

**FOUNDATION INVESTIGATION REPORT  
PROPOSED UNDERPASS  
MISSISSAUGA ROAD OVER HIGHWAY 401  
MISSISSAUGA, ONTARIO  
SITE NO. 24-125**

**Prepared For:**

**GIFFELS ASSOCIATES LIMITED**

**Prepared by:**

**SHAHEEN & PEAKER LIMITED**

**Project: SPT1023  
October 10, 2001**

**250 Galaxy Boulevard  
Etobicoke, Ontario  
M9W 5R8  
Tel: (416) 213-1255  
Fax: (416) 213-1260**

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SITE NO. 24-125**

**1. INTRODUCTION**

Shaheen & Peaker Limited (S&P) was retained by Giffels Associates Limited (Giffels) to carry out a geotechnical investigation for a proposed underpass which will replace the existing structure that carries Mississauga Road over Highway No. 401. The site is located in the City of Mississauga, Regional Municipality of Peel, west of the Highway 401 / Highway 10 (Hurontario Street) interchange.

The proposed underpass will be part of the Mississauga Road Widening project which will begin at Highway 401 and extend northerly to south of Hwy 407.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of sampled boreholes. The findings of the investigation are presented in this report.

**2. PHYSIOGRAPHY AND SITE DESCRIPTION**

The site is located in the physiographic region known as the Peel Plain which is generally level or gently undulating and in which heavy-textured clayey soils comprise the upper zones of the overburden. The underlying material is a glacial till which contains shale and limestone fragments and can be characterized as a shaley till underlain by the bedrock. The bedrock is known to belong to the Queenston formation, which is generally reddish shale interbedded with limestone and siltstone layers. The Queenston shale was deposited in the Upper Ordovician Age (i.e. approximately 450 million years ago).

The site is a landscaped interchange with heavy traffic on both Highway 401 and on Mississauga Road. All access roads of the interchange are paved and the unused areas are grassed with some trees and bushes. South of the interchange the area is developed with commercial and industrial buildings.

### **3. INVESTIGATION PROCEDURES**

The fieldwork for the project was performed during the period between April 23 and May 15, 2001, and consisted of drilling and sampling 14 boreholes (numbered 1 through 10, plus 4A, 5A, 7A and 9A). The locations of the boreholes are shown on Drawing No. 2.

The boreholes were located in the field by S&P personnel, using preliminary drawings prepared by and obtained from Giffels Associates Limited ("Giffels"), Consulting Engineers. On completion of the field work the location coordinates and geodetic ground elevations were determined by Vlado Vujeva, OLS.

The depth of boreholes ranged from 2.4 m to 14.2 m below existing ground surface.

With the exception of Borehole 1 which was accessible by truck-mounted drilling equipment, the boreholes were advanced using a Bombardier-mounted drilling rig. Both were equipped with 150 mm dia. solid stem augers and standard soil testing tools.

Soil samples in the boreholes were taken at frequent intervals (i.e. generally at 0.76 m) of depth, starting at the ground surface. Samples were taken with a 51 mm o.d. split spoon (SS) which was driven into the bottom of the borehole in general accordance with the Standard Penetration Test Method (ASTM D 1586). In this method the driving energy is standardized and is generated by a 63.5 kg hammer freely dropping 0.76 m. The number of blows of the hammer required to drive the sampler into the ground by a vertical distance of 0.30 m is the Standard Penetration Resistance or N-value of the soil which indicates the consistency of cohesive soils (clays and clayey soils), or the compactness condition of cohesionless soils (sands, gravels and non-plastic silts). In some cases auger samples (AS) were obtained by collecting cuttings from the blades of the auger.

Shale bedrock was encountered in 13 of the 14 boreholes. (Borehole 1 was drilled through the approach embankment and did not extend sufficiently deep to the shale bedrock.) The shale could be penetrated by the solid stem auger or was explored by diamond core drilling, using washwater for cooling the drilling equipment

and bringing up the cuttings. Wire-line core barrel was used to obtain NQ size (63.5 mm dia.) core in Boreholes 2, 3, 4A, 5, 5A, 6, 7A, 8 and 9A. The -A series of boreholes (i.e. Boreholes 4A, 5A, 7A and 9A) were drilled adjacent to boreholes bearing the same number either because the recovery of shale samples from the split spoon was generally low and adequate bedrock information could only be obtained by coring (Borehole 4A), or the core recovery rate was very low because the core barrel was blocked by cobbles or fractured limestone (Borehole 5A), or auger refusal was reached probably due to a hard limestone layer (in Boreholes 7A and 9A).

The groundwater conditions were observed in the open boreholes after completion of the borehole. To allow for long-term observation of the groundwater conditions, standpipe piezometers were installed in Boreholes 4, 7A, and 8.

In selected boreholes downhole gas readings were taken using a SCOTT AVIATION Model D 15 gastester to test for the presence of combustible gas concentration. No gas was detected by this method.

The fieldwork was carried out under the full time supervision of experienced geotechnical personnel from S & P who observed the work, directed the sampling, kept records of field tests and described and cared for the samples.

Upon their completion, the boreholes were backfilled to about 8 m below the ground surface with soil brought up by augering (i.e. auger cuttings). The upper 8 m of the open boreholes was then grouted using a cement/bentonite mixture.

The soil profiles, sampling data, N-values and other pertinent information are shown on the Borehole log sheets, Appendix A.

The soil samples and the rock cores were shipped to our laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content, Atterberg Limits tests and grain-size analyses was carried out on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log sheets. The

plasticity charts and the grain size distribution sheets are also presented on separate data sheets in Appendix B. Core photographs are given in Appendix C.

#### **4. SUBSURFACE CONDITIONS**

The subsurface conditions were explored at 10 locations (Boreholes 1 through 10). Boreholes 1, 2, 3, 8, 9 and 10 were located on top or in the side of the approach embankment therefore these encountered road fill materials. Boreholes 4, 5, 6 and 7 were drilled for the two piers, approximately at the level of Highway 401 and, in these holes, the thickness of fill was minor.

The borehole locations and profile are shown on a Plan, see Drawing No. 2. Sections A-A, B-B, C-C and D-D indicating the inferred subsurface soil stratigraphy are given on Drawing No. 2A. The sections summarize the soil profiles presented in detail on the borehole log sheets which were prepared separately for each borehole.

The six boreholes (nos. 1, 2, 3 and 8, 9, 10), which were located within the footprint of the approach embankment, penetrated through 1.8 to 5.5 m deep fill or possible fill which was placed on the existing ground to the underpass level.

The four boreholes (nos. 4, 5, 6 and 7) drilled at the pier locations encountered 0.8 to 1.0 m road construction related fill which was placed when Highway 401 was constructed. The ground had topsoil cover at most locations.

At the northern half of the bridge site, the findings of Boreholes 6 to 10 show the presence of shale bedrock at Elevations ranging between 187.5 and 186.8 m. At Boreholes 8, 9 and 10, the bedrock is overlain by a 0.5 to 2.4 m thick layer of clayey silt till overburden. On the south side, however, the surface of the bedrock was contacted at lower Elevations ranging between 184.3 and 182.8 m and the bedrock is overlain by a relatively thick (i.e. generally about 2 to 4 m thick) fine sand layer. From this it may be surmised that there may have been a watercourse here which ran in an approximately east-west direction at the bridge site. It is possible that this watercourse cut a channel into the overburden which was subsequently filled with alluvial deposits (mostly fine sands). These fine sands,

which are much more pervious than the clayey tills encountered mainly on the north side of the site, continue to convey groundwater.

The details of the subsurface conditions encountered in the boreholes are presented on the Borehole Log sheets in Appendix A. The various soil deposits encountered in the boreholes are described in the following paragraphs.

#### 4.1 TOPSOIL

Topsoil was encountered in Boreholes 2, 3, 5, 6, 7, 8, 9 and 10. The thickness of the topsoil ranged from approximately 50 to 150 mm (average: about 90 mm). The topsoil is clayey, and contains roots and humus.

The remains of topsoil were also encountered at greater depth, below the fill comprising the approach embankment, in Boreholes 1 and 3 (in both holes at 3.4 m depth) and in Borehole 10 (at 2.4 m depth). The topsoil was found in the soil samples, where their thickness was 80, 150 and 450 mm thick, respectively. In Borehole 10 topsoil remains were also encountered at about 3 m depth. The actual thickness of the topsoil could be more (i.e. extend beyond the sampled depth). The topsoil was well compressed by the weight of the approach embankment which was placed several decades ago.

#### 4.2 PAVEMENT AND BASE COURSE

Borehole 1 was drilled through asphalt pavement of about 280 mm thickness. The asphalt is underlain by 300 mm crushed stone base course.

#### 4.3 SURFICIAL SAND FILL

In Boreholes 3, 7 and 10 sand fill and in Borehole 5 sand, identified as possible fill, were found immediately below the topsoil layer. In Boreholes 3 and 7, this granular material consists mainly of fine sand with traces of gravel, while in Borehole 10 it is mixed with gravel size particles. From the recorded values of 13 to 16 blows/0.3 m, the sand fill in Boreholes 3 and 10 appears to have received some compaction, while in Borehole 7, the recorded N-value is 3 blows/0.3 m indicating a very loose condition. In Boreholes 5 and 7 the material was very similar in

composition to the on-site alluvial fine sand material from which it was probably obtained. The thickness of the sand fill ranged from 0.25 to 0.9 m at the borehole locations. A 0.3 m layer of sand fill was also encountered below clayey silt fill in Borehole 6, at 0.5 m depth.

Possible fill, consisting of fine sand, was also encountered between 3.4 m and 5.5 m depth, (i.e. between Elevations 186.7 and 184.6 m) in Borehole 2, below the clayey silt fill (discussed below). Although its gradation (95 % sand and 5 % silt) is very similar to that of the in-situ natural fine sand deposit, the low N-values of 6 and 7 blows may indicate the man-made origin of the fine sand. In the other borings the fine sand deposit was found to be very dense, as will be described later in this report.

These deposits are classified as granular (non-cohesive) soils.

#### 4.4 CLAYEY SILT FILL

At the ground level, below the topsoil, pavement structure or surficial sand fill, clayey silt fill was encountered at all borehole locations except for Boreholes 5 and 7. The origin of this material is local and the clayey silt was probably excavated from nearby borrow areas or construction sites. It was used as the principal construction material for the approach embankments and for regrading the site. The color of the material is brown and reddish brown, mottled, and it has some sand content and traces of gravel. Occasional rootlets and topsoil remains are telltale signs of the man-made origin of the deposit. Its thickness was found to range from 0.4 m (in BH 6) to 3.4 m (in BH 10) at the eight borehole locations where it was encountered and it extended to Elevations ranging from 186.7 m to 189.9 m. The average thickness of the clayey silt fill is 2.2 m.

The grain-size distribution of two samples was determined as follows (see Fig. 1, Appendix B).

Gravel:	1	and	3 %
Sand:	25	and	34%
Silt:	44	and	39%
Clay:	30	and	24%



The clayey silt fill has low plasticity as indicated by two Atterberg Limits test results:

Liquid Limit:	27% (both samples)
Plastic Limit:	18% and 19%
Plasticity Index:	9% and 8%

These results are characteristic of clayey soils of low plasticity, as shown in Figure 2, Appendix B. It is basically a cohesive material.

The measured natural moisture contents of the clayey silt fill samples were between 7 and 22%.

The N-values obtained in the clayey silt fill ranged from 5 to 29 blows indicating that the fill was unevenly compacted and that the consistency ranges from firm to very stiff.

#### 4.5 CLAYEY SILT TILL

Clayey silt till was encountered in Boreholes 1 and 3 (south side), and in Boreholes 8, 9 and 10 (north side), at depths ranging from 1.8 m to 3.8 m (average: 3.2 m). Its average thickness is 1.6 m, ranging from 0.5 m (in BH 8) to 2.4 m (in Borehole 9). The average elevation of the top and bottom of the deposit is 188.9 m (range: 187.3 m to 189.9 m) and 187.3 m (range: 186.7 m to 188.1 m), respectively.

Clayey silt till was not encountered in the "low" boreholes (4, 5, 6 and 7), which were drilled along Highway 401 and in Borehole 2.

The clayey silt till is a cohesive material. It is brownish red in color, and has the following composition from a sample tested (see Figure 3, Appendix B):

Gravel:	8 %
Sand:	40%
Silt:	39%
Clay:	13%

Being of glacial origin, the clayey silt till can be expected to contain random cobbles and boulders.

The following Atterberg Limits were obtained on a tested sample:

Liquid Limit:	27%
Plastic Limit:	18%
Plasticity Index:	9 %

The results indicate clayey soils of low plasticity, as indicated in Figure 4, Appendix B. The natural moisture content of samples recovered from the deposit is about 12%.

The N-values generally range from 31 blows to in excess of 60 blows for 0.3 m penetration, indicating hard consistency due to the heavy overconsolidation by the weight of the ice sheets which covered the area in the geological past. Some high N-values may have been caused by oversize particles (cobbles, for instance). The clayey silt till has favourable engineering properties, such as high shear strength, very low compressibility and very low permeability.

#### 4.6 FINE SAND

A fine sand deposit was encountered in all the boreholes drilled on the south side (i.e. in Boreholes 1 through 5). It was probably deposited by an old watercourse therefore it could be of alluvial origin which is also indicated by the layered structure: thin silt and occasional gravelly layers were encountered in the recovered samples. In Borehole 2, it was identified as being a possible fill material.

At the borehole locations the fine sand deposit was encountered between the average depths of 3.2 m (range: 0.9 to 5.5 m) and 6.8 m (range: 4.0 to 9.7 m), hence its average thickness is 3.7 m (range: 2.9 to 4.4 m). The elevation of the top of the deposit was quite consistent: the average is El. 187.7 m (range: 187.3 to 188.1 m). The bottom average elevation is 184.0 m (range: 182.8 to 184.7 m).

Typical grain size distribution curves of the material are shown on Figure 5, Appendix B. The curves indicate that the deposit consists of approximately

93 to 95% primarily fine sand and 5 to 7% silt. The uniformity of the deposit is indicated by the  $C_u (=D_{60}/D_{10})$  value of 2.4. This deposit is considered a relatively fine-grained granular (i.e. non-cohesive) material.

Generally, the recorded N-values were quite high: the lowest value was 35 blows per 0.3 m and most were well in excess of 50 blows. The highest N-value was 107 for 230 mm penetration. In Borehole 2 the recorded N-values are 6 and 7 but, as was discussed before, the material in this borehole was identified as possible fill.

The deposit is water bearing. Based on the grain size distribution, the material has high permeability and is very erodible. The permeability is probably higher in the horizontal direction due to the layered structure of the deposit, particularly through the occasional coarser layers present.

Above the ground water table the fine sand deposit has low natural water content (3 to 6%) but below the water table, the water content is higher, up to 15 %.

#### 4.7 COBBLES, GRAVEL OR BROKEN LIMESTONE

In Boreholes 2, 4A and 5, a 0.13 to 0.6 m thick layer consisting of broken limestone or cobbles and gravel, was found on top of the bedrock. The presence of fractured material is a frequent occurrence on the top of the bedrock. Although such materials are very dense, they are much more pervious than the underlying bedrock and significant water seepage can be expected through them.

#### 4.8 SHALE BEDROCK

The bedrock is part of the Queenston formation which was deposited in the Upper Ordovician Age. It consists of red or reddish brown shale, with interbeds of more competent siltstone and greyish hard limestone. It is also known to contain occasional thin clay seams. The harder layers/seams are usually less than about 100 mm to 150 mm thick but some layers are much thicker. These are actually lenses and can vary significantly in thickness over short distances. Stress relief features, such as folds and faults are also found in the Queenston Formation. In

these features, the rock is heavily fractured and sheared, and contains layers of shale rubble and clay.

The surface of the bedrock was encountered at depths ranging from 0.8 m to 9.7 m, with its top at the average elevation of 185.2 m. The bedrock surface appears to slope to the south; the bedrock is higher at the north abutment (about El. 187.1 m), and lower at the south abutment (about El. 183.8 m). A (possibly local) low point was encountered at Borehole 4 (El. 182.8 m). The bedrock surface elevations should be considered approximate only as in some cases they were inferred from resistance to augering.

At most locations the shale bedrock could be penetrated by the solid stem augering method. In some cases auger refusal was encountered in the shale bedrock (e.g. in Boreholes 7 and 9, at 2.4 and 8.2 m depth, respectively) which was probably caused by the presence of a limestone layer. In some cases when refusal was encountered higher than the planned depth, the borehole was continued by diamond core drilling. It is of interest to note that, even where the bedrock could be penetrated by augering (e.g. in Borehole 4), the core recovery rate was 96% with an RQD value of 72%. This indicates that the penetrability of the bedrock by the auger does not necessarily indicate poor rock quality.

Where explored by diamond core drilling, the general recovery rates ranged from 82 to 100%. (In Borehole 5 the core barrel was blocked twice hence the recorded low recovery rates in this borehole cannot be considered representative, as indicated by the findings in the adjacent Borehole 5A). Eleven of the 15 RQD (Rock Quality Designation) values ranged from 53 to 90% (indicating "fair to good" quality) with the average at 76% ("good" quality). If the four low RQD values (24 to 38%) were included in the analysis, the average RQD value would be 64%, indicating "fair" rock quality. It is of interest to note that the lower RQD values were generally obtained in the boreholes put down on the south side, where the surface of the bedrock was contacted at relatively lower elevations and where an old watercourse may have existed.

The cores obtained from the shale bedrock indicate that it contains frequent and relatively more competent siltstone and grayish hard limestone layers

and/or lenses. The thickness of the limestone layers/lenses was found to be generally 40 to 100 mm, but some were as thin as 10 mm or as thick as 300 mm. In some of the boreholes (e.g. Boreholes 6 and 7), the presence of limestone and siltstone layers was noted one after another. The siltstone layers/lenses were found to be generally between 25 and 600 mm thick but some were thicker (e.g. Borehole 7).

The shale bedrock can be classified as a cohesive material for engineering analysis.

The geotechnical index properties of the shale bedrock material were determined using combined SS samples. The following grain size distribution results were obtained (see Figure 6, Appendix B).

Gravel:	4 %
Sand:	54%
Silt:	37%
Clay:	5 %

The Atterberg Limits tests indicate material of low plasticity (see Figure 7, Appendix B).

Liquid Limit:	20%
Plastic Limit:	14-15%
Plasticity Index	5-6%

The natural water content of the shale bedrock is about 3%. Occasionally higher values (up to 12%) were obtained on the SS samples but these are not considered representative because they were either due to the presence of minor seepage through the horizontal fractures of the shale or due to seepage from above, from the wet overburden.

The bedrock has mainly subhorizontal bedding partings, very closely to closely spaced, with occasional fractures inclined about 60 degrees to the core axis. Based on visual and tactile examination, along with point load test results on the

rock core samples (which yielded uniaxial compressive strength values of between 35 and 105 MPa) the rock is considered medium strong to strong (25 to 100 MPa).

The properties of the Queenston shale are discussed in detail by J. A. Franklin and J. Gruspier in their monograph: "Evaluation of Shales for Construction Projects – An Ontario Shale Rating System" (published by the Ontario Ministry of Transportation and Communications, 1983). Accordingly, the Queenston shale is predominantly brick-red in colour with green bands. The formation consists of 90% to 100% shale, with occasional limestone layers. The mineralogical composition is as follows: about 60% clay minerals (mostly illite with chlorite and minor expansive clays), 12 to 34% quartz, 0 to 1 % feldspar and dolomite and 2 to 30% calcite.

#### 4.9 GROUNDWATER CONDITIONS

Groundwater was encountered at the site. The observations were either direct, made in the open boreholes after completion of the boring (see the logs of Boreholes 4, 7 and 9), in piezometers installed in the boreholes (see the logs of Boreholes 7A and 8), or indirect, based on the observed condition (i.e. wetness) of the recovered samples (see the logs of Boreholes 1, 2, 3, and 5). In some cored boreholes (see the logs of Boreholes 4A, 5A, 6 and 9A) the water level did not stabilize after completion therefore these observations were disregarded.

All the recorded water levels are shown on the individual borehole log sheets. These show that, at the time of the field work, i.e. in April-May, 2001, the groundwater levels ranged from El. 184.5 m (in Borehole 5) to El. 186.9 m (in the piezometer installed in Borehole 7A). Heavy rainfalls before May 29 may have caused the high water level in Borehole 7A; the water level at the same location was El 185.7 m two weeks before, which fits better in the general pattern of readings. Based on these observations, it is our opinion that the range of water levels at the time of our investigations was between El. 184.9 m and El. 186.3 m.

It should be pointed out that the groundwater level can be expected to fluctuate seasonally and in response to weather events and can rise after prolonged wet periods and at the time of spring thaw.

**Yours truly**

**SHAHEEN & PEAKER LIMITED**



L.S. Rolko, P.Eng.



Zuhtu Ozden, P.Eng.

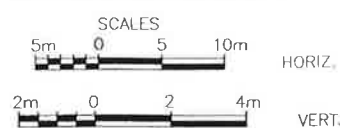


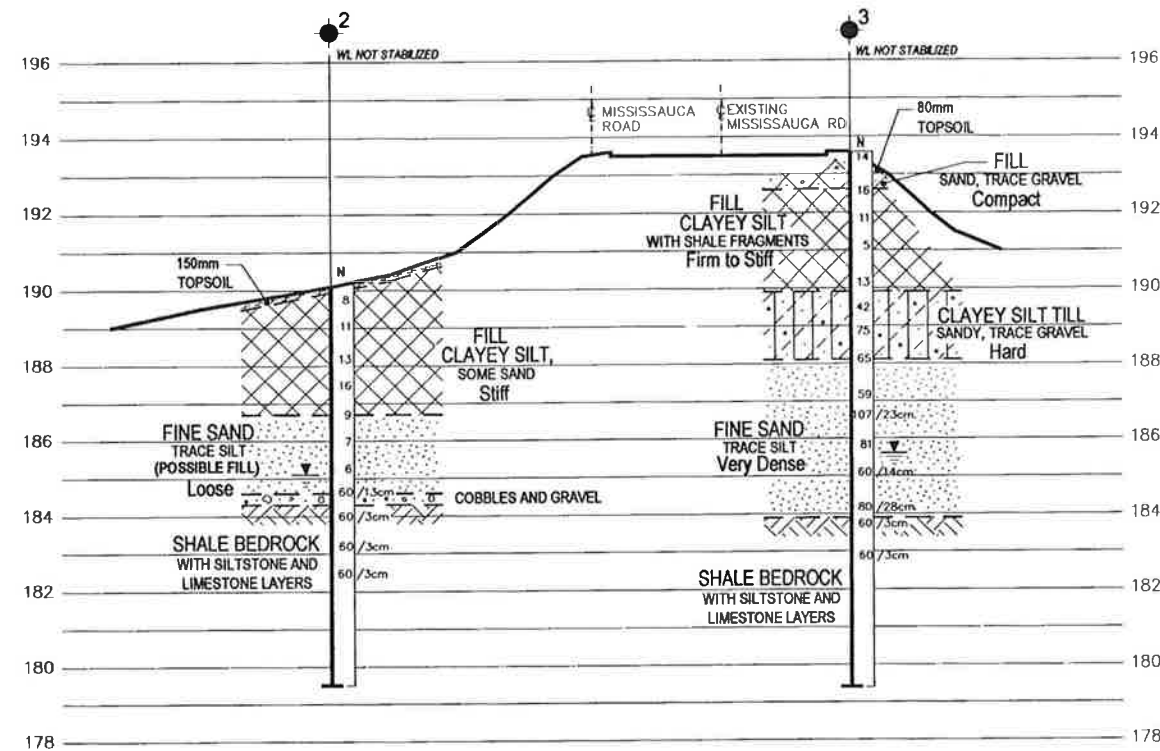
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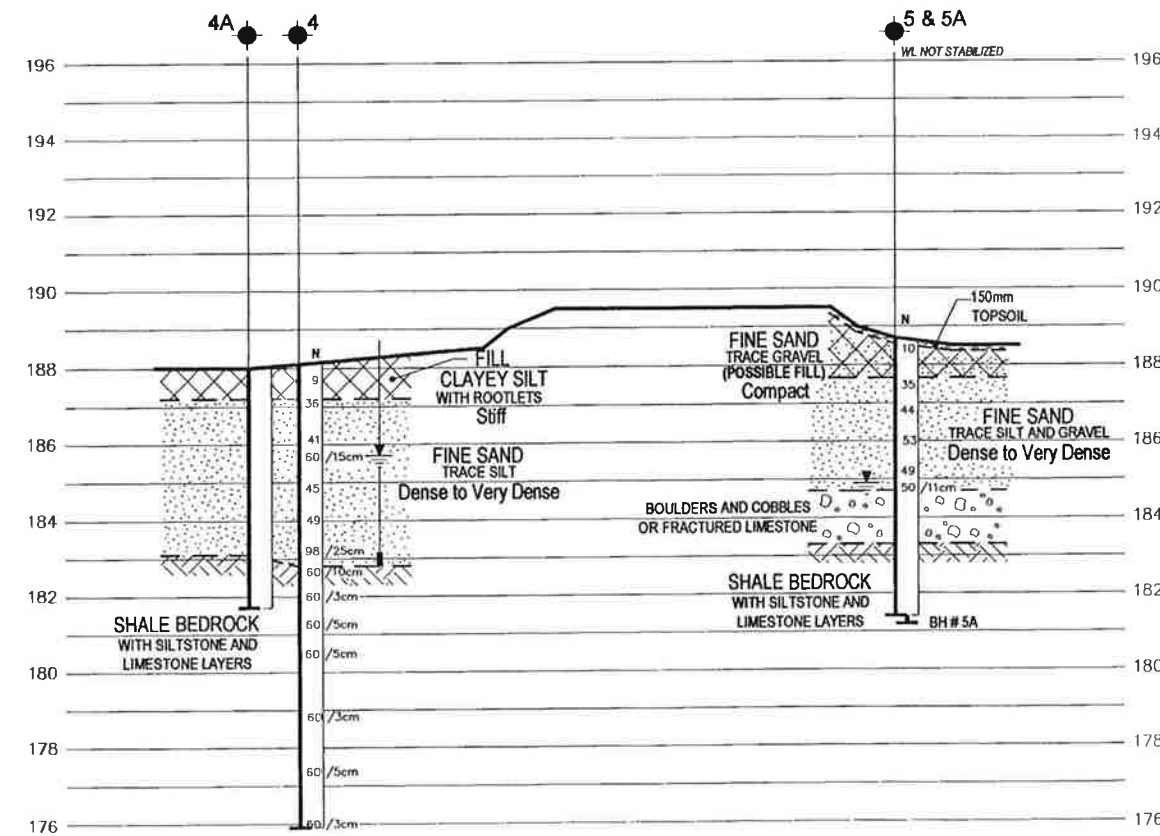
BORE HOLE LOCATIONS &amp; SOIL STRATA

REV.				
	DATE	BY	DESCRIPTION	
Geocres No. 30M12-256				
HWY No 401				DIST C.R.
SUBMD ZO	CHECKED LR	DATE Jul., 2001	SITE 24-125	
DRAWN JTW	CHECKED JP	APPROVED	DWG 2	

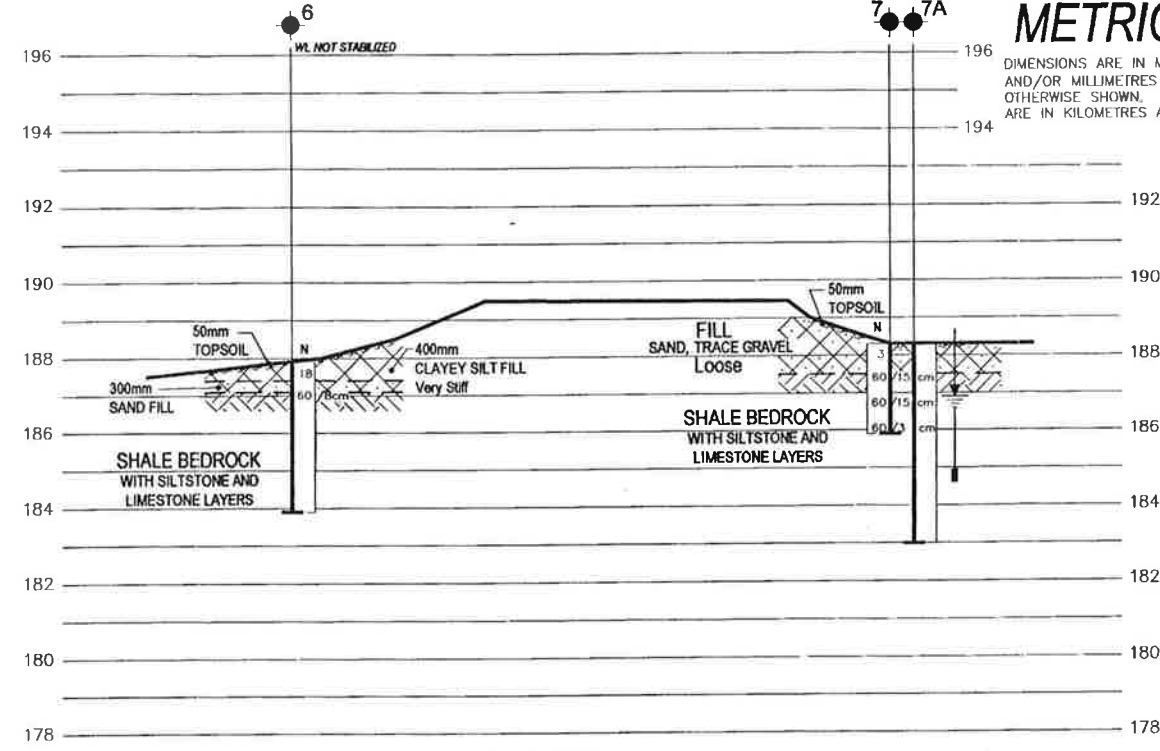




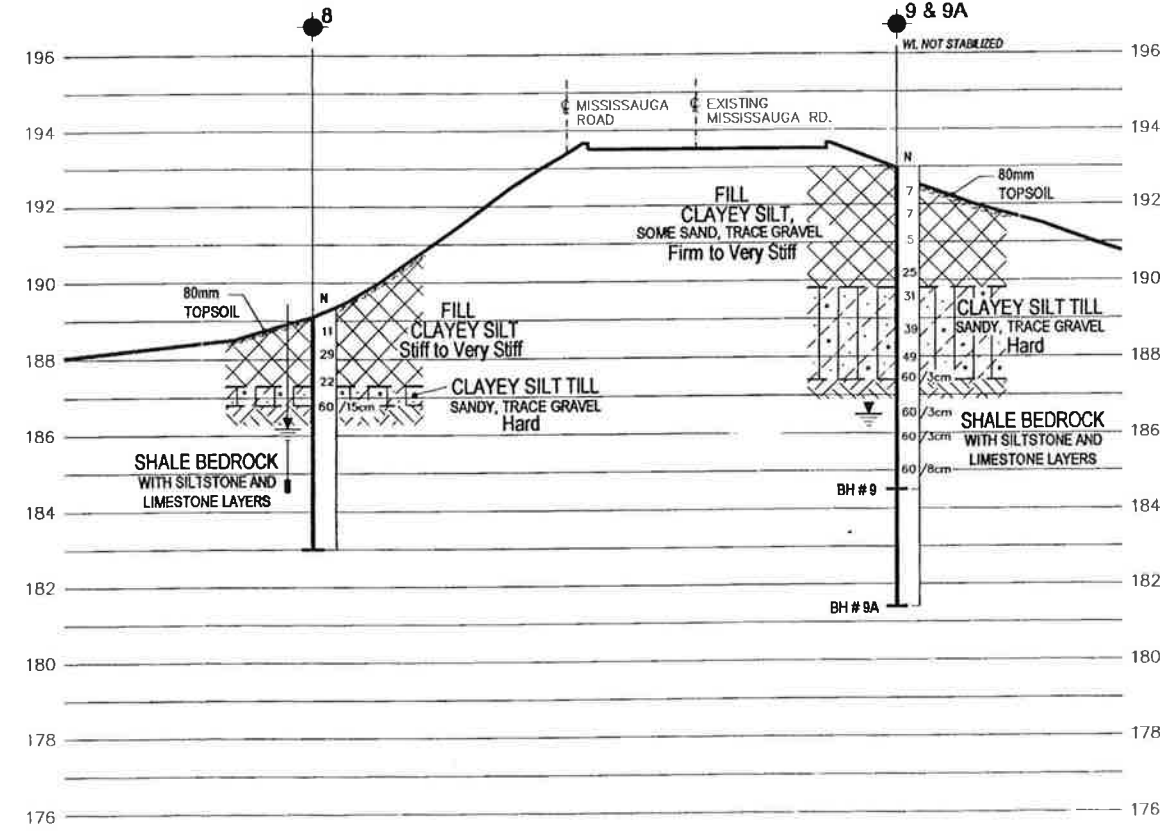
SECTION A - A



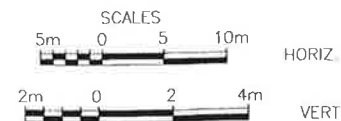
SECTION B - B



SECTION C - C



SECTION D - D



NOTE:

For Plan, Profile and Borehole Locations refer to Drawing 2.

CONT No.  
WP

HWY 401 - MISSISSAUGA ROAD  
UNDERPASS  
SECTIONS A-A, B-B, C-C AND D-D

SHEET

Shaheen & Peaker Limited

FOR KEY PLAN SEE  
DRAWING 2.

KEY PLAN

LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- ▽ W L at time of investigation Apr. and May 2001
- ▽ W L in Piezometer
- ⊕ Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	192.9	4 828 543.6	601 239.4
2	190.1	4 828 550.6	601 201.5
3	193.6	4 828 572.1	601 236.8
4	188.1	4 828 575.3	601 177.1
4A	188.0	4 828 573.1	601 173.9
5	188.7	4 828 600.4	601 217.3
5A	188.7	4 828 600.7	601 218.3
6	187.9	4 828 611.8	601 141.1
7	188.3	4 828 632.6	601 183.5
7A	188.4	4 828 633.4	601 185.2
8	189.1	4 828 633.9	601 116.7
9	192.7	4 828 659.8	601 155.0
9A	193.0	4 828 662.6	601 152.1
10	192.2	4 828 667.2	601 107.0

NOTE

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

DATE	BY	DESCRIPTION
Geocres No.	30M12-256	
HWY No.	401	DIST C.R.
SUBM'D TO	CHECKED LR	DATE Jul., 2001
DRAWN JTW	CHECKED JP	APPROVED
		SITE 24-125
		UWG 2A

# APPENDIX A

## Records of Boreholes

# RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 543.6; E 601 239.4 ORIGINATED BY ZA  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY GT  
 DATUM Geodetic DATE 27.04.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
192.9	Ground Surface													
192.6	280 mm Asphalt Pavement		1	SS	15									
192.3	300 mm Crushed Gravel													
0.6	FILL  Clayey Silt, sandy, firm to very stiff red and brown, some grey, damp to moist		2	SS	20									
			3	SS	18									
			4	SS	18									
			5	SS	6									
189.2	organic topsoil, dark brown													
3.7	CLAYEY SILT TILL Sandy, trace gravel, red, mottled moist, hard		6	SS	45									
			7	SS	60/23									
187.7														
5.2	FINE SAND with silt layers, dense to very dense brown  damp to moist wet		8	SS	48									
			9	SS	52									
184.8														
8.1	End of borehole Borehole open to 7.9 m on completion *Water level not stabilized Ground water probably at 7.5 m from sample moisture condition													

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 550.6; E 601 201.5 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Core COMPILED BY GT  
 DATUM Geodetic DATE 27.04.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
EL.EV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
190.1	Ground Surface													
0.0	150 mm <b>Topsoil</b>		1	SS	8		190							
	<b>FILL</b> Clayey Silt, some sand, trace of organics to ~1.0 m depth, stiff, brown, mottled damp to moist		2	SS	11		189							
			3	SS	13		188							1 25 44 30
			4	SS	16		187							
186.7			5	SS	9		186							
3.4	<b>FINE SAND</b> trace Silt, loose, brown, (possible FILL)		6	SS	7		185							0 95 5 0
		moist --- wet	7	SS	6		184							
184.6			8	SS	60/13		183							
5.5 184.3	<b>COBBLES AND GRAVEL: wet</b>		9	SS	60/13		182							
5.8			10	SS	60/13		181							SS 9, 10, 11 combined
	<b>SHALE BEDROCK</b> reddish, with siltstone and greyish limestone layers		11	SS	60/13		180							
			12	NQ RC	Rec. 98%									RQD=77%
			13	NQ RC	Rec. 98%									RQD=38%
179.5														
10.6	End of borehole *Water level not stabilized Ground water probably at ~ 5.0 m from sample moisture condition													

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 572.1; E 601 236.8 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Core COMPILED BY GT  
 DATUM Geodetic DATE 15.05.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
193.6	Ground Surface						20	40	60	80	100	20	40	60						
0.0	80 mm <b>Topsoil</b> <b>FILL</b> Sand, trace gravel brown, moist, compact		1	SS	14							○								
192.6			2	SS	16															
1.0	<b>FILL</b> Clayey Silt with shale fragments, firm to stiff, brown, damp to moist		3	SS	11															
			4	SS	5							○								
			5	SS	13								○							
189.9	<b>topsoil</b>																			
3.7	<b>CLAYEY SILT TILL</b> Sandy, trace gravel, brown, some reddish, hard, damp to moist		6	SS	42							○								
			7	SS	75							○								
188.1			8	SS	65															
5.5	<b>FINE SAND</b> trace silt, brown, very dense		9	SS	59							○								
			10	SS	107/23															
			11	SS	81							○								
		damp ----- wet	12	SS	60/14															
			13	SS	80/28							○								
183.9			14	SS	60/3															
9.7	<b>SHALE BEDROCK</b> reddish, with siltstone and greyish limestone layers		15	SS	60/3							○								
			16	NQ RC	Rec. 92%															
			17	NQ RC	Rec. 97%															
179.4																				
14.2	End of borehole Borehole cave at 7.4 m on completion																			

+<sup>3</sup> × 3<sup>3</sup> Numbers refer to  
Sensitivity

20  
15 0.5  
10 (%) STRAIN AT FAILURE

\*Water level not  
stabilized  
Ground water  
probably at 8.0 m  
from sample  
moisture condition

RQD=77%

RQD=75%



# RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 575.3; E 601 177.1 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY GT  
 DATUM Geodetic DATE 27.04.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
188.1	Ground Surface							20	40	60	80	100					
0.0	<b>FILL</b> Clayey Silt, with rootlets, stiff, brown, moist		1	SS	9		188										
187.2			2	SS	36		187										
0.9	<b>FINE SAND</b> trace silt with gravelly zone at 2.4 m brown  dense ----- very dense damp ----- wet		3	SS	41		186										
			4	SS	60/15		185										
			5	SS	45		184										
			6	SS	49		183										
			7	SS	98/25		182										
182.8			8	SS	60/10		181										
5.3			9	SS	60/3		180										
			10	SS	60/5		179										
			11	SS	60/5		178										
	<b>SHALE BEDROCK</b> reddish, with inferred siltstone and greyish limestone layers		12	SS	60/3		177										
			13	SS	60/5		176										
175.9			14	SS	60/3		175										
12.2	End of borehole Water level on completion at 2.4 m Piezometer plugged at 2.9 m depth Borehole moved ~3.0 m West and BH4A advanced by augering (without sampling) to 5.0 m and then cored (see BH4A log) Piezometer installed at 5.3 m																

+ 3, X 3; Numbers refer to  
Sensitivity

20  
15 10 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 4A

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 573.1; E 601 173.9 ORIGINATED BY RA  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Core COMPILED BY GT  
 DATUM Geodetic DATE 14.05.01 CHECKED BY LSR

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
						20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>			
188.0 0.0	Ground Surface															
	Augering to 4.9 m without sampling (see BH4)															
183.1 4.9	130 mm limestone cobbles or fractured rock  <b>SHALE BEDROCK</b> reddish, with siltstone and greyish limestone layers		1	NQ RC	Rec. 96%											RQD=72%
181.8 6.2	End of borehole Borehole 4A located 3m west of borehole 4 *Water level not stabilized after coring Water used for coring															



# RECORD OF BOREHOLE No 5

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 600.4; E 601 217.3 ORIGINATED BY RA  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Core COMPILED BY GT  
 DATUM Geodetic DATE 14.05.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								20 40 60 80 100							
								20 40 60 80 100							
188.7	Ground Surface														
0.0	150 mm <b>Topsoil</b> <b>FINE SAND</b> trace gravel, compact, brown, (possible FILL)		1	SS	10										
187.7			2	SS	35										
1.0	<b>FINE SAND</b> trace silt and gravel greyish brown  dense ----- very dense		3	SS	44										
			4	SS	53										
			5	SS	49										
184.7	damp to moist to 3.8 m, wet below		6	SS	50/11										
4.0	BOULDERS AND COBBLES OR FRACTURED LIMESTONE														
183.3			7	NQ RC	Rec. 13%									Core barrel blocked	
5.4	<b>SHALE BEDROCK</b> reddish, with inferred siltstone and greyish limestone layers		8	NQ RC	Rec. 17%									Core barrel blocked	
181.4															
7.3	End of borehole Auger refusal at ~4.0 m depth Borehole moved East 0.5 m East Auger refusal at 4.6 m depth Begin coring *Water level not stabilized Ground water level probably at ~3.8 m from sample moisture condition Borehole moved again ~0.5 m East because of poor recovery BH5A advanced by augering (without sampling) to 4.6 m and then cored (see BH5A log)														

+ <sup>3</sup> × <sup>3</sup> : Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 5A

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 600.7; E 601 218.3 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Core COMPILED BY GT  
 DATUM Geodetic DATE 15.05.01 CHECKED BY LSR

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
188.7	Ground Surface													
0.0														
	Augering to 4.6 m without sampling (see BH5)													
184.1														
4.6	LIMESTONE BOULDERS AND COBBLES		1	NQ	Rec.									RQD=0%
183.3	OR FRACTURED LIMESTONE			RC	50%									
5.4	280 mm fractured limestone at 5.2 m		2	NQ	Rec.									RQD=53%
				RC	93%									
	SHALE BEDROCK reddish, with siltstone and greyish limestone layers		3	NQ	Rec.									RQD=90%
				RC	100%									
181.2														
7.5	End of borehole *Water level not stabilized after coring Water used for coring													

+ 3 x 3 Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 6

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 611.8; E 601 141.1 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Core COMPILED BY GT  
 DATUM Geodetic DATE 24.04.01 CHECKED BY LSR





SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								20 40 60 80 100										
187.9	Ground Surface																	
0.0	50 mm Topsoil		1	SS	18	*												
187.4	400 mm Clayey Silt Fill: brown, very stiff		2	SS	60/8		187											
0.5																		
187.1	300 mm Sand Fill: brown																	
0.8																		
	SHALE BEDROCK reddish, with siltstone and greyish limestone layers		3	NQ RC	Rec. 100%		186									RQD=33%		
							185											
	mostly limestone		4	NQ RC	Rec. 100%		184									RQD=24%		
183.9																		
4.0	End of borehole *Water level not stabilized After coring to 4.0 m depth borehole re-augered from ground surface Auger refusal at ~3.2 m																	

# RECORD OF BOREHOLE No 7

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 632.6; E 601 183.5 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY GT  
 DATUM Geodetic DATE 23.04.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
188.3	Ground Surface						20	40	60	80	100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>		
0.0	50 mm <b>Topsoil</b> <b>Fill</b>		1	SS	3	*	188									
187.5	Sand, trace gravel, <b>brown, loose, damp</b>		2	SS	60/15											
0.8	<b>SHALE BEDROCK</b> reddish, with inferred siltstone and greyish limestone layers		3	SS	60/15		187									
185.9			4	SS	60/3		186									
2.4	End of borehole Refusal to augering at 2.4 m probably on a limestone layer *Borehole dry on completion Borehole moved ~ 3.0 m East and BH7A advanced by augering (without sampling) to 2.4 m and then cored (see BH7A log)															

+<sup>3</sup> ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 7A

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 633.4; E 601 185.2 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Core COMPILED BY GT  
 DATUM Geodetic DATE 23.04.01 & 26.04.01 CHECKED BY LSR













SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
188.4	Ground Surface							20	40	60	80	100					
0.0							188										
187.5	Probable BEDROCK Surface						187										
0.9	Augering to 2.4 without sampling (see borehole BH7)						186										
186.0	75 mm limestone layer at 2.4 m						185										
2.4	<b>SHALE BEDROCK</b> reddish, with siltstone and greyish limestone layers 50 mm clay layer at 3.7 m		1	NQ RC	Rec. 82%		184										RQD=25%
			2	NQ RC	Rec. 100%		183										RQD=90%
183.0																	
5.4	End of borehole Piezometer installed at 3.7 m Water level in piezometer at 2.7 m on May 15/2001 and at 1.5 m on May 29/2001																

# RECORD OF BOREHOLE No 8

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 633.9; E 601 116.7 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Core COMPILED BY GT  
 DATUM Geodetic DATE 24.04.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								20 40 60 80 100							
189.1	Ground Surface														
0.0	80 mm <b>Topsoll</b> <b>Fill</b> Clayey Silt, brown, mottled, stiff to very stiff, moist to wet		1	SS	11		189								
			2	SS	29		188								3 34 39 24
187.3			3	SS	22										8 40 39 13
1.8	<b>CLAYEY SILT TILL</b> sandy, trace gravel, reddish brown, damp, hard		4	SS	60/15		187								
2.3	<b>SHALE BEDROCK</b> reddish, with siltstone and greyish limestone layers														
			5	NQ RC	Rec. 95%		186								RQD=72%
			6	NQ RC	Rec. 100%		185								
							184								RQD=85%
183.0															
6.1	End of borehole Piezometer installed at 4.6 m Water level in piezometer at 3.2 m on May 15/2001 and at 2.9 m on May 29/2001														

+ 3, x 3, Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 9

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 659.8, E 601 155.0 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY GT  
 DATUM Geodetic DATE 23.04.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	W P	W	W L		
192.7 0.0	Ground Surface																
	80 mm Topsoil FILL		1	SS	7		192						o				
	Clayey Silt, some sand, trace gravel, rootlets, brown, mottled firm to very stiff		2	SS	7								o				
	moist		3	SS	5		191						o				
	moist to wet		4	SS	25		190						o				
189.8 2.9	CLAYEY SILT TILL		5	SS	31		189						o				
	sandy, trace gravel, brown, hard damp to moist,		6	SS	39		188						o				
			7	SS	49		187						o				
187.4 5.3	SHALE BEDROCK		8	SS	60/3		186						o				
	reddish, with siltstone and greyish limestone layers		9	SS	60/3		185						o				
			10	SS	60/3								o				
			11	SS	60/8								o				
184.5 8.2	End of borehole Refusal to augering @ 8.2 m *Water level at 6.2 m upon completion not stabilized Borehole moved 2 m West and BH9A advanced by augering (without sampling) to 8.5 m and then cored (see BH9A log)																

+<sup>3</sup> ×<sup>3</sup>: Numbers refer to  
Sensitivity

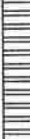
20  
15 5  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 9A

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 662.6; E 601 152.1 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Core COMPILED BY GT  
 DATUM Geodetic DATE 26.04.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
193.0	Ground Surface						193	20	40	60	80	100					
0.0																	
							192										
							191										
							190										
							189										
							188										
187.4	Probable BEDROCK Surface						187										
5.6							186										
	Augering to 8.5 m without sampling (see borehole 9)						185										
184.5							184										
8.5							183										
	<b>SHALE BEDROCK</b> reddish, with siltstone and greyish limestone layers		1	NQ RC	Rec. 100%		182										RQD=60%
			2	NQ RC	Rec. 100%												RQD=85%
181.4																	
11.6	End of borehole BH9A drilled 3.0 m North of BH9 Refusal to augering @ 8.5 m *Water level not stabilized after coring Water used for coring																

+<sup>3</sup> × 3<sup>3</sup> : Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No 10

1 OF 1

METRIC

W.P. \_\_\_\_\_ LOCATION Hwy 401-Mississauga Road Underpass-Co-ords: N 4 828 667.2; E 601107.0 ORIGINATED BY GI  
 DIST Toronto HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY GT  
 DATUM Geodetic DATE 25.04.01 CHECKED BY LSR

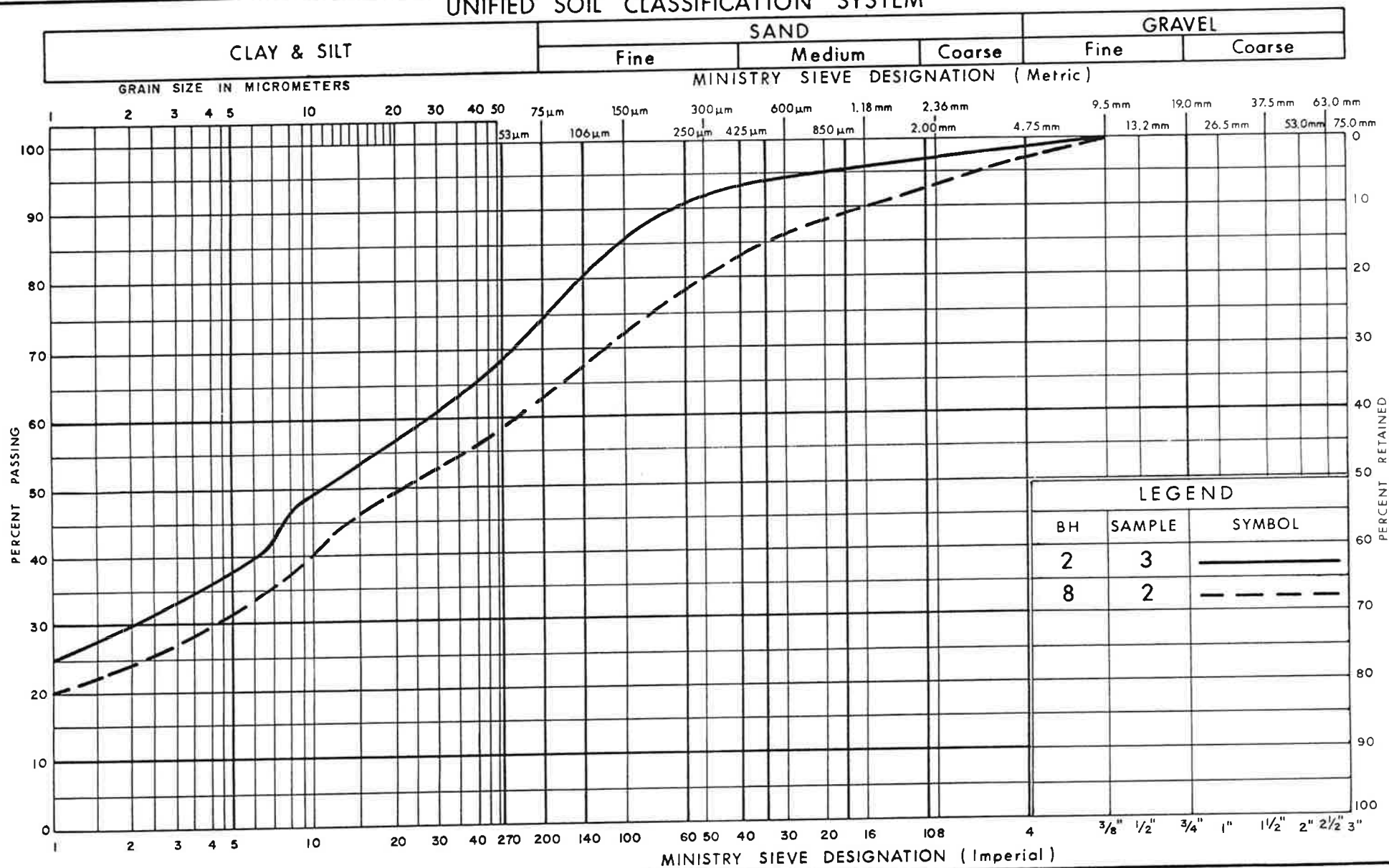
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
FLEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
192.2	Ground Surface							20	40	60	80	100					
0.0	100 mm <b>Topsoil</b>		1	SS	13		192										
191.8	<b>250 mm Sand and Gravel Fill</b>																
0.4	FILL																
	Clayey Silt, some sand, stiff reddish brown, damp to moist		2	SS	14		191										
			3	SS	10		190										
	rootlets and topsoil remains, dark grey in SS4		4	SS	14		189										
			5	SS	16		188										
188.4																	
3.8	<b>CLAYEY SILT TILL</b>		6	SS	47		188										
	hard with shale fragments, reddish, some grey, damp		7	SS	56		187										
186.7																	
5.5	<b>SHALE BEDROCK</b>		8	SS	80/10		186										
	reddish, with siltstone and greyish limestone layers						185										
184.5			9	SS	80/5												
7.7	End of borehole Borehole dry upon completion *Water level not stabilized																

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 (% STRAIN AT FAILURE

# APPENDIX B

## Laboratory Test Results

## UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION  
FILL - CLAYEY SILT, SOME SAND

FIG No 1

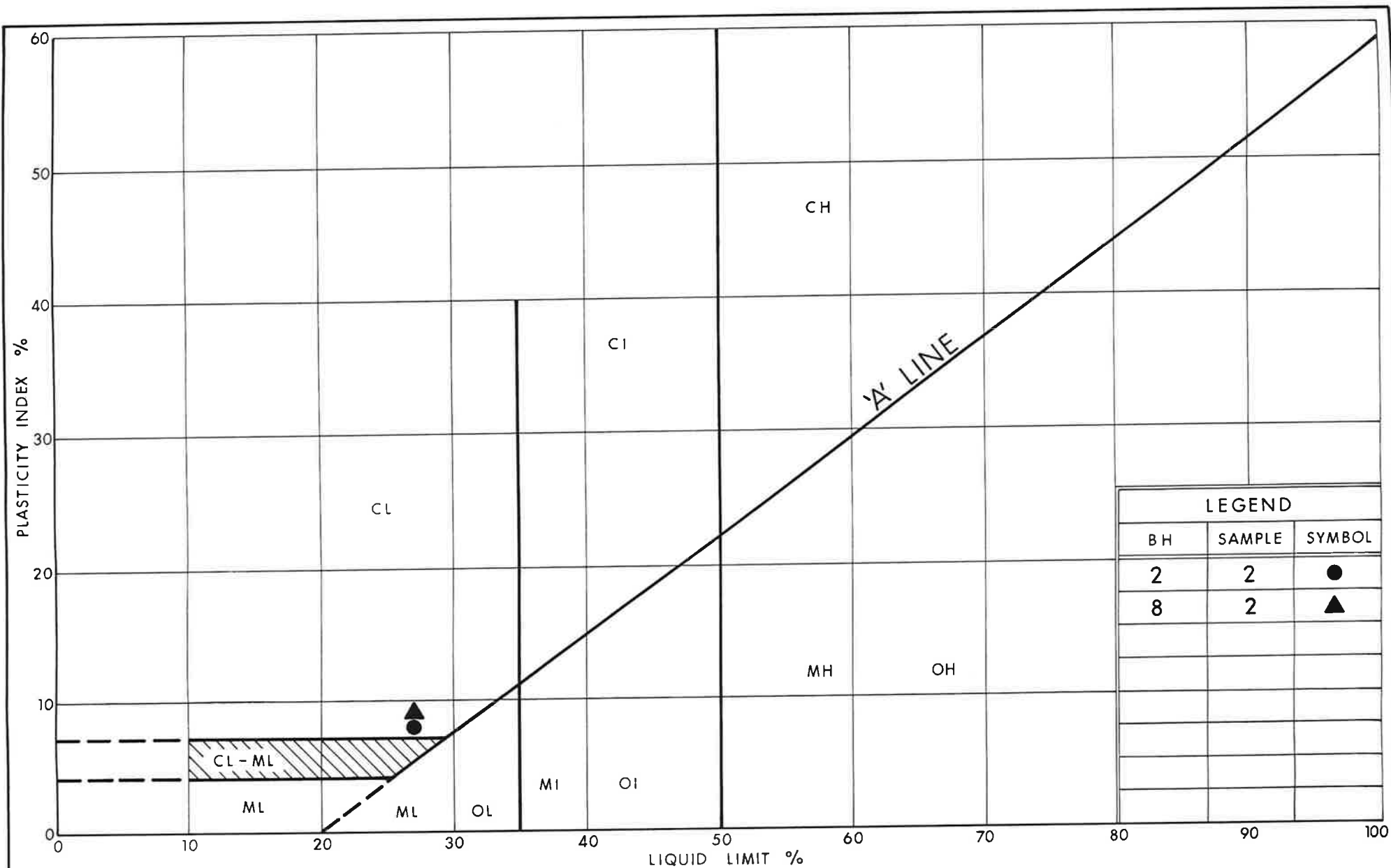
W P

SPT 1023



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Transportation



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Ontario

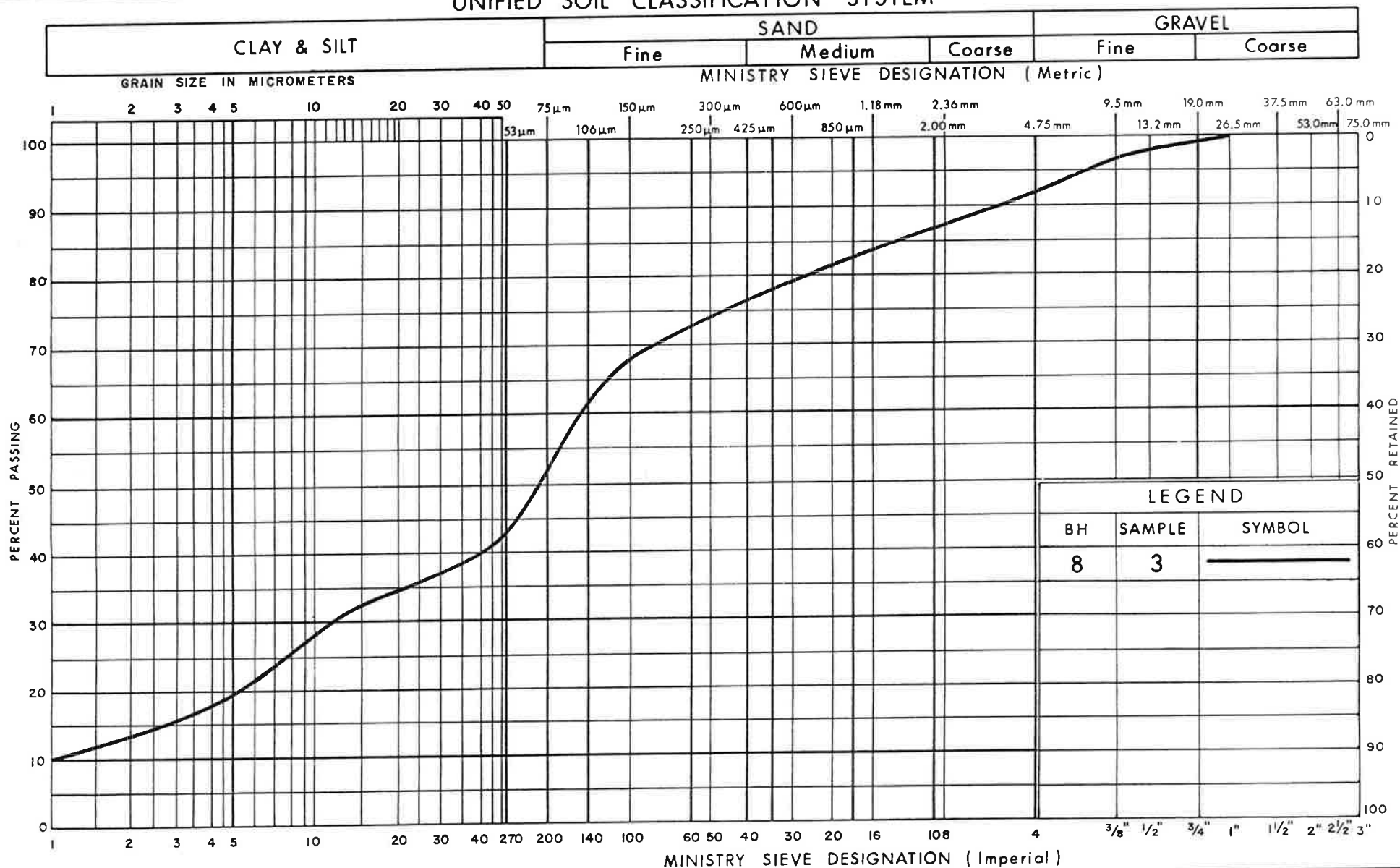
# PLASTICITY CHART FILL-CLAYEY SILT, SOME SAND

FIG No 2

W P

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## UNIFIED SOIL CLASSIFICATION SYSTEM



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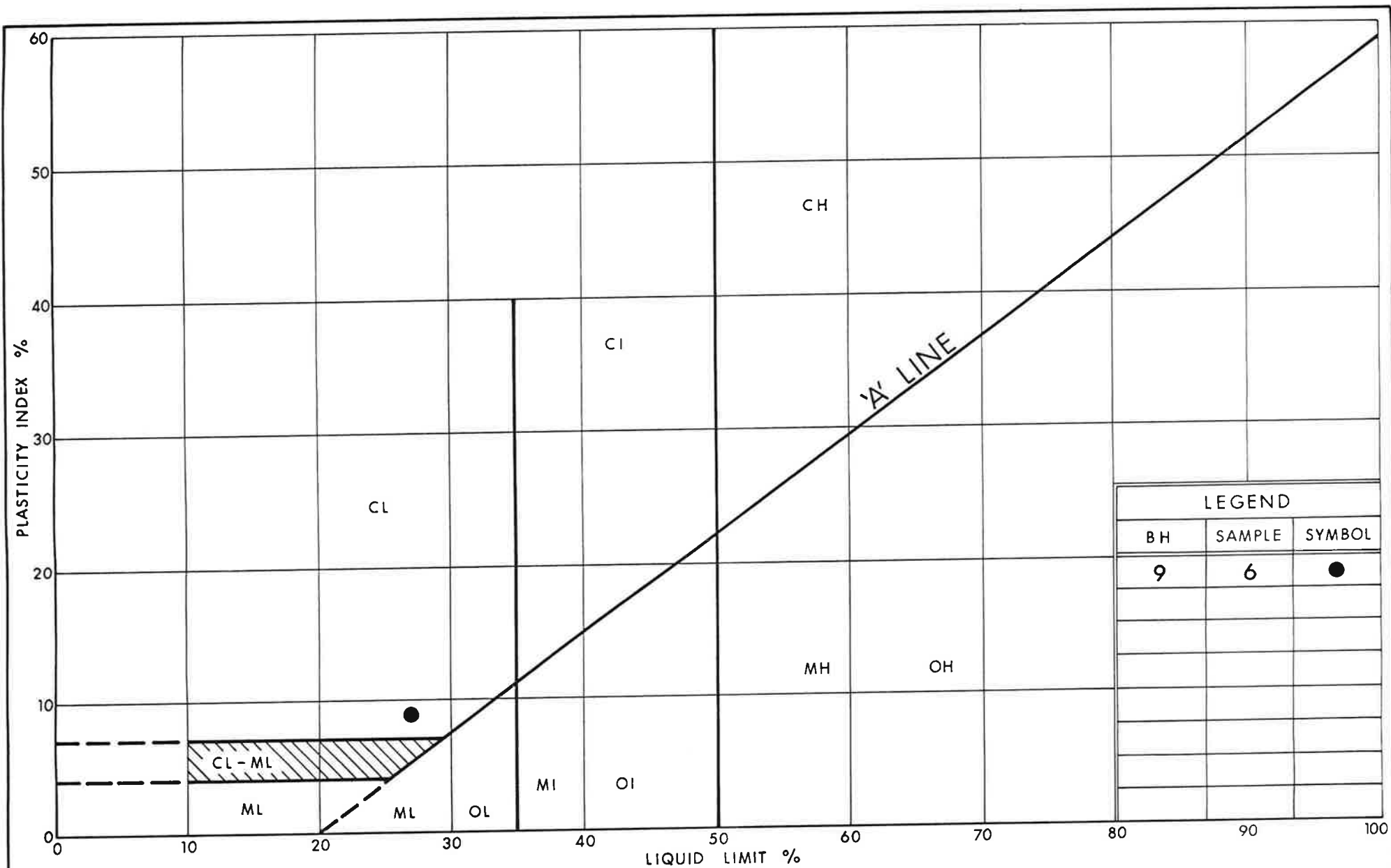
## GRAIN SIZE DISTRIBUTION

### CLAYEY SILT TILL

FIG No 3

W P

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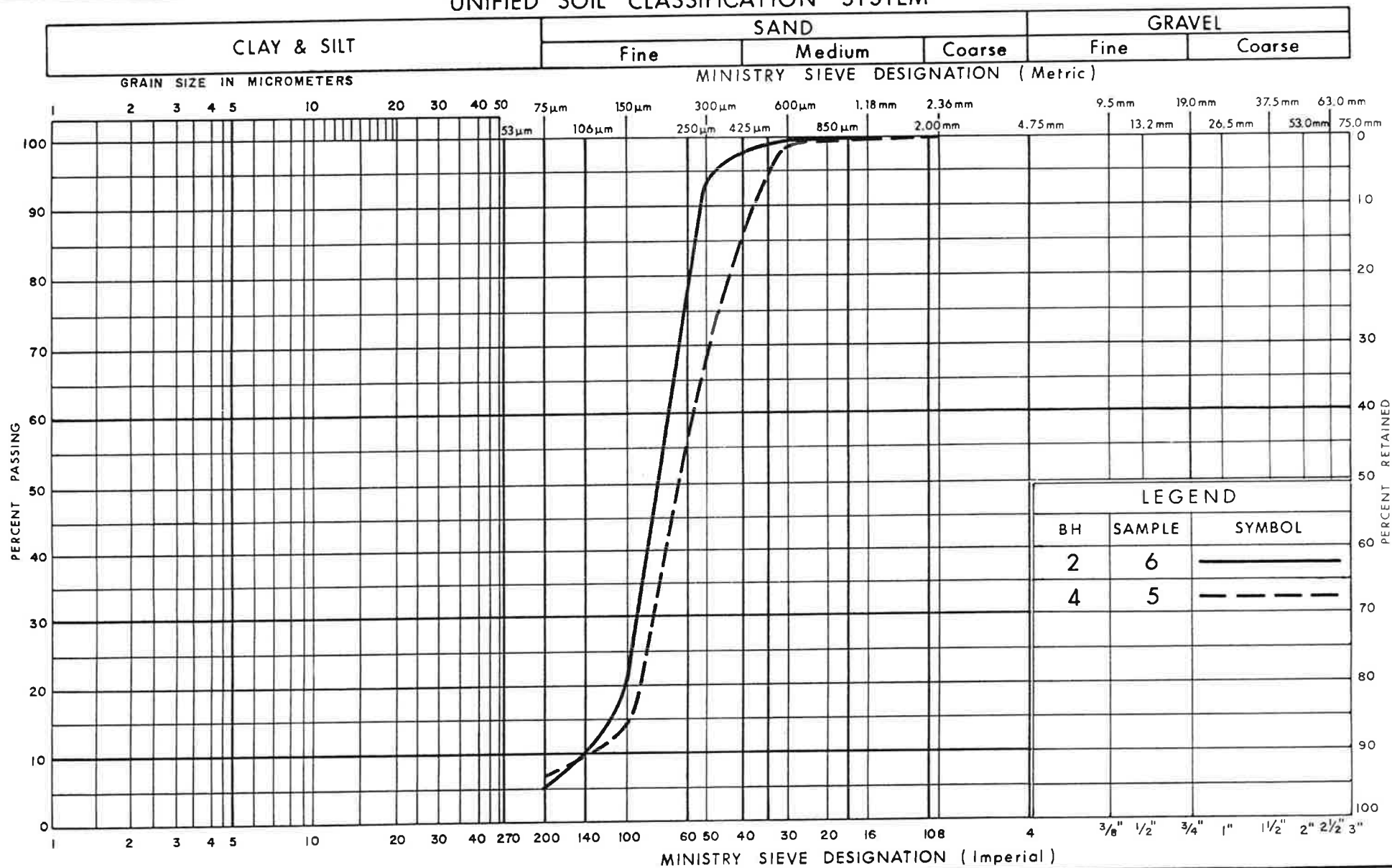
# PLASTICITY CHART CLAYEY SILT TILL

FIG No 4

W P

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## UNIFIED SOIL CLASSIFICATION SYSTEM



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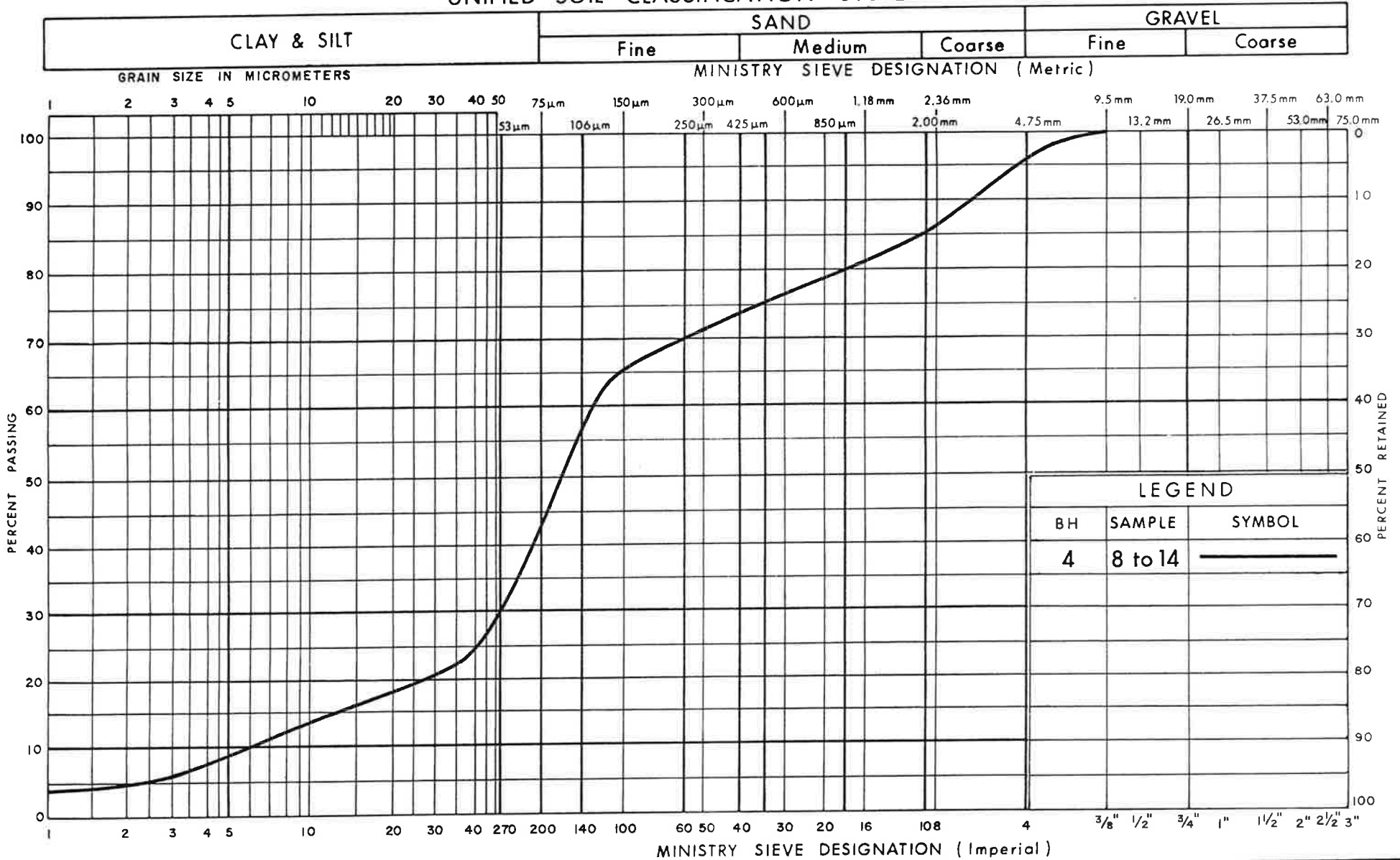
## GRAIN SIZE DISTRIBUTION FINE SAND

FIG No 5

W P

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# UNIFIED SOIL CLASSIFICATION SYSTEM



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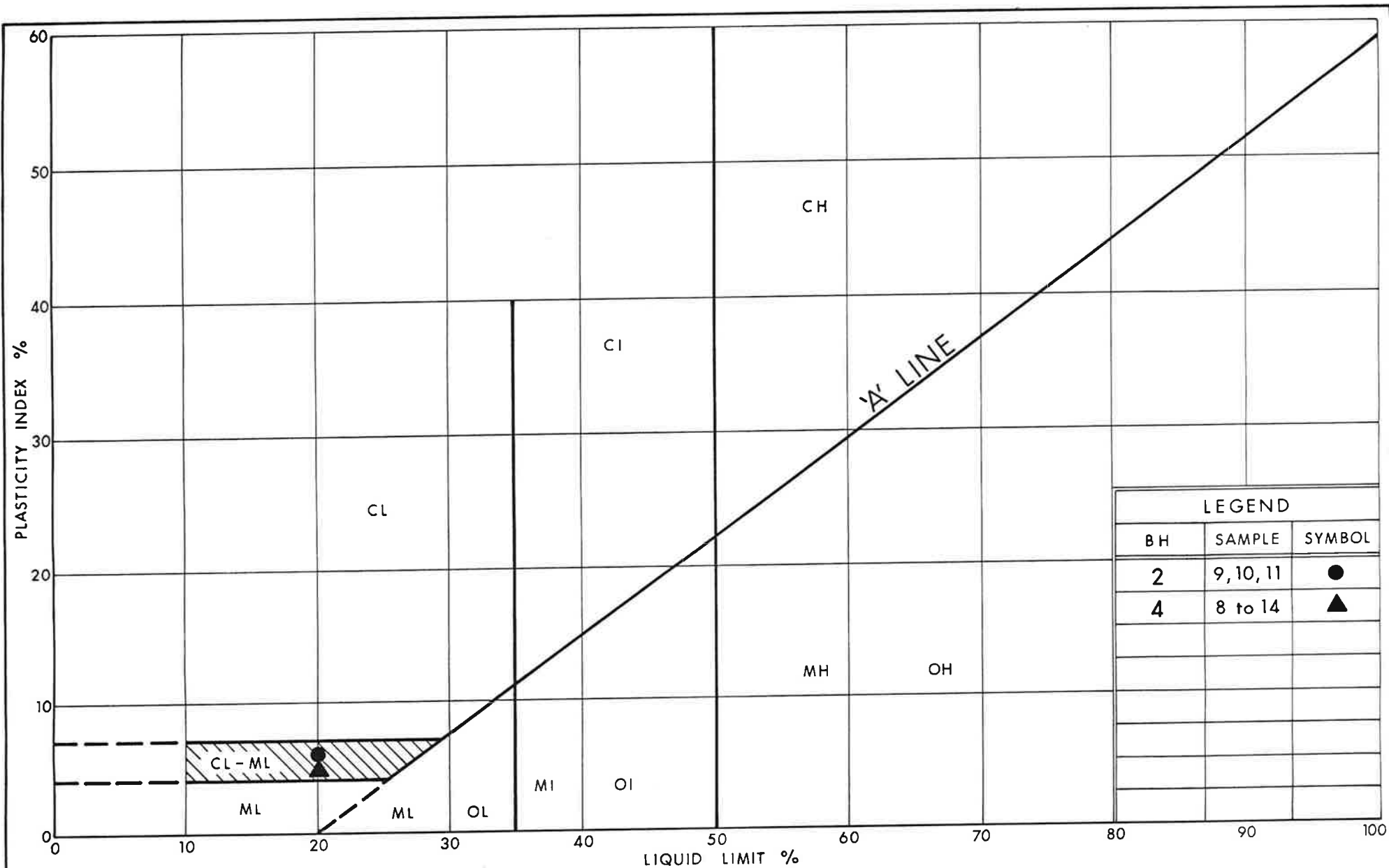
GRAIN SIZE DISTRIBUTION  
SHALE BEDROCK

FIG No 6

W P

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# PLASTICITY CHART SHALE BEDROCK

FIG No 7

W P

SPT 1023

# APPENDIX C

## Core Photographs



PHOTOGRAPHS OF CORE IN BOREHOLES 2 AND 3  
(BH2: ELEVATION 182.4 TO 179.5 M)  
(BH3: ELEVATION 182.5 TO 179.4 M)



PHOTOGRAPHS OF CORE IN BOREHOLES 4A AND 5A  
(BH4A: ELEVATION 183.1 TO 181.8 M)  
(BH5A: ELEVATION 183.3 TO 181.2 M)





**PHOTOGRAPHS OF CORE IN BOREHOLES 6 AND 7A  
(BH6: ELEVATION 186.4 TO 183.9 M)  
(BH7A: ELEVATION 186.0 TO 183.0 M)**



**PHOTOGRAPHS OF CORE IN BOREHOLES 8 AND 9A**  
**(BH8: ELEVATION 186.1 TO 183.0 M)**  
**(BH9A: ELEVATION 184.5 TO 181.4 M)**

## APPENDIX D

# Explanation of Terms Used in Report

## EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

**JOINTING AND BEDDING:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$\gamma_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_i$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kn/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kn/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kn/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kn/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kn/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m <sup>2</sup>	SEEPAGE FORCE
$\gamma'$	kn/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						



**FOUNDATION DESIGN REPORT  
PROPOSED UNDERPASS  
MISSISSAUGA ROAD OVER HIGHWAY 401  
MISSISSAUGA, ONTARIO  
SITE NO. 24-125**

**Prepared For:**

**GIFFELS ASSOCIATES LIMITED**

**Prepared by:**

**SHAHEEN & PEAKER LIMITED**

**Project: SPT1023  
October 10, 2001**

**250 Galaxy Boulevard  
Etobicoke, Ontario  
M9W 5R8  
Tel: (416) 213-1255  
Fax: (416) 213-1260**

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**FOUNDATION DESIGN REPORT  
PROPOSED UNDERPASS –  
MISSISSAUGA ROAD OVER HIGHWAY 401  
MISSISSAUGA, ONTARIO  
SITE NO. 24-125**

**5. DISCUSSION AND RECOMMENDATIONS**

**5.1 CONCEPTUAL DESIGN OF STRUCTURE**

The conceptual plans received on May 22, 2001, from Giffels Associates Limited, Consulting Engineers and Architects, (Project No: 00-4030, General Arrangement and Profiles), indicate that the proposed structure will consist of three spans. The three spans will be 35.5 m – 45.0 m - 35.5 m long, and the principal supporting structural elements will be continuous steel box girders, with reinforced concrete bridge deck. The existing bridge is an approximately 66.6 m long, four-span structure with three piers founded on spread footings and two perched abutments supported by piles. The new bridge will be about 37.5 m wide and the widening will take place towards the west side. The existing 4 to 5 m high approach embankments will be raised by about 2 m, hence the new approach embankments will be about 6 to 7 m high. At the abutments the fill is planned to be retained by the RSS false abutments concept.

The high point of the top of pavement of the underpass structure will be approximately El. 196.3 m and the high point of the pavement of Highway 401 is about El. 188.3 m, i.e. about 6 m below the deck.

According to the conceptual design drawings, the top of pier foundations of the proposed structure is anticipated to be about El. 187.3 m.

Since the traffic will have to be maintained over the existing structure, the underpass will be constructed in two phases: first the existing approach embankment will be widened towards the west and the westerly half of the new bridge will be built west of the existing one. After traffic is diverted to the new structure, the existing bridge will be demolished and replaced by the easterly half of the new structure. Finally, the full width underpass will be opened to traffic.

## 5.2 STRUCTURE FOUNDATIONS

The boreholes show that the subsurface conditions at the north and south sides of the site are quite different. On the north side, a shallow clayey till overburden is underlain by bedrock at relatively high elevation, while on the south side the bedrock is lower and the overburden above the bedrock includes an extensive deposit of water bearing fine sand.

In consideration of the variable soil conditions and different foundation criteria, the foundation conditions will be discussed separately at the abutments and the two piers. Although the conceptual drawings indicate that piled foundations are the preferred alternative at the abutments which are to be designed in accordance with the “integral abutment” concept, other alternative foundation methods will also be discussed.

### 5.2.1 SOUTH ABUTMENT FOUNDATIONS (BOREHOLES 2 AND 3)

#### 5.2.1.1 SPREAD FOOTINGS ON NATURAL DEPOSITS

Borehole 3, contacted competent overburden at about Elevation 189.5 m below which the soil is capable of supporting spread footing foundations. The surface of bedrock was contacted at about Elevation 183.9 m. In Borehole 2, however, below the fill the overburden consists of loose sand (identified as possible fill) and therefore the footing must be extended into the underlying bedrock at or below Elevation 184.1 m, as shown in Table 5.2.1.1 below. Since the placement of part of the footing on bedrock and part on sand is undesirable, the entire footing should be placed on bedrock which will entail extensive excavation. Furthermore, the excavation would have to be extended through wet sand, which would require dewatering. For these reasons, the use of spread footings is considered to be impractical and is not recommended.

Table 5.2.1.1.1

## Recommended Soil and Rock Resistances - Spread Footings for South Abutment

BH No	Existing Grade Elevation BH location (m)	Recommended Footing Base (Bottom) Depth below Existing Ground Surface at Borehole Location (m)	Recommended Founding (Footing Base) Elevation (m)	Factored Bearing Resistance at U.L.S. (kPa) *	Bearing Resistance at S. L. S. (kPa)	Subgrade Material
2	190.1	6.1	184.0 or lower	1500	1500**	Shale bedrock (GW*** at ~5.0 m)
3	193.6	4.1	189.5 or lower	800	400	Clayey silt till underlain by fine sand (GW*** at ~8.0 m)
		9.7	183.9 or lower	1500	1500**	shale bedrock

\*Incorporating a resistance factor of 0.5 as per Ontario Highway Bridge Design Code (OHBDC) 3<sup>rd</sup> Edition.

\*\* Total settlements should be less than 12 mm for foundation placed on properly prepared bedrock surface.

\*\*\*Groundwater level recorded/estimated at the time of investigation.

In the overburden, the serviceability condition is based on the premise that the maximum total settlements will not exceed 25 mm. This can be achieved provided that founding subgrade is undisturbed during the construction.

#### 5.2.1.2 SPREAD FOOTINGS ON ENGINEERED FILL

Spread footings could also be supported on engineered Granular A fill, well compacted in accordance with MTO standard procedures. In our opinion, however, this foundation alternative will result in differential settlements, which could be potentially detrimental to the continuous structure, therefore it is not recommended.

### 5.2.1.3 CAST-IN-PLACE CONCRETE CAISSON FOUNDATIONS

This alternative is feasible for supporting the abutment. The caissons should extend into the bedrock minimum 1.5 caisson diameter, and should be founded on the properly cleaned and competent shale bedrock. The surface of the shale bedrock was encountered at about Elevation 184.3 and 183.9 m at Boreholes 2 and 3, respectively. For such caissons the axial factored bearing resistance at ULS is 7000 kPa and the settlement of the caissons would be small (i.e. not more 12 mm) providing that the bases of caissons are properly inspected, approved and cleaned of any debris and loose rock. To utilize these rock resistances, the minimum penetration into the rock should be 1.5 m and the minimum caisson length below the finished grade should be 6 m. Further comments on the design, installation and inspection of the caissons are given in Sections 5.4 and 5.7 of this report.

### 5.2.1.4 DRIVEN PILE FOUNDATIONS

According to the conceptual drawings, the preferred alternative at the abutment is piled foundations, and the structure would be designed and constructed in accordance with the "integral abutment" concept.

The following axial resistances can be assumed, providing that the piles are driven to practical refusal in the bedrock

Pile Type	= Steel HP 310 x 110
Factored Axial Resistance at ULS	= 1700 kN/pile
Axial Resistance at SLS	= 1200 kN/pile

The shale has some limestone interbeds and lenses, which will render the penetration of the pile into the bedrock rather erratic, unpredictable and in many cases insufficient. Some piles may rest on hard limestone while others would be driven into less competent shale. This can cause unpredictable load distribution among the piles.

Furthermore, in Borehole 2, underlying the embankment fill below Elevation 186.7 m, loose fine sand (suspected to be fill) was contacted overlying a

layer of cobbles, boulders or limestone slab at Elevation 184.6 m followed by bedrock at Elevation 184.3 m. In such conditions, where the overburden is shallow and weak and where a short pile is driven through the weak overburden, it may 'walk' over the hard bedrock surface, rather than penetrating it.

For these reasons, the use of driven pile foundations without pre-augering into the bedrock is considered to be impractical and is not recommended.

In order to provide fixity, the piles will need to penetrate the bedrock a sufficient distance as determined by the structural engineer, and in order to ensure this, pre-augering to a sufficient depth will be required.

Normally, pre-augering in the overburden consists of a 200 to 300 mm diameter hole drilled to a sufficient distance above the anticipated refusal elevation (e.g. 1.5 m). In this instance, however, since the depth of penetration of the pile below the pre-augered level would be unpredictable and the wet sand will cave into the pre-augered hole (i.e. will prevent concreting the hole), the following approach is recommended.

Pre-augering could be carried out with an auger slightly larger than 0.6 m diameter (e.g. 0.76 m diameter) and extend a short distance into the shale bedrock. A steel casing would then be extended into the bedrock to effectively seal water ingress and cave-in from the overburden, especially the wet sand deposit and the cobbles, gravel and/or limestone slabs zone immediately above the bedrock. In order to form an effective seal, screwing (i.e. proper seating) the casing into bedrock may be necessary. The hole can then be extended into the bedrock by a 0.6 m diameter auger to the desired depth, but not less than 3.0 m into the bedrock. The augered hole can then be properly cleaned with the auger and the steel H-pile can be placed in the augered hole, centered and tapped gently into place with the pile driving equipment. Since the pile will only be tapped into place (i.e. no hard driving) the tip need not be reinforced and if desired a lighter section such as HP 310 x 79 can be used, rather than 310x110. The hole can then be filled with concrete to provide fixity and adhesion for pile capacity. The steel casing can then be slowly withdrawn, as more concrete is placed, similar to a caisson installation. If concrete backfill is not necessary above the bedrock level, the hole can be filled, above the

bedrock surface level, with sand. For such a caisson/pile combination the following axial resistances are recommended.

$$\begin{aligned}\text{Factored Axial Resistance at U.L.S.} &= 2000 \text{ kN per caisson/pile} \\ \text{Axial Resistance at S.L.S.} &= 2000 \text{ kN per caisson/pile}\end{aligned}$$

S.L.S. is based on a settlement not exceeding 20 mm. To utilize these values, as mentioned before, the minimum penetration of concreted section into the bedrock should be 3.0 m and the minimum diameter of the concreted hole should be 0.6 m.

The minimum spacing between the caisson/pile holes should be 1.5 m center to center.

If a different hole size than 0.6 m diameter (or penetrating into the bedrock a greater depth) is to be utilized, then the design can be based on a similar adhesion value (i.e. 530 kPa) around the caisson hole circumference, but the adhesion from the top 1.0 m of the bedrock should be ignored. We will be pleased to discuss this further if you require it.

## 5.2.2 PIER FOUNDATIONS (BOREHOLES 4, 5, 6 AND 7)

### 5.2.2.1 SPREAD FOOTINGS ON NATURAL DEPOSITS

Both piers can be supported by spread footings placed at shallow depths below existing ground level. On the conceptual drawings the top of footings at both the south pier and north pier locations is indicated to be at El. 187.3 m and, therefore, the bottom (i.e. founding elevation) of the footings will probably be at about Elevation 186.3 m.

It can be seen from the findings of the four pertinent boreholes that the subsurface conditions are variable at the southerly and northerly piers, therefore the recommended founding elevations will be shown separately for each borehole. Table 5.2.2.1 indicates the recommended founding elevations, founding materials and bearing resistances.



Table 5.2.2.1.1

## Recommended Soil and Rock Resistance – Spread Footings for Pier Foundations

BH No	Existing Grade Elevation at Borehole Location (m)	Recommended Footing Base (Bottom) Depth below Existing Ground Surface at Borehole Location (m)	Recommended Founding (Footing Base) Elevation (m)	Factored Bearing Resistance at U.L.S. (kPa) *	Bearing Resistance at S. L. S. (kPa)	Subgrade Material
4 South Pier	188.1	1.8	186.3 or below	800	400	fine sand wet below Elev.185.7 (GW** at 2.4 m) shale bedrock
		5.5	182.6 or below	1500	1500	
5 South Pier	188.7	2.0	186.7 or below	800	400	damp fine sand, wet below Elev.184.3 (GW** at 3.8 m) shale bedrock
		5.4	183.3	1500	1500	
6 North Pier	187.9	1.2	186.7 or lower	1500	1500	shale bedrock
7 North Pier	188.3	1.2	187.1 or lower	1500	1500	shale bedrock (GW @ El. 186.9 m)

\*Incorporating a resistance factor of 0.5 as per Ontario Highway Bridge Design Code (OHBD) 3<sup>rd</sup> Edition.

\*\* GW-Groundwater level recorded/estimated at the time of our investigation.

The serviceability condition is based on the premise that the maximum settlements for foundations placed on undisturbed founding subgrade in the overburden will not exceed 25 mm, while settlements for foundations placed on properly prepared bedrock surface should not exceed 12 mm.

We recommend that all footings should rest on the properly prepared bedrock surface to minimize differential settlements which could be potentially detrimental to the continuous design structure.

At the south pier location, the excavations will have to extend through water bearing sand deposit, which will require dewatering in an area immediately adjacent to Highway 401 traffic. For this reason, the use of spread footing foundations at the south pier location is not recommended. Instead, the use of

caisson foundation socketed into bedrock would be a better solution, as discussed in the next section (Section 5.2.2.2)

#### *5.2.2.2 CAST-IN-PLACE CONCRETE CAISSON FOUNDATIONS*

The use of cast-in-place concrete caisson foundation is feasible at both pier locations. In particular, this would be an attractive alternative at the south pier location (Boreholes 4 and 5) where excavations for spread footings would have to extend through water bearing sand layers to the bedrock surface, requiring dewatering immediately adjacent to Highway 401 traffic. The caissons should extend minimum one and a half caisson diameter and not less than 1.5 m into the bedrock and should be founded on the properly cleaned and competent shale bedrock surface. Shale bedrock was encountered at Elevation 182.8 m and at Elevation 183.3 m at Boreholes 4 and 5, respectively at the south pier location, and near the ground surface (i.e. at Elevation 187.1 and 187.5 m) at Boreholes 6 and 7 at the north pile location. For such caissons the recommended vertical bearing resistance at ULS is 7000 kPa and the settlement of the caissons should not exceed 12 mm provided the bases of the caisson are properly inspected, approved and cleaned of any debris and loose rock. To utilize the recommended rock resistance the minimum caisson length below the finished grade should be 6 m.

For example, a 1.50 m diameter caisson, constructed in accordance with the above recommendations, can be expected to carry a load of about 12400 kN.

Further comments on the design, installation and inspection of caissons are given in Sections 5.4 and 5.7 of this report.

#### *5.2.2.3 DRIVEN PILE FOUNDATIONS*

Because of the presence of bedrock at or near the ground surface the use of driven piles is not recommended.

## 5.2.3 NORTH ABUTMENT (BOREHOLES 8 AND 9)

## 5.2.3.1 SPREAD FOOTINGS ON NATURAL DEPOSITS

The soil conditions are variable at this abutment but to a less extent than at the south abutment. Table 5.2.3.1.1 indicates the recommended founding elevations and resistances. It can be seen that the foundation base levels would be at different elevations and, if this alternative were chosen, we recommend to place the footings in the clean and competent shale to eliminate potentially detrimental differential settlements.

Table 5.2.3.1.1.

## Recommended Soil and Rock Resistances - Spread Footings for North Abutment

BH No	Existing Grade Elevation at Borehole Location (m)	Recommended Footing Base (Bottom) Depth below Existing Ground Surface at Borehole Location (m)	Recommended Founding (Footing Base) Elevation (m)	Factored Bearing Resistance at U.L.S. (kPa) *	Bearing Resistance at S. L. S. (kPa)	Subgrade Material
8	189.1	2.1 3.1	187.0 186.0 or lower	800 1500	400 1500	clayey silt till shale bedrock (GW** at 2.9 m)
9	192.7	4.0 5.5	188.7 187.2 or lower	800 1500	400 1500	clayey silt till shale bedrock (GW** at 6.2 m) not stabilized

\*Incorporating a resistance factor of 0.5 as per Ontario Highway Bridge Design Code (OHBD) 3<sup>rd</sup> Edition.

\*\*Groundwater level recorded/estimated at the time of investigation.

Settlements of footings placed on the undisturbed overburden and properly prepared rock surface, as discussed above, should not exceed 25 mm and 12 mm, respectively.

## 5.2.3.2 SPREAD FOOTING ON ENGINEERED FILL

Spread footings could be supported on well compacted and engineered fill, consisting of Granular A fill. This foundation alternative is, however,

not recommended because of the possibility of differential settlements which could be potentially detrimental to the continuous structure.

#### 5.2.3.3 CAST-IN-PLACE CONCRETE PILE FOUNDATIONS (CAISSONS)

Caisson alternative is feasible at this location and we recommend that the caissons be founded in the properly cleaned and competent shale bedrock, extending into the bedrock minimum 1.5 caisson diameter. Shale was encountered at El. 186.8 m, and 187.4 m in Boreholes 8 and 9, respectively.

For caissons designed and constructed in accordance with the above recommendations, the vertical bearing resistance at ULS is 7000 kPa. To utilize this rock resistance, the minimum penetration into the bedrock should be 1.5 m and the minimum caisson length below the finished grade should be 6 m. The settlement of caissons should be small (i.e. not more than 12 m). Further comments on the design, installation and inspection of the caissons are given in Sections 5.4 and 5.7 of this report.

#### 5.2.2.4 DRIVEN PILE FOUNDATIONS

According to the conceptual drawings, the preferred alternative is piled foundations for the abutment, and the underpass would be designed and constructed in accordance with the "integral abutment" concept.

The following axial resistances can be used for HP310x110 steel H-piles driven to partial refusal on the bedrock.

Factored Axial Resistance at U.L.S. =	1700 kN per caisson/pile
Axial Resistance at S.L.S. =	1200 kN per caisson/pile

As was discussed for the south abutment foundations in Section 5.2.1.4 of this report, the penetration of the driven piles into the bedrock will be erratic. In addition, the surface of the bedrock, in Boreholes 8 and 9 was recorded at Elevations 186.6 and 187.4 m, respectively and, according to conceptual design drawings by Giffels, the bottom of the CSP (flex zone) will be about 189.5 m. Therefore, the length of the piles beneath the flex zone to provide

sufficient fixity will be rather short and unpredictable. For these reasons, we are of the opinion that pre-augering will be required. The details of this approach, which is a caisson/pile combination, were given in Section 5.2.1.4 of this report and will not be repeated here. This is the recommended foundation scheme for both abutments and provides the following geotechnical resistances for a 0.6 m diameter hole, extending 3.0 m into the bedrock.

$$\begin{aligned}\text{Factored Axial Resistance at U.L.S.} &= 2000 \text{ kN per caisson/pile} \\ \text{Axial Resistance at S.L.S.} &= 2000 \text{ kN per caisson/pile}\end{aligned}$$

The design of other than 0.6 m diameter caissons or caissons extending more than 3 m into the bedrock was discussed in Section 5.2.1.4 of this report.

### 5.3 SUMMARY OF RECOMMENDED FOUNDATION SCHEMES

#### 5.3.1 ABUTMENTS

For an integral abutment type structure, the use of driven H-piles is normally required. With the prevailing subsurface conditions, however, the use of driven piles is considered impractical and therefore a pre-augered hole concept is recommended. It is believed that for this project the most practical approach would be to use a pre-augered hole, which would be filled with concrete, after placing the steel H-pile in the hole, in order to provide fixity and sufficient rock resistance primarily through adhesion, as detailed in Sections 5.2.1.4 and 5.2.3.4 of this report.

This approach is suitable for both the north and south abutments.

#### 5.3.2 SOUTH PIER

Because of the presence of water bearing sand layer overlying the bedrock, we recommend that the south abutment be supported on caissons socketed into bedrock to avoid possible extensive dewatering immediately adjacent to Highway 401.

### 5.3.3 NORTH PIER

The north pier can be supported on caissons socketed in the bedrock or alternatively on spread footing foundations on bedrock.

## 5.4 GENERAL COMMENTS ABOUT FOUNDATIONS

**Spread footings.** The serviceability condition for footings constructed on the properly prepared and approved natural soil subgrade is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 18 mm, respectively. The total and differential settlement of footings placed on properly prepared and approved shale bedrock surface should not exceed 12 mm. In order to minimize differential settlements of the continuous structure, we recommend that all foundations be supported on bedrock.

Where the surface of the bedrock is lower than the proposed founding level or where the rock is shattered and/or weathered, the unsuitable materials should be removed and replaced with mass concrete.

Allowance should be made to place a 100 to 150 mm thick concrete mudmat in all footing excavations within about four hours of excavation to minimize disturbance. The shale is prone to weathering and disturbance and therefore, a similar mudmat would be required on founding surfaces placed on shale.

Under inclined loading conditions the Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6-8.4.2 of O.H.B.D.C.

For the evaluation of the sliding resistance of the foundation (O.H.B.D.C. 6-8.4.3), the ultimate angle of friction between the underside of the foundation and the clean shale bedrock surface (or between concrete surfaces) can be taken as 25 degrees. Horizontal shear resistance can be increased, if required, by penetrating the bedrock (i.e. keying-in and utilizing passive rock resistance) and/or shear in grouted dowels and rock anchors. The minimum dowel length below the underside of the footing should be 3 m.

The unfactored horizontal resistance against sliding between concrete and approved till surface can be calculated by assuming a friction angle of

28 degrees. The unfactored horizontal resistance against sliding between concrete and very dense fine sand surface can be calculated assuming a friction angle of 29 degrees.

If there are net uplift forces which are to be resisted by rock anchors, the factored rock/grout bond capacity at U.L.S. in the bedrock can be taken as 500 kPa (incorporating a factor of safety of 2 against ultimate failure condition) and S.L.S. will not govern. The upper 0.5 m of rock should, however, not be included in calculating the resistance and the minimum embedment depth should be into 2.5 m into shale bedrock (below the underside of the footing). The anchors should also be checked for rock wedge pull-out condition assuming a 60 degree apex cone/wedge. Group effects should be considered where appropriate.

**Caisson foundations.** Caissons should be founded in the shale bedrock, on the competent shale bedrock surface which is “dry” (the maximum depth of water at the base of caissons at the time of placing concrete is 50 mm), was cleaned manually, and was evaluated by the geotechnical engineer. The caisson bore should be protected by temporary lining to prevent caving and permit personnel descending into the caisson bore for dewatering, cleaning and base evaluation. The minimum caisson diameter should be 760 mm to facilitate cleaning and inspection.

The settlement of caissons properly constructed is expected not to exceed 12 mm.

The resistance to lateral movement is provided by the passive resistance developed on the face of the caisson. The unfactored passive resistance developed over the width of the caisson can be computed assuming  $2c_u$  at the surface, increasing linearly to  $9c_u$  at a depth of 3 diameter and beyond. This pressure can be converted into a passive resistance by using a bearing width equal to the caisson diameter.

The unfactored  $c_u$  (i.e. undrained shear strength) for the shale can be taken as 1000 kPa within a depth of 1.5 m of the bedrock surface and 1500 kPa below.

Passive resistance developed within 1.2 m of the ground surface (i.e. within the frost zone) should be ignored.

Coefficient of horizontal subgrade reaction (force per volume) can be calculated from the following expression:

$$k_s = 67c_u/d \quad \text{kN/m}^3$$

Where

$k_s$ = coefficient of subgrade reaction	$\text{kN/m}^3$
$c_u$ = undrained shear strength of the material	$\text{kPa}$
$d$ = pile diameter	$\text{m}$

Applicable  $c_u$  values are given in Table 5.4.1 of this report. The expression and values given are applicable to single caisson units only.

**Piled foundations.** As mentioned before, the use of driven piles is not recommended. Nonetheless, the following information is provided for the sake of completeness. Piles driven into the shale bedrock will undergo tough driving therefore piles should be provided with reinforced tips (as per MTO Standard OPSD 331.00). For the same reason (i.e. difficult driving) lightweight piles - e.g. HP 310x79 sections - are not recommended.

All pile driving should be carried out in accordance with SP 903S01. Depending on the design, any pre-augered hole may need to be protected against cave-ins with a temporary casing, which would be withdrawn after the pile is driven to the required depth and, depending on the details of the design and flexibility required, the bore may need to be filled with unshrinkable fill.

The piles should be driven with a suitably heavy hammer capable of delivering a rated energy of at least 55 kJ/blow, but not more than 70 kJ/blow and the driving of the piles should be controlled by a recognized pile driving formula, such as the Hiley formula. The estimated ultimate resistance of the piles using the Hiley formula can be calculated by dividing the recommended axial resistance at ULS by a resistance factor of 0.5, as per current MTO practice. Using this criterion,



the estimated axial resistance of a HP 310x110 section is 3400 kN per pile as per the Hiley formula (i.e. 1700 divided by 0.5 = 3400).

The monitoring of driving applying the Hiley formula should begin when the piles reach the shale bedrock at the bottom of the pre-bored hole. The pile driving should be carried out in accordance with MTO Standard SP 903S01, and the monitoring in accordance with MTO Standards SS 103-10 and SS 103-11.

During the driving process the piles, which have already been driven, should be monitored to determine if pile heaving was caused by the driving of adjacent piles. If heaving occurred, the heaved piles should be redriven to the original tip elevation, or lower, to the original final driving resistance. At least 10 % of the piles (but not less than two piles) should be re-tapped 24 or more hours after original driving, as per OPSS-903S01, to check if relaxation has occurred. If relaxation has occurred, all piles should be re-tapped. It may be necessary to stagger driving of the piles.

In case of the pre-augered caisson/pile foundations to support the abutments, the section of the hole below the bedrock level will be filled with concrete and in this case for the calculation of horizontal resistances the expressions and data given for caissons earlier in this section of the report can be used. For the section above the concreted section, the method of backfilling the hole above the rock surface will play a significant role. The following expression and data are for driven piles without pre-augering or with pre-augering with a small diameter hole (e.g. 200 mm diameter) and are not applicable for pre-augered holes of larger diameter.

In cohesionless soils the coefficient of horizontal subgrade reaction can be estimated from:

$$k_s = n_h z / d$$

Where $k_s$ = coefficient of horizontal subgrade reaction	kN/m <sup>3</sup>
$z$ = depth	m
$d$ = pile width	m
$n_h$ = coefficient related to soil density as given in Table 5.4.1	kN/m <sup>3</sup>

Also presented in the same table are estimated values for angle of internal friction and bulk unit weights.

Where the soil is primarily cohesive, the undrained shear strength of the soil is given.

Table 5.4.1

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m <sup>3</sup> )	Angle of Internal Friction Degrees (Φ)	Recommended $n_h$ Value (MN/m <sup>3</sup> )	Recommended Undrained Shear Strength (kPa)
South Abutment BH2	190.0-186.7	clayey fill	20.0			50
	186.7-184.6	fine sand	18.5	28	1.3	
	184.6-184.3	cobbles & gravel	19.0	33	11.0	
	184.3-182.8	shale	23.0			1000
	below 182.8	shale	23.0			1500
BH3	193.5-192.6	sand fill	20.0	30	6.6	
	192.6-189.9	clayey fill	19.0			40
	189.9-188.1	clayey silt till	22.0			250
	188.1-186.0	fine sand	20.0	32	18.0	
	186.0-183.9	fine sand	20.0	32	11.0	
	183.9-182.4	shale	23.0			1000
	below 182.4	shale	23.0			1500
North Abutment BH8	189.0-187.3	clayey fill	20.0			60
	187.3-186.8	clayey silt till	22.0			250
	186.8-185.3	shale	23.0			1000
	below 185.3	shale	23.0			1500
BH9	192.6-189.8	clayey fill	19.0			40
	189.8-187.4	clayey silt till	22.0			250
	187.4-185.9	shale	23.0			1000
	below 185.9	shale	23.0			1500

In accordance with MTO requirements (MTO Structural Office Standard), piles for **integral abutments** require a 3 m long flex zone. According to current MTO standards, the flex zone should consist of an annular space between two concentric corrugated steel pipes (CSP-s). One CSP (e.g. 600 mm dia) would be placed as a sleeve directly on the H-pile while the other, somewhat larger CSP (typically, 800 mm diameter) would surround the smaller diameter inner CSP. The annular space between the two CSP's is the 3 m long flex zone.

We do not anticipate structural/foundation damage to the existing bridge due to vibrations caused by pile driving for the new structure at the proposed abutment locations. We recommend, however, that in the event pile driving is

contemplated, vibration specialist and/or a pile driving contractor be consulted in this matter. We also recommend that the vibrations should be monitored during pile driving to confirm the theoretical assumptions.

## 5.5 LATERAL EARTH PRESSURES

The lateral earth pressures acting on the abutments and wingwalls will depend on the type and method of placement of the backfill materials and on the subsequent lateral movement of the structure. The lateral earth pressures to be used in the design should be computed in accordance with Section 6-7 of the OHBDC.

Granular backfill should be placed behind the retaining walls to conform to the minimum requirements illustrated in OPSD 3501.00 and OPSD 3504.00. The granular backfill should conform to OPSS Form 1010 for either Granular 'A' or 'B' Type 1. To maintain free draining characteristics in these granular fill materials, the maximum percentage passing the No. 200 sieve (75 µm) should be limited to 5%.

The backfill should be placed in accordance with OPSS 501. A perforated subdrain should be installed behind the base of the walls as shown in OPSD 3504.00 to maintain the granular fill in a drained condition. The subdrain should be directed through a positive outlet into a municipal sewer or highway drainage system.

The lateral earth pressure  $p_h$  may be computed using the equivalent fluid pressures presented in Section 6-7-4 of the OHBDC, or employing the following expression based on unfactored earth pressure distribution.

$$p_h = K(\gamma H + q) \quad \text{kPa}$$

where

K	=	Earth pressure coefficient , use values from table below
$\gamma$	=	Unit Weight of soil = 21.2 kN/m <sup>3</sup> for Granular 'B'
		22.8 kN/m <sup>3</sup> for Granular 'A'

H = Depth below top of wall, m  
 q = Unit surcharge pressure kPa

Table 5.5.1  
 Coefficient of Lateral Earth Pressure  
 Compacted Granular 'A'

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.27$	$K_a=0.34$	$K_a=0.40$
$K_b=0.35$	$K_b=0.44$	$K_b=0.50$
$K_o=0.43$	$K_o=0.56$	$K_o=0.62$
$K^*=0.45$	$K^*=0.60$	$K^*=0.66$

Table 5.5.2  
 Coefficient of Lateral Earth Pressure  
 Compacted Granular 'B'

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.33$	$K_a=0.42$	$K_a=0.54$
$K_b=0.41$	$K_b=0.52$	$K_b=0.64$
$K_o=0.50$	$K_o=0.66$	$K_o=0.76$
$K^*=0.57$	$K^*=0.74$	$K^*=0.86$

NOTE:  $K_a$  is the coefficient of active earth pressure  
 $K_b$  is the backfill earth pressure coefficient for an unrestrained structure including compaction effects

$K_o$  is the coefficient of earth pressure at rest

$K^*$  is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

These values are based on the assumption that the backfill behind the retaining structure is free-draining granular material and adequate drainage is provided. Allowance should be made for traffic loads.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. Structures supported on bedrock can be considered unyielding. If the abutments are restrained and do not allow lateral yielding, then at rest pressures should be used as per Clause C6-7.1 of the OHBDC, 3<sup>rd</sup> Edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients. The use of vibratory compaction equipment behind retaining structures should be restricted in size as per current MTO practice and as specified in OPSS 501.

As an alternative to conventional retaining walls, MTO's Retained Soil System may be used. The following should be included in the Contract Documents:

- identify longitudinal extent in plan of the Retained Soil System
- identify in plan transverse space constraints (top of wall and bottom of wall)
- identify elevation of top of wall and bottom of wall
- include NSSP for Retained Soil Systems in Contract Documents

The Retained Soil System should be of high performance and high appearance.

## 5.6 APPROACH EMBANKMENTS

As mentioned before, it is likely that the west half of the new bridge will be constructed while the existing Mississauga Road bridge is in use. After the west half of the bridge is completed and is open to traffic, the existing bridge will be removed and replaced with the new structure.

On both the south and north sides of the existing bridge, the construction will involve the widening of the existing embankments, and raising the existing approach embankments by a maximum of approximately 2 m. The existing

grade at the toe of the embankment is about El. 189 to 190 m. Since the new grade will be approximately El 196 m, the widening will involve constructing a new approach embankment of about 6 to 7 m height.

Based on the findings in the boreholes, no foundation failures or excessive settlements are anticipated for the proposed 6 to 7 m high embankments, provided that all organic soils, weak or otherwise unsuitable materials are removed as per MTO Standards before placing the new fill.

Assuming that the approach embankments will be constructed from properly compacted, acceptable inorganic earth fill materials, 2 (H) to 1 (V) side slopes should be stable. Proper erosion control measures should be implemented both during the construction and permanently. This should be achievable by seeding or sodding (OPSS 572) of the completed and fine graded earth surfaces immediately after construction of the embankment.

All organic, wet, soft, loose and other unsuitable soils should be removed within an envelope defined by an imaginary slope not steeper than 1:1, beginning at the toe of the proposed embankment as shown on the sketch presented in Appendix F. The average thickness of the unsuitable soils to be stripped can be assumed to be about 0.2 m. After stripping, the exposed subgrade should be inspected, approved and compacted using a suitably heavy compactor, under the supervision of a geotechnical engineer who is familiar with the findings of this report and is appointed by the Contract Administrator.

Provided that all organic and otherwise unsuitable materials are removed and the subgrade is compacted from the surface to a satisfactory degree (as indicated by proofrolling) as detailed above, the settlement of the foundation materials (i.e. not including the settlement of the embankment material under its own weight) should not exceed about 50 mm. It is estimated that about two-thirds of this settlement should be completed during the construction of the abutment and within about one to two months of placing the embankment fill to its full height. The settlement of the embankment fill under its own weight can be expected to be about 40 mm. Such settlements are considered acceptable therefore preloading or surcharging should not be necessary.

No major dewatering problems are anticipated during the construction of the embankment because the highest groundwater level recorded in the boreholes was at 1.5 m depth (in Borehole 7A) at the time of the fieldwork. After rainy periods the water level could rise and puddling could occur in low-lying areas, but any groundwater can be controlled by pumping, if required. To control the surface water, allowance should be made for gravity drainage and pumping from open sumps to remove water from a surface and/or perched water source, should it be needed. In addition, if the construction is carried out during a wet season, the first lifts of the fill may have to consist of free-draining granular soil materials.

The materials used for the construction of the approach embankments should consist of approved, acceptable earth borrow, free of cobbles and boulders, frozen materials, organic soils, etc. Oversize materials (i.e. larger than 75 mm size) should not be used in embankment fills through which piles would be driven. The fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. The degree of compaction within the top 0.5 m of the fill (i.e. the subgrade immediately beneath the granular subbase) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under the supervision of a geotechnical engineer appointed by the contract administrator.

The sides of the existing embankment should be properly benched prior to placing the fill for the widening of the approach embankments, as per MTO procedures and Ontario Provincial Standards (OPSD-208.01).

## 5.7 CONSTRUCTION COMMENTS

The natural soils encountered in some areas at the site consist mainly of clayey silt deposits and shale bedrock which have very low permeability therefore, in these deposits, no major dewatering problems are anticipated. Any water seepage, including due to percolation from precipitation, perched water or seepage from the ground surface, can be collected in temporary sumps by gravity drainage and removed by pumping.

At the **south abutment and south pier location** (i.e. in Boreholes 1 through 5), a water bearing fine sand deposit was encountered. At the time of field work, the groundwater level in the deposit was encountered at about El. 186± m and, if any excavation extends below the ground water level, the fine sand deposit will have to be dewatered prior to excavation and during construction of the foundations. For example, if spread footing foundations resting on bedrock would be the selected alternative, the fine sand deposit would have to be dewatered.

Because the fine sand is highly erodible and appears to extend below the entire southerly approach embankment, direct pumping from the fine sand deposit is not recommended since this could cause the removal of particles from below the approach embankment. One approach would be to use well points. Since, however, it would be very costly to extend the wellpoints into the bedrock, some water seepage would occur at the bedrock surface and that portion of the fine sand immediately overlying the bedrock would not be properly dewatered. It may be possible to collect this water in temporary trenches and remove by pumping from temporary sumps. In this case, the toe of the existing embankment and the shoring of the excavation should be protected with filtered ballast to minimize loss of soil from below the approach embankment and the toe of shoring. Protection by a suitable geotextile could also be considered. A better alternative would be to use properly filtered deep wells extending into the bedrock. In any event, the dewatering scheme should be designed and installed and operated by qualified personnel.

At the **north pier and north abutment location** excessive seepage into temporary excavations is not anticipated and seepage can be handled by gravity drainage and where necessary by pumping from open sumps. Allowance should also be made to handle perched water in the fill materials.

Based on the existing information obtained from Giffels, there is no interference between the new pier footings and the existing abutment footings.

The **interference between the existing and new pier foundations** was examined. We understand that the existing south pier is supported on spread footing with a bottom elevation of about El. 186.4 m. The borings indicate that, at this elevation, the existing footing would be resting on dense sand. We also



understand that the new bridge will be supported on caissons socketed into the bedrock, as per our recommendations. In this instance, we recommend that the caissons be installed with care and good workmanship, and precautions be taken to prevent any loss of (sand) soils, especially below the groundwater table to avoid undermining of the existing footing. This is particularly true during the installation of the easternmost caisson which will be about 2.5 m away from the edge of the existing footing. It may be prudent to monitor the existing footing during and shortly after the installation of the caissons for any signs of settlement.

At the north pier footings the underside of the new footing will be about 0.3 m below the bottom of the existing north pier footing (according to the available information or the existing structure) and the existing pier footing will be removed when constructing the second phase structure (i.e. the east half of the new bridge).

When installing caissons, the temporary steel liners should keep the groundwater out. Since the caissons would extend into the practically impervious shale bedrock, the water seepage into the lined bore should be minimal and it is believed that it can be removed by pumping or with bailing equipment. Care should be exercised, however, when the concrete is poured into the caissons and the liner is withdrawn, to ensure that the top of concrete inside the liner is above the external water level to prevent ingress of ground water into the fresh concrete of the caisson.

The permissible maximum depth of water at the base of caissons is normally 50 mm at the time of placing concrete.

A minimum caisson diameter of 0.76 m will be necessary and temporary steel liners will have to be employed so that the bases of the caisson holes can be cleaned, inspected and approved. It should also be pointed out that the walls of the caissons in the bedrock will have to be cleared of excessive overburden smear and we recommend that an allowance be made for this purpose. The temporary liners can be withdrawn after inspection and approval of the caisson base and walls by the geotechnical engineer and while pouring concrete. We recommend that the concrete be poured without undue delay after the completion of the caisson hole to prevent the deterioration of shale. The presence of hard layers (e.g. limestone layers) will present problems during the installation of the caissons

since these hard layers will significantly increase the time required to auger the caissons and may necessitate the implementation of special pressures, such as coring equipment. Problems due to excessive groundwater seepage are known to occasionally occur in this formation, which will necessitate dewatering measures such as vigorous pumping.

All excavations, shoring and backfilling should be carried out in conformance with the safety regulations of the province, as well as the following specifications.

SP 539S01 – Protection Schemes

SP 902S01 – Excavation and Backfilling to Structures

The surficial granular fill materials above groundwater table and the clayey silt fill can be classified as Type 3 soil above the ground water table. Clayey silt till deposits can be classified as Type 2 soil. The deeper lying fine sand deposit can be characterized as Type 3 and Type 4 material, above and below the ground water level, respectively. In the bedrock excavations should remain temporarily stable with nearly vertical walls for the duration of construction.

The Queenston bedrock formation encountered at the site could be penetrated by normal augering equipment and based on these findings and on experience, digging into the bedrock should be practicable (although not without some difficulty) by heavy excavating equipment, occasionally using pneumatic jackhammer as required. Limestone layers or lenses may be encountered, but these are mostly less than 150 mm thick, or if thicker, they are likely to be discontinuous. We recommend that unit prices should be included in the contract documents to cover extra costs incurred by removing limestone layers which cannot be excavated by normal equipment. Similar conditions are expected to exist when augering for caissons.

Temporary support will likely be necessary when excavating for the foundations of the piers along Highway 401 where uninterrupted traffic has to be maintained. If there is sufficient room, the excavation can be carried out with sloping sides. If the room is insufficient, the walls of the excavation may have to be

shored and braced. Shoring will also be required to support the sides of the existing embankment during the construction of the foundations of the structure, both during the construction of the first half of the structure and especially during the second half. In all instances, the shoring should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structure performance level. In this case, the Performance Level should be 2. Special care and precautions will be required so as not to undermine the foundations of the existing bridge when constructing the foundation of the west half of the bridge. In this case, Performance Level 1 may be necessary depending on the nature and proximity of the excavation to the existing foundations.

The use of caisson foundations for the piers will minimize the excavations adjacent to the existing bridge foundations especially on the south side.

Locally, temporary shoring systems generally consist of support provided by conventional soldier piles and timber lagging. Depending on the depth of excavation, and on the depth of embedment of the soldier piles below the level of excavation, the soldier piles can be designed as cantilever structures, or they can be strutted or supported by rakers, if they are not cantilevered.

Table 5.7.1 indicates the recommended earth pressure coefficients for temporary shoring design.

**Table 5.7.1 - Unfactored Parameters for Temporary Shoring Design**

Soil Type	K <sub>a</sub>	K <sub>o</sub>	K <sub>p</sub>	Y (kN/m <sup>3</sup> )
Granular Fill and Fine Sand	0.30	0.45	3.3	22.0
Clayey Fill	0.33	0.50	3.0	21.0
Clayey Silt Till	0.30	0.50	3.0	22.0
Shale Bedrock	0.15	0.45	4.5	23.0

If raker support is required, the inclination of rakers should be about 1 to 1, and the raker footings should extend into the natural and undisturbed soil or bedrock, minimum 0.6 m below the excavation level. For the design of raker footings, the following resistances are recommended.

**Table 5.7.2 – Geotechnical Resistances for Raker Footings**

<b>Type of Soil and BH Nos.</b>	<b>Factored Bearing Resistance at U.L.S. kPa</b>	<b>Bearing Resistance at S.L.S. kPa</b>
Clayey silt till (BHs 3,8, 9 and 10)	300	150
Fine sand (BH 4 and 5)	250*	150*
Shale bedrock (BH 6 and 7)	600	400

\*raker footing minimum 1 m wide.

If the shoring system is supported by temporary tieback anchors, we recommend that the bond length of the tiebacks should be in the shale bedrock. The U.L.S. value of the bond of 500 kPa can be used in the shale bedrock and S.L.S. will not govern. Special measures will be necessary to prevent cave-ins when installing tiebacks through sand deposits, especially under the groundwater table.

The shoring system should be designed by a professional engineer experienced in this type of work.

The Paleozoic sedimentary rocks of southern Ontario, including the Queenston Formation, exhibit an expansion in the horizontal direction, due to a phenomenon known as residual stress relief, which can result in damage to structures. This aspect should be taken into consideration if any walls are to be placed directly against vertical bedrock face.

Support will likely be necessary to retain the elevation difference between the two stages of the embankment construction. This could consist of conventional cantilevered shoring consisting of soldier piles and lagging which, if desired, can be removed during the construction of the second half of the embankment. This should be constructed in conformance with SP 539 501 – Protection Schemes and Performance Level 2 is believed to be adequate. The earth pressure parameters given in Table 5.7.1 can be used for the design. We understand that consideration is being given to a low performance retained soil system (RSS) to retain the elevation difference between the two stages of the

embankment construction. This is considered acceptable. In this instance the supplier of RSS should be responsible for the design, including backfill materials, reinforcement, internal and external stability.

If temporary falsework is required along Highway 401 during the removal of the existing bridge, Boreholes 6 and 7 show, outside the paved area the presence of about 0.8 m deep fill followed by bedrock. On the south side, Boreholes 4 and 5, show the presence of up to 1 m fill followed by dense sand.

**Vegetation** should be established on all slope faces to protect against surficial erosion as per OPSS 572.

## 5.8 FROST PROTECTION

Design frost protection for the general area is 1.2 m, therefore, a permanent soil cover of 1.2 m or its thermal equivalent is required for frost protection of foundations, including those founded on the shale bedrock.

## 6. CLOSURE

We recommend that once the details of the project are finalized, our recommendations be reviewed for their specific applicability.

The limitations of Report, as quoted, in Appendix F, are an integral part of this report.

### Shaheen & Peaker Limited



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L.S. Rolko, P.Eng.

Handwritten signature of Zuhtu Ozden in blue ink.

Zuhtu Ozden, P.Eng.

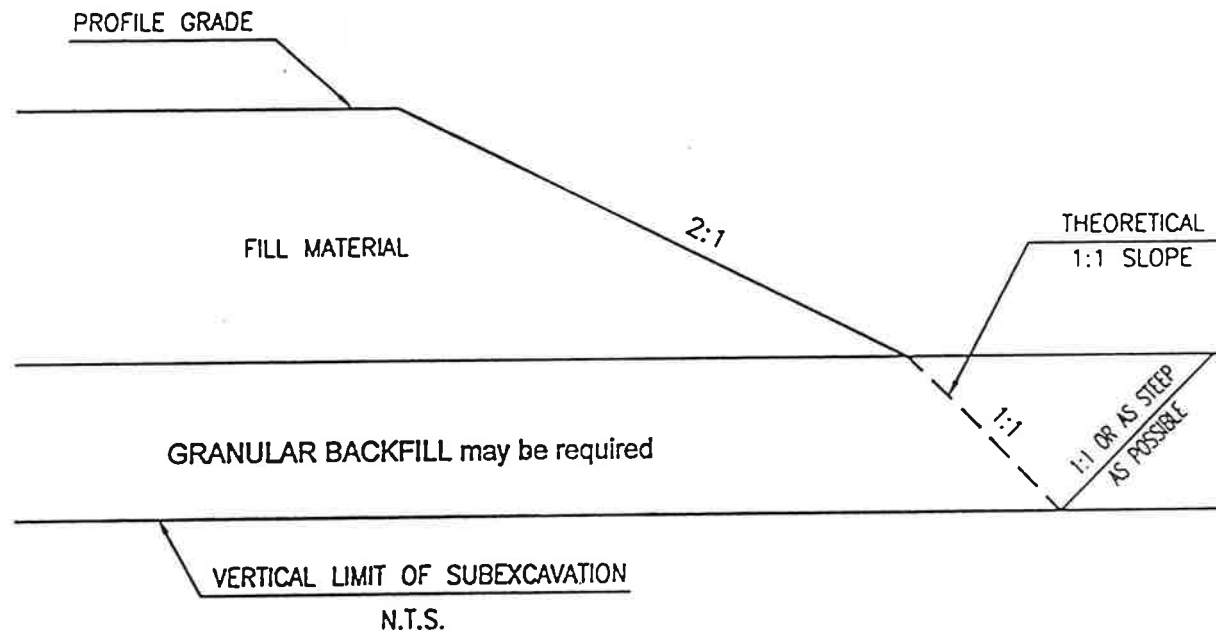


**Foundation Design Report, Proposed Underpass-Mississauga Road over Highway 401,  
Mississauga, Ontario**

**Giffels Associates Limited**

## Appendix E

# Removal of Unsuitable Soils From Beneath Approach Fills



REMOVAL OF UNSUITABLE SOILS  
FROM BENEATH APPROACH FILLS  
N.T.S.

# Appendix F

## Limitations of Report



## **LIMITATIONS OF REPORT**

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.