

**Discussion of a Selection of Construction Issues Associated with the
Waverley Underpass and other Manitoba Infrastructure**

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Abstract

The Waverley Street Underpass project is one of the latest major capital projects undertaken by the City of Winnipeg Public Works Department. The project entailed the design and construction of a new Waverley Street underpass crossing of the Canadian National Railway (CNR) Rivers rail line (Rivers 3.89 Subdivision) amongst many other associated items of work such as roadway, active transportation, and other improvement works. Throughout the construction of the bridge structure in particular, a number of technically challenging or otherwise intriguing situations arose; particularly, unforeseen issues associated with the bridge structure's caisson foundation construction as well as issues generally seen around Manitoba, Canada with respect to the use of hot-dip galvanized steel reinforcing were encountered. While these items are not necessarily novel to the profession, there is benefit to be had from additional discussion.

This paper provides a brief overview of the Waverley Street Underpass project with the purpose of providing context for the caisson issue discussion. Additionally, material issues related to hydrogen embrittlement of hot-dip galvanized rebar and hydrogen evolution of hot-dip galvanized rebar in plastic concrete are discussed. The impacts each of these issues had on the project and suggestions for mitigating similar issues on future projects are also presented.

1.0 Introduction

The Waverley Street Underpass project is one of the latest major capital projects undertaken by the City of Winnipeg Public Works Department. The project entailed the design and construction of a new Waverley Street underpass crossing of the Canadian National Railway (CNR) Rivers rail line (Rivers 3.89 Subdivision) amongst many other associated items of work such as roadway, active transportation, and other improvement works. Throughout the construction of the bridge structure in particular, a number of technically challenging or otherwise intriguing situations arose; particularly, unforeseen issues associated with the bridge structure's caisson foundation as well as issues generally seen around Manitoba, Canada with respect to the use of galvanized steel reinforcing (hydrogen embrittlement and hydrogen evolution in plastic concrete) were encountered. While these items are not necessarily novel to the profession, there is benefit to be had from additional discussion. Generally, additional measures could have been taken that may have eliminated or mitigated the issues encountered.

While the concerns with the galvanized reinforcing steel are material concerns in nature and independent of the specific project, the caisson issues were very much tied into the overall project. Thus, a brief project background is provided in the following section to provide the necessary context required in order to appreciate the challenges encountered.

2.0 Project Background

Due to rapid growth in the southwest quadrant of Winnipeg, Manitoba, coupled with increasing traffic congestion due to heavy train operations, the Waverley Street crossing of the CNR Rivers rail line has become a high priority in recent years.

Waverley Street crosses two mainline railway tracks at Mile 3.89 of the CNR Rivers Subdivision. Traffic movements over the previously existing at-grade crossing were controlled by grade crossing warning signals with gates. There are two track cross-over's present immediately east of the crossing with train movements at this location controlled by a centralized traffic control (CTC) railway signal system governing when, on which track, and at what speed trains pass through the crossing. At present, there are approximately 35 to 40 train movements on any given day which contributes to significant delays to the approximately 30,000 vehicle movements through the crossing on Waverley Street daily.

In order to address the growing concerns and congestion in the southwest quadrant of the City due to the at-grade rail crossing, the City of Winnipeg initiated a preliminary design study for the Waverley Street Underpass Project in early 2014. The project team, led by Dillon Consulting Limited as the lead proponent, was tasked with developing a high-level design for the grade separation of Waverley Street and the CNR Rivers rail line amongst many other associated items of work. A map of the overall project area is shown in **Figure 1**.

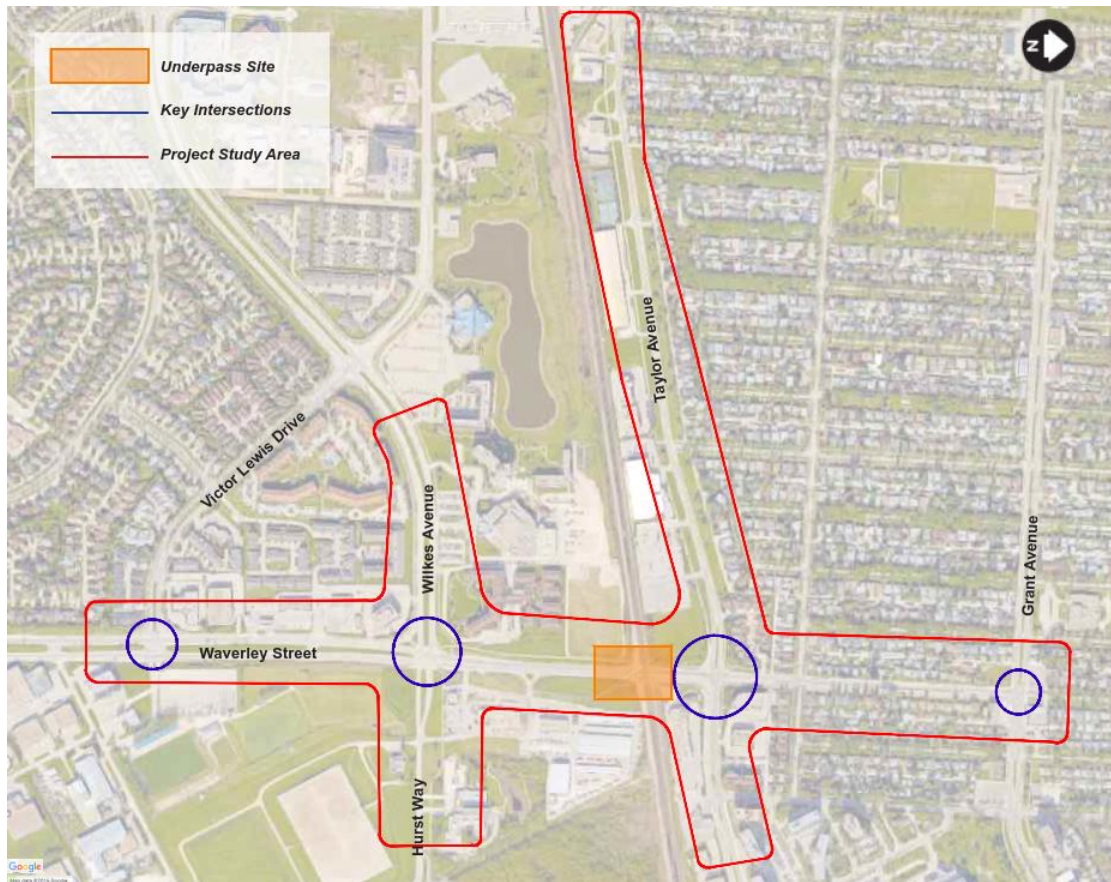


Figure 1: Map of Project Study Area

In 2015, the Federal Government and the Province of Manitoba committed a combined total of \$92M of the approximated \$155M (2015 Dollars) expected cost of the proposed underpass works developed as part of the preliminary design study. With significant third-party funding in hand, the City subsequently fast-tracked the project although it had not officially been on its approved Capital budget. The detailed design was completed in 2017 and construction commenced shortly and is scheduled to be substantially complete by October, 2019.

3.0 Preliminary Design Phase

Two significant items considered closely during the preliminary design phase were the determination of the preferred bridge superstructure and substructure types. Each of these are discussed in further detail herein.

3.1 Superstructure

Three superstructure options were considered for the underpass structure. These options were twin thru-plate girders, precast prestressed concrete box girders, and steel I-beams encased in concrete with all three options consisting of four simply-supported spans. For a variety of reasons

not discussed here, the steel I-beams encased in concrete were ultimately recommended as the preferred superstructure type during the preliminary phase.

3.2 Substructure

A geotechnical investigation and assessment consisting of three phases was completed by the team's geotechnical subconsultant. The three geotechnical investigation phases were set-up to examine various aspects of the overall project including the bridge structure foundations, railway embankment and railway detour, roadway considerations and potential retaining walls. The bridge structure foundations in particular were considered in Phase I of the investigation in which three deep test holes (total) were drilled at both ends of the bridge. The test holes were completed using 125 mm diameter solid stem augers and HQ wireline for rock coring.

The three deep test holes were advanced past the till roughly 6 m into the underlying limestone bedrock which forms an artesian aquifer. The bedrock formation is a Paleozoic Carbonate rock formation known as the Upper Carbonate Aquifer. The depth to bedrock surface was about 18 m below existing grade or approximately at elevation 215.5 m. A layer of hard clay (shale) infill was encountered within the bedrock at an approximate elevation of 212 m or 21.5 m below existing grade. The hard clay infill zone was 0.3 m thick at 3.5 m below bedrock surface in one test hole and 0.8 m thick at 2.6 m below bedrock surface in another. The top 5 m of the bedrock formation was observed as highly decomposed and based on the calculated RQD values for the recovered rock cores, the rock quality was very poor to fair. Low RQD values were generally calculated over the entire length of rock cores indicating very poor to poor rock quality. Uniaxial compressive strength tests completed on three samples of rock cores recovered indicated compressive strengths of 114, 121 and 194 MPa, respectively. The recovered rock cores and associated RQD values from one of the deep test holes are presented in **Figure 2**.



Rock Core (Run)	Material	Run Depth below Ground level (m)	RQD (%)
C1	TILL	13.9-15.2	NA
C2	TILL	15.2-16.8	NA
C3a	TILL	16.8-18	NA
C3B	LIMESTONE	18-18.3	0
C4	LIMESTONE	18.3-19.1	23
C5	LIMESTONE	19.1-20.6	22
C6	LIMESTONE	20.6-21.7	0
C7	LIMESTONE	21.7-22.6	23
C8	LIMESTONE	22.6-24.2	60
C9	LIMESTONE	24.2-25.8	26

Figure 2 – Rock Cores and Associated RQDs for Deep Test Hole TH14-04

Both cast-in-place belled caissons and cast-in-place rock socketed caissons were investigated as potential pier foundation options. Generally, belled caissons were expected to present constructability challenges and were deemed likely unfeasible for the project. As such, they will not be discussed further. On the other hand, drilled caissons socketed into sound bedrock

appeared feasible and have a history of use on similar rail underpasses within the City. Local practice is to design the drilled shafts based on values of maximum allowable end bearing and/or shaft adhesion of 3.0 MPa and 1.0 MPa, respectively, provided that downhole inspection and assessment of the rock competency is undertaken. The assessment of the rock competency consists of small diameter proof drilling to 2 m below the socket base to detect the presence of voids or clay/silt layers of any significance and determine if deeper socket boring is required. In the event that the socket cannot be visually inspected, inspection of the recovered rock core and/or downhole video monitoring can confirm the competency of the bedrock. In this situation, caissons founded in sound bedrock should be designed on the basis of a reduced allowable shaft adhesion of 0.60 MPa with no contribution from end bearing. It's noted that safety concerns related to man entry into the boring (e.g., high level of gas) generally precludes undertaking the visual inspection.

Based on the finding of the three test holes advanced into the bedrock the top 5 m of the bedrock is dominated by very poor to poor quality rock as previously discussed. Socket length, at least at the location of these test holes, should be expected to be developed below elevation 211.0 m and measures to maintain socket wall stability and groundwater control should be anticipated. The geotechnical subconsultants advised that depth to sound bedrock should be expected to vary across the site and it should be recognized that the presence of the heavily fractured rock and infill material above the socket length may require that a permanent steel casing be left in the ground so that the integrity of the shaft is maintained. In this regard, the basis for measurement and payment for the rock socket installation should be established in the contract preparation stage to recognize that the bedrock conditions at some rock socket locations may require unanticipated extra effort and materials for their completion. The payment mechanisms ultimately utilized for the caisson construction are presented in Section 4.0.

Finally, if rock socketed caissons were to be advanced to the detailed design phase as the preferred pier foundation, a test caisson(s) was highly recommended by the geotechnical subconsultants to verify design assumptions, examine the feasibility of construction and assist in the selection of adequate equipment and proper construction practices.

4.0 Detailed Design Phase

Upon completion of the preliminary design phase, the same project team was retained to undertake the detailed design and to provide contract administration services during construction for the overall project. The following sections describe the detailed design efforts associated with the underpass superstructure and substructure designs. Additionally, payment mechanisms developed for construction of the caisson foundations are also presented for discussion.

4.1 Superstructure

The preferred superstructure type recommended in the preliminary design phase – steel I-beams encased in concrete – was developed further with the final superstructure cross-section as shown in **Figure 3**.

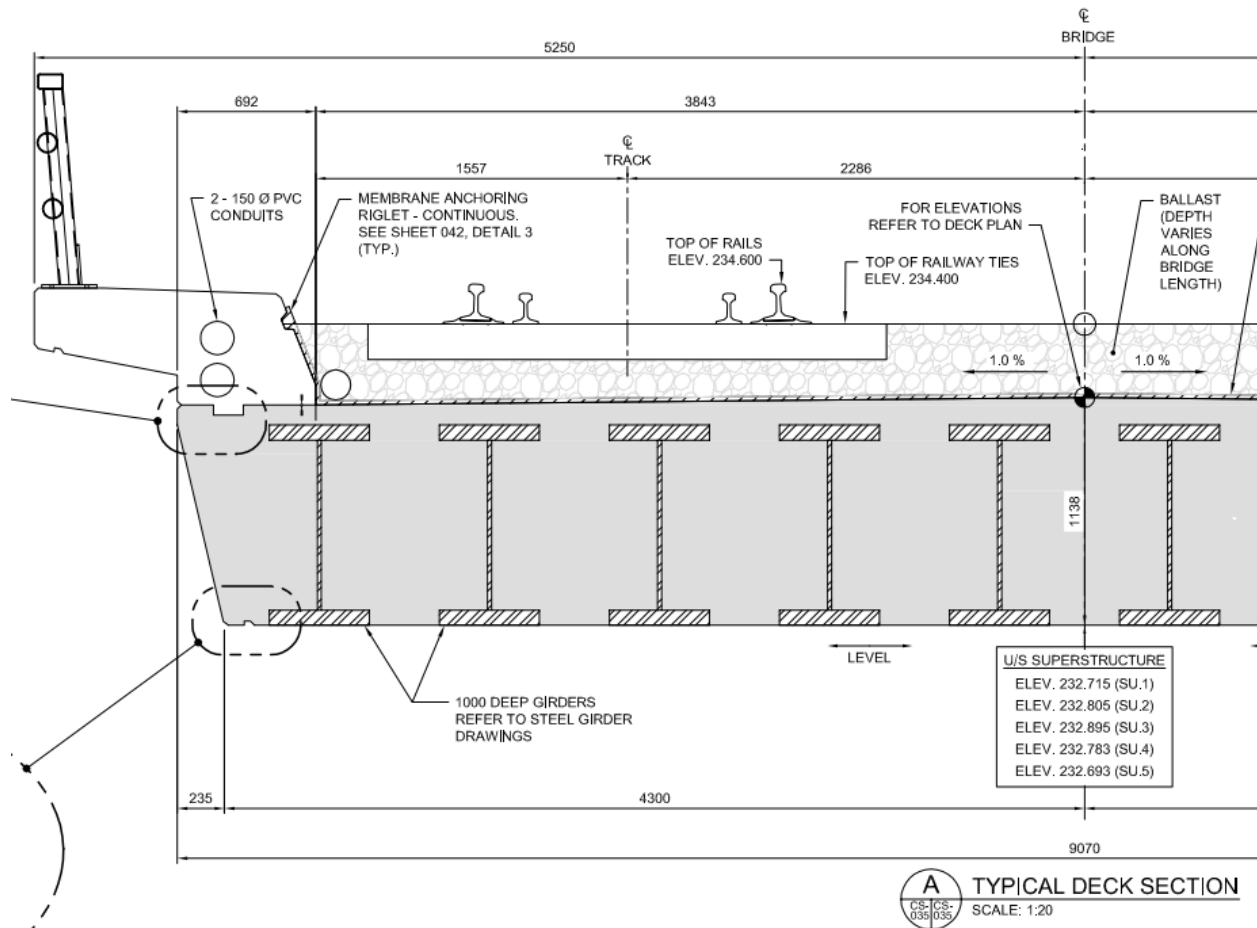


Figure 3: Final Superstructure Cross-Section (Symmetrical About Centreline)

For this particular superstructure design, only the simply-supported steel girders were considered for Ultimate Limit States design checks. In other words, the steel girders were designed to carry the entirety of the AREMA Cooper E90 locomotive live load and dead load. The cast-in-place infill slab that encases the steel spans was only considered for Service Limit States design checks. In other words, the concrete infill was provided solely for deflection control and to improve the aesthetics of the structure. The project specifications called for the steel girders to be either metallized or hot-dip galvanized. The concrete infill was also fairly heavily reinforced with hot-dip galvanized reinforcing steel.

4.2 Substructure

As per the geotechnical subconsultant's recommendations during the preliminary design phase, a test caisson was completed early in the detailed design phase. The test caisson was located to the southeast of the proposed bridge structure approximately 35m south of the proposed centerline. The test caisson was completed using a Soilmec R-516 HD piling rig equipped with a 1200/910 mm diameter flight auger and 810 mm core barrel.

The test caisson was advanced through the clay overburden and till layer with augers to practical refusal into the bedrock at a depth of 17.2 m below surface. The core barrel was then employed to core into the bedrock from 17.2 m to a termination depth of 30.2 m below ground surface. The caisson was sleeved with a 1.2 m diameter outer temporary casing. The temporary safety casing extended from ground surface to a depth of 5.0 m below surface. An inner (permanent) sleeve was inserted into the test caisson to support the walls of the test hole at deeper depths. The inner (permanent) sleeve was 36 inch (0.91 m) in diameter and extended into the bedrock to a depth of 21.0 m below ground surface. The rock socket below depth of 21.0 m was advanced without the use of a sleeve (permanent) or casing to support the side walls of the hole. After a video inspection, the caisson was backfilled with concrete/bentonite mixture and abandoned.

Findings and recommendations of this test caisson exercise were summarized as follows:

- A permanent sleeve from ground surface into the weathered/highly fractured bedrock will be required to maintain a stable excavation.
- Video inspection of the test caisson is recommended to confirm the quality of the rock socket. However, if pumping of groundwater to inspect the socket would tend to destabilize the excavation due to pumping of fine sand from the fractured zones, an alternate method to confirm the quality of the socket core should be utilized. This should be combined with maintaining an extra water head inside the inner casing and probing the base of the socket with a weighted steel probe bar after cleaning and immediately before tremie concrete placement.
- Tremie placement of concrete will be required due to the large amount of water seepage from the bedrock aquifer; and,
- The depth to the bedrock (poor to fair quality bedrock) is expected to vary across the site and it should be recognized that the test holes advanced at the proposed bridge location are more representative of expected ground conditions at that location.

With the recommendations of the preliminary design phase geotechnical assessment and findings of the test caisson in hand, the three piers were designed, each consisting of three 1.2m caissons complete with 1.07m diameter rock sockets framed into a reinforced concrete pier cap as shown in **Figure 4**. The caissons were expected to be approximately 21m long plus approximately 4.5-5m of socket. A partial elevation of the bridge structure is also shown in **Figure 5**.

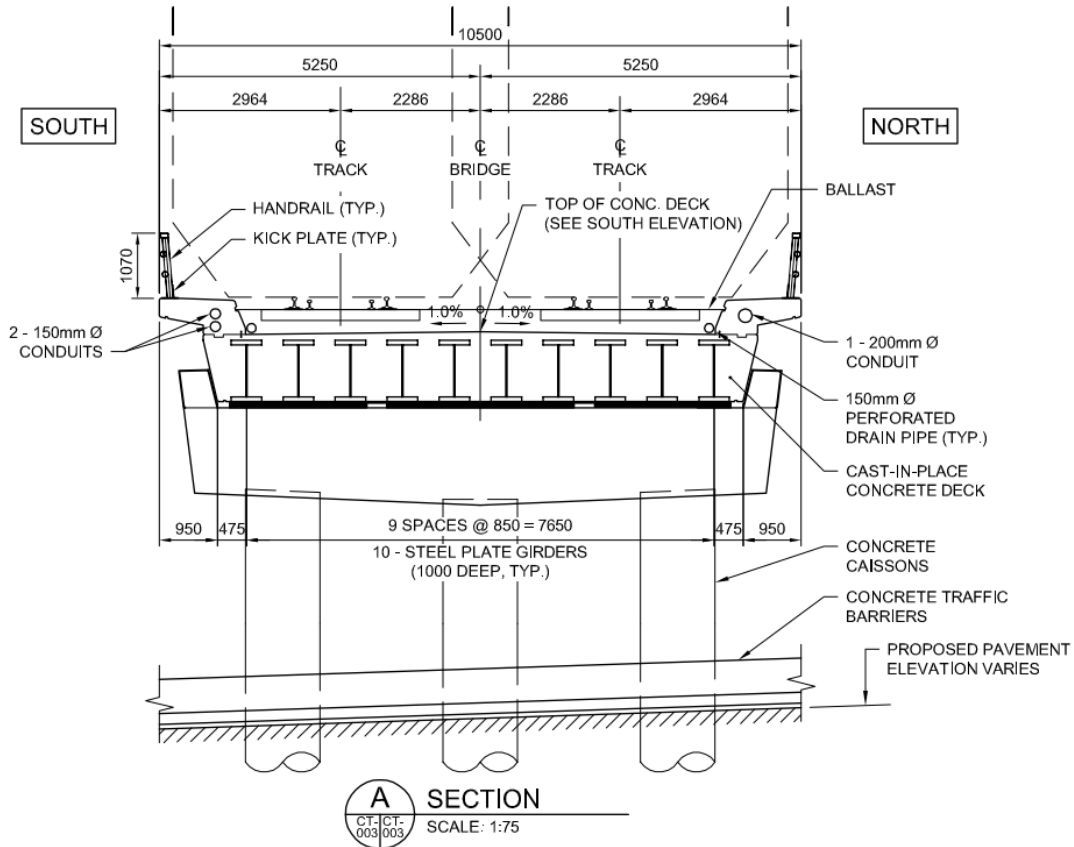


Figure 4: Final Bridge Cross-Section at Pier

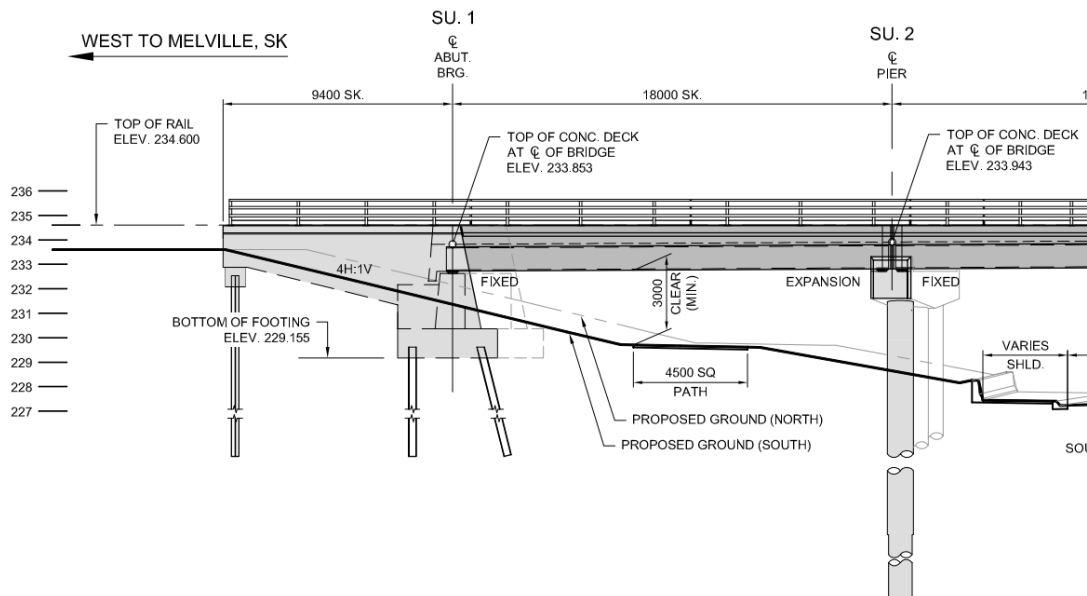


Figure 5: Final Bridge Partial Elevation

In light of the variable bedrock conditions likely to be present, the payment mechanisms for the caisson construction were carefully considered by the project team; the advertised tender document had the following provisional pay items with the following associated definitions:

Pay Items:

- Base tender Lump Sum Price on number of caissons shown on the Drawings including all labour and material required to install rock-socketed caissons to Elevations as per Contract Drawings;
- Provide unit price for additional length of rock-socket into sound bedrock, including coring, rock removal, reinforcing and concrete;
- Provide unit price for additional length of the steel casing only into weathered rock zone; and,
- Provide unit price for reduced length of rock socket into bedrock, including a reduction of coring, rock removal, reinforcing and concrete to be credited to the City.

Definitions:

- Weathered Rock Zone: Weathered rock encountered above the sound bedrock including voids and soil filled cavities which would require permanent steel casing to support the caisson hole; and,
- Sound Rock: Rock which may contain fractures but a casing is not required to support the caisson hole.

With these provisional pay items, the project team felt that the risks associated with the potential for variable sound bedrock elevations were mitigated. If the Contractor was required to extend the caisson due to the presence of a weathered rock zone above sound bedrock, a unit price per linear meter would already be established simplifying the overall change order process. Similarly, if for whatever reason the rock socket length into sound bedrock had to be extended, a unit price per linear meter would already be established. As is discussed in further detail in Section 5.1, the subsurface conditions for the partial construction of one pier were so exceptional that it fell outside the reasonable application of these provisional pay items leading to a financial conflict made worse by the critical path nature of the caisson construction.

5.0 Construction Phase

The overall project construction for the Waverley Street Underpass Project commenced in May 2017 with work beginning on the Waverley Street detour around the eventual location of the underpass. The rail shoofly was also constructed in the early phases of construction to accommodate excavation work and eventual bridge construction. With the Waverley Street detour and railway shoofly in place, the underpass structure was able to be constructed in generally greenfield conditions; the underpass structure was actually constructed at grade with the underpass being excavated after completion of the structure once rail traffic was returned from the shoofly to the original track alignment.

The construction of the rail bridge was scheduled for the 2018 construction season with the Critical Date for substantial completion of the bridge construction being August 17, 2018. The critical date was subsequently extended as a result of unforeseen subsurface conditions discussed in Section 4.1; the critical date was extended to September 4, 2018 and the works for that date were completed early on August 31, 2018. A detailed look at the construction works and issues encountered during construction of the bridge are provided in the subsequent sections.

5.1 Rock Socketed Caissons C4 and C5

While a test caisson had been installed during the detailed design phase to further determine the subsurface conditions in the area and to verify the constructability of the design – previously discussed in Section 4.2 – Manitoba, like many geographic areas, is known to have formations of bedrock that are not uniform. Typical soil stratigraphy in the Winnipeg area consists of silts and various clays in the near-surface strata with an underlying layer of lacustrine clay of ten or more meters before till and limestone bedrock are encountered. All caissons except caisson C4 and C5 (the southern two caissons at the central pier as shown in **Figure 6**) had final tip elevations of the rock-socket generally similar to the anticipated elevation of 210.0 m. Caisson C4 and C5, however, ended up being in the location of a karst system in which groundwater had eroded the limestone bedrock causing a collection of limestone fragments. As such, sound bedrock was not encountered as expected.

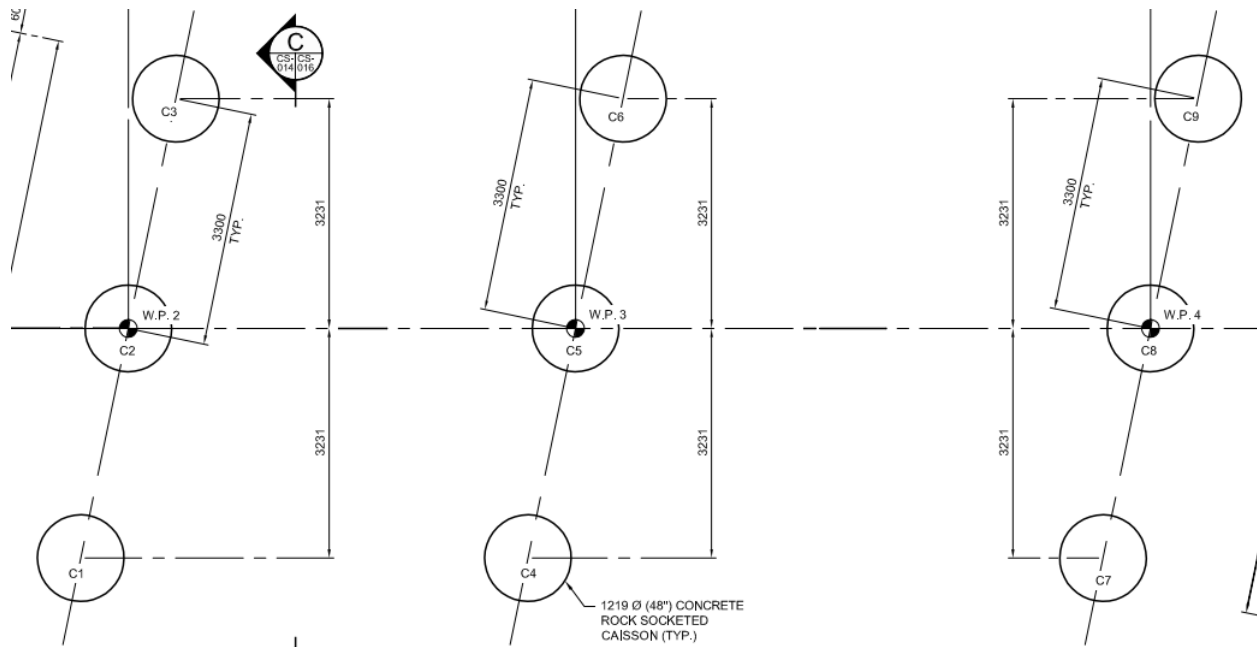


Figure 6: Plan of Caissons (Three Caissons per Pier) – Not to Scale

Due to as expected construction of caisson C6 at the same pier, less than 3.5m away, the team was hopeful that sound bedrock would be discovered shortly below the expected elevation. Thus, the foundation contractor continued to advance the 1.2 m working sleeve into what would be determined to be the karst system and the strata was verified periodically using a soup can to remove the bedrock sluff. This approach continued until the foundation contractor became concerned about the possibility of needing to advance significantly deeper associated with the risks of having an open excavation for an extended period of time coupled with a depth where maintaining a plumb caisson is extremely difficult.

Eventually the foundation contractor ceased work on caissons C4 and C5 as a result of their concerns noted above and concerns additionally noted by the Owner and Contract administrator. At this point, it appeared as though significant delays to the project were inevitable. The entire project team, including the contractors, met to determine a course of action to complete the caisson foundations and to find a way to address the financial concerns regarding the unanticipated caisson depth. The foundation contractor was concerned that the provisional pay items previously described in Section 4.2 did not address the special situation at hand and the contractor was reluctant to continue with any work until the finances could be reviewed which would further delay the project. After candid discussions, the foundation contractor agreed to proceed with work in good faith. It was decided that two test boreholes drilled with a 2-inch hollow stem auger would be carried out adjacent to the C4 and C5 caissons to verify the depth that competent bedrock would be encountered at each caisson. Once the elevation was determined, the course of action for installation would be determined and instruction provided.

The two test borehole investigations determined that the elevation needed for competent bedrock was close to the elevation that had been reached when works on site had ceased. The depth needed, however, necessitated redesigning the two caissons as the current 1.2m sleeve in the ground was not long enough to advance deeper to allow for the rock-socket to be installed. To

reach the necessary elevation the 1067mm sleeve was telescoped with an additional 1015mm sleeve to complete the rock socket. The rock socket diameter was reduced to 1016 mm (from 1067mm) and the length of the 1067mm sleeve was extended when competent bedrock was found at approximately 196.5m, 13.5m below the anticipated elevation. The redesigned caisson configuration is shown in **Figure 7**.

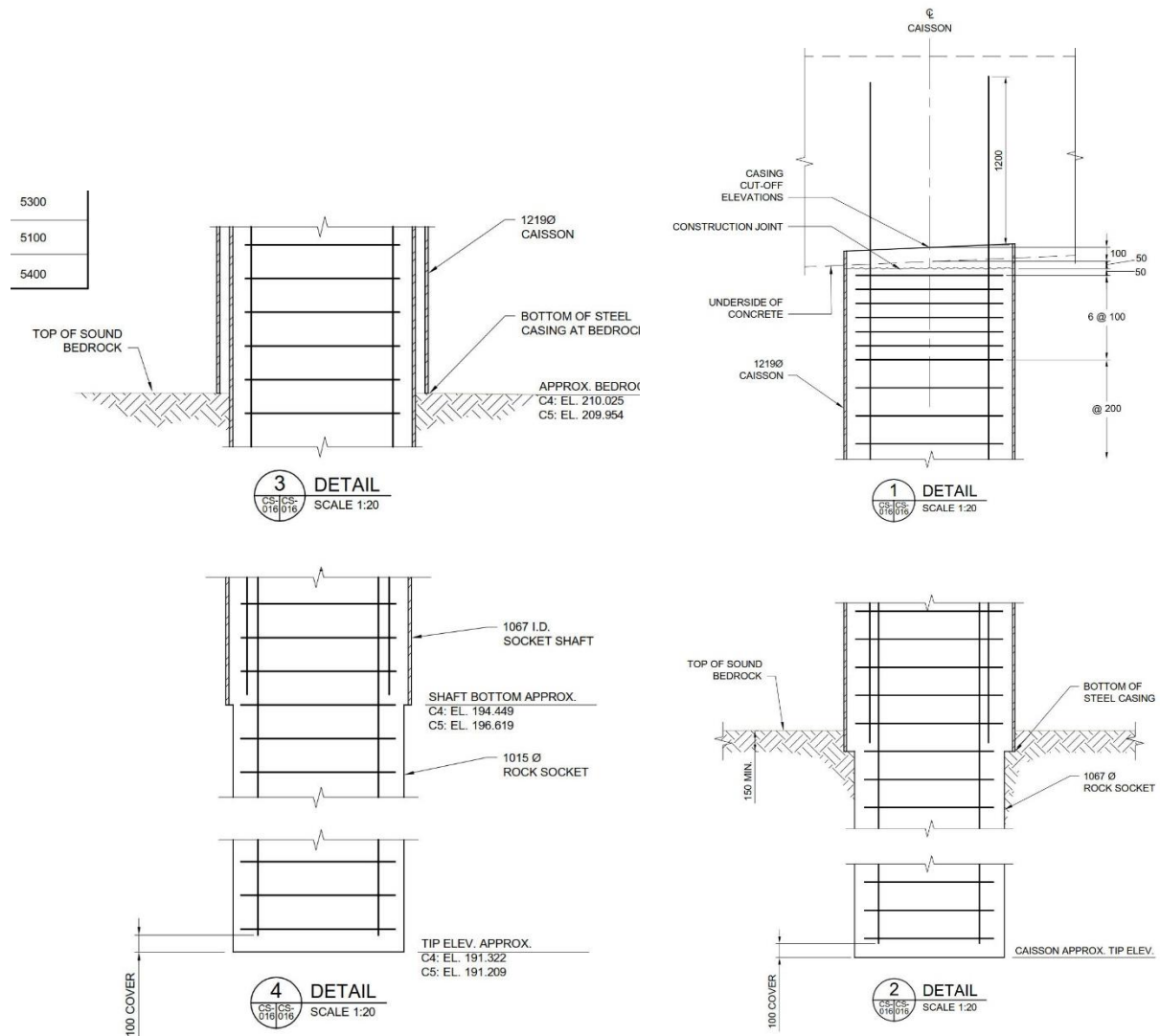


Figure 7: Caisson C4 and C5 Redesign (Left) and Initial Caisson Design (Right)

In order to avoid safety and stability concerns around caisson construction it is recommended that on future projects test boreholes, similar to those carried out once the problem was identified, are conducted ahead of construction at each caisson location so that a more certain estimate can be made with respect to the amount of effort. The designs can also be further optimized in terms of the lengths of various sleeves. It is expected that had the karst system been identified ahead of

construction the delays incurred could have mitigate and construction sequencing could have been adjusted proactively.

The unexpected depth of caisson C4 and C5 constituted the largest construction challenge faced by the team. Diligent communication and work in good faith allowed the schedule impacts to be mitigated and the project to remain on track for the subsequent Critical Dates.

5.2 Hydrogen Embrittlement of Galvanized Reinforcing Steel

The City of Winnipeg has long been a proponent of the use of hot-dip galvanized (HDG) reinforcing steel in its bridge structures. Although HDG reinforcing steel is fairly commonplace in the industry, the use of HDG reinforcing steel in the Waverley Underpass and other projects being managed simultaneously by the Project Team identified challenges and possible improvements associated with the *ASTM A767/A767M-16 Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement* – particularly with respect to the manner in which hydrogen embrittlement is tested for. Based on the Project Team’s experience on the matter, concerns were raised regarding the HDG reinforcing as there were a number of recent galvanized rebar failures identified on sites during routine inspections of other projects. A typical failure is shown in **Figure 8**. The failure behaviour was noted to be brittle, resembling the deleterious effects of hydrogen embrittlement.



Figure 8: Typical Failure of HDG Rebar (Note bent bars are fractured in two)

For the purposes of this project designed for a service life of 75 years, embrittlement of reinforcement poses significant long-term durability concerns for the composite deck superstructure. As defined by *ASTM A143/A143M-07 Standard Practice for Safeguarding Against Embrittlement of Hot-Dip Galvanized Structural Steel Products and Procedure for Detecting Embrittlement*, embrittlement is “the loss or partial loss of ductility in a steel where an embrittled product characteristically fails by fracture without appreciable deformation”. Any reduction in the ductility and load-bearing capacity can lead to sudden failure below the material’s yield stress.

Common ways in which hydrogen embrittlement is typically derived in reinforcing steel includes aging phenomena, cold working of reinforcing, and specifically for the case in question, absorption of hydrogen. Signs of hydrogen embrittlement may also appear after pickling reinforcement with an excessive immersion time.

During the fabrication of the HDG reinforcing steel, a series of pre-treatment methods were employed including visual inspection, caustic degreasing, sulfuric acid pickling, and zinc-ammonia chloride fluxing, after which, zinc galvanizing was applied in accordance with CAN/CSA Standard G164 and ASTM A767M-16 to a retention equal to a Class II level (610 gm/m²). The HDG reinforcing steel was chromate quenched immediately following galvanizing in accordance with the project’s specifications; this fabrication process was consistent for all HDG reinforcement on the project including 15M, 20M, 25M, and 35M rebar sizes. It’s noted that in the superstructure alone, all four rebar sizes were present as shown in **Figure 9**. The general pre-treatment and surface protection process was consistent for each bar sizes.

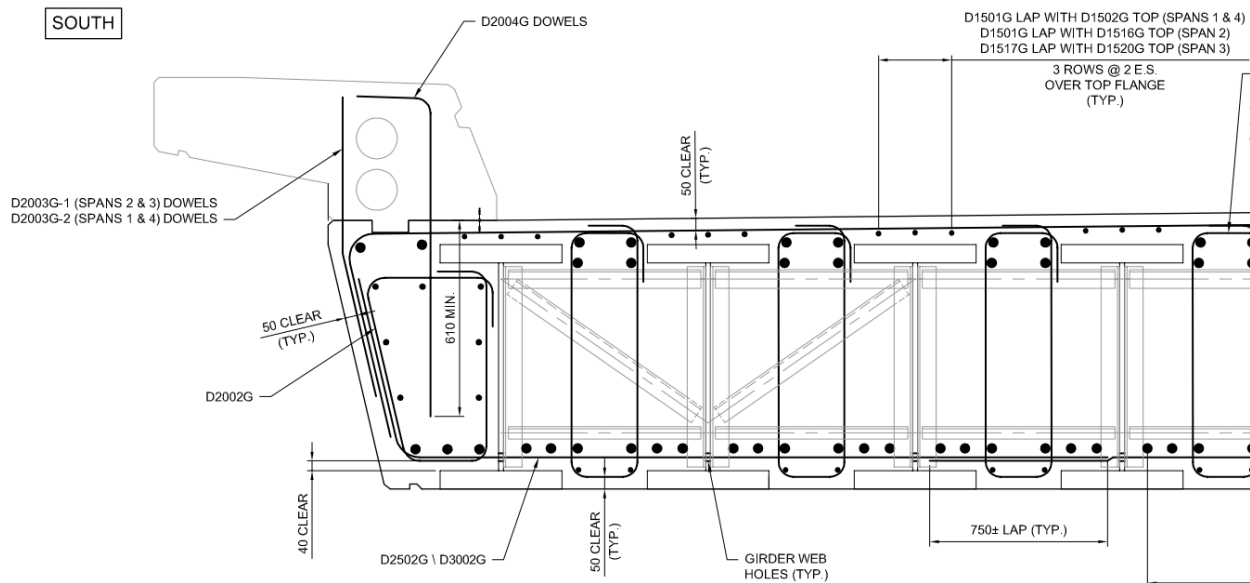


Figure 9: Superstructure Concrete Infill Reinforcement Layout (Partial Section)

Embrittlement of reinforcing steel is commonly detected after the fabrication process. *Section 9* of *ASTM A143M-07* outlines the bend test procedure commonly used in practice to detect

embrittlement. Representative photos of the recommended test are provided in **Figure 10** and the test is generally summarized as follows:

- Clamp the article in a vise (if necessary);
- Bend using a lever until cracking of the base steel occurs or until the bend reaches 90° (whichever occurs first);
- In the case of previously bent articles (that is, reinforcing steel bent prior to any bend-testing), the test shall be accomplished by reverse bending the previously bent section. Interestingly, this step was present in the 2003 version of ASTM A143M-03 but later removed in the 2007 update; and finally,
- Compare the galvanized article's degree of bending to the same for a similar ungalvanized article. If damage is present on both articles, then it can be concluded that the damage was not caused as a result of the hot-dip galvanizing process.

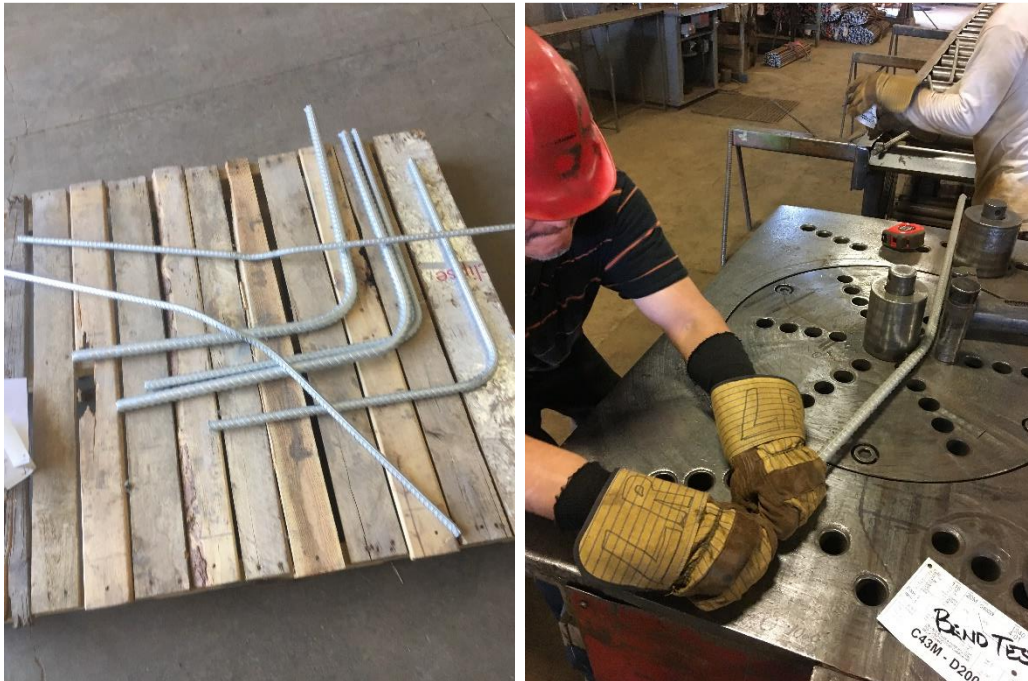


Figure 10: Waverley Underpass 15M and 20M Bend Test Specimens (Left); ASTM A143M-07 Bend Test in Progress (Right)

Additional key stipulations from ASTM A143M-07 regarding hydrogen embrittlement include:

- *A lot is a unit of production from which a sample may be taken for testing... a lot is one or more articles of the same type and size comprising a single order or a single delivery load, whichever is smaller, unless the lot identity, established in accordance with the above, is maintained and clearly indicated in the shipment by the manufacturer.*
- *If one test specimen should be found embrittled by these tests, two additional specimens should be tested. Failure of either the second or the third specimen shall be cause for rejection of the whole lot that the samples represent.*

In our experience, acquiring test results and quality control documentation at a regular frequency from fabricators has proved challenging on projects. Smaller reinforcing steel sizes have been generally selected for bend testing on past projects and it is noted that testing a single 15M bar is not representative of the entire reinforcement range of products for the overall project. Based on our experience, larger bars with increased surface area exposure tend to be more susceptible to hydrogen embrittlement and therefore are the generally the critical cases that requiring testing.

Additionally, and most importantly, bending of a straight HDG reinforcing steel bar is not equivalent to the reverse-bending of a previously bent and then galvanized reinforcing steel bar. The embrittlement failure tends to occurs particularly at the bends in reinforcement galvanized *after* bending. Although the recommendations outlined by the ASTM Standards have been adhered to in their entirety, the testing completed is not truly indicative of the actual reinforcing steel conditions which complicates the problem resolution process when a fractured bar is discovered in the field.

In the future, additional language is recommended to be included in project specifications modifying the bend test methodology outlined in ASTM A143M-07: Particularly, the 2003 methodology of reverse-bending a previously bent and galvanized reinforcing steel bar would be considered prudent. Above all, good quality control during the fabrication process – such as strict monitoring of pickling immersion time – is the best way to minimize the occurrence of embrittlement issues. Additionally, the use of “bendable” galvanized reinforcing steel, in which the bending occurs *after* the galvanizing process, could be explored.

5.3 Galvanized Reinforcing Steel Hydrogen Off-Gassing

Linked to the provision of HDG reinforcing steel, another reinforcement challenge encountered was hydrogen off-gassing from the reinforcing steel leading to “boils” presenting themselves in the plastic concrete during concrete placement.

Generally, as per ASTM A767M-16, chromate quenching is optional and is not completed unless specified by the Owner. When it is specified, ASTM A767M requires that the chromating process take place immediately after galvanizing. The principle advantage of the chromating process is

the passivation of the zinc surface resulting from the surface chromate film which has been shown to have a “self-healing” characteristic. Specifically, it is often used to prevent formation of wet storage stains during the first six weeks after galvanizing, the period in which the fresh zinc coating is most vulnerable to the elements.

Additionally, chromating is frequently utilized to prevent hydrogen evolution that occurs as a result of the chemical reactions which take place between wet concrete and zinc which could potentially lead to subpar bond between the concrete and reinforcing steel. In high pH scenarios, the zinc in the HDG coating is attacked by the alkalis in the fresh concrete leading to the formation of calcium hydrozincite and hydrogen. This reaction only occurs while the concrete is plastic and ceases once the concrete has set.

During the superstructure concrete pours for the Waverley Underpass specifically, surface boils appeared at all of the deck and trainman’s walkway pours as shown in **Figure 10 (Left)**. It is important to note that the Waverley Underpass reinforcing steel was chromate quenched in accordance with ASTM A767M and confirmation from the fabricator was obtained confirming so. Upon further investigation into other projects with similar issues, it was hypothesized that the boils were the result of the fresh concrete reacting with the zinc-rich paint that was used to touch up small defects (knicks, holidays, etc.) of the galvanized reinforcing steel or potentially fresh concrete reacting with the areas where the protective chromate layer provided by the chromate quenching of the reinforcement had been compromised or otherwise no longer present. Additionally, literature suggests that the chromating film is typically consumed within six weeks; therefore, if a chromate quenched bar is not cast into concrete within six weeks, the benefit of the chromating may be nullified.

Upon further investigation, the project team did not find any significant research study into the long-term durability concerns associated with these hydrogen gas boils; it is not clear whether these boils result in a direct pathway from the reinforcing steel to the concrete surface allowing for ingress of deleterious substances resulting in accelerated corrosion. In light of this, all boils on concrete components not scheduled to be covered with a waterproofing membrane were repaired by drilling 50mm into each boil and injecting the holes with high-strength two-part epoxy; a completed repair is shown in **Figure 11 (Right)**



Figure 11: Hydrogen Gas Boils on Concrete Deck Surface Immediately After Pouring (Left); Epoxied Repair of Boils (Right)

Interestingly, the off-gassing issues were not encountered during the substructure construction which also utilized HDG reinforcing steel; that is, the formation of boils was limited only to the superstructure concrete pours. It is currently not understood why this is the case, but this may be associated with the fact that the substructure concrete and superstructure concrete were two different mix designs. Seeing as the chemical reaction resulting in hydrogen evolution is strongest the higher the fresh concrete pH, variations in concrete mixes may play a role in the hydrogen formation. Additionally, it is interesting to note that in our experience, these boils have been generally limited to components cast using the redi-mix concrete of a specific local supplier which may further suggest the issue lies with the concrete mix and not necessarily the HDG reinforcing itself.

While the exact cause of the hydrogen gas boils on the Waverley Underpass is not conclusively known, following chromating and material care management best practices should be emphasized in project specifications to mitigate potential issues. Additionally, the Contractor's responsibility to repair hydrogen gas boils should also be explicitly written in the project specifications. Otherwise, as in the case of the Waverley Underpass, the contractor may argue that if the boils are due to a chemical reaction between the HDG reinforcing steel and fresh plastic concrete – both specified in detail by the Engineer – then the Contractor should not be responsible for repairing the gas boils.

6.0 Conclusion

The construction of the Waverley Underpass will alleviate traffic congestion and improve safety of the public. The works will also provide improved connectivity of active transportation in the area. The bridge portion of the project came up against several challenges including subsurface

conditions impacting the installation of two caissons, hydrogen embrittlement testing of galvanized reinforcement, and hydrogen evolution in plastic concrete.

The lessons learned through the issues encountered and the supplemental research of the issues will inform and improve the process of future works. In particular, it is recommended that a test borehole be conducted at each caisson location ahead of the start of construction so that the subsurface conditions are more accurately known. Further, improved language in the specifications with respect to the testing of galvanized reinforcement would alleviate much of the concern around hydrogen embrittlement in bent bars and pursuing alternative reinforcement in the form of “bendable” galvanized reinforcement may also improve the quality of the end product. Finally, hydrogen off-gassing can be best mitigated by following best practices for material management. Specifications should also have language expressly addressing the Contractor’s responsibility to repair any hydrogen gas boils.