APPENDIX B

Past Condition Inspection Reports

Bridge No. D-8

Dunsmuir Viaduct

between
Beatty St. & Main St.

Construction Date and Orientation:	Built in 1969	Bridge Orientation: East-West		
Feature Supported:	Dunsmuir Street – three lanes [2 westbound and 1 bike lane].			
Feature Crossed:	Quebec St., Expo Blvd., Carrall St., Pat Quinn Way (Abbott St.).			
Substructure:	Concrete pier on concrete footing with concrete (or timber) piles.			
Superstructure:	Precast concrete I-girders (prestressed & post tensioned) with a 200 mm thick reinforced cast-in-place concrete deck.			
Wearing Surface:	Concrete.			
Approaches:	Asphalt roadway.			
Dimensions:	*No. of Spans:*	21		
	Deck Area:	848.4 m [long] x 13.9 m [wide] = 11,793 m^2		
	Skew Angle:	0º		
	Sidewalks:	North side only [1.22 m wide]		
General:	*Bearings:*	Reinforced Neoprene Pads (abutments only)		
	Bank/Pier Protection:	None		
	Guardrail:	Precast Concrete		
	Curb:	None		
	Utilities:	Unknown		
	Clearance:	Unknown (and varies)		
	Posted Speed Limit:	50 km/h		
	Sign Posting:	None		
	Design Load:	AASHTO HS-25		

Major Future Improvements Needed:	Detailed seismic assessment and retrofit design.			
Anticipated Remaining Service Life:	15 –20 years			
Estimated Bridge Replacement Cost:	$52,679,000			
Screening Level Seismic Assessment:	Priority:	High	Priority Index:	80
Bridge Condition Index [BCI] Rating:	Previous BCI:	1.44	Current BCI:	2.06

Updated: 2016 June

BRIDGE CONDITION INSPECTION

CITY OF VANCOUVER

Structure Number	D-8	Structure Name	Dunsmuir Viaduct	Inspection Date (yyyy/mm/dd)	2016 May 18

COMPONENT / **PERCENT CONDITION RATING**

Enter % in each condition.
See CoV User Guide

	COMPONENT	E	G	F	P	V	X	N
	CHANNEL							
1	Debris Risk							100
2	Bank/Bed							100
3	Dolphins/Fenders							100
	SUBSTRUCTURE							
4	Foundat'n Movement		98	2				
5	Abutments		100					
6	Wing/Retaining Wall		25	50			25	
7	Footings/Piling							100
8	Piers/Columns		100					
9	Bearings		100					
10	Caps		100					
11	Corbels							100
	SUPERSTRUCTURE							
12	Floorbeams							100
13	Stringers							100
14	Girders		100					
15	Portals							100
16	Bracing/Diaphragms							100
17	Truss Chords/Arch							100
18	Arch Ties							100
19	Truss Diagonals							100
20	Truss Rods/Verticals							100
21	Cables							100
22	Panels							100
23	Pins/Bolts/Rivets							100
24	Camber/Sag		100					
25	Live Load Vibration		100					
26	Coating (structure)							100
	DECK							
27	Sub Deck/Cross Ties		98	2				
28	Wearing Surface		100					
29	Deck Joints				100			
30	Curbs/Wheelguards							100
31	Sidewa k(s)		100					
32	Railings/Parapets		60	40				
33	Median Barrier		100					
34	Drains/Pipes		98	2				
35	Coating (Railings)							100
	APPROACHES							
36	Signing/Lighting		99	1				
37	Roadway		100					
38	Roadway Flares							100

INSPECTION NOTES BY COMPONENT

All poor or very poor conditions should be explained with notes and documented by photos. Label explanation(s) with component no.'s.

4. Foundation Movement: Settlement of approach structure at east end of on-ramp abutment (DM2) measured at 89 mm.

5. Abutments: Restrictive enclosure fencing on north side of on-ramp abutment (DM2) compromised in a 1m^2 area, possibly with assistance of a fire [Photo D1].

6. Retaining Wall: Failure of retaining wall at base of east abutment (D2) has permitted slope creep of hillslope resulting in the moderate undermining of approach structure elements. Footing on each side approach undercut for appx. 1500 mm. Issue has been known for a long period; no structural concerns observed.

9. Bearings: One bearing pad at west abutment (D23) overhung the bearing seat by 5 mm. Not a concern given the size of the pads [Photo D2].

10. Caps: Small spall with exposed rebar on south overhang soffit of Pier DM5 (above skate park) [Photo D3].

27. Sub Deck: Transverse cracks with efflorescence consistent along north and south overhangs of on-ramp approach [Photo D4].

27. Sub Deck: Large spall of deck overhang at base of north barrier in two locations of on-ramp approach (DM1 -DM2). Surrounding concrete appears sound [Photo D5].

Continued on next page (if necessary)

General Inspection Notes (Monitoring Notes):

Access to southern half of bearings at the west abutment not accessible due to limitations in reach of equipment.

Utility Concern Notes (Contact Utility Owner):

Condition Codes	
E Excellent	V Very Poor
G Good	X Not Insp.
F Fair	N n/a
P Poor	

For Condition Guidelines see CoV User Guide

Temperature

+ 14 °C

Weather

Partly Cloudy

Time of Day

7:30 am

Todd McCrimmon, P.Eng. / Aaron Pettis, P.E. – COWI North America

Lead Inspector / Inspector - Firm (please type or print)

Signature (Lead)

Inspection Notes by Component (continued):

27. Sub Deck: Black soot stains on deck soffit and surrounding members due to fire lit beneath north side of abutment [Photo D6].

27. Sub Deck: Transverse deck soffit cracks with efflorescence observed periodically throughout structure. No evidence of spalling or corrosion staining [Photo D7].

27. Sub Deck: Large deck soffit spall with exposed reinforcement on east side of Pier D6 at transition with off-ramp (above skate park). All loose material removed exposing new 200 mm x 200 mm area with (uncoated) corroded reinforcement [Photo D8].

27. Sub Deck: Exhaust staining and transverse cracks with efflorescence typical in Span D7-D8 above Skytrain tracks.

27. Sub Deck: No change since previous inspection report to timber formwork (appx. 400 mm x 300 mm) from through-deck repair still in place in Span D17-D18. Appears securely fastened. Similar formwork (appx. 200 mm x 400 x mm) in place in Span D22-D23.

27. Sub Deck: Several locations of deck soffit (and Piers) in Span D17-D18 have been mapped for embedded steel to avoid conflict when mounting decorative lighting for Rogers Arena Plaza [Photo D9].

27. Sub Deck: Removed loose mortar and confirmed soundness of three small patches on soffit overhang on south side of Span D18-D19 (over Rogers Arena Plaza) [Photo D10], and four small patches on soffit overhang on south side of Span D19-D20 (over Costco sidewalk) [Photo D11]. Two of the patches in Span D19-D20 are delaminated but could not be removed by hand [Photo D12].

27. Sub Deck: Confirmed soundness of spalled concrete areas over 3.5 m long with exposed reinforcement on north overhang of Span D20-D21 [Photo D13].

29. Deck Joints: Surface corrosion on all deck joint steel typical at deck joint overhangs [Photo D14].

29. Deck Joints: All deck joint seals are in poor condition. Most seals are visibly torn or recessed and filled with debris [Photo D15].

32. Parapets: Damage to top rail of barrier on south side of on-ramp approach entrance. No risk of fall at this location [Photo D16].

32. Parapets: Spalling on outside face of barrier observed at ten locations along south side of on-ramp approach.

32. Parapets: Loose concrete removed from 9 existing spalls on outside face south barriers, and one failed deck soffit patch in Span D14-D15 over parking lot. Four locations in Span D15-D16 and 5 locations over Abbott St. not worked on due to cars below [Photo D17].

32. Parapets: Shallow surface spalling with exposed reinforcement on south face of north parapet typical on 185 of 464 panels throughout length of structure. Majority of bars are coated with zinc-rich paint but have continued to corrode [Photo D18].

33. Median Barrier: Slight misalignment of barrier between vehicle traffic and bike lane around Pier D15 [Photo D19].

34. Drains/Pipes: Drain basin at DM5 filled with debris and vegetation [Photo D20].

36. Signing: Broken hazard sign at beginning of eastbound bike lane taper at top of on-ramp [Photo D21].

39. Utilities: Concrete utilities access box below roadway on north side of Pier D9 has corrosion stains and efflorescence. Concrete condition appears sound [Photo D22].

Inspection Photos

See attached photo log.

Structure Number	D-8	Structure Name	Dunsmuir Viaduct	Inspection Date (yyyy/mm/dd)	2016 May 18

Remedial Work Activity List

Component	Location	Activity Description	Qty.	Unit	D	%	R	U	S	Photo #
4. Foundation Movement	DM2	Monitor – Settlement of approach structure at east end of abutment.	89	mm	1	100	1	M		
5. Abutments	DM2-N	Determine if repairs to the fencing is necessary.	1	m²	3	5	1	R		D1
6. Retaining Wall	D2	Monitor – Undermining of east abutment and approach structures due to loss of soil.	1	ea.	1	100	1	M		
10. Caps	DM5-S	Monitor – Existing spall (with exposed rebar) which will fall onto skate park if future spalling occurs.	0.01	m²	1	1	1	M		D3
27. Sub Deck	D6	Monitor – Existing spall (with exposed rebar) which will fall onto skate park if future spalling occurs.	0.2	m²	2	1	2	M		D8
27. Sub Deck	D19-D20 South	Monitor – Two 200 mm x 200 mm delaminations of previous patch repairs in deck overhang above sidewalk outside Costco.	0.1	m²	1	1	2	M		D11 D12
29. Deck Joints	All	Monitor – Condition of all expansion joint seals is poor. Ensure seals are not protruding above roadway surface or resulting in other hazard for bridge users.	8	ea.	4	100	1	M		D15
32. Parapets	D15-D17 South	Monitor – Nine spall locations were not assessed for loose concrete due to vehicles below. Based on condition of nearby spalls, current threat of loose material assumed to be low.	9	ea.	2	1	2	M		D17
34. Drains	DM5	Clear catch basin blockage.	1	ea.	3	1	1	R		D20
36. Signing	DM5	Repair hazard sign for bike lane taper.	1	ea.	1	1	1	R		D21

Location Legend: DM = On-Ramp Pier; D = Mainline Pier; N = North; S = South

Rating System Legend:

Rating		Rating "D" – Degree of Condition		Rating "R" – Relevancy		Rating "U" – Urgency		Rating
		Degree of Severity of Defect		Structural Integrity and Safety of User		Maintenance Priority and Urgency of Repair	Monitor	M
E	E	Excellent	No defects, as new condition.	No defects, as new condition.		Routine maintenance work.	Routine	R
G	1	Good	Normal wear and deterioration not requiring maintenance/repair.	Minimum Relevancy	No structural integrity or safety issues.	Work not required before next detailed inspection.	≥ 5 yrs.	1
F	2	Fair	Functioning as intended. Minor maintenance/repair required.	Moderate Relevancy	Minor impact on structural integrity or safety issue.	Work required within specified time period.	< 3 yrs.	2
P	3	Poor	Not functioning as intended. More extensive repair required.	Major Relevancy	Structural integrity or safety issue compromised.	Work required within specified time period.	< 2 yrs.	3
V	4	Very Poor	Not functioning as intended. Major repair required.	Maximum Relevancy	Structural integrity and safety severely compromised. Collapse imminent and/or danger to users.	Immediate repair required.	ASAP	4

X – not inspected; N – not applicable; % – percentage of component with rating "D".
Safety Column [S]: S – Safety related defect; MS – Make safe a "S" related defect; ST – Make safe a structural integrity related defect. Otherwise left blank.

General Conditions Photo Log

COWI

Structure: D-8 Dunsmuir Viaduct

Date: 2016 May 18

G1. View west along on-ramp approach from Main Street.

G2. Typical bearing condition at on-ramp abutment (DM2).

G3. View of on-ramp span DM4-DM5 over Quebec St.

G4. General view of east abutment at D2.

G5. Typical bearing condition at east abutment (D2).

G6. On-ramp junction with mainline at Pier D6.

General Conditions Photo Log

Structure: D-8 Dunsmuir Viaduct

Date: 2016 May 18

G7. General view looking west from Pier D8.

G8. Gap between piers at expansion joint; Pier DM5 shown.

G9. General view of deck looking west from D14.

G10. East face of Pier D21 north of the Costco building.

G11. General view of west abutment at D23.

G12. Deck joint at west abutment (D23).

2016 Deficiency Photo Log

Structure: D-8 Dunsmuir Viaduct

Date: 2016 May 18

D1. Damaged fencing at on-ramp abutment enclosure (DM2).

D2. Slight overhang of bearing pad at west abutment (D23).

D3. Pier cap soffit spall with exposed rebar at Pier DM5.

D4. Cracks on deck overhang along on-ramp approach.

D5. Deck overhang spall on on-ramp approach.

D6. Staining of deck soffit from fire at on-ramp abutment.

2016 Deficiency Photo Log

COWI

Structure: D-8 Dunsmuir Viaduct

Date: 2016 May 18

D7. Deck soffit cracks of on-ramp span between DM4-DM5.

D8. Deck soffit spall with exposed rebar at Pier D6.

D9. Steel mapping outlines on pier and deck soffit D17-D18,

D10. Deck soffit spall at south overhang in span D18-D19.

D11. Deck soffit spall at south overhang in span D19-D20.

D12. Deck soffit spall at south overhang in span D19-D20.

2016 Deficiency Photo Log

COWI

Structure: D-8 Dunsmuir Viaduct

Date: 2016 May 18

D13. Spalled concrete on north deck overhang span D20-D21.

D14. Typical corrosion staining at expansion joint locations.

D15. Typical condition of deck joints; torn seals and debris.

D16. Damage to barrier concrete of on-ramp approach (DM1).

D17. Loose concrete at existing parapet spalls D14-D15.

D18. Typical spalls on south face of north parapets.

2016 Deficiency Photo Log

Structure: D-8 Dunsmuir Viaduct

Date: 2016 May 18

D19. Slight misalignment of traffic barrier near Pier D15.

D20. Drain basin filled with debris and vegetation at DM5.

D21. Broken hazard sign at on-ramp bike lane taper.

D22. Staining of concrete at base of utility box at Pier D9.

Detailed Inspection Results for the City of Vancouver - 2016 Inspections by COWI.

Structure #: D-8 Inspection Date: 2016 May 18

5 Year Repair, Rehabilitation and Maintenance Plan

Structure Name: Dunsmuir Viaduct

Component	Location	Activity Description	Qty.	Unit	Unit Rate		Base Cost+	D	%	R	U	S	Photos
5 Abutments	DM2-N	Determine if repairs to enclosure fencing is necessary.	1	m^2	$250	=	$250	3	5	1	R		D1
34 Drains	DM5	Clear catch basin blockage.	1	ea.	$250	=	$250	3	1	1	R		D20
36 Signage	DM5	Repair hazard sign for bike lane taper.	1	ea.	$250	=	$250	1	1	1	R		D21
					Routine Maintenance Sub-Total		**$750**	**(by 2017)**					

Total of Base Costs	$750
Engineering Design and Supervision (15%)	$113
Traffic Management and Site Establishment (20%)	$150
Sub-Total	$1,013
Contingency and Inflation (15%)	$152
Bridge Total Rounded Up (nearest $1,000)	**$2,000**

+ Base cost shown is best estimate and may deviate from unit rate projection when projection is deemed an unrealsitic estimate of expected repair cost.

Location Legend: DM = On-Ramp Pier

N = North

Bridge No. D-8

Dunsmuir Viaduct

between
Beatty St. & Main St.

Notes:

COWI

City of Vancouver - Engineering Services
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FAX +1 604 986 1302
WWW cowi-na.com

DATE 2016 April 04

REF 2068-015-RPT-002-0
PROJECT NO 2068

Attention: Dane Doleman, P.Eng.

Re: Bridge Inspection Consultant - Provisional Task PV1 - Monitoring Inspections

Please find enclosed inspection reports for the monitoring inspections completed as part of the 2015 Task PV1 scope for the City of Vancouver. These inspections have been carried out on all structures in the City's bridge structures network that are not receiving detailed inspections during the project year (2015), with the exception of the Burrard and Granville Bridges.

Enclosure 1 contains individual monitoring inspection reports for each of the following twenty-seven (27) bridges:

AO-3 Hastings Viaduct	D-7 Canada Place 2 Viaduct	GC-4 Commercial Drive Bridge
AO-4 Marine Dr-Bndry Rd OP	D-8 Dunsmuir Viaduct	GC-5 Broadway Bridge
D-1 Howe Street Viaduct	D-9 Georgia Viaduct	GC-6 Victoria Drive Bridge
D-2 Canada Place 1 Viaduct	D-10 Main St OP (Dunsmuir)	GC-7 Lakewood Drive Bridge
D-3 Cordova 1 Viaduct	D-11 Main St OP (Georgia)	GC-8 Nanaimo St Bridge
D-4 Cordova 2 Viaduct	FC-2 Cambie Bridge	P-1 Boundary Rd Ped OP
D-5 Cordova 3 Viaduct	GC-1 Grandview Viaduct	P-9 SEFC Canoe Bridge
D-5a Thurlow Viaduct	GC-2 Clark Drive Bridge	P-10 SEFC Weir Bridge
D-6 Burrard Viaduct	GC-3 Woodland Drive Bridge	P-11 Still Creek Ped Bridge

Significant issue(s) identified:

- D-10 Main Street Overpass (Dunsmuir): Delaminated concrete observed in three locations on the deck overhang at the south end of the west abutment appears as though it is loose enough to fall (some already has). As the hazard is above an area openly accessible to the public the loose concrete should be removed.

General Notes:

- These cursory inspections were carried out primarily for the purpose of monitoring deficiencies that were identified in previously completed detailed inspections and may therefore need to be read in conjunction with the most recent detailed inspection completed for that bridge. Reference to the 2013 and 2014 monitoring inspections may also be beneficial.
- A comprehensive inspection of all bridge elements was not undertaken such as would be typically undertaken in a detailed inspection. Access platforms were

not used and as such, bridge deck soffits and upper sections of piers were inspected from ground level with the aid of binoculars if necessary. Traffic control was also not used; therefore, all bridge wearing surface observations were made from a sidewalk, if available. In cases where a sidewalk was not present, the topside of the bridge deck was not inspected.

All inspections were undertaken by the undersigned.

Yours truly,

Todd McCrimmon, P.Eng.
COWI North America, Ltd.

Encl. 1: Monitoring Inspections for each structure, listed in order of bridge identification number.

2015 Monitoring Inspection Report

Structure: D-8 Dunsmuir Viaduct

Insp. Date: 2016 Mar 27 Insp. By: TGM

GA: View of .

FOLLOW-UP DEFICIENCIES (From Previous Inspections):

1. [5] Abutments: Encampments have remained at east abutment [P1].
2. [5] Abutments: Erosion of soil around the east abutment due to defective retaining wall [P2 &P3].
3. [8] Piers: Corrosion stains typical down the sides of expansion joint piers.
4. [29] Deck Joints: No significant change in condition of deck joints, though additional deterioration noted (as observed from the deck). Most joint seals missing/detached and recessed, filled with debris [P4].
5. [32] Parapets: Spalling of interior and/or exterior faces of parapets exposing reinforcement typical [P5].

NEW DEFICIENCIES & OBSERVATIONS (Key Notes):

1. [5] Abutments: Access to west abutment restricted [P6].
2. [10] Caps: Expansion pier at Rogers Arena plaza with joint seal applied along pier cap [P7].
3. [31] Sidewalks: Large cracks in sidewalk near east end of on-ramp.

RECOMMENDED SHORT TERM ACTIONS:

1. No safety issues. Durability repairs not recommended as structures expected to be removed from service.

P1. Encampment established at east abutment of on-ramp.

P2. Erosion of soil from north side of east abutment.

P3. Deficient retaining wall at base of east abutment that is permitting the erosion of soil.

P4. Typical deck joint with large recesses and filled with debris.

P5. Spalling to outside face of parapet at north sidewalk north of Rogers Arena.

P6. General view of west abutment.

P7. Pier cap joint seal installed at Rogers Arena Plaza.

P8. General view of main line (left) and on-ramp (right junction above the skate park.

APPENDIX D
D-8 DUNSMUIR VIADUCT

1 Introduction

This report summarizes the findings of the 2009 detailed inspection for the Dunsmuir Viaduct (D-8).

2 Existing Information

The City of Vancouver provided Associated Engineering with the following record information prior to the commencement of the inspection. An 11 x 17 copy of the GA drawing(s) is included with this report.

2.1 Past Inspection Reports

- Completed Routine Inspection Form (Form BIF 20) dated June 5, 2009
- Completed Routine Inspection Form (Form BIF 20) dated December 6, 2008
- Completed Routine Inspection Form (Form BIF 20) dated June 6, 2008
- Completed Routine Inspection Form (Form BIF 20) dated December 6, 2007
- Completed Routine Inspection Form (Form BIF 20) dated October 12, 2006
- Completed Routine Inspection Form (Form BIF 20) dated April 18, 2005

2.2 Record Drawings

- BB206-1 Site Location Plan
- BB206-2 Layout
- BB206-3 Key Plan
- BB206-4 Long Section
- BB206-5 Highway Profiles
- BB206-6 Drawing List
- BB206-7 Key Plan
- BB206-8 Typical Section 1
- BB206-9 Typical Sections 2
- BB206-10 Typical Sections 3
- BB206-11 Typical Sections 4
- BB206-12 Borehole Locations
- BB206-13 Borehole Logs
- BB206-14 Horizontal Alignment
- BB206-15 Vertical Alignment
- BB206-16 Foundation Layout 1
- BB206-17 Foundation Layout 2
- BB206-18 Alterations To Existing Facilities At D10, D12, D13, & D15

- BB206-19 Alterations To Existing Facilities At D16, D17, D18, D19, D20, D21, & D22
- BB206-23 Pier Foundation Schedule & Details
- BB206-24 Pier Foundation Misc.
- BB206-32 Dunsmuir Ramp Layout D2-D3, DM2-DM3
- BB206-33 Dunsmuir Ramp Layout D3-D5
- BB206-34 Dunsmuir Ramp Layout DM3-DM5
- BB206-35 Dunsmuir Ramp Layout D5 & DM5 To D7
- BB206-36 Dunsmuir Layout D7-D9
- BB206-37 Dunsmuir Layout D9-D11
- BB206-38 Dunsmuir Layout D11-D13
- BB206-39 Dunsmuir Layout D13-D15
- BB206-40 Dunsmuir Layout D15-D17
- BB206-41 Dunsmuir Layout D17-D19
- BB206-42 Dunsmuir Layout D19-D21
- BB206-43 Dunsmuir Layout D21-D23
- BB206-44 Dunsmuir Layout D23-Beatty
- BB206-60 Foundations Piers G3, GM3, D3, DM3
- BB206-61 Lane Markers D9 / G9 To Main Street
- BB206-62 Pier Shaft Details 1
- BB206-63 Lane Markers D20 / G22 To Beatty Street
- BB206-64 Pier Shaft Details 3
- BB206-65 Pier Shaft Details 2
- BB206-66 Pier Cross Girder Outline
- BB206-67 Pier Cross Girder Reinforcement
- BB206-68 Pier Cross Girder At D6, G6
- BB206-70 Precast Concrete Deck Stringer - Outline
- BB206-71 Precast Concrete Deck Stringer – Pre-stressing
- BB206-72 Guardrail Detail 1
- BB206-73 Guardrail Detail 2
- BB206-74 Handrails
- BB206-75 Deck Drainage 1
- BB206-76 Deck Drainage 2
- BB206-77 Deck Drainage 3
- BB206-78 Deck Drainage 4
- BB206-79 Deck Drainage 5
- BB206-80 Deck Drainage 6
- BB206-81 Deck Drainage 7
- BB206-82 Dunsmuir Ramp West Abutment Layout
- BB206-83 Dunsmuir Ramp West Abutment East Wall
- BB206-84 Dunsmuir Ramp West Abutment South Wall
- BB206-85 Dunsmuir Ramp West Abutment North Wall
- BB206-86 Dunsmuir Ramp West Abutment Tie Beam
- BB206-87 Dunsmuir Ramp West Abutment Misc.

- BB206-90 Dunsmuir Ramp East Abutment Plans And Elevations
- BB206-91 Dunsmuir Ramp East Abutment Sections
- BB206-94 Dunsmuir Main Ramp East Abutment Plans And Elevations
- BB206-95 Dunsmuir Main Ramp East Abutment Sections
- BB206-96_GM Ramp East Abut- Georgia West Abut
- BB206-100 Plumbing Landscape Development 1
- BB206-101 Plumbing Landscape Development 2
- BB206-102 Plumbing Landscape Development 3
- BB206-103 Plumbing Landscape Development 4
- BB206-104 Electrical 1
- BB206-105 Electrical 2
- BB206-106 Electrical 3
- BB206-107 Electrical 4
- BB206-108 Electrical 5
- BB206-109 Deck Diaphragms
- BB206-110 Expansion Joints
- BB206-111 Erection Sequence
- BB206-114 Lighting Pylon Foundations
- BB206-115 Lighting Masts
- BB206-116 Lighting Poles
- BB206-117 Lighting Pylon
- BB206-118 Pier G2 Footing

3 Brief Structural Description

The viaduct structure carries westbound traffic from Main Street to Beatty Street via an elevated roadway. The viaduct comprises 21 spans (counted from the east). A sidewalk is located on the north elevation and protected from traffic via a concrete parapet with steel bridge rail.

At the eastern end of the bridge, a ramp provides access from Main Street, the ramp and main viaduct structure merge in span S4. The left-hand traffic lane is closed to traffic across the bridge using temporary concrete no post barriers; this closure starts at the Main Street Ramp Merge and extends to span S17.

The structure comprises precast, prestressed concrete I-girders that span continuously between cast-in-place concrete piers. The girder ends are built-in to the pier cap diaphragms and, except for at the abutments, there are no bearings. A cast-in-place concrete slab that cantilevers out from the exterior girders completes the deck. A concrete parapet with steel bridge rail protects the traffic from the grade separation at each elevation.

Bridge expansion piers formed from two independent concrete columns accommodate thermal movement through flexure of the columns. The original joint seal at the expansion piers comprises a multi-part compression seal. The seal was left insitu during the installation of a new concrete deck overlay and a new multi-part compression seal installed at deck level above the original seal.

4 Condition Summary and Discussion

The bridge is generally in good to fair condition with some of the secondary components showing signs of deterioration.

4.1 East Approach

The area of fill retained by concrete retaining walls to the east of the structure and the Main Street ramp structure are settling. Large deflections in the bridge rail over the transition between the back of the abutment structure and approach structures are the most obvious signs. In addition to the deflections in the steel bridge rails, there are measureable height differences in adjacent units of the concrete bridge rail, the asphalt wearing-surface is cracking and a notable bump is developing in the travel lanes.

We suggest that the City seek geotechnical advice on the settlement and possibly develop a monitoring regime to give a better understanding of the problem.

4.2 Bridge Rail

The existing concrete bridge rail and traffic barriers are in fair to poor condition with delaminated and spalling concrete in isolated patches over the length of the bridge. The age and condition of the concrete combined with insufficient concrete cover is the primary cause of the spalling.

The majority of the spalling is occurring on the inside (traffic) face of the barriers, however, of particular concern are the large flakes of concrete that are spalling from the outside face of the south rail around the entrance to G.M. Place and an Impark parking lot. The deterioration discussed does not currently affect the structural capacity of the rail to retain errant vehicles.

While it is possible to undertake patch repairs to the concrete, these repairs will only provide a temporary solution with a life expectancy of around 10 to 15 years. Given the extent of the current concrete spalling and the likely future spalling along the 2.4 km of bridge rail, the City should expect to undertake extensive patching as part of an ongoing maintenance program.

In addition to concrete patching, the application of a corrosion inhibitor may slow the progression of the deterioration. To determine the cost benefit of such a product, the City may wish to trial a product over the worst attached areas of the viaduct rail. A liquid applied product such as the Sika Ferogard 903 could be applied easily by City crews for this trial.

Depending upon the future aspirations for the structure from a functional perspective the city should consider the wholesale replacement of the bridge rails. As a guide, we recommend that the City start making a financial plan for a replacement in five to ten years time.

The spalling of the outer face of the south bridge rail between G.M Place and Carrall Street raises public safety concerns, given the potential for spalling concrete to fall onto members of the public attending events at GM Place. Given the public liability issues, short life expectancy for patch repairs, and poor condition of the concrete, we recommend that the City replace this section of the rail as a priority.

In the interim to replacing this bridge rail, the City should undertake regular monitoring to identify and remove delaminated concrete that has become loose. To help minimize further concrete deterioration where reinforcement is exposed we recommend cleaning and painted the exposed reinforcement with a zinc-rich primer such as Zinga.

4.3 Soffit Patch Repairs

In a similar manor to the delamination and spalling on the bridge rail there are several existing repair patches in the soffit of the south deck cantilever that are starting to de-bond. We recommend that the City monitor the condition of these patches regularly and remove any loose concrete.

4.4 Cracking in Girders

During the inspection, we identified diagonal cracking (<0.25 mm) in the webs of the precast girders adjacent to the pier diaphragms. The cracking starts at around the mid-height of the girder web and propagates diagonally upwards away from the piers at around 45º. The cracking is more prevalent at the piers without expansion joints and can probably be attributive to the post tensioning details at these locations. The cracking is not exposed to road spray and the reinforced concrete deck slab provides additional continuity and structural redundancy over the piers. We recommend that these cracks are monitored as part of the 5 year principal inspections on the structure.

4.5 Deck Joints

The deck joints are in a poor condition, all of the joint seal elements at deck level are missing or badly torn. The inspection team could not inspect original seals below due to the debris filling the joint gap. However, given that the original seals were paved over and the expansion piers all have water staining we believe that these lower seals have also failed.

We recommend the replacement of all of the deck joints and armour at deck level. A multi-cell strip seal will provide the City with a more durable solution in this high traffic area.

5 Further Investigation or Testing Works

We recommend the City seek geotechnical advice on the settlement and possibly develop a monitoring regime to give a better understanding of the problem.

We also recommend that the City monitor and remove any spalling and loose concrete on the outside edge of the bridge rails until a more permanent solution is implemented.

6 Maintenance and Repairs

6.1 Make Safe Repairs

Table 6-1 presents repairs and recommendations required to protect members of the public. The City should treat these repairs as a high priority and completed them as soon as is practical.

Table 6-1
Make Safe Repairs

Year	Item	Location	Activity Description	Comments	Report Photographs	Quantity	Units	Total Cost
2009	4.9 Traffic Barrier or Guardrail	S12	Realign no-post traffic barrier.	Barrier out of alignment from vehicle impact.	D-18	10	m	$1,500
	5.3 Expansion Joint Cover Plates	NA - D21	Re-secure cover plates over longitudinal edge joint in sidewalk between bridge structure and adjacent sidewalk structure.	Reattached plates using Hilti nails or similar.	D-01, D-02	3	m	$1,500
2011	4.3 Underside of Deck Slab	S17	Remove timber formwork from full depth deck slab repair in Bay C, near to D18.	Timber could fall into carpark below suggest removed when man lift next rented for bridge works.		1	Ea	$1,500

Year	Item	Location	Activity Description	Comments	Report Photographs	Quantity	Units	Total Cost
2014	4.8 Parapets or Railings	AS	Consider partial replacement of existing south exterior bridge rail between G.M Place concourse and Carrall Street to remove liability from deteriorating and falling concrete.	Flakes of concrete are falling from the exterior of the concrete bridge rail due to shallow concrete cover depths and deterioration. These fakes could potentially fall onto the public around G.M Place and in the adjacent parking lot.	D-23, D-24, D-25	17 0	m	$204,000

6.2 Routine Maintenance

Table 6-2 presents identified routine maintenance items specific to the Dunsmuir Viaduct. While these items should be addressed within the next 12 months, the City should plan to repeat these activities on a regular basis.

Table 6-2
Routine Maintenance Items

Item	Location	Activity Description	Comments	Report Photographs	Quantity	Units	Total Cost
4.7 Drainage	D5	Clear deck drain and replace missing grate, in south shoulder.		D-39	1	Ea	$750
4.8 Parapets or Railings	AS	Touch-up anchor bolts for top rail with zinc-rich paint such as Zinga.		D-26	1	Ea	$1,000

6.3 General Maintenance (5-Year Plan)

Table 6.3 identifies general maintenance activities required within the next 5 years for the bridge.

Table 6-3
5-Year Maintenance Plan

Year	Item	Location	Activity Description	Comments	Report Photographs	Quantity	Units	Total Cost
2011	4.8 Parapets or Railings	ALL	Patch repair isolated spalls and delaminations by saw cutting around the exterior of the repair, chipping out around the exposed reinforcement, clean all corrosion products, and patching with a concrete repair mortar containing corrosion inhibitors.	Consider replacement of bridge rail in this section as alternative, see discussion. Consider using SikaTop® 123 Plus or equivalent for repair.	D-19 to D-25	150	m^2	$225,000
2012	4.1 Wearing Surface	S18	Repair asphalt pothole forming in south shoulder.		D-17	2	m^2	$1,500
2012	5.4 Expansion Joint Sealant or Element	ALL	Replace existing deck joints with strip seals. The existing deck joints were covered with a new system when the deck was overlaid; the upper joint has totally failed.	Replace with multi celled strip seal such as D.S. Brown's A2R-0 cellular strip seal.	D-03 to D-16	110	m	$110,000

Year	Item	Location	Activity Description	Comments	Report Photographs	Quantity	Units	Total Cost
2013	1.9 Approach Barriers	East Ramp	Patch repair isolated spalls and delaminations by saw cutting around the exterior of the repair, chipping out around the exposed reinforcement, clean all corrosion products, and patching with a concrete repair mortar containing corrosion inhibitors.	Consider SikaTop® 123 Plus or equivalent patch repair mortar.	D-19 D-21	10	m^2	$15,000
2013	2.8 Cantilevers	DM2	Patch repair isolated heavy spalls with exposed reinforcement in the Main Street ramp structure by saw cutting around the exterior of the repair, chipping out around the exposed reinforcement, clean all corrosion products, and patching with a concrete repair mortar containing corrosion inhibitors.	Located at first construction joint in approach retaining wall behind abutment. Consider SikaTop® 123 Plus or equivalent.	D-37	1	m^2	$1,500
2013	4.8 Parapets or Railings	ALL	Apply corrosion inhibitor to all exposed surfaces of bridge steel.	Following patching, consider Sika Ferogard 903 or equivalent.	D-19 To D-25	4,000	m^2	$27,500

Year	Item	Location	Activity Description	Comments	Report Photographs	Quantity	Units	Total Cost
2013	4.12 Deck Cantilevers	S20	Patch repair isolated spalls in north overhang by saw cutting around the exterior of the repair, chipping out around the exposed reinforcement, clean all corrosion products, and patching with a concrete repair mortar containing corrosion inhibitors.	Consider SikaTop® 123 Plus or equivalent.	D-27	1	m^2	$2,000

6.4 Monitoring Items

Table 6-4 identifies items that require monitoring.

Table 6-4
Monitoring Items

Component	Location	Activity Description	Further Comments	Report Photographs	Monitoring Frequency
4.8 Parapets or Railings	AS	Monitor outer face of bridge parapet for loose concrete spalls above public areas.	Monitor to ensure that new concrete delaminations are identified and removed before they can pose a risk to the general public. Recommend using a man lift such as the Genie S65 with the 5' boom extension to aid access over G.M. Place concourse.	D-23 D-24 D-25	6 mos
2.3 Front Wall	DM2	Water staining on abutment for ramp structure, monitor condition of concrete.	As part of principal inspection.	I-23	5 yrs
2.9 Traffic Barriers	S1	Monitor efflorescence and cracking in deck cantilevers.	As part of principal inspection.	D-38	5 yrs
3.2 Cap/corbel	D5 DM5 D7 D9 D11 D14 D17 D19 D21 D23	Monitor condition of concrete pier caps for deterioration below leaking deck joints.	As part of principal inspection.	D-28 D-35	5 yrs

Component	Location	Activity Description	Further Comments	Report Photographs	Monitoring Frequency
4.3 Underside of Deck Slab	Ramp S4, S4	Monitor rust staining in soffit of slab at delta where ramp structure joins main bridge structure.	As part of principal inspection.	D-33	5 yrs
4.3 Underside of Deck Slab	Ramp S4, S4	Monitor transverse cracking with efflorescence in bays A, B and C of both spans.	As part of principal inspection.	D-31 D-32	5 yrs
4.4 Exterior long membrane	All	Monitor fine diagonal cracks in girder webs at ends.	As part of principle inspection, believed hairline and 0.25 m wide.		5 yrs
4.12 Deck Cantilevers	S6	Monitor cracking and efflorescence staining on deck overhang above sky train tracks.	As part of principal inspection.	D-36	5 yrs
4.12 Deck Cantilevers	S19	Monitor stability of concrete patch repairs on edge of south deck cantilever.	Using boom lift as part of principal inspection.	D-27	5 yrs

7 Estimated Remaining Life

The Dunsmuir Viaduct was constructed circa 1973 and is generally in good condition. There do not appear to be any significant functional deficiencies at present that would limit the life expectancy of the structure.

We estimate the remaining service life of the structure to be 15 years before major rehabilitation or replacement is required.

8 Estimated Replacement Costs

Please reference Section 5 of the 2004 Detailed Bridge Inspection Program Report for details of costs included within these estimates.

Table 8-1
Order of Magnitude, Bridge Replacement Costs

Costs Item	Quantity	Estimated Costs (2009-Dollar)
Construction - Deck	7680 m²	$4,000,000
Construction - Rail/ Barrier	2400 m	$2,400,000
Construction - Joint (incl. armour)	120 m	$120,000
Construction - Superstructure/ Substructure	1	$23,430,000
Construction - Bearings	16	$50,000
Demolition		$2,000,000
	Sub Total:	**$32,000,000**
10% City Overhead		$3,200,000
10% Contingency		$3,200,000
15% Engineering and Project Delivery		$4,800,000
	TOTAL:	**$43,200,000**

Attachments:

- City of Vancouver Field Inspection Sheet
- Bridge Inventory Pictures
- Bridge Defect Pictures
- General Arrangement Drawings (two sheets)

The City of Vancouver Field Inspection Sheet

Name:	Dunsmuir Viaduct	
ID No:	D-8	BRIDGE

Current Inspection			Inspectors	Firm	Date
Principal	×	Monitoring	S. Cook, T. Aucott	Associated Eng.	July 23, 2009
Last Principal					
Last Monitoring					

Bridge type:	Concrete (precast) - continuous	No. of Spans:	21
Orientation:	East - West	Total Length:	723 m
Description:	Prestressed concrete bridge connector between Prior St and Dunsmuir St	Overall Width:	10 m
Features Supported:	Road	Deck Area:	7680 m^2
Feature Crossed:	Road, Rail, Parking Lots		

Inspection Item	R(%)				R(%)				R(%)				R(%)				R(%)				R(%)				R(%)				R(%)			
	4	3	2	1	4	3	2	1	4	3	2	1	4	3	2	1	4	3	2	1	4	3	2	1	4	3	2	1	4	3	2	1
1 Approach	1.1 Horizontal Alignment				1.2 Vertical Alignment				1.3 Embankment				1.4 Drainage				1.5 Guardrail				1.6 Sidewalk				1.7 Wearing Surface				1.8 Slope Protection			
East	100				E				E				100				100				X				E				X			
West	E				E				X				E				E				95	5			E				E			
East Ramp	100				E				100				E				75	25			E				E				100			
1 Approach	1.9 Approach Barriers				1.10 Utilities				1.11 Signage				1.12 Retaining Walls																			
East	50	50			E				E				50	50																		
West	E				E				E				X																			
East Ramp	50	50			E				E				50	50																		
2 Abutment	2.1 Bearing Seats				2.2 Back Wall				2.3 Front Wall				2.4 Piles				2.5 Foundation				2.6 Stability				2.7 Drainage				2.8 Cantilevers			
East	E				E				100				N				N				100				E				90	10		
West	N				N				100				N				N				E				E				100			
East Ramp	E				E				100				N				N				100				E				100			
2 Abutment	2.9 Traffic Barriers				2.10 Wearing Surface																											
East	75	25			E																											
West	E				E																											
East Ramp	75	25			E																											
3 Pier	3.1 Column				3.2 Cap/corbel				3.3 Foundation				3.4 Piles				3.5 Stability															
DM3	E				E				N				N				E															
DM4	E				E				N				N				E															
DM5	E				100				E				N				E															
D3	E				E				N				N				E															
D4	E				E				N				N				E															
D5	E				100				E				N				E															
D6	E				E				E				N				E															
D7	E				100				N				N				E															
D8	E				E				N				N				E															
D9	E				100				N				N				E															
D10	E				E				N				N				E															
D11	90	10			100				N				N				E															
D12	E				E				N				N				E															
D13	90	10			E				N				N				E															
D14	E				100				N				N				E															
D15	E				E				N				N				E															
D16	E				E				N				N				E															
D17	E				100				N				N				E															
D18	E				E				E				N				E															
D19	95	5			100				E				N				E															
D20	E				E				E				N				E															
D21	E				100				E				N				E															
D22	E				E				E				N				E															
4 Span	4.1 Wearing Surface				4.2 Wearing Surface Rideability				4.3 Underside of Deck slab				4.4 Exterior Long Members				4.5 Interior Long Members				4.6 Diaphragms				4.7 Drainage				4.8 Parapets or Railings			
Ramp-S1	E				E				100				E				E				E				E				100			
Ramp-S2	E				E				100				E				E				E				E				100			
Ramp-S3	E				E				100				E				E				E				E				90	10		
Ramp-S4	E				E				100				E				E				E				E				90	10		
S1	E				E				100				E				E				E				E				100			
S2	E				E				100				E				E				E				E				90	10		

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Span	4.1 Wearing Surface				4.2 Wearing Surface Rideability				4.3 Underside of Deck slab				4.4 Exterior Long Members				4.5 Interior Long Members				4.6 Diaphragms				4.7 Drainage				4.8 Parapets or Railings			
S3	E				E				100				E				E				E				E				90	10		
S4	E				E				100				E				E				E				E				90	10		
S5	E				E				E				E				E				E				E				90	10		
S6	E				E				E				E				E				E				E				90	10		
S7	E				E				E				E				E				E				E				100			
S8	E				E				E				E				E				E				E				100			
S9	E				E				E				E				E				E				75		25		100			
S10	E				E				E				E				E				E				E				100			
S11	E				E				E				E				E				80	20			E				100			
S12	E				E				E				E				E				E				E				90	10		
S13	E				E				E				E				E				E				E				100			
S14	E				E				E				E				E				E				E				100			
S15	E				E				E				E				E				E				E				100			
S16	E				E				E				E				E				E				E				80		20	
S17	E				E				100				E				E				E				E				70	10	20	
S18	98		2		E				E				E				E				E				E				90	10		
S19	E				E				E				E				E				E				E				80		20	
S20	E				E				E				E				E				E				E				80		20	
S21	E				E				E				E				E				E				E				80		20	
4 Span	4.9 Traffic Barrier or Guardrail				4.10 Signage				4.11 Utilities				4.12 Deck Cantilevers																			
Ramp-S1	75	25			E				E				E																			
Ramp-S2	75	25			E				E				E																			
Ramp-S3	75	25			E				E				E																			
Ramp-S4	75	25			E				E				E																			
S1	75	25			E				E				E																			
S2	75	25			E				E				E																			
S3	75	25			E				E				E																			
S4	75	25			E				E				E																			
S5	75	25			E				E				E																			
S6	75	25			E				E				100																			
S7	85	15			E				E				E																			
S8	75	25			E				E				E																			
S9	75	25			E					100			E																			
S10	75	25			E				E				E																			
S11	75	25			E				E				E																			
S12	75	25			E				E				E																			
S13	75	25			E				E				E																			
S14	75	25			E				E				E																			
S15	75	25			E				E				E																			
S16	75	25			E				E				E																			
S17	75	25			E				E				95	5																		
S18	75	25			E				E				95	5																		
S19	75	25			E				E				100																			
S20	75	25			E				E				95	5																		
S21	75	25			E				E				95	5																		
5 Support	5.1 Bearing Performance				5.2 Expansion Joint Nosing				5.3 Expansion Joint Cover Plates				5.4 Expansion Joint sealant or element				5.5 Expansion Joint Performance				5.6 Expansion Joint Gap @ Deck End											
East Abut	E				E				X						100				100		E											
East Ramp	E				E				X						100				100		E											
DM3	X				X				X				X				X				X											
DM4	X				X				X				X				X				X											
DM5	X				E				X						100				100		E											
D3	X				X				X				X				X				X											
D4	X				X				X				X				X				X											
D5	X				E				X						100				100		E											
D6	X				X				X				X				X				X											
D7	X				90		10		X						100				100		E											
D8	X				X				X				X				X				X											
D9	X				E				X						100				100		E											
D10	X				X				X				X				X				X											
D11	X				E				X						100				100		E											
D12	X				X				X				X				X				X											
D13	X				X				X				X				X				X											
D14	X				E				X						100				100		E											
D15	X				X				X				X				X				X											
D16	X				X				X				X				X				X											

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Support	5.1 Bearing Performance				5.2 Expansion Joint Nosing				5.3 Expansion Joint Cover Plates				5.4 Expansion Joint sealant or element				5.5 Expansion Joint Performance				5.6 Expansion Joint Gap @ Deck End											
D17	X				E				X						100				100		E											
D18	X				X				X				X				X				X											
D19	X				E				X						100				100		E											
D20	X				X				100				X				X				X											
D21	X				E				75	25			25		75		25		75		E											
D22	X				X				100				X				X				X											
West Abut	N						100		100						100				100		E											

Remedial Work Activity List

Item	Location	Activity description	Qty	Unit	U	Make Safe	Comments	Report photographs	Monitor Freq.
1.9 Approach Barriers	East Ramp	Patch repair isolated spalls and delaminations by saw cutting around the exterior of the repair, chipping out around the exposed reinforcement, clean all corrosion products, and patching with a concrete repair mortar containing corrosion inhibitors.	10	m²	3	No	Consider SikaTop® 123 Plus or equivalent patch repair mortar	D-19, D-21	
2.3 Front Wall	DM2	Water staining on abutment for ramp structure, monitor condition of concrete.	1	Ea	M	No	As part of principal inspection	I-23	5yrs
2.8 Cantilevers	DM2	Patch repair isolated heavy spalls with exposed reinforcement in the Main Street ramp structure by saw cutting around the exterior of the repair, chipping out around the exposed reinforcement, clean all corrosion products, and patching with a concrete repair mortar containing corrosion inhibitors.	1	m²	3	No	Located at first construction joint in approach retaining wall behind abutment. Consider SikaTop® 123 Plus or equivalent patch repair mortar	D-37	
2.9 Traffic Barriers	S1	Monitor efflorescence and cracking in deck cantilevers.	1	Ea	M	No	As part of principal inspection.	D-38	5 yrs
3.2 Cap/corbel	D5,DM5, D7,D9, D11,D14, D17,D19, D21,D23	Monitor condition of concrete pier caps for deterioration below leaking deck joints.	1	Ea	M	No	As part of principal inspection.	D-28, D-35	5 yrs
4.1 Wearing Surface	S18	Repair asphalt pothole forming in south shoulder.	2	m²	2	No		D-17	
4.3 Underside of Deck slab	S17	Remove timber formwork from full depth deck slab repair in Bay C, near to D18.	1	Ea	3	Yes	Timber could fall into carpark below.		
4.3 Underside of Deck slab	Ramp S4, S4	Monitor rust staining in soffit of slab at delta where ramp structure joins main bridge structure.	1	Ea	M	No	As part of principal inspection.	D-33	5 yrs
4.3 Underside of Deck slab	Ramp S4, S4	Monitor transverse cracking with efflorescence in bays A, B and C of both spans.	1	Ea	M	No	As part of principal inspection.	D-31, D-32	5 yrs
4.4 Exterior Long Members	All	Monitor fine diagonal cracking in webs of girders at girder ends.	1	Ea	M	No	Existing Cracks between Hairline and 0.25 mm wide.	D-28, D-29	5 yrs
4.7 Drainage	D5	Clear deck drain and replace missing grate, in south shoulder.	1	Ea	R	No		D-39	
4.8 Parapets or Railings	AS	Touchup anchor bolts for top rail with Zink rich paint such as Zinga.	1	Ea	R	No		D-26	
4.8 Parapets or Railings	All	Patch repair isolated spalls and delaminations by saw cutting around the exterior of the repair, chipping out around the exposed reinforcement, clean all corrosion products, and patching with a concrete repair mortar containing corrosion inhibitors.	150	m²	3	No	Consider replacment of bridge rail in this section as alternative, see discussion. Use SikaTop® 123 Plus or equivalent patch repair mortar.	D-19 to D-25	
4.8 Parapets or Railings	AS	Monitor outer face of bridge parapet for loose concrete spalls above public areas. Monitor repair patches on bridge deck overhang (soffit). Several patches are debonding.	1	Ea	M	Yes	Monitor to ensure that new concrete delaminations are identified and removed before they can pose a risk to the general public. Recommend using a man lift such as the Genie S65 with the 5' boom extension to aid access over G.M. Place concourse.	D-23, D-24, D-25	6 Mon

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Remedial Work Activity List

Item	Location	Activity description	Qty	Unit	U	Make Safe	Comments	Report photographs	Monitor Freq.
4.8 Parapets or Railings	AS	**Alternative:** Consider partial replacement of existing south exterior bridge rail between G.M Place concourse and Carrall Street to remove liability from deteriorating and falling concrete.	170	m	2	Yes	Flakes of concrete are falling from the exterior of the concrete bridge rail due to shallow concrete cover depths and deterioration. These fakes could potentially fall onto the public around G.M Place and in the adjacent parking lot.	D-23, D-24, D-25	
4.9 Traffic Barrier or Guardrail	S12	Realign no-post traffic barrier.	10	m	1	Yes	Barrier out of alignment from vehicle impact.	D-18	
4.12 Deck Cantilevers	S6	Monitor cracking and efflorescence staining on deck overhang above sky train tracks.	1	Ea	M	No	As part of principal inspection.	D-36	5 yrs
4.12 Deck Cantilevers	S19	Monitor stability of concrete patch repairs on edge of south deck cantilever.	1	Ea	M	Yes	Using boom lift, as part of principal inspection.	D-27	5 yrs
4.12 Deck Cantilevers	S20	Patch repair isolated spalls in north overhang by saw cutting around the exterior of the repair, chipping out around the exposed reinforcement, clean all corrosion products, and patching with a concrete repair mortar containing corrosion inhibitors.	1	m²	3	No	Consider SikaTop® 123 Plus or equivalent patch repair mortar.	D-27	
5.3 Expansion Joint Cover Plates	NA - D21	Re-secure cover plates over longitudinal edge joint in sidewalk between bridge structure and adjacent sidewalk structure.	3	m	1	Yes	Reattached plates using Hilti nails or similar.	D-01, D-02	
5.4 Expansion Joint sealant or element	ALL	Replace existing deck joints with strip seals.	110	m	2	No	The existing deck joints where covered with a new system when the deck was overlaid, the upper joint has totally failed. Replace with multi celled strip seal such as D S Brown's A2R-0 cellular strip seal.	D-03 to D-16	
4.8 Parapets or Railings	All	Consider using spry-on corrosion inhibitor on bridge rail to slow concrete deterioration.	4000	m²	3	No	Consider Sika Ferrogard 903 or equivalent.		

1.1 Horizontal Alignment	East Ramp	**Inspection Note:** Intersection at end of ramp structure, affecting horizontal alignment for bridge approach.				No	No action required.		
3.1 Column	D11	**Inspection Note:** Minor chipping at base of concrete column.				No	No action required.		
3.1 Column	D19	**Inspection Note:** Concrete chipping on corners of column near base.				No	No action required.		
4.5 Interior Long Members	S11	**Inspection Note:** Concrete chip to bottom flange of girder C at eastern diaphragm location.				No	No action required.		

Inspectors Assessment and Further Comments

East Approach

The area of fill retained by concrete retaining walls to the east of the structure and the Main Street ramp structure are settling. Large deflections in the bridge rail over the transition between the back of the abutment structure and approach structures are the most obvious signs. In addition to the deflections in the steel bridge rails, there are measureable height differences in adjacent units of the concrete bridge rail, the asphalt wearing-surface is cracking and a notable bump is developing in the travel lanes.

Bridge Rail

The existing concrete bridge rail and traffic barriers are in fair to poor condition with delaminated and spalling concrete in isolated patches over the length of the bridge. The age and condition of the concrete combined with insufficient concrete cover is the primary cause of the spalling.

The majority of the spalling is occurring on the inside (traffic) face of the barriers, however, of particular concern are the large flakes of concrete that are spalling from the outside face of the south rail around the entrance to G.M. Place and an Impark parking lot. The deterioration discussed does not currently affect the structural capacity of the rail to retain errant vehicles.

Whilst it is possible to undertake patch repairs to the concrete, these repairs will only provide a temporary solution with a life expectancy of around 10 to 15 years. Given the extent of the current concrete spalling and the likely future spalling the 2.4 km of bridge rail on the viaduct the City should expect to undertake extensive patching as part of an ongoing maintenance program.

# The City of Vancouver Field Inspection Sheet	Name:	Dunsmuir Viaduct	
	ID No:	D-8	BRIDGE

Inspectors Assessment and Further Comments

Depending upon the future aspirations for the structure from a functional perspective the city should consider the wholesale replacement of the bridge rails. As a guide, we recommend that the city start making finical plan for a replacement in 5 to 10 years time.

The spalling of the outer face of the south bridge rail between G.M Place and Carrall Street raises public safety concerns, given the potential for spalling concrete to fall onto members of the public attending events at G.M. Place. Given the public liability issues, short life expectancy for patch repairs, and poor condition of the concrete, we recommend that the City replace this section of the rail as a priority.

In the interim to replacing this bridge rail the City should undertake regular monitoring to identify and remove delaminated concrete that has become loose. To help minimize further concrete deterioration where reinforcement is exposed we recommend cleaning and painted the exposed reinforcement with a Zink rich Primer such as Zinga.

Soffit Patch Repairs

In a similar manor to the delamination and spalling on the bridge rail there are several existing repair patches in the soffit of the south deck cantilever that are starting to de-bond. We recommend that the City monitor the condition of these patches regularly and remove any loose concrete.

Cracking in Girders

During the inspection, we identified diagonal cracking (<0.25 mm) in the webs of the precast girders adjacent to the pier diaphragms. The cracking starts at around the midheight of the girder web and propagates diagonally upwards away from the piers at around 45º. The cracking is more prevalent at the piers without expansion joints and can probably be attributive to the post tensioning details at these locations. The cracking is not exposed to road spray and the reinforced concrete deck slab provides additional continuity and structural redundancy over the piers. We recommend that these cracks are monitored as part of the 5 year principal inspections on the structure.

Deck Joints

The deck joints are in a poor condition, all of the joint seals elements at deck level are missing or badly torn. The inspection team could not inspect original seals below due to the debris filling the joint gap. However, given that the original seals were paved over and the expansion piers all have water staining we believe that these lower seals have also failed.

We recommend the replacement of all of the deck joints and armor at deck level. A multi-cell strip seal will provide the City with a more durable solution in this high traffic area.

Rating "R"		Structural integrity and safety of user	Functionality - secondary to structural integrity. Does it perform as originally designed?	Maintenance priority and urgency of repair	Urgency "U"	
					Monitor	M
E		No Defects	New Condition - No Defects	Not applicable	Routine	R
4	G	Min Relevancy	Acceptable, functioning as intended but maintenance required	Not required before next principal inspection	5ys or >	4
3	F		Functioning as intended. Minor to more extensive rehab required to upgrade to new	Preventative maintenance required within specified period	< 3yrs	3
2	P		Unacceptable, not functioning as intended. Major rehabilitation required	Work required within specified period	< 2yrs	2
1	V	Max Relevancy	Immediate action. Collapse imminent	Danger to users - immediate repair required	ASAP	1

N - Not Applicable; X - Not applicable; % - Percentage of Element Representing Rating "R";
NA - North Abutment; AL - All Supports: AS - All Spans; S1 - Span 1; P1 - Pier 1; AP - All Piers; P1-3 - Piers 1 to 3

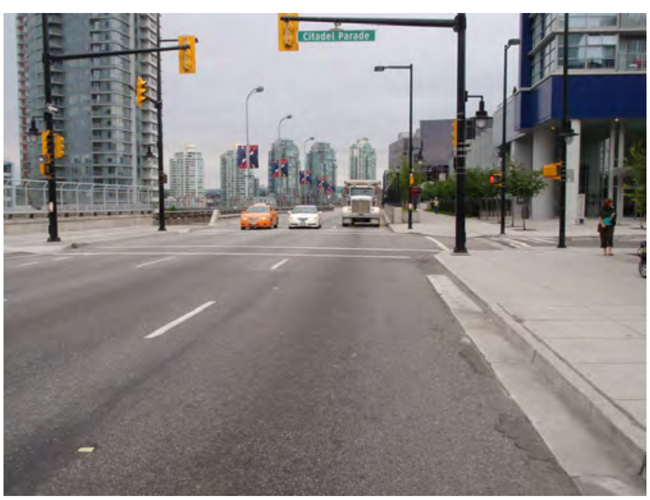
City of Vancouver
2009 Detailed Bridge Inspections - Inventory Pictures
D-8 Dunsmuir Viaduct

D-8 I-01.jpg
North elevation looking west.

D-8 I-02.jpg
North elevation, looking east.

D-8 I-03.jpg
South elevation.

D-8 I-04.jpg
Looking west from east abutment at main structure and ramp.



D-8 I-05.jpg
West approach.

D-8 I-06.jpg
Longitudinal joint between deck and sidewalk structure at west approach.

City of Vancouver
2009 Detailed Bridge Inspections - Inventory Pictures
D-8 Dunsmuir Viaduct

D-8 I-07.jpg
Compression seal in longitudinal joint between deck and sidewalk structure.

D-8 I-08.jpg
View looking east along viaduct.

D-8 I-09.jpg
View looking east along viaduct.

D-8 I-10.jpg
View Looking West, note misaligned No-Post barriers.

D-8 I-11.jpg
View looking west along viaduct.

D-8 I-12.jpg
East approach.

D-8 I-13.jpg
Main Street ramp - looking east.

D-8 I-14.jpg
Sidewalk flair and replacment bridge rail in west approach.

D-8 I-15.jpg
Typcial sidewalk on ramp.

D-8 I-16.jpg
Typical wearing surface.

D-8 I-17.jpg
Typical catch basin.

D-8 I-18.jpg
Typcial sidewalk drain.

D-8 I-19.jpg
Main Street ramp.

D-8 I-20.jpg
Typical soffit.

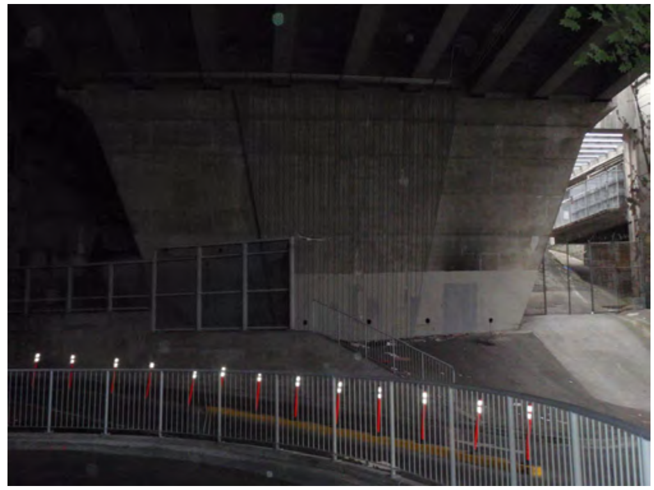
D-8 I-21.jpg
D6, south elevation.

D-8 I-22.jpg
D13 west elevation.

D-8 I-23.jpg
Main Street ramp, east abutment (DM2).

D-8 I-24.jpg
Typical abutment bearing.

D-8 I-25.jpg
Enclosure at east abutment of main street ramp.

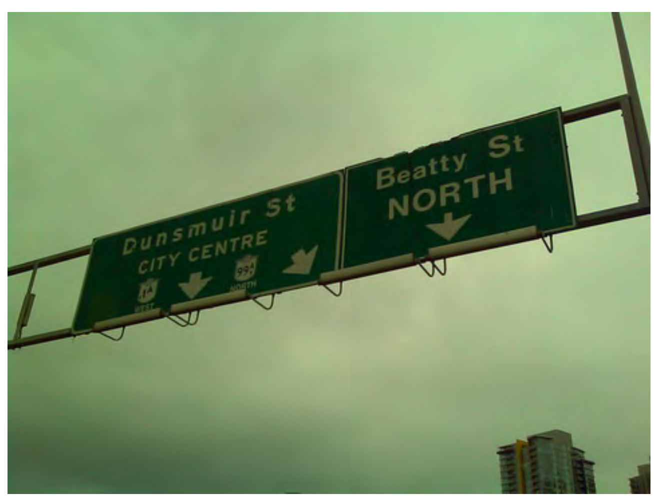


D-8 I-26.jpg
West abutment.

D-8 I-27.jpg
Cage around west abutment.

D-8 I-28.jpg
D2, east abutment.

D-8 I-29.jpg
Typical bearing at Main Street ramp east abutment.



D-8 I-30.jpg
Typical sign.

D-8 D-01.jpg
Longitudinal joint in sidewalk, note cover plate hold down screws missing.

D-8 D-02.jpg
Longitudinal joint in sidewalk, note cover plate lifting.

D-8 D-03.jpg
D23 west abutment deck joint.

D-8 D-04.jpg
Debris in west abutment deck joint, note no seal at deck level.

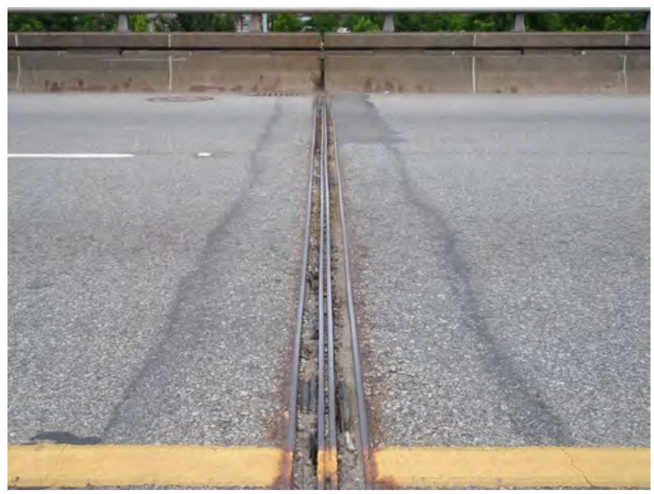
D-8 D-05.jpg
D21 Deck joint, note failed seal element and debris filling joint gap.

D-8 D-06.jpg
D19 Deck joint, note failed seal element and debris filling joint gap.

D-8 D-07.jpg
D17 Deck joint, note failed seal element and debris filling joint gap.

D-8 D-08.jpg
D14 Deck joint, note failed seal element and debris filling joint gap.

D-8 D-09.jpg
D11 Deck joint, note failed seal element and debris filling joint gap.



D-8 D-10.jpg
D9 Deck joint, note failed seal element and debris filling joint gap.

D-8 D-11.jpg
D7 Deck joint, note failed seal element and debris filling joint gap.

D-8 D-12.jpg
D7 kink in deck joint armour.

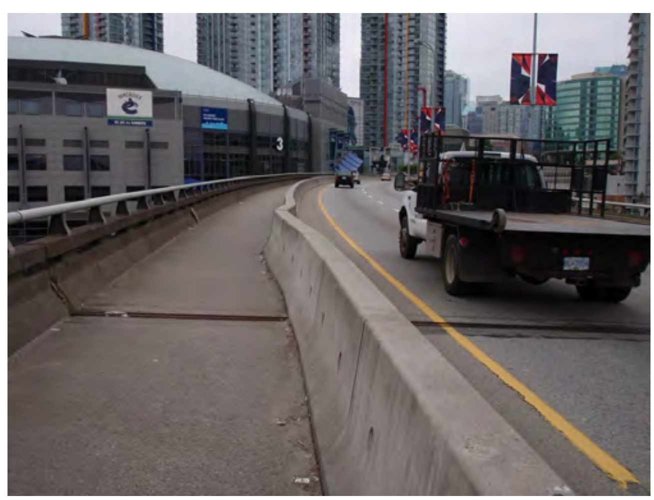
D-8 D-13.jpg
D5 Deck joint, note failed seal element and debris filling joint gap.

D-8 D-14.jpg
DM5 Deck joint, note failed seal element and debris filling joint gap.

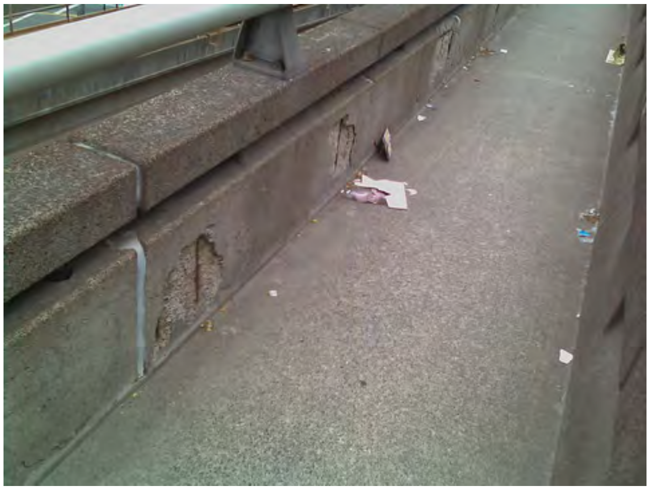
D-8 D-15.jpg
DM2, Ramp, east abutment joint , note missing seal and debris in gap.

D-8 D-16.jpg
D2, East abutment deck joint, note missing seal, corroded armor and gap filled with debris.

D-8 D-17.jpg
Minor deterioation of aspahlt wearing surface in span S18.



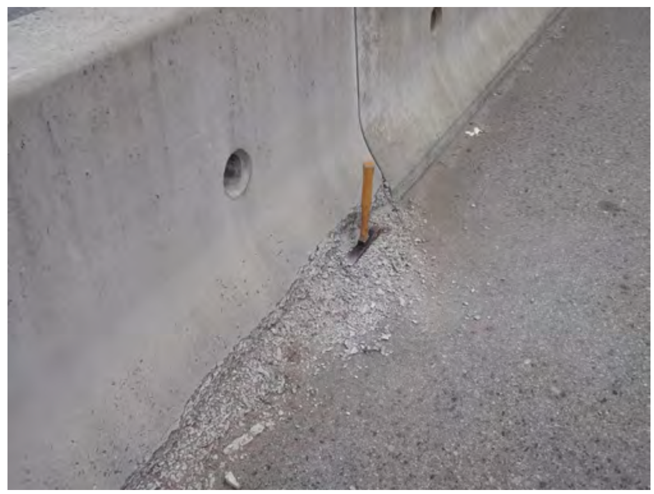
D-8 D-18.jpg
No-Post traffic barriers missaligned from vehicle impact near D18.



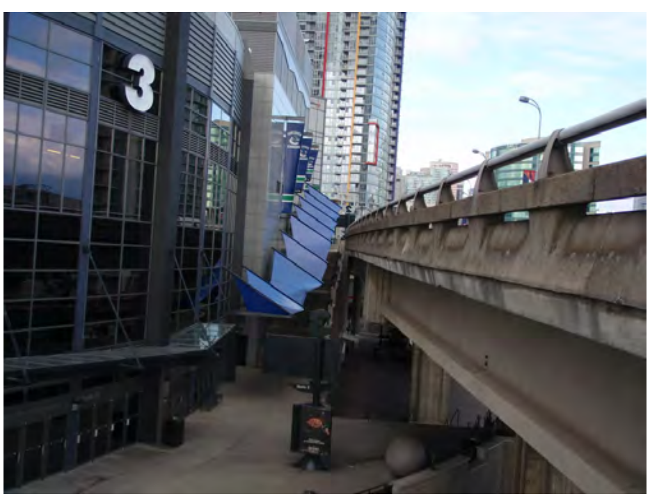
D-8 D-19.jpg
Typical spalls with exposed reinforcement in sidewalk rail.

D-8 D-20.jpg
Spalling around Expansion joint mointoring device.

D-8 D-21.jpg
Typical spalling along top of bridge rail.



D-8 D-22.jpg
Deterioration of No-Post barrier due to poor quality concrete.



D-8 D-23.jpg
South elevation above G.M. Place concourse.

D-8 D-24.jpg
Typcial spall in outside of south bridge rail above G.M. Place and parking lots.

D-8 D-25.jpg
Example of spalling concrete from outside face of bridge rail.

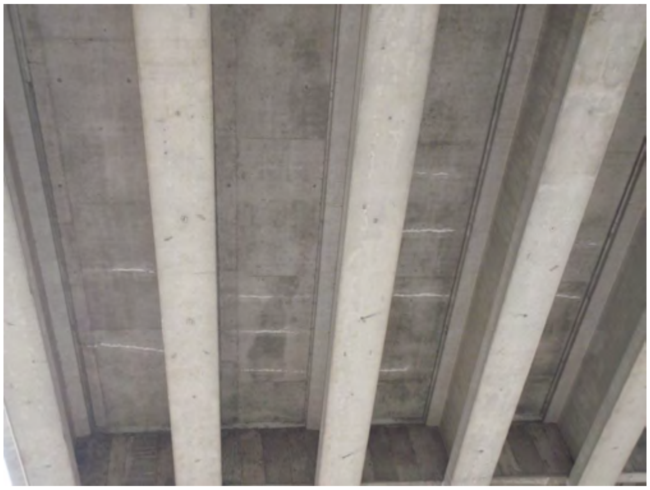
D-8 D-26.jpg
Surface Corrosion on bolts for bridge rail.

D-8 D-27.jpg
Debonding patch on south deck cantilever above G.M Place concourse.

D-8 D-28.jpg
Water staining and efflorescence around catch basin in bay E at pier D9.

D-8 D-29.jpg
Note cracking and efflorescence in catch basin chamber at east abutment.

D-8 D-30.jpg
Water and rust staining from cold joint between catch basin chamber and precast girder at pier D8.



D-8 D-31.jpg
Transverse cracking with efflorescence in subdeck of S5.

D-8 D-32.jpg
Transverse cracking and efflorescence (1m centres) in subdeck of ramp structure.

D-8 D-33.jpg
Rust staining at construction joint where ramp and main viaduct diverge.

D-8 D-34.jpg
Typical spall on deck overhang.

D-8 D-35.jpg
Typcial water staining on expansion joint pier.

D-8 D-36.jpg
Water staining and efflorescence at expansion joint on D7.

D-8 D-37.jpg
Large concrete spall with exposed reinforcement at back of east abutment of ramp, note settelment at construction joint.

D-8 D-38.jpg
Note cracking and efflorescence stainnig on retaining wall overhang at east approach.

D-8 D-39.jpg
Missing catch basin grid in north shoulder at D7

D-8 D-40.jpg
Sag in PVC surface water drain by Pier D8

D-8 D-41.jpg
Typical encampment at east abutments.

D-8 D-42.jpg
Crack in manhole cover in S11.

s.13(1)

s.13(1)

Detailed Design of Road & Utilities in the Northeast False Creek Project

17M-00475-01 | PS20161278

DUNSMUIR VIADUCT

Seismic Retrofit Strategy Report

NEFCAP-MMM-S-RPT-017_0

Rev 0 | Oct 30, 2017

REVISION HISTORY:

Revision	Date	Description	Author	Approver
0	Oct 30, 2017	Re-issued as Rev. 0	Reza Saiedi	Monty Knaus
C	Oct 24, 2017	Revised as per City Comments	Reza Saiedi	Monty Knaus
B	Oct 16, 2017	Revised as per City Comments	Reza Saiedi	Monty Knaus
A	June 15, 2017	Draft Submission	Reza Saiedi	Monty Knaus

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APPENDICES:

- Appendix A: General Arrangement Drawings
- Appendix B: Structural Design Brief
- Appendix C: Geotechnical Report

EXECUTIVE SUMMARY

At the request of the City of Vancouver (the City) and as a component of the "Detailed Design of Roads and Utilities in the Northeast False Creek" (RFP No. PS20161278), WSP completed a seismic assessment and developed a retrofit design for a six span segment of the Dunsmuir Viaduct between Citadel Parade (abutment D23) and Pat Quinn Way (pier D17) in downtown Vancouver, BC.

Seismic performance criteria were defined as those for a "major-route" structure, required to facilitate immediate post-earthquake emergency response after inspection. Accordingly, service and damage criteria were defined for the 2475 and 975 year return period hazard levels.

A simplified 3D model was created to represent the existing structure including topside modifications for the parks design. Columns were conservatively assumed to be fixed at their base. The gaps between the three two-span continuous frames were captured using tension and compression models. Design response spectra were calculated for Site Class C ground conditions, assuming ground improvements are implemented where necessary. A response spectrum analysis was then performed to obtain demands at various seismic hazard levels. These demands were compared with unfactored nominal member capacities. No pushover or time-history analyses were carried out for this initial assessment.

The following seismic deficiencies were identified, and where possible, expressed as demand to capacity ratios:

- Columns D22 to D18:
  - Poor detailing of reinforcing steel
  - Insufficient shear and torsional capacity
  - Insufficient flexural capacity
- Liquefaction potential at D18 and D17
- Foundations D22 to D17: Insufficient resistance to overturning and sliding
- Pull-out of girder reinforcement from pier caps at D21 and D19
- Insufficient bearing seat length at abutment
- Insufficient transverse resistance at abutment shear key
- Insufficient gap at interface with Costco / Spectrum building

Seismic retrofits put forward to rectify these deficiencies include:

- Column jackets at D22 to D18
- Foundation anchors and overlays at D22 to D17
- Ground improvements at D18 and D17
- Girder - pier cap connectors at D21 and D19
- Link slabs at D21 and D19
- Abutment: Seat length extensions
- Abutment: New shear keys
- New joint at interface with Costco / Spectrum building
- New pier at D17 to support the Dunsmuir Elevated Park

In addition to the above, general rehabilitation required includes:

- Above-deck modifications as per parks design
- New joint at abutment
- Miscellaneous defect repairs
- Deck rehabilitation (extent to be confirmed following deck evaluation)

The proposed seismic retrofits tie the three two-span frames together to form one continuous six-span structure, stiffen the columns, stabilize the foundations, and release longitudinal movement at D17. Structural confirmation checks were carried out on the proposed retrofit scheme, to verify both seismic and in-service performance. While pushover analyses were not carried out as part of this initial assessment, the retrofitted viaduct is expected to achieve the performance criteria for major-route bridges at the 2475 year return period hazard level. Thermal expansion and contraction of the structure is expected to cause limited cracking in the superstructure during the coldest and hottest months of the year. However, these cracks are not expected to affect the load carrying capacity of the bridge, and can be monitored and repaired if necessary.

The suggested seismic retrofits, general rehabilitation, and deck rehabilitation are estimated to cost approximately $4.2M, $0.3M, and $2.1M, respectively. This is a "Class D" cost estimate (+30%). The tender price for all these works including 5% for general costs is $6.9M, and the total cost with 30% contingency is $9.0M.

1. INTRODUCTION

At the request of the City of Vancouver (the City) and as a component of the "Detailed Design of Roads and Utilities in the Northeast False Creek" (RFP No. PS20161278), WSP have completed a seismic assessment and developed a retrofit strategy for a six span segment of the Dunsmuir Viaduct between Citadel Parade (abutment D23) and Pat Quinn Way (pier D17) in downtown Vancouver, BC (Figure 1). The required scope of work is itemized in the City's Request for Proposals dated October 14, 2016, and our Technical Proposal dated November 21, 2016, and includes the following:

- Performing all structural, geotechnical, and soil structure interaction analysis and assessment work to develop the Seismic Retrofit Strategy according to the latest applicable design codes.
- Attending a meeting with the City following the preliminary seismic retrofit strategy to reach consensus for the final design.
- Providing cost estimates for seismic retrofit to the different performance levels specified in the design criteria.

1.1. Bridge Location

Figure 1 shows the portion of the Dunsmuir Viaduct proposed to be retained for future use by cyclists and pedestrians.

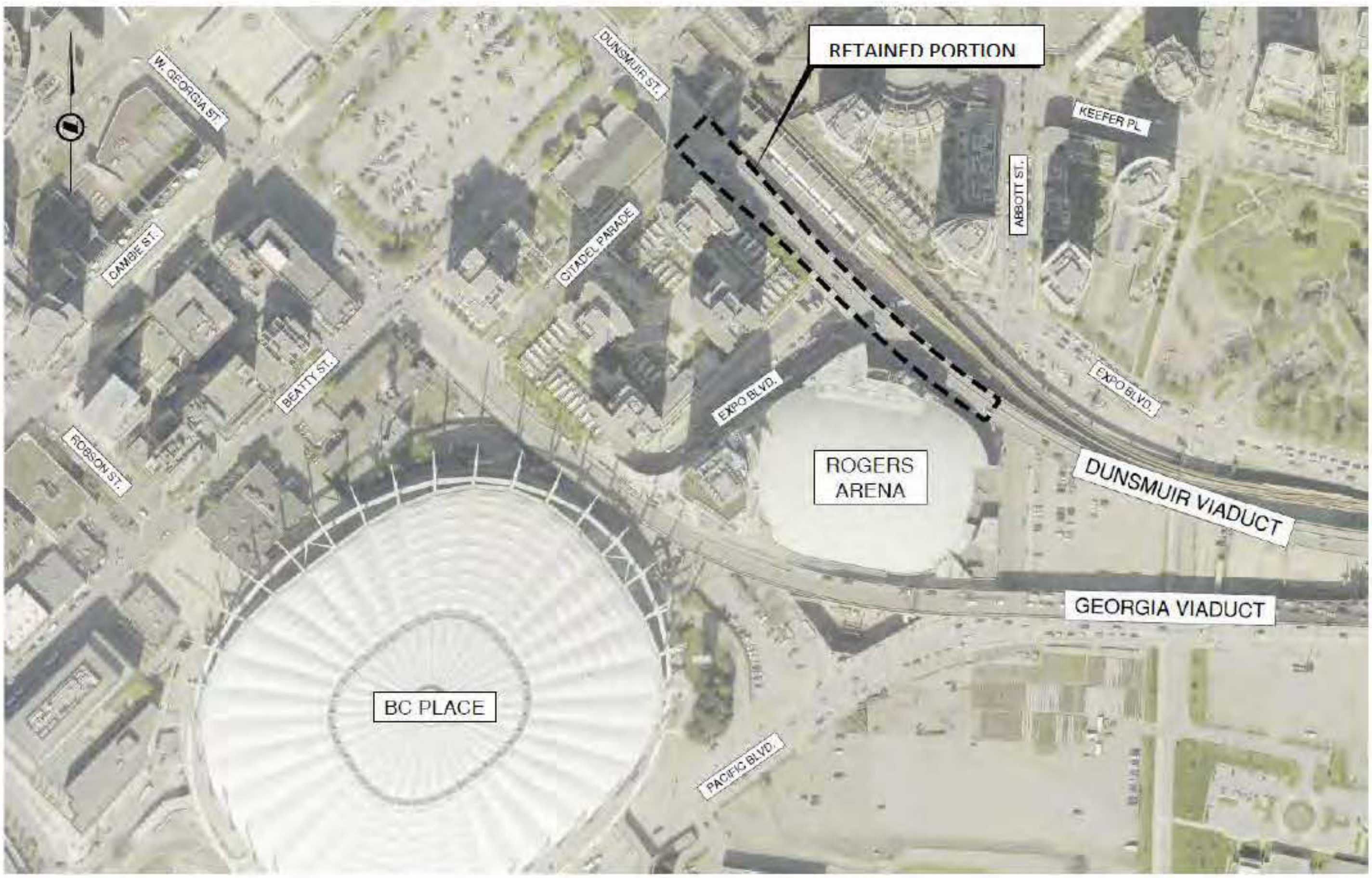


Figure 1. Bridge location, Downtown Vancouver

1.2. Review of Existing Information

The following information received from the City was reviewed in detail:

- Original record drawings:
  - City of Vancouver (1969-1973). "*Georgia Viaduct Replacement, Contract No. 2*", Drawings 6857-200 to 299, 2100 to 2109, 801 to 805.
- Previous seismic assessment reports:
  - MMM Group (2015). "*Georgia and Dunsmuir Viaducts, Limited Seismic Scope Study, Final Report*", Project No. 5014103-003, May 2015.
  - MMM Group (2014). "*Georgia Viaduct Seismic Assessment Report*", Project No. 5013312-001, June 2014.
  - Cochrane Engineering (2004). "*Conceptual Design Report – Seismic Assessment of the Georgia and Dunsmuir Viaducts*", Feb 2004.

Detailed visual inspections of the viaduct were performed on February 2 and March 30, 2017. The Condition Assessment Report (WSP, 2017) summarizes our findings.

1.3. Bridge Configuration

The Dunsmuir Viaduct was built in 1970-1971. The 1.0 km long structure starts at Gore Avenue in the east, spans over Main Street, Quebec Street, Expo Boulevard and Pat Quinn Way, and ends at Citadel Parade in the west. The portion from the west abutment D23 (at Citadel Parade) to pier D17 (at Pat Quinn Way) is proposed to be retained, while the remaining portions east of D17 will be demolished and replaced with the new Dunsmuir Elevated Park (Figure 2).

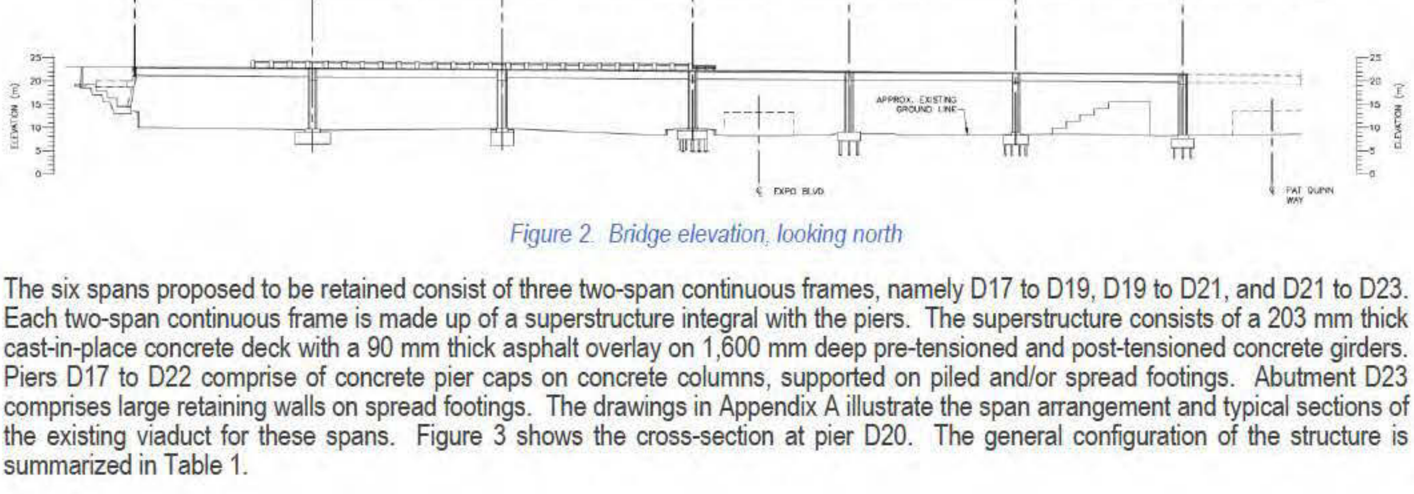


Figure 2. Bridge elevation, looking north

The six spans proposed to be retained consist of three two-span continuous frames, namely D17 to D19, D19 to D21, and D21 to D23. Each two-span continuous frame is made up of a superstructure integral with the piers. The superstructure consists of a 203 mm thick cast-in-place concrete deck with a 90 mm thick asphalt overlay on 1,600 mm deep pre-tensioned and post-tensioned concrete girders. Piers D17 to D22 comprise of concrete pier caps on concrete columns, supported on piled and/or spread footings. Abutment D23 comprises large retaining walls on spread footings. The drawings in Appendix A illustrate the span arrangement and typical sections of the existing viaduct for these spans. Figure 3 shows the cross-section at pier D20. The general configuration of the structure is summarized in Table 1.

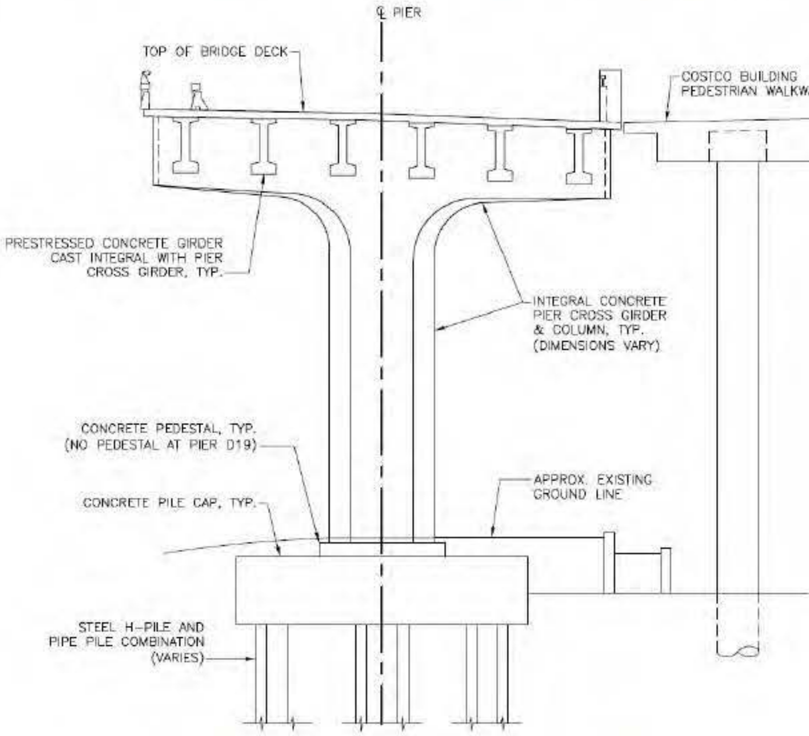


Figure 3. Cross-section at pier D20, looking east

Table 1: Bridge Configuration Summary

Element	Configuration & Material
Construction Date	1970-1971
Span Arrangement	D17-D19: 35.8 m, 35.8 m D19-D21: 33.5 m, 40.9 m D21-D23: 40.7 m, 38.1 m All span lengths noted are between centrelines of supporting piers / abutment bearings.
Lane Configuration	Existing configuration: From south to north: Shoulder (~0.76 m), two westbound vehicle lanes (~3.65 m each), bi-directional cycling lane (~3.04 m clear), pedestrian walkway (1.22 m clear min.) Final Configuration: Bi-directional cycling lane (4.5 m clear), pedestrian walkway (4.5 m clear min.), landscaped area (varies)
Piers (D17 to D22)	Cast-in-place concrete piers founded on expanded base piles at D17, D18, D19, H-piles and expanded base piles at D20, and spread footings at D21, D22
Abutment (D23)	Cast-in-place concrete abutment wall and wing walls founded on spread footings
Girders	1600 mm deep prestressed concrete I-girders integral with piers
Bearings	Laminated elastomeric bearings at abutment D23
Deck	203 mm cast-in-place concrete deck, 90 mm (estimated) asphalt overlay
Joints	Neoprene compression seals with steel armouring at abutment D23 and piers D21, D19 and D17. Neoprene compression seal at longitudinal joint above Costco / Spectrum building.

2. DESIGN CRITERIA

The seismic assessment design criteria is based on Section 4 of CAN/CSA S6-14 and BC MoTI Supplement to S6-14. The City of Vancouver has categorized the bridge as a major-route bridge. Major routes are those required for immediate post-earthquake emergency response. This categorization mandates the following performance criteria:

- 2475 year return period hazard level:
  - Service disruption. The bridge shall be usable for restricted emergency traffic after inspection. The bridge shall be repairable. Repairs to restore bridge to full service might require bridge closure.
  - Extensive damage: Inelastic behaviour is expected. Members might have extensive visible damage, such as spalling of concrete and buckling of braces but strength degradation is not permitted. Members shall be capable of supporting dead loads plus one lane of live load in each direction (to account for emergency vehicles), including P-delta effects, without collapse.
- 975 year return period hazard level:
  - Service limited: The bridge shall be usable for emergency traffic and be repairable without requiring bridge closure. At least 50% of the lanes, but not less than one lane shall remain operational. If damaged, normal service shall be restored within a month.
  - Repairable damage: The bridge may experience inelastic behaviour, however, primary members shall be repairable in place and shall be capable of supporting the dead load plus live load corresponding to the service performance criteria during repairs.

Detailed seismic design criteria can be found in Appendix B: Design Brief.

3. METHODOLOGY

3.1. Structural Model

A 3D model of the bridge was created using CSiBridge software (Figure 4).

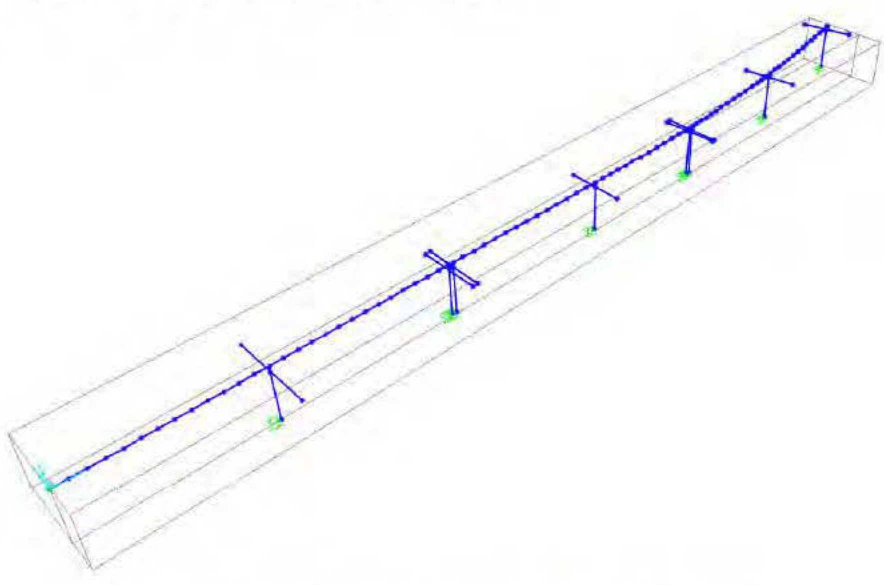


Figure 4. 3D model of existing structure (left to right: D23 to D17)

The superstructure was modelled as a spine element of equivalent cross-sectional area and moments of inertia, located at its centre of mass. This element captures the overall geometry of the bridge structure, including its plan curvature from D20 to D17, and the varying deck width and number of girders from D22 to D20. Pier caps and columns were modelled as beam elements and connected to the superstructure with rigid links. Each half of the split piers was modelled individually. The columns were assigned effective stiffnesses based on their cracked sections.

For this initial assessment, the superstructure and piers were modelled in their current condition. The deck, however, was modelled in its proposed configuration for future use. It was assumed that the existing 203 mm thick deck and 90 mm thick asphalt overlay would remain, and a strip of landscaping (4.5 m wide x 0.4 m deep) consisting of saturated soil would be added along the centreline of the deck. The asphalt and landscaping weight were included as an additional mass.

3.2. Boundary Conditions

3.2.1. Supports

The geotechnical report (Appendix C) recommends foundation springs for use in structural models. For this initial assessment, however, the pier columns were conservatively assumed to be fixed at their base. Therefore, all pier foundations were effectively excluded from the structural model. While this is a conservative assumption for the existing structure, it is a more accurate representation of the retrofitted structure in which foundations would be tied down using soil anchors (Section 5.4), and are therefore fixed against translation and rotation.

The west abutment was modelled as fixed transversely to reflect the presence of a shear key, and as free longitudinally to reflect the superstructure on elastomeric bearings.

3.2.2. Gap at Split Piers

The six-span structure comprises three sets of two-span continuous frames, with gaps in between frames. For simplicity, the frames were assumed not to interact with each other transversely. To capture the effects of these gaps in the longitudinal direction, two models were made:

- Tension model, in which frames were not connected and could move completely independently of each other, to capture the out-of-phase longitudinal response
- Compression model, in which frames were structurally connected in the longitudinal direction, to capture the in-phase longitudinal response

The envelope of the demands from these two models was then used as the demands.

3.3. Modal Analysis

The model was discretized to give sufficient accuracy for modal analysis: each span was divided into segments not exceeding 3 m in length, and each column was divided into five segments. The number of modes was increased until mass participation exceeded 90% for all six degrees of freedom. The Complete Quadratic Combination (CQC) method was used to combine modes.

3.4. Design Response Spectra

Horizontal spectral acceleration values for the bridge site were obtained from the Geological Survey of Canada. Design response spectra were then developed based on CAN/CSA S6-14 (*Figure 5*). Site class C ground conditions were assumed at all piers on the instruction of the geotechnical engineer, assuming the liquefaction potential at piers D18 and D17 would be mitigated by ground improvements (see Sections 4.3 and 5.3). See Appendix C: Geotechnical Report for details.

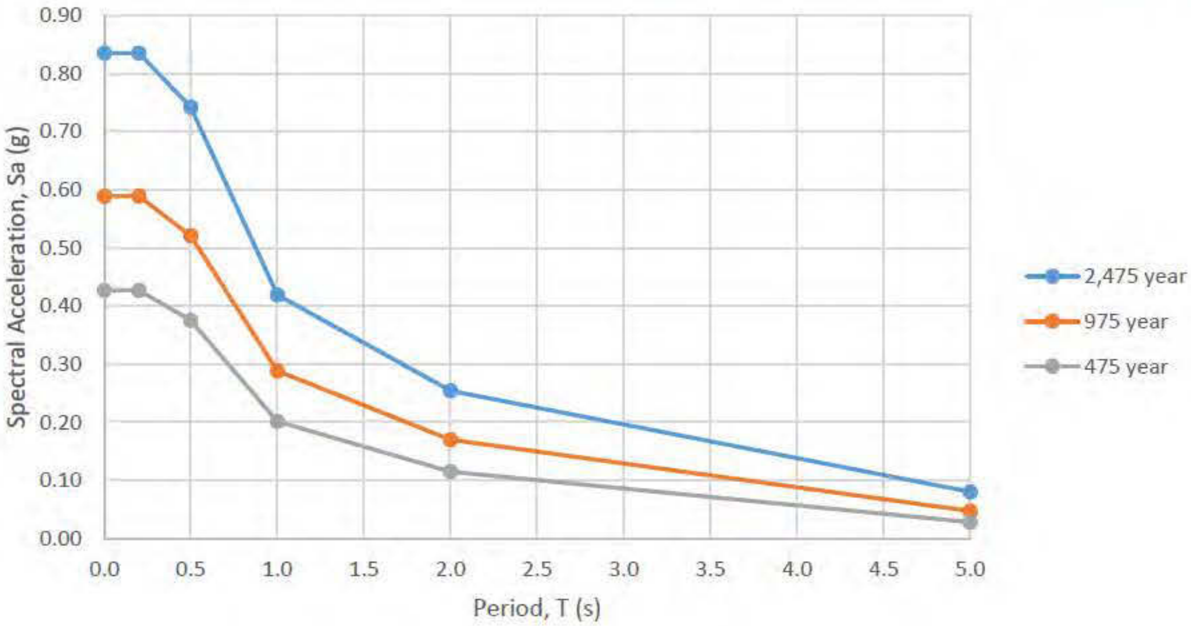


Figure 5. Design response spectra

3.5. Response Spectrum Analysis

Response spectrum analyses were performed for the 2475, 975 and 475 year return period design hazard levels using their respective design spectra. The resulting elastic seismic demands were then combined with dead load demands using 1.0D + 1.0EQ, and compared against member capacities. This was considered adequate for the purposes of this initial assessment, with no need for inelastic static pushover analysis. The reason for this is explained in Section 4.2.1.

Seismic excitations were imposed in two perpendicular global directions U1 and U2. The horizontal elastic seismic effects on each of the principal axes of a component resulting from analyses in the two perpendicular horizontal directions were combined within each direction from the absolute values to form two load cases as follows:

- 100%U1 + 30%U2: 100% of the absolute value of the effects resulting from an analysis in one of the perpendicular directions combined with 30% of the absolute value of the force effects from the analysis in the second perpendicular direction; and
- 30%U1 + 100%U2: 100% of the absolute value of the effects from the analysis in the second perpendicular direction combined with 30% of the absolute value of the force effects resulting from the analysis in the first perpendicular direction.

3.6. Seismic Load Paths

The seismic load path from the superstructure to the ground is as follows: from composite deck and girders into the integral pier caps, into the integral columns, and finally, as the case may be, into spread footings or pile caps and piles. The west abutment also acts to resist transverse seismic load at the superstructure level via its shear keys.

3.7. Member Capacities

Member capacities were taken as their unfactored nominal resistances assuming material resistance factors for concrete and reinforcing bars of 1.0, and based on material strengths specified on the record drawings. These values are listed in Appendix B: Design Brief. Yield strength of rebar and compressive strength of concrete were not increased beyond the specified values because the structure is not expected to exhibit ductile behavior (Section 4.2.1), and no material testing records were available.

4. RESULTS AND DEFICIENCIES

4.1. Overview

The deficiencies can be summarized as follows:

- Columns D22 to D17:
  - Poor detailing of reinforcement
  - Insufficient shear and torsional capacity
  - Insufficient flexural capacity
- Liquefaction potential at D18 and D17
- Foundations D22 to D17: Insufficient resistance to overturning and sliding
- Pull-out of girder reinforcement from pier caps at D21 and D19
- Insufficient bearing seat length at abutment
- Insufficient transverse shear resistance at abutment shear key
- Insufficient gap at interface with Costco / Spectrum building

The following sections describe these deficiencies in detail and in numerical terms, where applicable. Results in this section are generally expressed as a demand / capacity ratio. Where this ratio exceeds 1, demand is greater than capacity, and the value is highlighted to indicate a potential deficiency.

4.2. Columns

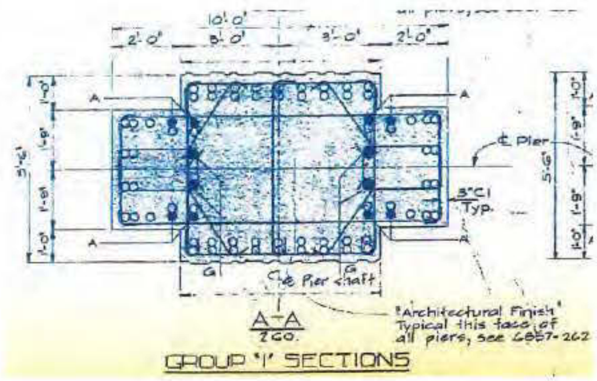
4.2.1. Poor Detailing of Reinforcement

The Dunsmuir Viaduct was designed in the early 1970s to the code requirements of the day. As such, it does not meet the seismic detailing requirements of today's codes and is not expected to be resilient to seismic events.

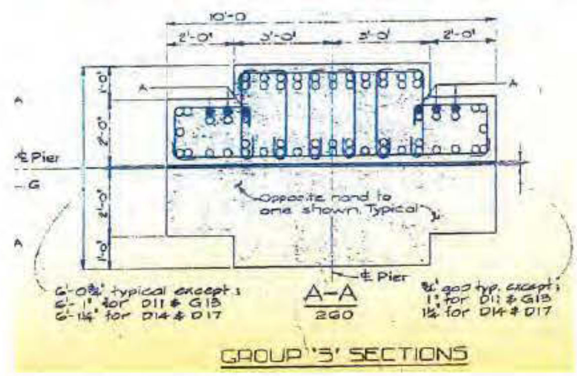
In particular, the transverse reinforcement (stirrups) in the columns does not provide adequate confinement to the concrete core and does not prevent longitudinal bars from buckling under reversed cyclic loading. As a result, the columns cannot develop any meaningful ductility, and cannot form plastic hinges to dissipate energy. The columns are therefore expected to experience the full elastic demands from seismic excitation. This also means an inelastic static pushover analysis is of no practical use for the existing structure, as the columns will likely fail as soon as they have reached their design capacities.

Figure 6 illustrates typical reinforcement details at base of "full" and "split" columns. The stirrups around the column perimeter do not form closed stirrups anchored back into the core; rather they are a series of discreet bars lapped around the column perimeter. When elastic demands exceed design capacity, after a few cycles of seismic loading, the concrete cover will spall off, and these perimeter stirrups will lose their anchorage and tensile capacity. Then, any confinement they provided to the concrete core is lost, allowing the concrete core to crush and longitudinal reinforcement to buckle. At this point, virtually all flexural, shear and torsional capacity is lost.

The column reinforcement includes internal stirrups (cross-ties) that form 180 and 90 degree hooks around longitudinal bars on the two sides of the column. However, these hooks do not alternate sides, as is current seismic practice. Hence, these stirrups do not provide adequate confinement to the longitudinal bars or concrete core.



(a) "Full" piers D20 and D18



(b) "Split" piers D19 and D17

Figure 6. Typical column details

4.2.2. Shear and Torsional Capacity

As mentioned in Section 4.2.1, the poor detailing of stirrups in columns will result in virtually all flexural, shear and torsional capacity being lost when elastic demands exceed design capacity. For the first few cycles, however, the section maintains some resistance.

An attempt was made to estimate the shear resistance using the refined methodology outlined in Priestley et al. (1996), Section 7.4.8. In this method, we considered only the deeper rectangle of the cruciform section in each loading direction and assumed maximum concrete degradation (k = 0.05) for Vc (strength of concrete shear resisting mechanisms), used only the stirrups in this rectangle for Vs (strength of shear resisting mechanisms involving transverse reinforcement), and included the axial load component Vp (shear strength provided by axial force in member).

Resulting shear demand / capacity ratios are listed in Table 2. It is observed that all columns have insufficient shear capacity at the 2475, 975 and 475 year return period hazard levels for combined transverse and longitudinal shear.

Table 2: Shear demand / capacity ratios in columns

	Transverse			Longitudinal			Combined Trans. and Long.		
	Return Period (years)			Return Period (years)			Return Period (years)		
	2475	975	475	2475	975	475	2475	975	475
D22	1.05	0.73	0.53	1.64	1.21	0.89	2.01	1.43	1.04
D21 West	2.84	1.99	1.43	2.10	1.61	1.28	3.68	2.65	1.97
D21 East	1.38	0.96	0.69	1.93	1.51	1.23	2.24	1.68	1.31
D20	1.39	0.97	0.70	2.68	1.91	1.43	3.31	2.37	1.74
D19 West	2.66	1.86	1.33	1.52	0.99	0.76	3.29	2.35	1.73
D19 East	2.66	1.51	1.08	1.35	1.16	0.93	2.90	2.08	1.55
D18	1.49	1.05	0.75	2.52	1.81	1.35	3.10	2.16	1.54
D17 West	2.10	1.46	1.04	1.46	1.10	0.87	2.96	2.13	1.58

4.2.3. Flexural Capacity

Table 3 summarizes the range of flexural demand / capacity ratios for the base of all columns under bi-axial loading. It is observed that flexural capacity is inadequate in all columns at the 2475 year return period hazard level, but adequate at the 975 and 475 year return period hazard levels.

Table 3: Flexural demand / capacity ratios at base of columns under bi-axial loading, Margin of error: +-10%

	Return Period (years)		
	2,475	975	475
D22	**1.43**	0.94	0.62
D21 West	**1.40**	**1.01**	0.75
D21 East	**1.30**	0.95	0.71
D20	**1.45**	0.98	0.67
D19 West	**1.35**	0.92	0.64
D19 East	**1.10**	0.78	0.58
D18	**1.51**	**1.02**	0.70
D17 West	**1.14**	0.77	0.57

4.2.4. Compression-Only Splices

The "G-Loc" mechanical butt splices used on longitudinal column rebar are compression-only and do not provide tensile strength (Figure 7). The splices are staggered at various heights in the column (Figure 8). Several bars are spliced within 1.4 to 2.2 m from the base of columns. This results in a reduction in flexural capacity at these locations.

Figure 7. "G-Loc" compression-only butt-splices

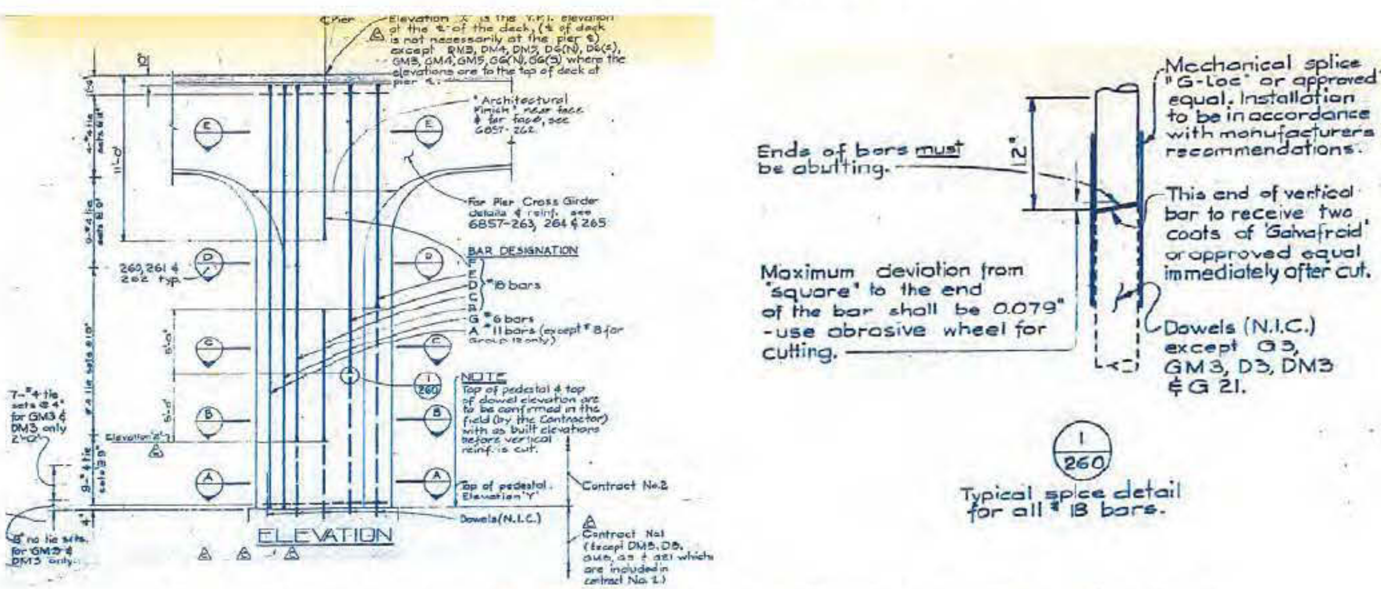


Figure 8. Compression-only butt-splices were used in longitudinal column rebar, staggered at different heights

4.3. Liquefaction Potential

The geotechnical engineer's preliminary assessment indicates liquefaction of fill below the water table and strain-softening of silty clays at piers D18 and D17 at both the 975 and 2475 year return period hazard levels. This would result in a loss of lateral support in piles (significant) and pile cap (moderate to significant), which could lead to pier collapse.

The foundations at piers D19 and D20 have potentially liquefiable material between the bottom of the pile cap and the till-like soil. As such, loss of lateral support under liquefied soil conditions is a possibility.

Further details can be found in the geotechnical report in Appendix C.

4.4. Foundation Stability

Piers D22 and D21 are on spread footings, while piers D20 to D17 are supported by H-piles and expanded base piles. The geotechnical engineer recommended the pile caps at D20 and D19 can be treated as spread footings for this initial assessment due to their short pile lengths. Pile caps at D18 and D17 can be treated as spread footings if jet grouting is used.

The proposed retrofit strategy includes jacketing all columns (Section 5.2). This changes the stiffness of the structure and results in greater inertial loads being attracted to the foundations (Section 7.1). Therefore, foundation stability was checked under the elastic demands from the response spectrum analysis for the proposed retrofit strategy. Overturning and sliding checks were completed for all foundations. Sliding checks conservatively relied on frictional resistance and not on passive resistance of soil.

Results of the overturning checks are presented in Table 4. It was concluded that all foundations are unstable in overturning at the 2475, 975 and 475 year hazard levels.

Table 4. Foundation overturning check: Eccentricity limits

	Eccentricity limit (S6-14 Cl. 6.10.3.4)					
	Transverse, e_B / 0.3B			Longitudinal, e_L / 0.3L		
	30%U1+100%U2			100%U1+30%U2		
	Return period (years)			Return period (years)		
	2475	975	475	2475	975	475
D22	**1.98**	**1.36**	0.96	**2.32**	**1.48**	**1.00**
D21	**2.52**	**1.75**	**1.25**	**5.60**	**3.73**	**2.60**
D20	**3.21**	**2.22**	**1.58**	**2.58**	**1.78**	**1.25**
D19	**4.59**	**3.17**	**2.26**	**3.36**	**2.32**	**1.65**
D18	**5.53**	**3.82**	**2.73**	**2.42**	**1.68**	**1.20**
D17	**4.27**	**2.91**	**2.06**	**4.73**	**3.19**	**2.24**

Results of the sliding checks are shown in Table 5. Foundations at D21 through D17 were found to be unstable in sliding at the 2475 year return period hazard level in various directions (see table), but not at the 975 or 475 year return period hazard levels.

Table 5. Foundation sliding demand / capacity ratios for bi-directional loading

	30%U1+100%U2			100%U1+30%U2		
	Return period (years)			Return period (years)		
	2475	975	475	2475	975	475
D22	0.61	0.42	0.33	0.90	0.58	0.44
D21	**1.03**	0.71	0.56	**1.45**	0.96	0.75
D20	0.85	0.59	0.46	**1.10**	0.75	0.58
D19	**1.32**	0.91	0.72	**1.11**	0.76	0.60
D18	**1.06**	0.73	0.58	0.95	0.66	0.53
D17	**1.22**	0.83	0.65	**1.23**	0.83	0.65

Pile caps at D17 to D20 could attract negative bending moment in a seismic event if some piles go into tension. However, these pile caps were not checked for negative bending capacity, as the tensile geotechnical resistance of piles was not available.

Spread footings and pile caps were checked in positive bending for the full geotechnical bearing capacity of the soils under them. This is an upper-bound approach. All foundations passed this check, except at D22 (demand / capacity = **1.18**). The spread footing at D22 will likely not need any strengthening after detailed analysis is done.

4.5. Girder - Pier Cap Connections

The proposed retrofit strategy includes jacketing of all columns (Section 5.2). This changes the stiffness of the structure, and results in greater inertial loads being attracted to the girder - pier cap connections (Section 7.1). Therefore, girder - pier cap connections were checked under the elastic demands from the response spectrum analysis for the retrofitted columns.

Girder - pier cap connections were checked for both positive and negative bending during a seismic event. Positive bending exerts tension at the bottom of the girder – pier cap interface, while negative bending induces tension in longitudinal deck rebar above the girder.

Dead loads apply negative bending in the superstructure at piers, while seismic loads in the longitudinal direction put equal positive and negative bending in the superstructure at the two sides of the pier. The combined dead and seismic loads were applied at the girder - pier cap connections for each hazard level.

At the bottom of precast girder ends, dowels and pre-tensioning strands project into the pier caps. While the dowels are fully developed at this interface, the pre-tensioning strands are not, so their tensile capacity was adjusted accordingly. At the top of precast girder ends, post-tensioning strands project into the pier caps. These strands provide connectivity between the pier cap and the superstructure and help resist negative bending at the interface.

Table 6 lists the flexural demand / capacity ratios at girder – pier cap connections.

Table 6: Flexural demand / capacity ratios at girder - pier cap connection

	Positive Bending			Negative Bending		
	Return period (years)			Return period (years)		
	2475	975	475	2475	975	475
D22 West	0.00	0.00	0.00	0.82	0.73	0.66
D22 East	0.00	0.00	0.00	0.90	0.78	0.69
D21 West	**2.87**	**1.94**	**1.32**	**1.45**	**1.05**	0.79
D21 East	**1.75**	**1.08**	0.64	**1.16**	0.88	0.69
D20 West	0.00	0.00	0.00	0.90	0.79	0.71
D20 East	0.00	0.00	0.00	0.74	0.66	0.61
D19 West	**2.47**	**1.73**	**1.25**	0.94	0.66	0.47
D19 East	**1.82**	**1.16**	0.73	0.96	0.72	0.56
D18 West	0.00	0.00	0.00	0.73	0.65	0.60
D18 East	0.00	0.00	0.00	0.83	0.73	0.67
D17 West	**2.56**	**1.81**	**1.31**	0.92	0.65	0.46

The following observations can be made:

- The girder - pier cap connections at D22, D20 and D18 will experience no net positive bending, because the negative moments induced by dead loads at these piers exceed the positive moments induced by seismic loads.
- The girder - pier cap connections at D21, D19 and D17 will fail in positive bending at the all three hazard levels. The failure mechanism is the abrupt pull-out of pre-tensioning strands from the pier cap, followed by yielding of the bottom flange dowels.
- The girder - pier cap connections at D21 will fail in negative bending at the 2475 year return period hazard level.

4.6. Abutment Seat Length

Girder seat lengths at the west abutment were checked using the empirical method outlined in CAN/CSA S6-14, and were found to be deficient for some girders. Due to the varying geometry of the abutment front wall (Figure 9), six of the nine girders at the abutment (all but the three middle girders) have insufficient bearing seat length, which could lead to span D23-D22 collapsing during a seismic event.

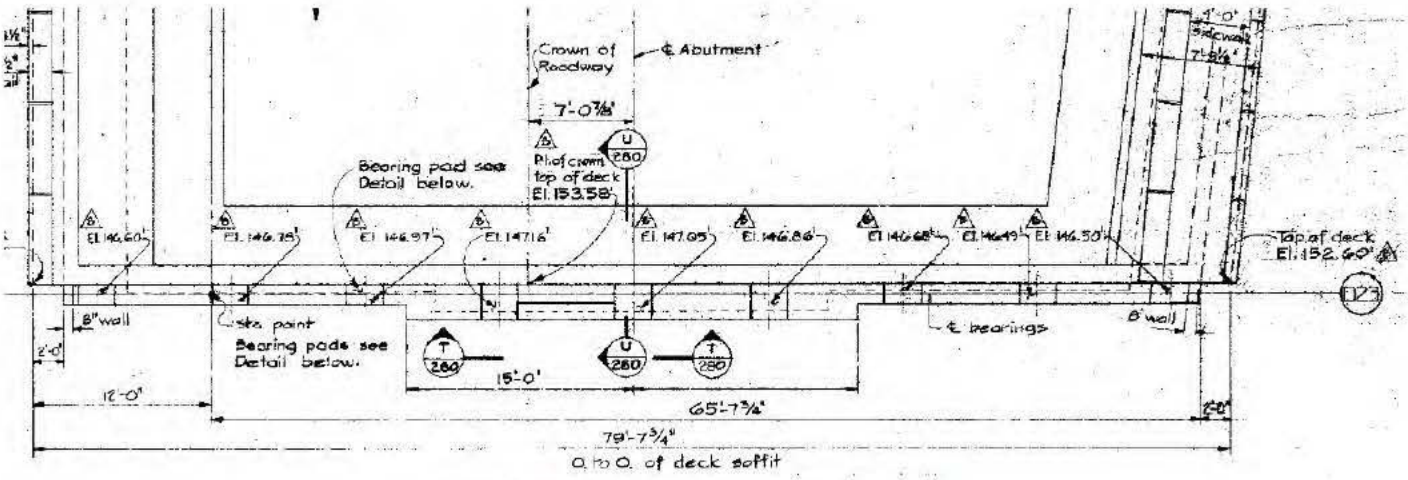


Figure 9. West abutment girder seat lengths, plan view

4.7. Abutment Shear Key

The existing shear key at the west abutment does not have sufficient interface shear resistance to restrain the superstructure transversely (Table 7).

Table 7. Shear demand / capacity ratios at west abutment shear keys

	Return Period (years)		
	2475	975	475
D23	**5.87**	**4.14**	**2.99**

4.8. Joint at Costco / Spectrum Building

There is a longitudinal joint between the Dunsmuir Viaduct and the adjacent Costco / Spectrum building. The joint is at deck level along the south edge of the viaduct, beginning at the west end of the south wall of the west abutment D23, continuing along the roof of the Costco / Spectrum building (which also serves as the sidewalk south of the viaduct), and ending close to pier D20 on the pedestrian bridge connecting the sidewalk to Rogers Arena. This joint has a gap width of 75 mm, measured during our condition inspection on March 30, 2017. In order to avoid pounding between the two structures during a seismic event, the sum of the maximum transverse displacement of each structure at the design earthquake should not exceed this amount.

The transverse movement of the viaduct along this length is affected by rotation of the foundations. However, our model of the existing structure assumes columns are fixed at their base (Section 3.2.1). Hence, this model cannot predict the amount of lateral movement.

In any case, the existing columns have poor detailing of reinforcement (Section 4.2.1), and need to be jacketed (Section 5.2). Response spectrum analysis for the retrofitted structure indicates the maximum transverse movement of the viaduct along this length during the 2475 year return period hazard level is roughly 30 mm. This leaves approximately 75 – 30 = 45 mm for the movement of the Costco / Spectrum building to prevent pounding.

The Costco / Spectrum building was likely designed to the National Building Code of Canada (1995), which mandates a maximum lateral deflection limit of 0.020hs between storeys at the 475 year return period hazard level, where hs is the storey height. At the height of the joint (12.5 m), this corresponds to a lateral deflection of about 250 mm, which exceeds the 45 mm limit calculated above. Calculating the transverse deflection of the Costco / Spectrum building at the 2475 year return period hazard level will require an analysis of the building structure, which is beyond our scope.

For this initial assessment, we have assumed the existing gap is insufficient to prevent pounding between the two structures.

4.9. Column Connections

The joint stresses at the column-foundation and column - pier cap connections will be checked during detailed design. The cost of any potential retrofits to these connections can be considered part of the design contingencies (Section 8).

5. SEISMIC RETROFITS

5.1. Overview

Figure 10 shows an overview of the proposed retrofit strategy for spans D17 to D23. Seismic retrofits include:

- Column jackets at D22 to D18
- Foundation anchors at D22 to D17
- Ground improvements at D18 and D17
- Girder - pier cap connectors at D21 and D19
- Link slabs at D21 and D19
- Abutment: Seat length extensions
- Abutment: New shear keys
- New joint at interface with Costco / Spectrum building
- New pier at D17

The following sections describe each proposed retrofit measure in detail.

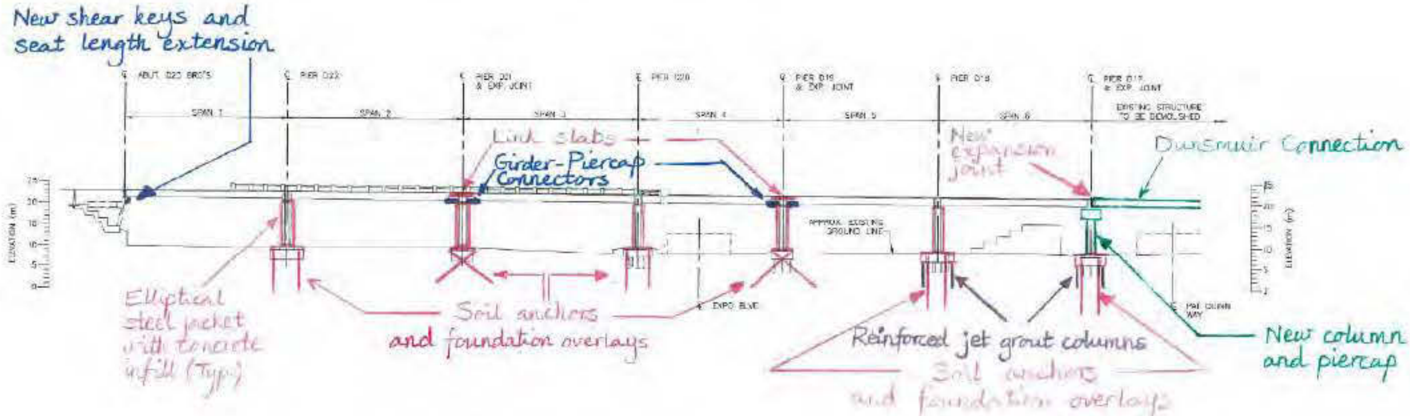


Figure 10. General arrangement of seismic retrofit scheme, elevation view

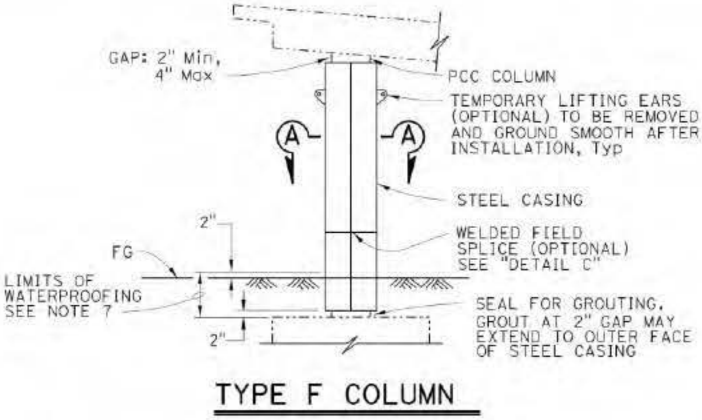
5.2. Column Jackets

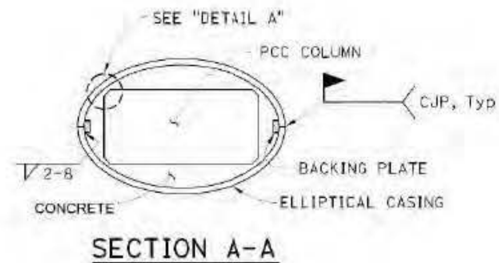
We recommend installing elliptical steel jackets on columns D22 to D18. The jacket would provide passive lateral confinement to concrete in the compression zone under flexure, thus allowing plastic hinges to form in the tops and bottoms of columns. It would also enhance shear resistance by resisting the lateral column dilation associated with development of diagonal shear cracks (Priestley et al., 1996).

5.2.1. Background

Steel column casings are the most commonly used column retrofit method in the state of California. They are an effective retrofit strategy for enhancing shear capacity, confinement, and preventing slipping of lap splices (CalTrans, 2011).

For rectangular columns, the recommended practice is to use an elliptical jacket that provides a continuous confining action. Two half shells of steel plate rolled to specific radii are positioned over the columns and are site-welded up the vertical seams. The space between the jacket and existing column is filled with normal concrete. *Figure 11* shows typical steel jackets used to retrofit rectangular columns.





(a) Type F casing provides fixed end conditions

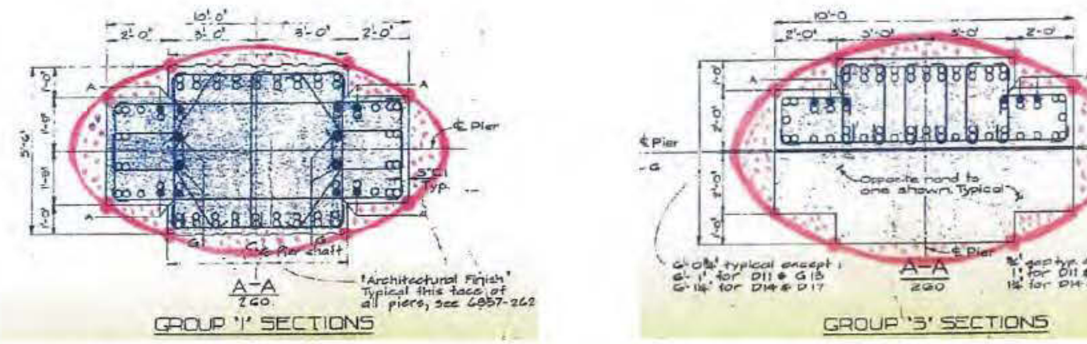
(b) Elliptical casing for rectangular columns

Figure 11. Typical steel jackets for columns
Source: Caltrans (2015)

5.2.2. Application

In the Dunsmuir viaduct, the full columns at D22, D20 and D18 have a doubly-symmetrical cruciform cross-section. The split piers at expansion joints D21 and D19 form the same shape when the two adjacent split piers are grouped together. In order to maximize concrete confinement, an elliptical jacket must be fitted to the eight corners of this shape. The radii of this ellipse (a and b) were obtained by fitting the equation of an ellipse $(x/a)^2 + (y/b)^2 = 1$ to the co-ordinates of these corners (Figure 12).

Once the steel jacket has been installed, the gap between the jacket and existing columns should be filled with normal concrete. At split piers, the gap between split columns should first be cleared of any debris, flushed with water and injected with a cement grout so as to fill the space completely.



(a) Full piers (typical)

(b) Split piers (typ.)

Figure 12. Elliptical shapes were fitted to the cruciform columns of the viaduct

All viaduct columns have a transversely flared shape under the pier cap. The steel jacket should extend over this length as well. In this region, the jacket will take the shape of an elliptical cone, which will require special forming and horizontal site weld at its interface with the regular jacket (Figure 13).

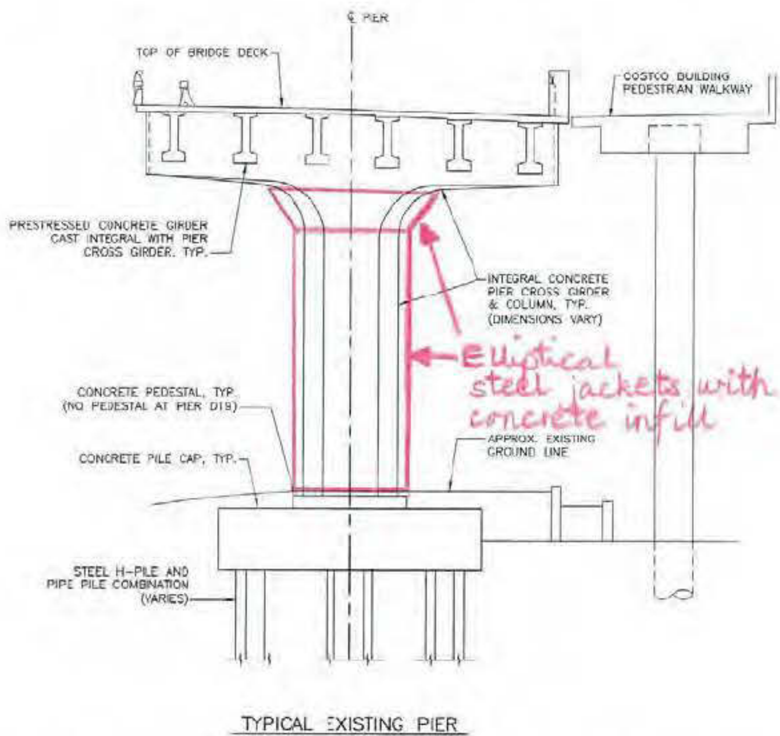


Figure 13. Steel jackets on flared columns of the viaduct

A 50 mm gap should be provided between the end of jacket and the top of the pedestal and the bottom of the pier cap to avoid the possibility of the jacket acting as compression reinforcement by bearing against the supporting member at large drift angles. This is to avoid excessive flexural strength enhancement of the plastic hinge region, which could result in increased demands in footings and cap beams under seismic response (Priestley et al., 1996).

The steel casing will be designed to develop the full flexural resistance of the plastic hinge regions. A series of closely spaced steel rings welded to the inside of the casings will help develop the full tensile capacity required within a short distance from the top and bottom of the jackets, ahead of the compression-only splices on longitudinal bars (Section 4.2.4).

5.3. Ground Improvements

The geotechnical engineer recommends ground improvements be completed at D18 and D17 to reduce the effects of seismic liquefaction and strain-softening and thereby increasing the lateral support of the existing piles. Ground improvement options could include:

- Installing a ring of jet-grout columns around each pier that would act as a new caisson-type foundation (Figure 14). The ring would be reinforced with steel I-beams to structurally connect it to the pile cap. Jet grouting can be used in low headroom areas; however, there may be challenges related to the containment and disposal of soils and cement generated from the jet grouting process in a congested area. The results of environmental testing indicate that concentrations of several contaminants are above residential standards. As such, the soil and groundwater spoils may require special disposal requirements, which may increase costs. These increased costs are variable depending on actual soil groundwater conditions during drilling and what methods the contractor uses to dispose of or contain the material. As such, these increased costs are not included in the cost estimate for jet grouting, and will be covered by the construction contingency instead.

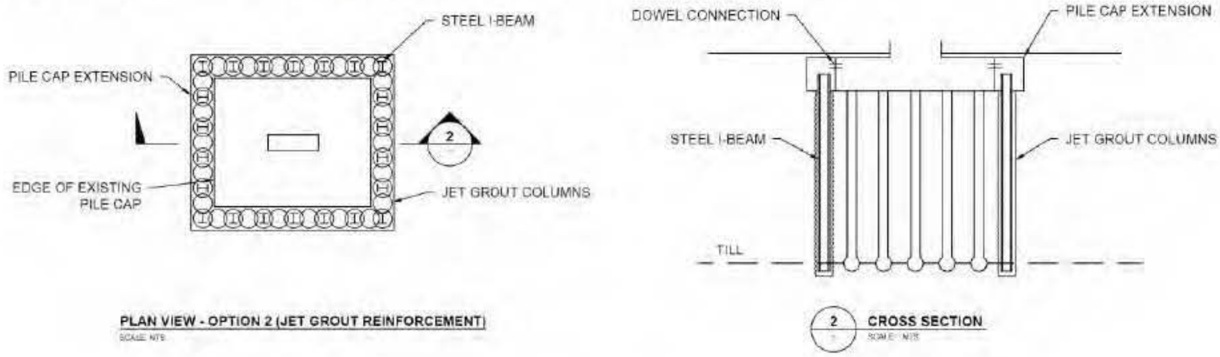


Figure 14. Jet grouting
Source: Thurber Engineering (2017), Fig. D2

- Installing four drilled shafts at each pier (Figure 15). Each shaft would be at least 1.2 m in diameter to resist lateral loads and extend five to ten pile diameters into till-like soil to achieve lateral fixity. There will be a cost-premium associated with construction in areas of low headroom and limited access. Drill cuttings will require special disposal requirements.

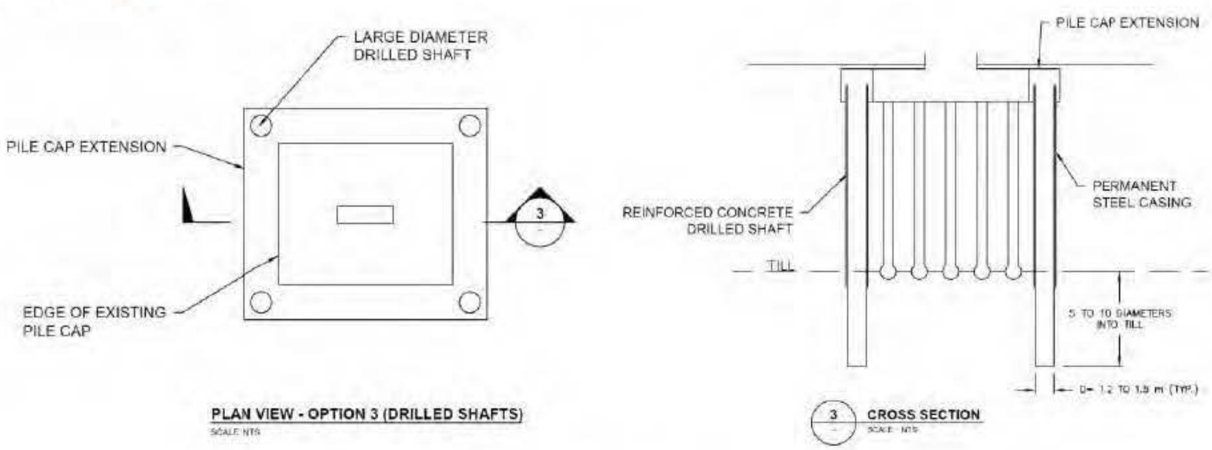


Figure 15. Drilled shafts
Source: Thurber Engineering (2017), Fig. D2

Drilled shafts are likely to be more reliable than jet-grout columns at these piers. However, they are more costly to install. We recommend using jet grouting instead of drilled shafts to minimize construction costs. Further details can be found in Appendix C: Geotechnical Report.

5.4. Foundation Anchors

To protect the foundations against overturning, we recommend the foundations at all piers (D17 to D22) be tied down using soil anchors (Figure 16). "Foundation" here refers to the spread footing or pile cap at each pier, as the case may be. These anchors would resist the uplift component of overturning moments on one side of the foundation, and the underlying soil or piles would resist the compressive component on the other side. Where additional sliding resistance is required, diagonal soil anchors could be used to resist the horizontal load.

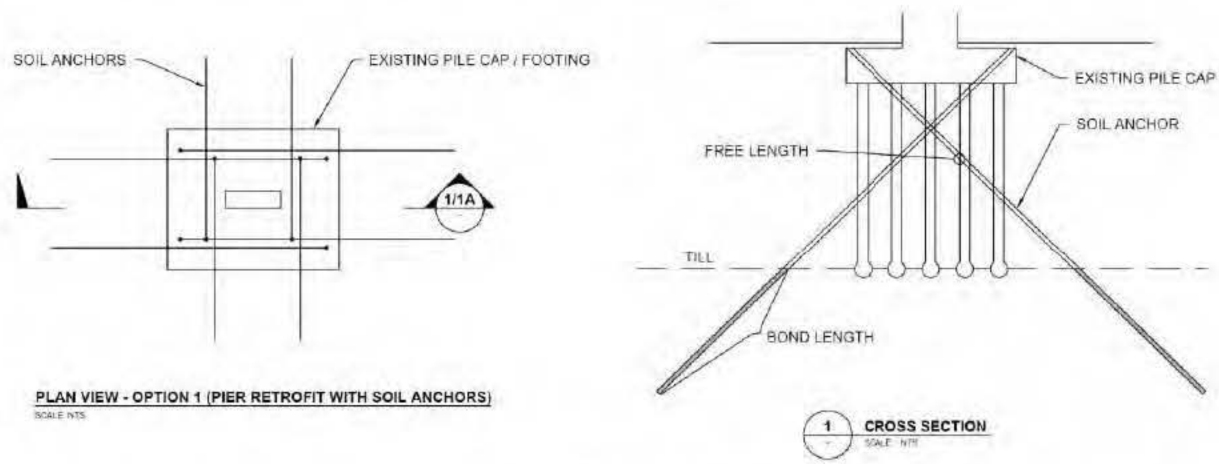


Figure 16. Soil anchors
Source: Thurber Engineering Ltd. (2017), Figure D1

Sliding was a concern at piers D21, D20, D19 and D18 at the 2475 year hazard level. However, the pre-tension applied to the soil anchors during installation will increase the normal force on the foundation base, and thus, its frictional resistance. We expect that only piers D21 and D19 will require diagonal soil anchors to resist sliding at the 2475 year hazard level. Other piers can have vertical anchors, which are less expensive to install and test.

Each anchor would be stressed against a plate or steel assembly at the top of the foundation, pass through a hole cored into the foundation, have a free (unbonded) length beneath the foundation, and end in a bonded length inside till-like soil (Figure 17). The top of the anchors would be embedded in a concrete foundation overlay dowelled into the existing pile cap. This overlay would protect the anchor assembly against corrosion, and include rebar to resist negative bending moments generated by the uplift force.

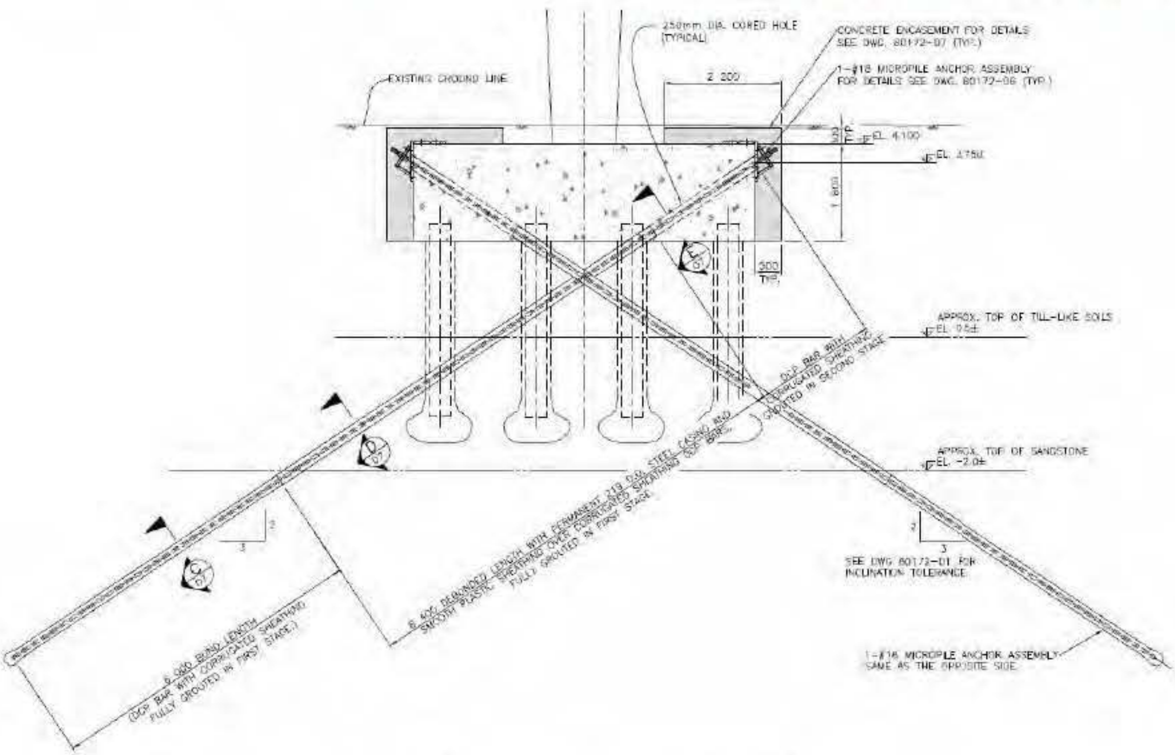


Figure 17. Example of foundation anchor detail
Source: MMM Group (2008), Drawing 80172-04

At piers D19 and D20 additional measures such as permanent casing or more anchors may be required to address buckling and loss of lateral support under liquefied soil conditions.

Piers D17 to D20 have piled foundations. A review of available utility information around all piers revealed a hydro transmission duct runs along the north edge of the pile cap at D17. Record drawings show a 36" outer diameter pipe running under, through or near foundations at D22 to D18. Soil anchors and jet grouting can be arranged to avoid conflicting with these piles and utilities, which should be confirmed during detailed design. Should the presence of utilities preclude soil anchoring, foundations could be enlarged to decrease maximum bearing pressure or new piles could be installed to resist uplift. Should the presence of utilities preclude jet grouting, drilled shafts could be used.

5.5. Girder - Pier Cap Connectors

We recommend connecting the girders to pier caps at piers D21 and D19 using steel brackets. Brackets are required at the top of girders at pier D21 and at the bottom of girders at both piers.

Figure 18 shows one potential detail, with the bracket connected to the bottom flange of the precast prestressed girders via anchor bolts and to the vertical face of the pier cap via high-strength bars. At D19, the girders on the east and west sides of the pier cap line up such that the same bars could be used to connect the brackets on the two sides. Surface scanning and caution must be exercised so as not to cut load-carrying bars or post-tensioning tendons in the pier cap.

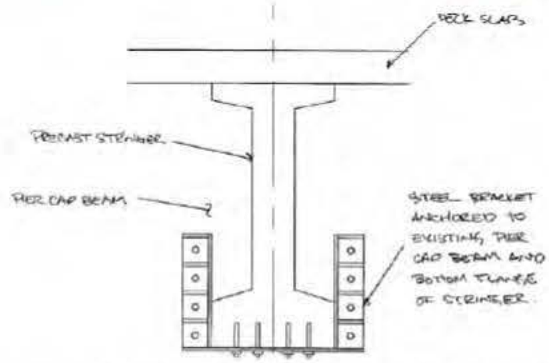


Figure 18. Example of steel bracket detail connecting girders to pier caps
Source: Cochrane (2004), Appendix C

5.6. Link Slabs

We recommend link slabs be installed at deck level at expansion joints D19 and D21, and the gap between split pier caps be grouted down to the jacket level.

Jacketing the adjacent split columns at these locations turns them into one integral unit (Figure 12.b), and in so doing changes the bridge articulation as described below. The proposed link slabs would help resist and transfer the resulting effects of this new articulation.

- The superstructure becomes continuous for longitudinal thermal movements. A link slab would help transmit resulting longitudinal forces between spans at the deck level: via concrete compression for expansion, and via rebar tension for contraction.
- The superstructure becomes continuous for longitudinal and transverse seismic movements. The link slab would help transfer resulting forces between spans at the deck level, through concrete compression and rebar tension for longitudinal forces, and through shear and bending for transverse forces (with the link slab acting as a very deep beam).
- The previously separate split columns become much stiffer in bending, increasing end fixity for rotation of the superstructure at these locations. This attracts significant negative moments under live loads to the relatively weak "neck" of split pier caps projecting from the top of the jacket, which could overstress them. Installing a link slab at deck level and grouting the gap between split pier caps would create a tension-compression moment couple to resist these negative bending moments, and essentially makes the superstructure continuous for longitudinal bending under live loads.

Moreover, link slabs would solve a long-term maintenance problem by eliminating the leaking expansion joints. Our condition inspection on 30 March 2017 revealed all expansion joints are leaking, leading to wetness and corrosion staining at split piers.

5.7. Abutment: Seat Length Extension

The three most interior girders have sufficient seat length. We recommend extending the seat length at the six remaining girders by at least 300 mm. This can be accomplished by building a concrete corbel at the bearing seat level and dowelling it into the abutment front wall (Figure 19). Figure 20 shows an example developed for a different project.

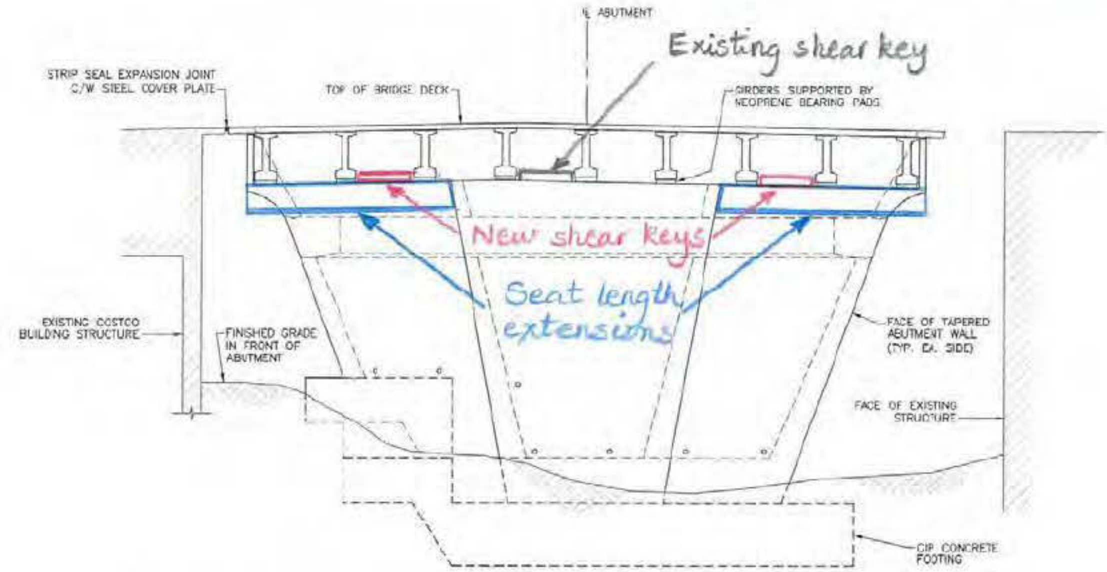


Figure 19. Seat length extensions and new shear keys at west abutment, elevation view

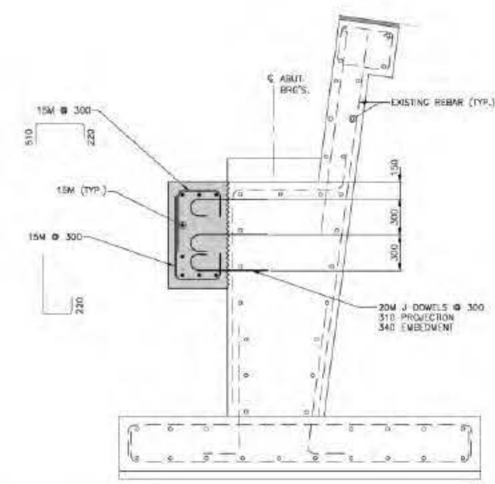


Figure 20. Example of seat length extension detail, cross-section view
Source: MMM (2016), Drawing 02448-113

5.8. Abutment: New Shear Keys

We recommend supplementing the existing shear key with two new shear keys, one on each side (Figure 19). These shear keys would be connected to the existing bearing ledge via vertical dowels, and extend on either side to the bottom flanges of adjacent girders. The gap at this interface should match at all three shear keys, so as to maximize the possibility of simultaneous engagement and load sharing between shear keys in a seismic event.

5.9. New Joint at Costco / Spectrum Building

We recommend replacing the existing compression seal joint between the Dunsmuir Viaduct and the adjacent Costco / Spectrum building with a steel cover plate expansion joint, consisting of a sliding steel plate spanning a wider gap. Figure 21 shows an example of this detail. The gap between the concrete structures (e.g. 300 mm) will be adjusted to accommodate the transverse seismic movement of the two structures and prevent them from pounding against each other. The south edge of the bridge deck will have to be cut back accordingly. The steel plate will be designed to resist pedestrian / vehicular loads over the gap. If the 25 mm gap between the steel plate and pavers on the Spectrum structure closes during a seismic event, the steel plate will ride up on top of the pavers. If the joint is damaged in a seismic event, it can be repaired or replaced.

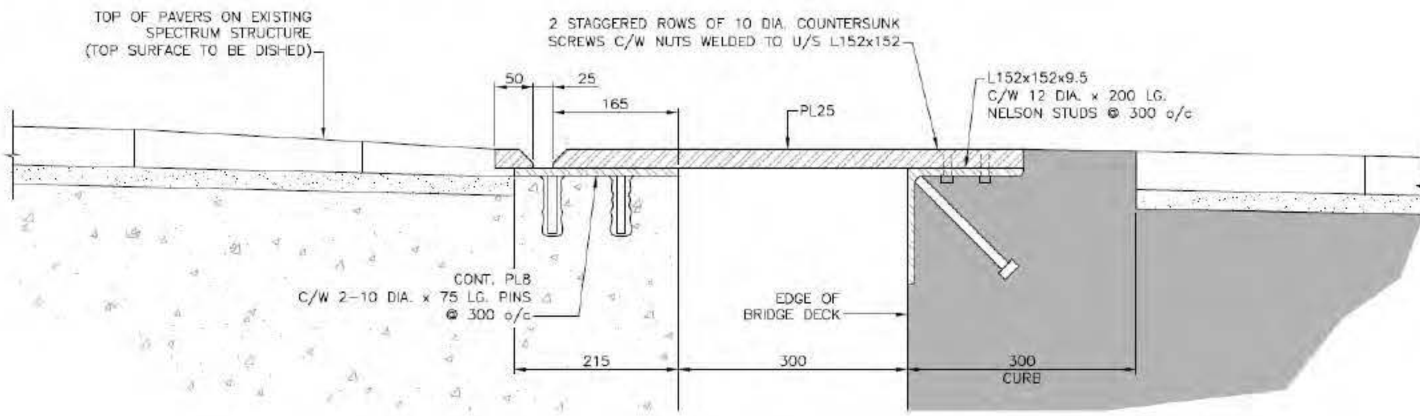


Figure 21. New joint between Georgia Ramp and Costco / Spectrum building

5.10. New Pier at D17

The new "Dunsmuir Elevated Park" structure will connect to the retained spans of the Dunsmuir viaduct at pier D17, just west of Pat Quinn Way. We recommend demolishing the existing split columns at D17, building a new column and pier cap to support both the existing span D18-D17 and the new first span of the Dunsmuir Elevated Park. Both spans would be supported on bearings that allow longitudinal movement but provide transverse restraint. A strip seal expansion joint would separate the two spans at deck level. The existing foundation could be retrofitted (see Sections 5.3 and 5.4) and re-used to minimize costs.

Building a new pier at D17 has several advantages over retaining the existing pier:

- The new column if well detailed would not have the seismic deficiencies of the existing column.
- The new pier cap could provide ample bearing seat length to both spans.
- Removing the longitudinal fixity at D17 reduces the bending demand from thermal movement on columns D22 to D18 (see Section 7.3).
- Connecting the new span to the existing split pier D17 East would be very challenging.

6. GENERAL REHABILITATION

In addition to the seismic retrofits, a host of modifications are required above the deck to prepare it for its intended final configuration, and joint replacement, defect repairs and possibly deck rehabilitation are required to extend the service life of the bridge. These general rehabilitation items are described below.

6.1. Above-Deck Modifications

Several modifications are required above the deck level to convert the viaduct into an elevated park and active transportation bridge for cyclists and pedestrians with integration into adjacent properties, structures, and Rogers Arena concourse. These include:

- Removing existing barriers
- Removing existing asphalt and waterproofing membrane
- Installing new waterproofing membrane
- Installing new asphalt on the bike path
- Installing new concrete pavers on the sidewalk
- Installing drainage layer and landscaping for the planters
- Installing new railings on outside edges of the deck.

6.2. New Joint at Abutment

As stated in the Condition Assessment Report (NEFCAP-MMM-S-RPT-003), at the west abutment (D23) expansion joint, corrosion staining and spalling of the north overhang soffit was observed. The expansion joint between the exterior girders could not be inspected, as the girder end diaphragm prevents access. The bearing seats were generally dry, with some efflorescence and corrosion staining.

We recommend replacing this expansion joint with a strip seal expansion joint to prevent further deterioration of deck and bearing seats. This is consistent with the City's intent as outlined in the project RFP. Any deteriorated deck concrete should be replaced during this process.

6.3. Miscellaneous Defect Repairs

As stated in the Condition Assessment Report (NEFCAP-MMM-S-RPT-003), a few minor defects were noted, for which we recommend the following repairs:

- West abutment: Repair cracks on front wall under bearing seat for girder C.
- Deck: Repair spalling in south overhang in span D19-D20.
- Drainage: Seal and caulk leaking PVC pipe connection near D19. Replace waterproofing membrane in catch basin at pier D17.

6.4. Deck Rehabilitation

The 203 mm thick reinforced concrete deck is topped with an asphalt overlay in most places. A visual inspection and hammer-sounding of the deck soffit from a manlift was conducted on March 30, 2017. The inspection identified:

- Algal growth and wetness on vertical edges of the deck along the entire viaduct
- Previous patch repairs and new concrete spalls in the deck soffit of the south overhang in span D18-D19.
- No defects on the deck soffit between girders that would suggest conditions of structural distress or other deterioration mechanisms
- Robust concrete in all areas when struck with a hammer, with no signs of deterioration.

Nevertheless, the condition of concrete in the middle and top of deck and the condition of rebar inside the deck is currently unknown. A detailed deck condition evaluation should be completed to acquire a better understanding of the deck's condition. This should include a delamination survey (chain-drag), half-cell potential survey, and chloride ion content testing on core samples taken from the deck.

Depending on the results of this evaluation, a partial-depth concrete overlay may be recommended to extend the service life of the deck. This would remove the concrete near the deck surface which typically has higher chloride concentrations and halts corrosion in the top mat of deck rebar. Works would typically include removal of existing concrete by hydro-demolition from the top of the deck to below the top mat of deck rebar, surface preparation of the sub-deck by high-pressure water blasting, and supply and placement of a high-performance concrete overlay. Figure 22 shows an example of a partial-depth concrete overlay for a different project.

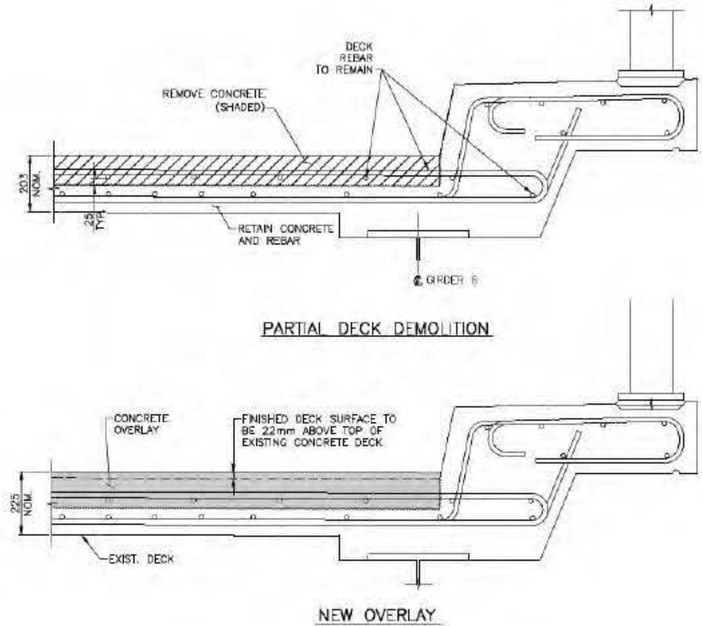


Figure 22. Example of partial-depth deck overlay
Source: MMM Group (2016), Drawing 02448-142

7. CONFIRMATION OF RETROFIT STRATEGY

7.1. Introduction

Seismic retrofit and general rehabilitation design is an iterative process by nature. First, the structure is analyzed in its existing condition, deficiencies are found, and a retrofit strategy is proposed to rectify them. Then, the structure is re-analyzed in its retrofitted configuration, member capacities are checked against the new demands, and any new deficiencies discovered are addressed by final design. This process is repeated until the entire retrofitted structure is proven to resist the demands it attracts. The retrofitted structure is expected to exhibit acceptable performance both in a seismic event and during regular service. This section outlines the structural confirmation checks carried out on the retrofit strategy proposed for the viaduct.

The proposed retrofit strategy will change the seismic and in-service behaviour of the bridge as follows:

- With split columns combined together by jackets, link slabs installed at deck level, and the gap between pier caps grouted, the bridge will behave as one continuous six-span structure rather than three discrete two-span frames.
- Jacketed columns will be stiffer, both in longitudinal and transverse directions, because (a) they have larger cross-sections, and (b) the concrete inside is confined and hence remains uncracked under shear and flexure. This effect is pronounced for split columns in the longitudinal direction, where two singular columns are joined together in an ellipse encasing the two.
- With ground improvements and soil anchors in place, the foundations will be stabilized against sliding and overturning, i.e. translations and rotations.
- With the new column and pier cap constructed at D17, the superstructure of the existing viaduct will be free to translate longitudinally and rotate about the vertical and transverse axes.

7.2. Seismic Performance

7.2.1. Capacity Design

Modern seismic retrofit philosophy is based on *capacity design*, where locations (plastic hinges) of potential inelastic flexural deformation are selected, while undesirable plastic hinge locations, or undesirable inelastic deformation mechanisms, such as shear, are inhibited by providing them with an appropriate strength margin above that corresponding to the plastic hinge strength. This strategy makes the structure as insensitive as possible to the unknown characteristics of the seismic input excitation (Priestley et al., 1996).

In the retrofit scheme proposed for the viaduct, the intended plastic hinges are at the top and bottom of the jacketed columns. The girder - pier cap connections and foundation stability should then be checked for the probable flexural strength (over-strength) of these plastic hinges. The maximum demand exerted on these elements is the minimum of the elastic demand from a response spectrum analysis and the overstrength of these joints. For this initial assessment, we have used elastic demands. The detailed design will have to consider the effect of our strength capacity.

7.2.2. Model

A new model was created in CSiBridge to represent the retrofitted structure (Figure 23). Response spectrum analyses were then repeated at the three hazard levels to obtain revised seismic demands. These revised demands were then used to obtain a preliminary design of retrofits for the girder - pier cap connections and foundations. As noted earlier, the results presented in Sections 4.3, 4.5, and 4.7 represent the elastic demands on the retrofitted structure.

Support conditions were the same as outlined in Section 3.2, except for the changes at D17 explained above.

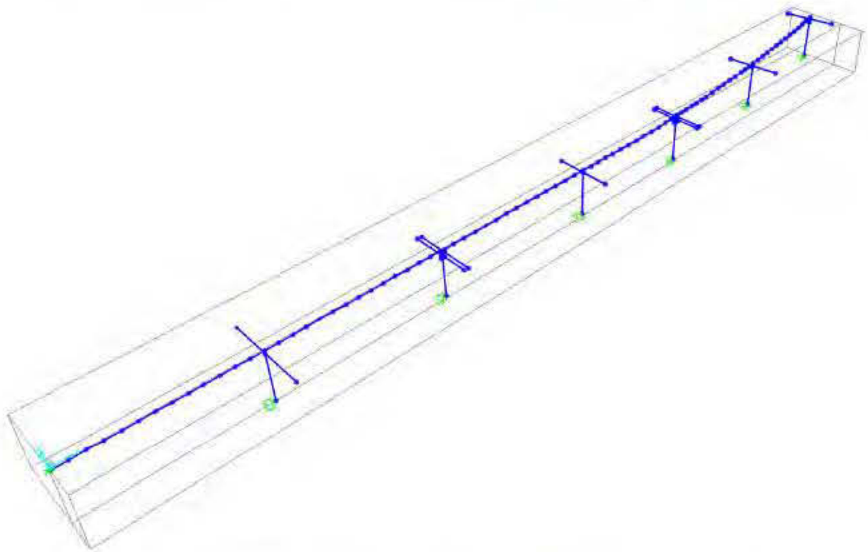


Figure 23. 3D model of retrofitted structure (left to right: D23 to D17)

7.2.3. Performance Level

We have not carried out inelastic plastic pushover analyses to ascertain the strain levels in the concrete and reinforcing steel. However, based on our experience, we expect the retrofitted structure to meet the performance requirements of a major-route bridge, as outlined in the design criteria (Section 2).

Table 8 lists performance criteria required for major-route bridges at the 2475 year return period hazard level and compares these with our expectations of the performance of the retrofitted viaduct.

Table 8. Performance criteria for extensive damage and expected performance of retrofitted viaduct at the 2475 year return period hazard level

Category	Performance Criteria	Expected Performance
General	Inelastic behavior is expected. Members may have extensive visible damage, such as spalling of concrete and buckling of braces but significant strength degradation is not permitted. Members shall be capable of supporting the dead load plus 1 lane of live load in each direction, including P-Delta effects without collapse.	Jacketed columns may form plastic hinges at their tops and bottoms. Pushover analyses are required to verify displacement (curvature) capacities are not exceeded, and measure displacements required to calculate P-Delta effects.
Concrete Structures	Extensive concrete spalling is permitted but the confined core concrete shall not exceed 80% of its ultimate confined strain limit. Reinforcing steel tensile strains shall not exceed 0.05.	Jacketed columns will not experience concrete spalling. Limits on compressive strains in concrete and tensile strains in steel listed in CAN/CSA S6-14 do not apply to jacketed columns. Strain limits will be confirmed in detailed design.
Connections	There may be significant joint distortions but damaged connections must maintain structural integrity under gravity loads.	Column-foundation, column-pier cap, and girder - pier cap connections will be designed using the capacity design principle.
Structural Displacements	There may be permanent structural offsets as long as they do not prevent use by restricted emergency traffic after inspection or the bridge, nor preclude return of full service to the bridge after major repairs.	Structural displacements will be validated during detailed design.

Bearings and Joints	Bearings may be damaged or girders may become unseated from bearings, but girders shall have adequate remaining seat length and connectivity to carry emergency traffic. Bearings and joints may require replacement.	Girders will have adequate seat length to carry emergency traffic.
Restrainers	Restraining systems might suffer damage but shall not fail	Girder - pier cap connectors will be designed not to fail at the overstrength of the columns at their top.
Foundations	Foundation lateral and vertical movements must be limited such that the bridge can be used by restricted emergency traffic. Foundation offsets shall be limited such that repairs can bring the structure back to the original operational capacity.	Soil anchors will be designed not to fail at the overstrength of the columns at their bases. They will limit lateral sliding and uplift of foundations. Ground improvements will limit movement of soils.

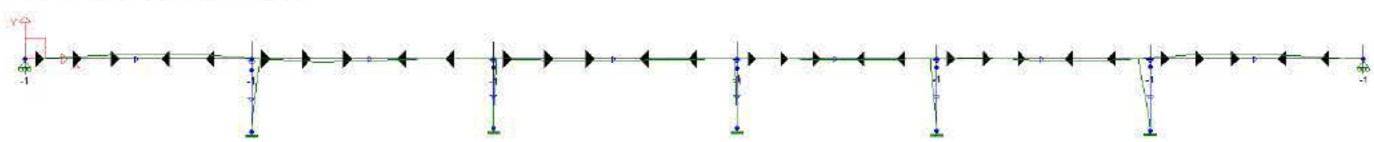
7.3. In-Service Performance

The retrofitted six-span structure has a greater longitudinal thermal movement length (225 m) than the existing two-span frames (79 m, 74 m, 72 m). As such, piers D22 to D18 are subject to greater seasonal thermal movement than in the existing configuration. This movement attracts larger bending moments to the outer columns and the superstructure. This is exacerbated by the fact that jacketed columns are stiffer than the original columns. The columns and superstructure have to be checked for these demands.

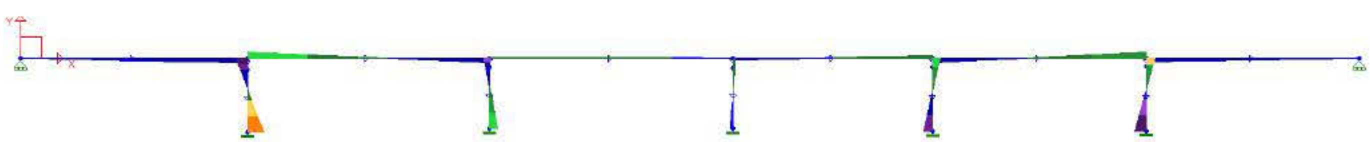
The magnitude of thermal effects is linearly proportional to the temperature range the structure is subjected to. The range starts at the temperature at which the split columns at D21 and D19 are made continuous by pouring concrete in their steel jackets. The range ends at the maximum or minimum effective temperatures defined by CSA S6-14.

7.3.1. Model

To quantify the effects of thermal movement, a 2D model of the retrofitted structure was created in S-Frame software (Figure 24). A thermal load of 1°C was applied to the superstructure, and resulting moments were linearly scaled for expansion and contraction over the desired temperature range.



(a) Thermal loading applied (contraction)



(b) Resulting bending moment diagram

Figure 24. 2D model of retrofitted bridge, elevation view (left to right: D23 to D17)

7.3.2. Capacity

While the jacketed portion of columns have ample bending and shear capacity, the 50 mm gap left at the top of pedestal and bottom of pier cap (section 5.2.2) leaves a vulnerable weak point where moment is also at its greatest. The split columns are thinner than the full columns and are therefore weaker in this regard.

One way to strengthen these sections would be to join the split columns together to act as one integral unit by way of high-strength bars placed in longitudinal holes passing through them. However, this poses the risk of cutting critical vertical column rebar, especially on the inside face of columns inaccessible to surface scanning equipment. To avoid this invasive measure, a refined analysis approach was used to verify flexural capacity at the bottom of split columns. At the top of split columns, the link slabs at deck level help share the bending between the two halves.

The superstructure was checked at ultimate and serviceability limit states.

7.3.3. Results

The "gap" sections at tops and bottoms of all columns were found to have sufficient capacity at the ultimate limit state to resist effects of thermal loads in combination with dead, live, and wind loads. This is subject to the construction temperature being between 6 and 16°C when split columns are jacketed. This condition can be met by timing this activity for the spring or fall months and is not considered to be onerous for the contractor.

Some cracking was predicted at the bottom of girders near piers D22 and D18 during the coldest and warmest months of the year. However, these cracks are expected to close under prestressing load as temperatures moderate. This shortcoming is associated with the serviceability limit state, not the ultimate limit state. Hence, these cracks are not expected to affect the load carrying capacity of the bridge, and can be monitored and repaired if deemed necessary.

The elastomeric bearings at abutment D23 were found to have sufficient effective height to accommodate the longitudinal thermal movements.

8. COST ESTIMATE

A Class D cost estimate (+30%) of the proposed seismic retrofits was developed for the purpose of program planning, to establish a more specific definition of client needs, and to inform the renewal options analysis. The cost estimate is the summation of all identifiable project elemental costs, which were in turn estimated using our best estimates of quantities and unit prices, quotes from suppliers, fabricators and contractors, construction costs for similar projects, and cost guides developed by BC and Alberta Ministries of Transportation.

Table 9 lists the detailed cost estimate for seismic retrofit and general rehabilitation items. Foundation earthwork includes excavation, backfill and surface restoration associated with accessing foundations for installing foundation anchors and ground improvements. Deck rehabilitation is presented as a provisional sum, because it is contingent on the results of the upcoming deck condition evaluation. The cost of seismic retrofits for the 475 year return period hazard level are close to the 2475 and 975 year return period hazard levels.

Table 9. Cost estimate for seismic retrofits and general rehabilitation

Seismic Retrofits

	Hazard level return period (years)
	2475 and 975
Column Jackets	$1,350,000
Foundation Earthwork	$340,000
Foundation Anchors	$970,000
Ground Improvements	$690,000
Girder - Pier Cap Connectors	$250,000
Link Slabs	$90,000
Abutment: Seat Length Extension	$20,000
Abutment: New Shear Keys	$10,000
New Joint at Costco / Spectrum Building	$240,000
New Pier at D17	$240,000
Total	**$4,200,000**

General Rehabilitation

Remove Barriers	$50,000
Remove Asphalt	$150,000
New Joint at Abutment	$50,000
Miscellaneous Defect Repairs	$20,000
Total	**$270,000**

Provisional Sum

Deck Rehabilitation	**$2,120,000**

The methodologies used to estimate the more costly components of the work are as follows:

- Steel jackets for columns: Using input from a prominent local steel fabricator
- Soil anchors for foundations: Using construction costs for the "Cambie Street Bridge, Foundation Seismic Retrofit" project, as the soil conditions and proposed anchor configuration are similar to those at the Dunsmuir Viaduct
- Ground improvements: Jet grouting, using input from the geotechnical engineer and a contractor
- Deck rehabilitation: Quotes from local contractors, and Alberta Unit Price Averages Reports.

Table 9 includes costs with general costs and contingencies included. General costs include mobilization, traffic management, quality management, and other costs that form part of the tender price. Contingencies are estimated at 30% and include:

- Design contingency: Cost of design changes during the detailed design phase.
- Construction contingency: Cost of unforeseen changes during construction, such as those arising from sub-surface issues, geotechnical conditions and existing utilities.

Table 10. Cost estimate with general costs and contingencies included

	No Deck Rehab.	With Deck Rehab.
General: 5% of Tender Price	$240,000	$350,000
Seismic Retrofits	$4,200,000	$4,200,000
General Rehabilitation	$270,000	$270,000
Deck Rehabilitation	$0	$2,120,000
Tender Price	**$4,710,000**	**$6,940,000**
Contingencies: 30% of Tender Price	$1,410,000	$2,080,000
Total Cost	**$6,120,000**	**$9,020,000**

While great effort was expended in developing this estimate, there are uncertainties associated with it:

- The quantities are based on this preliminary seismic assessment study, and not a detailed design. Many components were not designed, and were simply assumed to be similar to past designs for similar projects.
- Future bid prices depend on many factors, such as market prices of materials, labour, and equipment, the contractor's risk tolerance, and how busy contractors are with other projects. Because these factors can fluctuate from one year to another, bid prices may vary from past project experience.

9. CLOSURE

Please contact the undersigned should you have any questions or comments regarding this report.

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