Sid Buckwold Bridge Rehabilitation

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Stantec

INTRODUCTION

Senator Sid Buckwold Bridge or Idylwyld Bridge (the Bridge) Rehabilitation was one of the most complex capital bridge rehabilitation projects undertaken by the City of Saskatoon (the City). The complexity of the project mainly stemmed from the high volume of daily traffic, monumental importance of the structure and its surrounding, and the unknowns associated with the structural elements. As this bridge is one of the four primary crossings connecting the east and west of Saskatoon by crossing South Saskatchewan River, it was crucial to complete the project efficiently to reduce the negative economic impact.

As a part of the rehabilitation, the two ramp structures were also serviced. For the purpose of this paper, the focus will be on the main structure. The paper will briefly go into all stages of the project while going into greater details for some of the unique or complex challenges and/or methods utilized by the consultant.

BRIEF HISTORY

Idylwyld Bridge was built in 1965, in place of the old CNR Bridge that was built in 1890. The opening of this bridge kick started the commercial development of the west bank. Following an agreement signed between the City and Canadian National Railways (CNR), the existing railway truss bridge was demolished in 1962. Following this, the current Idylwyld Bridge along with Idylwyld Freeway was built to meet the demand created by the growing population. Eventually the surrounding area was heavily commercialized as part of the downtown development. Currently, Sid Buckwold Bridge stands as a show piece to what is currently known as the River Landing Development, one of the main attractions of the City.

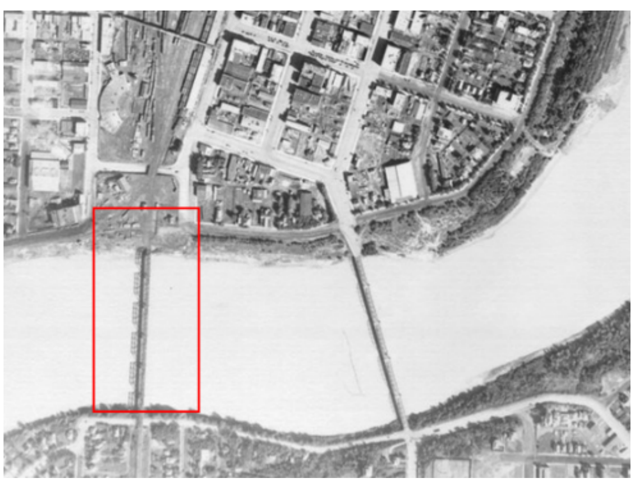

Figure 1 - 1927 Aerial View - 1890 CN Bridge (Tank, 2019)

Figure 2 - Present Aerial View (Google, 2021)

THE STRUCTURE

The main bridge structure, crossing South Saskatchewan River is connected to 2 ramp structures: Idylwyld Ramp and 1St Avenue Ramp. The bridge length is 183m long between expansion joints; however, the 13m long abutment vaults at both ends makes the total length of the bridge to be 209m. This 3-span bridge consists of 51.8m-79.2m-51.8m spans. The superstructure and substructure are made of reinforced concrete. The original design was in accordance with AASHO 1963 and CSA A135 -1962 (By Canadian Precast/Prestressed Concrete Institution, CPCI). The design vehicle used was HS-20.

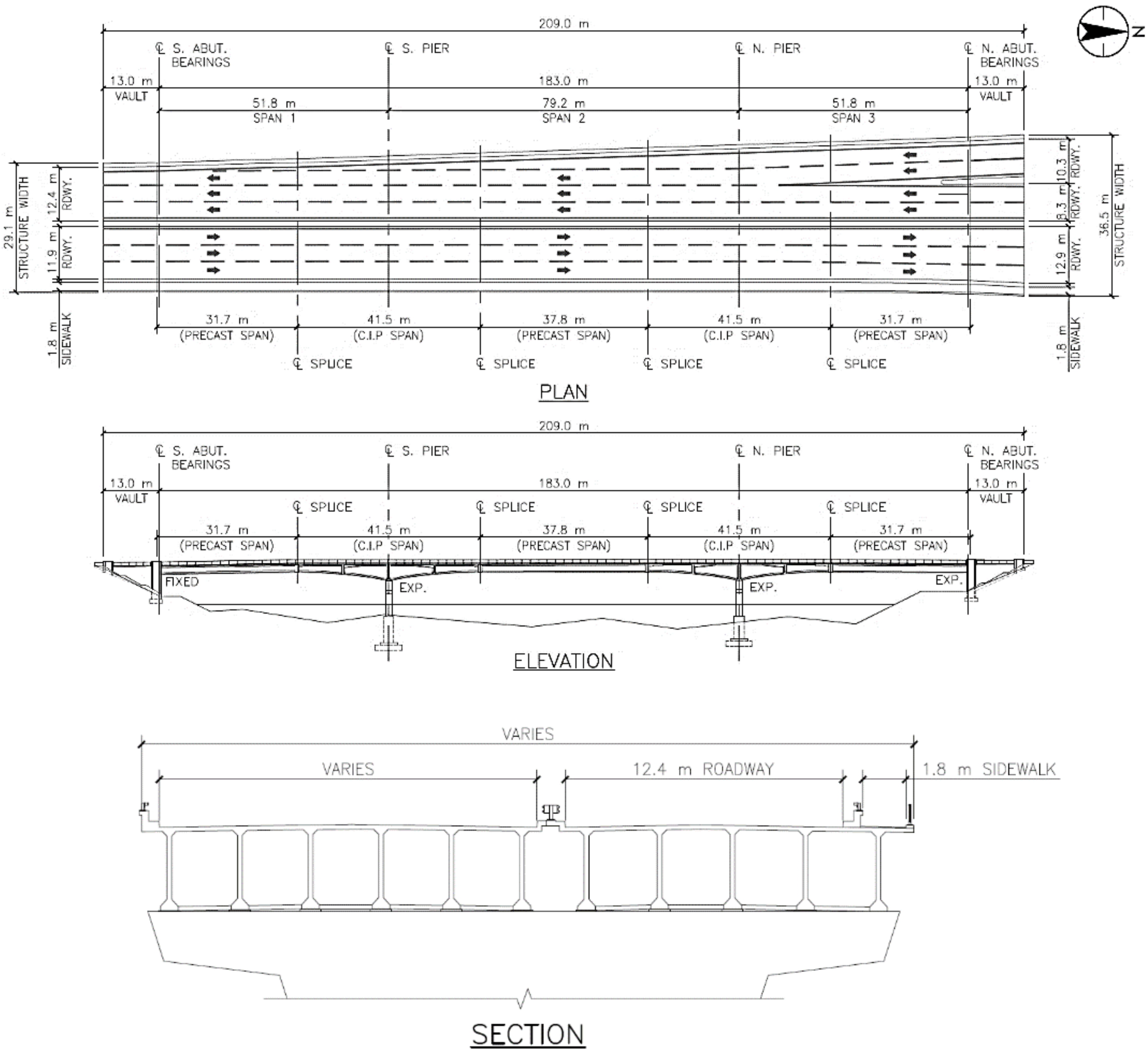

Figure 3 – Sid Buckwold Bridge

There are two separate superstructures, one for northbound and one for southbound traffic. The southbound structure is of varying width due to an on-ramp (19^{th} Street On-Ramp) at the north end, whereas the northbound structure has a consistent width with an attached sidewalk. The girders are post-tensioned with both precast and cast-in-place (CIP) sections and are continuous over the piers.

The two CIP spans consist of concrete T-girders that are integral with the deck. CIP girder sections have a bottom slab as it gets closer to the pier supports, making it a box section with a cavity. In addition, CIP sections are reinforced with post-tension cables and conventional black reinforcing steel.

As can be seen in Figure 3, the three precast T-shaped spans which are spliced to the CIP sections, are reinforced with conventional black steel, prestressed and post-tensioned cables. The spaces between these precast T-girders are infilled with CIP concrete slabs. The deck was 7” deep and at the time of construction, with a specified minimum clear cover of 1” (25mm). Strip-seal expansion joints are found at both ends of the bridge deck.

The piers are cast-in-place “hammerhead” style structures, supported on a spread footing that bears directly on a glacial till layer in the riverbed. The conventional abutments are supported on steel H-piles. The south pier bearings are fixed, and the rest are expansion pot style bearings.

The two ramp structures connected to the main bridge structure are approximately 200m long. Both ramps have integral T-beam superstructures. The substructure is made of reinforced concrete bell footings for the piers, and CIP concrete abutments supported on timber piles. The approaches consist of retaining walls that are connected to the wingwalls. Combined, both ramps have 7 strip seal expansion joints, with one shared joint at the south end. Both structures have a hot rubberized waterproofing membrane and asphalt wearing surface.

MAINTENANCE HISTORY

THE “NEED” FOR REHABILITATION

The structure has been in service for 50+ years. The City’s Inspection and Deck Testing Program had identified Sid Buckwold Bridge to require a major rehabilitation by 2018. Considering that this structure is one of the main river crossings into and out of the core, it was determined that combining the repairs to all structures would have the least detrimental effect on the downtown traffic demands.

During the 2014 OSIM Inspection, severe deterioration and delaminations were identified along the underside of the bridge deck and girders. In addition, scaling and cracks were noted on the piers and abutments. All 9 expansion joints showed signs of leakage through the strip seals. Moreover, due to height and access restrictions, complete inspection of the superstructure and piers was not conducted during the regular inspection cycles. Therefore, as a part of the consultant’s scope of work for this service agreement, additional investigations were required.

THE CLIENT’S NEEDS AND RECOMMENDATIONS

During the kickoff meeting, the following noteworthy items were discussed to ensure the design and the rehabilitation strategies met their expectations:

- Prioritization of public disruptions, traffic accommodation strategy, and budget.
- With the opening of the new Circle Drive South Bridge, Idylwyld Drive is no longer considered a major truck route; therefore, there was no need for these bridges to service vehicles heavier than the Saskatchewan Ministry of Highways non-permit (NP) vehicles or legal loads.
- The city did not want to consider increasing the deck width.
- For life cycle cost analysis purposes, the City requested the bridge to be considered “irreplaceable” and assume ongoing future repairs will continually restore protection systems resulting in an infinite service life. The City requested that the proposed deck rehabilitation should increase the service life by minimum 30 years without requiring any major repairs.
- The bridge was home to a very large pigeon population for a long period. Therefore, pigeon feces clean-up and bird exclusions at the piers and abutments were to be included in the design.

PRE-DESIGN & DESIGN

To kick off the preliminary design phase, a number of investigations were selected to refine and confirm the scope of the rehabilitation. The selection of the additional investigations stemmed from the extensive desktop study conducted to review the existing bridge and rehabilitation plans, previous deck testing reports, and OSIM inspections.

DESKTOP STUDY

Review of Original and Rehabilitation Drawings

Upon review of the construction drawings, the following key findings were made:

- The existing median and traffic barrier geometry and height were substandard. The height of the railing along the exterior edge of the sidewalk was substandard. In addition, the curb style barriers are not recommended by Alberta Transportation’s Road Design Guide. It is also deemed spatially inefficient considering its width.
- Lane geometry did not meet Transport Association of Canada (TAC) guidelines. Driving lane widths and shoulder widths were substandard for the 80km/hr design speed (or 70km/hr posted). The cross-slope did not meet the minimum recommended 2.0%. The Idylwyld Ramp has quite a considerable superelevation.
- There are bonded post-tensioning cables in both the CIP concrete and precast concrete girders. Some of the cables are draped and others are horizontal. The ducts closest to the top of the girder surface are in the CIP girder sections, above the piers. These ducts are horizontal and specified to be installed at the mid-point of the top flange resulting in a theoretical 56mm clear cover to the top of the duct. Due to the

concrete removal in previous rehabilitation and construction tolerances, the tops of the ducts were suspected to be within 20mm to 50mm of the top of the girder surface.

- The top mat of reinforcing within the CIP concrete sections consists of 19mm or 16mm transverse bars spaced at 152mm in the upper layer, and 13mm diameter longitudinal bars spaced at 457mm in the lower layer. The transverse bars are the main reinforcing steel for transferring forces within the bridge deck.
- An unsealed longitudinal joint separates the northbound and southbound structures. This joint was potentially a cause for the deterioration of underlying structural components.
- The existing strip seal expansion joints have been in service for 32 years.
- There is a utility gallery/walkway below the median consist of structural steel supports attached to the concrete girders and open grating. A steel structure also supports a 600mm diameter sanitary force main.
- The bridge has undergone major rehabilitation in 1986 and 1995. Key repair items noted were the removal of 11mm of deck concrete (6mm in 1986 + 5mm in 1995), replacement of unsealed finger joints with strip seals, and membrane and wearing surface replacement.

- In addition to the aforementioned, minor rehabilitation work was completed in 2004, 2007, and 2008 to complete spot repairs and to replace the wearing surface.
- The ramp structures had undergone major rehabs in 1986, 1994 and 2015. The repairs included deck concrete repairs, deck drain modifications, bearing replacements, expansion joint gland replacement, and replacement of asphalt wearing surface.

Previous Testing and Inspection Reports

There were 2 previous deck testing reports, conducted in 2006 and 2011, and an OSIM Inspection from 2014. From the deck testing reports, the following items were highlighted:

- The average deck clear cover measured was 10mm in 2006 and 21mm in 2011.
- 2006 deck testing results were compared to previous results in 2001, and it indicated that the deck deterioration had progressed, as expected. However, when comparing 2011 to 2006, the deterioration had regressed. As there were no concrete repairs completed, it was determined that this unexpected regression was a result of testing different lanes of the deck.
- Comparison of 2006 chloride content suggested that at the time the contamination was significantly increasing. In 2011, it was found that the contamination was generally low. However, increasing results were observed in localized areas, suggesting that the membrane had failed locally.

- Both 2006 and 2011 chain drag tests indicated debonding of asphalt, which further confirmed the localized membrane failures.
- In 2006, half-cell testing was not conducted, therefore, was unable to compare the results with the 2011 testing. In 2011, the data indicated a 90% probability of active corrosion.

Below key items were identified upon the review of the 2014 OSIM Inspection:

- Abutments had isolated delaminations and medium to wide cracks propagating from access hatches.
- Barriers, median and curbs had isolated delaminations. The median railing had deformed W-Beam sections and posts, very severe corrosion on posts, and missing connections.
- Girder ends had isolated delaminations. Soffit elements had staining and light scaling ($46m^2$ quantified in "Poor" rating).
- Wearing Surface had light to severe cracks, potholes, and ravelling.
- Expansion joints had leaking glands and abraded concrete end dams.
- Abutment Bearings had light to medium corrosion with less than 5 de-bonded stainless steel sliding plates.

FIELD INVESTIGATIONS

Upon the aforementioned reviews of the existing data, the following investigations were conducted to further determine the extent of the repairs needed and to evaluate the bridge element capacities.

Detailed Visual Inspection

A thorough OSIM level including a special access inspection was conducted to reach the areas that were inaccessible during the routine inspections. In addition, this was bundled with deck testing and PT cable evaluation to inspect underneath the wearing surface. Some of the key findings are summarized below:

- A manlift secured to a barge was utilized to inspect the soffit, girders, girder cavities, diaphragms and bearings that were unreachable by land or the utility walkway. It was found that the soffit had severe delimitation and spalls throughout. Particularly, components surrounding the median longitudinal joint were severely deteriorated.
- The concrete surfaces within the box section of the girders, or girder cavities, over the piers, were not inspected due to a substantial build-up of pigeon feces and active pigeon nests. Therefore, to properly inspect these areas, the pigeon flock had to be removed and droppings had to be cleaned. However, since the box sections were formed using 2x4 planks, the inspector was able to count the marks left by the planks to estimate the volume of feces in these cavities without having to enter these areas. The volume of feces per girder cavity was estimated at approximately ~$25m^3$ for a total of ~$900m^3$ in all 36 girder cavities.

Figure 4 - Girder Cavities

- Isolated light pattern cracking with associated minor depressions in precast concrete girder sections (bottom-up defects) were noted on the deck surface. Delaminations and spalls were observed within the test pits created for the post-tensioned cable evaluation. Deterioration appeared more prevalent in the area 1.8m from the curb and a lack of clear cover (5mm to 15mm) appears to be contributing to the deterioration. The condition of the soffit and deck-top within the test pits suggests isolated partial depth repairs will be required to approximately 10% to 20% of the area.

- Isolated delaminations were noted on precast concrete girders in the thickened areas for the post-tensioning anchors.

- Expansion joints were in poor condition. Non-uniform gaps at abutment 2 suggested damages to internal components. End dams had severe abrasion damages, the joint anchorage was exposed and had detached from the deck. Seals were leaking and deteriorated areas were found in the abutment below.

- Hairline to wide diagonal shear cracks propagating from shafts towards outer bearings of the concrete piers.
- At the abutments, light to very severe delaminations and spalls with surface deposits, and corrosion on reinforcing were noted along ballast wall to bearing seat construction joint (interior face within vault). Wide cracks and medium to severe delaminations of bearing pedestals were observed, which undermined the masonry plates.

Deck Testing

To confirm the previous deck testing data and progression of deterioration, a deck test was completed. The timing would also match the current sequence the City had used for the deck testing program for the bridge. However, this deck testing was expanded to include the bridge deck, sidewalk, abutments, and girder ends. The following were included within the test program:

- Delamination test in accordance with ASTM D4580-03 (limited to the soffit, girders, and abutments)

- Half-Cell (CSE) Test in accordance with ASTM C876-91

- The expansion joints were used as ground points as they are connected to the deck reinforcing, which was confirmed by minimal resistance readings.
- Continuity between the deck surface and deck reinforcement can be affected by a functioning membrane. The effect a membrane has on this test was confirmed by taking a reading through the membrane and asphalt and comparing it to a reading on the bare concrete deck at the same location. This additional test was completed at 4 locations to determine the effect the membrane had on the test

- Rapid Chloride Test in accordance with AASHTO T-260
  - A total of 42 locations were tested, including the bridge deck (24), sidewalk (6), girders (4), and abutments (8).
- Rebar Depth Testing (Clear-Cover Measurements) using GPR
- Carbonation Test using phenolphthalein solution on the freshly cored concrete at 2 locations

Below is a summary of the key findings:

- Deck soffit in precast sections indicated that ~7% (~158m²) of the area is delaminated while CIP indicated ~1% (22m²) of the area was delaminating.
- The average chloride concentration at the deck rebar depth was measured at 0.191. Whereas the concentration at the girder rebar depths was recorded at 0.016. Tests conducted within the abutment also showed that rebar was chloride contaminated, with an average concentration of 0.124.
- 16.7% of the deck area was marked as >95% probability of corrosion when considering the chloride testing results.
- Half-cell testings suggested that 32.8% of the deck area had a >90% probability of corrosion, with an average voltage reading of -349mV.
- The average asphalt thickness was measured at 91mm and the average clear cover was measured at 19mm.

Post-Tensioning Evaluation

Due to the proximity of PT cables to the deck surface and the indication of chloride-induced corrosion of deck reinforcing, samples were tested at the PT cable ducts to confirm the corrosion status of the PT cables. Test locations were carefully selected to reduce the disruption to traffic and keep the testing as least invasive as possible while considering the representability of the data to the entire bridge deck. Therefore, the selection process considered the severity of chloride exposures and the severity of the members (i.e., primary versus secondary). The inspection areas were in the CIP concrete portions, near the piers, where the ducts are nearest to the deck surface. The cables consist of bundled steel 7mm diameter wires and ≈75mm diameter corrugated steel ducts filled with grout.

Below is a summary of the key findings:

- The membrane was well-adhered to the concrete deck and appeared to be providing some level of protection; 92mm to 95mm of concrete cover existed above the PT cable ducts.
- No cracking or deterioration of the concrete was observed over the PT cables. The ducts and grout had no observed material defects and the ducts were fully grouted.
- The impact echo testing did not identify areas with suspected voided or soft grout; the high ratio of girder thickness to PT duct diameter made void detection difficult.
- The bundled cables/wires had light superficial corrosion.
- Low concentrations of chlorides were identified in the grout, which was most likely to be attributed to background chlorides.
- No carbonation was identified on the grout (pH >11). The petrographic analysis determined the grout to be of high quality and in good condition.

Based on the findings from test data, no repairs were warranted.

Figure 5 - Exposing Post-Tension Cable Duct

Underwater Investigation & Bathymetric Survey

The piers are supported on spread footings, bearing directly on the riverbed. These are backfilled with approximately 3.2m of granular material and riprap for protection. Therefore, an underwater investigation was conducted to determine any sign of scour or protection failures. It should be noted that this crossing marks the narrowest point of the South Saskatchewan River from Medicine Hat, Alberta to Codette Lake, Saskatchewan; thus, comparably higher flow velocity and scour was to be expected. In addition to the scour investigation, the survey was also utilized to identify any concrete defects underwater.

A local marine investigation company, Inland Marine Technologies, conducted the bathymetric survey. A combination of Kongsberg-Mesotech software (MS1000) and Sonar Heads (1171 Hi-res Multi-Frequency) was used to gather bridge pier and riverbed imagery. To obtain accurate results, 32 images were taken by repositioning the workboat 8 times per pier. The software was able to properly scale and add an as-built overlay for comparison purposes. Distance measurements were based on a pre-calculated speed of sound through water for the conditions at the time of the survey. All relevant parameters such as temperature, depth, salinity, and turbidity were factored in for this analysis. It should be noted that larger anomalies can be measured at a reasonable level of accuracy, whereas the minor erosion or smaller imperfections in concrete (<1cm) are merely "educated guesses" based on other similar-measured anomalies and viewer's experience.

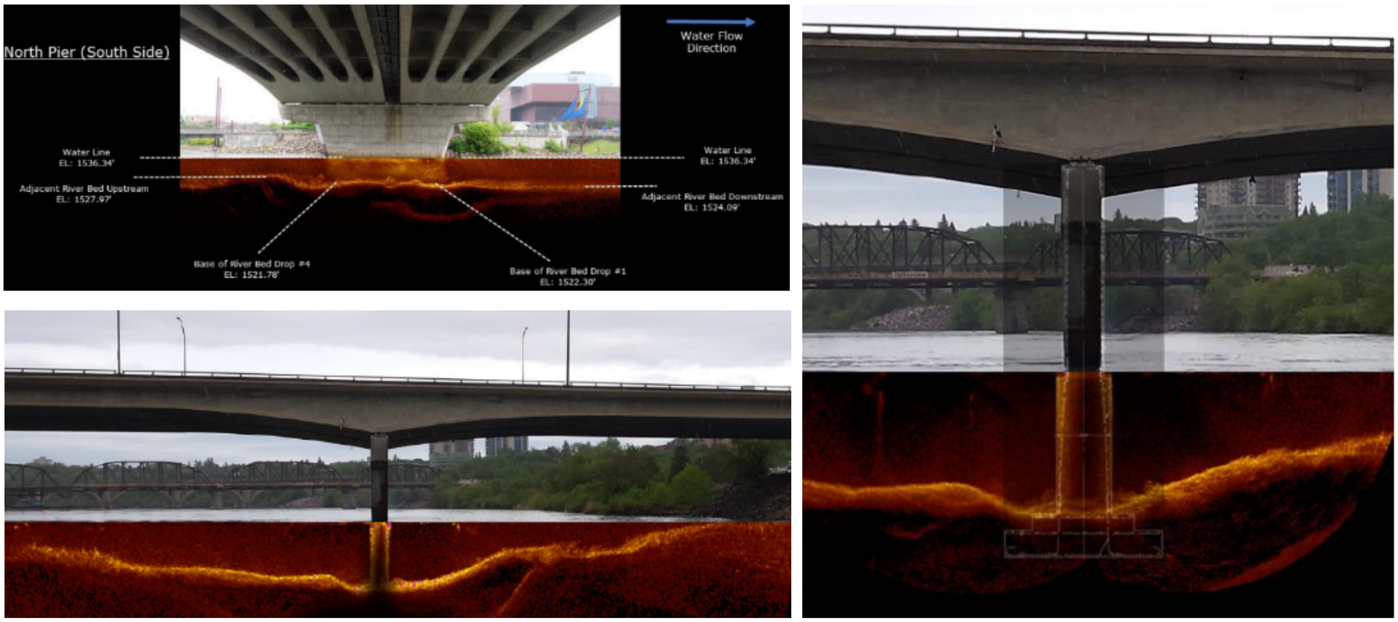

Figure 6 - Imagery From Bathometric Survey

Key findings from the bathymetric survey were as follows:

- No suspected concrete deterioration of piers below water level.
- No suspected movement of the pier footings.
- Substantial loss of rip rap around piers in comparison to as-built drawings.
- Scour was found at the upstream ends of both footings to a depth of ~1.2 metres, leaving approximately 2.0m of streambed material above the bottom of the footings.
- No undermining of footing was observed at the time of the investigation.
- Remains of an abandoned bridge pier (1890 CNR Bridge) have daylighted near the south side of the north pier. This appears to be contributing to the scour.

Stantec recommended restoring the lost riprap at the pier footings but this was not selected for the scope of this rehabilitation by the City. It was recommended to the City to continue underwater assessments at reasonable time intervals to ensure the footings are continuing to perform as intended. It should be noted that currently there are no Canadian specifications for such investigations, though there are a vast number of major structures that are supported on shallow foundations. For this investigation, Federal Highway Administration (FHWA) specifications were utilized as a guide.

Traffic Accommodation and User Impact Study

This crossing is considered to be one of the busiest in the city. A comprehensive study was conducted to analyze the potential traffic accommodation strategies and their impact. Seven different options were evaluated using a qualitative analysis using five assessment criteria:

- Construction cost: Impact on the construction cost for selected phasing and obstructions to construction work by the particular strategy. (Rating 35 out of 100)
- Road user impact: Criterion was based on how much an option would inconvenience the everyday road user due to increased travel delay caused by slower driving speeds, less capacity/fewer driving lanes provided, or needing to take a longer route. (Rating 30 out of 100)
- Impact on southbound underpass: Rating was based on how long the option negatively impacted the operation of the underpass ramp connecting 19th Street to southbound Idylwyld Drive. (Rating 20 out of 100)
- Schedule: Rated the estimated construction time for each option, depended on mobilization, familiarization with the project, and the ability to overlap different tasks of the rehabilitation. (Rating 10 out of 100)
- Traffic accommodation cost: This considers the direct cost of traffic accommodation equipment, crossover construction costs, barriers, traffic signals, maintenance costs among others. (Rating 5 out of 10)

This evaluation was proven to be effective during construction with minimal impact on the traffic system, especially during peak hours. It should be also noted that the City provided and maintained the traffic accommodation setup throughout. The Contractor was only to move and adjust the signs within the construction zone as required.

LOAD RATING (PRIOR TO THE LOAD TESTING)

In accordance with Table 1.1 of CHBDC, for an AADT of 37,900 (main structure only), this section of the roadway is classified as a Class A highway. It is also considered to be a primary weight highway route by the Ministry of Highways (MoH). However, the City had requested not to consider this crossing as a major truck route. The original HS-20 design vehicle has three axles and a Gross Vehicle Weight (GVW) of 32.2 tonnes.

The following elements were then evaluated for vertical shear and longitudinal moments:

- Cast-in-place box girders
- Precast concrete girders
- Concrete deck
- Abutment vault roof T-beams
- Pier caps

The finite analysis software package, CSiBridge (by Computer & Structures Inc.) was used to model the entire structure. Due to the separation of the superstructure and substantial difference in deck geometry, 2 separate models were created for the east and west structures. The associated dead loads were derived from record drawings and CHBDC. For the live load, MoH's BE-100 – Bridge Evaluation Guideline, was used and the non-permit truck NP-8 (the heaviest) was used. NP-8 is nearly double the GVW of the original HS-20 design truck. NP-8 truck is shown in the below figure 7.

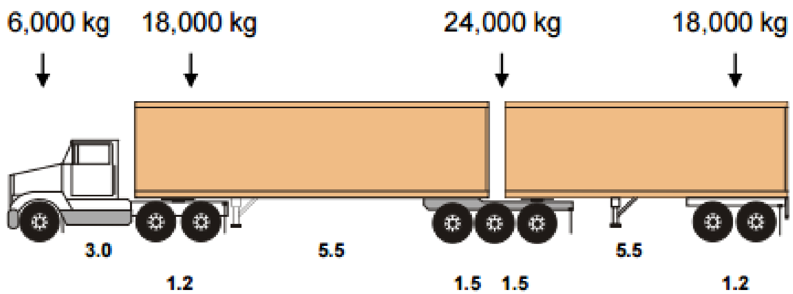

Figure 7 - NP-8 Truck (Ministry of Highways, 2018)

Target Reliability Index (β) was calculated for each element on each structure. This index was based on the system behaviour, the element behaviour, and the inspection level for each element in accordance with CHBDC. Then β was used to determining the dead and live load factors for the analysis. The index varied from 2.75 (deck shear and moment) to 3.50 (Abutment Roof T-Beam for Shear).

The capacity at ULS of each element was assessed by calculating the live load capacity factor (F-factors) at critical locations of section change or regions of high stress in accordance with CHBDC. It was found that the girders near the pier in the east structure had an F-factor of 0.52 and the west structure had a factor of 0.6. This was highly concerning as it was well below the serviceable rating of 1. Pier caps, however, were found to have an F value above 5 for both east and west structures. This did not match what was observed during the detailed visual inspections. Girders near the piers did not show any signs of shear cracks, while the piers showed a considerable amount of shear cracking. Hence the recommendations were made to do strengthen both the piers and girder within the predesign report.

PIGEONS – THE CHALLENGES

Having the girder cavities open since the original construction in 1965, the pigeon population has been growing to approximately 2,000 birds. The cavities seemed to have accumulated the most droppings due to their confining geometry. It was imperative to clean the structure and install bird exclusions. This large of a pigeon flock has the potential to spread diseases, produce negative effects on the aesthetics of the structure and its surrounding, and specifically in this case – a substantial increase in the dead load acting on the structure. In addition, the droppings were known to have uric acid, which has been known to cause damage to the underlying concrete.

However, it was not a case of simply cleaning and installing exclusions, as the flock would find new habitat in the surrounding structures. The construction of Remai Modern, an Avant-garde museum building and a theatre, and Traffic Bridge was just completed. Therefore, the only option was to reduce or eliminate the flock to ensure that the flock does not move their habitat to these iconic structures and the surrounding dense residential areas.

Unfortunately, this was not a common issue within Canada and had little to no usable local data or information. Eventually, the designer was able to track down another Stantec associate in California who had experience with such issues. The studies of pigeons indicated that the flock cannot feasibly be relocated, as they have a 500km+ range for tracking back to their home. Therefore, to reduce the flock, the only option was to trap and euthanize the birds and then install the exclusions. It was confirmed that pigeons are considered a pest upon consulting a conservation officer. Therefore, humane euthanizing techniques to control them were acceptable.

PREDESIGN RECOMMENDATIONS

After the completion of the analysis done during predesign stage, the following key recommendations were made:

- Deck was determined to be a critical element of the deck rehabilitation; hence, removal of 25mm and the addition of 65mm to increase the rebar cover along with the completion of partial and full-depth isolated deck repairs was recommended. Conventional waterproofing and galvanic protection were chosen to protect the deck.

- Girder and Pier strengthening, where the option comparison considered the cost of construction, aesthetics, and constructability:

    - Two practical options, Fibre-reinforced Polymer (FRP), and external post-tensioned stirrups with FRP strips on exterior girders were considered. The FRP option was found to be the most effective as it was comparably less expensive and was not visible as the strips. This was due to the fact that the FRP would be installed inside the cavities between the girders adjacent to the piers. The preliminary estimated cost for the FRP system was $1.6M.

    - For the pier strengthening, the two options considered were FRP and a CIP concrete overbuild. The FRP option was chosen as the cost was comparatively lower while the aesthetics were also comparable. Bundling the FRP repairs with the girder repairs resulted in a saving of $60k over performing the repairs separately.

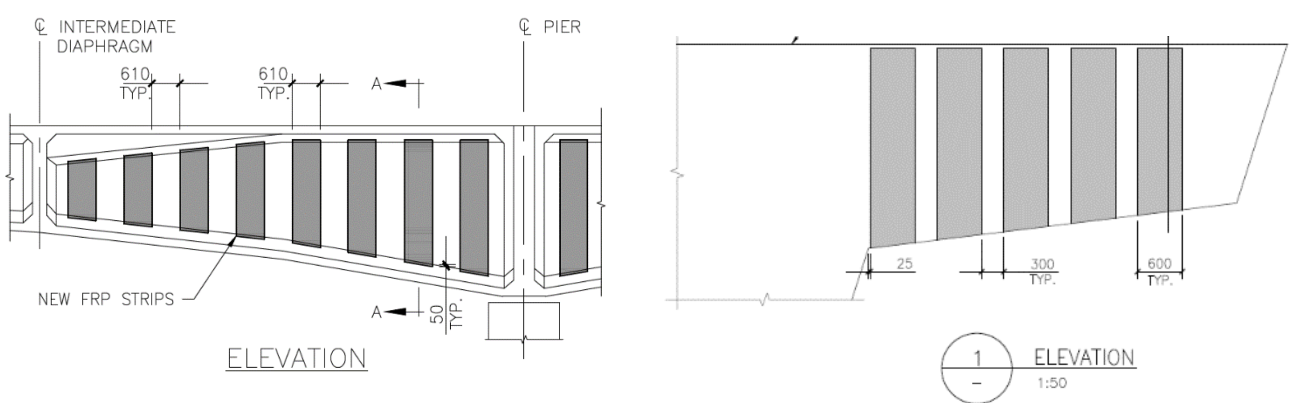

Figure 8 - Proposed FRP Strengthening Options

- All existing barriers were to be replaced due to their condition, performance, and sufficiency. This included both traffic and pedestrian barrier systems for both main and ramp structures. Approximately 2km of barriers were to be replaced.

- With the barrier replacement, an existing overhead sign structure attached to the existing barriers was to be replaced.

- All nine expansion joints (including the ramp structures) were to be replaced with cover-plated V-seal deck joints. Evaluations were done to determine the possibility of eliminating 2 joints of the main structure by converting it to an integral system. However, due to the difficulty in construction, cost, and schedule constraints this option was eliminated. In addition, an unsealed joint with a drainage system was also considered but eliminated due to space restrictions.

- The 12 existing deck drains were to be removed and 28 new deck drains were to be installed while extending the drainpipe to ensure the runoff is directed away from any structural components. The drainpipe systems at each abutment were to be replaced as well.

- Bearings were found to be in fairly serviceable conditions and did not warrant any replacements. Though, the abutments bearing were to be cleaned, blasted, and repainted to extend its service life.

- Concrete spot repairs were also warranted for isolated areas at the abutments, piers, girders, and pedestrian stairs at each end. Areas covered in pigeon feces at the time were to be reevaluated once the structure was cleaned. During this stage, it was difficult to determine the extent of the bearing pedestal deterioration, therefore, it was recommended to request separate pricing for jacking the abutment if required to complete the repairs during the construction phase.

- The abutment concrete repairs posed a unique challenge due to the presence of structural components and pigeon exclusions, and it was not desirable to complete future patchwork after the rehabilitation as the City was to apply permanent artwork to the walls. Abutment concrete was sound, but chloride contaminated; hence the probability of chloride-induced corrosion was considerably high. Therefore, with the help of Vector Corrosion Technologies, a cathodic protection system was recommended.
    - Initially, the system considered utilizing Galvashield CC anodes to match the corrosion needs. However, to service 30 years or more would require a substantially large number of anodes. Instead, with the help of specialists at Vector Corrosion, Galvashield Fusion T2 was recommended. These anodes are designed to provide short-term Impressed Current Cathodic Protection (ICCP) to re-passivate the steel and provide long-term galvanic protection. This would increase the service life of the abutment by 40 years or more.

- It was recommended to trap and euthanize the pigeon flock. The girder cavities to be then cleaned of pigeon feces and permanent pigeon exclusions to be installed to prevent future habitations. Approximately 800m^3 of pigeon feces from girder cavities and other structural members were estimated to be removed.

CHANGES AND CHALLENGES DURING THE DESIGN PHASE

The transition from predesign recommendation to detailed design was fairly straightforward for most items. This section focuses mainly on the challenges and some unique approaches that were utilized during the design phase.

Deck Rehabilitation

It was quite evident that the longitudinal joint between the two structures was causing substantial deterioration to the underlying structural components. Initially, the designers looked at ways to eliminate the joint by connecting the superstructures longitudinally. Joining the two separate structures would result in connecting two different shaped decks together, which have separate thermal expansion ranges, especially in the transverse direction. Therefore, it would result in a complex design that would require a change to the bearings to allow multidirectional rotation and translations. This did not align with the City's budget expectations, hence was not pursued.

Barriers & Deck Drains

The existing barrier system with curbs was deemed inefficient for the space used in comparison to the level of protection provided. The existing median barriers had an overall width of 1.219m, with the longitudinal separation joint on one side. This was changed to a thinner (0.615m) and taller barrier system which moved the longitudinal joint to be in between the barriers. East and west barriers were also designed to decrease the overall width by ~38%. Therefore, an additional 630mm width was added to the driving surface for the southbound lanes. For the northbound lanes, the space-saving was distributed to both driving lanes (+302mm) and the pedestrian walkway (+328mm). Along the bottom of the median barriers, a drip edge was placed to better direct the water away from the superstructure components.

All but two existing deck drains were designed to be removed and replaced with 28 new deck drains. It was chosen to keep the two as is since there were near-surface post-tension cables running underneath these two drains. This would eliminate the risk of damage during demolition.

Strengthening and Load Testing

The inspection reports and findings were revisited as the strengthening design began. This reopened the discussions about the lack of shear cracking at the girder while the live load capacity was nearly a half. Typically, for shear deficiencies, the cracks would be a prominent indicator. Moreover, this 53-year-old structure utilized unique components compared to a typical beam bridge. As mentioned earlier, spans near the piers were cast in place with the shape, both height and width, are parabolically thickening towards the supports. In addition, the slabs at the girder bottoms closer to the piers would also help distribute the load. Therefore, the modelling of CHBDC live load distribution factors to the actual distribution was questioned. Therefore, a load test was proposed to the City to confirm the results or potentially eliminate the need for girder strengthening.

Stantec then contracted Bridge Diagnostics Inc. (BDI) to instrument all 5 girders on the east structure with Electrical-resistance Strain Gauges (ESGs). The sensors were used to monitor girder strains when a vehicle of known geometry and weight travelled along the bridge. The sensors were installed at three locations along the bridge. It should also be mentioned that the lead designer was previously employed with a similar testing company. Therefore, he had a great understanding of the process and the equipment. He was able to give specific recommendations to properly measure the resistance values at the most critical locations. Little adjustments were made by the crew due to accessibility issues. The first instrumented section was located 12ft (3.7m) from the north abutment. The second instrumented section was located 42ft-6in (13m) from the north abutment with the third section located in the critical shear region 17ft (5.2m) on the north side of the north pier. The first two sections had ESGs on the underside of the girders as well as at the top of the web immediately below the flange taper. A 3-axle gravel truck was used as the test vehicle.

As the west-most girder of the east structure is the critical girder, it was used for the analysis. Upon completion of the test, the theoretical live load distribution was compared with the real-time measured distribution. A comparison sample is shown in Figure 9. As such, actual measured Live Load Distribution Factors (LLDFs) were used in the calculations instead of the CHBDC factors. During predesign analysis using CHBDC factors, the LLDF was determined at 0.919, whereas the test results found that the actual distribution was only 0.366. Therefore, the designer was able to reduce the load acting on this exterior girder by close to 60%. As such, the F factor was revised from 0.52 to 1.306. This suggested that no shear strengthening work was required for the girders, eventually saving ~$1.6M of strengthening cost.

In addition, since the FRP system was not required for the girders, it was decided to go ahead with CIP concrete overbuild for the piers. To complete FRP strengthening only for the piers would increase the cost to be above the estimate for the CIP overbuild. The CIP option was approximately $20K less than the FRP option when it's not bundled with the girder strengthening work. In addition, CIP overbuild could be done by most competent contractors with minimal supervision.

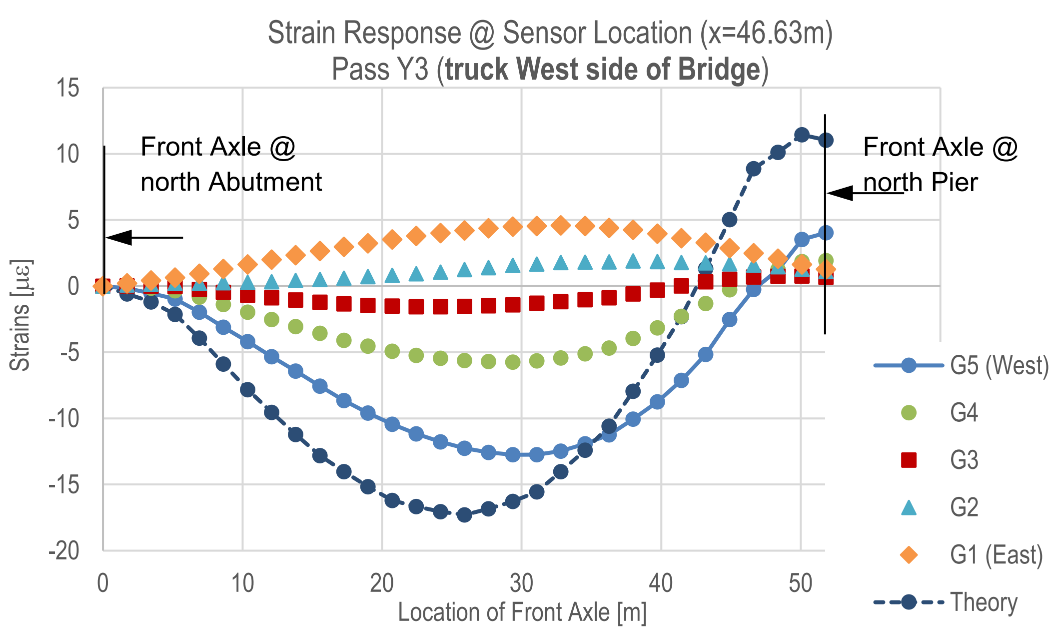

Figure 9 - Strain Response Measured vs Theoretical

Separate Pricing for Bid Items

The City's primary goal was to rehabilitate the main structure, while the scope for ramp structures was budget-dependent. This approach was economical while being focused on the main goal of the project. In addition, the inclusion of corrosion-resistant reinforcing steel was also tendered as a separate pricing item. Therefore, due to the unknowns associated with the scope of construction, some unique challenges had to be addressed.

With the introduction of 5 separate pricing bid packages, it was quite difficult to determine the dependencies to elements when accepting, for example, none to all 5 items. The biggest challenges were associated with the barrier transitions and associated elements, and staging plans. A number of elements were connected to the barriers including but not limited to light fixtures, deck drains, expansion joints and an overhead sign structure. For the staging plan, traffic accommodation strategies would substantially change if a ramp was to be open for traffic versus closed for construction. Therefore, the initial proposed approach was to create 3 separate packages to address the primary changes and adjust as required during the post-tender and preconstruction period. However, due to budgetary constraints, this approach was not chosen as it would have a considerable increase in the engineering fees. Instead, one design package was created with the intention of modifying post-award with tri-party discussions. Some of the challenges faced with this approach are further discussed within the next section, Construction – Challenges.

CONSTRUCTION – CHALLENGES

The final value of the construction was over $18M. The construction began in April 2019 and was completed by September 2020, with a significantly reduced construction scope through the winter months due to weather restrictions. During this 2-year construction period, numerous challenges were faced due to a variety of reasons. Some of the key challenges are discussed within this section.

Record Drawings and Field Conditions

This is a common issue with most bridge rehabilitations, especially of similar aged structures. Fortunately, for this structure, the key structural components remained unchanged or comparably similar to what was shown on the record drawings. However, numerous differences were found throughout the components of the bridge that had financial effects. The differences included missing rebar or changes to rebar configuration, drainage outlets and pipes, abutment details, barriers details, girder thicknesses of ramp structures and electrical plan. In addition, the surrounding area has undergone major changes since the bridge was constructed. Stairs connecting River Landing and Rotary Park were quite different from what was shown on the record drawings. Most of these details were not available as they have been misplaced over the years. Therefore, challenges were faced with respect to tie-ins, dimensions, demolition, verification of existing components and their serviceability. Continuous and transparent communication between all parties and effective management of contractor and engineering resources played a major role in resolving these issues to meet the expectations.

Post-Tension Cables and Reinforcing Steel

During demolition and deck milling, a few post-tensioning ducts were damaged and/or found to be severely deteriorated. The timing of this was particularly concerning due to its implication on the progress at the time. The first step to resolve this issue was to properly locate these post-tension cables, which was done using Ground Penetrating Radar (GPR). Once mapped, the severity of the cables was evaluated. It was found that the cables were only used for construction purposes and did not have structural importance. Therefore, a decision was made to keep them as is.

As with most rehabilitations, the newly constructed elements rely heavily on the existing reinforcing steel to tie into the existing structure that is to remain in place. During the predesign, it was clear that the construction would need to take into consideration the extremely low clear cover. While this was expected, it was found that at the expansion joints and barriers on the main structure and the ramps, some of the primary reinforcements were either missing or not placed as per the record drawings. In addition, certain reinforcements were epoxy coated, which would affect the lap lengths. As the construction progressed and similar issues surfaced, continuous engineering analysis was needed to determine the severity of these variations and whether additional rebar was needed.

Pigeons and the Public

Even though the pigeons are classified as pests, the euthanizing process attracted the public's attention in particular through social media. Numerous professionals with different opinions weighed in on the euthanization process, potential health issues from pigeon droppings, and animal rights considerations. Some had suggested using birth control to control the flock instead of euthanizing the flock, while some suggested hiring falcon handlers. A few petitions were also put in place by animal rights activists to stop the euthanization process. After months of discussions and media presence, the decision was finalized to continue with euthanizing of the flock as it was the only option considering the flock size and its leftovers.

Overhead Sign Foundation and Watermain

With the barriers being replaced and the existing overhead sign support had its foundation incorporated into the existing barriers, the contractor was required to design and construct the new foundation for the new overhead sign structure. The initial design consisted of a deep pile foundation but was deemed risky due to underlying utilities.

In the tender package, the City had identified a 300mm diameter water main in the vicinity of the Idylwyld Ramp structure, which was built in 1907. However, the city had no reliable data pertaining to the exact location of the water main. The only information the City was able to provide was that for this particular era of construction, the typical cover used was 2.5m to 3.0m for such cast-iron pipes. However, since 1907, the surrounding area had undergone major changes along with a major bridge replacement. Therefore, it was impossible to accurately determine the depth and actual clearance of the pipe with the historic records. Therefore, the contractor attempted to physically locate the pipe by using hydro-vac excavation. Upon excavating approximately to a depth of 12ft, a rock layer was encountered. The contactor believed that this may be a layer protecting the water main, therefore, seized the hydro-vac operations. These findings then lead to the foundation being redesigned to have a shallow footing foundation. After few iterations, the final design consisted of T-shaped footing, with a jersey barrier traffic protection system for the pile cap.

OUTCOME

Even though there were numerous challenges throughout the rehabilitation process, the project was completed on time and within the allocated budget. The City, Stantec and the contractor worked cohesively to ensure the project was successful, especially during the midst of the COVID-19 Pandemic surfaced during the second construction season. Special recognition must be made to the contractor's staff who took extensive measures to ensure the health of all site personnel.

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