FINAL REPORT

Foundation Investigation and Design Report
Front Street / CNR Overpass (WBL), Site 14-363/2
Highway 402
City of Sarnia, Ontario
District - London
G.W.P. 3038-03-00

STANTEC CONSULTING LTD.

PROJECT NO. 1012607
GEOCRES NO. 40J16-79

REPORT NO. 1012607

REPORT TO	**Stantec Consulting Inc.** **1400 Rymal Road East** **Hamilton, ON** **L8W 3N9**
FOR	**Foundation Investigation and Design Report**
ON	**Front Street / CNR Overpass (WBL)** **Site 14-363/2, Highway 402** **City of Sarnia** **District – London, Ontario** **G.W.P. 3038-03-00** **Geocres. No. 40J16-79**

August 25, 2008

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1.0 INTRODUCTION

Jacques Whitford Limited (Jacques Whitford) was retained by Stantec Consulting Ltd., to complete a Foundation Investigation and Design Report for the widening of the Highway 402 Front Street / CNR Overpass (WBL), site 14-363/2, in the City of Sarnia, Ontario, (GWP No. 3038-03-00).

The work was carried out under Agreement No. 3005-E-0029 and in general accordance with the Subconsultant Agreement dated May 24, 2006. Authorization to proceed with the investigation was provided by Mr. David Emery, P.Eng., of Stantec Consulting Ltd., the prime consultant on this detailed design assignment.

The scope of work for the foundation investigation is incorporated within Stantec's project, which forms part of the above noted subconsultant agreement.

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 location is on Highway 402 at the Front Street Interchange (IC-1) in the City of Sarnia, Ontario.

Highway 402 at Front Street is a semi-urban freeway with wide gravel shoulders and a wide grass covered centre median. Highway 402 is built on embankments that are approximately 8 m to 9 m high at the bridge location. Highway 402 is generally oriented in an east west direction with two east bound lanes, two west bound lanes, and a west bound ramp all passing over Front Street. The east and west bound lanes of the highway at this location are super-elevated, curving to the north.

Front Street is five lanes wide at the structure. The street is generally at or near the ground surface elevation of the surrounding lands and is built with an urban section with concrete curbs. Drainage is provided by concrete gutters and a series of catchbasins located along the sides of the road.

Drainage for Highway 402 is provided by ditches located along the sides and in the central median. The ditches are sloped towards a limited number of catch basins located along the existing highway. Regional drainage is towards the St. Clair River located approximately 0.8 km southwest of the project site.

There are 2 bridge structures that convey Highway 402 over Front Street; one structure for the eastbound lanes and one structure for the west bound lanes. Previously, the bridges also conveyed the highway over CNR tracks, which have since been replaced with a walking path. The former CNR alignment is located on the east side of Front Street. The structures were reportedly constructed in 1981 and are of similar construction consisting of 5 spans supported on abutments on the east and west side with 4 central piers. The spans range in length from approximately 10 m to 15 m, with the shorter spans located near the abutments. There is approximately 12 m between the two bridge structures. The bridges are reportedly constructed of reinforced and pre-stressed concrete.

Based on Drawing Sheet No. 107, WP 347-65-02 by Nisbet Lethan Limited, it is understood that the existing bridge structures are supported on 325 mm diameter steel pipe piles driven to the underlying bedrock at depths in the range of 38.1 m, (Elev. 140 m) to 47.2 m (Elev. 136.5 m). The drawing indicates that there are 9 piles at each abutment location and 10 piles at each pier location. The piles were installed with an incline of 1:4 (Horizontal to Vertical), and were filled with concrete that was to have a 28 day compressive strength of 30 MPa.

Representative colour photographs of the bridges are provided in **Appendix D**.

3.0 PHYSIOGRAPHY

Based on the physiography of Southern Ontario by Chapman and Putnam (1984), this section of Highway 402 is situated in the physiographic region known as the Huron Fringe, a narrow geological strip between Lake Huron and the adjacent St. Clair Clay Plains. The Huron Fringe is composed mainly of surficial sands, silts and gravels, underlain by lacustrine clayey silt and silty clay.

The bedrock in the area of the site consists of laminated, thinly bedded shale that is black to grey in colour and is of the Kettle Point Formation.

4.0 BACKGROUND

Previous Investigations

A previous assessment of the site was carried out by Golder Associates Ltd. The results of the assessment were provided in a written report titled:

> *Preliminary Foundation Investigation and Design Report, Proposed Front Street/CNR Overpass, Structure Widening, Highway 402, Geocres No. 40J16-66, GWP 3038-03-00, Agreement Number 3005-A-000394,* Dated July 26, 2006.

This preliminary report was based on factual data contained within the Ministry of Transportation's, Geocres Library, MTO Report Geocres No. 40J16-58 titled:

> *Foundation Investigation Report for Front Street Overpass and CNR Overhead, W.P. 347-65-02/03, Site 14-363, Highway 402, District 1, Chatham,* Dated August 1977.

Nine boreholes were drilled at the site in 1977. The reported subsurface conditions included the following strata types:

Strata	Layer Thickness (m)	Depth to underside of Strata (m)
Sand	1.8 to 3.0	1.8 to 3.0
Clayey Silt	13 to 17	16.8-17.1 5 boreholes were terminated in this layer
Silty Clay	18 to 21	34.7-37.5
Sand	1.5 to 1.9	37 to 40
Bedrock	n/a	Termination depth of the boreholes

The upper sand was generally reported to be very loose to compact, with moisture contents in the range of approximately 11% to 32%.

The clayey silt layer was stated to be firm to very stiff, with actual reported measurements indicating stiff to very stiff conditions. In situ shear vane tests ranged from approximately 31 kPa to >105 kPa (the limit of the equipment), with an average of approximately 64 kPa. Laboratory tests were reported as follows:

- Moisture contents:
    - Crust: approximately 10% to 20%, average of about 16%;
    - Below the crust: approximately 12% to 28%, average of about 22%;
- Average plastic limit of approximately 15%;
- Average liquid limit of approximately 30%;
- Bulk densities ranging from approximately 2,000 to 2,100 kg/m^3; and,
- Confined and unconfined triaxial testing yielded shear strengths of approximately 34 kPa to 115 kPa.

The silty clay layer was stated to be stiff. In situ shear vane tests ranged from approximately 55 kPa to 101 kPa with an average of 74 kPa. Laboratory tests were reported as follows:

- Moisture contents ranging from 17% to 36%, average of about 27%;
- Average plastic limit of approximately 20%;
- Average liquid limit of approximately 39%;
- Bulk density of approximately 1,900 kg/m^3; and,
- Confined and unconfined triaxial shear strengths ranging from approximately 69 kPa to 79 kPa.

The bedrock was identified as black shale of the Kettle Point formation. The total rock recoveries reported were 77% to 100%.

Groundwater was reported at depths in the range of about 0.8 m to 2.3 m below ground surface, corresponding to elevations between 176 m and 178 m.

5.0 INVESTIGATION PROCEEDURES

5.1 Field Program

The fieldwork for the present investigation was carried out between November 1 to 23, 2006, December 12 to 19, 2006 and January 30, 2007 to February 1, 2007. A total of 5 boreholes were advanced to depths ranging from approximately 40 m to 50 m below existing grade, using track and truck mounted drill rigs equipped with 250 mm (outside diameter), hollow-stem augers and mud-rotary drilling techniques. The drill rigs were supplied and operated by Aardvark Drilling Inc. and London Soils.

Prior to commencing the field investigation, the borehole locations were established in the field by Jacques Whitford personnel. 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. Relatively undisturbed samples were obtained by pushing thin walled sample tubes in general accordance with ASTM D1587.

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 HQ wire line 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 Front Street or Highway 402, as noted on the Record of Borehole sheets. Offsets were referenced looking up chainage. The borehole locations are also referenced to Northing and Easting co-ordinates, which 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 either the benchmark or temporary benchmark noted below:

- BM 860, Cross cut on the southwest corner of the west abutment of the westbound bridge over Front Street, with a reported Geodetic elevation of 186.53 m, as identified on a survey drawing by J. D. Barnes, titled “Pre-Engineering, Sta.: 11+000 to Sat 11+700”, with a survey date of August, 2004; and,

- Temporary BM, a catchbasin on the east side of Front Street, approximately 20 m north of Pier 3, with a Geodetic elevation of 177.54 m, as provided by Stantec, February 2006.

The location of the benchmark and temporary benchmark are shown on Drawing No. 1 in **Appendix A**.

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

Three rock core samples were submitted for unconfined compressive strength testing. The results of the unconfined compressive strength testing are provided on Figure No. 11 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 used on the Record of Borehole sheets is also provided in **Appendix B**. Copies of the Record of Borehole sheets from the 1977 investigation have been included in **Appendix B** for reference.

A Borehole Location Plan and a Strata Plot of the soils encountered in the boreholes are provided on Drawing No. 1 in **Appendix A**.

A summary of the soil and groundwater conditions encountered in the 2006/2007 boreholes is provided below.

6.2 Soil

6.2.1 Asphalt

Asphalt was encountered at the ground surface in Boreholes FS-2 and FS-3 and was approximately 150 mm and 165 mm thick, respectively.

6.2.2 Topsoil

Topsoil was encountered at the ground surface in Boreholes FS-1, FS-4 and FS-5. The thickness of the topsoil ranged from approximately 0.8 m to 1.5 m.

6.2.3 Sand Fill (SW)

Sand fill was encountered underlying the asphalt or topsoil in all boreholes. The sand fill ranged in thickness from approximately 0.7 m to 1.5 m.

The sand fill generally consisted of sand and gravel directly below the asphalt, grading to sand fill, trace gravel with increasing depth. The sand fill contained varying amounts of silt, with to trace, and was generally moist to damp.

Based on the N-Values obtained from the Standard Penetration Tests (SPTs), the compactness of the sand fill was variable ranging from loose to very dense, but was more typically compact to dense.

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

- Moisture Content:
    - 7% to 29%/
- Grain Size Distribution
    - 8% gravel;
    - 64% sand; and,
    - 28% 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 are provided on Figure 1 in **Appendix C**.

6.2.4 Clayey Silt Fill (CL)

Two layers of clayey silt fill were encountered in Borehole FS-5. The first layer was encountered at a depth of approximately 1.4 m, elevation of approximately 184.5 m and was approximately 0.7 m thick. The second layer of clayey silt fill was encountered in Borehole FS-5 at a depth of approximately 2.9 m, elevation of approximately 182.9 m and was approximately 1.2 m thick.

The clayey silt fill contained trace to some sand, trace to some gravel, trace to some organics, topsoil and plant debris, and was generally moist.

Based on the N-Values obtained from the SPTs, the consistency of the clayey silt fill was stiff.

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

- Moisture Content:
    - 8% and 18%.

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

6.2.5 Silty Sand Fill (SM)

A layer of silty sand fill was encountered underlying the sand fill in Borehole FS-1 at a depth of approximately 1.5 m, elevation of approximately 183.9 m. The silty sand fill was approximately 2.3 m thick.

The silty sand fill contained some gravel, varying amounts of fly and bottom ash and trace organics. The silty sand fill was generally moist.

Based on the N-Value obtained from a single SPT, the compactness of the silty sand fill was compact.

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

- Moisture Content:
  - 10% to 18%

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

6.2.6 Fly and Bottom Ash (SP-SM)

Fly and bottom ash was encountered in Boreholes FS-1 and FS-5 at depths of approximately 3.8 m and 4.1 m below existing grade, or elevations of approximately 181.7 and 181.6, respectively. The thickness of the fly and bottom ash was approximately 3.1 m in FS-1 and 4.6 m in FS-5.

The fly and bottom ash could be characterised as a sandy silt / silty sand with trace to some gravel and trace clay sized particles. The ash was generally moist to wet.

Based on the N-Values obtained from the SPTs, the compactness of the fly and bottom ash ranged from very loose to very dense, but was more typically loose.

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

- Moisture Content:
  - 15% to 37%.
- Grain Size Distribution:
  - 3% to 16% gravel;
  - 31% to 74% sand;
  - 9% to 62% silt; and,
  - 1% to 5% clay.
- Atterberg Limits:
  - Liquid Limit: 26%;
  - Plastic Limit: 15%; and,
  - One sample tested indicated non-plastic.

The results of the moisture content tests, 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 and Atterberg Limits tests are provided on Figures 2 and 3 in **Appendix C**.

6.2.7 Native Sand (SM)

Native sand was encountered underlying the fill in Boreholes FS-1, FS-2 and FS-3. The sand was encountered at depths of 8.4 m, 1.5 m, and 1.4 m, respectively, elevations of approximately 176.2 m to

177.1 m. The thickness of the sand ranged from approximately 0.7 m to 2.3 m. The sand was encountered at greater depth in Borehole FS-1, as this borehole was drilled on the top of the approach embankments.

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

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

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

- Moisture Contents:
    - 13% to 24%.
- Grain Size Distribution:
    - 1% gravel;
    - 70% to 82% sand; and,
    - 17% to 30% fines (silt and clay).

The results of the moisture content tests 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 also provided on Figure 4 in **Appendix C.**

6.2.8 Silt (ML)

A localized deposit of silt was encountered in Borehole FS-2 at a depth of approximately 6.9 m, elevations of approximately 170.9 m and was approximately 1.5 m thick.

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

Based on the N-Values obtained from a single SPT, the compactness of the silt was assessed to be compact.

Laboratory testing performed on a single sample consisted of a moisture content test, a grain size distribution and Atterberg Limits test. The test results are as follows:

- Moisture Content:
    - 18%.
- Grain Size Distribution:
    - 1% gravel;
    - 26% sand;
    - 64% silt; and,
    - 9% clay.
- Atterberg Limits:
    - Non-plastic.

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 test are also provided on Figure 5 in **Appendix C.**

6.2.9 Silty Clay (CL)

Silty clay was encountered underlying the sand or fill in all boreholes. The silty clay was encountered at depths in the range of approximately 2.1 m to 10.7 m below existing grade, or elevation of approximately 174.3 m to 177.1 m. The silty clay was approximately 28.2 m to 37.9 m thick and extended to depths in the range of approximately 36.6 m to 46.6 m or elevations of approximately 139.2 m to 141.2 m.

The silty clay generally contained some sand and trace gravel near the contact with the sand stratum described above, then containing less sand with increasing depth. The silty clay was generally moist to damp.

Based on the N-Values obtained from the SPTs, the consistency of the silty clay was variable ranging from very soft to very stiff. The upper 2 m of the silty clay generally could be classified as very stiff, indicating this layer is likely a desiccated layer.

In situ shear vane testing was carried out in the silty clay. The results of the testing indicated that the shear strength of the silty clay was variable ranging from approximately 50 kPa to >100 kPa (the upper limit of the testing equipment). The in situ shear vane testing indicated that the consistency of the silty clay could be described as firm to very stiff.

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

- Moisture Content:
    - 13% to 37%.
- Grain Size Distribution:
    - 1% to 17% gravel;
    - 10% to 36% sand;
    - 29% to 48% silt; and,
    - 19% to 51% clay.
- Atterberg Limits:
    - Liquid Limits: 24% to 42%; and,
    - Plastic Limits: 14% to 23%.
- Unit Weight Measurements:
    - 22.3 to 22 kN/m^3.

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

The results of the grain size distribution tests are provided on Figures 6 and 7 in **Appendix C**. The results of the Atterberg Limits tests are provided on Figures 8 and 9 in **Appendix C**. The results of the unit weight tests are provided on Figure 10 in **Appendix C**.

6.2.10 Bedrock

Bedrock was encountered in all boreholes at depths of approximately 36.6 m to 46.6 m below existing grade, corresponding to elevations of approximately 141.2 to 139.2 m. The bedrock consisted of black shale of the Kettlepoint formation.

Core samples of the bedrock were obtained from Boreholes FS-2, FS-3 and FS-4. The observations of the rock cores are summarized as follows:

- Total Core Recovery (TCR): 15% to 100%, Mean of about 71%;
- Solid Core Recover (SCR): 11% to 99%, Mean of about 53%;and,
- Rock Quality Designation (RQD): 0% to 95%, Mean of about 49%.

Three samples of the rock were submitted for testing the unconfined compressive strength. The test results were as follows:

- Unconfined compressive strength:
  - Sample from FS-2: 95 MPa;
  - Sample from FS-3: 97 MPa;
  - Sample from FS-4: 99 MPa; and,
  - Mean: 97 MPa.

The results of the rock core analysis and unconfined compressive strength test results are provide on the Record of Borehole sheets in **Appendix B**.

The results of the unconfined compressive strength tests are also provided on Figure 11 in **Appendix C**.

6.3 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; the use of drilling mud also prohibited the installation of functional monitoring wells. However, wet conditions were encountered in the boreholes during drilling at depths in the range of approximately 1.4 m to 8.7 m below existing grade, corresponding to elevations in the range of approximately 176.8 m to 177.1 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 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:

Raymond Haché, P.Eng., M. Sc., PMP
Principal, Geotechnical Service Director, and
Designated Principal
MTO Foundations Contact

GC/RH/dd

Enclosures

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FOUNDATION DESIGN REPORT

Front Street / CNR Overpass (WBL)
Site 14-363/2, Highway 402
City of Sarnia, Ontario
G.W.P. 3038-03-00
District – London

8.0 DISCUSSION

8.1 General

There are 2 bridge structures that convey Highway 402 over Front Street and the former CN rail line, now a walking path; one structure for the eastbound lanes and one structure for the west bound lanes. There is approximately 12 m between the two bridge structures.

Based on drawings provided by Stantec, the existing bridge structures were built about 1981 and are of similar construction; consisting of 5 spans supported on abutments and 4 piers. The spans range in length from approximately 10 m to 15 m, with the shorter spans located near the abutments. The bridges are reportedly constructed with a reinforced concrete deck and pre-stressed concrete girders. The drawings indicate that the existing structures are supported on 325 mm diameter concrete filled steel pipe piles driven to the underlying bedrock.

Highway 402 at Front Street is built on high embankments to a semi-urban freeway section with wide gravel shoulders and a wide grass covered centre median. The embankments are approximately 8 m to 9 m high at the overpass location. Highway 402 is generally oriented in an east west direction and passes over Front Street. The east and west bound lanes of Highway 402 at this location are independently super-elevated, curving to the north.

Front Street is four lanes wide, at the structure. The street is generally at or near the ground surface elevation of the surrounding lands and is built with an urban cross section with concrete curbs.

8.2 Proposed Development

The Ministry of Transportation (MTO) is proposing to widen and upgrade a section of Highway 402 from the Blue Water International Bridge property eastward to Indian Road, a total distance of approximately 3.1 km (Sta. 10+500 to Sta. 13+600).

As part of the highway widening work, two additional westbound lanes will be added to Highway 402 on a new structure over Front Street. It is understood that the proposed lanes will be located between the existing bridge structures. The proposed structure will be similar to the existing bridge structures consisting of a 5 span pre-stressed concrete girders with reinforced concrete deck. The proposed bridge will include semi-integral abutments.

Due to the super-elevation of Highway 402 at the bridge location, the inside shoulder of the widened west bound lanes will be approximately 2 m to 2.5 m higher than the inside shoulder of the east bound lanes. To accommodate the grade difference on the approach embankments, it is understood that a retaining structure, such as a retaining wall or retained soil system, will be constructed.

It is also understood that a storm water management pond will be constructed in the central portion of the East-North/South ramp. The investigation and design report for the storm water management pond will be provided in a separate report under separate cover.

8.3 Subsurface Conditions

The subsurface conditions encountered at the Highway 402 Front Street / CNR Overpass generally consisted of embankment fill to depths in the range of approximately 1.4 m to 8.7 m (bottom elevation of approximately 176.5 m to 177.7 m) overlying loose to compact native sand, overlying a deep, firm to very stiff, silty clay deposit. The fill was generally comprised of sand, silty sand, clayey silt and a combination of fly and bottom ash. The bottom of the sand underlying the fill was noted at elevations between approximately 174.7 m and 175.6 m. Firm to very stiff silty clay was encountered in all boreholes underlying the fill or native sand. The silty clay extended to depths in the range of approximately 36.6 m to 46.6 m, corresponding to elevations of approximately 139.2 m to 141.2 m. Bedrock, comprised of dark grey to black weathered shale, was encountered underlying the silty clay.

Groundwater was inferred in the boreholes at depths ranging from about 1.4 m to 8.7 m below existing grade or elevations ranging from approximately 176.4 m to 177.1 m. It was not possible to measure the groundwater levels in the boreholes, given the techniques use to drill the boreholes.

8.4 Preliminary Foundation Design Report

A previous assessment of the site was carried out by Golder Associates Ltd. The results of the assessment were provided in a written report titled:

> *Preliminary Foundation Investigation and Design Report, Proposed Front Street/CNR Overpass, Structure Widening, Highway 402, Geocres No. 40J16-66, GWP 3038-03-00, Agreement Number 3005-A-000394,* Dated July 26, 2006.

The Golder preliminary report was based on factual data contained within the Ministry of Transportation's, Geocres Library, MTO Report Geocres No. 40J16-58 titled:

> *Foundation Investigation Report for Front Street Overpass and CNR Overhead, W.P. 347-65-02/03, Site 14-363, Highway 402, District 1, Chatham,* Dated August 1977.

The preliminary recommendations provided by Golder included the following:

- Shallow spread footings would likely be impractical at this location, given the relatively high loads imposed from the bridge structure and the relatively low bearing capacity of the subsurface soils.
- Placing spread footings on granular rafts or mats would have enhanced geotechnical resistance. However, there were environmental limitations with this option, associated with excavation and disposal of fly ash and bottom ash from the site. In addition, differential movement between the existing bridge structures and the new structure would occur, given that the existing bridge structures are founded on piles.

- Steel 325 mm diameter concrete filled pipe piles driven to the underlying bedrock at an elevation of about 140 m are feasible. The following factored geotechnical axial resistance was provided:
    - 2000 kN ULS (the structural capacity of the pile): and,
    - SLS not provided as rock was considered an unyielding material.

The preliminary report indicated that the pipe pile option is preferable for this project as this would provide similar horizontal deformation characteristics as the existing piles.

The preliminary report also provides design recommendations for retaining walls and embankments.

8.5 Foundation Assessment

Additional information has now been obtained by drilling five more boreholes. The preliminary recommendations have been reviewed.

Based on the conditions observed and reported, the site is generally suitable for the use of either spread footings placed on the native soils or a piled foundation. However, the geotechnical resistance for spread footings may be inadequate to effectively support the anticipated bridge loads.

An enhanced geotechnical resistance for spread footings at the piers and abutments could be achieved by placing the shallow foundations on compacted granular pads. The construction of the granular pads at the abutment locations would be relatively straight forward. However, it would be necessary to dispose of fly and bottom ash at an appropriate off-site facility if encountered. At the pier locations it would be necessary to sub-excavate below the underside of the existing pile caps to install the granular pad. In addition, excavations may be difficult adjacent to the traveled lanes of Front Street, and would require the use of temporary shoring.

The use of semi-integral abutments is being contemplated for this location. The preferred option would be to support the abutments and piers on driven piles, which would be consistent with the foundation for the existing bridge structures.

The following table provides a summary of the foundation options under consideration:

Foundation Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
Spread Footings supported on native sand	Lowest cost.	Potential for settlement of the underlying soils May require the use of settlement mitigation measures. Geotechnical resistance at ULS and SLS are considered very low. May not be feasible due to low geotechnical resistance, even if settlement mitigation measures are taken.	Low	May encounter settlement above tolerable limits, even with mitigation measures taken.

Foundation Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
Spread footings on granular pads	Higher geotechnical resistance. Lower anticipated settlement.	Potential for settlement of the underlying soils. Potential to encounter bottom ash and fly ash, which should be removed and disposed off-site prior to construction.	Medium	Significant excavation of the existing embankment material required. Potential conflict with existing pile caps and Front Street could require shoring.
Steel H-Piles Driven to the underlying bedrock	High geotechnical resistance. Minimal settlement.	Potential to encounter cobbles and boulders during driving. Would have different deformation characteristics that the existing pipe piles.	High	Possible tip damage during driving which would require a driving shoe. Potential for down drag forces.
Steel Pipe Piles Driven to the underlying bedrock	High geotechnical resistance. Minimal settlement. Similar horizontal deformation characteristics as the existing piles	Potential to encounter cobbles and boulders during driving. Difficult to driver with closed bottoms.	High	Possible tip damage during driving which would require a driving shoe. Potential for down drag forces.
Friction piles founded in the clayey silt / silty clay	Some cost savings, reduction in number of splices.	Lower geotechnical resistance Some settlement under design loads. Differential settlement between the existing structure and widening, given that the piles for the existing structure were reportedly driven to the underlying bedrock.	High	Piles likely spaced closer together, which would result in reduction of capacity for pile groups.

9.0 RECOMMENDATIONS

9.1 Pile Foundations

9.1.1 End Bearing Piles

Given the conditions encountered during this investigation, it is recommended to found the new bridge on piles driven to bedrock. This option would be consistent with the foundations for the existing structures.

The abutments and piers could be founded on either HP310x110 piles or 324 mm diameter circular piles driven to the underlying bedrock at depths in the range of about 36.6 m to 46.6 m below existing grade or elevations of approximately 139.2 m to 141.1 m.

Steel H-Piles

HP310 x 110 Steel H-Piles for the abutments and piers driven to the underlying bedrock may be designed using a factored geotechnical resistance at ULS of 2000 kN. The ULS value includes a resistance factor of 0.4.

An unfactored SLS geotechnical resistance for piles driven to rock is not provided since the rock is considered to be unyielding. Geotechnical resistance at SLS will not govern since it will be greater that the factored ULS value. The structural engineer should consider the structural compressibility of the pile as part of the design.

Circular Pipe Piles

324 mm diameter steel pipe piles with a wall thickness of 13 mm for the abutments and piers driven to the underlying bedrock may be designed using a factored geotechnical resistance at ULS of 1700 kN. The ULS value includes a resistance factor of 0.4.

An unfactored SLS geotechnical resistance for piles driven to rock is not provided since the rock is considered to be unyielding. Geotechnical resistance at SLS will not govern since it will be greater that the factored ULS value. The structural engineer should consider the structural compressibility of the pile as part of the design.

9.1.2 Down Drag Forces

The westbound embankments will be widened into the centre median of Highway 402. Based on the sections provided by Stantec, it is understood that approximately 2.5 m of fill will be required in the central median. It is also understood that the super-elevation of the road will result in the inside shoulder of the west bound lanes being approximately 1.2 m higher than the inside shoulder of the east bound lanes with the grade separation achieved using a retaining wall.

The placement of the fill material will induce settlement of the existing embankments and the underlying soils. Down drag forces, induced as a result of the settlement of the underlying soils under the approach fill, must be considered. Calculations using the consolidation parameters provided in Section 9.4 indicate that the widened section embankment may induce settlements in the silty clay of approximately 20 mm to 30 mm.

Steel H-Piles

Based on the settlement estimates provided above, the unfactored negative skin friction calculated for an HP310X110 steel H-pile supporting the abutments is approximately 750 kN. The Neutral Plain is located at the contact elevation of the underlying bedrock. The calculation for the negative skin friction was applied to the perimeter of a theoretical box around the pile and presumes that 10 mm of relative movement between the pile and surrounding soil is required to mobilize the drag forces. Based on the settlement profile, 10 mm of relative movement is calculated to occur at an elevation of approximately 167 m geodetic.

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

Steel Pipe Piles

Based on the settlement estimates provided above, the unfactored negative skin friction calculated for a 324 mm diameter circular pipe pile supporting the abutments is approximately 620 kN. The neutral

Plain is located at the contact elevation of the underlying bedrock. The calculation for the negative skin friction was applied to the circumference of the pile and presumes that 10 mm of relative movement between the pile and surrounding soil is required to mobilize the drag forces. Based on the settlement profile, 10 mm of relative movement is calculated to occur at an elevation of approximately 167 m geodetic.

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

The neutral plane has been noted at the interface of the contact surface of the bedrock. This is supported in an article by Bengt H. Fellenius titled “Pile Foundation” in the Foundation Engineering Handbook, Second Edition, compiled by Hsai-Yang Fang Ph. D. that states:

> *“Normally, the neutral plane lies below the mid-point of a pile. The extreme case is for a pile on rock, where the location of the neutral plane is at the bedrock elevation. For a dominantly shaft-bearing pile “floating” in a homogenous soil with linearly increasing shear resistance, the neutral plane lies at a depth which is about equal to the lower third point of the pile embedment length.”*

9.1.3 Friction Piles

The use of friction piles is not recommended for this site, given that the existing piles are reportedly driven to the underlying bedrock and that any movement of the piles heads will result in differential settlement of the bridge structure.

9.1.4 Lateral Forces

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

Parameter	OPSS Granular B Type II	Native Sand	Silty Clay
Bulk Unit Weight (kN/m3)	21	18	20
Angle of Internal Friction, Φ	35^{o}	30^{o}	-
Coefficient of passive earth pressure	3.7	3.0	-
Design Undrained Shear Strength (kPa)	-	-	50

9.1.5 Lateral 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 4400 kN/m^3 for compact to loose sandy soils (Table 20.3, p. 315, of the Canadian Foundation Engineering Manual 1992)
z = depth below grade
d = pile diameter

9.1.6 Group Effects on Lateral Deflections

If piles are spaced at less than 8 pile diameters, center to center, parallel to the direction of lateral load, or less than 4 pile diameters, center to center, perpendicular to the lateral load, group effects will need to be considered and the lateral load at a specific deflection may need to be 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

$e_s = (e_B^2 \cos^2\alpha + e_p^2 \sin^2\alpha)^{½}$

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.

9.1.7 Tensile Resistance

Resistance to tensile loads should be calculated based on the shaft resistance of the piles in accordance with Section 6.8.5 of the CHBDC. The following factored ULS values may be used for design of piles in tension:

Pile Type	Location	Pile Length (m)	Pile size	Factored ULS Tensile Resistance (kN)
H-Piles	Abutment	45	HP310X110	650
	Piers	36	HP310X110	550
Circular Pipe Piles	Abutment	45	324 mm Diameter	600
	Piers	36	324 mm Diameter	440

A ULS resistance factor of 0.3 has been applied to the values provided above. The self weight of the piles has not been included in the ULS values provided above.

9.1.8 Piling Notes

Steel H-piles should be equipped with Type II reinforced flanges as per OPSD 3000.100. Steel pipe piles should be equipped with Type II Reinforced tip as per OPSD 3001.100.

The piles are anticipated to be approximately 35 m to 45 m in length, which will require the piles to be spliced during driving. Welded splices for steel H-piles should be in accordance with OPSD 3000.150. Welded splices for steel pipe piles should be in accordance with OPSD 3001.150.

Piles should be driven in accordance with OP903S01.

For semi-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".

9.2 Earth Pressure Design

To prevent hydrostatic pressure build-up, backfill against the abutments should consist of free draining granular materials. OPSS Granular A or OPSS Granular B, Type II is recommended. The zone of granular backfill must be constructed in accordance with Figure C6.9.1(I) (CHBDC Commentary) and OPSD 3501.000, using a frost penetration depth, f, of 1.2 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 Sand and Sand Fill	Fly Ash Fill*	Silty Clay
Unit Weight (kN/m^3)	22	21	18	14	20
Effective Angle of Internal Friction, Φ	35^o	35^o	30^o	30^o	26^o
Horizontal Backslope					
Coefficient of Active Earth Pressure, Ka	0.27	0.27	0.33	0.33	0.39
Coefficient of Passive Earth Pressure, Kp	3.69	3.69	3.0	3.0	2.56
Coefficient of Earth Pressure at Rest, Ko	0.43	0.43	0.5	0.5	0.56
2H:1V backslope					
Coefficient of Active Earth Pressure, Ka	0.39	0.39	0.54	0.54	0.17
Coefficient of Passive Earth Pressure, Kp	10.82	10.82	7.46	7.46	5.71

***Fly ash properties were obtained from the following references:**

Toth, P.S. et al. (1988) "Coal ash as structural fill with special reference to Ontario Experience" Canadian Geotechnical Journal Vol. 25, pp. 694 – 704.

Kim, B. et. al. (2005) "Geotechnical Properties of Fly and Bottom Ash Mixtures for Use in Highway Embankments" Journal of Geotechnical and Environmental Engineering, ASCE Vol 131 No. 4 pp. 914 – 924.

Leonards and Baily (1982) "Pulverized Coal Ash as Structural Fill", journal of Geotechnical and Environmental engineering, ASCE, Vol 108, No. 4, pp. 517 - 531.

9.3 Seismic Forces and Soil Profile Type

The zonal acceleration ration for Sarnia is 0.00 as per CHBDC Table A3.1.7.

It is recommended that Soil Profile IV as defined in CHBDC Section 4.4.6 be used in the seismic design of this site.

9.4 Embankment Design and Construction

The westbound embankment will be widened into the centre median of Highway 402. Based on the sections provided by Stantec, it is understood that approximately 2.5 m of fill will be required in the central median. It is also understood that the super elevation of the road will result in the inside shoulder of the west bound lanes being approximately 1.2 m higher than the inside shoulder of the east bound lanes with the grade separation achieved using a retaining wall.

It is recommended that normal weight fill such as Granular B Type I, Select Subgrade Material or approved borrow material be used for the planned widening.

However, the placement of the fill material will induce settlement of the existing embankments and the underlying soils. Calculations using the following consolidation parameter indicate that the widened embankment section may induce total settlements of the silty clay and the existing embankment material of approximately 80 mm near the centre of the widening, to 40 mm at the outer edge of the proposed widening.

Parameters*	Value
Initial Void Ratio	0.6
Compression Index, Cc	0.14 - 0.17
Re-Compression Index, Cr	0.035
Coefficient of Consolidation, Cv	5×10^{-7} m^2/sec

Note:
- The initial void ration is a theoretical value based on the soil moisture content
- the compression index and recompression index values are based on a review of correlations published in the “Manual on Estimating soil properties for foundation Design” by F. H. Kulhawy and P. W. Mayne; the results of the consolidation tests carried out at the nearby Christina Street/Highway 402 interchange; and comments provided in “Coal ash as structural fill with special reference to Ontario Experience” published in the Canadian Geotechnical Journal referenced herein.
- A Range of Compression Index values were used throughout the profile depth corresponding to the range of moisture contents measured with depth.
- Based on the consolidation test results at the Christina Street interchange and the similar soil and moisture content profile observed at this site, the soils were considered to be normally consolidated and therefore the compression index was used to calculate all settlement estimates.

It is anticipated that approximately 20 mm to 50 mm of the total settlement will occur in the existing embankment fills. This settlement should occur over a short period of time (during the construction period), given the granular nature of the material.

The longer term consolidation of the underlying silty clay is anticipated to be in the range of 20 mm to 30 mm.

Based on the coefficient of consolidation values anticipated for this site and assumed drainage path, the compression period is anticipated to be in the order of 2 to 12 months. It is noted that settlement monitoring carried out at the Front Street interchange and reported by Toth, P.S. et al. (1988) “Coal ash as structural fill with special reference to Ontario Experience” Canadian Geotechnical Journal Vol. 25, pp. 694 – 704, indicated the following:

- 295 mm settlement over a 10 month monitoring period; and,
- 2/3 of the settlement occurred within the first month after the placement of the fill.

Based on these observations, it is anticipated that most of the long term settlement will occur within the first month after placement of the fill.

Prior to placing the additional 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 in accordance with OPSS 501 and SP206S03.

The new embankment fill material should be benched into the existing embankments in accordance with OPSD 208.010.

Where slopes are practical, they should be constructed with 2H:1V slopes, and should be vegetated to prevent surface erosion. Maintenance will be required until the vegetative root mat is established on the slopes.

9.5 Retaining Structures

Given the super-elevation between the east and westbound embankments, it will be necessary to use a retaining structure to achieve the grade separation. It is understood that the preferred option is to use a retained soil system wall or a concrete retaining wall.

9.5.1 Retained Soil System (RSS)

A retained soil system is a mechanically reinforced soil retaining system that generally consists of granular fill placed and compacted in layers, and reinforced with fabric or metal strips or geogrid. The wall is faced to prevent loss of the fill material. The facing generally consists of either pre-cast concrete panels or decorative concrete blocks; both are mechanically fastened to the reinforcing strips or grids.

It is recommended that the RSS wall be founded on shallow spread footings provided the wall is able to accommodate the calculated settlements noted herein. The base of the RSS wall at this site will generally be founded in the sand and fly ash fill. Therefore, to provide a uniform base for the proposed retaining structure, it is recommended that the wall be constructed on a granular mat.

It is recommended that an additional 0.5 m of existing fill be removed and replaced with compacted structural fill. The base of the sub-excavation should be inspected and proof rolled prior to placing structural fill. The structural fill should consist of OPSS Granular B Type I placed in 200 mm loose lifts and compacted to 100% Standard Proctor Maximum Dry Density (SPMDD).

9.5.2 Geotechnical Resistance

The front face of an RSS wall is typically supported on a strip footing placed below the ground surface in front of the wall. The footing must be founded below any topsoil, loose fill or unsuitable native soils. For an assumed width of 0.6 m for the face footing and presuming that the footing is placed on the compacted granular sub-grade prepared in accordance with the recommendation herein, a factored geotechnical resistance at ULS of 300 kPa may be used for design. The ULS value includes a resistance factor of 0.5.

Presuming that the reinforced soil acts as a unit, and the width of the reinforced soil mass, is approximately 2/3 of the wall height, a factored geotechnical resistances at ULS of 300 kPa may be used for the design of the RSS wall founded on the prepared compacted structural fill. The ULS value includes a resistance factor of 0.5.

Settlement of the RSS wall including the reinforced mass and the concrete facing footings will occur as a result of the loading due to the embankment, since the walls are incorporated into the embankment. The geotechnical resistance at SLS of 100 kPa may be considered. The settlement of the underlying silty clay resulting from the combined RSS wall and the embankment widening, will likely be in the range of approximately 20 mm to 30 mm.

9.5.3 Resistance to Lateral Loads

The resistance to lateral loads between the compacted granular fill and the sub-grade should be calculated in accordance with section 6.7.5 of the CHBDC. The un-factored co-efficient of friction between the compacted granular fill of the RSS wall and the granular mat may be taken as 0.51.

9.5.4 Stability

We have carried out a global stability analysis using Slope/W a commercially available slope stability analysis program. A factor of safety of 1.3 was calculated, based on a 1.5 m high retaining wall reinforced with 3 layers of uniaxial geogrid reinforcement.

The internal stability of the retained soil wall is the responsibility of the RSS supplier / designer.

9.5.5 Concrete Retaining Walls

Alternatively, the retaining wall could be constructed using a conventional concrete retaining wall founded on shallow spread footings, provided the wall can accommodate the settlements noted herein. A conventional wall designed in accordance with the recommendations and using the geotechnical parameters provided herein would be subject to the same settlements caused by the embankment widening and grade increase.

10.0 CONSTRUCTION RECOMMENDATIONS

10.1 Open Cut Excavations

Earth excavation, if required, should be carried out in accordance with SP206S03. 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 any excavations below the anticipated water level at elevations of approximately 176 m to 177 m 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.

The construction should be subject to time constraints such that temporary excavations are open for no longer than 10 calendar days. 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.

10.2 Staging

Through discussions with representatives of Stantec, it is understood that Stage 1 will consist of the construction of the new bridge. Traffic will be diverted to the new bridge during the rehabilitation of the existing westbound structure, Stage 2. Once the rehabilitation work is complete, the existing rehabilitated westbound structure will be connected to the new widened structure, Stage 3, and the new widened bridge will be opened.

10.3 Shoring

Shoring may be required to support Highway 402 during sub-excavation and construction of the retaining system between the east and west bound lanes. Shoring will also be required along Front Street for the installation of the piles and pile caps at the pier locations.

It is recommended that shoring consist of soldier piles and lagging. The soldier piles will be installed in the underlying sand and clayey silt deposits.

The temporary shoring may be designed using the lateral earth pressure parameters provided in Section 9.2, entitled Earth Pressure Design.

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

10.4 Groundwater Control

It was not possible to measure the ground water level in the boreholes given the drilling techniques that were used. However, wet conditions were encountered in the boreholes at depths in the range of approximately 1.4 m to 8.7 m below existing grade, elevations in the range of approximately 176.4 m to 177.1 m. Therefore, excavations below an elevation of about 176.4 m to 177.1 m will likely encounter groundwater. In addition, excavations above this elevation, may encounter perched groundwater within the fill materials.

Given the soil conditions, seepage above the elevations noted above is anticipated to be slow and therefore should be readily handled by conventional sumps and pumping techniques. Excavations below the elevations noted above may be difficult given the presence of wet native sands. Therefore, some form of dewatering, in addition to conventional sumps and pumping techniques, will likely be required.

The following table provides an estimate of the hydraulic conductivity of the various soils encountered, based on the results of the grain size distribution tests and information provided in the Supplement to the Ontario Building Code 2006 edition:

Soil type	Hydraulic Conductivity (cm/sec)
Sand fill	10^{-3} to 10^{-5}
Fly Ash Fill	10^{-4} to 10^{-6}
Native Silt	10^{-5} to 10^{-6}
Native Sand	10^{-1} to 10^{-3}
Native Silty Clay	10^{-6} or less

10.5 Erosion control

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.

10.6 Frost Protection

The site is location in an area with a mean freezing index of between 250 and 500 Degree days (^{o}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 1.2 m.

10.7 Construction Monitoring

It is recommended that the existing bridge structure be monitored to ensure that the construction of the bridge widening does not adversely impact the existing structure. The monitoring program should include a visual assessment of the existing structure on a regular basis, along with vibration monitoring during the installation of the piles for the planned widening.

11.0 CLOSURE

Use of this report is subject to the Statement of General Conditions attached. It is the responsibility of Stantec Consulting Ltd. 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:

Raymond Haché, P.Eng., M. Sc., PMP
Principal, Geotechnical Service Director, and
Designated Principal
MTO Foundations Contact

GC/RH/dd

Enclosures

P:\CMiC Jobs\1010xxx\1012607\Front Street\Geotech Report - Front Street - MTO Submission - Final Report.doc

Appendix A

Drawings

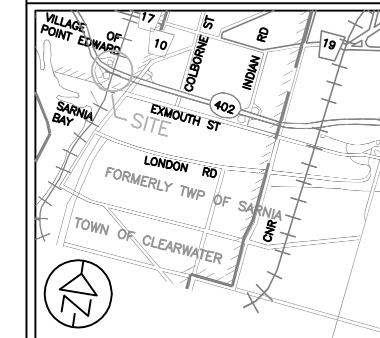
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

HWY 402
CONT No —
WP No 3038–03–00

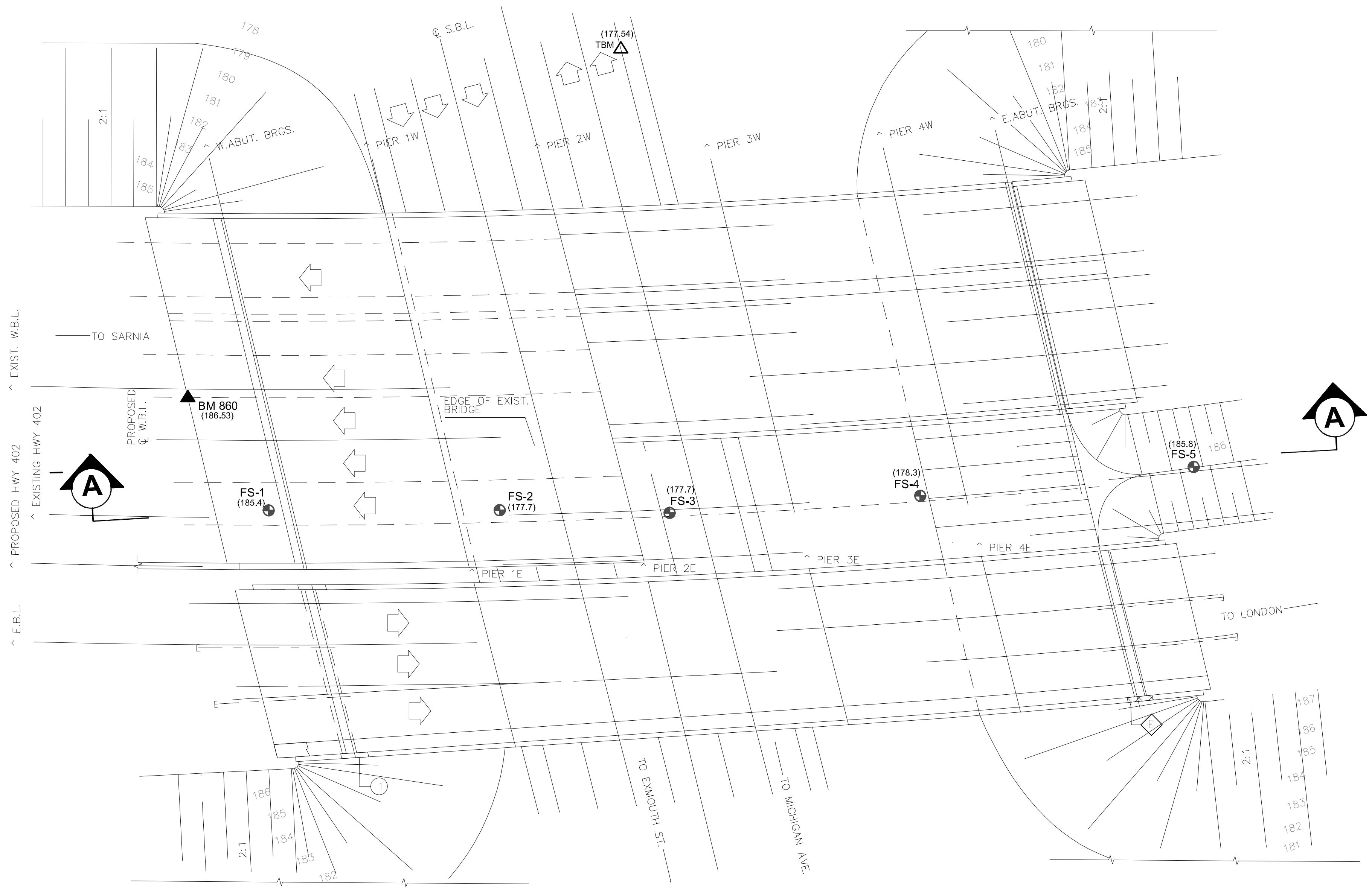
BORELOCATION PLAN
HIGHWAY 402 OVERPASS WIDENING AT FRONT STREET

SHEET

Jacques Whitford



KEY PLAN N.T.S



PLAN

LEGEND

- BOREHOLE (BY JACQUES WHITFORD)
- BM BENCH MARK
- TBM TEMPORARY BENCH MARK
- (178.3) ELEVATION (m)
- WL at time of investigation

No	ELEVATION (m)	COORDINATES	
		NORTHING	EASTING
FS–1	185.4	4 760 954.0	312 612.3
FS–2	177.7	4 760 941.8	312 639.0
FS–3	177.7	4 760 948.0	312 654.5
FS–4	178.3	4 760 934.7	312 670.6
FS–5	185.8	4 760 934.1	312 700.8

0 10 m 20 m

NOTES

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

NOTES: 1) 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.

2) Base plan provided by Stantec Consulting Ltd.

3) This drawing is for subsurface information only. Surface details and features are for conceptial illustration.

REVISIONS	DATE	BY	DESCRIPTION
	——	—	

GEOCRES No 40J16–79

HWY No 402			DIST LONDON
SUBM'D GC	CHECKED	DATE 2008–08–22	SITE — — — —
DRAWN PC	CHECKED	APPROVED	DWG 1012607–GEO–FS–01

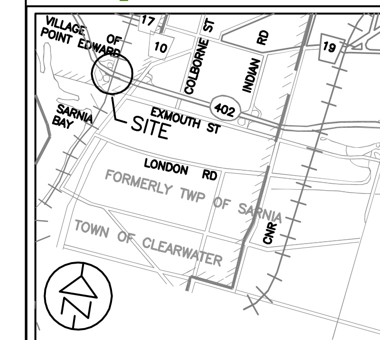
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

HWY 402
CONT No —
WP No 3038–03–00

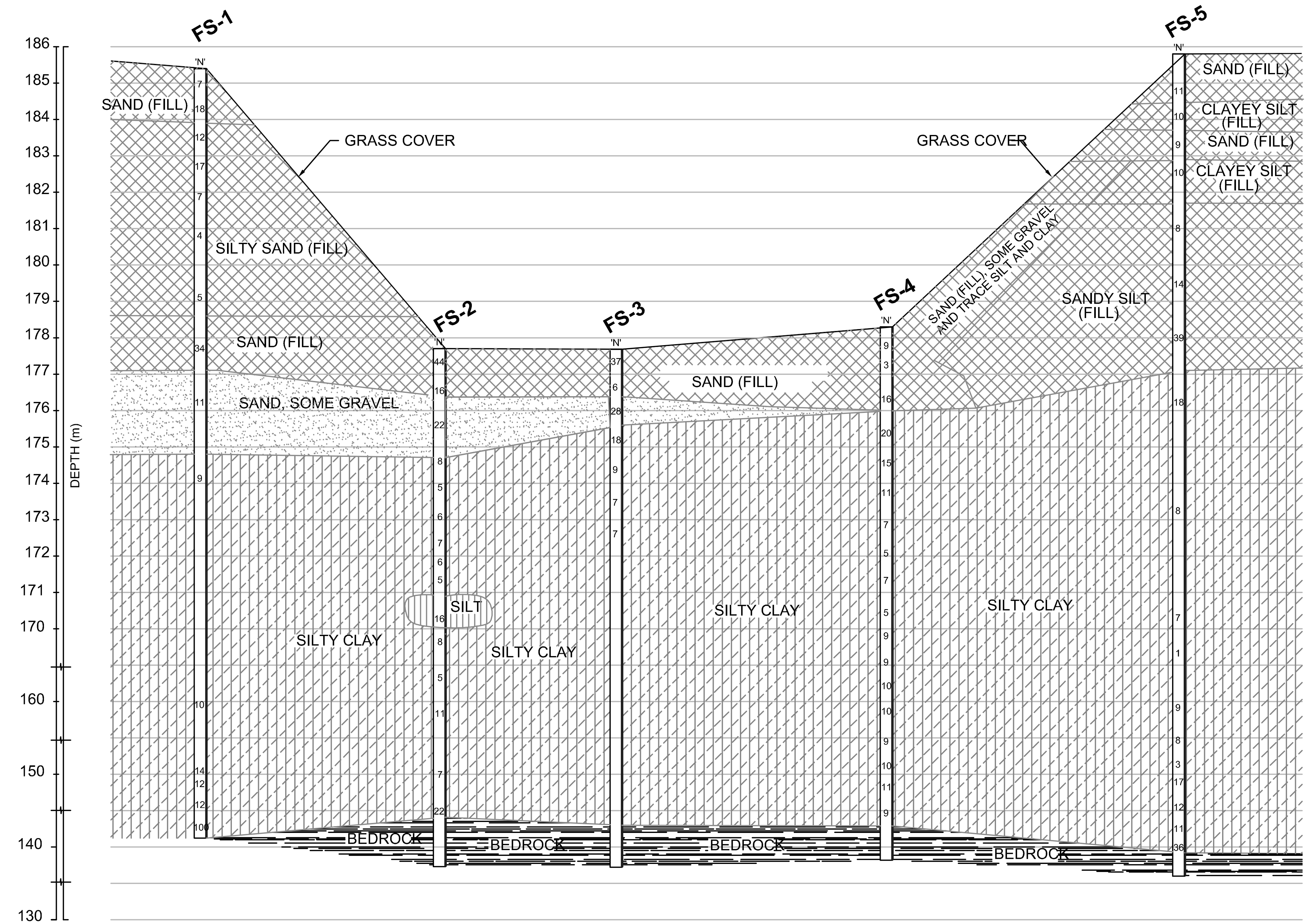
BOREHOLE LOCATION PLAN
HIGHWAY 402 OVERPASS
WIDENING AT FRONT STREET

SHEET

Jacques Whitford



KEY PLAN N.T.S



A-A CROSS SECTION

LEGEND

- BOREHOLE (BY JACQUES WHITFORD)
- BM BENCH MARK
- TBM TEMPORARY BENCH MARK
- (178.3) ELEVATION (m)
- WL at time of investigation

No	ELEVATION (m)	COORDINATES NORTHING	COORDINATES EASTING
FS–1	185.4	4 760 954.0	312 612.3
FS–2	177.7	4 760 941.8	312 639.0
FS–3	177.7	4 760 948.0	312 654.5
FS–4	178.3	4 760 934.7	312 670.6
FS–5	185.8	4 760 934.1	312 700.8

0 20m

NOTES

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

NOTES: 1) 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.

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REVISIONS DATE	BY	DESCRIPTION
--	-	

GEOCRES No 40J16–79

HWY No 402			DIST LONDON
SUBM'D GC	CHECKED	DATE 2008–08–22	SITE – – – –
DRAWN PC	CHECKED	APPROVED	DWG 1012607–GEO–FS–02

Appendix B

Terms and Symbols Used on the Record of Borehole Sheet

Record of Borehole Sheets

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3 m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR.*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Jacques Whitford

2006/2007 Investigation

Record of Borehole Sheets

Ministry of Transportation Ontario

RECORD OF BOREHOLE No FS-1

METRIC

W.P.	3038-03-00	LOCATION	Highway 402, Stn: 10+760 o/s C.L., N4760954.01, E312612.3
DIST	London HWY 402	BOREHOLE TYPE	Hollow Stem Auger, Mudrotary Casing, Split Spoon
DATUM	Geodetic	DATE	1.30.07 - 2.1.07
ORIGINATED BY	RM	COMPILED BY	OL
CHECKED BY	GC		

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: O UNCONFINED, ● QUICK TRIAXIAL, ✖ FIELD VANE, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
185.4 / 0.0	760 mm TOPSOIL, brown, wet	1	SS	7				
184.7 / 0.8	SAND (SM) (FILL), trace to some gravel, trace silt, brown, compact, moist	2	SS	18				
183.9 / 1.5	Silty SAND (SM) (FILL), some gravel, trace organics, with fly ash, brown to dark brown, compact to loose, moist	3	SS	12				
		4	SS	17				
		5	SS	7				
181.6 / 3.8	Silty SAND/Sandy SILT (SM) (FLY ASH), trace gravel, trace slag fragments, black, very loose to loose, moist	6	SS	4				
		7	SS	5	4	31	62	3
178.6 / 6.9	SAND (SP-SM) (FILL), trace to some gravel, brown, very dense, moist	8	SS	34				
177.1 / 8.4	SAND (SM), trace gravel, trace to some silt, brown, compact, wet	9	SS	11				
174.8 / 10.7	Silty CLAY (CL), trace sand, brown, stiff, moist	10	SS	9	3	17	46	34
		11	TW					

Elevation scale: 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No FS-1

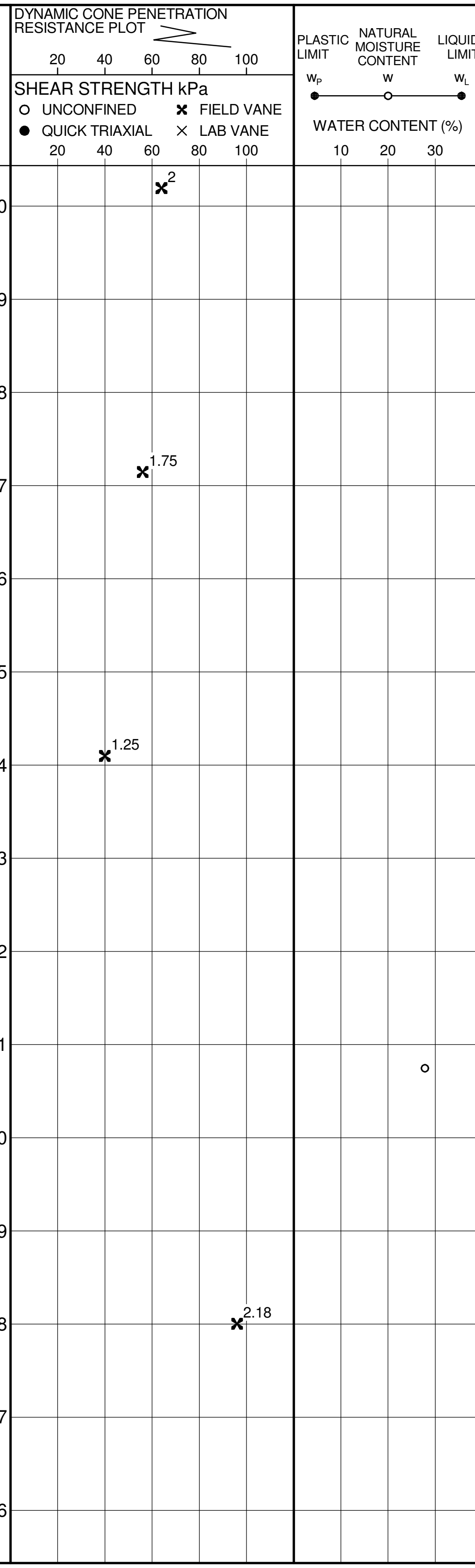
METRIC

W.P. 3038-03-00
LOCATION Highway 402, Stn: 10+760 o/s C.L., N4760954.01, E312612.3
ORIGINATED BY RM

DIST London HWY 402
BOREHOLE TYPE Hollow Stem Auger, Mudrotary Casing, Split Spoon
COMPILED BY OL

DATUM Geodetic
DATE 1.30.07 - 2.1.07
CHECKED BY GC

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	Silty CLAY (CL), trace sand, brown, stiff, moist *(continued)*				
		12	VT		170
					169
					168
		13	VT		167
					166
					165
		14	VT		164
					163
					162
		15	SS	10	161
					160
					159
		16	VT		158
					157
					156



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No FS-1

METRIC

W.P. 3038-03-00	LOCATION Highway 402, Stn: 10+760 o/s C.L., N4760954.01, E312612.3		ORIGINATED BY RM
DIST London HWY 402	BOREHOLE TYPE Hollow Stem Auger, Mudrotary Casing, Split Spoon		COMPILED BY OL
DATUM Geodetic	DATE 1.30.07 - 2.1.07		CHECKED BY GC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS GR SA SI CL
	Silty CLAY (CL), trace sand, brown, stiff, moist *(continued)*				155			
		17	TW					
					154			
					153			
					152	Field vane 1.54		
		18	VT					
					151			
					150			
					149			
		19	SS	14				
					148			
					147			
					146			
		20	SS	12			42	2 12 40 46
					145			
					144			
					143			
		21	SS	12				
					142			
141.2 44.2	END OF BOREHOLE at approximately 44.2 m on inferred bedrock.	22	SS	50 / 25 mm				

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, ● QUICK TRIAXIAL, ✖ FIELD VANE, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

RECORD OF BOREHOLE No FS-2

METRIC

W.P. 3038-03-00	LOCATION Front St. Stn.: 10+005 o/s: 9.0 m Rt, Twp of Sarnia; N4760941.8, E312639.0	ORIGINATED BY JP	
DIST London HWY 402	BOREHOLE TYPE Hollow Stem Auger, Tricone, Mudrotary Casing, Split Spoon, HQ Core Barrel	COMPILED BY MW	
DATUM Geodetic	DATE 11.17.06 - 11.23.06	CHECKED BY GC	

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
177.7	Front St. S.B. Rt. D.L.					
177.6 / 0.0	150 mm ASPHALT					
0.2	SAND (SW) (FILL), with gravel, trace silt and clay, wet Brown Dense	1	SS	44		
176.9 / 0.8	- brown and grey, some silt and clay, trace gravel compact	2	SS	16		
176.2 / 1.5	SAND (SM), some silt and clay, trace gravel, wet Brown Compact to loose	3	SS	22		1 70 (30)
		4	SS	8		
174.7 / 3.0	Silty CLAY (CL), with sand, some gravel, moist Brown Firm	5	SS	5	22.3	
		6	SS	6		
		7	SS	7		17 36 29 19
172.4 / 5.3	- grey, firm	8	SS	6		
		9	SS	5		
170.9 / 6.9	SILT (ML), with sand, trace gravel, wet Grey Compact	10	SS	16		1 26 64 9
169.3 / 8.4	Silty CLAY (CL), some sand, trace gravel, moist Grey Firm	11	SS	8		
		12	SS	5		

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164, 163); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

✖^{3}, ×^{3}: Numbers refer to Sensitivity ○$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No FS-2

METRIC

W.P.	3038-03-00	LOCATION	Front St. Stn.: 10+005 o/s: 9.0 m Rt, Twp of Sarnia; N4760941.8, E312639.0
DIST	London	HWY	402
BOREHOLE TYPE	Hollow Stem Auger, Tricone, Mudrotary Casing, Split Spoon, HQ Core Barrel		
DATUM	Geodetic	DATE	11.17.06 - 11.23.06
ORIGINATED BY	JP	COMPILED BY	MW
CHECKED BY	GC		

SOIL PROFILE: ELEV DEPTH | DESCRIPTION | STRAT PLOT — SAMPLES: NUMBER | TYPE | "N" VALUES — GROUND WATER CONDITIONS — ELEVATION SCALE — DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) — SHEAR STRENGTH kPa (○ UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100) — PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10, 20, 30 — UNIT WEIGHT γ kN/m³ — REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Description: Silty CLAY (CL), some sand, trace gravel, moist
Grey
Firm *(continued)*
- stiff

Sample Number	Type	Elevation scale (approx.)	Field vane annotation
13	VT	162	2.17
14	VT	159	2.1
15	VT	156	>100 kPa
16	VT	153	1.9
17	VT	150	2.1

Elevation scale: 162, 161, 160, 159, 158, 157, 156, 155, 154, 153, 152, 151, 150, 149, 148

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

RECORD OF BOREHOLE No FS-2

METRIC

W.P. 3038-03-00	LOCATION Front St. Stn.: 10+005 o/s: 9.0 m Rt, Twp of Sarnia; N4760941.8, E312639.0		ORIGINATED BY JP
DIST London HWY 402	BOREHOLE TYPE Hollow Stem Auger, Tricone, Mudrotary Casing, Split Spoon, HQ Core Barrel		COMPILED BY MW
DATUM Geodetic	DATE 11.17.06 - 11.23.06		CHECKED BY GC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa / DCPT	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty CLAY (CL), some sand, trace gravel, moist Grey Firm *(continued)*				147				
		18	VT			Field vane ≈78, sensitivity 1.5			
					146				
					145				
					144				
		19	VT			Field vane ≈90, sensitivity 3			
					143				
					142				
141.1 36.6	Weathered SHALE (BEDROCK)	20	SS	84/ 250mm	141		≈17		
140.3 37.4	TCR= 43% SCR= 23% RQD= 23%	21	HQ		140				
139.7 38.0	TCR= 100% SCR= 57% RQD= 0%	22	HQ						
139.4 38.3	TCR= 100% SCR= 89% RQD= 83%	23	HQ		139				
138.5 39.2	TCR= 89% SCR= 79% RQD= 60%	24	HQ		138				
137.3 40.5	END OF BOREHOLE at approximately 40.4 m								

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, ● QUICK TRIAXIAL, ✖ FIELD VANE, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30.

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

RECORD OF BOREHOLE No FS-3

METRIC

W.P. 3038-03-00	LOCATION Front St. Stn.: 9+995 o/s: 3.0 m Lt, Twp of Sarnia; N4760948.0, E312654.5	ORIGINATED BY JP
DIST London HWY 402	BOREHOLE TYPE Hollow Stem Auger, Tricone, Mudrotary Casing, Split Spoon, HQ Core Barrel	COMPILED BY MW
DATUM Geodetic	DATE 11.10.06 - 11.15.06	CHECKED BY GC

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH / WATER CONTENT	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
177.7	Front St. N.B. P.L.										
177.6 / 0.0	165 mm ASPHALT										
0.2	SAND (SW) (FILL), some gravel, trace silt and clay, trace asphalt fragement, moist; Brown; Dense	1	SS	37							
177.0 / 0.8	- 75 mm black fly ash at approximately 0.6 m below grade	2	SS	6	177						
176.4 / 1.4	- brown to dark brown, some silt and clay, some organic matter; SAND (SM), some silt and clay, trace gravel, wet; Brown; Compact	3	SS	28	176			1	82	(17)	
175.6 / 2.1	Silty CLAY(CL), trace to some sand, trace gravel, damp; Brown; Firm to very stiff	4	SS	18	175						
		5	SS	9		Wn = 17%	22.0	3	19	48	30
		6	SS	7	174						
173.0 / 4.7	- 300 mm sand seam at approximtaely 4.4 m below grade; - grey, firm below 4.7 m	7	SS	7	173						
					172						
	- some sand, trace gravel	8	VT		171	3.375					
		9	VT		170	1.88					
		10	VT		169, 168	1.89					
					167, 166						
		11	SS	9	165						
					164, 163						

Continued Next Page

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

RECORD OF BOREHOLE No FS-3

METRIC

W.P. 3038-03-00 LOCATION Front St. Stn.: 9+995 o/s: 3.0 m Lt, Twp of Sarnia; N4760948.0, E312654.5 ORIGINATED BY JP

DIST London HWY 402 BOREHOLE TYPE Hollow Stem Auger, Tricone, Mudrotary Casing, Split Spoon, HQ Core Barrel COMPILED BY MW

DATUM Geodetic DATE 11.10.06 - 11.15.06 CHECKED BY GC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (field vane)	WATER CONTENT	GR	SA	SI	CL
	- grey, firm below 4.7 m *(continued)*										
		12	VT		162	2.08					
					161						
					160						
		13	VT		159	1.5					
					158						
					157						
		14	VT		156	>100 kPa					
					155						
					154						
		15	VT		153	1.93					
					152						
					151						
	- with sand	16	SS	7	150			3	23	38	36
					149						
					148						

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, ● QUICK TRIAXIAL, FIELD VANE, LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

Continued Next Page

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

RECORD OF BOREHOLE No FS-3

METRIC

W.P. 3038-03-00	LOCATION Front St. Stn.: 9+995 o/s: 3.0 m Lt, Twp of Sarnia; N4760948.0, E312654.5	ORIGINATED BY JP
DIST London HWY 402	BOREHOLE TYPE Hollow Stem Auger, Tricone, Mudrotary Casing, Split Spoon, HQ Core Barrel	COMPILED BY MW
DATUM Geodetic	DATE 11.10.06 - 11.15.06	CHECKED BY GC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)
	- grey, firm below 4.7 m *(continued)*						
		17	VT		147	1.32 (field vane)	
					146		
					145		
		18	VT		144	2.14 (field vane)	
					143		
					142		
	- some sand -75 mm grey sand seam, trace gravel, trace silt and clay, wet	19	SS	22	141		
140.3 37.4	SHALE (BEDROCK) TCR= 15% SCR= 11% RQD= 0%	20	HQ		140		
139.8 37.9	TCR= 100% SCR= 99% RQD= 95%	21	HQ		139		
138.9 38.9	TCR= 64% SCR= 61% RQD= 63%	22	HQ				
138.3 39.5	TCR= 46% SCR= 24% RQD= 42%	23	HQ		138		
137.3 40.4	END OF BOREHOLE at approximately 40.4 m						

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10, 20, 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL; GROUND WATER CONDITIONS; STRAT PLOT.

✖^{3}, ×^{3}: Numbers refer to Sensitivity   O$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No FS-4

METRIC

W.P. 3038-03-00 LOCATION Front St. Stn.: 10+004 o/s: 23.0 m Lt, Twp of Sarnia; N4760934.7, E312670.6 ORIGINATED BY JP

DIST London HWY 402 BOREHOLE TYPE Hollow Stem Auger, Tricone, Mudrotary Casing, Split Spoon, HQ Core Barrel COMPILED BY MW

DATUM Geodetic DATE 11.1.06 - 11.8.06 CHECKED BY GC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
178.3	Front St. N.B. Rt. Shldr.				
0.0	TOPSOIL, with sand, some gravel, trace asphalt fragement, moist Very loose to loose Dark brown to black	1	SS	9	
		2	SS	3	
176.8 / 1.5	SAND (SM) (FILL), with silt and clay, trace gravel, wet Compact Grey	3	SS	16	8 64 (28)
176.0 / 2.3	Silty CLAY (CL), some sand, trace gravel, damp Very stiff Grey	4	SS	20	
		5	SS	15	
174.2 / 4.1	Silty CLAY (CL), some sand, trace gravel, wet Firm to stiff Grey	6	SS	11	
		7	SS	7	1 17 47 35
		8	VT		
		9	VT		
		10	SS	5	
		11	SS	7	
		12	SS	5	

Elevation scale: 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100
Shear Strength kPa: ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
Field vane sensitivity: 2.46, 1.45

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

✖^{3}, ×^{3}: Numbers refer to Sensitivity ○$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No FS-4

METRIC

W.P. 3038-03-00 LOCATION Front St. Stn.: 10+004 o/s: 23.0 m Lt, Twp of Sarnia; N4760934.7, E312670.6 ORIGINATED BY JP

DIST London HWY 402 BOREHOLE TYPE Hollow Stem Auger, Tricone, Mudrotary Casing, Split Spoon, HQ Core Barrel COMPILED BY MW

DATUM Geodetic DATE 11.1.06 - 11.8.06 CHECKED BY GC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, ✖ FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty CLAY (CL), some sand, trace gravel, wet Firm to stiff Grey *(continued)*						163				
			13	SS	9				Wn = 33%		1 15 42 43
							162				
							161				
							160				
			14	SS	9						
							159				
							158				
							157				
			15	SS	10						
							156				
							155				
							154				
			16	SS	10						
							153				
							152				
							151				
			17	SS	9						
							150				
							149				

Continued Next Page

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No FS-4

METRIC

W.P.	3038-03-00	LOCATION	Front St. Stn.: 10+004 o/s: 23.0 m Lt, Twp of Sarnia; N4760934.7, E312670.6	ORIGINATED BY	JP
DIST	London HWY 402	BOREHOLE TYPE	Hollow Stem Auger, Tricone, Mudrotary Casing, Split Spoon, HQ Core Barrel	COMPILED BY	MW
DATUM	Geodetic	DATE	11.1.06 - 11.8.06	CHECKED BY	GC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT)	GR	SA	SI	CL
	Silty CLAY (CL), some sand, trace gravel, wet Firm to stiff Grey *(continued)*								
		18	SS	10	41	1	10	37	51
		19	SS	11					
		20	SS	9					
140.2 38.1	SHALE (BEDROCK), slightly weathered Grey TCR=70% SCR=27% RQD=40%	21	HQ						
139.2 39.1	TCR=87% SCR=64% RQD=84%	22	HQ						
138.2 40.1	END OF BOREHOLE at approximately 40.1 m								

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (148, 147, 146, 145, 144, 143, 142, 141, 140, 139); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF BOREHOLE No FS-5

METRIC

W.P. 3038-03-00	LOCATION Highway 402, Stn.: 10+835 o/s: C.L., Twp of Sarnia; N4760934.1, E312700.8	ORIGINATED BY KH
DIST London HWY 402	BOREHOLE TYPE Hollow Stem Auger, Tricone, Split Spoon	COMPILED BY MW
DATUM Geodetic	DATE 12.12.06 - 12.19.06	CHECKED BY GC

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
185.8	Hwy 402 median Ditch							
0.0	Borehole daylighted to approximately 0.8 m below grade							
185.1 / 0.8	SAND (SP) (FILL), trace gravel, trace to some silt, black, compact, moist	1	SS	11				
184.4 / 1.4	Clayey SILT (CL) (FILL), some sand, trace gravel, some topsoil and organics, black, firm, moist	2	SS	10				
183.7 / 2.1	SAND (SP), trace silt, trace gravel, brown, loose, moist, black sand particles, moist	3	SS	9				
182.9 / 2.9	Clayey SILT (CL) (FILL), some sand, trace to some gravel, brown, moist, plant debris and trace black sand	4	SS	10				
181.7 / 4.1	Sandy SILT (SP-SM) (FILL), trace gravel, grey to black, loose to dense (fly ash)	5	SS	8				
		6A	SS	14	3	31	62	5
	- 50 mm clayey silt seam	6B						
	- some gravel	7	SS	39	16	74	9	1
177.1 / 8.7	Silty CLAY (CL), trace gravel, trace sand, wet Firm to stiff Brown	8	SS	18				
		9	SS	8	4	22	43	30

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No FS-5

METRIC

W.P. 3038-03-00 LOCATION Highway 402, Stn.: 10+835 o/s: C.L., Twp of Sarnia; N4760934.1, E312700.8 ORIGINATED BY KH

DIST London HWY 402 BOREHOLE TYPE Hollow Stem Auger, Tricone, Split Spoon COMPILED BY MW

DATUM Geodetic DATE 12.12.06 - 12.19.06 CHECKED BY GC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty CLAY (CL), trace gravel, trace sand, wet Firm to stiff Brown *(continued)*	10	SS	7	170	
169.1 16.8	- grey				169	
					168	
		11	TW		167	
		12	SS	1	166	1 12 45 42
					165	
		13	TW		164	
					163	
					162	
		14	VT		161	
					160	
					159	
		15	SS	9	158	
					157	
					156	

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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Continued Next Page

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

RECORD OF BOREHOLE No FS-5

METRIC

W.P. 3038-03-00 LOCATION Highway 402, Stn.: 10+835 o/s: C.L., Twp of Sarnia; N4760934.1, E312700.8 ORIGINATED BY KH

DIST London HWY 402 BOREHOLE TYPE Hollow Stem Auger, Tricone, Split Spoon COMPILED BY MW

DATUM Geodetic DATE 12.12.06 - 12.19.06 CHECKED BY GC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	- grey *(continued)*	16	SS	8	155			
					154			
					153			
	- soft	17	SS	3	152			
	- stiff	18	VT		151	1.67		
					150			
	- very stiff	19	SS	17	149			
					148			
					147			
	- stiff	20	SS	12	146			9 19 39 33
					145			
					144			
		21	SS	11	143			
					142			
					141			

Continued Next Page

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

RECORD OF BOREHOLE No FS-5

METRIC

W.P.	3038-03-00	LOCATION	Highway 402, Stn.: 10+835 o/s: C.L., Twp of Sarnia; N4760934.1, E312700.8	ORIGINATED BY	KH
DIST	London HWY 402	BOREHOLE TYPE	Hollow Stem Auger, Tricone, Split Spoon	COMPILED BY	MW
DATUM	Geodetic	DATE	12.12.06 - 12.19.06	CHECKED BY	GC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	- grey *(continued)*						
		22	SS	36		140	
	- with black sand seam at approximately 46 m below grade						
139.2 / 46.6	SHALE (BEDROCK) Grey Tricone approximately 3.2 m into the bedrock					139	
						138	
						137	
136.0 / 49.8	END OF BOREHOLE at approximately 49.8 m					136	

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3

✖^{3}, ×^{3}: Numbers refer to Sensitivity ○$^{3\%}$ STRAIN AT FAILURE

1977 Investigation

Record of Borehole Sheets

Jacques Whitford

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 1

WP 347-63-02/03 LOCATION Co-ords N 15 619 207 E 1 025 870 ORIGINATED BY PJS

DIST 1 HWY 402 BORING DATE April 28 1977 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER ELEV	REMARKS % GR SA SI CL
585.9	Ground Level 178.6m (0.0m)					
0.0	Sand Pockets of Silt and Gravel Very Loose				▼	9·77 (14)
		1	SS	2	580	
579.9 / 6.0	176.8m (1.8m) Clayey Silt Some Sand Trace of Gravel Occasional Sand Seams Very Stiff to Firm	2	SS	22		
		3	SS	32		0 24 46 30
		4	SS	16		
		5	SS	10	570	
		6	SS	11		
		7	SS	10	560	
		8	SS	9		
		9	SS	9	550	
		10	SS	31		
					540	
		11	SS	21		0 14 42 44
530.9 / 55.0	161.8m (16.8m) Silty Clay Trace of Sand				530	
		12	SS	12		
					520	
		13	SS	9		
					510	
		14	SS	12	500	
					490	
		15	SS	15		
481.9 / 104.0	146.9m (31.7m) Continued					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w

WATER CONTENT %: 10 20 30

UNIT WEIGHT γ

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS – ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 1 cont

WP 347-65-02/03 LOCATION Co-ords N 15 619 207 E 1 025 870 ORIGINATED BY PJS

DIST 1 HWY 402 BORING DATE April 28, 1977 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT % (10 20 30)	UNIT WEIGHT γ	REMARKS % GR SA SI CL
481.9	146.9m (31.7m) Continued									
104.0	Silty Clay Trace of Sand Stiff					480				
			16	SS	16	470				
462.9	141.1m (37.5m)									
123.0	Black Sand					460				
456.6	139.2m (39.4m)									
129.3	Black Shale Bedrock			BXL	77%					
450.9	137.4m (41.1m)		17	RC	Rec					
135.0	End of Borehole									

15 ○▲ % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION · ENGINEERING MATERIALS OFFICE · SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 2

WP 347-65-02/03	LOCATION Co-ords N 15 619 230 E 1 025 803	ORIGINATED BY PJS
DIST 1 HWY 402	BORING DATE May 5, 1977	COMPILED BY PJS
DATUM Geodetic	BOREHOLE TYPE Hollow Stem Augers	CHECKED BY

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH PSF: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 1000 2000

LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT %: 10 20 30

UNIT WEIGHT γ

REMARKS: % GR SA SI CL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEV	SHEAR STRENGTH	REMARKS
583.7	Ground Level 178.5m (0.0m)						
0.0	Sand Pockets of Silt and Gravel very Loose to compact	1	SS	2			Org 9.6% ; 0 88 (12)
		2	SS	7	580		0 76 (24)
577.7	176.1m (2.4m)	3	SS	26			2 30 39 29
8.0	Clayey Silt Some Sand Trace of Gravel Occasional Sand Seams Very Stiff to Firm	4	SS	26			
		5	SS	23			
		6	SS	12			
					570	>2200	
		7	SS	10		+s=1.8	
		8	SS	12		+s=1.6	
					560		
		9	SS	10		+s=2.0	
		10	SS	8		+s=1.7	
		11	SS	7	550	+s=1.7	
		12	SS	6		+s=2.1	
					540	+s=2.3	
		13	SS	10		+s=1.8	
529.7	161.5m (17.1m)				530	+s=1.6	
56.0	Silty Clay Trace of Sand Stiff	14	SS	12		+s=1.3	
					520		
					510		
					500		
					490		
481.7	Continued 146.8m (31.7m)						
104.0							

20 / 15 ○ 5 / 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 2 cont

WP 347-65-02/03	LOCATION Co-ords N 15 619 230 E 1 025 803	ORIGINATED BY FJS
DIST 1 HWY 402	BORING DATE May 5, 1977	COMPILED BY FJS
DATUM Geodetic	BOREHOLE TYPE Hollow Stem Auger	CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER / ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
481.7	Continued 146.8m (31.7m)									
104.0	Silty Clay Trace of Sand Stiff					480				
						470				
465.7 / 120.0	141.9m (36.6m) Black Sand									
460.7	140.4m (38.1m) Probably Bedrock		Refusal to Augering			460				
460.0 / 125.8	End of Borehole 140.2m (38.3m)									

20 15 ⌀ 5 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 3

WP 367-65-02/03 LOCATION Co-ords N 15 619 246 E 1 025 726 ORIGINATED BY PJS

DIST 1 HWY 402 BORING DATE May 6, 1977 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER ELEV	SHEAR STRENGTH PSF	WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
584.2	Ground Level 178.1m (0.0m)									
0.0	Sand Pockets of Silt and Sand Loose to Compact		1	SS	21	580				
			2	SS	12					
576.2 / 8.0	175.6m (2.4m) Clayey Silt Some Sand Trace of Gravel Occasional Sand Seams Very stiff to firm		3	SS	27					
			4	SS	16					
			5	SS	13	570	+ >2200			
			6	SS	8		+ s=1.7			
			7	SS	9		+ s=1.5			
			8	SS	9	560	+ s=1.7			
			9	SS	9		+ s=1.6			
			10	SS	10		+ s=1.9			
			11	SS	5	550	+ s=2.0			
			12	SS	11	540	+ s=2.3			
			13	SS	14		+ >2200			
531.2	161.9m (16.2m)		14	SS	12		+ s=2.3			
53.0	End of Borehole									

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH PSF: o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 1000 2000

LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w — WATER CONTENT % 10 20 30

15 ◇ 5 (20, 10) % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

HIGHWAY ENGINEERING DIVISION · ENGINEERING MATERIALS OFFICE · SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 4

WP 347-65-02/03 LOCATION Coords N 15 619 288 E 1 025 623 ORIGINATED BY PJS

DIST 1 HWY 402 BORING DATE May 2, 1977 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

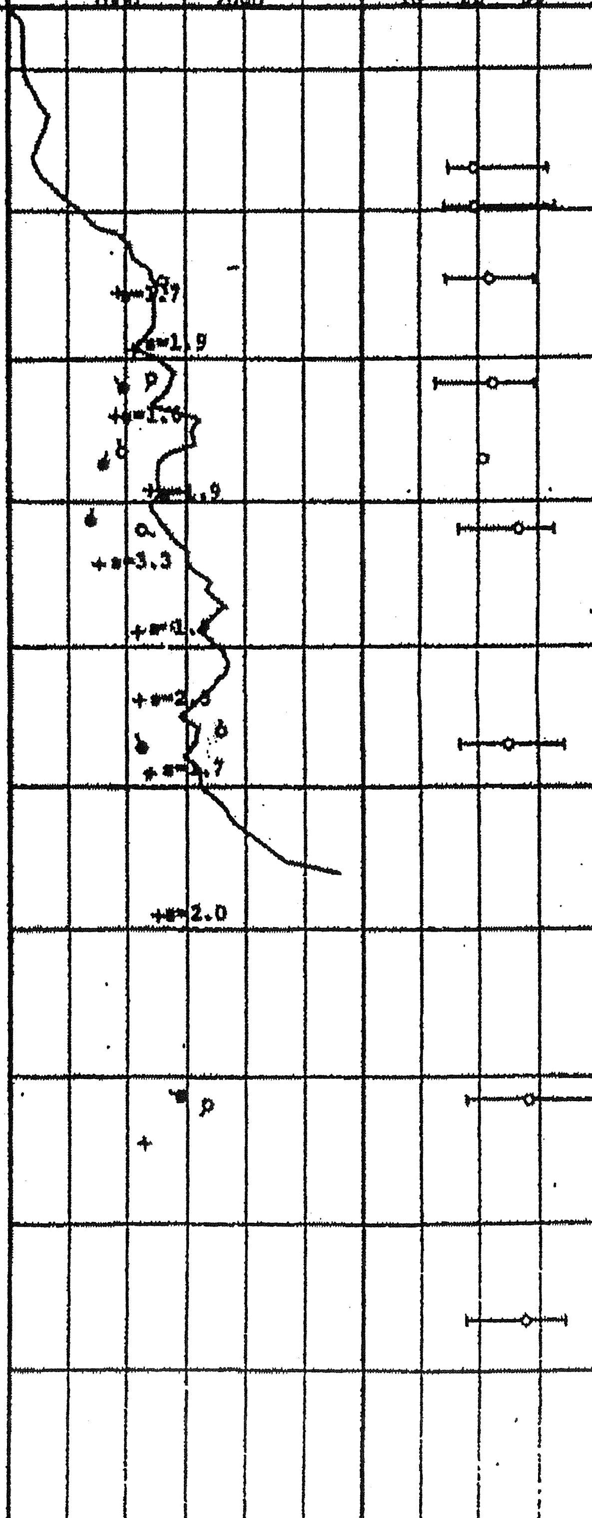
SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
584.2 / 0.0	Ground Level 178.1m (0.0m)							
	Sand Pockets of Silt and Gravel Loose		1	SS	7	580		
			2	SS	6			
574.2 / 10.0	175.1m (3.1m)							
	Clayey Silt Some Sand Trace of Gravel Occasional Sand Seams Very Stiff to Firm		3	SS	11			
			4	SS	10	570		
			5	TW	PH			
			6	TW	PH		130	
			7	SS	7	560		
			8	TW	PH		129	
			9	TW	PH		129	
			10	TW	PH	550	127 124	
			11	TW	PH	540		
			12	TW	PH			
			13	TW	PH		123 125	
530.2 / 54.0	161.6m (16.5m)					530		
	Silty Clay Trace of Sand Stiff		14	TW	PH	520		
			15	TW	PH	510	119	
						500		
			16	SS	12	490		
480.2 / 104.0	146.4m (31.7m) Continued							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH PSF: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 1000 2000

LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W

WATER CONTENT % 10 20 30



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS - ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No. 4cont

WP 347-65-02/03 LOCATION Co-ords N 15 619 288 E 1 025 623 ORIGINATED BY PJS

DIST 1 HWY 402 BORING DATE May 2, 1977 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT % 10 20 30	UNIT WEIGHT γ	REMARKS % GR SA SI CL
481.7	Continued 146.4m (31.7m)									
104.0	Silty Clay Trace of Sand Stiff		17	SS	14	480				
471.7	143.8m (34.7m)									
114.0	Black Sand					470				
464.2	141.5m (36.6m)		18	SS			Sample Disturbed			7 81 (12)
120.0	Black Shale Bedrock			BXL	100%	460				
456.3	139.1m (39.0m)		19	NC	Rec					
127.9	End of Borehole									

20 15 ○ 5 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION · ENGINEERING MATERIALS OFFICE · SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 6

WP 347-65-02/03 LOCATION Co-ords N 15 619 172 E 1 025 632 ORIGINATED BY PJS

DIST 1 HWY 402 BORING DATE May 10, 1977 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
584.4	Ground Level 178.1m (0.0m)						
0.0	Sand, Pockets of Silt and Gravel Loose to Compact	1	SS	6	580		
		2	SS	8			0 35 60 5
575.6 / 9.0	175.4m (2.7m) Clayey Silt Some Sand Trace of Gravel Occasional Sand Seams Very Stiff to Firm	3	SS	18			
		4	SS	16			
		5	SS	13	570		
		6	TW	PH		131 134	
		7	TW	PH		132	
		8	TW	PH	560	130	
553.4 / 31.0	168.7m (9.5m) End of Borehole	9	TW	PH			

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH PSF ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 1000 2000

+ s=1.5 + s=2.2 + s=1.8 + s=2.0

LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W

WATER CONTENT % 10 20 30

15 20 5 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 7

WP 347-65-02/03 LOCATION Co-ords N 15 619 156 E 1 025 689 ORIGINATED BY PJS

DIST 1 HWY 402 BORING DATE May 10, 1977 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	ELEV	SHEAR STRENGTH PSF (+ FIELD VANE)	WATER CONTENT %	REMARKS % GR SA SI CL
586.5	Ground Level 178.2m (0.0m)							
0.0	Sand, Pockets of Silt and Gravel Compact	1	SS	14	580			
		2	SS	10				
576.5	175.7m (2.4m)	3	SS	23			o	1 56 39 4
8.0	Clayey Silt Some Sand Trace of Gravle Occasional Sand Seams Very Stiff to Firm	4	SS	15			o	6 19 42 33
		5	SS	16				
		6	SS	11	570	+>2200	w_P–w–w_L	
		7	SS	9		+ s=1.6	o	
		8	SS	9		+ s=2.3	o	
		9	SS	19	560			
		10	SS	55				
		11	SS	12	550	+ s=2.0		
		12	SS	8		+ s=1.8		
		13	SS	7	540	+ s=1.8		
						+ s=1.7		
530.5	161.7m (16.5m)				530			
54.0	Silty Clay Trace of Sand Stiff	14	SS	15		+ s=1.5		
					520			
					510			
					500			
					490			
530.5 / 54.0	Continued 146.5m (31.7m)							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH PSF: o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 1000 2000

LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w; WATER CONTENT % 10 20 30

UNIT WEIGHT γ

15 20 5 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

HIGHWAY ENGINEERING DIVISION · ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 7 cont

W.P. 347-65-02/03 LOCATION Co-ords N 15 619 156 E 1 025 689 ORIGINATED BY PJS

DIST 1 HWY 402 BORING DATE May 10, 1977 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W / WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES					
480.5	146.5m (31.7m) Continued									
104.0	Silty Clay Trace of Gravel Stiff					480				
						470				
464.5 / 120.0	141.6m (36.6m) Black Sand									
459.5 / 125.0	140.1m (38.1m) End of Borehole Probable Bedrock					460				
	NOTE Water Level not established									

15 ⌀ 5 % STRAIN AT FAILURE (20)

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION · ENGINEERING MATERIALS OFFICE · SOIL MECHANICS SECTION 19.

RECORD OF BOREHOLE № 8

WP 347-65-02/03 LOCATION Co-ords N 15 619 135 E 1 025 792 ORIGINATED BY PJS

DIST 1 HWY 402 BORING DATE May 9, 1977 COMPILED BY PJS

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH PSF (1000 2000) ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L / PLASTIC LIMIT W_P / WATER CONTENT W, WATER CONTENT % (10 20 30)	UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
585.8	178.6m (0.0m) Ground Level									
0.0	Sand, Pockets of Silt and Gravel Loose 176.7m (1.8m)		1	SS	6	580				1 38 56 5
579.8 / 6.0	Clayey Silt Some Sand Trace of Gravel Occasional Sand Seams Very Stiff to Firm		2	SS	19					
			3	SS	26					6 27 41 26
			4	SS	27					
			5	SS	18					
			6	SS	12	570				
			7	TW	PH		s=1.3		131	
							s=1.7			
			8	TW	PH		s=1.4		131	
			9	TW	PH	560	s=1.8		129	
			10	TW	PH		s=1.8			
552.8 / 33.0	168.5m (11.6m) End of Borehole									

20 15 ◇ 5 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 9

WP 347-65-02/03 | LOCATION Co-ords N 15 619 135 E 1 025 864 | ORIGINATED BY FJS

DIST 1 HWY 402 | BORING DATE May 9, 1977 | COMPILED BY FJS

DATUM Geodetic | BOREHOLE TYPE Hollow Stem Auger | CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH PSF ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 1000 2000	LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W / WATER CONTENT % 10 20 30	UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEV				
586.5	Ground Level 178.8m (0.0m)								
0.0	Sand, Pockets of Silt and Gravel Compact								
	176.6m (2.1m)	1	SS	12	580				
579.5 / 7.0	Clayey Silt, Some Sand Trace of Gravel Occasional Sand Seams Very Stiff To Firm	2	SS	28					
		3	SS	30					
		4	SS	16					
		5	TW	PH	570			131	
		6	TW	PH		+s=2.0		132	
		7	SS	10		+s=1.6			
		8	TW	PH	560	+s=1.7		131	
		9	SS	10		+s=1.6			
		10	SS	10	550	+s=1.7			
563.5	165.7m (13.1m)	11	TW	PH		+s=1.7			
43.0	End of Borehole								

20
15 ○ 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION • ENGINEERING MATERIALS OFFICE • SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 11

WP 347-65-02/03 LOCATION Co-ords N 15 619 257 E 1 025 649 ORIGINATED BY PJS

DIST 1 HWY 402 BORING DATE May 11, 1977 COMPILED BY PJS

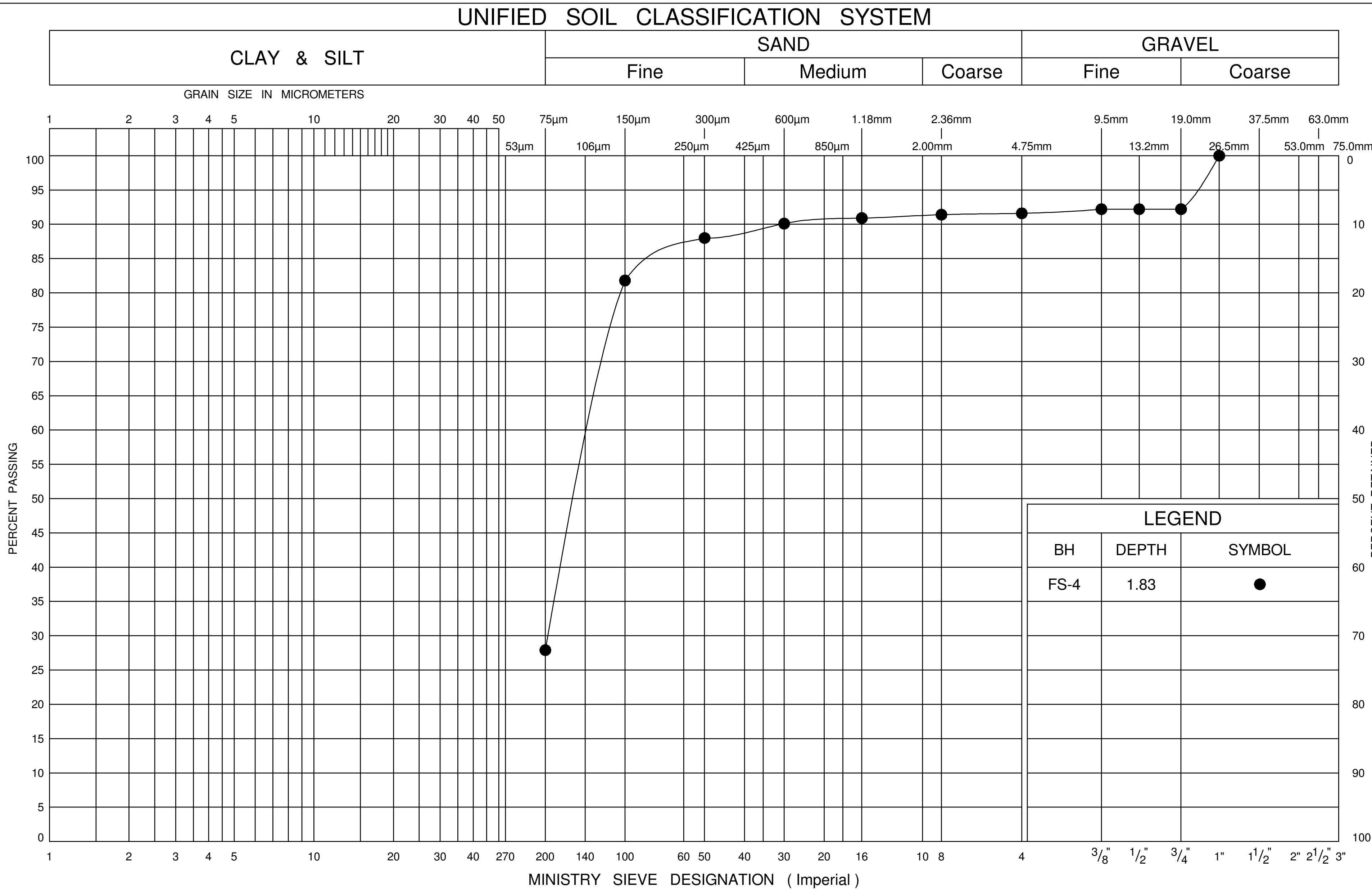
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT. PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH PSF (1000 2000) o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w – WATER CONTENT % (10 20 30)	UNIT WEIGHT γ	REMARKS % GR SA SI CL
585.5 / 0.0	178.5m (0.0m) Ground Level									
	Sand, Pockets of Silt and Gravel Loose to Compact		1	SS	5	580				
576.5 / 9.0	175.7m (2.7m)		2	SS	13	▽				6 68 23 3
	Clayey Silt Some Sand Trace of Gravel Occasional Sand Seams Very Stiff to Firm		3	SS	13					
			4	SS	11	570	+ s=2.2			
			5	SS	9		+ s=2.2			
			6	SS	9		+ s=2.0			
			7	SS	8	560	+ s=2.2			
557.5 / 28.0	169.9m (8.5m) End of Borehole		8	SS	8		+ s=1.7			

20
15 ◇ 5 % STRAIN AT FAILURE

Appendix C

Geotechnical Laboratory Test Results



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

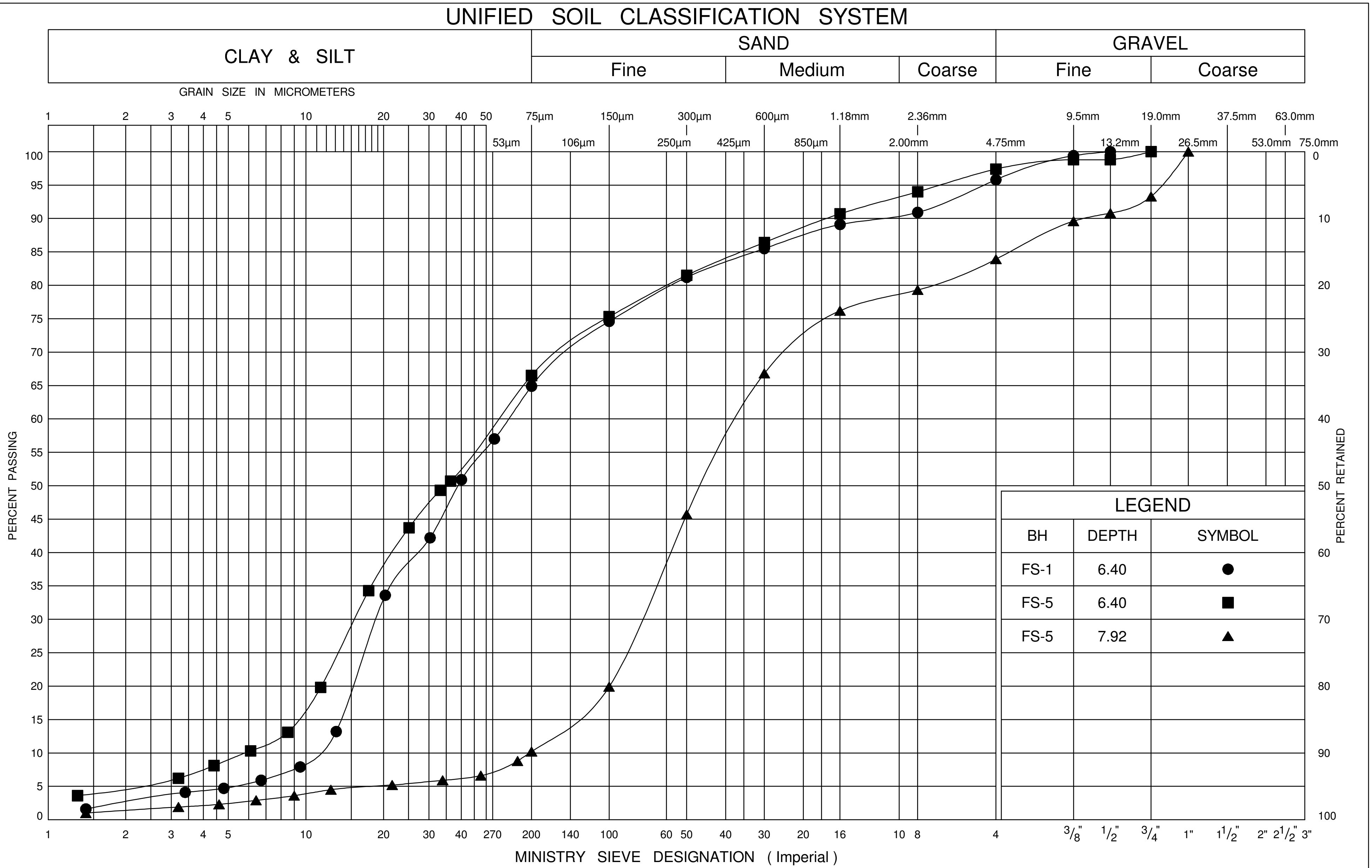
SAND (Fill)

FIG No 1

W P 3038-03-00

Hwy 402, Township of Sarnia

78 12 M



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GRAIN SIZE DISTRIBUTION

Fly and Bottom Ash

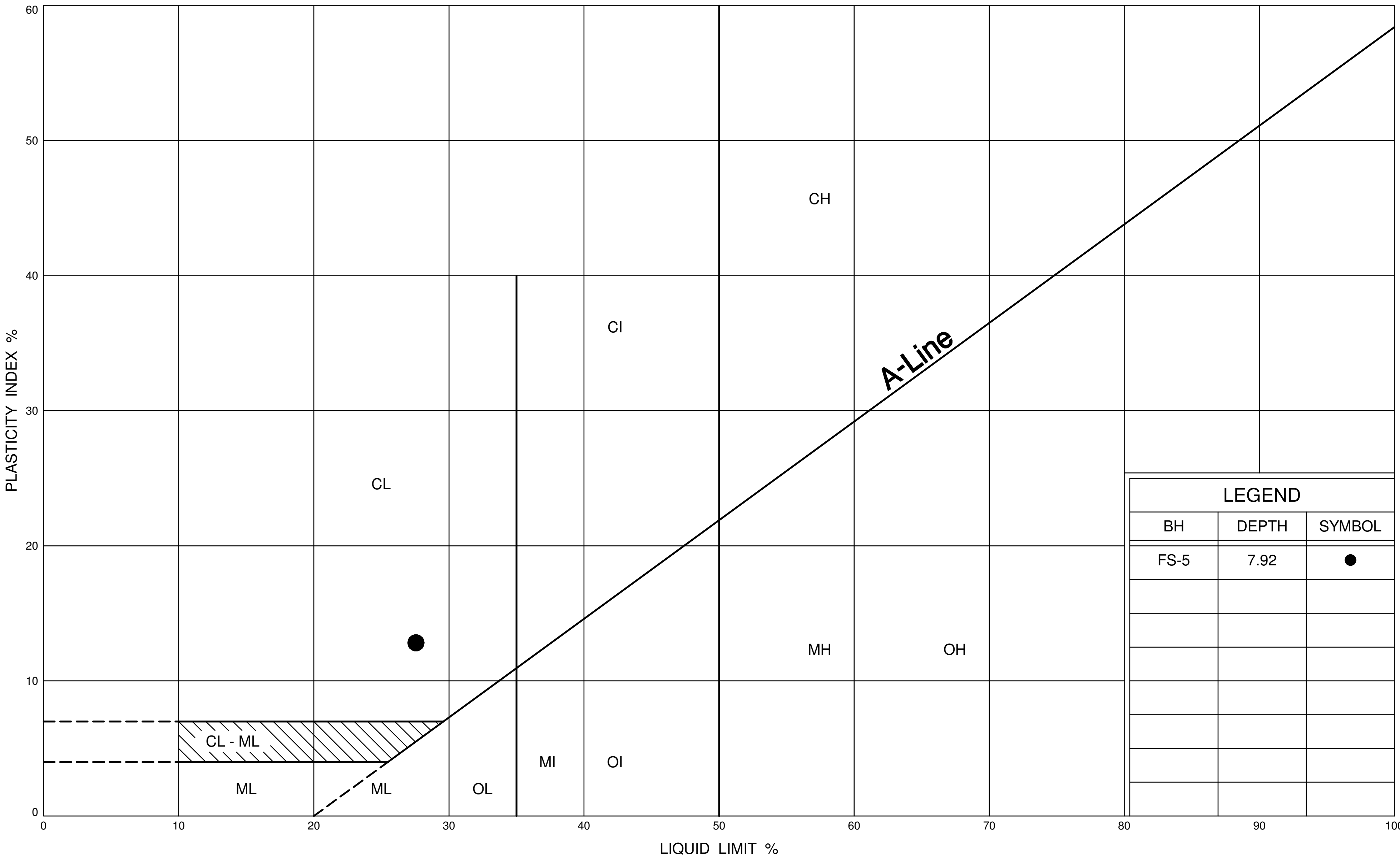
FIG No 2

W P 3038-03-00

Hwy 402, Township of Sarnia

ONTARIO MOT GRAIN SIZE 1012607.GPJ ONTARIO MOT.GDT 11/19/08

Oct 75, FF - S - 21



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PLASTICITY CHART

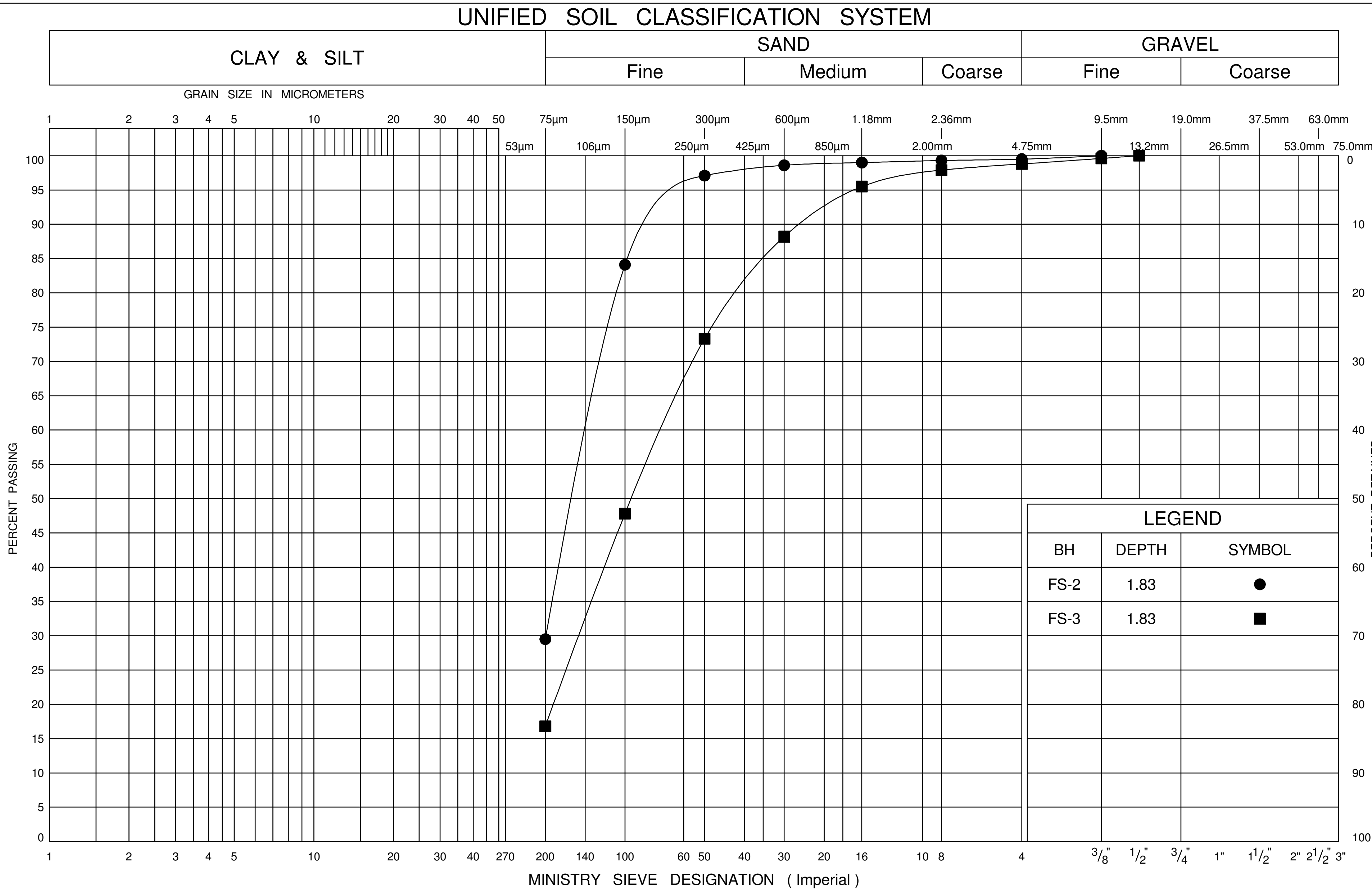
Fly and Bottom Ash

FIG No 3

W P 3038-03-00

Hwy 402, Township of Sarnia

ONTARIO MOT PLASTICITY CHART 1012607.GPJ ONTARIO MOT.GDT 11/19/08



Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

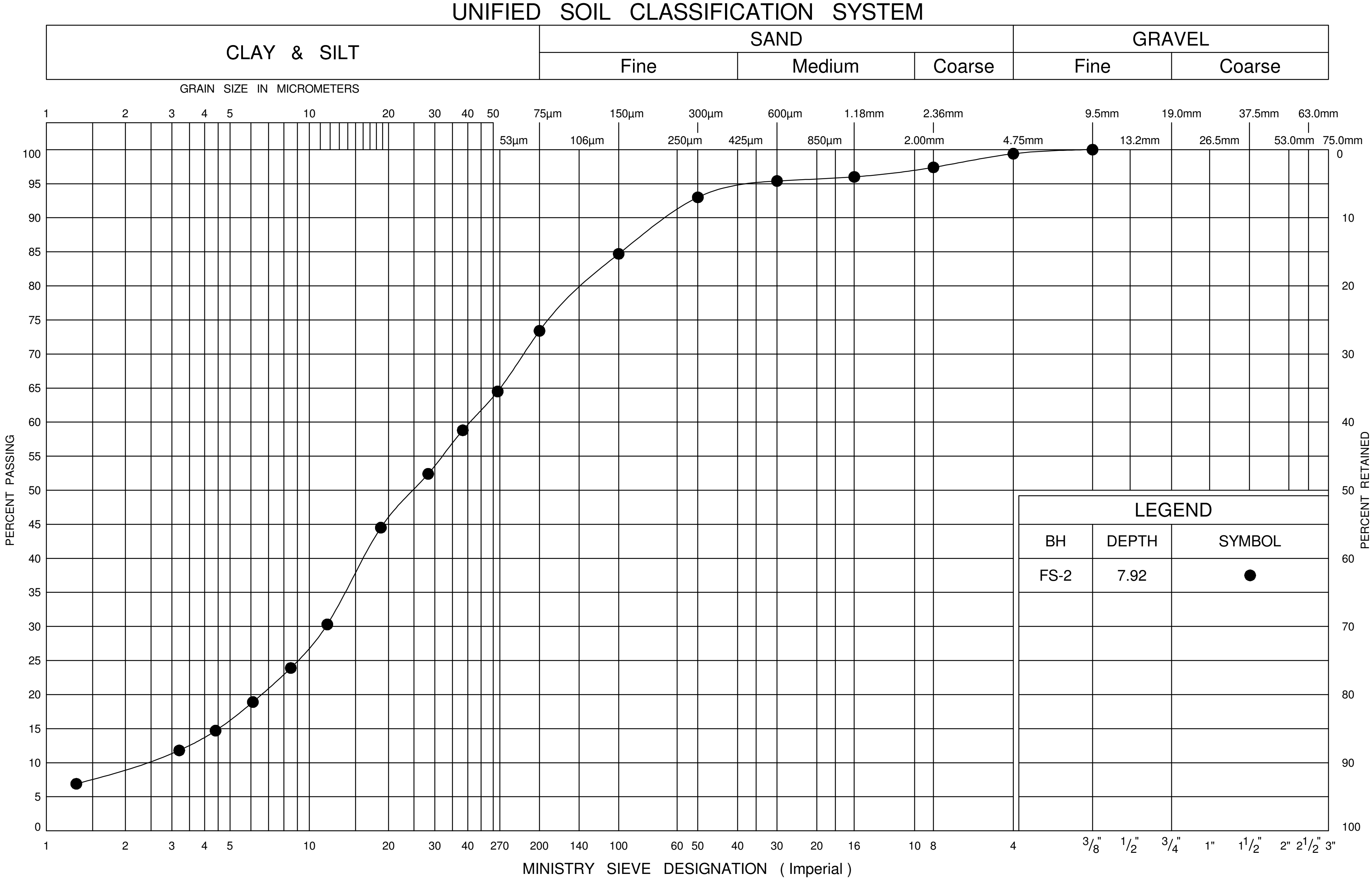
Native SAND

FIG No 4

W P 3038-03-00

Hwy 402, Township of Sarnia

78 12 M



ONTARIO MOT GRAIN SIZE 1012607.GPJ ONTARIO MOT.GDT 11/19/08

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

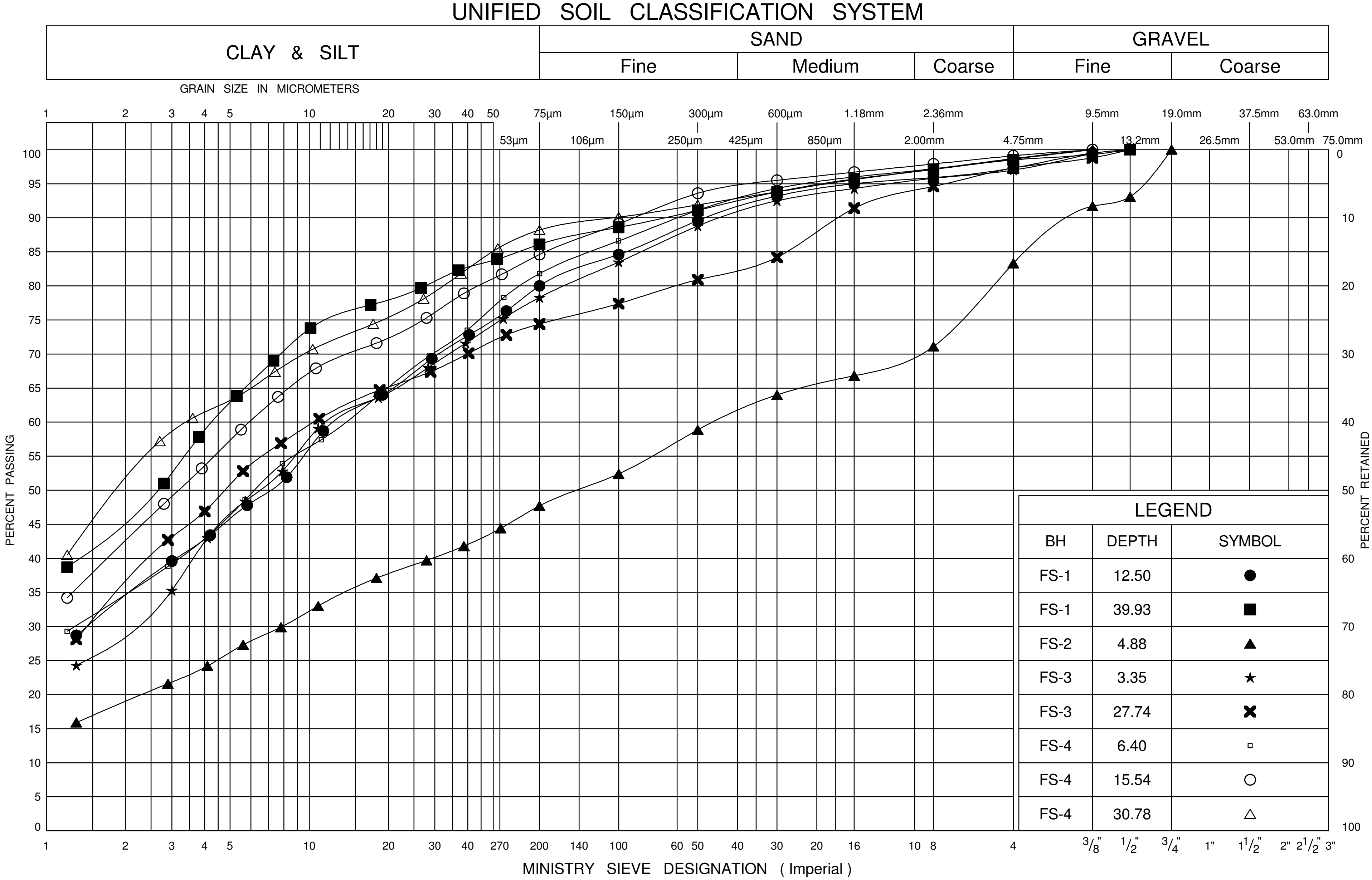
SILT

FIG No 5

W P 3038-03-00

Hwy 402, Township of Sarnia

78 12 M



LEGEND

BH	DEPTH	SYMBOL
FS-1	12.50	●
FS-1	39.93	■
FS-2	4.88	▲
FS-3	3.35	★
FS-3	27.74	✖
FS-4	6.40	▫
FS-4	15.54	○
FS-4	30.78	△

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Silty CLAY, some sand, trace gravel

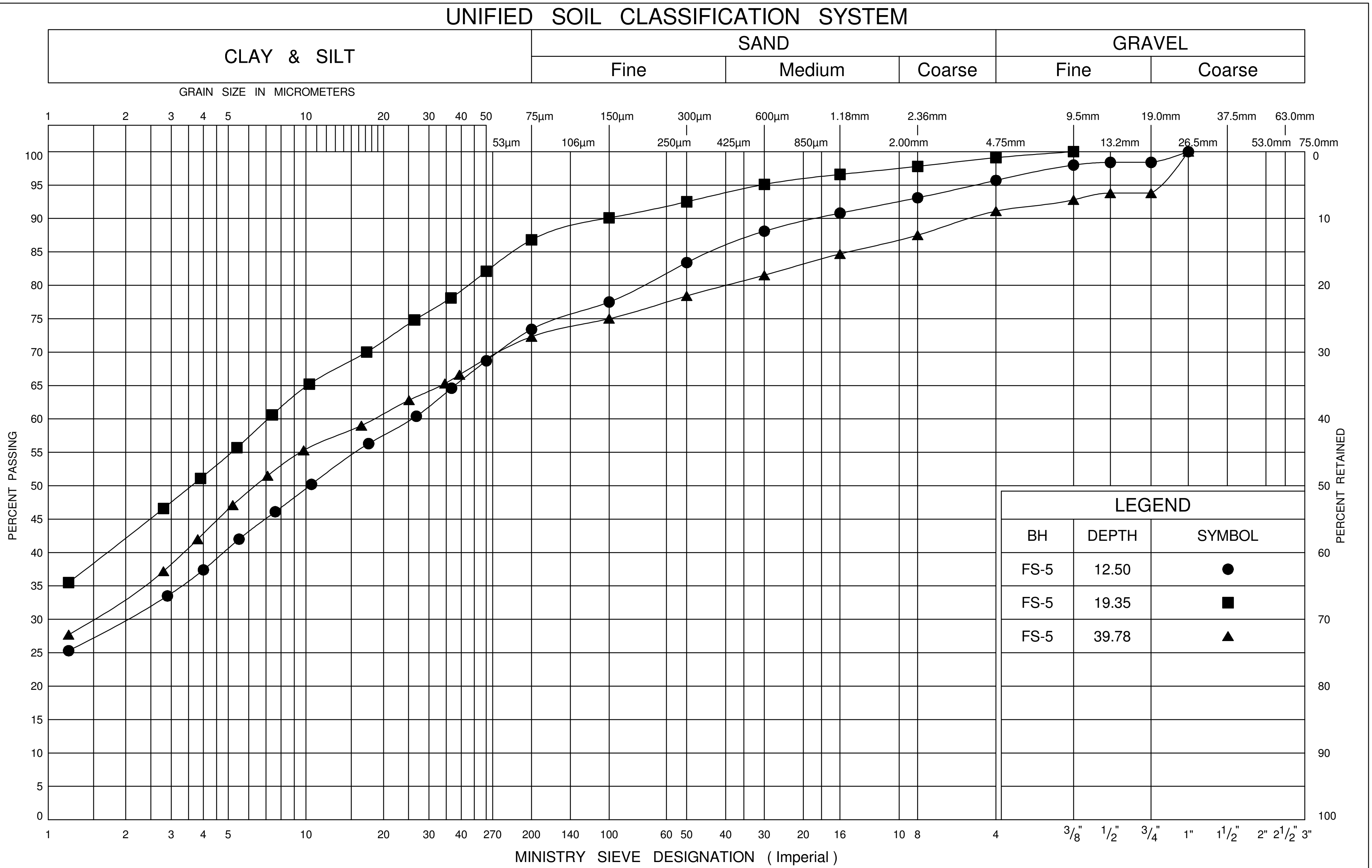
FIG No 6

W P 3038-03-00

Hwy 402, Township of Sarnia

ONTARIO MOT GRAIN SIZE 1012607.GPJ ONTARIO MOT.GDT 11/19/08

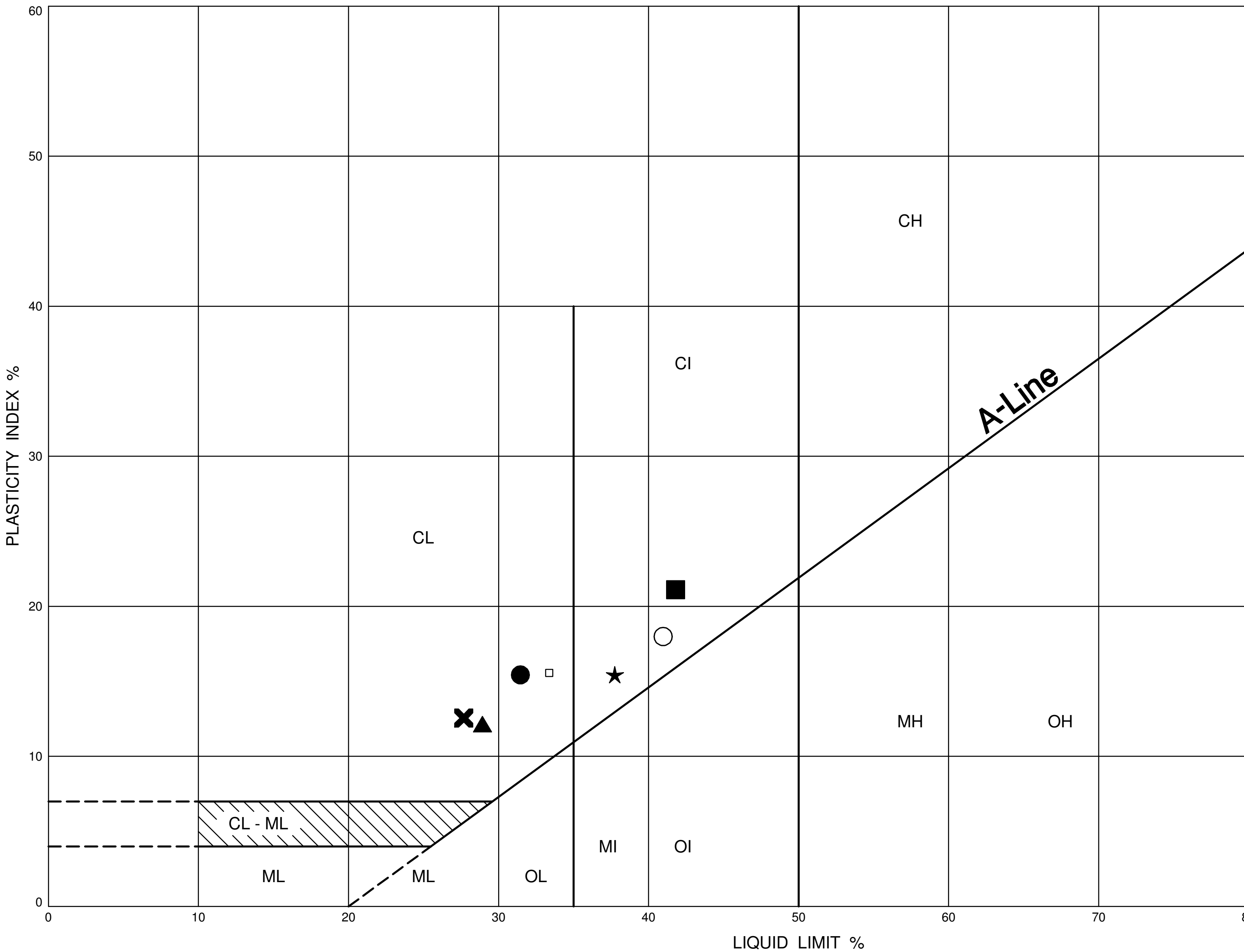
78 12 M



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION	FIG No 7
	Silty CLAY, some sand, trace gravel	W P 3038-03-00
		Hwy 402, Township of Sarnia

ONTARIO MOT GRAIN SIZE 1012607.GPJ ONTARIO MOT.GDT 11/19/08

Oct 75, FF - S - 21



LEGEND		
BH	DEPTH	SYMBOL
FS-1	12.50	●
FS-1	39.93	■
FS-3	3.35	▲
FS-3	27.74	★
FS-4	6.40	✖
FS-4	15.54	▫
FS-4	30.78	○

Ministry of Transportation
Ontario

PLASTICITY CHART

Silty CLAY, some sand, trace gravel

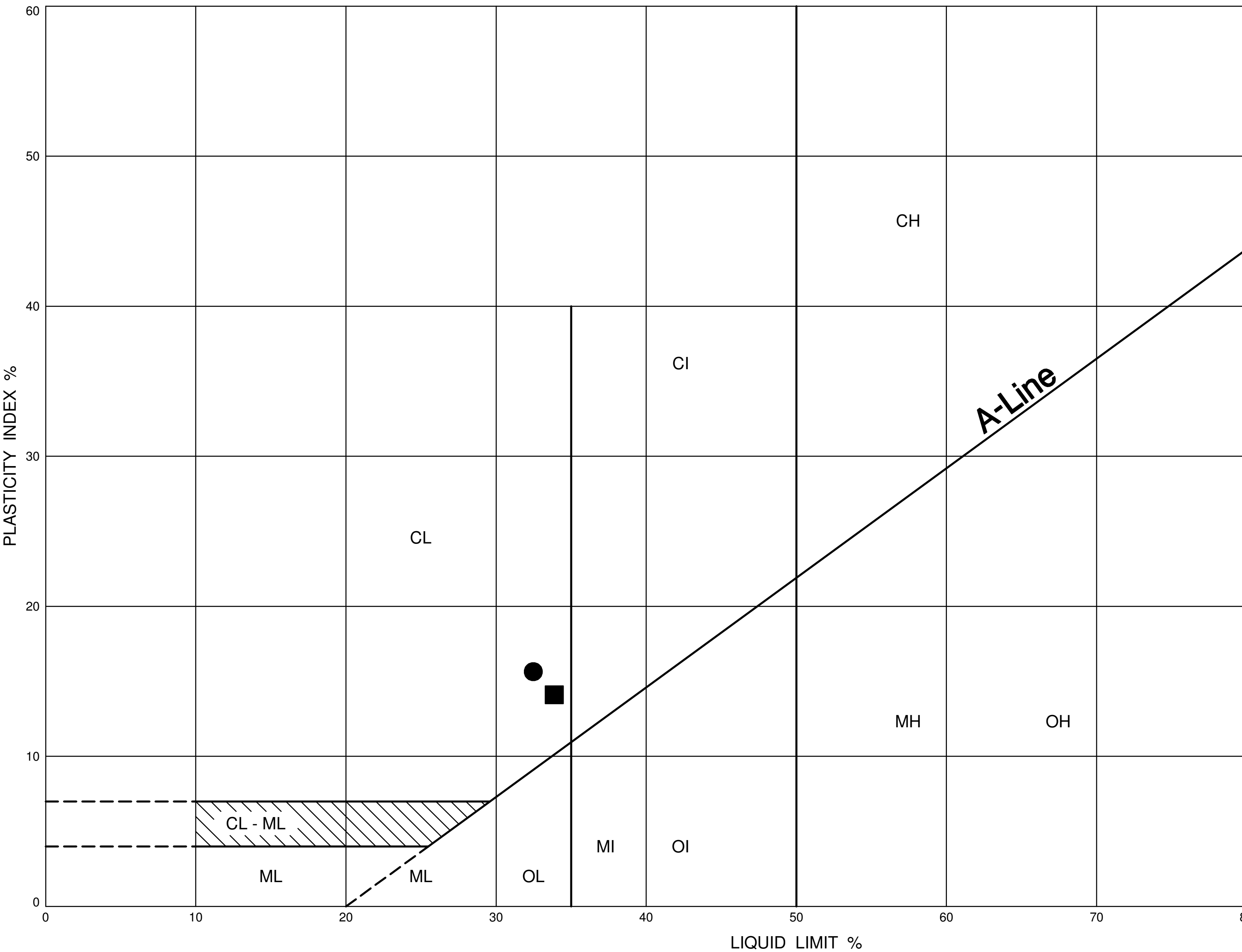
FIG No 8

W P 3038-03-00

Hwy 402, Township of Sarnia

ONTARIO MOT PLASTICITY CHART 1012607.GPJ ONTARIO MOT.GDT 11/19/08

Oct 75, FF - S - 21



LEGEND		
BH	DEPTH	SYMBOL
FS-5	19.35	●
FS-5	39.78	■

Ministry of Transportation
Ontario

PLASTICITY CHART

Silty CLAY, some sand, trace gravel

FIG No 9

W P 3038-03-00

Hwy 402, Township of Sarnia

ONTARIO MOT PLASTICITY CHART 1012607.GPJ ONTARIO MOT.GDT 11/19/08

Jacques Whitford Limited

7271 Warden Ave,
Markham, Ontario
L3R 5X5
Tel: (905) 474 -7700
Fax: (905) 479-9326

Density/Bulk Unit Weight Of Soil Specimen

Figure: 10

Location: Highway402, Sarnia, Ontario. **Project No.: 1012607**

Date Sampled: 01 Dec. 2006 **Date Tested: 20 Dec. 2006** **Tested By: HW**

	Unit		1	2	3
Borehole No.			BH(FS2-5)	BH(FS3-5)	BH(FS4-5)
Weight of soil specimen in air	*gms*	**A**	**193.2**	**153.4**	**123.7**
Weight of soil specimen in liquid (oil)	*gms*	**B**	**119.0**	**93.7**	**74.6**
Mass of Liquid displaced	*gms*	**(A-B)**	**74.2**	**59.7**	**49.1**
Specific Gravity of Liquid (oil)		γ_L	**0.8714**	**0.8714**	**0.8714**
Density of soil sample	*Kg/m³*	$D = 1000A*\gamma_L/(A-B)$	**2269**	**2239**	**2195**
Unit Weight of soil sample	*KN/m³*	$U = D*0.009807$	**22.3**	**22.0**	**21.5**

Jacques Whitford Limited

7271 Warden Ave,
Markham, Ontario
L3R 5X5
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Rock Core Compressive Strength Test Report

Figure: 11

Location: Highway 402, Sarnia, Ontario

Project No.: 1012607

Core Number	FS-2	FS-3	FS-4
Average Height (mm)	58.84	70.75	91.85
Average Diameter (mm)	47.18	47.17	47.17
H/D Ratio	1.247	1.500	1.947
Correction Factor	0.929	0.960	0.996
Compressive Strength (MPa)	102.2	101.1	99.2
Corrected Compressive Strength (MPa)	94.7	97.1	98.8

Appendix D

Representative Site Photographs

Photo 1: Front Street / CNR Overpasses - May 2006

Photo 2: Font Street / CNR Overpasses - July 2006