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Appendix ‘G’ – Geotechnical Assessment

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REPORT

Geotechnical Assessment

Three Grand River Crossings (Lorne Bridge, Brant's Crossing Bridge, and TH&B Railway River Crossing Bridge), City of Brantford, Ontario

Submitted to:

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1.0 INTRODUCTION

This report was prepared as part of a submission for the Environmental Assessment (EA) Study being carried out to assess the structural conditions and to determine the extent of the proposed works for bridge rehabilitation or replacement of three bridges crossing the Grand River (the Lorne Bridge, Brant's Crossing Bridge, and the Toronto, Hamilton, & Buffalo Railway (TH&B) Bridge No. 75) in Brantford, Ontario. The locations of the bridge sites are shown on the Location Plan, Figure 1.

Based on the information provided, the proposed work(s) will include either bridge rehabilitation of the structural members and bridge deck or complete bridge replacement(s) at the three sites. An overview of the existing geotechnical conditions within the project area, specifically the three bridge crossing locations, is provided in this report based on a site reconnaissance and a review of existing geotechnical data. No new intrusive geotechnical exploration or testing activities were carried out under the current scope of work for this assignment. This report includes preliminary geotechnical recommendations for the assessment and conceptual design of the proposed project. GM BluePlan Engineering Limited (GMBP) has been retained by the City of Brantford (the City) to complete a conceptual study on the rehabilitation and/or replacement options for the bridge structures at the site. Authorization for Golder Associates Ltd. (Golder) to proceed with the project, in accordance with our proposal dated August 19, 2019, was provided by GMBP.

This report should be read in conjunction with the attached document "Important Information and Limitations of This Report", which comprises an integral component hereof. The reader's attention is specifically drawn to this material, as it is essential for proper use and interpretation of the information presented and discussed herein. This report addresses only the geotechnical aspects of the referenced works.

2.0 METHODOLOGY

Existing available geotechnical information for the areas of the bridge(s) and nearby sites and the existing bridge design drawings were compiled and reviewed. The information consisted of topographical mapping, soils and bedrock mapping, geological data, and site-specific geotechnical data from previous site investigations carried out on, or adjacent to, the site. The previous site investigations, inspection report and drawings are identified as follows:

- Lorne Bridge, Brant's Crossing Bridge and TH&B Railway Bridge No. 75 original construction plans/drawings
- Dominion Soil Investigation Inc. Report No. 78-4-K2 titled "Subsurface Investigation, Proposed Reconstruction of Lorne Bridge, Brantford, Ontario", dated May 1978.
- Peto MacCallum Ltd. Report No. 89 F 251 titled "Geotechnical Investigation Foundation Subgrade Evaluation, Lorne Bridge, Brantford, Ontario", dated 1986
- Golder Report No. 881-3443 titled "Geotechnical Investigation, Proposed Brantford Flood Control Works, Lorne Bridge to Market Street, Brantford, Ontario", dated January 1989;
- Golder Report No. 881-3443-1 titled "Geotechnical Investigation, Proposed Lorne Dam Removal, Brantford, Ontario", dated May 1989;
- Lorne Bridge Engineering Condition Assessment Reports (2004 and 2015);

- Lorne Bridge, Brant’s Crossing Bridge and TH&B Railway Bridge No. 75 enhanced Ontario Structure Inspection Manual (OSIM) Inspection Reports (2017); and
- Lorne Bridge, Brant’s Crossing Bridge and TH&B Railway Bridge No. 75 Ministry of Transportation of Ontario (MTO) Site Investigation Reports (March 2018).

Original design drawings of selective pages and relevant boreholes drilled at and adjacent to the site of the bridges were provided in Appendix A and B, respectively. In addition, a field reconnaissance was carried out by a geotechnical engineer from our staff on April 24, 2020. Select photographs are attached in Appendix C.

3.0 SITE DESCRIPTION

Based on the information provided and site reconnaissance, the Lorne Bridge (also known as Colborne Street West) is located between Brant Avenue/Icomm Drive and Ballantyne Drive/Fordview Drive and is classified as a minor arterial roadway. It is defined by three structures, including: the Lorne Bridge pedestrian underpass, the Lorne Bridge arch, and the Lorne Bridge girder section. The Brant’s Crossing Bridge is approximately 130 metres south of the Lorne Bridge and is currently closed. It was used as a pedestrian crossing and east/west cycling trail connection and consists of four spans, including two that are steel truss construction and two that are half-through deck girder construction. The TH&B Railway Bridge is located approximately 270 metres south of the Brant’s Crossing Bridge. It was a former railway bridge that has been converted and is currently being used as a pedestrian and cycle crossing.

3.1 Lorne Bridge

The Lorne Bridge consists of a concrete box culvert pedestrian underpass, a single span concrete box girder bridge that crosses over an abandoned rail line and a three-span continuous concrete open spandrel arch bridge over the Grand River in Brantford, Ontario. The bridge was originally built in 1924. It has spans of approximately 41.8, 46.9 and 41.8 metres for a total deck length of approximately 130.5 metres. The superstructure is founded on two cast-in-pace concrete piers and two abutments. The travel width of the structure is 17.4 metres and the overall width of the structure is 22.9 metres. The roadway accommodates two lanes of traffic in each of the eastbound and westbound directions. There is a 2.1-metre-wide sidewalk on both the north and south sides. The single span concrete box girder structure was originally used for a railway grade separation and was constructed in 1924 and replaced in 1980. The abutment of the original structure was left in place.

Previous rehabilitation works were completed on this structure including deck replacement (1980); east expansion joint replacement (1988) and west expansion joint replacement (1994).

3.2 Brant’s Crossing Bridge

The Brant’s Crossing Bridge was originally part of the Brantford, Norfolk, and Port Burwell Railway (BN&PBR). The bridge was constructed between 1912 and 1913 and was in operation for rail transport into the 1980s, when it was purchased by the City. It was then converted to a pedestrian and cyclist bridge, remaining in this function until 2018 when it was closed after an ice-dam event. The superstructure is founded on three cast-in-pace concrete piers and two abutments. The total deck length of the bridge is about 121.4 metres with four spans of approximately 23.3, 37.4, 37.4 and 23.3 meters in length. The roadway width of the structure is 2.5 metres and the overall width of the structure is 5.8 metres.

The bridge is oriented east-west and situated at the northern end of a relatively straight section of the river, at a point of a slight bend from a southeasterly to a southerly flow. The view up-river to the north is of the City's downtown and the Lorne Bridge, which carries Colborne Street West across the river. The view down-river to the south is of the TH&B Railway Bridge and beyond to the Veterans Memorial Parkway bridge before the river bends further west and out of sight.

3.3 TH&B Railway Bridge

Based on the information provided, we understand that the TH&B Railway Bridge was constructed in 1890. The bridge consists of four spans which are comprised of a series of floor beams and stringers supported by two through plate girders. The superstructure is founded on one cast-in-place concrete pier (west pier), two steel tube pile piers (centre and east piers) and two abutments. The total deck length of the structure is about 124.8 metres with each span approximately 31.2 metres in length. The roadway width of the structure is 5.4 metres and the overall width of the structure is 5.8 metres. The TH&B Railway Bridge currently functions as a pedestrian bridge.

The TH&B Railway Bridge is oriented east-west and the view up-river to the north is of the City's downtown, the Brant's Crossing Bridge and the Lorne Bridge, which carries Colborne Street West across the river. The view down-river to the south is of the Veterans Memorial Parkway Bridge before the river bends further west and out of sight.

4.0 SUBSURFACE CONDITIONS

4.1 Sources of Subsurface Information

Existing geotechnical information for the area of the bridge site readily available from our files and provided by GMBP and the City was compiled and reviewed. The information consisted of topographical mapping, aerial mapping, soils and bedrock mapping, geological data and site-specific geotechnical data from previous site investigations carried out on or adjacent to the site. The previous site investigations are identified as follows:

Site 1: Lorne Bridge

- Lorne Bridge original construction plans/drawings;
- Dominion Soil Investigation Inc. Report No. 78-4-K2 titled "Subsurface Investigation, Proposed Reconstruction of Lorne Bridge, Brantford, Ontario", dated May 1978;
- Peto MacCallum Ltd. Report No. 89 F 251 titled "Geotechnical Investigation, Foundation Subgrade Evaluation, Lorne Bridge, Brantford, Ontario", dated 1986;
- Golder Report No. 881-3443 titled "Geotechnical Investigation, Proposed Brantford Flood Control Works, Lorne Bridge to Market Street, Brantford, Ontario", dated January 1989; and
- Golder Report No. 881-3443-1 titled "Geotechnical Investigation, Proposed Lorne Dam Removal, Brantford, Ontario", dated May 1989.

Site 2: Brant's Crossing Bridge

- Brant's Crossing Bridge original construction plans/drawings;

- Golder Report No. 881-3443 titled "Geotechnical Investigation, Proposed Brantford Flood Control Works, Lorne Bridge to Market Street, Brantford, Ontario", dated January 1989; and
- Golder Report No. 881-3443-1 titled "Geotechnical Investigation, Proposed Lorne Dam Removal, Brantford, Ontario", dated May 1989.

Site 3:TH&B Railway Bridge

- TH&B Railway Bridge No. 75 original construction plans/drawings; and
- Golder Report No. 881-3443 titled "Geotechnical Investigation, Proposed Brantford Flood Control Works, Lorne Bridge to Market Street, Brantford, Ontario", dated January 1989.

The locations of boreholes used in the development of this report are illustrated on Figure 1. Copies of relevant borehole records from the previous subsurface explorations are included in Appendix B.

4.2 Site Geology

The general site is located in the physiographic region of Southwestern Ontario known as the Norfolk Sand Plain. In this area, the Grand River flows through former glacial spillways which were previously eroded into the underlying till plains. The surficial soils along the subject sections of the bridge sites consist of more recently deposited alluvial deposits of gravel, sand and silt.

Bedrock in the surrounding area of the site consists of limestone, dolostone, and shale belonging to the Salina Formation of the upper Silurian Age. Previous drilling at the bridge locations encountered bedrock between elevation 194.0 metres and 196.0 metres in the boreholes, as shown on the Record of Borehole sheets in Appendix B.

4.3 Subsurface Stratigraphy

Details of the subsurface conditions encountered in the boreholes drilled in 1978, 1986 and 1989 are provided in Appendix B. It should be noted that the soil conditions indicated on the Record of Borehole sheets have been inferred from non-continuous sampling and observations of drilling resistance and have been simplified for the purpose of geotechnical design. The boundaries shown typically indicate transitions from one soil type to another and should not be interpreted to indicate exact planes of geological change. Further, post-exploration construction activities may have altered the subsurface conditions from those shown on the Records of Boreholes.

4.3.1 Lorne Bridge

In 1986, Peto MacCallum Ltd. (Peto) drilled four boreholes 1, 2, 3 and 4A/4B at the locations shown on Figure 1. The boreholes were located at each of the abutments and piers. Borehole 1 was drilled at the east abutment (railway underpass) and was terminated at a depth of 17.3 metres below the road surface due to the poor rock quality, difficult drilling and progressed only 200 millimetres into the underlying bedrock.

Boreholes 2 and 3 were drilled at the west abutment and east pier, respectively, and penetrated approximately 1.2 and 1.0 metres into the underlying rock and terminated at depths of about 15.5 and 17.6 metres below the bridge deck. Borehole 4A/4B was drilled at the west pier and was terminated due to drilling difficulties at a depth of about 6.1 metres below the bridge deck. The drawings of the original bridge show that the abutments and piers were to be founded on bedrock at elevations of 193.26 to 193.87 metres.

Based on boreholes 1, 2, 3, and 4A/4B and our review of the construction drawings, we understand that the east and west abutments, as well as the east pier, where fully penetrated by the boreholes and are founded on limestone bedrock at the following levels:

Location – Lorne Bridge	Design Founding Elevation (metres)	Actual Founding Depth (metres below bridge deck or pavement)/Elevation (metres)
East Abutment (Borehole 1)	193.87	17.1/194.0
East Pier (Borehole 3)	193.87	16.6/194.5
West Pier (Borehole 4A/4B)	193.57	Not defined during drilling. Based on the construction drawings, it is anticipated that the west pier is founded on bedrock.
West Abutment (Borehole 2)	193.26	14.3/194.0

In 1988, Golder drilled two boreholes (boreholes 19 and 20) adjacent to the east abutment of the Lorne Bridge at the approximate locations shown on Figure 1. Borehole 19 was drilled adjacent to the east abutment and encountered some 5.0 metres of sand and gravel fill overlying reinforced concrete in which the borehole was terminated due to difficult drilling. Borehole 20 was drilled immediately east of borehole 19 in an attempt to avoid the reinforced concrete. Borehole 20 encountered an approximately 0.9-metre-thick layer of sand and gravel fill overlying strata of clayey silt, silty clay and silt. Bedrock was encountered at a depth of about 7.5 metres below ground surface or at an elevation of about 194.56 metres.

In addition to the above, boreholes 7, 8 and 11 were drilled at, or near, the east abutment and borehole 9 was drilled at the west abutment by Dominion Soil Investigation Inc. in 1978 at the approximate locations shown on Figure 1.

Boreholes 7 and 8 were drilled adjacent to the abandoned railway tracks where they pass under the Lorne Bridge. Subsurface soils encountered in these boreholes consisted of firm to very stiff clayey silt overlying very stiff to hard silt till layers which extended to depths of about 7.6 and 9.0 metres below the track surface or to elevations of about 194.5 and 193.5 metres, respectively. At these elevations, both boreholes encountered auger refusal on the inferred limestone bedrock surface. Borehole 11 was drilled behind the bridge abutment wall and encountered compact to dense sand and gravel fill extending to a depth of about 4.3 metres below surface, which was underlain by a stratum of stiff grey silt with clay seams. Borehole 11 was terminated in silt with clay seams after exploring the stratum for about 0.8 metres.

Borehole 9 was drilled at the bridge’s west approach within the low-lying west bank of the river. Fill materials with a thickness of about 4.5 metres were encountered beneath the asphalt and concrete pavement. Below the fill, strata of fine sand and silty sand were encountered overlying compact fine sand with gravel.

Groundwater levels at the bridge abutment and pier locations generally corresponded to the river water level at the time of the previous explorations. Groundwater levels should be expected to fluctuate seasonally and rise during snow melt and wet periods of the year.

4.3.2 Brant’s Crossing Bridge (Former Railroad Bridge)

In 1988, Golder drilled five boreholes, boreholes 15, 16, 17, 18 and 22, as part of the nearby dam removal program at the Brant’s Crossing Bridge site at the approximate locations shown on Figure 1.

Boreholes 15, 16, and 17 were drilled through the west, centre, and east piers. Borehole 22 was drilled through the east abutment and borehole 18 was drilled immediately behind the east abutment.

In borehole 15, an approximately 0.5-metre-thick layer of concrete was encountered overlying the limestone block masonry which is understood to comprise the former abutment of a former structure. The limestone block masonry was fully penetrated to a depth of about 6.7 metres below the top of the pier. Beneath the masonry, a 0.4-metre-thick layer of hard silty clay was encountered. In boreholes 16, 17 and 22, cast-in-place concrete was encountered for the full height of the piers and abutment.

Borehole 18 was drilled behind the east abutment and encountered some 5 metres of inferred very loose sand fill material overlying an approximately 1.5-metre-thick layer of sand and gravel, and sand strata of silt, clayey silt and silty clay.

Bedrock was encountered beneath the silty clay in boreholes 15 and 19 and underlying the concrete in boreholes 16, 17 and 22. The bedrock surface was encountered between elevations of about 194.3 metres and 195.6 metres in the boreholes. The bedrock generally consisted of fresh, massive, grey to light grey, fine-grained limestone.

Groundwater levels at the bridge abutment and pier locations generally corresponded to the river water level at the time of the previous exploration and are expected to fluctuate seasonally and rise during snow melt and wet periods of the year.

4.3.3 TH&B Railway Bridge

No site-specific geotechnical information (specific boreholes at abutment or pier locations) was available for the TH&B Railway Bridge. Based on our review of the drawings, available reports and site reconnaissance, we understand that the substructure at this bridge location consists of one cast-in-pace concrete pier (west pier), two steel tube pile piers (centre and east pier) and two abutments. Based on the general arrangement drawings (from 1901) for the piers and abutments, we understand that the existing abutments and pier foundations are supported directly on bedrock at depths ranging between about 9.4 and 12.1 metres below the bridge deck (the bedrock surface slopes downward from west to east) or at approximately elevations 194.3 metres to 191.6 metres.

In 1988, Golder advanced two boreholes, boreholes 5 and 6, at the north and south sides of the existing west abutment as part of the flood control works at the approximate locations shown on Figure 1. The subsurface soil conditions encountered in boreholes 5 and 6 consisted of fill and topsoil overlying strata of sand, sand and gravel, silt, silty clay and clayey silt till. Boreholes 5 and 6 were terminated at depths of about 8.1 and 8.2 metres below the ground surface or at elevations of about 194.1 and 194.2 metres, respectively.

Groundwater levels at the abutment and pier locations are anticipated to generally correspond to the river water level and should be expected to fluctuate seasonally and rise during snow melt and wet periods of the year.

5.0 SEISMIC SITE CLASSIFICATION

Subsurface ground conditions for seismic site characterization were established based on the results of the previous boreholes drilled in the vicinity the site. Based on the anticipated foundation levels on/within the limestone bedrock, the site may be classified as Site Class C (very dense soil and soft rock) in accordance with Table 4.1 of the Canadian Highway Bridge Design Code (CHBDC) (2014).

Based on the locations of the bridges, the reference Site Class C spectral acceleration values were obtained based on the 5th generation seismic hazard maps published by the Geological Survey of Canada (GSC).

In accordance with Section 4.4.3.4 of the CHBDC (2014), the peak ground velocity (PGV), peak ground acceleration (PGA) values and design spectral acceleration (Sa) values for Site Class C are presented below.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGV (m/s)	0.026	0.041	0.070
PGA (g)	0.032 g	0.054 g	0.096 g
Sa (0.2) (g)	0.055 g	0.089 g	0.153 g
Sa (0.5) (g)	0.035 g	0.054 g	0.088 g
Sa (1.0) (g)	0.020 g	0.031 g	0.048 g
Sa (2.0) (g)	0.009 g	0.015 g	0.024 g
Sa (5.0) (g)	0.002 g	0.003 g	0.006 g
Sa (10.0) (g)	0.001 g	0.001 g	0.002 g

Given the Site Class C designation, the values presented above do not need to be factored using site-specific acceleration or velocity coefficients.

6.0 DISCUSSION

6.1 General

This section of the report provides our interpretation of the available geotechnical data and it is intended for the guidance of the design engineer for conceptual design within the context of the overall EA Study. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project.

It is understood that as part of the EA, consideration is being given to rehabilitating the existing bridge structure(s) and or full replacement(s).

Based on our review of the information provided and the subsurface information, we understand the following:

- The Lorne Bridge currently carries traffic on Colborne Street West across the Grand River with a 30-tonne load limit in the winter;
- The Brant's Crossing Bridge was closed in February 2018 following a flooding and ice jam event; the bridge formerly carried pedestrian and cyclist traffic over the Grand River and would require structural repairs in order to be re-opened; and
- The TH&B Railway Bridge currently carries pedestrian and cyclist traffic over the Grand River and has been identified as requiring structural repairs to maintain the existing crossing.

The preliminary ranges of the geotechnical design parameters provided below have been developed in the absence of current geotechnical test data and are based only on observations of existing structure performance, construction drawings and general subsurface soil condition descriptions. During future studies, a better understanding of loads on the existing piers and abutments will allow development of more refined lower bounds for the geotechnical design parameters. If an existing bridge is to be rehabilitated, the existing abutment and pier footings should be acceptable based on geotechnical considerations, pending site specific geotechnical testing, structural evaluations, and consideration of future load changes, if any. Future inspections and repairs should focus on the conditions of the existing footings, if visible, and ice and scour protection. If the superstructure of a bridge is to be replaced, consideration may be given to reuse of the existing footings with minor structural and erosion protection repair works or construction of new footings. The geotechnical aspects of these options are discussed in more detail below.

6.2 Existing Foundations

Golder has not carried out any current intrusive exploration, testing or underwater assessment of the abutment and pier foundations at three bridge locations as part of this study. The details of the bridge structures including dimensions, material type and conditions were summarized and provided in the Lorne Bridge, Brant's Crossing Bridge and TH&B Railway Bridge enhanced OSIM Inspection Reports, respectively. We understand that GMBP and the City are aware of these reports and results.

Based on the existing information from the previous geotechnical explorations, OSIM inspection reports and design drawings, it would appear that the spread footing or steel tube pile foundations for all of the abutments and piers were founded on bedrock at the three bridge sites. The foundations were designed using a factored Ultimate Limit States (ULS) factored resistance of 2,500 kilopascals (kPa). The Serviceability Limit States (SLS) geotechnical reaction for 25 millimetres of settlement generally exceeds the ULS factored resistance for foundations bearing on sound bedrock.

We recommend that for structural evaluation of these abutment and pier foundations for superstructure replacement and rehabilitation without replacement of the substructure with minor structural (including erosion) and aesthetic repairs at three bridges, a factored ULS resistance of up to 3,000 kPa may be used provided that the abutment and pier foundations are in good condition. As noted above, the geotechnical reaction at SLS (for 25 millimetres of settlement) may be assumed to exceed the ULS resistance.

6.3 New Foundations

The design drawings for the three bridges indicate that the abutments and piers have been founded on bedrock. Based on the subsurface conditions encountered during the previous geotechnical explorations carried out in the vicinity of the project site, both shallow and deep foundation options have been considered for the replacement of the bridges.

Based on the information obtained from the previous boreholes, the overburden soils are not considered suitable to support new foundation structures at the bridge sites. Both shallow and deep foundation options bearing on the limestone bedrock underlying the overburden materials have been considered for support of the new bridge(s). Driven steel H-piles have not been considered for support of any of the foundations at these bridge sites due to their lower cross-sectional area compared to drilled shafts and potential installation difficulties. Further, sufficient lateral resistance may not be developed given the relatively shallow depth of overburden (i.e., less than 8 metres). The following provides a discussion on the foundation options (shallow and deep) with regard to constructability and risks associated with respect the structures(s):

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability /Risk
Spread/strip footings founded on native soil deposits	Not feasible for support of piers or abutments due to the unknown thickness and low strength of the overburden soils and impractical considering the depth to bedrock.			
Spread/strip footings founded on competent bedrock	• Feasible for support of the piers; however, requires cofferdams to complete excavation and footing construction in the dry adjacent to/within the river. • Feasible for support of the abutments; however, requires temporary roadway protection and may require temporary protection of existing bridge footings, where the new footings are not constructed at the same elevation as the existing adjacent footings.	• Higher geotechnical resistance than for shallow foundations bearing on native soil deposits. • Straightforward method of construction; however, extensive excavation, including into weathered bedrock, required.	• Significant excavation depths through existing fills and native soils and into bedrock required. • At the abutments, temporary roadway protection systems required along the edge of existing bridge approach embankments • Groundwater control (cofferdams) required at pier locations and may be difficult to “seal” cofferdam at the bedrock surface or prevent upward seepage inflows; mitigation measures such as concrete plug placed by tremie methods may be required. • Precludes use of integral abutments, although may permit semi-integral abutments; potentially greater maintenance required at abutments. • Greater volume of excavation spoil; and concrete than for deep foundations. • More groundwater control required than for deep foundations option.	• Conventional excavation and construction techniques. • Risk of groundwater / river water inflow through gaps at the cofferdam-bedrock interface or through bedrock fractures at the piers.

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability /Risk
Steel H-piles founded on shale bedrock; Steel tube (pipe) piles on shale bedrock	• Not feasible for support of the piers or abutments due to the shallow depth to bedrock, low lateral support from the overburden soils, and potential obstructions/difficult driving conditions. • Would require pre-drilling and socketing steel piles into concrete filled rock sockets in bedrock.			
Caissons (drilled shafts) socketed into bedrock	• Feasible for support of abutments and piers.	• Higher bearing resistances than for steel H-piles, requiring fewer pile elements. • Will result in less volume of excavation and spoils generation than for spread footing option. • At abutments, may reduce temporary protection system requirements, in particular, the need for dowelling to pre-support excavation face. Minor groundwater seepage anticipated within pile cap excavation - pumping from filtered sumps will provide adequate groundwater control. • At piers, may result in smaller footprint/working area than for spread footing option and overall volume of excavation may be less; will also reduce cofferdam and groundwater control requirements. • Pile caps could potentially be eliminated if the pier columns extended directly up from the top of the drilled shafts.	• Permanent liners would be required during construction to control potential ground loss in overburden soils and to mitigate groundwater (and river water) inflows. • More expensive coring/churn drilling required to form bedrock socket through the inferred limestone bedrock at the sites. • Precludes use of integral abutments. • The rock socket is required to be cleaned (potentially by airlift methods) and inspection with a video camera would be required. • Concrete would have to be placed by tremie methods.	• Conventional construction methods for caisson foundations; temporary or permanent liners required for ground and groundwater control. • Restriction of use of barge/floating platform in the river due to low levels during summer seasons.

6.3.1 New Shallow Foundations on Bedrock

Shallow foundations comprised of strip/spread footings founded on the slightly weathered to fresh limestone bedrock at the three bridge locations are feasible for support of the abutments and piers. However, significant depths of excavation (ranging from about 8 to 10 metres deep at the abutments and piers) through existing fills, native soils and into the bedrock will be required. The deep excavations at the piers would require cofferdams extending to, and sealed into, bedrock to reduce groundwater/river water inflows. Excavations at the abutments will be achievable and groundwater control will be easier to manage; however, temporary protection systems will be required to support the adjacent roadway approach embankments and additional protection measures may be required to ensure adequate support of the existing abutment foundations. Based on the above, shallow footings on bedrock are considered a suitable foundation alternative for support of new abutments, but not for the piers.

In the absence of current geotechnical test data, relatively conservative values of the engineering properties of the founding soils have been estimated. A factored geotechnical resistance at ULS ranging from 2,500 to 3,000 kPa may be used for conceptual design of shallow foundations placed on bedrock at the bridge locations. The SLS reaction does not generally apply to the design of foundations on the bedrock, provided the bedrock surface is properly cleaned of soil and highly weathered bedrock at the time of construction. For the ULS sliding resistance of a cast-in-place footing placed on the limestone bedrock, an unfactored friction coefficient of 0.7 can be used for dead and sustained live loads.

All footings would have to be provided with a minimum of 1.2 metres of earth cover or a thermally equivalent thickness of insulation for frost protection purposes. Construction of spread foundations for the piers would require installation of cofferdams and the use of either subaqueous excavation and construction techniques or significant dewatering/unwatering work. Scour protection would also be required. Therefore, while spread foundations are feasible, this foundation type may not be practical for new piers.

6.3.2 New Deep Foundations – Caissons (Drilled Shafts)

Deep foundations may be used for support of new abutments and piers. Drilled shafts (caissons) socketed into the limestone bedrock are considered feasible and the preferred option for the support the new bridge foundations at all locations.

Caissons socketed approximately two diameters or greater into the bedrock should be designed based on end-bearing resistance using a factored axial resistance at ULS of 2.5 MPa for preliminary design purposes. For a 1.5-metre diameter caisson, this would equate to a factored axial resistance at ULS of about 5,000 kilonewtons (kN). The SLS geotechnical reaction for 25 millimetres of settlement does not apply to rock-socketed caissons founded the limestone bedrock since the SLS reaction would be greater than the factored axial resistance at ULS.

It is recommended that the abutments and piers be supported on drilled shaft foundations socketed into the fair quality limestone bedrock to a minimum depth equal to two times the diameter of the drilled shaft below the bedrock surface. A steel liner extending into the bedrock surface will be required in order to prevent “necking” of the concrete. Water inflow into the drilled shafts should be expected given the proximity to the Grand River; therefore, placement of concrete by the tremie method will be required to install drilled shafts.

The centre to centre spacing between the drilled shafts should be greater than 2.5 times the drilled shaft diameter to limit interaction between drilled shafts. Provided this minimum drilled shaft spacing within a group is maintained, the efficiency factor for the pile group is expected to be 1.0 (i.e., no reduction for group effects is required).

6.4 Excavations and Groundwater Control

Excavations for shallow foundations and for pile caps may encounter near surface fills and or the existing pavement structure, topsoil, clay, silt, sand, and gravel. Based on the age of the existing structures and various historical activities that are known that have taken place at the sites, it is also possible that excavations may encounter existing foundations, remnants of temporary structures used during the original construction or other rubble.

Based on the historic water levels at the bridge locations and depending on the time of year construction is carried out, excavations for the abutments may extend below the groundwater level. The water level should be expected to fluctuate seasonally and due to climatic variations. Excavations for pier foundations will extend below the river and groundwater levels and groundwater flow from the native granular soils should be expected. A Permit to

Take Water would likely be required. Excavations for piers at all three bridge locations will require coffer dams in order to carry out construction.

Temporary protection systems will be required along the existing bridge foundations and roadways to facilitate safe construction of the new abutment foundations and maintain operation of the existing adjacent roadways and bridges.

The existing and proposed pier foundations are located within the Grand River and below the normal river water level and as such, the pier foundations could experience some erosion/scour throughout the design life for the structures if the adjacent soils are eroded. Scour protection should be provided around the pier foundations for the bridge structure. Riprap should be provided on the river channel banks and around the piers. The riprap should extend from the channel banks to at least 1.0 metres above the design flood level at the structure locations.

7.0 ADDITIONAL GEOTECHNICAL INPUT

This report has been prepared to assist with geotechnical aspects that may influence planning, early conceptual design, and early cost estimating in support of the Environmental Assessment for this project. A detailed and thorough geotechnical exploration and confirmation of the abutment geometry, backfill conditions, existing pavement structures and the like (to the extent practicable) and areas that might include future retaining walls will be required for each bridge location during subsequent phases of design to address and refine all geotechnical issues discussed within this report.

Signature Page

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
Feb. 22/2021
M. A. SWALLOW
PROVINCE OF ONTARIO

Mark A. Swallow, P.E., P.Eng.
Principal and Senior Practice Leader

VN/MAS/vf/cr

https://golderassociates.sharepoint.com/sites/113598/project files/6 deliverables/geotechnical/19128292-1000-r01-rev0 geo assessment-three grand river crossings_22feb2021.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Ground Water Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

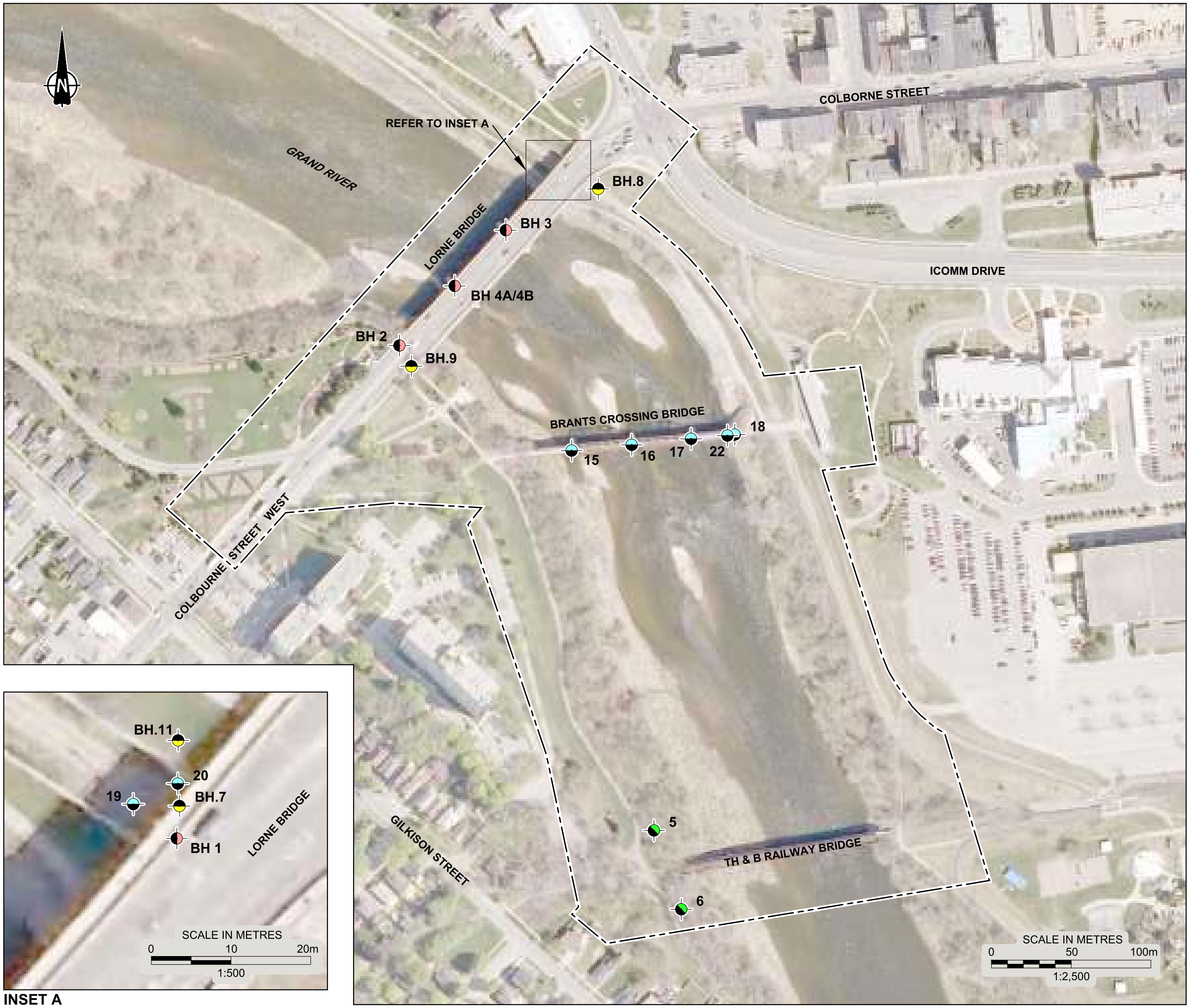
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client’s expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder’s report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder’s report.

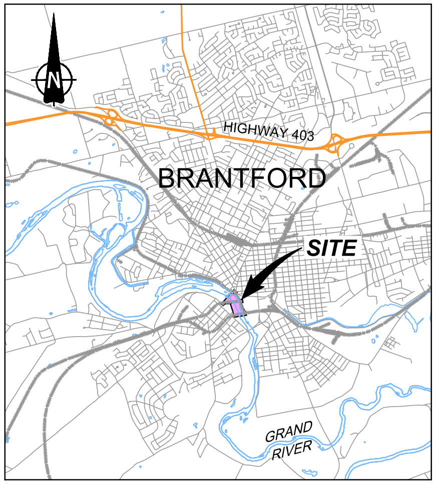
During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder’s report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder’s report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder’s responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



INSET A



KEY PLAN

LEGEND

- - - - APPROXIMATE LOCATION OF STUDY AREA

BOREHOLES (PREVIOUS GOLDER EXPLORATIONS):

- ◐ 881-3443-1
- ◐ 881-3443

BOREHOLES (BY OTHERS):

- ◐ PETO MACCALLUM TLD 1986
- ◐ DOMINION 1978

REFERENCE

DRAWING BASED ON THE GRAND RIVER CONSERVATION AUTHORITY 2006 ORTHOGRAPHIC IMAGERY;
PETO MACCALLUM LTD., FOUNDATION SUBGRADE EVALUATION REPORT, DECEMBER 9 - 1986;
DOMINION SOIL INVESTIGATION INC., REPORT ON SUBSURFACE INVESTIGATION, MAY 1978; AND
CANMAP STREETFILES V2008.4.

NOTES

THIS DRAWING IS SCHEMATIC ONLY AND IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

ALL LOCATIONS ARE APPROXIMATE.

PROJECT

GEOTECHNICAL ASSESSMENT
THREE GRAND RIVER CROSSINGS EA
BRANTFORD, ONTARIO

TITLE

LOCATION PLAN

GOLDER

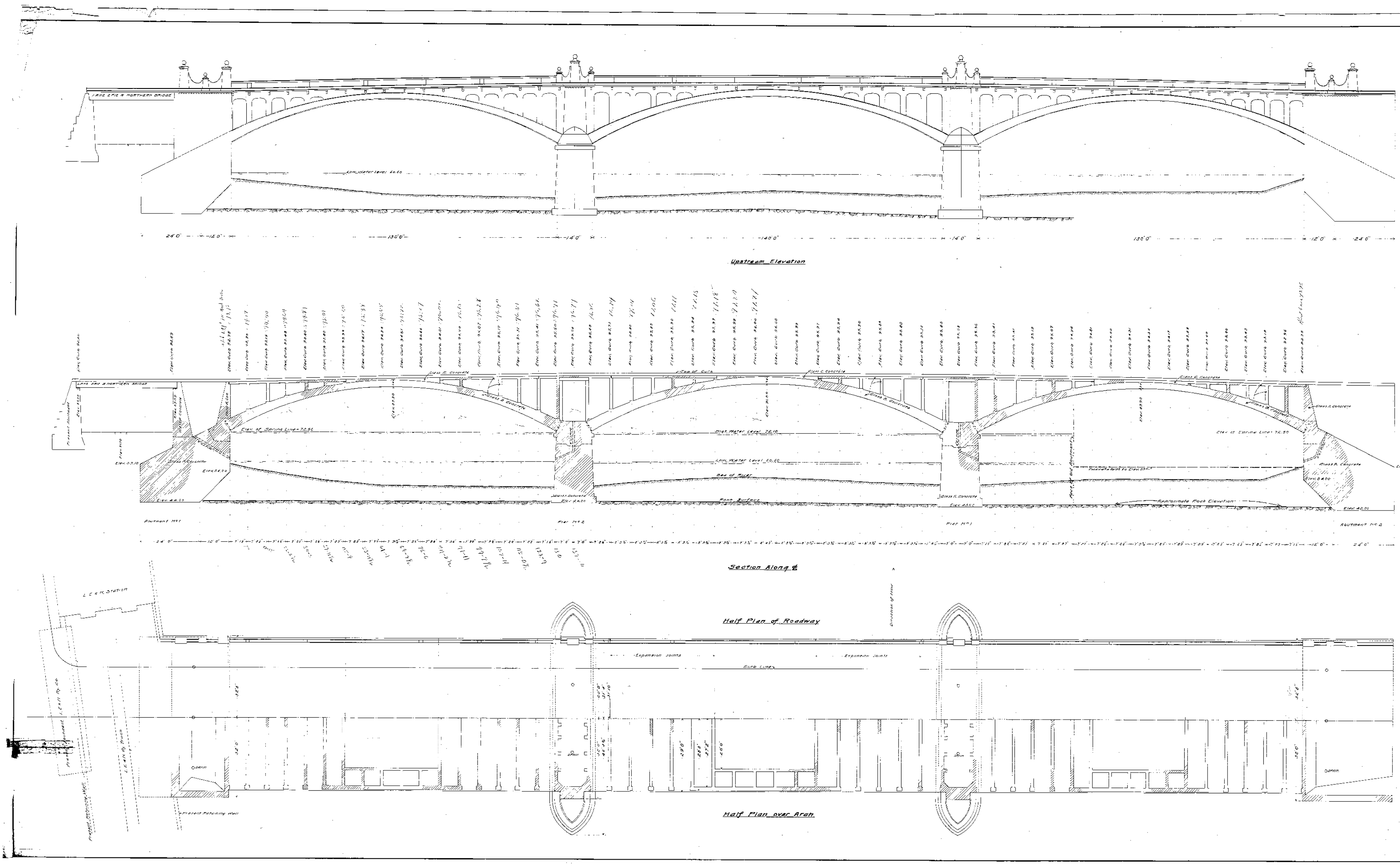
PROJECT No.	19128292	FILE No.	19128292-1000-R01001
		SCALE	AS SHOWN
CADD	DCH	May 22/20	REV.
CHECK			

FIGURE 1

Drawing file: 19128292-1000-R01001.dwg May 22, 2020 – 4:57pm 0 25mm Original Format is Tabloid 279mm x 432mm Client: Client: GM Blueplan Engineering Limited

APPENDIX A

Original Design and Relevant Bridge Structure Drawings



LORNE BRIDGE
BRANTFORD ONT.
GENERAL ELEVATION, SECTION,
AND PLAN.

SCALE. 1 IN. = 10 FT. | DATE. Feby 26 1925.

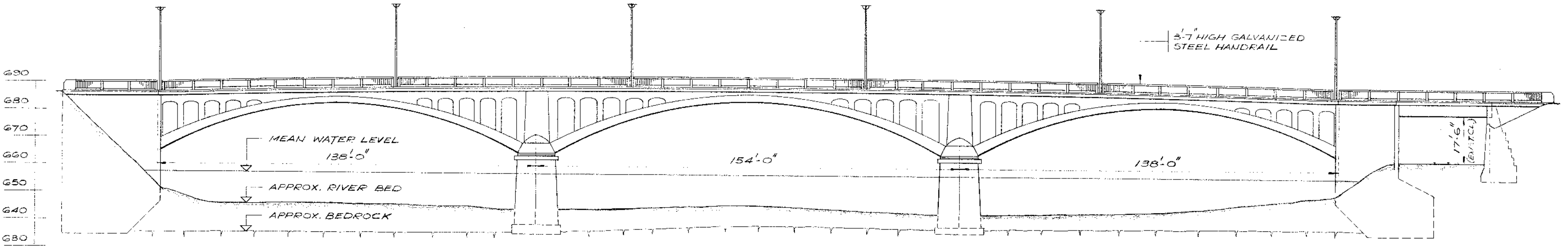
F. P. Adams
CITY ENGINEER

DRAWN BY:
E. T. McLaren

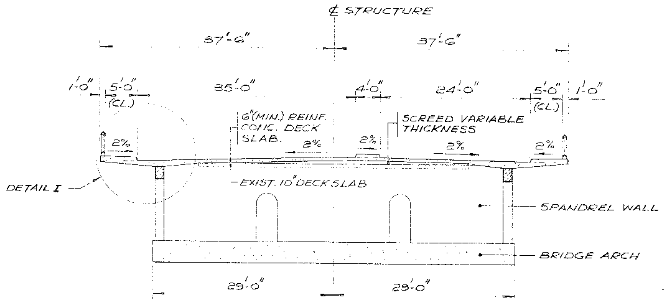
No. LB. 5.

FEB X 1972

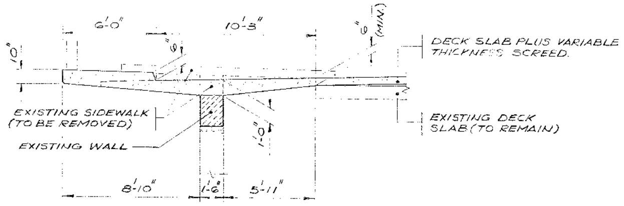
1-833



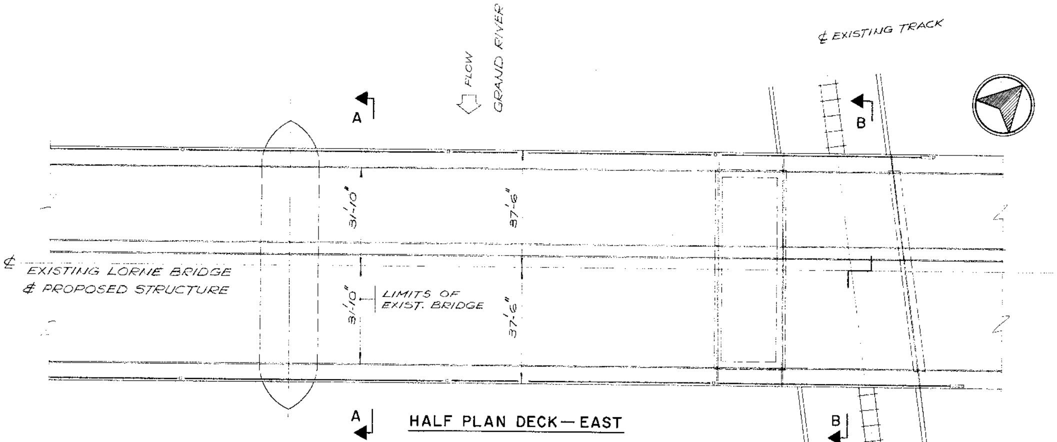
SOUTH ELEVATION – PROPOSED BRIDGE



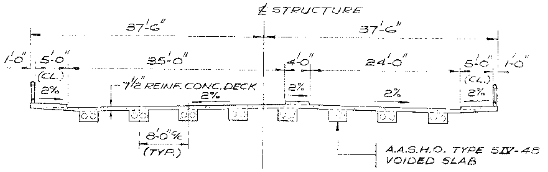
SECTION A-A



DETAIL I



HALF PLAN DECK – EAST



SECTION B-B

PRINT RECORD		
lo.	FOR	DATE
1	MEETING 2.6.72	1.6.72

CONSULTING ENGINEERS AND PLANNERS
DAMAS AND SMITH LIMITED

REV.	BY	DATE	DESCRIPTION

CITY OF BRANTFORD

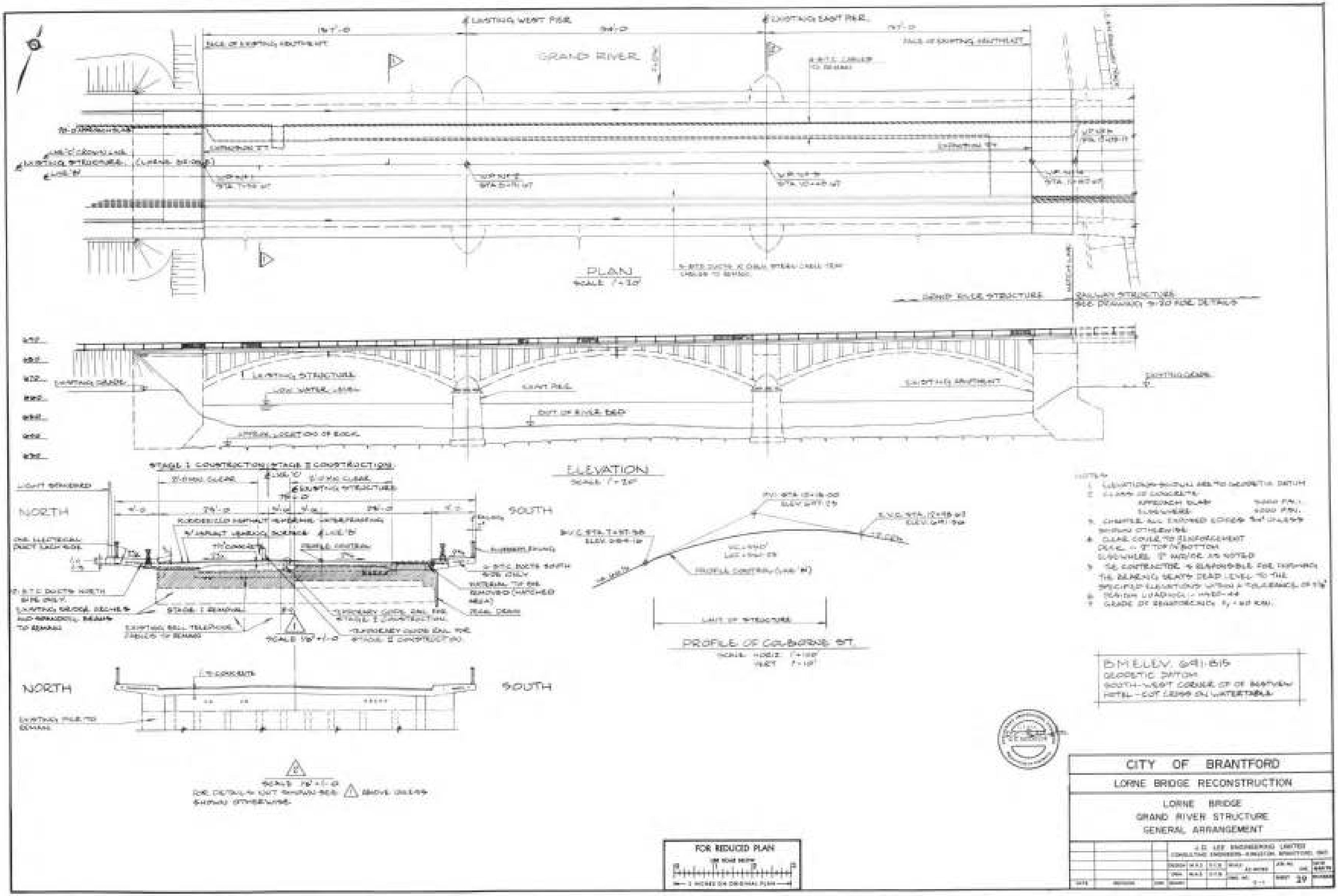
PROPOSED LORNE BRIDGE IMPROVEMENTS

WIDENING TO FIVE LANES

DRAWN R.A.P.
CHECK B.F.
DATE FEB.1972
ACTIVITY 717
SCALES: ELEVATION, PLAN: 1"=20' SECT'S. 1"=10', DETAIL 1¼"=1'
DRAWING No. 7102-F002

E-21

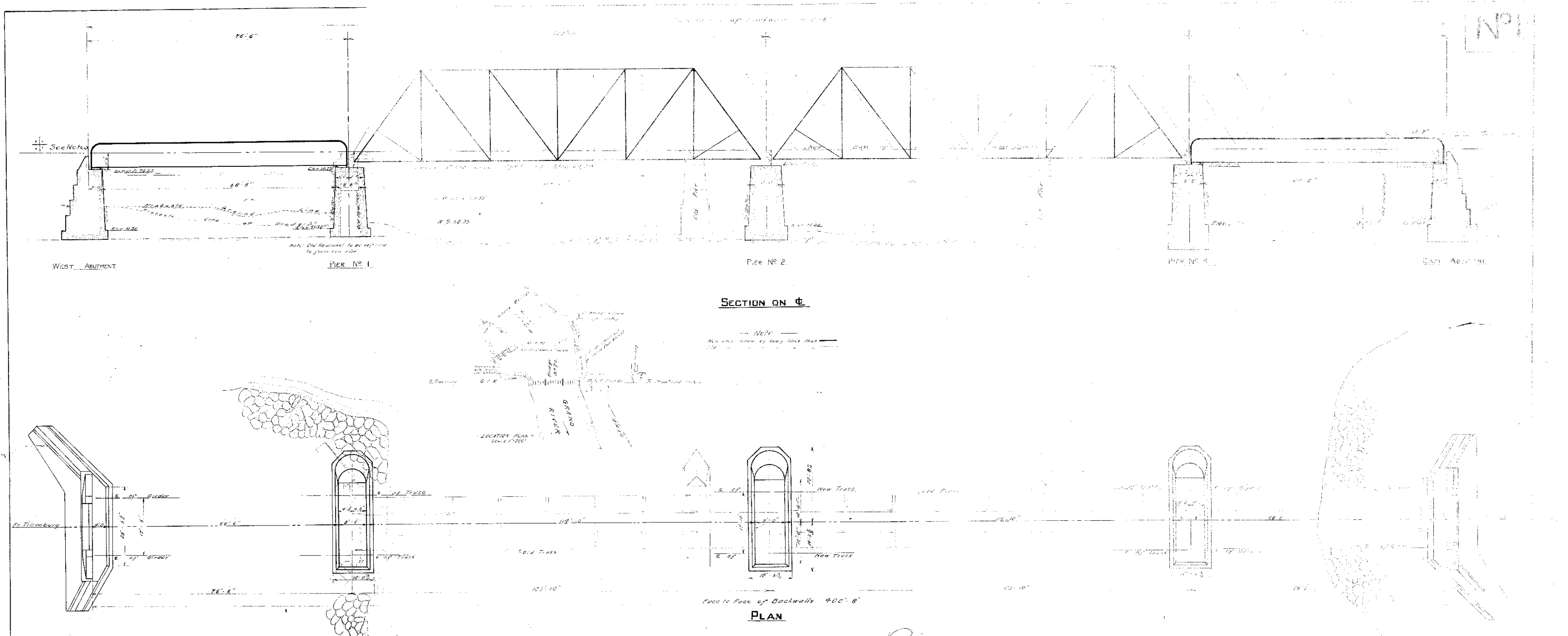
1-1491-4



NOTES
1. ELEVATIONS SHOWN ARE TO GEODETIC DATUM
2. CLASS OF CONCRETE
   APPROACH SLAB 5000 P.S.I.
   ELSEWHERE 4000 P.S.I.
3. CHAMFER ALL EXPOSED EDGES 3/4" UNLESS SHOWN OTHERWISE
4. CLEAR COVER TO REINFORCEMENT DECK = 2" TOP 1½" BOTTOM ELSEWHERE 2" AND/OR AS NOTED
5. THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITHIN A TOLERANCE OF 1/8"
6. DESIGN LOADING - HS20-44
7. GRADE OF REINFORCING f_y = 60 KSI.

B.M. ELEV. 691.815
GEODETIC DATUM
SOUTH-WEST CORNER OF [illegible] HOTEL - CUT CROSS ON WATERTABLE

CITY OF BRANTFORD
LORNE BRIDGE RECONSTRUCTION
LORNE BRIDGE
GRAND RIVER STRUCTURE
GENERAL ARRANGEMENT
J.D. LEE ENGINEERING LIMITED
CONSULTING ENGINEERS [illegible] BRANTFORD, ONT.
SCALE AS NOTED
SHEET 29



Examined R. [illegible] Masonry Engineer

Approved H. R. [illegible] Chief Engineer

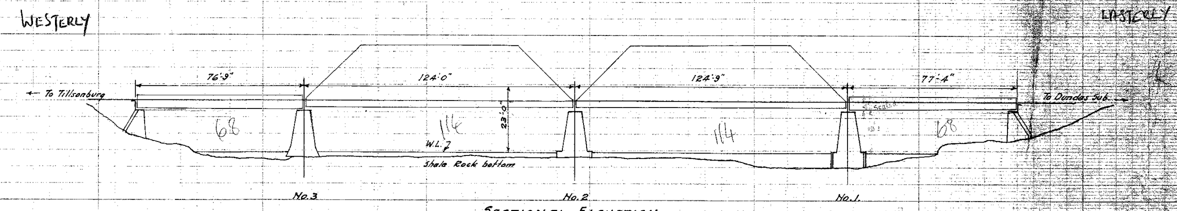
Vice President

GRAND TRUNK RAILWAY
MIDDLE DIVISION 20th DISTRICT
BRIDGE No. 75 — MILE 87
GRAND RIVER — BRANTFORD
GENERAL PLAN [illegible]

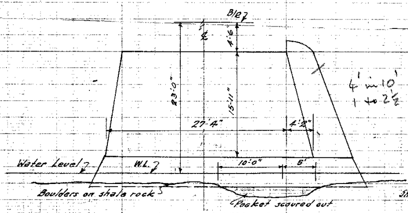
Montreal [illegible]

Amended Aug. 16th, 1912.
No. The Rail Elevation at site is not definitely settled and may be altered slightly.
Rail Elev. 75.30 for Base of rail on truss spans and 74.50 at Abutment ends of Girders accepted by City Engineer and approved. Seat level of Abut. and Piers remains unchanged (El. 70.25)

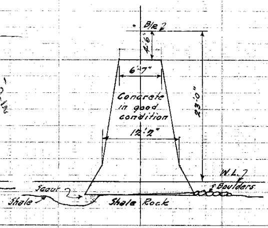
Amended G.J.D.
[illegible] C.H.
[illegible] by R.W.
[illegible] Superstructure [illegible]



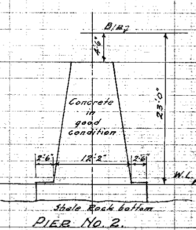
SECTIONAL ELEVATION.
Scale 1" = 30'



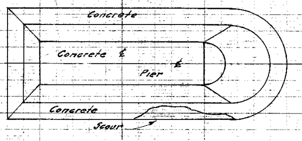
SIDE ELEVATION, PIER No.3



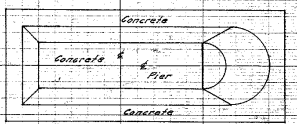
PIER No. 3.



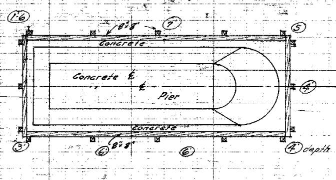
PIER No. 2.



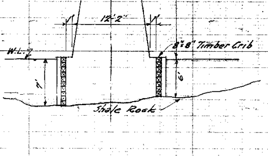
PLAN, PIER No.3.



PLAN, PIER No. 2.



PLAN, PIER No.1.



SECTION, PIER No.1.

SCALE 1" = 10'

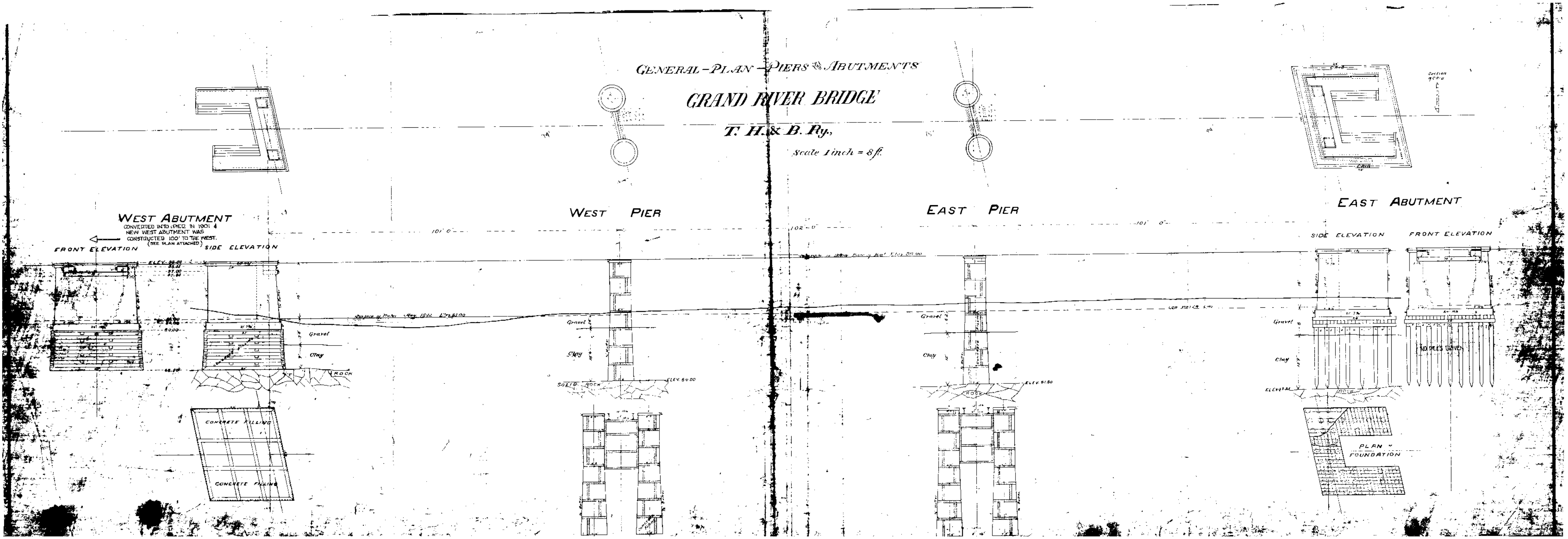
CANADIAN NATIONAL RAILWAYS
CENTRAL REGION
LONDON DIV'N. BURFORD SUBDIV.

BRIDGE M.P. ~~23~~ 9.5
OVER GRAND RIVER
AT BRANTFORD

INFORMATION FROM INSPECTION BY DIVING
CREW MAY 25th 1934

Drawn by H.P.

Inspector



GENERAL-PLAN-PIERS & ABUTMENTS

GRAND RIVER BRIDGE

T. H. & B. Ry.

Scale 1 inch = 8 ft.

WEST ABUTMENT
CONVERTED INTO PIER IN 1901 & NEW WEST ABUTMENT WAS CONSTRUCTED 100' TO THE WEST. (SEE PLAN ATTACHED)

APPENDIX B

Record of Boreholes (Previous Investigations)

RECORD OF BOREHOLE 5

SHEET 1

LOCATION - SEE FIGURE 1

BORING DATE NOV. 29, 1988

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 881-3443

(Golder Report No. 881-3443)

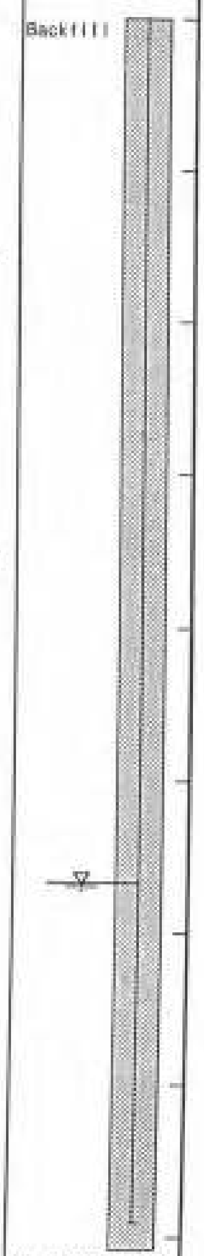
BORING METHOD: POWER AUGER (HOLLOW STEM)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (M)
GROUND SURFACE	202.18
Black silty TOPSOIL	0.00
Black sand & gravel (FILL)	0.15 201.42
Loose black foundry sand occ. gravel (FILL)	0.76 200.81
Loose silty topsoil (FILL)	1.37
Loose brown sandy silt occ. gravel & topsoil (FILL)	1.61 199.90
Loose to very dense black and brown cinders and ash metal, roofing shingles, tar and coal (FILL)	2.28 197.79
Loose grey fine to medium SAND with silty topsoil layers trace organics	4.39 196.75
Compact to very dense brown SAND AND GRAVEL trace to some silt	5.43 194.71
Very dense grey SILT with silty clay layers	7.47 194.10
END OF BOREHOLE	8.08

SAMPLES NUMBER	TYPE	BLOWS/0.3M
1	50 DO	8
2	50 DO	4
3	50 DO	69/250mm
4	50 DO	6
5	50 DO	9
6	50 DO	8
7	50 DO	89
8	50 DO	24
9	50 DO	48
10	50 DO	75

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



WL ENCOUNTERED
AT ELEV. 196.75
DURING DRILLING
NOV. 29, 1988

WL IN STANDPIPE
AT ELEV. 196.61
DEC. 6, 1988

DEPTH SCALE
1 : 50

Golder Associates

LOGGED D.J.M.

CHECKED

RECORD OF BOREHOLE 6

SHEET 1

LOCATION - SEE FIGURE 1
BORING DATE NOV. 29, 1988
DATUM GEODETIC
SAMPLER HAMMER, 63.5kg, DROP, 760mm
PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

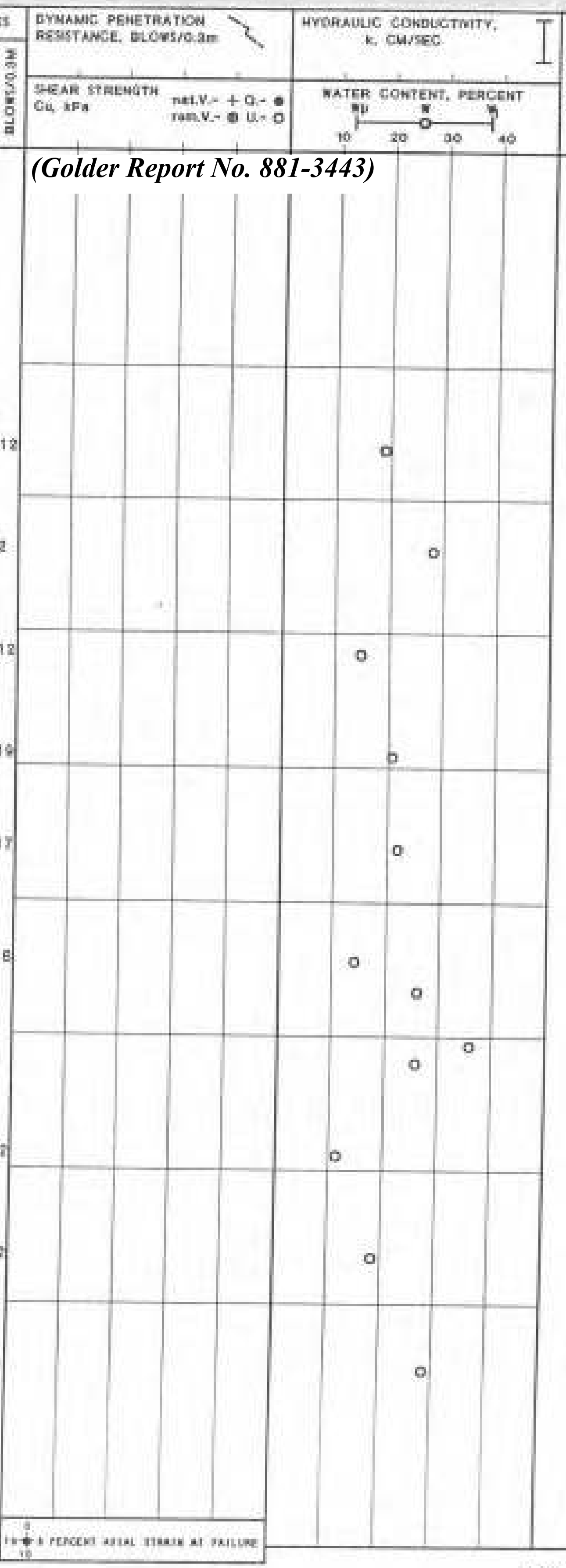
PROJECT 881-3443

(Golder Report No. 881-3443)

BORING METHOD: POWER AUGER (HOLLOW STEM)

DESCRIPTION	ELEV. DEPTH (M)
Black silty TOPSOIL	
GROUND SURFACE	202.44
	0.03
Compact brown sandy silt some gravel occ. metal and glass occ. cloth (FILL)	201.01 1.43
Very loose to compact black and brown cinders and ash with sandy silt layers, glass and shingles (FILL)	197.78 4.66
Compact black SANDY SILT trace organics	196.81 5.63
Loose black SAND occ. silt seams occ. shell fragments	196.40 6.04
Compact to dense grey SAND AND GRAVEL trace silt	194.21 8.23
END OF BOREHOLE	

NUMBER	TYPE	BLOWS/0.3M
1	50 DO	12
2	50 DO	2
3	50 DO	12
4	50 DO	19
5	50 DO	17
6	50 DO	16
7	50 DO	8
8	50 DO	32
9	50 DO	32
10	50 DO	21



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

WL ENCOUNTERED AT ELEV. 196.88 DURING DRILLING NOV. 28, 1988

DEPTH SCALE
1 : 50

Golder Associates

LOGGED D.J.M.
CHECKED

RECORD OF BOREHOLE 15

SHEET 1

LOCATION - SEE FIGURE 1

BORING DATE MAY 3, 1989

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 881-3443-1

(Golder Report No. 881-3443-1)

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat.V.- + Q.- ● rem.V.- ⊕ U.- ○)	HYDRAULIC CONDUCTIVITY, k, CM/SEC / WATER CONTENT, PERCENT Wp W Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		TOP OF WEST PIER		201.90							
	ROTARY DRILLING NXL ROCK CORE	CONCRETE		0.00							
1		Grey LIMESTONE BLOCK MASONRY		201.41 0.49	1	Nx RC	-	T.C.R. 100 % S.C.R. 33 % R.Q.D. 7 %			
2					2	Nx RC	-	T.C.R. 100 % S.C.R. 40 % R.Q.D. 33 %			
3											
4					3	Nx RC	-	T.C.R. 100 % S.C.R. 73 % R.Q.D. 55 %			BOREHOLE GROUTED TO SURFACE FOLLOWING DRILLING
5					4	Nx RC	-	T.C.R. 98 % S.C.R. 83 % R.Q.D. 78 %			
6					5	Nx RC	-	T.C.R. 50 % S.C.R. 13 % R.Q.D. 0 %			
		Hard grey SILTY CLAY with silt layers		195.32 6.58							
7		Fresh, massive, grey to light grey fine grained LIMESTONE small specs of quartz thoughout, some chert inclusions, some shaly partings		194.89 7.01	6	50 DO	100/175mm		O		
8					7	Nx RC	-	T.C.R. 98 % S.C.R. 92 % R.Q.D. 89 %			
		END OF BOREHOLE		193.34 8.56							
9											

PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE

1 : 50

Golder Associates

LOGGED D.J.M.

CHECKED

RECORD OF BOREHOLE 16

SHEET 1

LOCATION - SEE FIGURE 1

BORING DATE MAY 4, 1988

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 881-3443-1

(Golder Report No. 881-3443-1)

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○			HYDRAULIC CONDUCTIVITY, k, CM/SEC / WATER CONTENT, PERCENT Wp W Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		TOP OF CENTRE PIER		201.91 0.00									
1					1	Nx RC	-	T.C.R. 100 %	S.C.R. 80 %	R.Q.D. 87 %			
2					2	Nx RC	-	T.C.R. 100 %	S.C.R. 100 %	R.Q.D. 100 %			
3	ROTARY DRILLING NxL ROCK CORE	CONCRETE											BOREHOLE GROUTED TO SURFACE FOLLOWING DRILLING
4					3	Nx RC	-	T.C.R. 100 %	S.C.R. 98 %	R.Q.D. 98 %			
5					4	Nx RC	-	T.C.R. 100 %	S.C.R. 90 %	R.Q.D. 83 %			
6				195.57 6.34	5	Nx RC	-	T.C.R. 72 %	S.C.R. 33 %	R.Q.D. 33 %			
7		Fresh, massive, grey to light grey, fine grained LIMESTONE vuggy between 6.7 and 7.0m with light green quartz and/or chert inclusions	▦		6	Nx RC	-	T.C.R. 100 %	S.C.R. 93 %	R.Q.D. 93 %			
8		END OF BOREHOLE		193.83 8.08									
9													

3
15 ✛ 5 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE
1 : 50

Golder Associates

LOGGED D.J.M.
CHECKED

RECORD OF BOREHOLE 17

SHEET 1 OF 2

LOCATION - SEE FIGURE 1

BORING DATE MAY 6, 1989

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 881-3443-1

(Golder Report No. 881-3443-1)

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	TYPE	BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○	HYDRAULIC CONDUCTIVITY, k, CM/SEC / WATER CONTENT, PERCENT Wp W Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		TOP OF EAST PIER		201.92							
	ROTARY DRILLING NxL ROCK CORE	CONCRETE			1	Nx RC	-	T.C.R. 100 % S.C.R. 100 % R.Q.D. 100 %			BOREHOLE GROUTED TO SURFACE FOLLOWING DRILLING
					2	Nx RC	-	T.C.R. 100 % S.C.R. 100 % R.Q.D. 100 %			
					3	Nx RC	-	T.C.R. 100 % S.C.R. 95 % R.Q.D. 82 %			
					4	Nx RC	-	T.C.R. 100 % S.C.R. 100 % R.Q.D. 85 %			
					5	Nx RC	-	T.C.R. 78 % S.C.R. 58 % R.Q.D. 36 %			
					6	Nx RC	-	T.C.R. 46 % S.C.R. 29 % R.Q.D. 0 %			
		Fresh, massive grey to light grey LIMESTONE vuggy below 10.2m		194.33 7.59							
					7	Nx RC	-	T.C.R. 97 % S.C.R. 78 % R.Q.D. 77 %			
9				193.05 8.87							

CONTINUED ON SHEET 2

15 ⊕ 3 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE

1 : 50

Golder Associates

LOGGED D.J.M.

CHECKED B

RECORD OF BOREHOLE 17 Cont...

SHEET 2 of 2

LOCATION - SEE FIGURE 1

BORING DATE MAY 5, 1989

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 881-3443-1

(Golder Report No. 881-3443-1)

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	TYPE	BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○	HYDRAULIC CONDUCTIVITY, k, CM/SEC WATER CONTENT, PERCENT Wp W Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
8											
9		CONTINUED FROM SHEET 1		193.06 8.87							
	ROTARY DRILLING NxL ROCK CORE	Fresh, massive grey to light grey LIMESTONE vuggy below 10.2 m			8	Nx RC	-	T.C.R. 90 % S.C.R. 87 % R.Q.D. 83 %			
10											
		END OF BOREHOLE		191.53 10.39							
11											
12											
13											
14											
15											
16											
17											
18											

0
10 ⊕ 3 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE
1 : 50

Golder Associates

LOGGED D.J.M.
CHECKED

RECORD OF BOREHOLE 18

SHEET 1 OF 2

LOCATION - SEE FIGURE 1
BORING DATE MAY 6, 1989
DATUM GEODETIC
SAMPLER HAMMER, 63.5kg, DROP, 760mm
PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

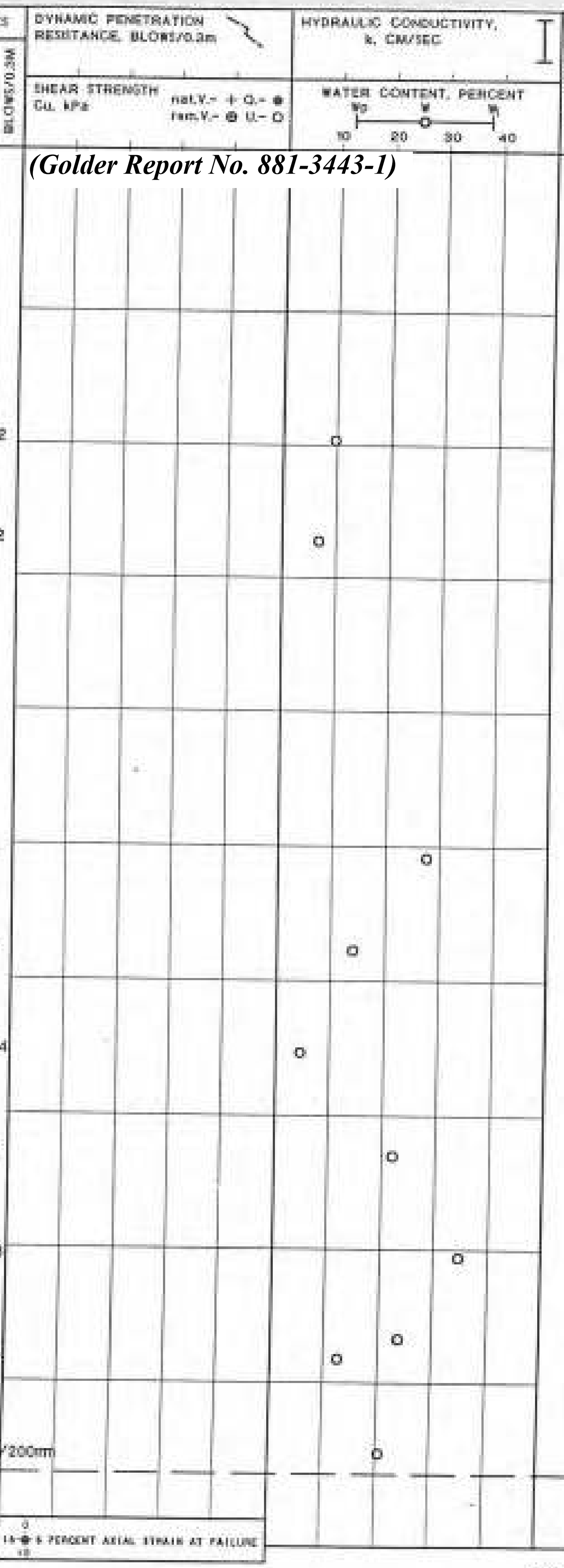
PROJECT 881-3443-1

(Golder Report No. 881-3443-1)

BORING METHOD: ROTARY DRILLING / POWER AUGER; NxL ROCK CORE / (HOLLOW STEM)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (M)
GROUND SURFACE	203.03
Brown sand & gravel (FILL)	0.00
Very loose to loose brown sand trace to some silt occ. gravel, wood, sawdust and cinders (FILL)	0.15
Very dense brown SAND AND GRAVEL	197.85 5.18
Compact grey SILT with clayey silt layers	196.32 6.71
Stiff grey CLAYEY SILT trace sand occ. gravel	195.57 7.46
Very stiff grey SILTY CLAY with silt layers	195.23 7.80
CONTINUED ON SHEET 2	194.31 8.72

SAMPLES NUMBER	TYPE	BLOWS/0.3M
1	50 DO	2
2	50 DO	2
3	50 DO	1
4	50 DO	7
5	50 DO	84
6	50 DO	7
7	50 DO	13
8	50 DO	24
9	50 DO	128/200mm



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

WL ENCOUNTERED AT ELEV. 196.93 DURING DRILLING MAY 6, 1989

DEPTH SCALE 1 : 50

Golder Associates

LOGGED D.J.M.
CHECKED

RECORD OF BOREHOLE 18 Cont...

SHEET 2 of 2

LOCATION - SEE FIGURE 1

BORING DATE MAY 8, 1989

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 881-3443-1

(Golder Report No. 881-3443-1)

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○	HYDRAULIC CONDUCTIVITY, k, CM/SEC / WATER CONTENT, PERCENT Wp W Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
8											
		CONTINUED FROM SHEET 1		194.31 8.72							
9	ROTARY DRILLING NXL ROCK CORE	Fresh, massive, light grey fine grained LIMESTONE light green chert inclusions between 8.8 and 9.7 m and between 10.0 and 10.4 m small vuggy section at 10.1 m			10	Nx RC	-	T.C.R. 97 % S.C.R. 75 % R.Q.D. 70 %			
10					11	Nx RC	-	T.C.R. 100 % S.C.R. 33 % R.Q.D. 33 %			
		END OF BOREHOLE		192.18 10.85							
11											
12											
13											
14											
15											
16											
17											
18											

15 ⊕ 3 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE
1 : 50

Golder Associates

LOGGED D.J.M.
CHECKED

RECORD OF BOREHOLE 19

SHEET 1

LOCATION - SEE FIGURE 1

BORING DATE MAY 8, 1989

DATUM GEODETIC

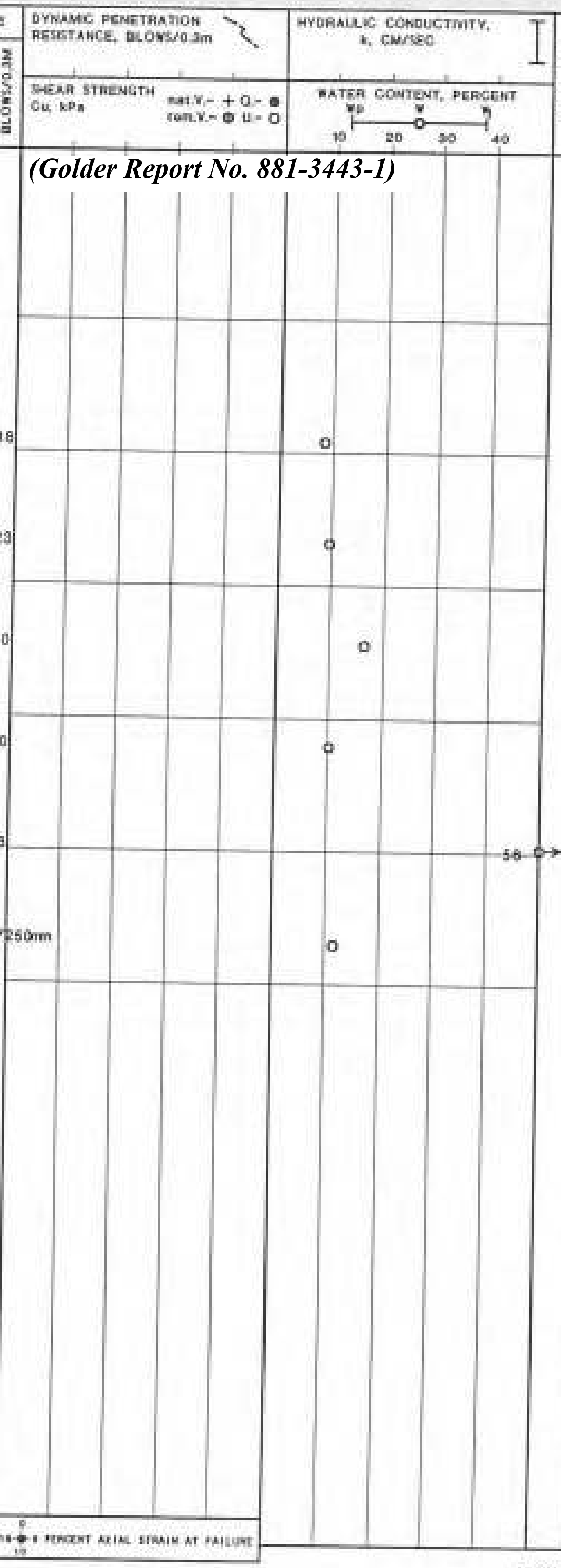
SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 881-3443-1

(Golder Report No. 881-3443-1)

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (M)	SAMPLES NUMBER	TYPE	BLOWS/0.3M
0	ROTARY DRILLING / POWER AUGER NxL ROCK CORE / (HOLLOW STEM)	GROUND SURFACE	202.07 0.00			
1		Compact brown sand and gravel occ. topsoil and clay lumps (FILL)		1	50 DO	18
2				2	50 DO	23
				3	50 DO	30
3		Compact to very dense brown sand and gravel trace silt with wood & concrete (FILL)	199.18 2.89	4	50 DO	50
4				5	50 DO	16
				6	50 DO	92/250mm
5		Probably Reinforced Concrete	197.10 4.97			
6		END OF BOREHOLE (Practical Auger Refusal at 5.33m Triconed to 5.79m)	196.28 5.79			
7						
8						
9						



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

WL ENCOUNTERED AT ELEV. 199.93 DURING DRILLING MAY 8, 1989

DEPTH SCALE

1 : 50

Golder Associates

LOGGED D.J.M.

CHECKED

RECORD OF BOREHOLE 20

SHEET 1 OF 2

LOCATION - SEE FIGURE 1

BORING DATE MAY 8, 1989

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

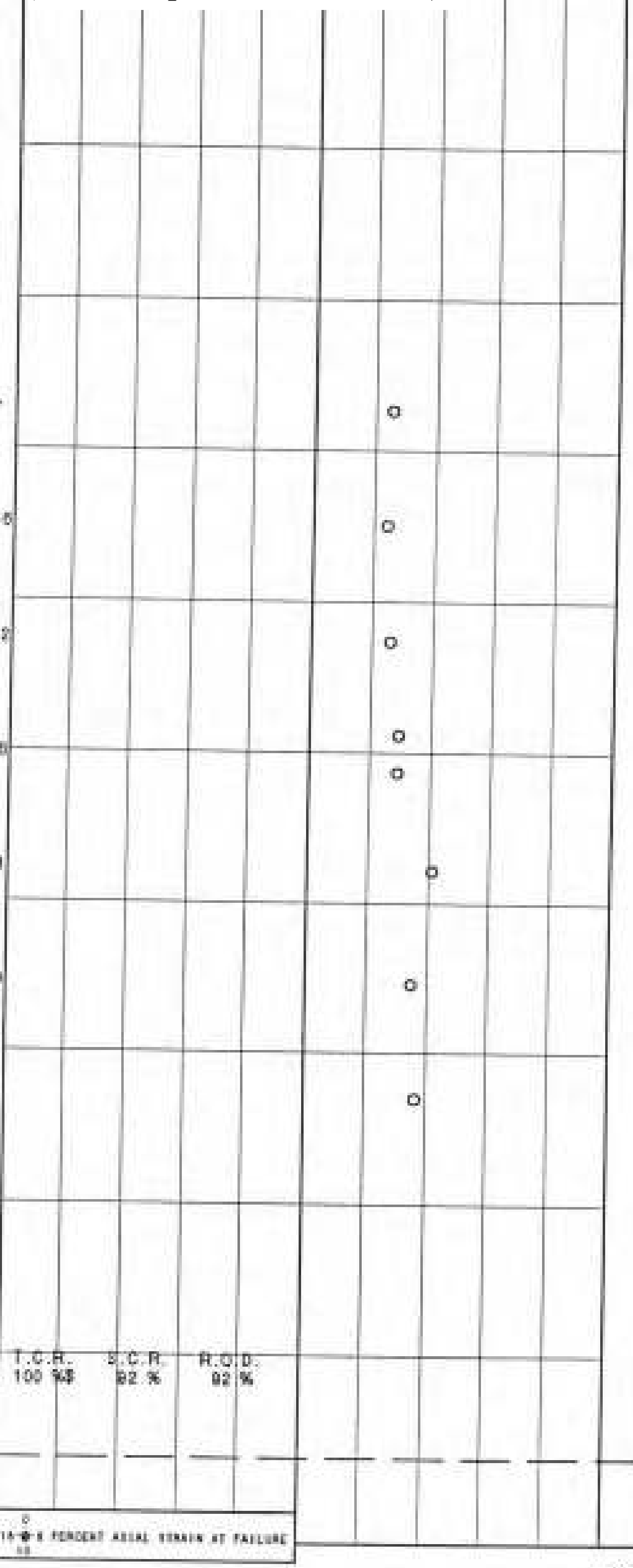
PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 881-3443-1

(Golder Report No. 881-3443-1)

BORING METHOD: ROTARY DRILLING / POWER AUGER; NxL ROCK CORE / (HOLLOW STEM)

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (M)	SAMPLES NUMBER	TYPE	BLOWS/0.3M
0	GROUND SURFACE	202.02 0.00			
	Brown sand & gravel (FILL)				
1	Hard grey CLAYEY SILT (TILL)	201.11 0.91	1	AS	-
2			2	50 DO	66
3	Hard grey INTERLAYERED SILTY CLAY AND SILT	199.13 2.89	3	50 DO	82
4	Very dense grey SILT with silty clay layers	197.97 4.05	4	50 DO	78
5			5	50 DO	51
			6	50 DO	59
6	Hard grey SILTY CLAY with silt layers	196.08 5.94	7	50 DO	37
7			8	50 DO	61
	Fresh, massive, grey to light grey LIMESTONE , chert inclusions at 7.9 m and vuggy section at 9.7 m	194.56 7.46			
8			9	Nx RC	-
	CONTINUED ON SHEET 2	193.31 8.71			
9					



T.C.R. 100 %$ S.C.R. 92 % R.Q.D. 92 %

15-⊕-5 PERCENT AXIAL STRAIN AT FAILURE

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

WL ENCOUNTERED AT ELEV. 198.21 DURING DRILLING MAY 8, 1989

DEPTH SCALE
1 : 50

Golder Associates

LOGGED D.J.M.
CHECKED

RECORD OF BOREHOLE 20 Cont...

SHEET 2 of 2

LOCATION - SEE FIGURE 1

BORING DATE MAY 8, 1989

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 881-3443-1

(Golder Report No. 881-3443-1)

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○	HYDRAULIC CONDUCTIVITY, k, CM/SEC; WATER CONTENT, PERCENT Wp W Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
8											
		CONTINUED FROM SHEET 1		193.31 8.71							
9	ROTARY DRILLING NxL ROCK CORE	Fresh, massive, grey to light grey LIMESTONE, chert inclusion at 7.9 m and vuggy section at 9.7 m			10	Nx RC	-	T.C.R. 90 % S.C.R. 85 % R.Q.D. 80 %			
10				192.02							
		END OF BOREHOLE		10.00							
11											
12											
13											
14											
15											
16											
17											
18											

$\overset{3}{\underset{10}{19 \oplus}}$ % PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE

1 : 60

Golder Associates

LOGGED D.J.M.

CHECKED

PETO MACCALLUM LTD.
CONSULTING ENGINEERS

LOG OF BOREHOLE No. 1

JOB NAME LORNE BRIDGE
LOCATION Brantford, Ontario
BORING METHOD Continuous Flight Hollow Stem Augers and BW Wash Boring
BORING DATE Oct. 9 & 10/86
JOB No. 86 F 251
ENGINEER T. Lee-Bun
TECHNICIAN S. Burt

SOIL PROFILE				SAMPLES			SHEAR STRENGTH C_u ▲ / DYNAMIC CONE PENETRATION • / STANDARD PENETRATION TEST • (BLOWS/FOOT 20 40 60 80)			LIQUID LIMIT W_L / PLASTIC LIMIT W_P / WATER CONTENT W (WATER CONTENT % 10 20 30)	GROUNDWATER OBSERVATIONS AND REMARKS
DEPTH	DESCRIPTION Borehole 1 - East Abutment GROUND ELEVATION: 692.7	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/FOOT N VALUES	Run (%)	Recovery (%)	Mod. RQD (%)		
	3 in. Asphaltic Concrete 6 in. Structural Concrete	Δ;Δ			RC						
	(Railway Underpass)										
29.8	FILL: Predominantly sand and gravel	▨	660 655								
38.5	CONCRETE: Sound 2 and 3 ft. thick boulders encountered		650 645 640		RC						
56.2											
56.8	LIMESTONE: Very poor quality/ very fractured, grey to blue grey limestone, high strength		635		RC*		7	100	0		*4 attempts
	Borehole terminated at 56.8 ft.										

NOTES Modified R.Q.D. = $\frac{\text{Total length of BXL core longer than 4 in.}}{\text{Total Drill Run}}$ X 100%

CHECKED BY:

PETO MacCALLUM LTD.
CONSULTING ENGINEERS

LOG OF BOREHOLE No. 2

JOB NAME LORNE BRIDGE
LOCATION Brantford, Ontario
BORING METHOD Continuous Flight Hollow Stem Augers and BW Wash Boring
BORING DATE Oct. 10 & 14/86
JOB No. 86 F 251
ENGINEER T. Lee-Bun
TECHNICIAN S. Burt

SOIL PROFILE				SAMPLES					
DEPTH	DESCRIPTION Borehole 2 - West Abutment GROUND ELEVATION: 683.4	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/FOOT N VALUES	SHEAR STRENGTH C_u ▲ DYNAMIC CONE PENETRATION • STANDARD PENETRATION TEST • BLOWS/FOOT 20 40 60 80	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W W_P W W_L WATER CONTENT % 10 20 30	GROUNDWATER OBSERVATIONS AND REMARKS
	5 in. Asphaltic Concrete								
	FILL: Predominantly sand and gravel with cobbles/ boulders								
21.0	CONCRETE: Sound		660 655						
			650 645 640		RC				
46.8	LIMESTONE: Hard, sound, grey to blue grey limestone, high strength, excellent quality occ. vugs infilled with crystal		635		RC	48	100 100		
50.8	Borehole terminated at 50.8 ft.		630			Run (%)	Recovery (%) Mod. RQD (%)		

Depth scale: 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70

NOTES

CHECKED BY

PML 504

MEMBER OF THE ASSOCIATION OF CONSULTING ENGINEERS OF CANADA

PETO MACCALLUM LTD.
CONSULTING ENGINEERS

LOG OF BOREHOLE No. 3

JOB NAME LORNE BRIDGE
LOCATION Brantford, Ontario
BORING METHOD Continuous Flight Hollow Stem Augers and BW Wash Boring
BORING DATE Oct. 15/86
JOB No. 86 F 251
ENGINEER T. Lee-Bun
TECHNICIAN S. Burt

SOIL PROFILE

DESCRIPTION Borehole 3 - East Pier
GROUND ELEVATION 692.5

SAMPLES: NUMBER, TYPE, BLOWS/FOOT N - VALUES

SHEAR STRENGTH C_u ▲
DYNAMIC CONE PENETRATION •
STANDARD PENETRATION TEST •
BLOWS/FOOT 20 40 60 80

LIQUID LIMIT W_L
PLASTIC LIMIT W_P
WATER CONTENT W
WATER CONTENT % 10 20 30

GROUNDWATER OBSERVATIONS AND REMARKS

DEPTH	DESCRIPTION	ELEVATION	TYPE	Run (%)	Recovery (%)	Mod. RQD (%)
	3 in. Asphaltic Concrete 13.5 in. Structural Concrete		RC			
	Open chamber under bridge deck.					
20						
		670				
22.8	CONCRETE: Sound, occ. steel reinforcing bars encountered					
25						
		665				
30			RC			
		660				
35						
		655				
40						
		650				
45						
		645				
50						
	12 in. thick layer of wood between 53.0 and 54.0 ft.	640				
54.2 54.4	2 in. layer of rounded gravel					
55		635	RC	41	100	100
57.8	LIMESTONE: hard, sound, grey to blue grey limestone, high strength, excellent quality occ. small vugs infilled with crystals					
60		630				
	Borehole terminated at 57.8 ft.					
65						
70						

NOTES

CHECKED BY

PETO MacCALLUM LTD.
CONSULTING ENGINEERS

LOG OF BOREHOLE No. 4A/4B

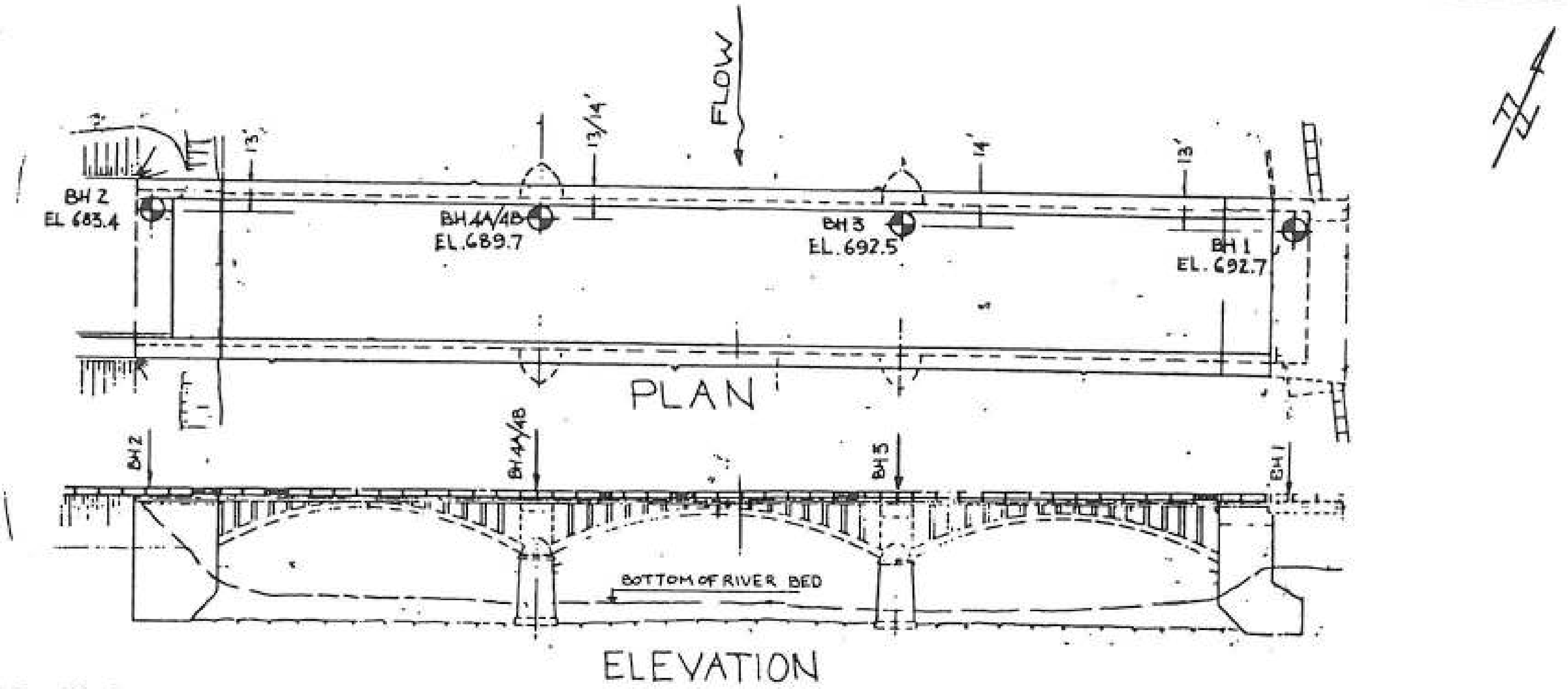
JOB NAME LORNE BRIDGE
LOCATION Brantford, Ontario
JOB No 86 F 251
BORING METHOD Continuous Flight Hollow Stem Augers and BW Wash Boring
BORING DATE Oct. 15/86
ENGINEER T. Lee-Bun
TECHNICIAN S. Burt

SOIL PROFILE

DEPTH	DESCRIPTION Borehole 4A/4B - West Pier GROUND ELEVATION 689.7	LEGEND	ELEVATION	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/FOOT N VALUES	SHEAR STRENGTH C_u ▲ DYNAMIC CONE PENETRATION × STANDARD PENETRATION TEST ● BLOWS/FOOT 20 40 60 80	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W WATER CONTENT % 10 20 30	GROUNDWATER OBSERVATIONS AND REMARKS
	3 in. Asphaltic Concrete 15 in. Structural Concrete				RC				
5	Open chamber under bridge deck.		685						
10			680						
15			675						
19.8 20	TOP OF CONCRETE		670						
	Borehole terminated at 19.8 ft. Unable to seat boring equipment into concrete.								
25									
30									
35									
40									
45									
50									
55									
60									

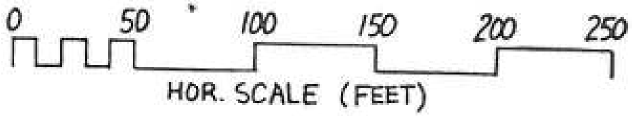
NOTES

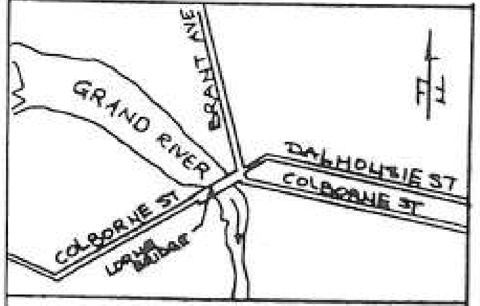
CHECKED BY



Reproduced from City of Brantford Dwg S-1, Sheet 29, Rev 1 dated March '79, Titled "Lorne Bridge Reconstruction".

BH1: Cut cross on north end of concrete retaining wall, 3.6 m east of LE & N Railway, 95.15 m north of Lorne Bridge. Elevation 668.03 ft. provided by client.





KEY PLAN

THE CORPORATION OF THE CITY OF BRANTFORD

FOUNDATION SUBGRADE EVALUATION
LORNE BRIDGE BRANTFORD ONT.

BORENOLE LOCATION PLAN

PETO MacCALLUM LTD.
CONSULTING ENGINEERS

DRAWN MB CHECKED APPROVED	DATE	SCALE	JOB NO.	DRAWING NO.
	NOV/86	AS SHOWN	86F251	1

LOG OF BOREHOLE 7

Our Reference Nº 78-4-K2

Enclosure Nº 8

CLIENT: J.D. Lee Eng. Ltd.
PROJECT: Lorne Bridge Reconstruction
LOCATION: Brantford, Ontario
DATUM ELEVATION: Geodetic

DRILLING DATA
Method: Augering
Diameter: 6½"
Date: April 7, 1978

SUBSURFACE PROFILE					SAMPLES			PENETRATION RESISTANCE Blows/Foot 20 40 60 80 100 UNDRAINED SHEAR STRENGTH lbs/sq.ft. + FIELD VANE TEST ● COMPRESSION TEST	WATER CONTENT % PLASTIC LIMIT w_P NATURAL w LIQUID LIMIT w_L	REMARKS
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows/Foot			
		GROUND SURFACE								
663.0	0	Brown SAND and GRAVEL (FILL)								Standpipe
661.5	1.5	Firm brown clayey SILT								
660.5	2.5	Stiff mottled brownish grey clayey SILT			1	SS	17	o		
658.0	5.0	Very stiff grey clayey SILT (TILL)			2	SS	25	o		
656.0	7.0	Hard grey SILT (TILL) occasional gravel			3	SS	33	o		
	10			▽	4	SS	57	o		
649.1	14.9	Very stiff grey SILT, trace of clay, clay seams below 16 feet		El. 651.6 Apr 13/78	5	SS	26	o		
	20.0				6	SS	24	o		
638.0	25.0	limestone fragments			7 8	[illegible] AS	200/0			Auger refusal at 25 feet, possible bedrock
		END OF BOREHOLE								

VERTICAL SCALE: 1 inch to 5 feet

DOMINION SOIL INVESTIGATION INC.

MADE: J.R.A. CHECKED: R.N.R.

LOG OF BOREHOLE 8

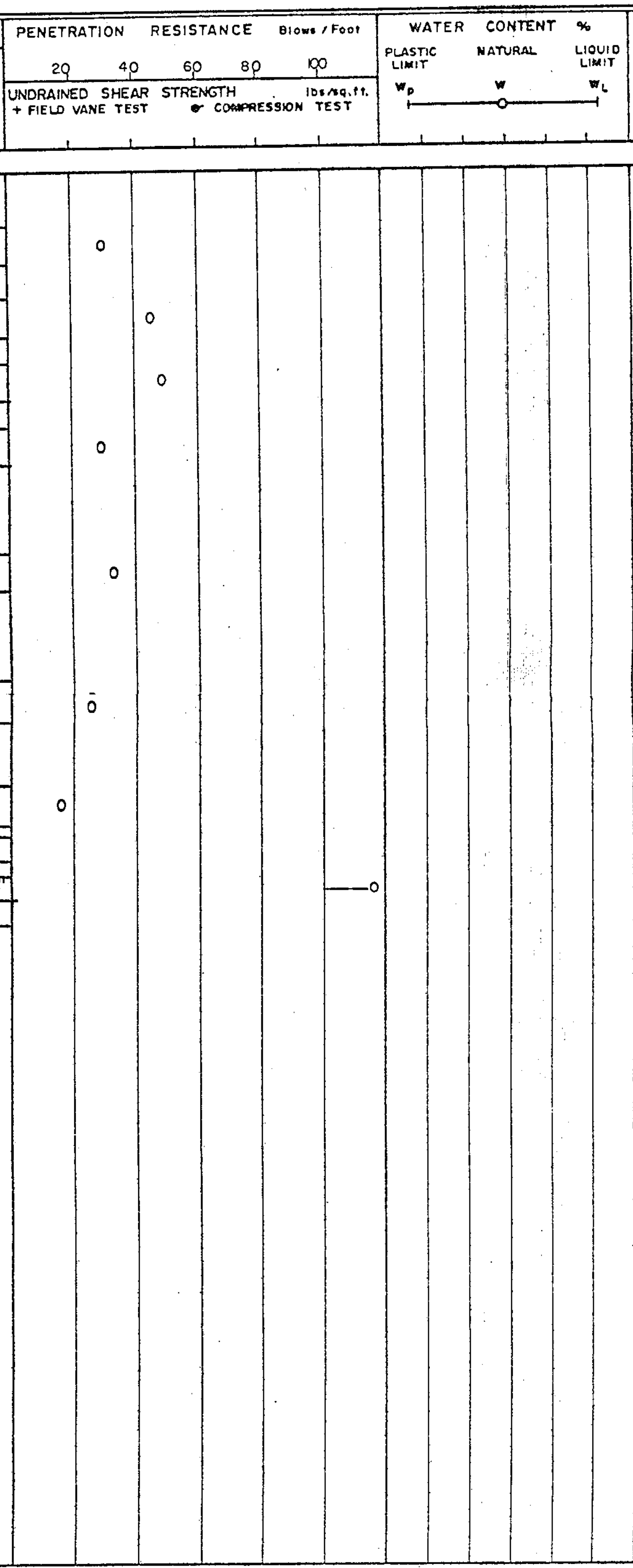
Our Reference No 78-4-K2

Enclosure No 9

CLIENT: J.D. Lee Eng. Ltd.
PROJECT: Lorne Bridge Reconstruction
LOCATION: Brantford, Ontario
DATUM ELEVATION: Geodetic

DRILLING DATA
Method: Augering
Diameter: 6½"
Date: April 7, 1978

SUBSURFACE PROFILE					SAMPLES			REMARKS
ELEVATION Ft	DEPTH Ft	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows / Foot	
564.4	0	CONCRETE SURFACE CONCRETE		▽ El. 655.6 Apr 13/78				Standpipe
563.9	0.5	Brown SAND & GRAVEL FILL						
562.5	1.5	Stiff brown clayey SILT			1	SS	29	
561.4	3.0	Black & brown uniform fine SAND						
559.9	4.5	Hard brown clayey SILT (TILL)			2	SS	43	
556.9	7.5	Very stiff to hard grey clayey SILT (TILL)			3 4	SS SS	48 29	
550.9	13.5	Very stiff grey SILT, with ½ in. sand seams 6 in. apart			5	SS	36	
545.4	19.0	Very stiff grey SILT			6	SS	25	
539.9	24.5	Very stiff grey silty CLAY, layers of silt			7 8	SS TW	18	
536.9	27.5	Hard grey clayey SILT (TILL)			9 10	SS AS	35/3"	Auger refusal at 29.5 feet, possible bedrock
534.9	29.5	END OF BOREHOLE						



VERTICAL SCALE: 1 inch to 5 feet

MADE: J.R.A.

CHECKED: R.N.R.

DOMINION SOIL INVESTIGATION INC.

LOG OF BOREHOLE ....9.......

Our Reference Nº 78-4-K2 ....

Enclosure Nº 10 .....

CLIENT: J.D. Lee Eng. Ltd.
PROJECT: Lorne Bridge Reconstruction
LOCATION: Brantford, Ontario
DATUM ELEVATION: Geodetic

DRILLING DATA
Method: Augering
Diameter: 6¼"
Date: April 11, 1978

SUBSURFACE PROFILE					SAMPLES			PENETRATION RESISTANCE Blows / Foot 20 40 60 80 100 UNDRAINED SHEAR STRENGTH lbs./sq.ft. + FIELD VANE TEST ● COMPRESSION TEST	WATER CONTENT % PLASTIC LIMIT w_P NATURAL w LIQUID LIMIT w_L	REMARKS
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows/Foot			
		PAVEMENT SURFACE								
81.6	0.0	[illegible]								
81.3	0.3	concrete								
80.9	0.7	wood								
80.1	1.5	crushed stone								
79.6	2.0	Compact brown fine to medium SAND, some gravel (FILL)		Dry Upon Completion	1	SS	18			
	5.0				2	SS	15			
746	7.0	Compact to dense brown sandy SILT some gravel, trace topsoil (FILL)			3	SS	32			
	10.0				4	SS	16			
	15.0				5	SS	100/[illegible]"			
66.1	15.5	Compact brown fine SAND, trace silt								
	20.0				6	SS	19			
59.1	22.6	Compact brown fine SAND, some gravel			7	SS	19			
57.1	24.5	END OF BOREHOLE								
	25									

VERTICAL SCALE: 1 inch to 5 feet

MADE: J.R.A. CHECKED: R.N.R.

DOMINION SOIL INVESTIGATION INC.

LOG OF BOREHOLE 11

Enclosure No 12

Our Reference No. 78-4-K2

CLIENT: J.D. Lee Eng. Ltd.
PROJECT: Lorne Bridge Reconstruction
LOCATION: Brantford, Ontario
DATUM ELEVATION: Geodetic

DRILLING DATA
Method: Augering
Diameter: 6½"
Date: April 11, 1978

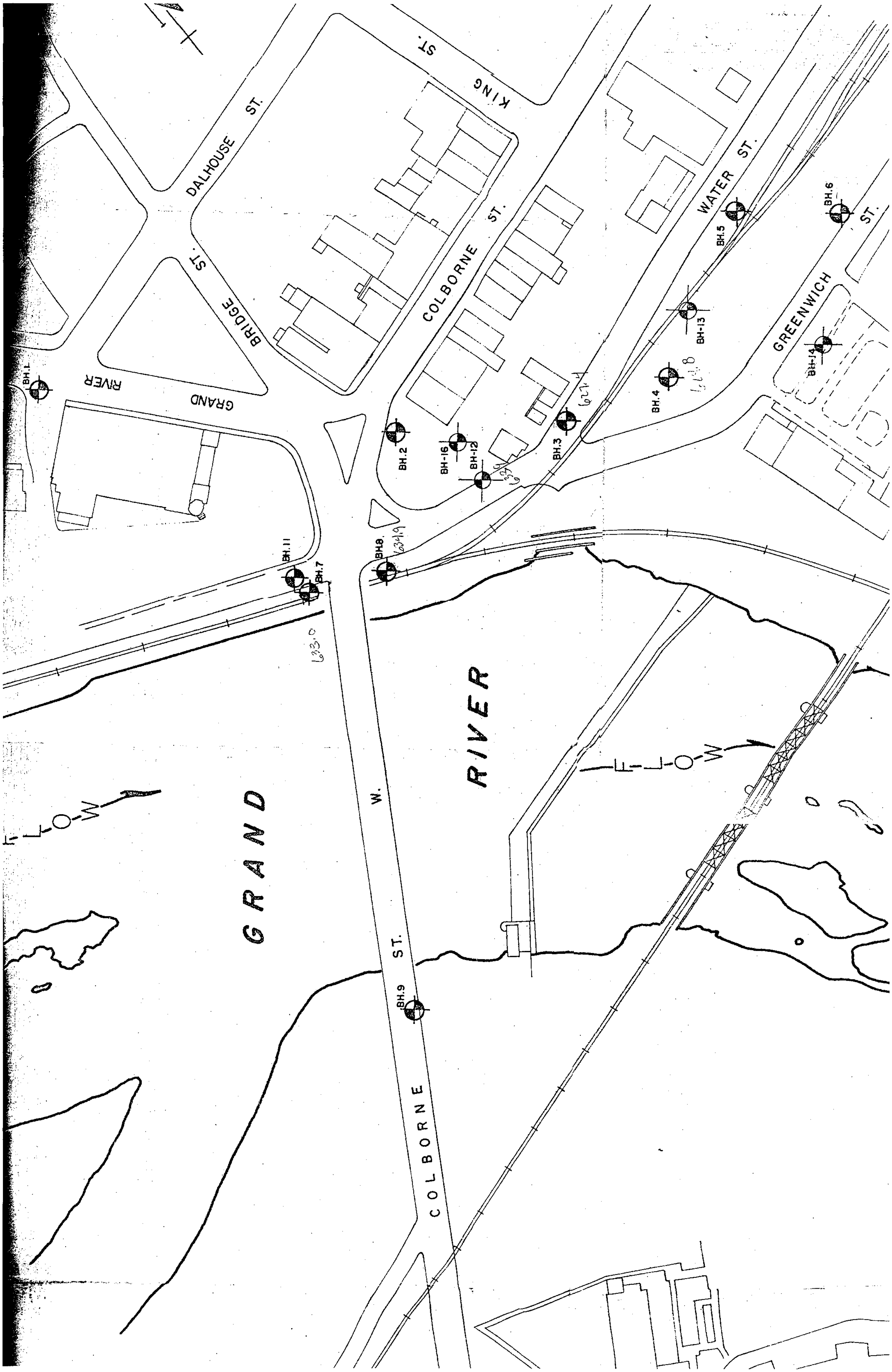
SUBSURFACE PROFILE					SAMPLES			PENETRATION RESISTANCE Blows/Foot 20 40 60 80 100 UNDRAINED SHEAR STRENGTH p.s.f. + FIELD VANE TEST ◉ COMPRESSION TEST	WATER CONTENT % PLASTIC LIMIT W_P NATURAL W LIQUID LIMIT W_L	REMARKS
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows/Ft.			
683.5	0	PAVEMENT SURFACE CONCRETE		Dry Upon Completion						
682.5	1.0	CRUSHED STONE								
682.0	1.5	Compact to dense fine to medium SAND and GRAVEL trace of silt (FILL)			1	SS	19			
	5.0				2	SS	37			
					3	SS	38/4"			
	10.0	trace topsoil			4	SS	19			
669.5	14.0	Stiff grey SILT with CLAY seams								
	15.0				5	SS	15			
667.0	16.5	END OF BOREHOLE								
	20.0									

VERTICAL SCALE: 1inch to 5 feet

DRAWN: J.R.A.

CHECKED: R.N.R.

DOMINION SOIL INVESTIGATION INC.


DALHOUSE ST.
KING ST.
BRIDGE ST.
GRAND RIVER
COLBORNE ST.
WATER ST.
GREENWICH ST.
BH.1
BH.2
BH.3
BH.4
BH.5
BH.6
BH.7
BH.8
BH.9
BH.11
BH-12
BH-13
BH-14
BH-16
633.0
634.9
633.9
627.4
628.8
GRAND
RIVER
FLOW
FLOW
COLBORNE W. ST.

APPENDIX C

Site Photographs

Photograph 1: View up-river to Lorne Bridge and downtown Brantford from the west abutment, facing north.

Photograph 2: View from the former railbed of the LE&N atop the stone embankment to the northeast of the bridge, facing southwest.

Photograph 3: View down-river to TH&B Crossing Bridge to the northeast to east of the bridge.

Photograph 4: View of the Brant’s and Lorne bridge from the TH&B Crossing Bridge to the south, facing north.

Photograph 5: Lorne bridge – Looking at east pier and abutment from upstream side of the bridge.

Photograph 6: Lorne bridge – Looking at east pier and west pier from upstream side of the bridge.

Photograph 7: Lorne bridge – Looking at east pier and west pier from downstream side.

Photograph 8: Lorne bridge - Looking towards east pier and abutment from downstream side.

Photograph 9: View of the Brant’s bridge from the Lorne bridge to the south, facing northeast.

Photograph 10: Brant’s bridge- Looking at west pier from the upstream side of the bridge and west abutment.

Photograph 11: Brant’s bridge- Looking at east pier and east abutment from upstream side.

Photograph 12: Brant’s bridge – Looking at central pier looking from upstream side.

Photograph 13: Brants bridge – Looking at west pier and west abutment from upstream side.

Photograph 14: TH&B Crossing Bridge– Looking at central pier, east pier, and east abutment from downstream side.

Photograph 15: TH&B Crossing Bridge – Looking at central pier, east pier and east abutment from upstream side.

Photograph 16: TH&B Crossing Bridge – Looking at west pier and west abutment from downstream side.

Photograph 17: TH&B Crossing Bridge – Looking west.

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