GREER GALLOWAY
CONSULTING ENGINEERS

Hudson Bridge
Preliminary Design Report

GANANOQUE

Submitted to:

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Submitted by:

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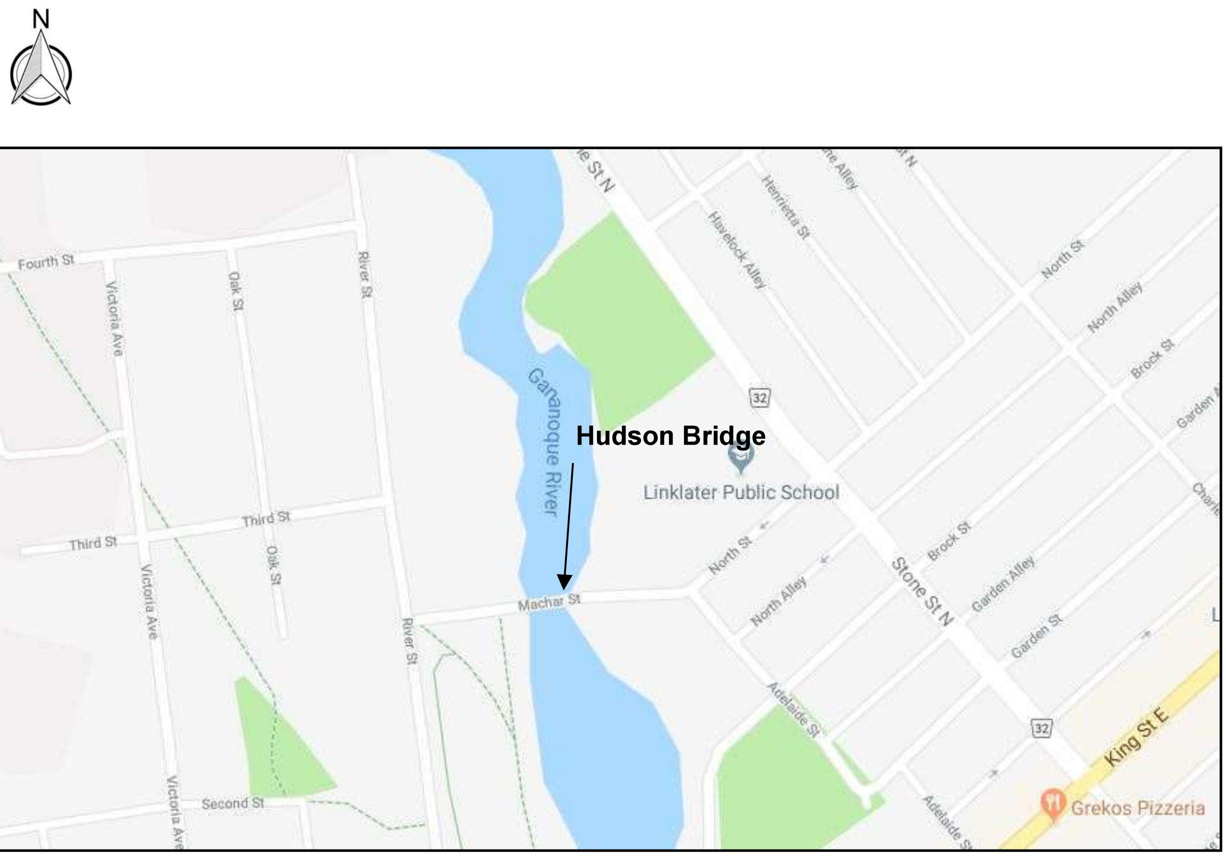
KEY PLAN
Hudson Bridge, Machar Street
Gananoque, Ontario



1. PROJECT OVERVIEW AND OBJECTIVE

Greer Galloway Group Inc. was retained by the Town of Gananoque to undertake a Municipal Class Environmental Assessment (MCEA), preliminary design, detailed design, contract preparation, and tendering for the replacement of the Hudson Bridge. The current structure is exhibiting significant structural deficiencies which require the bridge to be replaced.

In 2014, the Biennial Bridge Inspection Report, completed by Keystone Bridge Management Corp. identified several deficiencies in the structure. A cautionary load posting of 20 tonnes was established as a result of this inspection, due to observed irregular deflection values on the north side of the bridge. This load posting means that the bridge is not suitable for use by emergency vehicles such as fire trucks.

In 2016, the inspection by Keystone Bridge Management Corp. determined that the structural integrity of the bridge had been compromised by the deterioration and recommended that the structure be replaced by 2020. The report identified numerous deficiencies in the bridge structure including: rutting in the deck surface, impact damage to the approach guiderail, corrosion of the floor beams, top and bottom chord, stringers, and abutment bearings. Due to a variety of factors including structural, public safety, and economic considerations, the Town considers the replacement of the structure to be the most viable option.

The objective of the project is to address the identified structural concerns as they pertain to the replacement of the bridge so that it may continue to safely convey present and future traffic flows.

2. BACKGROUND

2.1 Site Location

The existing Hudson bridge structure is a single span, wrought-iron, pin-connected, Pratt through truss bridge with additional steel stringers and joists. The function of the bridge is to carry Machar Street over the Gananoque River. A pedestrian walkway was added to the structure during past rehabilitation work. The structure is owned and maintained by the Town of Gananoque. The structure spans across the Gananoque river over one (1) continuous span with a crossing length of 39.1m and an observed clearance of 1.4m from the water level. The bridge is roughly perpendicular to the river at the crossing. The road over the structure carries one (1) lane of traffic in the east-west direction with an ADT of 1095 based on town estimates. The percentage of truck volume was not determined presumably because existing postings limit the size and number of trucks that would use the bridge. The speed limit posting at this location is 25 km/hr and the weight limit posting is 20 tonnes.

The surrounding neighborhood is primarily residential, with a public school located immediately North East of the bridge and a municipal park immediately South West of the bridge. The embankments on the West slope at the corners of the structure consists of a natural slope with natural vegetation. The North East embankment is the steepest embankment, comprised of large rocks with some grass and vegetation throughout. Several of the rocks show signs of settlement and cracking. The South East embankment features a gabion retaining wall beyond the sidewalk. Beyond the gabion is a grassy, gentle slope which leads down to the area below the East end of the bridge. The embankments and slope protection are in fair condition with only minor erosion present.

As maintaining traffic is critical, a detour route will be provided throughout the duration of the replacement on the Hudson Bridge.

2.2 Background Information

The original steel structure was built in 1883 by the Dominion Bridge Company for the Thousand Island Railway (TIR) and was initially located further down river from its current location. In 1925 the bridge was sold to the town by TIR and was moved to its current location to help service the newly developing residential areas in the town’s North Ward.

An inspection report prepared by McCormick Rankin Corporation in 2010 records that reconstruction and rehabilitation work on the bridge was undertaken in 1996. The work included installing a new wood deck, steel deck joists, steel stringers, a guide rail system and rebuilding the concrete abutments.

3. EXISTING CONDITIONS

3.1 OSIM Inspection 2016

The findings of the 2016 OSIM Inspection completed by Keystone Bridge Management Group can be found in their report, issued June 2016. The OSIM found severe corrosion and rust perforation throughout the bridge’s primary structural components including the floor beams and truss compression members. The report stated that the bridge could not be relied on to carry “traffic of any description after 2030”, with a recommendation that the town replace the bridge by 2020 to avoid assuming unnecessary risk.

Timber Laminated Deck Surface

At the time of inspection, the timber had major rutting, allowing screws and steel spacers to protrude through the deck. It was recommended that asphalt padding be installed until the deck could be replaced.

Soffit

Laminated timber on steel tie deck. Lack of waterproofing is allowing water to reach steel stringers below deck surface.

Thrie Beam G/R (2) Railings
The railings were in good condition and secure. At the time of inspection, the approach guiderail in NE corner exhibited impact damage.

Bottom Chord
Eye Bars have uniform tension. Were found to be in reasonable condition given age. One eye bar in SE corner damaged (bent) from handling. 100% of bottom chord featured moderate corrosion.

Diagonal/Post/Hangers
100% of elements feature moderate corrosion. Load test completed in 2014 determined that all hangers receive tension.

Top Chord
95% of top chord features moderate to major corrosion. 5% features moderate perforation. Perforations were located in web and top flange of end diagonals.

Portals
Portals were in good condition.

Steel Floor Beams (6)
95% of the surface area of the floor beams exhibited moderate corrosion. 5% exhibited minor perforations or moderate section loss. Perforations were noted on two west most floor beams. The report recommended a boat inspection to review condition of all floor beams from up close.

Steel Deck Ties
5% of steel deck ties exhibited minor corrosion. These members are part of the deck system as part of a retrofit to the bridge. They did not cause any concern at time of inspection.

Steel Stringers (3)
10% exhibited moderate corrosion. The stringers are not original to the bridge and it was believed that the corrosion was caused by lack of waterproofing on the deck.

RC Abutment Wall (2)
5% of abutment wall exhibited signs of minor leaching cracks and minor scaling.

RC Ballast Wall (2)
No concerns at time of inspection.

Steel abutment bearings (4)

90% Moderate corrosion, 10% moderate section loss. The Abutment bearings were severely corroded. Debris around the bearings is increasing the rate of corrosion in bearings and end diagonals.

Water Channel
Deep channel with current.

Embankment
No defects detected in embankments. Well vegetated.

Signs
4 signs in place. Cautionary load posting sign in place at both ends of bridge.

4. HERITAGE

The Cultural Heritage Evaluation Report (CHER) prepared by Golder Associates Ltd. (June 21, 2018) determined that the Hudson Bridge has cultural heritage value. Per the Heritage Impact Assesement (HIA), prepared by Golder Associates Ltd (June 21, 208):

> *The heritage value lies in its rarity as one of the oldest surviving steel bridges in Ontario, as an early example of a pin connected Pratt through truss bridge, for its historical connections to the Thousand Islands Railway, for its links to other historic bridges in Town, and for its qualities as a landmark.*

It was determined that replacement of the bridge would have a significant adverse effect on the structures heritage attributes. Per the recommendations of the HIA, Greer Galloway is considering ways to incorporate the scale, massing, materials and finishes of the original Bridge where possible and appropriate, into the final design.

5. ABUTMENT CORING

St. Lawrence Testing conducted bridge coring on the structure on October 10th. Cores were taken from both the bridge abutments and the ballast wall (labelled in the report as lower abutment and upper abutment respectively). Laboratory testing included compressive strength, air void system, and chloride ion content analysis. The results of these tests can be found in the Geotechnical Report in **Appendix C.**

The initial core test failed to produce a sufficient amount of data for the existing conditions of the abutment (labelled lower abutments in geotechnical report). As such additional cores were taken from the bridge abutments on December 17th, 2018. A representative from Greer Galloway was on site to assist with location and number of cores.

Cores taken from the East abutment consisted of two types of concrete; older unreinforced concrete with large natural boulders, and a layer (approximately 250mm thick) of newer, reinforced concrete at the outside face. The West abutment was made of the same older unreinforced concrete with only a thin layer of new unreinforced concrete patch material on the outside face.

The results for core testing of the Hudson Bridge are outlined below. Full results can be found in **Appendix D**.

5.1 Concrete Compressive Strength

Concrete core samples taken from the existing abutments were tested for compressive strength in accordance with CSA A23.2-14C. Results were provided for both the newer patch material, as well as the original concrete. Compressive strengths for the bridge abutment cores are found in table 5.1, below.

Core Section	Concrete Type (patch or original)	Strength (MPa)
3	Patch	48.9
3	Original	16.0
4	Patch	46.8
5	Patch	51.9
6	Original	14.4
9	Patch	45.2
14	Original	22.2
16.1	Original	19.5
16.2	Original	24.5
17.1	Original	23.0
17.2	Original	22.1

Table 5.1 Compressive Strength (from Dec 17, 2018)

From the table above, the concrete patch material from the East abutment has a much higher compressive strength compared to the original abutment material. At this time, Greer Galloway is working to confirm the suitability of reusing the existing abutments based on the test results. Based on preliminary calculations we can determine that the existing abutments will be able to be re-used to some extent with some re-facing (patch) and placement of additional, reinforced concrete patching at both East and West abutments.

If existing abutments are deemed unsuitable, new cast-in-place C-1 Concrete abutments will need to be placed. New abutments would be excavated minimum 5 ft below grade, or to sound bedrock to provide frost protection.

5.2 Air Void System in Hardened Concrete

The Air Void System in the hardened concrete was tested as per ASTM C457. Air voids in the hardened concrete above 3.0% is generally preferable, and the applicable A23.1 states 5-8% for 14-20mm aggregate size on structural concrete exposed to chlorides and freeze and thaw cycles. The results for the bridge are found in table 5.2, below.

Core Section	Concrete Type (patch or original)	Air Content (%)	Air Void Spacing Factor (mm)	Assessment
2	Patch	4.4	0.117	NOT acceptable
2	Original	12.7	0.275	NOT acceptable
6	Original	16.1	0.146	NOT acceptable
8	Patch	5.3	0.121	Acceptable
13	Patch & Original	5.8	0.569	Acceptable
18	Original	2.2	0.687	NOT acceptable

Table 5.2 Air Void System (from Dec 17, 2018)

Air content values of both the patch material and original concrete are within expected ranges based on the age of each section. The air content of the original concrete is significantly greater than that of the patch material. The patch material is within acceptable limits however which bodes well for re-use of the abutments.

5.3 Chloride Ion Content in Hardened Concrete

The chloride ion content in the hardened concrete was tested as per ASTM C 1202. An exposure class of C-1 was used to assess the existing abutments. Per CSA A23.1, for C-1 exposure the chloride ion penetrability must be less than 1500 Coulombs. The results for the bridge are found in table 5.3, below.

Core Section	Concrete Type (Original or Patch)	RCP (Coulombs)	Assessment
3	Patch	571	Acceptable
3	Original	5440	NOT acceptable
5	Patch	865	Acceptable
9	Original	3951	NOT acceptable

11	Original	4328	NOT acceptable
18	Original	2103	NOT acceptable

Table 5.3 Chloride Ion Content (from Dec 17, 2018)

From the results above, only the patch material has acceptable RCP values. This is not surprising given the age and condition of the original concrete. These results indicate that the suitability of existing abutments for reuse will be based primarily on the “patch” concrete. Leading to the idea of refacing with a structural mass of concrete and leaving the existing poor concrete as lateral bracing to be doweled into.

5.4 Reinforcing

After reviewing the condition of the existing concrete rebar scanning was deemed unnecessary. This is primarily because the existing original concrete was deemed unsuitable for reuse as abutments for the new structure. From the results above it is now known that existing rebar in the East abutment patching is not corroded. As such, a scan should be ordered to determine the location and size of existing rebar in the patching.

6. GEOTECHNICAL INVESTIGATION

6.1 Scope

St. Lawrence Testing. conducted a geotechnical investigation at Hudson Bridge on October 10th, 2018. A Geotechnical Report was completed and is included in **Appendix C.**

Two boreholes were drilled, one each at the East and West end of the Bridge.

6.2 Stratigraphy and Borehole Locations

Borehole 1 was put down 3m East of the East abutment and 0.6m North of the road centre line. Borehole 2 was placed 3m West of the West abutment and 0.6m North of the road centre line. Both locations showed similar stratigraphy at each end of the bridge.

Borehole stratigraphy indicated 150mm to 200mm of asphalt at the surface underlain by gravel fill to approximately 500mm below the road surface. At both sides there was fill found below the gravel. Fill in Borehole 1 was brown, with some black, moist, loose silty sand and gravel down to 2.44m. Fill in Borehole 2 was brown, moist, compacted silty sandy gravel to 3.10m.

At Borehole 1, a steel plate was found at 2.44m below grade, above poor-quality granite which extended down to 2.8m. Excellent quality granite was found from 2.8m to 4.34m below grade. At borehole 2, the granite bedrock was of poor quality down to 4.2m, then of excellent quality to 5.73m below grade.

For specific borehole information refer to the borehole logs provided in the geotechnical report, found in **Appendix C.**

6.3 Bearing Capacities

The bearing capacities of the granite bedrock subgrade are found and summarized in the table below.

Factored ULS Bearing Capacity	1000 kPa
SLS Allowable Bearing Capacity	1000 kPa

Table 6.1 Bearing Capacities

6.4 Dewatering

At this time there is no significant in water work anticipated. Dewatering procedures will be established if required.

6.5 Roadway and Backfill

If any new foundations are built, or if extensive excavations are done a new roadway will be required. Backfill for roadway should consist of Granular "B" Type 2 up to the existing road subbase. All compactions hall be to 95% Standard Proctor Density in maximum 250mm lifts.

A frost tape will be required from the new gravel into the existing roadway. This taper should be at a 4 to 1 slope.

The roadway should consist of 300mm of Granular "B" Type 2 subbase and 150mm of Granular "A" base, each compacted to 100% Standard Proctor Density. The asphalt should match the thickness at each end, which currently varies from 150mm to 200mm thick. Asphalt to be HL3 with maximum lift thickness of 40mm and compaction to be to 96% Marshall Density.

7. TRAFFIC AND ROADSIDE SAFETY

7.1 Lane Width

At present, the Hudson Bridge carries one lane of predominantly vehicular traffic, and one pedestrian walkway across the Gananoque River, over one (1) continuous span with a crossing length of 39.1 m and an observed clearance of 1.4 m from water level. The deck has a travel width of 4 m and pedestrian walkway of 1.9m. There is approximately 50mm overlapping between the walkway and roadway. The total deck riding surface is 156.4 m^2.

7.2 Traffic Counts

Based on town estimates the bridge has an ADT of 1095 as of Autumn 2014, after the posting of the bridge. The percentage of truck volume was not determined presumably because existing postings limit the size and number of trucks that would use the bridge. The speed limit posting at this location is 25 km/hr and the weight limit posting is 20 tonnes. It is advisable that upon completion of this project, a new traffic study be completed including percentage of truck volume, for town records.

The current ADT puts the bridge in Class B as per S6-14 (CHBDC). Our preliminary projections of traffic growth yield an ADT of 2000 within the next 50 years and 2500 at year 75. This projection suggests that the bridge will stay within Class B for its whole new lifecycle.

7.3 Sidewalks

At present, there is a sidewalk on the South side of the bridge. The Greer Galloway Group supports the town's request to maintain pedestrian access across the bridge.

7.4 Guiderails

At present, the bridge features guide rails on both the North and South sides of the traffic lane. The North side features a thrie beam guide rail along the length of the structure which ties into the guard rail at the East and West approaches. The South side features a thrie beam guard rail similar to the North, as well as an additional thrie beam above, creating greater separation between vehicle and pedestrian traffic.

8. CONSTRUCTION STAGING AND UTILITIES

8.1 Utilities

Utilities could be an issue at this location due to the proximity of the Bell/Hydro poles and lines to the structure. These existing utilities will need to be protected by the Contractor during construction unless design decisions warrant the need for relocation. Additionally, there is a dormant gas main buried within the roadway. Contractor will need to be aware of the location of this gas main as it may interfere with site work depending on final scope.

8.2 Detour and Staging

The Greer Galloway Group identified the need to maintain traffic throughout the project. Due to the bridge being a single lane closure is unavoidable during construction. A detour route will need to be provided throughout the duration of the bridge replacement.

9. PERMITS

9.1 Conservation Authority

A meeting with the Cataraqui Region Conservation Authority has not yet taken place, however, it will be required since the east abutment rehabilitation will involve a limited amount of activities in the water. The minutes and any relevant information from these meetings will be incorporated into this report as part of the Appendices.

10. COST ESTIMATE AND EXPECTED LIFE SPAN

10.1 Cost Estimate

As the 60% design is being completed, cost estimates cannot be provided at this time. This PDR will be updated with all relevant cost estimates once prepared.

10.2 Life Span

As the superstructure of this bridge is being replaced, a new lifecycle of 75 years can be expected, provided the necessary rehabilitation is conducted on the substructure to bring it up to the standards of Class B highway bridge.

11. RECOMMENDATIONS

In consideration of all influencing factors, a set of criteria was established early on to compare and narrow down the alternatives to the most suitable design options. The design options that were carried forward for final evaluation are those that were rated the highest with respect to the following criteria:

- Cost effectiveness,
- Ability to sufficiently address the structural deficiencies on the substructure and mitigate causes of these deficiencies,
- Minimal or no anticipated impact on heritage value of the structure,
- Minimal anticipated impact on the environmental footprint, and
- Minimal or no relocation of existing utilities.

Accordingly, the options that have been carried forward for further review and evaluation are:

1. Alternative 1: Replacement of the structure with a Mabey compact 200 style pre-engineered modular bridge.
2. Alternative 2: Replacement of the structure with a custom vehicle truss bridge
3. Alternative 3: Replacement of structure with steel girder with concrete deck bridge

The Greer Galloway Group recommends Alternative 1: Replacement of the structure with Mabey Compact 200 style pre-engineered modular bridge. All of the alternatives will be designed to support adequate vehicular and pedestrian loads over the existing span as per the applicable road class.

12. CONCLUSIONS

Based on past performance, it is expected that some of the existing substructure can be utilized by the new superstructure with little to no alteration. Namely, the ballast walls. Base on our preliminary calculations and the results from core testing we have determined that the abutments may be able to be reused. The extent of their usefulness is dependent on repairing the areas affected by corrosion. Based on the same coring tests the original concrete has been deemed not usable.

Our intent would be to reface the parts of the existing abutments which are suitable for reuse. Refacing would involve placement of new concrete and reinforcement. This approach is ideal as it limits the scope of required concrete removals. However, if we are unable to provide enough load transfer to the bedrock, in order to maintain the existing foot print significant removals would be required. We would need to remove both the newer patch sections as well as the original concrete up to 1500mm below grade (frost line) or to sound bedrock. New abutments would then be poured. In the event that such actions are required, it would be our preference to preserve the existing ballast walls. We are in the process of analyzing the stability of the ballast walls for this requirement and we have reasons to assume with some certainty that they will be okay.

The three proposed alternatives for the new bridge structure are all suitable for the existing span and anticipated loading. Additionally, they can all be designed with appropriately wide carriageways and pedestrian access. The Mabey structure is more economical but features a significantly different aesthetic from the existing structure. The Mabey bridge features a central carriageway with a cantilevered walkway similar to the original structure and comes only in galvanized finish. While it is our preferred option, much of the heritage characteristics and aesthetic appeal of the bridge would be lost with this alternative.

A custom vehicle truss bridge is more stylistically similar to the Pratt Through Truss of the original structure which is the source of much of the heritage/cultural value. While more expensive, this type of bridge may appeal more to the public, especially those who would object to the removal of the existing structure on visual grounds. This alternative also has the option of a weathered steel exterior coating/finish evocative of the original Thousand Islands Railway (TIR) bridge.

Both the Mabey bridge and the custom vehicle truss bridge are pre-engineered, pre-manufactured structures. For both alternatives the bulk structure is produced off site and simply assembled at bridge location. This reduces the closure time for the crossing, as well as the amount of work done on site.

The third alternative, a steel girder bridge with concrete deck would have to be custom made and will require a specialized erection/installation procedure, which will result in higher costs. This alternative would also feature a higher dead load which would have implications on the abutment design/rehabilitation. Additionally, the concrete elements would all need to be cast-in-place which could potentially increase the construction time.

All the above alternatives will be designed to support adequate vehicular and pedestrian loads over the existing span. Vehicular loading has been calculated in accordance with CSA S6-14 - The Canadian Highway Bridge Design Code (CHBDC), and incorporates dynamic load effects and support of a CL-625 (transport) truck. Pedestrian walkway loads were analyzed in accordance with Ontario Building Code (OBC) requirements for footbridges (4.8 kPa). The dead load used for analysis depends on the type of bridge selected. Once a final decision has been made, the specific dead load can be incorporated into the final design calculations.

We trust this Preliminary Design Report provides sufficient information and is to your complete satisfaction.

Respectfully Submitted,

GREER GALLOWAY GROUP

CONSULTING ENGINEERS
Tony de la Concha, P.Eng.
Project Manager

Appendix A
Geotechnical Report

St. Lawrence Testing
& Inspection Co. Ltd.
P.O. Box 997, Cornwall, ON, Canada K6H 5V1
814 Second Street W., Phone (613) 938-2521
E-mail: slt@ontarioeast.net Fax (613) 938-7395

October 31, 2018

Mr. Bryson Lee, P. Eng.
Greer Galloway & Associates Ltd.
640 Cataraqui Woods Drive
Unit 2A
Kingston, ON
K7P 2Y5

RE: Machar St. Bridge, Gananoque, ON
Geotechnical Subsurface Investigation
Report No. 18C257

Dear Mr. Lee:

In accordance with verbal and e-mail instructions received from you, this report is submitted outlining the results of a geotechnical subsurface investigation carried out at the Machar St. Bridge, connecting North St. to River St. in Gananoque, ON.

A) DESCRIPTION OF FIELD WORK

Prior to starting drilling, service locates were carried out. There were some difficulties in carrying this out because of the gas company requiring a lot of notice and the drilling company availability. The Town of Gananoque stepped in and established a date for drilling. There was a gas company representative on site full time on the day of drilling. The gas main that had been present was no longer in use but was still buried within the roadway. The gas company representative indicated the location of the old gas main and remained on site full time during the drilling.

We had 2 crews on site to carry out the drilling. In addition to the regular boreholes, we were also requested to obtain concrete cores from the abutments. The coring contractor was Tillaart Core Drilling from Cornwall, ON. The drilling contractor was Eastern Ontario Diamond Drilling from Hawkesbury, ON. They had a truck mounted CME 55 auger drill. Supervision of the drilling and the coring was by the undersigned geotechnical engineer.

The boreholes were advanced through the overburden to the top of the bedrock. Coring then took place until there was full recovery and a high RQD. The samples were visually identified on site and recorded. The results are found attached in the borehole logs.

The core locations were recorded on site and returned to our lab for further tests. It was requested to obtain the measured air content of the concrete in addition to the compression tests. Our lab carried out the compression tests. The air content analyses were subbed out to a specialty firm. The results will be forwarded when the test report is received.

B) STRATIGRAPHY AND BOREHOLE LOCATIONS

The stratigraphy is fairly similar at the site based on one borehole put down at each end of the bridge.

Borehole 1 was put down 3 m. East of the East end abutment and 0.6 m. North of the road centre line.

Borehole 2 was put down 3 m. West of the West end abutment and 0.6 m. North of the road centre line.

The stratigraphy at the boreholes indicates 150 to 200 mm. of asphalt at the surface underlain by a gravel fill to approximately 0.50 m. below the surface.

There is a fill below the gravel. At Borehole 1, the fill is a brown, with some black, moist, loose silty sand with gravel down to 2.44 m. At Borehole 2 the fill is a brown, moist, compact silty sandy gravel to 3.10 m.

There was a steel plate above the granite bedrock at Borehole 1 at 2.44 m. below grade. The granite was of a poor quality down to 2.8 m. then was of excellent quality down to 4.34 m. with a full recovery and an RQD of 100.

The granite bedrock at Borehole 2 was of a poor quality down to 4.2 m. then was of excellent quality to 5.73 m. with a full recovery and an RQD of 94.

For the specific information at each borehole the borehole logs should be referred to.

C) GEOTECHNICAL DISCUSSION

1) General

It is our understanding that it is proposed to put up a new bridge at this location. The possibility exists that if the existing abutments are suitable, they will be reused for the new bridge.

2) Foundations

If new foundations are to be built, they can be constructed on footings bearing on the granite bedrock.

The nature of granite in this area is that it is highly variable in elevation. The core recovery and RQD indicate that the upper surface may be highly fractured. It will be necessary to excavate to the bedrock and expose the upper surface. Any loose granite should be removed. There will likely be a requirement to hoe ram some areas in order to provide a reasonable flat surface. Once the loose granite is removed, a modified high strength mud slab should be poured in order to provide a uniform flat surface for the new footing. It is likely that rock anchors be drilled in the granite bedrock, to extend not only into the mud slab, but to extend into the new footings as well. This is to ensure no horizontal shift in the bridge.

The bearing capacity of the footings on the granite bedrock surface is 1000 KPa S.L.S. and U.L.S. The seismic factor is Site Class A.

3) Roadway

If new foundations are to be built, a new roadway will be required where the road is excavated.

Once the foundations are completed, the backfill should consist of Granular "B" Type 2 within the roadway up to the existing road subbase. All compaction should be to 95% Standard Proctor Density in maximum 250 mm. lifts.

There will be a frost taper required from the new gravel into the existing roadway. This should be at a 4 to 1 side slope. As an example, if the new gravel starts down at 2.0 m. below grade and the bottom of the road gravel is at 0.50 m. below grade, the frost taper should be 1.5 x 4 = 6 m. long.

The roadway should consist of 300 mm. of Granular "B" Type 2 subbase and 150 mm. of Granular "A" base, each compacted to 100% Standard Proctor Density. The asphalt should match the thickness at each end. It currently varies from 150 to 200 mm. thick. The asphalt should be HL3 throughout because of the small quantity. Maximum lift thickness should be 40 mm. and compaction should be to 96% Marshall Density.

D) CONCRETE TEST RESULTS FROM EXISTING FOUNDATIONS

We extracted 4 cores from each end of the bridge. Two cores were taken from the lower, older abutments and two cores were taken from the upper, newer abutments. One core from each set was taken on the North side and one core was taken on the South side.

The examined cores showed a slight difference between the older foundations and the newer foundations. The older concrete had a slightly more variable colour appearance. In some cases there was very large aggregate stone in the older concrete.

Following is a summary of the core locations.

West Abutment

Core # 1	Upper abutment – North side
Core # 2	Lower abutment – North side
Core # 3A	Upper abutment – South side. This core broke off in the extraction process. We cored another close by.
Core # 3B	Upper abutment – South side
Core # 4	Lower abutment – South side

East Abutment

Core # 5	Upper abutment – North side
Core # 6	Lower abutment – North side
Core # 7	Upper abutment – South side
Core # 8	Lower abutment – South side

Following are the compressive test results of the cores

Core # 1

Unit weight:	146.8 lbs./cu. ft.
Compressive strength:	6860 p.s.i. or 47.3 MPa

Core # 3B

Unit weight:	151.5 lbs./cu. ft.
Compressive strength:	7030 p.s.i. or 48.5 MPa

Core # 6

Unit weight:	156.6 lbs./cu. ft.
Compressive strength:	7400 p.s.i. or 51.0 MPa

Core # 7

Unit weight:	154.7 lbs./cu. ft.
Compressive strength:	6600 p.s.i. or 45.5 MPa

The high unit weights of the concrete indicate that there was no likelihood of air entrainment in the concrete. This is understandable since the use of air entrainment only started around 1960. The age of the bridge appears considerably older than 1960.

E) CONSTRUCTION CONTROL

In order to ensure that the recommendations of this report are adhered to, it is recommended that our firm be retained to inspect and test for the possible footings and road backfill to ensure that the recommendations are followed.

Respectfully submitted
ST. LAWRENCE TESTING & INSPECTION CO. LTD.

REGISTERED PROFESSIONAL ENGINEER
G. G. McINTEE
PROVINCE OF ONTARIO

G.G. McIntee, P. Eng.
GGM:njw
Attachments

St. Lawrence Testing & Inspection Co. Ltd.

OFFICE BOREHOLE RECORD

REPORT NO. 18C257
BOREHOLE NO. 1
CASING HF Auger
DATUM

CLIENT Greer Galloway & Associates Ltd.
LOCATION Machar St. Bridge, Gananoque, ON
DATE OF BORING October 10, 2018 DATE OF WL READING

DEPTH	ELEVATION / DEPTH	SOIL DESCRIPTION	STRAT. PLOT	WATER CONDITIONS	CONDITION	TYPE	NUMBER	RECOVERY	N - VALUE	LABORATORY TESTS PERFORMED
0		200 mm. Asphalt Gravel Fill								
	.51	Silty Sand Fill Brown, some black, moist, loose, with gravel								
1						SS	1	50	7	
						SS	2	60	8	
2						SS	3	40	9	
	2.44	25 mm. Steel Plate Granite Bedrock				C	4	73	RQD 0	
3										
						C	5	100	100	
4										
	4.34	Termination of coring								
5										

SAMPLES

LAB TEST RESULTS

WATER CONTENT & ATTERBERG LIMITS.
WP W WL

DYNAMIC PENETRATION TEST BLOWS PER FOOT.. .K...
0 20 40 60 80

APPENDIX

St. Lawrence Testing & Inspection Co. Ltd.

OFFICE BOREHOLE RECORD

REPORT NO. 18C257

CLIENT Greer Galloway & Associates Ltd.

BOREHOLE NO. 2

LOCATION Machar St. Bridge, Gananoque, ON

CASING HF Auger

DATE OF BORING October 10, 2018

DATE OF WL READING

DATUM

SOIL PROFILE: ELEVATION / DEPTH	SOIL DESCRIPTION	STRAT. PLOT	WATER CONDITIONS	SAMPLES: CONDITION	TYPE	NUMBER	RECOVERY	N - VALUE	LABORATORY TESTS PERFORMED
0	150 mm. Asphalt Gravel Fill								
.46	Silty Gravel Fill Brown, moist, compact, sandy, becoming wet below 2.7 m.				SS	1	70	23	
					SS	2	65	11	
					SS	3	50	22	
					SS	4	85	53	
								RQD	
3.10	Granite Bedrock				C	5	57	0	
					C	6	100	94	
5.73	Termination of coring								

LAB TEST RESULTS

WATER CONTENT & ATTERBERG LIMITS.
WP W WL

DYNAMIC PENETRATION TEST BLOWS PER FOOT.. .K ...
0 20 40 60 80

APPENDIX

Appendix B
Abutment Core Testing Report

DAVROC
TESTING LABORATORIES INC.

CONSULTING ENGINEERS

Materials Testing
and Inspection

File: L19-0239MT **January 18, 2019**

St. Lawrence Testing & Inspection Co. Ltd.
814 Second Street West
P.O. Box #997
Cornwall, Ontario
K6H 5V1

Attn.: Mr. Gib McIntee, P.Eng.

Dear Sir;

Preliminary Report
Concrete Core Testing from
Hudson Bridge Abutments
Machar Street, Gananoque, Ontario

Further to the receipt of nineteen (19) 100 mm nominal diameter concrete core samples to our laboratory on January 3 and 4, 2019, Davroc Testing Laboratories Inc. (Davroc) is pleased to provide the results of the testing carried out on these core samples. This letter summarizes our test results and observations.

It is our understanding that on Dec 17, 2018 the subject core samples were drilled and extracted from the bridge abutment walls of the Hudson Bridge located on Machar Street, Gananoque, Ontario. Using field information emailed to us December 21, 2018 by you, core sample identification numbers and field core locations are presented in Table 1 below.

Table 1

Core No.	Bridge Abutment	Abutment Face	Core Location Description/Comments
1	East	South	20" North of South end, 9" below top. Core broke on rebar
2			22" North of South end, 15" below top
3			30" North of South end, 22" below top
4			41" North of South end, 22" below top
5			49" North of South end, 11" below top
6		North	89" North of South end, 14" below top
7			89" North of South end, 42" below top
8			125" North of South end, 12" below top
9			127" North of South end, 26" below top
10	West	South	35" North of South end, 7" below top
11			35" North of South end, 15" below top
12			28" North of Core 11, 24" below top
13			27" North of Core 10, 12" below top
14			53" North of Core 13, 17" below top
15		North	55" North of Core 12, 28" below top
16			23" North of Core 15, 29" below top
17			27" North of Core 14, 16" below top
18			44" North of Core 17, 24" below top
19			44" North of Core 18, 12" below top

Note: all information in this table was provided by the client

Visual Observation Comments

The core samples received were visually noted to consist of one of three different concrete horizon (layer) compositions:

1. Old concrete the entire length of the core (assumed to be the ~100 year old original concrete), or
2. Newer patching concrete material layer (up to 255 mm in thickness) followed by older (assumed original) concrete, referenced from the exterior face of the abutment (the newer patching concrete included reinforcing steel), or
3. A thin veneer (up to 75 mm in thickness) of patching grout followed by the older (assumed original) concrete, referenced from the exterior face of the abutment.

Cores consisting of some or all of the old concrete were observed to have no reinforcing steel embedded within the old concrete however, occasional large natural rocks were observed in the concrete mix (see attached photographs appended to this report).

Bonding between the old concrete and patching grout and bonding between the rock and old concrete was noted to be very weak with total separation observed in many cases.

Laboratory Testing and Results

Upon receipt of the core samples in our laboratory each core was measured to determine the overall length and the length of each horizon of different concrete/patching material. Selected samples were then prepared for specific laboratory testing following a testing plan pre-approved by the client.

The results of the core specimen measuring and laboratory testing are attached to this report as Appendix 2, Table 2.

We trust that the above information is satisfactory. Should you have any further questions, please do not hesitate to contact the undersigned.

Yours very truly,
Davroc Testing Laboratories Inc.

Greg Wuisman, P.Eng.
V.P. of Materials Engineering and Testing

Sal Fasullo, C. E. T.
Executive Vice President

GW/SF/gw
19-0239-1 St Lawrence

Appendix 1
Photographs

Photograph 1
Core #11 showing embedded natural rock with poor bond to concrete

Photograph 2
Core #16 showing embedded natural rock

Photograph 3.
Core #17 showing embedded natural rock

Photograph 4.
Core #13 showing embedded rock completely debonded from concrete

Appendix 2

Table 2

Core No.	Length (mm)			Core Horizon Tested	Description of Concrete Tested (patch or original)	Test Conducted			
	Total Core	Patch/ Concrete Material	Original Concrete			Compressive Strength (MPa)	Air Voids system		RCP (Coulombs)
							Total Air Content (%)	Spacing Factor (mm)	
1	116	0 to 116	-	-	Patch	No Test			
2	290 -320	0 to 220	220 to 320	0 to 150	Patch	-	4.4	0.117	-
				220 to 320	Original	-	12.7	0.275	-
13	395 – 430	0 to 210	210 to 430	10 to 60	Patch		-	-	571
				65 to 210	Patch	48.9	-	-	-
				220 to 270	Original	-	-	-	5440
				275 to 395	Original	16.0	-	-	-
4	247 - 255	0 to 255	-	25 to 170	Patch	46.8	-	-	-
5	240 - 260	0 to 245	245 to 260	10 to 60	Patch	-	-	-	865
				120 to 245	Patch	51.9	-	-	-
6	400 -410	0 to 200	200 to 410	210 to 280	Original	-	16.1	0.146	-
				285 to 395	Original	14.4	-	-	-
7	90 – 220	0 to 220		-	Patch	No Test			
8	208 - 225	0 to 225		0 to 150	Patch	-	5.3	0.121	-

Core No.	Length (mm)			Core Horizon Tested	Description of Concrete Tested (patch or original)	Test Conducted			
	Total Core	Patch/ Concrete Material				Compressive Strength (MPa)	Air Voids system		RCP (Coulombs)
							Total Air Content (%)	Spacing Factor (mm)	
9	310 - 380	0 to 250	250 to 380	10 to 60	Patch				-
				110 to 250	Patch	45.2	-	-	-
				255 to 305	Original				3951
10	125-225	-	0 to 225	-	-		No Test		
11	270-320	-	0 to 320	20 to 70	Original				4328
12	160 -205	0 to 20	20 to 205	-	-		No Test		
13	295 -315	0 to 30	30 to 315	0 to 110	Patch & Original		5.8	0.569	-
14	270 - 300	0 to 30	30 to 300	30 to 220	Original	22.2	-	-	-
15	210 - 240	0 to 35	35 to 240	-	-	No Test			
16	300 - 375	0 to 30	30 to 375	25 to 150	Original	19.5	-	-	-
				155 to 300	Original	24.5			
17	345 - 405	0 to 35	35 to 405	35 to 220	Original	23.0	-	-	-
				225 to 340	Original	22.1			
18	235 - 260	0 to 30	30 to 260	40 to 90	Original		-	-	2103
				217 to 300	Original		2.2	0.687	-
19	265 - 275	0 to 75	75 to 275	-	-	No Test			