REPORT

April 4, 2018

FOUNDATION INVESTIGATION AND DESIGN REPORT

REHABILITATION OF SHEKAK RIVER BRIDGE - SITE NO. 38N-007
HIGHWAY 631, TOWNSHIP OF LARKIN, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5569-09-00, WP 5175-13-01

Submitted to:
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L3R 9R9

GEOCRES No.: 42F-53

Report Number: 1661607-R06

Distribution:

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

Golder Associates

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FOUNDATION INVESTIGATION REPORT
REHABILITATION OF SHEKAK RIVER BRIDGE – SITE NO. 38N-007
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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 the Shekak River Bridge (Site No. 38N-007). The bridge is located on Highway 631, 88 km North of Highway 17 or about 12 km south of the Town of Hornepayne, and in the Township of Larkin, Ontario. 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 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 Shekak River Bridge 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.

It should be noted that the orientation (i.e., north, south, east, west) stated in the text of the report is referenced to project north and therefore may differ from magnetic north shown on Drawing 1. Highway 631 is generally oriented in a north-south direction, however the highway/bridge is oriented in a skewed east-west direction at the Shekak River crossing.

2.0 SITE DESCRIPTION

The existing Shekak River Bridge consists of an approximately 32 m long by 10 m wide, three-span rolled steel I-girder structure with spans of 9 m, 14 m and 9 m that was constructed in 1965. 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-Shekak GA-01.dwg) on August 21, 2017, the bridge deck is at Elevations 318.4 m and 318.2 m at the west and east abutments, respectively. The existing approach embankments are about 3 m to 4 m high relative to the river. Views at the bridge site are shown on Photographs 1 to 4, following the text of this report.

3.0 INVESTIGATION PROCEDURE

The fieldwork was carried out on June 13 and June 14, 2017, during which time two boreholes (SH-1 and SH-2) were advanced at the locations shown in Drawing 1.

The boreholes were advanced from the existing roadway platform using a truck mounted CME-55 drill rig supplied and operated by Landcore Drilling of Chelmsford Ontario. The boreholes were advanced using 108 mm inner diameter hollow stem augers and soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using 50 mm outer diameter split-spoon samplers driven by an automatic hammer, in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). The groundwater level in the open boreholes was observed during and immediately following the drilling operations as described on the Record of Borehole sheets in Appendix A. The boreholes were backfilled with bentonite and soil cuttings 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, Atterberg limits 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.

The borehole locations and elevations were measured and surveyed by a member of our technical staff, referenced to the highway centerline and existing bridge abutments. 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		
SH-1	5443221.5	247499.6	49.124460	-84.785134	318.3	9.8
SH-2	5443197.4	247551.4	49.124248	-84.784421	318.1	9.8

4.0 SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain (NOEGTS)[1] mapping, the Shekak River Bridge site is located within a glaciolacustrine plain deposit consisting primarily of sands and silts, bordered by bedrock knobs to the east and west of site.

Based on geological mapping by the Ontario Ministry of Northern Development and Mines (MNDM)[2], the site is underlain by foliated gneissic tonalite to granodiorite bedrock with minor supracustal inclusions.

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

[1] Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 41JNW
[2] Ontario Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey – Map 2543

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 SH-1 and SH-2 is presented below.

4.2.1 Subsoil Conditions

A description of the soil deposits encountered in the boreholes is provided below.

Deposit/Layer Description	Boreholes	Deposit Surface Elevation (m)	Deposit Thickness (m)	N Values (blows) / Relative Density	Laboratory Testing
Asphalt	SH-1 & SH-2	318.3 & 318.1	0.05	n/a	n/a
RAP	SH-1 & SH-2	318.25 & 318.05	0.125 & 0.1	n/a	n/a
(FILL) Gravelly sand and silty sand to sand	SH-1 & SH-2	318.1 & 317.9	3.0 & 2.5	N = 11 – 29 **Compact**	w = 4% & 14% 2 – M (Fig. B1)
Silt to Sandy Silt[1,2]	SH-1 & SH-2	315.1 &315.4	5.5 & 7.1 (borehole terminated in deposit in SH-2)	N = 3 – 68 **Very loose to Very dense**	w = 16% – 58% 3 – MH (Fig. B2) 1 – AL (NP)
(TILL) Sandy Silt	SH-1	309.6	1.1 (borehole terminated in this deposit)	N = 10 **Compact**	w = 12% 1 – MH (Fig. B3)

Where:
N = SPT 'N' values; number of blows for 0.3 m of penetration
w = natural moisture content (%)
M = sieve analysis for particle size
MH = combined sieve and hydrometer analysis
AL = Atterberg Limit Tests
NP = non-plastic test result in Atterberg limits

Notes:
1) Augers grinding from 4.4 m to 4.6 m depth in Borehole SH-2.
2) A 50 mm thick seam of peat was encountered at 3.4 m depth in Borehole SH-1 near the surface of the silt deposit.

4.2.2 Groundwater Conditions

The depths to/elevations of unstabilized groundwater levels measured in the open boreholes upon completion of drilling are presented below. Water levels should be expected to vary depending on the time of year and precipitation events.

Borehole No.	Depth to Unstabilized Groundwater Level (m)	Approximate Groundwater Elevation (m)
SH-1	3.1	315.2
SH-2	6.0	312.1

The river water level was surveyed by others at Elevation 314.1 m in November 2016.

5.0 CLOSURE

The field drilling program was supervised by Mr. Shane Albert. This Foundation Investigation Report was prepared by Ms. Aronne-Kay De Souza, EIT, 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., a Designated MTO Foundations Contact and Senior Consultant of Golder, conducted an independent quality control review and technical audit of this report.

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n:\active\2016\3 proj\1661607 lea_5015-e-0049_ne region\foundations\reporting\r06-shekak\final\1661607-r06 r-rev0 lea shekak river bridge rehabilitation fidr 04apr_18.docx

PART B

FOUNDATION DESIGN REPORT
REHABILITATION OF SHEKAK RIVER BRIDGE – SITE NO. 38N-007
HIGHWAY 631, TOWNSHIP OF LARKIN, ONTARIO
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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 Shekak River Bridge located on Highway 631 about 12 km south of the Town of Hornepayne. 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 Shekak 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 embankment.

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 314 m. As referenced in Section 4.2.2, the river water level was surveyed by others in November 2016 at Elevation 314.1 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. 42F00-010: "Foundation Investigation, Proposed Bridge, WP142-64, Sec. Hwy. 631, Shekak River" by William A. Trow Associates Ltd. (Note that the report date is not available on the scanned copies; the text of the report indicates that the subsurface investigation was completed in April 1964.). 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 March, 1965 for construction of the existing bridge: Contract 65-319, Drawing Nos. D5531-1 to -5 (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 5.

The 1965 Shekak River bridge drawings indicate that the bridge is supported by both 324 mm (12.75") and 356 mm (14") outer diameter steel tube piles with 6 mm (0.25") and 10 mm (.375") thick walls, respectively, filled with concrete. There was no reference in the text of the report nor on the above-noted drawing to the piles being fitted with pile driving shoes. The piles were to be driven below about Elevation 306.4 m (although it is noted that due to illegibility of the drawing, it is not clear whether the piles were to be driven below Elevation 1005' or 1008'), to a specified load of about 490 kN (50 tons). The drawing specifies "the required pile penetration was to be calculated using the Hiley formula with a factor of safety of 2.5 or 3 as determined by the engineer". Based on the GEOCRES information, we infer that the piles were driven into the very dense sand and gravel till stratum, which reportedly contains boulders up to 600 mm size, the surface of which we have interpreted as being present at about Elevation 305.4m (1002ft) at the North Abutment and Elevation 300.8m (987ft) at the South Abutment. While we did not find, nor were provided, any information supporting the tip elevation to which the piles were driven, it is also possible that the piles may have been driven tod bedrock (at Elevation 296.9m (974ft) at the North Abutment and Elevation 295.8m (970.5ft) at the South Abutment) if they penetrated the till and did not hang up on the boulders.

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
Shekak River Bridge	1430	970

In assessing the available geotechnical resistances of the existing piles, we have assumed that the piles have penetrated at least 2 m into the very dense sand and gravel till stratum. Further, we have 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.

Based on the above, we estimate that the available f-ULS and f-SLS geotechnical resistances will exceed the maximum design loads/required geotechnical resistances provided by LEA, as summarized above. SLS will not apply if the piles are founded on bedrock or near the bottom of the till in close proximity to the bedrock surface.

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 will be at the approach embankments is likely at or near the elevation of the river water level. The groundwater level is 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., gravelly sand and silty sand to sand fill) 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 (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. 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 Gravelly Sand and Silty Sand to Sand- Fill (compact)	19	30	-	0.33	0.50	3.00
Silt to Sandy Silt (loose to very dense)	18	29	-	0.35	0.52	2.88
Sandy Silt – TILL (Compact)	19	30	-	0.33	0.50	3.00

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 deposit, with little to no risk of pile adhesion and therefore full removal of the protection system should be implemented.

7.0 CLOSURE

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

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REFERENCES

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

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

OPSS.PROV 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resource Act

Regulation 903 Wells (as amended)

MINISTRY OF TRANSPORTATION, ONTARIO

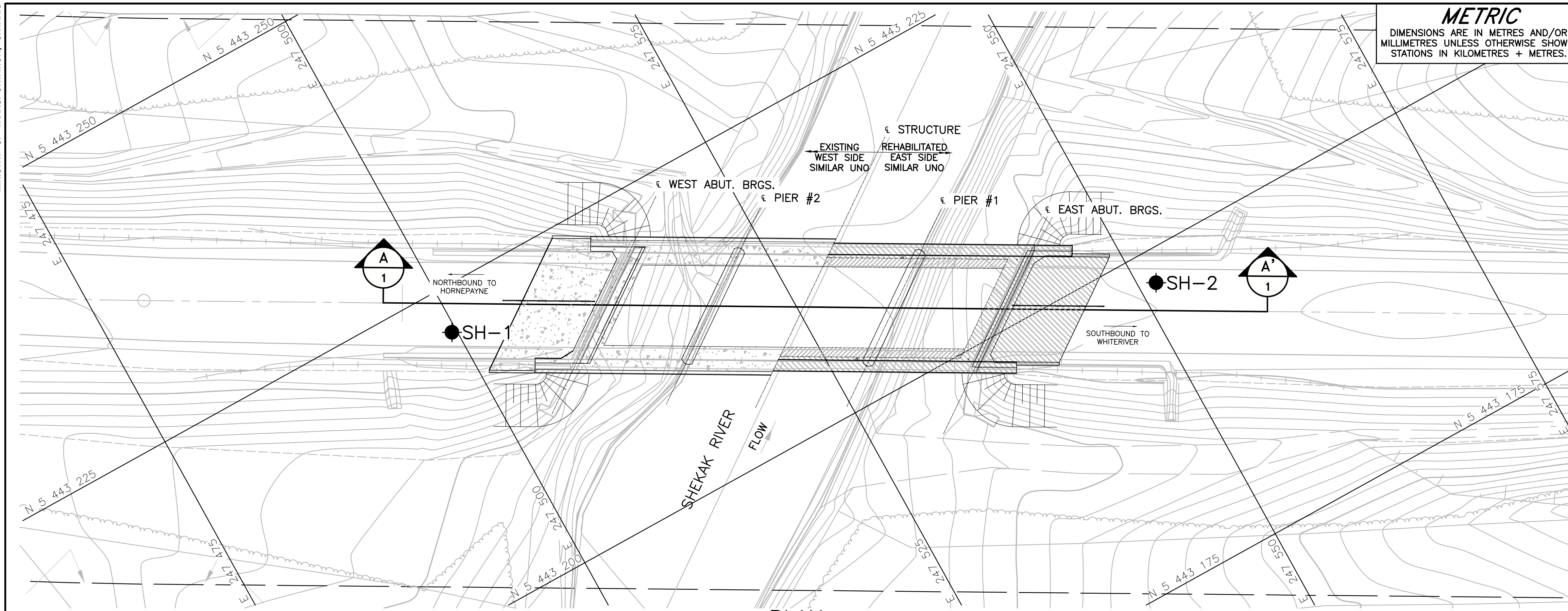
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5569-09-00

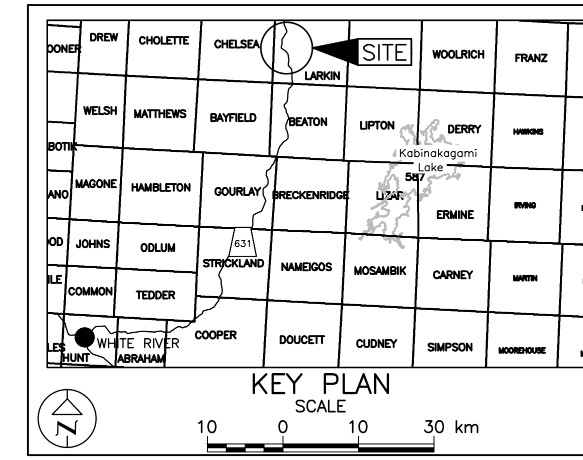
SHEET

HWY 631
SHEKAK RIVER BRIDGE
LAT. 49.124355, LONG. -84.784777
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER



PLAN
SCALE
4 0 4 8 m



KEY PLAN
SCALE
10 0 10 30 km

LEGEND

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

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE 13)

No.	ELEVATION	NORTHING	EASTING
SH-1	318.3	5443221.5	247499.6
SH-2	318.1	5443197.4	247551.4

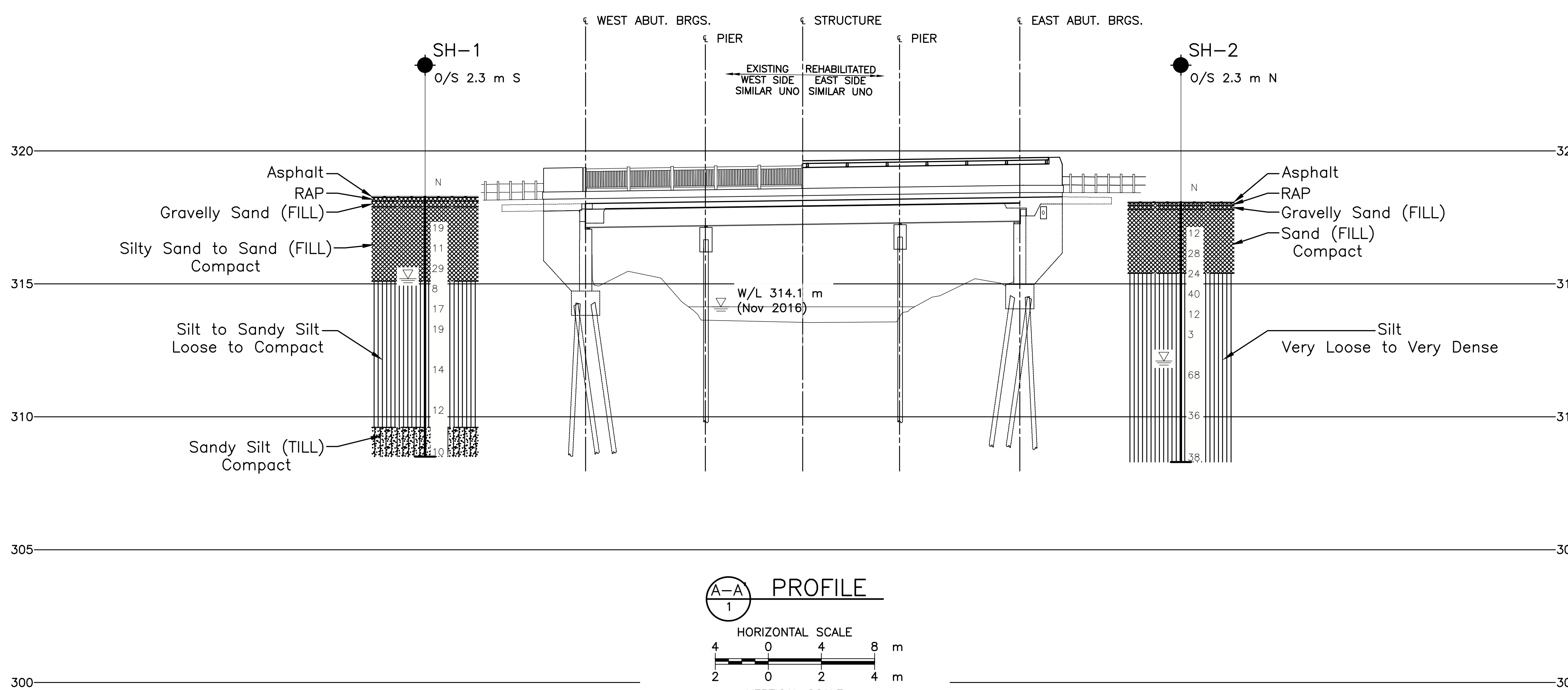
NOTES

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.

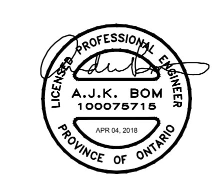
REFERENCE

Base plans provided in digital format by LEA Consulting LTD., drawing file nos. 17197-Shekak GA-01.dwg, received AUG 15, 2017. General arrangement plan file no. 17197-Shekak-R01-General Arrangement.dwg, received APR 04, 2018.



A-A' 1 PROFILE

HORIZONTAL SCALE
4 0 4 8 m
2 0 2 4 m
VERTICAL SCALE



NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 42F-53

HWY.		PROJECT NO. 1661607	DIST.
SUBM'D. AC	CHKD.	DATE: 4/4/2018	SITE: 38N-007
DRAWN: JJL	CHKD. AB	APPD. JMAC	DWG. 1

PLOT DATE: April 4, 2018

Photograph 1: Shekak River Bridge
East approach, looking west (June 2017)

Photograph 2: Shekak River Bridge
Bedrock outcrop north of bridge, looking west (June 2017)

Photograph 3: Shekak River Bridge
North side of bridge, looking west (June 2017)

Photograph 4: Shekak River Bridge
East approach embankment looking southwest (June 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

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

(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 δ
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

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	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

Golder Associates

RECORD OF BOREHOLE No SH-1

 METRIC

PROJECT 1661607
G.W.P. 5569-09-00
LOCATION N 5443221.5; E 247499.6 MTM ZONE 13 (LAT. 49.12446; LONG. -84.785134)
ORIGINATED BY SA
DIST ___ HWY 631
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY AC
DATUM GEODETIC
DATE June 13, 2017
CHECKED BY AB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
318.3	GROUND SURFACE					
0.0	ASPHALT (50 mm)					
	RAP (125 mm)					
317.9 / 0.4	Gravelly sand (FILL) Brown Moist					
	Silty sand to sand, some gravel, some silt (FILL) Compact Brown Moist	1	SS	19		
		2	SS	11		
		3	SS	29		0 78 (22)
315.1 / 3.2	SILT to Sandy SILT, trace clay Loose to compact Brown to grey Wet. A 50 mm peat seam was encountered at 3.4 m depth.				▽ (water level approx. 315.2)	
		4	SS	8		
		5	SS	17		
		6	SS	19		
		7	SS	14		0 21 75 4
		8	SS	12		
309.6 / 8.7	Sandy SILT, trace gravel, some clay (TILL) Compact Grey Wet					
		9	SS	10		9 33 48 10
308.5 / 9.8	END OF BOREHOLE					

Note:

1. Water level at a depth of 3.1 m below ground surface (Elev. 315.2 m) upon completion of drilling.

Elevation scale: 318, 317, 316, 315, 314, 313, 312, 311, 310, 309

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

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

Golder Associates

RECORD OF BOREHOLE No SH-2

1 OF 1 **METRIC**

PROJECT 1661607
G.W.P. 5569-09-00
LOCATION N 5443197.4; E 247551.4 MTM ZONE 13 (LAT. 49.124248; LONG. -84.784421)
ORIGINATED BY SA
DIST HWY 631
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY AC
DATUM GEODETIC
DATE June 14, 2017
CHECKED BY AB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
318.1	GROUND SURFACE								
0.0	ASPHALT (50 mm)					318			
	RAP (100 mm)								
0.3	Gravelly sand (FILL) Brown Moist								
	Sand, some silt, some gravel (FILL) Compact Brown Moist	1	SS	12		317			
		2	SS	28			○		13 73 (14)
		3	SS	24		316			
315.4 / 2.7	SILT, some sand, trace gravel, some clay Very loose to very dense Brown to grey Moist to wet	4	SS	40		315	○		2 16 74 8
	Trace oganics from 2.7 m to 3.0 m depth.	5	SS	12		314			
	Auger grinding from 4.4 m to 4.6 m depth.	6	SS	3		313	○		
					∇	312			
	Silty sand layer encountered in Sample 7.	7	SS	68		311			
		8	SS	36		310	○	NP	1 5 85 9
		9	SS	38		309			
308.3 / 9.8	END OF BOREHOLE								

Note:

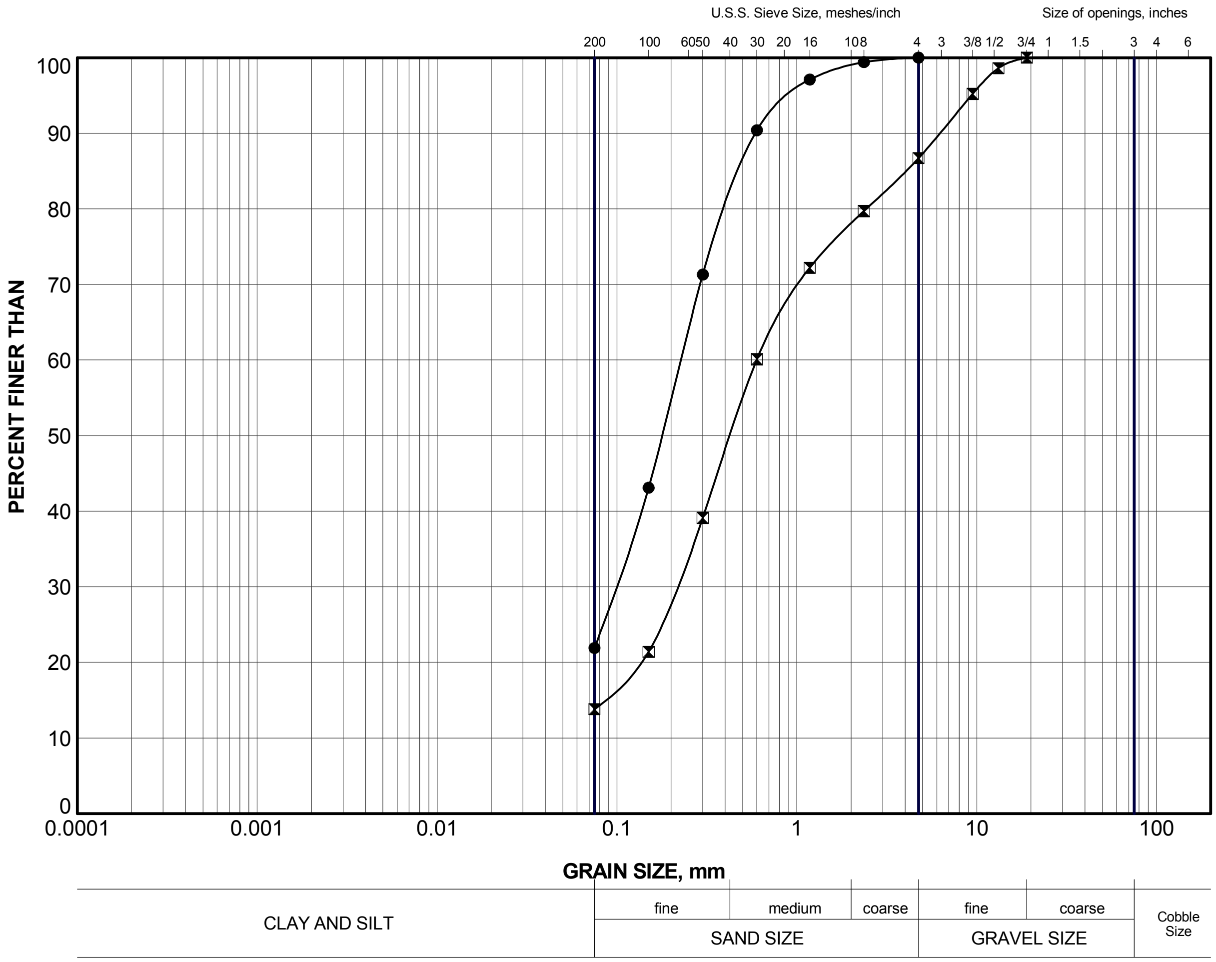
1. Water level at a depth of 6.0 m below ground surface (Elev. 312.1 m) upon completion of drilling.

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

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

APPENDIX B

Laboratory Test Results



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	SH-1	3	315.7
⧗	SH-2	2	316.3

PROJECT

HIGHWAY 631
SHEKAK RIVER BRIDGE

TITLE

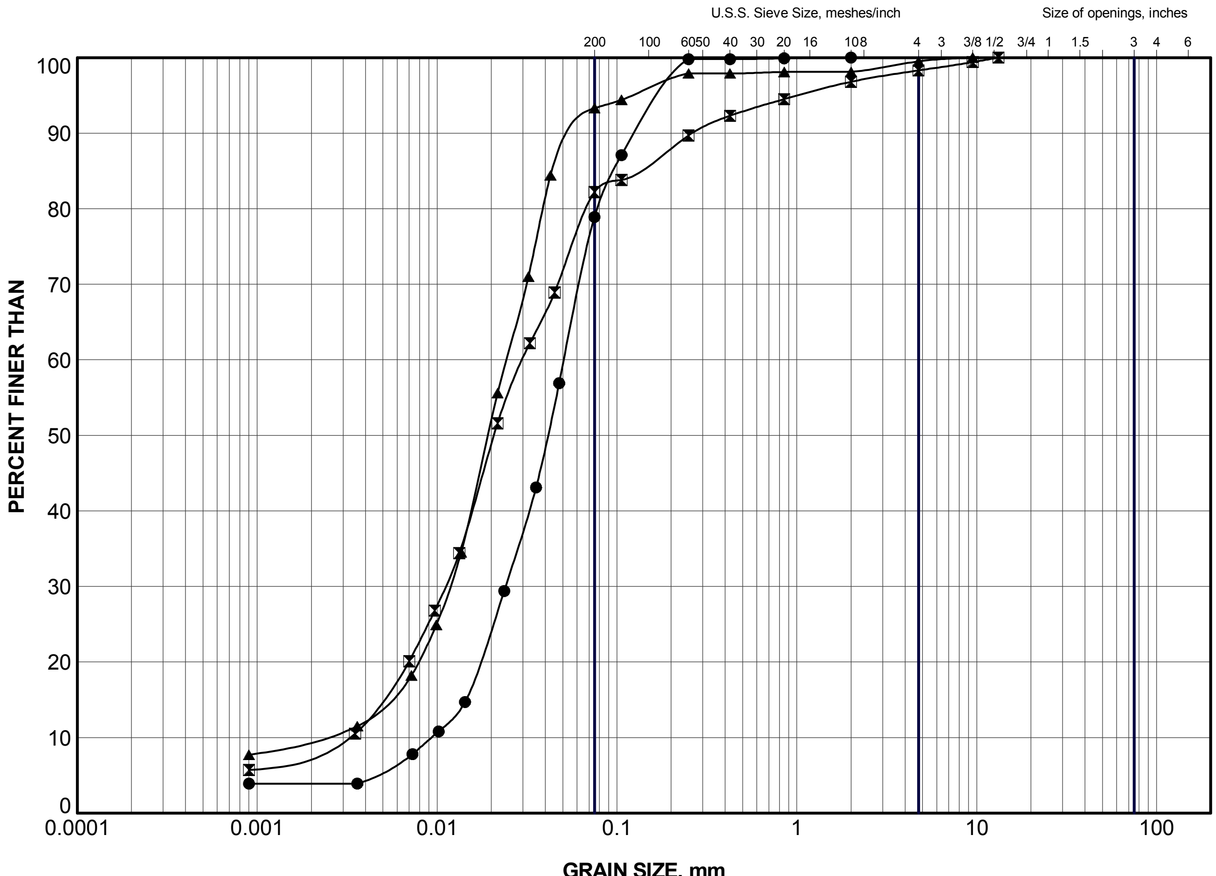
GRAIN SIZE DISTRIBUTION

SILTY SAND to SAND (FILL)

Golder Associates
SUDBURY, ONTARIO

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

SUD-MTO GSD (2016) GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	SH-1	7	311.9
⧗	SH-2	4	314.8
▲	SH-2	8	310.2

PROJECT

HIGHWAY 631
SHEKAK RIVER BRIDGE

TITLE

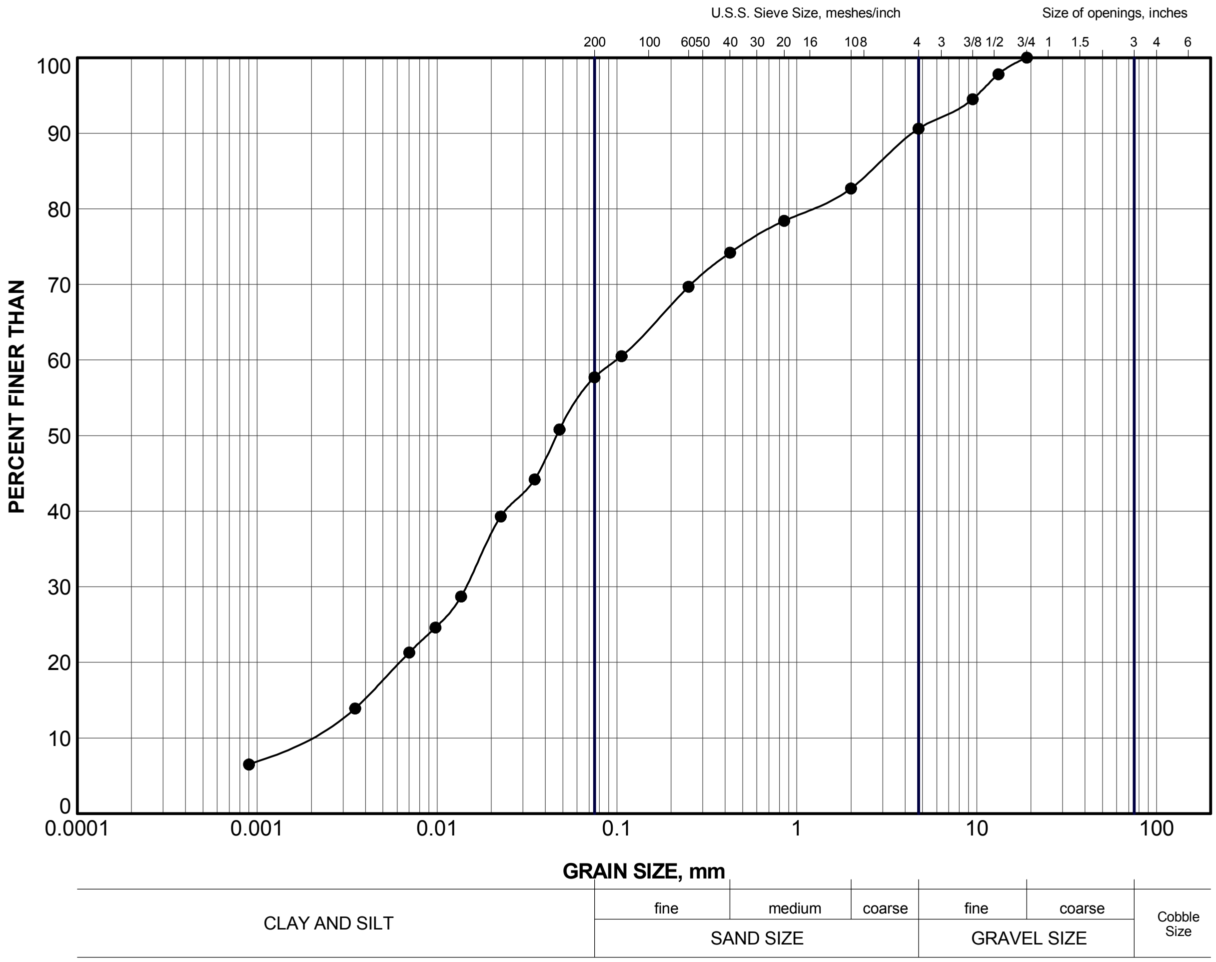
GRAIN SIZE DISTRIBUTION

SILT to SANDY SILT

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

Golder Associates
SUDBURY, ONTARIO

SUD-MTO GSD (2016) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	SH-1	9	308.8

PROJECT

HIGHWAY 631
SHEKAK RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION
SANDY SILT (TILL)

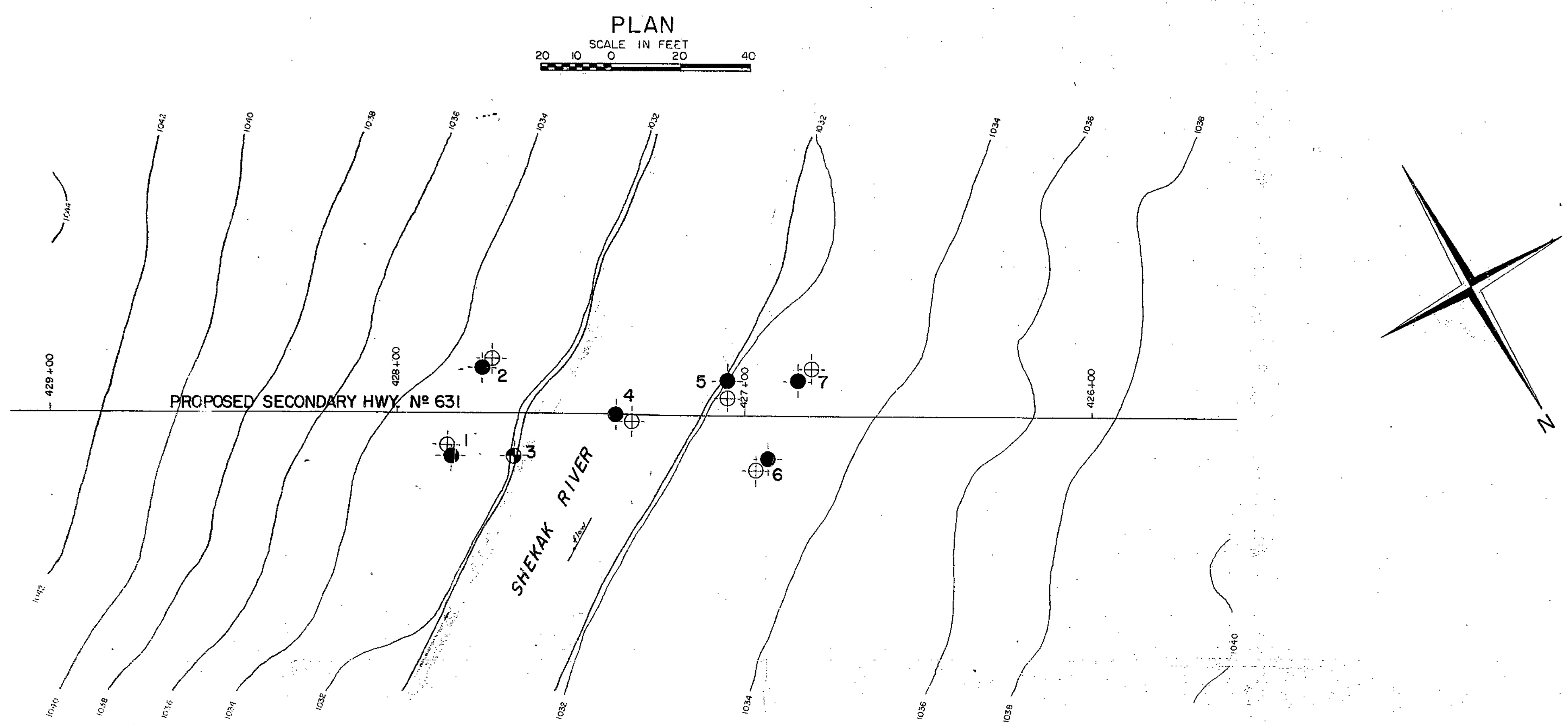
Golder Associates
SUDBURY, ONTARIO

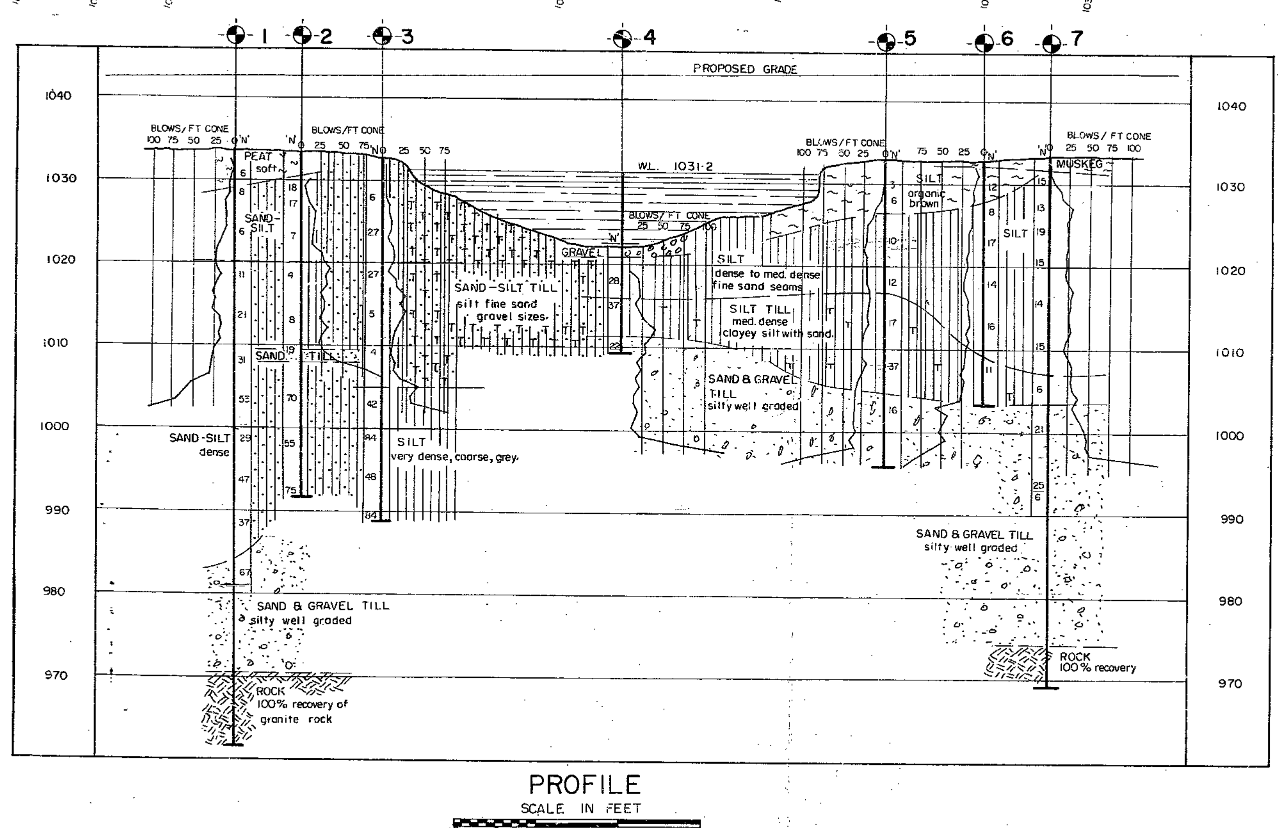
PROJECT No.		1661607	FILE No.	1661607.GPJ
DRAWN	TB	Oct 2017	SCALE N/A	REV.
CHECK	AC	Oct 2017		
APPR	JMAC	Oct 2017	**FIGURE B3**	

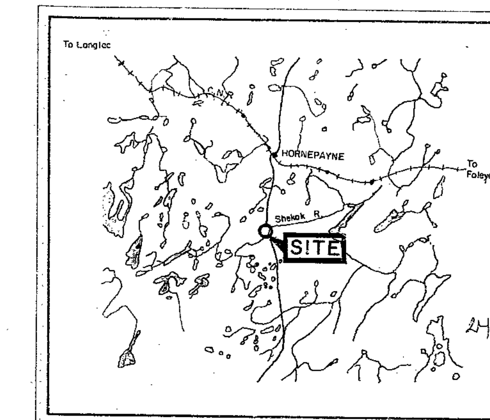
SUD-MTO GSD (2016) GLDR_LDN.GDT

APPENDIX C

Borehole Location Plan/Profile Drawing (GEOCRES 42F-010) and Select Structural Drawings







KEY PLAN
SCALE IN MILES
8 4 0 8 16

LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation.

NO.	ELEVATION	STATION	OFFSET
1	1033·6	427+84	12' Lt
2	1033·2	427+76	13' Rt.
3	1032·9	427+66	12' Lt.
4	1032·1	427+37	℄
5	1032·9	427+05	10' Rt
6	1032·8	426+93	12' Lt.
7	1033·2	426+85	10' Rt.

– 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.

REVISIONS		
DATE	BY	DESCRIPTION

PRINT RECORD		
NO.	FOR	DATE

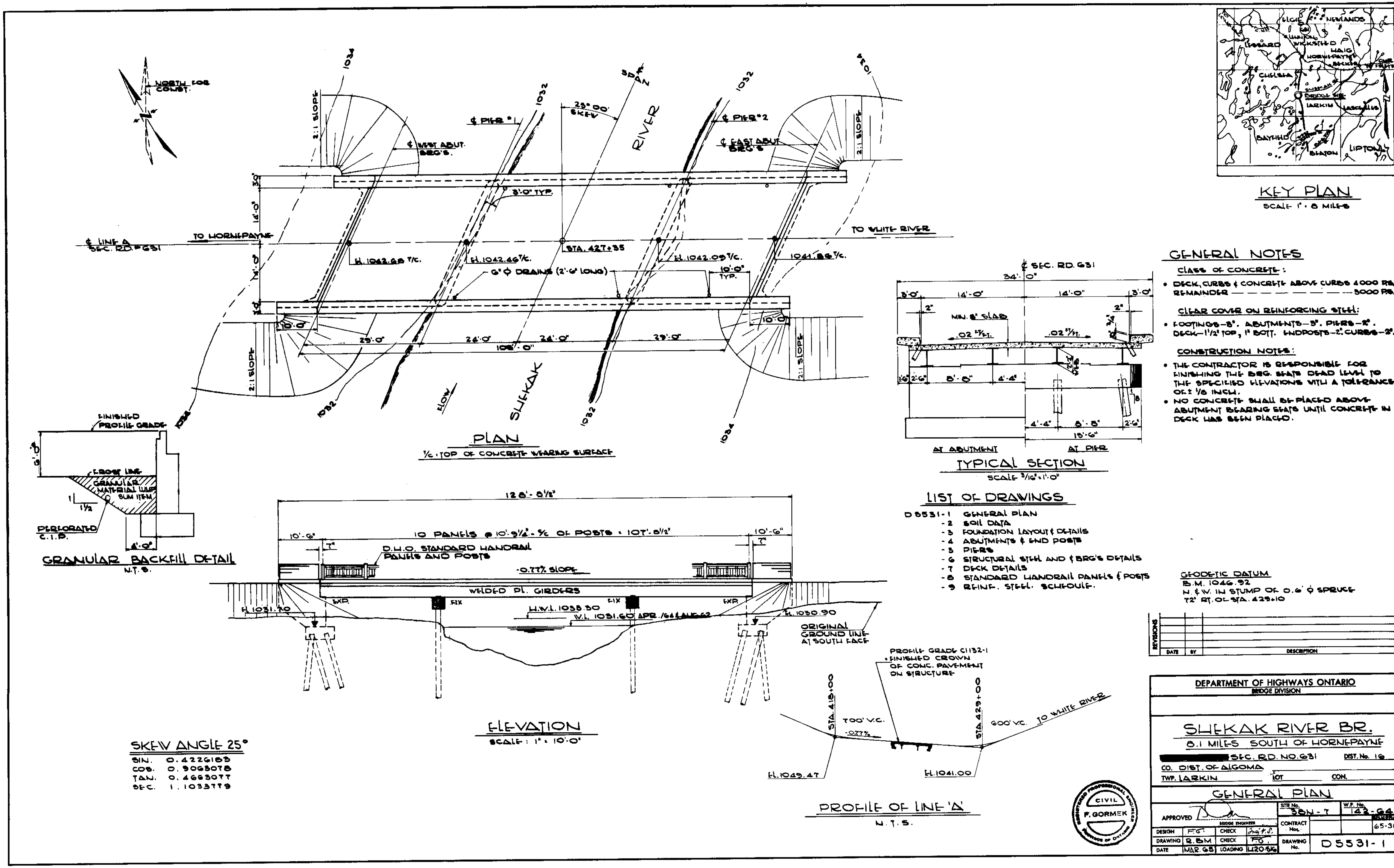
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & RESEARCH DIVISION – FOUNDATION SECTION

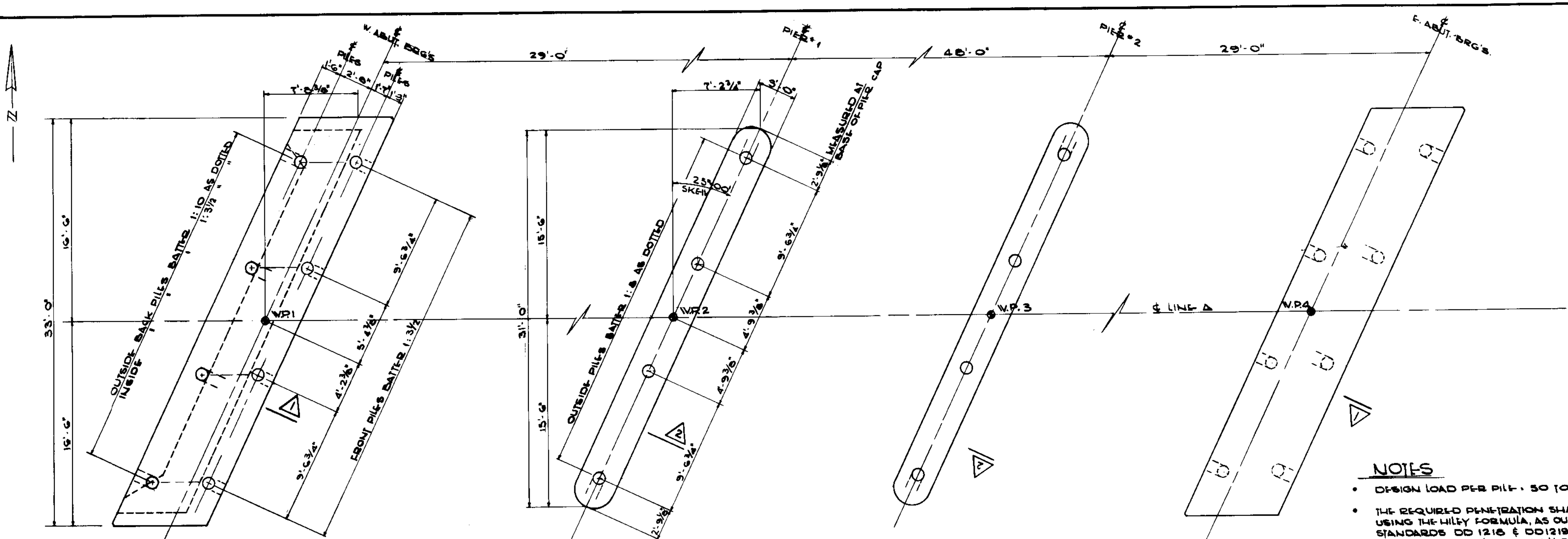
SHEKAK RIVER

KING'S HIGHWAY NO. 631 LINE A DIST NO. 18
DIST. ALGOMA
TWP. LARKIN LOT CON.

PROJ. 1410 WILLIAM A. TROW AND ASSOCIATES LIMITED

SUBM'D	CHECKED D.H.S.	W.P. NO. 142 - 64	M.&R. DRAWING NO.
DRAWN E.F.K.	CHECKED	JOB NO.	
DATE		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	


KEY PLAN
SCALE 1" = 8 MILES
GENERAL NOTES
CLASS OF CONCRETE:
• DECK, CURBS & CONCRETE ABOVE CURBS 4000 PSI.
REMAINDER ———— 3000 PSI.
CLEAR COVER ON REINFORCING STEEL:
• FOOTINGS-3". ABUTMENTS-3". PIERS-2".
DECK-1 1/2" TOP, 1" BOTT. ENDPOSTS-2". CURBS-2".
CONSTRUCTION NOTES:
• THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BRG. SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8 INCH.
• NO CONCRETE SHALL BE PLACED ABOVE ABUTMENT BEARING SEATS UNTIL CONCRETE IN DECK HAS BEEN PLACED.
NORTH FOR CONST.
PLAN
1/c TOP OF CONCRETE WEARING SURFACE
SHEKAK RIVER
FLOW
℄ LINE A SEC. RD. #631
TO HORNEPAYNE
TO WHITE RIVER
STA. 427+35
25°00' SKEW
℄ SPAN
℄ PIER #1
℄ PIER #2
℄ WEST ABUT. BRG'S.
℄ EAST ABUT. BRG'S
6" ϕ DRAINS (2'-6" LONG)
2:1 SLOPE
108'-0"
TYPICAL SECTION
SCALE 3/16" = 1'-0"
℄ SEC. RD. 631
AT ABUTMENT
AT PIER
LIST OF DRAWINGS
D5531-1 GENERAL PLAN
-2 SOIL DATA
-3 FOUNDATION LAYOUT & DETAILS
-4 ABUTMENTS & END POSTS
-5 PIERS
-6 STRUCTURAL STEEL AND & BRG'S DETAILS
-7 DECK DETAILS
-8 STANDARD HANDRAIL PANELS & POSTS
-9 REINF. STEEL SCHEDULE.
GRANULAR BACKFILL DETAIL
N.T.S.
FINISHED PROFILE GRADE
PERFORATED C.I.P.
GRANULAR MATERIAL LUMP SUM ITEM
ELEVATION
SCALE: 1" = 10'-0"
128'-8 1/2"
10 PANELS @ 10'-9 1/2" - ℄ OF POSTS = 107'-8 1/2"
D.H.O. STANDARD HANDRAIL PANELS AND POSTS
-0.77% SLOPE
WELDED PL. GIRDERS
H.W.L. 1035.30
W.L. 1031.60 APR./64
ORIGINAL GROUND LINE AT SOUTH FACE
SKEW ANGLE 25°
SIN. 0.4226183
COS. 0.9063078
TAN. 0.4663077
SEC. 1.1033779
PROFILE OF LINE 'A'
N.T.S.
PROFILE GRADE C1132-1 FINISHED CROWN OF CONC. PAVEMENT ON STRUCTURE
TO WHITE RIVER
GEODETIC DATUM
B.M. 1046.92
N. & W. IN STUMP OF 0.6' ϕ SPRUCE 72' RT. OF STA. 425+10
REVISIONS
DATE BY DESCRIPTION
DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION
SHEKAK RIVER BR.
8.1 MILES SOUTH OF HORNEPAYNE
[redacted] SEC. RD. NO. 631 DIST. No. 16
CO. DIST. OF ALGOMA
TWP. LARKIN LOT CON.
GENERAL PLAN
APPROVED BRIDGE ENGINEER
SITE No. 38N-7
W.P. No. 142-64
65-318
DESIGN F.G. CHECK
DRAWING R.B.M. CHECK F.G.
DATE MAR. 65 LOADING H20-S16
CONTRACT No.
DRAWING No. D5531-1
CIVIL
F. GORMEK



FOUNDATION PLAN & LAYOUT

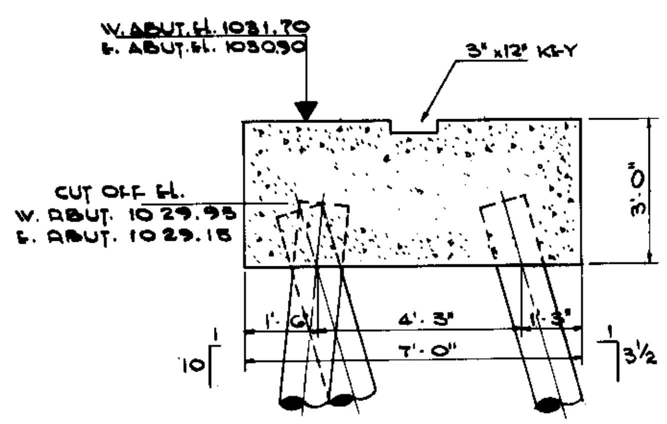
¼" = 1'-0"

● W.P. = WORKING POINT FOR LAYOUT PURPOSES.

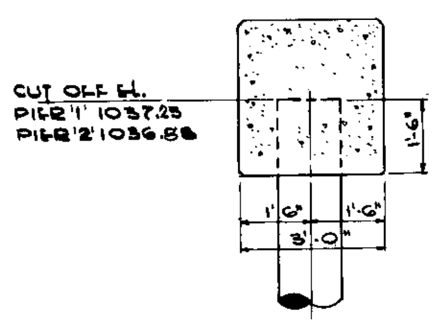
NOTES

- DESIGN LOAD PER PILE = 50 TONS.
- THE REQUIRED PENETRATION SHALL BE CALCULATED USING THE HILEY FORMULA, AS OUTLINED IN D.H.O. STANDARDS DD 1218 & DD1219. THE FACTOR OF SAFETY TO BE EMPLOYED SHALL BE 2.5 OR 3 AS DETERMINED BY THE ENGINEER. THE PILES SHALL BE DRIVEN TO AT LEAST ELEVATION 1005.00 FOR SCOUR PURPOSES.
- NUMBER OF STEEL TUBE PILES:
  - 6 12¾" O.D. ¼" WALL @ 42' W. ABUT.
  - 6 12¾" O.D. ¼" WALL @ 33' E. ABUT
  - 4 14" O.D. ⅜" WALL @ 46' PIER # 1
  - 4 14" O.D. ⅜" WALL @ 40' PIER # 2

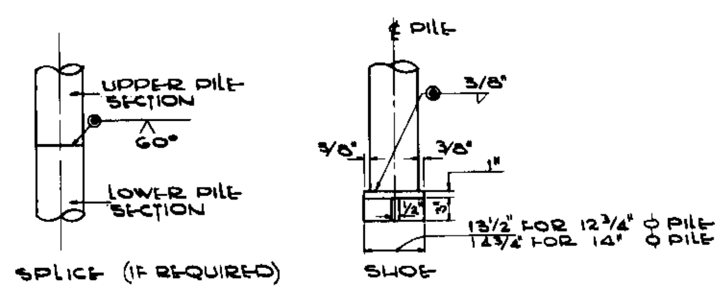
  ALL PILES TO BE FILLED WITH CONCRETE (3000 P.S.I.)



1 ½" = 1'-0"



2 ½" = 1'-0"



TUBE PILE

N.T.S.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

SUEKAK RIVER BRIDGE

[redacted] SEC. RD. NO. 631 DIST. No. 16
CO. DIST. OF ALGOMA
TWP. LARKIN LOT CON.

FOUNDATION LAYOUT & DETAILS

APPROVED	BRIDGE ENGINEER			SITE No. 38N-7	W.P. No. 142-64
DESIGN	F.G.	CHECK	J.V.S.	CONTRACT Nos.	65-319
DRAWING	R.B.M.	CHECK	F.G.	DRAWING No.	D5531-3
DATE	MAR.65	LOADING	H20-S16		

CIVIL F. GORMEK PROFESSIONAL ENGINEERS PROVINCE OF ONTARIO

APPENDIX D

Notice to Contractor

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.

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

For more information, visit golder.com

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