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SEISMIC DESIGN OF THE PITT RIVER BRIDGE

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ABSTRACT

The new Pitt River Bridge (PRB) was designed and constructed between early 2007 and late 2009 a design-build delivery contract for the Ministry of Transportation of British Columbia in the Metro Vancouver area. The bridge carries Highway 7 over the Pitt River on a seven-lane crossing comprising a steel composite cable-stayed bridge and concrete approach spans.

The Province specified a performance-based design philosophy building on the minimum requirements in the Canadian Highway Bridge Design Code. Requirements included a three-level seismic design criteria for this “lifeline” structure with minimal damage, repairable damage, and no collapse performance levels corresponding to design earthquakes having 10%, 5% and 2% probability of exceedence in 50 years. Requirements included mitigation of liquefaction-induced deformation and other effects. To achieve these requirements, site-specific response spectra and earthquake records were derived. CHBDC minimum seismic inputs, including the Importance Factor, were replaced with the explicit performance-based design approach. This paper discusses the seismic requirements, a selection of key aspects of the seismic design, some innovations achieved, and how the required seismic performance was met and demonstrated.

Introduction

The new Pitt River Bridge is a seven-lane 506 m-long steel bridge comprising a three span, 380 m of composite cable-stayed bridge with four concrete approach spans crossing a relatively deep river (16 m, nominally twice as deep as the nearby Fraser River). It was opened to four lanes of traffic in September, 2010, and all seven lanes in October, 2010. It was designed and built in a design-build delivery model from notice of award between January 2007 and October of 2009. The preliminary design for tendering was performed between June and September of 2006. The bridge site is on a 100 m-deep deposit of sands and soft, compressible soils, the upper layers of which are considered liquefiable. The seismic risk in the region is relatively high, being near the Cascadia Subduction zone, and having a history of significant seismic activity. The Seismic Performance Zone within CHBDC is Zone 4 having a PGA of 0.25 g for a risk of exceedence of 10% in 50 years. The bridge is classified as a Lifeline Structure for seismic design.
The B.C. Ministry of Transportation and Infrastructure provided project-specific seismic design

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and performance requirements that differed significantly from the CHBDC. The structure's seismic performance requirements exceeded the minima in the CHBDC. The site-specific seismic inputs were based on 10%, 5%, and 2% probabilities of exceedence in 50 years. Firm ground uniform hazard spectra were provided, and the design-build team derived local seismic inputs (spectra and time-histories) considering the soil conditions. The seismic hazard and PBD design approach specified we believe were more appropriate for mitigation of ground deformations and for structural design. It is unlikely that a reasonable design could have been achieved for liquefaction-induced ground deformations using CHBDC seismic inputs. Soil-structure interaction is also important for structural design, and a consistent seismic risk and PBD approach was needed. This paper briefly outlines the seismic design requirements, our approach to the seismic design, some discussion on key design challenges, and some conclusions on both PBD and on its application to this project.

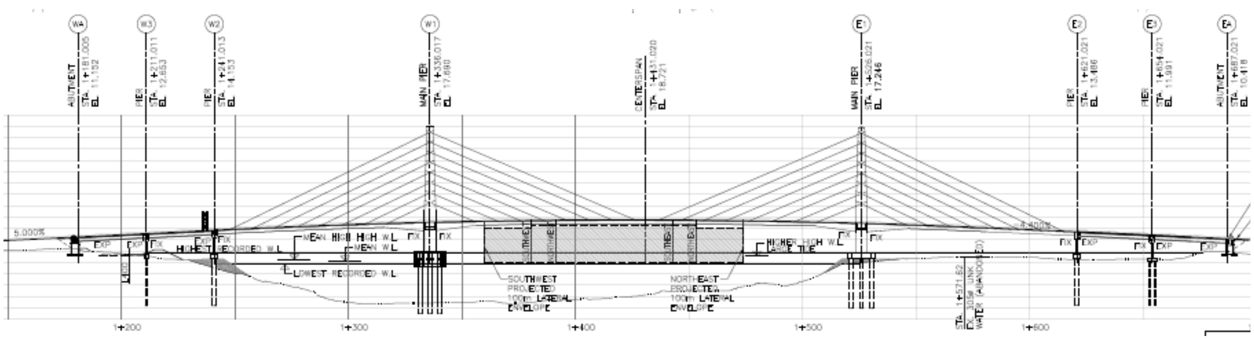


Figure 1: Pitt River Bridge

Figure 2: Bridge (looking south-east) during construction showing substructure arrangements

Bridge Systems

The river bridge features are summarized below. The design of selected components of interest to the seismic performance is described subsequently.

- Seven spans between supports – West abutment (WA), W3, W2 (transition), W1 (river pier in 16 m water depths), E1 (east riverbank pier), E2 (transition pier), E3, east abutment (EA).
- Piers W2, W1, E1 and E2 are supported on 1830 mm-diameter steel piles driven to till. Pier W3 is supported on 914 mm-diameter friction piles. Pier E3 is supported on 610 mm-diameter friction piles driven to length.
- Pier (or pylon) W1 is located in 16 m of water within the river, and features a "perched" pile cap, constructed above low water level. The 1830 mm concrete-filled piles extend through the water to resist lateral loads (seismic and vessel impact) in flexure. Pylon E1 is located on the east river bank.
- Stone columns were installed adjacent to both river banks and under abutments. Additional in-river stone columns were installed on the west side.
- Abutments do not resist seismic longitudinal or transverse loads.
- Voided concrete abutments on spread footings will control long-term settlements.
- Concrete approach spans were used on each side, with two spans each side of 30 m and 33 m. Superstructure comprises precast, pre-stressed concrete I-girders, cast-in-place concrete structural deck on precast concrete 'stay-in-place' forms. The west end approach girders are flare in plan to suit the geometry for the highway exit ramp.
- Approach piers comprise several slender 600 x 1400 reinforced concrete columns.
- Three plane cable-stayed bridge with 95 m – 190 m – 95 m spans.
- Cable stayed bridge spans comprise 50 mm cast-in-place concrete overlay on precast concrete deck panels supported on a grillage of weathering steel floor beams and longitudinal girders.
- Seven traffic lanes with provision for an eighth lane for a rapid bus lane.

Seismic Requirements

The Province included seismic design requirements that exceed the minimum requirements of the Canadian Highway Bridge Design Code (CHBDC). CHBDC is a prescriptive force-based approach using seismic loads corresponding to a 10% chance of exceeding in 50 years (475-year return period). Seismic loads are scaled up by an importance factor of 3.0 for a lifeline bridge, and design forces are reduced by ductility factors, typically 3 to 5 in most bridges. Pier components are detailed to be ductile and to have a clearly defined plastic mechanism for seismic loads. It implicitly provides for limited damage for modest earthquakes, and collapse prevention for a "large" earthquake. The CHBDC commentary indicates that an event having a probability of exceeding 5% in 50 years (1,000-year return period) is considered a "large" event.

For this project, seismic performance objectives and requirements followed an explicit performance-based design approach. The approach is outlined in the project requirements, and supersedes the CHBDC. In British Columbia, the CHBDC is not adopted by as a legal document, and therefore becomes a contractual requirement. The code does recognize that "the Authority" may provide guidance and supplementary requirements. The seismic requirements for the Pitt River Bridge were based on, and expanded on, those specified for the Golden Ears Bridge project, written by the first author of this paper.

The Province specified three design earthquake levels, plus a fourth event (subduction event) for assessment of ground deformations. Performance objectives are specified for each design event, and described in terms of use of the bridge by traffic, and by the level of structural damage. The requirements were outlined in the following documents, in decreasing order of precedence:

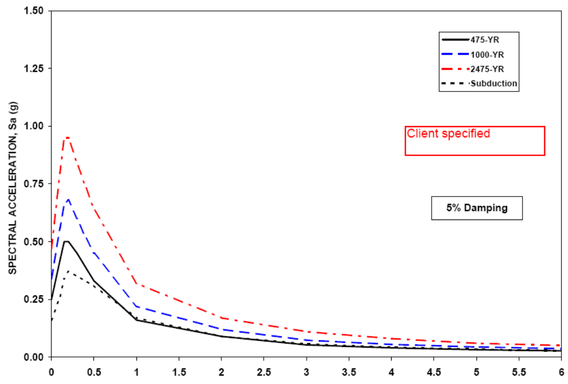
- Project-specific criteria (Schedule A).
- MoT Supplement to CAN/CSA-S6-00 (CHBDC, Canadian Highway Bridge Design Code).
- CAN/CSA-S6-00, Canadian Highway Bridge Design Code
- ATC-32": Improved Seismic Design Criteria for California Bridges; Prov'l Recommendations
- MCEER / ATC-49-I and II, Recommended LRFD Guidelines for the Seismic Design of Highway Bridges Part I: Specifications and Commentary (2003).

The river bridge seismic performance requirements for service after an earthquake and for the associated damage was as tabulated below, with service and damage descriptions following.

Probability of Exceedence in 50 Years (Return Period)	Service Level (1)	Damage Level (2)
Lifeline Structure		
10% (475 years)	Immediate	Minimal
5% (975 years)	Limited	Repairable
2% (2475 years)	Possible Loss of Service	Significant (No Collapse)

Immediate: Full access to all traffic is available following the design earthquake.
Limited: Access for highway loading in a designated lane, albeit recognizing that a post-earthquake inspection would be expected first. Full access to traffic within days.
Significantly Limited: Limited access to emergency traffic within days. Full access after repairs.
Possible loss of service: Access to traffic is not required following the design earthquake.
Minimal Damage: Essentially elastic performance. Permanent structural offets are not present, except that retaining walls and embankments may have up to 100 mm of residual offset.
Repairable Damage (no span or component collapse): Inelastic response was permitted but the structure was to be restorable to its pre-earthquake condition without replacement of primary structural members or requiring complete closure.
Significant Damage: Damage does not cause collapse of any span or part of the structure. Permanent offsets may occur and damage consisting of cracking, yielding, and major spalling of concrete. Extensive repairs would be anticipated.

Site-specific spectra and near-surface accelerograms were derived by the Designers for inputs to the above analyses. Design spectra in CHBDC, and limitations to minimum ordinates (§4.4.7.1 and §4.4.7.3) were not specified or used for the project. Figures 3 through 6 below show the specified firm ground inputs (uniform hazard spectra), and the site-specific spectra derived by the Designers.



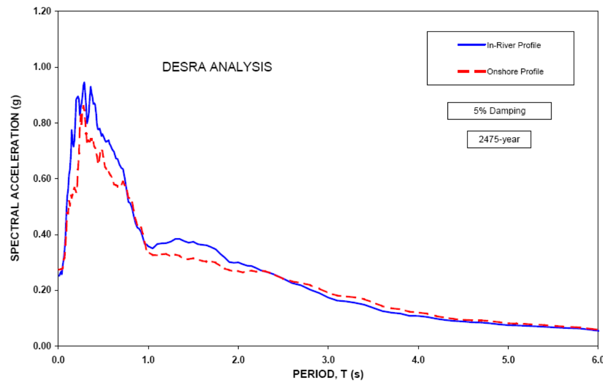
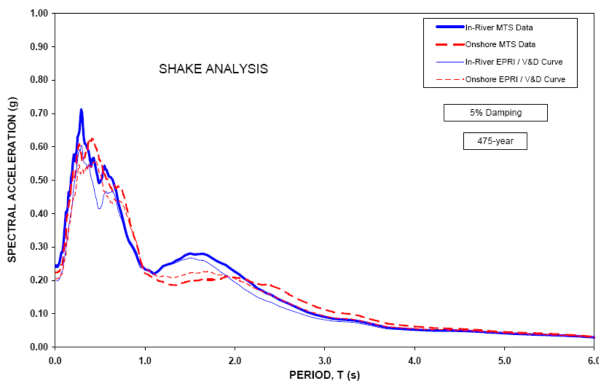


Figure 3. Firm ground spectra

Figure 4. 2% in 50 years site spectra



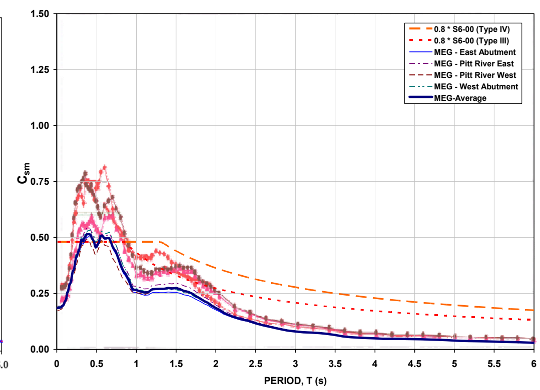


Figure 5. 10% in 50 years site spectra

Figure 6. 10% vs CHBDC minima

Seismic Design

Bridge Articulation:

Important to the lateral load design was in setting the articulation of the bridge. Most cable-stayed bridges adopt expansion joints at or near the end (tie-down) piers. That was also the case for the Green Bridge in Brisbane, which was the basic structural model for the river spans of the Pitt River Bridge. For this site, Pier W1 was designed to be laterally flexible to 16 m of free pile length through the river. Pier E1 was laterally much stiffer. This made the river span lateral load system highly irregular. Torsional effects in the bridge plan view were anticipated, both for elastic response and potentially being exacerbated during inelastic response. In addition, the largest single load applied to the bridge, initially predicted to be as much as 40 MN, was vessel impact directly on pier W1. This load needed to be shared among adjacent supports. Our solution was to design the concrete deck as structurally continuous between abutments, a distance of 506 m. This was the key aspect of the lateral load design of the bridge, including inertial and ground deformation demands, and this strategy was instrumental in allowing us to adopt the extended pile and perched pile cap arrangement at W1. Reference designs, and the writing of the project requirements by the Owner, all assumed massive in-river piers to address vessel impact demands, all constructed in cofferdams. The bridge cost would not have been awarded at all without this solution, coupled with the cable-stayed solution and strong motivation by the Contractor.

One challenge in achieving this arrangement was the design for longitudinal thermal expansion and contraction of the bridge at all supports. Expansion joints (finger joints) are provided only at the two abutments, where they are most accessible for inspection and maintenance. Deformations at interior supports were accommodated by flexural in columns and piles.

Analyses:

A suite of dynamic analyses was specified as a minimum to demonstrate that the performance targets would be met, as tabulated below.

Seismic Ground Motion Probability of Exceedence in 50 Years	Analysis Method(s)
10% (475 year return period)	- Elastic Dynamic Analysis
5% (975 year return period)	- Elastic Dynamic Analysis - Inelastic Static Analysis (pushover)
2% (2450 year return period)	- Elastic Dynamic Analysis - Inelastic time History (Response History) Analysis - Inelastic Static Analysis
Cascadia Subduction Zone Event	- Non-linear Ground Response Analyses to assess the Impact of Long-duration Ground Motions on Soil Liquefaction and Foundation Performance

Approaches adopted in the analyses are outlined following.

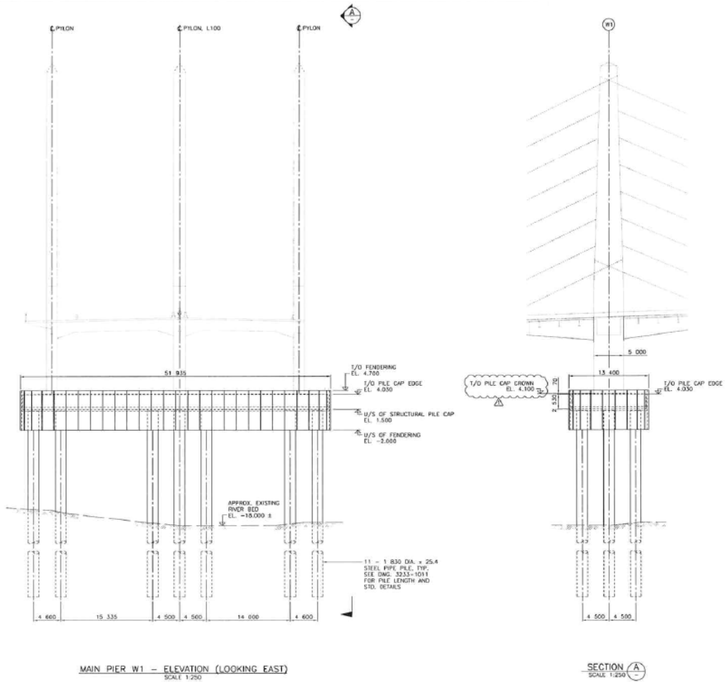
- Soil spectra derived were site-specific spectra for surface motions and neglected the potential benefits of soil-pile-structure interaction. DESRA-derived spectra were adopted for the 2% in 50 year event, SHAKE spectra for other events.
- Ground deformation analyses used FLAC with a locally customized sand constitutive relationship (University of British Columbia "UBC Sand") that accounts for pore-pressure build-up. The design team further customized this model to account for lab and field testing capturing confinement effects at depth (MEG, 2007).
- Response spectra, time-history, and non-linear static analyses were performed for design and design validation. Analyses used SAP2000 and LARSA, using linear and non-linear behaviours in both programs. SAP pushover analyses were checked against manual incrementing of a linear model, with excellent correlation of results.
- Analyses input uniform spectra or records at all supports. The 'on shore' spectral ordinates were used, being higher than for the 'river' spectra owing to the greater depth (16 m) of soil column, and our expectation that most of the seismic energy imparted to the bridge was expected to be transferred through Pier E1.

- Accelerations in time history analyses used near-surface motions. Selected record depths varied for piers with extended piles and piers with pile caps. Acceleration records and displacement records were used, with good agreement from each method.
- All piles were modeled individually in part because there were relatively few, large diameter piles at river span supports. Piles were modeled as composite sections in the upper concrete-filled sections, and as steel shells below. The potential for plastic hinging at the top, gapped pile-to-pilecap connection, and lower down throughout composite and non-composite sections, was assessed.
- Softened local elements were included at the pile-pile cap interfaces. The value of this refinement is debateable, but was helpful in subsequent push-over analyses.
- Distributed springs were modeled along each pile. Spring constants used secant values, consistent with deformation demands and derived from non-linear curves. Independent analyses were performed using 6x6 foundation stiffness matrices.
- Non-linear static (push-over) analyses were performed in both directions of each pier, with separate analyses to assess liquefaction-induced soil pressures on piles. Soil pressures were developed interactively by geotechnical and structural engineers.

In most substructure elements the demands were modest compared to the capacities. RSA, time history and inelastic static analyses combined to provide a good representation of the expected seismic behaviour of the bridge system, and are believed to be sufficient and appropriate for this irregular lifeline bridge on challenging soils.

W1 and E1 Foundation and Pylon Design:

Figure 7 illustrates the structural arrangement of the river pier, showing the free pile lengths through 16 m of river depth, the perched pile cap, and the concrete pylon supported. Figure 8 shows the pile to pile-cap connection detail at Pier E1, which was located at grade on the east river bank. While the corresponding detail at Pier W1 is similar, that pier had additional challenges arising from detailing for precast concrete stay-in-place panels to support the wet concrete of the W1 pile cap, and from the need to design for local bow impacts of vessels (laden barges) on the steel pile shells.



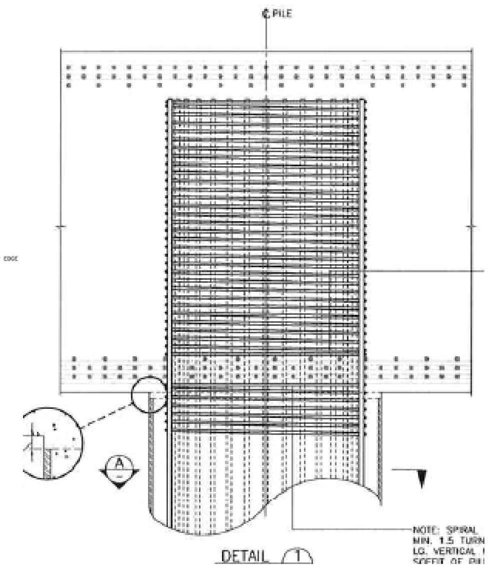


Figure 7. W1 Pier Arrangement

Figure 8. E1 Pile / Pile Cap Connection

Pier W1 has 11 – 1830 diameter steel pipe piles, concrete filled in the upper levels where potential plastic hinging would otherwise occur. The plastic hinges are designed to be at the tops of the piles at the pile to pile cap interface. A gap was provided between the pile shells and pile cap, similar to those shown by testing for steel jacket retrofits (Priestley and Calvi, 1996), to ensure that potential hinging would occur at that location, allowing adjacent members to be designed as capacity protected. Attention was paid to the design, detailing and construction to ensure the transfer of gravity and other vertical loads from the reinforced concrete core into the steel pile shell. While the factored loads are limited to 0.6 P_u of the pile, P_u was near to the yield capacity of the pile shells, requiring that very large loads be transferred from the concrete to the shells.

As described previously, resistance of the lateral loads applied to the W1 pier were critical to the economic design of the bridge, and the overall seismic performance. For seismic loads, ground motions will be imparted to the bridge both upwards through W1, and also through E1 and approach piers, through the deck and down through W1. Had a solid concrete pier been used, the concrete weight to be supported by piles would have been much larger. Axial loads from overturning of such a pier would also have been much larger than will occur with our Pier W1 arrangement. Vessel impact loads must also be resisted at W1 and be transferred up through the deck and down through other piers. Under all lateral load cases, the associated axial loads in the W1 piles are small, which contributed to the overall economy of the seismic design of the bridge.

W2 and E2 Piers and Foundation Design:

The detailing of approach piers was key to the seismic design and the articulation of the entire bridge. Several different arrangements and pier types were considered to address the various levels of seismic performance, including consideration of the expected global elastic and inelastic global bridge response. Other demands related to service loads, secondary deformations at deck level from longitudinal pier deformations, and the need for structural continuity between abutments were critical. Figure 9 illustrates the typical arrangement adopted for the four approach piers. Figure 10 shows the reinforcing for the W2 columns, which was typical for the four approach piers. Note the detailing of the cross-ties, that the tie spacing increases in the middle section of the column, and the straight bar extensions into the beam-column joints. The latter was demonstrated as acceptable owing to the available depth of the beam, and through explicit design of the beam-column joint (Priestley and Calvi, 1996).

Figure 9 illustrates that the load paths for gravity and for loads were unconventional. Vierendeel frame action was relied on in both cases. Columns were 600 x 1400 mm, and provided the necessary longitudinal flexibility to accommodate thermal, creep and shrinkage deformations. Reinforcing ratios up to 2.5% were necessary. Transverse push-over analyses were used in combination with global analyses to determine axial and flexural (longitudinal and transverse) demands in columns. It was recognized that significant column plastic hinging under seismic demands would change the lengths of the columns, and in turn column axial loads and capacities. This effect would vary for each column owing to the different support stiffnesses near and away from the three piles - and therefore the shear demands for which capacity design would be necessary. Column elongation effects are typically not accounted for in either static or dynamic non-linear analyses, as such constitutive relationships are rarely implemented. In this case the transverse ductility demands were estimated to be less than two, and therefore column elongation during post-elastic response was assumed to be manageable. Columns between piles have less axial load and therefore hinge earlier, which implies greater elongation and increased axial load, therefore mitigating the initial effects. Sensitivity analyses on flexural and shear column capacities and demands (capacity design) were undertaken to add confidence to the design.

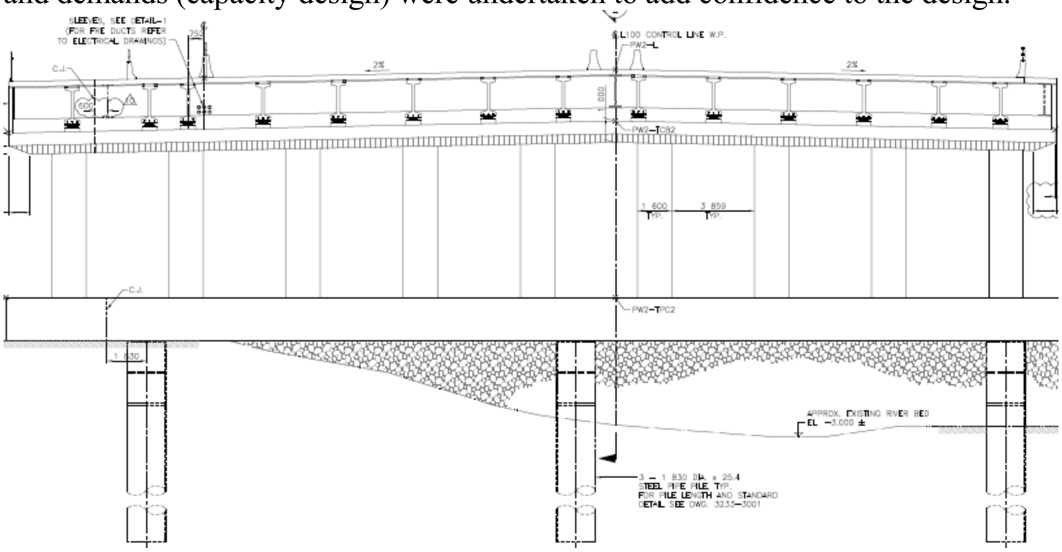


Figure 9. W2 pier arrangement

Figure 10. W2 pier arrangement

Figure 11. Column damage achievable

A related issue was that of spalling of cover concrete in columns under longitudinal demands from the 10% in 50 year event. Performance requirements specified essentially elastic response. For the columns located nearest to piles, with the highest axial stresses from gravity loads, spalling of concrete at the upper plastic hinges was assessed. The contract required that any deviation from specified requirements be identified and a variance requested. This was done and the Ministry granted the variance; one of the very few variances accepted for the entire project. Our justification included an assessment of the longitudinal seismic performance under all three design events, the confinement and flexural reinforcing details provided, a discussion of damage and repair, and evidence that similar columns (for example as seen following the Northridge Earthquake in Figure 11), having less confining steel, have performed well in past earthquakes.

Conclusions

A sampling of the seismic design challenges and solutions has been presented for the Pitt River Bridge. Selected conclusions regarding the seismic design of important bridges arising from this project include:

- The performance-based design approach was appropriate, and a clarified framework for inclusion of PBD in the CHBDC for important bridges, including the use of site-specific demands and removal of prescriptive minimum values, would be an important addition to this code.
- The specified seismic performance requirements for the bridge were, we believe, fully met or exceeded for this design-build delivery project.
- There was significant financial risk inherent in the design-build delivery model for the seismic design of this project for both the DB Contractor and the design team. The contract price was fixed, as is normal in design-build, based on a minor level of preliminary design. Detailed design met the contract requirements, while optimizing construction cost for the Contractor. Tensions between those competing factors required care, experience and considerable design effort following contract award.
- Explicit seismic design for components in which capacity design may not be economically achievable, or where post-seismic repairs may be difficult, is recognized in the CHBDC, and

these provisions were important for this project.

- Central to the successful seismic design was the adoption of a robust and conceptually well thought out structural system. Follow-up with careful and time-consuming detailed design and detailing was essential.
- Economically addressing the seismic and construction constraints of this project contributed to the eventual success of this project. The global structural system, and the seismic design and detailing of W1 and of approach piers and foundations, was critical. The adoption of a continuous deck system between abutments suited both the lateral load design and the design for long term durability of the bridge, and contributed to the success of this project.

Acknowledgements

Peter Kiewit Sons Ltd was the Design-Build Contractor, and we gratefully acknowledge their vision and expertise in the successful design and construction of this project. MMM Group is the Prime Consultant. Associated Engineering (AE) and International Bridge Technology were the river bridge designers (foundations/lateral load design/approaches/independent checking and river span superstructure designers, respectively). Thurber Engineering Ltd and Marine Geoscience Group (MEG) provided geotechnical and geotechnical seismic design. Charles Redfield and Richard Nutt of NRV provided independent reviews to AE, and their insightful contributions were appreciated. Sharlie Huffman, P.Eng. and Don Gillespie, P.Eng. of the Ministry of Transportation worked with the DB Team throughout design, and their support is also acknowledged.

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