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Seismic Isolation and Half-Joint Elimination of Cambie Bridge in Vancouver, BC

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ABSTRACT

The Cambie Bridge is approximately 870 m long and carries Cambie Street over False Creek. Constructed circa 1985, this structure contains brittle detailing and does not comply with current seismic performance criteria. The City of Vancouver identified the bridge as a critical piece of infrastructure for movement of people and goods between South Vancouver and the Downtown Core. Accordingly, the City selected the bridge as a candidate for seismic retrofit with a target performance of limited, repairable damage and relatively fast return to post-earthquake service. In expectation of the increased seismicity of the South Coast of British Columbia between the 2015 and 2020 National Building Code seismic hazard models, the City also desired a retrofit strategy that reduced sensitivity to potential future seismicity increases. Given the significant investment in seismic resiliency improvements to the bridge, the City also sought opportunities for general rehabilitation and service life extension.

The brittle structural detailing and stringent performance criteria make a ductility-based seismic retrofit very challenging to achieve. A low-damage seismic retrofit strategy was developed that seismically isolates the superstructure, thus reducing forces transmitted to the substructure and significantly reducing challenging below-grade retrofit work. Rearticulation of the superstructure with flexible bearings presented the opportunity to eliminate the existing half-joints by converting them to continuous spine beam connections and making the superstructure continuous over its entire length. This retrofit eliminates the risk of road salts further compromising the longitudinal post-tensioning anchorages and removes future inspection and maintenance items at these locations. Additionally, seismic performance is improved through increased superstructure connectivity and thus load distribution to the bearings.

Given the Cambie Bridge's length and geometry, half-joint elimination and installation of isolation bearings significantly changes not only the longitudinal and transverse lateral seismic response of the bridge, but also the lateral service load responses, particularly thermal response. This paper will focus on the changes to the bridge response, and the supporting rigorous linear and nonlinear analysis and assessment that was performed to understand bridge's behaviour and validate the retrofit design.

Keywords: Earthquake, Bridge, Retrofit, Isolation, Analysis

INTRODUCTION

Background

Cambie Bridge is one of three City of Vancouver-owned bridges crossing False Creek and is a critical piece of infrastructure for movement of people and goods between South Vancouver and the Downtown Core. As part of the it's seismic resiliency program, the City identified Cambie Bridge as a candidate for seismic retrofit with a target performance of limited, repairable damage and relatively fast return to post-earthquake service. In recognition of the significant investment in seismic resiliency improvements to the bridge to achieve the target performance, the City also sought opportunities for general rehabilitation and service life extension. Associated Engineering was retained by the City to provide preliminary and detailed design services for the seismic retrofit and general rehabilitation of Cambie Bridge.

Structure Description

Cambie Bridge is approximately 870 m long and carries Cambie Street over False Creek in Vancouver, BC. The main bridge comprises 21 approach spans with a typical length of 39 m, and a marine span of 84 m, as shown in Figure 1 below. The main bridge deck is 30.5 m wide and includes three lanes of south-bound and three lanes of north-bound traffic, a multi-use pathway on the east side, and a sidewalk on the west side. The bridge also has several ramps: 2nd Avenue On-ramp, 2nd Avenue Off-ramp, Nelson Ramp, Smithe Ramp, and Taylor Ramp. For a complete bridge plan, see Figure 2.

(a)

(b)

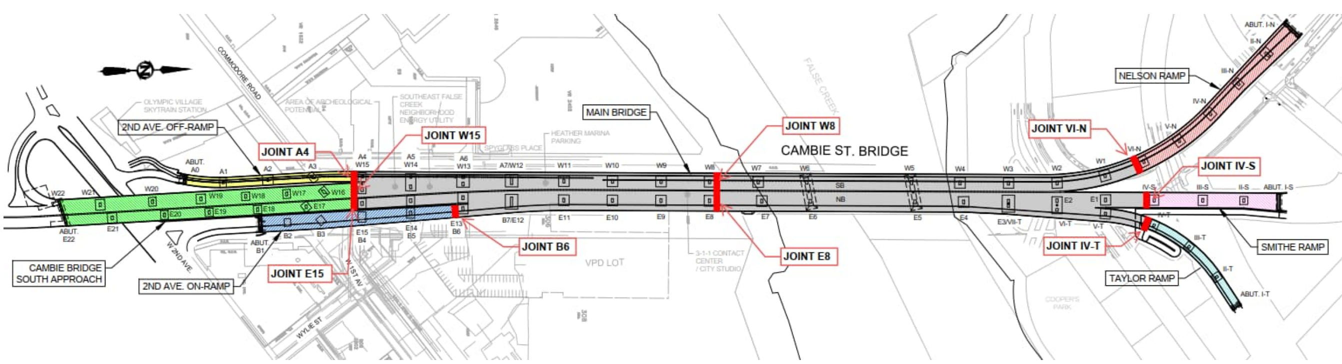
Figure 1. Marine Span Photos (a) facing North, (b) facing South.

The superstructure system is longitudinally post-tensioned concrete spine beams with transversely post-tensioned and reinforced concrete deck cantilevers. The spine beam soffits are typically 2.0 m or 3.0 m wide, and are supported by 65 reinforced concrete mono-columns, as shown above. Spine beam connections to the mono-columns vary, with 25 columns being integral with the superstructure, and 40 columns connected with disc bearings.

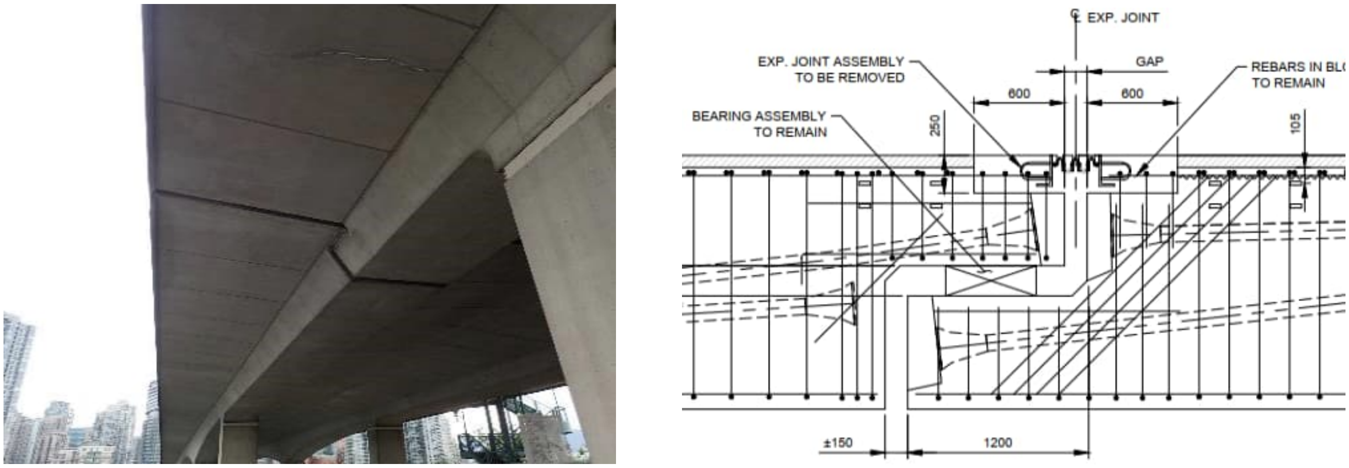
The site soil is characterized by a till layer which is located near grade at the south end of the bridge, and which gets progressively deeper moving north. The till is overlain by sand, clayey silt, and variable fill layers. Accordingly, the southern abutments and pier columns up to Pier W16 are supported on reinforced concrete spread footings, while the structure north of Pier W16 is supported on groups of unreinforced expanded base piles with reinforced concrete pile caps.

Existing Articulation and Half-joints

The bridge comprises eight segments, which are separated from one another with in-span half-joints, as shown in Figure 2 below. These half-joints contain guided bearings, allowing the transfer of vertical and transverse loads, while permitting independent longitudinal movement. The half-joints include modular expansion joints at each location to accommodate this movement.



(a)



(b) *(c)*

Figure 2. Existing in-span half-joints (a) plan locations, (b) typical joint photo, (c) representative details

At the center-of-rigidity of each bridge segment, the pier columns are monolithic with the superstructure, providing a fixed connection. Moving away from this center and towards each half-joint, the superstructure is supported on expansion disc bearings that are longitudinally free, but provide transverse movement restraint.

As shown in Figure 2(c) above, the half-joints also contain longitudinal spine beam post-tensioning anchorage zones, which are located directly below the existing modular expansion joints. Expansion joint seals are prone to leaking, and the existing joints showed evidence of water staining and debris on the soffit and half-joint corbels. The criticality of the structural components within the half-joint, combined with associated the inspection and maintenance challenges present a durability concern for Cambie Bridge.

RETROFIT DESIGN PARAMETERS

Seismic Performance Criteria

The City's target performance of Cambie Bridge was to meet or exceed new bridge design criteria for 'Major-route bridges', as defined by the Canadian Highway Bridge Design Code S6:19 (S6:19)[1]. Given the detailing limitations of the approach pier foundations, project-specific performance criteria were developed to meet or exceed 'Major-route bridges' wherever practical, with a lower standard accepted for liquefaction-induced flow slides. Table 1 below shows the project-specific performance levels. Damage Levels marked with (*) correspond to liquefaction load cases.

Table 1. Project-specific Seismic Performance Levels

Earthquake Return Period (years)	Damage Level			Service Level
	Superstructure, Pier Columns, Abutments & Marine Span Pier Foundations	Approach Pier Foundations	Joints & Bearings	
975	Minimal	Minimal	Minimal	Service Limited
2475	Repairable	Repairable / Extensive*	Minimal	Service Disruption

In light of the increased seismicity of the South Coast of British Columbia between the NBC 2015[2] and 2020[3] seismic hazard models, the City's retrofit criteria also required that the above Damage and Service Levels remain valid for moderate future increases in site seismicity.

Seismic Design Input

Site-specific seismic design inputs were developed for the Cambie Bridge. A total of 30 spectrally-matched ground motion records were used as the basis for seismic analysis. The motions were categorized into three suites: crustal (10), in-slab (10), and interface (10). Given the variable site stratigraphy and foundation types, extensive nonlinear site-response analyses were conducted to scale the ground motions for accurate site-response. Figure 3 below shows the resultant design acceleration response spectrum for the 2475-year return period earthquake, and the calculated bounding spectra based on site-response analysis. The NBC 2020[3] Site Class C spectrum is also shown.

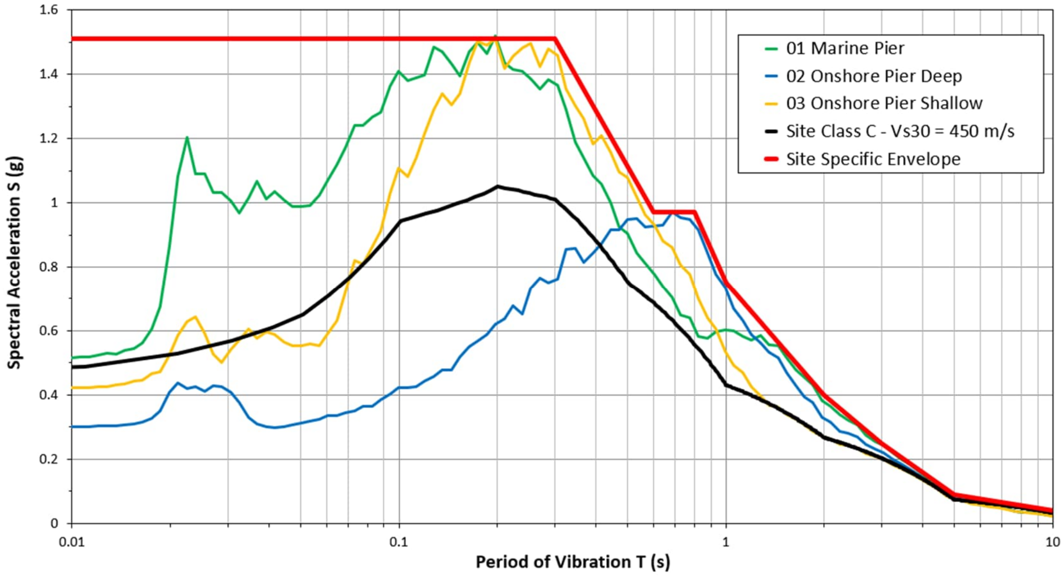


Figure 3. Site-specific design acceleration response spectrum

SEISMIC RETROFIT STRATEGIES CONSIDERED

Ductility-based Retrofit Strategy

A ductility-based seismic retrofit strategy was considered, whereby seismic energy would be dissipated through inelastic structural deformation of the pier columns, with the foundations capacity protected from elastic demands. Preliminary assessment identified several challenges in achieving the City's target seismic performance levels.

The large 1.5 m deep by 3.0 m wide typical rectangular columns were proportioned and detailed for the heavy solid concrete superstructure weight and service loads. While vertical bar detailing is generally adequate, with no lap splices or bar curtailment in high-moment regions, horizontal tie detailing and spacing is inadequate to provide reliable core concrete confinement or prevent vertical bar buckling. Further, the existing bearings at the 40 expansion piers do not have capacity to transmit the required lateral demands to the substructure. Thus, retrofits to achieve acceptable ductility capacity at each of the 65 pier columns would prove extremely challenging and involve highly invasive intervention.

The typical approach pier foundations comprise large, nominally reinforced concrete pile caps atop groups of unreinforced expanded base piles. The expanded base pile connection to the pile caps is intended for vertical load transfer only, with only nominal casing embedment for lateral load transfer, as shown in Figure 4 below.

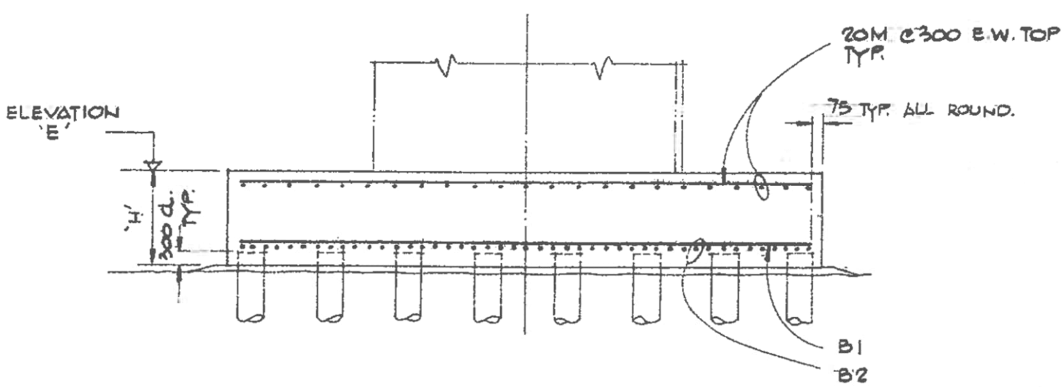


Figure 4. Typical pier foundation elevation

The pile caps do not contain shear ties, and many do not contain sufficient horizontal reinforcement to transfer column yield demands to the piles. Based on the existing details, capacity protection of the piers would be even more challenging than the corresponding column retrofits, and would involve large, deep excavations in a dense urban environment.

The structure is laterally stiff, with significantly-contributing vibration modes having periods ranging from 0.7 seconds to 1.0 second. Not only do these short periods attract significant seismic force demands, they also correspond with frequency-dominant earthquake hazards and as such, indicate that the existing bridge is very sensitive to changes in hazard. Given that a key goal of the retrofit is to account for or mitigate the effects of future increases in site seismicity, the inherent lateral stiffness presents a serious risk to the performance.

Based on the above identified challenges, a ductility-based retrofit strategy was considered cost-ineffective and impractical to achieve the desired seismic performance goals.

Low-damage Retrofit Strategy

A low-damage isolation-based retrofit strategy was identified as an attractive alternative approach for the Cambie Bridge, whereby laterally flexible bearings could be installed at the column-tops, increasing the bridge's fundamental period of vibration and significantly reducing forces transmitted to the stiff, brittle substructure components. Preliminary assessment and concept design indicated that this approach could shift the bridges fundamental period of vibration to at least 3.0 seconds. When combined with supplemental damping, this period shift would not only reduce force demands to within the substructure elements' respective elastic ranges, but also reduce the retrofit structure's sensitivity to changing seismic hazards.

Employment of an isolation-based retrofit strategy would require modifications to each of the column tops to accommodate new isolation bearings, in addition to significant retrofits to each of the six abutments to accommodate the increased lateral displacement demands. Additionally, temporary superstructure support and access to modify the columns and install new bearings would be challenging. However, these component locations are generally above-grade, and thus relatively accessible for retrofits when compared to a ductility-based approach.

A key challenge as part of the isolation-based retrofit strategy development was selection of a bearing type. Cambie Bridge presented several constraints which guided consideration of bearing types. As previously discussed, the superstructure is generally solid concrete spine beams, thus self-weight vertical reactions are large. Resisting vertical loads within the existing column-top plan area restricted maximum bearing size, which would limit other bearing properties, including bearing height, stiffness, deformation capacity, damping, and re-centering. After careful investigation of these considerations and others, circular lead-rubber bearings were selected for this application. Figure 5 below shows a conceptual bearing configuration for a typical pier column.

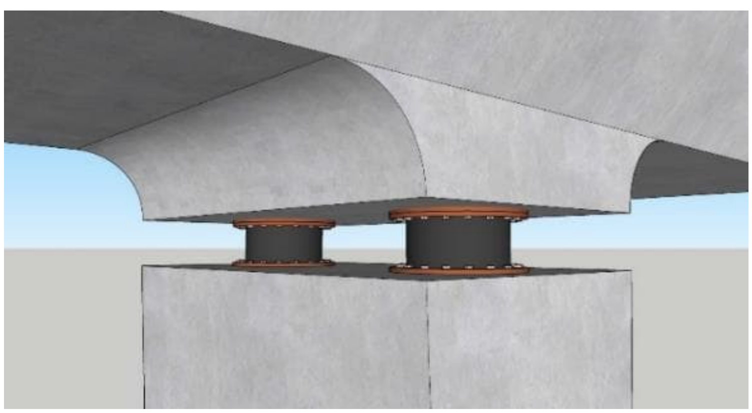


Figure 5. Conceptual bearing configuration at a typical pier column

Based on the above merits and considerations, a low-damage isolation-based retrofit strategy was selected for the Cambie Bridge.

HALF-JOINT ELIMINATION AND REARTICULATION

Half-Joint Elimination Benefits

An isolation-based seismic retrofit strategy presented the possibility of elimination of the existing in-span half-joints by converting the joints to continuous, monolithic superstructure connections. This would protect the longitudinal post-tensioning anchorages from future chloride ingress and eliminate future inspection and maintenance of the half-joint bearings and expansion joint components. Additionally, it would create a continuous deck diaphragm for lateral response, with seismic performance benefits including

1. Improved distribution of seismic demands from superstructure to pier columns;
2. promotion of in-phase seismic response over a long and variable site, elimination of in-span joint pounding risk;
3. removal of in-span joint guided bearings from seismic load path; and
4. improved loss-of-span protection in extreme liquefaction-induced flow slide event (near-shore piers).

While many of the benefits of half-joint elimination were apparent, it was recognized that the superstructure rearticulation caused by this work would completely change both vertical and lateral bridge response to not only seismic demands, but also service demands, particularly those induced by temperature changes.

Rearticulation Considerations

As presented previously, the Cambie Bridge is highly irregular in plan. Thus, converting the superstructure to be continuous over its length would introduce important lateral response behaviour changes, including

1. Increased thermal and seismic demands at all six abutments and many of the piers;
2. Significant bi-directional displacement demands at the abutment joints; and
3. Lateral flexure and shear demands in the superstructure, especially at the curved northern ramps.

Figure 6 below illustrates the rearticulation changes by showing the theoretical lateral center-of-rigidity of the superstructure following elimination of the half joints.

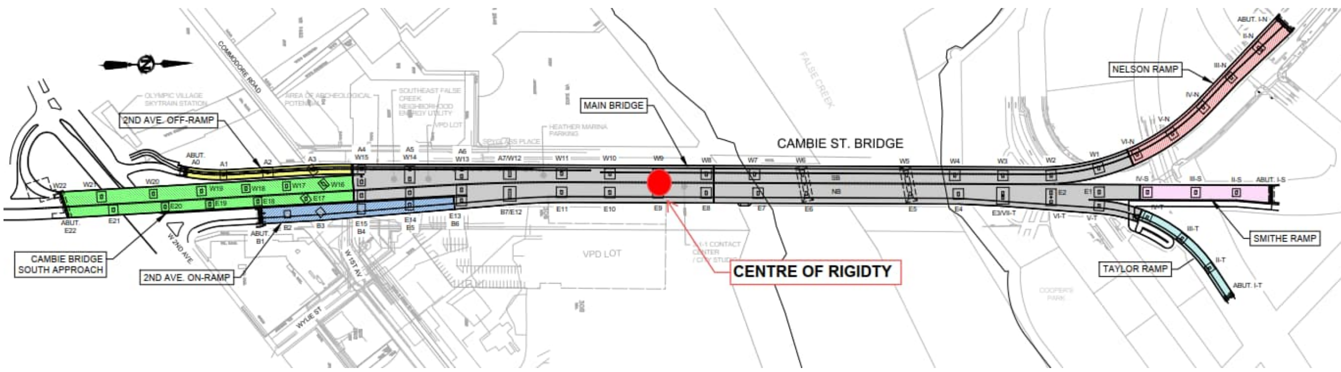


Figure 6. Theoretical lateral center-of-rigidity following half-joint elimination

Superstructure displacement and force demands at the piers could be addressed through isolation bearing design, however accommodation of large bi-directional displacements at the abutment joints would be very challenging, as the seismic performance criteria requires minimal joint damage at all earthquake levels. Additionally, large bi-directional thermal displacements would pose major challenges for bridge alignment with the approaches, including not only roadway alignment, but also barrier and utility transitions. To eliminate this challenge, a longitudinally-guided abutment design was proposed, wherein longitudinal guide bearings and concrete shear keys would be installed at each abutment, restricting out-of-plane superstructure displacements, and thus maintaining joint travel square to the abutment backwall for both thermal and seismic response.

It was recognized that out-of-plane restraints at each of the abutments would create potentially significant restraint forces in the guide bearings and shear keys. Additionally, they could increase lateral flexural and shear superstructure demands, especially at the curved Nelson and Smithe Ramps. It was determined that detailed analysis of the rearticulated bridge response would be required to understand the behaviour and confirm the viability of half-joint elimination with guided abutments.

ANALYSIS METHODOLOGY

Midas Civil[4] software was used to perform a variety of finite element analyses on the Cambie Bridge. Preliminary seismic demand analysis was conducted using multi-mode elastic response spectral analysis (RSA) using site specific elastic uniform acceleration hazard spectra based on NBC 2015[2] seismic hazard values. The hazard spectra values were reduced for isolated periods of vibration based on estimated global damping ratios. RSA models included isolation bearings with effective shear stiffness, in addition to equivalent elastic multi-degree-of-freedom foundation springs. Iterative analysis was conducted to refine the bearing and foundation spring stiffness based on expected displacement demands, and to conduct sensitivity analysis.

Detailed seismic demand analysis was conducted using nonlinear time-history analysis (NTHA) using 30 spectrally-matched and scaled ground motion records based on NBC 2020[3], as presented previously. NTHA models included nonlinear isolation bearing springs and multi-degree-of-freedom foundation springs. Damping from the isolation bearings, soils, and reinforced concrete structure were explicitly modeled. Several analyses were conducted to account for variations in bearing properties, foundations springs, and system damping. For all seismic demand models, cracked concrete pier column stiffnesses were used, based on guidance by Priestley et al.[5] and verified by nonlinear section analysis tools.

Thermal demand analysis was conducted using global finite element analyses, using both linear and nonlinear isolation bearing springs. Temperature demands were based on those prescribed in S6:19[1].

Concrete section capacity assessment was conducted using a number of tools including Midas General Section Designer[6], Response-2000[7], and in-house tools.

SEISMIC PERFORMANCE ANALYSIS RESULTS

Vibration Properties

RSA models estimated a retrofit fundamental period of vibration of approximately 3.10 seconds, and an equivalent global damping ratio of approximately 20%. Refined NTHA generally confirmed these properties, reporting a theoretical fundamental period of 3.22 seconds and a corresponding modal damping ratio of 20.7%. In total, 300 modal properties were calculated as part of the analysis. Table 2 below presents the first 10 modes.

Table 2. Dynamic Properties – Nominal Bearing Stiffness

Mode	Period [seconds]	Long. Modal Mass Participation [%]	Trans. Modal Mass Participation [%]	Modal Damping Ratio [% Critical]
1	3.22	13.38	56.98	20.7%
2	3.18	76.48	9.87	19.7%
3	2.96	0.60	2.04	19.8%
4	2.56	0.07	14.53	15.0%
5	2.01	0.34	0.16	9.5%
6	1.82	0.28	0.09	8.9%
7	1.55	0.03	0.93	6.9%
8	1.36	0.01	0.06	5.5%
9	1.34	0.01	0.00	5.2%
10	1.21	0.01	1.31	4.6%

As shown above, the retrofit structure is controlled by the isolated modes, with the first four modes capturing 91% and 83% of the longitudinal and transverse modal mass participation, respectively. These modes also benefit from the significant damping provided by the lead cores of the isolation bearings.

With reference to Figure 3 presented previously, shifting the existing structure period from approximately 0.70 seconds to 3.22 seconds corresponds to a reduction in spectral acceleration from approximately 0.95 g to 0.25 g, significantly reducing forces transferred to the brittle substructure elements. Sensitivity to seismic hazard uncertainty is also demonstrably reduced, as a 20% increase in spectral acceleration represents an increase of approximately 0.19 g in the existing configuration, but only 0.05 g in the proposed retrofit configuration.

Pier Column Performance Results

Table 3 below presents demand-capacity ratios at the base of select representative pier columns for the 2475-year return period earthquake load case. Existing articulation results are based on RSA demands, while proposed retrofit articulation results are based on NTHA demands. Demand-capacity ratios exceeding 1.0 indicate elastic demands exceed expected yield capacities.

Table 3. Selected pier column seismic demand-capacity ratios

Pier	Flexure				Shear			
	Longitudinal		Transverse		Longitudinal		Transverse	
	Existing	Retrofit	Existing	Retrofit	Existing	Retrofit	Existing	Retrofit
A2	**2.96**	0.31	**3.59**	0.24	**1.99**	0.11	**1.41**	0.11
B4	**2.45**	0.43	**2.25**	0.23	**1.24**	0.10	0.90	0.12
E5	**1.46**	0.21	**1.05**	0.20	0.73	0.07	0.38	0.09
W7	0.78	0.72	**3.51**	0.42	0.14	0.07	0.63	0.11
V-N	**2.30**	0.37	**1.76**	0.21	**1.37**	0.11	**1.43**	0.12
III-S	**2.16**	0.27	**1.20**	0.11	**1.80**	0.12	0.75	0.11
III-T	**2.20**	0.27	**2.12**	0.16	**1.71**	0.11	**1.20**	0.11

As shown above, the existing selected pier columns exceed their expected yield capacities, often by a significant margin. In the retrofit articulation, the column demands are reduced to within the elastic range.

Superstructure Performance Results

As previously discussed, lateral demands in the superstructure were an important consideration for the proposed retrofit articulation. Performance was checked at the existing half-joint locations, where new reinforced concrete continuity deck connections provide lateral resistance. Performance was also checked throughout the existing spine beams, where existing longitudinal deck reinforcement is much lighter than the continuity slab locations, but the longitudinal post-tensioning provides considerable additional reinforcement and concrete section precompression. Table 4 below presents transverse flexural demand-capacity ratios at select spine beam locations. NTHA demands for the 2475-year return period earthquake were used.

Table 4. Selected spine beam seismic demand-capacity ratios – transverse flexure

Superstructure	Location	Comment	Demand / Capacity
2nd Ave Off-ramp	Joint A4	Half-joint	0.81
	Midspan between Pier A1 and A2	Existing spine beam	0.82
2nd Ave On-ramp	Joint B6	Half-joint	0.88
	South of Joint B6	Existing spine beam	0.79
Nelson Ramp	Joint VI-N	Half-joint	0.61
	Pier III-N	Existing spine beam	0.50
Smithe Ramp	Joint IV-S	Half-joint	0.67
	North of Joint IV-S	Existing spine beam	0.62
Taylor Ramp	Joint IV-T	Half-joint	0.52
	Midspan between Pier III-T and IV-T	Existing spine beam	0.59

Detailed analysis showed that, while the guided abutment configuration does induce transverse flexure in the spine beams, the isolation bearings provide little restraint at each pier location, thus significantly reducing the stiffness of the superstructure. As a result, induced forces are well-distributed across the spine beams, and held to within the elastic range. This relatively low superstructure stiffness also limits restraint forces required to be resisted by the proposed abutment shear keys and guide bearings. Shear demand-capacity ratios are not presented above, but are all significantly lower than 1.0.

THERMAL PERFORMANCE ANALYSIS

Thermal Coefficient Testing

S6:19[1] prescribes a thermal coefficient of linear expansion value of 10×10^{-6} /°C in lieu of physical testing. The actual thermal coefficient of concrete is variable, and highly dependent on the thermal properties of the aggregate used, with the S6:19[1] design value being conservative for most common types of concrete aggregate. Given the length of continuous superstructure in the rearticulated configuration, it was recognized that thermal superstructure deformations would be of the same order-of-magnitude as expected seismic deformations, and thus the thermal coefficient value used could significantly impact design of the isolation bearings and abutment joints. Thermal coefficient testing of 10 concrete cores from the Cambie Bridge was conducted in accordance with AASHTO T 336-15[8]. Based on testing and statistical analysis of results, a thermal coefficient of 8.61×10^{-6} /°C was selected.

Thermal Performance Results

Figure 7 below shows the representative theoretical deformation resulting from temperature changes (expansion shown), with the deformed superstructure shown in red and the undeformed superstructure shown in grey (deformation exaggerated for clarity). As shown, transverse restraint is required at all six abutment locations to maintain superstructure deformation square to the abutment backwall, given the highly irregular superstructure geometry. This is especially apparent at the curved Nelson and Taylor Ramps, where unrestrained out-of-plane joint deformations would be large, and abutment restraint would induce transverse spine beam force demands.

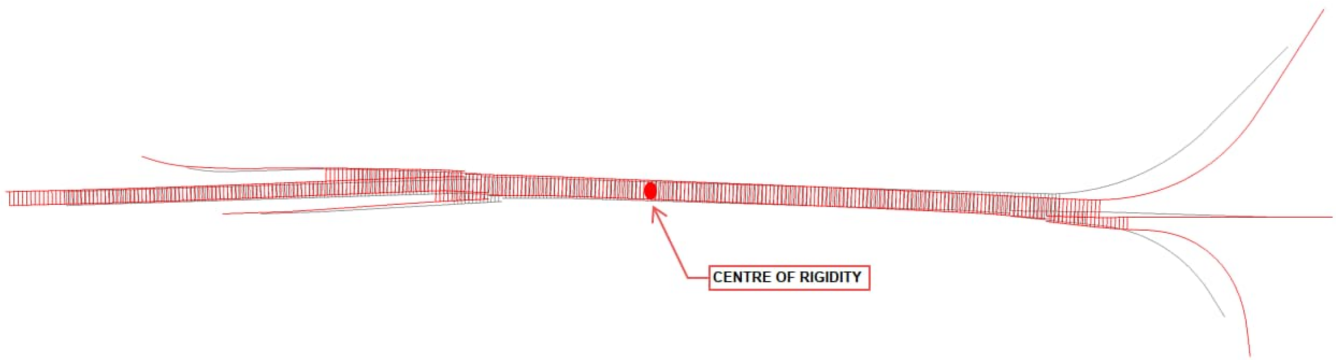


Figure 7. Retrofit superstructure thermal expansion

Detailed thermal analysis showed similar phenomena to that found in the global seismic analysis: the flexibility of the isolated superstructure allows the spine beams to undergo considerable deformation without attracting excessive flexure and shear demands. The spine beam demands and lateral forces transmitted to the pier columns by thermal deformation were found to be significantly lower than the seismic deformation.

CONCLUSIONS AND DISCUSSION

The City of Vancouver selected Cambie Bridge for seismic retrofit, with target performance criteria to permit relatively fast return to post-earthquake service with minimal repairs to key structural components. Following an initial response spectrum analysis and assessment of existing structural details, it became clear that achieving the required performance with a ductility-based retrofit strategy would be impractical. Instead, a low-damage isolation-based approach was proposed to leverage the inherent substructure stiffness and high superstructure irregularity into a cost-effective seismic retrofit that not only achieves the stringent target performance requirements, but also reduces sensitivity to uncertainty in the seismic hazard.

The use of isolation bearings presented an opportunity to eliminate the existing in-span half-joints, thus protecting the longitudinal post-tensioning anchorage zones from future chloride ingress and eliminating future inspection and maintenance of the half-joint bearings. It was recognized that this rearticulation of the superstructure would significantly alter the bridge's response to imposed loads and deformations, particularly seismic- and temperature-induced demands. A variety of linear and nonlinear global and local analyses were conducted to understand the retrofit bridge's lateral response and optimize the design of the bearings, joints, and half-joint continuity connections.

The deformation capacity and low lateral stiffness of the proposed isolation bearings enable not only a seismic retrofit strategy that addresses the City's performance goals for the bridge, but also a high-value general rehabilitation strategy that improves the structures durability and eliminates several future maintenance challenges.

ACKNOWLEDGMENTS

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