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FOUNDATION INVESTIGATION AND DESIGN REPORTS, PROPOSED WATERMAIN INSTALLATION, HIGHWAY 405 ENCROACHMENT AT ST. PAUL AVENUE, TOWN OF NIAGARA-ON-THE LAKE, ONTARIO, PROJECT NO. 091854

Prepared for: R.V. Anderson Associated Ltd., St. Catharines

GEOTBURL01244AA
August 6, 2010

Coffey Geotechnics Inc.
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SPECIALISTS MANAGING THE EARTH

August 6, 2010

R.V. Anderson Associates Ltd.
1 St. Paul Street, Suite 702
St. Catharines, ON L2R 7L2

Attention: Mr. Vince Grande, P.Eng.

Dear Sir:

RE: Foundation Investigation and Design Report, Proposed Watermain Installation, Highway 405 Encroachment at St. Paul Avenue, Town of Niagara-on-the-Lake, Ontario, Project No. 091854

Please find attached our Foundation Investigation and Design Report for the above mentioned project site. This report addresses the geotechnical investigation undertaken and provides an evaluation and assessment of the proposal to install, by open cut, a 300 mm diameter watermain. This watermain will be encased in a 500 mm diameter steel pipe along St. Paul Avenue, and will be in close proximity to the footing foundations of the west piers of the two Hwy 405 Overpass structures.

We trust that the foregoing information provided is satisfactory for your purpose. If you have any questions or require any clarification, please do not hesitate to contact us.

For and on behalf of Coffey Geotechnics Inc.

Levent Ozcan, P.Eng.
Geotechnical Engineer

Attachment: Foundation Investigation and Design Report, Proposed Watermain Installation, Highway 405 Encroachment at St. Paul Avenue, Niagara-on- the-Lake, Ontario, Project No. 091854

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FOUNDATION INVESTIGATION REPORT

PROPOSED WATERMAIN INSTALLATION

HIGHWAY 405 ENCROACHMENT AT ST. PAUL AVENUE

TOWN OF NIAGARA-ON-THE-LAKE, ONTARIO

1 INTRODUCTION

This report presents the results of a geotechnical investigation commissioned by R.V. Anderson Associates Ltd. on behalf of the Town of Niagara-on-the-Lake and undertaken by Coffey Geotechnics Inc. (Coffey). The purpose of the investigation was to obtain information about subsurface conditions at the site by means of testpits and, based on the information obtained, to assess the engineering characteristics of the subsurface soils. The proposed 300 mm diameter watermain is to be located along St. Paul Avenue in close proximity to the west piers of the two Hwy 405 overpass structures carrying eastbound and westbound traffic over St. Paul Avenue. Reference on drawings in this report to Four Mile Creek Watermain, Four Mile Creek Road or St. Paul Avenue refers to the same location addressed in this report.

The findings of the investigation are presented in this report. It should be pointed out that this investigation was undertaken for design assistance only and not for contract purposes, and contractors should, therefore, not rely solely upon this information in planning and/or executing their construction work.

2 SITE DESCRIPTION AND GEOLOGICAL SETTING

The site is located at the intersection of the Hwy 405 overpasses and St. Paul Avenue between the Niagara Town Line and Steele Road as shown in Drawing 1 in Appendix B. The section between the Niagara Town Line and Steele Road is within the Hwy 405 right-of-way and within the lowlands adjacent to the Niagara Escarpment. Within this section of the overall project limits, St. Paul Avenue is a 2-lane asphalt paved roadway with gravel shoulders. The piers of the Hwy 405 overpasses are located to the outside shoulders of St. Paul Avenue. The location of the proposed watermain is also shown on Drawing 1, both in plan and section. The existing pier footing is shown in Section A of Drawing 1.

The site is located broadly between the Niagara Escarpment and Lake Erie in the physiographic regions of Southern Ontario, comprising the Haldimand Clay Plain and the Iroquois Plain. The geologic setting of the site consists, according to the text "Physiography of Southern Ontario" by Chapman and Putnam, bedrock of the Salina Formation of Upper Silurian Age. Based on the location of the project site and the map outlining the Physiography of Southern Ontario, the site appears to be situated within a sand plain or a till moraine plain.

3 INVESTIGATION METHODOLOGY

3.1 General

The field work was carried out on May 21, 2010. The geotechnical investigation was undertaken by testpitting rather than the use of conventional drill holes since it was desirable to mimic the proposed installation by open cut. It was also felt that testpits rather than boreholes would allow for better observations to be made of the subsurface ground conditions and their behaviour when subjected to the excavation process.

The investigation consisted of digging a total of three Testpits (Testpits 1, 2 and 3) along the South Bound (SBL) shoulder of St. Paul Avenue, underneath the Highway 405 overpasses and adjacent to the existing foundations of the piers of the Highway 405 bridges. Testpits were dug using a 310 FG rubber tire backhoe equipped with an approximately 1 m wide flat bottomed bucket. This backhoe was provided to Coffey by DeRose Bros. General Contracting on behalf of the Town of Niagara-on-the-Lake. During the excavations of the Testpits, Mike Kowalcyzk, a representative of R.V. Anderson and Associates Ltd., was on the site along with Coffey's Engineering Staff – Vishnu Diyaljee, Levent Ozcan (Geotechnical Engineers) and Vince Hicks (Senior Technician).

The depths of the testpits varied between 2.1 m and 3.1 m. The locations of these testpits are shown on Drawing No. 1, Appendix B with relevant photographs taken during the testpitting included in Appendix E.

It should be noted that the geotechnical investigation was limited to within the Highway 405 underpass area.

4 SOIL LABORATORY TESTING

The soil samples recovered from the testpits were sealed and transferred to Coffey's Soil Laboratory in Burlington for further verification and classification testing. Four soil samples were tested for grain size distribution. Two samples were taken from Testpit 1, from depths of 0-240 mm and 300-900 mm, the third sample was taken from Testpit 2, from depths of 0-300 mm, and the last sample was taken from Testpit 3, from depths of 900 mm-1.2 m.

The laboratory test results are summarized on the Record of Testpits in Appendix C.

5 SUBSURFACE CONDITIONS

5.1 General

Generalized descriptions of the subsurface conditions encountered in the testpits are provided in Subsections 5.1.1 to 5.1.4, while the Record of Testpits are provided in Appendix C. The testpit records indicate the subsurface conditions only at the testpit locations and include textural descriptions of the subsoils in accordance with MTO Soil Classification Manual and indicate the soil boundaries inferred from non-continuous sampling and observations during the testpit advancement. These boundaries typically represent approximate transition zones from one material type to another and should not be regarded as exact planes of geological change. It should also be noted that the subsurface conditions may vary across this site. When reading this report the Explanation of Terms and Symbols provided in Appendix C should be referenced.

In general, the site consisted of predominantly of varying size granular materials ranging from gravel to silt to the depths of investigation in each testpit.

A summary of the soil types and groundwater conditions encountered in the testpits is provided in the sub-sections below.

5.1.1 Granular Base/Subbase

A granular base/subbase layer below the ground surface was encountered in the three testpits. The thickness of this base/subbase layer was about 240 mm and consisted of gravelly sand, some silt and clay or sand and gravel, some silt and clay. A sample of this material was recovered from Testpit 1 from between a depth of 0 to 240 mm below the ground surface. This sample was subjected to laboratory testing for water content and grain size distribution. The results of these tests are presented in the corresponding Record of Testpits in Appendix C.

The test results are summarized as follows:

Water Content

- 10%

Grain Size Distribution

- 20% gravel;
- 61% sand;
- 19% silt and clay

The grain size distribution curve of this granular base/subbase material is presented in Figure 1 in Appendix D.

Another sample of the base course/subbase material was recovered from Testpit 2 between 0 to 300 mm below the ground surface. This sample was subjected to laboratory testing for water content and grain size distribution. The results of these tests are presented in the corresponding testpit log in Appendix C.

The test results are summarized as follows:

Water Content

- 4.7 %

Grain Size Distribution

- 30% gravel;
- 55% sand;

- 15% silt and clay

The grain size distribution of this granular base/subbase material is presented in Figure 2 in Appendix D.

5.1.2 Granular Fill Layer

Underneath the granular base/subbase layer, a thick layer of gravel and sand (fill) was encountered in the testpits. A sample of the granular fill material was recovered from Testpit 1 from depths between 300 to 900 mm. This sample was subjected to laboratory testing for water content and grain size distribution. The results of these tests are presented in the corresponding Record of Testpit in Appendix C.

Water Content

- 8.4 %

Grain size distribution

- 40% gravel;
- 50% sand;
- 10% silt; and clay

The grain size distribution curve of this granular fill is presented in Figure 3 in Appendix D. The material is considered a granular (non-cohesive) soil type.

At the Testpit 2 location, the granular fill, which was found underneath the old pavement layer, continued to the termination elevation of the testpit, which was about 3.1 m below the ground surface.

5.1.3 Pavement Layer

At the bottom of the granular fill, an old asphalt pavement layer was encountered in the three testpits. In the absence of any factual data, this layer was reasoned to represent a previous roadway, presumably one that existed prior to the construction of the Hwy 405 overpasses. This roadway surface elevation was about 900 mm to 1.5 m lower than the existing shoulder (top) elevation. While digging Testpit 1, a high tar like smell was noted. This smell was observed to come from the pavement layer and appeared to be a tack coat. However, on closer examination and reasoning this material was thought to be a constituent of the pavement layer, which was likely of macadam construction. It should be noted that the tar like odour was not noticed when this layer was penetrated in the other testpits. The pavement layer at those locations appeared to be of asphaltic concrete composition. It should be noted that Testpit 1 was the furthest from the roadway and close to the apron of the bridge headslope fill. It is also likely that this tar product might be isolated.

5.1.4 Gravelly Sand Fill Layer

Below the granular base/subbase layer of the old pavement layer in Testpits 1 and 3, a predominantly gravelly sand fill was encountered. The gravelly sand fill started at about 1.5 to 1.6 m below the ground surface. It was mixed with 50 mm Crusher Run Limestone (CRL) in Testpit 1 and contained traces of wood pieces in Testpits 2 and 3. Testpit 1 was terminated at about 2.1 m, Testpit 2 at about 3.1 m and Testpit 3 at about 2.4 m depth below the ground surface.

A sample of the gravelly sand mixed with 50 mm CRL material was recovered from Testpit 3 from depths between 0.9 m to 1.2 m. This sample was subjected to laboratory testing for water content and grain size distribution. The results of these tests are presented in the corresponding testpit log in Appendix C.

The test results are as follows:

Water Content

7 %

Grain size distribution

- 37 % gravel;
- 53 % sand;
- 10 % silt and clay

The grain size distribution curve of this granular fill is presented in Figure 4 in Appendix D. It is a granular (i.e. non-cohesive) soil type.

In Test Pit 1, below a depth of 1.6 m, the fill was found mixed with some clayey silt.

6 STRATIGRAPHIC PROFILE

An approximate stratigraphic profile of the subsurface linking the soils encountered in the testpits is shown in Drawing 2, Appendix B in relation to the ground profile.

7 GROUNDWATER CONDITIONS

Visual observations of groundwater were made during and after the digging of each of the testpits. The testpits were not backfilled on completion but left open until the final testpit was undertaken. During this period, which lasted for about 4 hours, periodic checks were made on each pit and at each time interval groundwater measurements were taken. Seepage was observed in the three testpits during the excavation and when the excavation was below the old pavement layer. Above the old pavement layer, fairly dry conditions were encountered.

Testpit 1 had only 100 mm of standing water just after the initial excavation, seepage was noted and water level was measured to be approximately 1.3 m below the top of the existing ground level after about 3.5 hrs. The depth of water in the testpit was approximately 800 mm.

In Testpit 2, while the standing water was only 50 mm at the end of the excavation, the water level was measured to be about 2.7 m below the ground level after 4 hours, with an approximate 400 mm depth of water in the testpit.

In Testpit 3, seepage noted at approximately 1.9 m below the ground level during the excavation, with water level at approximately 2.5 m below ground level after approximately 2.5 hours. The depth of water in the testpit was less than about 100 mm.

8 HISTORIC SUBSURFACE INFORMATION

Historic information on the subsurface conditions at this site was determined from the foundation report for the Hwy 405 Overpass Structures. This investigation was undertaken in January 1961 for the geotechnical design of the foundations for the present Hwy 405 Overpasses. At the time of the investigation St. Paul Avenue/Four Mile Creek roadways was known as Hwy 8. A copy of this report, obtained from the MTO project files, is provided in Appendix F for reference.

As shown on the Hwy No. 8 Overpass, General Plan, Drawing 6, Appendix B, several boreholes were put down to facilitate the understanding of the subsurface conditions at this site. Of these, Boreholes 2, 7 and 8 are relevant to the present project since these were drilled the closest to the location of the east piers. These boreholes show that the subsurface materials below the topsoil depth to consist essentially of clayey silt and sandy silt materials within a depth of 3 m below the topsoil layer. Below this depth, essentially granular materials were encountered to the depth of sandstone and red shale bedrock, which was contacted at around 20 m below the ground surface.

Artesian conditions were encountered at around 8 -10 m below ground level. This artesian condition produced a head of water about 3 m above the ground level. Stationary water table measurements in these boreholes showed dry conditions in Borehole 2 and groundwater levels at approximately 1.2 m and 1.5 m below ground level. The Standard Penetration 'N' values within the top 3 m at averaged 30 in Borehole 2, 20 in Borehole 7, and 8 in Borehole 8. For construction of the spread footings, recommended for the foundations of the overpass structures, the existing ground was stripped of the organics and compacted gravel fill used to bring the existing ground to about 1 m or slightly higher in the vicinity of the east piers.

There was no pavement layer recorded in these or any of the other boreholes as noted in the borehole logs of the "Report from the 1961 Geotechnical Design of Foundations for The Present Hwy 405 Overpasses" in Appendix F. It should be noted that at the time of the geotechnical investigation, Hwy 8 was in existence and would likely have had a paved surface/hard surface. Following the overpass construction, this roadway would likely have been buried by the compacted granular fill used to raise the grade of the existing ground prior to the construction of the overpasses.

For and on behalf of Coffey Geotechnics Inc.

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FOUNDATION DESIGN REPORT

PROPOSED WATERMAIN INSTALLATION

HIGHWAY 405 ENCROACHMENT AT ST. PAUL AVENUE

TOWN OF NIAGARA-ON-THE-LAKE, ONTARIO

9 DISCUSSION

The purpose of the Design Report is to provide an evaluation and assessment of the proposed location and method of installation of the watermain. In the following sections we present and discuss our findings to demonstrate whether the concept as proposed is feasible without impacting the existing pier foundations.

9.1 Features of Proposed Watermain Location

The location of the watermain and relevant plan-profile, sections and testpit locations are shown in Drawing 1 in Appendix B. Essentially, the purpose of the investigation was to determine whether the proposed location and the open cut method of installing the watermain would have any detrimental impact on the integrity of the existing piers of Hwy 405 Overpass Structures. Section A in Drawing 1 illustrates typical details of the proposed watermain location from the SBL piers and the proposed depth of installation. As shown, the location of the watermain is to be offset approximately 6.5 m from the centreline of the piers and at a depth of 2 m below the ground surface at the centreline of the proposed installation. The excavation of the watermain is shown as an open cut with a sloping section to a depth of 1 m after which the excavation is shown with vertical sides to below the depth of the watermain. Based on the above, the top edge of the excavation will be approximately 4.0 m east of the inside face of the SBL pier footings and with the centreline of the proposed watermain approximately 5.28 m east of the inside face of the SBL pier footings. These measurements are illustrated in Drawing 3, Appendix B.

As shown on Drawing 4, Appendix B, the proposed watermain will be located above stress/pressure distribution lines 1:1, 1.43:1, and 3:1 from the underside of the SBL pier footing foundation. Typically, a 2:1 pressure distribution line is used as a benchmark in deciding on a suitable distance of a new structure to be constructed adjacent to an existing one or for an open excavation to be made adjacent to an existing structure. It is quite clear from tis drawing that the offset of the proposed installation will not be influenced by stresses from the existing footing, nor will the existing footings be influenced by the new installation from a load associated perspective.

9.2 Features of Existing Watermain Location

Also shown in Drawing 3, Appendix B is the existing 500 mm diameter watermain, encased with a 700 mm diameter steel liner. This watermain, from discussions with Vince Grande of R.V Anderson Associates Ltd., was installed some 20 years ago by the open cut method. According to the as-built drawings provided by the Engineer of Record at the time and as well that the MTO provided a permit for this installation. This watermain is offset 5.34 m east of the centreline of the pier and offset approximately 4.12 m east of the inside face of the SBL pier footing. The depth of installation was approximately 3.85 m below the ground surface. In relation to the various pressure distribution lines, it is noted that the location of this existing watermain was not influenced by the footing pressure nor was the footing area influenced by the pressure of the installed watermain structure.

9.3 Features of Testpit Excavations

As discussed in Part 1, the three testpits were put down along the alignment of the SBL piers or within a metre east of the face of the piers. The photographs in Appendix E show that the testpits remained open during the period of testpitting which varied from 4 to 4.5 hours. It should be noted that there was about an hour time lag between the completion of Testpit 3 and the start of Testpit 2 as a result of the need to change the flat bottom bucket of the backhoe to one with teeth to facilitate the excavation of the old pavement layer encountered with depth.

Observations were made of each testpit during and following its completion while excavation was in progress at another pit location.

Asphaltic concrete was encountered in all testpits between 0.9 to 1.6 m below ground at the testpit locations. This indicated the existence of an old road below the present roadway which was likely buried by a grade raise to construct the existing roadway. There was no evidence of this roadway when the drilling was undertaken in 1961 for the design and construction of the Hwy 405 overpasses, details of which are provided in Section 8, Part 1 of this report. It would appear, that this roadway was initially constructed after the overpass was constructed, and with a subsequent reconstruction of the roadway showing its present surface.

The important aspects from the testpit investigation that are pertinent to decision making are as follows:

1. Testpits were excavated to depths below the proposed depth of excavation for the watermain installation without any caving of the sidewalls even over a period of 3 to 4 hours for Testpits 1 and 2.
2. Seepage was observed in all testpits at around a depth of 1.9 m in Testpit 3 and 2.1 m in Testpit 2. Seepage in Testpit 1 was encountered at around 1.3 m below ground surface.
3. A tar based pavement layer was encountered in Testpit 1. This pavement layer produced a high smell which was assumed to be from a tar based product. This appeared to be isolated and occurred only in the vicinity of Testpit 3.
4. Compared with the proposed watermain, the testpits were excavated closer to the SBL pier foundations, although outside of the location of the piers, and did not collapse over a period of 4 hours.
5. The soils encountered were essentially granular in nature consisting of varying sizes ranging from sand to large sized gravels.
6. Actual digging of the testpits to the depth of the proposed excavation with a rubber tired back hoe took about 1 to 1.5 hours, with most of the time spent in digging through the old pavement layer. The longer time that was reported for Testpit 2 was due to stoppage of digging due to the inability to dig through the pavement layer with a flat bottom bucket, and changing of the backhoe bucket when the same condition was encountered in Testpit 3.

7. The amount of seepage collected in the testpits varied from 100 mm in Testpit 3 to 400 mm at Testpit 2 to about 900 mm in Testpit 3.

9.4 Inferences from Testpits and Existing Installation on Proposed Installation

While theoretical or semi-empirical approaches can be used to determine whether the offset of the proposed excavation for the proposed watermain installation is sufficient to prevent any impact on the existing foundations, the field observations from the testpit program have, in many ways, substantiated that there should be no problems with the intended installation. Based on information provided, the existing watermain, which is closer to the piers than the proposed, by at least 1 m, and reportedly installed by open cut, did not result in construction problems or any reported impacts on the pier foundations. However, there is also no evidence to completely support that the excavation for the existing watermain was not braced or a trench box used. No photographs of the existing watermain installation are available.

The deduction from the exercise and information that open cut was used in the previous installation suggests that, with the proposed watermain being further away from the piers, there should be no detrimental impact on the existing foundations of the Overpasses. Had the sides of the testpits undertaken shown collapse, this could have questioned the method of installation of the existing watermain and further question the likely behaviour of the proposed installation by the open cut approach.

While the information provided that the existing watermain was installed by open cut, this information can be considered to be somewhat questionable for the section of watermain that was installed in proximity to the SBL piers. This watermain was installed some 20 years ago to a depth of 3.85 m below the ground surface at a location slightly west of the proposed installation as shown in Drawing 3, Appendix B.

To determine if the existing installation was undertaken as reported at this location, we show in Drawing 4 the open cut likely to have been undertaken. This is shown as an excavation to ground surface with vertical sidewalls. Since the depth of installation was deeper than which a worker can enter unprotected, we have also shown possible sloped sidewalls of the excavation to allow for a safe excavation for a worker to enter such an excavation. With granular bedding used below the existing watermain, it is feasible that workers would have been required to enter the excavation. With sloped sidewalls, the excavation would be close to the pier and this would likely not have been allowed. It is feasible also that the section of open cut along St. Paul Avenue parallel to the SBL pier footing of the east bound Hwy 405 overpass could have been shored with a trench box. This would have assured that the excavation remained intact for the duration of installation of the watermain.

9.4.1 Evaluation of the Safe Setback Distance of Watermain from Pier Footings

As shown in Drawing 3, Appendix B, it is proposed to slope the excavation within the top 1 m, which is desirable from a worker safety viewpoint, as well to ensure that sloughing of the surface layers that can possibly occur will not result in soil entering the excavation prior to the bedding for the watermain being properly placed. The adequacy of the offset of the excavation from the pier footings is examined below in relation to empirical and theoretical approaches.

9.4.2 Empirical Approach

One of the empirical approaches used in determining a safe offset distance of placing a new shallow foundation/structure near an existing one is to use a distance away from the existing foundation equal to the larger of the width of the proposed or the existing foundation. In this case, the minimum offset width would be 2.44 m from the inside face (roadway side) of the footing foundation. The offset distance from the top edge of the proposed excavation is 4 m and hence can be considered satisfactory. The proposed offset distance is shown on Drawing 3, Appendix B.

9.4.3 Theoretical Approach

A theoretical approach in determining the safe offset distance is based on the preservation of the soil within the zone that will be mobilized in determining the soil bearing resistance at the Ultimate Limit State remaining intact during excavation. This is achieved by evaluating the distance of the theoretical rupture surface from the inside face of the existing footing foundation. In preserving the integrity of this zone, excavation should not be made within this established distance. The concept utilizes the determination of the rupture surface as shown in Drawing 5, Appendix B. If a friction angle of zero degrees is used, then this offset distance would be about 2.2 m from the inside face of the pier footing.

However, if we use an angle of friction of about 32 degrees, which would be more realistic since the material below the footing is granular in nature, then the horizontal distance from the edge of the footing to the theoretical line of rupture would be about 4 m. The distance from the outside edge of the proposed open cut toward the outside of footing would be about 4 m as well. It is to be noted that the soil above the footing base is not considered to contribute to the ultimate bearing resistance of the footing. This suggests that the proposed excavation as shown in Drawing 5, Appendix B will not have any negative impact on the integrity of the existing footings.

While excavations for buried utilities can be undertaken closer than the theoretical offset, these are often done in short sections of about 3 m so that backfilling can be done quickly following excavation. This method may be suitable for this site.

10 INSTALLATION BY TRENCHLESS METHODS

The jack and bore approach used for installation of watermains and other utilities crossing highways would eliminate the need for the open cut method and is attractive where disruption to the normal flow of traffic cannot be tolerated such as on major highway series and on roadways that deep utility installations are to be undertaken. Directional drilling is also a preferred approach. In the situation in question, this approach with the soils encountered may be problematic and hence would be an overall costlier approach than the open cut approach. While both trenchless and open cut installation methods are acceptable from a reliability point of view, from a cost consideration the proposed open cut method would be the more economical one.

11 GROUND SETTLEMENT CONCERNS

Seepage is always a sign of concern that has to be addressed in any construction operation. The seepage observed was slow and hence can be removed by sump pumping. However, with possible sand seams within the excavation depth, pumping has to be done through a coarser aggregate blanket at the base of the excavation so that fines are not removed during pumping. If this is not recognized pumping can result in the development of cavities beyond the excavation walls due to loss of ground. For this reason, we recommend that when the excavation for the watermain proceeds to near the pier locations, the trench should be opened in relatively short sections and backfilled as rapidly as practical.

As an alternative to avoid a sloped open cut excavation, a trench box is recommended, which will not only assist in maintaining vertical trench walls, but also provide protection to workers working in or adjacent to the excavation.

We have discussed with R.V Anderson Associates Ltd. previously that the watermain installation can, perhaps, be located at approximately 1.2 m below the existing ground level to make use of the old pavement structure as a base for the new installation. This depth however, may not allow for acceptable structural cover over the top of the pipe. This location provides a condition with no interference from seepage zones and hence reduces the possibility of intersecting water. However, with the nature of granular material, it is not expected that minor pumping will result in settlement of the soils below the footings. This proposed location may require protection of the watermain from freezing. This can be offset by the use of rigid insulation around the pipe. This approach and insulation installation procedures are well documented by Manufacturers such as Dow Chemical, for example.

In the event that that the decision is install the pipe to the design depth as shown in Drawing 1, there is always the potential for ground settlement when dewatering of any site is undertaken. For this site, with essentially granular soils and seepage to be encountered within the proposed depth of installation of the watermain, any likely ground settlement will occur more or less simultaneously with drawdown. The concern of settlement at this site would be its impact, if any, on the footing foundations of the existing Hwy 405 structures. This impact has been examined by determining the magnitude of settlement that would likely occur during dewatering from the proposed trench excavation.

Drawing 6 shows the drawdown curve that would likely result due to pumping. This curve shows the radius of influence being a distance of about 9 m from the source of pumping. As shown also in Drawing 6, this drawdown would result in an approximately 0.5 m drop in water level below the centre of the footings. The likely movement of the pier foundations of the bridges due to the 0.5 m drop in water level has been determined to be less than 1 mm. If we assume that the drawdown below the pier is to the maximum depth of excavation say 2 m, then the likely settlement would be around 3 mm. This settlement has been determined using the allowable bearing pressure of 2 tons/ft^2 (217 kPa) recommended for the design of the pier foundations (See Foundation Report in Appendix F).

12 RECOMMENDATIONS AND CONCLUSIONS

While from a practical viewpoint we have deduced that the proposed open cut as proposed and as shown in the Drawings in Appendix B would be satisfactory and result in no adverse impacts on the existing SBL pier foundations, we also recognize that the risk factor for a bridge structure is relatively high if there are problems associated with the satisfactory performance of its foundations.

We, therefore, recognize the MTO's concern that open cut unsupported excavations within close proximity of major bridge structures resting on spread footings could lead to instability and/or serviceability problems of the existing footings through loss of bearing resistance and excessive settlement. Both of these could occur if the bearing resistance of the ground below the footings is compromised.

This study has shown that these problems should not occur and is based on our understanding, as well, that the previous installation, which was much closer to the pier foundations, was carried out by open cut construction. To alleviate any doubts and to make the recommendations of accepting the proposed installations unequivocal, it would be of value to obtain the records of the conditions of permit issued by MTO for the installation of the existing watermain. This opinion is strengthened by our observations of the testpits standing with vertical sidewalls despite some seepage within or close to the proposed depth of excavation for the watermain installation. This is further reinforced by the empirical and theoretical setback distances from the footings of the piers which are exceeded by the proposed location of the watermain installation.

In summary, we have examined the proposed installation of the 300 mm PVC watermain to be encased in a 500 mm diameter steel casing, which will be situated approximately at an offset distance of 6.5 m from the centreline of the SBL piers of the Hwy 405 overpasses. Based on our observations made during the testpitting, subsurface soils encountered, evaluation and assessment of location and depth of the proposed watermain installation, with respect to practical and theoretically derived offset distances, we anticipate no effects on the foundations of the existing SBL piers during or after the duration of this work.

We caution, however, that the short term observations and deductions made from the testpits can often differ when full scale excavation and installation is undertaken. Ground conditions that may be different to that encountered in the testpits may require changes to be made. Construction activity and weather conditions may also result in conditions warranting changes to be made. In this respect, there is a need to provide full time inspection of this operation by a Geotechnical Engineer who is familiar with the findings of this report and who will be able to recognize problems that could increase the level of risk during the excavation and installation phase of the watermain and to evaluate and decide what needs to be done, if such occurs. The removal and disposal of the tar based pavement sections that will be encountered during the open cut method will also require monitoring to ensure its proper disposal and this will require the full time inspection by an Environmental Officer. Consideration can also be given to the monitoring of the affected bridge piers, immediately before, during and several weeks after the construction of the watermain. A suggested monitoring program for the bridge piers during is provided below in Section 13 below.

13 MONITORING PROGRAM FOR BRIDGE PIER FOUNDATIONS

To guard against possible collapsing ground that could occur during excavation, which, however, is not anticipated; it is proposed to undertake monitoring of the site through visual and survey methods. The visual approach will entail monitoring by Coffey's site representative on a continuous basis during the excavation, installation and backfilling of the watermain during the period of construction from its inception to completion.

In addition to visual observation, surface movement markers will be installed prior to construction as shown in Drawing 8. These markers will consist of 20 mm rebar driven into the ground adjacent to the footing foundations on the west side of St. Paul Avenue. Four markers are proposed adjacent to the footings of each of the eastbound and westbound overpasses. These will be situated about 1 m east of the outside (roadside) edge of the footing and 1 m to the west from the west face of the excavation in the array shown in Drawing 8.

These markers will be installed to a minimum depth of 750 mm below ground and the tops spray painted for identification. These markers will be surveyed by an Ontario Legal Surveyor in relation to coordinates (Northings and Eastings), and elevation. The survey will be referenced to established permanent survey monuments. These markers will be initialized prior to excavation, rechecked on the day excavation is to start and surveyed during the period of excavation, installation and backfill of the watermain over the period of the construction. Frequent survey checks will be made at approximately one hour intervals or shorter. Both Coffey and the Survey company representatives will be onsite full time during the entire construction operation.

The survey data will be evaluated on site to determine if any ground movements are occurring. If movements are found to be occurring, the excavation or installation operation will be stopped and the situation evaluated. However, with the proposal for using trench boxes as the excavation progresses, it is not expected that lateral movement will be of a concern. Survey monitoring will be done once a day for a period of week following the backfilling of the excavation.

In terms of vertical settlement, this should not be allowed to exceed 25 mm. If the movement accumulates to about 6 mm in a progressive manner, this would signal that the dewatering may be creating a problem. In that case, the dewatering by pumping will be ceased. However, it is not anticipated that this situation will arise based on the calculated settlements. Photographs will be taken prior to the excavation and during the course of the installation, as well as other tape measurement field checks will be made to ensure that sufficient redundancy is incorporated in the monitoring.

14 CLOSURE

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

For and on behalf of Coffey Geotechnics Inc.

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Appendix A

Important Information about your Coffey Report

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Important information about your **Coffey** Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Important information about your **Coffey** Report

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way.

Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment.
Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

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Appendix B

Testpit Location Plan (Drawing 1)

Stratigraphic Profile (Drawing 2)

Partial Section A (Drawing 3)

Section A (Drawing 4)

Theoretical Approach to Determine Setback Distance (Drawing 5)

Hwy 8 Overpass General Plan (Drawing 6)

Partial Section A – Drawdown Curve (Drawing 7)

Surface Monitoring Points (Drawing 8)

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Appendix C

Explanation of Terms and Symbols

Notes on Sample Descriptions

Compactness and Consistency Tables

Record of Testpits

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EXPLANATION OF TERMS AND SYMBOLS

N VALUE - SANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO DRIVE A STANDARD 51-mm O D SPLIT SPOON SAMPLER 0 3 m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HMNNER WITH A MASS OF 63 5 kg FALLING FREELY A DISTANCE OF 0 76 m FOR PENETRATION LESS THAN 0 3 m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED AVERAGE N VALUE IS DENOTED AS 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 COMPACTNESS

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

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

COMPACTNESS COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF COMPACTNESSAS INDICATED BY SPT N VALUES AS FOLLOWS

N (BLOWS /0 3m)	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

<u>RECOVERY</u> SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN

<u>MODIFIED RECOVERY</u> SUM OF THOSE INTACT CORE PIECES 100mm + IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN THE ROCK QUALITY DESIGNATION (RQD) FOR MODIFIED RECOVERY IS

RQD(%)	0-25	25-50	50-75	75-90	90-100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

<u>JOINTING AND BEDDING</u>

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

<u>FIELD SAMPLING</u>

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

<u>MECHANICAL PROPERTIES OF SOIL</u>

m_v	Pa-1	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	COEFFICIENT OF SECONDARY CONSOLIDATION
C_v	m2/sec	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_a	kPa	APPARENT COHESION INTERCEPT
φ_a	°	APPARENT ANGLE OF INTERNAL FRICTION
c_R	kPa	IDUAL SHEAR STRENGTH
τ_r	kPa	DED SHEAR STRENGTH
S_t	1	$\text{SENSIVITY} = \frac{C_u}{\tau_r}$

<u>STRESS AND STRAIN</u>

u	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
$\varepsilon_1\varepsilon_2\varepsilon_3$	%	PRINCIPAL STRAINS
ν	1	POISSON'S RATIO
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
M	1	COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

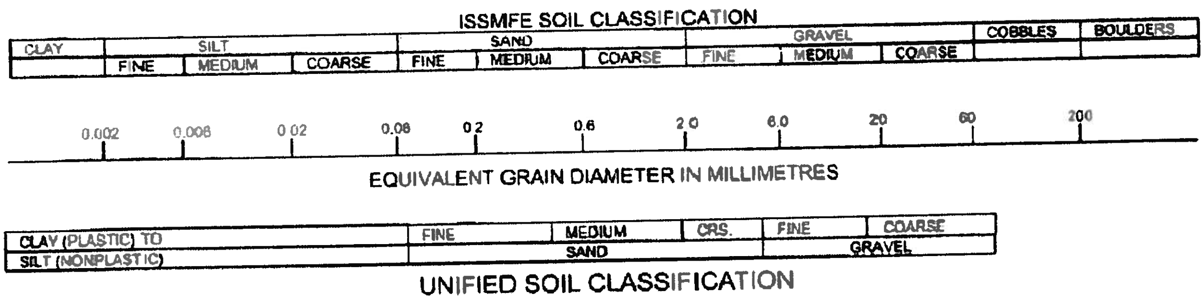
ρ_s	g/m³	DENSITY OF SOLID PARTICLES
γ_s	kg/m³	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m³	DENSITY OF WATER
γ_w	kg/m³	UNIT WEIGHT OF WATER
ρ	kg/m³	DENSITY OF SOIL
γ	kg/m³	BULK UNIT WEIGHT OF SOIL
ρ_d	kg/m³	DENSITY OF DRY SOIL
γ_d	kg/m³	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m³	DENSITY OF SATURATED SOIL
γ_{sat}	kg/m³	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m³	DENSITY OF SUBMERGED SOIL
γ'	kg/m³	UNIT WEIGHT OF SUBMERGED SOIL

e		VOID RATIO
n		POROSITY
w	%	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	%	$\text{LIQUIDITY INDEX} = \frac{w - w_P}{I_P}$
I_C	%	$\text{CONSISTENCY INDEX} = \frac{w_L - w}{I_P}$
e_{max}	%	VOID RATIO IN LOOSEST STATE

e_{min}	%	VOID RATIO IN MOST DENSE STATE
I_D	1	$\text{DENSITY INDEX} = \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
Dn	mm	n PERCENT - DIAMETER
Cu	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m³/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULID GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m³	SEEPAGE FORCE

Notes on Sample Descriptions

1. Sample descriptions included in this report follow the Canadian Foundation Engineering Manual Soil Classification System, in general. This system follows the standard proposed by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Laboratory grain size analyses provided by Coffey Geotechnics Inc. also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, the samples were classified visually. Visual classification is, however, not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



2. **Fill:** Where fill is designated on the borehole log, it is defined as indicated by the samples recovered during the boring process. The consistency or compactness of the fill material has been based on the description of the collected samples i.e. cohesive or cohesionless materials. The reader is cautioned that fills are heterogeneous in nature and variable in gradation, density and degree of compaction. The borehole description may, therefore, not be applicable as a general description of on site fill materials. All non-engineered fills should be expected to contain obstructions such as wood, large concrete pieces or subsurface basements, floors, tanks, etc. None of these may have been encountered in the boreholes unless noted. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas during the exploration program and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings may suggest the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated landfill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken, if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. **Till:** The term till on the borehole logs indicates that the material originates from a geological process associated with glaciations. Because of this geological process the till must be considered heterogeneous in composition, and as such, may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200mm) or boulders (over 200mm). Contractors may, therefore, encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is, therefore, essential when dealing with sensitive excavations or dewatering programs in till materials. Note glacial till deposits frequently contain isolated pockets of water bearing sands and gravel which may be encountered during construction.

TABLE 3.1 *Compactness Condition of Sands from Standard Penetration Tests*

Compactness Condition	SPT N-INDEX (blows per 0.3 m)
Very loose	0 – 4
Loose	4 - 10
Compact	10 30
Dense	30 50
Very dense	Over 50

TABLE 3.2 *Approximate Consistency of Cohesive Soils*

Consistency	Field Identification
Very soft	Easily penetrated several cent meters by the fist
Soft	Easily penetrated several centimeters by the thumb
Firm	Can be penetrated several centimeters by the thumb with moderate effort
Stiff	Readily indented by the thumb but penetrated only with great effort
Very stiff	Readily indented by the thumb nail
Hard	Indented with difficulty by the thumbnail

TABLE 3.3 *Consistency and Undrained Shear Strength of Cohesive Soils*

Consistency	Undrained Shear Strength (kPa)	Spt N-Index (blows/0.3m)
Very soft	< 12	<2
Soft	12 - 25	2 4
Firm	25-50	4 - 8
Stiff	50 - 100	8 - 15
Very stiff	100 - 200	15 - 30
Hard	> 200	>30

Source: CANADIAN FOUNDATION ENGINEERING MANUAL
4th EDITION 2006, Canadian Geotechnical Society

Ministry of Transportation
Ontario

RECORD OF TESTPIT No. 1

METRIC

G W P GEOTBURL01244AA
LOCATION Sta. 0+470 HWY 405
ORIGINATED BY V.H.
DIST N.O.T.L. HWY 405
TESTPIT TYPE RUBBER TIRE BACKHOE
COMPILED BY L.O.
DATUM Geodetic
DATE MAY 21, 2010
CHECKED BY V.D.

ELEV. DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
141.20	Ground Surface						
140.96	GRANULAR FILL: gravelly sand, some silt and clay Compact Brown	1	GRAB			141	20 61 (19)
140.30	GRANULAR FILL: sand and gravel, some silt and clay Compact Brown; seeping between 800-900mm	2	GRAB				40 50 (10)
140.20	Asphaltic Concrete 100mm						
139.90	FILL: crusher run limestone 50mm with sandy silt, some clay Compact Reddish Brown				*	140	
139.60	seeping at 1.3m	3	GRAB				
139.10	FILL: clayey silt/sand, mixed with 50mm crusher run limestone Compact Light Brown						
	End of testpit 2.1m No Detectable Gas					139	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): o UNCONFINED, + FIELD VANE, • QUICK TRIAXIAL, x LAB VANE: 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³

Elevation scale: 141, 140, 139, 138, 137, 136, 135, 134

* GROUNDWATER CONDITIONS

DATE	GROUNDWATER ELEVATION (m)
on completion	139.90
11:10 am	139.40
12:27 pm	139.90

$+^7$, x^5: Numbers refer to Sensitivity

20 / 15 -o- 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF TESTPIT No. 2

METRIC

G W P GEOTBURL01244AA LOCATION Sta. 0+440 HWY 405 ORIGINATED BY V.H.

DIST N.O.T.L. HWY 405 TESTPIT TYPE RUBBER TIRE BACKHOE COMPILED BY L.O.

DATUM Geodetic DATE MAY 21, 2010 CHECKED BY V.D.

ELEV. DEPTH	SOIL PROFILE 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 + FIELD VANE • QUICK TRIAXIAL x LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
142.0	Ground Surface						142				
141.77	GRANULAR FILL: sand and gravel, some silt and clay Compact Brown		1	GRAB					o		30 55 (15)
	GRANULAR FILL: gravel and sand, trace of clay Compact Brown		2	GRAB			141		o		
140.50	wet between 1.3-1.5m										
140.40	Asphaltic Concrete 100mm										
139.90	FILL: sand and gravel, trace of clay, trace silt Compact Brown						140				
139.30	seeping below 2.1m		3	GRAB		* ▽			o		
138.90	End of testpit No Detectable Gas						139				
							138				
							137				
							136				
							135				

* GROUNDWATER CONDITIONS

DATE	GROUNDWATER ELEVATION (m)
on completion	139.90
11:25 am	139.30

$+^7$, $\times^5$: Numbers refer to Sensitivity

20 / 15 –◇– 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF TESTPIT No. 3

METRIC

G W P GEOTBURL01244AA
LOCATION Sta. 0+410 HWY 405
ORIGINATED BY V.H.
DIST N.O.T.L. HWY 405
TESTPIT TYPE RUBBER TIRE BACKHOE
COMPILED BY L.O.
DATUM Geodetic
DATE MAY 21, 2010
CHECKED BY V.D.

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
142.60	Ground Surface					
	GRANULAR FILL: sandy gravel, granular base/subbase Compact Brown	1	GRAB			
142.35	GRANULAR FILL: gravel and sand, trace to some silt, trace wood pieces, trace clay Compact, Brown	2	GRAB		142	
	wet between 1.3-1.5m	3	GRAB			37 53 (10)
141.29	Asphaltic Concrete 50mm					
141.25	FILL: gravelly sand, some silt and clay, trace wood pieces Compact Dark Brown				141	
	seeping below 1.9m	4	GRAB			
138.90	End of testpit 2.4m No Detectable Gas				140	

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) o UNCONFINED + FIELD VANE • QUICK TRIAXIAL X LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

Elevation scale: 142, 141, 140, 139, 138, 137, 136

* GROUNDWATER CONDITIONS

DATE	GROUNDWATER ELEVATION (m)
on completion	140.70
10:25 am	140.20

$+^7$, x^5: Numbers refer to Sensitivity

15 -◇- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

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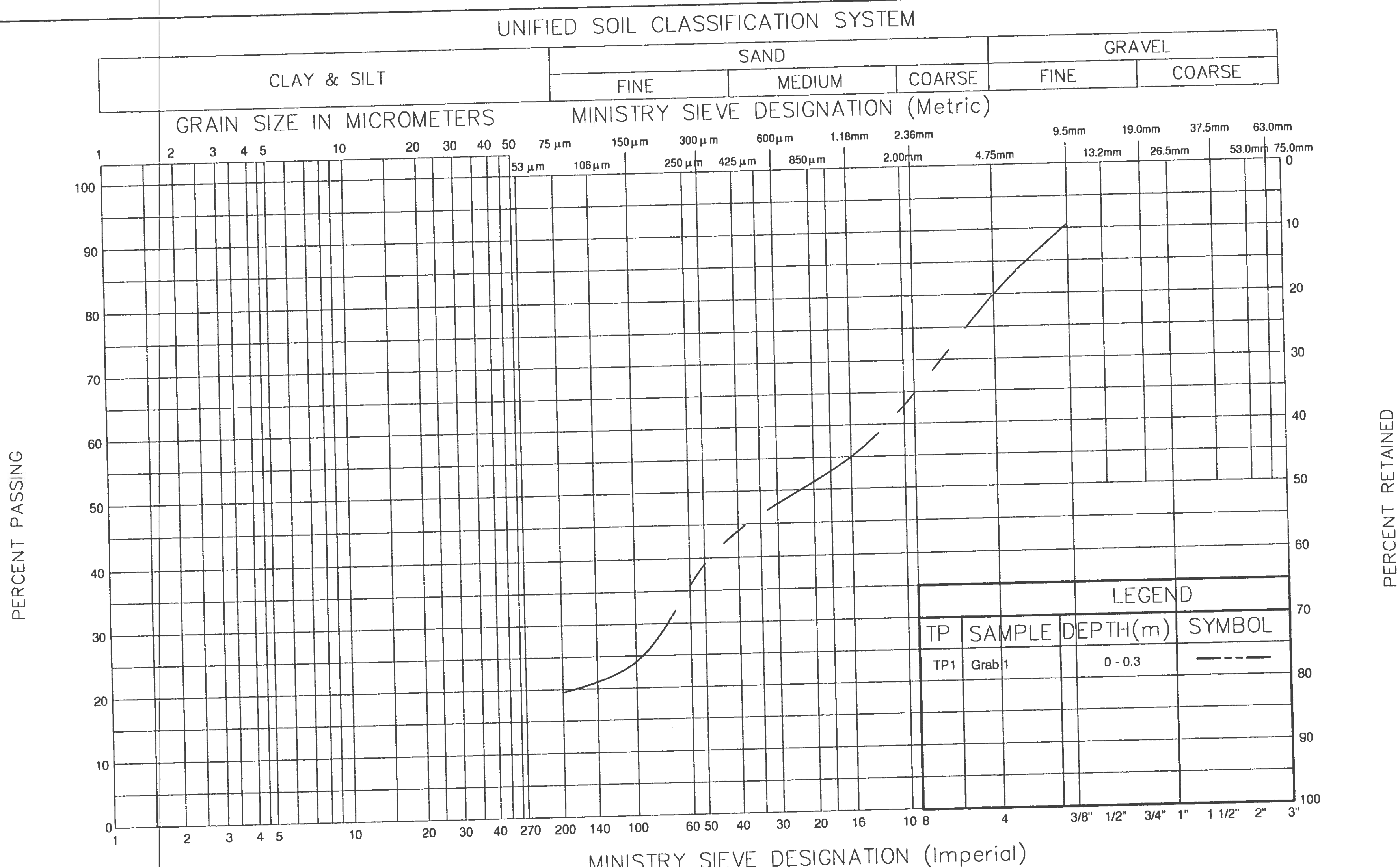
Appendix D

Soil Laboratory Test Results (Figures 1-4)

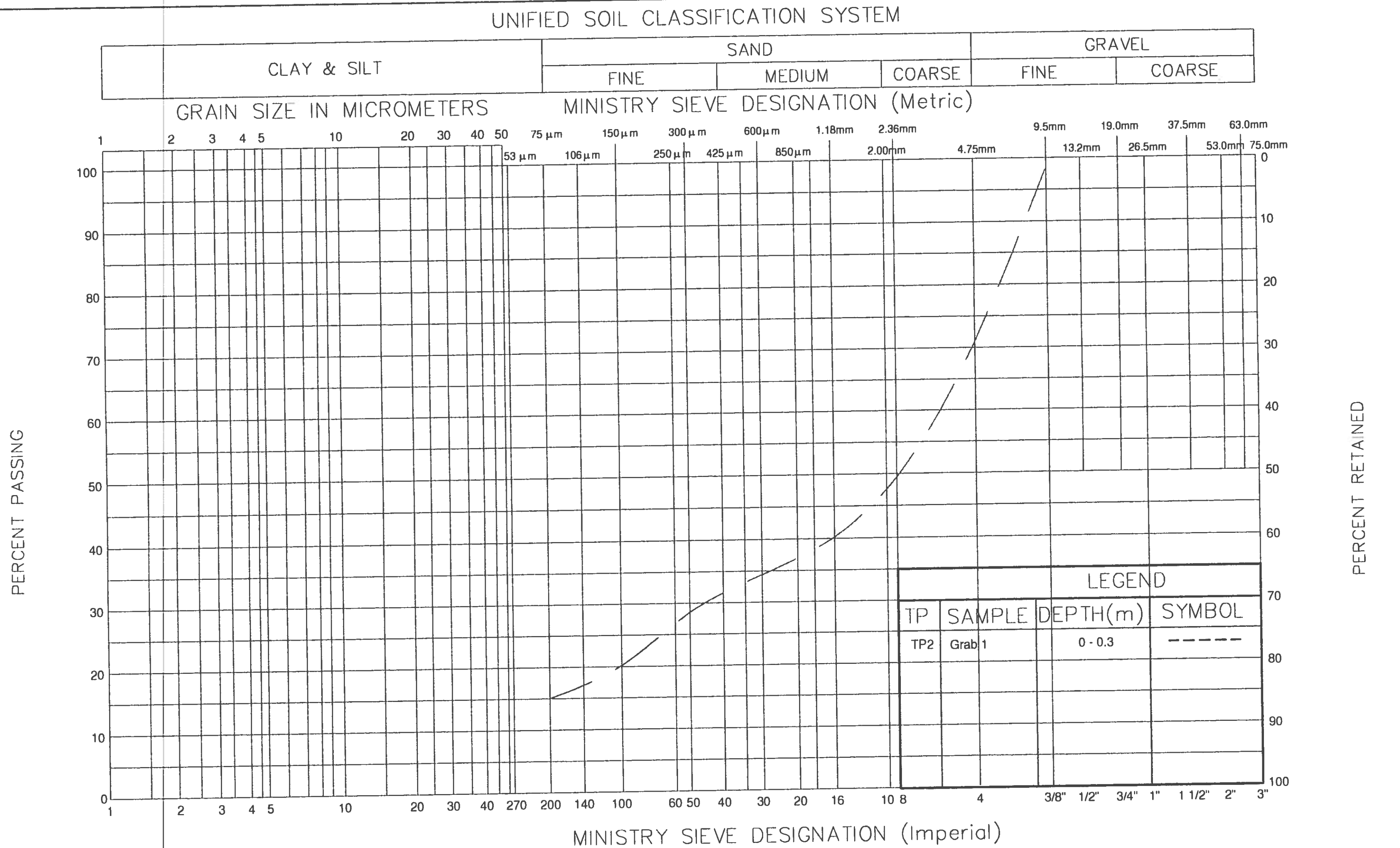


HWY 405
GEOTBURL01244AA
FIGURE No. 1

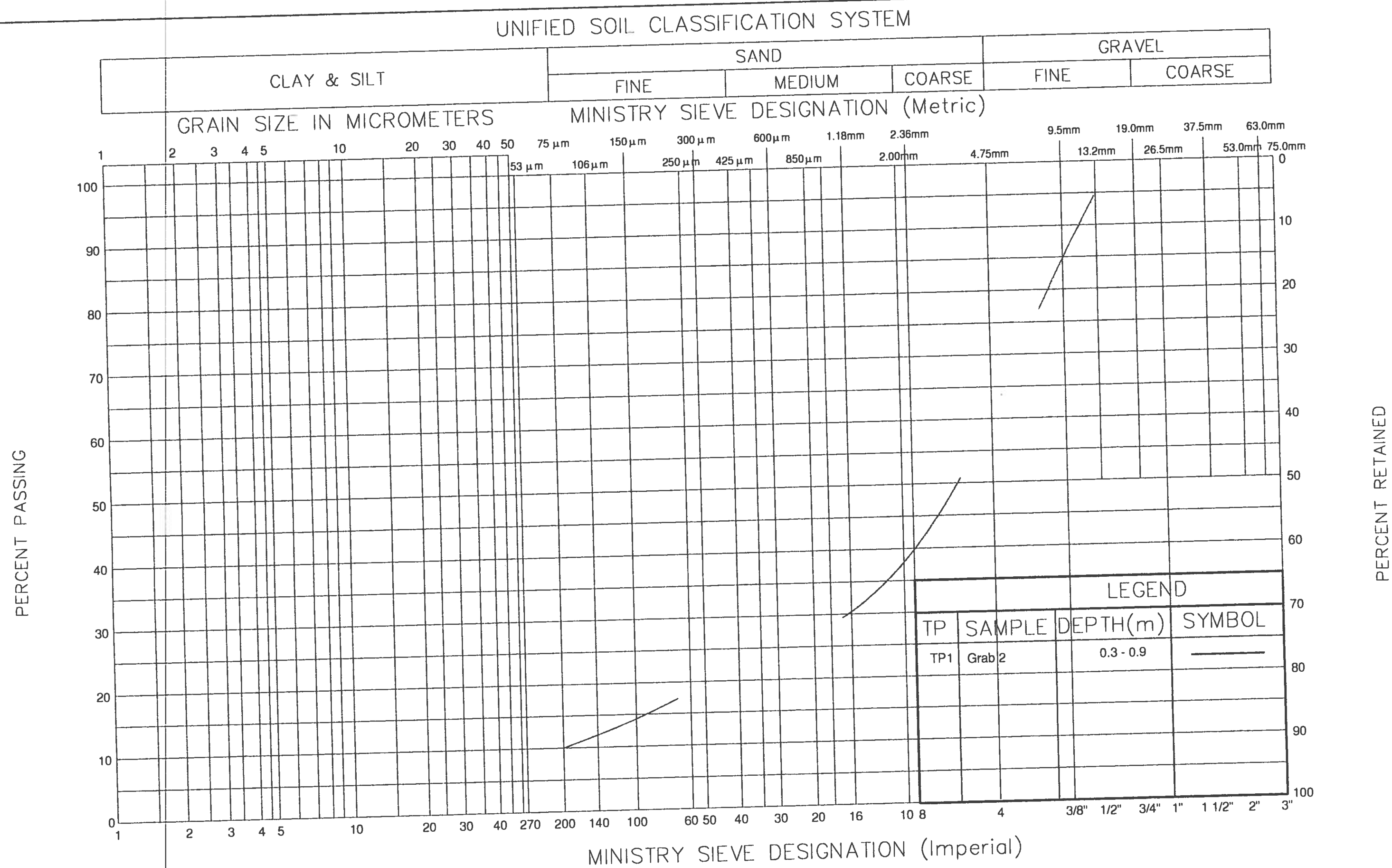
Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

GRANULAR FILL: Gravelly sand, some silt and clay



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION GRANULAR FILL: Sand and gravel, some silt and clay	HWY 405 GEOTBURL01244AA FIGURE No. 2



GRAIN SIZE DISTRIBUTION

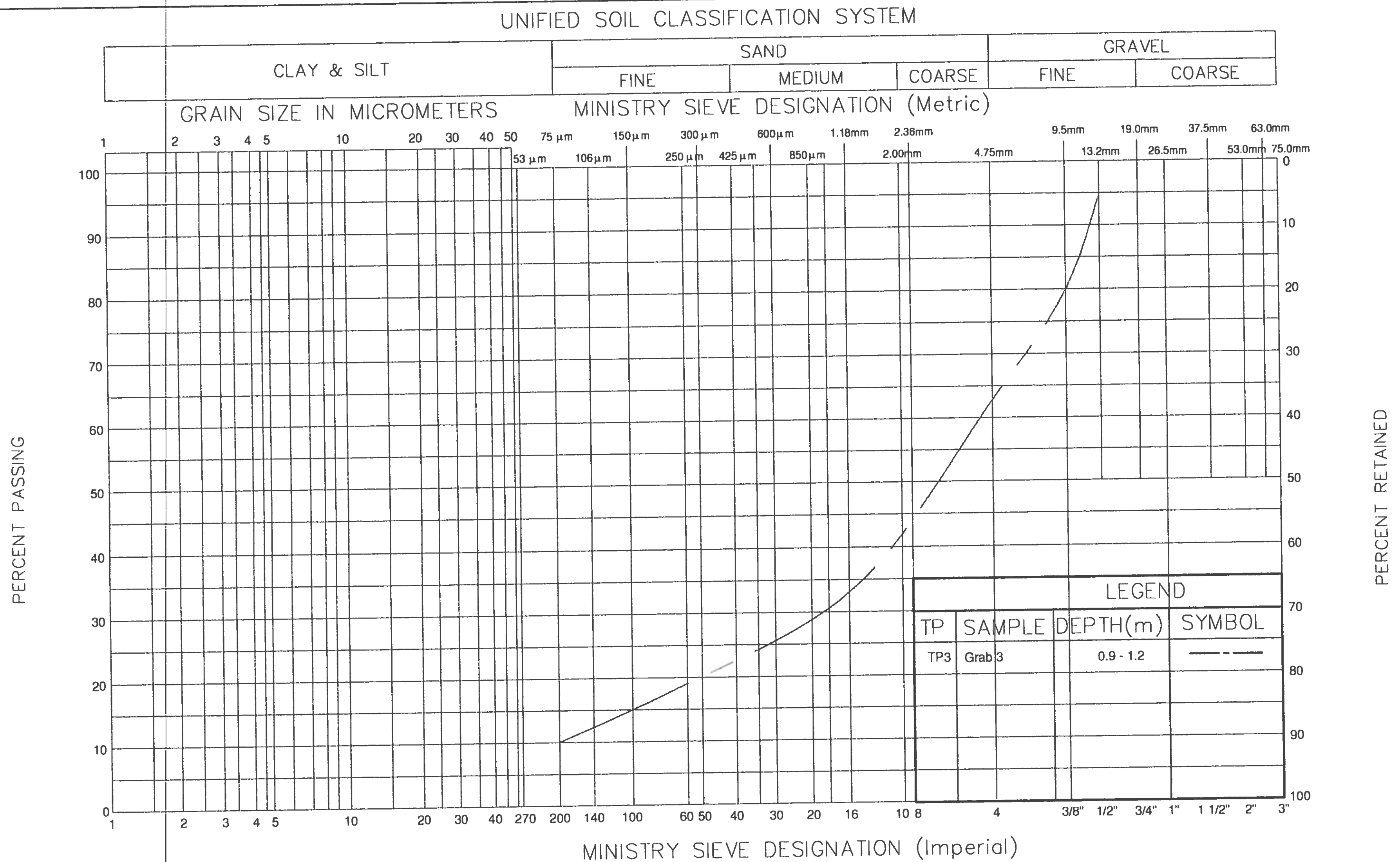
GRANULAR FILL: Sand and gravel, some silt and clay

Ministry of Transportation Ontario

HWY 405

GEOTBURL01244AA

FIGURE No. 3



Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

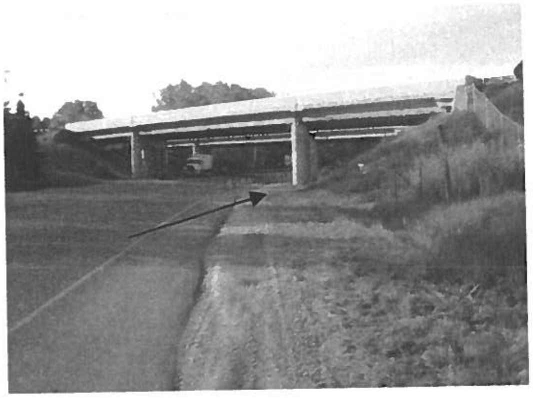
FILL: Sand and gravel, some silt and clay

HWY 405	
GEOTBURL01244AA	
FIGURE No. 4	

Appendix E

Photographs

TEST PIT 1 (May 21, 2010)



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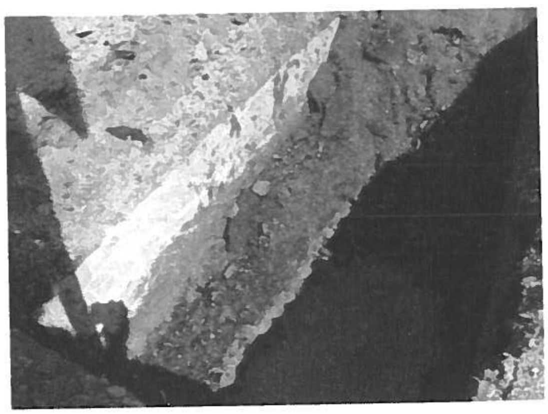
Photograph 2, Time: 7:58 A.M.



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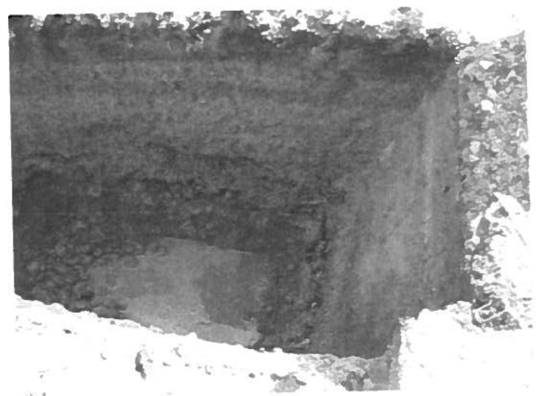

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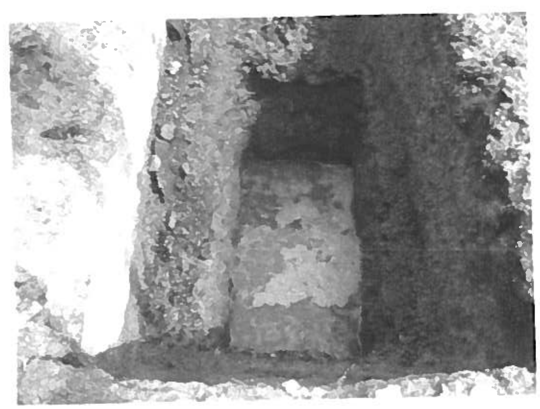
Photograph 5, Time: 10:28 A.M.



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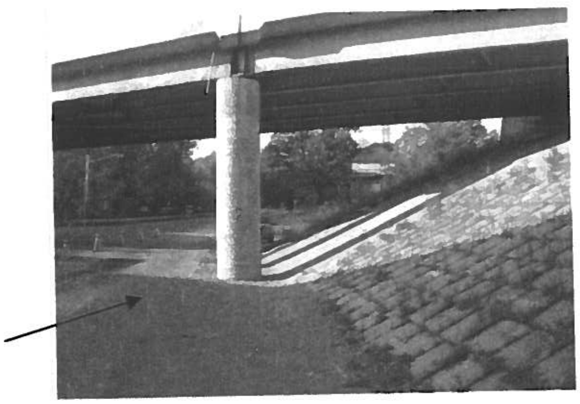
TEST PIT 2 (May 21, 2010)



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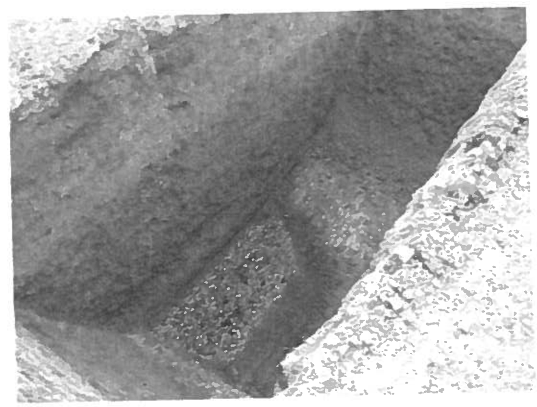
Photograph 2, Time: 8:17 A.M.

Photograph 3, Time: 8:18 A.M.



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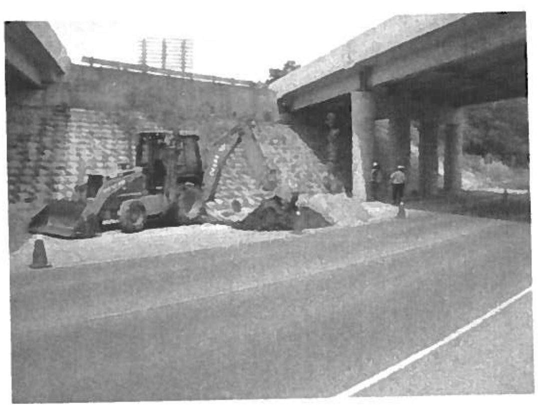


Photograph 5, Time: 11:06 A.M.

Photograph 6, Time: 11:09 A.M.

TEST PIT 3 (May 21, 2010)

Photograph 1, Time: 7:28 A.M.

Photograph 2, Time: 7:43 A.M.

Photograph 3, Time: 7:45 A.M.

Photograph 4, Time: 7:49 A.M.

Photograph 5, Time: 7:50 A.M.

Photograph 6, Time: 11:07 A.M.

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Appendix F

Report from January 1961 Geotechnical Design of the Foundations

for the present HWY 405 Overpasses

cc: Mr. A. M. Toye (2nd copy)

BA 1197

March 28, 1961.

Mr. A. M. Toye,

Bridge Engineer,

Materials & Research Section.

D.H.O. FOUNDATION INVESTIGATION REPORT -
W.J. 61-F-6 -- J.P. 235-59.

Attention: Mr. S. McCombie.

Re: Overpass at Hwy. 8 and Q.E.W. Extension (Hwy.405) Twp. of Stamford, Cty. of Welland, District No. 4.

Attached hereto, we are forwarding to you the Soil Investigation Report for the above mentioned location.

The recommendations and summary contained in this report are self-explanatory, and we trust they will prove adequate for your future design work.

Should any queries arise in connection with this project, please do not hesitate to contact our Office.

L. G. Soderman,
PRINCIPAL FOUNDATION ENGR.
Per:

A.G. Stermac

(A. G. Stermac,
SUPERVISING FOUNDATION ENGR.

AGS/MdeF
Attach.
cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
I. C. Campbell
J. C. Thatcher
T. J. Kovich
A. Watt

Foundations Office
Gen. Files.

TABLE OF CONTENTS

FOUNDATION INVESTIGATION

For

Overpass at Hwy. 8 and Q.E.W. Extension (Hwy. 405)
Twp. of Stamford, County of Welland, District No.4
W.J. 61-F-6 -- W.P. 285-59.

1. <u>INTRODUCTION</u>:

A bridge structure is planned to carry the proposed Hwy. 405 over Highway No. 8, approximately 2 miles South of junction of Hwy. 8 and 8A, in St. David (Sta. 46 + 36.53, Plan E 3952-1). At this location, the proposed new Line 'C' of Hwy. 8 passes between Lots 3 and 4, and approximately 23 ft. east of the present Hwy. 8.

A subsoil investigation was carried out at the site and this report contains the field and laboratory findings and recommendations concerning the foundations of the structure.

2. <u>DESCRIPTION OF SITE</u>:

Highway No. 8 runs in the North-South direction at the investigated site. There are a few buildings on the west side of the highway, while on the east side of the road, the area is generally slightly lower than the surrounding land, and is mostly wasteland, overgrown with weeds and smaller trees. The ground is slightly undulating and there are small creeks running in the area.

The proposed bridge site is part of the lowland adjacent to the Niagara Escarpment.

cont'd. /2 ...

3. FIELD AND LABORATORY WORK:

Field work consisted of nine sampled boreholes with Dynamic Cone Penetration test adjacent to each borehole. These boreholes were located by the Surveying Crew of the Location Section, from an available sketch prepared by the Bridge Planning Section.

The exploration programme was carried out by a standard coredrill machine adapted for soil sampling. Conventional wash boring procedure was followed. Samples were recovered at depths required by means of a 2-inch O.D. split spoon sampler. The dimension of this spoon sampler and the energy used in driving it, conform to the requirements of the Standard Penetration Test. A 2" I.D. thin-walled Shelby sampler was also used at rare times, when the subsoil conditions permitted, to obtain an undisturbed sample. Use of AXT core-barrel had to be made for recovering rock samples. It may be mentioned that boring 5 was drilled only to a depth of 25 ft. to verify the strength of the upper strata in this region.

Samples were visually examined and identified in the field before being transported to the laboratory. Upon receipt in the laboratory, necessary tests were carried out on a selection of both disturbed and undisturbed samples for the determination of moisture content, grain size distribution curves, Atterberg limits, and undrained triaxial compression tests.

Laboratory and field test results have been summarized and are included in this report under Appendix I.

Drawing No. 61-F-6A shows the borehole locations, their respective elevations and the estimated subsoil stratigraphy.

cont'd. /3 ....

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.3) Silty Sand and Gravel:

The thickness of this layer also varies over the investigated area from approximately 35 feet in Borehole 7 to about 75 feet in Borehole 4. The composition of the layer is not uniform because some sandy silt layers and gravel seams were encountered at different and irregular intervals.

During the investigation, artesian water pressures were encountered in this layer causing the loosening up of the material. Because of this, many penetration tests had to be ruled out and discarded. However, all the reliable tests indicate that the layer is dense to very dense.

4.4) Sandy Till with Boulders:

In Boreholes 4, 6 and 9, a layer of sandy till with boulders was encountered. It is hard to establish any pertinent property of this layer due to the presence of relatively large boulders. The average thickness of this layer overlying bedrock is 5 feet. In Borehole 6, it was not quite definite whether the borehole was terminated in bedrock or whether the recovered rock core was still part of some large boulders. Assuming that all of it are boulders, the thickness of the layer at this particular location, is at least 17 feet.

4.5) Bedrock:

Except in Borehole 5, which was a shallow one, and in Borehole 6, bedrock was found and proved in all the other boreholes. In Borehole 6, underlying the bouldery till, the boring

cont'd. /5 ...

4. SUBSOIL CONDITIONS:

4.1) General:

The field investigation and the subsequent laboratory testing have shown the general stratification of the subsoil to be basically regular. The main soil strata encountered were a thin surface layer of topsoil, a clayey silt and sandy silt layer, a layer of sandy silt and silty sand with sporadic sandy and gravelly seams generally increasing in amount with depth, and a layer of sandy bouldery till overlying bedrock. All these layers with the exception of bouldery till and bedrock, were encountered in all boreholes, but their respective thicknesses were not equal in the different locations. A more detailed description of these layers, with the exception of the surface layer, is given below:-

4.2) Clayey Silt & Sandy Silt:

This layer was encountered in all boreholes but its thickness varied from approx. 9 feet in Borehole 8, to about 25 feet in Borehole 7. In most cases, the boundary between the clayey silt and the sandy silt could not have been clearly defined, but sometimes there was a clear separation of the cohesive from the non-cohesive material. Because of these conditions, the recorded 'N' values are somewhat erratic. However, it can be concluded that on the average, this layer is in a medium dense state. The average moisture content of the layer is 18%, and the average Atterberg limits of the cohesive part are 25.5% and 15.0%, thus giving a liquidity index of 0.29. The shear strength as determined on one sample is 1,200 p.s.f. This value is certainly not wholly representative, but it can be considered as indicative.

cont'd. /4 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.5) Bedrock: (cont'd.) ...

revealed sandstone and red shale. It could not be positively established whether this is bedrock or boulders.

As mentioned above, bedrock was proven in all the other boreholes, and below are given the respective bedrock elevations:-

Borehole	Bedrock Elevation (ft.)
1	391.8
2	392.0
3	372.5
4	350.7
7	396.2
8	395.4
9	391.8

It can be seen that there is a difference in bedrock surface elevations of up to 45 feet over the investigated area.

Bedrock that was found was identified either as light grey sandstone or as red shale.

5. GROUND WATER CONDITIONS:

During the investigation, observations and measurements were carried out to determine the ground water conditions. In some of the boreholes, a stationary ground water level could be observed, and this was recorded. At greater depth, invariably artesian water was encountered and pressures corresponding to a water level of about 10 feet above ground level, were recorded. All this indicates that a perched water layer exists in the upper

cont'd. /6 ...

5. GROUND WATER CONDITIONS: (cont'd.) ...

soil material, while in the underlying material, artesian conditions prevail.

Given below, are elevations at which a static water level was first encountered and also elevations at which an artesian pressure was first observed:-

Borehole No.	Elevations - Ground Water	Elevations - Artesian Water
1	-	432.3
2	-	425.0
3	446.7	426.0
4	451.7	415.7
5	-	429.0
6	-	432.5
7	456.4	440.0
8	457.7	433.4
9	458.0	438.5

The difference in artesian water elevations - i.e., where artesian conditions were first noticed - is most probably due to certain permeability differences of the material in question. Due to more permeable seams and interbedded layers, the detectible artesian condition may vary quite considerably.

6. DISCUSSION AND RECOMMENDATIONS:

In the preceding paragraph, a quite detailed description of the materials and their most important properties was given. From this description, it can be seen that the subsoil is fairly good and that the structure can be built at the proposed location

cont'd. /7 ...

6. <u>DISCUSSION AND RECOMMENDATIONS</u>: (cont'd.) ...

without undue effort or expense. The density and strength of the pertinent layers are such that spread footings can be used. A safe maximum load of 2.0 tons per square foot is recommended for footings of an average width of 6 - 8 feet. The footings should be placed at least 5 feet below the existing ground level to provide for frost protection and to assure that no topsoil is left under the footings. Due to the predominantly granular character of the subsoil, settlements will be of short duration. As it was pointed out earlier, the depth of the different layers varies throughout the investigated area, and it is most likely that the settlements will be non-uniform. It is therefore recommended that simply supported beams be used in the design of the multi-span structure.

The investigation has shown that there is most probably a perched water table in the upper layer. It is impossible to predict what the water conditions will be during construction, and it is therefore recommended that measures be taken to de-water the foundation excavations if such a necessity should arise. In such a case, it is recommended to drive sheet piles around and below the bottom of the excavation for a distance equal to the distance of the ground water level above the bottom of the excavation. It is believed that the amount of water entering the excavation could be handled with ordinary sump pumps.

The alternative of founding the structure on friction piles was discarded because of the encountered artesian water conditions at greater depth. The cost of end-bearing piles

cont'd. /8 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...
becomes prohibitive because the length of the piles in places would be more than 100 feet.

No stability problems are foreseen in connection with the construction of the approach fills. If stage construction is feasible, it is strongly recommended. The influence of the approach fills on the bridge footings would thus be greatly decreased if not even completely eliminated.

7. SUMMARY:

1. The stratification of the subsoil is, in general, uniform although the thickness of the different layers varies throughout the investigated area. The layers and their properties are described in detail in the report.

2. Spread footings, 6 - 8 ft. wide, placed at a minimum of 5 feet below the present ground surface, and with a maximum load of 2.0 T/sq. ft., are recommended.

3. Simply supported beams are recommended for the multi-span structure because of the possible non-uniform settlements.

4. Stage construction, approach embankments being placed at least 6 months ahead, is recommended in order to eliminate the influence of the approach fills on the structure's footings.

5. In case of a high ground water table recommendations pertaining to the sheet piling as given in the report, should be followed.

6. No stability problems for the approach fills are anticipated.

cont'd. /9 ...

8. **MISCELLANEOUS:**

The field work was commenced on January 23, 1961, and completed by February 28, 1961, under the supervision of Mr. B. Ghadiali and Mr. M. Devata, of the Foundation Sub-section.

Equipment was owned and operated by D.H.O. drilling crew.

March 1961. REPORT PREPARED BY: B.M. Ghadiali

B. Ghadiali,
PROJECT FOUNDATION ENGR.

REPORT APPROVED BY: AGStermac

A. G. Stermac,
SUPERVISING FOUNDATION ENGR.

APPENDIX I.

FORM OB—ML—222
FEB 59
1M-58-5569

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 285-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
1	S1	3'-4.5'	Sand-gravel-clay silt. Fill material.	19	-	-	-	-	-	
	S2	6'-7.5'	Silty fine sand. Loose. Brown.	11	-	-	-	-	-	
	S3	10'-11.5'	Clay silt, fine sand & trace of fine gravels. Loose. Br. grey.	6	-	-	-	-	-	
	S4	15'-16.5'	Silty sand. V. dense. Br. red.	50	-	-	-	-	-	
	S5	20'-21.5'	Clay silt & trace of fine gravels. Dense. Brown.	39	19.5	-	-	-	-	
	S6	25'-26.5'	Silty sand. Dense. Br. red.	44	-	-	-	-	-	
	S7	30'-31.5'	Silty sand. Br. red.	2	-	-	-	-	-	
	S8	35'-36.5'	" "	2	-	-	-	-	-	
	S9	40'-41.5'	" "	10	-	-	-	-	-	

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 285-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
1	RC10	67'-72'	Sandstone bedrock. Light grey.	-	-	-	-	-	-	
	RC11	72'-77'	" " " "	-	-	-	-	-	-	
2	S1	3'-4.5'	Clay silt with sand-gravels-cinders. Med. dense. Brown.	18	-	-	-	-	-	
	S2	6'-7.5'	Clay silt with some sand & gravels. Dense. Brown.	45	15.7	-	-	-	-	
	S3	10'-11.5'	Clay silt & sand. Med. dense. Br. grey.	18	-	-	-	-	-	
	S4	15'-16.5'	" " " "	13	20.1	-	-	-	-	
	S5	20'-21.5'	Silty sand. Med. dense. Br. red.	29	-	-	-	-	-	
	S6	25'-26.5'	Sandy silt. Med. dense. Br. red.	29	12.5	-	-	-	-	
	S7	30'-31.5'	Silty fine sand. Dense. Br. red.	33	-	-	-	-	-	
	S8	35'-36.5'	Silty fine sand. Quick condition. Br. red.	P	-	-	-	-	-	
	S9	40'-41.5'	No Recovery.	P	-	-	-	-	-	Lost.
	RC10	63'-68'	Sandstone with interbedded shale. Grey.	-	-	-	-	-	-	

FORM OS—ML—222
FEB 59
1M-58-5569

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 285-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS, FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
3	S1	3'-4.5'	Clay sand-silt & organic material. Br. black.	16	-	-	-	-	-	
	S2	6'-7.5'	Clay silt & trace of organic material. Stiff. Grey.	24	-	-	-	-	-	
	S3	10'-11.5'	Clay silt & sandy silt. V. dense. Br. grey.	51	-	-	-	-	-	
	S4	15'-16.5'	Clay silt. Med. stiff. Grey.	16	18.5	14.7	25.1	-	132.0	
	S5	20'-21.5'	Clay silt & fine sand. Med. stiff. Grey.	12	17.0	-	-	-	-	
	T6	25'-26'	Clay silt & sandy silt. Med. dense. Grey-Br. red.	16	14.4	-	-	-	139.0	
	S7	30'-31'	Sandy silt. V. dense. Br. red.	86	-	-	-	-	-	
	S8	35'-36.5'	Silty sand. Dense. Br. red.	31	-	-	-	-	-	
	S9	40'-41.5'	Silty sand. Br. red.	P	-	-	-	-	-	
	RC10	80.7'-86'	Shale rock. Red & Grey.	-	-	-	-	-	-	47% Recovery

FORM OB—ML—222
FEB 59
IM-58-5869

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 285-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
4	S1	3'-4.5'	No recovery.	3	-	-	-	-	-	Lost.
	S2	7'-8.5'	Clay silt with sand & gravels. Med. dense. Grey.	21	-	-	-	-	-	
	S3	10'-11.5'	Clay silt with fine sand layers. Loose. Brown & grey.	13	22.1	-	-	-	-	
	S4	15'-16.5'	" " "	12	-	-	-	-	-	
	S5	20'-21.5'	Clay silt with some sand & gravels. Med. stiff. Br. grey.	9	17.1	13.2	22.2	-	-	
	S6	25'-26.5'	Silty sand. V. dense. Br. red.	54	-	-	-	-	-	
	S7	30'-31.5'	" " "	75	-	-	-	-	-	
	S8	35'-36.5'	Sandy silt. Dense. Br. red.	44	23.5	-	-	-	-	
	S9	40'-41.5'	Sandy silt. Br. red.	9	-	-	-	-	-	
	S10	50'-51.5'	Silty sand. Dense. Br. red.	35	-	-	-	-	-	

FORM OB—ML—222
FEB 59
1M-58-5869

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 285-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
4	S11	60'-61.5'	Silty sand & fine gravels. Dense. Br. red.	37	-	-	-	-	-	
	RC12	97.3'-101.3'	Sandstone boulder & sandy till V. dense. Grey & Br. red.	-	-	-	-	-	-	
	RC13	101.3'-105.5'	Shale bedrock. Red.	-	-	-	-	-	-	
5	S1	3'-4.5'	Silt sand & gravels. Dense. Brown	37	14.3	-	-	-	-	
	S2	6'-7.5'	Clay silt with sand & gravel in trace. Dense. Br. grey.	36	-	-	-	-	-	
	S3	9'-10.5'	" " " "	39	-	-	-	-	147.5	
	S4	12'-13.5'	" " " "	27	-	-	-	-	-	
	T5	15'-16.5'	Silty clay with some fine sand & gravels. Stiff. Br. grey.	P	20.3	17.8	28.6	1395 1760	133.2	
	S6	20'-21.5'	Silty clay & fine sand & gravel in trace. V. dense. Br. red.	66	-	-	-	-	-	

FORM OB—ML—222
FEB 59
1M-58-8869

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 285-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
6	S1	3'-4.5'	Clay silt & fill material. Soft. Br. grey.	2	-	-	-	-	-	
	S2	6'-7.5'	Sandy silt. Med. dense. Br. red.	23	-	-	-	-	-	
	S3	10'-11.5'	Sandy silt & silty sand. Med. dense. Br. red.	26	-	-	-	-	-	
	S4	15'-16.5'	Sandy silt & fine gravels. V. dense. Br. red.	60	-	-	-	-	-	
	S5	20'-21.5'	Silty sand & sandy silt. Dense. Br. red.	43	-	-	-	-	-	
	S6	25'-26.5'	Silty sand, sandy silt & fine gravels Dense. Br. red.	31	-	-	-	-	-	
	S7	30'-31.5'	" " "	P	-	-	-	-	-	
	S8	35'-36.5'	Sandy silt & fine gravels. Dense. Br. red.	36	-	-	-	-	-	
	S9	40'-41.5'	Silty sand & fine gravels. Br. red.	28	-	-	-	-	-	
	S10	45'-46.5'	" " "	10	-	-	-	-	-	
	S11	55'-56.5'	" " "	42	-	-	-	-	-	

FORM OS—ML—222
FEB 59
IM-59-5569

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 285-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.i.	UNIT WEIGHT p.c.f.	REMARKS
6	S12	60'-61.5'	Sand & gravels. Br. red.	P	-	-	-	-	-	
	RC13	66.5'-71.5'	Boulder stones.	-	-	-	-	-	-	
	RC14	72.7'-77.7'	Sandstone boulders with red shale.	-	-	-	-	-	-	
	RC15	77.7'-83.5'	" " "	-	-	-	-	-	-	
7	S1	3'-4.5'	Sand & gravel fill. Loose.	13	-	-	-	-	-	
	S2	6'-7.5'	Sandy silt & fine gravels. Med. dense. Brown.	28	-	-	-	-	-	
	S3	10'-11.5'	Clay silt & sandy silt. Dense. Br. grey.	29	16.4	-	-	-	-	
	S4	15'-16.5'	Clay silt & silty fine sand. Dense. Grey-Br. red.	26	-	-	-	-	-	
	S5	20'-21.5'	Sandy silt & silty sand. V. dense. Br. red.	56	-	-	-	-	-	
	S6	25'-26.5'	" " "	66	19.5	-	-	-	-	
	S7	30'-31.5'	Silty sand. Br. red.	2	-	-	-	-	-	

FEB 59
1M-58-5559

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 285-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
7	S8	35'-36.5'	Silty sand & fine gravels. Br. red.	8	-	-	-	-	-	
	S9	40'-41.5'	Silty sand. Br. red.	6	-	-	-	-	-	
	S10	50'-51.5'	" "	21	-	-	-	-	-	
	S11	60'-61.5'	" "	12	-	-	-	-	-	
	RC12	65'-70'	Sandstone bedrock. Light grey.	-	-	-	-	-	-	
8	S1	3'-4.5'	Clay silt, sand & organic material. Loose. Brown.	9	-	-	-	-	-	
	S2	6'-7.5'	Clay silt & sandy silt. Loose. Grey	6	18.5	-	-	-	-	
	S3	10'-11.5'	" " "	4	-	-	-	-	-	
	S4	15'-16.5'	Clay silt & sandy silt. Med. dense. Br. grey	13	23.4	-	-	-	-	
	S5	20'-21.5'	Silty sand & fine gravels. Dense. Br. red.	46	-	-	-	-	-	

FORM OB—ML—22.
FEB 59
1M-58-5569

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 285-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
8	S6	25'-26.5'	Silty sand. Br. red.	16	-	-	-	-	-	
	S7	30'-31.5'	" "	8	-	-	-	-	-	
	S8	35'-36.5'	Silty sand & sandy silt. Br. red.	5	-	-	-	-	-	
	S9	40'-41.5'	Silty sand. Br. red.	13	-	-	-	-	-	
	S10	50'-51.5'	Silty sand & sandy silt. Br. red.	13	-	-	-	-	-	
	S11	60'-61.5'	Silty sand & gravels. V. dense. Br. red.	91	-	-	-	-	-	
	RC12	63'-68'	Sandstone bedrock. Light grey.	-	-	-	-	-	-	
9	S1	3'-4.5'	Clay silt, sand & organic material. Med. dense. Brown.	13	-	-	-	-	-	
	S2	6'-7.5'	Sandy silt & silty clay. Stiff. Br. grey.	16	-	-	-	-	-	
	T3	10'-11.5'	" " "	P	16.0	12.4	26.1	525	136.1	

FORM OB—ML—222
FEB 59
1M-58-5869

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 285-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
9	VANE	10'		-	-	-	-	1200	-	Sens: 2.7
	S4	15'-16.5'	Silty fine sand. Med. dense. Br. red.	24	-	-	-	-	-	
	S5	20'-21.5'	Silty sand, sandy silt & gravels. Med. dense. Br. red.	26	-	-	-	-	-	
	S6	25'-26.5'	Sandy silt. Br. red.	15	-	-	-	-	-	
	S7	30'-31.5'	Silty sand & gravels in trace. Br. red.	12	-	-	-	-	-	
	S8	35'-36.5'	Silty sand, sandy silt & gravels. Br. red.	20	-	-	-	-	-	
	S9	40'-41.5'	Silty sand & gravels. Br. red.	12	-	-	-	-	-	
	S10	50'-51.5'	No Recovery.	28	-	-	-	-	-	Lost.
	S11	60'-61.5'	Silty sand & gravels. V. dense. Br. red.	75	-	-	-	-	-	
	RC12	63.7'-68.7'	Sandstone boulder.	-	-	-	-	-	-	

FORM OB-ML-126 (REV. 1959.) 59-5008

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 285-59
JOB 61-F-6
DATUM 458.8'
BORING DATE Jan. 23/61

BORE HOLE NO. 1
STATION 156+70 (58' Rt.)
COMPILED BY B.K.
CHECKED BY B.M.G.

LEGEND

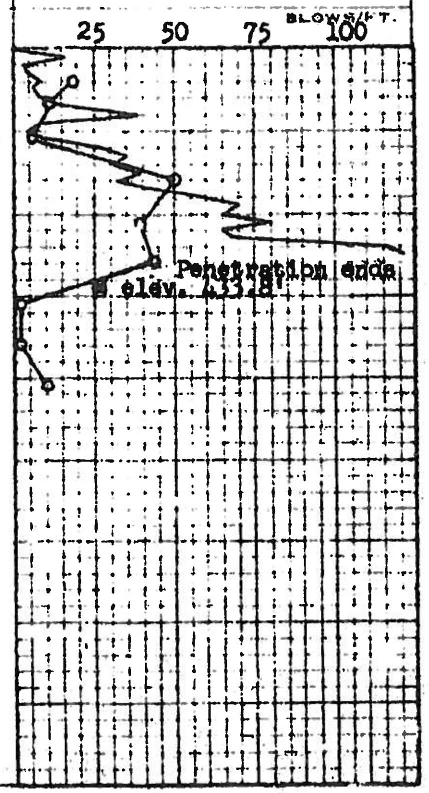
2" DIA SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

1/2 UNCONFINED COMPRESSION (Qu) O
VANE TEST (C) AND SENSITIVITY (S) +S
NATURAL MOISTURE AND LIQUIDITY INDEX LI X
LIQUID LIMIT
PLASTIC LIMIT

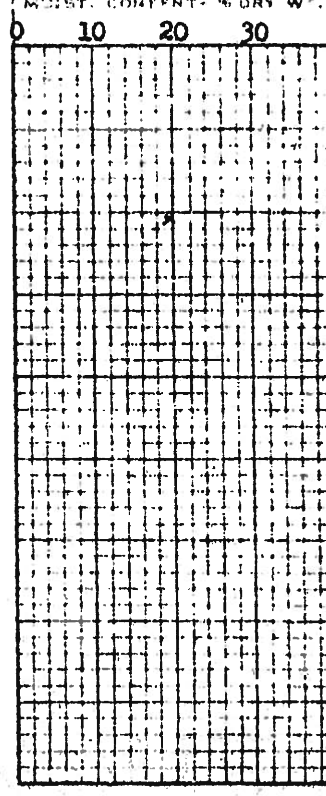
Symbol	Description	Elev. Feet	Depth Feet	Sample	Natural Unit Wt. P.C.F.
	↓ Groundlevel	458.8	0		
	Top Soil & Fill material	452.8		S1	-
	Clay silt, fine sand & trace of fine gravels. Loose. Br. Grey.	446.3	10	S2	-
	Silty sand. V. dense. Br. Red.	440.8		S3	-
	Clay silt & trace of fine gravels Dense. Br. Grey.	433.8	20	S4	-
	Artesian Condition	432.3		S5	-
	Silty sand. Dense. (Quick condition) Br. Red	416.8	30	S6	-
				S7	-
			40	S8	-
				S9	-
	Sandy fine gravels.	409.8	50		
	Silty sand. Dense to V. dense Br. Red	391.8	60		
	Sandstone bedrock. Light grey.	381.8	70	RC10	-
				RC11	-
	End of borehole.		80		

Penetration resistance profile shown; obtained by driving a 2" dia. cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow.

STRENGTH AND PENETRATION RESISTANCE — P.S.F.



CONSISTENCY



FEB 59
1M-58-5569

SUMMARY OF FIELD & LABORATORY TESTS

JOB 61-F-6
W.P. 28?-59

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.i.	UNIT WEIGHT p.c.f.	REMARKS
9	RC13	68.7'-73.7'	Sandstone bedrock. Light grey.	-	-	-	-	-	-	
			S denotes split spoon sample. T " shelby tube sample. RC " rock core.							

FORM 08-ML-126 (REV. 1959) -59-5008

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 285-59
JOB 61-F-6
DATUM 455.0'
BORING DATE Jan. 27/61

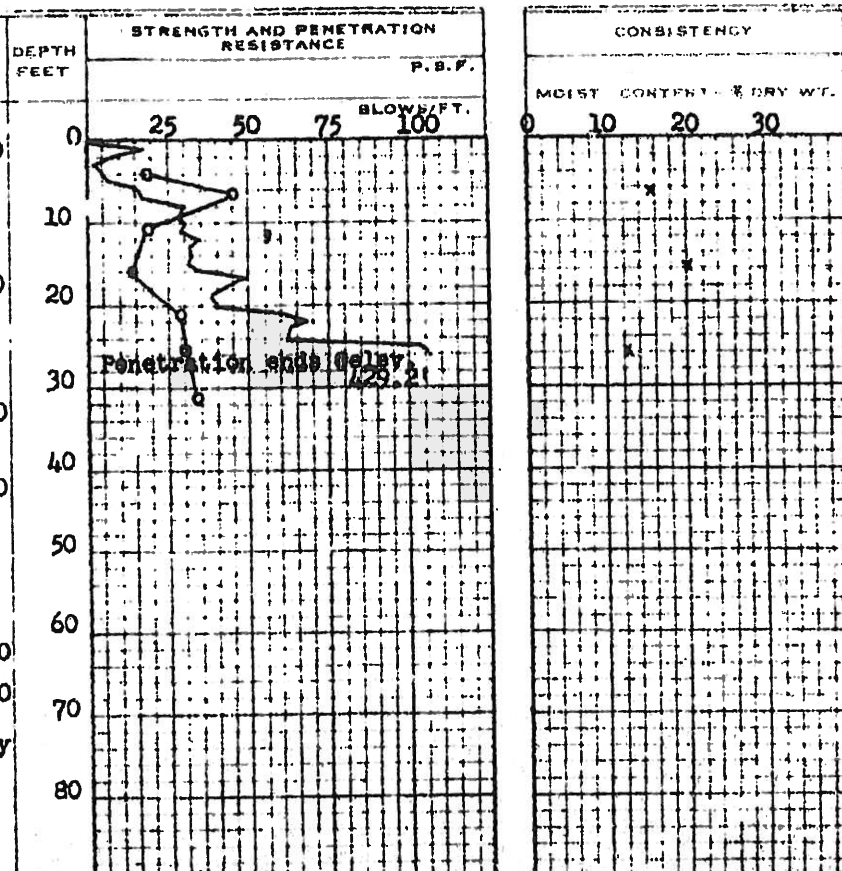
BORE HOLE NO. 2
STATION 155+48 (59' Lt.)
COMPILED BY B.K.
CHECKED BY B.M.G.

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) O
VANE TEST (C) AND SENSITIVITY (S) +S
NATURAL MOISTURE AND LIQUIDITY INDEX LI X
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	Groundlevel	455.0		
	Top Soil	454.0	S1	-
	Clayey silt & fine sand Fine gravels up to 8' depth Med. dense. Br. grey	437.0	S2, S3, S4	-
	Silty sand & sandy silt Dense Br. Red Artesian Condition	424.0	S5, S6, S7	-
		412.0	S8, S9	-
	Silty sand. Fine gravels up to 50' V. dense. Br. Red	392.0		
	Sandstone with interbedded shale. Grey	387.0	RC10	-
	End of borehole			



Penetration resistance profile shown; obtained by driving a 2" dia. cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow.

FORM OB-ML-126 (REV 1959) -59-5008

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 285-59
JOB 61-F-6
DATUM 453.2'
BORING DATE Jan. 31/61

BORE HOLE NO. 3
STATION 155+91 (62' Lt.)
COMPILED BY B.K.
CHECKED BY B.M.G.

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

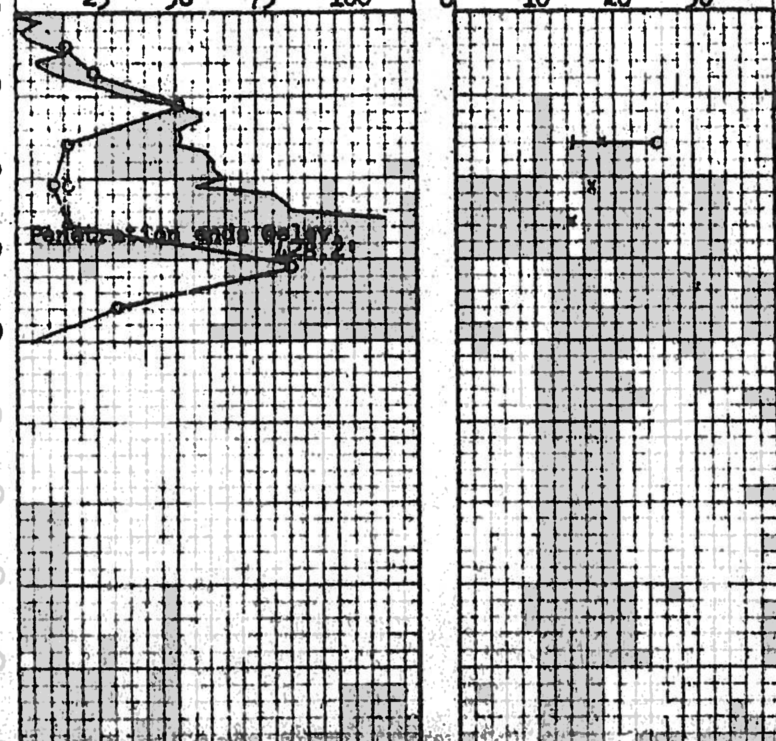
LEGEND

1/2 UNCONFINED COMPRESSION (Qu) — O
VANE TEST (C) AND SENSITIVITY (S) — +S
NATURAL MOISTURE AND LIQUIDITY INDEX — X LI
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	↓ Groundlevel	453.2	0		
	Top Soil & fill material. Brown to Clay-silt-sand & organic matter, black.	448.7		S1	-
	W.L.	446.7		S2	-
	Clay silt & sandy silt Stiff to v. stiff Br. grey to grey		10	S3	-
				S4	132.0
			20	S5	-
		427.7		T6	139.0
	Artesian Condition	427.2	30	S7	-
	Sandy silt, silty sand & some intermittent layers of gravels at further depths V. dense to dense Br. red			S8	-
			40	S9	-
			50		
			60		
			70		
		372.5	80		
	Shale bedrock Red & Grey	367.2		RU10	-
	End of borehole				

STRENGTH AND PENETRATION RESISTANCE — P.S.F. — BLOWS/FT. 25 50 75 100

CONSISTENCY — MOIST. CONTENT - % DRY WT. 0 10 20 30



Penetration resistance profile shown obtained by driving a 2" dia cone from groundlevel to depth noted with an energy of 50 ft / lbs per blow.

MATERIALS AND RESEARCH SECTION

W.P. 285-59
JOB 61-F-6
DATUM 452.7'
BORING DATE Feb. 2/61

BORE HOLE NO. 4
STATION 156/41 (66' Lt.)
COMPILED BY B.K.
CHECKED BY B.M.G.

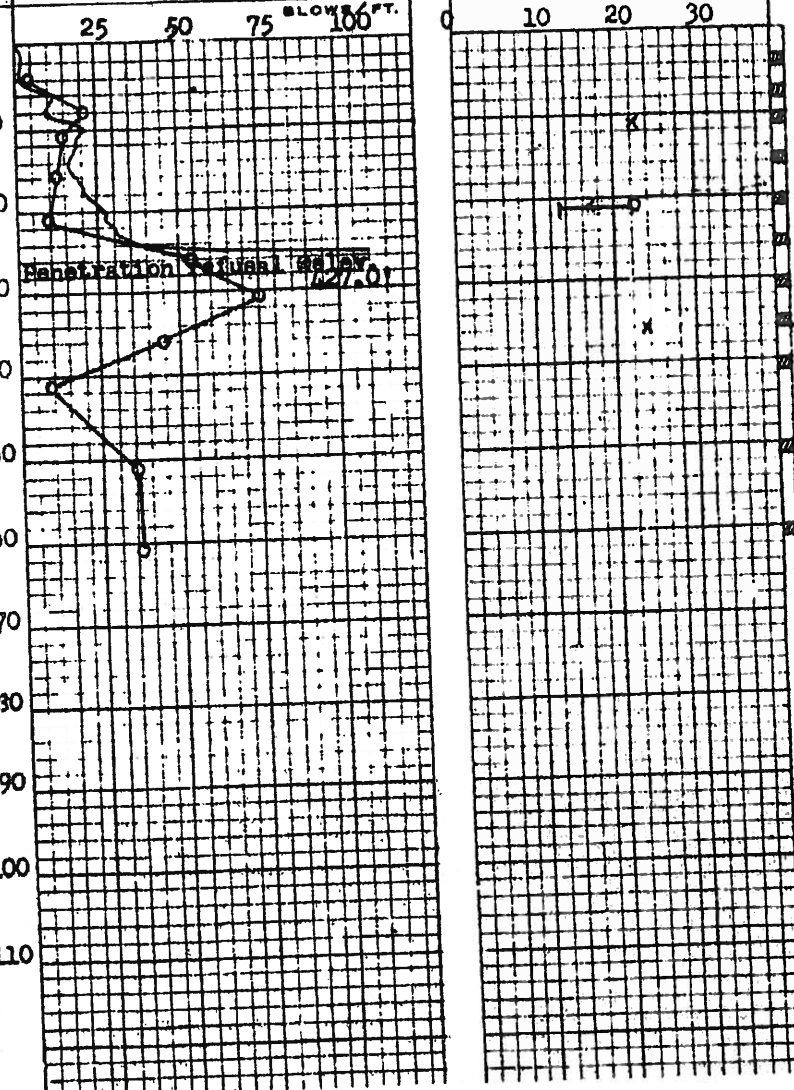
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	↓ Groundlevel	452.7	0
	Top Soil W.L.	451.7 / 450.0	
	Clay silt & sandy silt. Trace of fine gravels (15'to20') Loose to med. dense. Brown changing to grey with depth	427.7	10, 20
	Silty fine sand & sandy silt Dense to v. dense. Br. red. Artesian Condition	415.7 / 396.2	30, 40, 50
	Silty sand (coarse) & fine gravels. V. dense. Br. red.	355.4	60, 70, 80, 90
	Sandstone boulders & sandy till V. dense. Br. grey to Br. red.	350.7	100
	Shale bedrock. Red.	347.2	
	End of borehole.		110

Penetration resistance profile shown; obtained by driving a 2" dia. cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow.



SAMPLE	NATURAL UNIT WT. R. C. F.
S1	-
S2	-
S3	-
S4	-
S5	-
S6	-
S7	-
S8	-
S9	-
S10	-
S11	-
RC12	-
RC13	-

FORM OB-ML-126 (REV 1959) - 59-5008

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 285-59
JOB 61-F-6
DATUM 452.7'
BORING DATE Feb. 2/61

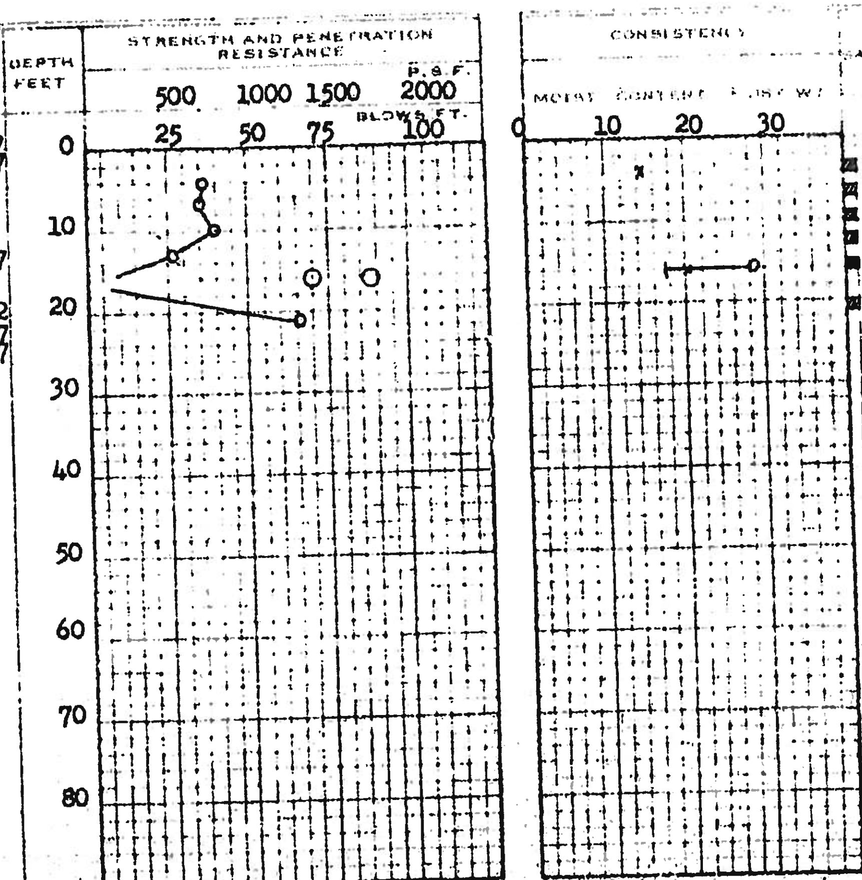
BORE HOLE NO. 5
STATION 156+57 ℄
COMPILED BY B.K.
CHECKED BY B.M.G.

2" DIA SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) O
VANE TEST (C) AND SENSITIVITY (S) +
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SOIL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	↓ Groundlevel	452.7	0
	Top Soil	451.7	
	Clayey silt with sand & gravels. (fine) Dense Br. Grey	438.7	
	Silty clay with some fine sand & gravels. Stiff. Br. Grey.	432.2	
	silty sand. V. dense Artesian Cond.	428.7	
	Br. Red.	427.7	
	End of borehole.		



SAMPLE	NATURAL UNIT WT. P.C.F.
S1	-
S2	-
S3	147.5
S4	-
T5	133.2
S6	-

FORM 06-ME-126 (REV. 1959) — 59-5008

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 285-59

JOB 61-F-6

DATUM 461.2'

BORING DATE Feb. 20/61

BORE HOLE NO. 7

STATION 156+24 (68' Lt.)

COMPILED BY B.K.

CHECKED BY B.M.G.

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2" DIA. CONE

2" SHELBY

CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) O

VANE TEST (C) AND SENSITIVITY (S) +S

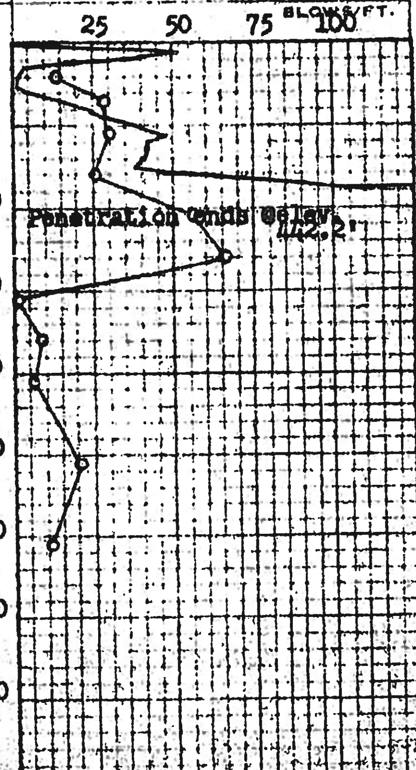
NATURAL MOISTURE AND LIQUIDITY INDEX LI X

LIQUID LIMIT

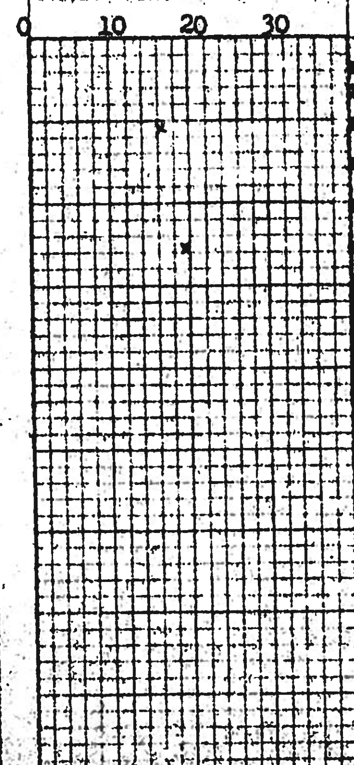
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	↓ Groundlevel	461.2	0
	Topsoil & fill material. Loose. Brown w.l. ▽	456.4 / 451.2	
	Clay silt & sandy silt. Trace of fine gravels. Med. dense to dense. Br. grey to grey.	445.2	10
	Artesian Condition	440.2	20
	Sandy silt changing to silty sand with depth. Trace of fine gravels. V. dense. Br. Red.	396.2	30, 40, 50, 60
	Sandstone Bedrock. Light grey.	391.2	70
	End of borehole. Penetration resistance profile shown; obtained by driving a 2" dia. cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow.		80

STRENGTH AND PENETRATION RESISTANCE — P.S.F. — BLOWS/FT.



CONSISTENCY — MOIST. CONTENT - % DRY WT.



SAMPLE	NATURAL UNIT WT. P.C.F.
S1	-
S2	-
S3	-
S4	-
S5	-
S6	-
S7	-
S8	-
S9	-
S10	-
S11	-
RC12	-

FORM OB-ML-126 (REV. 1959.) - 59-5008

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 285-59
JOB 61-F-6
DATUM 456.5'
BORING DATE Feb. 14/61

BORE HOLE NO. 6
STATION 157+33 (66' Rt.)
COMPILED BY B.K.
CHECKED BY B.M.G.

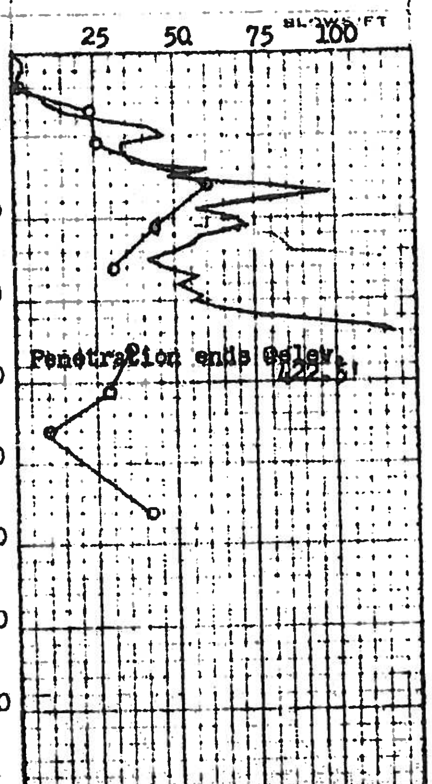
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Groundlevel	456.5	0
	Top Soil & fill. Soft Br. grey	451.5	
	Sandy silt & silty sand Some gravels. Med. dense to v. dense. Br. Red. Art. Cond.	433.5	
		420.5	
	Silty sand & gravels. V. dense. Br. Red.	390.0	
	Boulders & sandy till.	383.8	
	Sandstone boulders & red shale rock.	373.0	
	End of borehole.		

STRENGTH AND PENETRATION RESISTANCE — P.S.F. — BLOWS/FT



CONSISTENCY — MOIST. CONTENT % DRY WT.	SAMPLE	NATURAL UNIT WT. P.C.F.
	S1	-
	S2	-
	S3	-
	S4	-
	S5	-
	S6	-
	S7	-
	S8	-
	S9	-
	S10	-
	S11	-
	S12	-
	RC13	-
	RC14	-
	RC15	-

Penetration resistance profile shown; obtained by driving a 2" dia. cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow.

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 285-59
JOB 61-F-6
DATUM 458.4'
BORING DATE Feb. 22/61

BORE HOLE NO. 8
STATION 154+87 (66' Lt.)
COMPILED BY B.K.
CHECKED BY B.M.G.

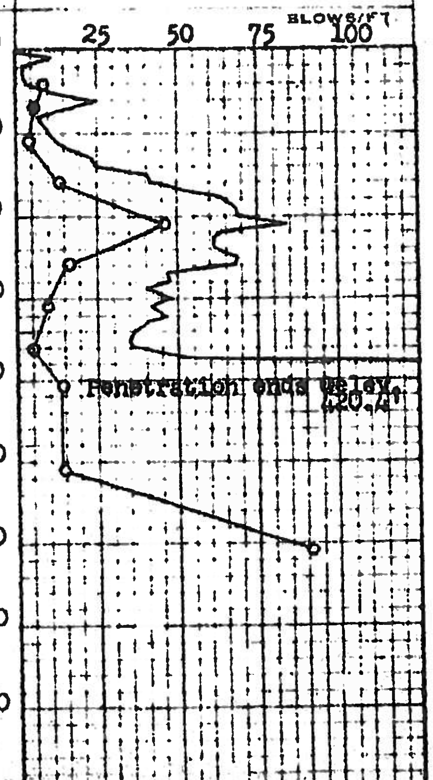
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) ○
VANE TEST (C) AND SENSITIVITY (S) +S
NATURAL MOISTURE AND LIQUIDITY INDEX LI X
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	↓ Groundlevel	458.4	0		
	Topsoil & fill material.	453.4		S1	-
	W.L.	452.4		S2	-
	Clayey silt & sandy silt Loose. Grey.		10	S3	-
		442.4		S4	-
	Sandy silt & silty sand.		20	S5	-
	Artesian Condition	433.4		S6	-
	Intermittent layers of fine gravels.		30	S7	-
	Med. dense to v. dense at further depths. Br. Red.			S8	-
			40	S9	-
			50	S10	-
			60	S11	-
		395.4			
	Sandstone bedrock Light grey.	390.4		RC12	-
	End of borehole. Penetration resistance profile shown; obtained by driving a 2" dia. cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow.		70		
			80		

STRENGTH AND PENETRATION RESISTANCE P.S.F.



CONSISTENCY

MOIST. CONTENT - % DRY WT.

0 10 20 30

FORM OB-ML-126 (REV. 1959) - 59-5008

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 285-59 BORE HOLE NO. 9

JOB 61-F-6 STATION 155/11 (45' Rt.)

DATUM 460.5' COMPILED BY B.K.

BORING DATE Feb. 27/61 CHECKED BY B.M.G.

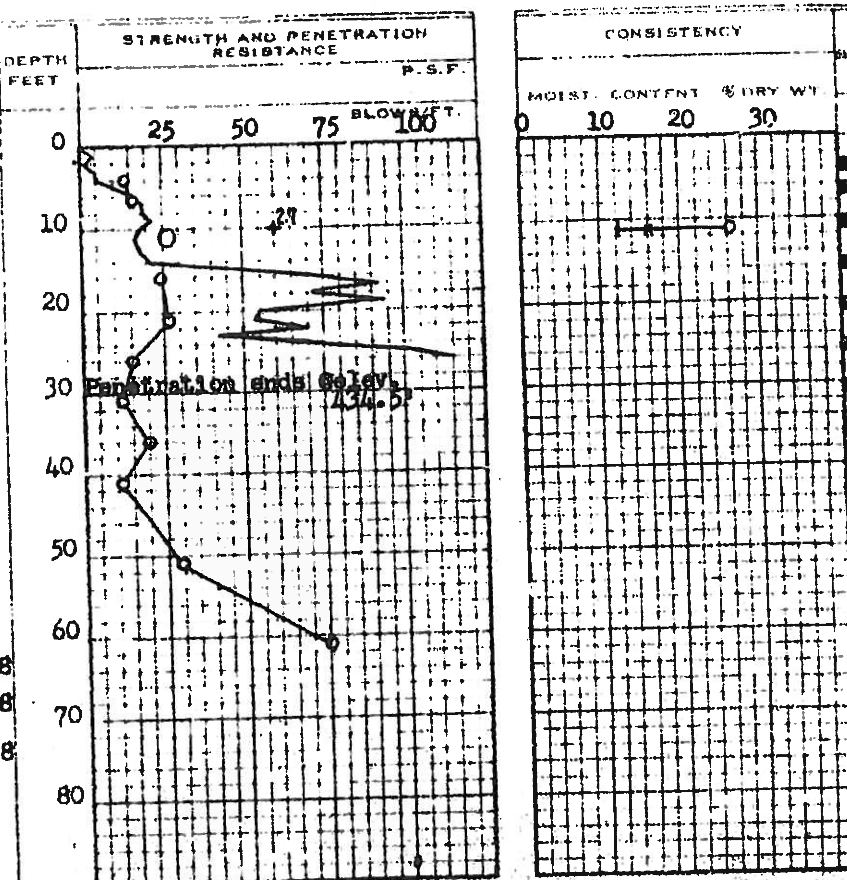
LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

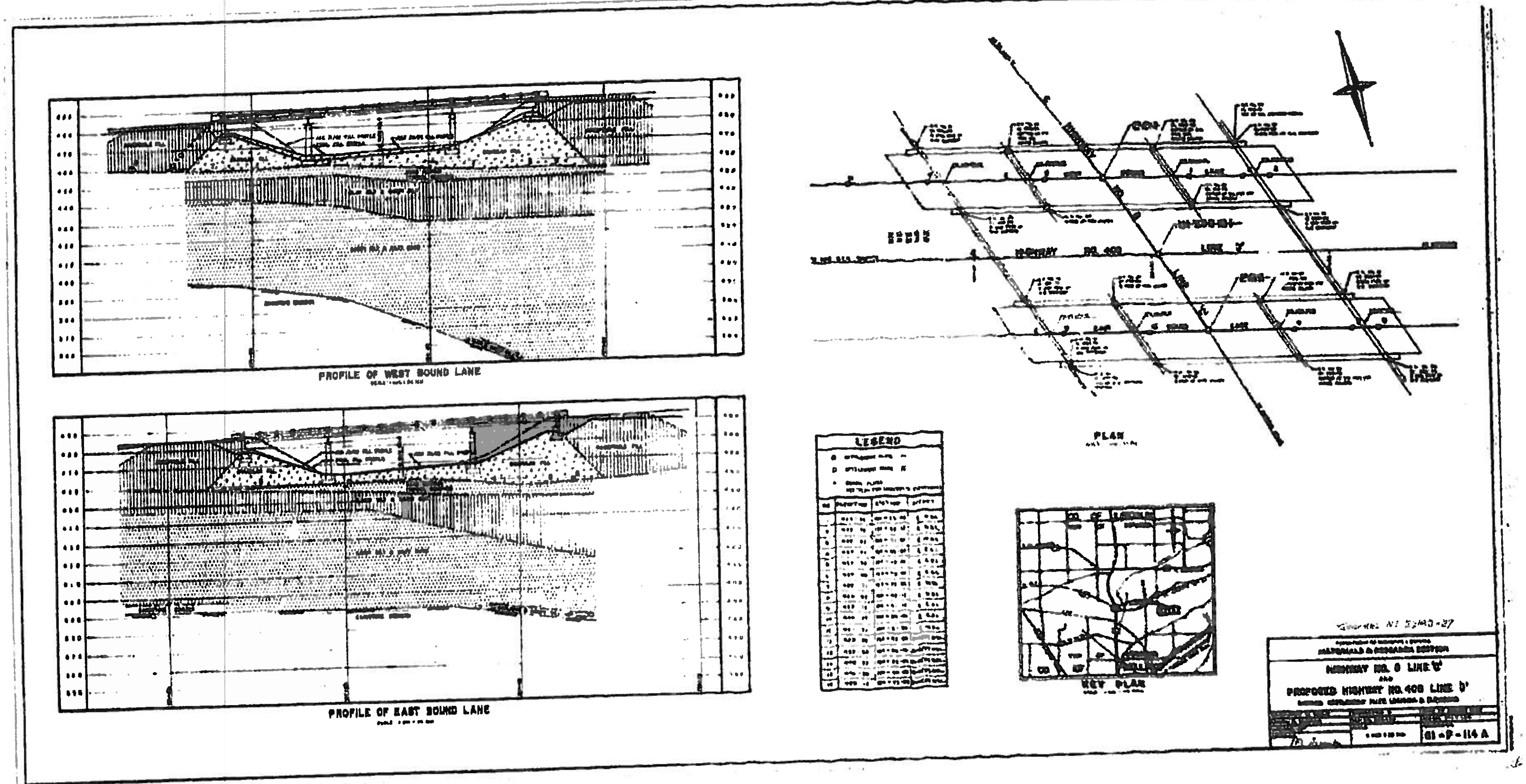
1/2 UNCONFINED COMPRESSION (Qu) — O
VANE TEST (C) AND SENSITIVITY (S) — +
NATURAL MOISTURE AND LIQUIDITY INDEX — X
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET
	Groundlevel	460.5
	Top Soil	459.0
	Sandy silt & silty sand. W.L. Some org. matter. Med. dense. Brown	458.0
	Silty clay & sandy silt. Stiff. Br. grey to grey.	453.5
	Sandy silt, silty sand & gravels. Med. dense to v. dense. Br. red.	446.5
	Sandstone boulder.	396.8
	Sandstone bedrock. Light grey.	391.8
	End of borehole.	386.8

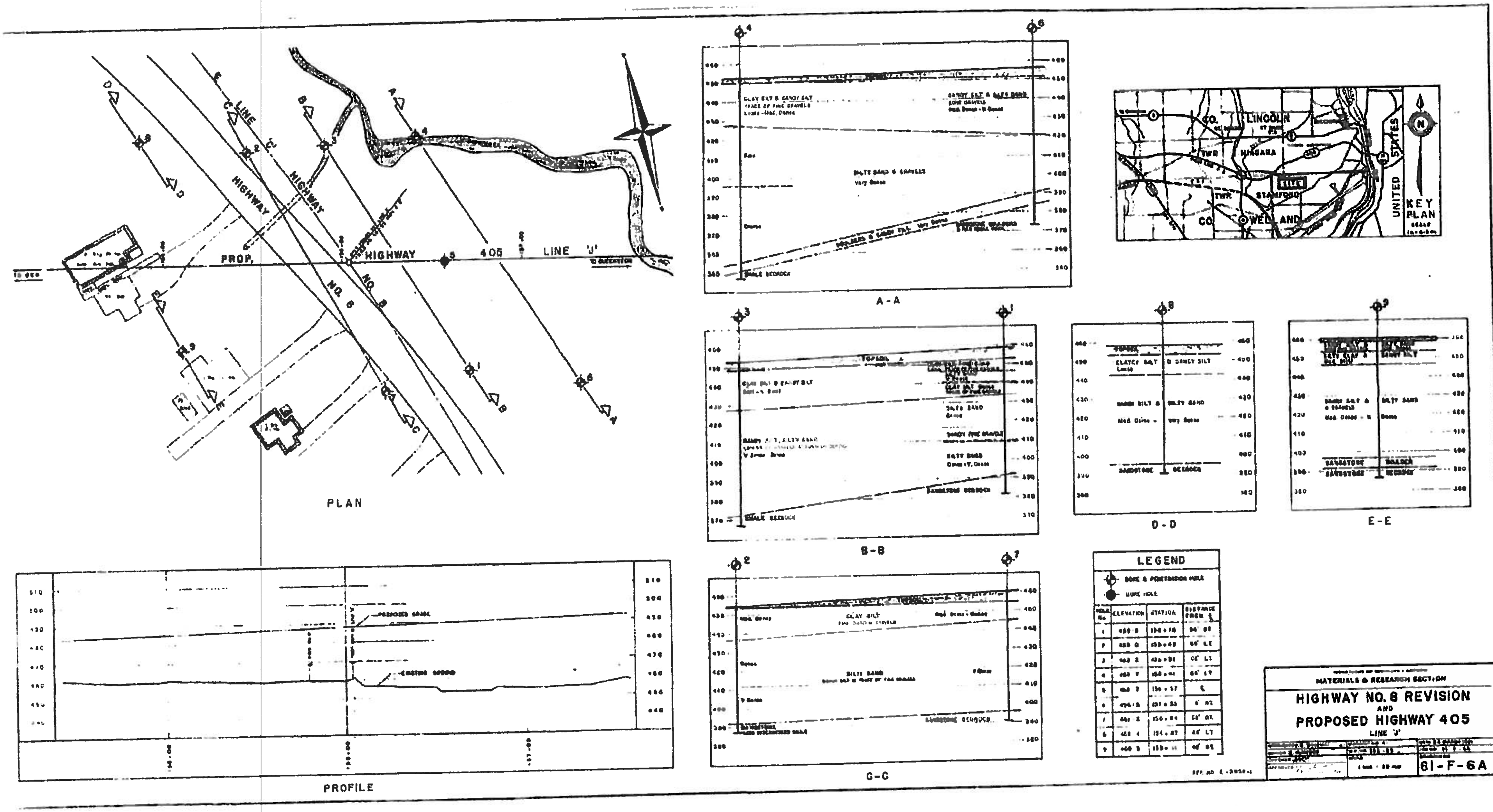
Penetration resistance profile shown; obtained by driving a 2" dia. cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow.



SAMPLE	NATURAL UNIT WT. P.C.F.
S1	-
S2	-
T3	136.1
S4	-
S5	-
S6	-
S7	-
S8	-
S9	-
S10	-
S11	-
RC12	-
RC13	-



SOME DEFECTS IN NEGATIVE DUE
TO CONDITION OF ORIGINAL DOCUMENTS


PLAN
PROP. HIGHWAY 405 LINE 'J'
HIGHWAY NO. 8
LINE
PROFILE
PROPOSED GRADE
EXISTING GROUND
A - A
B - B
C - C
D - D
E - E
KEY PLAN
UNITED STATES
LINCOLN
NIAGARA
STAMFORD
WELLAND
LEGEND
BORE HOLE
MATERIALS & RESEARCH SECTION
HIGHWAY NO. 8 REVISION
AND
PROPOSED HIGHWAY 405
LINE 'J'
61-F-6A

#61-F-6
W.P. #285-59
Hwy. #8
OVERPASS &
Q.E.W. EXT.
(Hwy. #405)