6th Line Municipal Class Environmental Assessment

County Road 27 to St John's Road
Town of Innisfil, ON

September 6, 2016

APPENDIX G:
DESKTOP STUDY FOUNDATION ASSESSMENT

TECHNICAL MEMORANDUM

DATE	September 24, 2015	**PROJECT No.**	1413283
TO	Cheryl Murray, P.E., Highway Practice Lead HDR Corporation		
CC			
FROM	Kevin J. Bentley, P.Eng.	**EMAIL**	kbentley@golder.com

DESKTOP STUDY FOUNDATION ASSESSMENT
6TH LINE AND CN RAIL OVERHEAD STRUCTURE WIDENING / REPLACEMENT
6TH LINE MUNICIPAL CLASS ENVIRONMENTAL ASSESSMENT
TOWN OF INNISFIL, ONTARIO

Golder Associates Ltd. (Golder) has been retained by HDR Corporation (HDR) on behalf of the Town of Innisfil to provide foundation engineering services in support of the Municipal Class Environmental Assessment (Class EA) for proposed improvements (e.g. widening) of 6th Line from County Road 27 to St. John's Road in Innisfil, Ontario.

This technical memorandum summarizes the results of a desktop study of available subsurface information and a limited site investigation, and provides preliminary foundation recommendations for the proposed widening/replacement of the existing 6th Line/CN Rail (or Go Rail) Overhead structure. The preliminary geotechnical / foundation recommendations provided in this technical memorandum are based on limited information about the project and are intended for planning purposes only.

The reader is referred to the attached "Important Information and Limitations of This Report" that follows the text of this technical memorandum and forms an integral part of this document.

1.0 PROJECT BACKGROUND

Currently 6th Line is a 2-lane road with a posted speed of 80 km/h. Based on predicted future uses, the segment of roadway between 20 Sideroad to St. John's Road (approximately 3 km in length, and including the planned Sleeping Lion Development) is anticipated to have future urbanized characteristics, while the segment from County Road 27 to 20 Sideroad (approximately 12 km in length, with mostly agricultural properties) will operate as a rural section.

Based on the recommendations from the 2013 Transportation Master Plan, and additional assessment conducted through this EA study, the Town is proposing to widen 6th Line, between 20th Sideroad and St. John's Road, from a 20 m 2-lane local rural road to a 26-30 m wide 4-lane urban major collector road, and proposing to reconstruct 6th Line, between County Road 27 and 20th Sideroad, from a 20 m 2-lane local rural road to a 2-lane rural arterial road with paved shoulders and 30 m right-of-way protection.

In addition to confirming the cross section and preliminary conceptual design of the roadway, the study will review the need for the following corridor features:

- Bike lanes or multi-use trails;
- Potential need for a future interchange at Highway 400;
- New structure or structure widening over the existing GO rail line;
- Intersection improvements.

2.0 SITE DESCRIPTION

The existing 6th Line / CN Rail(or Go Rail) Overhead structure is located about 1 km east of 20th Sideroad and about 2 km west of St. John's Road, in the Town of Innisfil, Ontario (see Key Plan on Figure 1). The existing structure is a three-span bridge that carries the 6th Line over a single railway track (see Photograph 1 below).

The existing ground surface surrounding the bridge structure is generally flat and is occupied by farm fields. The CN Rail line appears to have been constructed in cut, given the depressed profile in comparison to the grade of the surrounding farm fields (see Photograph 2 below).

Based on a digital terrain model provided by HDR, the 6th Line road surface at the bridge is at approximately Elevation 249.7 m (Geodetic Datum). The ground surface adjacent to the roadway and bridge generally ranges between Elevation 249 m and 250 m. The ground surface at the CN rail level is generally ranges between Elevation 242 m and 243 m.

Photograph 1: Looking West from South Side

Photograph 2: Looking East on South Side

3.0 REVIEW OF AVAILABLE INFORMATION

A desktop study search of available subsurface information at the MTO GEOCRES library found an existing geotechnical report that was prepared for a previous bridge replacement at the site. The report titled “Foundation Investigation, Canadian National Railway Overpass, Sixth Line Road, Township of Innisfil, MTO GEOCRES No. 31D-153”, dated December 1968, was prepared by Dominion Soil Investigation Limited. A copy of the factual portion of the geotechnical report is included in Appendix A.

Based on the 1968 report, we understand that the current three-span bridge replaced a previous five-span timber structure at the site. Referring to the inferred subsoil profile drawing included in the report, the existing bridge approaches were constructed as fill embankments about 5 m above the surrounding grade at the abutments and the piers were located within the CN Rail corridor which is shown to have been constructed in about 3 m of cut. As part of the previous geotechnical investigation for the bridge replacement, a total of four (4) boreholes were drilled and two (2) Dynamic Cone Penetration Tests were advanced at the site. A copy of the Borehole Logs and plan drawing showing the borehole locations is provided in Appendix A. It is noted that the borehole elevations provided in the 1968 report are referenced to a local benchmark and are not consistent with the datum used for the current ground surface elevations that are referenced to Geodetic datum. A summary of the subsurface conditions encountered at the borehole locations are summarized in Section 4.0.

4.0 CURRENT INVESTIGATION

As part of the pavement investigation for the current project (provided in a separate report), one deep borehole (designated BH104) was advanced on the south side of 6th Line near the existing overhead structure as shown on Figure 1. The field work was carried out by Golder on April 14, 2015 during which time Borehole BH104 was advanced using a CME-45D truck mounted drill rig, supplied and operated by KC Drilling Ltd. from Innisfil, Ontario. The borehole was advanced using 154 mm diameter solid stem augers and soil samples were obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586).

The groundwater condition in the open borehole was observed throughout the drilling operations and a piezometer was installed in Borehole BH104 to permit monitoring of the groundwater level at the site.

The field work was observed on a full-time basis by a member of Golder's technical staff who located the borehole in the field, arranged for the clearance of underground utilities, directed the drilling, sampling and in situ testing operations, and logged the borehole. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Barrie for further examination and laboratory testing. Index and classification tests (water contents and grain size distributions) were carried out on selected soil samples.

The borehole location was measured relative to on-site features and the ground surface elevation was obtained from the Digital Terrain Model for the site provided by HDR and should be considered approximate. The location (referenced to the project Stationing system) and elevation of the borehole (referenced to Geodetic datum) are summarized below.

Borehole No.	Location Station	Description	Ground Surface Elevation (m)	Borehole Depth (m)
BH104	23+358 on 6th Line	1.50 m Right of Centre Line	249.2	8.0 m

The detailed subsurface soil and groundwater conditions encountered in the borehole advanced as part of the current investigation and the results of in situ and laboratory testing are given on the Record of Borehole sheet and Figure 2 following the text of this technical memorandum.

5.0 SUBSURFACE CONDITIONS

Based on the previous boreholes advanced in 1968 (Borehole Nos. 1, 2, 3 and 4) and Borehole BH104 advanced during the current study, the subsurface conditions at the site generally consist of a thin layer of topsoil underlain by a sand and silt till to sandy silt till deposit. At the approach embankment location, a surficial layer (about 25 mm thick) of pavement surface treatment underlain by granular fill (about 0.5 m thick) was present, underlain by clayey silt to sand and silt fill which is underlain by the sand and silt till deposit.

The fill was encountered to depths of 3.1 m and 5.6 m below road surface at Borehole 1 and BH104 respectively, which were both located at the bridge approach embankment. The fill contained topsoil inclusions and cobbles / boulders were inferred at a depth of about 3.7 m below ground surface (Elevation 245.5 m) in BH104. The Standard Penetration Test (SPT) 'N'-values measured within the sandy clayey silt fill ranged between 32 and 50 blows per 0.13 m of penetration suggesting a hard consistency. The SPT 'N'-values measured within the silt and sand fill ranged between 4 and 18 blows per 0.3 m of penetration indicating a very loose to compact relative density. The natural water content of the fill soils ranges from about 7 per cent to 14 per cent.

The sand and silt till to sandy silt till deposit extends to the termination of the boreholes which ranged from a depth of 6.6 m to 8.1 m below ground surface. The glacial till deposit is described as containing variable amounts of clay and gravel, and contains cobbles and boulders. The SPT ‘N’-values measured within the silt and sand till to sandy silt till deposit ranged between 26 blows per 0.3 m and greater than 50 blows per 0.1 m of penetration indicating a compact to very dense relative density, but generally a dense to very dense relative density. A grain size distribution performed on a sample of the glacial till is shown on Figure 2 and indicates the predominant silt and sand content of the deposit. The natural water content of the glacial till ranges from about 6 per cent to 11 per cent.

The Log/Record of Borehole sheets indicates Boreholes 1, 4, and BH104 were dry on completion of drilling operations. Boreholes 2 and 3 indicate groundwater was measured at depths of 4.3 m and 3.2 m below ground surface (i.e., about 1.5 m below the CN rail elevation at the time of the investigation) shortly after drilling operations in October 1968. It is noted that the Datum used for groundwater elevations in the 1968 investigation report is referenced to a local benchmark. The groundwater levels measured upon or shortly after completion of drilling are not considered to have stabilized.

A standpipe piezometer was installed in BH104 and was screened within the sand and silt till unit. The groundwater level was measured at a depth of 6.2 m below ground surface (Elevation 243.0 m) on April 28, 2015. Although the groundwater level was measured about two weeks after drilling operations, groundwater levels are anticipated to fluctuate throughout the season will be higher during the Spring season and during periods of precipitation and snowmelt.

6.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

We understand that HDR / Town of Innisfil require preliminary foundation recommendations for planning purposes for rehabilitating / replacing the existing 6th Line / CN Rail Overhead structure as part of the widening / improvements to 6th Line in this area.

The geotechnical comments and recommendations provided herein are based on interpretation of the factual data available from the existing borehole information obtained from a desktop study and limited current investigation at the site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary for conceptual design of the EA study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at the bridge site as the design progresses.

It is important to note that no design or construction drawings for the existing bridge and/or foundations were available at the time this technical memorandum was prepared. The type of foundations used for the existing bridge are currently unknown and should be confirmed prior to detail design as the existing foundation type, size, and depth may influence the proposed widening / replacement foundation options.

6.1 Preliminary Foundation Options

Underlying the topsoil and fill materials, the native subsoils at the 6th Line / CN Rail Overpass site generally consist of dense to very dense silt and sand till to sandy silt till. Based on these subsurface conditions, it is recommended that new foundations (i.e., abutments and piers) for the proposed widened / replacement structure be founded on spread footings placed on the dense to very dense silt and sand till to sandy silt till.

Consideration could be given to the use of perched abutments founded on spread footings placed on a compacted granular pad above the dense to very dense glacial till soils within the approach embankments;

however, spread footings founded on the existing fill soils is not considered an option given that the existing fill soils are variable in composition are contain pockets of organics.

Alternatively, abutment and pier foundations could be supported on steel H-piles driven into the very dense silt and sand till to sandy silt till. Difficulties penetrating through the glacial till soils and fill containing cobbles/boulders should be expected and pre-augering techniques may be required to achieve a minimum pile embedment length (especially at pier locations).

For preliminary design, spread footings founded on the dense to very dense silt and sand till to sandy silt till may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 700 kPa and geotechnical resistance at Serviceability Limit States (SLS) of 450 kPa (for 25 mm settlement). The preliminary geotechnical resistances provided above assume a 3 m wide footing and assume there will be no influence from the existing bridge configuration / foundations. The footings should be placed as high as possible within the dense to very dense glacial till, provided a minimum 1.5 m of soil cover is provided to protect the founding subgrade against frost penetration, to reduce temporary dewatering efforts during construction of the footings. Groundwater conditions at the foundation locations will have to be confirmed during the detail design stage but are not anticipated to be a major concern given that the groundwater level recorded in the piezometer in Borehole BH104 was measured to be below the top of the dense to very dense silt and sand till layer.

For preliminary design, steel HP310x110 piles driven through the fill soils and compact native soils to a minimum 3 m embedment into the very dense silt and sand till deposit (where SPT ‘N’ values are greater than 100 blows / 0.3 m of penetration) may be designed using a factored axial geotechnical resistance at ULS of 900 kN and at SLS (for 25 mm of settlement) of 600 kN. As previously discussed, the presence of cobbles/boulders may result in difficulties achieving the target penetration depth to achieve the design capacities and/or minimum pile embedment for the structure type and pre-augering or perched abutments may need to be considered. The base of pile caps should have a minimum 1.5 m of soil cover to protect against frost penetration.

The settlement of any new foundations will be dependent on the footing size and configuration, and on the applied loads. This settlement will be differential with respect to the existing overpass structure foundations if consideration is being given to leaving the existing structure in place. As previously mentioned, the existing footing types, sizes and configuration should be confirmed prior to detail design. In addition, foundation / protection systems located within the CN right-of-way will likely need to be designed using the AREMA guidelines in addition to the Canadian Highway Bridge Design Code (CHBDC).

7.0 CLOSURE

This technical memorandum was prepared by Mr. Qasim Cheema, P.Eng., a geotechnical engineer with Golder. Mr. Kevin J. Bentley, P.Eng., a geotechnical engineer and Associate with Golder provided a senior review of the technical memorandum. We trust the above information meets with your current requirements, but should you have any questions, please do not hesitate to contact us.

GOLDER ASSOCIATES LTD.

Qasim

Qasim Cheema, P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
SEPT. 24, 2015
K. J. BENTLEY
PROVINCE OF ONTARIO

Kevin J. Bentley, P.Eng.
Geotechnical Engineer, Associate

QC/KJB/rb

Attachments:

Important Information and Limitations of This Report
Method of Soil Classification, Abbreviations and Terms, List of Symbols
Record of Borehole BH104
Figure 1 – Borehole Location Plan
Figure 2 – Grain Size Distribution – Silt and Sand Till
Appendix A – Previous Geotechnical Investigation

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ATTACHMENTS

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder can not be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

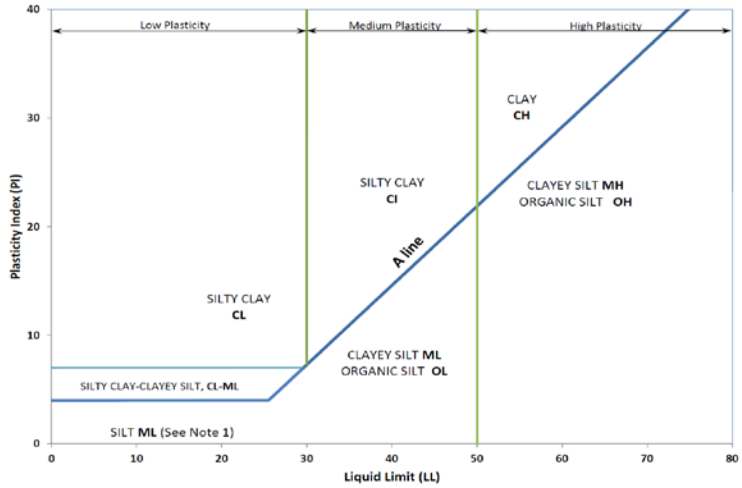
Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators					Organic Content	USCS Group Symbol	Primary Name
				Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)			
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% **(see Note 2)**	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.
Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.
For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.
For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.
A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to ~~or~~ indicate~~s~~ a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL, SAND and CLAY)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.).

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
FS	Foil sample
GS	Grab Sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size
TP	Thin-walled, piston – note size
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 - 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects.
2. Definition of compactness descriptions based on SPT 'N' ranges from Terzaghi and Peck (1967) and correspond to typical average N_{60} values.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson’s ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l – w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w – w_p) / I_p$
I_C	consistency index = $(w_l – w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} – e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 0 analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes:

1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

---

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

PROJECT: 1413283

RECORD OF BOREHOLE: BH104

SHEET 1 OF 1

LOCATION: 23+358 1.50 m Rt of C/L

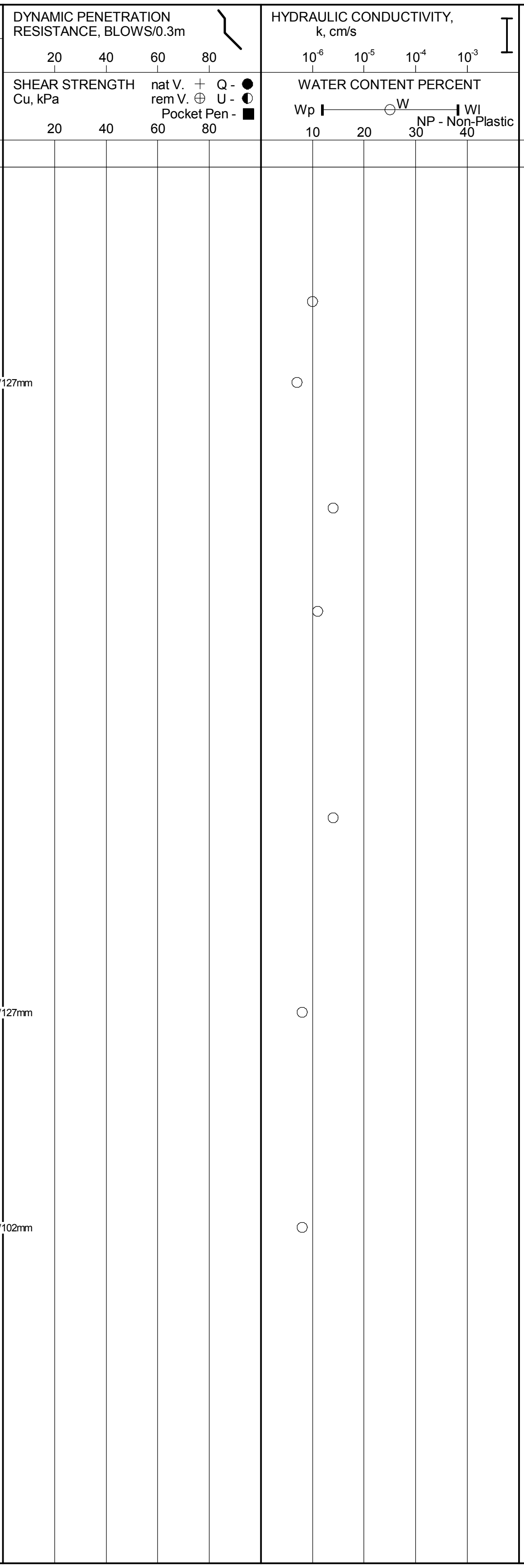
BORING DATE: April 14, 2015

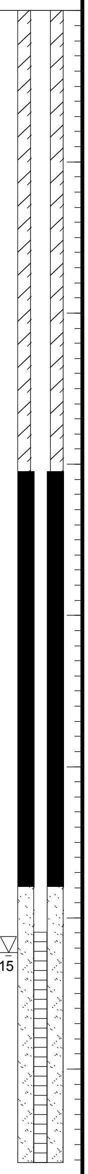
DATUM: Geodetic

SPT Hammer: Mass, 140lbs.; DROP, 30in.

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface	249.21					
0	Truck Mounted CME 45D 150 mm O.D. Solid Stem Auger with SPT Sampling	SURFACE TREAMENT	0.03	1	GS			
		FILL-(SW-GW) SAND and GRAVEL, crushed, some silt; brown; (BASE), non-cohesive, moist, compact	0.15	2	GS			
		FILL-(SM) gravelley SILTY SAND; brown; (SUBBASE), non-cohesive, moist, compact	248.68 0.53					
1		FILL-(CL-ML) Sandy CLAYEY SILT, slight plasticity, some sub-angular to sub-rounded gravel; brown, oxidation staining; cohesive, w<PL, hard		3	SS	32		
				4	SS	50/127mm		Cuttings
2				5	SS	43		
3		Brown to black; below 3.05 mbgs		6	SS	39		
		Suspected boulders and cobbles at 3.66 mbgs						
4		FILL-(ML/SM) SILT and SAND, trace sub-angular to sub-rounded gravel, some clay; light brown to black; non-cohesive, moist, compact	245.17 4.04					Bentonite
5				7	SS	18		
		(ML/SM) SILT and SAND, trace sub-rounded gravel, some clay; light brown to grey; (TILL), non-cohesive, moist, very dense	243.65 5.56					Silica Sand
6				8	SS	50/127mm	MH	GR=3% SA=45% SI=44% CL=8% Apr. 28/15
7								Screen
				9	SS	50/102mm		
8		End of Borehole.	241.18 8.03					
9								
10								

NOTE:
1. Free water not encountered in open borehole upon completion of drilling.
2. Free water measured at 6.23 mbgs in peizometer on April 28, 2015
3. Auger Refusal on suspected boulders and cobbles at 3.66 mbgs. Drilled second hole and continued sampling 2 m east.





DEPTH SCALE

1 : 50

LOGGED: DM

CHECKED: KSB

(KEY PLAN SCALE 1:200000 m)

0 50 100 150

1:2500 METRES

LEGEND

⌖ Approximate Borehole Location
— Road
— Watercourse
🟦 Water Area, Permanent
▨ Wetland, Permanent

REFERENCE

Datum is UTM NAD 83 Zone 17
OBM Features ESRI Geography Network
Satellite Imagery Simcoe County GIS, 2012

CLIENT
HDR

PROJECT
6th LINE FROM COUNTY ROAD 27 TO ST. JOHN'S ROAD,
INNISFIL, ONTARIO

TITLE
BOREHOLE LOCATION PLAN

CONSULTANT
Golder Associates

YYYY-MM-DD	2015-07-02
PREPARED	STB
DESIGN	
REVIEW	DM
APPROVED	KJB

PROJECT No.	Phase	Rev.	Figure
141-3283	-	AA	1

Path: \golder.gds\gal\Barrie\CAD□rojects\2014\141-3283 (Innisfil, 6th Line Contaminant Overview)\-BB-\ | File Name: 1413283BBOBM.dwg

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A

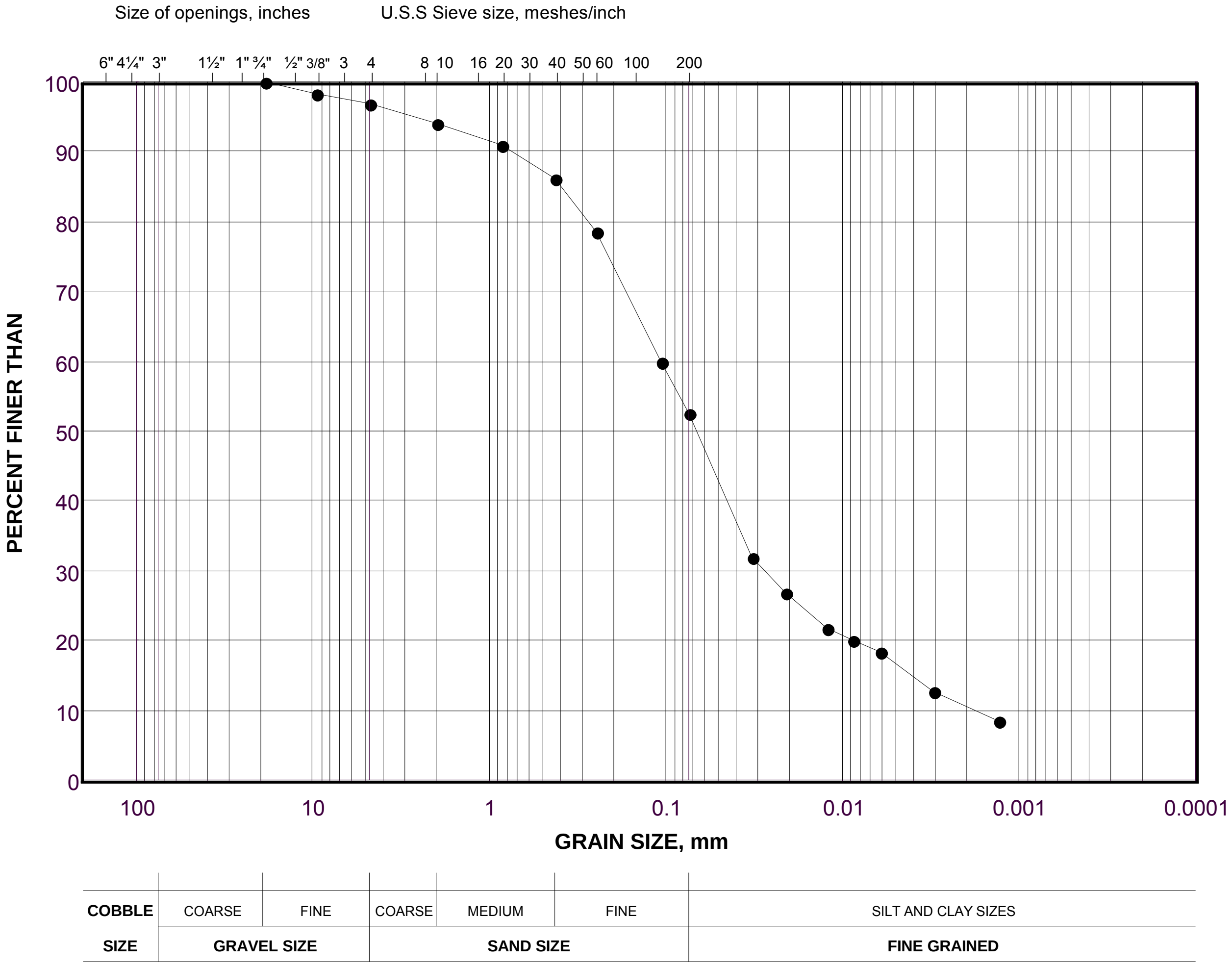
25 mm

0

GRAIN SIZE DISTRIBUTION

(ML/SM) SILT and SAND

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	BH-104	SS8	6.10 - 6.55

Project Number: 1413283

Checked By: ______________________

Golder Associates

Date: 14-May-15

APPENDIX A

Previous Geotechnical Investigation

MTO GEOCRES NO. 31D-153

[illegible]
Site 30-413

DOMINION SOIL INVESTIGATION LIMITED
CONSULTING SOIL & FOUNDATION ENGINEERS

HEAD OFFICE
104 CROCKFORD BLVD.
SCARBOROUGH, ONT.
CANADA
TELEPHONE: 751-6565

BRANCH OFFICE
369 QUEENS AVE.
LONDON, ONT.
TELEPHONE: 433-3651

ASSOCIATED COMPANY
SOIL TESTING AND ENGINEERING LTD.
39 BRENTFORD ROAD
KINGSTON 5, JAMAICA
WEST INDIES

Our Ref. No: 8-10-11
December 1968.

FOUNDATION INVESTIGATION

CANADIAN NATIONAL RAILWAY OVERPASS

SIXTH LINE ROAD, TOWNSHIP OF INNISFIL

PREPARED FOR:

AINLEY & ASSOCIATES LIMITED
CONSULTING ENGINEERS
104 HURONTARIO STREET
COLLINGWOOD, ONTARIO.

Distribution: 5 copies - Ainley & Associates Limited
2 copies - Dominion Soil Investigation Limited

Our Ref. No: 8-10-11

CONTENTS

ENCLOSURES

1.0. INTRODUCTION

In a letter dated 4th October 1968 Mr. W.M. Trotter, P.Eng. of Ainley and Associates, requested that Dominion Soil Investigation Limited carry out a foundation investigation for a 120 ft. span road bridge.

The bridge on Line 6 Road, Township of Innisfil will replace the existing old narrow wooden structure which crosses the Canadian National Railways line.

This report describes all aspects of the investigation and test work. Foundation conditions are discussed and recommendations are made for foundation design.

2.0. THE SITE

The site is located about 9 miles south-east of the City of Barrie and about 1½ miles west of the shore of Lake Simcoe.

Within the immediate area of the bridge, the terrain is fairly flat. The existing humpbacked bridge is at a maximum height of about 14 ft. above ground level and its steep approach fills increase to about 10 ft. thickness at the 'abutments'.

The railway line has been constructed in cut and is about 8.5 ft. below general ground level. Cut side slopes are at about 2:1 and show no signs of instability.

A survey party from the office of Ainley and Associates provided chainage points on the road centreline to which the survey for this site investigation was referred. A benchmark was also given and this was later referred to geodetic datum (spike in tree at elevation 500.93 ft.).

3.0. SITE WORK

Test drilling was carried out between the 10th and 15th October 1968, by a Dominion Soil Investigation Limited drilling crew using a trailer mounted diamond drill type machine. All work was under the direction of a field technician who kept in close liaison with the project engineer.

Four boreholes numbered 1 to 4 were drilled near the proposed bridge foundation positions and adjacent boreholes were alternated either side of the road centreline. Each borehole had a cone test made alongside to enable comparison of results with those of additional cone test holes Nos. 5 and 6.

The borehole and cone test locations are shown in plan on Enclosure No. 1.

Sampling in each borehole was effected by Standard Penetration test methods with samples being taken at 2.5 to 5.0 ft. intervals of depth. In the cone test a 2-inch diameter, 60° solid cone was driven continuously from the surface using the same driving energy as that for the Standard Penetration test and the blows for each foot of penetration were recorded.

All soils exposed during drilling were described by the technician who also sealed samples for transport to the laboratory. Water level observations were made continuously during the course of the investigation.

Detailed borehole logs have been drawn up for all test holes and these are enclosed.

4.0. LABORATORY TESTING

A small amount of laboratory testing was carried out to help identify soil types and to observe trends in the subsoil profile. Tests made were natural moisture content determinations, Atterberg limits and a particle size analysis of a typical sample of subsoil.

Results are shown on the borehole logs except for the result of the particle size analysis which is enclosed separately in graphical form.

5.0. SUBSOIL CONDITIONS

Subsoil conditions are favourable and uniform over the site. A subsoil profile has been drawn up on Enclosure No. 2 to aid interpretation of the conditions.

The existing approach fills which are up to 10 ft. deep, are loose and are composed of brown sandy silt with inclusions of topsoil.

The natural subsoil within the depth investigated (30 ft. below general ground level) is a very dense sandy silt glacial till containing traces of gravel. At slight depths below the surface the till is less compact but otherwise 'N' values range from about 40 to well over 100 blows per foot.

In borehole 3, a 2 ft. thick zone of hard silty clay till was encountered within the sandy silt. No silty clay was established in other boreholes indicating that the zone is not continuous.

The groundwater level was established at about elevation 485 ft. This elevation corresponds approximately with the level below which the till has a uniform grey colour and shows no signs of oxidation.

Enclosures.

Our Ref. No. 8 – 10 – 11

Enclosure No. 1

Prep. By Z. A.

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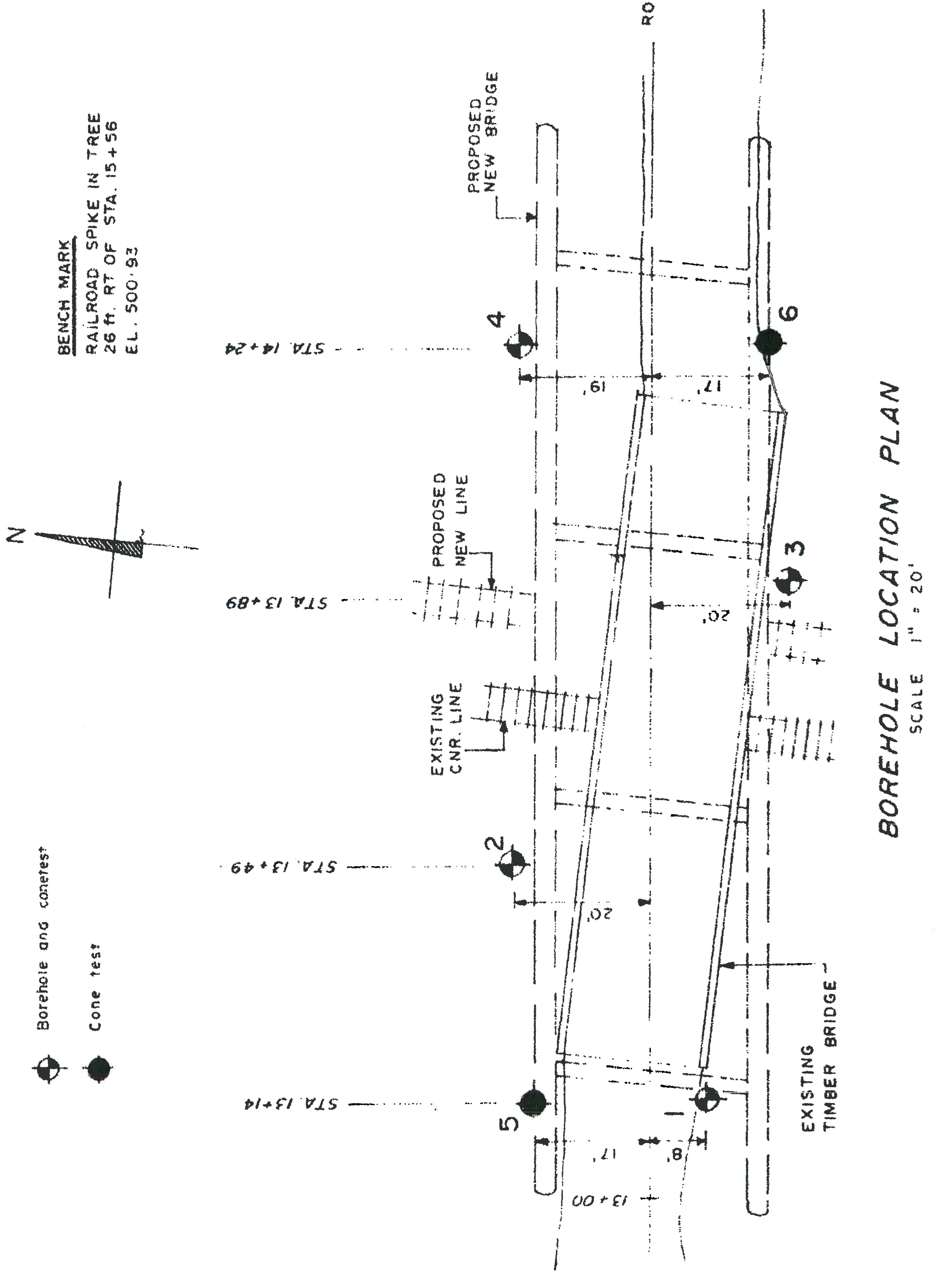
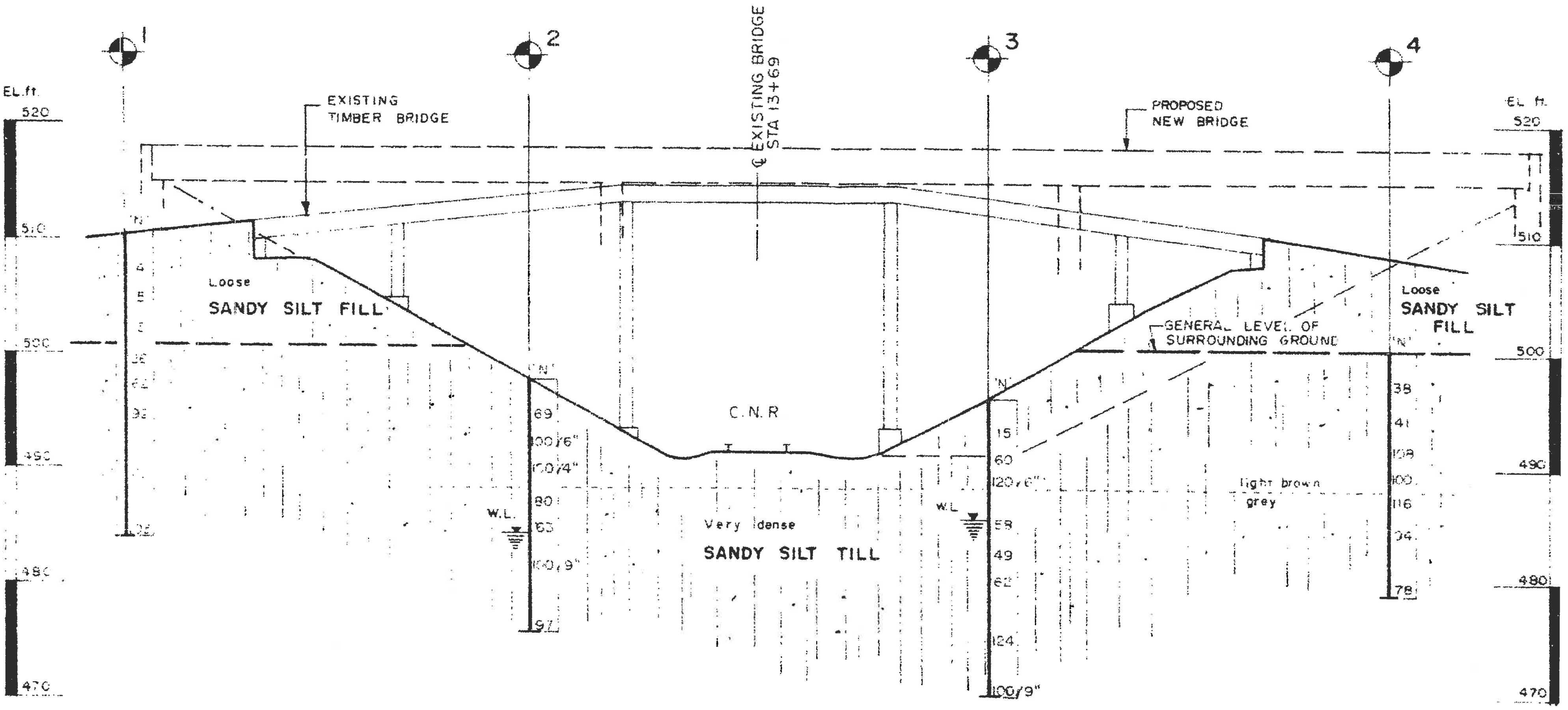
CONDITION OF ORIGINAL DOCUMEN



BOREHOLE LOCATION PLAN

SCALE 1" = 20'

DOMINION SOIL INVESTIGATION LIMITED



INFERRED SUBSOIL PROFILE

SCALE 1IN.=10FT.

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DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

LOG OF BOREHOLE ....1.......

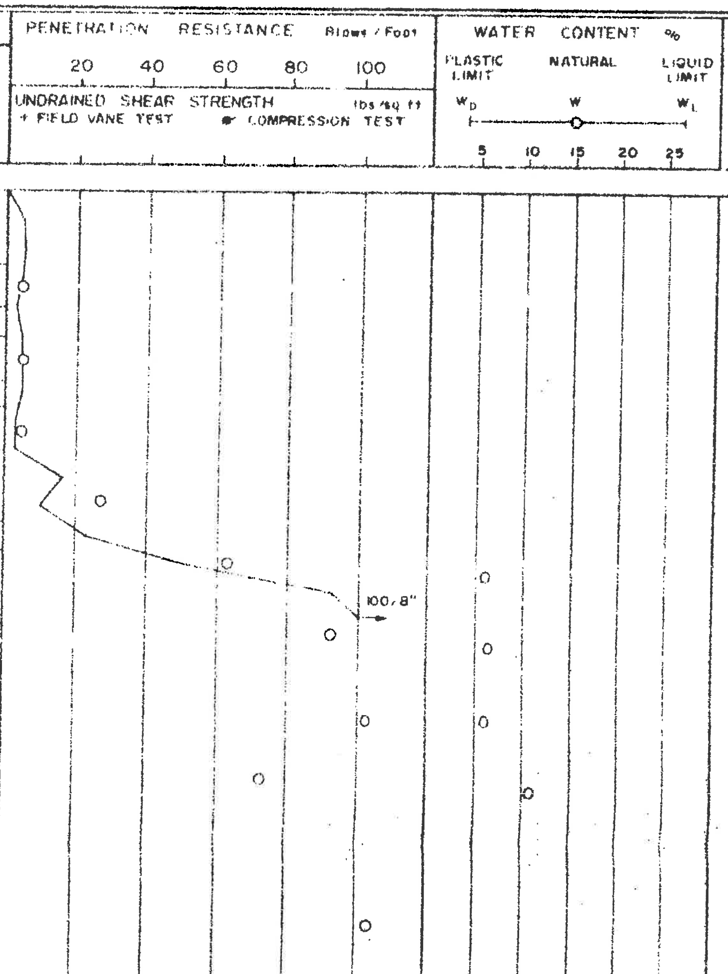
Our Reference Nº 8-10-11

Enclosure Nº 3

CLIENT: AINLEY & ASSOCIATES LTD
PROJECT: BRIDGE OVER C N R
LOCATION: TOWNSHIP OF INNISFIL, ONTARIO
DATUM ELEVATION: SPIKE IN TREE 500·93'

DRILLING DATA
Method DRY BORING
Diameter 4"
Date OCT. 15, 1968

SUBSURFACE PROFILE				SAMPLES			REMARKS
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	GROUND WATER	NUMBER	TYPE	N Blows/Foot	
510·4	0	GROUND SURFACE					
		6" GRAVEL SURFACE					
		LOOSE Brown **SANDY SILT FILL** topsoil inclusions		1	SS	4	
				2	SS	5	
				3a 3b	SS	5	
500·2	10·2	VERY DENSE **SANDY SILT** light brown / grey embedded grits and gravel (glacial till)	BOREHOLE DRY ON COMPLETION	4a 4b	SS	26	
				5	SS	62	
				6	SS	92	
				7	SS	102	
				8	SS	72	
				9	SS	103	
483·9	26·5	END OF BOREHOLE					



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LOG OF BOREHOLE 2

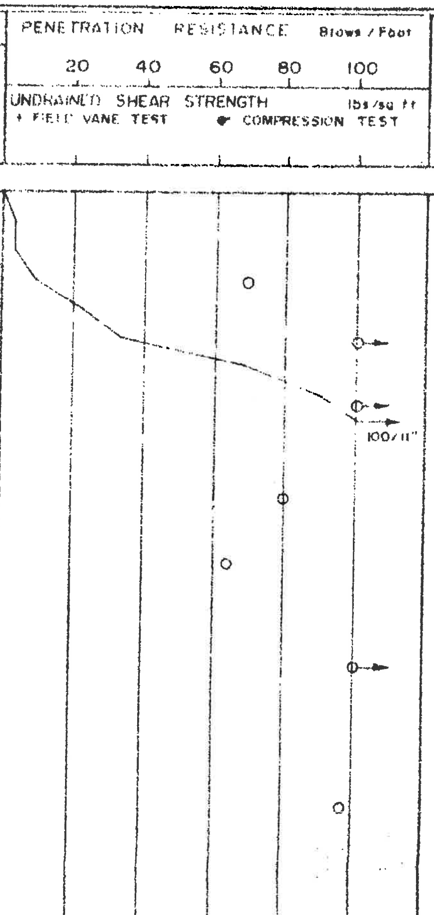
Our Reference Nº 8-10-11

Enclosure Nº 4

CLIENT: AINLEY & ASSOCIATES LTD
PROJECT: BRIDGE OVER C N R
LOCATION: TOWNSHIP OF INNISFIL, ONTARIO
DATUM ELEVATION: SPIKE IN TREE 500·93'

DRILLING DATA
Method WASHBORING
Diameter 3"
Date OCT 11, 1968

SUBSURFACE PROFILE					SAMPLES			WATER CONTENT %	REMARKS
ELEVATION Ft.	DEPTH Ft	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows / Foot	PLASTIC LIMIT W_p — NATURAL W — LIQUID LIMIT W_L	
498·0	0	GROUND SURFACE							
		VERY DENSE **SANDY SILT** embedded grits and gravel occasional cobbles		WL. EL. 484·3 OCT. 15, 68	1	SS	69		
	5				2	SS	100/6"		
		light brown			3	SS	100/4"		
	10	grey (glacial till)			4	SS	80		
				▼	5	SS	63		
	15				6	SS	100/9"		
	20				7	SS	97		
475·5	22·5	END OF BOREHOLE							
	25								



LOG OF BOREHOLE 3

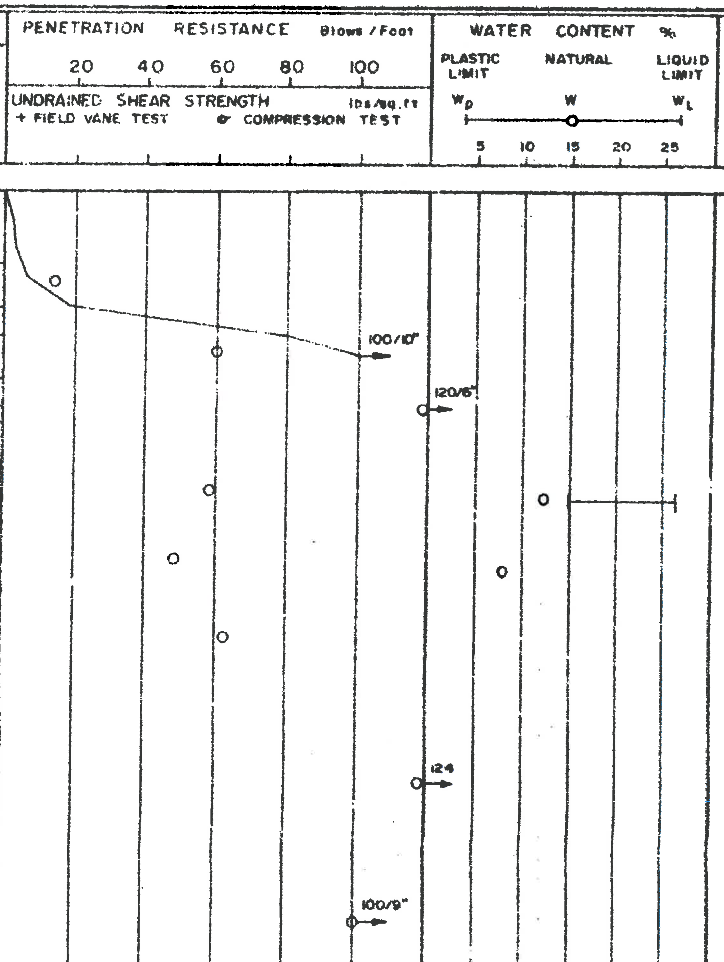
Our Reference Nº 8-10-11

Enclosure Nº 5

CLIENT: AINLEY & ASSOCIATES LTD.
PROJECT: BRIDGE OVER C N R
LOCATION: TOWNSHIP OF INNISFIL, ONTARIO
DATUM ELEVATION: SPIKE IN TREE. 500·93'

DRILLING DATA
Method: DRY BORING
Diameter 3"
Date OCT 10, 1968

SUBSURFACE PROFILE					SAMPLES		
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows/Foot
496·1	0	GROUND SURFACE					
		6" TOPSOIL Loose SILT TILL Dark brown (organic)					
494·1	2·0	Light brown Compact Very dense **SANDY SILT** grit traces (glacial till)			1 2 3	SS SS SS	15 60 120/6"
485·6	10·5	Hard, grey **SILTY CLAY** (till)		W.L. EL. 485·5 OCT. 15, 1968	4	SS	58
483·6	12·5	Very dense grey **SANDY SILT** embedded grits and gravels trace of clay (glacial till)			5 6 7 8	SS SS SS SS	49 62 124 100/9"
469·8	26·3	END OF BOREHOLE					



REMARKS

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

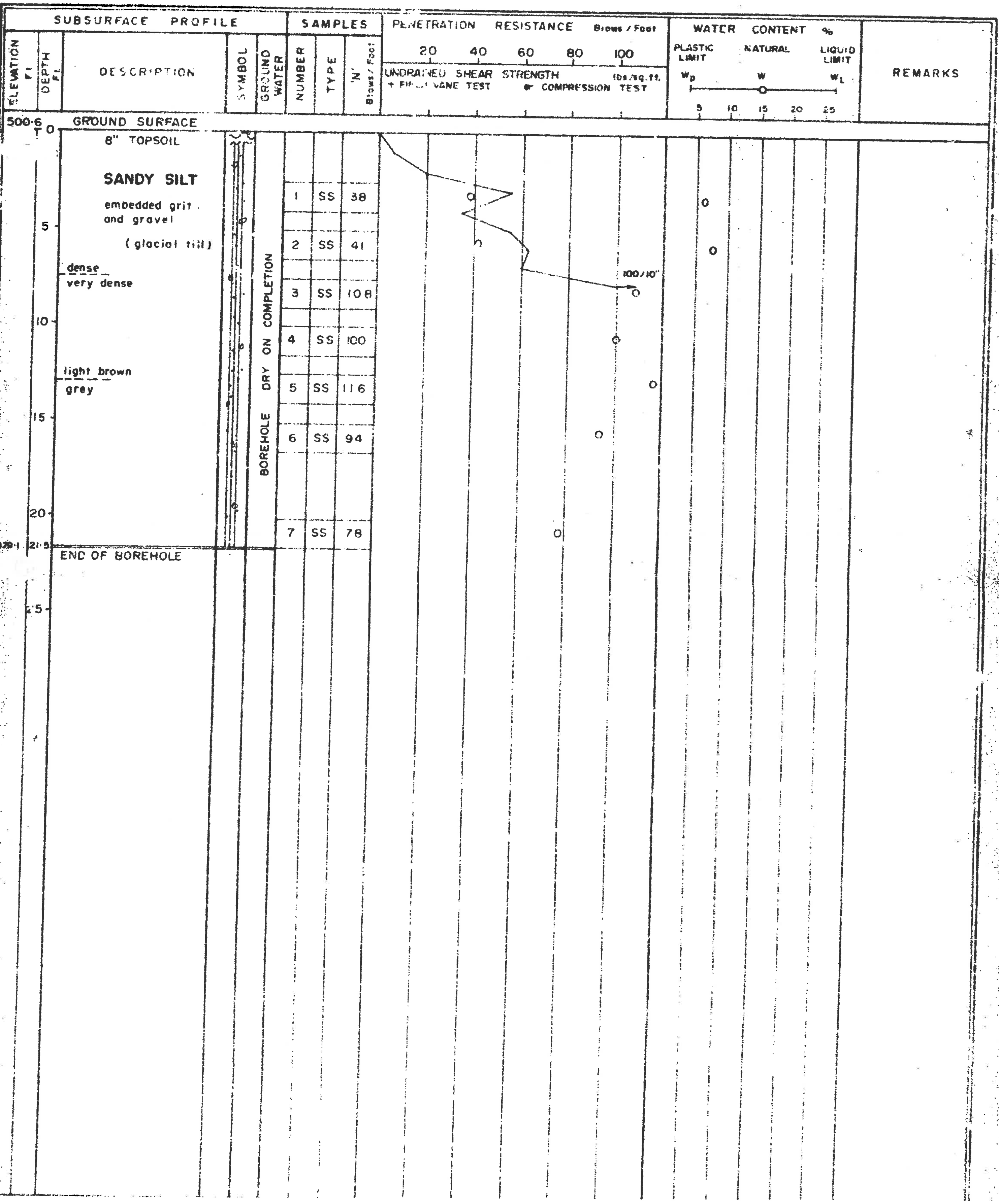
LOG OF BOREHOLE 4

Our Reference Nº 8-10-11

Enclosure Nº 6

CLIENT: AINLEY & ASSOCIATES LTD
PROJECT: BRIDGE OVER C N R
LOCATION: TOWNSHIP OF INNISFIL, ONTARIO
DATUM ELEVATION: SPIKE IN TREE 500·93'

DRILLING DATA
Method: DRY BORING
Diameter: 3"
Date: OCT. 15, 1968



ELEVATION Ft	DEPTH Ft	DESCRIPTION	GROUND WATER	NUMBER	TYPE	'N' Blows / Foot	REMARKS
500·6	0	GROUND SURFACE					
		8" TOPSOIL					
		SANDY SILT embedded grit and gravel (glacial till)	BOREHOLE DRY ON COMPLETION	1	SS	38	
	5			2	SS	41	
		dense very dense		3	SS	108	
	10			4	SS	100	
		light brown grey		5	SS	116	
	15			6	SS	94	
	20			7	SS	78	
479·1	21·5	END OF BOREHOLE					
	25						

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

LOG OF CONE TEST 5

Our Reference Nº 8-10-11

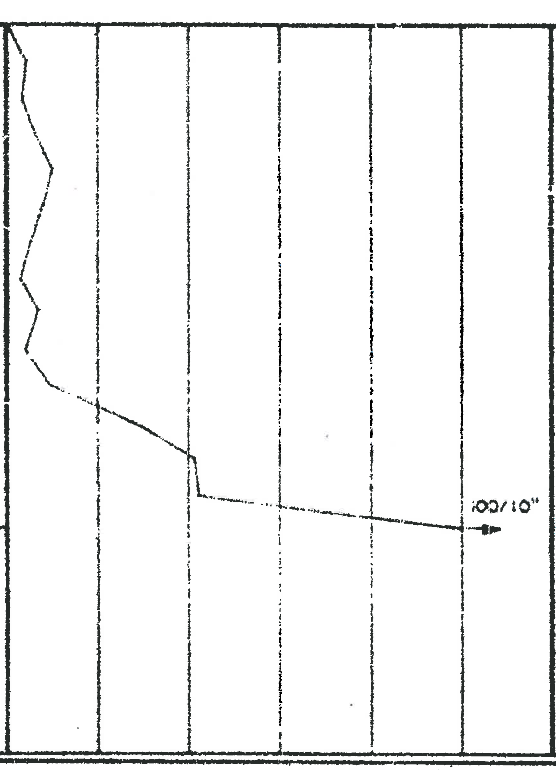
Enclosure Nº 7

CLIENT: AINLEY & ASSOCIATES LTD.
PROJECT: BRIDGE OVER C.N.R.
LOCATION: TOWNSHIP OF INNISFIL, ONTARIO
DATUM ELEVATION SPIKE IN TREE 500·93'

DRILLING DATA

Method:
Diameter:
Date: OCT. 15, 1968

SUBSURFACE PROFILE					SAMPLES			PENETRATION RESISTANCE Blows / Foot 20 40 60 80 100 UNDRAINED SHEAR STRENGTH lbs/sq.ft. + FIELD VANE TEST ● COMPRESSION TEST	WATER CONTENT % PLASTIC LIMIT Wp, NATURAL W, LIQUID LIMIT WL	REMARKS
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows / Foot			
510·4	0	GROUND SURFACE								
	5	LOOSE FILL (inferred)								
500·4	10	Dense to very dense TILL (inferred)								
496·6	13·8	END OF CONE TEST								
	15									



VERTICAL SCALE: 1 inch to 5 feet

MADE: S.O. CHECKED:

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CONDITION OF ORIGINAL DOCUMENT

LOG OF CONE TEST 6

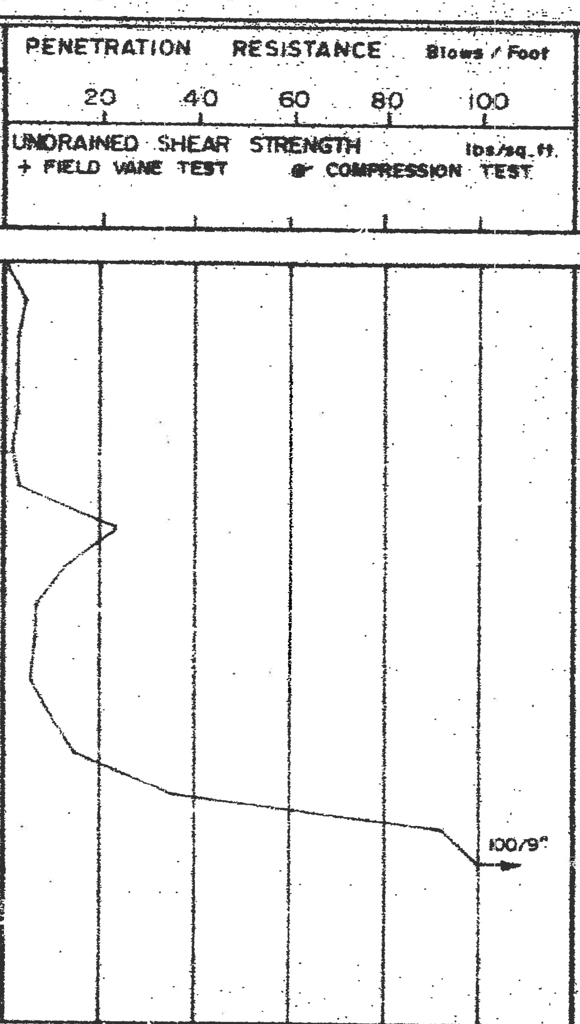
Our Reference Nº 8-10-11

Enclosure Nº 8

CLIENT: AINLEY & ASSOCIATES LTD.
PROJECT: BRIDGE OVER C.N.R.
LOCATION: TOWNSHIP OF INNISFIL, ONTARIO
DATUM ELEVATION: SPIKE IN TREE 500·93'

DRILLING DATA
Method:
Diameter:
Date: OCT. 15, 1968

SUBSURFACE PROFILE					SAMPLES			PENETRATION RESISTANCE Blows / Foot	WATER CONTENT %	REMARKS
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows / Foot	20 40 60 80 100 UNDRAINED SHEAR STRENGTH lbs/sq.ft. + FIELD VANE TEST ⊕ COMPRESSION TEST	PLASTIC LIMIT W_p NATURAL W LIQUID LIMIT W_L	
509·4	0	GROUND SURFACE								
	5 10	LOOSE FILL (inferred)								
497·4	12·0 15	Dense to very dense TILL (inferred)								
493·6	15·8	END OF CONE TEST								



VERTICAL SCALE: 1 inch to 5 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE: S.O. CHECKED:

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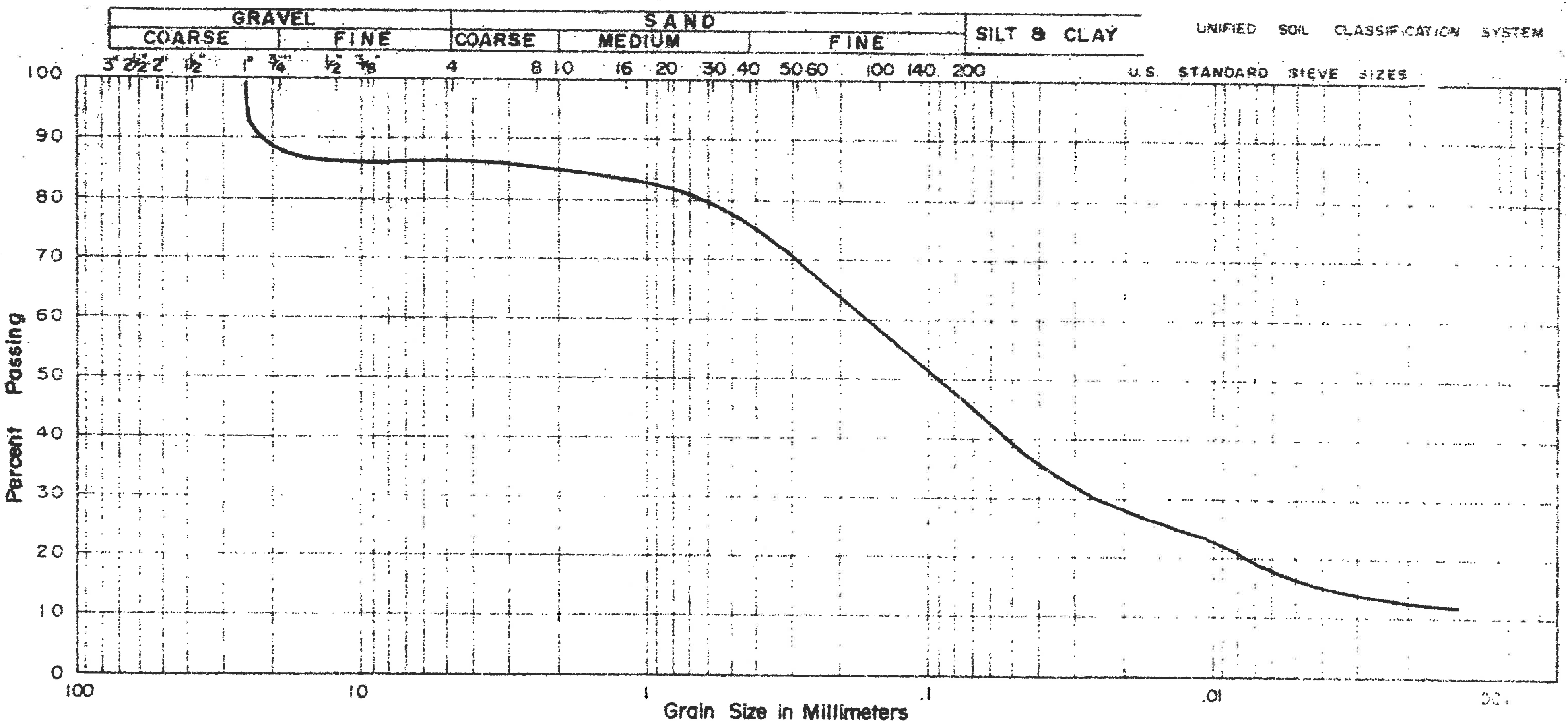
GRAIN SIZE DISTRIBUTION

OUR REFERENCE Nº 8-10-11



PROJECT: BRIDGE OVER C.N.R.

LOCATION: TWP. OF INNISFILL

BOREHOLE Nº: 1

SAMPLE Nº: 7

DEPTH: 19'

ELEVATION:

COEFFICIENT OF UNIFORMITY:

COEFFICIENT OF CURVATURE:

Classification of Sample and Group Symbol:

SANDY SILT- trace of gravel

PLASTIC PROPERTIES

LIQUID LIMIT % =

PLASTIC LIMIT % =

PLASTICITY INDEX % =

MOISTURE CONTENT % =

ENCLOSURE Nº 9

DATE September 24, 2015 **PROJECT No.** 1413283

TO Cheryl Murray, P.E., Highway Practice Lead
HDR Corporation

CC

FROM Kevin J. Bentley, P.Eng. **EMAIL** kbentley@golder.com

DESKTOP STUDY FOUNDATION ASSESSMENT
6TH LINE AND HIGHWAY 400 OVERPASS STRUCTURE WIDENING / REPLACEMENT
6TH LINE MUNICIPAL CLASS ENVIRONMENTAL ASSESSMENT
TOWN OF INNISFIL, ONTARIO

Golder Associates Ltd. (Golder) has been retained by HDR Corporation (HDR) on behalf of the Town of Innisfil to provide foundation engineering services in support of the Municipal Class Environmental Assessment (Class EA) for proposed improvements (e.g., widening) of 6th Line from County Road 27 to St. John's Road in Innisfil, Ontario.

This technical memorandum summarizes the results of a desktop study of available subsurface information and provides preliminary foundation recommendations for the proposed extension / replacement of the existing Highway 400 / 6th Line Overpass structure. The preliminary geotechnical / foundation recommendations provided in this technical memorandum are based on limited information about the project and are intended for planning purposes only.

The reader is referred to the attached "Important Information and Limitations of This Report" that follows the text of this technical memorandum and forms an integral part of this document.

1.0 PROJECT BACKGROUND

Currently 6th Line is a 2-lane road with a posted speed of 80 km/h. Based on predicted future uses, the segment of roadway between 20 Sideroad to St. John's Road (approximately 3 km in length, and including the planned Sleeping Lion Development) is anticipated to have future urbanized characteristics, while the segment from County Road 27 to 20 Sideroad (approximately 12 km in length, with mostly agricultural properties) will operate as a rural section.

Based on the recommendations from the 2013 Transportation Master Plan, and additional assessment conducted through this EA study, the Town is proposing to widen 6th Line, between 20th Sideroad and St. John's Road, from a 20 m 2-lane local rural road to a 26-30 m wide 4-lane urban major collector road, and proposing to reconstruct 6th Line, between County Road 27 and 20th Sideroad, from a 20 m 2-lane local rural road to a 2-lane rural arterial road with paved shoulders and 30 m right-of-way protection.

In addition to confirming the cross section and preliminary conceptual design of the roadway, the study will review the need for the following corridor features:

- Bike lanes or multi-use trails;
- Potential need for a future interchange at Highway 400;
- New structure or structure widening over the existing GO rail line;
- Intersection improvements.

2.0 SITE DESCRIPTION

The existing Highway 400 / 6th Line Overpass structure is a single span bridge that carries Highway 400 (six-lane configuration) over the 6th Line (see Photograph 1 below) in the Town of Innisfil, Ontario (see Key Plan included on Drawing 1 - Borehole Location Plan in Appendix A).

The existing ground surface surrounding the bridge structure is generally flat and is occupied by farm fields / grassed areas. The 6th Line appears to have been constructed in cut up to about 4 m deep, given the depressed profile in comparison to the grade of the surrounding fields (see Photograph 2 below).

Based on a digital terrain model provided by HDR, the 6th Line road surface at the bridge is at approximately Elevation 291 m (Geodetic Datum). The ground surface surrounding 6th Line ranges between about Elevation 294 m to 295 m and the ground surface at the Hwy 400 grade is at approximately Elevation 296.5 m.

Photograph 1: Looking East from 6th Line

Photograph 2: Looking West from 6th Line

3.0 REVIEW OF AVAILABLE INFORMATION

A desktop study search of available subsurface information at the MTO GEOCRES library found an existing geotechnical report that was prepared for the proposed widening of Highway 400 at the site. The report titled “Preliminary Foundation Investigation Report, Innisfil Sixth Line Overpass, Structure Site 30-211, Highway 400 Widening from 1 km South of Highway 89 to Highway 11, G.W.P. 30-95-00”, dated January 2002, was prepared by Golder. A copy of the Preliminary Foundation Investigation Report is included in Appendix A and the results of the investigation are summarized below.

4.0 SUBSURFACE CONDITIONS

Two boreholes, designated B3-1 and B3-2, were advanced on 6th Line at the east and west side of the existing bridge, respectively. The locations of the boreholes are shown on the Borehole Location Plan provided in Appendix A, along with a copy of the Record of Boreholes.

Based on the existing boreholes, the subsurface conditions at the site consist of a surficial layer of asphalt, underlain by sand and gravel to silty sand fill, underlain by clayey silt till.

The sand and gravel fill was 300 mm to 500 mm thick and is considered to be part of the asphalt pavement structure for 6th Line. A thicker layer of silty sand fill was encountered below the sand and gravel in Borehole B3-2, but it is likely that this silty fill is associated with utility trench backfill as this borehole was located near a catch basin. The measured Standard Penetration Test (SPT) ‘N’-values were 11 and 28 blows per 0.3 m of penetration, indicating that the silty sand fill has a compact relative density.

The clayey silt till deposit encountered below the fill extended to the bottom of the boreholes that terminated at depths of 8 m and 11 m in Boreholes B3-1 and B3-2 respectively. The top of the till was encountered at Elevation 290.7 m in Borehole B3-1, located on the east side of the highway bridge. Borehole B3-2, located on the west side of the highway bridge, encountered inferred utility trench backfill; outside of the utility trench areas, it is expected that the surface of the till deposit will be encountered immediately below the road base fill. The clayey silt till contains variable amounts of sand and gravel, and cobbles were inferred during drilling operations. The natural moisture content measured on samples of the till ranged from 6 per cent to 9 per cent. Atterberg Limits testing performed on samples of the till indicate that the material is of low plasticity. The SPT 'N'-values measured within the clayey silt till deposit ranged between 67 blows to 138 blows per 0.3 m of penetration, but were typically greater than 100 blows per 0.3 m of penetration, suggesting that the clayey silt till is hard.

The Record of Boreholes document the groundwater level measured in the open boreholes following drilling operations in October 2000. The groundwater level was measured in Borehole B3-1 at a depth of 6.9 m below ground surface (about Elevation 284.5 m) and was rising; the groundwater level in Borehole B3-2 was at 4 m depth (about Elevation 287 m). A piezometer was installed at the time of drilling but was subsequently destroyed before any stabilized groundwater level readings could be taken. As a result, the reported groundwater levels are not considered to have stabilized and groundwater levels are anticipated to fluctuate seasonally and will be higher in the Spring season and during periods of precipitation and snowmelt.

5.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

We understand that HDR / Town of Innisfil require preliminary foundation recommendations for planning purposes for lengthening / replacing the existing Highway 400 / 6th Line Overpass structure as part of the widening / improvements to 6th Line in this area.

We also understand that the Ministry of Transportation Ontario is considering widening Highway 400 from six lanes up to eight (or possibly twelve) lanes and subsequently widening / replacing this same structure.

The geotechnical comments and recommendations provided herein are based on interpretation of the factual data available from the previous borehole investigation at the site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary for conceptual design of the EA study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at the bridge site as the design progresses in collaboration with the MTO.

It is important to note that no design or construction drawings for the existing bridge and/or foundations were available at the time this technical memorandum was prepared. Based on information provided in the 2002 Preliminary Foundation Investigation report, it is understood that the existing single-span structure abutments and associated retaining walls are supported on spread footings which are founded at about Elevation 289.4 m. More details on the existing foundations should be made available as the design progresses as the existing foundations may influence the proposed lengthening / widening / replacement foundation options.

5.1 Preliminary Foundation Options

Underlying the asphalt and fill materials, the native subsoils at the Hwy 400 / 6th Line Overpass site generally consist of hard clayey silt till, with SPT 'N'-values typically greater than 100 blows per 0.3 m of penetration. Based on these subsurface conditions, it is recommended that new foundations for the proposed bridge extension / widening / replacement structure be founded on spread footings placed on the hard clayey silt till. Consideration could also be given to the use of perched abutments founded on spread footings placed on a compacted granular pad above the clayey silt till soils within the approach embankments.

Alternatively, for a new Overpass or structurally-separate widening / extension, new foundations could be supported on steel H-piles driven into the hard clayey silt till. Difficulties penetrating through the glacial till soils and fill containing cobbles/boulders should be expected and pre-augering techniques may be required to achieve a minimum pile embedment length. It should be noted that the boreholes advanced as part of the previous preliminary investigation were drilled from the 6th Line grade (i.e., cut level). If perched footings or integral abutments supported on deep foundations are considered viable options, determination of the subsoil conditions between the Highway 400 grade and the 6th Line cut will be required during the detailed design stage.

For preliminary design, spread footings founded on the hard clayey silt till (at or below Elevation 289.4 m) may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 900 kPa and geotechnical resistance at Serviceability Limit States (SLS) of 600 kPa (for 25 mm settlement). The preliminary geotechnical resistances provided above assume a 3 m wide footing and assume there will be no influence from the existing bridge configuration / foundations. The bottom of the footings should be provided with a minimum 1.5 m of soil cover to protect the founding subgrade against frost penetration. Groundwater conditions at the foundation locations will have to be confirmed during the detail design stage but are not anticipated to be a major concern given that the groundwater level recorded in the boreholes at the time of investigation was measured to be below the recommended founding level. If widening of the existing overpass structure is being considered (i.e. to accommodate the widening of Hwy. 400), the founding level of the existing footings should be matched; however, it is likely that both widening and lengthening of the bridge will be required suggesting that a complete bridge replacement will be needed. Where fill is encountered below the footing founding level, it should be removed and replaced with lean concrete. Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings. Based on the existing information, a well-compacted granular pad will be required to support the wing wall or retaining wall footings above Elevation 289.4 m.

For spread footings placed within the approach embankments on a compacted Granular 'A' pad, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of well compacted Granular' A' pad and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design.

Alternatively, for preliminary design, steel HP310x110 piles driven through the fill soils and native soils to a minimum 3 m embedment into the hard clayey silt till deposit (where SPT ‘N’ values are greater than 100 blows / 0.3 m of penetration) may be designed using a factored axial geotechnical resistance at ULS of 1,600 kN and at SLS (for 25 mm of settlement) of 1,400 kN. For preliminary design, a pile tip level at Elevation 287 m may be assumed, although the actual pile tip elevation may vary depending on the results of the detail design investigation. As previously discussed, the presence of cobbles/boulders may result in difficulties achieving the target penetration depth to achieve the design capacities and/or minimum pile embedment for the structure type and pre-augering or perched abutments may need to be considered. The base of pile caps should have a minimum 1.5 m of soil cover to protect against frost penetration.

The settlement of any new foundations will be dependent on the footing size and configuration, and on the applied loads. This settlement will be differential with respect to the existing overpass structure foundations if consideration is being given to leaving the existing structure in place. As previously mentioned, the existing footing types, sizes and configuration should be confirmed and geotechnical resistances assessed during detail design (if existing footings are to be used and remain in place) and new foundations should be designed in accordance with the latest version of the Canadian Highway Bridge Design Code (CHBDC).

6.0 CLOSURE

This technical memorandum was prepared by Mr. Qasim Cheema, P.Eng., a geotechnical engineer with Golder. Mr. Kevin J. Bentley, P.Eng., a geotechnical engineer and Associate with Golder provided a senior review of the technical memorandum. We trust the above information meets with your current requirements, but should you have any questions, please do not hesitate to contact us.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
SEPT. 24, 2015
K. J. BENTLEY
PROVINCE OF ONTARIO

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QC/KJB/rb

Attachments:
Important Information and Limitations of This Report
Appendix A – Previous Geotechnical Investigation

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder can not be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client’s expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder’s report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder’s report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder’s report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder’s report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder’s responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

APPENDIX A

Previous Geotechnical Investigation

PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
INNISFIL SIXTH LINE OVERPASS
STRUCTURE SITE 30-211
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

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PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT

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LIST OF FIGURES

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening and / or replacement of the existing Innisfil Sixth Line overpass structure. A foundation site investigation has been carried out, in which two boreholes were advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Innisfil Sixth Line overpass structure is located about 7 km north of the Highway 89 interchange and about 8.5 km south of the Molson Park Drive interchange, in the Town of Innisfil, Simcoe County. The MTO has designated this overpass as Structure Site No. 30-211.

At this site, the original ground surface was at about Elevation 294 m to 295 m. Innisfil Sixth Line has been constructed in a cut up to 4 m deep, with its grade at about Elevation 291 m under Highway 400. The Highway 400 grade is at about Elevation 296.5 m at the structure site.

The existing single-span overpass structure was constructed in the early 1950s. According to the general layout drawings for this existing structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments and associated retaining walls are supported on spread footings which are founded at about Elevation 289.4 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in October 2000, at which time two boreholes were drilled. Boreholes B3-1 and B3-2 were advanced in the vicinity of the north and south abutments, on the east and west sides of the highway, respectively. The boreholes were advanced to between 8 m and 11 m below the Innisfil Sixth Line cut grade.

The investigation was carried out using a bombardier-mounted B-57 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. The boreholes were advanced using solid stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole B3-1 to permit monitoring of the groundwater levels at the site.

The field work was supervised on a full-time basis by a member of our staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits tests and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The borehole locations, together with elevations and northing and easting co-ordinates, are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek (about 1 km north of Highway 89) and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field, in which the Innisfil Sixth Line site is located, consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory testing carried out on selected soil samples, are given on the Record of Borehole sheets and Figure 1. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes B3-1 and B3-2 were advanced on the east and west sides of Highway 400, respectively, from approximately Innisfil Sixth Line grade. The locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the site is underlain at the borehole locations by sand and gravel to silty sand fill, overlying clayey silt till. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Beneath the asphalt in Boreholes B3-1 and B3-2, 300 mm to 500 mm of sand and gravel road base fill was encountered.

Underlying this road base fill in Borehole B3-2, a 1.8 m thick layer of silty sand fill, containing some gravel and trace clay, was encountered. It is likely that this silty fill is associated with utility trench backfill, as Borehole B3-2 is located near a catch basin. The measured Standard Penetration Test 'N' values were 11 and 28 blows per 0.3 m of penetration, indicating that the silty sand fill has a compact relative density.

4.2.2 Clayey Silt Till

A deposit of clayey silt till was encountered below the fill in both boreholes. The surface of the till is at Elevation 290.7 m in Borehole B3-1, on the east side of the highway. On the west side of the highway, Borehole B3-2 encountered utility trench backfill; outside of the utility trench areas, it is expected that the surface of the till deposit will be encountered immediately below the road base fill. The till deposit extends to the maximum depth investigated, to about Elevation 283 m and 279.5 m in the boreholes on the east and west sides of Highway 400, respectively. The till deposit is at least 8 m to 11 m thick.

The clayey silt till contains a significant proportion of sand, and trace to some gravel. The result of a grain size distribution test carried out on a representative sample is shown on Figure 1. The natural moisture contents measured on samples of the clayey silt till ranged from 6 to 9 per cent. Atterberg Limits testing was carried out on three samples. The plastic limits ranged from 11 to 12 per cent, the liquid limits from 14 to 15 per cent and the plasticity indices from 3 to 4 per cent. The results of the Atterberg Limits testing indicate that the clayey silt till is inorganic and of low plasticity.

The measured SPT 'N' values ranged from 67 to 138 blows, but were typically greater than 100 blows per 0.3 m of penetration, indicating that the clayey silt till is hard.

4.3 Groundwater Conditions

The groundwater conditions were observed in the open boreholes following drilling operations in October 2000. At that time, the water level measured in Borehole B3-1 was at 6.9 m depth (about Elevation 284.5 m) and rising; the water level measured in Borehole B3-2 was at 4 m depth (about Elevation 287 m). The piezometer which was installed in Borehole B3-1 could not be founded in January or March 2001. This piezometer is presumed to have been destroyed. Therefore, the stabilized groundwater level could not be determined

The colour change in the soil from brown to grey at a relatively shallow depth indicates that the piezometric groundwater level is likely in the upper portion of the till.

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to be higher during wet periods of the year.

GOLDER ASSOCIATES LTD.

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LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π = 3.1416

ln x, natural logarithm of x

$\log_{10}$ x or log x, logarithm of x to base 10

g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity)of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

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

(a) Index Properties (con't.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_P	plasticity Index = $(w_l - w_p)$
w_S	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(d) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_S	swelling index
C_α	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1. $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = (Compressive strength)/2

RECORD OF BOREHOLE No B3-1

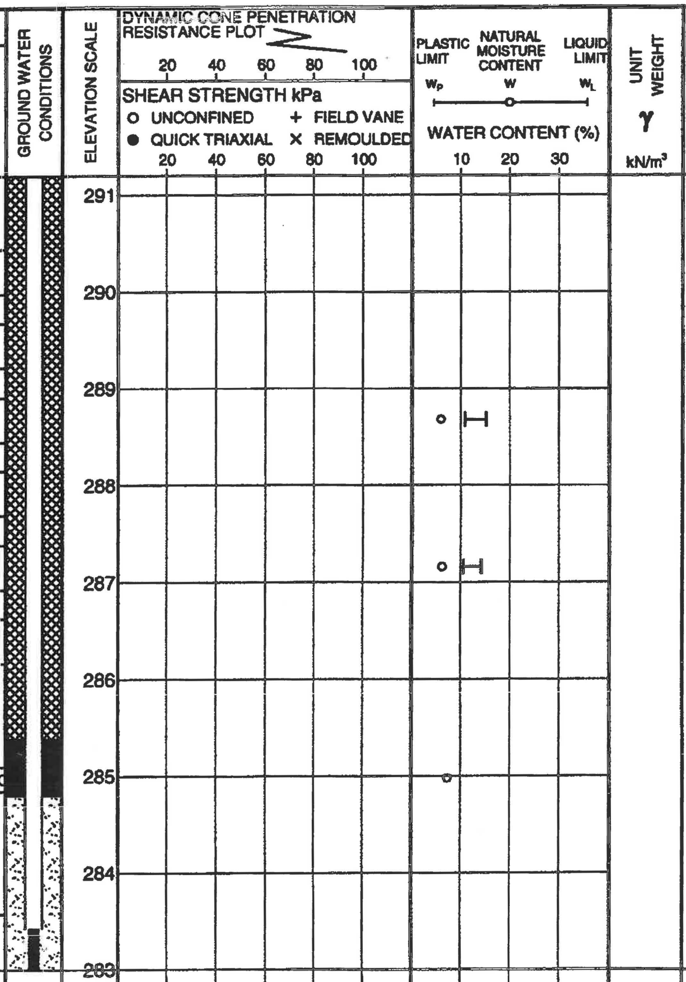
1 OF 1 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic

LOCATION N 4902360.3; E 290986.9
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
DATE Oct.25/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
291.2	GROUND SURFACE							
0.0	Asphalt							
	Sand and Gravel (Fill)							
290.7 0.5	Clayey Silt with sand, trace to some gravel (Till) Hard Brown becoming grey at 1.4m depth Moist	1	SS	67				
		2	SS	130				
		3	SS	138	3	52	31	14
		4	SS	104				
		5	SS	138				
		6	SS	135				
		7	SS	100/.1				
	Cobble at 6.4m depth							
		8	SS	101				
283.0 8.2	END OF BOREHOLE							



Notes:
1. Water level in open borehole at 7.4m depth (Elev.283.8m) immediately after completion of drilling. Water level rose to 6.9m depth (Elev.284.3m) about 10 minutes after completion of drilling.
2. Piezometer could not be found on January 19 or March 15, 2001; piezometer presumed destroyed.

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

RECORD OF BOREHOLE No B3-2

1 OF 1 **METRIC**

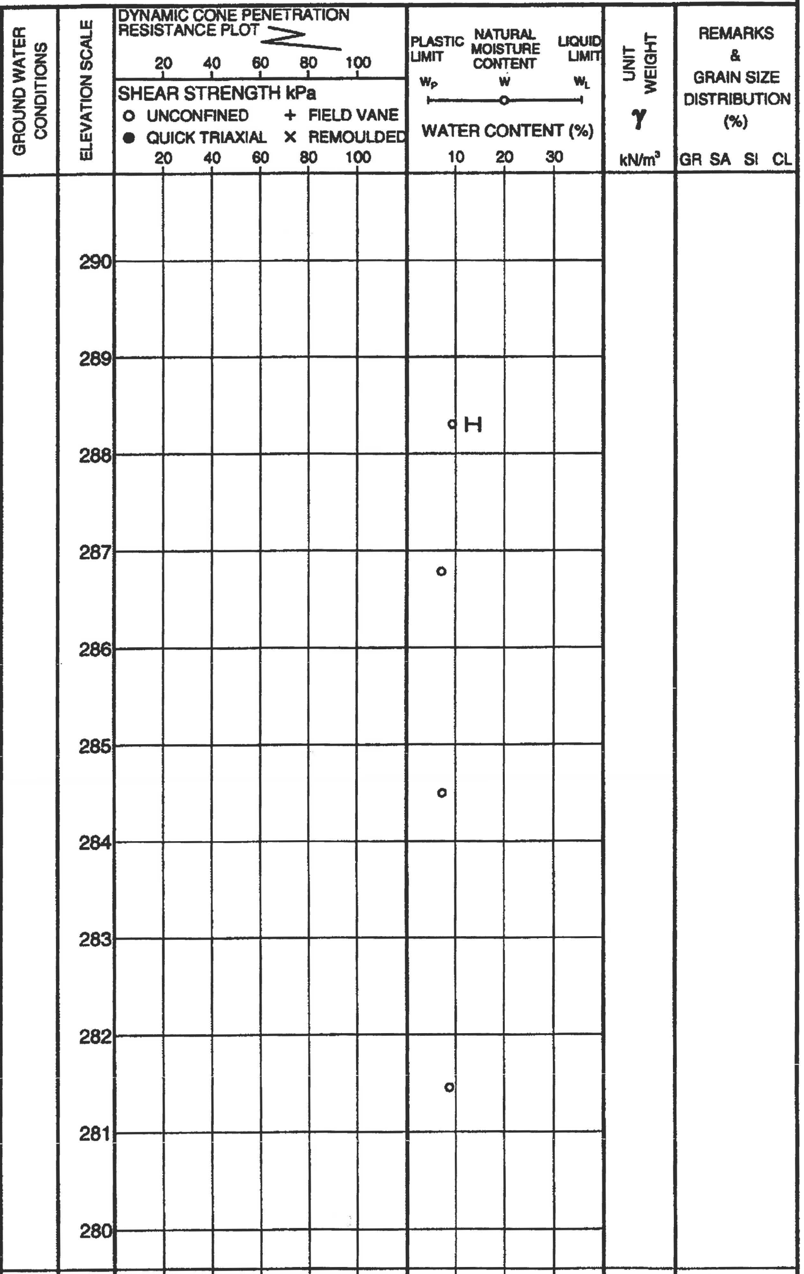
PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic

LOCATION N 4902360.3; E 290986.9
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
DATE Oct.25/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

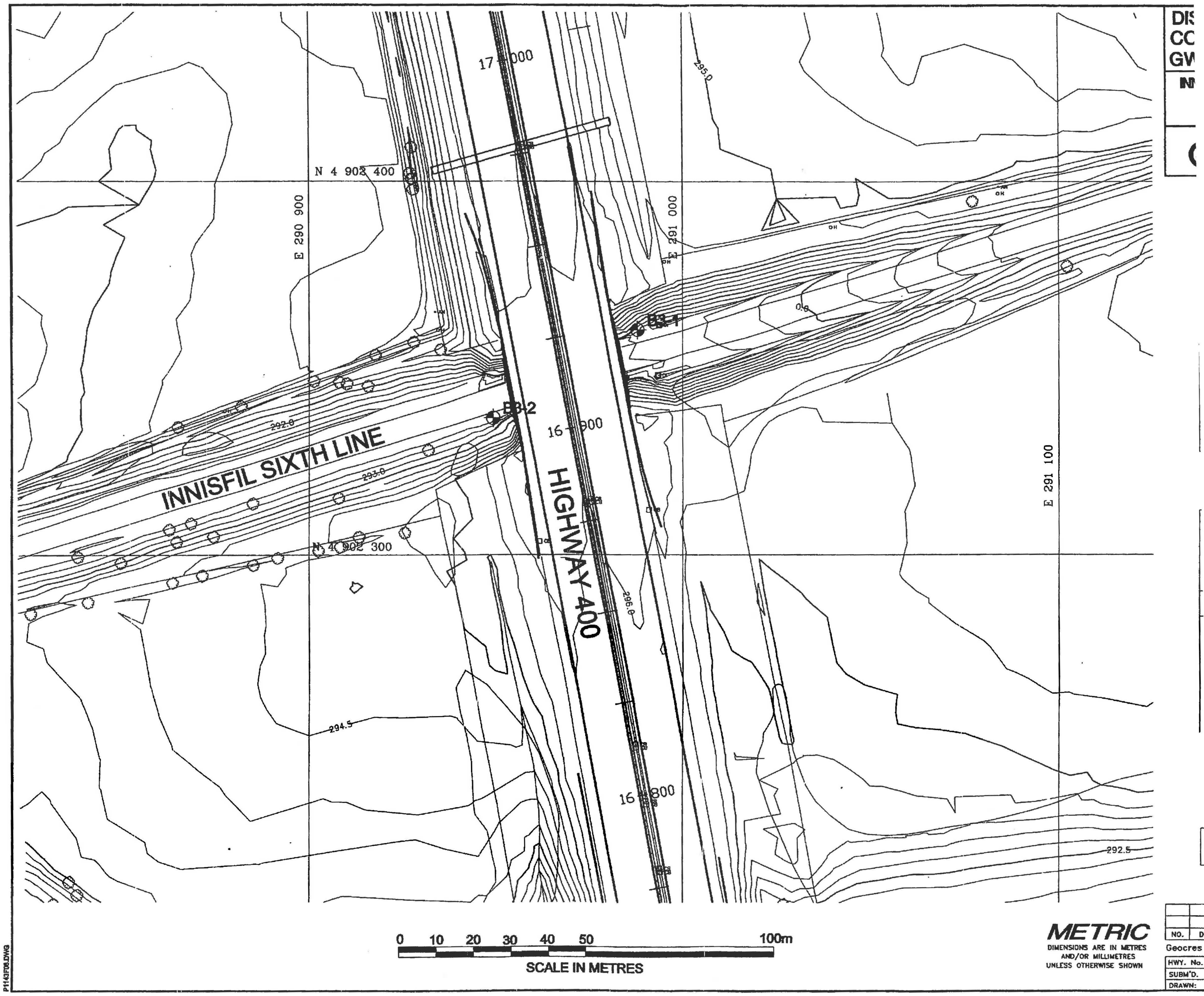
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
290.9	GROUND SURFACE			
0.0	Asphalt			
	Sand and Gravel (Fill)			
0.3	Silty Sand, some gravel, trace clay (Fill) Compact Brown Moist	1	SS	11
	Cobble at 2.1m depth	2	SS	28
288.8 / 2.1	Clayey Silt with sand, trace to some gravel (Till) Hard Brown becoming grey at 4.6m depth Moist	3	SS	77
		4	SS	78
		5	SS	106
		6	SS	130
		7	SS	71
		8	SS	135
	Cobble at 8.7m depth	9	SS	104
		10	SS	105
279.6 / 11.3	END OF BOREHOLE			

Notes:
1. Water level in open borehole at 4.0m depth (Elev.286.9m) on completion of drilling operations.



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

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

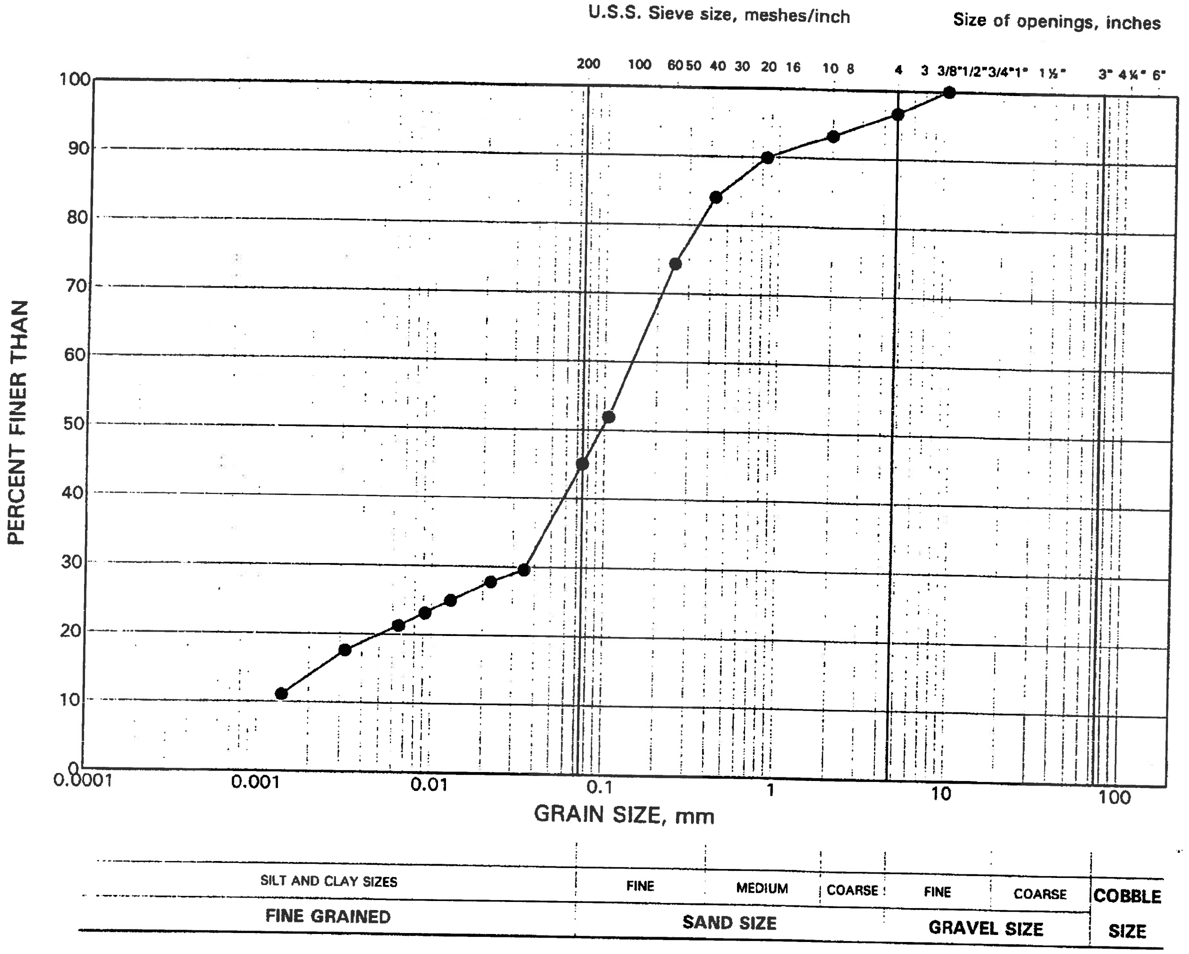


DIS
CO
GW
IN
17+000
N 4 902 400
E 290 900
E 291 000
295.0
BH-1
BH-2
292.0
16+900
INNISFIL SIXTH LINE
293.0
HIGHWAY 400
N 4 902 300
296.0
E 291 100
294.5
16+800
292.5
0
10
20
30
40
50
100m
SCALE IN METRES
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN
NO.
D
Geocres
HWY. No.
SUBM'D.
DRAWN:
P1143F08.DWG

GRAIN SIZE DISTRIBUTION TEST RESULT

Clayey Silt Till

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B3-1	3	288.7