

FINAL REPORT

Preliminary Foundation
Investigation and Design Report
Opasatika River Bridge
Replacement
Site No. 39W-062
Highway 11
District – Cochrane

G.W.P. 319-85-00

LEA CONSULTING LTD.

PROJECT NO. 1015345
GEOCRES NO. 42G-27



PROJECT NO. 1015345

REPORT TO

**Lea Consulting Ltd.
625 Cochrane Drive
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Markham, Ontario
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FOR

**Preliminary Foundation Investigation and
Design Report**

ON

**Opasatika Bridge Replacement
Site 39W-062, Highway 11
District – Cochrane, Ontario
G.W.P. 319-85-00
Geocres. No. 42G-27**

June 5, 2008

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Record of Borehole Sheets
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PRELIMINARY FOUNDATION INVESTIGATION REPORT

Opasatika River Bridge Replacement Site No. 39W-062, Highway 11 Near Opasatika, Ontario G.W.P. 319-85-00 District – Cochrane

1.0 INTRODUCTION

Jacques Whitford Limited (Jacques Whitford) was retained by Lea Consulting Ltd., to complete a Preliminary Foundation Investigation and Design Report for the replacement of the Opasatika River Bridge located on Highway 11 near Opasatika, approximately 34 km west of Kapuskasing, Ontario, (GWP No. 319-85-00).

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 preliminary 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 Opasatika River near Opasatika, Ontario, which is approximately 34 km west of Kapuskasing, Ontario.

Highway 11 is generally oriented in an east west direction with one east bound lane and one west bound lane. Highway 11 at the Opasatika River is built on embankments to a rural highway section with wide gravel shoulders and is generally 3 m to 4 m 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 to the Opasatika River.

The existing bridge consists of a concrete deck, approximately 10 m wide, supported on four continuous cast concrete girders that are supported on three piers and two abutments. The distance between the abutments and the outer most piers is approximately 12 m to 13 m, while the distance between the piers is approximately 18 m to 19 m. The existing bridge was constructed in 1950 with no significant rehabilitation reported.

A construction drawing dated September 25, 1941, revised June 18, 1950, indicates that the existing bridge is supported on cruciform shaped caissons. The drawing indicates that the soil conditions at the site consist of "blue clay" overlying "hard pan", which is underlain by "Gravel & Layers of Sand & Boulders". However, the drawing does not indicate the bearing stratum of the caissons.

There is a parking and rest area located to the northwest of the bridge location. Overhead hydro and buried telephone cables are located along the north side of the highway right-of-way. There is an Environment Canada water monitoring station near the northeast portion of the existing bridge. An Ontario Northland Railway (ONR) line is located approximately 40 m to the south of the centreline of the existing highway and generally runs parallel to the existing highway.

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 Opatatika River is situated on the boundary between deep water glaciolacustrine and glaciomarine deposits of clay-silt and a glacial till deposit generally comprised of unsorted boulders, gravel, 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 paragneisses and migmatites.

4.0 INVESTIGATION PROCEDURES

4.1 Field Program

The fieldwork for this investigation was carried out between July 18 and 21, 2007, and August 9 to 14, 2007. A total of 6 boreholes were advanced for this investigation using a combination of truck, track, barge and skid mounted drill rigs equipped with either 250 mm (outside diameter) continuous flight hollow-stem augers, 100 mm (outside diameter) continuous flight solid-stem augers or wash-boring equipment, supplied and operated by Walker Drilling Inc. and Abraflex Inc. Three boreholes were drilled on each side of the Opatatika River to depths of approximately 17.2 m to 21.3 m below existing grade, elevations of approximately 199.4 m to 204.9 m.

Prior to commencing the field investigation, the borehole locations were cleared of underground utilities by the various utility companies. The borehole locations were established in the field by Jacques Whitford personnel.

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 by coring the bedrock using standard NQ size diamond tipped coring equipment. The Total Core Recovery (TCR), Solid Core Recovery (SCR) and Rock Quality Designation (RQD) were recorded for the bedrock cores recovered from the boreholes.

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 the Ontario Ministry of the Environment Regulation 903, using a cement/bentonite slurry.

All soil samples recovered from the boreholes were placed in moisture-proof bags and transported 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.

4.2 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 to the existing centre line of the highway looking up chainage. The borehole chainage and off-sets are provided on the Drawing Nos. 1 and 2 in **Appendix A** and on the Record of Borehole sheets in **Appendix B**.

The ground surface elevation at the borehole locations were inferred from a survey plan provided to Jacques Whitford by Lea Consulting Limited. It is understood that the survey plan was referenced to a Geodetic datum.

4.3 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 moisture content determination, grain size distribution and Atterberg Limits testing. The laboratory results are provided on the Record of Borehole sheets in **Appendix B**. The results of the grain size analyses and Atterberg Limits tests are shown on Figure Nos. 1 through 10 in **Appendix C**.

Unless requested in advance, all samples will be stored in our laboratory for a period of 12 months from the issue date of this report.

5.0 RESULTS OF THE INVESTIGATION

5.1 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 used on the Record of Borehole sheets is also provided in **Appendix B**.

A Borehole Location Plan and Soil Strata Plots 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.

5.2 Soil

5.2.1 Topsoil

Topsoil was encountered at the ground surface in Boreholes OPA-07-1 and OPA-07-5. The topsoil was approximately 125 mm thick in both boreholes.

5.2.2 Asphalt

Asphalt was encountered at the ground surface in Boreholes OPA-07-3 and OPA-07-6, and was approximately 175 mm and 65 mm thick, respectively.

5.2.3 Sand and Gravel Fill and Sand Fill

Sand and gravel fill and sand fill was encountered in Boreholes OPA-07-1, OPA-07-3, OPA-07-4, OPA-07-5 and OPA-07-6. The thickness of the sand and gravel fill and sand fill was approximately 0.4 m to 4.5 m and extended to depths of approximately 1.2 m to 4.6 m below existing grade, elevations of about 217.2 m to 219.6 m.

The sand contained trace to with silt, trace to some clay and trace gravel. Asphalt fragments were encountered in the sand fill sample obtained from Borehole OPA-07-4.

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

- Moisture Content:
 - 7% and 34%
- Grain Size Distribution
 - 1% to 6% gravel;
 - 44% to 78% sand;
 - 20% to 51% fines (silt and clay)

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

5.2.4 Silty Clay Fill

A localized layer of silty clay fill was encountered in Borehole OPA-07-5 at a depth of approximately 1.2 m below existing grade, an elevation of approximately 219.1 m. The thickness of silty clay fill was approximately 0.5 m and extended to depth of approximately 1.7 m below existing grade, elevation of approximately 218.6 m.

Based on visual examination the silty clay fill was found to be moist.

5.2.5 Peat

A layer of non-fibrous peat was encountered underlying the fill in Boreholes OPA-07-3 and OPA-07-4 at depths of approximately 3.7 m and 1.4 m below existing grade, elevations of about 219.6 m and 217.2 m. The peat was approximately 2.4 m and 3.7 m thick and extended to depths of approximately 5.1 m and 6.1 m below existing grade, elevations of approximately 213.5 m and 217.2 m.

The non fibrous peat contained wood fragments, some silty clay, trace sand and was saturated.

Based on the N-values obtained from the SPTs, the compactness of the non-fibrous peat was determined to be very loose to loose.

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

- Moisture content:
 - 31% to 259%
- Atterberg Limits:
 - Liquid Limit: 59% and 92%
 - Plastic Limit: 41% and 64%
 - Plasticity Index: 18% and 28%

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

The results of the Atterberg Limits Test are provided on Figure 2 in Appendix C.

5.2.6 Silty Clay (CL-ML to CI)

Silty clay was encountered in all boreholes. The silty clay was approximately 1.9 m to 6.1 m thick and extended to depths ranging from approximately 6.1 m to 11.8 m below existing grade, elevations in the range of 211.5 m to 216.2 m.

The silty clay contained varying amounts of sand (sandy to trace sand), trace to with organic matter, and was generally moist to wet. Strata of sand and silt were encountered within the silty clay layer. These are described separately below.

Based on the N-Values obtained from the SPT, the consistency of the silty clay was determined to be firm to stiff.

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:
 - 5% to 53%
- Grain Size Distribution:
 - 0% to 3% gravel;
 - 4% to 36% sand;
 - 34% to 65% silt; and,
 - 18% to 59% clay.

- Atterberg Limits:
 - Liquid Limit: 18% to 39%
 - Plastic Limit: 13% to 22%
 - Plasticity Index: 5% to 18%

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 Nos. 3, 5 and 7 in **Appendix C**. The results of the Atterberg Limits tests are provided on Figure Nos. 4, 6 and 8 in **Appendix C**.

5.2.7 Silt (ML)

A layer of silt was encountered in Borehole OPA-07-2 within the silty clay at a depth of approximately 2.3 m below existing grade, an elevation of about 216.2 m. The silt was approximately 1.4 m thick and extended to a depth of approximately 3.7 m below existing grade, an elevation of about 214.9 m.

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

Based on the N-Values obtained from the SPTs, the silt was determined to be compact.

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

- Moisture Content:
 - 19%
- Grain Size Distribution:
 - 2% gravel;
 - 1% sand;
 - 84% silt; and,
 - 12% clay.

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 test is provided on Figure 9 in **Appendix C**.

One sample of the silt was submitted for Atterberg Limits testing. The test results indicated that the silt was non-plastic.

5.2.8 Sand (SP) and Silty Sand (SM)

A layer of sand and silty sand was encountered within the silty clay in Boreholes OPA-07-2, OPA-07-4 and OPA-07-6 at depths of approximately 3.7 m to 6.1 m below existing grade, elevations of about 213.5 m to 216.9 m. The thickness of the sand and silty sand ranged from approximately 0.9 m to 3.0 m and the unit extended to depths of approximately 5.2 m to 9.1 m, elevations of about 212.6 m to 213.9 m.

Based on the N-Values obtained from the SPTs, the silty sand was determined to be variable ranging from very loose to very dense.

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

- Moisture Content:
 - 22% to 33%

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

5.2.9 Silty Sand Till (SM)

Silty sand till was encountered in all boreholes at depths of approximately 6.1 m to 11.8 m below existing grade, elevations of approximately 211.5 m to 214.6 m. The thickness of the silty sand till ranged from approximately 4.4 m to 10.7 m and extended to the depths in the range of approximately 13.9 m to 19.8 m, elevations of approximately 202.8 m to 208.0 m.

The silty sand till strata contained trace to some gravel and clay, and was wet. Rock fragments, cobbles and boulders were also encountered.

Based on N-values obtained from the SPTs, the compactness of the silty sand till was determined to be compact to very dense, but was generally dense.

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

- Moisture Content:
 - 6% to 12%
- Grain Size Distribution:
 - 2% to 16% gravel;
 - 28% to 60% sand;
 - 21% to 51% silt; and,
 - 3% to 20% clay.

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 test are provided on Figure 10 in **Appendix C**.

5.3 Bedrock

Bedrock was cored in all boreholes except Borehole OPA-07-3. The bedrock was encountered at depths of approximately 13.9 m to 16.8 m below existing grade, elevations of approximately 208.0 m to 202.8 m.

The bedrock generally consisted of metamorphic gneiss.

Core samples of the bedrock were obtained from all boreholes, except OPA-07-3. The observations of the rock cores are summarized as follows:

- Total Core Recovery (TCR): 62% to 100%, average of approximately 95%

- Solid Core Recover (SCR): 32% to 95%, average of approximately 72%
- Rock Quality Designation (RQD): 26% to 89%, average of approximately 59%

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

5.4 Groundwater

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

Borehole Number	Groundwater First Encountered	
	Depth Below Existing Grade (m)	Elevation (m)
OPA-07-1	2.1	218.6
OPA-07-2	River Surface	218.5
OPA-07-3	4.8	218.5
OPA-07-4	River Surface	218.6
OPA-07-5	1.8	218.5
OPA-07-6	3.0	220.1

It should be noted that groundwater levels and the water level of the river are subject to seasonal fluctuations and responses to precipitation events.

6.0 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 ground water 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

PRELIMINARY FOUNDATION DESIGN REPORT

Opasatika River Bridge Replacement Site No. 39W-062, Highway 11 Near Opasatika, Ontario G.W.P. 319-85-00 District – Cochrane

7.0 DISCUSSION

7.1 General

The site is located on Highway 11 at the Opasatika River near Opasatika Ontario, which is approximately 34 km west of Kapuskasing, Ontario.

The existing bridge consists of a concrete deck, approximately 10 m wide, supported on four continuous cast concrete girders that are supported on three piers and two abutments. The distance between the abutments and the outer most piers is approximately 12 m to 13 m, while the distance between the piers is approximately 18 m to 19 m. The existing bridge was constructed in 1950 with no significant rehabilitation reported.

A construction drawing dated September 25, 1941, revised June 18, 1950, indicates that the existing bridge is supported on cruciform shaped caissons. The drawing indicates that the soil conditions at the site consist of “blue clay” overlying “hard pan”, which is underlain by “Gravel & Layers of Sand & Boulders”. However, the drawing does not indicate the bearing stratum or elevation of the caissons.

There is a parking and rest area located to the northwest of the bridge location. Overhead hydro and buried telephone cables are located along the north side of the highway right-of-way. There is an Environment Canada water monitoring station near the northeast portion of the existing bridge. An Ontario Northland Railway (ONR) line is located approximately 40 m to the south of the centreline of the existing highway and generally runs parallel to the existing highway.

7.2 Proposed Development

The Ministry of Transportation (MTO) is planning to replace the existing bridge at the Opasatika River, with a new bridge.

Based on the a technical memorandum by Lea Consulting titled “Technical Memorandum: Development and Evaluation of Alternative Roadway and Structure Alignments”, dated February 2007, it is understood that the new bridge will likely be a three span (22.5 m – 45 m – 22.5 m) bridge constructed with a reinforced concrete deck, and either CPCI pre-cast concrete I-girders or steel I-

girders. The new bridge will likely be constructed on the existing alignment, once a temporary detour bridge has been constructed.

The temporary detour bridge will likely be constructed to the south of the existing bridge location. Based on the technical memorandum referenced above, the temporary detour bridge will likely consist of a two span (39.6 m – 39.6 m) bridge, supported on abutments and a central trestle, which will likely be constructed by extending the piles above the water line and incorporating them as the trestle support legs. The detour bridge will likely have three truss panels on either side of the driving lanes, and steel deck units that will be covered with an aggregate epoxy driving surface.

Along with the temporary detour bridge, a temporary detour alignment will also be constructed. It is understood that the temporary detour alignment will be to the south of the existing alignment. The detour alignment will be approximately 750 m long from Sta. 12+350 to Sta. 13+100, will be approximately 12 m wide and will likely be constructed on embankments. Preliminary sketches indicate that the detour embankment will likely be constructed as a widening to the existing embankments.

7.3 Subsurface Conditions

The subsurface conditions encountered are shown on the Record of Borehole sheets in **Appendix B**. The following is a brief summary of the soil and groundwater conditions encountered.

The conditions encountered on both the east and west side of the existing bridge at the Opakatika River generally consisted of fill overlying deposits of firm to stiff clay over a dense to very dense silty sand till. Localized deposits of peat were encountered in 2 boreholes generally underlying the fill material. Localized sand and silt strata were observed within the silty clay layer. Bedrock was encountered underlying the silty sand till at depths of approximately 13.9 m to 16.8 m below existing grade, elevations of about 202.8 m to 208.0 m.

Wet conditions were first encountered during drilling on the spoon in the boreholes advanced on the land at depths in the range of approximately 1.8 m to 4.8 m below existing grade, elevations in the range of approximately 218.5 m to 220.1 m. The elevation of the water within the river was approximately 218.5 m at the time of the investigation.

7.4 Foundation Options

The following table provides a comparison of the foundation options considered for both the replacement bridge and the detour bridge.

Foundation Option	Advantages	Disadvantages	Relative Cost	Risks/ Consequences
Spread Footings supported on native soils	Lower cost than using deep foundations	Excavations may be difficult in water bearing silts and sand. Low geotechnical resistance as a result of the underlying soils. Settlement of the underlying soils will occur due to soft and organic soil deposits. Will require the use of settlement mitigation measures. Not compatible with trestle design for detour pier. Not compatible with integral abutment design.	Low	Excavation may be difficult in water bearing silts and sand. Likely not be feasible due to low geotechnical resistance, even if settlement mitigation measures are taken.
Spread footings on granular pads	Higher geotechnical resistance. Lower anticipated settlement.	Potential for settlement of the underlying soft, loose and organic soils. Difficult to excavate soils at the pier locations particularly for the detour bridge. Dewatering Challenges. Not compatible with trestle design for detour pier. Not compatible with integral abutment design.	Medium	Presence of water bearing soils. Settlements will occur even with mitigation measures.
Piles Driven to be end bearing on silty sand till	Minimal settlement Piles would be in the order of about 11 m long and would likely not require splicing. Compatible with trestle design for detour pier, as the piles could be extended above the water line and incorporated as the legs of the trestle structure. Compatible with integral abutment design.	The geotechnical resistances would be moderate. Down drag loads may be induced on the piles depending on the approach fill requirements.	High	Potential tip damage during driving. Potential bending, doglegging, or breaking of the piles, due to the presence of boulders.
Piles Driven to be end bearing on Bedrock	High geotechnical resistance. Minimal settlement. Compatible with trestle design for detour pier. Compatible with integral abutment design.	May be difficult to drive piles through the dense glacial till. Down drag loads may be induced on the piles depending on the approach fill requirements.	High	Possible tip damage during driving. Potential bending, doglegging, or breaking of the piles due to the presence of boulders.

Foundation Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
Friction piles in dense silty sand till	Some cost savings, reduction in number of splices. Compatible with trestle design for detour pier. Compatible with integral abutment design.	Piles would be relatively short and would have a low geotechnical resistance. Piles likely spaced closer together, which would result in reduction of capacity for pile groups. Down drag loads may be induced on the piles depending on the approach fill requirements.	Medium to High	Potential bending, doglegging, or breaking of the piles due to the presence of boulders.
Caissons founded on the underlying bedrock or hard/dense strata	High geotechnical resistance.	Water bearing silts and sands will be difficult to drill through. Steel liners would be required to keep the caisson open. May require mud drilling techniques to minimize base instability towards the river. Caisson length likely in the order of 9 to 12 m long, slightly longer than typical caissons. Not compatible with trestle design for detour pier. Not compatible with integral abutment design. High down drag loads may be induced on the caissons depending on the approach fill requirements.	High	Steel Liners would be required to keep the caisson open. Will likely encounter problems due to the presence of boulders.

Given the soil and water conditions at this site, it is concluded that H-piles will provide the most economical foundation system for the preferred structural concepts for both the replacement and detour bridges. For preliminary purposes it is recommended that the design proceed with steel H-piles driven into the silty sand till and supported by both friction and end bearing.

8.0 PRELIMINARY RECOMMENDATIONS

8.1 Axial Foundation Design

8.1.1 Piles - Geotechnical Resistance

Given the conditions encountered during this investigation, the replacement and detour bridges could be founded on steel HP310X110 piles driven into the dense silty sand till. For preliminary purposes pile tip elevations should be assumed to be at approximately 207 m for all foundation elements.

HP310 x 110 Steel H-Piles for the abutments and pier driven into the dense silty sand till may be designed using a factored geotechnical resistance of ULS of 1800 kN. The ULS value includes a resistance factor of 0.4.

A geotechnical reaction at SLS of 1600 kN is recommended for piles driven into the dense silty sand till. The toe of the pile is expected to settle less than 20 mm at the SLS value. The structural engineer should consider the structural compressibility of the pile as part of the design.

8.1.2 Piles - Down Drag Forces

The placement of the fill material for the detour embankments will induce settlement of the soils underlying the detour embankments.

Down drag forces, induced as a result of the settlement of the organic soils underlying the approach fills, must be considered. Preliminary calculations indicate that the embankment widening for the detour may induce settlement of the organic soils will be variable and could be in the order of approximately 10 mm at the existing rounding.

Based on these settlement estimates, down drag forces on the foundation of the existing abutments are not anticipated.

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

A grade raise is proposed at the replacement bridge. This will induce approximately 30 mm of settlement and create down drag forces in the pile foundation of the new abutment. The unfactored down drag forces on an HP310x110 steel piles is estimated to be 60 kN.

8.1.3 Piles - Tensile Resistance

There is the potential for ice on the river to impose loads on the trestle of the temporary bridge, which may require that some of the piles will be required to resist tensile loads.

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 450 kN and 120 kN, may be considered for HP 310x110 H-piles in tension for piles at abutment and pier locations, respectively. The ULS tensile resistance was calculated using a geotechnical resistance factor of 0.3.

The above values do not include the weight of the piles.

8.2 Lateral Foundation Design

8.2.1 Lateral Resistance for Vertical Piles

Passive lateral resistance for vertical piles should be calculated as per C6.8.7.2 (Static Analysis) of the CHBDC using the following unfactored geotechnical soil parameters:

Parameter	Native Sand and Silts	Silty Clay	Silty Sand Till
Bulk Unit Weight (kN/m ³)	18	20	20
Effective friction angle	28°	-	32°
Coefficient of passive earth pressure	2.8	-	3.3
Design Undrained Shear Strength (kPa)	-	100	-

An assessed horizontal passive resistance at ULS of 200 kN is recommended for the HP310 x 110 piles at this site, based on Table C6.4 of the CHBDC.

8.2.2 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 = 100 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 4,400 kN/m³ and 11,000 kN/m³ for compact and dense sandy soils, respectively (Table 20.3, p. 315, of the Canadian Foundation Engineering Manual, 3rd Edition, 1992)

z = depth below grade

d = pile diameter

An assessed horizontal reaction at SLS of 110 kN is recommended for the HP310 x 110 piles at this site based on Table C6.4 of the CHBDC.

8.2.3 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 2.5 d (where d =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, Isenhowe 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.

8.3 Piling Notes

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

Welded splices for steel H-piles should be in accordance with OPSD 3000.150.

Where integral abutments are considered, 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 space between the pile and the liner should be filled with loose sand, such as OPSS concrete sand.

Piles should be supplied and installed in accordance with SP903S01. The piles should be driven in accordance with standard SS103-11 using an ultimate geotechnical resistance equal to twice the design load at ULS.

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

“Piles to be driven in accordance with Standard SS103-11 using an ultimate geotechnical resistance of 3600 kN per pile, but must be driven below elevation 207.”

8.4 Earth Pressure Design

To prevent hydrostatic pressure build-up, backfill against the abutments should consist of free draining granular materials and a subdrain should be installed as per OPSD 3102.100. OPSS Granular A or OPSS Granular B, Type II (both with less than 5% passing the 75 μm sieve) 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 6.6.

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)	22.0	22.0	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

8.5 Soil Profile Type and Seismic Forces

The zonal acceleration ratio for Hearst and Kapuskasing is 0.00 as per CHBDC Table A3.1.1.

Soil Profile Type I as defined in Section 4.4.6 of the CHBDC would be appropriate for this site.

8.6 Embankment Design and Construction

The existing embankments are constructed at 3H:1V to 4H:1V side slopes and exhibit no visible signs of instability.

Replacement Bridge

The placement of the fill material to accommodate the widening for the detour alignment will induce settlement of the organic soils underlying the existing embankments. Preliminary calculations indicate that the settlement at the centreline of the existing road would be in the order of about 5 mm or less, while the settlement of the existing rounding would be in the order of approximately 10 mm.

It is understood that the vertical profile will be raised by approximately 0.5 m for the replacement bridge. This will induce approximately 30 mm of settlement of the underlying soils. The majority of that settlement will occur during construction. Nevertheless, it is recommended that the fill material for the replacement bridge be placed early in the construction schedule, to provide time for the settlement to occur before construction of the bridge.

As part of the construction, the existing backfill behind the abutments will be excavated and later replaced. Self settlement of the back fill of as much as 15 mm will occur. This settlement will occur during the construction period.

Detour Bridge

The embankments for the detour bridge will be constructed to the south of the existing embankments, and will be approximately 4 m high immediately behind the abutments. Preliminary drawings and sketches indicate that the embankments for the planned detour will likely be constructed as a widening of the existing embankments. The detour embankment will be approximately 12 m wide and the side slope will be approximately 2H:1V.

Prior to placing the fill, all topsoil, loose, wet, organic and other deleterious material, including the peat, 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.

Embankment fill should consist of OPSS Select Subgrade Material or clean granular fill such as OPSS Granular B. The use of rock fill could also be considered, however, it is anticipated that rock fill will not likely be available at this site. The embankment side slopes for the detour constructed of soil, should be no steeper than 2H:1V.

Preliminary calculations indicate that the placement of the fill material for the detour embankment will induce settlements of the underlying soils as outlined in the following table:

Location	Anticipated Settlement (mm)
Existing Rounding	10
Existing Toe of Slope	30

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

9.0 CONSTRUCTION RECOMMENDATIONS

9.1 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 surficial native soils should be considered Type 3 soils. Temporary excavations should be made with side slopes no steeper than 1H:1V from the base of the excavation.

Where encountered, the existing peat and organic soils should be considered Type 4 soils. Temporary excavations should be made with side slopes no steeper than 3H:1V from the base of the excavation.

Flatter side slopes will be required for open cut excavations in loose wet sand and silt 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 provided herein, when excavations are in close proximity to existing infrastructure and embankments.

9.2 Staging

Through discussions with representatives of Lea Consulting, the staging of the project had not been determined at the time this report was prepared. For preliminary consideration, it is anticipated that Stage 1 will likely consist of the construction of the detour embankments and detour bridge to the south of the existing bridge; Stage 2 will consist of the demolition of the existing bridge and construction of the replacement bridge; and, Stage 3 will consist of the removal of the temporary detour and detour bridge.

9.3 Groundwater Control

The water level in the river at the time of the investigation in early August, 2007, was determined to be at an elevation of approximately 218.5 m.

Wet conditions and groundwater were encountered in the boreholes advanced on land at depths in the range of approximately 1.8 m to 4.8 m below existing grade, elevations in the range of approximately 218.5 m to 220.1 m.

Excavation above elevations of about 218.5 m to 220.1 m may encounter perched water within the soil and fill materials. Given the soil conditions, seepage above these elevations is anticipated to be slow and therefore should be readily handled by conventional sumps and pumping techniques.

Excavations below elevations of about 218.5 m to 220.1 m will encounter groundwater and excavations below a depth of approximately 218.5 m will be below the water level in the river, as observed in August 2007. Excavations below the river level may be difficult given the presence of wet, saturated silts and sands. Therefore, some form of dewatering, in addition to conventional sumps and pumping techniques will be required.

9.4 Shoring and Cofferdams

It is anticipated that shoring will be required for the construction of the abutments, especially during the removal of the peat for the detour abutments.

Given that the proposed piers will likely be in the river, it is anticipated that cofferdams will be required for the construction of the foundation elements below the water line. It is recommended that both the shoring and cofferdams consist of sheet piles.

Shoring design, including the use of sheet piles, is the responsibility of the contractor and may be designed using the parameters provided in Section 8.4 Earth Pressure Design. Variation in the water level of the river should be taken into account when designing the shoring and cofferdams.

Shoring should meet the requirements of Performance Level 2 as per SP105S19.

Hydrostatic forces associated with the water level in the river should be taken into account when designing the shoring.

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.

Encroachment of excavation into the forward and side slopes of the existing bridge will require special attention. Excavations will not be permitted within the influence zone of the existing abutments. The influence zone includes all materials below an imaginary line drawn at an angle of 1H:1V downward and away from the vertical edges of the abutments.

9.5 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 on the slopes as soon as possible after completion of the embankments in order to control surface erosion. Erosion control should be in accordance with OPSS 572.

For preliminary design purposes, the river slopes within 3 m of the structures surface should be surfaced with rip-rap at least 300 mm thick placed on a Class II non-woven filter fabric. At other locations, as previously noted, vegetation should be established as soon as possible after completion of the embankment fills in order to control surface erosion.

Where foundation elements are in close proximity to flowing water they should be protected from scour. Flood water velocities were not available at the time this report was prepared. For preliminary design purposes, and based on experience with similar projects, rock protection with particles ranging from 100 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. These recommendations should be reviewed with the flood water velocities.

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.

9.6 Frost Protection

The site is located in an area with a mean freezing index of about 2000 Degree-days (°Days), (Figure 13.6, Canadian Foundation Engineering Manual, 2006). Based on Figure 3.5 of the MTO Pavement Design and Rehabilitation Manual, the frost penetration depth for this area is 2.6 m.

10.0 ADDITIONAL INVESTIGATION

The scope of work for the preliminary foundation investigation and design report was limited to advancing boreholes at six locations. Boreholes were not advanced at all foundation locations.

Therefore, it is recommended that an additional foundation investigation be carried out once the abutment and pier locations have been finalized for both the replacement and temporary detour bridges.

Additional laboratory testing should also be carried out. Specifically, consolidation testing of the organic soils encountered in the area of the planned detour bridge and embankments should be carried out. The results will be used to better understand both the magnitude and duration of the settlement caused by the placement of the detour embankments.

The additional work should include boreholes at the pier locations, within the river, to determine the depth to a competent bearing stratum at the pier locations.

The potential for scour around the bridge foundation elements should be evaluated during detailed design. The preliminary recommendations provided herein should be re-assessed based on measured and anticipated water velocities.

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 of 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

GC/FG/ct

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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.

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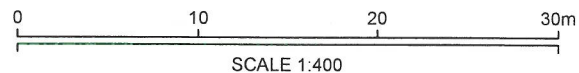
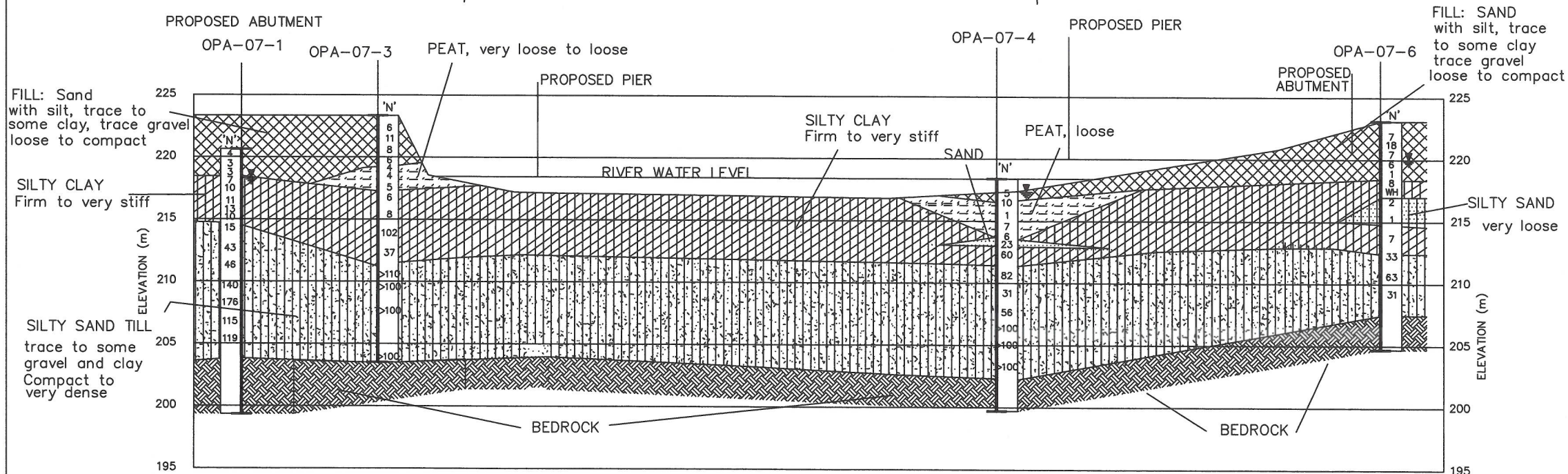
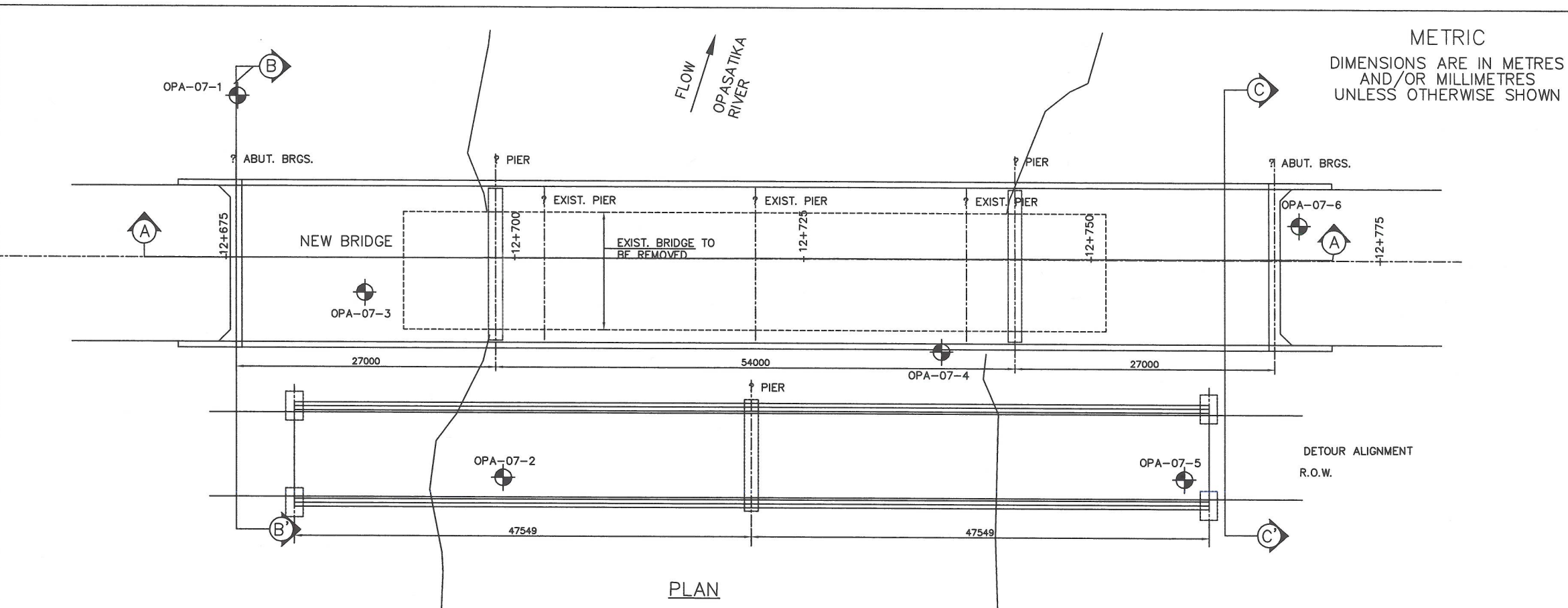
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PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Jacques Whitford, sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this 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 -
WP No 319-85-00

OPASATIKA RIVER BRIDGE
REPLACEMENT-BOREHOLE
LOCATIONS AND SOIL STRATA
(SECTION A-A')

SHEET
-

KEY PLAN N.T.S.

LEGEND

- BOREHOLE
- GROUNDWATER LEVEL

BOREHOLE NO.	STATION	OFFSET	ELEVATION
OPA-07-1	12+676	14m Lt.	220.7
OPA-07-2	12+695	19m Rt.	218.5
OPA-07-3	12+687	3m Rt.	223.3
OPA-07-4	12+737	8m Rt.	218.6
OPA-07-5	12+758	19m Rt.	220.3
OPA-07-6	12+768	3m Lt.	223.1

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. 42G-27

REVISIONS	DATE	BY	DESCRIPTION

DESIGN B.D. CHK R.T.K. CODE CHBDC 00 LOAD ONT CL-625 DATE DECEMBER 2008
DRAWN A.A. CHK B.D. SITE 39W-062 STRUCT - SCHEME - DWG 1

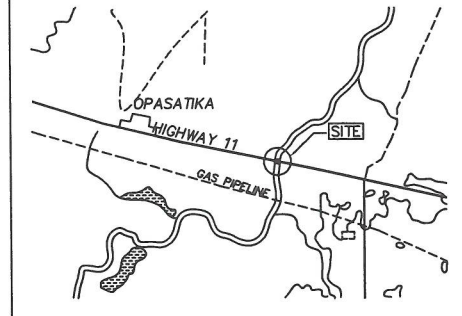
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No -
WP No 319-85-00



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

SHEET
-



KEY PLAN N.T.S.

LEGEND

- BOREHOLE
- GROUNDWATER LEVEL

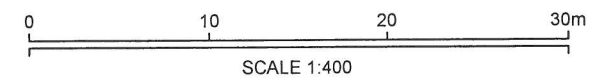
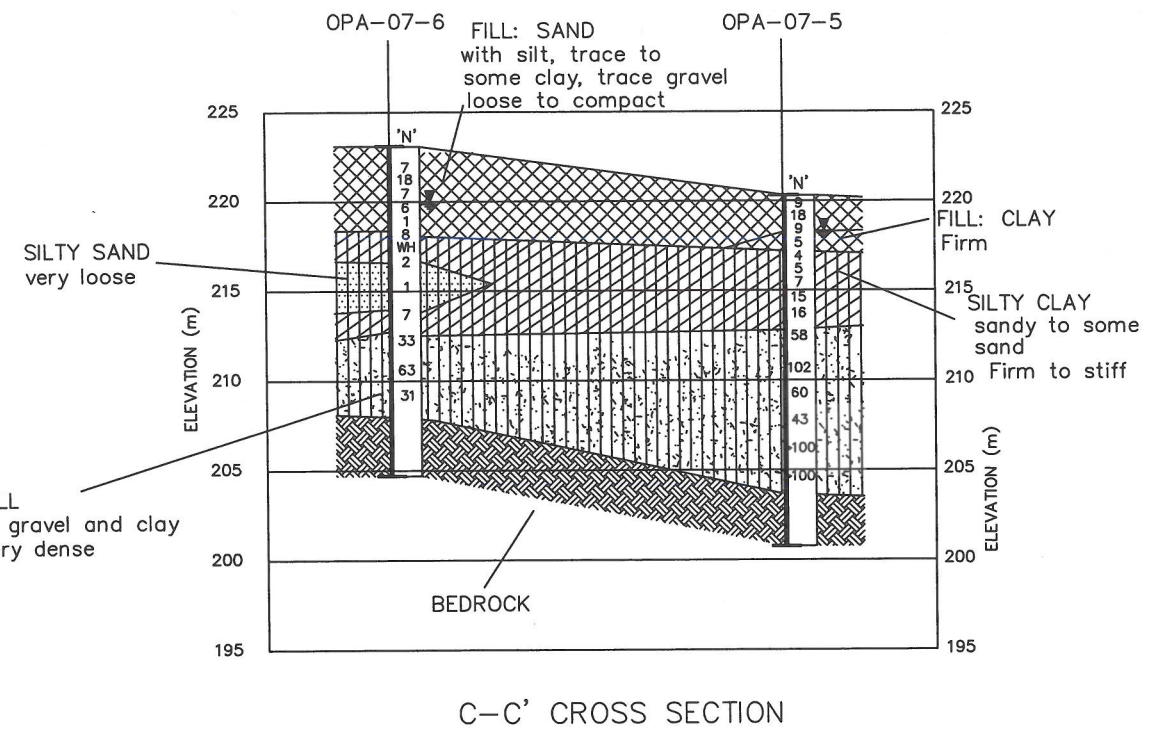
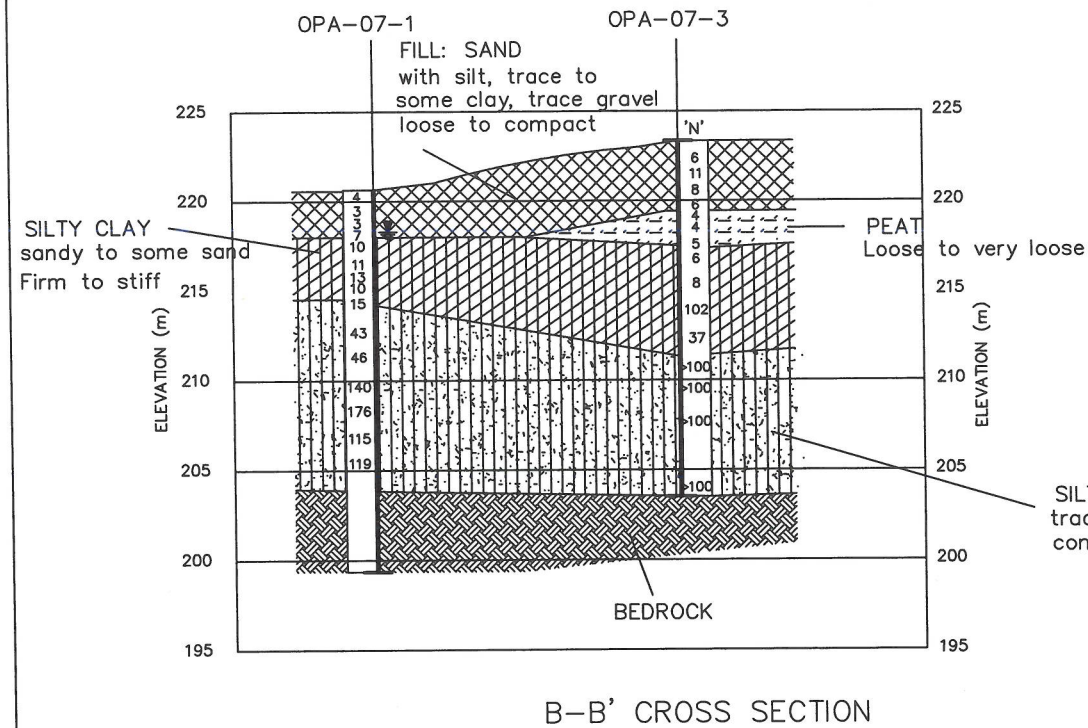
BOREHOLE NO.	STATION	OFFSET	ELEVATION
OPA-07-1	12+676	14m Lt.	220.7
OPA-07-2	12+695	19m Rt.	218.5
OPA-07-3	12+687	3m Rt.	223.3
OPA-07-4	12+737	8m Rt.	218.6
OPA-07-5	12+758	19m Rt.	220.3
OPA-07-6	12+768	3m Lt.	223.1

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.
- Offsets measured from existing centreline

GEOCRES NO. 42G-27

REVISIONS	DATE	BY	DESCRIPTION
DESIGN B.D.	CHK R.T.K.	CODE OHBC 00	LOAD ONT CL-825 DATE DECEMBER 2008
DRAWN A.A.	CHK B.D.	SITE 39W-082	STRUCT - SCHEME - DWG 2



Appendix B

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

RECORD OF BOREHOLE No OPA-07-1

1 OF 2

METRIC

W.P. 319-85-00 LOCATION Opasatika River Sta. 12+676 o/s 14 m Lt ORIGINATED BY NH
 DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY NH
 DATUM Geodetic DATE 7.21.07 - 7.21.07 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							WATER CONTENT (%)
220.7	Grass							20 40 60 80 100	20 40 60 80 100	10 20 30					
220.0	125 mm TOPSOIL: silty sand trace gravel with high organic content FILL: brown sand, some silt and clay, trace gravel, damp to moist		1	SS	4	▽	220	○ UNCONFINED	× FIELD VANE					1 78 (22)	
0.1			2	SS	3				● QUICK TRIAXIAL	× LAB VANE					6 74 (20)
			3	SS	3										
218.7	Silty CLAY (CL) trace to some sand, with organic matter, wet Firm to stiff Grey		4	SS	7			218							
2.0			5	SS	10			217							0 16 54 30
			6	SS	11			216							
			7	SS	13			215							
			8	SS	10			214							
214.6	Silty SAND (SM) TILL, trace to some clay, trace gravel, occasional cobbles and boulders, wet Compact to very dense Grey		9	SS	15			213							
6.1			10	SS	43			212							
			11	SS	46			211							5 39 45 10
			12	SS	140			210							
			13	SS	176			209							
			14	SS	115			208							
							207								
							206								

Continued Next Page

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

RECORD OF BOREHOLE No OPA-07-1

2 OF 2

METRIC

W.P. 319-85-00 LOCATION Opasatika River Sta. 12+676 o/s 14 m Lt ORIGINATED BY NH
DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY NH
DATUM Geodetic DATE 7.21.07 - 7.21.07 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									
								20	40	60	80	100					
								20	40	60	80	100					
	</																

METRIC

Continued Next Page

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

2 OF 2

METRIC

ELEV DEPTH	SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT	LQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
								20 40 60 80 100 					
								○ UNCONFINED ✕ FIELD VANE ● QUICK TRIAXIAL ✕ LAB VANE					
								20 40 60 80 100 					

[illegible]

RECORD OF BOREHOLE No OPA-07-3

1 OF 2

METRIC

W.P. 319-85-00 LOCATION Opatatika River Sta. 12+687 o/s 3.0 m Rt ORIGINATED BY RG
DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY OL
DATUM Geodetic DATE 7.19.07 - 7.19.07 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
223.3	Asphalt							20 40 60 80 100						
223.0	175 mm ASPHALT							○ UNCONFINED × FIELD VANE						
0.2	FILL: brown sand, with silt, some clay, trace gravel		1	AS	--		223	● QUICK TRIAXIAL × LAB VANE					kN/m ³	GR SA SI CL
			2	SS	6		222							
			3	SS	11		221							4 56 32 8
			4	SS	8		220							
			5	SS	6		219							
219.6							218							
3.7	Non fibrous PEAT some silty clay, trace sand Loose Brown		6	SS	4		217						WL = 92 Wp = 64	
			7	SS	4		216						44.5 WL = 59 Wp = 41	
			8	SS	5		215						88.3	
217.2							214							
6.1	Silty CLAY (CL-ML) some sand, trace gravel Firm to Hard Brown to grey		9	SS	6		213							
			10	SS	8		212							
			11	SS	102		211							3 12 65 20
			12	SS	37		210							
211.5							209							
11.8	Silty SAND (SM) TILL some clay, trace gravel, with occasional to frequent cobbles and boulders Very dense Brown to grey		13	SS	110 / 250 mm									
			14	SS	60 / 50 mm									

Continued Next Page

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

ONTARIO MOT 1015345 OPASATIKA JULY 2007 GPJ ONTARIO MOT.GDT 1/3/08

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METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	UNIT WEIGHT	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	 w _p ————— w ————— w _L 10 20 30	γ	GR SA SI	

[illegible]

RECORD OF BOREHOLE No OPA-07-4

1 OF 2

METRIC

W.P. 319-85-00 LOCATION Opatatika River Sta. 12+737 o/s 8 m Rt ORIGINATED BY NH
 DIST New Liskeard HWY 11 BOREHOLE TYPE Steel Casing, Split Spoon COMPILED BY NH
 DATUM Geodetic DATE 8.10.07 - 8.12.07 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	× FIELD VANE						
218.6	0.0	WATER													
217.6	1.0	FILL: black sand with asphalt					218								
217.2	1.4	Non fibrous PEAT some silty clay, trace sand, with wood fragments Very loose to loose Grey		1	SS	5									
				2	SS	10	217								
							216						117		
				3	SS	1							166.7		
							215						259		
				4	SS	7									
							214						89.5		
213.5	5.1	SAND (SP) trace silt, wet Compact Grey		5	SS	6									
				6	SS	23	213								
212.6	6.0	Silty CLAY (ML-CL), wet Hard Grey		7	SS	60	212								
211.6	7.0	Silty SAND (SM) TILL some gravel, trace clay, with rock fragments at 14.9 m to 15.9 m, occasional cobbles and boulders, wet Dense to very dense Grey		8	SS	82	211								
							210								
				9	SS	31	209								
							208								
				10	SS	56	207								
							206								
				11	SS	100/ 127 mm	205								
				12	SS	100/ 150 mm	204								

Continued Next Page

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

RECORD OF BOREHOLE No OPA-07-4

2 OF 2

METRIC

W.P. 319-85-00 LOCATION Opasatika River Sta. 12+737 o/s 8 m Rt ORIGINATED BY NH
DIST New Liskeard HWY 11 BOREHOLE TYPE Steel Casing, Split Spoon COMPILED BY NH
DATUM Geodetic DATE 8.10.07 - 8.12.07 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								WATER CONTENT (%)			
			13	SS	100/ 100 mm			20	40	60	80	100	10	20	30	GR	SA	SI	CL
202.8			1	NQ			203												
15.8	Fair to poor quality pinkish GNEISS - very close to moderate joint spacing, dip 0 degrees to 80 degrees - slightly weathered to fresh - highly fractured at 16.9 m to 18.7 m TCR = 96% SCR = 66% RQD = 52%		2	NQ			202												
	TCR = 100% SCR = 52% RQD = 26%		3	NQ			201												
199.9							200												
18.7	END OF BOREHOLE at approximately 18.7 m																		

RECORD OF BOREHOLE No OPA-07-5

1 OF 2

METRIC

W.P. 319-85-00 LOCATION Opatatika River Sta. 12+758 o/s 19 m Rt ORIGINATED BY NH
 DIST New Liskeard HWY 11 BOREHOLE TYPE Steel Casing, Split Spoon COMPILED BY NH
 DATUM Geodetic DATE 8.13.07 - 8.14.07 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
220.3	Grass					▽	220	○ UNCONFINED	✕ FIELD VANE						
220.0	125 mm TOPSOIL: silty sand with high organic content		1	SS	9		219								
219.1	FILL: brown sand and gravel, moist - moist to 1.2 m		2	SS	18		218								
218.6	FILL: silty clay, moist						217								
218.0	Silty CLAY (CL-ML) with sand, wet Firm to stiff Brown to grey		3	SS	9		216								
213.7	Sandy Silty CLAY (CL-ML) Firm to stiff Grey		4	SS	5		215								
6.6	Silty SAND (SM) TILL trace gravel and clay, boulder at 13.1 m to 13.3 m, occasional cobbles and boulder, wet Dense to very dense Grey		5	SS	4		214								
			6	SS	5		213								
			7	SS	7		212								
			8	SS	15		211								
			9	SS	16		210								
			10	SS	58		209								
			11	SS	102		208								
			12	SS	60		207								
			13	SS	43	206									
			14	SS	100/150 mm										

ONTARIO MOT 1015345 OPASATIKA JULY 2007 GPJ ONTARIO MOT GDT 1/3/08

Continued Next Page

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

RECORD OF BOREHOLE No OPA-07-5

2 OF 2

METRIC

W.P. 319-85-00 LOCATION Opasatika River Sta. 12+758 o/s 19 m Rt ORIGINATED BY NH
 DIST New Liskeard HWY 11 BOREHOLE TYPE Steel Casing, Split Spoon COMPILED BY NH
 DATUM Geodetic DATE 8.13.07 - 8.14.07 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			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100		W _p W W _L				
								○ UNCONFINED × FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								20 40 60 80 100		10 20 30				

ONTARIO MOT 1015345 OPASATIKA JULY 2007 GPJ ONTARIO MOT GDT 1/3/08

RECORD OF BOREHOLE No OPA-07-6

1 OF 2

METRIC

W.P. 319-85-00 LOCATION Opatatika River Sta. 12+768 o/s 3.0 m Lt ORIGINATED BY RG
DIST New Liskeard HWY 11 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY OL
DATUM Geodetic DATE 7.18.07 - 7.18.07 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	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						
223.1	Asphalt							20 40 60 80 100							
222.9	63 mm ASPHALT FILL: brown sand, some silt and clay, trace gravel		1	AS	--		223								
			2	SS	7		222							5 71 (24)	
			3	SS	18		221								
			4	SS	7		220								
			5	SS	6		219								
			6	SS	1		218								
218.5	Silty CLAY (CL-ML) some sand, trace organic matter Firm Brown to grey		7	SS	8		217								
			8	SS	WH		216								
216.9	Silty SAND (SM) with clay, wet Very loose Grey		9	SS	2		215								
			10	SS	1		214								
213.9	Silty CLAY (CL-ML) some sand Firm Grey		11	SS	7		213								
							212								
212.4	Silty SAND (SM) TILL trace to some gravel, trace clay, with occasional cobbles and boulders, wet Dense to very dense Grey		12	SS	33		211								
			13	SS	63		210								
			14	SS	31		209								

Continued Next Page

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

2 OF 2

METRIC

DATUM Geodetic DATE 7.18.07 - 7.18.07 CHECKED BY GTC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT	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					
208.9 18.7	Fair to poor quality pinkish grey GNEISS - close to moderate joint spacing - fresh TCR = 100% SCR = 95% RQD = 89%	///	1	NQ		208							
		///				207							
	TCR = 100% SCR = 63% RQD = 67%	///	2	NQ		206							
204.9 18.1	END OF BOREHOLE at approximately 18.1 m Groundwater first encountered at a depth of approximately 3.0 m (El. 220.1 m)	///				205							

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

Appendix C

Geotechnical Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

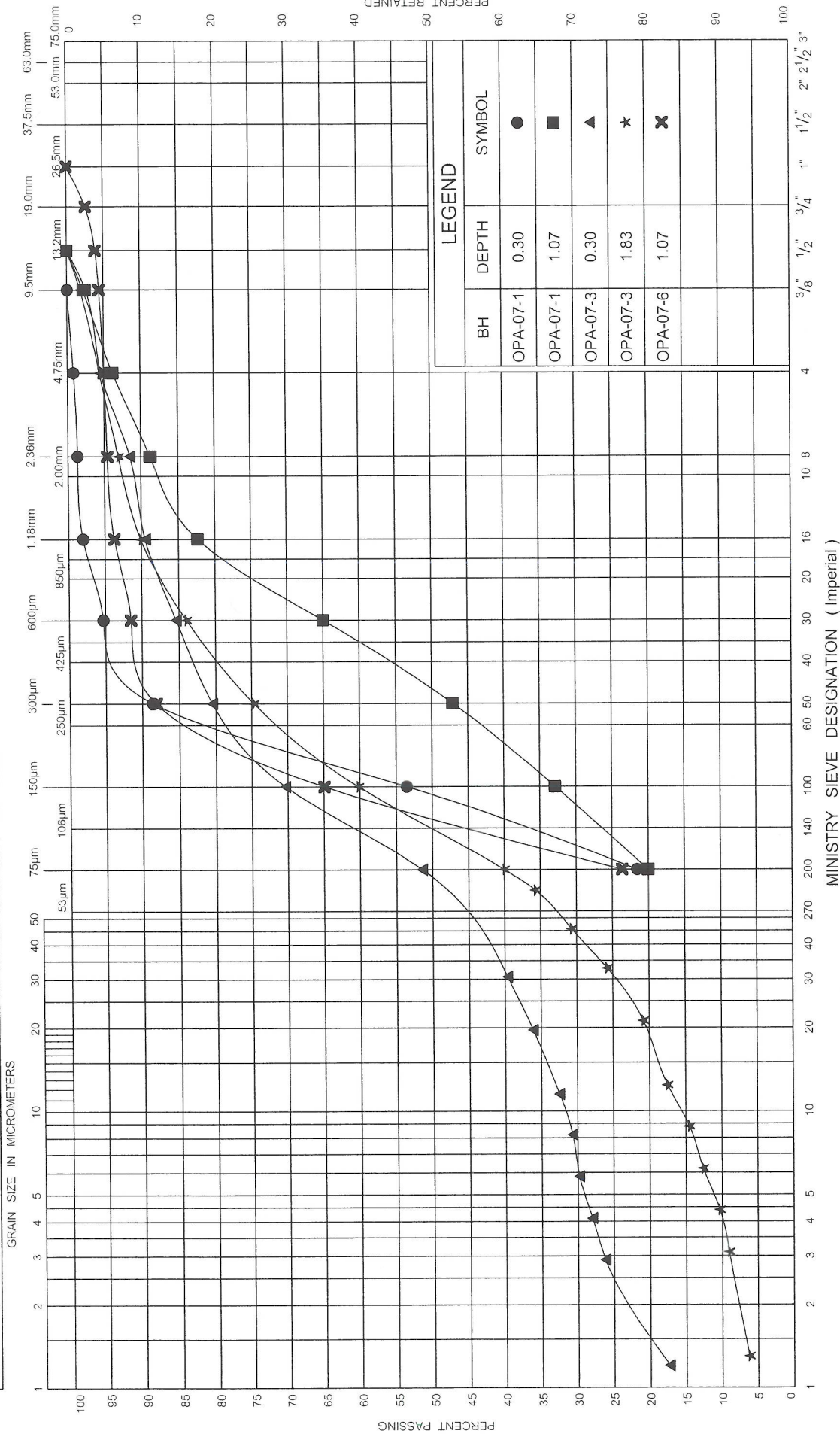


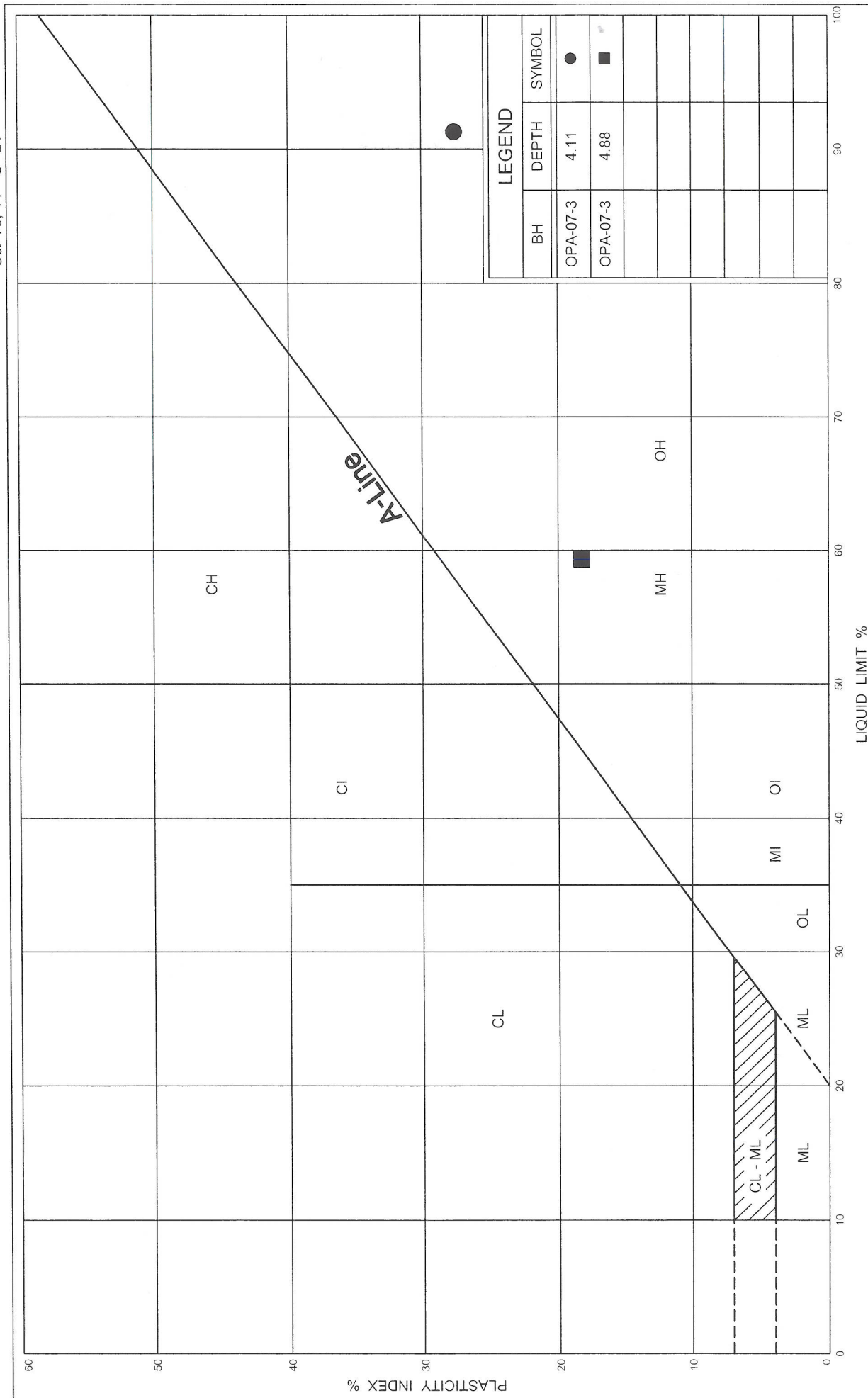
FIG No 1

GRAIN SIZE DISTRIBUTION

Sand Fill

W P 319-85-00

Opasatika Bridge/Hwy 11, Hearst, Ont.



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

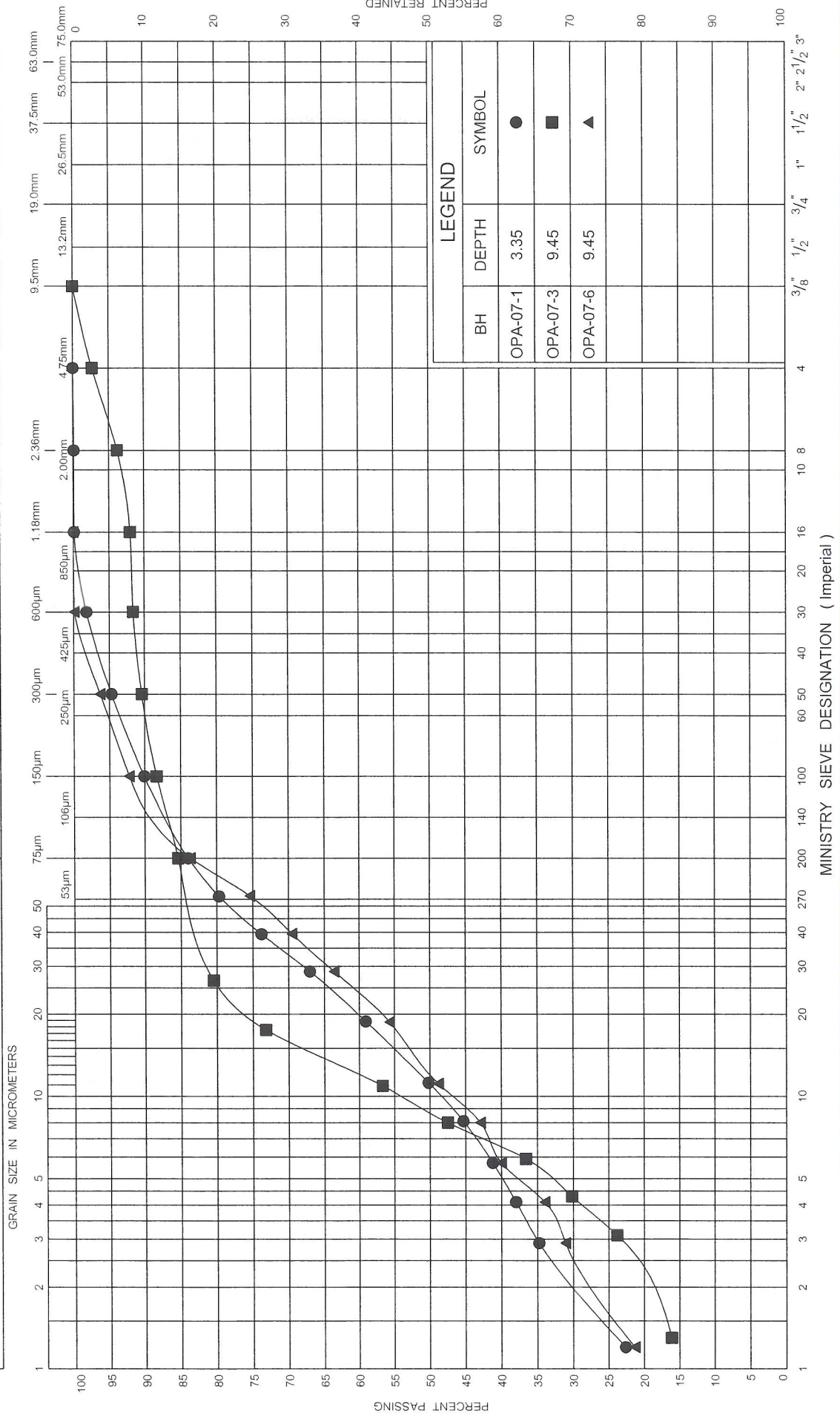
Fine

Medium

Fine

Coarse

Coarse



GRAIN SIZE DISTRIBUTION

Silty Clay (CL - ML)

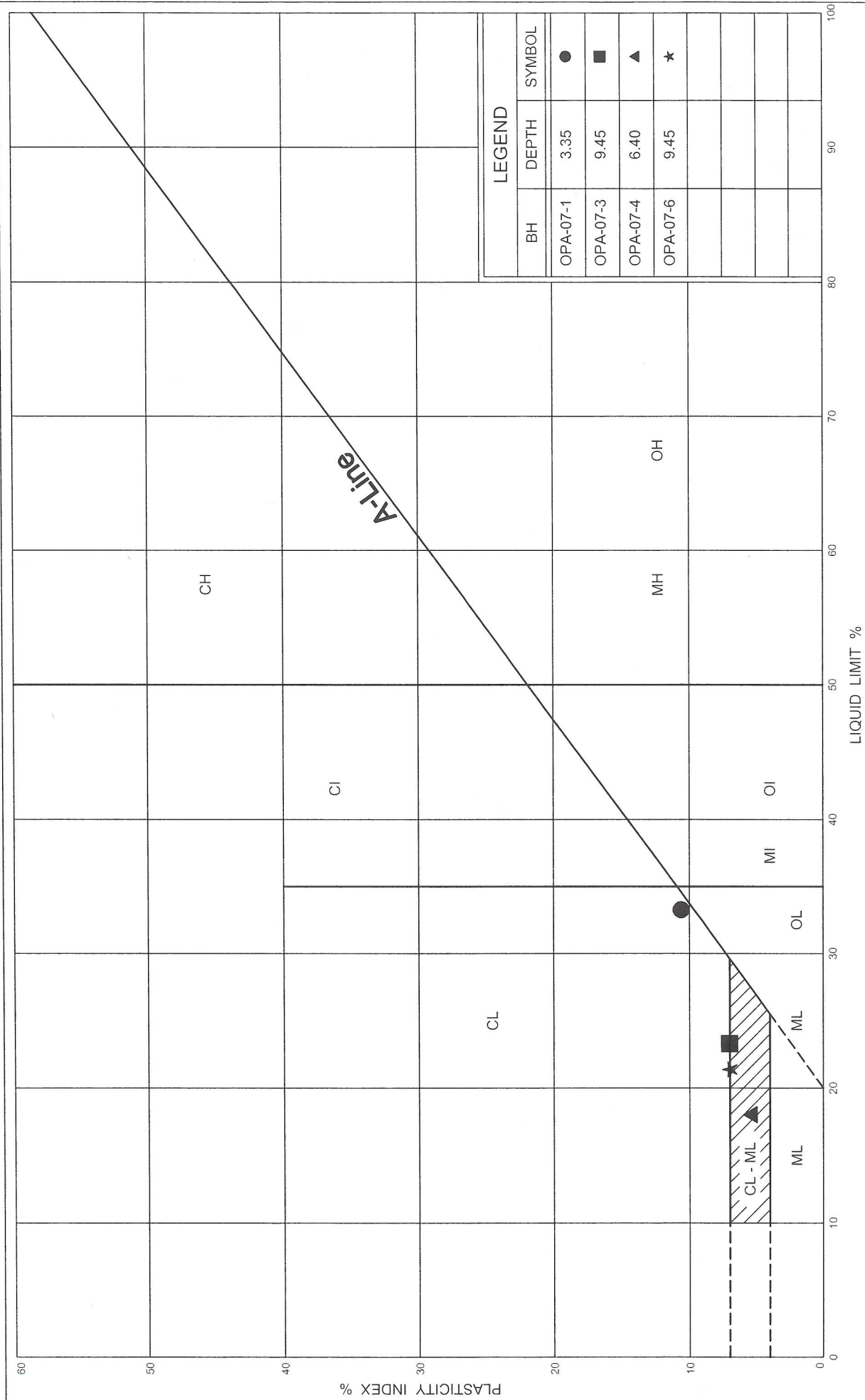
FIG No 3

W P 319-85-00

Opasatika Bridge/Hwy 11, Hearst, Ont.

Ministry of
Transportation





PLASTICITY CHART
Silty Clay (CL - ML)

FIG No 4

W P 319-85-00

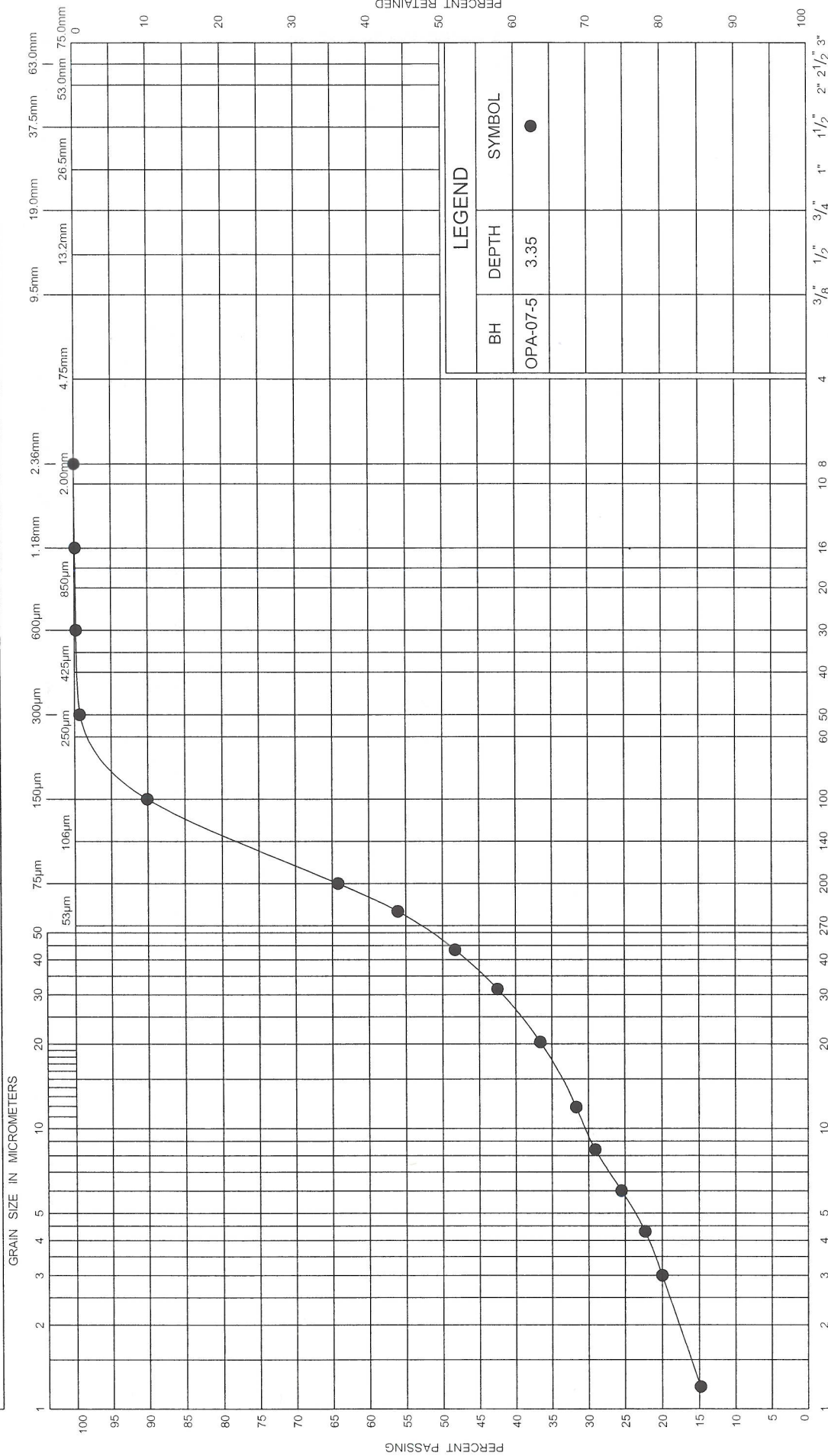
Opasatika Bridge/Hwy 11, Hearst, Ont.

Ministry of
Transportation

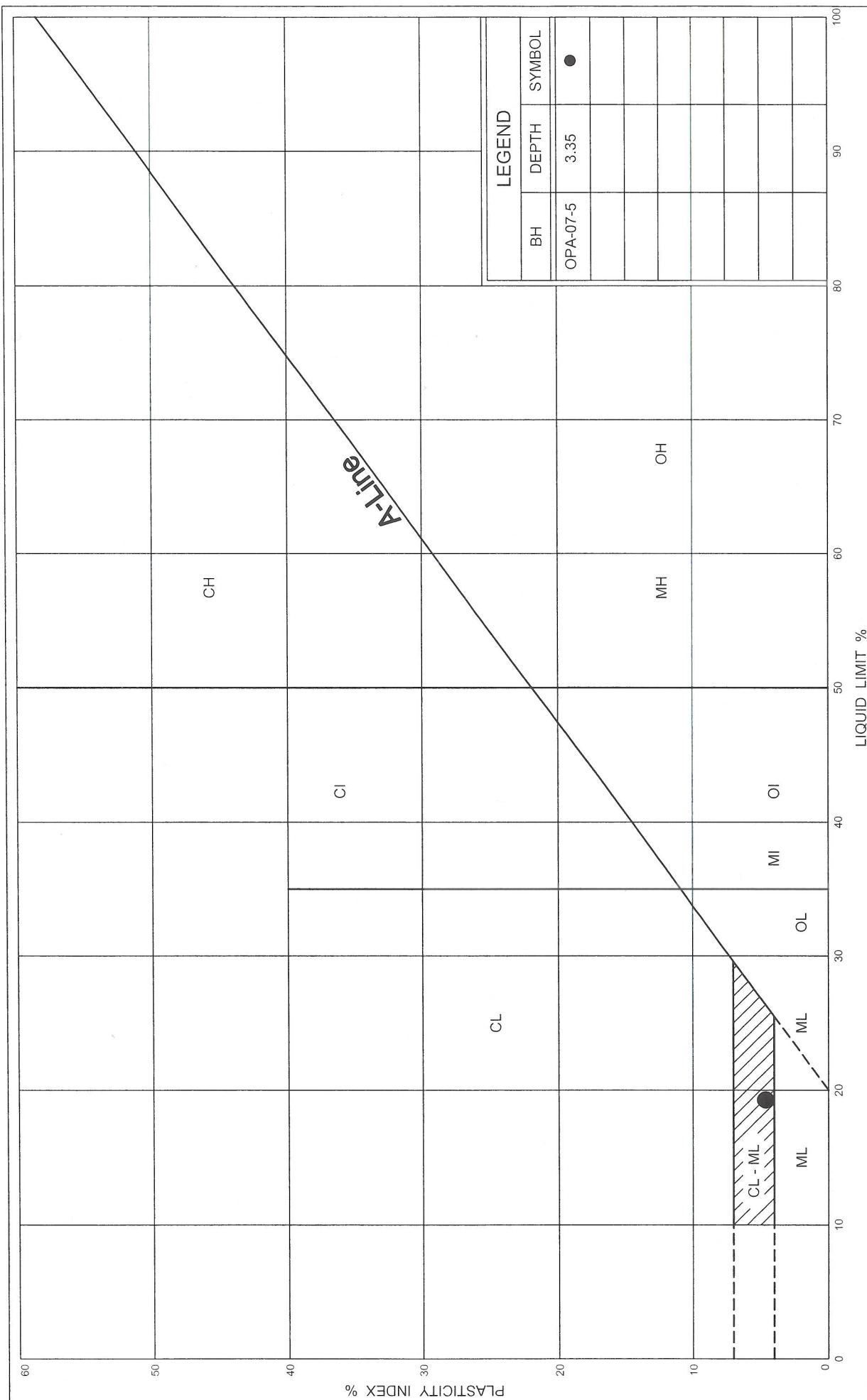


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse



BH	DEPTH	SYMBOL
OPA-07-5	3.35	●



PLASTICITY CHART

Ministry of
Transportation

Sandy Silty Clay (CL - ML)

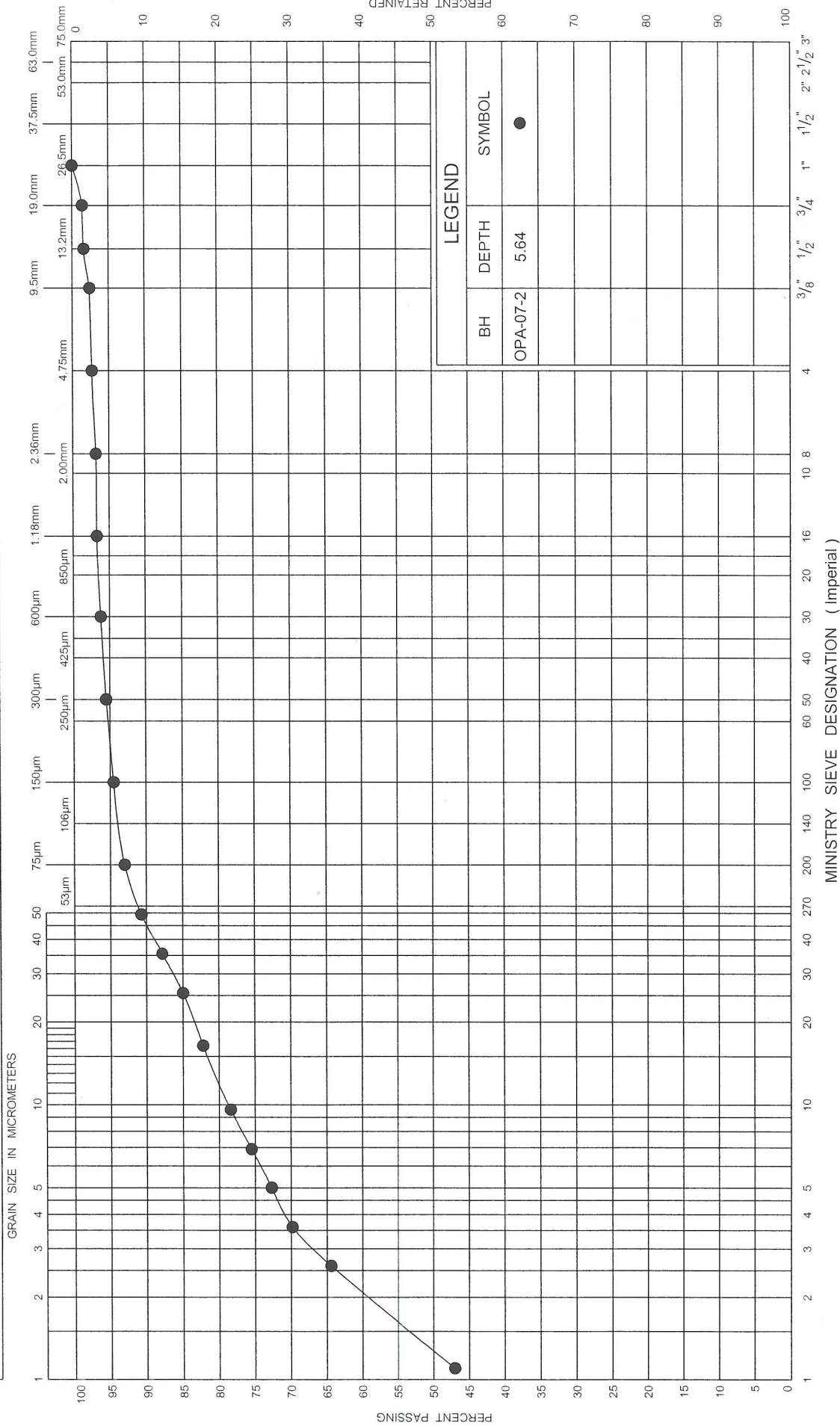
FIG No 6

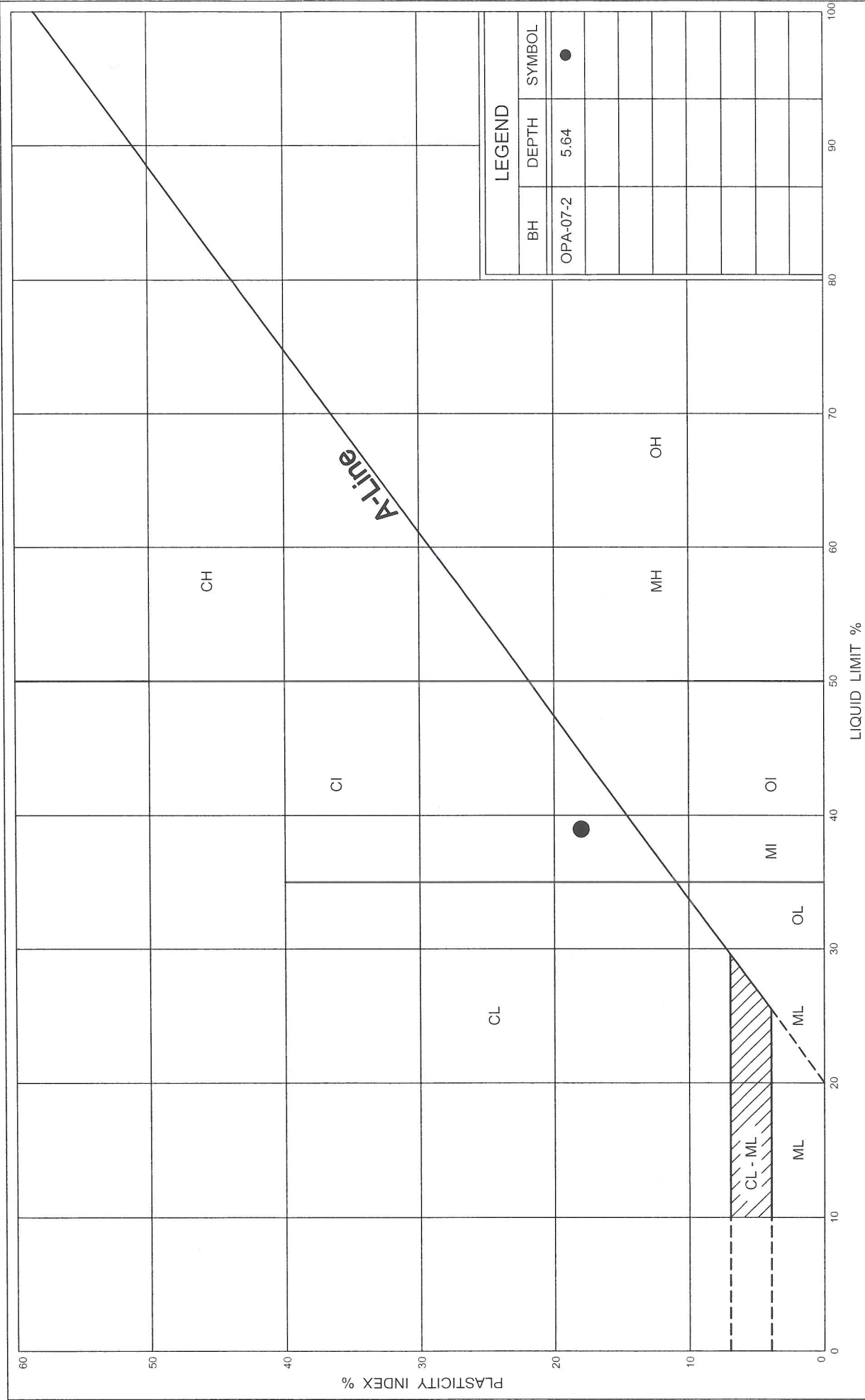
W P 319-85-00

Opasatika Bridge/Hwy 11, Hearst, Ont.

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse





PLASTICITY CHART

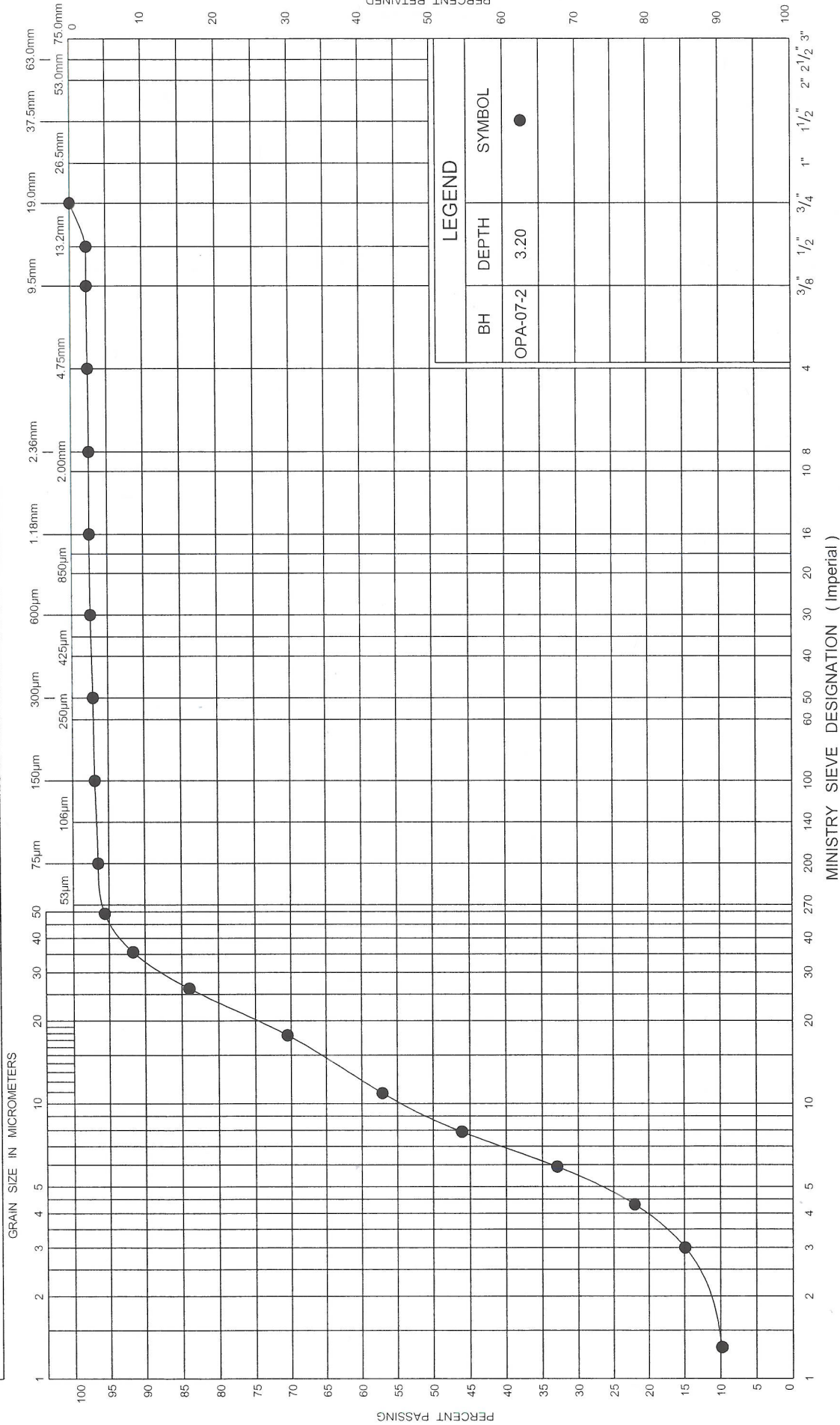
FIG No 8

WP 319-85-00

Opatika Bridge/Hwy 11, Hearst, Ont.

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



GRAIN SIZE DISTRIBUTION

Silt (ML)

Ministry of
Transportation



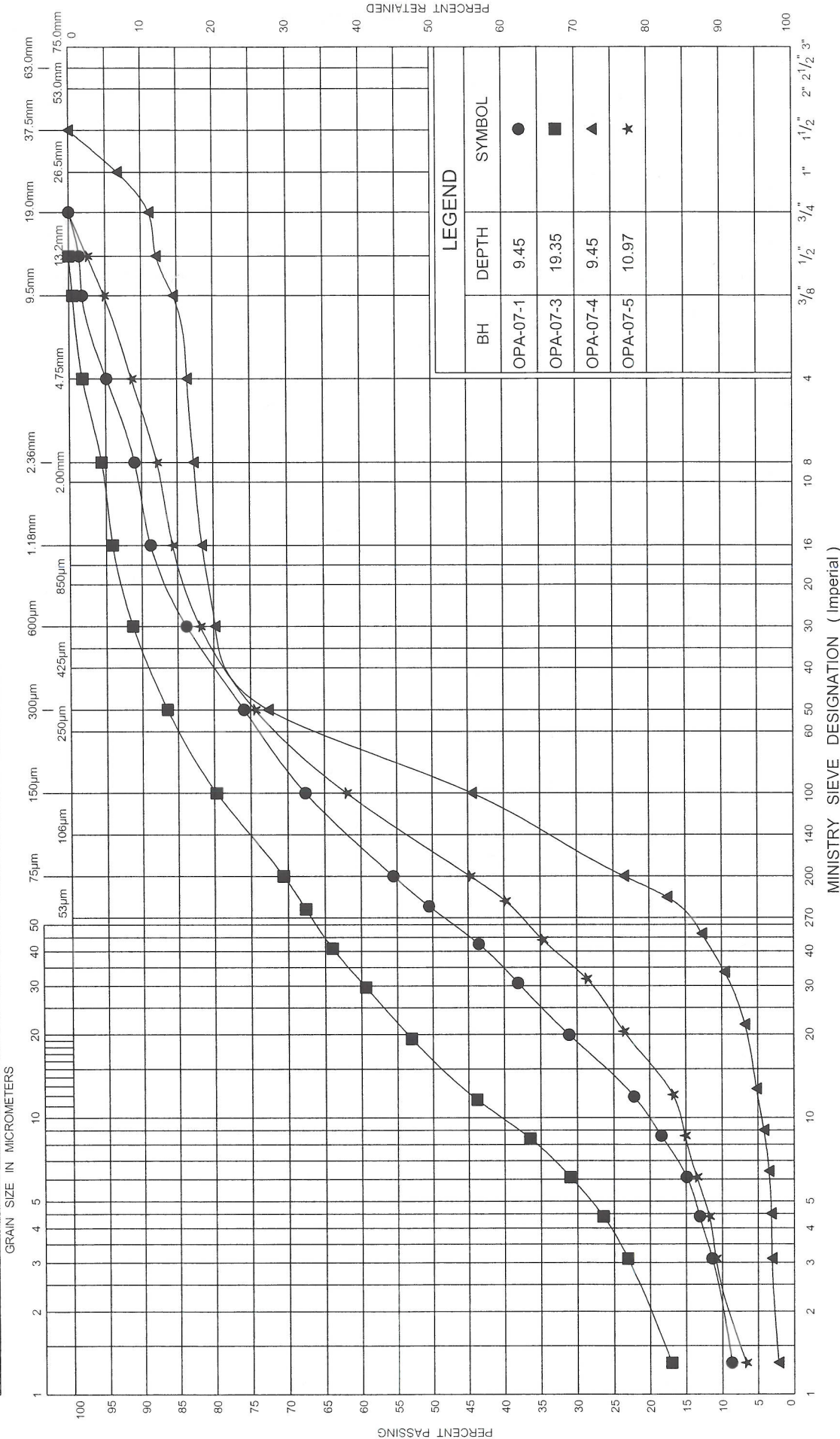
FIG No 9

W P 319-85-00

Opasatika Bridge/Hwy 11, Hearst, Ont.

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse



Appendix D

List of Standard Specifications and Drawings
Reference in the Report

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 Opasatika River near Opasatika, Ontario.

Standard Drawings:

OPSD3000.100

OPSD3000.150

OPSD3101.150

OPSD3102.100

Standard Specifications:

OPSS206

OPSS501

OPSS572

Special Provisions:

SP105S19

SP902S01

SP903S01

SS103-11