

REPORT

Foundation Investigation and
Design Report
Kabinakagami River Bridge
Replacement
Site No. 39W-009
Highway 11
District – New Liskeard

G.W.P. 5411-04-00

LEA CONSULTING LTD.

PROJECT NO. 1015345
GEOCRES NO. 42F-19

PROJECT NO. 1015345

REPORT TO

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FOR

**Draft Foundation Investigation and Design
Report**

ON

**Kabinakagami River Bridge Replacement
Site 39W-009, Highway 11
District – New Liskeard
G.W.P. 5411-04-00
Geocres. No. 42F-19**

January 15, 2008

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FOUNDATION INVESTIGATION REPORT

**Kabinakagami River Bridge Replacement
Site No. 39W-009, Highway 11
Near Hearst, Ontario
G.W.P. 5411-04-00
District – New Liskeard**

1.0 INTRODUCTION

Jacques Whitford Limited (Jacques Whitford) was retained by Lea Consulting Ltd., to complete a Foundation Investigation and Design Report for the replacement of the Kabinakagami River Bridge on Highway 11, located approximately 32 km west of Hearst, Ontario, (GWP No. 5411-04-00).

The work was carried out under Agreement No. 5005-E-0025. Authorization to proceed with the investigation was provided by Mr. Peter Ojala, P.Eng., Vice President, Head of Bridges and Structures, of Lea Consulting Ltd, the prime consultant on this design assignment.

This foundation investigation report has been prepared specifically and solely for the project described herein. It contains the factual results of the foundation investigation and the laboratory testing.

2.0 SITE DESCRIPTION

The site is located on Highway 11 at the Kabinakagami River, approximately 32 km west of Hearst, Ontario.

Highway 11 at the Kabinakagami River is built on shallow embankments to a rural highway section with wide gravel shoulders. Highway 11 is generally oriented in an east west direction with one east bound lane and one west bound lane. The highway is generally higher than the surrounding lands. Drainage for Highway 11 is provided by ditches located along the sides of the highway, which are sloped to drain towards the Kabinakagami River.

The existing bridge at the Kabinakagami River is a five span structure with steel girders supporting a reinforced concrete deck. The existing bridge is approximately 107 m long and 10 m wide. The bridge conveys one westbound lane and one eastbound lane of Highway 11 over the Kabinakagami River. The structure was reportedly constructed in 1942. The northern half of the bridge deck was reportedly replaced in 2002.

Based on a drawing dated December 1978, it is understood that the existing bridge structure is likely supported on a combination of shallow and deep foundations. The drawings indicate that the East Abutment and Pier 4 are likely supported by shallow foundations placed on bedrock. The drawings indicate the West Abutment and Piers 1, 2 and 3 are likely supported by deep foundations. The drawing does not indicate what the deep foundations are bearing on and the drawing does not identify the type of deep foundation.

3.0 PHYSIOGRAPHY

Based on Map 2518, titled "Surficial Geology of Northern Ontario", dated 1987, by the Ministry of Northern Development and Mines, Highway 11 at the Kabinakagami River is situated on the boundary between a Clay-Silt deposit and a Till Deposit. The clay-silt deposit is noted as a Glaciolacustrine deposit, while the till deposit is noted as an unsorted mixture of boulders, sand, silt and clay sized particles.

Based on Map 2543, titled "Bedrock Geology of Ontario, East-Central Sheet", dated 1991, by the Ontario Ministry of Northern Development and Mines, the bedrock at the site is noted as Metasedimentary rock comprised of wacke, arkos, argillite, slate, marble, chert, iron formation and minor metavolcanic rock intrusions.

4.0 BACKGROUND

A preliminary investigation was carried out by Jacques Whitford Limited. The results of the preliminary investigation were provided in the following draft Preliminary Foundation Investigation and Design report:

- *Draft Report*
Preliminary Foundation Investigation and
Design Report
Kabinakagami River Bridge Replacement
Site No. 39W-009
Highway 11
District – New Liskeard
GWP 5411-04-00
Jacques Whitford project number 1015345
Draft report dated: April 4, 2007

The factual results from the draft preliminary foundation report, including the Record of Borehole sheets and Laboratory Test data, have been incorporated into this report.

5.0 INVESTIGATION PROCEDURES

5.1 Field Program

The fieldwork for the preliminary investigation was carried out from January 14 to 20, 2007 and February 2 to 23, 2007. The fieldwork for the detailed foundation investigation was carried out from July 5 to 8, 2007 and on July 18 and 19, 2007. Supplemental investigation at the east abutment was carried out on November 29 and 30, 2007.

A total of 14 boreholes have now been advanced at the site using truck, track and barge mounted drill rigs equipped with 250 mm (outside diameter) continuous flight, hollow-stem augers, 150 mm (outside diameter) continuous flight, solid-stem augers, steel casings and mud-rotary drilling. The following table outlines the borehole locations numbers etc.:

Side of River	Borehole Number	Element
East Side of River	KB-06-4, KB-06-5, KB06-6, KB-07-100, KB07-101, KB-07-102, KB-07-103	East Abutment
	KB07-2	East Pier
	KB-07-4	East Approach Fills
West side of river	KB-06-1, KB-06-2, KB06-3	West Abutment
	KB07-1	West Pier
	KB-07-3	West Approach Fills

Prior to commencing the field investigations, the borehole locations were cleared of underground utilities by the various utility companies.

Soil samples were recovered from the boreholes at regular intervals using a 50 mm Outside Diameter split-tube sampler by conducting Standard Penetration Tests (SPTs) in general accordance with the procedures outlined in ASTM specification D1586-99.

Where cohesive soils were encountered, in situ shear vane testing was carried out using a vane meeting the MTO N-Vane design requirements and following the procedures outlined in ASTM D2573-94.

Rock cores were obtained using standard NQ rock coring equipment.

Jacques Whitford field personnel recorded the conditions encountered in all boreholes at the time of the investigation. Soils were described in accordance with the MTO Soils Classification System for foundation reports.

The groundwater levels, where encountered and where practical, were measured in the boreholes during and on completion of drilling. All boreholes were backfilled in accordance with Ontario Regulation 903, using cement/bentonite slurry.

All soil samples recovered from the boreholes were placed in moisture-proof bags and returned to our laboratory for detailed classification and testing as required. All rock cores were placed in rock core boxes and transported to our laboratory for detailed examination and selected laboratory testing.

5.2 Survey

The borehole locations were established by Jacques Whitford personnel and referenced to the stations on Highway 11, as noted on the Record of Borehole sheets. Offsets were referenced looking up chainage left or right of the centreline of the proposed highway alignment. The borehole locations are provided on the Drawing No. 1 in **Appendix A** and on the Record of Borehole sheets in **Appendix B**.

The ground surface elevation at the borehole locations were surveyed by Jacques Whitford Personnel. The boreholes were surveyed to the following benchmark:

- Geodetic Canada Benchmark No. 84U024, with a reported Geodetic elevation of 244.859 m.

5.3 Laboratory Testing

All samples returned to the laboratory were subjected to detailed visual examination and classification. Approximately 25% of the soil samples were submitted for routine testing including grain size distribution, Atterberg Limits and moisture content determination testing. In addition, samples of the rock core were submitted for unconfined compressive strength testing.

The laboratory results are provided on the Record of Borehole sheets in **Appendix B**. The results of the grain size analyses, Atterberg Limits and unconfined compressive strength tests are shown on Figure Nos. 1 through 7 in **Appendix C**.

Unless requested in advance, all samples will be stored in our laboratory for a period of 12 months, after issuance of this report.

6.0 RESULTS OF THE INVESTIGATION

6.1 Subsurface Conditions

The subsurface conditions encountered in the boreholes are summarized on the Record of Borehole sheets provided in **Appendix B**. An explanation of the terms and symbols used on the Record of Borehole sheets is also provided in **Appendix B**.

A Borehole Location Plan and a Strata Plot of the soils encountered in the boreholes are provided on Drawing Nos. 1 and 2 in **Appendix A**.

A summary of the soil and groundwater conditions encountered in the boreholes is provided below.

6.2 Soil

6.2.1 Topsoil

Topsoil was encountered at the ground surface in Boreholes KB-06-01 to KB-06-06. The thickness of the topsoil ranged from approximately 50 mm to 150 mm, with a mean of about 100 mm.

6.2.2 Sand, and Sand and Gravel Fill

Sand and sand and gravel fill was encountered at the ground surface in Borehole KB-07-4 and KB-07-03 and underlying the topsoil in Boreholes KB-06-2, KB-06-3 and KB-06-6. The thickness of the sand and sand and gravel fill ranged from approximately 0.6 m to 2.0 m.

The sand and sand and gravel fill was generally moist. Wood debris was encountered in the fill material in Borehole KB-06-2.

Based on the N-Values obtained from the Standard Penetration Tests (SPTs), the compactness of the sand and sand and gravel fill was assessed as loose to compact.

Laboratory testing performed on selected samples consisted of moisture content tests. The test results are as follows:

- Moisture Content:
 - 9% to 25%

The results of the moisture content tests are provided on the Record of Borehole sheets in **Appendix B**.

6.2.3 Sandy Silt and Silt Fill

Sandy silt and silt fill was encountered underlying the topsoil in Borehole KB-06-5 and underlying the sand and gravel fill in Borehole KB-07-4. The thickness of the sandy silt and silt fill ranged from approximately 1.8 m to 2.9 m.

The sandy silt and silt fill contained trace to some gravel and was generally moist. Fragments of asphalt and wood were generally encountered within the fill.

Based on the N-Values obtained from the SPTs, the compactness of the fill ranged from loose to compact.

Laboratory testing performed on selected samples consisted of moisture content tests and a grain size distribution test. The test results are as follows:

- Moisture Content:
 - 16% to 20%
- Grain Size Distribution:
 - 2 % gravel;
 - 11 % sand;
 - 77 % silt; and,
 - 10 % clay.

The results of the moisture content tests are provided on the Record of Borehole sheets in **Appendix B**. The results of the grain size distribution tests on the sandy silt are also provided on Figure 1 in **Appendix C**.

6.2.4 Peat

A 25 mm thick seam of peat was encountered underlying the fill material in Borehole KB-06-6 at depth of approximately 1.6 m below existing grade, elevation of approximately 244.0 m

6.2.5 Sandy Silt (ML)

Sandy silt was encountered underlying the topsoil or sand and gravel fill in Boreholes KB-06-1 to KB-06-4 and KB-06-6, and underlying the silty clay (discussed below) in Borehole KB-07-3 at depths in the range of approximately 0.1 m to 2.7 m, elevations of approximately 240.0 m to 247.2 m). Sandy silt was also encountered at the bottom of the river in Boreholes KB-07-1 and KB-07-2, at depths of approximately 1.1 m and 0.9 m below the surface of the river at the time of the investigation, elevations of approximately 239.3 m and 239.5 m.

The sandy silt generally contained trace gravel, trace clay, and was generally wet to saturated. Trace organic material was encountered in the boreholes advanced in the river.

Based on the N-Values obtained from the SPTs, the compactness of the sandy silt was variable ranging from very loose to very dense, but was typically loose to compact.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution and Atterberg Limits tests. The test results are as follows:

- Moisture Contents:
 - 12% to 33%, (higher moisture contents were associated with the samples obtained from the river bottom).

- Grain Size Distribution:
 - 0% to 2% gravel;
 - 18% to 36% sand;
 - 53% to 76% silt; and,
 - 6% to 9% clay.

Atterberg Limits testing was attempted on 3 selected samples. The results indicated the material was non-plastic.

The results of the moisture content and grain size distribution tests are provided on the Record of Borehole sheets in **Appendix B**.

The results of the grain size distribution tests on the sandy silt are also provided on Figure 2 in **Appendix C**.

6.2.6 Silty Clay (ML-CL)

Silty clay was encountered underlying the fill in Borehole KB-06-5, the sandy silt and silt in KB-06-1, KB-06-2, KB-06-6, KB-07-1, KB-07-2 and KB-07-3. The silty clay was encountered at depths in the range of approximately 2.0 m to 4.7 m, elevations of approximately 236.7 m to 243.0 m. The thickness of the silty clay was variable ranging from approximately 0.2 m to 5.5 m with the thickest silty clay deposit encountered in Borehole KB-06-2.

The silty clay generally contained trace sand and was generally moist to wet. Rock fragments were encountered in the silty clay obtained from Borehole KB-07-2.

In situ shear vane testing was carried out in the thickest deposit of silty clay. The results of a single test indicated that the shear strength of the silty clay was approximately 40 kPa. The in situ shear vane testing indicated that the consistency of the silty clay could be described as firm. It is noted that N-values from the SPTs indicated that the consistency of the silty clay increased with depth from soft to become very stiff to hard.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution and Atterberg Limits tests. The test results are as follows:

- Moisture Content:
 - 19% to 68%
- Grain Size Distribution:
 - 0% to 9% gravel;
 - 3% to 13% sand;
 - 40% to 65% silt; and,
 - 30% to 38% clay.
- Atterberg Limits:
 - Liquid Limits: 24 to 26
 - Plastic Limits: 14 to 15

The results of the moisture content, grain size distribution and Atterberg Limits tests, are provided on the Record of Borehole sheets in **Appendix B**.

The results of the grain size distribution tests are provided on Figure 3 in **Appendix C**. The results of the Atterberg Limits tests are provided on Figure 4 in **Appendix C**.

6.2.7 Silt (ML)

Silt was encountered underlying the silty clay in Boreholes KB-06-1, KB-06-2 and KB-07-1 at depths of in the range of approximately 3.8 m to 10.1 m, elevations of approximately 239.2 m to 232.5 m. The thickness of the silt ranged from approximately 2.1 m to 5.3 m.

The silt contained trace sand and trace to some clay. Trace rock and wood fragments were encountered in Borehole KB-06-2.

Based on the N-Values obtained from the SPTs, the compactness of the silt was very loose to compact.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution and Atterberg Limits tests. The test results are as follows:

- Moisture Contents:
 - 18% to 24%
- Grain Size Distribution:
 - 0% to 1% gravel;
 - 0% to 7% sand;
 - 81% to 94% silt; and
 - 6% to 17 % clay.

Atterberg Limits testing was attempted on two selected samples. The results indicated the material was non plastic.

The results of the moisture content and grain size distribution tests are provided on the Record of Borehole sheets in **Appendix B**.

The results of the grain size distribution tests are provided on Figure 5 in **Appendix C**.

6.2.8 Sand (SM)

Sand was encountered underlying the silt and sandy silt in Boreholes KB-06-1 and KB-06-3. The sand was encountered at depths of approximately 9.1 m and 8.7 m, corresponding to elevations of approximately 233.9 m and 235.4 m, in Boreholes KB-06-1 and KB-06-3, respectively. The sand was approximately 2 m thick in all boreholes.

The sand generally contained some silt, trace gravel, and was generally saturated.

Based on the N-Values obtained from the SPTs, the compactness of the sand was loose to compact.

Laboratory testing performed on selected samples consisted of moisture content tests. The test results are as follows:

- Moisture Content:
 - 11% to 15%.

The results of the moisture content tests are provided on the Record of Borehole sheets in **Appendix B**.

6.2.9 Glacial Till

Glacial till was encountered in Boreholes KB-06-1 to KB-06-3, KB07-1 and KB-07-3 drilled on the west side of the river. The glacial till generally consisted of sandy silt to silty sand till and clayey silt till.

6.2.9.1 Silty Sand Till and Sandy Silt Till (SM)

Silty sand till and sandy silt till was encountered in Boreholes KB-06-1 to KB06-3 and KB-07-3. The Silty sand till and sandy silt till was encountered at depths in the range of approximately 7.3 m to 12.2 m below existing grade, elevations in the range of approximately 231.4 m to 237.8 m. The till ranged in thickness from approximately 3.8 m to 6.7 m.

The till contained varying amounts of gravel and clay, and was generally wet to saturated.

Cobbles and boulders were encountered throughout the till during drilling.

Based on the N-Values obtained from the SPTs, the compactness of the till was variable ranging from compact to very dense.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

- Moisture Content:
 - 8% to 21%
- Grain Size Distribution:
 - 6% to 25% gravel;
 - 31% to 53% sand;
 - 19% to 45% silt; and,
 - 3% to 16% clay

Atterberg Limits testing was attempted on a single selected sample of the sandy silt till. The results indicated the material was non-plastic.

The results of the moisture content and grain size distribution tests are provided on the Record of Borehole sheets in **Appendix B**.

The results of the grain size distribution tests are provided on Figure 6 in **Appendix C**.

6.2.9.2 Clayey Silt (ML to ML-CL)

Clayey silt till was encountered in Boreholes KB-06-2, KB-07-1 and KB-07-3 at depths in the range of approximately 10.7 m to 16.8 m below existing grade, elevations of approximately 226.8 m to 234.0 m. The till ranged in thickness from approximately 5.8 m to 8.8 m. Borehole KB-07-3 was terminated in the clayey silt till stratum at a depth of approximately 11.3 m, elevation of approximately 233.7 m.

The clayey silt till contained varying amounts of sand and trace amounts of gravel, and was generally moist to wet.

Cobbles and boulders were encountered throughout the till during drilling.

Based on the N-Values obtained from the SPTs, the consistency of the silty clay will was considered to be very stiff to hard.

Laboratory testing performed on selected samples consisted of moisture content tests. The test results are as follows:

- Moisture Content:
 - 5% to 18%

The results of the moisture content and grain size distribution tests are provided on the Record of Borehole sheets in **Appendix B**.

6.3 Bedrock

Bedrock was encountered in all boreholes except Borehole KB-07-3 and consisted of grey and pink gneiss. The following table outlines the depth and elevation at which bedrock was encountered:

Area	Borehole Number	Depth Below Existing Grade to Bedrock Surface (m)	Elevation of Bedrock Surface (m)
West side of River (West Abutment)	KB-06-1	17.4	225.6
	KB-06-2	22.6	221.0
	KB-06-3	17.4	226.7
West Pier	KB-07-1	19.5	220.9
East Pier	KB-07-2	4.4	236.0
East side of River (East Abutment)	KB-06-4	3.4	243.9
	KB-06-5	3.8	241.4
	KB-06-6	4.9	240.8
	KB-07-4	2.4 *	244.2*
	KB-07-100	3.5 *	241.9*
	KB-07-101	6.5	241.6
	KB-07-102	4.6*	240.4*
	KB-07-103	5.8	238.5

Notes: * Boreholes terminated at auger refusal on inferred bedrock surface.

All boreholes except Boreholes KB-07-3, KB-07-4, KB-07-100 and KB-07-102 were terminated in the bedrock after confirmation coring was completed. Borehole KB-07-4, KB-07-100 and KB-07-102 were terminated at auger refusal, at a depths of approximately 2.4 m to 4.6 m below existing grade, elevations of approximately 244.2 m to 240.4 m, on inferred bedrock.

The observations of the rock cores are summarized as follows:

- Total Core Recovery (TCR): 100% to 86%, mean of approximately 100%;
- Solid Core Recover (SCR): 100% to 41%, mean of approximately 76%; and,
- Rock Quality Designation (RQD): 99% to 24%, mean of approximately 72%.

Laboratory testing performed on two samples of the rock consisted of unconfined compressive strength tests. The test results are as follows:

- Unconfined Compressive Strength:
 - 68 MPa and 104 MPa

The results of the rock core analysis are provided on the Record of Borehole sheets in **Appendix B**.

The results of the unconfined compressive strength are provided on Figure 7 in **Appendix C**.

6.4 Groundwater

It was not practical to measure the ground water on completion of drilling, given the methods employed to drill the boreholes included the use of drilling mud. However, water was encountered on the split spoon sampler during drilling at the depths and elevations noted in the following table:

Borehole Number	Groundwater First Encountered	
	Depth Below Existing Grade (m)	Elevation (m)
KB-06-1	1.5	241.5
KB-06-2	0.8	242.8
KB-06-3	1.5	242.6
KB-06-4	3.0	244.3
KB-06-5	1.5	243.7
KB-06-6	2.0	243.7
KB-07-1	Drilled in river	240.4 surface of river
KB-07-2	Drilled in river	240.4 surface of river
KB-07-3	3.9	241.1
KB-07-4	2.2	244.4

The river level was surveyed in July, 2007.

The drawings provided indicate that the water level in July 2006 was at elevation 240.3 m.

7.0 CLOSURE

A soil investigation is a limited sampling of a site. The information is gathered at specific borehole locations and can only be extrapolated to an undefined limited area around the borehole locations. The extent of the limited area depends on the variability of the soil and groundwater conditions as influenced by geological processes, as well as the history of the site reflecting natural conditions, construction activities and site use. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information.

We trust the above information meets with your present requirements. Should you have any questions or require further information, please do not hesitate to contact us at your convenience.

Regards,

JACQUES WHITFORD LIMITED

Original Signed By:

Geoffrey Creer, P.Eng.
Geotechnical Engineer

Original Signed By:

Fred J. Griffiths, Ph. D., P.Eng.
Designated Principal
MTO Foundations Contact



FOUNDATION DESIGN REPORT

Kabinakagami River Bridge Site 39W-009, Highway 11 Near Hearst, Ontario G.W.P. 5411-04-00 District – New Liskeard

8.0 DISCUSSION

8.1 General

The existing bridge at the Kabinakagami River is a five span structure with steel girders supporting a reinforced concrete deck. The existing bridge is approximately 107 m long and 10 m wide. The bridge conveys one west bound lane and one east bound lane of Highway 11 over the Kabinakagami River. The structure was reportedly constructed in 1942. The northern portion of the bridge deck and the protective barrier was reportedly replaced in 2002.

Based on a drawing dated December 1978, it is understood that the existing bridge structure is likely supported on a combination of shallow and deep foundations. The drawings indicate that the East Abutment and Pier 4 are likely supported by shallow foundations placed on bedrock. The drawings indicate the West Abutment and Piers 1, 2 and 3 are likely supported by deep foundations. The drawing does not indicate what the deep foundations are bearing on and the drawing does not identify the type of deep foundation.

The water level in the Kabinakagami River was reported to be at elevation 240.3 m in July 2006 and was surveyed at elevation 240.4 m in July 2007.

8.2 Proposed Development

The Ministry of Transportation (MTO) is planning to replace the existing bridge structure at the Kabinakagami River, located approximately 32 km west of Hearst, Ontario, with a new bridge structure. It is understood that the new structure will be constructed to the south of the existing structure and will include the construction of a permanent realignment of Highway 11.

Based on the preliminary structural design report, the new bridge will be a three span (30 m - 41 m – 30 m) bridge constructed with pre-cast pre-stressed concrete girders and a reinforced concrete deck. Finished grade at the east and west abutments will be at elevations of approximately 249.0 m and 247.7 m, respectively.

It is understood that the preferred option to support the new bridge will likely be a combination of shallow and deep foundations. The East Abutment and Pier 2 of the replacement bridge structure will be founded on spread footings resting on the underlying bedrock, while the West Abutment and Pier 1 will be supported on piles driven to bedrock. It is understood that the replacement structure will likely have semi-integral abutments.

The underside of the pile caps at the West Abutment and Pier 1 will be at elevations of approximately 240.5 m and 236.8 m, respectively. The underside of the footings for the East Abutment and Pier 2 will be at elevations of approximately 241.4 m and 236.0 m, respectively.

The realigned highway will be approximately 1.3 km in total length and extend from Sta. 22+900 to Sta. 24+200. The realigned section will be approximately 10 m wide and will likely be constructed on shallow embankments. Preliminary cross sections indicate that the embankments at the east abutment will be approximately 3 m to 3.5 m high, and the embankments at the west abutment will be approximately 4 m to 4.5 m high.

8.3 Subsurface Conditions

The subsurface conditions encountered at the Kabinakagami River Bridge are variable across the river.

On the east side of the river the soil profile consists of topsoil overlying fill, overlying sandy silt, overlying a thin layer of silty clay over bedrock. Bedrock was encountered at elevations in the range of approximately 236.0 m and 244.2 m

On the west side of the river bedrock was encountered much deeper at elevations in the range of approximately 220.9 m and 226.7 m. The overburden consists of several units of loose sandy and silty soils and firm low plastic silty clay overlying a dense glacial till.

Groundwater was first encountered on the split spoon sampler during drilling at depths in the range of approximately 0.8 m to 3.9 m below existing grade, elevations in the range of approximately 241.1 m to 244.4 m. The river level was at an elevation of approximately 240.4 m at the time of the investigation in July 2007.

8.4 Foundation Options

The following table provides a comparison of the foundation options considered for the replacement bridge structure.

Foundation Option		Advantages	Disadvantages	Relative Cost	Risks/Consequences
Spread Footings supported on bedrock	East side of river	Foundations can be placed on the underlying bedrock. Geotechnical Resistance at ULS and SLS are considered to be high. Requires the excavation of approximately 4 to 5 m of material to expose the underlying bedrock for the eastern portion of the bridge.	Excavations may be difficult in water bearing silts and sand.	Low	Excavation may be difficult in water bearing silts and sand to depths near to and below the river level.
	West side of river		Not practical due to depth to bedrock.	N/A	N/A

Foundation Option		Advantages	Disadvantages	Relative Cost	Risks/ Consequences
Spread footings on overburden soil	East side of river	Reduces excavation requirements	Lower Geotechnical resistance than footings on bedrock.	Low	Excavations will be below groundwater level. May require shoring to aid in dewatering.
	West side of river	Lower cost than using deep foundations	Low geotechnical resistance as a result of the underlying soils. Potential for settlement of the underlying soils. May require the use of settlement mitigation measures.	Medium	Likely not feasible due to low geotechnical resistance, even if settlement mitigation measures are taken.
Piles Driven to the underlying bedrock	East side of river	High geotechnical resistance Minimal settlement	Would require the use of very short piles Potential tip damage during driving.	High	Potential tip damage during driving.
	West side of river	High geotechnical resistance Minimal settlement	Would require the use of long piles that may require splicing. Drag Loads may be induced on the piles depending on the approach embankments. Possible tip damage, bending, doglegging, or breaking of the piles due to the presence of boulders.	High	Possible tip damage, bending, doglegging or breaking of the piles due to the presence of boulders. Require tip reinforcement
Friction piles	East side of river		Not practical given that bedrock was encountered at depths of about 3 to 4 m below existing grade	N/A	N/A
	West side of river	Reduces splicing	Reduction in geotechnical resistance per pile, increased number of piles. Piles likely spaced closer together, which would result in reduction of capacity for pile groups.	High	Potential bending, doglegging, or breaking of the piles due to the presence of boulders. Piles will require tip reinforcement.
Caissons founded on the underlying bedrock	East side of river	High geotechnical resistance. Augering to the bedrock should be relatively straight forward.	Water bearing silts and sands will be difficult to auger through. Steel liners would be required to keep the caisson open. May require mud drilling techniques to minimize base instability towards the river.	High	Steel Liners would be required to keep the caisson open. May require tremie concrete to develop groundwater seal.
	West side of river	High geotechnical resistance.	Not Practical due to anticipated depth.	N/A	N/A

Based on our review of the options, it is recommended that Pier 2 and the East Abutment be supported on shallow spread footings on bedrock. It is recommended that the West Abutment and Pier 1 be supported on H-piles end bearing on bedrock.

9.0 RECOMMENDATIONS

A list of the standard specifications, drawings and provisions referenced in this report is provided in **Appendix D**.

9.1 Axial Foundation Design

Given the subsurface conditions encountered at the site, it is recommended that the replacement structure be founded on a combination of shallow footings and deep foundations both placed on the underlying bedrock. It is recommended that the eastern portion of the bridge be founded on shallow spread footings and the western portion of the bridge be founded on steel H-piles driven to the underlying bedrock.

9.1.1 Spread Footings

The east portion of the replacement bridge may be supported on spread footings founded on the underlying bedrock. Bedrock was encountered in the boreholes drilled at the location of the planned East Abutment at depths in the range of approximately 3.5 m to 6.5 m, elevations of approximately 238.5 m to 241.9 m. Bedrock was encountered in the borehole advanced in the river for the planned Pier 2 at a depth of approximately 4.4 m, an elevation of about 236.0 m.

Spread footings 3.5 m in width, 10 m long and founded on the surface of the underlying sound bedrock at the depths outlined above, may be designed using a factored Ultimate Limit States (ULS) resistance of 3,000 kPa. The ULS value includes a resistance factor of 0.5. The Serviceability Limit States for 25 mm settlement exceeds the ULS value. Therefore, ULS will govern the design.

Where footings are constructed at multiple elevations and depths below grade, they should be stepped in accordance with Section 6.7.1 of the CHBDC.

9.1.1.1 Rock Anchors

It is understood that rock anchors may be required to assist in resisting overturning moments of the spread footings for the East Abutment and Pier No. 2. The following parameters are provided for the design of rock anchors:

- Ultimate (unfactored) bond strength 2,000 kPa
- Minimum bonded anchor length 3.0 m

Rock anchors should be constructed and tested in accordance to OPSS 942.

9.1.2 Piles

Given the conditions encountered during this investigation, it is recommended that the western portion of the replacement bridge be founded on piles driven to bedrock.

The West Abutment and Pier 1 could be founded on HP310x110 piles driven to the underlying bedrock. The piles for the West Abutment will have tip elevations in the range of approximately 221.0 m to 226.2 m. The piles for Pier 1 will likely have tip elevations of approximately 220.9 m.

HP310 x 110 Steel H-Piles for the West Abutment and Pier 1 driven to the underlying bedrock may be designed using a factored geotechnical resistance at ULS of 2,000 kN. The ULS value includes a resistance factor of 0.4.

For piles set on bedrock, the pile tip is not anticipated to settle. Therefore, the geotechnical resistance value at SLS will not govern the design. However, the structural engineer will need to evaluate the elastic compression of the pile.

9.1.2.1 Drag Forces on Piles

The placement of the fill material for the realigned highway embankments will induce settlement of the underlying soils. The majority of the settlement is expected to occur relatively quickly, likely during the construction of the embankments, given the sandy and silty nature of the underlying soils.

Therefore, if the embankments are constructed early in the schedule, drag forces are not anticipated at the abutment locations.

Drag forces are also not anticipated at the pier locations, as it is presumed that the grades at these locations will remain unchanged.

9.1.2.2 Tensile Resistance

Resistance to tensile loads should be calculated based on the shaft resistance of the piles in accordance with the CHBDC. A geotechnical resistance at ULS of 160 kN, may be considered for HP 310x110 H-piles in tension on the west side of the river.

The above ULS tensile resistances were calculated using a geotechnical resistance factor of 0.3 and a pile length of 17.5 m.

The above value does not include the weight of the pile.

9.2 Lateral Foundation Design

9.2.1 Spread Footings

The unfactored horizontal resistance for spread footings may be calculated using an unfactored coefficient of friction of 0.7 between cast-in-place concrete and sound bedrock.

9.2.2 Lateral Resistance for Vertical Piles

Passive lateral resistance for vertical piles should be calculated as per C6.8.7.1 (Static Analysis) of the CHBDC using the following unfactored geotechnical soil parameters:

Parameter	OPSS Granular B Type II	Native Sand and Silts	Silty Clay
Bulk Unit Weight (kN/m ³)	22	18	20
Effective friction angle	35°	28°	-
Coefficient of passive earth pressure	3.7	2.8	-
Design Undrained Shear Strength (kPa)	-	-	50

9.2.3 Lateral Pile Deflections

The coefficient of horizontal subgrade reaction that is used for deflection calculations may be estimated for cohesive soils as follows:

$$k_s = 67 C_u/d$$

Where k_s = the coefficient of horizontal subgrade reaction (force per volume)

C_u = undrained shear strength of the soil = 50 kPa for this application

d = pile diameter

The coefficient of horizontal subgrade reaction that is used for deflection calculation for non-cohesive soils may be estimated as follows:

$$k_s = n_h(z/d)$$

Where k_s = the coefficient of horizontal subgrade reaction (force per volume)

n_h = Co-efficient related to soil density. This may be taken as 1300 kN/m³ for compact to loose sandy soils (Table 20.3, p. 315, of the Canadian Foundation Engineering Manual, 3rd Edition, 1992)

z = depth below grade

d = pile diameter

9.2.4 Group Effects on Lateral Deflections

As per Section 6.8.9.2 of the CHBDC, the interaction effects of the piles must be considered where the centre to centre spacing is less than 2.5d (where d = the pile diameter) or 750 mm. The interaction generally results in the lateral load at a specific deflection being decreased.

The nature of pile-soil-pile interaction is complex, however is generally broken down into the following main components:

- alteration of the soil state due to pile installation and the potential overlap of the alterations when nearby piles are driven; and,
- superposition of strains and alterations of the soil failure zones when nearby piles are simultaneously loaded.

Studies (Reese, Isenhower and Wang, 2006) have reported the following reduction between single piles and pile groups.

Condition No. 1: Load is parallel to pile spacing

Pile Spacing c/c	Trailing Pile Group Pile Efficiency, e_T	Lead Pile Group Pile Efficiency, e_L
7d	1.0	1.0
4d	0.8	1.0
3d	0.7	0.9
2d	0.6	0.8

Condition No. 2: Load is perpendicular to pile spacing

Pile Spacing c/c	Group Pile Efficiency, e_p
4d	1.0
3d	0.9
2d	0.75

Where piles are on a skew to each other relative to the direction of load the Group Pile Efficiency may be calculated based on

$$e_s = (e_B^2 \cos^2 \alpha + e_p^2 \sin^2 \alpha)^{1/2}$$

where

e_B = either e_T or e_L from above

α = angle between direction of loading and the skew

Note that when piles are more than 3.3 pile diameters apart perpendicular to the direction of the load, the skew correction is not necessary. The lateral load at a specific deflection for each individual pile must consider the interaction of all piles within the group.

The reduction factor applied to a pile is the product of the efficiencies of all of the interactions of piles within that pile group.

9.3 Piling Notes

Steel H-piles should be equipped with Type II reinforced flanges as per OPSD 3000.100.

The piles on the west side of the river for the replacement structure are anticipated to be approximately 17 m to 23 m in length, which may require the piles to be spliced during driving. Welded splices for steel H-piles should be in accordance with OPSD 3000.150.

For integral abutments, it is recommended that the upper 3 m of the pile (immediately below the pile cap) be placed in a pre-augered hole lined with a corrugated steel pipe liner. The liner should have a diameter larger than that of the piles. The space between the pile and the liner should be filled with loose sand, such as OPSS concrete sand.

The following note should be added to the pile foundation drawings:

“Piles to be driven to bedrock”.

9.4 Earth Pressure Design

To prevent hydrostatic pressure build-up, backfill against the abutments should consist of free draining granular materials and a sub-drain should be installed as per OPSD 3102.100. OPSS Granular A or OPSS Granular B, Type II is recommended. The zone of granular backfill must be constructed in accordance with Figure C6.20 (CHBDC Commentary) and OPSD 3101.150, using a frost penetration depth, f , of 2.6 m

Earth pressure coefficients are provided below for different backslope conditions. In order to use the coefficients of pressure for a particular granular material, the granular backfill must be provided within a wedge extending from the base of the abutment at 45° (or smaller) to the horizontal. If a smaller wedge is used, the coefficients of earth pressure of the materials outside the backfill wedge must be used for lateral pressure design calculations.

For rigidly tied structures (e.g. bridge abutments), the at-rest pressure should be used for design, unless the wall can deflect enough (approximately 0.05% of the wall height) to establish the active pressure. The effect of compaction should be accounted for as per CHBDC Figure 6.9.3.

Lateral earth pressures may be calculated using the parameters in the following table:

Parameters	OPSS Granular A	OPSS Granular B, Type II	Native Sands and Silts
Unit Weight (kN/m ³)	22	22	18
Angle of Internal Friction, ϕ	35°	35°	28°
Horizontal Backslope			
Coeff. of Active Earth Pressure, K_a	0.27	0.27	0.36
Coeff. of Passive Earth Pressure, K_p	3.69	3.69	2.77
Coeff. of Earth Pressure at Rest, K_o	0.43	0.43	0.53
2H:1V backslope			
Coeff. of Active Earth Pressure, K_a	0.39	0.39	0.63
Coeff. of Passive Earth Pressure, K_p	10.82	10.82	6.51

9.5 Soil Profile Type and Seismic Forces

The zonal acceleration ratio for Hearst is 0.00 as per CHBDC Table A3.1.1. Therefore, seismic design is apparently not required.

Soil Profile IV as defined in CHBDC Section 4.4.6 would be appropriate for this site.

9.6 Embankment Design and Construction

The existing embankments are constructed at 2H:1V and exhibit no signs of instability.

The embankments for the replacement structure will be constructed to the south of the existing embankments, and will be approximately 3 m to 3.5 m high at the east abutment and approximately 4 m to 4.5 m high at the west abutment.

The placement of the fill material will induce settlement of the underlying soils. Calculations indicate that the realignment embankment section may induce total settlements of the underlying sands, silts and clayey silts in the order of approximately 75 mm to 100 mm near the centre of the proposed realignment. However, the majority of the settlement is anticipated to occur quickly, likely during the construction of the embankments, given the sandy and silty nature of the native soils.

Prior to placing the fill, all topsoil, loose, wet, organic and other deleterious material should be removed from the area of the proposed embankment. The exposed subgrade of the embankment should be proof rolled, inspected and certified in accordance with SP902S01, prior to the placement of any fill materials.

The embankment should be constructed of OPSS Select Subgrade Material or earth fill in accordance with OPSS 206 and 501.

It is recommended that the realigned embankments be constructed no steeper than 2H:1V.

10.0 CONSTRUCTION RECOMMENDATIONS

10.1 Open Cut Excavations

Earth excavation, if required, should be carried out in accordance with OPSS-206. Side slopes for open cut excavations should conform to the requirements of the Occupational Health and Safety Act and Regulations for Construction Projects current at the time of construction.

In accordance with the present act, the existing fill and native soils should be considered Type 3 soils. Temporary excavations should be made with side slopes no steeper than 1:1 (horizontal:vertical) from the base of the excavation. Flatter side slopes will be required for open cut excavations in loose sand deposits below the water line unless appropriate dewatering methods are employed.

Excavation side slopes should be protected from erosion and should be inspected regularly for signs of instability. Slopes should be flattened as required to maintain safe working conditions.

Shoring should be provided in accordance with the recommendations provide herein, when excavations are in close proximity to existing infrastructure and embankments.

10.2 Staging

Through discussions with representatives of Lea Consulting, it is understood that Stage 1 will consist of the construction of the new bridge structure and the majority of the planned highway realignment. Stage 2 will consist of connecting the new alignment to the existing highway. Stage 3 will consist of the removal of the existing bridge structure.

10.3 Groundwater Control

The water level in the river was determined to be at an elevation of approximately 240.4 m in July 2007.

The following table outlines the depth at which water or wet conditions were encountered during the investigation:

Foundation Element	Borehole	Water level or wet conditions encountered	
		Depth (m)	Elevation (m)
West Abutment	KB-06-2	0.8	242.8
	KB-06-3	1.5	242.6
Pier 1	KB-07-1	River Surface	240.4
Pier 2	KB-07-2		
East Abutment	KB-06-5	1.5	243.7
	KB-06-6	2.0	243.7

Given the depth to where water or wet conditions were encountered at the abutment locations and the planned underside of footing and pile cap elevations, water and seepage will be encountered at the abutment locations. However, it is anticipated that any water and seepage should be readily handled using conventional sumps and pumps.

The planed piers will be in close proximity to the water, and at the time of construction may even be in the river. In this respect, coffer dams will be required at the pier locations.

10.4 Shoring and Cofferdams

It is anticipated that shoring will be required for the construction of the abutments. Given the piers will be located within the river, cofferdams will be required for the construction of the piers. Therefore, it is recommended that both the shoring and cofferdams consist of steel sheet piles.

The sheet piles may be designed using the parameters provided in Section 9.4 Earth Pressure Design. The water level of the river should be taken into account when designing the shoring.

Where shoring is required, it should meet the requirements of Performance Level 2 as per SP105S19.

The design of shoring will need to account for basal heave due to flow of water beneath the sheet piling. Basal heave and seepage should be controlled by extending the shoring below the proposed excavation depth and placing a working mat of concrete, i.e., tremie concrete or a concrete mud mat, at the base of the excavation. Dewatering should be carried out from within the shoring.

Should the geometry of the foundations preclude the use of tremie to control groundwater flow, dewatering the excavation with well points could be achieved by installing a second set of sheet piles around the first set and installing the well points between the two sets of sheet piles.

It is anticipated that the construction of shoring and the piers will likely require a barge.

10.5 Erosion Control and Scour Protection

Slope protection and drainage measures will be required to ensure the long-term stability of the embankment slopes. Vegetation should be established as soon as possible after completion of the embankments in order to control surface erosion. Erosion control should be in accordance with OPSS 572.

The river slopes within 3 m of the structures should be surfaced with rip-rap at least 600 mm thick and placed on a Class II non-woven filter fabric. At other locations, as previously noted, normal slope vegetation should be established as soon as possible after completion of the embankment fills in order to control surface erosion.

The 50 year flood water velocity of the river in the vicinity of the bridge, reportedly ranges from approximately 1.0 m/s to 1.9 m/s, according to the Hydrology report by D. M. Wills, dated March 2007. Where foundation elements are in close proximity to flowing water they should be protected from scour. Rock protection with particles ranging from 200 mm to 400 mm in size should be provided within 3 m of all such foundation elements. The rock protection layer should be at least 600 mm thick.

The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt and sediment from running off the site.

10.6 Frost Protection

The site is located in an area with a mean freezing index of between 1250 and 1500 Degree days (°Days), (Canadian Foundation Engineering Manual 1992). Based on Figure 3.4 of the MTO Pavement Design and Rehabilitation Manual, the frost penetration depth for this area is 2.6 m.

11.0 CLOSURE

Use of this report is subject to the Statement of General Conditions attached. It is the responsibility of Lea Consulting Limited and the Ministry of Transportation Ontario, who are identified as “the Client” within the Statement of General Conditions, and its agents to review the conditions and to notify Jacques Whitford Limited should any these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying or unexpected site conditions
- Planning, design or construction

Regards,

JACQUES WHITFORD LIMITED

Original Signed By:

Geoffrey Creer, P.Eng.
Geotechnical Engineer

Original Signed By:

Fred J. Griffiths, Ph. D., P.Eng.
Designated Principal
MTO Foundations Contact

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STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Jacques Whitford Limited and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Jacques Whitford's present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Jacques Whitford is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Jacques Whitford at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

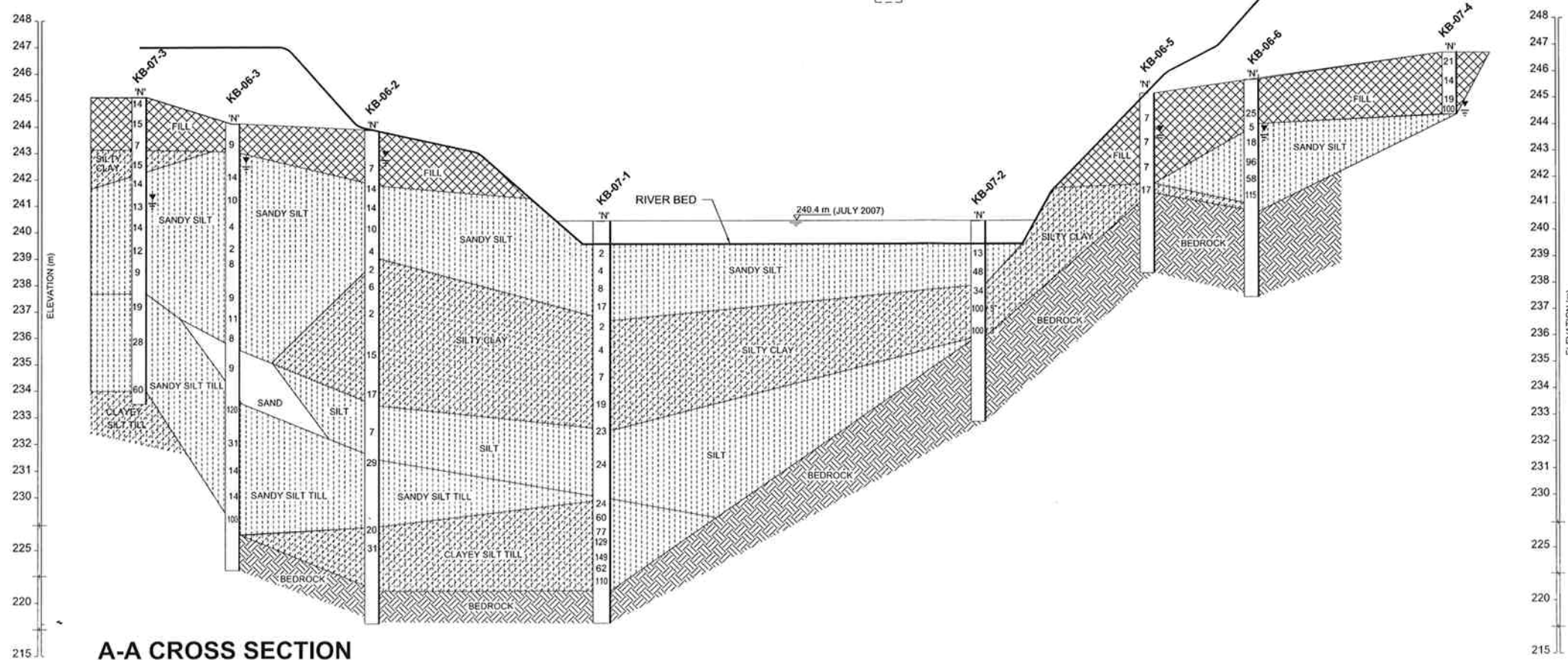
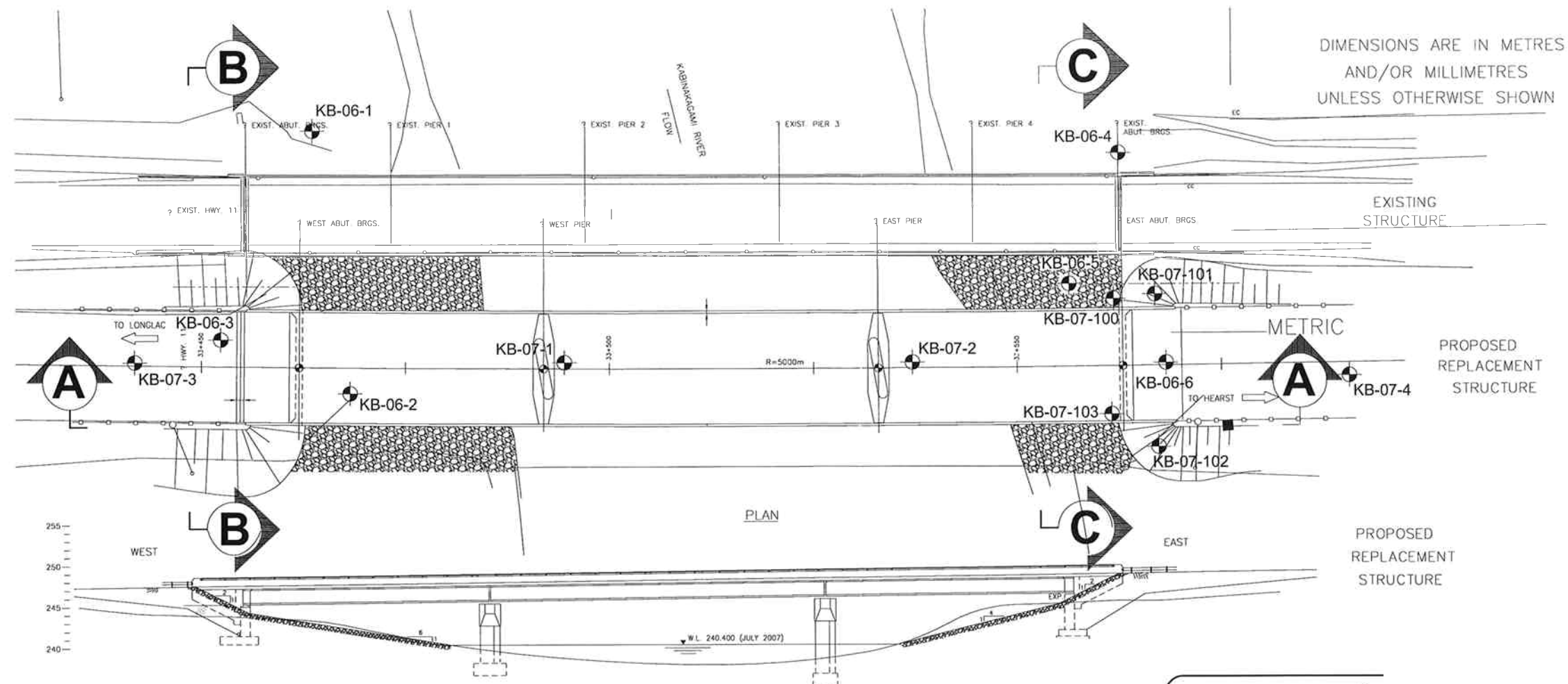
VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Jacques Whitford must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Jacques Whitford will not be responsible to any party for damages incurred as a result of failing to notify Jacques Whitford that differing site or sub-surface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Jacques Whitford, sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Jacques Whitford cannot be responsible for site work carried out without being present.



Appendix A

Drawings

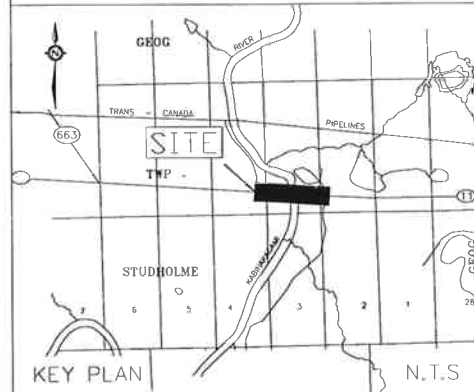


CONT No 2007-5104
 WP No 5411-04-00

KABINAKAGAMI RIVER
 REPLACEMENT BRIDGE
 BOREHOLE LOCATIONS AND
 SOIL STRATA



SHEET



LEGEND:



BOREHOLE



GROUNDWATER LEVEL

BOREHOLE No.	STATION	OFFSET	ELEVATION
KB-06-1	23+463	29m Lt	243.0 m
KB-06-2	23+468	3m Rt	243.6 m
KB-06-3	23+451	3m Lt	244.1 m
KB-06-4	23+562	21m Lt	247.3 m
KB-06-5	23+555	10m Lt	245.2 m
KB-06-6	23+567	0.5m Lt	245.7 m
KB-07-1	23+495	0.5m Lt	240.4 m
KB-07-2	23+537	0.5m Lt	240.4 m
KB-07-3	23+442	0.5m Lt	245.0 m
KB-07-4	23+590	1.5m Rt	246.6 m
KB-07-100	23+562	8m Lt	245.3 m
KB-07-101	23+567	9m Lt	248.1 m
KB-07-102	23+567	10m Lt	245.1 m
KB-07-103	23+562	6m Rt	244.3 m

NOTES:

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions.
- Base plan provided by LEA Consulting Ltd.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration only. The proposed structure location and features are shown for information purposes only.
- GEOCRE No. 42F-19



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	AA	CHK	CC
DRAWN	CC	CHK	FG
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LOAD	QNT	CL	E25
DATE	2008-01-10		
SITE	39W-009	STRUCT	SCHEME
DWG	1		

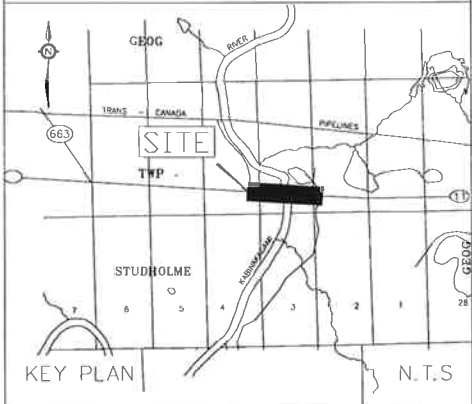
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UNLESS OTHERWISE SHOWN

CONT No2007-5104
WP No 5411-04-00

KABINAKAGAMI RIVER
REPLACEMENT BRIDGE
SOIL STRATA PLOTS
(SECTIONS B-B, C-C)



SHEET



LEGEND:



GROUNDWATER LEVEL

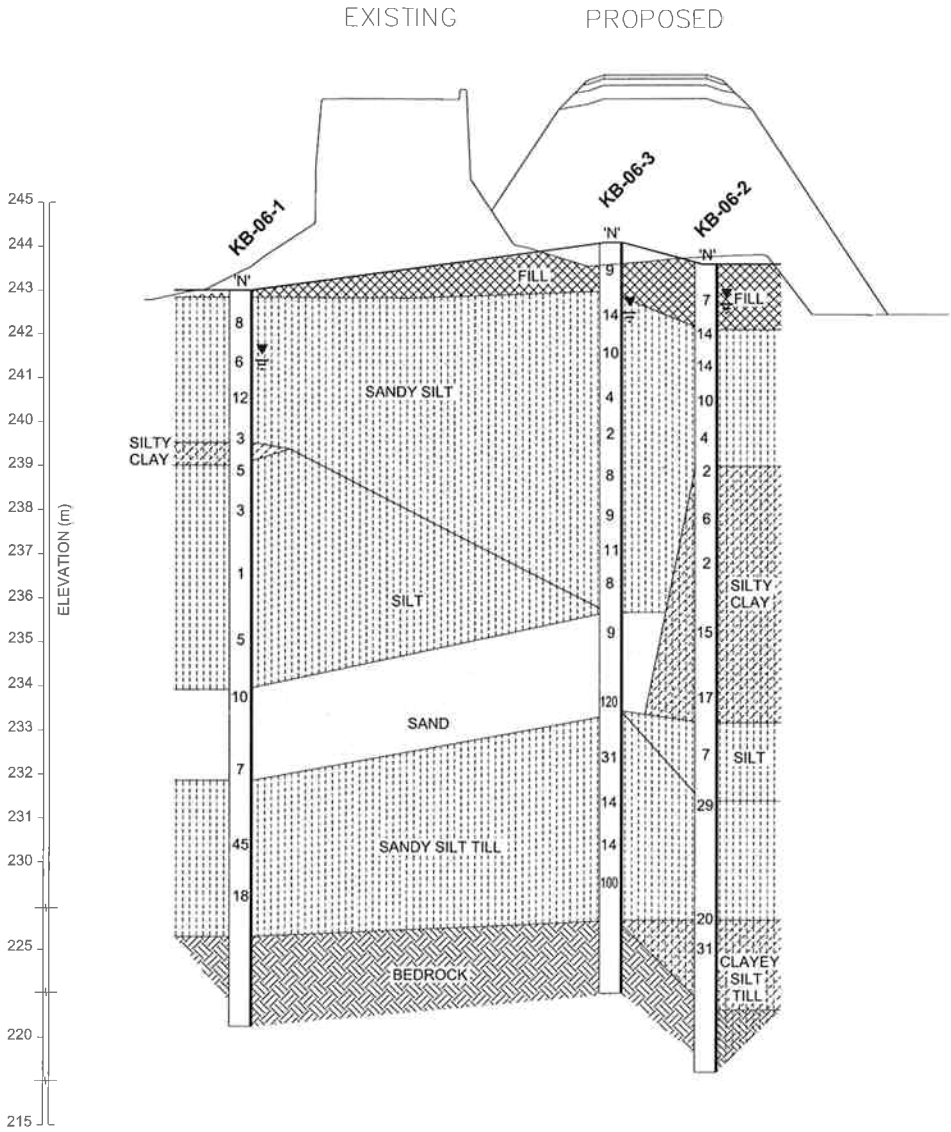
BOREHOLE No.	STATION	OFFSET	ELEVATION
KB-06-1	23+463	29m Lt	243.0 m
KB-06-2	23+468	3m Rt	243.6 m
KB-06-3	23+451	3m Lt	244.1 m
KB-06-4	23+562	21m Lt	247.3 m
KB-06-5	23+555	10m Lt	245.2 m
KB-06-6	23+567	0.5m Lt	245.7 m
KB-07-1	23+495	0.5m Lt	240.4 m
KB-07-2	23+537	0.5m Lt	240.4 m
KB-07-3	23+442	0.5m Lt	245.0 m
KB-07-4	23+590	1.5m Rt	246.6 m
KB-07-100	23+562	8m Lt	245.3 m
KB-07-101	23+567	9m Lt	248.1 m
KB-07-102	23+567	10m Lt	245.1 m
KB-07-103	23+562	6m Rt	244.3 m

NOTES:

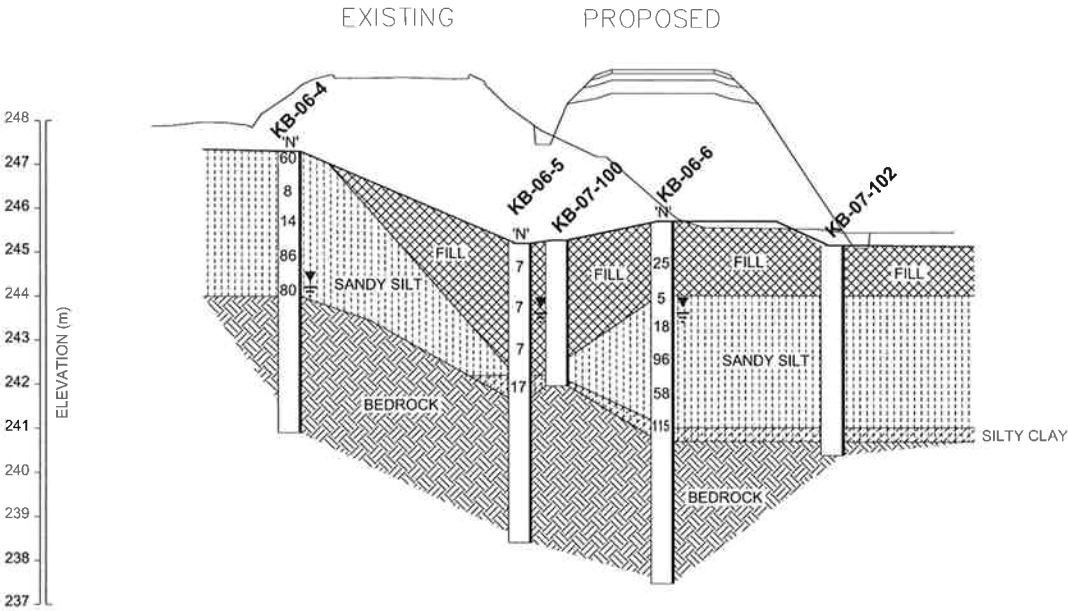
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions.
- Base plan provided by LEA Consulting Ltd.
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- GEOCREs No. 42F-19



B-B CROSS SECTION



C-C CROSS SECTION



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	AA	CHK CC	CODE CHBCC-00
DRAWN	EG	CHK EG	SITE 39W-009
			STRUCT
			SCHEVE
			DWG 2

Appendix B

Terms and Symbols Used on the Record of Borehole Sheet
Record of Borehole Sheets

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

<i>Topsoil</i>	-	mixture of soil and humus capable of supporting good vegetative growth
<i>Peat</i>	-	fibrous fragments of visible and invisible decayed organic matter
<i>Till</i>	-	unstratified and unsorted glacial deposit which may include particle sizes from clay to boulders
<i>Fill</i>	-	materials not identified as deposited by natural geological processes

Terminology describing soil structure:

<i>Desiccated</i>	-	having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	-	material breaks along plane of fracture
<i>Varved</i>	-	composed of regular alternating layers of silt and clay
<i>Stratified</i>	-	alternating layers or beds greater than 6mm (1/4") thick
<i>Laminated</i>	-	alternating layers or beds less than 6mm (1/4") thick
<i>Blocky</i>	-	material can be broken into small and hard angular lumps
<i>Lensed</i>	-	irregular shaped pockets of soil with differing textures
<i>Seam</i>	-	a thin, confined layer of soil having different particle size, texture, or color from materials above and below
<i>Well Graded</i>	-	having wide range in grain sizes and substantial amounts of all intermediate particles sizes
<i>Uniformly Graded</i>	-	predominantly one grain size

Soil descriptions and classification are based on the Unified Soil Classification System (USCS) (ASTM D-2488), which classifies soils on the basis of engineering properties. The system divides soils into three major categories: (1) coarse grained, (2) fine-grained, and (3) highly organic. The soil is then subdivided based on either gradation or plasticity characteristics. This system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification. The classification excludes particles larger than 76 mm.

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with the standard of the Ministry of Transportation of Ontario:

<i>Trace or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>With</i>	20-30%

The standard terminology to describe cohesionless soils includes the compactness as determined by the Standard Penetration Test 'N'-value*.

Compactness	'N'-value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

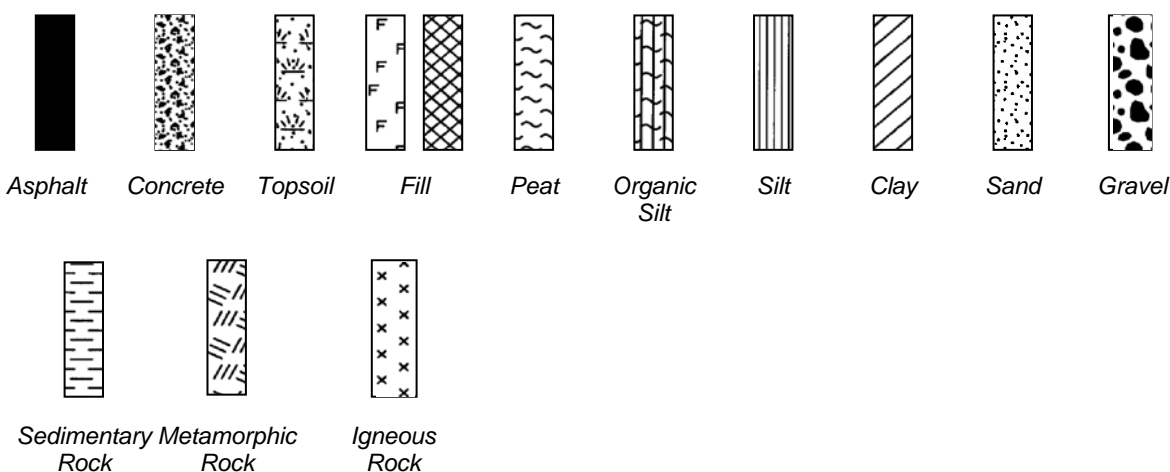
The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests or similar field and laboratory analysis. Standard Penetration Test 'N'-values* can also be used to provide an approximate indication of the consistency and shear strength of fine grained, cohesive soils.

Consistency	Undrained Shear Strength (kPa)	'N'-Value
Very Soft	<12.5	<2
Soft	12.5-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Note: **N'-VALUE- The Standard Penetration Test records the number of blows of a 140 pound (64kg) hammer falling 30 inches (760mm), required to drive a 2 inch (50.8mm) O.D. split spoon sampler 1 foot (305mm). For split spoon samples where full penetration is not achieved, the number of blows is reported over the sampler penetration in millimeters (e.g. 50/75).

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

SAMPLE TYPE

SS	Split spoon sample (obtained from the Standard Penetration Test)	BS	Bulk sample
TW	Thin Wall Sample or Shelby Tube	WS	Wash sample
PS	Piston sample	HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond drilling bits.
GS	Grab sample		
AS	Auger sample		
VT	Vane Test		

RECORD OF BOREHOLE No KB-06-1

1 OF 2

METRIC

W.P. 5411-04-00 LOCATION Kabinakagami River Sta. 23+463 o/s 29 m Lt. ORIGINATED BY DS
DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY JL
DATUM Geodetic DATE 07.01.20 - 07.02.20 CHECKED BY GTC

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
243.0	100 mm TOPSOIL		1	AS	-	▽	242	○ UNCONFINED	✕ FIELD VANE					0 29 65 6 Limit Results: Non-Plastic
242.0	Fine Sandy SILT(ML), moist to saturated Loose Brown		2	SS	8		241							
	- compact		3	SS	12		240							
	- loose		4	SS	6		239							
	- very loose		5	SS	3		238							
239.6	Silty CLAY, trace sand, moist Firm Brown		6	SS	5		237							
3.4			7	SS	3		236							
239.2	SILT(ML), trace sand, trace clay, saturated Loose to very loose Grey		8	SS	WH		235							
3.8			9	SS	1		234							
	- loose		10	SS	5		233							
233.9	SAND(SM), trace gravel, saturated Compact to loose Brown		11	SS	10	232								
			12	SS	7	231								
231.9	Sandy SILT(ML)(TILL), some gravel, wet to saturated Loose Grey - cobbles and/or boulders		13	SS	45	230								
	- dense		14	SS	18	229								


Continued Next Page

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

METRIC

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						
								20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED ✕ FIELD VANE ● QUICK TRIAXIAL ✕ LAB VANE 20 40 60 80 100 WATER CONTENT (%) 10 20 30		GR SA SI CL	

[illegible]

 ³,  ³: Numbers refer to Sensitivity
  ³% STRAIN AT FAILURE

RECORD OF BOREHOLE No KB-06-2

1 OF 2

METRIC

W.P. 5411-04-00 LOCATION Kabinakagami River Sta. 23+468 o/s 3 m Rt. ORIGINATED BY DS
 DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY JL
 DATUM Geodetic DATE 07.01.20 - 07.01.20 CHECKED BY GTC

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	✕ FIELD VANE						
							20 40 60 80 100	20 40 60 80 100							
							● QUICK TRIAXIAL	✕ LAB VANE							
243.6	50 mm TOPSOIL		1	AS	-	▽									
241.9	SAND and GRAVEL (FILL), some wood fragments, moist Loose to compact Brown		2	SS	7										
241.9	- 100 mm wood debris		3	SS	14										
1.7	Sandy SILT (ML), moist Compact to very loose Brown		4	SS	14										
	- 125 mm wood debris		5	SS	10										
	- Organic matter, wood fragments and roots.		6	SS	4										
239.0	Silty CLAY (CL), trace to some sand, wet Grey - firm		7	SS	2										
4.6			8	SS	6										
			9	SS	2										
	- some rootlets - stiff		10	SS	15										
	- very stiff		11	SS	17										
233.5	SILT (ML), some clay, trace rock fragments and wood fragments, saturated Loose Grey		12	SS	7										
231.4	Sandy SILT (SM) (TILL), some to trace gravel, moist Compact Grey - cobbles and/or boulders		13	SS	29										

ONTARIO MOT 1015345 KABINA 05 GPJ ONTARIO MOT.GDT 08/01/08

Continued Next Page

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

RECORD OF BOREHOLE No KB-06-2



2 OF 2

METRIC

W.P.	5411-04-00	LOCATION	Kabinakagami River Sta. 23+468 o/s 3 m RI.	ORIGINATED BY	DS
DIST	New Liskeard HWY 11	BOREHOLE TYPE	Hollow Stem Auger, Split Spoon	COMPILED BY	JL
DATUM	Geodetic	DATE	07.01.20 - 07.01.20	CHECKED BY	GTC

[illegible]

ONTARIO MOT 1015345 KABINA 06.GPJ ONTARIO MOT.GDT 08/01/08

 ³,  ³: Numbers refer to Sensitivity
  ³% STRAIN AT FAILURE

RECORD OF BOREHOLE No KB-06-3

1 OF 2

METRIC

W.P. 5411-04-00 LOCATION Kabinakagami River Sta. 23+451 o/s 3 m Lt. ORIGINATED BY DS
 DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY JL
 DATUM Geodetic DATE 07.02.15 - 07.02.15 CHECKED BY GTC

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	✕ FIELD VANE						
								● QUICK TRIAXIAL	✕ LAB VANE						
244.1							20 40 60 80 100							GR SA SI CL	
240.0	150 mm TOPSOIL		1	AS	-		244								
0.2	SAND and GRAVEL (FILL), moist Brown Loose														
243.0			2	SS	9		243								
1.1	Sandy SILT (ML), some gravel, moist Loose to compact Brown		3	SS	14		242								
			4	SS	10		241								
			5	SS	4		240								
			6	SS	2		239								
			7	SS	8		238								
			8	SS	9		237								
			9	SS	11		236								
			10	SS	8		235								
			11	SS	9		234								
			12	SS	120		233								
235.4						232									
8.7	SAND (SM), some silt, saturated Loose Brown		13	SS	31		231								
233.4			14	SS	14		230								
10.7	Sandy SILT (ML)(TILL), some clay, wet to saturated Dense to very dense Grey - cobbles and/or boulders														

ONTARIO MOT. 1015345 KABINA 06 GPJ ONTARIO MOT. GDT 08/01/14

Continued Next Page

×³, ×³: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

6 39 45 10
Limit Results:
Non-Plastic

RECORD OF BOREHOLE No KB-06-3

2 OF 2

METRIC

W.P.	5411-04-00	LOCATION	Kabinakagami River Sta. 23+451 o/s 3 m Lt.	ORIGINATED BY	DS
DIST	New Liskeard HWY 11	BOREHOLE TYPE	Hollow Stem Auger, Split Spoon	COMPILED BY	JL
DATUM	Geodetic	DATE	07.02.15 - 07.02.15	CHECKED BY	GTC

[illegible]

\times^3, \times^3 : Numbers refer to Sensitivity

 $\bigcirc^{3\%}$ STRAIN AT FAILURE

ONTARIO MOT 1015345 KABINA 05.GPJ ONTARIO MOT.GDT 08/01/14

RECORD OF BOREHOLE No KB-06-4

1 OF 1

METRIC

W.P. 5411-04-00 LOCATION Kabinakagami River Sta. 23+562 o/s 21 m Lt. ORIGINATED BY DS
DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY JL
DATUM Geodetic DATE 07.01.17 - 07.01.17 CHECKED BY GTC

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
247.3	100 mm TOPSOIL		1	NR	60	▽	247							2 36 53 9 Limit Results: Non-Plastic
246.1	Sandy SILT(ML), trace to some clay, trace gravel, wet Very dense Brown - loose		2	SS	8		246							
	- trace gravel and rock fragements compact		3	SS	14		245							
	- very dense		4	SS	86		244							
243.9	GNEISS BEDROCK		5	SS	80		243							
3.4	Grey and pink TCR = 100% SCR = 83% RQD = 75%		6	NQ	--		242							
	TCR = 100% SCR = 99% RQD = 97%		7	NQ	--		241							
240.9	END OF BOREHOLE at approximately 6.4 m													
6.4	Groundwater first encountered during drilling on spoon at a depth of approximately 3 m below existing grade, Elev. 244.3 m													

ONTARIO MOT 1015345 KABINA 06 GPJ ONTARIO MOT GDT 08/01/14

RECORD OF BOREHOLE No KB-06-5

1 OF 1

METRIC

W.P. 5411-04-00 LOCATION Kabinakagami River Sta 23+555 o/s 10 m Lt. ORIGINATED BY DS
 DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY JL
 DATUM Geodetic DATE 1.18.07 - 1.18.07 CHECKED BY GTC

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								20 40 60 80 100								
								○ UNCONFINED	× FIELD VANE	● QUICK TRIAXIAL						
245.2	100 mm TOPSOIL		1	NR	--		245									
240.9	Sandy SILT (FILL), some asphalt, moist Brown Loose		2	SS	7		244									
			3	SS	7		243									
	- some wood fragments		4	SS	7		242									
242.2	Silty CLAY (ML-CL), trace sand, some wood fragements, wet Very stiff Brown		5	SS	17		242									
241.4	GNEISS BEDROCK Grey and pink TCR = 100% SCR = 98% RQD = 78%		6	NQ	--		241									
	TCR = 100% SCR = 100% RQD = 99%		7	NQ	--		240									
238.3	END OF BOREHOLE at approximately 6.6 m Groundwater first encountered during drilling on spoon at a depth of approximately 1.5 m below existing grade, Elev. 243.7 m							239								
6.9																

ONTARIO MOT 1015345 KABINA 06 GPJ ONTARIO MOT.GDT 1/10/08

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

RECORD OF BOREHOLE No KB-06-6

1 OF 1

METRIC

W.P. 5411-04-00 LOCATION Kabinakagami River Sta. 23+567 o/s 0.5 m Lt. ORIGINATED BY DS
 DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY JL
 DATUM Geodetic DATE 07.01.20 - 07.01.20 CHECKED BY GTC

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								○ UNCONFINED	× FIELD VANE	× LAB VANE					
245.7	100 mm TOPSOIL		1	NR	-										
244.0	SAND (FILL), some gravel, some silt, moist Compact Brown		2	SS	25										
244.0	- loose		3	SS	5										
244.0	25 mm PEAT Brown		4	SS	18										
244.0	Sandy SILT (ML), some gravel, trace to some clay, moist Loose to compact Brown		5	SS	96/ 150 mm										
	- very dense - rock fragments		6	SS	58										
	- trace rock fragments		7	SS	115/ 150 mm										
241.0	150 mm varved Silty CLAY (CL), trace sand, moist Hard Grey		8	NQ	--										
	GNEISS BEDROCK Grey and pink TCR = 100% SCR = 93% RQD = 78%		9	NQ	--										
	TCR = 100% SCR = 83% RQD = 73%		10	NQ	--										
	TCR = 100% SCR = 93% RQD = 93%														
237.5	END OF BOREHOLE at approximately 8.2 m														
8.2	Groundwater first encountered during drilling on spoon at a depth of approximately 2.0 m below existing grade, Elev. 243.7 m														

ONTARIO MOT. 1015345 KABINA.06.GPJ ONTARIO MOT.GDT 08/01/08

ONTARIO MOT 1015345 KABINA 06.GPJ ONTARIO MOT.GDT 08/01/08

RECORD OF BOREHOLE No KB-07-1

1 OF 2

METRIC

W.P. 5411-04-00 LOCATION Kabinakagmi River Sta. 23+495 o/s 0.5 m Lt. ORIGINATED BY DS
DIST New Liskeard HWY 11 BOREHOLE TYPE Steel Casing, Split Spoon COMPILED BY NH
DATUM Geodetic DATE 07.07.05 - 07.07.07 CHECKED BY GTC

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED ● QUICK TRIAXIAL	✕ FIELD VANE ✕ LAB VANE					
240.4 0.0	River Surface WATER						20 40 60 80 100	20 40 60 80 100						
239.3 1.1	Sandy SILT (ML), trace organic matter, trace wooden fragments, saturated Very loose to compact Grey		1	SS	2									
			2	SS	4									
			3	SS	8									
			4	SS	17									
236.7 3.7	Silty CLAY (CL), trace gravel, trace sand, saturated Soft to very stiff Grey		5	SS	2									
			6	SS	4									
			7	SS	7									
			8	SS	19									
232.5 7.9	SILT (ML), some clay, trace sand, trace gravel, saturated Compact Grey		9	SS	23									
			10	SS	24									
229.7 10.7	Clayey SILT (ML) TILL, some sand, trace gravel, moist Very stiff to hard Grey		11	SS	24									
			12	SS	60									
			13	SS	77									
													</	

ONTARIO MOT 1015345 KABINA JULY07 - FOUNDATIONS GPJ ONTARIO MOT.GDT 08/01/11

Continued Next Page

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

RECORD OF BOREHOLE No KB-07-1

2 OF 2

METRIC

W.P.	5411-04-00	LOCATION	Kabinakagmi River Sta. 23+495 o/s 0.5 m Lt.	ORIGINATED BY	DS
DIST	New Liskeard HWY 11	BOREHOLE TYPE	Steel Casing, Split Spoon	COMPILED BY	NH
DATUM	Geodetic	DATE	07.07.05 - 07.07.07	CHECKED BY	GTC

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	w _p	w		
	Clayey SILT (ML) TILL, some sand, trace gravel, moist Very stiff to hard Grey (continued)		14	SS	129								
			15	SS	149								
			16	NQ	-								
			17	SS	62								
			18	SS	110								
			19	NQ	-								
220.9 19.5	- boulders and cobbles at 19.2 m to 19.5 m Poor to fair quality reddish grey GNEISS - close to moderate joint spacing, dip 10 degrees and 80 degrees - fresh TCR = 100% SCR = 71% RQD = 65% TCR = 100% SCR = 63% RQD = 58%		1	NQ	-								
			2	NQ	-								
218.0 22.4	TCR = 100% SCR = 75% RQD = 49% END OF BOREHOLE at approximately 22.4 m		3	NQ	-								

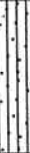


✕³, ✕³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

RECORD OF BOREHOLE No KB-07-2

1 OF 1

METRIC

W.P. 5411-04-00 LOCATION Kabinakagmi River Sta. 23+537 o/s 0.5 m Lt. ORIGINATED BY DS
DIST New Liskeard HWY 11 BOREHOLE TYPE Steel Casing, Split Spoon COMPILED BY NH
DATUM Geodetic DATE 07.07.08 - 07.07.08 CHECKED BY GTC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
240.4 0.0	River Surface WATER													
239.5 0.9	Sandy SILT (ML), trace clay, trace gravel, saturated Compact to dense Brown		1	SS	13									
238.0 2.4			2	SS	48									
236.0 4.4	Silty CLAY (CL-ML), some sand, trace gravel, trace rock fragments, saturated Hard Brown		3	SS	34									
			4	SS	100/5"									
			5	SS	100/3"									
232.8 7.6	Very poor to poor quality grey GNEISS - close to moderate joint spacing, dip 10 degrees and 90 degrees - thin bedding, dip 0-15 degrees - fresh to slightly weathered TCR = 100% SCR = 49% RQD = 49% TCR = 100% SCR = 56% RQD = 24%		1	NQ	-									
			2	NQ	-									
232.8 7.6	END OF BOREHOLE at approximately 7.6 m													

RECORD OF BOREHOLE No KB-07-3

1 OF 1

METRIC

W.P. 5411-04-00 LOCATION Kabinakagmi River Sta. 23+442 o/s 0.5 m Lt. ORIGINATED BY NH
 DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY NH
 DATUM Geodetic DATE 07.07.19 - 07.07.19 CHECKED BY GTC

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 (%)						
								○ UNCONFINED ● QUICK TRIAXIAL	✕ FIELD VANE ✕ LAB VANE	W _p	W	W _L				
245.0	Grass						20	40	60	80	100	10	20	30	GR SA SI CL	
0.0	SAND to Silty SAND Fill (SM), with gravel, moist to 1.9 m Compact Brown		1	SS	14	▽									11 31 42 16	
			2	SS	15											
243.0			3	SS	7											
2.0	Silty CLAY (CL-ML), wet Stiff Brown		4	SS	15											
242.3																
2.7	Sandy SILT (ML), some clay, moist to wet Loose to compact Brown		5	SS	14											
			6	SS	13											
			7	SS	14											
			8	SS	12											
			9	SS	9											
237.8																
7.2	Sandy SILT Till (ML), some gravel, some clay, wet Compact Brown to grey		10	SS	19											
			11	SS	28											
234.0			12	SS	60											
11.0	Clayey SILT Till (ML), trace sand, trace gravel, wet Hard Grey															
233.7																
11.3	END OF BOREHOLE at approximately 11.3 m Water first encountered on spoon at a depth of approximately 1.9 m, below existing grade, elevation of about 243.1 m Groundwater level was measured at a depth of approximately 3.9 m below existing grade, elevation about 241.1 m in borehole on completion of drilling.															

ONTARIO MOT 1015345 KABINA JULY07 - FOUNDATIONS.GPJ ONTARIO MOT.GDT 08/01/14

RECORD OF BOREHOLE No KB-07-4

1 OF 1

METRIC

W.P. 5411-04-00 LOCATION Kabinakagmi River Sta. 23+590 o/s 1.5 m RL ORIGINATED BY NH
 DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY NH
 DATUM Geodetic DATE 07.07.18 - 07.07.18 CHECKED BY GTC

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										WATER CONTENT (%)	
246.6	Grass							20	40	60	80	100							
0.0	SAND and GRAVEL Fill, trace silt, moist at 0.6 m		1	SS	21		246												
246.0	Compact Brown																		
0.6	SILT Fill, with wood fragments, moist to wet		2	SS	14			245											
	Compact Grey to brown																		
	- some sand, some clay, trace gravel		3	SS	19														
244.2	END OF BOREHOLE at approximately 2.4 m		4	SS	100														
2.4	Auger refusal at a depth of approximately 2.4 m below existing grade (Elev. 244.2 m) on inferred Bedrock.																		
	Groundwater level was measured at a depth of approximately 2.2 m below existing grade, (Elev. 244.4 m) in borehole on completion of drilling.																		

RECORD OF BOREHOLE No KB-07-100

1 OF 1

METRIC

W.P. 5411-04-00 LOCATION Kabinakagmi River Sta. 23+562 o/s 8 m Lt. ORIGINATED BY DS
DIST New Liskeard HWY 11 BOREHOLE TYPE Solid Stem Auger COMPILED BY OL
DATUM Geodetic DATE 11.30.07 - 11.30.07 CHECKED BY GTC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
245.3 0.0	Snow Covered Ground Surface Auger to bedrock.						245	20	40	60	80	100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L		
							244										
							243										
241.9 3.5	END OF BOREHOLE at approximately 3.5 m. Auger refusal at a depth of approximately 3.5 m below existing grade (Elev. 241.9 m) on inferred bedrock. Borehole caved to a depth of approximately 3.3 m (Elev. 242.0 m) below existing grade on completion of drilling.						242										

ONTARIO MOT 1015345 KABINA DEC07 - FOUNDATIONS GPJ ONTARIO MOT.GDT 1/10/08

RECORD OF BOREHOLE No KB-07-101

1 OF 1

METRIC

W.P. 5411-04-00 LOCATION Kabinakagmi River Sta. 23+567 o/s 9 m Lt ORIGINATED BY DS
 DIST New Liskeard HWY 11 BOREHOLE TYPE Steel Casing, Solid Stem Auger COMPILED BY OL
 DATUM Geodetic DATE 07.11.30 - 07.11.30 CHECKED BY GTC

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
						20	40	60	80	100			
248.1 0.0	Snow Covered Ground Surface Auger to bedrock.												
241.6 6.5	Very poor to poor quality grey GNEISS - close to moderate joint spacing, dip 10 degrees and 90 degrees - thin bedding, dip 0-15 degrees - fresh to slightly weathered TCR = 86% SCR = 41% RQD = 51% TCR = 100% SCR = 83% RQD = 92%		1	NQ	-								
239.7 8.5	END OF BOREHOLE at approximately 8.4 m.		2	NQ	-								

ONTARIO MOT 1015345 KABINA DEC07 - FOUNDATIONS.GPJ ONTARIO MOT.GDT 08/01/08

RECORD OF BOREHOLE No KB-07-102

1 OF 1

METRIC

W.P. 5411-04-00 LOCATION Kabinakagmi River Sta. 23+567 o/s 10 m Rt ORIGINATED BY DS
 DIST New Liskeard HWY 11 BOREHOLE TYPE Solid Stem Auger COMPILED BY OL
 DATUM Geodetic DATE 11.30.07 - 11.30.07 CHECKED BY GTC

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
245.1 0.0	Snow Covered Ground Surface Auger to bedrock													
							244							
							243							
							242							
							241							
240.4 4.6	END OF BOREHOLE at approximately 4.6 m. Auger refusal at a depth of approximately 4.6 m (Elev. 240.4 m) below existing grade on inferred bedrock. Borehole caved to a depth of approximately 3.5 m (Elev. 241.6 m) below existing grade on completion of drilling.													

ONTARIO MOT 1015345 KABINA DEC07 - FOUNDATIONS GPJ ONTARIO MOT.GDT 1/10/08

RECORD OF BOREHOLE No KB-07-103

1 OF 1

METRIC

W.P. 5411-04-00 LOCATION Kabinakagmi River Sta. 23+562 o/s 6 m Rt ORIGINATED BY DS
 DIST New Liskeard HWY 11 BOREHOLE TYPE Steel Casing, Solid Stem Auger COMPILED BY OL
 DATUM Geodetic DATE 07.11.30 - 07.11.30 CHECKED BY GTC

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
244.3 0.0	Snow Covered Ground Surface Auger to bedrock.							20 40 60 80 100						
							244							
							243							
							242							
							241							
							240							
							239							
238.5 5.8	Very poor to poor quality grey GNEISS - close to moderate joint spacing, dip 10 degrees and 90 degrees - thin bedding, dip 0-15 degrees - fresh to slightly weathered TCR = 98% SCR = 64% RQD = 96%	///	1	NQ	-		238							
	TCR = 100% SCR = 61% RQD = 67%	///	2	NQ	-		237							
235.4 8.8	END OF BOREHOLE at approximately 8.8 m.	///					236							

ONTARIO MOT 1015345 KABINA DEC07 - FOUNDATIONS GPJ ONTARIO MOT.GDT 08/01/08

ONTARIO MOT 1015345 KABINA DEC07 - FOUNDATIONS.GPJ ONTARIO MOT.GDT 08/01/08

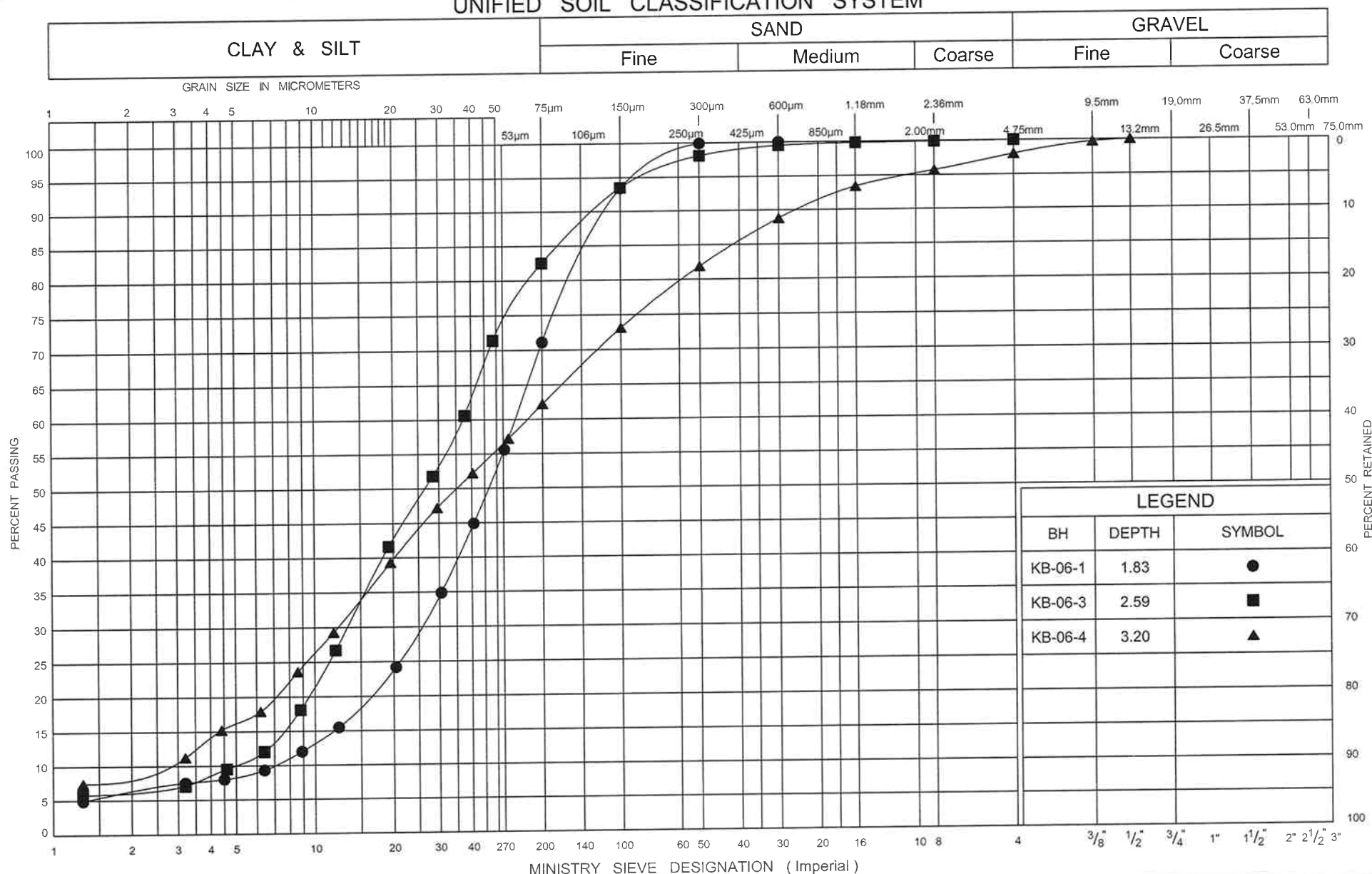
Appendix C

Geotechnical Laboratory Test Results



FIG No 1
W P 5411-04-00
Kabina River/Hwy 11, Hearst, Ont.

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

Sandy SILT (ML)

FIG No 2

W P 5411-04-00

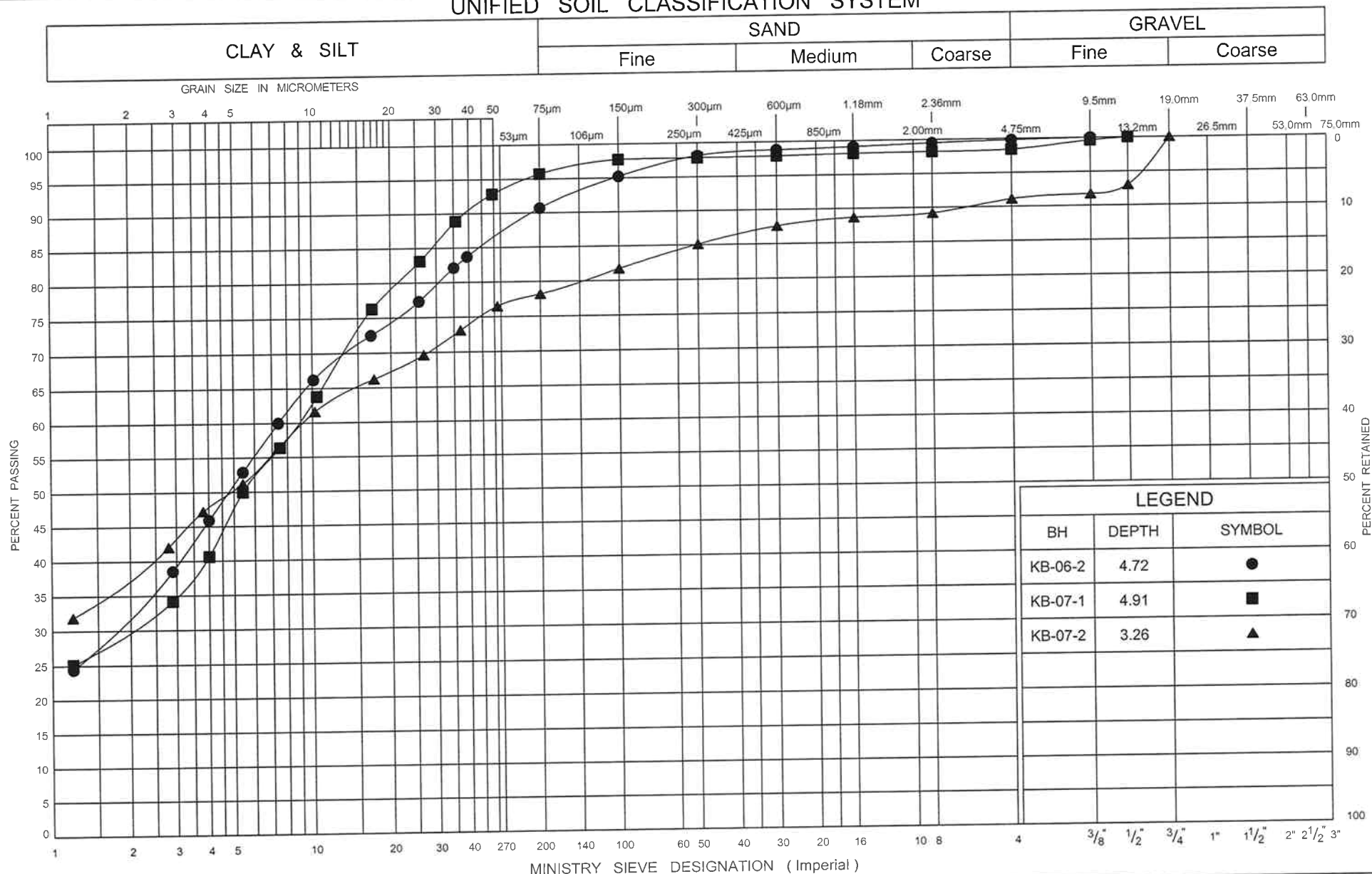
Kabina River/Hwy 11, Hearst, Ont.



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UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

Silty CLAY (CL)

FIG No 3

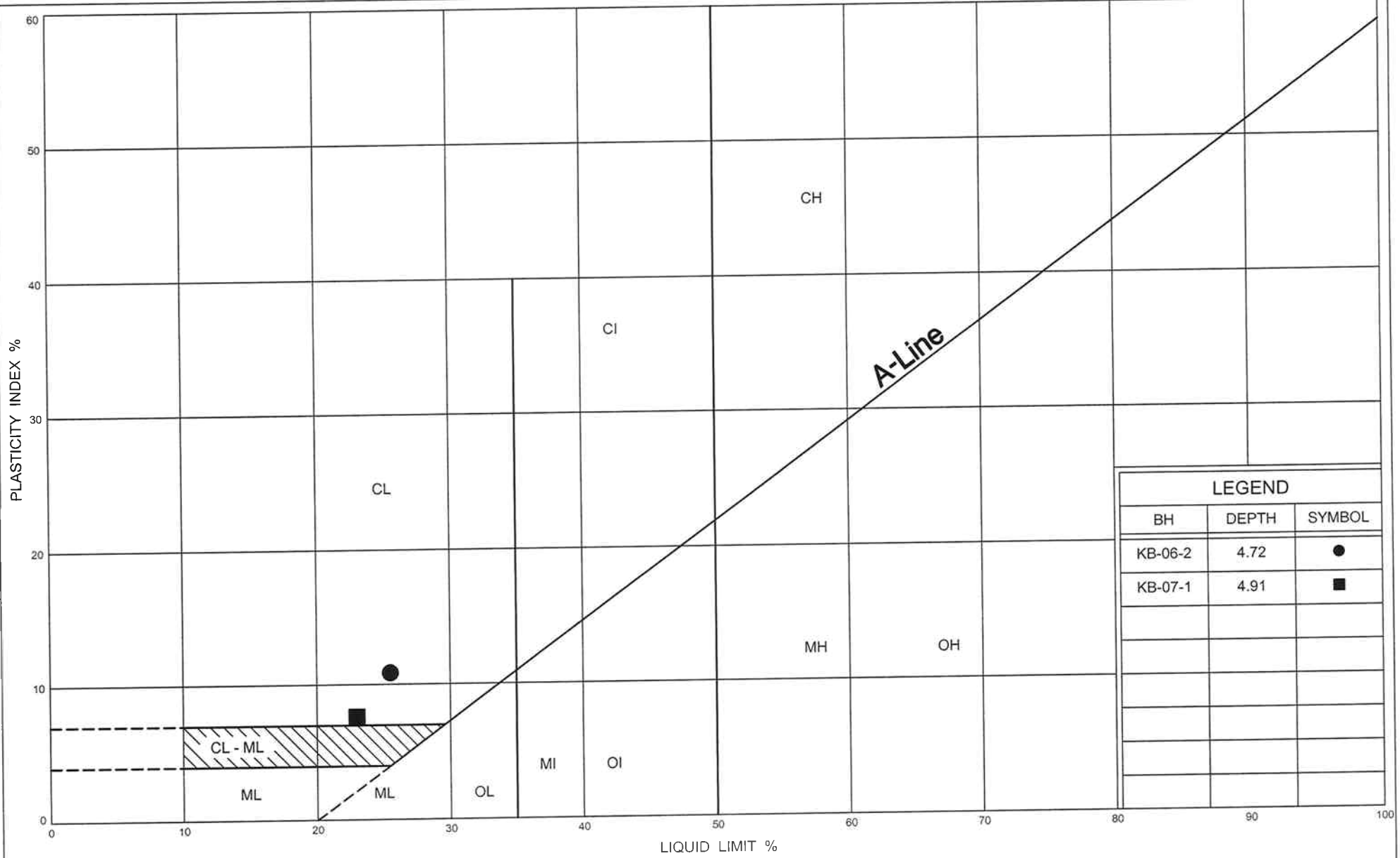
W P 5411-04-00

Kabina River/Hwy 11, Hearst, Ont.



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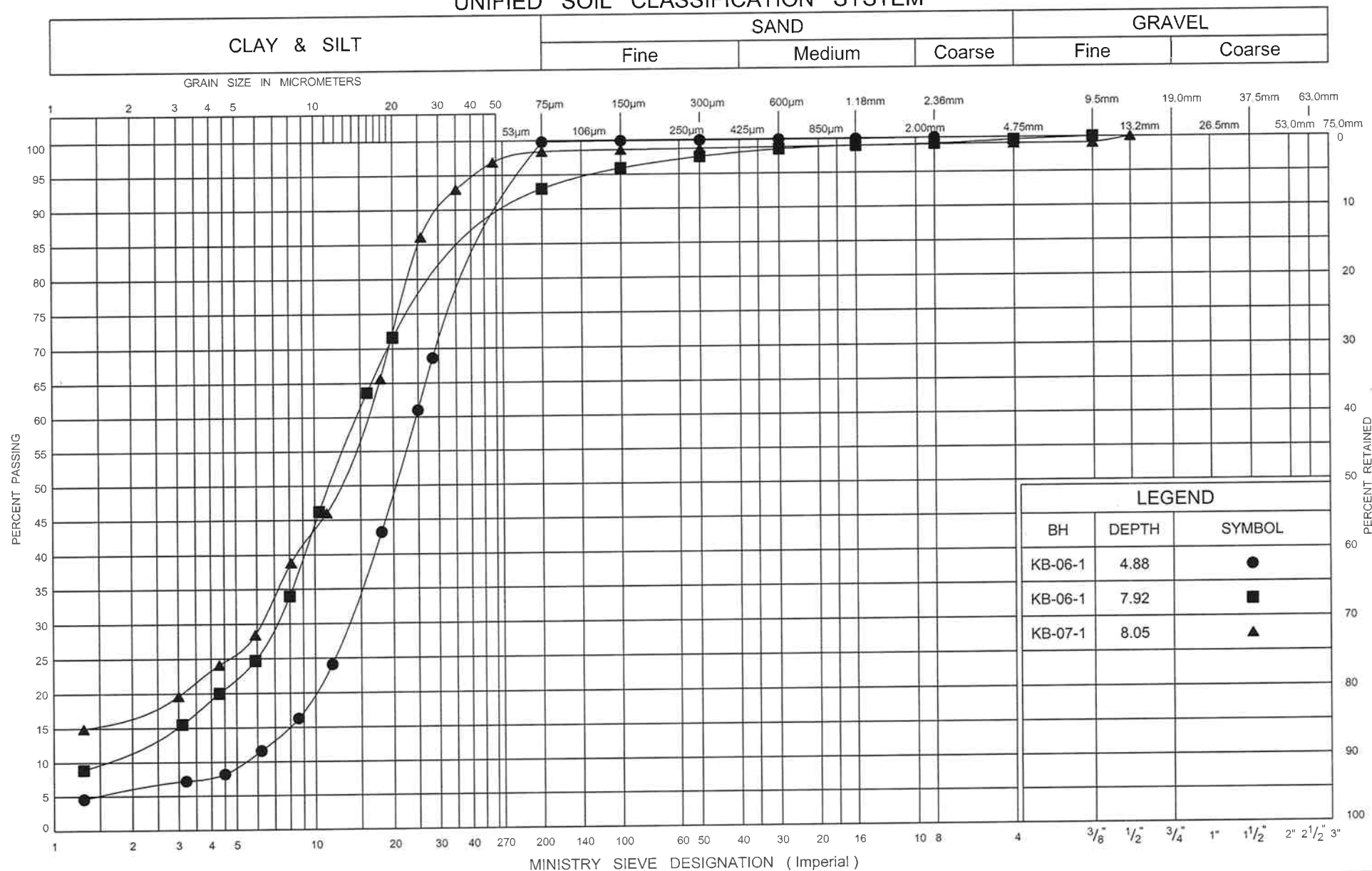
ONTARIO MOT PLASTICITY CHART 1015345 KABINA 06.GPJ ONTARIO MOT.GDT 08/01/08

PLASTICITY CHART Silty CLAY (CL)

FIG No 4
W P 5411-04-00
Kabina River/Hwy 11, Hearst, Ont.



UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

SILT (ML)

FIG No 5

W P 5411-04-00

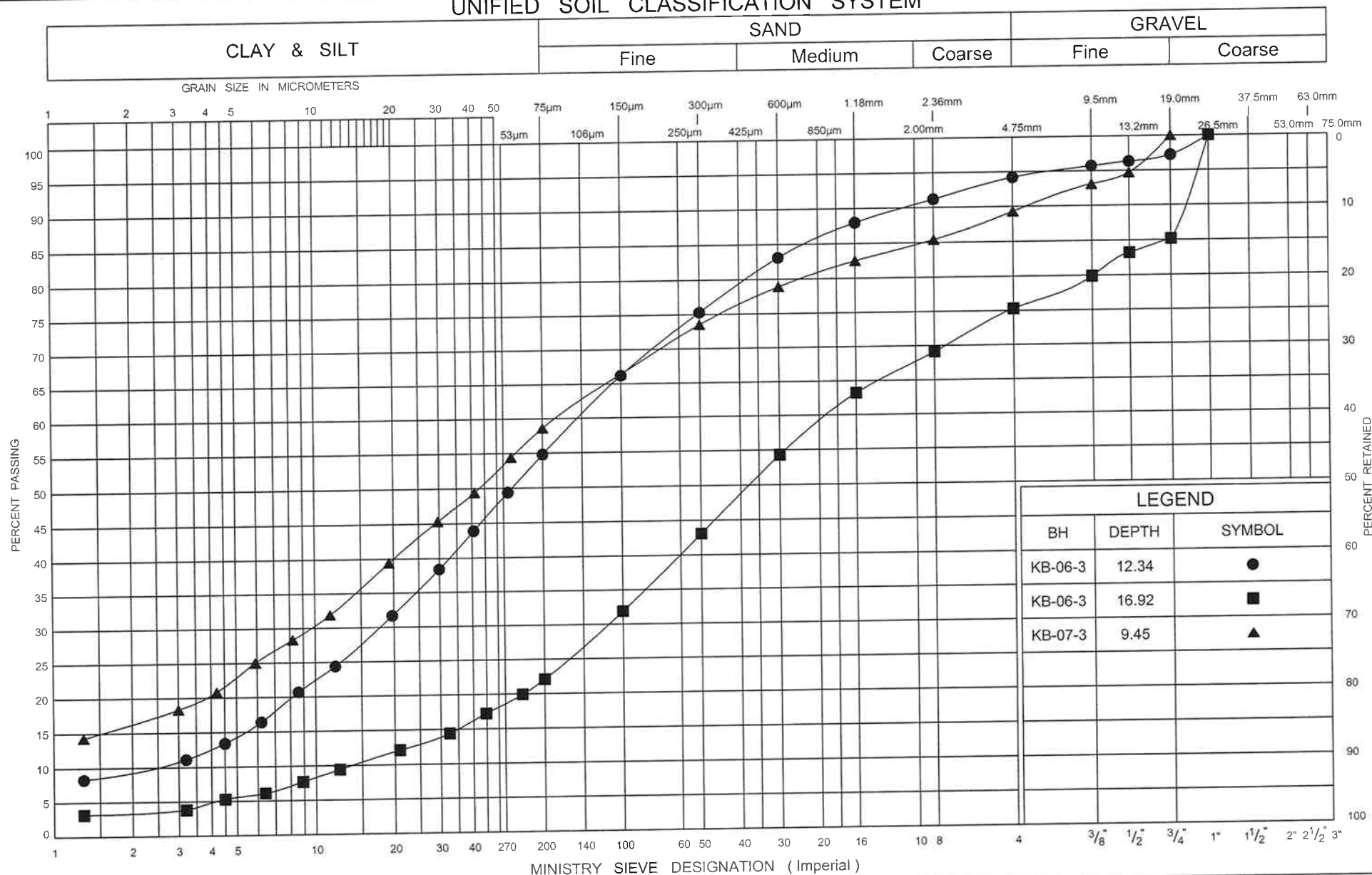
Kabina River/Hwy 11, Hearst, Ont.



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UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION
Sandy SILT Till (ML) to Silty SAND Till (SM)

FIG No 6

W P 5411-04-00

Kabina River/Hwy 11, Hearst, Ont.



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**Jacques Whitford
Limited**

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Markham, Ontario
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**Rock Core Compressive
Strength Test Report**

Figure 7

PROJECT NO: 1015345

Client: Lea Consulting Ltd. MTO

Project: Hwy 11, at the Kabinakagami River Bridge, Hearst, Ontario

Date Tested: 26 Mar, 2007

Core Number	KB-06-1 SA 4	KB-06-6 SA 1
Average Height (mm)	85.36	82.40
Average Diameter (mm)	48.22	47.52
H/D Ratio	1.770	1.734
Correction Factor	0.9816	0.9787
Compressive Strength (MPa)	105.8	69.1
Corrected Compressive Strength (MPa)	103.9	67.6

Tests carried out in accordance with CAN/CSA-A23.2 – 04, Unless otherwise noted

Appendix D

List of Standard Specifications and Drawings, and Provisions

Standard Specification and Drawings:

The following is a list of Standard Specifications and Drawings referenced in the Foundation Report for the replacement of the bridge on Highway 11 at the Kabinakagami River near Hearst, Ontario.

Standard Drawings:

OPSD3000.100

OPSD3000.150

OPSD3101.150

OPSD3102.100

Standard Specifications:

OPSS206

OPSS572

Special Provisions:

SP105S19