April 4, 2018

FOUNDATION INVESTIGATION AND DESIGN REPORT

WEST BEATON RIVER BRIDGE - SITE NO. 38N-008
HIGHWAY 631, ALGOMA DISTRICT
TOWNSHIP OF BEATON
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 5569-09-00, W.P. 5569-09-01

Submitted to:
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GEOCRES No.: **42F-52**

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Distribution:

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1 PDF Copy: Golder Associates Ltd., Sudbury, Ontario

REPORT

Golder Associates

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PART A

FOUNDATION INVESTIGATION REPORT
WEST BEATON RIVER BRIDGE REHABILITATION - SITE NO. 38N-008
HIGHWAY 631 ALGOMA DISTRICT
TOWNSHIP OF BEATON
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 5569-09-00, W.P. 5569-09-01

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA), on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the rehabilitation of West Beaton River Bridge (Site No. 38N-008), located on Highway 631, 73.4km North of Highway 17 in White River, Beaton Township. The key plan showing the general location of this section of Highway 631 and the location of the investigated area are shown on Drawing 1.

The purpose of this investigation is to establish the subsurface soil conditions at the existing bridge location by borehole drilling and laboratory testing on selected soil and samples.

The Terms of Reference and Scope of Work for the Foundation Investigation are outlined in MTO's Request for Proposal dated April 2016. Golder's proposal for foundation engineering services associated with rehabilitation of the West Beaton River Bridge structure is contained in Section 17.8 of LEA's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundations engineering services for this project, dated November 1, 2016.

2.0 SITE DESCRIPTION AND BACKGROUND INFORMATION

The existing West Beaton River Bridge consists of an approximately 26 m long and 10 m wide three-span structure that was constructed in 1968. In general, the topography of the site and surrounding area is relatively flat, with gently rolling terrain beyond the river. The area is surrounded by dense tree cover beyond the highway right-of-way.

Based on LEA's current General Arrangement dated August 2017, the existing abutments and piers are supported on driven steel tube piles. Based on the survey drawing provided by LEA (drawing 17197-West Beaton GA-S1.dwg) on October 16, 2017, the bridge deck is at Elevations 322.9 m and 323.0 m at the north and south abutments, respectively. The existing approach embankments are between about 4 m and 5 m high relative to the toe of slope at the river level. Views at the bridge site are shown on Photographs 1 to 2, following the text of this report.

3.0 INVESTIGATION PROCEDURE

The field work was carried out on August 19 and August 20, 2017, during which time two boreholes (WB-1 and WB-2) were advanced at the locations shown on Drawing 1.

The boreholes were advanced from the existing roadway platform using a Boart Longyear LF-70 DD drill rig supplied and operated by Downing Drilling Inc. (Downing) of Grenville-sur-la-Rouge, Quebec. The boreholes were advanced using 76 mm inside diameter hollow stem augers, and NW casing and wash boring techniques. Where coring through cobbles, boulders or bedrock was required, an NQ-size core barrel was used. Soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter split-spoon sampler operated by an automatic hammer, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). The groundwater level in the open boreholes was observed during the drilling operations as described on the borehole records in Appendix A. The boreholes were backfilled and grouted upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The field work was supervised on a full-time basis by a member of Golder's staff, who located the boreholes in the field, cleared the site for buried services, directed the drilling and sampling operations and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's Sudbury Laboratory for further examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Index and classification tests consisting of water content and grain size distribution were carried out on selected soil samples. The results of the laboratory testing on samples from the boreholes are presented on the borehole records in Appendix A, and on the grain size distribution figures in Appendix B. An unconfined compressive strength test was carried out on a specimen of the bedrock core obtained in Borehole WB-1.

Classification of the rock mass quality of the bedrock with respect to the Rock Quality Designation (RQD) and UCS are described based on Table 3.10 and Table 3.5, respectively, of the Canadian Foundation Engineering Manual (CFEM, 2006[1]). The degree of weathering of the bedrock samples (i.e., fresh to slightly weathered) and the strength classification of the intact rock mass based on field identification (i.e., strong to very strong) are described in accordance with Table B.3 and Table B.6, respectively, of the International Society for Rock Mechanics (ISRM[2]) standard classification system.

The borehole locations and elevations were measured and surveyed by a member of our technical staff, referenced to HCP100 Survey point. The borehole locations (referenced to the MTM NAD83, Zone 13 co-ordinate system), ground surface elevations (referenced to Geodetic datum) and borehole depths are presented on the borehole records in Appendix A, and summarized below.

Borehole	Location (MTM NAD 83, Zone 13)		Location World Geodetic System 84		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting	Latitude	Longitude		
WB-1	5429809.1	249204.1	49.004012	-84.759939	322.8	8.8*
WB-2	5429761.1	249206.1	49.003580	-84.759905	323.0	9.8

* Includes 3.2 m of bedrock core.

4.0 SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain (NOEGTS)[3] mapping, the West Beaton River Bridge site is located within a glaciolacustrine plain deposit consisting primarily of sands and silts, bordered by bedrock knobs to the north and south of the site.

Based on geological mapping by the Ontario Ministry of Northern Development and Mines (MNDM)[4], the site is underlain by gneissic tonalite rocks with minor supracrustal inclusions.

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[1] Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

[2] International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

[3] Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping.

[4] Ontario Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey – Map 2543

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil samples, are presented on the borehole records in Appendix A and the laboratory test sheets in Appendix B. The results of the in situ field tests (i.e., SPT ‘N’ values) as presented on the borehole records and in Section 4 are uncorrected. The stratigraphic boundaries shown on the borehole records sheets and on the interpreted stratigraphic profile on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations. A summary of the subsurface conditions as encountered in Boreholes WB-1 and WB-2 is presented below.

4.2.1 Subsoil Conditions

The soil deposits encountered in the boreholes are summarized below.

Deposit/Layer Description	Boreholes	Deposit Surface Elevation (m)	Deposit Thickness (m)	“N” – Values (blows)	Laboratory Testing
				Relative Density	
Asphalt	WB-1, WB-2	323.0 and 322.8	0.05	n/a	n/a
(FILL) Sand and gravel; sand to gravelly sand[1]	WB-1, WB-2	322.9 and 322.7	0.7 – 1.4; 2.1 – 3.8	N = 66 – 87; N = 17 – 50	w = 6% and 25% 2 – M (Fig. B1)
				Very dense; Compact to Dense	
Silt to Sandy Silt to Silt and Sand	WB-1, WB-2	320.0 and 317.8	2.8 – 4.6 (borehole terminated in deposit in WB-2)	N = 16 – 82	w = 11% – 50% 3 – MH (Fig. B2)
				Compact to very dense	

Where:

N = SPT ‘N’–values; number of blows for 0.3 m of penetration
w = natural moisture content
M = sieve analysis
MH = combined sieve and hydrometer analysis

Notes:

1) Cobbles and/or boulder were encountered within the gravelly sand deposit between 1.4 m and 1.9 and from 2.6 m to 2.8 m depths in Borehole WB-1.

4.2.2 Bedrock

Bedrock was cored in Borehole WB-1 and the depth/elevation of the actual/inferred bedrock surface is presented below.

Borehole No.	Depth to Bedrock Surface (below ground surface at borehole location) (m)	Bedrock Surface Elevation (m)	Refusal Condition (m)
WB-1	5.6	317.4	3.2 m length of bedrock core

The retrieved bedrock core from the borehole is described as foliated, medium grained, grey gneiss. More detailed descriptions of the bedrock core are presented on the Record of Drillhole WB-1 in Appendix A, including data regarding the discontinuity frequency and type. A photograph of the bedrock core samples is shown on Figure B3 in Appendix B. The bedrock properties, as encountered in the cored borehole, are summarized below. The results of unconfined compressive strength (UCS) testing are presented in Table B1 in Appendix B.

Borehole No.	Total Core Recovery (TCR)	Rock Quality Designation (RQD)	Quality Classification (Table 3.10 of CFEM 2006[5])	UCS (MPa)	Strength Classification (Table 3.5 of CFEM 2006[3])
WB-1	100%	94% - 100%	Excellent	57	(R4) Strong

4.2.3 Groundwater Conditions

The unstabilized groundwater levels measured in the open boreholes upon completion of NW casing and wash boring techniques, prior to and after NQ coring, was at ground surface; however, this is not considered representative of the in situ groundwater condition. Water levels should be expected to vary depending on the time of year and precipitation events.

The river water level was surveyed by others at Elevation 318.9 m, in November, 2016.

5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. Mat Riopelle. This Foundation Investigation Report was prepared by Mr. Tibor Berecz, and the technical aspects were reviewed by Mr. André Bom, P.Eng., a geotechnical engineer and Associate of Golder. Mr. Jorge M.A. Costa, P.Eng., and Ms. Lisa Coyne, P.Eng., both Designated MTO Foundations Contacts for Golder, conducted an independent quality control review and technical audit of this report.

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[5] Canadian Geological Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

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PART B

FOUNDATION DESIGN REPORT
WEST BEATON RIVER BRIDGE REHABILITATION - SITE NO. 38N-008
HIGHWAY 631 ALGOMA DISTRICT
TOWNSHIP OF BEATON
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 5569-09-00, W.P. 5569-09-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering recommendations for assessment of the existing bridge foundations, and discussion regarding temporary protection systems for the rehabilitation of the existing West Beaton River Bridge located on Highway 631 about 73 km north of Highway 17 in White River, Beaton Township. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented in this Foundation Design Report (Part B) are intended to provide MTO's designers with sufficient information to assess the existing structure and design the bridge rehabilitation, and to aid in the identification of feasible protection system alternatives and requirements to support the construction staging.

The discussion and recommendations contained in this Foundation Design Report (Part B) shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. The contractor must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A), as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of MTO to provide recommendations for the design of temporary protection systems for the rehabilitation of the West Beaton River bridge on Highway 631. The temporary protection systems will be required during construction of the rehabilitation works to convert the existing bridge abutments to a semi-integral abutment structure. As the structure will be rehabilitated in stages, with traffic reduced to one lane in the vicinity of and along the bridge, the excavations at the abutments and approach embankments will be supported by the temporary protection systems to maintain the stability of the existing roadway embankments.

Excavations will be required to expose the existing abutments. Based on the GA drawing provided by LEA, the underside of the existing abutments is at about Elevation 320 m. As referenced in Section 4.2.2, the river water level was surveyed by others in November 2016, at Elevation 318.9 m.

Subsequently, Golder was requested by LEA to assess the potentially available geotechnical resistances of the existing piles to aid LEA's structural design team in determining if the existing deep foundations supporting the abutments can accommodate LEA's proposed increased loading from widening the bridge. The results of our assessment of available geotechnical resistances of the existing deep foundations at the abutments were reported to LEA in a Technical Memorandum dated January 24, 2018; a summary of this assessment is presented in Section 6.2 of this report.

6.2 Geotechnical Resistance of Existing Piles

LEA's Structural Evaluation Summary, dated November 21, 2017, indicates that consideration is being given to widening the existing bridge deck to maximize the width of the temporary single lane during staged rehabilitation. The current bridge is approximately 50 years old and it is understood from LEA's summary that the currently proposed rehabilitation should extend the service life of the structure for another 40 to 50 years.

In addition, on December 1, 2017, LEA provided Golder with MTO's GEOCRES report and previous structural drawings for this site, as identified below. It is understood from LEA that as-built/pile driving records are not available for the site.

- MTO GEOCRES No. 42F00-001: "Report on Soil Investigation for Proposed Crossing of Hwy. 631 at West Beaton River, District of Algoma, Township of Beaton, Ontario (W.P.145-64)" by Dominion Soil Investigation Limited, dated March 1966. A copy of the foundation borehole location plan and soil profile drawing from this report is provided in Appendix C, for reference.
- Structural drawings dated April 1967 for construction of the existing bridge, Contract 67-188, Drawing Nos. D-5920-1 to -3 (select drawings provided in Appendix C for reference).
- Structural drawings dated May 1990 for rehabilitation of the existing bridge, Contract 91-206, Drawing Nos. 1 to 7.

The 1967 West Beaton River bridge drawings indicate that the bridge is supported by 324 mm (12.75") outer diameter steel tube piles with 6 mm (0.25") thick walls, filled with concrete. The tube piles were to be fitted with pile driving shoes and were to be driven to bedrock to a specified design load of about 490 kN (48 tons). Based on the existing GEOCRES information and Golder's Borehole WB-1 advanced on the north approach embankment, the piles are anticipated to be founded on the bedrock surface.

The 1990 drawings for the existing bridge do not identify any need for, or remedial works carried out for, rehabilitation due to movement of the foundations, either vertically or horizontally. It is further understood that during LEA's structural site review, no concerns of foundation settlement were observed.

The structural design requirements for the maximum ULS and SLS design loads, and consequently the minimum required factored ultimate geotechnical resistance (f-ULS) and factored serviceability geotechnical resistance (f-SLS, for 25 mm of settlement), for the proposed bridge widening were provided by LEA to Golder on December 12, 2017, as presented below.

Bridge Site	Maximum Design Loads/ Required Factored Pile Resistances (kN)	
	ULS	SLS, for 25 mm of settlement
West Beaton River Bridge	1060	680

Based on the design drawings and the assumption that the piles at West Beaton River bridge are supported on bedrock, the f-ULS resistance will meet or exceed the maximum design load provided by LEA. Serviceability Limit State (SLS) conditions will not apply for piles founded on the bedrock at this site. Further, Golder has assumed the following conditions apply to the foundations geotechnical resistances assessment:

- In accordance with Section 6.5 of the Canadian Highway Bridge Design Code CAN/CSA S6-14 (CHBDC (2014)) and its Commentary, the proposed bridge and foundation systems are classified as having a "typical consequence level".
- Based on the level of foundation investigation completed in the previous and current investigations in comparison to the degree of site understanding in Section 6.5 of CHBDC (2014), the level of confidence for design has been taken as a "low degree of site and prediction model understanding".
- The corresponding consequence factor, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of the CHBDC (2014) have been used for the assessment of the geotechnical resistance of the existing foundations.

The assessment of available geotechnical resistances assumes the piles are in good condition with no reduction in pile capacity due to age. Given the age of the existing bridge, it is recommended that the condition of the existing pile caps be assessed during the proposed rehabilitation. We understand from LEA that if required, to extend the life expectancy of the existing piles, concrete casing could be considered around the uppermost portion of the piles. If in the future larger resistances are required than those proposed by LEA above, the Foundations Consultant should be provided the opportunity to review the recommendations presented above and provide further discussion as may be applicable.

6.3 Excavations and Temporary Cut Slopes

The proposed works will require excavations through the embankment fill behind the existing abutments in order to rehabilitate the existing abutments and other components of the bridge. Depending on the depth of excavation and the river water level at the time of construction works, groundwater may be encountered, as the stabilized water level at the approach embankments is likely at or near the elevation of the river water level. The groundwater level will be subject to fluctuations and the depth of excavation below the groundwater will depend on the time of year of construction. Also, perched groundwater may be present within the granular fill deposits. Surface water runoff and seepage from the granular fill/native soil strata into the excavations should be expected and will be greater during periods of sustained precipitation. Pumping from properly filtered sumps located at the base of the excavations may be required to provide groundwater control, but these should be located outside of the actual excavation limits required for the rehabilitation works. Surface water runoff should be directed away from the excavations at all times. A Notice to Contractor should be included in the Contract to alert the contractor to the potential for surface water runoff and groundwater seepage conditions and that the excavation must be unwatered and the side walls and base kept stable during construction; an example Notice to Contractor is included in Appendix D.

All excavations should be carried out in accordance with the latest edition of the Ontario *Occupational Health and Safety Act* and Regulations for Construction Projects. The existing embankment fill and native soils are classified as Type 3 soils above the groundwater level and Type 4 soils below the groundwater level. Open cut excavation side slopes in the existing embankment fill (i.e., sand and gravel, gravelly sand, sand) should remain stable during construction if the temporary side slopes are cut back no steeper than 1 Horizontal to 1 Vertical (1H:1V) above the groundwater level; the excavation slopes should be flattened to 3H:1V below the groundwater level (if encountered).

During construction, stockpiles should be placed at a distance away from the edge of the excavation not less than 1.5 times the depth of excavation, and their heights should be controlled to prevent surcharging the sides of the excavation and/or overall slope.

As the temporary open cut excavations required to allow for bridge rehabilitation works to be carried out are expected to be of limited depth (to about 4 m) there are no expected issues with stability of the reconstructed portion of the embankment side slopes at an inclination of 2H:1V, nor with reconstruction of the fill portion of the roadway. The embankment fill open cut slopes parallel to the bridge abutments should be cut consistent with OPSD 208.010 (Benching of Earth Slopes) at the time of backfilling to integrate the new backfill into the existing roadway fill.

6.4 Temporary Protection Systems

The temporary protection system could consist of either driven steel sheet piling or soldier piles and lagging where the H-piles would be driven or placed in pre-bored holes to a suitable depth and horizontal lagging installed as the excavation proceeds. It is noted that the dense to very dense zones encountered in the fill and native soils, together with the presence of zones of cobbles, may pose challenges to the use of driven steel sheet piling at this site. Support to the system could be in the form of struts and wales or rakers and anchors.

The Contractor is responsible for the complete detailed design of the temporary protection system. The temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). Temporary protection systems should be designed to Performance Level 2 for any excavation adjacent to the existing roadway. Design of the temporary system should include an evaluation of base stability, soil squeezing stability and hydraulic uplift stability as defined in the Canadian Foundation Engineering Manual (CFEM 2006).

The temporary protection system may be designed using the following parameters:

Soil Type	Bulk Unit Weight (γ, kN/m^3)	Internal Angle of Friction (ϕ', degrees)	Undrained Shear Strength (Su, kPa)	Lateral Coefficient of Earth Pressure[1]		
				K_a (Active)	K_o (At Rest)	K_p [2] (Passive)
Existing Sand and Gravel and the Sand to Gravely Sand - Fill (compact to very dense)	19	30	-	0.33	0.50	3.0

Soil Type	Bulk Unit Weight (γ, kN/m^3)	Internal Angle of Friction (ϕ', degrees)	Undrained Shear Strength (Su, kPa)	Lateral Coefficient of Earth Pressure[1]		
				K_a (Active)	K_o (At Rest)	K_p [2] (Passive)
Silt to Sandy Silt to Silt and Sand (compact to very dense)	18	29	-	0.35	0.52	2.88

1. The lateral earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficient of earth pressure should be adjusted accordingly.
2. The total passive resistance below the base of the excavation (i.e., within and/or adjacent to the temporary protection system) may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

Consideration could be given to either partial or full removal of the temporary protection system upon completion of construction or each stage of construction (as required). Where possible, full removal of the temporary shoring system should be considered to mitigate potential impediments to future rehabilitation/reconstruction work at the bridge site. Given the limited depth of the proposed excavation for the bridge rehabilitation at this site, it is expected that the temporary shoring will be installed within the cohesionless granular embankment fill and native silt to sandy silt to silt and sand deposit, with little to no risk of pile adhesion and therefore full removal of the protection system should be implemented.

6.4.1 Obstructions

The embankment fill in Borehole WB-1 contains zones of cobbles and/or boulder as noted on the borehole record, which could affect the excavations and installation of temporary protection systems. An NSSP should be included in the Contract Documents to identify to the contractor the presence of cobbles and/or boulders within the embankment fill; an example is included in Appendix C.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. André Bom, P.Eng. Mr. Jorge Costa, P.Eng., and Ms. Lisa Coyne, P.Eng., Designated MTO Foundations Contacts for Golder, conducted an independent quality control review of this report and technical audit of this report.

Report Signature Page

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REFERENCES

Canadian Foundation Engineering Manual 2006, 4th Edition, Canadian Geotechnical Society.

Canadian Highway Bridge Design Code, 56-14. 2014. CSA Group.

International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2

Ministry of Natural Resources. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 41JNW.

Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey – Map 2543.

Occupational Health and Safety Act and Regulation for Construction Projects (as amended).

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils

Ontario Provincial Standard Drawings

OPSD 208.010	Benching of Earth Slopes

Ontario Provincial Standard Specifications (OPSS) – Provincial Oriented

OPSS.PROV 539	Construction Specification for Temporary Protection Systems

Ontario Water Resource Act

Regulation 903	Wells (as amended)

MINISTRY OF TRANSPORTATION, ONTARIO

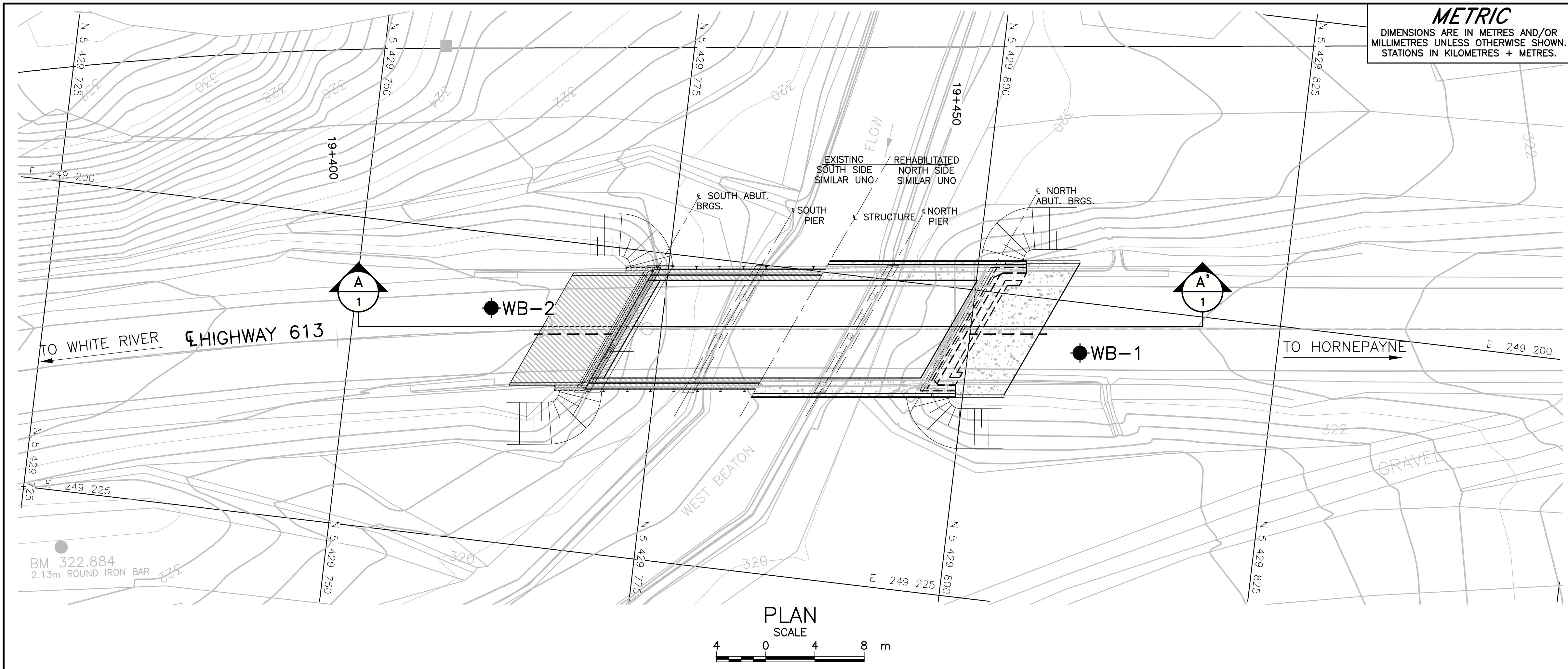
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

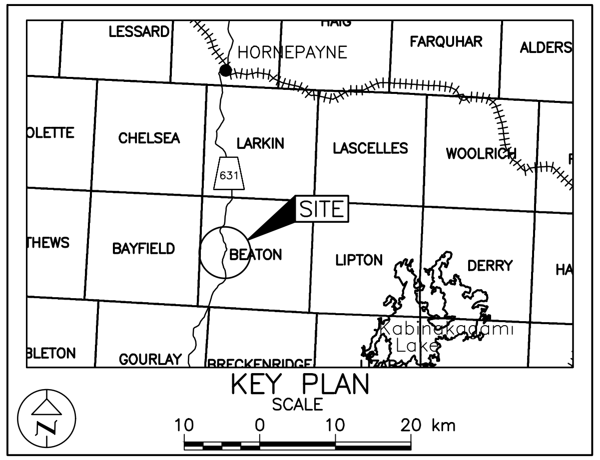
CONT No.
WP No. 5569–09–01

SHEET

HWY 631
WEST BEATON RIVER BRIDGE
LAT. 49.003801, LONG. –84.759927
BORHOLE LOCATIONS AND SOIL STRATA

GOLDER





LEGEND

●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▽	WL upon completion of drilling

BOREHOLE CO–ORDINATES (NAD83 MTM ZONE 13)

No.	ELEVATION	NORTHING	EASTING
WB–1	322.8	5429809.1	249204.1
WB–2	323.0	5429761.1	249206.1

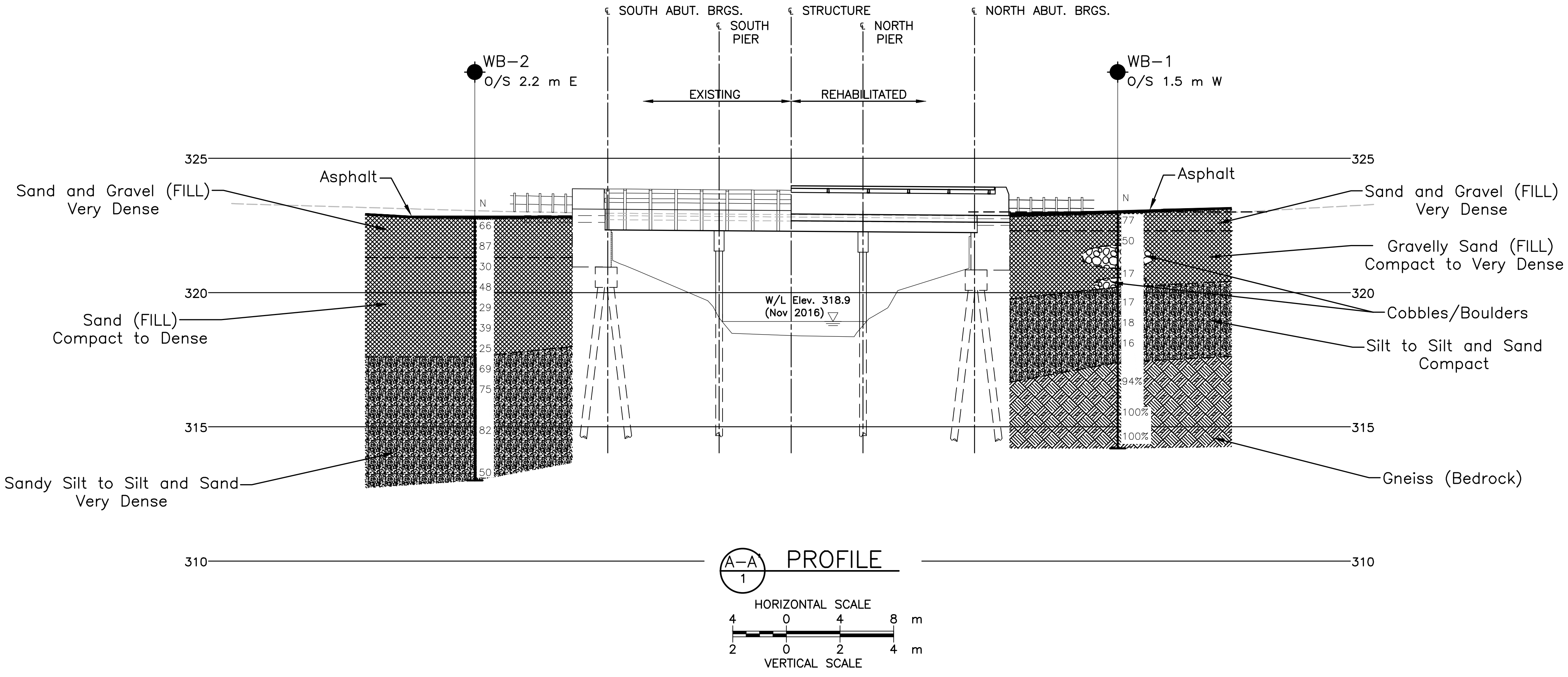
NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by LEA Consulting LTD., drawing file nos. 17197–West Beaton GA–S1.dwg, received OCT 16, 2017. General arrangement plan file no. 17197–West Beaton–R01–General Arrangement.dwg, received APR 04, 2018.



A–A 1 PROFILE

A.J.K. BOM
100075715
APR 04, 2018

J.M.A.COSTA
APR 04, 2018

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 42F–52

HWY. 631		PROJECT NO. 1661607	DIST. .
SUBM'D. TB	CHKD. .	DATE: 4/4/2018	SITE: 38N–008
DRAWN: TB	CHKD. AB	APPD. JMAC	DWG. 1

PLOT DATE: April 4, 2018
FILENAME: S:\Clients\MTO\1661607 LEA_5015–E–0049_NE Region\99_PROJ\1661607_LEA\40_PROD\West_Beaton\1661607–0006–BG–0001.dwg

Photograph 1: West Beaton River Bridge
North Approach Looking South (August 2017)

Photograph 2: West Beaton River Bridge
North Approach Looking North (August 2017)

APPENDIX A

Record of Boreholes

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan\delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes:
1 $\tau = c' + \sigma' \tan\phi'$
2 shear strength = (compressive strength)/2

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index Relative Density	N Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

Golder Associates

RECORD OF BOREHOLE No WB-1

1 OF 2 **METRIC**

PROJECT 1661607
W.P. 5569-09-01
DIST ___ HWY 631
DATUM GEODETIC
LOCATION N 5429809.1; E 249204.1 MTM ZONE 13 (LAT. 49.004012; LONG. -84.759939)
BOREHOLE TYPE NW Casing, Wash Boring and NQ Coring
DATE August 19, 2017
ORIGINATED BY MR
COMPILED BY TB
CHECKED BY AB

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – "N" VALUES	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P – NATURAL MOISTURE CONTENT w – LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
322.8 0.0	TOP OF ASPHALT ASPHALT (50 mm)								
0.1	Sand and gravel to gravelly sand (FILL) Very dense Brown Moist	1	SS	77					
322.1 0.7	Gravelly sand (FILL) Compact to very dense Brown Moist to wet Cobbles and/or boulder encountered between 1.4 m and 1.9 m depth and between 2.6 m and 2.8 m depth.	2	SS	50	322		○		22 70 (8)
		-	RC	-	321				
		3	SS	17					
		-	RC	-					
320.0 2.8	SILT, trace sand, trace gravel Compact Grey Wet	4	SS	17	320				
		5	SS	18	319		○		0 5 88 7
318.2 4.6	SILT and SAND, trace gravel Compact Grey Wet	6	SS	16	318				
317.2 5.6	GNEISS (BEDROCK) Bedrock cored from 5.6 m depth to 8.8 m depth. For coring details see Record of Drillhole WB-1.	1	RC	REC 100%	317				RQD = 94%
		2	RC	REC 100%	316 315				RQD = 100%
		3	RC	REC 100%					RQD = 100%
314.0 8.8	END OF BOREHOLE Note: 1. Wash water level at ground surface (Elev. 323.0 m) inside casing before bedrock coring. Not representative of stabilized condition.				314				

+3, ×3: Numbers refer to Sensitivity   ○3% STRAIN AT FAILURE

PROJECT: 1661607

RECORD OF DRILLHOLE: WB-1

SHEET 2 OF 2

LOCATION: N 5429809.1; E 249204.1
MTM ZONE 13 (LAT. 49.004012; LONG. -84.759939)
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: August 19, 2017
DRILL RIG: LF-70
DRILLING CONTRACTOR: George Downing Estate Drilling Ltd

DATUM: GEODETIC

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	
		REFER TO PREVIOUS PAGE	317.2				
5.6	LF-70 NW	GNEISS Strong Medium grained, foliated Grey	317.2 5.6	1	GREY 100%	JNIRRo JNIRRo JNIRRo JNIRRo	UCS = 57 MPa
6.9	LF-70 NQ Coring			2	GREY 100%	JNIRRo JNIRRo	
8.0	LF-70 NQ Coring			3	GREY 100%	JNIRRo	
8.8		END OF DRILLHOLE	314.0 8.8				

Column headings: DEPTH SCALE METRES; DRILLING RECORD; DESCRIPTION; SYMBOLIC LOG; ELEV. DEPTH (m); RUN No.; FLUSH; COLOUR % RETURN; RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX METRES (5 10 15 20); DISCONTINUITY DATA: B Angle (0 90 180 270), DIP w.r.t. CORE AXIS (0 30 60 90), TYPE AND SURFACE DESCRIPTION, Jr, Ja, Jn; HYDRAULIC CONDUCTIVITY k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.

Depth scale: 6 7 8 9 10 11 12 13 14 15 16 17

DEPTH SCALE
1 : 60

LOGGED: MR
CHECKED: AB

RECORD OF BOREHOLE No WB-2

1 OF 1 **METRIC**

PROJECT 1661607

W.P. 5569-09-01

LOCATION N 5429761.1; E 249206.1 MTM ZONE 13 (LAT. 49.00358; LONG. -84.759905)

ORIGINATED BY MR

DIST ____ HWY 631

BOREHOLE TYPE NW Casing and Wash Boring

COMPILED BY TB

DATUM GEODETIC

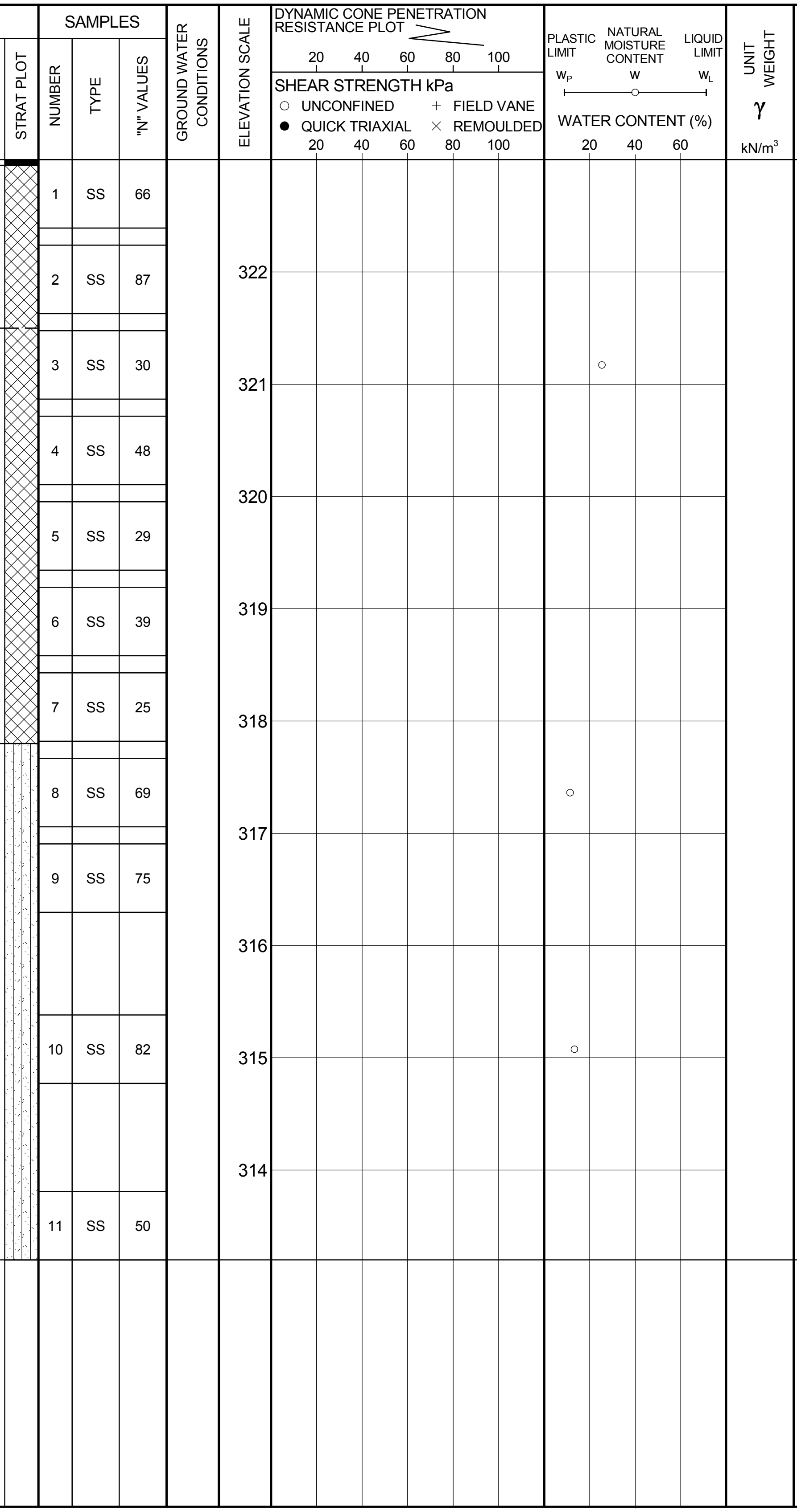
DATE August 20, 2017

CHECKED BY AB

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
323.0	TOP OF ASPHALT							
0.0 0.1	ASPHALT (50 mm)							
	Sand and gravel (FILL) Very dense Brown Moist	1	SS	66				
		2	SS	87				
321.5 1.5	Sand, trace silt, trace gravel (FILL) Compact to dense Brown Moist to wet	3	SS	30	1	91	(8)	
		4	SS	48				
		5	SS	29				
		6	SS	39				
		7	SS	25				
317.8 5.2	Sandy SILT to SILT and SAND, trace gravel Very dense Grey Wet	8	SS	69	1	24	70	5
		9	SS	75				
		10	SS	82	2	34	60	4
		11	SS	50				
313.2 9.8	END OF BOREHOLE							

Note:

1. Wash water level at ground surface (Elev. 322.8 m) inside casing before bedrock coring. Not representative of stabilized condition.



+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results

Golder Associates Ltd.
33 Mackenzie Street, Suite 100
Sudbury, Ontario, Canada P3C 4Y1
Telephone: (705) 524-6861
Fax: (705) 524-1984

TABLE B1 - SUMMARY OF ROCK CORE TEST DATA

PROJECT NO.: **1661607**

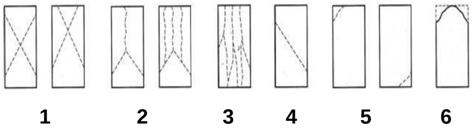
JOB NAME: **West Beaton River Bridge**

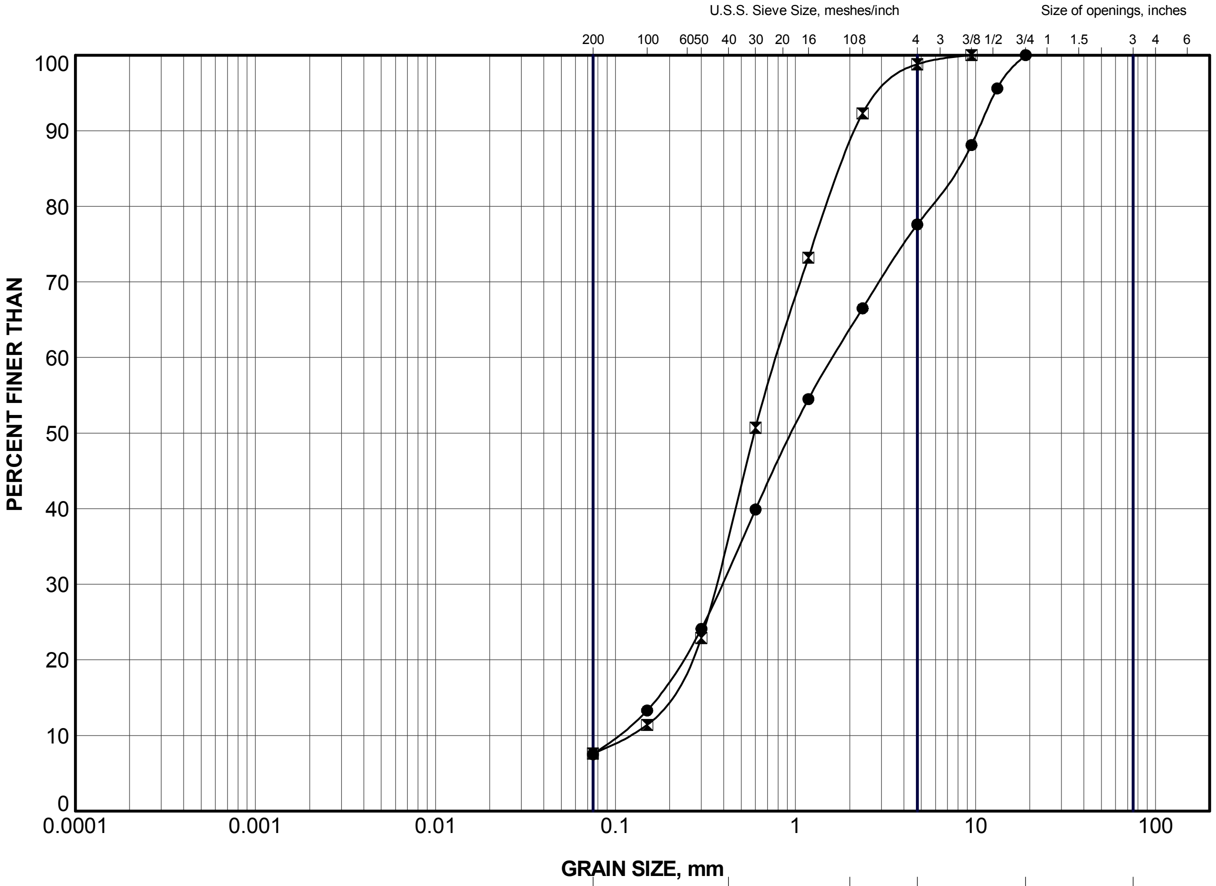
TYPE OF UNIT: **Bedrock Core**

BOREHOLE	**WB-1**					
GOLDER LAB #	**C1535**					
DATE TESTED	**Sept. 19, 2017**					
TESTED BY	JP					
DEPTH OF TESTED CORE (m)	6.6					
LENGTH (mm)	100.2					
DIAMETER (mm)	47.0					
DENSITY (kg/m3)	**2704**					
COMPRESSIVE STRENGTH (MPa)	**56.9**					
TYPE OF FRACTURE	**3**					

Checked by : AB

Type of Fracture





CLAY AND SILT	SAND SIZE			GRAVEL SIZE		Cobble Size
	fine	medium	coarse	fine	coarse	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WB-1	2	321.7
⧗	WB-2	3	321.2

PROJECT

HIGHWAY 631
WEST BEATON RIVER BRIDGE

TITLE

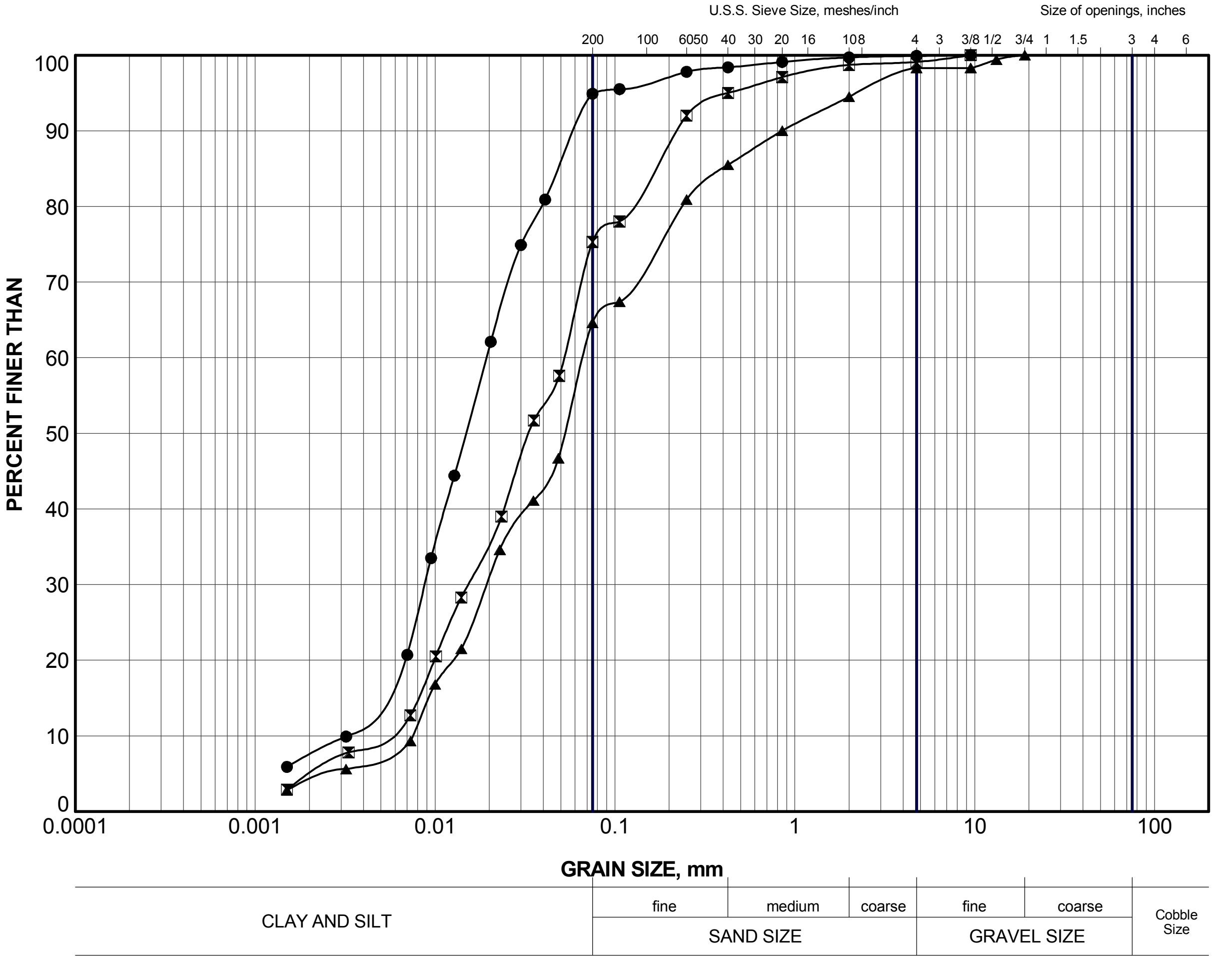
GRAIN SIZE DISTRIBUTION

SAND (FILL); and SAND and GRAVEL (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1661607	FILE No.	1661607.GPJ
DRAWN	TB	Oct 2017	SCALE N/A	REV.
CHECK	AB	Oct 2017		
APPR	JMAC	Oct 2017	**FIGURE B1**	

SUD-MTO GSD (2016) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WB-1	5	318.7
⊠	WB-2	8	317.4
▲	WB-2	10	315.1

PROJECT

HIGHWAY 631
WEST BEATON RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION
SILT to SILT and SAND

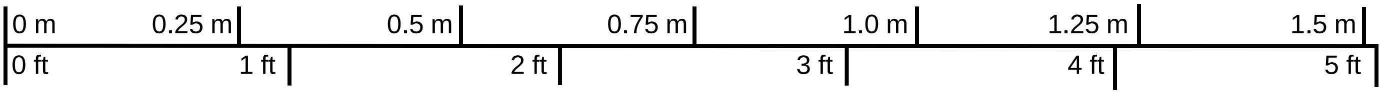
Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1661607	FILE No.	1661607.GPJ
DRAWN	TB	Oct 2017	SCALE N/A	REV.
CHECK	AB	Oct 2017		
APPR	JMAC	Oct 2017	**FIGURE B2**	

SUD-MTO GSD (2016) GLDR_LDN.GDT

Borehole WB-1

Box 1: 5.6 m – 8.8 m



Scale

PROJECT					
Highway 631 West Beaton River Bridge					
TITLE					
Bedrock Core Photograph					
Golder Associates	PROJECT No.	1661607-R05		FILE No. ----	
	DESIGN	TB	OCT 17	SCALE NTS	REV.
	CADD	-- --		**FIGURE B3**	
	CHECK	AB	OCT 17		
	REVIEW	JMAC	OCT 17		

REVISION DATE: Oct, 2017 BY: AB Project: 1661607-R05

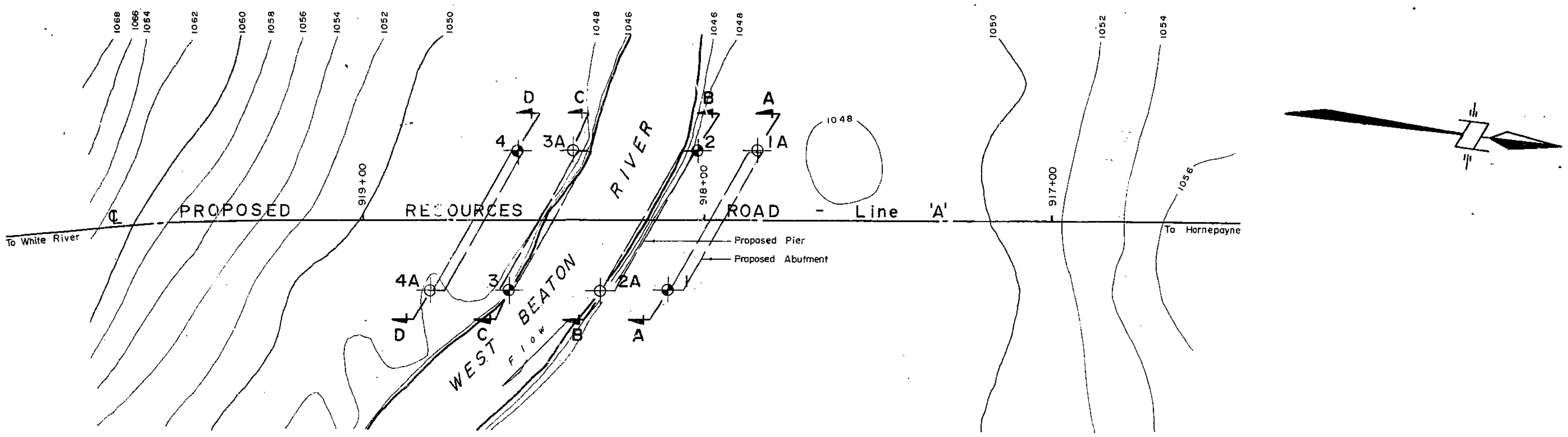
APPENDIX C

Borehole Location Plan/Profile Drawing (GEOCRES 42F-001) and Select Structural Drawings

663900 E
5430100 N

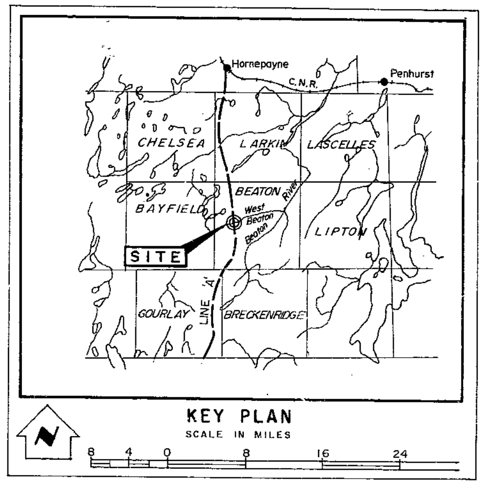
16

42F2



P L A N

SCALE: 1" = 20 Feet



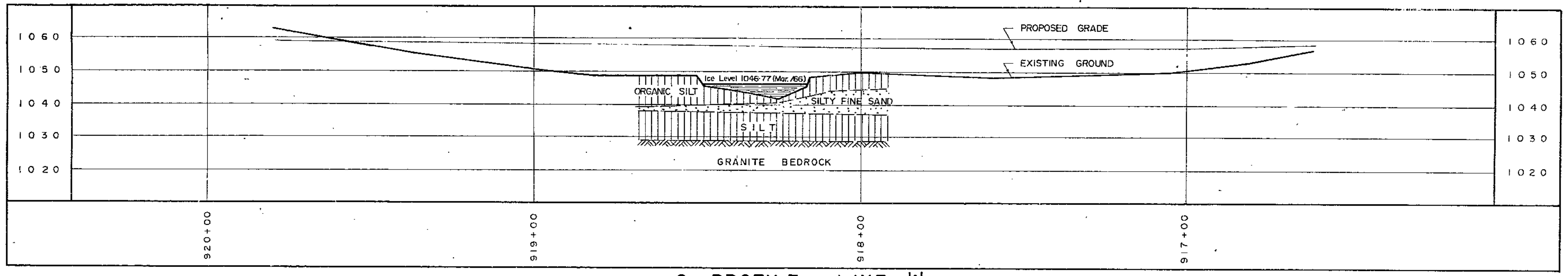
KEY PLAN

SCALE IN MILES

LEGEND

●	Bore Hole
⊕	Cone Penetration Hole
◐	Bore & Cone Penetration Hole
▼	Water Levels established at time of field investigation. (Mar. 10, 1966)

NO.	ELEVATION	STATION	OFFSET
1	1049·6	918 + 10	20' LT.
1A	1049·5	917 + 85	20' RT.
2	1049·3	918 + 02	20' RT.
2A	1046·5	918 + 30	20' LT.
3	1048·6	918 + 56	20' LT.
3A	1048·0	918 + 38	20' RT.
4	1048·1	918 + 54	20' RT.
4A	1048·1	918 + 79	20' LT.



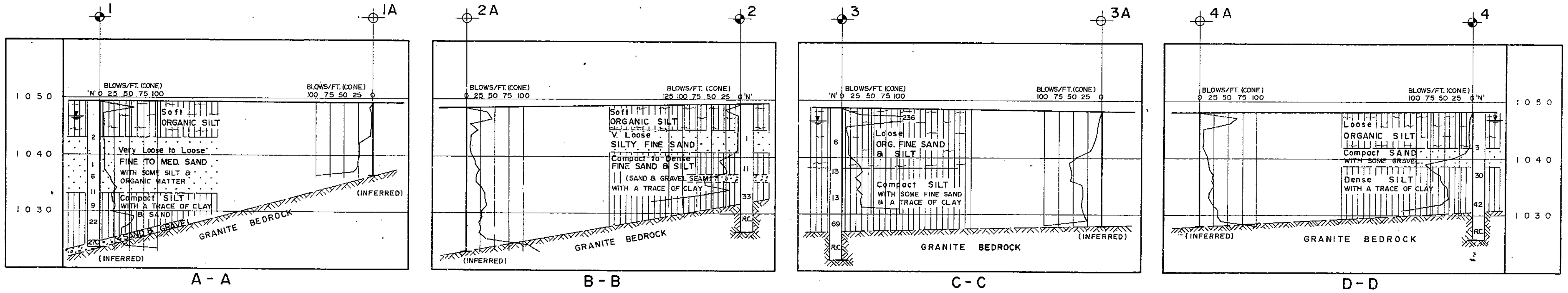
℄ PROFILE - LINE 'A'

SCALE 1" = 20 Feet

- NOTE -

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

PRINT RECORD		
NO.	FOR	DATE



A - A   B - B   C - C   D - D

SECTIONS

SCALE: 1" = 10 Feet

Ref. Nº E - 4208 - 1

REVISIONS		
DATE	BY	DESCRIPTION

DOMINION SOIL INVESTIGATION LIMITED

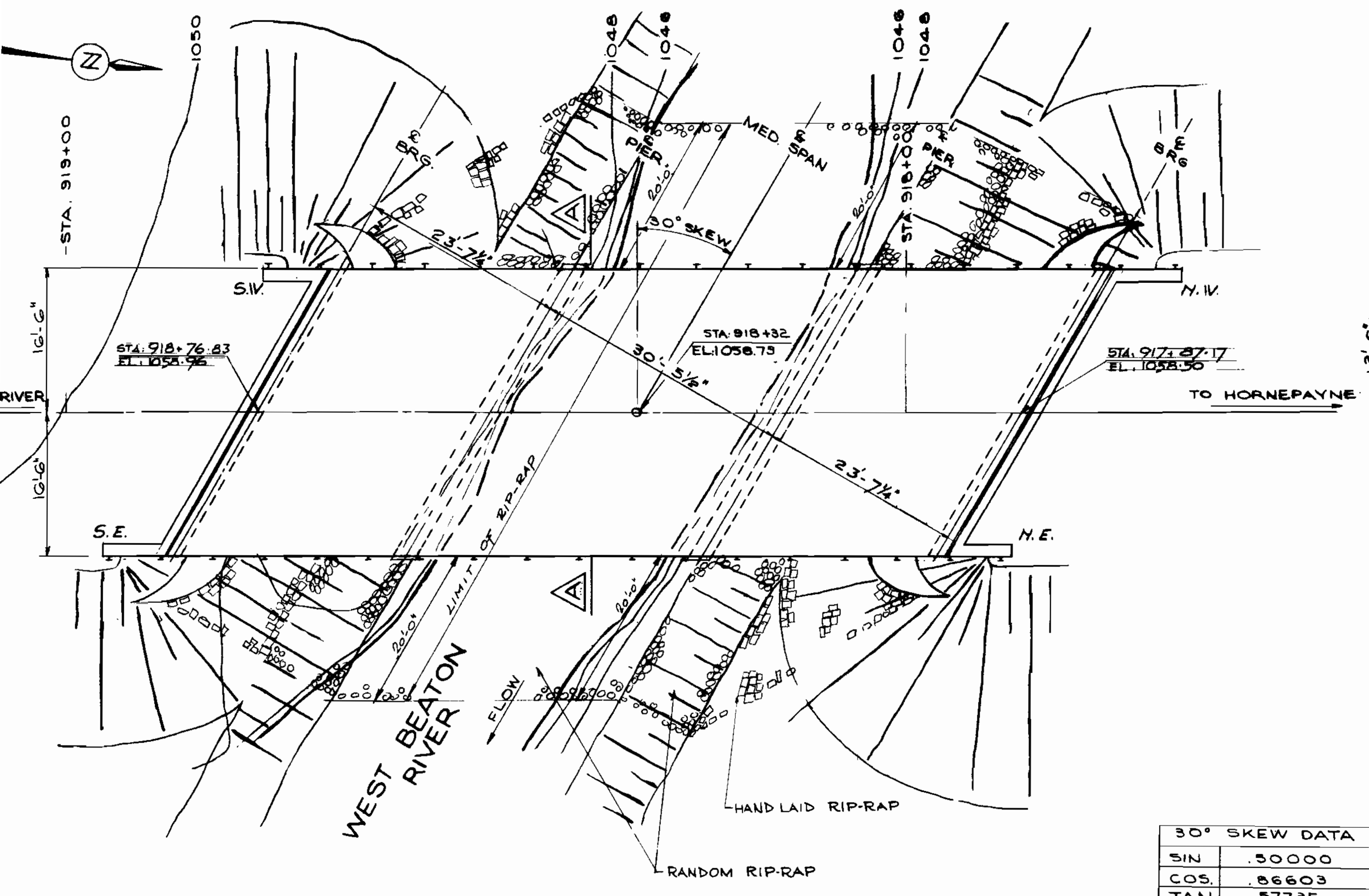
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

PROPOSED CROSSING AT
WEST BEATON RIVER
AND
PROPOSED ACCESS ROAD - Line 'A'

KING'S HIGHWAY NO. ______ DIST. NO. 18
DIST. ALGOMA
TWP. BEATON LOT ______ CON. ______

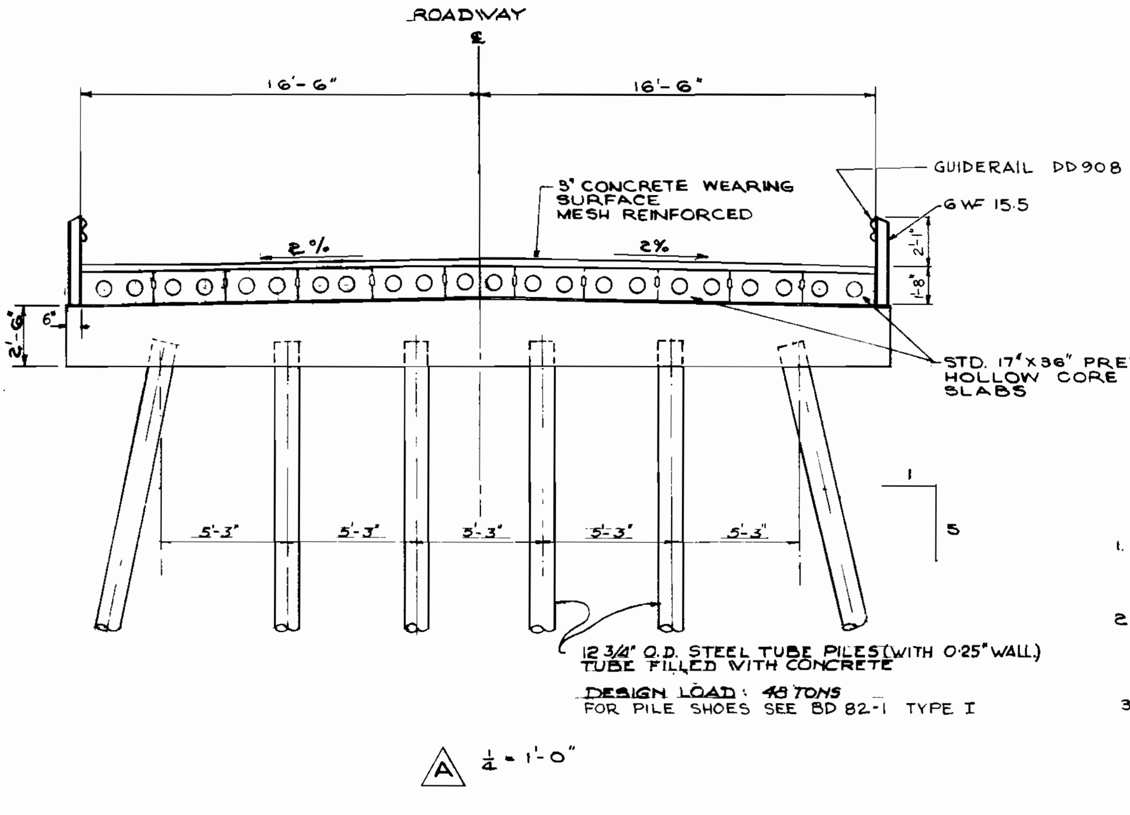
BOREHOLE LOCATIONS & SOIL STRATA

SUBM'D. I. L.	CHECKED	W.P. NO. 145 - 64	DRAWING NO. 6 - 3 - 3
DRAWN D. M.	CHECKED	JOB NO. 6 - 3 - 3	
DATE MAR. 24, 1966		SITE NO.	BRIDGE DRAWING NO.
APPROVED ______ PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	



PLAN
SCALE 1" = 10'-0"

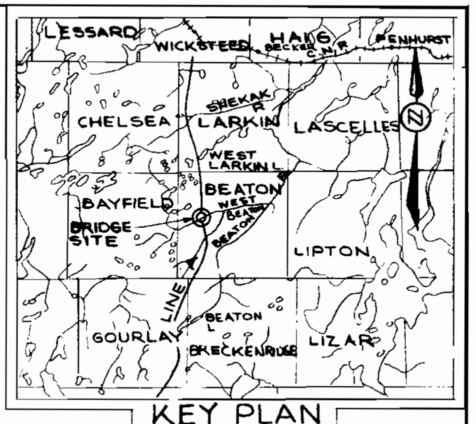
30° SKEW DATA	
SIN	.50000
COS.	.86603
TAN	.57735
SEC	1.15470



12 3/4" O.D. STEEL TUBE PILES (WITH 0.25" WALL.) TUBE FILLED WITH CONCRETE

DESIGN LOAD: 48 TONS
FOR PILE SHOES SEE BD 82-1 TYPE I

A ¼ = 1'-0"



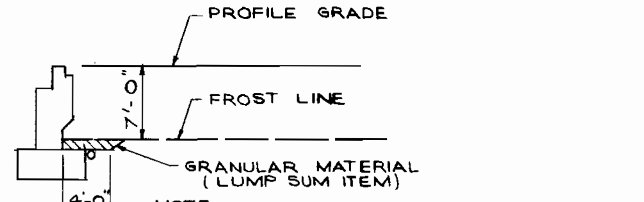
KEY PLAN
SCALE 1" = 8 MILES

NOTES

1. CLASS OF CONCRETE
   DECK - 4000 P.S.I.
   PRESTRESSED CONCRETE SLABS - 5000 P.S.I.
   ALL OTHER - 3000 P.S.I.
2. CLEAR COVER ON REINFORCING STEEL

FOOTINGS	3"
ABUTMENTS	3"
SIDEWALKS	1½"
PARAPET WALLS	1½"

3. THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH TOLERANCES OF ⅛" ±

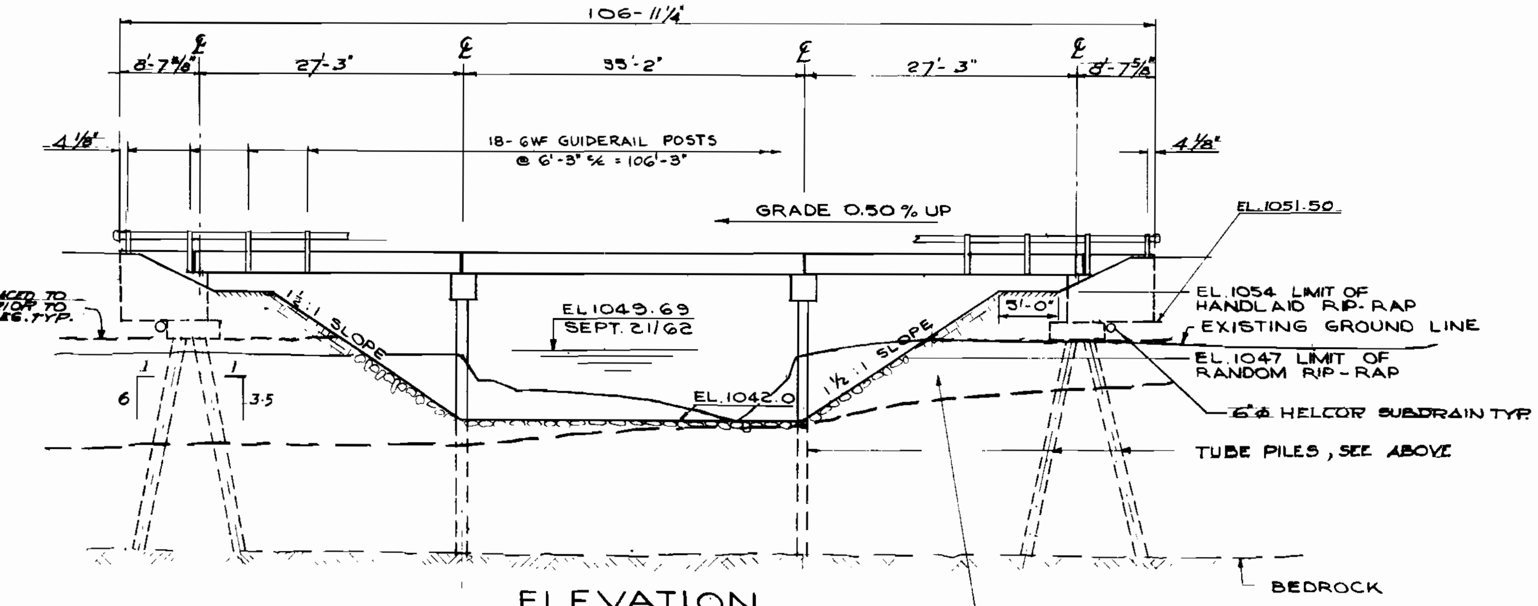


NOTE
SECTION PERPENDICULAR TO ABUTMENT. LATERAL LIMITS INSIDE FACE TO INSIDE FACE OF WING WALLS

DETAILS OF MINIMUM GRANULAR BACKFILL REQUIRMENT

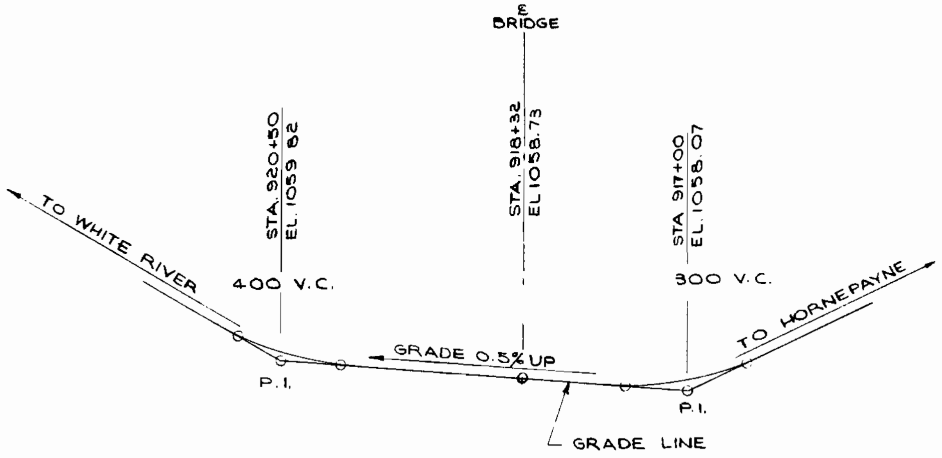
LIST OF DRAWINGS:

5920 - 1 GENERAL
" -2 BOREHOLE LOCATIONS & SOIL STRATA
" -3 FOUNDATION LAYOUT & PILE CAP
" -4 28'-0" PRESTRESSED CONC. SLAB UNITS
" -5 35'-0" " " "
" -6 ABUTMENT & WINGWAL DETAILS
" -7 STANDARDS DETAILS



ELEVATION
SCALE 1" = 10'-0"

ORGANIC MATERIAL TO BE SUBEXCAVATED AND REPLACED WITH GRANULAR MATERIAL (TYP.)



PROFILE GRADE
N.T.S.

D. S. MEDER

REVISIONS		
DATE	BY	DESCRIPTION

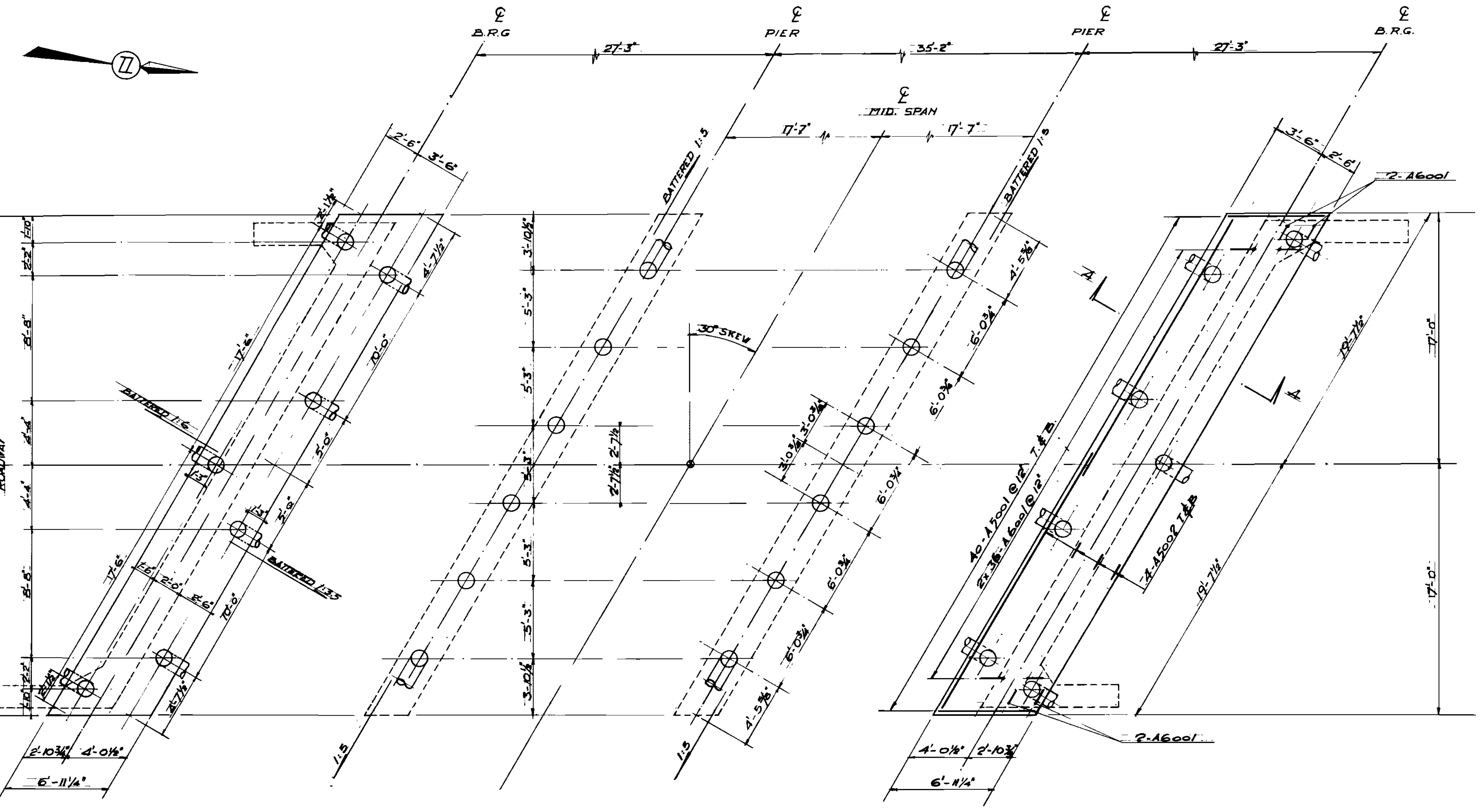
DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

WEST BEATON RIVER BRIDGE

KING'S HIGHWAY No. 631 DIST. No. 16
CO. ALGOMA
TWP. BEATON LOT CON.

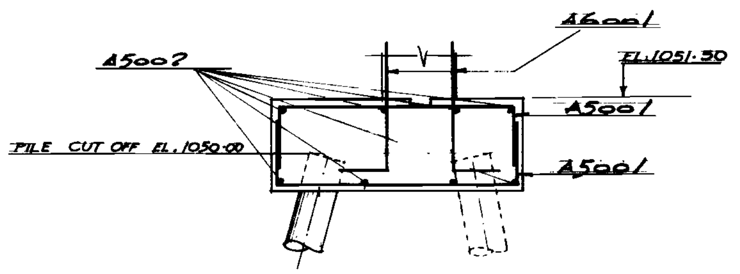
GENERAL

APPROVED	BRIDGE ENGINEER			SITE No. 38N-8	W.P. No. 145-64
DESIGN	D.S.M.	CHECK	W.T.H.	CONTRACT Nos.	67-188
DRAWING	P.A.H.	CHECK	D.S.M.	DRAWING No.	D-5920-1
DATE	APR. 67	LOADING	HS20-44		



FOUNDATION LAYOUT

SCALE: 1/4" - 1'-0"



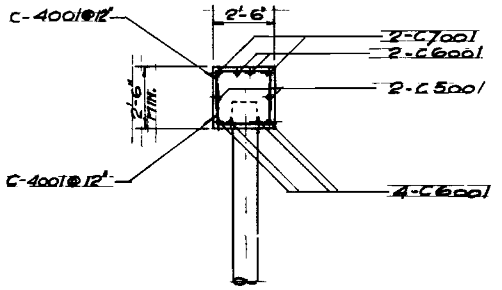
TYPICAL SECTION OF ABUTMENT A - A

PILE SCHEDULE

12 PILES	28'-0" LONG	PIERS
14 PILES	22'-0" LONG	ABUTM.

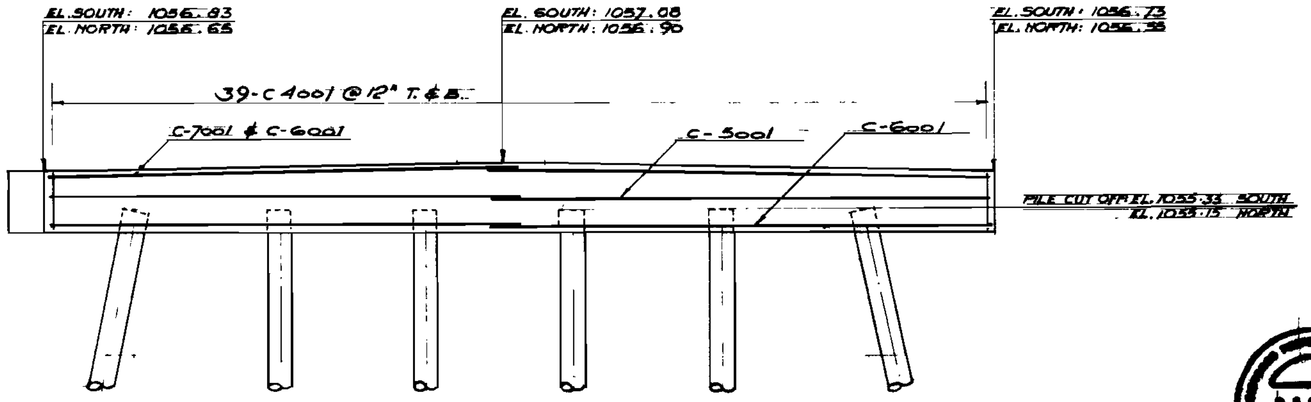
NOTES:

PILE SPACING AT ABUTMENTS TO BE MEASURED AT UNDERSIDE OF FOOTINGS.
PILE SPACING AT PIERS TO BE MEASURED AT UNDERSIDE OF DECK PIER CAPS.



TYPICAL SECTION OF PILE CAP

SCALE: 1/4" = 1'-0"



ELEVATION OF PILE CAP

SCALE: 1/4" = 1'-0"

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

WEST BEATON RIVER BRIDGE

KING'S HIGHWAY No. 631 DIST. No. 16
CO. ALGOMA
TWP. BEATON LOT CON.

FOUNDATION LAYOUT & PILE CAP

APPROVED
BRIDGE ENGINEER

DESIGN	D.S.M.	CHECK	W.T.H.	SITE No. 38N-8	W.P. No. 145-64
DRAWING	A.J.A.	CHECK	D.S.M.	CONTRACT Nos.	67-188
DATE	APR. 67	LOADING	HS20-44	DRAWING No.	D-5920-3

APPENDIX D

Non Standard Special Provisions

OBSTRUCTIONS

Non-Standard Special Provision

The Contactor is hereby notified that the embankment fill at the site of the West Beaton River bridge should be expected to contain cobbles and/or boulders, as encountered at one of the boreholes advanced at this site, which could affect excavations and the installation of temporary shoring and roadway protection systems. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for sub-excavation and installation of the foundation and temporary shoring and roadway protection systems.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

UNWATERING OF STRUCTURE EXCAVATION - Item No.

Notice to Contractor

Construction at the abutments for the existing bridge may require excavations to extend below the groundwater level and the adjacent river water level. The embankment fill and silt to sandy silt within the excavation may slough, run, boil or cave into the excavation unless appropriate groundwater controls are in place. The Contractor is to design and install an appropriate excavation protection and unwatering system to enable construction and prevent disturbance to the founding soils for the abutment pile caps.

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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