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NELSON CANYON BRIDGE SEISMIC RETROFIT

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ABSTRACT: A combined structural and geotechnical assessment was undertaken to quantify the seismic vulnerability of the Nelson Canyon Bridge, a 1956 steel-tower crossing near Horseshoe Bay in West Vancouver that serves as a lifeline because it carries a watermain supplying domestic and fire-flow demand to the surrounding community. The assessment, performed to current CSA S6:19 requirements, found the tall steel pier towers and bracing members overstressed and the pedestal foundations vulnerable to uplift under seismic overturning. Full replacement was ruled out on cost and environmental grounds. The retrofit replaces the reinforced-concrete deck with a lightweight prefabricated timber deck, reducing superstructure mass and the seismic inertial forces transferred to the towers and foundations, and introduces controlled rocking isolation connections at the tower bases. These connections use Triangular Added Damping and Stiffness (TADAS) devices as replaceable structural fuses that limit tension transfer while maintaining compression and shear resistance, complemented by FRP strengthening that confines the tapered pedestals against anchor pullout and concrete breakout. Because the lighter deck also shortens the fundamental period and can raise spectral acceleration, the net benefit was verified using response-spectrum and nonlinear time-history analyses. The results confirm reduced demands on the towers, bracing and foundations, controlled foundation uplift and improved confidence in post-earthquake watermain operability, offering a practical strategy for short- and medium-span lifeline bridges where replacement is hard to justify.

1. INTRODUCTION

Constructed in 1956, the Nelson Canyon Bridge spans Nelson Creek near Horseshoe Bay in West Vancouver, British Columbia. In 1974, a new alignment of Trans-Canada Highway 1 was constructed, rendering the bridge functionally obsolete for vehicular traffic. However, the bridge remains a critical asset due to the watermain carried along its deck, which supplies domestic water and fire-flow demand to the Horseshoe Bay area. As a result, the bridge is classified as a lifeline structure for the District of West Vancouver, and its functionality must be maintained following major seismic events.

The Nelson Canyon Bridge was originally designed without modern seismic provisions, and advanced corrosion of the steel floorbeams and extensive spalling of the concrete deck soffit, identified during close-proximity rope-access inspections, have prompted several replacement and rehabilitation studies over the past decade. Although full replacement alternatives, including single-span arch concepts, were evaluated, a Weighted Decision Matrix assessment favoured targeted rehabilitation as the more cost-effective and environmentally responsible option, avoiding the capital cost of a new crossing and the demolition and hazardous-material handling that replacement would entail directly above the sensitive Nelson Creek corridor, while preserving the existing alignment.

A comprehensive structural assessment of the existing bridge, performed in accordance with current seismic design requirements, revealed significant deficiencies within the substructure, particularly in the tall steel pier towers where several members were found to be overstressed under design-level earthquake loading. To address these vulnerabilities, a targeted retrofit strategy was developed to reduce seismic demands on the existing substructure while minimizing intervention to the bridge's primary structural system. The retrofit incorporates two complementary measures. First, the existing reinforced concrete deck is replaced with a lightweight timber deck, substantially reducing the superstructure mass and consequently the seismic inertial forces transmitted to the towers and foundations. Second, rocking isolation connections are introduced at the interface between the tower legs and the concrete pedestals. These connections are designed to limit uplift force transfer during seismic events, controlling foundation uplift while keeping the anchorage tension in the concrete pedestals low.

Controlled rocking is a well-established seismic protection mechanism, studied since Housner (1963) and since implemented in bridges such as the South Rangitikei Rail Bridge and the north approach of the Lions Gate Bridge, typically paired with supplemental energy dissipation. For this retrofit, Triangular Added Damping and Stiffness (TADAS) devices were selected as ductile structural fuses to control rocking at the tower-leg-to-pedestal connections and limit foundation uplift (Vargas 2006). This paper describes the existing bridge and its vulnerabilities, the retrofit concept and detailing, the analytical procedures used to evaluate it and the key findings.

2. EXISTING STRUCTURE AND SITE CONSTRAINTS

2.1 Bridge and Site

The Nelson Canyon Bridge is a six-span structure constructed in 1956 that carries a District of West Vancouver watermain across Nelson Creek within a steep rock canyon near Horseshoe Bay, British Columbia. It lies on a horizontal curve of 87 m radius with an 8% crossfall and has a total length of approximately 80 m, including a 12 m cast-in-place concrete jump span on the west approach. When the Upper Levels Highway was realigned to the south in 1974 the bridge was closed to regular vehicular traffic, and it has since been retained solely because it carries the watermain that supplies Horseshoe Bay and the surrounding area. This lifeline function classifies the structure as a lifeline bridge under CSA S6:19 and the National Building Code of Canada (NBCC 2020).

(a) (b)

Figure 1: Nelson Canyon Bridge (Existing Structure) a) top view - superstructure b) steel truss pier substructure

Apart from the jump span, the superstructure comprises a 225 mm cast-in-place concrete deck acting compositely with a welded steel transverse floorbeam system, which is in turn carried by two built-up steel longitudinal girder lines. The 10.1 m wide deck is edged by raised cast-in-place concrete curbs and by steel railing panels spanning between cast-in-place concrete posts, giving a combined curb and railing height of 1473 mm above a 305 mm curb. Transverse articulation is provided by deck joints, detailed as a joint filler at the fixed connections and a built-up angle and deck-plate assembly at the expansion joints. The superstructure is supported on two cast-in-place concrete abutments, a cast-in-place concrete pier at the end of the jump span and two tall steel truss pier towers built up from W-sections and lattice members, each founded on pyramid-shaped concrete pedestals cast directly onto the underlying bedrock (Figure 1). Member sizes and material grades were taken from the 1958 as-built drawings.

The bridge sits within a confined, steep-sided rock canyon, with Nelson Creek and its riparian corridor immediately below the structure. This setting is environmentally sensitive: any in-canyon work must protect water quality and aquatic habitat directly beneath the bridge, and because the 1950s steelwork is expected to carry lead-based coatings, any cutting or abrasive blasting of steel would require containment of lead-contaminated debris. These constraints limit construction access and staging and favour retrofit measures that minimize disturbance of both the existing steelwork and the canyon environment.

2.2 Geotechnical and Seismic Setting

The site is one of high seismicity, with crustal, in-slab and Cascadia subduction contributions, and was evaluated to CSA S6:19 as modified by the British Columbia Ministry of Transportation and Transit (BC MOTT) Supplement against a life-safety objective for the 975-year return period (five% probability of exceedance in 50 years) ground motion. Because the stiff steel towers give the bridge short fundamental periods near the peak of the design spectrum, the seismic response is governed by structural mass, which is the basis for the mass-reduction retrofit. A separate assessment found that an overhanging rock feature on the east canyon wall is expected to dislodge at the seven% in 50-year event; as its dislodgement is assumed to collapse the east pier, it effectively governs the residual risk to the bridge, a level accepted in consultation with the District.

2.3 Condition and Seismic Vulnerabilities

Close-proximity inspections in October 2022 found the bridge in overall poor-to-fair condition, with the worst deterioration at the floorbeams and deck soffit near the deck joints, the south girder flanges and the horizontal members of the pier towers. Corrosion has caused isolated full-section loss in floorbeam webs and flanges, deck-soffit delamination up to 50 mm over areas of about 4 m² with loss of the bottom reinforcement layer, and section losses of roughly 25% in the girder flanges and tower webs. These losses were approximated in the analysis model, and the observed bearing deterioration, though noted, was not modelled explicitly. Under the design-level event the structure develops two primary load paths: longitudinal demand shedding mainly to the abutments and transverse demand mainly through the pier tower legs to the pier foundations, which together impose substantial substructure demands not considered in the original design. Demand-to-capacity ratios evaluated for the tower members, with a 1.25 factor applied to brittle compression members per the BC MOTT Supplement, reveal the largest exceedances in the diagonal built-up bracing of Tower 1 and a similar pattern in the transverse members of Tower 2. The inertial force generated by the heavy concrete deck is a primary driver of these demands throughout the system.

At the foundations, seismic overturning generates localized tension at the pedestals large enough that uplift or loss of bearing at the pedestal–bedrock interface could be expected. Together, these structural and geotechnical vulnerabilities motivate a retrofit strategy aimed at both reducing seismic demand and limiting the transfer of uplift force to the foundations.

3. RETROFIT DESIGN AND DETAILING

Following the evaluation of rehabilitation and replacement alternatives, the preferred strategy for the Nelson Canyon Bridge was a targeted seismic retrofit consisting of replacement of the existing reinforced-concrete deck with a full-width lightweight timber deck and installation of rocking isolation connections at the pier pedestals. This strategy was selected because it addresses the principal structural and geotechnical vulnerabilities identified in the assessment while minimizing environmental disturbance, construction complexity and cost. The timber deck reduces dead load and seismic inertial forces transferred to the steel towers, bracing and foundations. However, because this mass reduction also shortens the fundamental period and may increase spectral acceleration, its seismic benefit was verified through dynamic analysis rather than assumed. The rocking isolation connections complement the mass-reduction strategy by limiting uplift and tension transfer to the existing pedestals and anchorage. Retaining the existing alignment further reduces disturbance to the canyon slopes and Nelson Creek riparian corridor, avoids the demolition and hazardous-material handling associated with full replacement, and supports a shorter and lower-cost construction program. The principal retrofit components including timber deck, expansion joints, railings, watermain support bench, rocking isolation connections and pedestal strengthening, are described below (Figure 2).

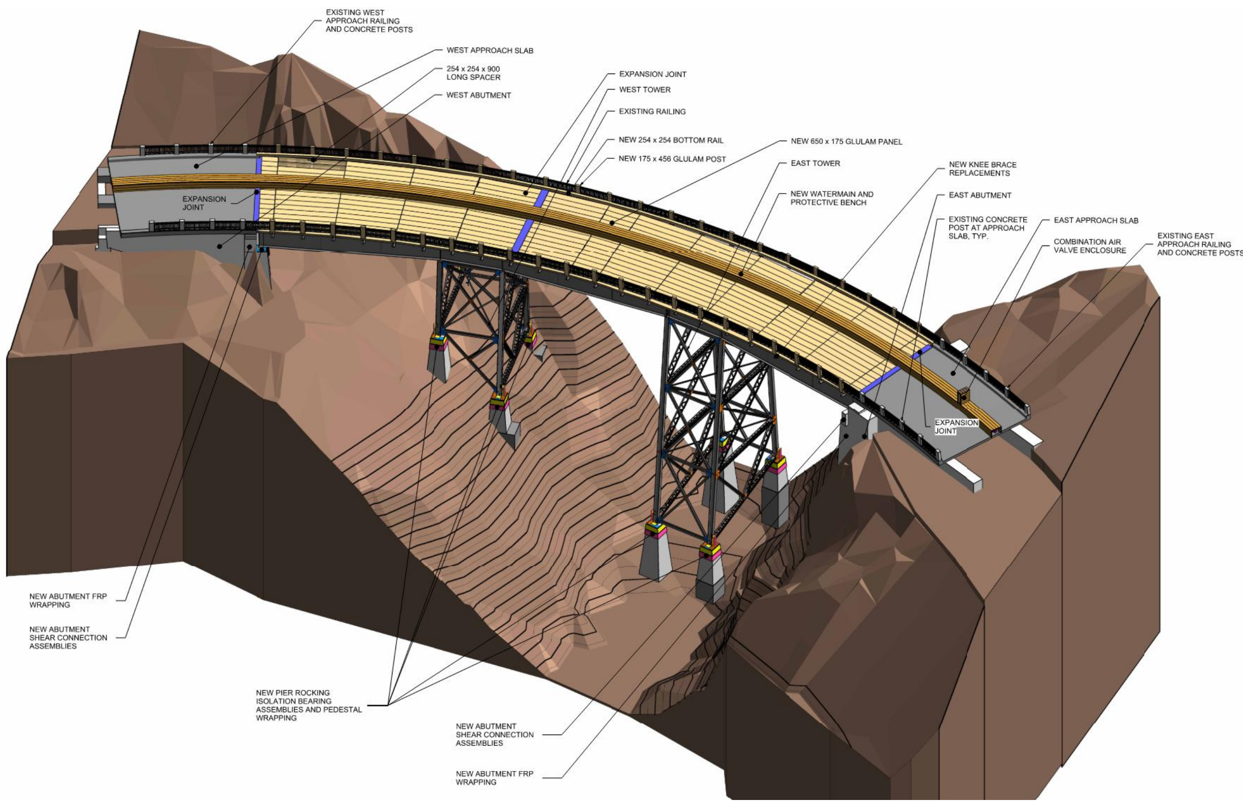

Figure 2: Nelson Canyon Bridge Retrofit Concept

3.1 Timber Deck Replacement

The existing reinforced concrete deck is demolished and replaced with prefabricated glulam deck panel cassettes, which reduced the seismic mass of the structure by approximately 70%. Shop-fabricating the cassettes and setting them in sequence shortens work over Nelson Creek and limits disturbance to the riparian corridor. Because the bridge follows an 87 m horizontal radius, the deck is curved in plan and set to an 8 percent crossfall: each cassette is cut longer along the outer (north) edge than the inner edge and is shaped to the crossfall. The panels are grouped by plan geometry into three families: Type 1, Type 2 and Type 3, with Type 3 forming the tapered panels at the span ends (Figure 3).

The glulam is manufactured from Douglas Fir-Larch, minimum grade 24F-1.8E. This grade satisfies bending and shear under the governing dead load and live load, including a maintenance vehicle, and meets the deflection serviceability limits for the deck.

Panel-to-panel connections in both the longitudinal and transverse directions transfer gravity load (dead load and live load, including the maintenance vehicle) and the maintenance-vehicle braking load between adjacent cassettes. Connections also deliver the deck loads and brake loads into the steel floor beams below. At the span ends, new glulam floorbeam is set to follow the crossfall of the bridge to support the underside of the Type 3 cassettes, so the end panels bear on a member rather than cantilevering off the last existing floor beam.

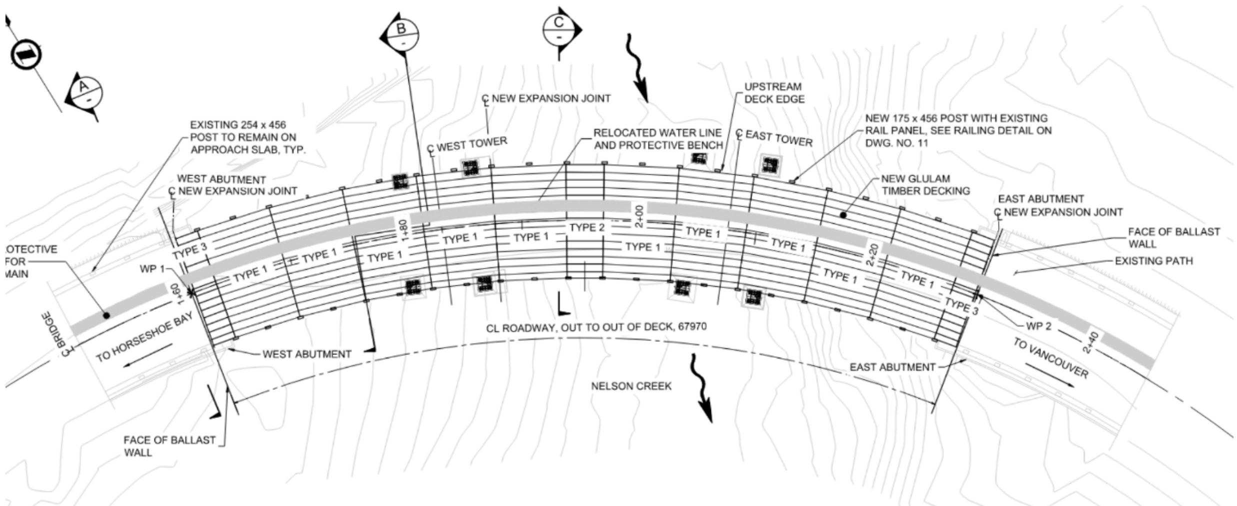

Figure 3: Curved deck framing plan showing the Type 1, Type 2 and Type 3 glulam cassette layout and the new expansion joint locations.

3.2 Expansion Joints

Three transverse expansion joints accommodate thermal and seismic movement: one at each approach-slab-to-deck interface and one deck-to-deck joint near mid-span (Figure 3). The movement ranges were set from a temperature-effect analysis of the bridge model run at the site maximum and minimum design temperatures, which gives a design gap of 30 to 100 mm at the two approach-slab-to-deck joints and 15 to 50 mm at the deck-to-deck joint. A Wabo SafetyFlex SFP-1000 system was selected for its movement capacity and, in particular, for its tolerance of the vertical offset that can develop from seismic uplift.

3.3 Railings

The existing steel fence panels are retained. Each panel is inspected, repaired for local damage as needed and given one coat of black paint before reinstallation. The heavy, deteriorated concrete rail posts and curbs are integral to the existing concrete deck and are demolished with it; new lighter glulam posts are installed on the timber deck to carry the retained steel panels, as shown in Figure 4. This keeps the railing consistent with the deck mass reduction while preserving the appearance of the original barrier.

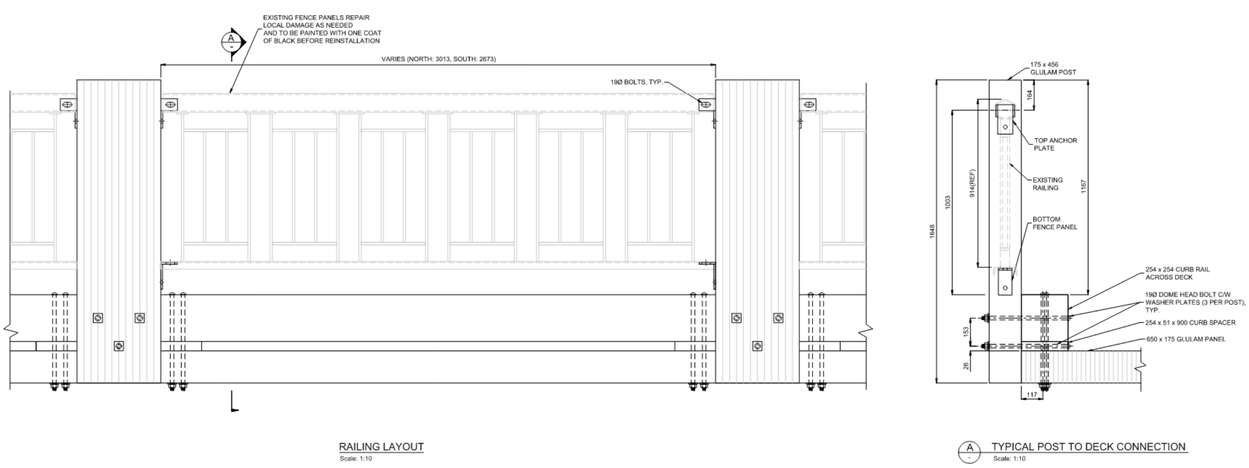

Figure 4. Timber railing design incorporating existing fence panels with new glulam posts.

3.4 Watermain Support Bench

The watermain is carried in a purpose-built support bench framed from lumber members with timber panel infill (Figure 4). Two air-valve access doors are provided, one at each end of the watermain, and a single flush, removable timber access panel at the midpoint allows internal maintenance. The watermain follows the curved alignment and is supported at 2500 mm centres. Because the deck carries an 8% transverse crossfall, at each support, a timber block is cut to seat the pipe level, with two triangular timber blocks at the sides to help centring it and restraining it laterally during a seismic event, while an 890 x 150 x 3 mm UHMW polyethylene sheet between the pipe and the blocks reduces friction and lets the pipe accommodate thermal expansion, contraction and seismic movement.

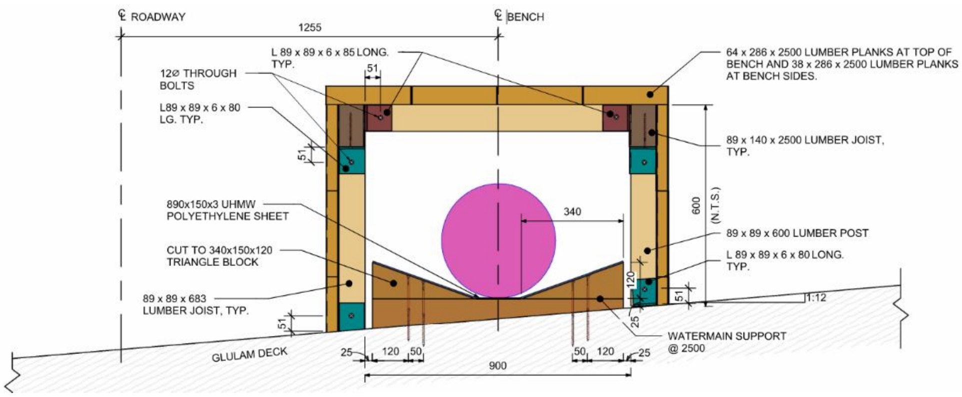

Figure 4: Watermain support bench — lumber framing, triangle-block centring and UHMW slip sheet on the 8% crossfall.

3.5 Rocking Isolation Foundation Connections

The pier towers are seismically isolated at their bases by controlled rocking connections that let the columns uplift in a controlled manner rather than transfer large tension into the foundations. Each connection combines a Triangular Added Damping and Stiffness (TADAS) device with a hold-down and bearing assembly anchored into the concrete pedestal. When a column tends to uplift, the triangular steel plates of the TADAS device engage and yield in flexure to dissipate seismic energy, while the hold-down clamps arrest the uplift once a stability threshold is reached; under compression the column bears directly onto the pedestal. A designed gap sets the uplift displacement at which the plates and hold-downs engage. By permitting controlled rocking, the connections relieve the tension and uplift demands that would otherwise overload the pedestals and their bedrock foundations. Because this anchorage concentrates tension in the pedestal, the pedestal is strengthened as described in the following subsection.

The triangular plates are proportionally sized to the bending-moment distribution of a cantilever under a point load, so that flexural yielding is uniform along their length (Pollino et al. 2010). The plastic shear force (Eq. 1), elastic shear force (Eq. 2) and elastic stiffness (Eq. 3) follow from the triangular-plate dimensions and material properties:

[1] $V_{pT} = \frac{N_T\, t_T^2\, b_T\, F_{yT}}{4\, L_T}$

[2] $V_{eT} = \frac{N_T\, t_T^2\, b_T\, F_{yT}}{6\, L_T}$

[3] $k_{eT} = \frac{E\, b_T\, N_T}{6}\left(\frac{t_T}{L_T}\right)^3$

where N_T = number of plates, t_T = plate thickness, b_T = plate width at fixed end; F_{yT} = yield stress of steel, L_T = length of plate from fixed end to free slot, and E = material elastic modulus. The ratio of plastic to elastic stiffness was confirmed to be approximately 0.05. Because the triangular plates yield in flexure and reload symmetrically in tension and compression without appreciable strength or stiffness degradation, a bilinear kinematic hardening hysteresis model with the backbone curve defined by Eq. 1 through Eq. 3 was adopted to represent the TADAS springs under the cyclic load reversals imposed in the nonlinear time-history analysis. The resulting spring response is shown in Figure 5(b).

At the base connection, a physical gap between the pier-column base plate and the hold-down clamps sets the uplift at which the triangular plates engage. Beyond this gap the hold-downs develop tension resistance, while in compression the column bears immediately onto the concrete pedestal.

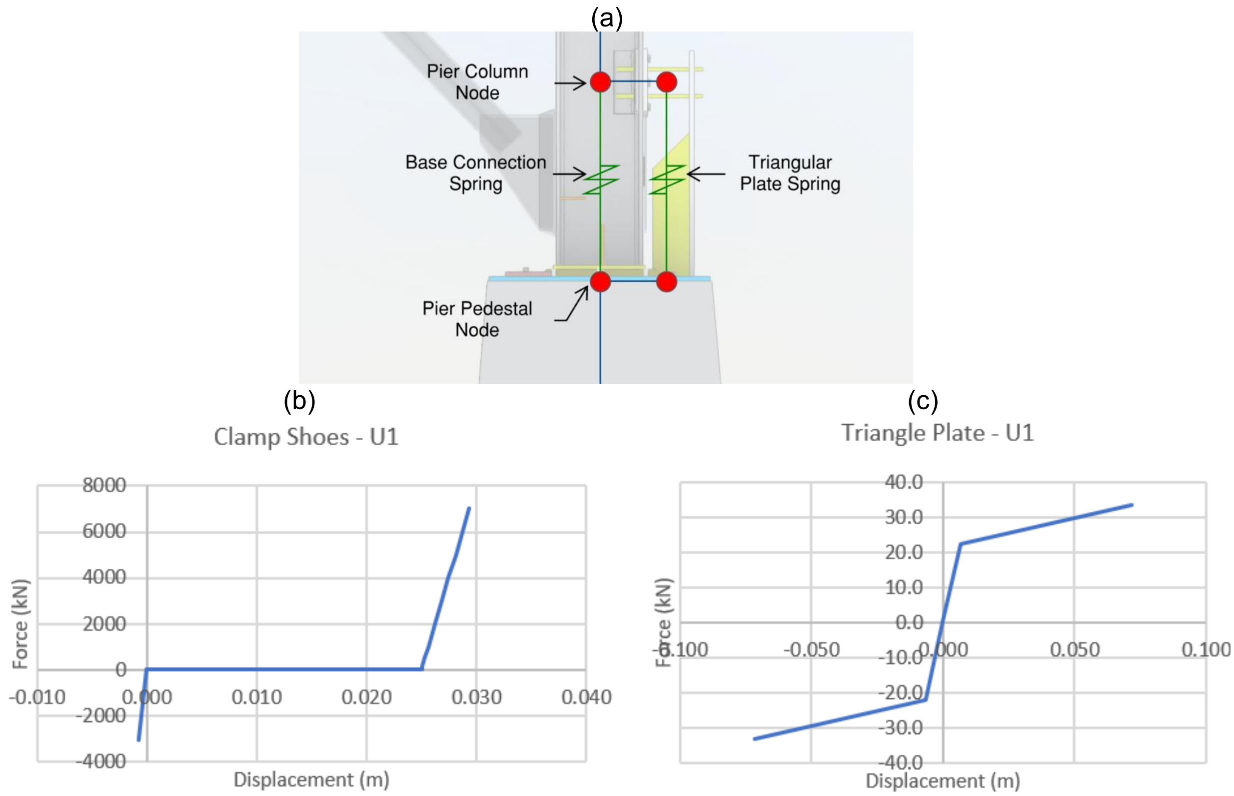

Figure 5: Rocking isolation connection modelling: (a) equivalent FEM link model; (b) triangular-plate (TADAS) spring force–displacement hysteresis; (c) base connection (hold-down clamp) force–displacement response.

3.6 Pedestal FRP Strengthening

The concrete pedestals that receive the rocking anchorage are strengthened with a fibre-reinforced polymer (FRP) wrap to confine the concrete, control breakout and edge failure. The pedestals are not prismatic and taper on a 1:12 incline, so a conventional unidirectional wrap cannot be applied in continuous layers parallel to the pedestal faces. A bidirectional FRP with fibres oriented at ±45° to the sheet's longitudinal axis was used instead. Referenced to the length of the sheet, the ±45° fibres resolve into tension components along both principal directions of the pedestal, with each fibre family carrying one direction, so a single layer develops the required capacity in both directions. Using one bidirectional layer in place of two unidirectional layers saves about \$650/m² over roughly 95 m², or on the order of \$62,000.

4. ANALYTICAL METHODOLOGY

4.1 Structural Modelling

Three-dimensional finite element models were developed to represent both existing and retrofitted bridge configurations. Frame elements were used to model the pier towers, girder and foundation pedestal members and shell elements were used to model the concrete deck. Frame elements were preferred in general due to their computational efficiency, whereas shell elements were preferred for the deck as they incorporate both in-plane membrane and out-of-plane bending deformations. The pier and abutment bearings, both expansion and fixed types, were modelled as links with shear and flexure releases to reflect the true behaviour of the respective bearings in the field. All element self-weight loads, including those for the watermain and railing, were accounted for through applied uniformly distributed and point model loads, which were also converted to masses for the seismic analysis. The models were calibrated using as-built drawings, field measurements and previous analytical studies. The controlled rocking isolation connections at the pier bases were represented by two parallel link elements: a triangular-plate (TADAS) spring and a base-connection spring, whose force-deformation properties follow the design presented in Section 3.5.

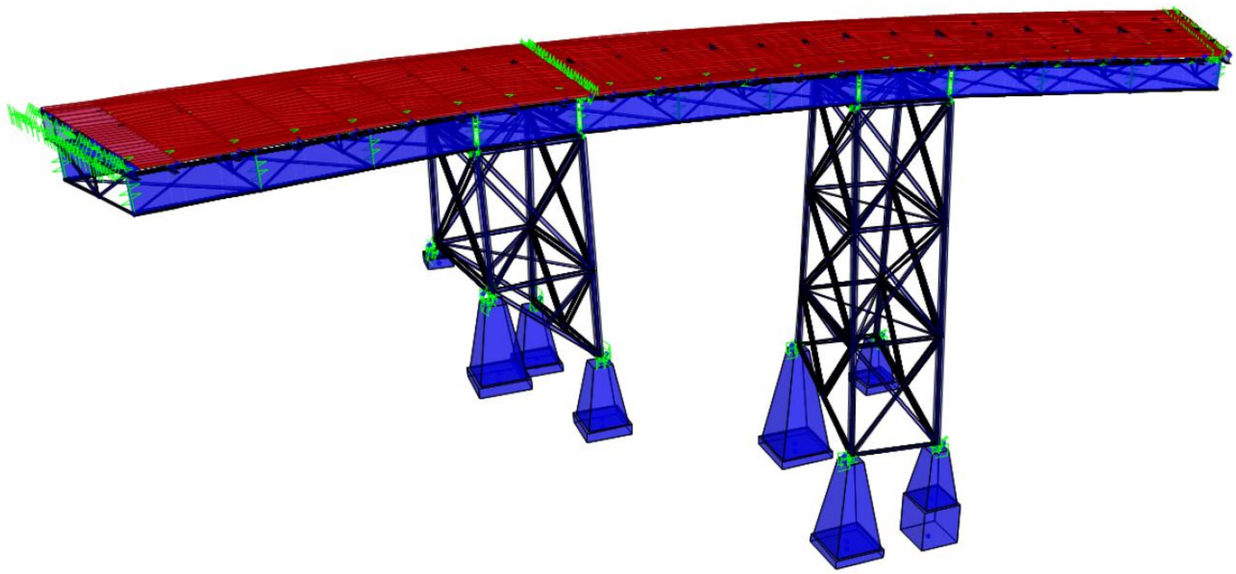

Figure 6: Finite element model of retrofitted bridge

4.2 Response Spectrum Analysis

A modal analysis was first carried out to evaluate the changes in dynamic properties resulting from the deck replacement, examining the fundamental periods and modal mass participation.

Response spectrum analyses were carried out using the sixth-generation seismic hazard demands converted to a design spectrum in accordance with CSA S6:19 §4.4.3.4. Since steel structures typically exhibit low inherent damping, the typical 5% design spectrum was modified to the 2% design spectrum per CSA S6:19 §4.4.3.5. The multi-mode spectral method was used to capture at least 90% participation of the contributing superstructure inertial mass in two orthogonal directions on the horizontal plane. The modal member forces and displacements were combined using the complete quadratic combination method, while the horizontal elastic seismic effects were directionally combined in accordance with CSA S6:19 §4.4.9.2 (i.e. the 100%-30% method). As discussed earlier, vertical seismic effects were not considered for this evaluation per CSA S6:19 §C4.11.6. Over 92% of the superstructure mass participated in the first eight modes over the horizontal plane.

4.3 Time History Analysis

Implementation of the proposed rocking isolation foundation connections required the use of nonlinear time-history analysis to accurately capture the system response and the highly nonlinear behaviour associated with foundation uplift and re-contact. A suite of eleven ground-motion records from nine earthquake events, selected to satisfy the site-specific seismic hazard criteria and scaled in accordance with the target spectral response recommended by Thurber Engineering, was used to evaluate the seismic performance of the retrofitted bridge. These analyses demonstrated that the rocking isolation connections, modelled using a combination of hysteretic and compression-only spring elements, were effective in substantially reducing tensile force transfer to the pier pedestals and the underlying foundation system. As a result, foundation uplift was controlled and only low tension reached the pedestals while compression and shear transferred in full, keeping the anchorage tension below the level that would cause pullout or concrete breakout in the pedestals during strong earthquake shaking.

5. DISCUSSION AND CONCLUSIONS

5.1 Discussion

The seismic retrofit of the Nelson Canyon Bridge was governed by the need to reduce demand on an aging steel substructure while limiting uplift at the pedestal–bedrock foundations and maintaining the post-earthquake function of the watermain. The assessment showed that the existing bridge is vulnerable under design-level shaking, particularly in the tall steel pier towers, bracing members and pedestal foundations. Although the foundations bear on competent bedrock, seismic overturning can generate uplift and tensile demand at the pedestal–bedrock interface, where the existing anchorage capacity is not adequate.

The selected retrofit addresses these vulnerabilities through a combined mass-reduction and force-limiting strategy. Replacement of the reinforced-concrete deck with prefabricated glulam deck cassettes substantially reduces superstructure mass and therefore lowers seismic inertial forces transferred to the towers, bracing and foundations. Because the lighter deck also shortens the fundamental period and can increase spectral acceleration, this benefit was confirmed analytically rather than assumed. The response-spectrum results showed that the reduction in seismic weight governed the overall response and reduced demands in the primary structural system. Foundation uplift remained a critical design issue because reducing deck weight also reduces stabilizing gravity load. The rocking isolation connections at the pier pedestals therefore form the key seismic-protection mechanism of the retrofit. By allowing controlled uplift at the tower bases, the TADAS devices and hold-down assemblies limit tension transfer into the existing pedestals while maintaining compression and shear resistance. Nonlinear time-history analyses confirmed that the rocking mechanism-controlled foundation uplift and limited the tension transferred to the pedestals. FRP strengthening of the tapered pedestals provides the local capacity to anchor the force-limiting devices and to resist pullout and concrete breakout in the existing foundation elements.

The retrofit also supports the bridge’s lifeline function and site constraints. The watermain support bench, centring blocks and UHMW slip sheets accommodate the bridge curvature, crossfall and expected thermal and seismic pipe movements. Prefabricated timber deck panels reduce construction duration over Nelson Creek, while retaining the existing alignment avoids the demolition, hazardous-material handling and environmental disturbance associated with full bridge replacement. Because the rocking isolation hardware is intended to act as replaceable sacrificial components, post-earthquake inspection and repair can focus on accessible connection elements rather than damage to the primary bridge structure.

5.2 Conclusions

The Nelson Canyon Bridge retrofit demonstrates that targeted seismic rehabilitation can improve the resilience of an aging lifeline bridge without requiring full replacement. The principal vulnerabilities were not related to static bearing capacity, but to seismic force demand in the steel tower system and the risk of foundation uplift at the pedestal–bedrock interface. Replacing the concrete deck with a lightweight timber deck reduces seismic inertial demand, while the rocking isolation connections limit foundation uplift and concentrate inelastic response in replaceable components. The analytical results confirm that the combined system reduces demands on the towers, bracing members and foundations while improving confidence in post-earthquake watermain operability.

This approach is well suited to short- and medium-span lifeline bridges where seismic vulnerability, environmental constraints and construction cost make full replacement difficult to justify. The project shows that combining lightweight deck replacement, controlled rocking isolation, localized foundation strengthening and utility-specific detailing can provide a practical and resilient retrofit strategy for critical utility crossings in high-seismicity regions.

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