FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 427 WIDENING
FROM FASKEN DRIVE TO STEELES AVENUE
ALBION ROAD OVERPASS
TORONTO, ONTARIO
G.W.P. 202-95-00

Geocres Number: 30M12-290

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a previous foundation investigation conducted by the MTO in 1982 for the design and construction of the existing mainline bridge structures of the Highway 427 overpass at Albion Road in Toronto, Ontario. This information has been used to develop foundation recommendations for the proposed inside widening between the two existing structures.

A model of the subsurface conditions has been developed for the site based on the 1982 data. This information has been used to prepare stratigraphic profile and cross-sections and a written description of the subsurface conditions. The 1982 records of boreholes, laboratory testing results and a plan showing the approximate locations of these boreholes are appended to this report.

Thurber carried out the study as a sub-consultant to SNC-Lavalin under the Ministry of Transportation Ontario (MTO) Agreement Number 2004-E-0071.

During the preparation of this report, reference has been made to the subsurface conditions from the 1982 investigation documented in the report below.

- MTO report titled "Foundation Investigation and Design Report, Albion Road Underpass Structure at Hwy 427, W.P. 153-80-02, Site No. 37-1110, GEOCRES 30M12-164, 1982 (Reference 1).

2 PROJECT AND SITE DESCRIPTION

The project involves the inside widening of the northbound and southbound (NBL and SBL) bridges of the Highway 427 overpass at Albion Road in the air gap between the existing bridges.

The site is located some 600 to 700 m south of the Highway 427 / Highway 407 interchange, and approximately 300 m north of the Highway 427 CNR Halton Overhead in Toronto, Ontario. Lands surrounding the site have been developed for commercial and industrial uses.

An aerial photograph of the site is included in Appendix F showing the general lay of the land adjacent to the site.

The site is situated within the South Slope physiographic region. The geology generally comprises a till plain consisting of clayey silt to silty clay till (Halton Till) grading into a sandy silt to silty sand till with depth. The underlying bedrock consists of grey shale with hard siltstone and limestone interlayers of the Georgian Bay Formation.

3 SITE INVESTIGATION AND FIELD TESTING

A site investigation was not carried out as part of the current project. Instead, borehole information from a previous investigation at the site in 1982 (Reference 1) has been used. The 1982 field program consisted of drilling and sampling six (6) boreholes (numbered 1 to 6) at the site prior to the construction of the mainline bridges and approach fills. Boreholes 5 and 6 were located at the north abutments of the existing bridges, while Boreholes 1 and 2 were located about 10 m to the south of the south abutments. Boreholes 3 and 4 were located near the centreline of the existing Albion Road. All boreholes were drilled from the original grade prior to any embankment and bridge construction.

Boreholes 1 to 6 were terminated at depths ranging from 9.4 to 18.6 m (Elevations 153.8 to 163.4 m). The approximate borehole locations are shown on the Borehole Locations and Soil Strata Drawing in Appendix G. The elevations of the boreholes shown on these drawings and on the individual Record of Borehole sheets in Appendix A were obtained from Reference 1. The northing and easting co-ordinates shown on the drawing in Appendix G have been converted to the NAD83 system from an older co-ordinate system that was in use at the time of preparation of Reference 1.

Hollow stem augers were used to advance all six boreholes. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

Groundwater conditions were noted upon completion of the drilling operations. Piezometers were installed in Boreholes 1 and 5 to permit monitoring of groundwater levels.

4 LABORATORY TESTING

Moisture content determinations were carried out on all soil samples. Grain size distribution analyses (sieve and hydrometer) and Atterberg Limits testing were carried out on selected samples. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A obtained from Reference 1. Details of the encountered soil stratigraphy are presented in this appendix and on the Borehole Locations and Soil Strata Drawing in Appendix G. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

In general, the native soil stratigraphy encountered at this site comprises silty clay till, with sand and trace gravel, overlying silty sand till, with gravel. The existing embankment fill was not investigated since the previous boreholes were drilled before the embankments were constructed. More detailed descriptions of the individual strata are presented below.

5.1 Silty Clay Till

Native brown, fissured silty clay till, with sand and trace gravel, containing thin fine sand layers was encountered below the topsoil (previously existing) in Boreholes 1 to 6. The thickness of this cohesive till ranged between 4.3 and 6.7 m. The base of the silty clay till ranged from Elevations 165.3 to 168.1 m.

SPT 'N' values in the till ranged from 10 to 59 blows per 0.3 m of penetration indicating a stiff to hard consistency. An occasional lower 'N' value of 7 was measured at about 1.5 m depth in Borehole 1 indicating a firm consistency. The natural moisture contents of the silty clay till samples ranged from approximately 10% to over 25% (at shallow depth).

Grain size distribution results for two samples of the silty clay till are presented on the Record of Borehole sheets in Appendix A and on Figures 1 and 2 in Appendix B. Atterberg Limit test results are presented on Table 1 in Appendix B. The results of laboratory tests are summarized as follows:

Soil Particles	Silty Clay Till (%)
Gravel	0 to 1
Sand	17 to 19
Silt	38 to 52
Clay	28 to 45

Liquid Limit	44 to 49
Plasticity Index	22 to 27

The above results show that the silty clay till is of medium plasticity with a group symbol of CI. Measured unit weights of selected till samples ranged from 19.1 to 22.2 kN/m^3. Table II shows results of several "quick" triaxial compression tests which indicate that the measured shear strength of the silty clay till varied from about 100 to 200 kPa.

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It is noted that glacial tills inherently contain cobbles and boulders which are expected to be present within the deposit, although these obstructions were not encountered in the boreholes.

5.2 Silty Sand Till

Native grey silty sand till with gravel was contacted below the silty clay till at 4.9 to 7.0 m depths (Elevations 165.3 to 168.1 m) in Boreholes 1 to 6. Boreholes 2 to 6 terminated within this till at depths of 9.4 to 15.7 m (Elevations 157.6 to 163.4 m). In Borehole 1, the till grades at a depth of 16.2 m (Elevation 156.2 m) into a fine sand with silt and thin layers of silty clay.

Based on SPT 'N' values ranging from 63 blows for 0.3 m of penetration to greater than 100 blows for less than 0.3 m penetration, the silty sand till and the underlying sand are described as very dense. The natural moisture contents of the silty sand till samples ranged approximately from 5% to 13%.

Grain size distribution curves for silty sand till and sand samples tested are presented on the Record of Borehole sheets and on Figures 3 and 4 in Appendix B.

The results of the laboratory tests are summarized as follows:

Soil Particles	Silty Sand Till (%)
Gravel	14 to 28
Sand	40 to 44
Silt	29 to 37
Clay	3 to 5

Soil Particles	Sand (%)
Gravel	0
Sand	65
Silt	22
Clay	13

5.3 Water Levels

Water levels were observed in the boreholes during and upon completion of drilling. Standpipe piezometers were installed in Boreholes 1 and 5 to monitor water levels after completion of drilling. The water levels measured in the piezometers are summarized in Table 5.1, along with the measurements in the boreholes upon completion of drilling.

Table 5.1 – Observed Groundwater Levels

Foundation Element	Borehole	Date	Water Level (m)		Comment
			Depth (m)	Elevation (m)	
South Abutment	1	February 13, 1982 February 19, 1982 February 26, 1982	7.3 6.9 5.3	165.0 165.4 167.0	In piezometer installed in silty sand till
	2	February 19, 1982	- *	- *	In open borehole
Mid Span (no foundation)	3	February 18, 1982	8.6	163.9	In open borehole
	4	February 19, 1982	6.8	165.8	In open borehole
North Abutment	5	February 19, 1982 February 26, 1982	3.0 2.4	169.9 170.5	In piezometer installed in silty sand till
	6	February 18, 1982	9.7	163.5	In open borehole

* No free water observed in borehole

Groundwater levels measured in the piezometers ranged from Elevations 167.0 to 170.5 m. These readings appear to suggest the presence of a downward hydraulic gradient within the silty sand till, which is not uncommon in the Toronto area.

Lower water levels were noted in open boreholes at elevations ranging from 163.5 to 165.8 m during the previous investigation.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may reach higher elevations after the spring snowmelt or after periods of heavy rainfall. Further, perched water may be encountered at higher levels in pockets or zones of more permeable sands and silts present within the heterogeneous tills, or within the fill.

6 MISCELLANEOUS

All borehole and laboratory testing information was obtained from Reference 1.

Interpretation of the subsurface data and preparation of this report were carried out by Dr. Sydney Pang, P. Eng.

Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects, reviewed the report.

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PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data from the 1982 MTO report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach fills for the proposed widening of the NBL and SBL bridges within the air gap at the Highway 427 overpass at Albion Road in Toronto, Ontario.

Based on the General Arrangement (GA) drawing provided by SNC-Lavalin, the existing structures are single span, welded steel box girder twin bridges carrying the Highway 427 southbound lanes (SBL) and northbound lanes (NBL) over Albion Road. Each of the SBL and NBL bridges measures approximately 32.5 m in span length between abutment bearings. Each bridge is supported on two abutments founded on spread footings. The approach embankments are in the order of 7 to 8 m high with a design inclination up to 2H : 1V for the side and forward slopes.

There is an existing median retaining wall (parallel to Albion Road) between the NBL and SBL abutments. It is understood that these two walls were only designed to retain the existing median fill slopes and are therefore underdesigned for providing adequate foundation support to the proposed widening structures.

Preliminary GA drawings, dated August 4, 2009, provided by SNC-Lavalin indicate that the proposed structure widenings will be located within the air gap between the two existing structures. It is understood that the existing retaining walls between the twin structures, barrier walls and the top portion of the wingwalls adjacent to the widening structures will be removed. New abutment walls for the widening structures will then be constructed.

The discussions and recommendations presented in this report are based on our understanding of the project and on the factual data obtained during the course of the investigation.

8 FOUNDATION DESIGN

Consideration was given to various alternate foundation systems, taking into account the site stratigraphy and the structure General Arrangement.

In general, the stratigraphy encountered at the site consists of embankment fill overlying native stiff to hard silty clay till which is underlain by very dense silty sand till. Piezometers installed in two boreholes within the silty sand till indicated that the measured groundwater levels decrease with depth from about Elevations 170.5 to 167.0 m. Perched water may be encountered at higher levels within the fill and the silty clay till.

Initial consideration was given to the following foundation types:

- Spread footings on undisturbed native soil
- Spread footings on engineered fill
- Augered caissons (drilled shafts)
- Driven steel H-piles

A comparison of these foundation alternatives based on advantages and disadvantages of each is included in Appendix D. In order to avoid undermining the existing bridge foundations during construction of the foundations for the bridge widening and to achieve consistency with the existing structures, it is recommended that the proposed foundation layouts of the abutments for the widening structures be similar to the existing ones, i.e. both abutments should be supported by spread footings founded on native stiff silty clay till.

8.1 Spread Footings on Native Soil

It is recommended that spread footings on native stiff silty clay till be used as foundation support for the north and south abutments of the bridge widening. Deep excavations up to the order of 10 m would be required to penetrate the fill, remove the existing retaining walls between the twin bridges, and to construct the spread footings on competent stiff to hard silty clay till. The GA drawings also indicate that the existing barrier walls (wing walls of the existing abutments) adjacent to the proposed widening structures will also be removed to facilitate foundation and structural connections between the old and new abutments.

It is understood that the existing abutment footings were designed to be founded on the native, stiff to hard silty clay till at approximate Elevation 171.0 m. Each of the existing footings is 1.0 m in thickness.

It is recommended that all new footings be founded at similar elevations as the existing footings such that the latter will not be undermined. It is critical for the designer to have accurate information on the existing footing base elevations and outlines of existing footing footprints to avoid interference between new and existing footings.

Spread footings should be founded on undisturbed stiff to hard silty clay till. Provided a minimum footing width of 2 m is maintained, footings founded on the above recommended stratum may be designed in accordance with the elevations and bearing resistances given in Table 8.1.

Table 8.1 – Bearing Resistances for Spread Footings

Foundation Element	Borehole	Founding Elevation (m)	ULS_f (kPa)	SLS (kPa)	Soil
South Abutment	1	171.0	450	300	Stiff to hard silty clay till
	2	171.0	450	300	
North Abutment	5	171.0	450	300	
	6	171.0	450	300	

* Based on approximate Albion Road centreline grade at Elevation 173 m.

The native, undisturbed silty clay till is typically very stiff to hard in consistency in all borehole locations except for the upper 2 m in Borehole 1 where lower SPT 'N" values of 7 and 10 blows per 0.3 m of penetration, indicating a firm to stiff consistency, were measured. Further recommendations on footing construction are provided in Section 8.1.1 of this report.

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

The geotechnical SLS resistance values given above are based on an estimated total settlement not exceeding 25 mm. This settlement is expected to be substantially complete by the end of construction. Differential settlement is not expected to exceed 20 mm in a 6 m span.

The sliding resistance at the interface between the mass concrete and the underlying native, undisturbed silty clay till may be computed on the basis of an ultimate coefficient of friction of 0.45. This is an "ultimate" value and requires a degree of sliding movement to occur to fully mobilize the resistance.

The bases of the foundation excavations should be inspected by a geotechnical engineer to confirm that the exposed footing subgrade conforms to the design requirements and has been adequately prepared to receive concrete. Where sub-excavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using engineered fill or mass concrete of the same class as the footing concrete.

The engineered fill must consist of OPSS Granular "A" placed in 150 mm lifts, compacted to 100% of its SPMDD at ±2% of the optimum moisture content.

8.1.1 Construction of Spread Footings for New Abutments

Removal of the existing retaining walls, removal of the abutment barrier walls, top portion of the wingwalls, and construction of the new abutments will require excavation of a portion of the existing median slopes. It is anticipated that a majority of the work will have to be carried out within temporarily supported excavations. The retained height of the median slopes will be up to the order of 10 m.

Special attention and care should be given to protection system installation and excavation operations in order not to damage the existing abutments nor to destabilize the adjacent forward slopes. The precautions that should be taken to minimize the risk of such occurrences are as follows:

- After partial removal of the existing approach slabs and top of the existing barrier (wing) walls, protection systems will have to be installed for each bridge in the median and in front of the existing retaining walls. Care should be taken to minimize the vibration associated with installation of the protection systems. Additional comments on protection system design are provided in Section 10.
- Bulk excavation and median retaining wall removal should not commence until all components of the required protection systems are installed, tested (where required) and approved. Consideration may be given to carrying out the new footing construction in shorter sections where practicable, i.e. attempts should be made to avoid removing the wall and opening up the excavation in its entirety all at once.
- Should any sub-excavation be required below the elevations allowed for in the protection system design, the Contract Administrator (CA) must first be informed who should then consult the protection system designer to decide on the appropriate course of action.
- Backfilling should commence as soon as the site conditions permit doing so.

Pre-construction condition survey of the existing abutments should be carried out to document the prevailing conditions. Settlement monitoring of the existing abutments should be carried out before (baseline conditions), during and after construction. Monitoring of potential vertical and lateral movements of the protection system should be considered. Visual inspection of the back slopes at the median should be carried out periodically to confirm stability.

8.2 Spread Footings on Engineered Fill

Spread footings on engineered fill are not recommended at this site. This is not a feasible option for this bridge widening due to the presence of competent native soils at shallow depths.

8.3 Augered Caissons (Drilled Shafts)

Augered caisson (drilled shaft) foundations socketted into the underlying very dense silty sand till may be used to support the structural loads at this site. Table 8.1 presents the recommended founding depths and elevations for caissons at each abutment location. Each caisson should be at least 6 m in length and should extend at least 4.0 m into the "100-blow" till.

Table 8.1 – Founding Depths and Elevations for Augered Caissons

Foundation Element	Borehole	Founding Elevation (m)
South Abutment	1	161.0
	2	161.0
North Abutment	5	161.0
	6	162.0

* Based on approximate Albion Road centreline grade at Elevation 173 m.

The following Table 8.2 presents the geotechnical resistances recommended for typical 1.2 and 1.5 m diameter caissons associated with the founding depths given in Table 8.1.

Table 8.2 – Vertical Geotechnical Resistance for Caisson Foundations

Caisson Diameter (m)	Axial Geotechnical Resistance For Caissons Founded within Native Silty Sand Till	
	Factored ULS_f (kN)	SLS (kN)
1.2	4,500	3,600
1.5	6,500	5,000

8.3.1 Caisson Installation

Caisson installation should be in accordance with Special Provision No. 903S01.

The soil providing the resistance, whether it is skin friction or end bearing, must be protected from disturbance.

The caisson installation equipment should be able to dislodge and remove any obstructions such as cobbles, boulders and rock slabs in the till. The silty sand glacial till is very dense at this site and augering might be laboured. The contract documents must contain a statement to alert bidders of these facts. The suggested wording for an NSSP addressing this issue is included in Appendix E.

The resistance value provided above is based on the assumption that the walls and base of each caisson are cleaned of loose material prior to placement of concrete. The caisson excavation should be unwatered (if necessary) to allow cleaning of the base and walls and prior to placing concrete. Concrete should be placed with a minimum delay after the socket is drilled and cleaned. A delay of 24 hours is considered to be the maximum permissible and the caisson must be maintained in an unwatered condition throughout the time lapse until concrete placement.

8.4 Driven Piles

Driven steel H-piles were also considered for providing foundation support to the new abutments. If used, such piles should be driven to refusal within the very dense silty sand till.

The existing abutments are founded on spread footings. In order to achieve refusal within the silty sand till, the piles would have to penetrate the silty clay till and the upper portion of the silty sand till. Vibration as a result of pile driving through these hard and very dense tills could have adverse effects on the adjacent existing footing foundations. Potential obstructions within the glacial tills may require pre-augering to reach the desired pile tip elevations.

As such, the use of driven pile foundations is not recommended for the bridge widenings at this site.

9 TEMPORARY EXCAVATION

9.1 General

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the soils within the likely depth of excavation at this site may be classed as Type 3 soils for fill and Type 2 for native stiff to hard silty clay till.

The selection of the method of excavation is the responsibility of the contractor and must be based on his equipment, experience and interpretation of the site conditions. Excavations should be inspected regularly for evidence of instability if they have been left open for extended periods of time and following periods of heavy rain or thawing. If required, remedial actions must be taken to ensure the stability of the excavation and the safety of workers.

The requirements for unwatering during excavation are discussed in Section 11.

Earth excavations for footing construction required at this site will penetrate through a variety of overburden soils including fill and native silty clay till. The soils, especially the tills, may contain cobbles, boulders and shale fragments. It is anticipated that temporary excavations at this site may be formed, where space permits, with temporary side slopes not steeper than 1H : 1V. Flatter slopes may be required at locations where the soils are less competent than what is assumed during design or where water seepage affects surficial stability.

An NSSP should be included in the contract alerting the Contractor to the possible presence of cobbles and boulders.

10 ROADWAY PROTECTION

Roadway protection will be required during construction at the existing abutment locations and the back slopes within the median. An item titled "Protection System" as per SP 105S19 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.02.01 and the alignment of the shoring be specified on the contract drawings.

The design of roadway protection should be the responsibility of the Contractor. However, one option that is considered to be suitable for use as protection system at this site is a soldier pile and lagging wall. It is anticipated that the soldier piles will need to be socketted into the very stiff to hard silty clay till or very dense silty sand till to develop the required toe resistance. The system may be stiffened by walers, cross bracings and/or soil anchors, where applicable.

A temporary soldier pile and lagging wall may be designed using the parameters given below:

γ	=	20 kN/m^3
γ_w	=	10 kN/m^3
K_a	=	0.35 (road embankment fill)
(active earth pressure)	=	0.33 (silty clay till)
K_p	=	2.9 (road embankment fill)
(passive earth pressure)	=	3.0 (silty clay till)

The designer of the roadway protection system should check whether the socket is sufficiently deep to provide base fixity.

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the protection system. All protection systems should be designed by a Professional Engineer experienced in such designs.

11 UNWATERING

Temporary excavations for footing construction are required to be carried out to approximate Elevation 171 m. The groundwater level can be as high as Elevation 170.5 m based on piezometer readings taken during the previous investigation. Perched water may be encountered at higher elevations within the cohesionless layers in the embankment fill.

Considering the consistency and low permeability of the silty clay till, groundwater control measures such as perimeter ditches and pumping from filtered sumps should be implemented to remove any accumulation of water from the footing excavation base prior to placing concrete. The possibility exists that additional pumps may be required if localized zones of perched water are encountered. Surface runoff should be diverted away from the excavations. All footings must be constructed in the dry.

The design of the unwatering systems is the responsibility of the Contractor.

12 APPROACH EMBANKMENTS

The foundation soils governing stability of the approach embankments consist of existing fill, native stiff to hard silty clay till underlain by very dense silty sand till. The proposed embankment heights are up to 7 to 8 m at the north and south approaches. Subsequent to excavation, retaining wall removal and new footing construction, new fill will have to be placed to reinstate the highway grade. Since the existing median is more or less at grade, the extra foundation load that may be induced due to additional fill placed to construct the new abutments should be minimal.

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002. It is recommended that earth fill should consist of SSM or granular materials in compliance with Special Provision 110F13, "Amendment to OPSS 1010 March 1993". Any existing fill slopes must be benched in accordance with OPSD 208.010 prior to placing new fill.

The embankment foundation soils are considered adequate to provide stability to new earth fills inclined at 2H : 1V or flatter.

Considering the embankment height and consistency of the foundation soils, settlement induced by embankment loading is expected to be less than 25 mm.

All topsoil and organic soils should be stripped from the footprint of the approach fills. Particular attention should be paid to removing all softened material from existing ditches that fall within the footprint of the new embankment.

Earth fill embankment slopes must be provided with erosion protection in accordance with SP572S01.

13 BACKFILL TO ABUTMENTS

Backfill to the abutments should consist of Granular A or Granular B Type II material meeting the requirements of Special Provision 110F13 "Amendment to OPSS 1010, March 1993". The backfill must be in accordance with OPSS 902 as amended by Special Provision 902S01, and placed to the extents shown in OPSD 3101.150.

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with SP105S01. The design of the abutment must include a subdrain as shown in OPSD 3102.100.

14 STATIC EARTH PRESSURE

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K\,.\,(\gamma h + q)$$

where:

P_h = horizontal pressure on the wall at depth h (kPa)
K = earth pressure coefficient (see Table 14.1)
γ = unit weight of retained soil (see Table 14.1)
H = depth below top of fill where pressure is computed (m)
Q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 14.1.

Table 14.1 – Earth Pressure Coefficients (K)

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A $\phi = 35°, \gamma = 22.8$ kN/m^3		OPSS Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°, \gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.27	0.40*	0.31	0.48*
At rest (Restrained Wall)	0.43	-	0.43	-	0.47	-
Passive	3.7	-	3.7	-	3.3	-

* For wing walls.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall.

The factors in Table 14.1 are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

15 SEISMIC CONSIDERATIONS

15.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.05

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

15.2 Liquefaction Potential

There is no potential for liquefaction of structures founded on very stiff to hard soils. The foundation soils at the site are assessed as not being prone to liquefaction.

15.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

For the design of retaining walls, the coefficients of horizontal earth pressure in Table 15.1 may be used:

Table 15.1 – Earth Pressure Coefficients for Earthquake Loading

Wall Condition	Earth Pressure Coefficients (K) for Earthquake Loading			
	OPSS Granular A or Granular B Type II $\phi = 35°$; $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; °, $\gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.29	0.44	0.33	0.53
Passive (K_{PE})	3.6	3.6	3.2	3.2
At Rest (K_{OE})**	0.59	-	0.63	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods

16 ADJACENT STRUCTURES AND BURIED UTILITIES

It is understood that an existing Bell Canada and an Etobicoke Hydro concrete duct banks are present adjacent to the toe of the south abutment walls just below Albion Road grade. Other buried utilities might also be present in the new foundation construction areas. It is recommended that the exact locations and elevations of these duct banks and utilities be established by the designer, and compared with the extent of the potential work zones related to the foundations of the proposed widening structures and associated works. The settlement and displacement/rotation tolerances of the duct banks and utilities should also be established.

It is recommended that the following be carried out prior to the commencement of construction:

- Carry out pre-construction condition survey including documentation of any existing distress associated with the existing structures and utilities. Any distress should be reported to and discussed with the structure/utility owner.
- Implement an instrumentation and monitoring program to include vibration and settlement monitoring during installation of shoring, excavation and new footing construction. Establish review and alert level criteria for allowable settlement and lateral movement following discussions with the owner of the structure/utility. Establish review and alert level criteria for vibration levels (in terms of peak particle velocity, ppv) during pile driving. Establish and agree on remedial action, if required, prior to start of construction.

- Carry out post-construction condition survey of the existing structures/utilities.

17 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

1. Destabilization of excavations

   Perched water may be encountered within the cohesionless fill layers. The impact of this perched groundwater might destabilized the sides and/or base of an excavation. The Contractor's unwatering plan must be available for rapid implementation should the need arise. Proper groundwater and surface water control measures must be in place prior to commencing excavation. All footings must be constructed in the dry.

2. Adjacent structures

   All new foundation footprints should be clear of any existing, adjacent structures and utilities. The existing abutment footings and buried duct banks must not be undermined. Vibration and settlement monitoring for adjacent structures and utilities, where required, should be provided by qualified personnel.

3. Existing slopes

   The retained back slopes at the median should be periodically inspected for potential instability during construction. Where necessary, remedial measures such as flattening of the slope inclination may be implemented. The forward and side embankment slopes should be inspected after construction for surficial disturbance. Where necessary, remedial measures such as re-vegetation and/or placement of gravel sheeting may be required for erosion control.

18 CLOSURE

Engineering analysis and preparation of this foundation design report was carried out by Dr. Sydney Pang, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

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Sydney Pang, P.Eng.
Associate, Senior Project Engineer

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
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PROVINCE OF ONTARIO

P.K. Chatterji, P.Eng.
Review Principal

Appendix A

Record of Borehole Sheets

(Previous Investigation)

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 1

03

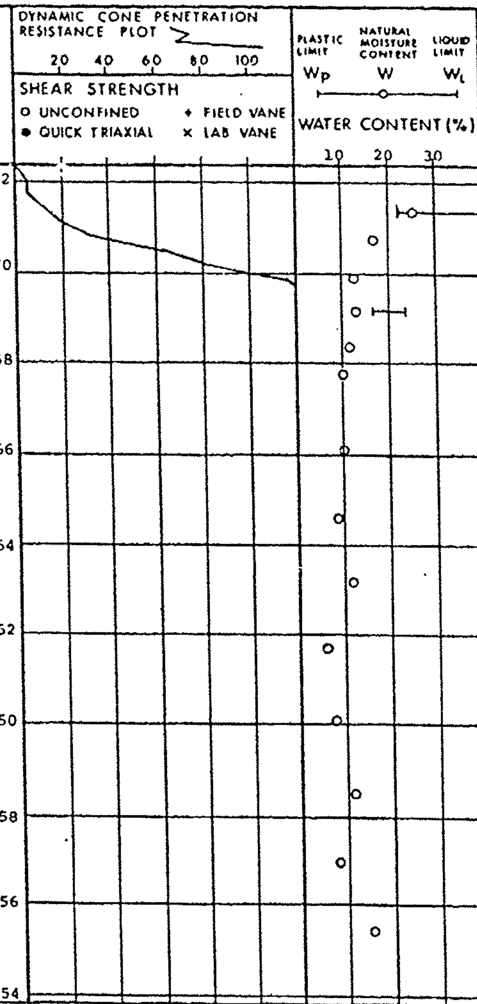
Metric

W P 153-80-02 LOCATION Co-ords. 4,845, 115N; 294, 281E ORIGINATED BY M.R.

DIST 6 HWY 427 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.P.

DATUM Geodetic DATE February 17, 1982 CHECKED BY SP

	ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0	172.32	Ground Level							
1	172.01 / 0.31	Topsoil, silty clay, low organic, dark brown					172		
		Silty clay with sand, trace gravel, fissured, thin fine sand layers, (Glacial Till)	1	SS	10			44	
			2	SS	7			19.1	0 17 38 45
7	170.19 / 2.13	Intermediate plasticity Stiff to Firm, Brown		SS	37		170		
		Silty clay with sand, trace gravel, fissured, thin sand layers (Glacial Till) Low plasticity	4	SS	53			22.2	1 19 52 28
			5	SS	30				
15	167.75 / 4.57	Hard Brown becoming very stiff, Grey	6	SS	26		168		
						▽			
			7	SS	27		166		
23	165.31 / 7.01	Silty sand fine to coarse with gravel, (Glacial Till) Very Dense Grey	8	SS	93				
							164	22.4	28 40 29 3
			9	SS	100/280 mm				
			10	SS	100/200 mm		162		14 44 37 5
			11	SS	100/280 mm		160		
			12	SS	100		158		
			13	SS	100/200 mm				
53	156.17 / 16.15	Sand, fine with silt, occasional thin layers of silty clay Very Dense Grey	14	SS	80/180 mm		156		0 65 22 13
60	153.75 / 18.57	End of Borehole	15	SS	100/280 mm		154		



Note:
½ hr. after sample 11, water at elevation 160.42 inside augers

Upon completion of augering, water at elevation 161.42 inside augers

Piezometer installed at elevation 154.03 seal at elevation 163.48

Date	Water Elevation
Feb. 13/82	165.02
Feb. 19/82	165.42
Feb. 26/82	167.02

+3, ×5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 2

Metric

W P 153-80-02 ~~03~~ LOCATION Co-ords. 4,845, 119N; 294, 317E ORIGINATED BY [illegible]

DIST 6 HWY 427 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.P.

DATUM Geodetic DATE February 19, 1982 CHECKED BY SP

	SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE	PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
0	172.70	Ground Level										
	172.09	Topsoil, silty clay, low organic Dark Brown						172				
2	0.61	Silty clay with sand, trace gravel, fissured, thin fine sand layers (Glacial Till) Intermediate plasticity Very Stiff Brown		1	SS	18						
				2	SS	28						
				3	SS	28		170				
10	169.65 / 3.05	Silty clay with sand trace gravel, fissured, thin fine sand layers (Glacial Till) Low plasticity Hard Brown to Grey		4	SS	42						
				5	SS	32						
				6	SS	35		168				
				7	SS	41		166				
23	165.69 / 7.01	Silty sand fine to coarse with gravel (Glacial Till) Very Dense Grey		8	SS	100		164				
30.?	163.28 / 9.42	End of Borehole		9	SS	100/130 mm						

Note:

After removal of augers upon completion of drilling, borehole caved at elevation 164.24, no free water

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

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

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 3

Metric

003

W P 153-80-02 LOCATION Co-ords. 4, 845, 141 N; 294, 277E ORIGINATED BY B.L.K.

DIST 6 HWY 427 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.P.

DATUM Geodetic DATE February 18, 1982 CHECKED BY SP

	ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³
0	172.52	Ground Level					
		Topsoil, silty clay, low organic, Dark Brown					
2.5	171.81 / 0.76	Silty clay with sand, trace gravel, fissured, thin fine sand layers (Glacial Till) Intermediate plasticity Very Stiff to Hard Brown	1	SS	30	172	
			2	TW	PH		20.9 20.6
			3	SS	34	170	
10	169.52 / 3.05	Silty clay with sand, trace gravel, fissured, thin fine sand layers (Glacial Till) Low plasticity Hard to Very Stiff Brown to Grey	4	SS	55		
			5	SS	48		
			6	SS	25	168	
21	166.17 / 6.40	Silty sand fine to coarse with gravel (Glacial Till) Very Dense Grey	7	SS	44	166	
			8	SS	100/250 mm		
						164	
30.8	163.18 / 9.39	End of Borehole	9	SS	100/250 mm		

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, 'N' VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 50 100 150 250 kPa; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Note:
After removal of augers upon completion of drilling, water level at elevation 163.89
Borehole caved at elevation 164.04

+³, ×⁵ : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 4

Metric

W P 153-80-02 -03   LOCATION Co-ords. 4, 845, 145 N. 294, 313E   ORIGINATED BY D.L.K.

DIST 6 HWY 427   BOREHOLE TYPE Hollow Stem Auger   COMPILED BY S.P.

DATUM Geodetic   DATE February 19, 1982   CHECKED BY SP

	ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
	172.59	Ground Level					
1	172.22	Topsoil, silty clay, low organic Dark Brown					172
	0.31	Silty clay with sand, trace gravel, fissured, thin fine sand layers (Glacial Till) Intermediate plasticity	1	SS	18		
			2	SS	21		
			[illegible]	SS	28		170
12	168.93	Very Stiff Brown	4	SS	24		
	3.66	Silty clay with sand, trace gravel, fissured, thin fine sand layers, (Glacial Till) Low plasticity	5	SS	39		
			6	SS	45		168
20	166.49	Hard Grey					
	6.10	Silty sand fine to coarse with gravel (Glacial Till)	7	SS	91	▼	166
			8	SS	88		
							164
31.2	163.09	Very Dense Grey	9	SS	100/200 mm		
	9.50	End of Borehole					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

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

Note:
After removal of augers on completion of drilling, water level and elevation 165.78 and borehole caved at elevation 165.68

OFFICE REPORT ON SOIL EXPLORATION

$+^3, \times^5$: Numbers refer to Sensitivity

15 $\diamond$ 5 (%) STRAIN AT FAILURE (20, 10)

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 5

Metric

W P 153-80-02 -03
LOCATION Co-ords. 4, 845, 161N; 294, 274E
ORIGINATED BY B.L.K.

DIST 6 HWY 427
BOREHOLE TYPE Hollow Stem Auger
COMPILED BY S.P.

DATUM Geodetic
DATE February 18, 1982
CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	ELEVATION SCALE
172.89	Ground Level				
172.28	Topsoil, silty clay, low organic — Dark Brown				172
0.61	Silty clay with sand, trace gravel, fissured, thin fine sand layers (Glacial Till) Intermediate plasticity Very Stiff to Hard — Brown	1	SS	26	
		2	SS	27	
169.84		3	SS	31	170
3.05	Silty Clay with sand, trace gravel, fissured, thin fine sand layers, (Glacial Till) low plasticity Hard — Brown	4	SS	49	
		5	SS	59	
168.01		6	SS	43	168
4.88	Silty sand, fine to coarse with gravel (Glacial Till)	7	SS	63	166
		8	SS	93	
163.36	Very Dense — Grey	9	SS	100/230 mm	164
9.53	End of Borehole				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, 'N' VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Handwritten margin: 0, 2, 10, 16, 31.3

Note:
After removal of augers upon completion of drilling, water level at elevation 164.97 and borehole caved at elevation 165.57

Piezometer installed at elevation 163.44 seal at elevation 171.06

Date	Water Elevation
Feb. 19/82	169.92
Feb. 26/82	170.51

(possible perched water infiltration)

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

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

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 6

Metric

W P 153-80-02 LOCATION Co-ords. 4. 845, 168N; 294; 309E ORIGINATED BY B.L.K.

DIST 6 HWY 427 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.P.

DATUM Geodetic DATE February 17/18, 1982 CHECKED BY SP

OFFICE REPORT ON SOIL EXPLORATION

	SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE (50 100 150 200 kPa)	PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES						
0	173.22	Ground Level									
2	172.61	Topsoil, silty clay, low organic Dark Brown									
	0.61	Silty clay with sand, trace gravel, fissured, thin fine sand layers (Glacial Till) Intermediate plasticity	1	SS	15		172				
			2	SS	21						
9	170.48	Very Stiff Brown	3	TW	PH					20.1	
	2.74	Silty clay, with sand, trace gravel, fissured, thin fine sand layers, (Glacial Till) Low plasticity	4	SS	59		170				
			5	SS	52						
17	168.1	Hard Brown to Grey	6	SS	27		168				
	5.18	Silty sand, fine to coarse with gravel (Glacial Till)	7	SS	88/250 mm						
							166				
		Very Dense Grey	8	SS	100/200 mm						
			9	SS	100/200 mm	▽	164				
			10	SS	100/180 mm		162				
			11	SS	94		160				
			12	SS	100/150 mm						
51.3	157.57		13	SS	100/250 mm		158				
	15.65	End of Borehole									

Note:
After removal of auger upon completion of drilling, water level at elevation 163.47 and borehole caved at elevation 167.43

$+^3, \times^5$: Numbers refer to Sensitivity

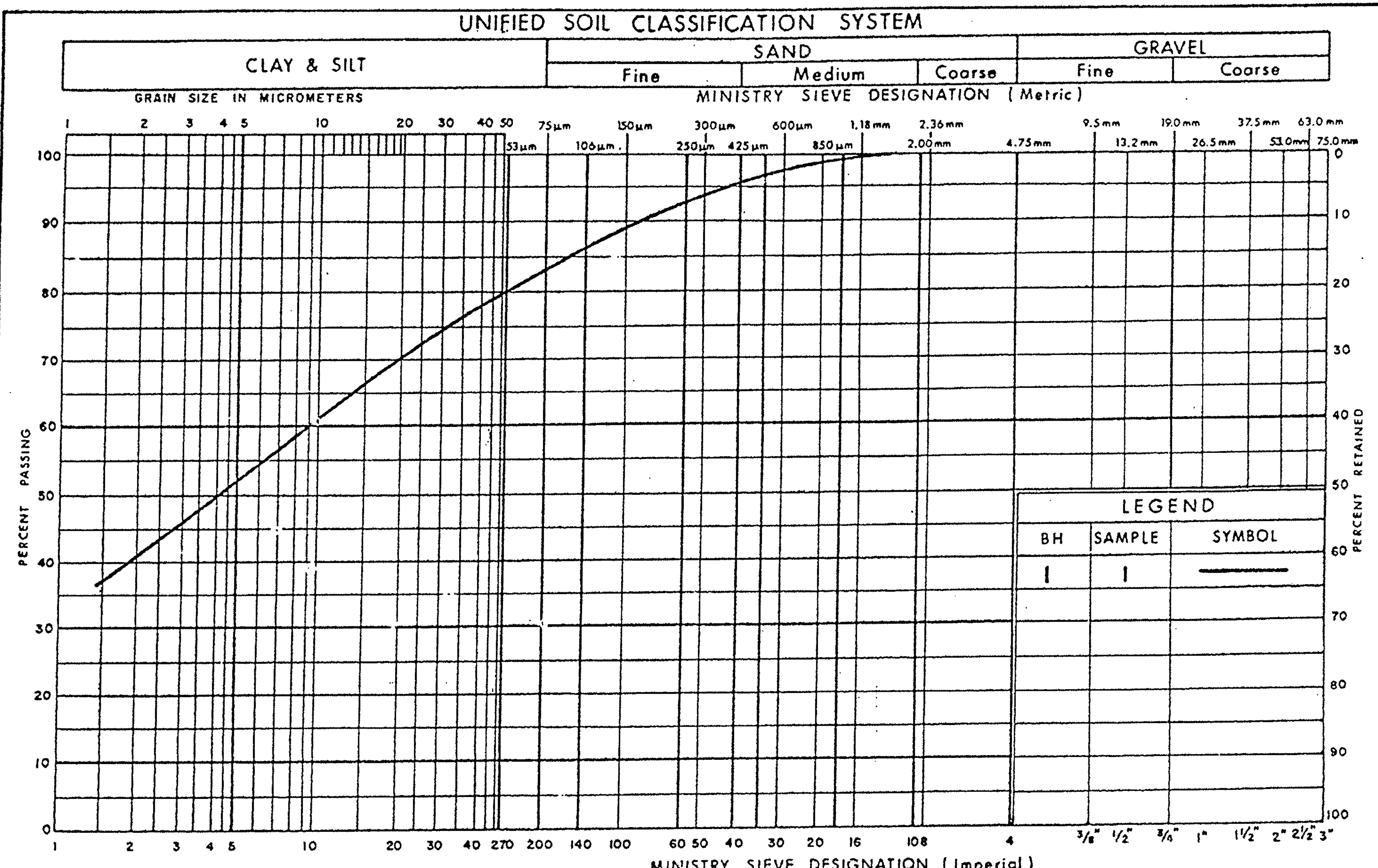
15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Appendix B

Laboratory Test Results

(Previous Investigation)

78 12 M



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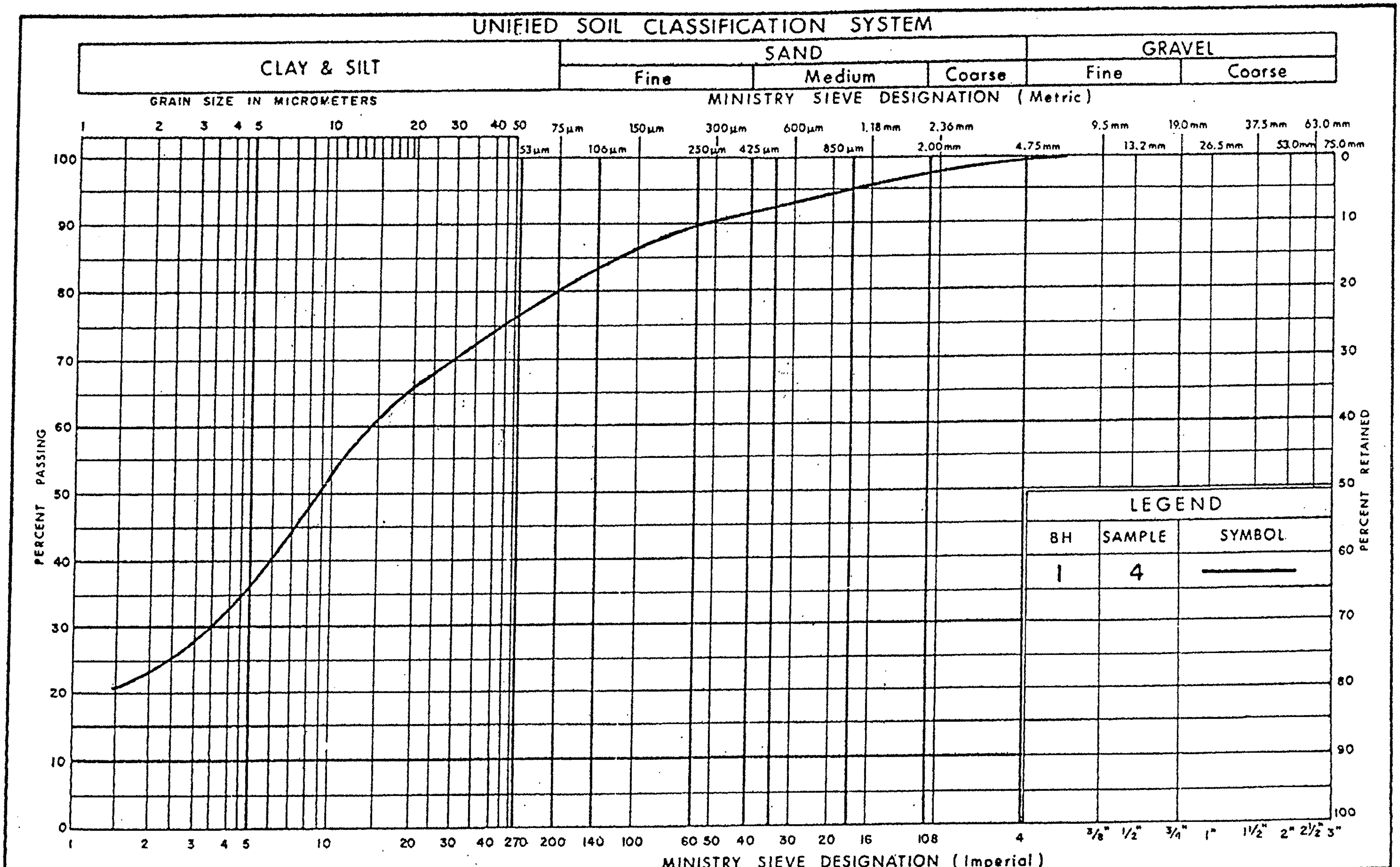
GRAIN SIZE DISTRIBUTION
SILTY CLAY (GLACIAL TILL)
WITH SAND

FIG No 1
WP 153-80-02

78 12 M



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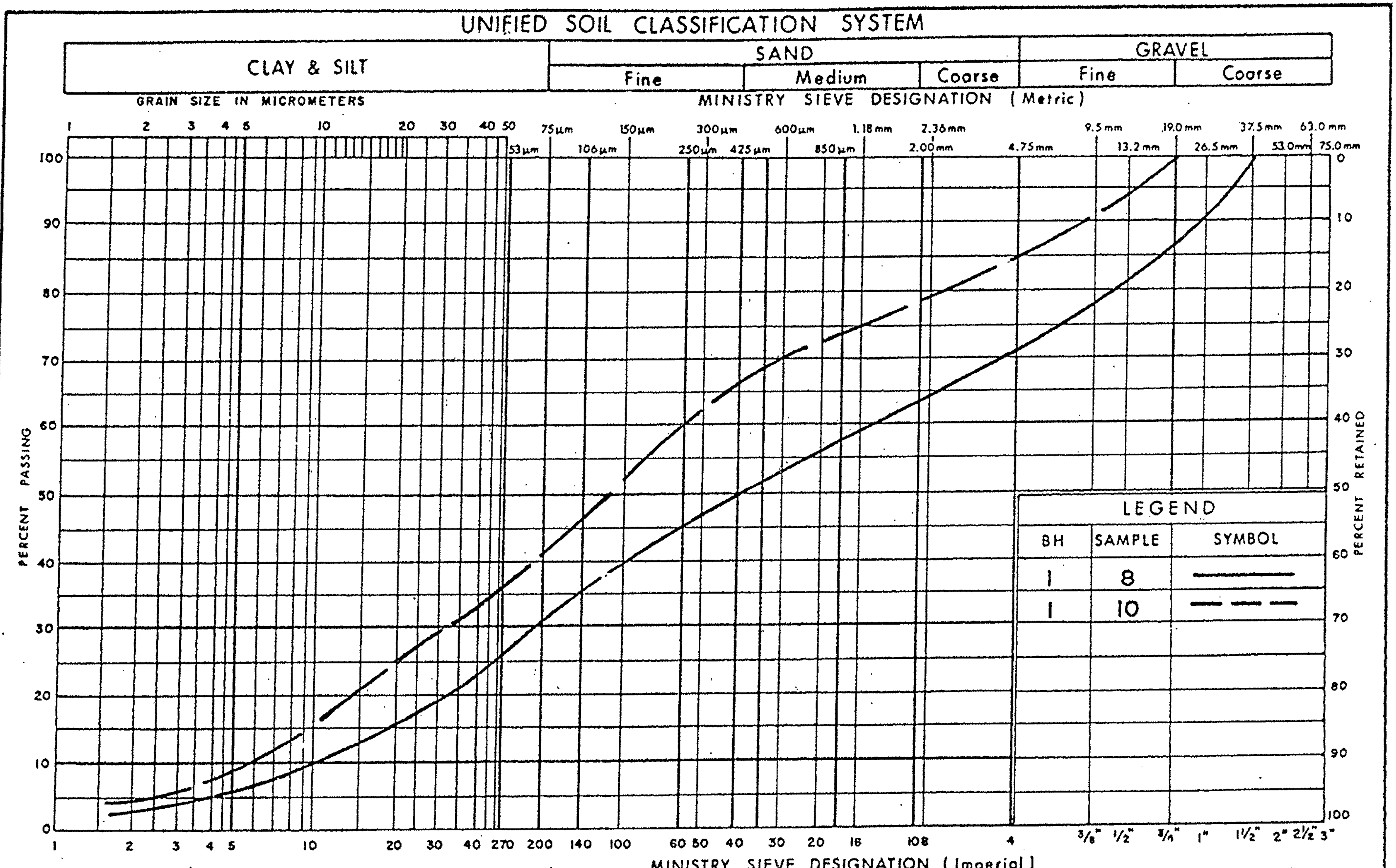
GRAIN SIZE DISTRIBUTION
SILTY CLAY (GLACIAL TILL) WITH SAND

FIG No 2

WP 153-80-02

78 12 M



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GRAIN SIZE DISTRIBUTION

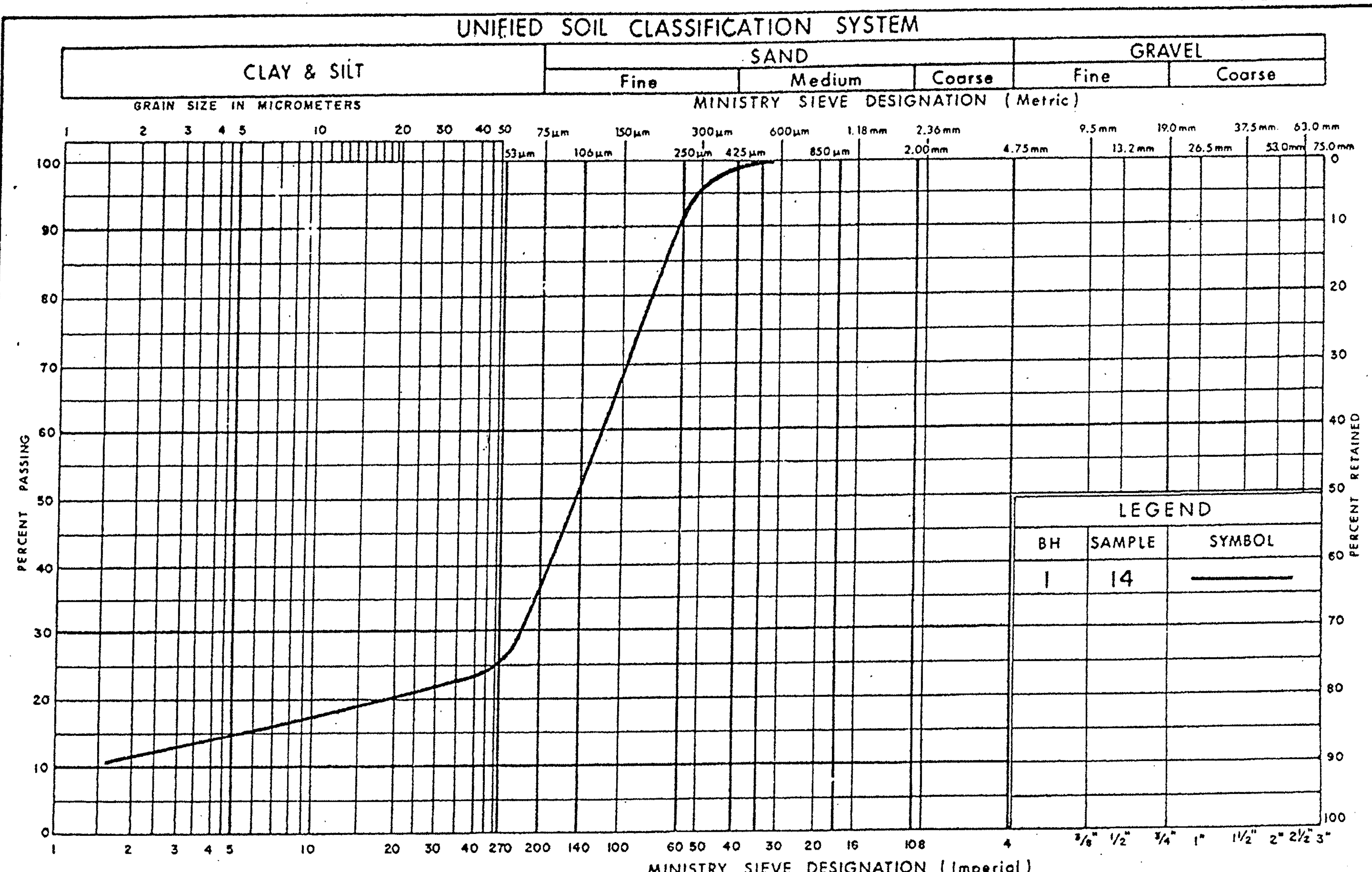
SILTY SAND (GLACIAL TILL)

FINE TO COARSE WITH GRAVEL

FIG No 3

W P 153-80-02

78 12 M



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Ontario

GRAIN SIZE DISTRIBUTION

SAND

FINE WITH SILT, OCCASIONAL THIN LAYERS OF SILTY CLAY

FIG No 4

W P 153-80-02

JOB NO. 82 F 18
MARCH, 1982

TABLE I

ATTERBERG LIMIT TEST RESULTS

Albion Road Underpass
at Highway 427

BOREHOLE NO.	SAMPLE NO.	DEPTH (m)	NATURAL WATER CONTENT (w) %	LIQUID LIMIT (w_L)	PLASTIC LIMIT (w_p)	PLASTICITY INDEX (I_p)	REMARKS
1	1	0.76-1.22	25	44	22	22	silty clay (CI)
3	2	1.52-1.98	22	49	22	27	silty clay (CI)
3	2	1.52-1.98	18	32	17	15	silty clay (CI)
6	3	2.29-2.74	21	32	16	16	silty clay (CI)
1	4	3.05-3.50	12	23	16	7	silty clay (CL)
1	8	7.62-8.07	9	Non-Plastic			silty sand

JOB NO. 82 F 18
MARCH, 1982

TABLE II

"QUICK" TRIAXIAL COMPRESSION TEST RESULTS

Albion Road Underpass
at Highway 427

BOREHOLE NO.	SAMPLE NO.	DEPTH (m)	NATURAL WATER CONTENT (w) (%)	UNIT WEIGHT WET (γ) (kN/m³)	UNIT WEIGHT DRY (γd) (kN/m³)	VOID RATIO (e)	DEGREE OF SATURATION (S_r) (%)	CELL PRESSURE ($\sigma 3$) (kPa)	FAILURE STRAIN (εf) (%)	SHEAR STRENGTH (τf) (kPa)	REMARKS
3	2	1.52-1.98	22.1	20.9	17.1	0.55	100	37.2	5.0	154	silty clay (CI)
			17.9	20.6	17.4	0.51	94	37.2	5.8	204	silty clay (CI)
6	3	2.29-2.74	20.6	20.1	16.6	0.59	94	49.6	5.2	99	silty clay (CI)

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Spread Footings on Native Soil	Spread Footings on Engineered Fill	Driven Piles	Augered Caissons
Advantages: i. Ease of construction. ii. Relatively high geotechnical resistance is available on the very stiff to hard till deposits. iii. Lower cost than deep foundations. iv. Consistent with existing foundation types	***Advantages:*** i. Generally less costly construction than deep foundation elements. ii. Founding level can be adjusted.	***Advantages:*** i. Piles will develop high geotechnical resistance in hard or very dense tills. ii. Foundation construction requires less volume of excavation than footings.	***Advantages:*** i. High geotechnical resistance is available for caissons founded within hard or very dense tills. ii. Foundation construction requires less volume of excavation than footings. iii. Likely requires smaller work zone than other alternatives during construction. iv. Sub-excavation of fill and variable material not required.
Disadvantages: i. Deep excavation will be required to penetrate extensive layer of fill. ii. Unwatering may be required, depending on the depth of excavation.	***Disadvantages:*** i. Potentially labouring sub-excavation through very stiff to hard native till will be required to accommodate the engineered fill pad. ii. Deep excavation will be required to penetrate the fill. iii. Unwatering may be required, depending on the depth of excavation. iv. Additional cost of engineered fill placement.	***Disadvantages:*** i. Higher unit costs than footings. ii. Potential adverse effects on the existing footing foundations due to vibrations associated with pile driving. iii. Potential difficulties for the piles to penetrate cobbles, boulders and hard zones within the tills.	***Disadvantages:*** i. Higher unit cost than footings. ii. Potential difficulties during augering to dislodge, remove or otherwise penetrate cobbles, boulders and hard zones within the tills. iii. More likely to encounter groundwater. iv. Potential difficulty in cleaning and inspecting bases.
RECOMMENDED	**NOT RECOMMENDED**	**NOT RECOMMENDED**	**FEASIBLE**

THURBER

Appendix D

List of SPs and OPSS

Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- SP 902 S01.
- SP 572 S01
- OPSS 120, 1994
- SP 105S19
- OPSS 206
- OPSS 1010
- OPSD 208.010
- OPSS 902
- Special Provision 903S01
- OPSD 3101.150.
- OPSD 3102.100

OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002.

All granular material should meet the specifications of Special Provision 110F13 "Amendment to OPSS 1010, March 1993".

2. Suggested Text for NSSP on "Impact on Adjacent Structure"

It is critical that Contractor's excavation and construction activities do not undermine or have any adverse impact on the integrity and performance of the following adjacent structures:

- *The lanes of the Highway 427 during excavation and foundation construction at the new north and south abutments.*
- *Protection of the existing structure foundations, back slopes at median, duct banks and other utilities during excavation and pile driving.*
- *Protection of existing approach fills.*

3. Suggested Text for NSSP on "Drilling of Caisson Sockets"

Caisson installation through the till may encounter cobbles, boulders or rock slabs and the installation equipment should be capable of dislodging and removing such obstructions.

Appendix E

Site Photographs

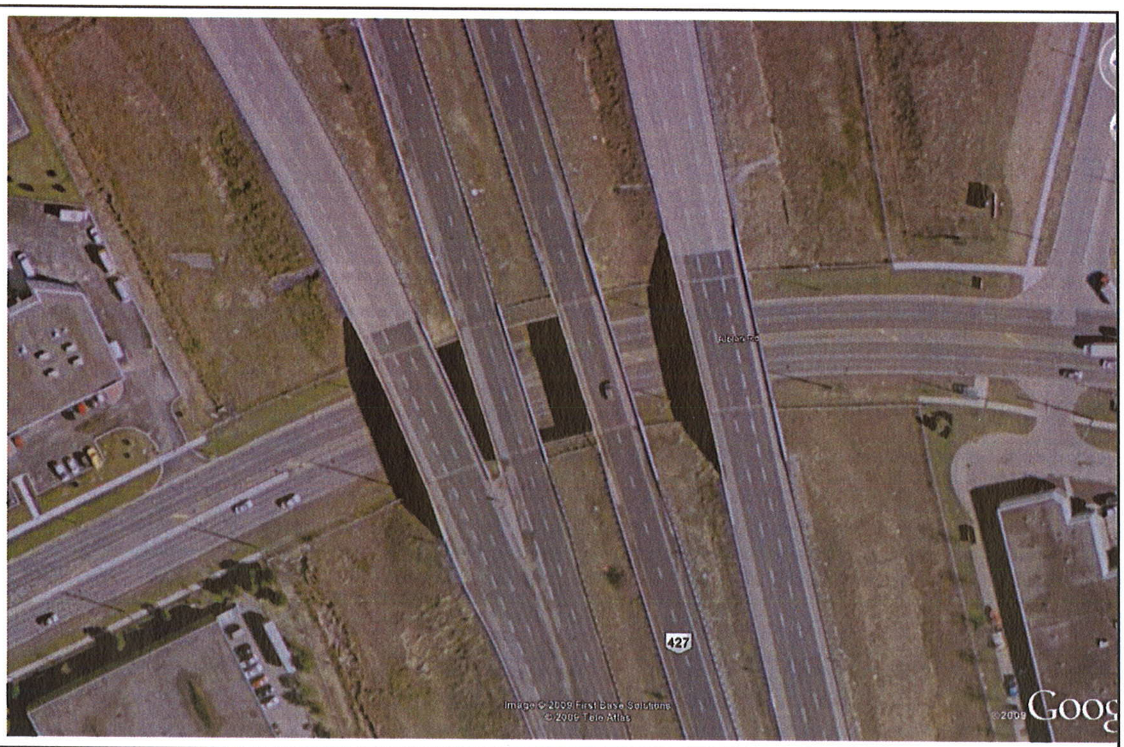


Photo 1. Aerial photograph of the site

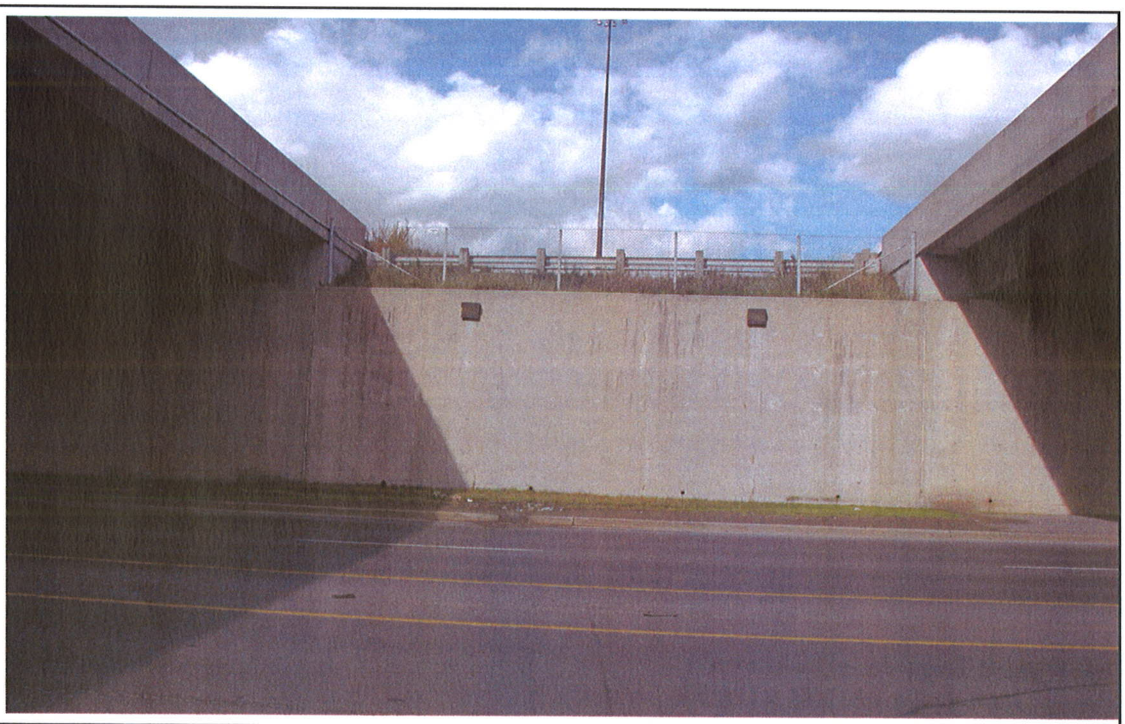


Photo 2. Photograph of existing north retaining wall

Appendix F

Borehole Locations and Soil Strata Drawings

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

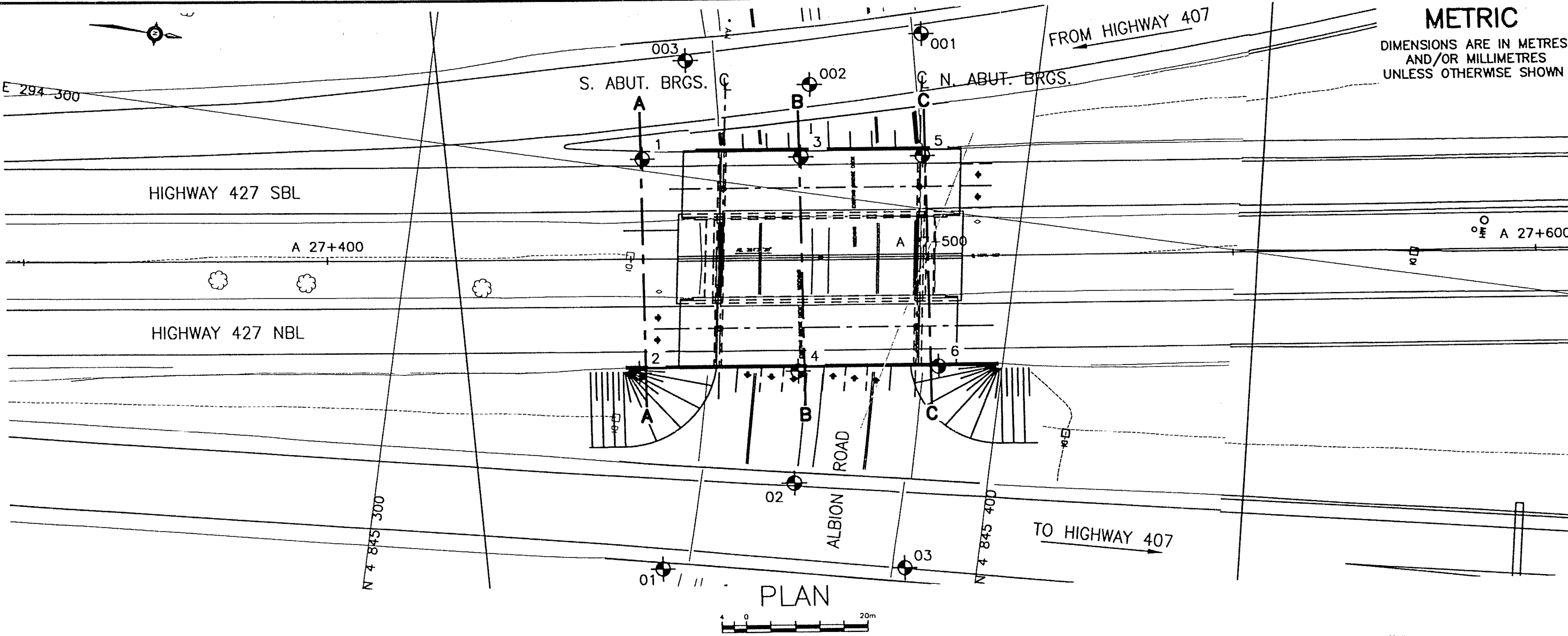
CONT No
GWP No 202-95-00

HIGHWAY 427
ALBION ROAD OVERPASS
REHABILITATION & WIDENING
BOREHOLE LOCATIONS AND SOIL STRATA

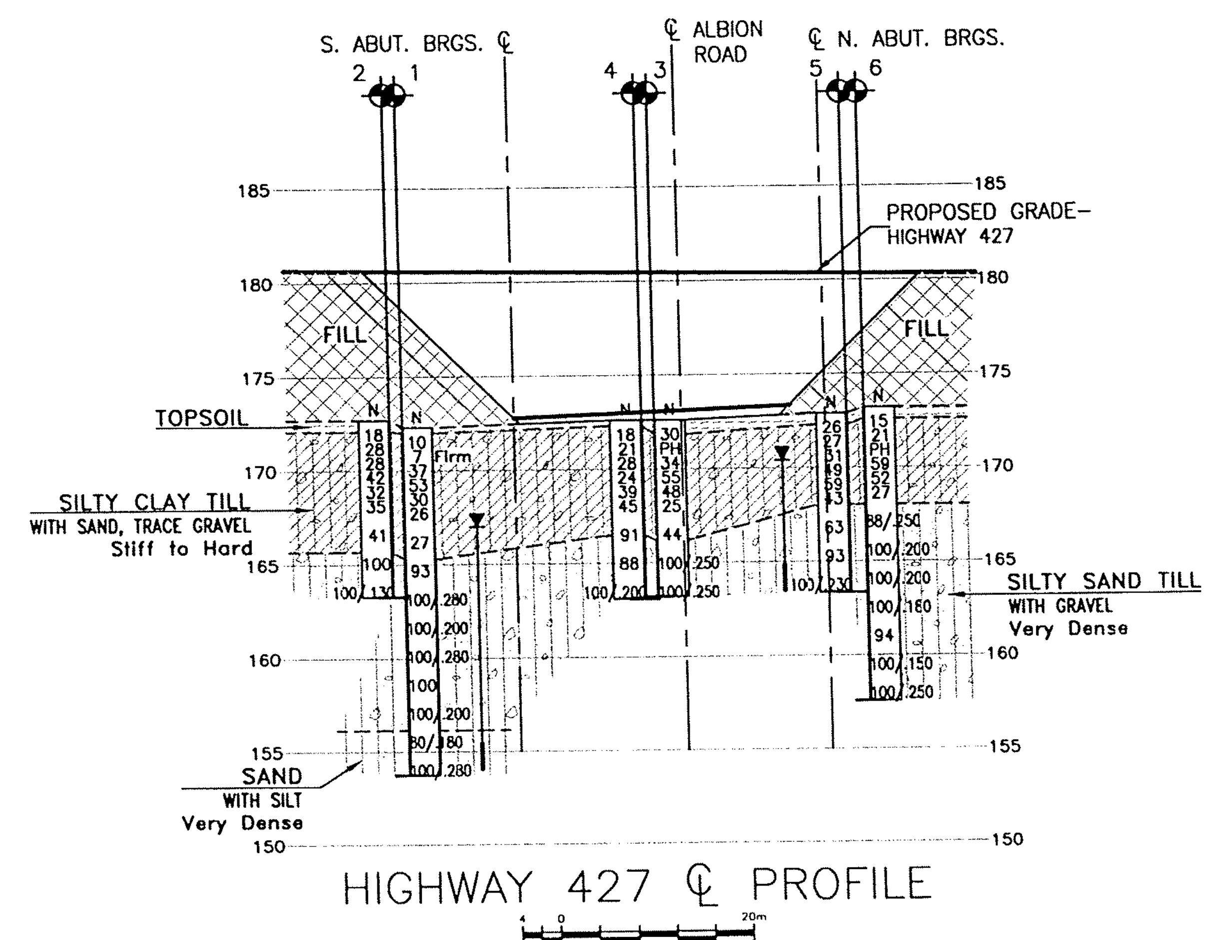
SHEET

SNC·LAVALIN

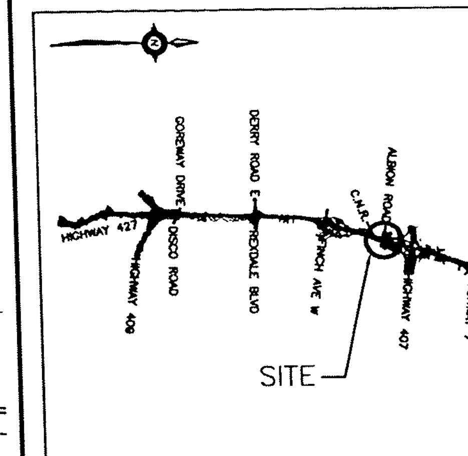
THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



PLAN



HIGHWAY 427 ℄ PROFILE



KEYPLAN

S. PANG
Nov. 26/09
LICENSED PROFESSIONAL ENGINEER
PROVINCE OF ONTARIO

P. K. CHATTERJI
Nov 26/09
LICENSED PROFESSIONAL ENGINEER
PROVINCE OF ONTARIO

LEGEND

	Borehole by Thurber (Present Investigation)
	Borehole by Others (Previous Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
1	172.3	4 845 336.6	294 298.1
2	172.7	4 845 340.6	294 334.1
3	172.6	4 845 362.6	294 294.1
4	172.6	4 845 366.6	294 330.1
5	172.9	4 845 382.6	294 291.1
6	173.2	4 845 389.6	294 326.1
01	172.6	4 845 348.6	294 366.2
02	172.8	4 845 368.3	294 348.9
03	173.2	4 845 388.2	294 360.5
001	172.1	4 845 379.9	294 270.6
002	172.1	4 845 362.5	294 281.7
003	172.2	4 845 341.7	294 280.5

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

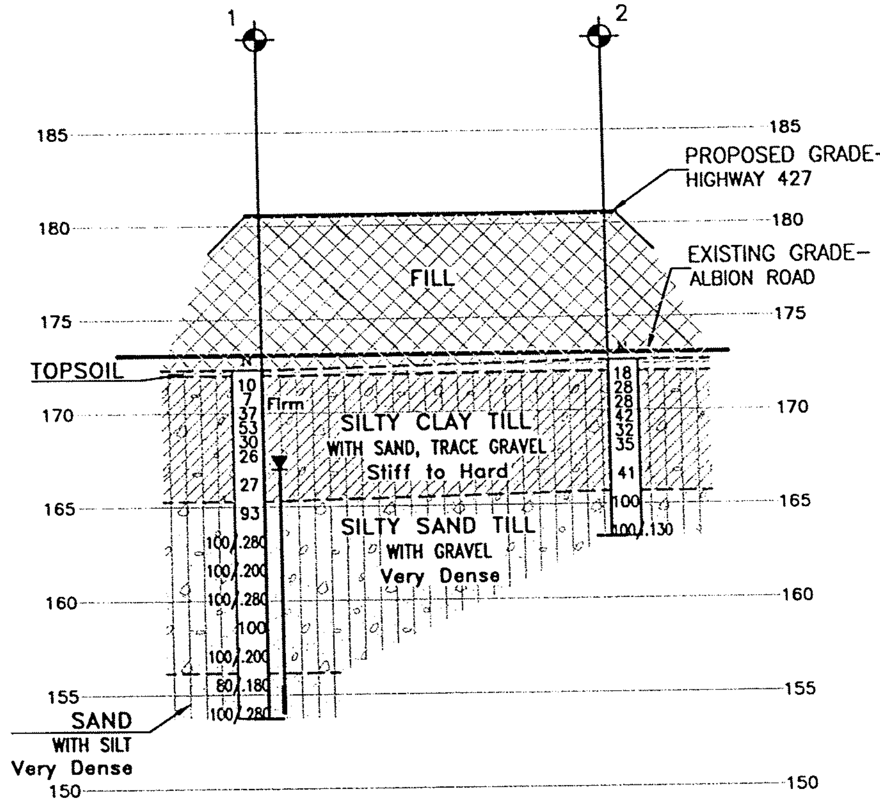
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 30M12-290

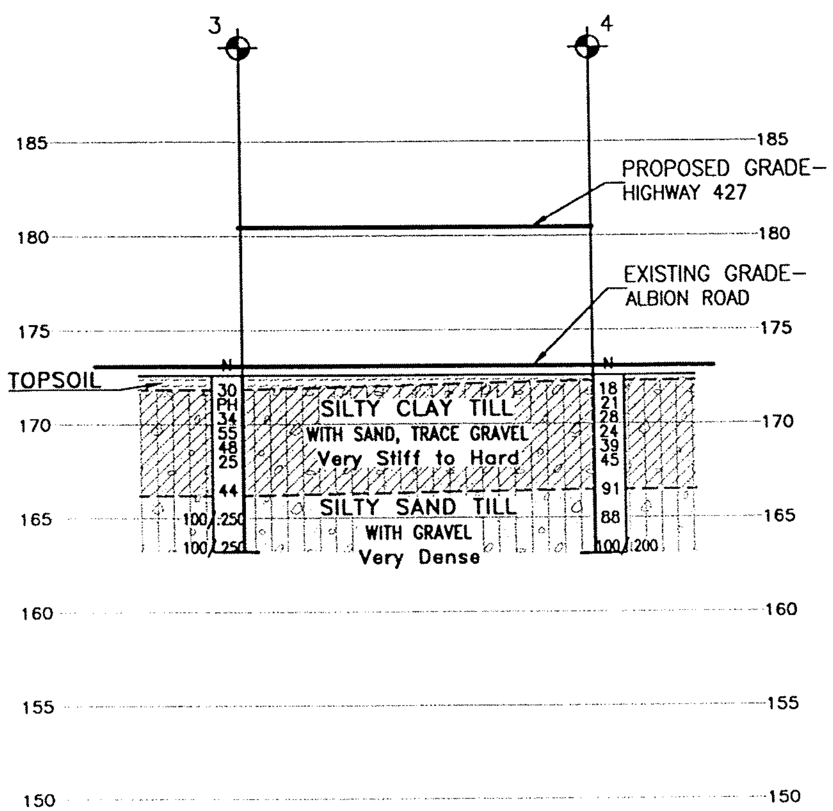
REVISIONS	DATE	BY	DESCRIPTION		
DESIGN SKP	CHK SKP	CODE	LOAD	DATE NOV. 2009	
DRAWN MFA	CHK PKC	SITE	STRUCT	DWG 1	

METRIC

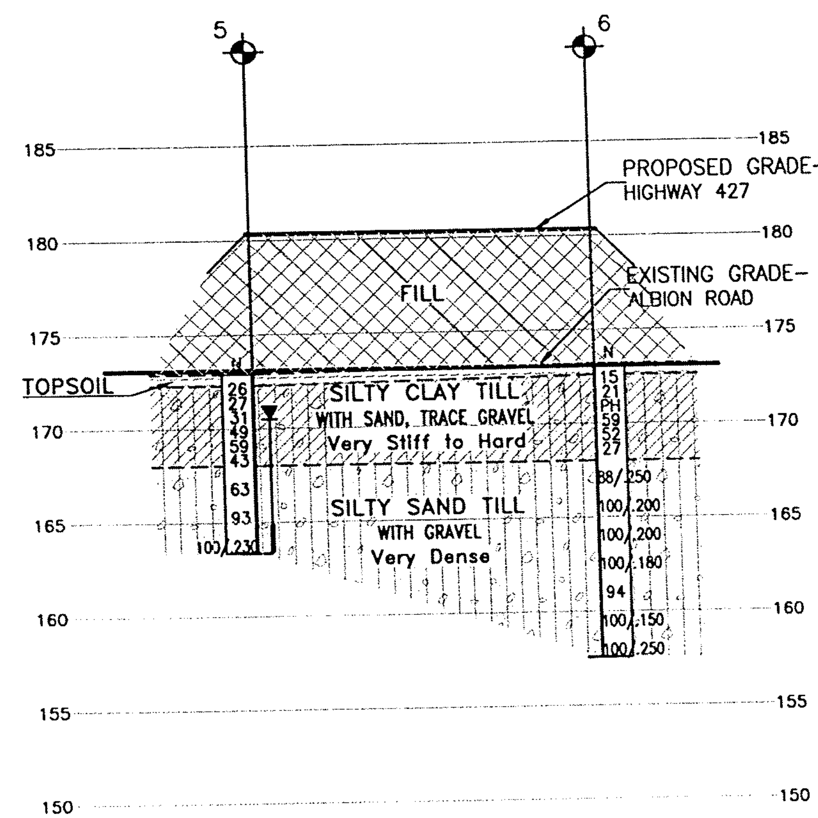
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



SECTION A–A



SECTION B–B



SECTION C–C

LICENSED PROFESSIONAL ENGINEER
S. PANG
Nov 26/09
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Nov 26/09
PROVINCE OF ONTARIO

CONT No
GWP No 202–95–00

HIGHWAY 427
ALBION ROAD OVERPASS
REHABILITATION & WIDENING
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

SNC·LAVALIN

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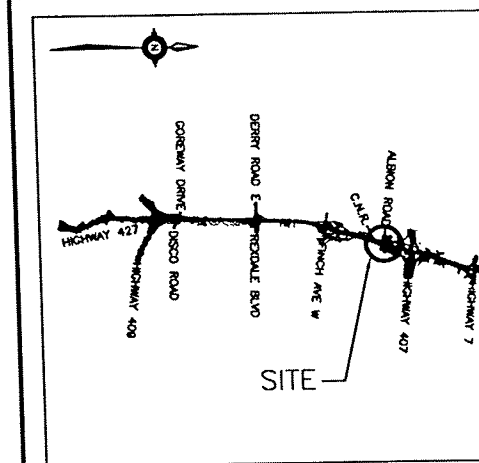

KEYPLAN

LEGEND

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	Borehole by Others (Previous Investigation)
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GEOCRES No. 30M12-290

REVISIONS	DATE	BY	DESCRIPTION

DESIGN SKP	CHK SKP	CODE	LOAD	DATE NOV. 2009
DRAWN MFA	CHK PKC	SITE	STRUCT	DWG 2