Photo: Horseshoe Bay Underpass

Upper Levels Highway Underpasses - Seismic Retrofit Project

Condition Assessment Report

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PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA K. A. RUSSELL # 54440

2022-05-27

Keith Russell, P.Eng.

Date

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA B. R. ROBAZZA # 47847

2022-05-27

2022-05-27

Brook Robazza, P.Eng.

Date

2022-05-27

Terrence Davies, P.Eng.

Date

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1 INTRODUCTION

TY Lin International (TYLI) was retained by the BC Ministry of Transportation and Infrastructure (BC MOTI) to conduct condition assessments of four bridge structures passing over Highway 1 in West Vancouver, British Columbia. This report summarizes the results of the condition assessments of the four structures.

1.1 Structures Inspected

The bridge names and their corresponding Ministry ID numbers are listed below, from west to east.

1. Horseshoe Bay Underpass (02339) – shortened as Horseshoe Bay U/P
2. Eagleridge Drive Underpass (02341) – shortened as Eagleridge U/P
3. Caulfeild Drive Underpass (02343) – shortened as Caulfeild U/P
4. Westmount Road Underpass (02347) – shortened as Westmount U/P

TYLI carried out a series of inspections to support the condition assessments. The primary inspections were carried out between November 2021 and January 2022, as described below:

Date of Inspection	Inspectors
November 16th, 2021	Brook Robazza, PhD, PEng Keith Russell, PEng
November 24th, 2021	Brook Robazza, PhD, PEng Juan-Carlos Carvajal, PhD, PEng (Thurber) Mehdi Amini, PhD, PEng (Thurber) Michaella Marchetta, EIT (Thurber)
November 27th, 2021	Brook Robazza, PhD, PEng Keith Russell, PEng Juan-Carlos Carvajal, PhD, PEng (Thurber)
January 25th, 2022	Brook Robazza, PhD, PEng Keith Russell, PEng

Weather conditions were clear on all inspection days. It is worth noting that the November 16th inspection was conducted immediately following a weekend of severe weather, which provided an opportunity for insight into the efficacy of drainage structures in and around the bridges.

1.2 Objectives

The primary objective for the condition assessments was to identify needs for structural renewal specific to seismic retrofit strategies that are being developed as part of TYLI's larger seismic retrofit assignment. TYLI's structural models and capacity calculations will be modified to account for drawing discrepancies and damaged components that were observed in the field during the condition assessments. Therefore, the condition assessments focused on items whose deficiencies would have the greatest effect on the seismic performance of the structure, i.e. pier legs, pedestals, end diaphragms, bracing, half-joints, bearings and foundations.

The condition assessments also identified general rehabilitation items that may affect the long-term performance of the structures but would not necessarily affect their performance during a seismic event. The intent is that these items could be repaired in tandem with the seismic retrofit works at BC MOTI's discretion.

A non-destructive deck evaluation (chain drag) of the Westmount U/P will also be completed in early 2022 when weather permits.

1.3 Methodology

The condition assessments were conducted on foot from the embankments and also from deck level. No specialized inspection equipment such as a boom lift or under-bridge inspection vehicle was employed. The general walkthrough procedure was as follows:

1.) Assess the condition of the abutment bearings and the abutment embankments.

2.) Note any embankment-retaining structures that could affect global stability.

3.) Review the abutment back walls/wingwalls for cracking/delamination.

4.) Review the abutment bearing shelves for cracking/delamination.

5.) Review the general condition of the deck and overhang soffit, including spalls, corrosion, efflorescence and cracking. Note any observed signs of leakage.

6.) Assess the condition of the inclined pier bearings and pedestals.

7.) Assess the general condition of the girders and inclined pier legs and note condition of coating and signs of corrosion/section loss.

8.) Assess the general condition of the running surface of the deck and sidewalks.

9.) Assess the deck joints and half-joints, with particular attention paid to locations where signs of leakage are present on the underside of the deck.

2 BRIDGE DESCRIPTIONS

2.1 Horseshoe Bay Underpass

Horseshoe Bay Underpass is located at the intersection of Marine Drive and Highway 1 in West Vancouver, BC.

The bridge consists of 5 spans supported by an east and west abutment, two inclined leg piers, and a central delta-shaped pier (see Exhibit 2.1-1). The superstructure is composed of steel girders made composite with a cast-in-place reinforced concrete deck slab through the use of shear studs along the length of the girders. The concrete deck carries one westbound and one eastbound lane that splits into two lanes at the east end, allowing drivers to turn north or south when exiting the bridge. The westbound lane cuts inwards towards the centerline of the bridge near the west end. The bridge is skewed, with the magnitude of the skew varying between spans, to accommodate a horizontal curve in the roadway. Three street light poles are located on the deck and are supplied by electrical conduits carried by the bridge. Four deck joints are present in the deck with one located over each abutment back wall and one located at both half-joints in the side spans (see Exhibit 2.1-2).

The east and west side spans are simply supported, spanning from the abutments to a half-joint support located 10 feet from the adjacent inclined pier legs. The remaining three spans are continuous and are rigidly connected to the delta pier and inclined pier legs via complete penetration field welds in the web plates. The continuous spans and west side span are composed of 6 haunched steel plate girders while the east side span, which flares towards the east abutment, has 9 girders including three that frame into the sixth girder from the south.

Articulation of the continuous spans is provided by pinned bearings at the base of the inclined pier legs and central delta-shaped pier legs. The top and bottom pinned bearing rocker plates are linked by ‘connector rings’ which provide uplift resistance. Pinned bearing translational resistance is provided by two ½” washers bolted to either end of the pin. Articulation of the side spans is provided by rotational freedom at the half-joint, and by sliding bearings at the abutments. Shear resistance is provided by a v-shaped abutment bearing supporting the 4th girder from the south on both side spans.

The delta and inclined pier pedestals are supported by spread footings on top of a shallow layer of imported fill. Both abutments are also supported on spread footings on top of a shallow layer of imported fill.

Exhibit 2.1-1: Horseshoe Bay U/P Looking South

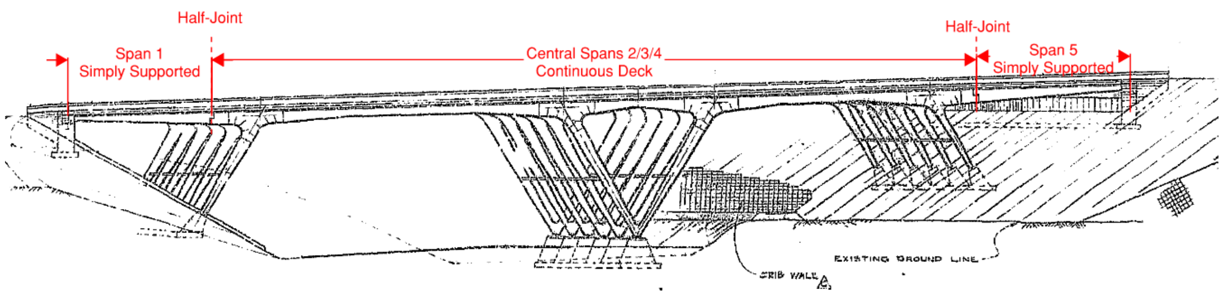


Exhibit 2.1-2: Horseshoe Bay U/P - Deck Continuity

2.2 Eagleridge Underpass

Eagleridge Underpass is located at the intersection of Eagleridge Drive and Highway 1 in West Vancouver, BC. The 3-span superstructure is composed of 5 parallel haunched steel plate girders, which are composite with a reinforced concrete deck. The deck carries one northbound and one southbound lane of traffic as well as a sidewalk located along the east side of the bridge. Based on the original design drawings, the deck slab is composed of a partial-depth, prestressed precast panels, and a cast-in-place reinforced slab. Transverse deck joints with compression seals are located over both the north and south abutment back walls.

The haunched steel plate girders are supported by bearings at the north abutment and south abutment, and by two inclined pier legs (Exhibit 2.2-1). Shear studs are arranged in groups along the length of the girder top flanges, except in negative moment regions above the two inclined pier legs where no shear studs are provided. Pockets in the precast panels accommodate the stud groups and are filled with concrete during the deck casting to make the girder composite. The girders and deck are therefore non-composite in the negative moment regions.

The inclined pier legs are integral with the plate girders forming a rigid frame superstructure. The girders and inclined pier legs are rigidly joined via complete penetration field welds. Bottom flange plates from the girders continue down the inclined pier legs forming the top and bottom flanges of the inclined legs (the side span girder bottom flange become the top flange of the pier leg while the centre span girder bottom flange becomes the bottom flange of the inclined leg). For the purpose of this report, the intersection of the inclined legs and girder is referred to as the “knee”.

Articulation of the superstructure is provided by pinned bearings at the base of the inclined pier legs and sliding bearings at the abutments. The pinned bearings are proprietary ‘Conenco’ bearings for which details are missing in the design drawing package. The bearings appear to be either spherical or disc bearings, and it not clear based on the design drawings whether transverse moment fixity is provided by these bearings.. Anchor bolts at the abutment bearings provide uplift resistance while lateral shear resistance is provided by two shear keys on the upper bearing plate of the third abutment bearing from the west. The abutment is supported on spread footings on bedrock with concrete fill while the pier leg pedestals are supported by spread footings on top of a shallow layer of imported fill.

Electronic signs for the nearby BC Ferries terminal at Horseshoe Bay are supported by the two westmost girders over the westbound lanes of Highway 1.

Exhibit 2.2-1: Eagleridge U/P Looking East

2.3 Caulfeild Underpass

Caulfeild Underpass is located at the intersection of Headland Drive and Highway 1 in West Vancouver, BC. The 3-span superstructure is composed of six parallel haunched steel plate girders, which are composite with a reinforced concrete deck. The deck carries one northbound and one southbound lane, a dedicated left turn lane at either end of the bridge, and a sidewalk which is located along the east side of the bridge. The deck slab is composed of partial-depth, prestressed precast panels, and a cast-in-place reinforced slab. Transverse deck joints with compression seals are located over both the north and south abutment back walls.

The haunched steel plate girders are supported by bearings at the north abutment and south abutment, and by two inclined pier legs (Exhibit 2.3-1). Shear studs are grouped along the length of the girder top flanges, except in negative moment regions above the two inclined pier legs where no shear studs are provided. Pockets in the precast panels accommodate the stud groups and are filled with concrete during the deck casting to make the girder composite. The girders and deck are therefore non-composite in the negative moment regions.

The inclined pier legs are integral with the plate girders forming a rigid frame superstructure. The girders and inclined pier legs are rigidly joined via complete penetration field welds. Bottom flange plates from the girders continue down the inclined pier legs forming the top and bottom flanges of the inclined legs (the side span girder bottom flange become the top flange of the pier leg while the centre span girder bottom flange becomes the bottom flange of the inclined leg). For the purpose of this report, the intersection of the inclined legs and girder is referred to as the “knee”.

Articulation of the superstructure is provided by pinned bearings at the base of the inclined pier legs and sliding bearings at the abutments. The top and bottom pinned bearing rocker plates are linked by ‘connector rings’ which provide uplift resistance. Pinned bearing translational resistance is provided by two ½” washers bolted to either end of the pin. Anchor bolts at the abutment bearings provide uplift resistance while lateral shear resistance is provided by two shear keys on the upper bearing plate of third abutment bearing from the west. The pier leg pedestals and abutments are supported on spread footings on top of a shallow layer of imported fill.

Utility pipes are carried by the bridge between the two westmost girders.

Exhibit 2.3-1: Caulfield U/P Looking East

2.4 Westmount Underpass

Westmount Underpass is located at the intersection of Westmount Road and Highway 1 in West Vancouver, BC. The 3-span superstructure is composed of five haunched steel plate girders, which are composite with a reinforced cast-in-place concrete deck. The bridge deck, which is skewed at approximately 32 degrees, carries one northbound and one southbound lane of traffic and a sidewalk located along the west side of the bridge. Transverse deck joints with compression seals are located over both the north and south abutment back walls.

The haunched steel plate girders are supported by bearings at the north abutment and south abutment, and by two inclined pier legs (Exhibit 2.4-1). Shear studs are only located in the positive moment region of the main span, and approximately the north half of the north side span. The girders and deck are non-composite in the regions without shear studs.

The inclined pier legs are integral with the plate girders forming a rigid frame superstructure with the girders and pier legs rigidly joined via complete penetration field welds. Bottom flange plates from the girders continue down the inclined pier legs forming the top and bottom flanges of the inclined legs (the side span girder bottom flange become the top flange of the pier leg while the centre span girder bottom flange becomes the bottom flange of the inclined leg). For the purpose of this report, the intersection of the inclined legs and girder is referred to as the "knee".

Articulation of the superstructure is provided by pinned bearings at the base of the inclined pier legs and sliding bearings at the abutments. The top and bottom pinned bearing rocker plates are linked by 'connector rings' which provide uplift resistance. Pinned bearing translational resistance is provided by two ½" washers bolted to either end of the pin. Hold-down bolts are present at the south abutment bearings and provide local uplift resistance. There are no hold-down bolts present at the north abutment bearings. Lateral shear resistance is provided by two shear keys on the upper bearing plate of the abutment bearing from the west. The north abutment and pier leg pedestals are supported on spread footings on bedrock with concrete fill. while the south pier leg pedestals are supported by spread footings on a shallow layer of imported fill.

Exhibit 2.4-1: Westmount U/P Looking East

3 ASSESSMENT FINDINGS

Observations and assessment findings have been summarized by element type in the following sections.

3.1 Horseshoe Bay Underpass

For the purposes of this report, the following nomenclature has adopted for Horseshoe Bay U/P.

- Girders are numbered increasing south to north.
- Inclined pier leg numbers correspond to the girder number they are integral with, and the end of the bridge at which they can be found (e.g. Inclined Pier Leg W2 would be in the west line of pier legs, integral with the 2nd girder from the south).
- Central delta-shaped pier numbers correspond to the girder number they are integral with and are denoted with a 'C' prefix (e.g. Pier C2 would the delta-shaped pier integral with the 2nd girder from the south).
- Abutment bearing numbers correspond to the girder number that they support and the end of the bridge at which they can be found (e.g. Abutment Bearing E4 would be found at the east abutment, supporting the 4th girder from the south).
- Pier bearing numbers correspond to the pier that they support (Inclined Pier Bearing W2 supports Inclined Pier Leg W2, Pier Bearing C2 supports delta-shaped pier C2 etc.).

3.1.1 Inclined Pier Bearings and Plinth Observations

- Central delta pier bearings were observed to be in generally good condition. Exposure to moisture and runoff has led to mildew and moss buildup on the bearings and plinths, but no exposed corrosion was observed on the bearing steel. However, localized bubbling of the paint around the anchor bolt washers suggested some corrosion was present (Exhibit 3.1-1, Exhibit 3.1-2).
- Central pier bearing plinths were observed to be in good condition. The grout layer at the top of the plinths immediately below the bearing had debonded from the parent concrete on all the plinths. However, these debonded sections did not undermine the bearing (Exhibit 3.1-3).
- The west inclined pier bearings were in good condition, having been recently repainted.
- Coating loss was present on approximately 50% the east pier inclined leg bearing rocker plates (Exhibit 3.1-4). Discolouration of the paint was observed on all east pier

bearings, suggesting that corrosion may be present below the coating. Coating loss and corrosion were also noted on some of the pin washer however, no section loss was observed.

- The grout layer immediately below the bearing had debonded from the parent concrete on all east pier inclined leg bearing plinths (Exhibit 3.1-5). At Pier 5, removal of the debonded concrete revealed a delamination with exposed, corroded rebar in the plinth (Exhibit 3.1-6). The delamination did not undermine the bearing.

Exhibit 3.1-1: Central Pier Bearing Plinth 6, Typical Condition

Exhibit 3.1-2: Central Pier Bearing 6, Typical Condition

Exhibit 3.1-3: Debonded Grout at Delta Pier 4 Plinth

Exhibit 3.1-4: Rocker Plate/Washer Corrosion, Inclined Pier Bearing E4

Exhibit 3.1-5: Inclined Pier Bearing E1, Superficial Delamination in Grout Pad

Exhibit 3.1-6: Inclined Pier Bearing E5, Exposed Rebar After Removal of Delaminated Concrete

3.1.2 Abutment Bearing and Bearing Shelf Observations

- Mild to moderate corrosion was present on all abutment bearings (Exhibit 3.1-7). Corrosion appeared to be less advanced at the west abutment. (Exhibit 3.1-8).

- Two vertical cracks were observed in the bearing shelf concrete just to the north of bearing E4 (Exhibit 3.1-9).

- Erosion was observed in the slope in front of the east abutment, with the grade at the top of the slope adjacent to the abutment wall appearing to exceed 45 degrees (Exhibit 3.1-10).

- The east abutment joint seal exhibited signs of leakage with water observed along the abutment wall at the south end (Exhibit 3.1-11). The west joint was observed to be functioning correctly.

Exhibit 3.1-7: Abutment Bearing E7, Moderate Corrosion of Base Plate and Sole Plate. Typical East Abutment Bearing Condition.

Exhibit 3.1-8: Abutment Bearing W6, Typical West Abutment Bearing Condition.

Exhibit 3.1-9: Vertical Crack Just North of Abutment Bearing E4

Exhibit 3.1-10: Erosion of East Abutment Slope.

Exhibit 3.1-11: Water on South End of East Abutment Bearing Shelf

3.1.3 Structural Steel Condition

- Localized coating discolouration was present on the soffit of the bottom flanges of all girders, suggesting corrosion beneath the coating. Section loss was not observed (Exhibit 3.1-12).
- Both central delta pier and inclined pier legs were in good condition (Exhibit 3.1-13).
- No significant corrosion or section loss was observed on the inclined pier legs or girders.

Exhibit 3.1-12: View of Steel Condition and Efflorescence on Soffit Looking East

Exhibit 3.1-13: West Inclined Pier Legs Observed in Good Condition

3.1.4 Deck Condition

- Efflorescence was observed on the deck soffit, and generally concentrated at the transverse cold joints used in the deck pour sequence (Exhibit 3.1-12).
- The deck wearing surface was in very good condition.
- The pedestrian railing at the north side of the bridge deck does not meet the Ministry height requirement for bicycle traffic.

3.2 Eagleridge Underpass

For the purposes of this report, the following nomenclature has adopted for Eagleridge U/P.

- Girders are numbered increasing west to east.
- Inclined pier leg numbers correspond to the girder number they are integral with and the end of the bridge at which they can be found (e.g. Inclined Pier Leg N2 is in the north line of pier legs, integral with the 2nd girder from the west).
- Abutment bearing numbers correspond to the girder number that they support and the end of the bridge at which they can be found (e.g. Abutment Bearing S4 is at the south abutment, supporting the 4th girder from the west).
- Inclined pier bearing numbers correspond to the pier that they support (e.g. Inclined Pier Bearing N2 supports Inclined Pier Leg N2 as defined above).

3.2.1 Inclined Pier Bearings and Plinth Observations

- Discolouration of the paint was observed on all pier bearings, suggesting that corrosion may be present below the coating. In locations where the coating had failed, corrosion was observed (Exhibit 3.2-1, Exhibit 3.2-2). No section loss was evident.
- The condition of the anchor bolts ranged from corroded with full coating loss to very good. No significant section loss was observed.
- Bearing steel was in generally good condition with protective coatings also in generally good condition.
- Bearing plinths were observed to be in good condition. However, the grout layer at the top of the plinths immediately below the bearing had debonded from the parent concrete on approximately 80% of the plinths (Exhibit 3.2-3). The debonded section did not appear to undermine the bearings, though further investigation may be required to confirm this.

Exhibit 3.2-1: Pier S4 Bearing, Minor Corrosion on Base Plate. Crack in the Debonded Layer.

Exhibit 3.2-2: Pier N3 Bearing, Corrosion on Base Plate

Exhibit 3.2-3: Pier S4 Bearing, Debonding of Grout Pad. Crack in the Debonded Layer.

3.2.2 Abutment Bearings and Bearing Shelf Observations

- Moderate corrosion was present on all bearings (Exhibit 3.2-4). Flaking of corroded steel on the top bearing plate was common and suggests possible deterioration of the sliding surface.

- Advanced corrosion of the anchor bolts was observed, resulting in complete failure of some of the bolts. Bolts have failed at bearings S1 and S4 (Exhibit 3.2-5). Several other bolts have not yet failed but showed signs of necking (section loss) where they enter the bearing shelf (Exhibit 3.2-6). No section loss is visible at the bolts holding down the girder 3 lateral restraint bearings (N3 and S3), however the interface between the bolt and the plinth where necking had occurred on other bolts was hidden from view by keeper plates.

- Anchor bolts extensions have been welded to the top of old anchor bolts (Exhibit 3.2-7).

- The keeper plates which provide a lateral load path from the girder 3 bearings at the abutment are heavily corroded. The impact on their capacity is unknown but should be further investigated if they are to continue to be relied upon. (Exhibit 3.2-8).

- The deck joint seals over the abutments appear to be functioning correctly.

- The south bearing shelf was in good condition, and delaminations were not observed. North bearing shelf had localized delaminations (approximately 500x500) below girder 3.

Exhibit 3.2-4: Abutment Bearing N4, Moderate Corrosion

Exhibit 3.2-5: Abutment Bearing S1, Failed Anchor Bolt

Exhibit 3.2-6: Abutment Bearing S5, Severe Necking at Base of Anchor Bolt.

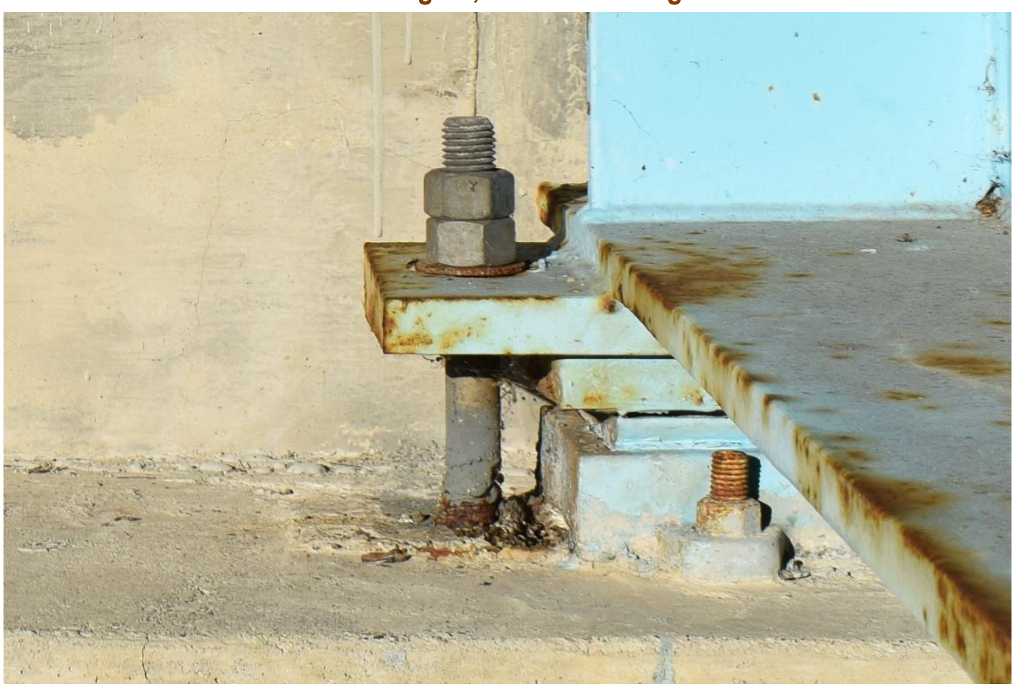


Exhibit 3.2-7: Abutment Bearing N1, New Threaded Rod Welded to Existing Anchor Bolt

Exhibit 3.2-8: Bearing S3, Corroded Keeper Plate

3.2.3 Structural Steel Condition

- Coating failure and corrosion were observed on all six girders along the interface between the deck soffit and the top flange (Exhibit 3.2-9), with the most widespread corrosion present on the exterior girders.

- The top flange of the outer two girders was observed to be locally deformed resulting in an undulating appearance. The deformations are believed to result from pack rust between the deck slab and the flange (Exhibit 3.2-10).

- Coating discolouration was present on the underside of the bottom flange of all girders, suggesting corrosion is occurring beneath the coating. Section loss was not observed (Exhibit 3.2-11).

- The pier leg coating was in poor condition with large areas of coating failure on the bottom flange. In these locations, corrosion product was flaking off (Exhibit 3.2-12). The corrosion product is not considered to currently affect the capacity of the member.

Exhibit 3.2-9: Girder 3 Looking North, Coating Loss and Corrosion Along Top Flange

Exhibit 3.2-10: Girder 5 Looking North, Undulating Top Flange Due to Pack Rust

Exhibit 3.2-11: Looking South, Girder Bottom Flange Corrosion

Exhibit 3.2-12: Pier Leg S3 Bottom Flange Corrosion

3.2.4 Deck Condition

- Deck soffit concrete had spalled in several locations, exposing transverse prestressed tendons with the largest spalls located at the south end of the west overhang (Exhibit 3.2-13.) and at the transverse joints of the west overhang between precast segments (Exhibit 3.2-14). The east overhang was in comparatively good condition. The extent to which corrosion may extend from zones of exposed tendons into the slab is unknown.

- The soffit between girders 4 and 5 had localized spalls with corrosion on the exposed prestressed tendons (Exhibit 3.2-15).

- Previously completed deck repairs on the wearing surface are performing well and the wearing surface was generally in very good condition with narrow (<0.3mm) transverse cracks observed.

- The pedestrian railing does not meet the Ministry height requirement for bicycle traffic.

Exhibit 3.2-13: West Deck Fascia at South End of Bridge, Spalling and Exposed PT

Exhibit 3.2-14: West Deck Fascia, Spalling and Exposed Prestressing Strands

Exhibit 3.2-15: Concrete Spall and Exposed Strand Deck Soffit Between Girders 4 and 5, Looking North

3.3 Caulfeild Underpass

For the purposes of this report, the following nomenclature has adopted for Caulfeild U/P.

- Girders are numbered from west to east.
- Inclined pier leg numbers correspond to the girder number they are integral with, and the end of the bridge at which they can be found (e.g. Inclined Pier Leg N2 is in the north line of pier legs, integral with the 2nd girder from the west).
- Abutment bearing numbers correspond to the girder number that they support and the end of the bridge at which they can be found (e.g. Abutment Bearing S4 is at the south abutment, supporting the 4th girder from the west).
- Inclined pier bearing numbers correspond to the pier that they support (e.g. Inclined Pier Bearing N2 supports Inclined Pier Leg N2 as defined above).

3.3.1 Inclined Pier Bearings and Plinth Observations

- Discolouration of the paint was observed on all pier bearings, suggesting that corrosion may be present below the coating (Exhibit 3.3-1). No section loss was observed, and the anchor bolts appeared to be in good condition.
- Coating failure was observed around the edges of the end washers on the pin, which provide lateral fixity of the pier legs. However no section loss was observed.
- Coating failure was observed on approximately 50% of the upper and lower bearing rocking plates (Exhibit 3.3-2), however no section loss was observed. This was also the location of the most advanced pier bearing corrosion.
- Concrete bearing plinths were observed to be in good condition. The grout layer at the top of the plinths immediately below the bearing had debonded from the parent concrete on approximately 50% of the plinths (Exhibit 3.3-3). The debonded section did not appear to undermine the bearings, though further investigation may be required to confirm this.

Exhibit 3.3-1: Inclined Pier Bearing S4, Typical Inclined Pier Bearing Condition

Exhibit 3.3-2: Inclined Pier Bearing S4, Corrosion on Lower Rocker Plate

Exhibit 3.3-3: Inclined Pier Bearing N5, Debonded Grout Pad

3.3.2 Abutment Bearings and Bearing Shelf Observation

- Mild to moderate corrosion was present on all abutment bearings, with the most advanced corrosion observed on the outermost bearings (Exhibit 3.3-4). Corrosion product was flaking off the top bearing plate and suggests possible deterioration of the sliding surface.

- Advanced corrosion and section loss of the anchor bolts was observed, resulting in complete failure of some of the bolts. Bolts have failed at bearings N4, S1 and S6 (Exhibit 3.3-5). Several other bolts have not failed but exhibit signs of necking (section loss) where they enter the bearing shelves (Exhibit 3.3-6). No section loss was visible at the bolts holding down the girder 3 lateral restraint bearings (S3 and N3), however the interface between the bolt and the plinth where necking was observed on other bolts was hidden from view by keeper plates. Several bolts had also been bent longitudinally towards or away from the abutment (Exhibit 3.3-7).

- Water ingress was observed at the north deck joint seal, with leaks near bearings N1 and N6. Water was also observed near bearings S1 and S6. Leaks were especially pronounced beneath the utilities at the northwest corner of the deck, where organic debris was collecting on the face of the bearing shelf (Exhibit 3.3-8).

- Anchor tabs connecting the bearing base plate to the bearing shelf were severely corroded or fractured in many cases (Exhibit 3.3-9).

- Vertical cracks and local delaminations were present on the front face of the bearing shelves (Exhibit 3.3-10).

Exhibit 3.3-4: Abutment Bearing N1, Heavily Corroded

Exhibit 3.3-5: Abutment Bearing S6, Anchor Bolt Failure

Exhibit 3.3-6: Abutment Bearing N6, Necking of Bolt Entering Bearing Shelf

Exhibit 3.3-7: Abutment Bearing N4, Bolt Bent Towards the North

Exhibit 3.3-8: Water Ingress Through Joint at North Abutment Beneath Utilities

Exhibit 3.3-9: Abutment Bearing S2, Fractured Tabs on Base Plate

Exhibit 3.3-10: South Bearing Shelf, Cracking, Delaminations and Past Repairs Visible

3.3.3 Structural Steel Condition

- Coating loss and corrosion were observed on all six girders along the interface between the deck soffit and the top flange (Exhibit 3.3-11), with the most extensive corrosion present on the exterior girders.
- Coating discolouration was present in a number of localized areas on the soffit of the bottom flanges of all girders, suggesting corrosion beneath the coating. Section loss was not observed (Exhibit 3.3-12).
- The coating of the pier legs was in fair condition, with localized patches of discolouration and coating loss. However, the coating condition was better than the soffit of the underside of the girder bottom flanges. Section loss was not observed (Exhibit 3.3-13).

Exhibit 3.3-11: Girder 6 Top Flange Corrosion, Looking North

Exhibit 3.3-12: Bottom Flange Corrosion/Coating Loss

Exhibit 3.3-13: South Pier Legs

3.3.4 Deck Condition

- Spalls were observed in the deck soffit concrete in several locations, exposing prestressing tendons. This was most frequently observed along the underside of the west deck overhang (Exhibit 3.3-14). By contrast, the east overhang was in comparatively good condition.

- The soffit between girders 4, 5 and 6 contained local spalls where exposed prestressing tendons were observed to be corroding (Exhibit 3.3-15). The extent to which corrosion may extend from zones of exposed tendons into the slab is unknown.

- A strip of discolouration was observed in the soffit concrete extending transversely across the full width of the deck from a large soffit spall on the south side of the bridge between girders 4 and 5. Corrosion was observed on the tendons at the spall. An arm’s length inspection may be able to confirm if the discolouration is indicative of delaminated concrete extending along the width of the deck, and/or corrosion of the rebar/tendons (Exhibit 3.3-16).

- There was damage to the asphalt in the southbound lane adjacent to the north deck joint armouring. The armouring itself was also fractured. (Exhibit 3.3-17).

- Deck surface repairs have previously been performed on an estimated 20% of the deck.

- The pedestrian railing does not meet the Ministry height requirement for bicycle traffic.

Exhibit 3.3-14: West Deck Fascia, Corroded Tendons Exposed at Concrete Spalls

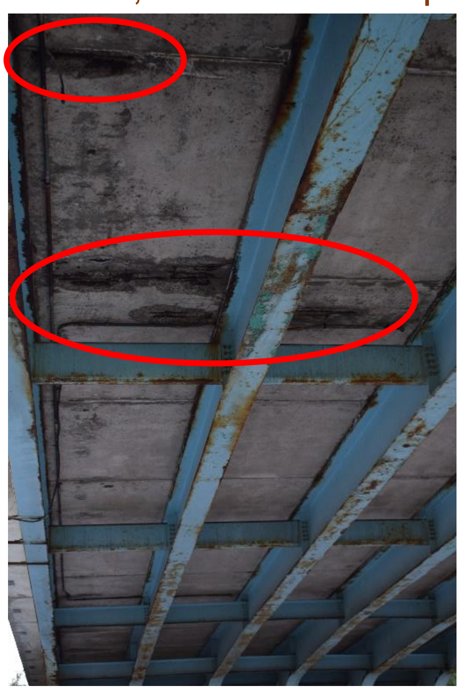


Exhibit 3.3-15: Soffit of Deck with Spalls between Girder 4/5/6, Looking South

Exhibit 3.3-16: Transverse Strip of Discolouration Along Soffit of Deck, South End of Bridge

Exhibit 3.3-17: Asphalt Damage and Fractured Armouring near North Joint

3.4 Westmount Road Underpass

For the purposes of this report, the following nomenclature has adopted for Westmount U/P.

- Girders are numbered increasing west to east.
- Inclined pier leg numbers correspond to the girder number they are integral with, and the end of the bridge at which they can be found (e.g. Inclined Pier Leg N2 is in the north line of pier legs, integral with the 2nd girder from the west).
- Abutment bearing numbers correspond to the girder number that they support and the end of the bridge at which they can be found (e.g. Abutment Bearing S4 is found at the south abutment, supporting the 4th girder from the west).
- Inclined pier bearing numbers correspond to the pier that they support (e.g. Inclined Pier Bearing N2 supports Inclined Pier Leg N2 as defined above).

3.4.1 Inclined Pier Bearing and Plinth Observations

- Discolouration of the paint was observed on all pier bearings, suggesting that corrosion may be occurring below the coating. Localized areas of coating failure and surface corrosion were observed on approximately 80% of the bearings. No section loss was observed.
- North bearing rocker plates had the most widespread coating loss and corrosion, but no section loss was visible (Exhibit 3.4-1).
- The lateral pin end washers, which provide lateral fixity of the pier legs, were corroded on most bearings (Exhibit 3.4-1). Section loss was not observed.
- Embankment fill was observed accumulating around north inclined pier bearing plinths.

Exhibit 3.4-1: Pier N3 Bearing, Corrosion on Pin Washer and Rocker Plates

3.4.2 Abutment Bearing and Bearing Shelf Observations

- Areas of moderate to severe corrosion were observed on all abutment bearings (Exhibit 3.4-2). The most advanced corrosion was observed on the bearings closest to deck joints (S1, S2, N2, N3, and N4). Corrosion product was commonly observed to be flaking off the top bearing plates.

- A polymer sheet in Abutment Bearing Assembly N2 had failed (Exhibit 3.4-3).

- Corrosion was observed at the base of the anchor bolts. However, no bolts were observed to have failed and no section loss was observed. Similarly, no section loss was visible at the bolts holding down the girder 3 bearings (S3 and N3), however the interface between the bolt and the plinth is hidden from view by keeper plates.

- The keeper plates which provide a lateral load path from the girder 3 bearings at the abutment were corroded (Exhibit 3.4-4).

- Discolouration of the abutment concrete and a buildup of debris indicate that water has leaked through the deck joint at the south abutment between girders 1 and 2 (Exhibit 3.4-5). Similar discolouration was also present between girders 2 to 3 and 3 to 4 at the north abutment (Exhibit 3.4-6, Exhibit 3.4-7).

- The concrete at the abutment bearing shelves appeared in good condition with minor cracking/delaminations.

Exhibit 3.4-2: Abutment Bearing N4, Corrosion of Bearing and Girder Flange

Exhibit 3.4-3: Abutment Bearing N2, Failed Polymer Sheet

Exhibit 3.4-4: Abutment Bearing S3, Corrosion on Keeper Plate

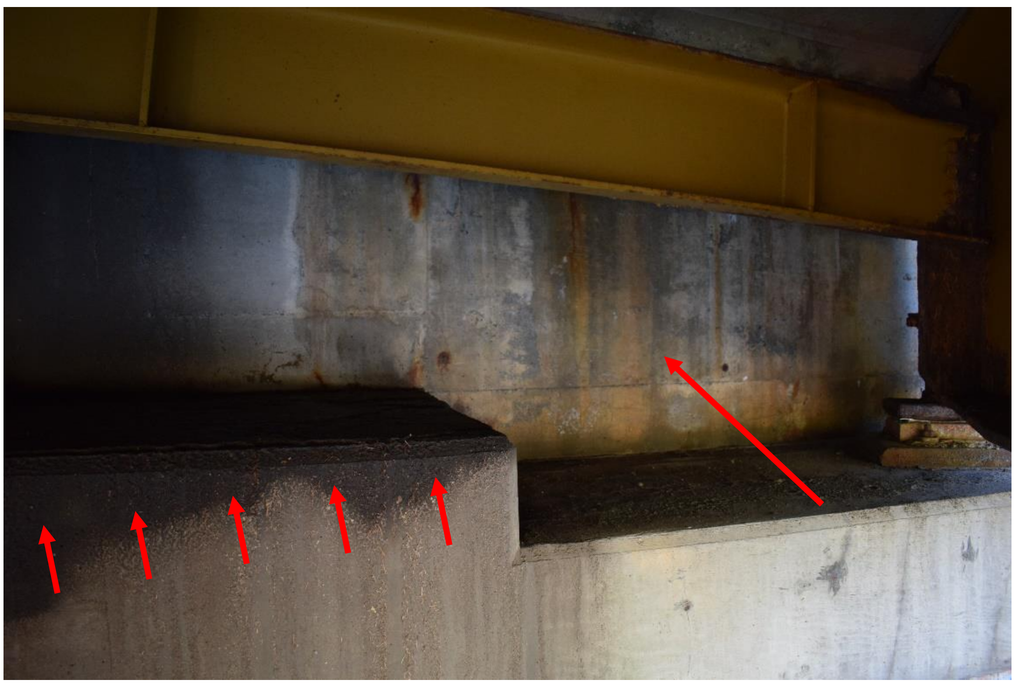


Exhibit 3.4-5: Water Stain Between Abutment Bearings S1 and S2

Exhibit 3.4-6: Water Stain Between Abutment Bearings N2 and N3

Exhibit 3.4-7: Water Stain Between Abutment Bearings N3 and N4

3.4.3 Structural Steel Condition

- Coating loss and corrosion were observed on all 5 girders along the interface between the deck soffit and the top flange (Exhibit 3.4-8).
- On girder 4, minor section loss was observed in a localized area of the top flange between the north piers and north abutment (Exhibit 3.4-9). The extent of the section loss could not be determined due to access limitations.
- Coating discolouration was present on the soffit of the bottom flanges of all girders, suggesting corrosion may be occurring under the coating. Coating failure and section loss were not observed (Exhibit 3.4-10).
- The coating of the pier legs was in fair to good condition, with a small number of localized patches of discolouration and coating loss. Section loss was not observed (Exhibit 3.4-10).

Exhibit 3.4-8: Between Girders 4 and 5 Looking South, Coating Loss Along Top Flange

Exhibit 3.4-9: Girder 4 Top Flange from North Abutment, Localized Flaking/Section Loss

Exhibit 3.4-10: North Abutment Pier Legs, Fair to Good Condition

3.4.4 Deck Condition

- Spalls were observed in the deck soffit concrete in several locations, exposing transverse reinforcement. The soffit between girders 3, 4 and 5 had localized spalls where corroded rebar was observed.

- Efflorescence was frequently observed on the deck soffit, generally concentrated at cold joints (Exhibit 3.4-11)

- A gap was observed at the joint between the south approach asphalt and the deck joint armouring. The gap was largest in the southbound lane. (Exhibit 3.4-12).

- Surface repairs had previously been completed on approximately 30% of the deck. However, asphalt has been used to fill in some potholes which is not considered a long-term solution. Some potholes had not been filled (Exhibit 3.4-13).

- Following a detailed survey (see APPENDIX A: Westmount Deck Delamination Survey), it was determined that approximately 34% of the deck was currently delaminated. Many of these delaminations border areas of previous surface repairs, which are not included in the 34% total.

- Deck soffit efflorescence suggests that some of the identified deck delaminations may extend through the full depth of the slab.

- The pedestrian railing does not meet the Ministry height requirement for bicycle traffic.

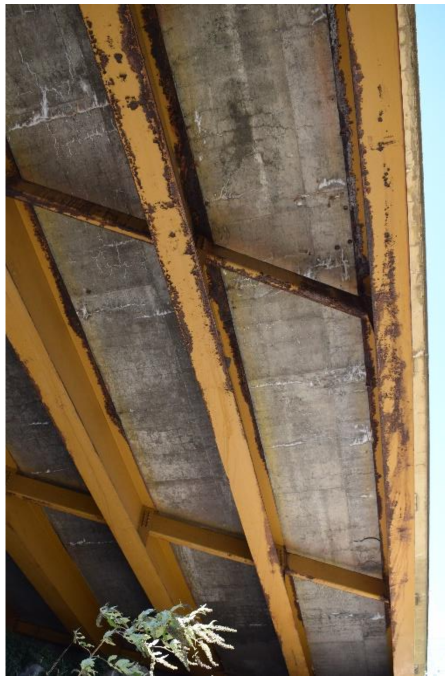


Exhibit 3.4-11: Efflorescence on Deck Soffit

Exhibit 3.4-12: Asphalt Deterioration at South Abutment Joint

Exhibit 3.4-13: North End of Deck Surface with Repairs Visible

4 RECOMMENDED REHABILITATION

This section outlines suggested rehabilitation items that have been identified through the condition assessments. These are broadly divided into two categories:

1. Seismic Performance Items – Rehabilitation items which affect the seismic performance of the structure.

2. General Rehabilitation Items – Rehabilitation items which do not affect the seismic performance of the structure but were identified by the inspection.

4.1 Horseshoe Bay

Seismic Performance:

- Remove delaminated concrete from the top of all inclined pier bearing plinths to determine the extent of delamination. Re-instate concrete. [Perform as part of upcoming seismic retrofits, 1-2 years]

- Perform localized coating repairs of abutment and inclined pier bearings. [Perform as part of upcoming seismic retrofits, 1-2 years]

- Remove accumulation of fill around east inclined pier bearing plinths. [Perform as part of upcoming seismic retrofits, 1-2 years]

General Rehabilitation:

- Perform localized coating repairs of structural steelwork. [5-10 years]

- Add cyclist dismount signs. [<1 year]

4.2 Eagleridge Underpass

Seismic Performance:

- Replace all abutment bearings with elastomeric bearings. [Perform as part of upcoming seismic retrofits, 1-2 years]

- Perform localized coating repairs on inclined pier bearings. [Perform as part of upcoming seismic retrofits, 1-2 years]

General Rehabilitation:

- Perform complete deck condition investigation including non-destructive deck evaluation (chain drag) to determine extent of delaminations. [Include in next BC MoTI inspection, 1-2 years]

- Perform localized coating repairs of structural steelwork. [5-10 years]
- Perform localized concrete delamination repairs on north abutment bearing shelf. [5-10 years]
- Annually remove loose concrete from deck soffit. Alternatively, install netting to catch falling debris.

4.3 Caulfeild Underpass

Seismic Performance:

- Replace all abutment bearings with elastomeric bearings. [Perform as part of upcoming seismic retrofits, 1-2 years]
- Perform localized coating repairs of inclined pier bearings. [Perform as part of upcoming seismic retrofits, 1-2 years]

General Rehabilitation:

- Perform complete deck condition investigation including non-destructive deck evaluation (chain drag) to determine extent of delaminations. [Include in next BC MoTI inspection, 1-2 years]
- Perform localized coating repairs of structural steelwork. [5-10 years]
- Replace deck joint seals at north and south abutments. [1-2 years]
- Repair approach asphalt adjacent to the north deck joint. Replace armouring at this location. [3-5 years]
- Perform localized concrete delamination repairs on front face of abutment bearing shelf. [5-10 years]
- Investigate source of leakage around utilities at the abutments. [Include in next BC MoTI inspection, 1-2 years]
- Annually remove loose concrete from deck soffit. Alternatively, install netting to catch falling debris.

4.4 Westmount Road

Seismic Performance:

- Replace all abutment bearings with elastomeric bearings. [Perform as part of upcoming seismic retrofits, 1-2 years]

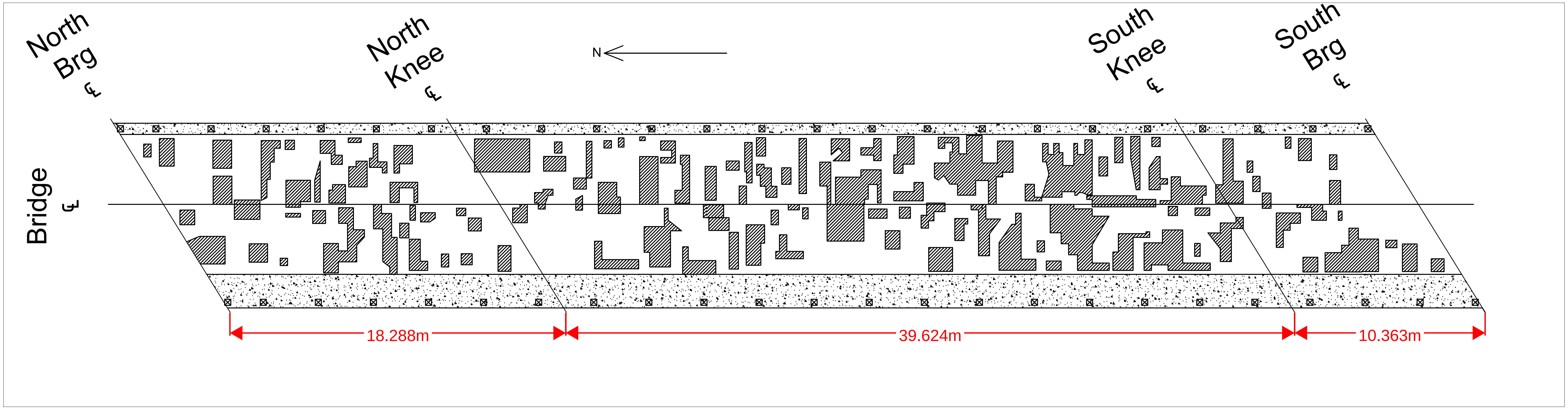
- Perform localized coating repairs of inclined pier bearings. [Perform as part of upcoming seismic retrofits, 1-2 years]

General Rehabilitation:

- Full deck resurfacing + full depth deck repairs as required [3-5 years].
- Perform localized coating repairs of structural steelwork. [5-10 years]
- Replace deck joint seals at north and south abutments. [1-2 years]
- Repair gap in approach asphalt adjacent to the south deck joint. [3-5 years]
- Annually remove loose concrete from deck soffit. Alternatively, install netting to catch falling debris.

APPENDIX A: WESTMOUNT DECK DELAMINATION SURVEY

Westmount Underpass: Deck Delaminations



North Span - Deck Delaminations

North End of Main Span - Deck Delaminations

South End of Main Span - Deck Delaminations

South Span - Deck Delaminations