



FOUNDATION INVESTIGATION AND DESIGN REPORT

West Beaton River Bridge - Site No. 38N-0008/B0
Highway 631, Township of Beaton, Algoma District
MTO Contract DB 2021-5168

Submitted to:

Facca Incorporated

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Submitted by:

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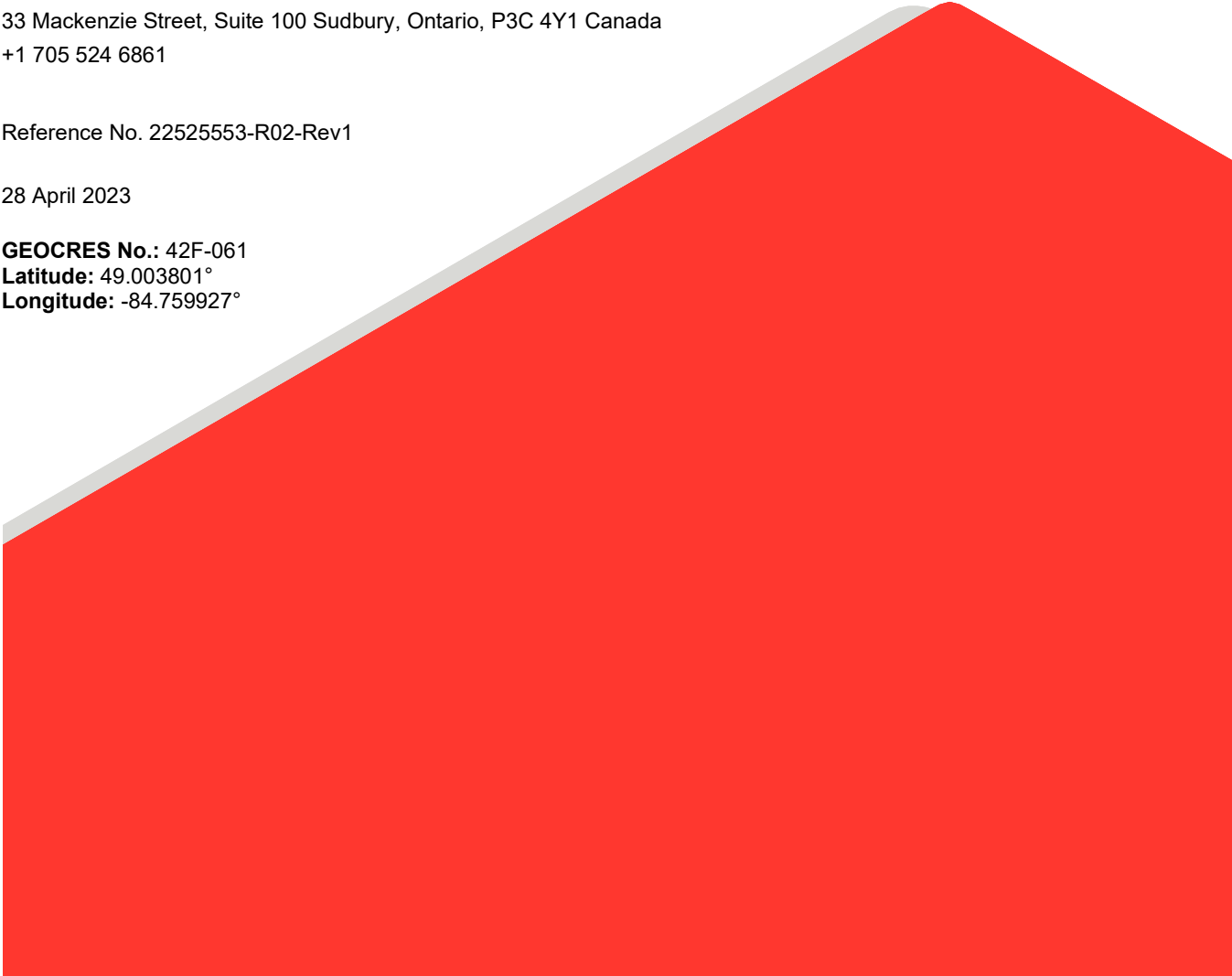
Reference No. 22525553-R02-Rev1

28 April 2023

GEOCRES No.: 42F-061

Latitude: 49.003801°

Longitude: -84.759927°

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1 PDF Copy: WSP Golder

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PART A

**FOUNDATION INVESTIGATION REPORT
WEST BEATON RIVER BRIDGE REHABILITATION
SITE NO. 38N-0008/B0
HIGHWAY 631, TOWNSHIP OF BEATON, ALGOMA DISTRICT
MTO CONTRACT DB 2021-5168**

1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd., now a member of WSP Canada Inc., hereafter referenced as WSP Golder) has been retained by Facca Incorporated (Facca), 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-0008/B0), located on Highway 631, 73 km North of Highway 17 in White River, Township of Beaton, Algoma District. 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.

WSP Golder was previously retained by LEA Consulting Ltd. (LEA) in 2017 on behalf of the MTO to carry out Foundation Investigation and Design services for the detail design of the replacement structure, as outlined in the Foundation Investigation and Design Report dated 27 June 2018 (Golder Report No. 1661607-R05, MTO GEOCRETS No. 42F-52).

2.0 SITE DESCRIPTION AND BACKGROUND INFORMATION

The existing West Beaton River Bridge consists of an approximately 27.3 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 the original structural design drawings for this bridge (Contract 67-188, Drawing Nos. D-5920-1 to -3), it is understood that the existing abutments and piers are supported on driven steel tube piles. The bridge deck is at approximately 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 19 August and 20 August 2017, during which time two boreholes (WB-1 and WB-2) were advanced at the locations shown on Drawing 1. Traffic protection was performed in general accordance with MTO's Ontario Traffic Control Manual Book 7 – Temporary Conditions.

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, with water obtained from the river. 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 WSP 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 WSP Golder's Sudbury Laboratory for further examination and laboratory testing. Index and classification tests consisting of water content and grain size distribution were carried out on selected soil samples, in accordance with MTO and/or ASTM standards as applicable. The results of the geotechnical 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. A uniaxial 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, 2006¹). 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 (ISRM²) standard classification system.

The borehole locations and elevations were measured and surveyed by a member of WSP Golder's 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 No.	Location (MTM NAD 83, Zone 13)		Location World Geodetic System 84		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)	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.

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.

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.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes, together with the results of the geotechnical 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.

The boreholes were drilled through the existing Highway 631 embankment and encountered asphalt underlain by sand and gravel to sand fill containing cobbles and boulders, overlying a deposit of compact to very dense sandy silt to silt and sand, in turn underlain by gneiss bedrock. A more detailed summary of the subsurface conditions as encountered in Boreholes WB-1 and WB-2 is presented below.

4.2.1 Fill and Native Soils

Below a 50 mm thick asphalt layer, the fill and native soil deposits encountered in the boreholes are summarized below.

Deposit/Layer Description	Boreholes	Surface Elevation (m)	Thickness (m)	SPT "N" – Values (blows/0.3 m)	Geotechnical Laboratory Testing
				Relative Density	
(FILL) Sand and gravel over sand to gravelly sand ¹	WB-1, WB-2	322.9 and 322.7	2.7 to 5.1	N = 17 – 87	w = 6% and 25% 2 – M (Fig. B1)
				Compact to very dense	
Silt to Sandy Silt to Silt and Sand	WB-1, WB-2	320.0 and 317.8	2.8 to 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.

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

4.2.2 Gneiss Bedrock

Bedrock was cored in Borehole WB-1 and the depth/elevation of the 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 uniaxial 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, were at ground surface; however, this is not considered representative of the stabilized 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

This Foundation Investigation Report was prepared by Mr. André Bom, P.Eng. Ms. Lisa Coyne, P.Eng., a Fellow and MTO Designated Foundations Contact for WSP Golder, conducted an independent technical review and quality control audit of this report.

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

Signature Page



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AB/LCC/ca

[https://golderassociates.sharepoint.com/sites/163211/project files/6 deliverables/ph 1000-fdns and pvmts/03-west beaton/final-rev1/22525553-r02-r-rev1-1000-west beaton river bridge rehabilitation fdr 28apr_23.docx](https://golderassociates.sharepoint.com/sites/163211/project%20files/6%20deliverables/ph%201000-fdns%20and%20pvmnts/03-west%20beaton/final-rev1/22525553-r02-r-rev1-1000-west%20beaton%20river%20bridge%20rehabilitation%20fdr%2028apr_23.docx)

PART B

FOUNDATION DESIGN REPORT
WEST BEATON RIVER BRIDGE REHABILITATION
SITE NO. 38N-0008/B0
HIGHWAY 631, TOWNSHIP OF BEATON, ALGOMA DISTRICT
MTO CONTRACT DB 2021-5168

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering recommendations for the proposed rehabilitation of the West Beaton River Bridge. The recommendations presented are based on interpretation of the factual data obtained from the boreholes advanced during the previous (2017) subsurface investigation at this site. The discussion and recommendations presented are intended to provide the Design-Builder with sufficient information to complete the foundation assessment and design for the proposed bridge rehabilitation.

6.1 General

The existing West Beaton River bridge will be rehabilitated, including conversion of the existing bridge abutments to a semi-integral configuration and an approximately 0.6 m widening of the bridge deck to accommodate the proposed half-and-half construction staging.

It is understood from WSP's structural team that the increase in dead load is approximately 48% at each pier, considered the critical case, relative to original design. This increase is greater than the 10% specified in MTO's Bridge Design Bulletin: Design and Evaluation of Foundations, dated August 20, 2013. Accordingly, WSP Golder assessed the factored geotechnical resistances of the existing piles to aid the structural design team in determining if the existing deep foundations supporting the abutments can accommodate the proposed increased loading from the proposed 0.6 m widening of the bridge deck. A summary of this assessment is presented in Section 6.2 of this report.

Temporary protection systems will be required along the approximate mid-line of Highway 631 during construction staging, to support the required excavations for conversion to a semi-integral abutment structure while maintaining a single lane of traffic. Excavations will be required to expose the existing abutments. It is understood that the underside of the existing abutment footings 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; the groundwater level adjacent to the river in the vicinity of the abutments may be similar to or higher than the river water level.

6.2 Geotechnical Resistance of Existing Piles

The following is a summary of MTO's GEOCRE report and previous structural design drawings for this site. It is understood that as-built drawings and pile driving records are not available for the site.

- MTO GEOCRE 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 (selected 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 design 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 GEOCRETS information and WSP 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 from site reviews by others that no concerns of foundation settlement have been observed.

Based on the design drawings and the assumption that the piles at West Beaton River bridge are supported on the strong gneiss bedrock, the factored ultimate geotechnical resistance may be taken as 2,000 kN. In the event that the piles at the south abutment or pier were terminated above bedrock in the very dense silt and sand deposit, the factored ultimate geotechnical resistance may be taken as 1,100 kN. It is understood that at both piers, WSP's structural designers require a factored ultimate geotechnical resistance of 1,250 kN, which can be met provided the piles terminated on bedrock; however, as-built records were not available from MTO to confirm this condition. Accordingly, we have considered the risks and probabilities of the piles reaching bedrock or encountering refusal on obstructions; based on MTO's Boreholes 2 and 3 at the piers, together with the other recent and historic boreholes and DCPTs at the site, the sites soils are considered to have a low risk of obstructions, and therefore it is expected that all piles would have a high probability of readily penetrating the compact to very dense soil to reach the bedrock, thus satisfying the design loading condition. In the event that an individual pile has encountered an obstruction above the bedrock, it is our opinion that a factored ultimate geotechnical resistance of 1,250 kN is locally acceptable; such a value corresponds to an ultimate (unfactored) geotechnical resistance of 2,750 kN, which would represent a resistance factor of 0.45 rather than 0.4 (or a factor of safety of 2.2 rather than 2.5) for selected individual piles. In addition, if an individual pile had met refusal on an obstruction within the overburden above the bedrock, it is likely that the ultimate and factored ultimate geotechnical resistance of that pile would be higher than the 1,100 kN value assessed above, further increasing our confidence that the design loading requirements are accommodated by the factored geotechnical resistances of the existing piles.

The factored serviceability geotechnical resistances provided above will be greater than the factored ultimate geotechnical resistance for piles founded on the strong gneiss bedrock or piles that have terminated above bedrock in the very dense soil. In completing this assessment, the following conditions are considered applicable to the foundations geotechnical resistances assessment:

- In accordance with Section 6.5 of the Canadian Highway Bridge Design Code CAN/CSA S6-19 (CHBDC (2019)) and its Commentary, the proposed bridge and foundation systems are classified as having a "typical consequence level".
- Based on the level of foundation investigation in comparison to the degree of site understanding in Section 6.5 of CHBDC (2019), the level of confidence for design has been taken as a "typical degree of site and prediction model understanding".
- The corresponding consequence factor, $\Psi = 1.0$, and geotechnical resistance factors, $\phi_{gu} = 0.4$ and $\phi_{gs} = 0.8$, from Tables 6.1 and 6.2 of the CHBDC (2019) have been used for the assessment of the geotechnical resistance of the existing foundations.

Based on assessed values in similar soil conditions (CHBDC 2006, Table C6-4), the factored ultimate horizontal passive resistance may be taken as 120 kN per pile. The factored SLS value for horizontal passive resistance may be taken as 40 kN for 10 mm of lateral movement at the pile cap level. Where applicable, group action for lateral loading should be evaluated as outlined in Section C6.11.3.4 of the *Commentary to CHBDC* (2019).

Following excavation and restoration of the site, foundations at this site should be provided with a minimum of 2.6 m of soil cover for frost protection as per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario).

6.3 Excavation and Groundwater Control

The proposed works will require excavations through the embankment fill behind the existing abutments associated with semi-integral abutment conversion. It is understood that this excavation will extend to a depth of approximately 1.3 m below the existing pavement surface. At this depth, the excavation will be maintained above the river level and groundwater level at this site.

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 if applicable, although such condition is not expected at this site. 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. Where there is insufficient space for an open-cut excavation, temporary protection systems will be required as discussed in Section 6.4.

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 temporary protection systems shall be designed and constructed in accordance with DBSP 539 (Temporary Protection Systems). Temporary protection systems shall 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 design of the temporary protection system is the responsibility of the Contractor's temporary works designer; the following geotechnical parameters are provided for guidance based on the subsurface conditions encountered in the boreholes at this site:

Soil Type	Bulk Unit Weight (γ , kN/m ³)	Internal Angle of Friction (ϕ' , degrees)	Undrained Shear Strength (S_u , kPa)	Lateral Coefficient of Earth Pressure ¹		
				K_a (Active)	K_o (At Rest)	K_p ² (Passive)
Sand and Gravel and Sand to Gravelly Sand Fill (Compact to very dense)	20	32	-	0.31	0.50	3.25
Silt to Sandy Silt to Silt and Sand (Compact to very dense)	18	30	-	0.33	0.50	3.0

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 C6.27 of the CHBDC (2019) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

6.4.1 Obstructions

The embankment fill in Borehole WB-1 contains zones of cobbles and/or boulder as noted on the borehole record, and similar obstructions should be expected elsewhere in the fill materials at this site; such obstructions could affect the excavations and installation of temporary protection systems. A Notice to Contractor 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 D.

6.5 Restoration of Approach Embankments

At the time of our foundation investigation in 2017 and in current conditions, the existing embankment have been observed to be performing satisfactorily from a geotechnical perspective, with no visual evidence of global instability or excessive settlement. The existing embankment side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V). Following excavation behind the abutments, the approach embankments should be restored with side slopes oriented no steeper than 2H:1V assuming the use of granular fill or earth fill/borrow material; the factor of safety for global stability of the reconstructed approach embankment side slopes will be greater than 1.5, thus satisfying the requirements of CHBDC (2019).

7.0 CLOSURE

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

Signature Page



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Geotechnical Engineer



Lisa Coyne, P.Eng.
Fellow, MTO Designated Foundations Contact

AB/LCC/ca

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REFERENCES

Canadian Standard Association (CSA) Group. 2019. *Canadian Highway Bridge Design Code*, CAN-CSA-S6-19 (CHBDC 2019) and *Commentary* on CAN-CSA-S6-19.

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.

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.

Ministry of Transportation, Ontario. August 13, 2013. Bridge Design Bulletin: Design and Evaluation of Foundations, date August 20, 2013.

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

ASTM International:

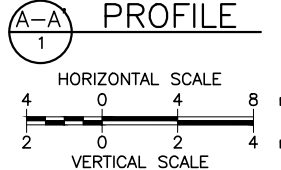
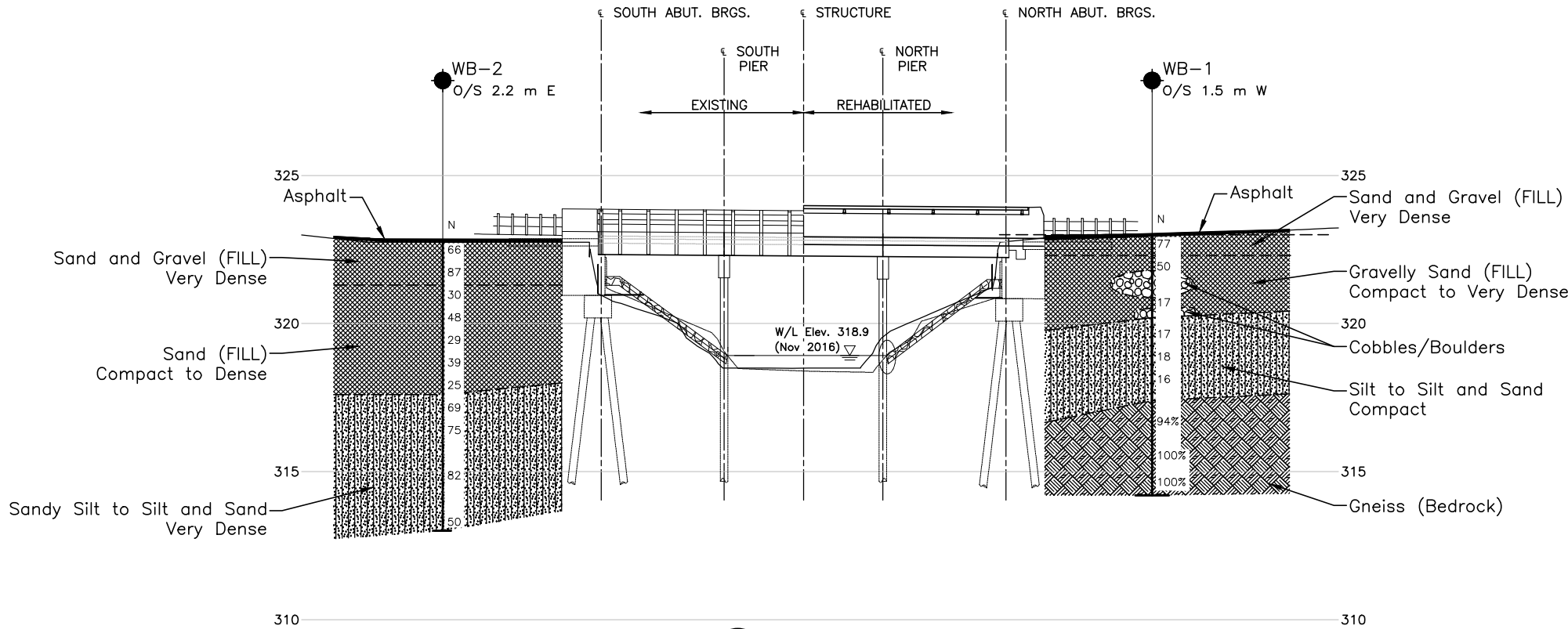
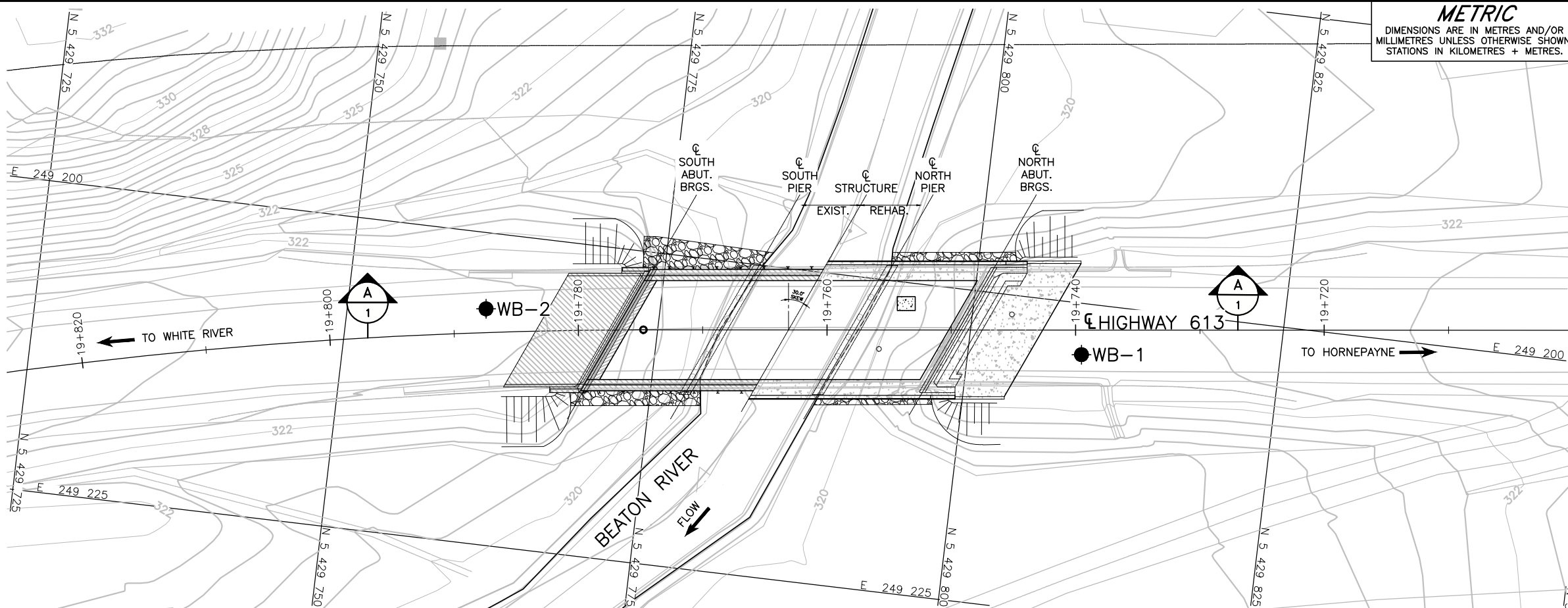
ASTM D1586 Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils

Ontario Provincial Standard Specifications (OPSS) – Provincial Oriented

DBSP 539 Construction Specification for Temporary Protection Systems

Ontario Water Resource Act

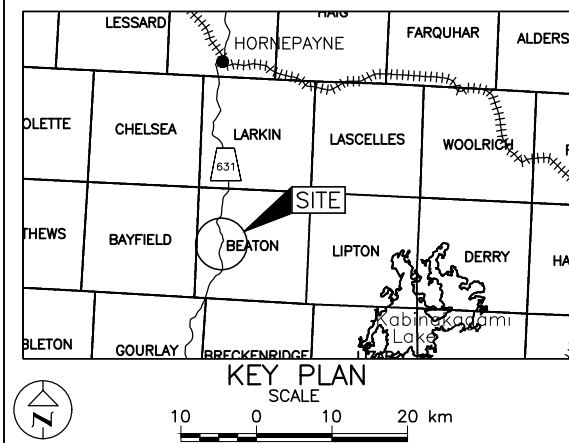
Regulation 903 Wells (as amended)



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

CONT No. DB 2021-5168
WP No.

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



LEGEND

- Borehole - Current Investigation
- Standard Penetration Test Value
- Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 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
- Topography provided in digital format by LEA Consulting, drawing file no. 17197-West Beaton GA-S1.dwg, Received October 16, 2017.
 - General Arrangement provided by WSP, drawing file no. S221-09193-00-303-0001GA.dwg, Received December 12, 2022.
 - Base plan provided in digital format by WSP, drawing file no. x17197 W Beaton Base_Preliminary.dwg, Received December 12, 2022.
 - Alignment provided in digital format by WSP, drawing file no. x 17197 West Beaton Alg Extn.dwg, Received December 12, 2022.

NO.	DATE	BY	REVISION
Geocres No. 42F-061			
HWY. 631	PROJECT NO. 22525553		DIST. .
SUBM'D. AB	CHKD. .	DATE: 3/13/2023	SITE: 38N-0008/B0
DRAWN: TR	CHKD. AB	APPD. LCC	DWG. 1



PHOTOGRAPHS

**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

PROJECT 1661607		RECORD OF BOREHOLE No WB-1				1 OF 2 METRIC											
W.P. 5569-09-01		LOCATION N 5429809.1; E 249204.1 MTM ZONE 13 (LAT. 49.004012; LONG. -84.759939)				ORIGINATED BY MR											
DIST HWY 631		BOREHOLE TYPE NW Casing, Wash Boring and NQ Coring				COMPILED BY TB											
DATUM GEODETIC		DATE August 19, 2017				CHECKED BY AB											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
322.8	TOP OF ASPHALT																
322.1	ASPHALT (50 mm)		1	SS	77												
322.1	Sand and gravel to gravelly sand (FILL) Very dense Brown Moist		2	SS	50												22 70 (8)
320.0	Gravelly sand (FILL) Compact to very dense Brown Moist to wet		-	RC	-												
320.0	Cobbles and/or boulder encountered between 1.4 m and 1.9 m depth and between 2.6 m and 2.8 m depth.		3	SS	17												
320.0	-		-	RC	-												
320.0	SILT, trace sand, trace gravel Compact Grey Wet		4	SS	17												
320.0	-		5	SS	18												0 5 88 7
318.2	SILT and SAND, trace gravel Compact Grey Wet		6	SS	16												
317.2	GNEISS (BEDROCK)		1	RC	REC 100%												RQD = 94%
317.2	Bedrock cored from 5.6 m depth to 8.8 m depth. For coring details see Record of Drillhole WB-1.		2	RC	REC 100%												RQD = 100%
317.2	-		3	RC	REC 100%												RQD = 100%
314.0	END OF BOREHOLE																
314.0	Note: 1. Wash water level at ground surface (Elev. 323.0 m) inside casing before bedrock coring. Not representative of stabilized condition.																

SHEET 2 OF 2

DATUM: GEODETIC

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd

CHECKED: AB

PROJECT 1661607		RECORD OF BOREHOLE No WB-2				1 OF 1 METRIC										
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			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								20	40	60	80	100	20	40		
323.0	TOP OF ASPHALT															
0.9	ASPHALT (50 mm)		1	SS	66											
	Sand and gravel (FILL)		2	SS	87											
	Very dense															
	Brown															
	Moist															
321.5																
1.5	Sand, trace silt, trace gravel (FILL)		3	SS	30											1 91 (8)
	Compact to dense		4	SS	48											
	Brown		5	SS	29											
	Moist to wet		6	SS	39											
			7	SS	25											
317.8																
5.2	Sandy SILT to SILT and SAND, trace gravel		8	SS	69											1 24 70 5
	Very dense		9	SS	75											
	Grey															
	Wet		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.															

APPENDIX B

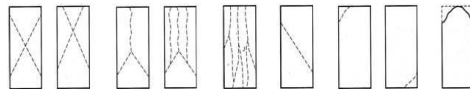
Laboratory Test Results

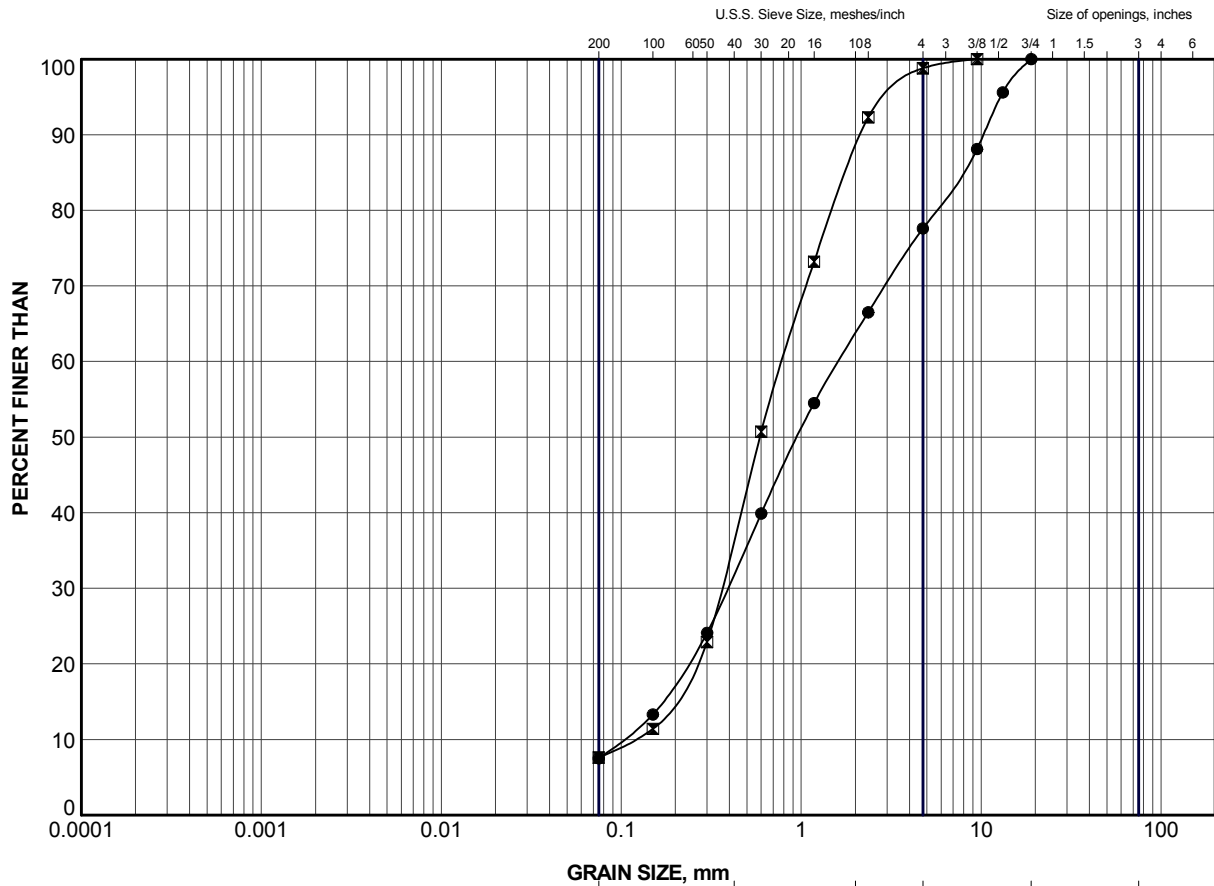
Golder Associates Ltd.

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***1****2****3****4****5****6**



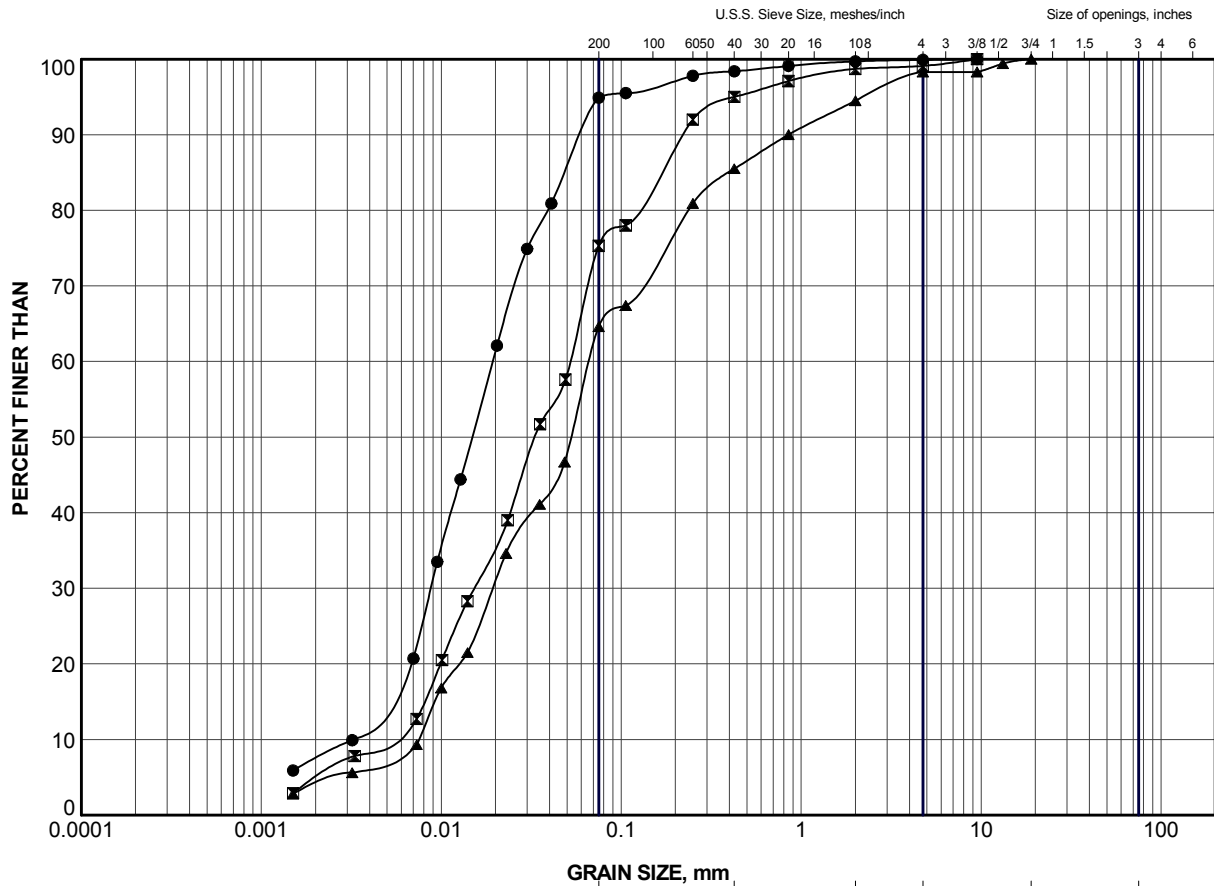
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

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)					
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		





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

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

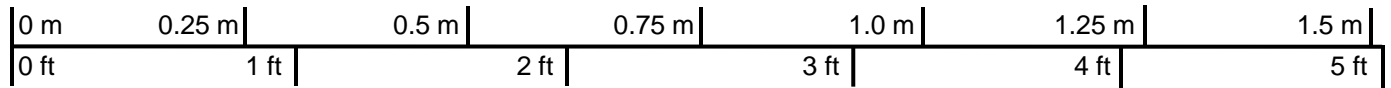
PROJECT No. 1661607			FILE No. 1661607.GPJ		
DRAWN	TB	Oct 2017	SCALE	N/A	REV.
CHECK	AB	Oct 2017	FIGURE B2		
APPR	JMAC	Oct 2017			

Borehole WB-1

TOR (5.6 m)




Box 1: 5.6 m – 8.8 m



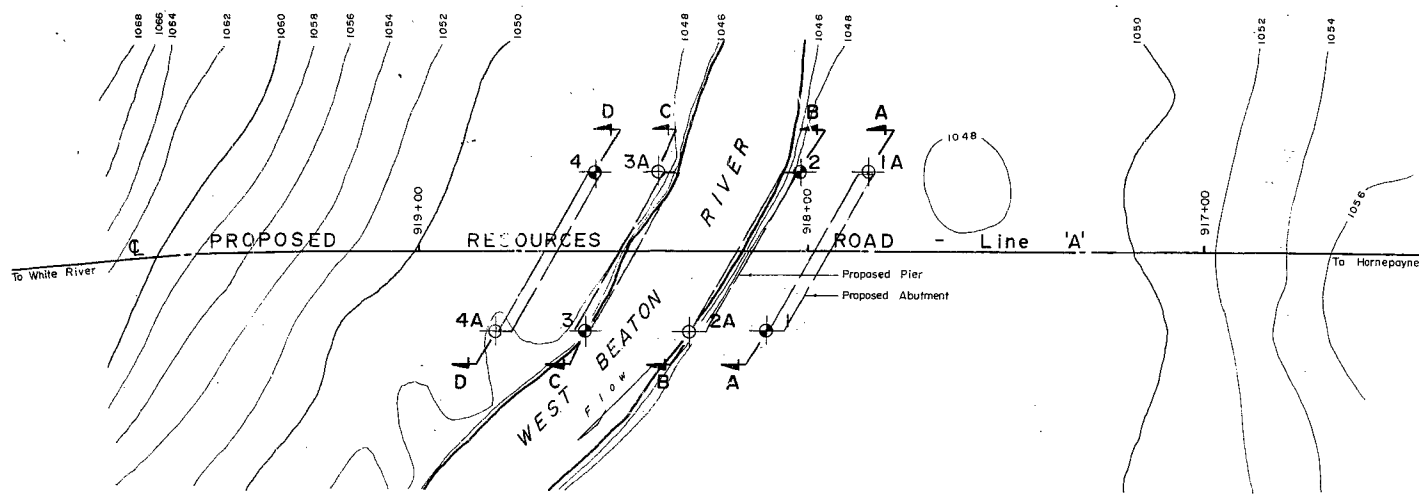
Scale

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

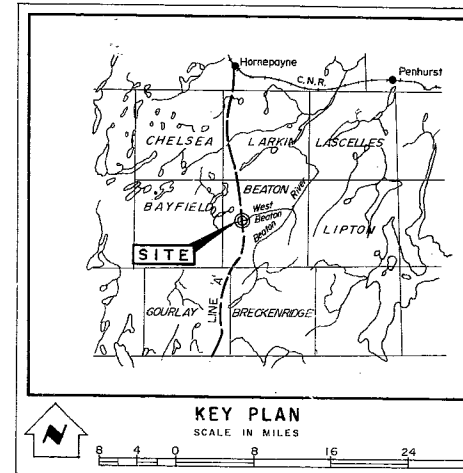
PROJECT					
Highway 631 West Beaton River Bridge					
TITLE					
Bedrock Core Photograph					
			PROJECT No. 1661607-R05		FILE No. ----
			DESIGN	TB	OCT 17
			CADD	--	--
			CHECK	AB	OCT 17
			REVIEW	JMAC	OCT 17
			SCALE NTS REV.		
			FIGURE B3		

APPENDIX C

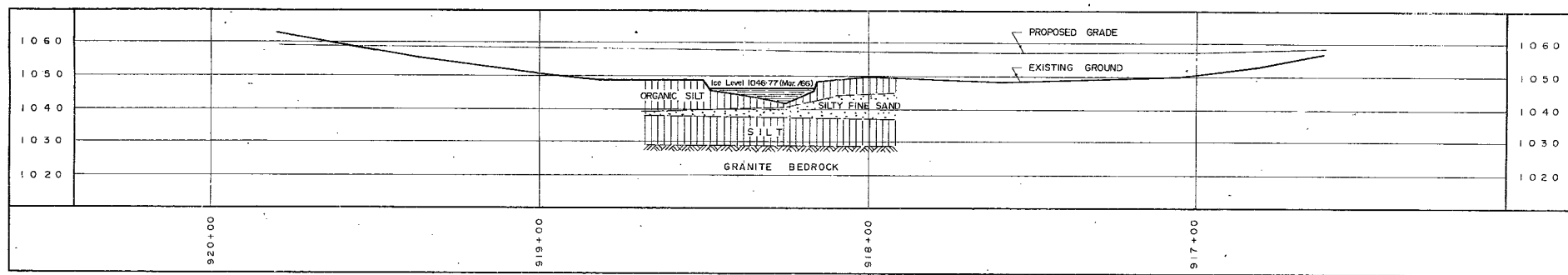
**Borehole Location Plan/Profile
Drawing (GEOCRES 42F-001) and
Select Structural Drawings**



PLAN
SCALE: 1" = 20 Feet



KEY PLAN
SCALE IN MILES



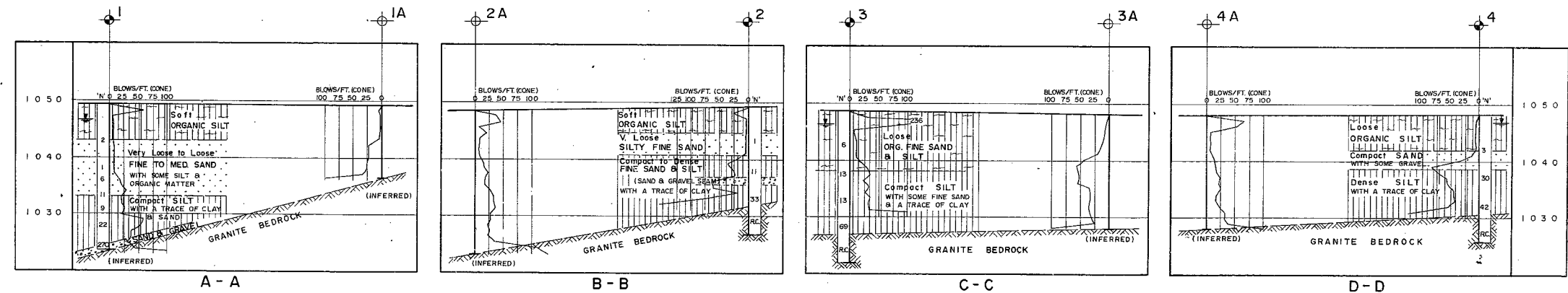
PROFILE - LINE 'A'
SCALE: 1" = 20 Feet

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.

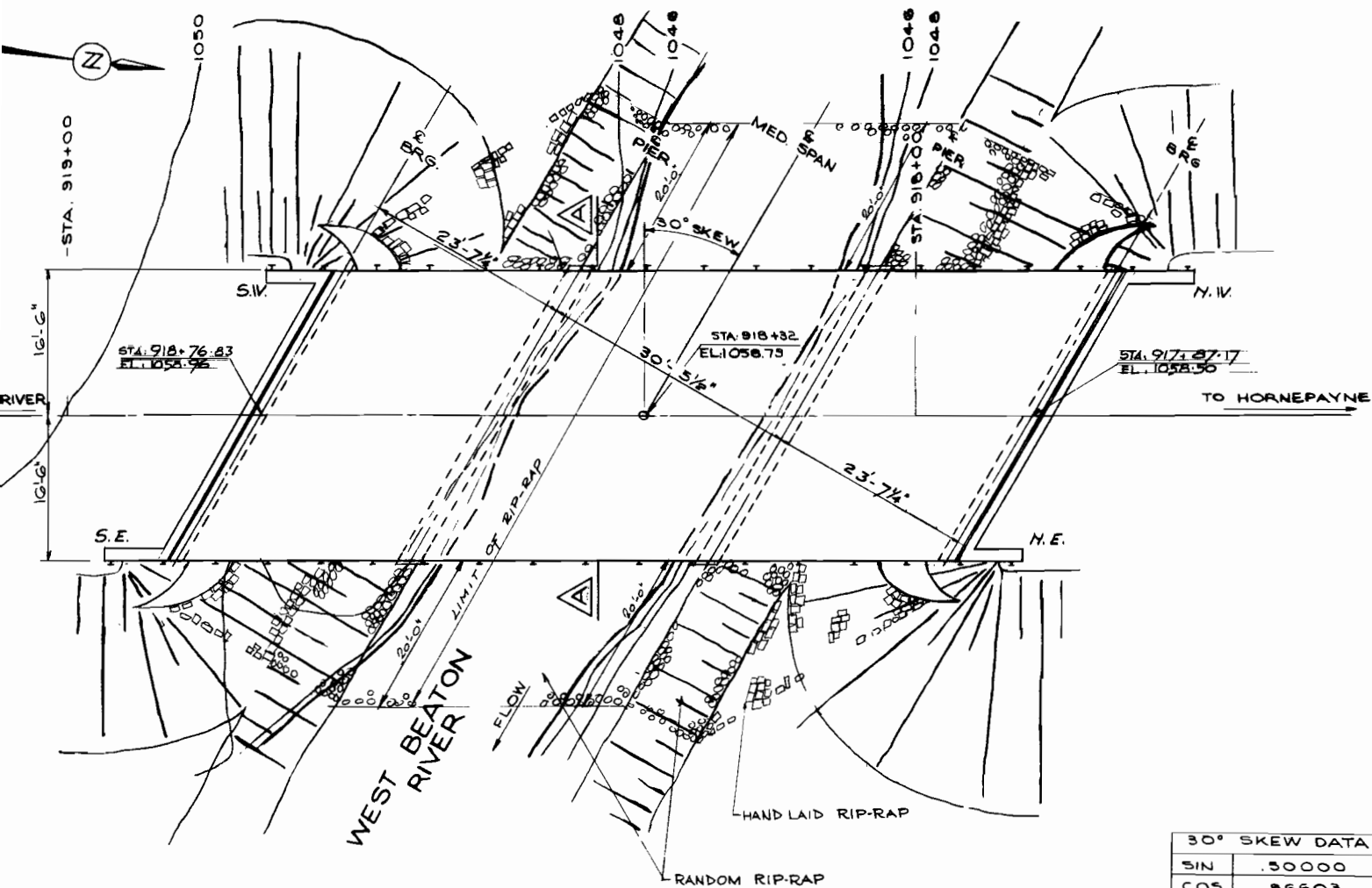
- NOTE -
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

PRINT RECORD		
NO.	FOR	DATE



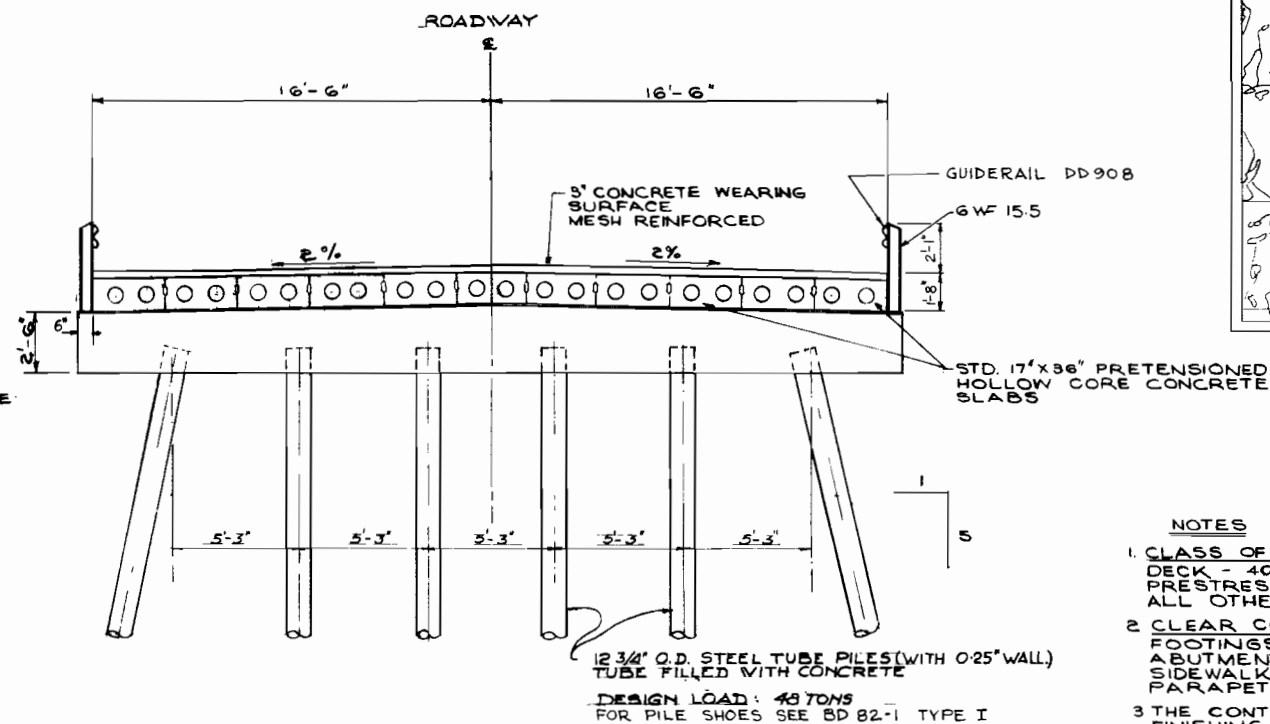
SECTIONS
SCALE: 1" = 10 Feet

REVISIONS	
DATE	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	
SUBMD. I.L. CHECKED	W.P. NO. 145-64
DRAWN D.M. CHECKED	JOB NO. 6-3-3
DATE MAR. 24, 1966	SITE NO.
APPROVED	CONTR. NO.
DRAWING NO. 6-3-3	
BRIDGE DRAWING NO.	

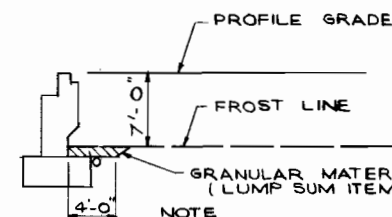


PLAN
SCALE 1"=10'-0"

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



A 1/4" = 1'-0"



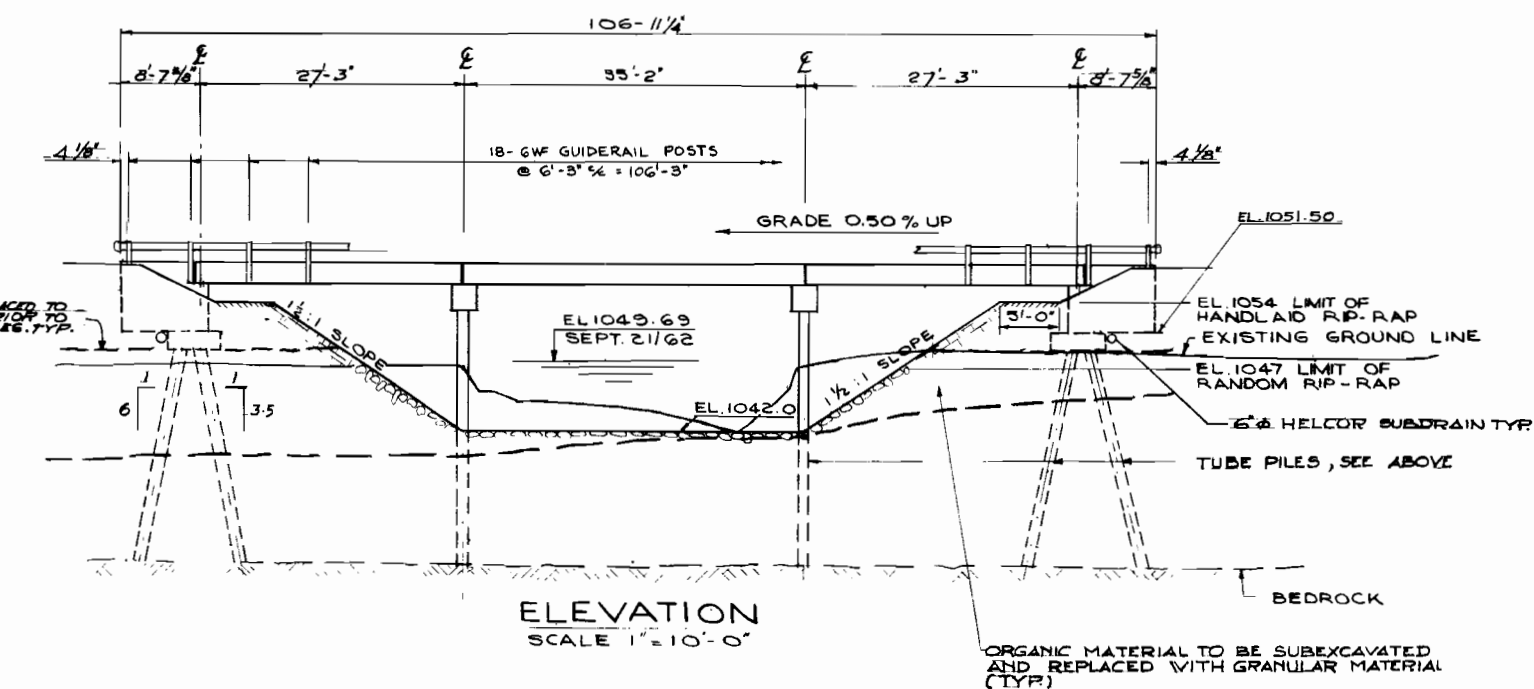
DETAILS OF MINIMUM GRANULAR
BACKFILL REQUIREMENT

NOTES

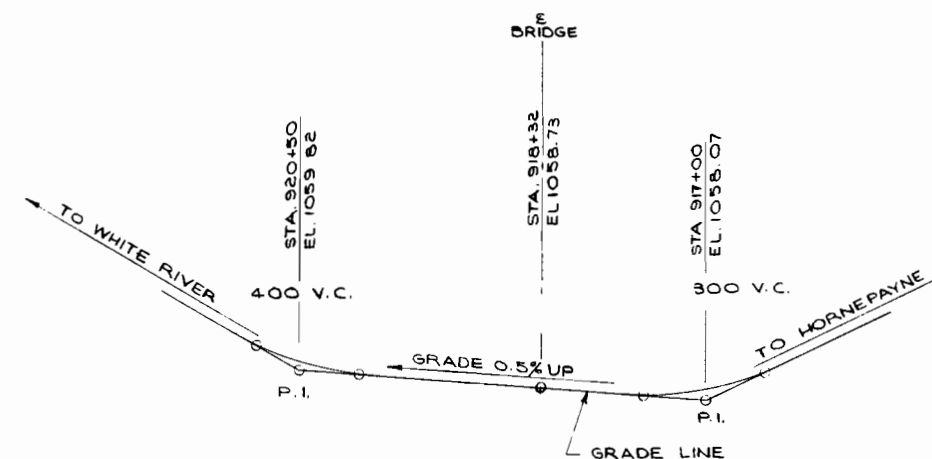
1. CLASS OF CONCRETE
DECK - 4000 P.S.I.
PRESTRESSED CONCRETE SLABS - 5000 P.S.I.
ALL OTHER - 3000 P.S.I.
2. CLEAR COVER ON REINFORCING STEEL
FOOTINGS 3"
ABUTMENTS 3"
SIDEWALKS 1 1/2"
PARAPET WALLS 1 1/2"
3. THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH TOLERANCES OF 1/8" ±

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 & WINGWALL DETAILS
- 7 STANDARDS DETAILS



ELEVATION
SCALE 1"=10'-0"



PROFILE GRADE
N.T.S.



REVISIONS	DATE	BY	DESCRIPTION

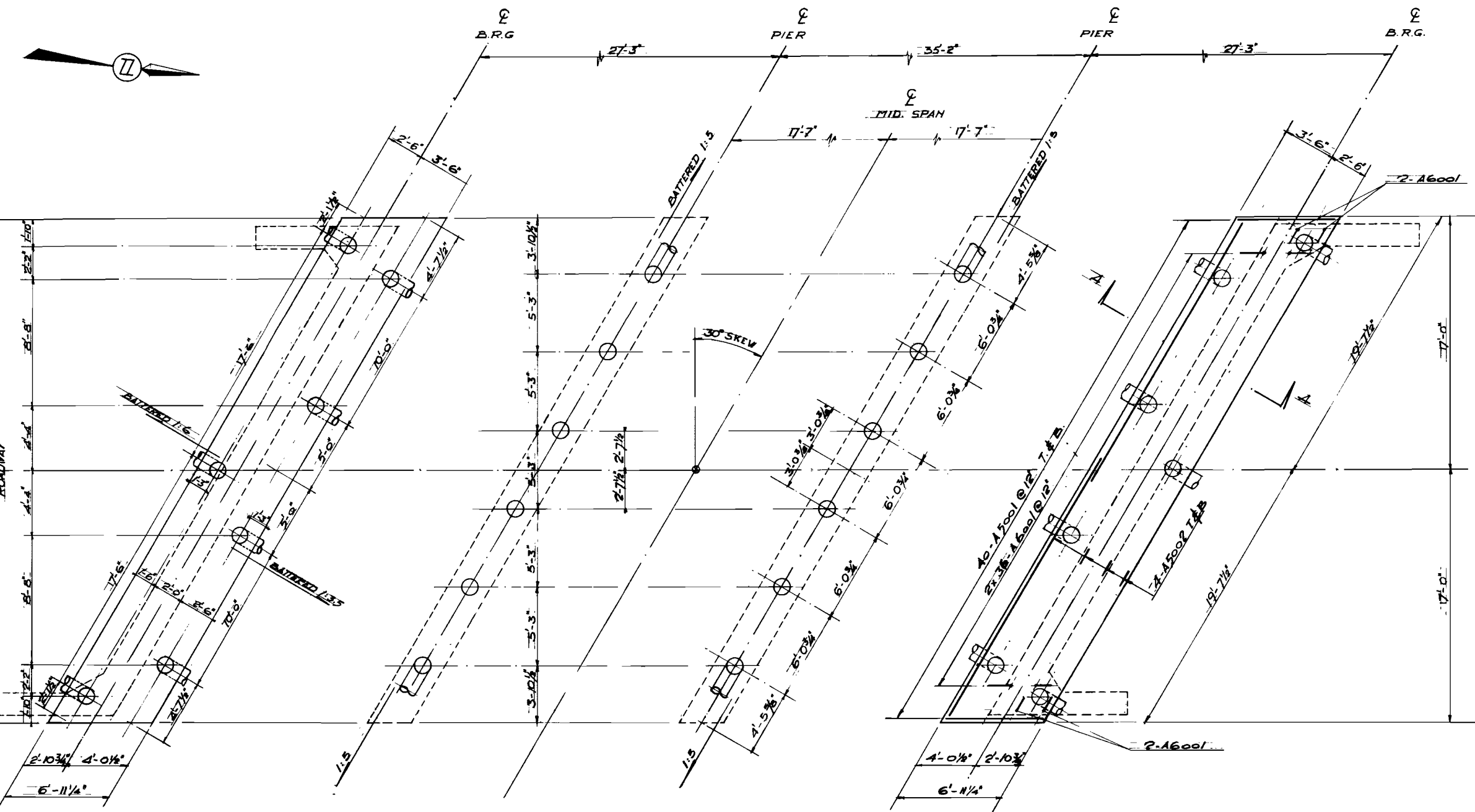
DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION

WEST BEATON RIVER BRIDGE

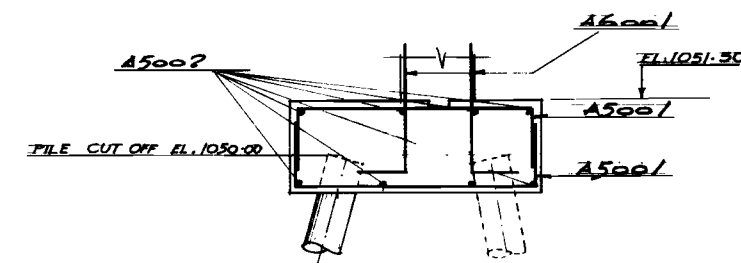
KING'S HIGHWAY No. 631 DIST. No. 16
CO. ALGOMA
TWP. BEATON LOT CON.

GENERAL

APPROVED	BRIDGE ENGINEER	SITE No. 36N-8	W.P. No. 145-64
DESIGN	DSM. CHECK W.T.H.	CONTRACT No.	67-188
DRAWING	P.A.H. CHECK DSM.	DRAWING No.	D-5920-1
DATE	APR. 67	LOADING	HS20-44



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



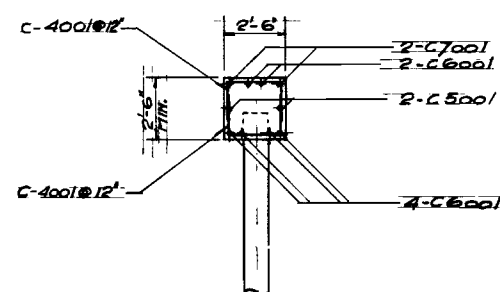
TYPICAL SECTION
OF ABUTMENT
A-A

PILE SCHEDULE

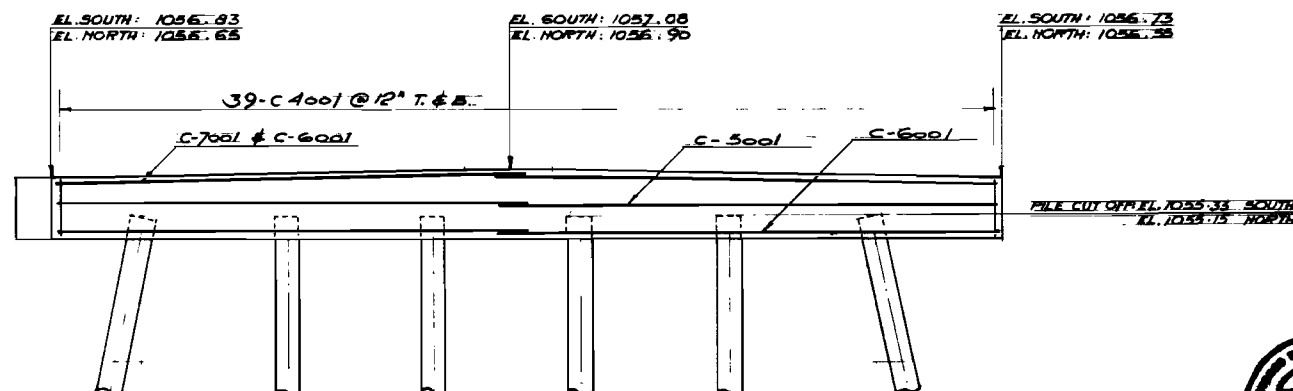
12 PILES	28'-0" LONG	PIERS
14 PILES	22'-0" LONG	ABUTM.

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

VIEST BEATON RIVER BRIDGE

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

FOUNDATION LAYOUT & PILE CAP

APPROVED	DESIGN	D.S.M.	CHECK	W.T.H.	SITE No.	38N-8	W.P. No.	145-64
DRAWING	A.J.A.	CHECK	DSM.	DSM.	CONTRACT	No.		67-178
DATE	APR. 67	LOADING	4520-44		DRAWING	No.		D-5920-3



APPENDIX D

Special Provisions

OBSTRUCTIONS

Notice to Contractor

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.



golder.com