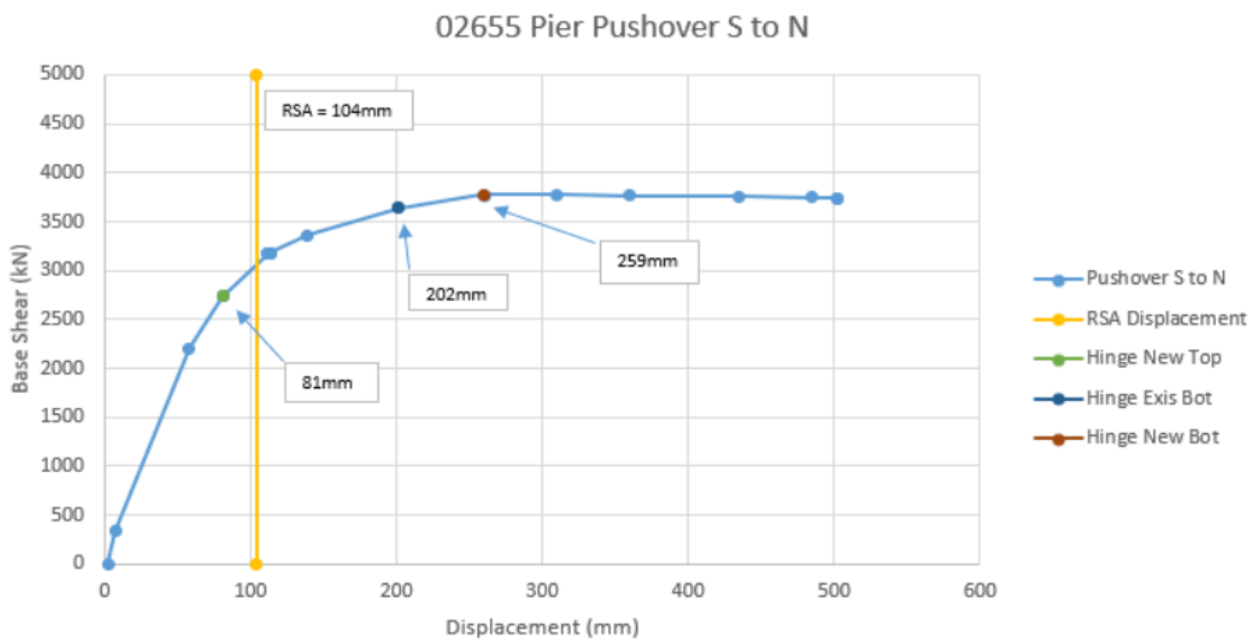


Figure 6. Transverse Pushover South to North at Central Pier 02655 Colquitz Bridge No. 2

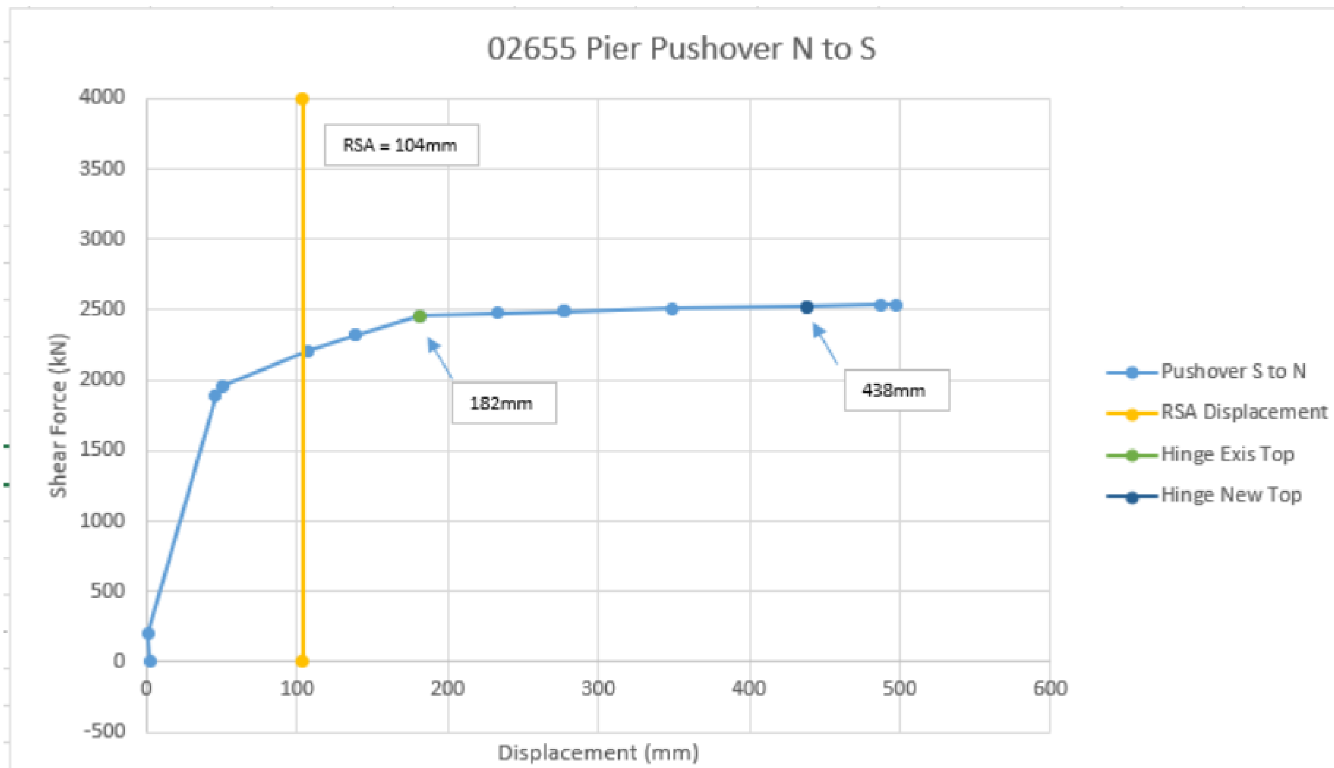


Figure 7. Transverse Pushover North to South at Central Pier 02655 Colquitz Bridge No. 2

Kinematically, WSP's analysis concluded that there is considerable lateral soil movement, specifically that the pile cap displaces between 80-110 mm and 35-45mm at the east and west abutments, respectively, for the 475-year event. For 2,475-year event, the displacement ranges from 160-220 mm and 60-90 mm at the east and west abutments, respectively. This is due to significant shear straining of the soft sensitive clay at the abutments, particularly, at the east abutment where the thickness of the clay is the greatest. Under the 2,475-year seismic event, the piles are anticipated lose their ability to carry axial loads. The pile caps at the abutments will serve as a footing once the piles are out of service.

Seismic Vulnerabilities and Proposed Retrofit

01378 Colquitz River No. 1

In the longitudinal direction, due to the low capacity of the backfill, the bridge longitudinal displacement led to unseating of the girders at abutments. Two actions are taken to improve the longitudinal resistance of the bridge at the embankment. Initially, a new drop wall is added to the end of the approach slab to work as a dead man and as an additional backwall. Additionally, the first 2 meters of the embankment material are replaced with competent compacted soil to improve the longitudinal resistance of the embankment. Bent#2 and the new Bent#1 are also engaged in the seismic load path. To engage the Bent#2, longitudinal bumpers are added to act as restrainers in longitudinal direction.

All these retrofit considerations led to reducing the displacement demand to 400 mm where the column reinforcement in the walls meets the extensive damage criteria.

In the transverse direction, the shear keys at Bent#2 become the first mode of failure as the bent is embedded into the ground, preventing rocking. To prevent the failure of the shear key, it was decided to isolate the columns of this bent in the transverse direction to up to a 4 m depth; it will behave similarly to Bent#3. Considering this retrofit work, the Bent#2 will also rock when the anchors yield.

02655 Colquitz River No. 2

The main structural vulnerabilities consist of a potentially unreliable transverse load path at the pier, some inelastic behavior of the abutment piles, and damage to the pier pile cap.

The geotechnical vulnerabilities include an unreliable load path, considerable lateral soil movement at the embankments, and potential failure of the abutment piles due to considerable lateral soil movement resulting in displacement of the pile caps under the 2475-year event.

The proposed retrofit strategies include the following:

- The pile cap at the pier requires strengthening to accommodate the additional seismic demands in-order to remain capacity protected; and,
- As the anchor bolt capacity at the pier is deficient under both the longitudinal and transverse seismic events, the addition of shear keys is required to produce a reliable load path, transversely. This will include concrete shear keys at the pier and replacing the existing steel k-bracing with steel plate diaphragms to provide the transverse load path.

Kinematically, WSP's analysis concluded that there is considerable lateral soil movement. This is due to significant shear straining of the soft sensitive clay at the abutments, particularly, at the East Abutment where the thickness of the clay is the greatest. Geotechnically, the piles at the abutments will continue to carry axial load. Under the 2475-year seismic event, the piles are anticipated lose their ability to carry axial loads however, the pile caps at the abutments will serve as a footing and the bridge will be able to carry the dead load and emergency vehicles, meeting extensive damage requirements for 2475-year event.

CONCLUSIONS

MMCL was retained by the BC MOTI to prepare a detailed design package for the widening of 01378 Colquitz River Bridge No. 1 and 02655 Colquitz River Bridge No. 2. This short stretch of highway between McKenzie Interchange, one km west, and Tillicum Road, 300m east, is currently the constraint which prevents continuation of adjacent dedicated shoulder bus lane.

The key objective of the project is to extend shoulder bus lane between Tillicum Road and Admirals-McKenzie Interchange in each direction. The work assignment involved geotechnical assessments from consultant WSP, civil design input from consultant Binnie, and seismic retrofit and rehabilitation design, and development of detailed design for both highway and structural works, including drawings, report, and cost estimates.

The detailed design for the 01378 Colquitz River Bridge No. 1 includes widening the bridge symmetrically by approximately 650 mm each side. This approach eliminates the need to provide additional girder lines or widen the substructure. Seismic retrofit items are limited to the addition of shear keys, construction of a new bent, and associated diaphragm modifications at each abutment.

The detailed design the 02655 Colquitz River Bridge No. 2 includes widening the bridge on both sides, including the provision of an additional girder line on the south edge. The widening is asymmetric to limit the additional girder lines to one and maximize the overhang width on the opposite edge of the bridge. The widening also requires the extension of the pier cap, supported on a new column and steel pipe pile, and a structural overlay of the pier foundation. Seismic retrofit items include the pier foundation strengthening as well as new shear keys and diaphragm replacement at the pier and shear modifications at the abutments.

Two types of analysis were performed on this bridge; a nonlinear static pushover and linear dynamic RS analysis. By comparing the results of the analyses, structural vulnerabilities were identified to outline a potential seismic retrofit plan.

The main structural vulnerabilities of the 01378 Colquitz River No. 1 Bridge consist of an unreliable longitudinal load path at the abutments causing unseating of the girders and damage to the existing shear keys at the bents. The proposed retrofit strategies include the following:

- Install a new drop wall to the end of the approach slab to work as a dead man and as an additional backwall.
- Replace the first 2 meters of the embankment material with competent compacted soil to improve the longitudinal resistance of the embankment.
- Install longitudinal bumpers at Bent#2 to act as restrainers in longitudinal direction.
- Isolate the columns of Bent#2 in the transverse direction to up to a 4 m depth as so it behaves similarly to Bent#3 and also rock when the anchors yield.

The main structural vulnerabilities of the 02655 Colquitz River No. 2 Bridge consist of a potentially unreliable transverse load path through the pier anchor bolts, some inelastic behavior of the abutment piles, and damage to the pier pile cap. The geotechnical vulnerabilities include an unreliable load path, considerable lateral soil movement at the embankments, and potential failure of the abutment piles due to considerable lateral soil movement resulting in displacement of the pile caps under the 2,475-year event. Under the 475-year event, at the abutments, it is expected that Minimal Damage performance can be met for structural elements. For the 2,475-year event, the abutment piles will meet extensive damage performance requirements. Under the 475-year and 2475-year events, at the pier, it is expected that Minimal Damage performance can be met for structural elements with the exception of the pile cap, which can meet the required performance levels with a retrofit.

The proposed retrofit strategies include the following:

- The pile cap at the pier requires strengthening to accommodate the additional seismic demands in-order to capacity protect the substructure and,
- As the fixed anchor bolt capacity at the pier is deficient under the transverse seismic events, the addition of shear keys is required to produce a transverse seismic load path. This will include concrete shear keys at the pier and replacing the existing steel k-bracing with steel plate diaphragms to provide the transverse load path.

The results of the evaluation indicate that with the suggested modifications to the structural load path, the retrofitted bridge will satisfy seismic performance requirements consistent with the BC MOTI supplement to S6-19 for existing bridge structures.

ACKNOWLEDGMENTS

WSP provided geotechnical support and completed both the preliminary and detailed geotechnical assessment on both structures. Binnie provided civil support and completed the detailed design on both structures. The authors wish to acknowledge the BC MOTI for giving permission to publish the paper.

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