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Seismic Performance Evaluation of Previously Retrofitted Oak Street Bridge

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ABSTRACT

The Oak Street Bridge, which connects the cities of Richmond and Vancouver over the north arm of the Fraser River, has undergone comprehensive seismic retrofits over five phases between 1993 and 2004, focusing on life-safety aspects for a 1 in 475-year seismic event. Over the two decades since the retrofits, the state of practice for bridge seismic design and analysis has advanced significantly, particularly in terms of seismic hazard levels, design methodology and performance requirements. The main objective of this evaluation was to examine seismic performance of the previously retrofitted bridge under the 2015 National Building Code of Canada seismic hazards, and to compare its performance to CHBDC S6-14 requirements for a lifeline bridge. This 1840m long, 4-lane highway bridge consists of three distinct segments: (a) South Approach (49 spans plus a 9-span off-ramp); (b) River Spans (8 spans); and (c) North Approach (27 spans plus a 7-span off-ramp). The soil conditions along the alignment vary from competent glacial till at the north end to more than 65m thick Fraser River deposits along the South Approach. Consequently, the existing foundations vary from shallow footings to deep foundations with different configurations and pile types. Due to the large number of combinations of foundation types, soil conditions, retrofitted structure types and seismic demands, representative 4-span continuous bridge segments were modelled to extrapolate performance for North and South approaches, and the 3-span continuous river crossing was the focus of the Main Span evaluation. Elastic dynamic analyses and inelastic static pushover analyses were performed for the structural assessment, which also accounted for soil-structural interactions, kinematic ground displacements, and post-seismic differential foundation settlements. The results indicated that the existing South Approach, North Approach and Main River Spans can achieve Service Disruption, Service Limited and Life Safety performance levels, respectively, for 1 in 2475-year seismic event, and Service Disruptions, Immediate Service, and Service Limited performance levels, respectively, for 1 in 975-year event.

Keywords: Bridges, Seismic Retrofit, Seismic Assessment, Seismic Analysis, Liquefaction

INTRODUCTION

The Oak Street Bridge first opened to traffic in 1957 and carries BC Highway 99 between the Cities of Richmond and Vancouver across the North Arm of Fraser River. The bridge has previously undergone comprehensive seismic retrofits over five phases between 1993 and 2004. The seismic retrofit works were completed in concert with BC Ministry of Transportation's implementation of designated Disaster Response Routes (DRRs) and Economic Sustainability Route (ESRs), which prioritized the seismic upgrade of regional routes and bridges that are vital to emergency response and post-disaster recovery. In these previous seismic retrofits, the focus was to minimize the risk of structural collapse and to minimize injuries and loss of life. As such, the earlier seismic retrofit criteria were based on a target performance of life safety/collapse prevention level for a 1 in 475-year design earthquake.

Along with the progress of seismic engineering practice, BC's earthquake emergency management framework has also made significant changes over the years. In 2015, the Province released the BC Earthquake Immediate Response Plan (BC-IRP), which was later updated in 2022 [1]. This identified Critical and Key Routes that will facilitate the flow of resources to impacted areas after a major earthquake. The BC-IRP, along with the corresponding Transportation Immediate Response Plan (TIRP), set-out updated functional expectations for the regional transportation networks in a post-earthquake scenario. With this background, a high-level seismic evaluation of Oak Street Bridge was carried out to account for the changes in seismic engineering practice, updates in codes and seismic hazards, and the current functional expectations on this critical regional infrastructure. The objective of this seismic evaluation was to conduct a high-level seismic assessment to better understand the performance of this previously retrofitted bridge under the contemporary evaluation methodology and performance criteria set out in the Canadian Highway Bridge Design Code [2] and subjected to the 975 and 2475-year return period hazards from the 2015 National Building Code of Canada [3].

The Oak Street Bridge is 1840m long and consists of three distinct segments. See Figure 1 below.

- The South Approach features concrete deck-on-girder superstructure with concrete wall-type and portal-frame piers supported on timber piles. The segment totals 49 spans along the main bridge alignment with an additional 9 spans for the south off-ramp.
- The River Spans consist of a 3-span continuous river crossing and a 5-span approach. The river spans feature steel plate girders with composite concrete deck. The river crossing is supported on large wall-type piers and the steel approach spans are supported on two-column portal-bents.
- The North Approach consists of 27 spans along the main bridge alignment with a 7-span north off-ramp. The segment features similar superstructure and substructure components as the south approach. Shallow foundations are used at most piers, with 8 piers close to the river channel retrofitted using battered H-piles.

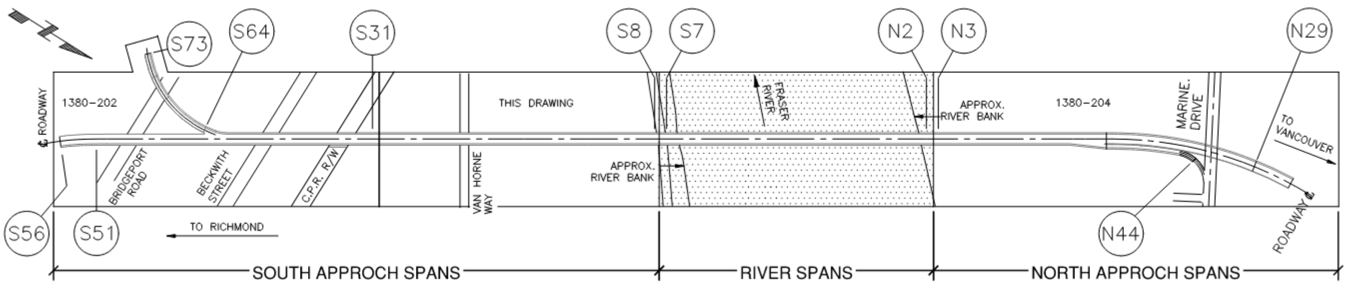


Figure 1 Oak Street Bridge Location and Segments

PREVIOUS RETROFITS

A brief description of previous retrofits is given below:

Foundation Retrofitting and Ground Improvement

- **Footing Overlay:** This was undertaken in Phases 1 and 3 to address strength and stiffness deficiencies in spread footings and pile caps. In Phase 1, concrete footing/pile cap overlay was undertaken between Bents S8 and S51, while Bent N3 and Bents N12 to N27 were retrofitted during Phase 3. The typical footing received a 762mm (30 inch) concrete overlay with "candy-cane" dowels into the original footing. The stirrups around the column and the vertical dowels into the existing footing.

- **Supplementary H-Piles with New Cap and Tie Beams**: From Bent N4 to N11, supplementary battered H-piles were driven around the existing piled foundation. The new piles were tied to the original bent columns via a new cast-in-place concrete pile cap which was located above the original foundation. A concrete tie beam connected the two pile caps (one at each column). Figure 2 shows the typical retrofitting details of a pile cap. The intent of this retrofit was to provide additional resistance against kinematic and inertial loading demands at Bents N4 to N11. The new H-piles and cap bypass the existing foundation and are designed to carry the full gravity and lateral loads of the bent [4].

- **Ground Densification and Seismic Drains:** Phase 5 works included ground improvement using stone columns around the south abutment (at Bent S55 and S56), between Bents S71 and S73 and at the steel approach spans (from Bent S2 to S7). Depth of ground densification range from 12m to 19m (40ft to 63ft). As it was difficult to implement ground densification around every bent, seismic drains were installed at other bents in the south approach – including the steel approach spans – from Bent S2 to S56. The implemented ground improvement scheme was limited in scope compared to that proposed along the entire south approach, in the river, and up the crest of the north bank. The prohibitively high cost of the full program and improved performance by alternative structural retrofits prompted to implement a scaled-down ground improvement program.

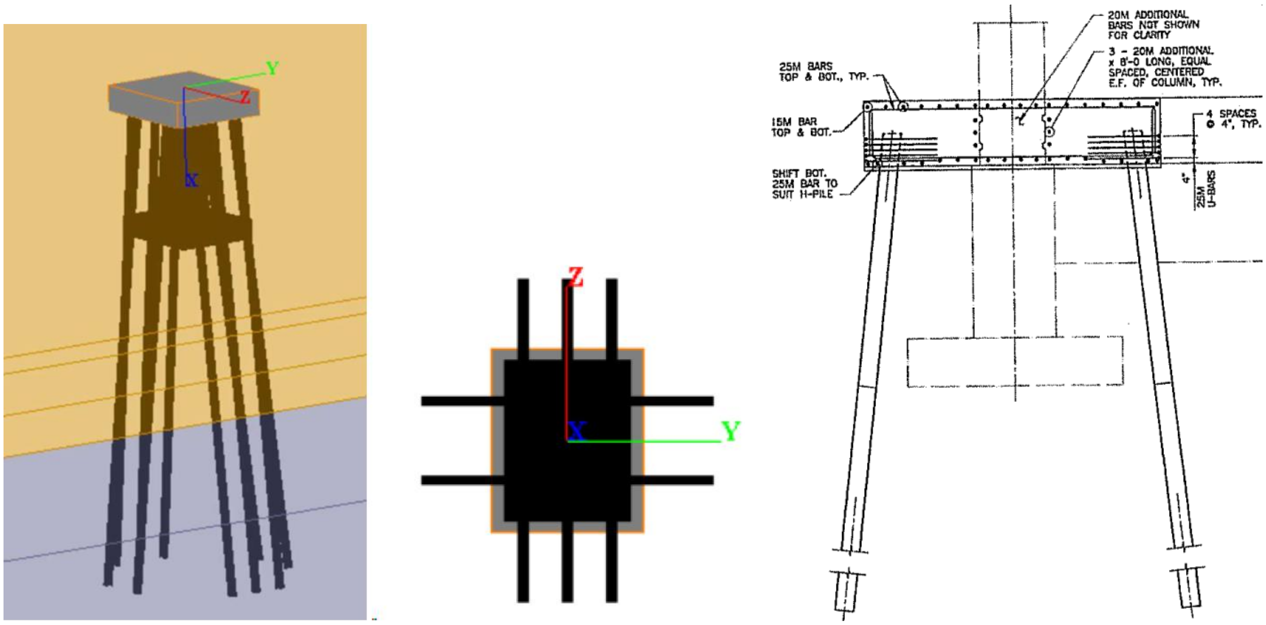


Figure 2 An example of existing footing retrofitted using batter piles and corresponding GROUP model used for analysis.

Substructure Retrofitting

- **Column Jackets:** steel jackets were installed on Bents S31 to S39 during Phase 1. The jacket uses a 1700mm (67-inch) diameter, 9.5mm (3/8-inch) thick steel circular pipe installed in two halves. Concrete infill was poured between the original column and the jacket. The jacketed portion extended full height of the column, from the base of the cap haunch to 50mm (2 inches) above the footing face. The gap with the footing is included to control the location for the onset of rotational hinging in the column. Figure 3 shows column jacket retrofit at a typical bent. The column jackets address deficiencies in poor column reinforcing details in the column to footing and column to cap connections – including short laps and insufficient shear and confining steel. Column jacketing was not applied to bents north of S31 since bents become taller towards the river crossing and did not attract as high moment demands [4].

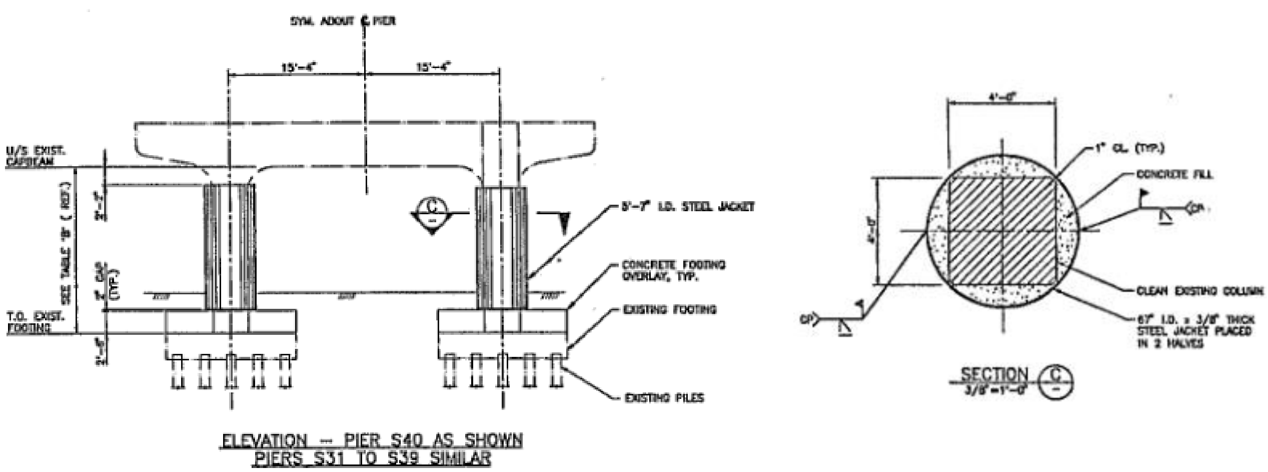


Figure 3 Typical Column Jacket Retrofit

- **Infill shear walls:** Infill walls were constructed at Bents S41 to S51 during Phase 1 and at Bents N18 to N27 during Phase 3. The concrete shear walls are 300mm (12 inches) thick and extend the full height of column from cap beam soffit to top of original footing. Two layers of 15M reinforcing matts were used in the shear walls, anchoring 300mm (12 inches) into existing elements. Figure 4 shows the elevation, cross-section, and reinforcing details of a typical infill shear wall. Infill shear walls address the same deficiency as column jackets for the much shorter bents at the south end of the structure. The shear walls provide a robust load path and produce a stiffer response, where the bent is expected to rock between the two column footings and experience additional damping via plunging and pull-out of the timber piles [4].

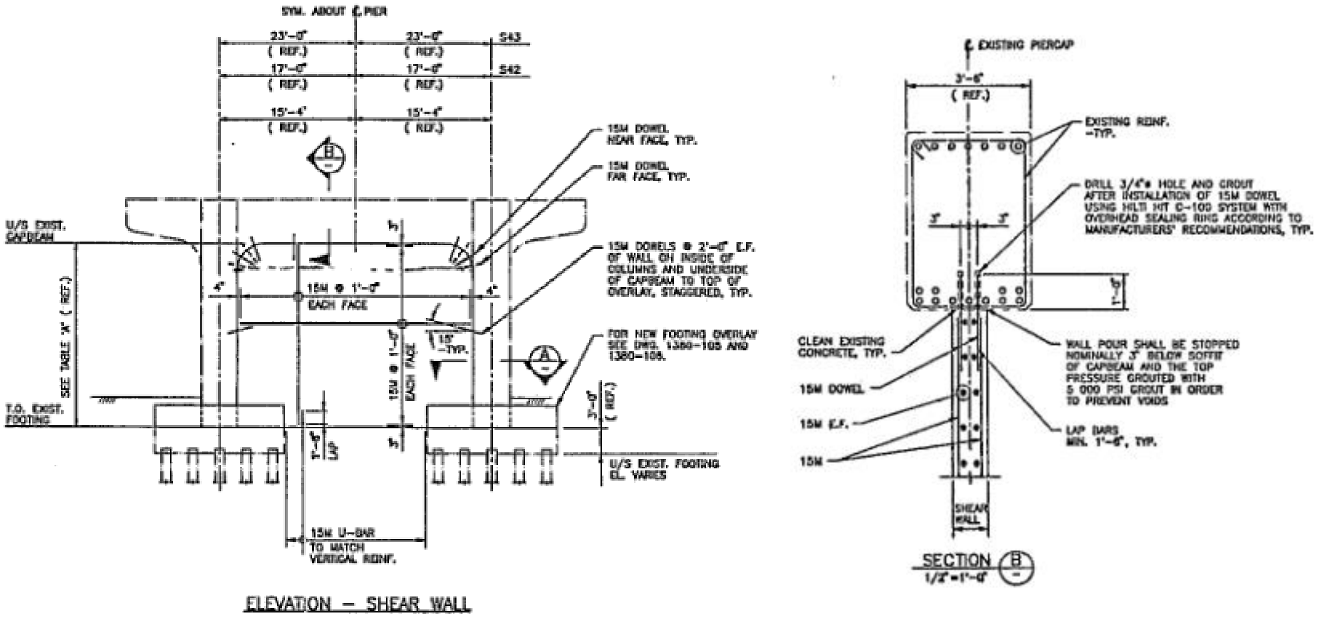


Figure 4 Typical Shear Wall Retrofit

- **Cap Beam Post-Tensioning:** vertical and horizontal cap beam post-tensioning were applied to nearly all South Approach Bents (S51 to S8) and North Approach Bents from N3 to N17. As shown in Figure 5 below, horizontal post-tensioning typically involves a single bonded strand installed by coring through the cap beam slightly above its centroid. Vertical post-tensioning uses Dywidag [5] thread rods installed along the cap beam, typically integrated with the anchorage for chevron braces. Cap beam post-tensioning addresses the issue of poor reinforcing details in the bent caps – including early termination of longitudinal reinforcing, inadequate shear reinforcing, and limited beam-column joint reinforcing. These deficiencies were confirmed by the UBC scaled model testing, which also verified the deformation performance of the retrofitted bents [6].

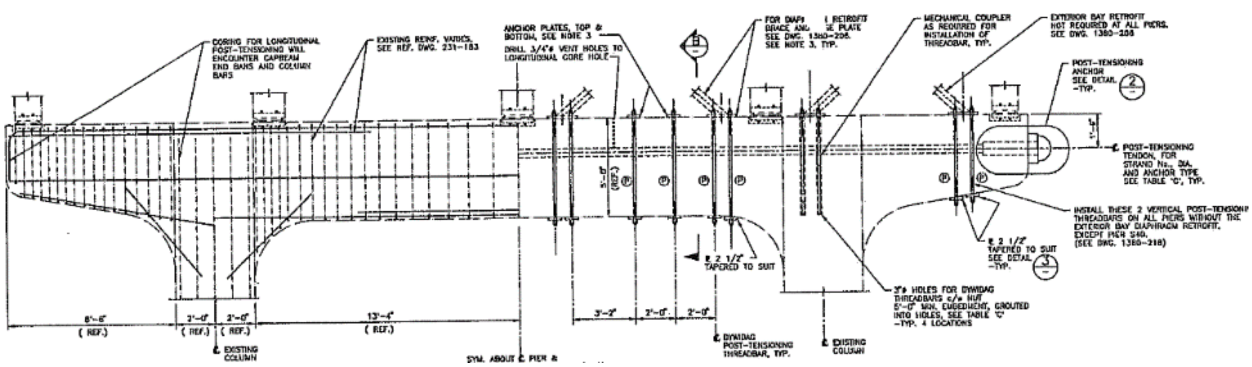


Figure 5 Typical Cap Beam Post-Tensioning

- **Chevron Diaphragm Braces, Shear Keys, Longitudinal Restraints, and Bearing Replacement:** During the first 4 phases of the original retrofits, chevron diaphragm braces, concrete shear key corbels, and longitudinal tie-rods were installed at the majority of north and south approach piers, including exit ramp piers to ensure adequate superstructure-to-substructure load path. Bearing replacement was done at the river spans to a similar effect.
- **River Pier Vertical Post-Tensioning**: Vertical post-tensioning of the river piers S1 and N1 involved coring approximately 16m (55 feet) through the shaft of the concrete caisson into the tremie footing, casting the infill concrete at the top of the caisson shaft, anchoring the thread bars by grouting the bottom 3.6m (12 feet), post-tensioning the 24 thread bars. As the original reinforcing in the pier shaft is deficient, this retrofit allows robust transfer of column longitudinal moment into the pier shaft and subsequently into the tremie foundation. Particularly in a rocking scenario, the vertical post-tensioning ensures the integrity of the pier during uplift [4].

Superstructure Retrofitting

- **Girder Fibre-Wrapping:** While some ground densifications were implemented near the south abutment and along the steel approach spans, relatively large displacements and differential settlements are still expected in the South Approach Bents. Strengthening of the concrete girders, which were deficient in shear capacity, were needed to resist the seismic forces caused by differential movements of the bents, particularly large shear forces induced by differential settlements. Glass-fibre reinforced polymer (GFRP) was selected as it is more flexible and more able to accommodate induced deformation without a brittle failure compared to the carbon fibre alternative. GFRP wrapping also provided cost savings relatively to ground improvement. Completed in 2002, GFRP wrapping was applied to all continuous concrete girders on the South Approach from Bent S8 to S53. Figure 6 shows GFRP wrapping arrangement on a typical girder.

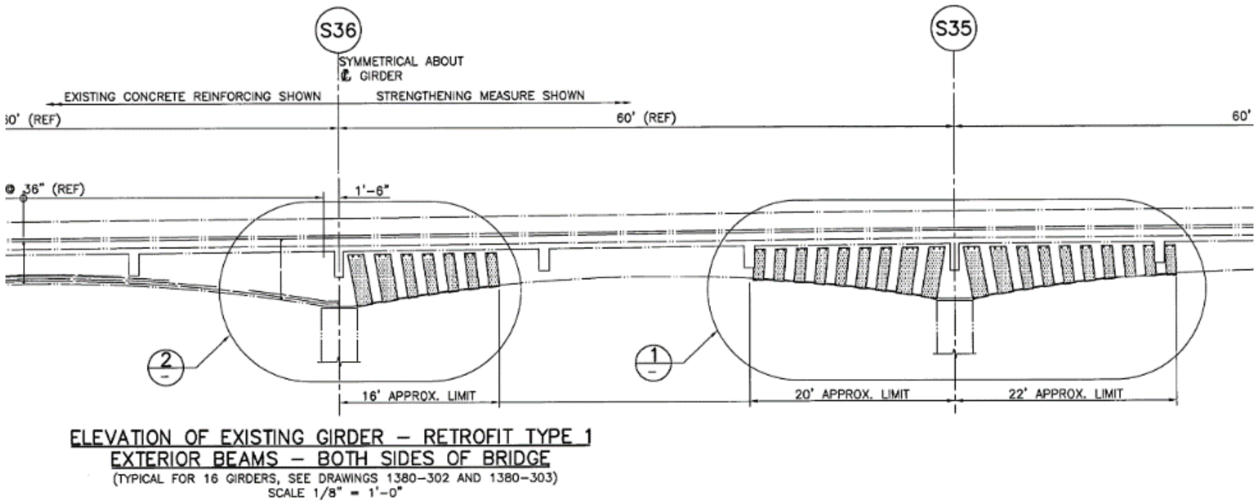


Figure 6 Typical Girder Fibre-Wrapping

COMPARISON OF SEISMIC HAZARD AND DESIGN CRITERIA

Since the completion of the previous retrofit works, the state of practice for seismic design for new bridges has evolved significantly, particularly in terms of design approach (e.g., change from force-based to performance-based), seismic performance requirements and seismic hazard levels. Seismic retrofit design has been following this same evolution of design philosophy. In 2014, CSA S6-14 [7] made drastic shift to performance-based seismic design that introduced three hazard levels of 475, 975, and 2475 year return periods and adopted different performance levels based on the importance category of the bridge.

Comparison of Evaluation Criteria to Design Criteria of Previous Retrofits

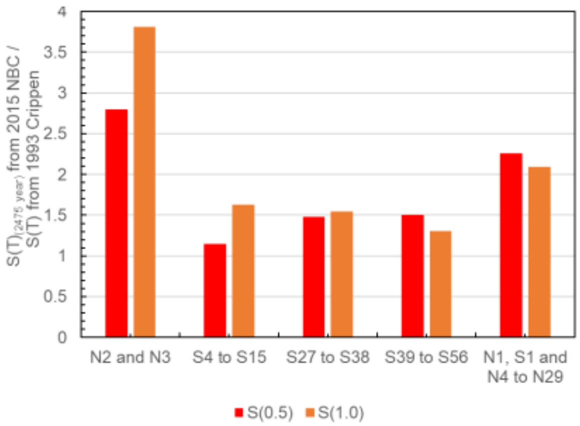
For this evaluation, the project team has developed the updated Seismic Performance Criteria with reference to requirements of CSA S6-14 [7] and the BC Supplement to CSA S6-14 [8] for a lifeline structure. Recognizing that previous retrofits were carried out targeting "Life Safety" for the 1 in 475 year return period event, the structures were evaluated to identify the seismic vulnerabilities for the 1 in 975 and 2475 year return period events. For these return period events, conceptual-level retrofitting options were developed targeting at least "Service Disruption" service level (i.e., Extensive Damage).

Changes to Design Response Spectra

Previous seismic retrofitting works were based on the uniform hazard response spectrum developed by BC Hydro International for the 1 in 475-year earthquake. At the time of the evaluation, fifth-generation seismic hazard model, referenced in the 2015 National Building Code of Canada [3] was in effect while draft values of the sixth-generation seismic hazard model were available for certain locations. The seismic assessment was primarily based on the seismic hazard values included in 2015 NBC for the 1 in 975 and 2475-year seismic events. The design response spectra for each section of the alignment were developed by conducting a series of site-specific ground response analyses. For comparison purposes, the design response spectra developed by Crippen International [9] for the previous retrofitting works was available for comparison.

To limit the discussion, Figure 8 shows only the increase in seismic demand for the 1 in 2475 and 975-year events [3] at periods of 0.5 and 1 second. In general, for the 2475-year event [3], the increase in seismic demand is generally expected to be about

1.5 to 2 times compared to the Crippen International [9] values. However, this increase at Piers N2 and N3 is about 2.7 to 3.8 times depending on the period of the structure. This difference is attributed to the use of site-specific response spectra versus firm ground ("till spectra") at these pier locations. For the 975-year event [3], the increase in seismic demand is generally expected to be about 1.0 to 1.5 times compared to the Crippen International [9] values, except at Bents N2 to N11 where the increase is 2.4 to 2.9 times depending on the period. Although not shown below, if 2020 NBC is adopted, the seismic demand is generally expected to increase by about 20 to 50 percent compared to the 2015 NBC [3] for 2475 and 975-year events.



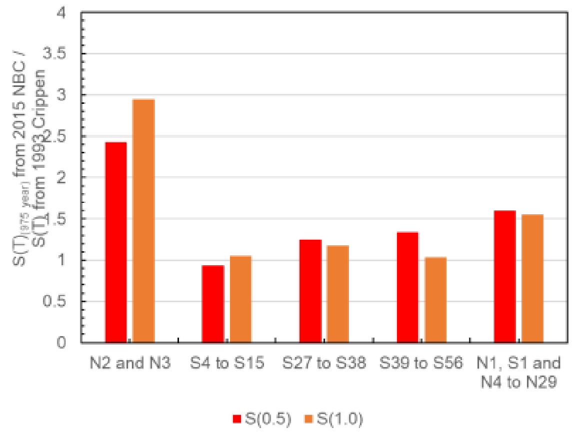


Figure 8. Comparison between design spectral accelerations used in previous retrofitting versus those recommended in the current study based on 2015 NBC

STRUCTURAL EVALUATION

Approach Spans Evaluation

The North and South Approaches of the Oak Street Bridge feature multi-span continuous concrete superstructure segments founded on relatively similar concrete bents. The evaluation methodology of the approach spans utilizes the repetitive characteristic of the structural arrangement and focuses analysis efforts on two representative structure segments – spans between Piers S27 to S31 for the South Approach and spans between Piers N10 to N14 for the North Approach.

Analysis models for each representative segment were created using CSiBridge, and response spectra analyses (RSA) were performed for 975-year and 2475-year earthquakes. Static pushover analyses (ISPA) were used to determine both capacity design demands and to assess structural behavior and damage at each stage of inelastic deformation of the lateral load resisting system by subjecting single-bent models to inertial displacement demands determined from the RSA analyses at the pier cap level. Lump plastic hinges were assigned at column ends to consider potential plastic hinges. Soil-structure interaction is captured using lumped, nonlinear foundation springs representing the combined behavior of pile group and pile cap. For the RSA analyses, equivalent secant stiffnesses of nonlinear foundation springs are derived through iterations.

For the kinematic loading and post-seismic settlement effects, the same representative structural segment models were used to determine the structural damage by imposed ground displacements. To capture the soil-pile interaction effects arising from kinematic loads, Caltrans [10] method was adopted. The method uses softened nonlinear p-y springs appropriate for the kinematic condition and apply discrete springs at regular depths on a single vertical frame element representing the structural properties of the combined pile group.

The RSA analyses were focused on the transverse response of the structure. For longitudinal dynamic response, the varying column lengths between segments would cause adjacent continuous segments to vibrate "out of phase" with one another due to different fundamental longitudinal periods and would lead to pounding at the expansion joints. Since the linear elastic RSA cannot meaningfully evaluate this gap behavior without significantly expanding the model (include the abutments and main spans), the longitudinal displacements were not closely examined.

Main seismic deficiencies identified by the evaluation are listed in the **Error! Reference source not found.** below. These deficiencies apply to both hazard levels: 1 in 975-year and 2475-year.

Main Spans Evaluation

The Main Span structure has three continuous spans supported on four wall-type piers from S2 to N2. Expansion joints are located at S2 and N2 to allow for longitudinal movements of the superstructure. The same seismic analyses using RSA and ISPA were performed for the Main Span evaluation with unique details noted below.

The RSA analysis for Main Spans includes responses of the structure under both longitudinal and transverse earthquake loading. The existing bearings at S2 and N2 were retrofitted by adding restrainers during Phase 2 retrofit, which limits the longitudinal movements of the superstructure. In this evaluation, two boundary conditions were considered to bound the seismic demands. The first scenario is that superstructure supports are restrained longitudinally at S1 and N1 but free to move longitudinally at S2 and N2. In the second scenario, the superstructure supports are restrained longitudinally at all piers. In the transverse direction, the superstructure supports are all restrained by diaphragms.

For the Main Spans, inelastic static pushover analyses were performed to evaluate ductile pier wall deformations in the week direction only. In the strong direction, the pier walls act as a non-ductile shear diaphragm.

The key seismic deficiencies identified for both target hazard levels (975-year and 2475-year) are mainly related to poor reinforcing details at the base of the walls (e.g. plastic hinge zone), including longitudinal lap splices and no transverse ties for confinement. S1 and N1 were retrofitted previously with vertical post-tensioning, however, N2 and S2 have not been retrofitted and are the most vulnerable.

Table 1 Approach and Main Spans Structural Seismic Vulnerabilities

Structural Element	South Approach	Main Span	North Approach
Bent Columns	Deficient hinge zone detailing at the column-foundation connections – namely longitudinal bar splice and confinement from transverse reinforcing. Bent columns deficient in shear under transverse inertial and kinematic loading.	Piers S1 and N1 can maintain life safety during a 2475-year earthquake. There are more uncertainties in Piers S2 and N2 because of insufficient confinement, short rebar lap length at hinge regions, and ductility demand greater than 1.0.	Deficient hinge zone detailing at the column-foundation connections – namely longitudinal bar splice and confinement from transverse reinforcing. Bent columns shear D/C ratio is between 0.95 and 1.0 under transverse earthquake only and could be deficient under 100%EQ (Transverse) +30%EQ (Longitudinal).
Superstructure	Limited superstructure ductility relative to the expected kinematic displacement and post-seismic settlement.	Not applicable	Not applicable

FOUNDATION EVALUATION

The greatest challenge for the foundation evaluation was the variation in soil conditions, extent of liquefaction and its consequences and different types of foundations. To demonstrate the variation in soil stratigraphy along the alignment, a simplified soil stratigraphy is shown in Figure 1, and different soil units are described below:

- **South Approach (Piers S2 to S73):** The south side of Fraser River is underlain by a Silt Crust (Unit 1) of 2.4 m to 5.2 m thick. Firm to soft silt crust was overlain by sand fill at some locations. Above soil unit is underlain by Sand and Silt (Unit 2) with thickness up to 7.3 m. This is underlain by fine to medium Sand (Unit 3) with thicknesses ranging between 9 m and 13.7 m. This is followed by a firm to stiff Silty Clay (Unit 4) with seams of fine sand, where the thickness was interpreted to exceed more than 60 m at the southern end of the alignment. The interpreted peak undrained shear strengths ranged from about 40 kPa at the top of the soil unit, increasing at a rate of 1.5 kPa/m as the depth increases. This is underlain by a very dense Glacial Till (Unit 5).
- **River Channel (Piers N1 and S6):** The river channel was underlain by a loose to compact fine medium sand layer (Units 3 and 6) with a combined thickness ranging from 8 to 10 m, followed by Glacial Till (Unit 5)
- **North Approach (Piers N44 to N2):** The north side is underlain by Fill (Unit 7) that ranges from 0 to 2.1 m, underlain by Peat and Organic Silt (Unit 8), that extends from Pier N25 towards the river. The thickness of this unit increases towards

the river where the maximum thickness reached approximately 9.6 m between Pier N6 and N7. This is underlain by a dense to very dense Sand and Gravel (Unit 9) with a maximum thickness of 15 m near the riverbank, followed by Glacial Till (Unit 5).

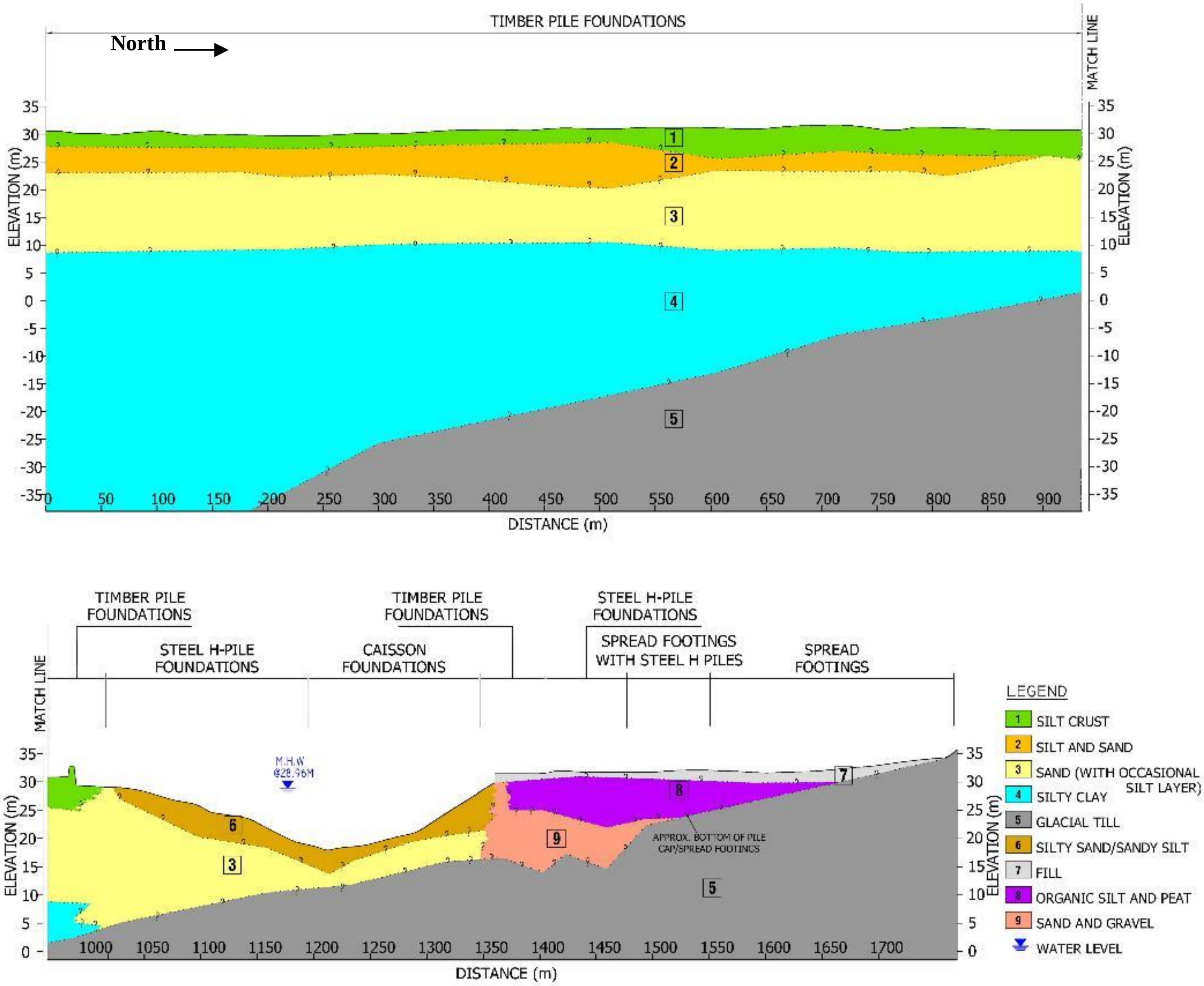


Figure 9. Interpreted soil profile along the alignment

Previous explorations indicated groundwater level vary from 1.5 m to 2.1 m below grade and is expected to fluctuate with the river water level and precipitation levels.

Soil Liquefaction Potential and Ground Deformations

A simplified liquefaction triggering assessment was completed using in situ penetration test results. The results indicated that sand-like units with loose to compact relative densities are classified as high-risk of liquefaction under the 975 and 2475 year return period events except at locations where ground improvement was undertaken. Clay-like soil units encountered at shallower depths were prone to significant cyclic softening in all seismic events. However, the degree and depth of cyclic softening of the deeper silty Clay unit depend on the seismic shaking intensity. Apart from the simplified analysis, nonlinear dynamic analysis was conducted using 2D FLAC 8.0 [11] program to address two key limitations that impacted the outcome of the simplified assessment: (i) to estimate the lateral spreading displacement along the alignment where the existing ground improvement is discontinuous and limited to zones near the bents and (b) to estimate the post-seismic settlement of timber pile foundations at the south side as they are founded above liquefiable soils. Details of these assessments are not discussed in this paper due to page limitations.

Foundation Vulnerabilities: North Approach

For this high-level assessment, the several analyses were performed targeting different foundation types to identify the vulnerabilities unique to these foundations.

At the northern end of the bridge, N12 to N44 piers were supported on spread footings that are founded on dense to very dense glacial till that is not susceptible to liquefaction or cyclic softening. The overturning of the spread footings along the bridge alignment is restrained by the superstructure while the framing action of the bent in the transverse direction will provide the required resistance against overturning. The likelihood of bearing capacity or sliding failure was considered low. No geotechnical vulnerabilities were identified in these foundations.

Bents N8 to N11 were originally supported on spread footings, with the bases of the footings founded on dense sand and gravel layer or glacial till. However, due to concerns related to kinematic and inertial loading, battered H-piles were installed surrounding the existing spread footings during the previous retrofitting works. Piles were installed with a batter of 1H:9V, and new piles were tied to the original bent columns via a new cast-in-place concrete pile cap at the ground surface. The current assessment has not identified any significant lateral spreading displacement at these bent locations. The lateral loads acting on the battered pile group are predominantly transferred to the piles as a compressive or tensile force. The GROUP [12] analysis conducted on these foundations indicated sufficient capacity against axial, flexural and shear failures modes of the pile group. Furthermore, pile cap connection failure was deemed unlikely as the top of the piles in the retrofit design had nelson studs to prevent pullout.

Further south, Bents N4 to N7 were supported on 13 to 16 H-piles per pier. As a part of the seismic retrofitting, additional battered piles (HP10x42) were installed to accommodate anticipated kinematic demands. On average, piles were about 10.5 m long and driven through the dense to very dense sand and gravel into the glacial till, where the driving resistance varied from 140 blows per foot to 200 blows per foot. The likely failure modes are similar to those described for Bents N8 to N11. The upper bound lateral spreading displacement estimated at these piers is about 0.3 m for 1 in 2475 year seismic event. With respect to seismic kinematic loading, GROUP [12] analysis indicated that seismic vulnerability of this group of foundations is low. Furthermore, no vulnerabilities were identified with respect to seismic inertial loading.

Bents N2 and N3 are located near the north riverbank. Pier N2 is supported on a single pile group consisting of 133 timber piles. Each column footing in Pier N3 is supported on 28 timber piles. These piers are underlain by about 4 m of fill, a mixture of peat, sand, silt and clay with organic, over dense to very dense sand and gravel. As a result, timber piles are only 4 m to 5.8 m long and likely driven into dense sand and gravel layer. FLAC and simplified analysis indicated approximately 0.3 m, 0.6 m and 1 m of displacement during 475, 975 and 2475 year events, respectively. One of the seismic deficiencies in this group of foundations is the weak connection between pile and pile cap. Record drawings show timber piles embedded only 150 mm into the pile cap, which is approximately 0.5 times the pile diameter (d_p). A series of lateral load tests conducted by Shama et al. [13] on pile cap connections indicated that if the pile embedment is about 1.5 times d_p or greater, the lateral capacity is expected to be governed by the flexural failure of the pile. For an embedment of 1.0 d_p, the specimen showed uplift as the horizontal force is increased and caused slipping of the pile within the embedment zone. According to the theoretical model developed by Shama et al. [13], the maximum lateral load that can be supported by a pile with an embedment of 0.5 times d_p is less than 10 kN. The structural loads estimated for Pier N2 indicate shear force varying from 66 kN/pile for the 1 in 2475-year event in the transverse direction and 36 kN/pile in the longitudinal direction. According to the theoretical model predictions, these loads are approximately six to four times the load that can be sustained by the connection. Since the actual tests were limited to embedment of 1.0 d_p and 1.5 d_p, there is some uncertainty related to the predictions made for an embedment of 0.5 d_p using the theoretical model.

Foundation Vulnerabilities: Main Span

The main span is supported on two main piers constructed over the river. Bent N1 is supported on a caisson foundation, with the base of the footing founded on glacial till. The caisson is about 6.5 m deep, 9.5 m wide in the longitudinal direction and 18.3 m long in the transverse direction and is supported on two rows of 12 steel H-piles (HP10x42). For the inertial loading, the estimated risk of overturning and bearing failure was estimated to be low. The FLAC analysis [11] has predicted several meters of lateral spreading displacements at this pier location in the longitudinal direction. The maximum bending moment estimated for the kinematic loading was smaller than the yield flexural resistance of the H-pile. Therefore, no seismic vulnerabilities have been identified for this foundation.

Bent S1 was supported on a caisson founded on glacial till although no additional H-piles have been installed as adjacent piers. Structural analysis has predicted considerably large shear forces and overturning moments under inertial loading conditions. In the transverse direction, the estimated foundation eccentricity was greater than 0.4 times the foundation width, which does not meet the requirements in CSA S6-14 [7] for seismic loading. Bearing capacity issues are not anticipated. Additional analysis should be conducted in future design phases to determine the actual risk of overturning after considering potential contributions from the superstructure. With respect to kinematic loading, the estimated ground displacements are relatively small at the middle of the river channel and away from the riverbanks.

At Bent S2, the caisson is supported by 90 H-piles (six rows of 15 H-piles). The average pile length was 7.3 m, and the caisson is 7.3 m wide and 18.9 m long. Ground densification was undertaken to a depth of 12 m. The estimated “free-field” horizontal ground displacement is about 1 m at the ground surface for the 1 in 975 and 2475-year events and becomes negligible at a depth of about 8 m. Therefore, almost the entire load arising from lateral spreading will be acting on the caisson and infill wall. As a result, the LPILE analysis indicated that flexural capacity of the piles is exceeded under kinematic and inertial force combinations.

Foundation Vulnerabilities: South Approach

Similar to Bent S2, Bents S3 to S6 were constructed by excavating the alluvial soils to a depth of 9 m to 12 m below ground surface and installing HP10x42 steel H-piles. Each pier was supported on 60 H-piles. Despite the ground densification carried out during previous retrofitting works, relatively large ground displacements have been predicted for the 975-year and 2475-year seismic events in FLAC analysis. For example, Figure 10 shows the ground displacements estimated for an inslab earthquake that represent the 975-year event. The estimated ground displacements extend below the pile cap level and impact the H-piles. According to LPILE analysis, the flexural capacity is exceeded under kinematic and inertial loading combinations. The likelihood of damage is low for the 1 in 475-year event where the magnitude of ground displacement is smaller and depth is shallower. This is consistent with the intentions of the original ground densification undertaken at this site that targeted a similar seismic return period.

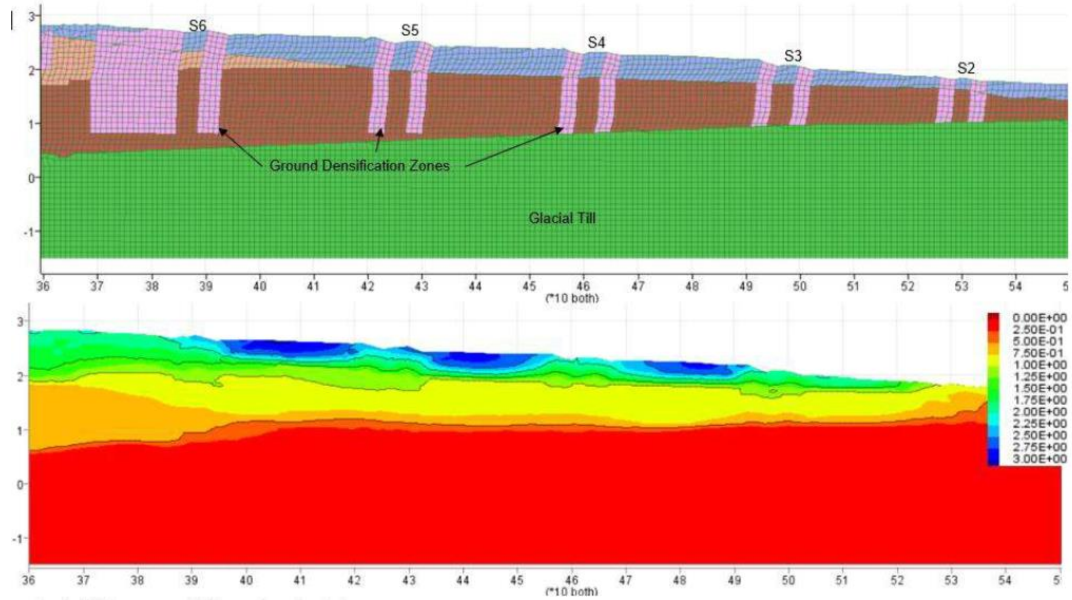


Figure 10: Horizontal ground displacements near S6 to S2 predicted for the 1 in 975 year seismic event (2015 NBC). Displacements are in meters.

Bents S7 to S73 are supported on untreated timber piles, and most of the pile groups consist of either 19 or 25 piles per group. Piles were installed with a very tight spacing of 0.9 m (~three pile diameters); such that liquefaction of soils within the pile group is unlikely to occur. For these bents, two failure modes are considered are likely: (a) the pile to pile cap connection failure similar to the Bents N2 and N3 at the north approach and/or (b) excessive settlement of the bent due to liquefaction and settlement of soils below the timber piles. Latter was considered a difficult task to quantify due to lack of current guidelines to estimate pile group settlement when liquefaction occurs below the pile group. A separate FLAC model was developed in the transverse direction to estimate the settlement of the pile group (Figure 11). The shear demands provided by the structural team for Bent S28 indicate a shear force of about 91 kN/pile for the 1 in 2475-year event which is significantly higher than the capacity predicted by Shama et al. [13] for a pile with a pile cap embedment of 0.5 times d_p. In addition, the post-seismic settlements caused by soil liquefaction below the pile cap can also increase the risk of separation.

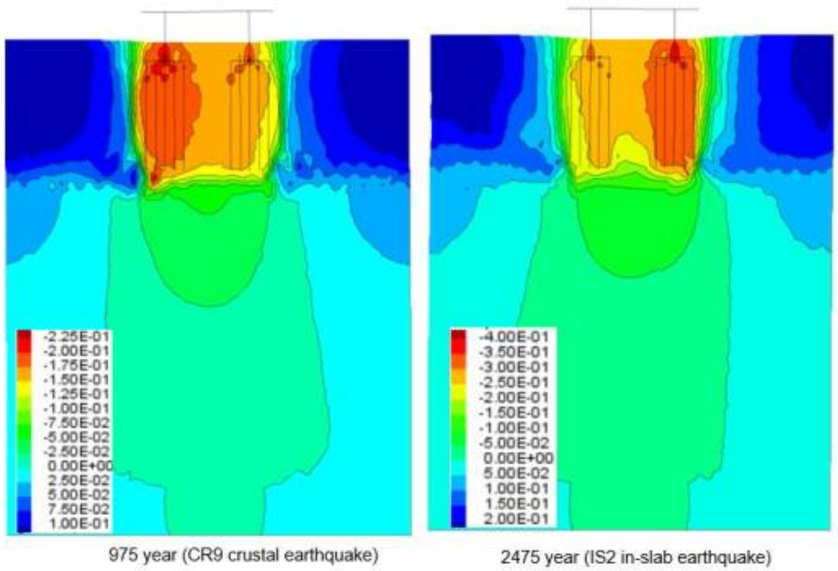


Figure 11: Post-seismic settlements estimated at end of shaking for 1 in 975 and 2475 year events in Bents supported on timber pile groups (2015 NBC)

SUMMARY OF EVALUATION RESULTS

The seismic deficiencies in the foundations are summarized below

Table 2 Summary of Foundation Deficiencies for the 975 and 2475 Year Events (for 2015 NBC)

Bents	Foundation Type	Seismic Deficiency
N12 to N44	Spread footings	No deficiencies
N11 to N7	Shallow footings on H piles. Additional H piles (battered) installed in 1995/1994.	No deficiencies
N4 to N6	H piles with a pile cap including extra battered H piles supported by a second pile cap.	No deficiencies
N2 and N3	Timber piles.	Potential deficiencies related to inertial and kinematic loads (pile-pile cap connection is vulnerable against shear (inertial), uplift and kinematic loads
N1	Caisson with H piles.	No deficiencies
S1	Caisson	Overturning concerns with respect to inertial loading in the transverse direction.
S2	Caisson with additional H piles	No deficiencies related to kinematic loads. Seismic inertial load demand is likely to exceed the flexural capacity of H piles.
S3 to S6	H piles.	Potential deficiencies related to kinematic loading, which may cause flexural failure of H piles. Seismic inertial load demand is likely to exceed the H pile flexural capacity.
S7 to S73	Timber piles	Deficiencies related to both inertial and kinematic loading: (i) weak pile-pile cap connection that is vulnerable against seismic shear loading, uplift, differential settlement of pile group and pile-cap, and kinematic loading; and (ii) differential settlement of the pile group.

After considering the structural deficiencies and geotechnical deficiencies, the overall seismic performance at each hazard level is shown in Table 3.

Table 3. Summary of Achievable Seismic Performances (for 2015 NBC)

Return Period	**South Approach Service Condition / Damage Level**	**North Approach Service Condition / Damage Level[(a)]**	**Main Span Service Condition / Damage Level[(b)]**
1 in 2475	Service Disruption / Extensive Damage	Service Limited / Repairable Damage	Life Safety / Probable Replacement
1 in 975	Service Disruption / Extensive Damage	Immediate Service / Minimal Damage	Service Limited / Repairable Damage

Notes:

(a) Except at Bents N2 and N3 supported on timber piles which will meet the Life-Safety/ Probable replacement only.
(b) A sensitivity study was performed for the Main Span and the worse Damage/Service levels are shown.

Based on the findings of the above seismic vulnerability assessment, several seismic retrofitting options were developed to achieve the desired seismic performance targets. Due to page limitations, the details are not provided in this paper. Additional assessments are recommended to confirm the findings of the high-level assessment completed for this project.

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