

**ENGINEERING MATERIALS OFFICE**  
**FOUNDATION DESIGN SECTION**

WP 384-89-01 DIST 4  
HWY QEW STR SITE 18-19

Jordan Harbour (Twenty Mile Creek)  
QEW WB and EB Structures

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# **FOUNDATION INVESTIGATION REPORT**

**FOR**

**Jordan Harbour (Twenty Mile Creek) -**

**QEW WB and EB Structures**

**W.P. 384-89-01, Site 18-19**

**District 4, Burlington**

## **INTRODUCTION**

This report summarizes the results of a foundation investigation conducted at QEW - Twenty Mile Creek crossing. The investigation was carried out at the proposed structure foundation locations and also within the approach areas extending from approximately stations 11+370 to 12+070.

## **SITE DESCRIPTION AND GEOLOGY**

The site is confined between Lake Ontario to the north and the Jordan Harbour to the south in the Town of Lincoln, Regional Municipality of Niagara. The site is situated between Victoria Ave and Jordan Road approximately ten kilometres west of the City of St. Catharines. The Jordan Harbour is present south of the site and its waters flow into Lake Ontario via the Twenty Mile Creek.

The site conditions have undergone significant changes within the time period in which the foundation investigation has taken place. Originally, the shoreline immediately north of the site was populated with tall deciduous trees but as a result of recent construction activity at the site, these trees have been removed. New fill material including shoreline protection materials (armour stone, rip rap) have been recently constructed along the proposed North Service Road/ Twenty Mile Creek crossing. Previous armour stone placed along the Twenty Mile Creek is also evident at the site. In addition, a pronounced breakwater rockfill wall is located within Lake Ontario northeast of the site area. This breakwater structure apparently has been recently constructed by the Beacon Hotel management to safeguard the hotel which is located north of the existing North Service Road just east of the site area.

The two existing QEW-Twenty Mile Creek structures are three span steel structures that carry two lane WBL and EBL respectively. Both structures have undergone considerable deterioration. It is understood that the existing structures are founded on concrete caissons bearing on bedrock.

The land surrounding the site is generally flat with grade increases in both west and east directions. The approach embankment fills to the existing structures reveal their slopes superimposed on the natural ground surface at the site.

The primary industry within the site area is agricultural and the area is reknown for its fruit growing. Orchards and vineyards are present within the generally site area, although not at the specific site location. Soil conditions and the regional climate allow grapes, peaches, cherries and pears to be grown extensively in the area. Motel accommodations and restaurants are also located within the area.

Physiographically, the site is located within the region known as the "Iroquois Plain". The Iroquois Plain is the product of the advance and retreat of the Wisconsinan ice sheet which covered the area during the Pleistocene epoch (over 12,000 years ago). The lowland bordering Lake Ontario, was inundated by the glacial lake called Lake Iroquois when the last glacier was receding at the site. Conditions in the old lake plain vary greatly within the Iroquois Plain. At the site location, the former lake bottom consisted of an organic silty clay to clay deposit which is underlain by undulating till plains and overlain by glaciofluvial sands to silty sands.

The overburden at the site is underlain by shale bedrock of the Queenston Formation at an elevation ranging from approximately 56 m to 67.9 m suggesting a highly variable and irregular bedrock surface. The overburden thickness ranges from approximately 7.8 m to 22.5 m.

## INVESTIGATION PROCEDURE

### General

The subsurface investigation conducted at the site was executed during several different mobilizations spanning time periods between 1965 and 1994. The borehole identification number groups that signify the five separate mobilizations that occurred at the site are summarized in Table 1 below. In each case, the physical and mechanical properties of the soil and/or rock were obtained both by in situ and laboratory testing as discussed below.

Table 1 - Investigation Time Periods			
BH Group #	BH	# of BH's	Time Period
1	1-10 incl.	10	65 11 02-19
2	101-104 incl.	4	92 06 29-92 07 03
3	105-114 incl.	10	93 06 10-29
4	301-302 incl.	4	93 08 12-13
5	501-508 incl.	8	94 01 17-94 02 01

The original field investigation at the site occurred in 1965, when a total of twenty-one boreholes were advanced for the proposed widening of the QEW structures at that time. Ten of these boreholes were advanced in the vicinity of the QEW EB and WB structures and have therefore been included in this report.

Since the original field investigation, four further investigations have been conducted between June 1992 and January 1994. Boreholes within Group #2 and #3 were advanced in an attempt to further define the properties and plan limits of organic materials at the site. Boreholes within Group #4 were advanced to retrieve specific subsoil data within the organic silty clay to clay stratum at the site and boreholes within Group #5 were advanced to retrieve subsurface information at the proposed structure foundation locations. A total of 26 boreholes have been advanced during the more recent investigations.

#### **Field Investigation**

The fieldwork for the original investigation in 1965 was carried out using both a diamond drill unit adapted for soil sampling purposes, and a Pennsylvania continuous flight drill auger which were conventional at the time. The more recent investigations were carried out employing track mounted diesel drilling units equivalent to a CME 55 and equipped with hollow stem augers to advance the boreholes. The fieldwork consisted of the advancement of several sampled boreholes ranging in depth from 9.6 m to 22.9 m.

Samples were retrieved at 0.7, 1.5 and 3 m intervals. Disturbed subsoil samples were retrieved using a standard 50.8 mm O.D. split spoon sampler driven in accordance with the Standard Penetration Test (SPT-ASTM D1586). Relatively undisturbed samples of the

organic silty clay to clay at the site were also retrieved using 57 mm and 73 mm diameter thin wall samples. The thin wall sampler was pushed either manually or hydraulically in accordance with the procedures outlined in ASTM D1587.

All subsoil samples were identified in the field and then properly sealed to preserve natural moisture contents in the soil. Disturbed samples were placed in sealed plastic containers and thin wall samples were capped and waxed. The samples were then transported to the laboratory where additional visual classifications were carried out and pertinent laboratory tests were conducted (see Laboratory Analyses).

In situ vane tests were also carried out to determine the undrained shear strength at selected intervals between the subsoil sample retrieval. The test was carried out in accordance with ASTM D2573 employing the standard MTO 'N' vane. Remoulded shear strengths were also obtained allowing the determination of soil sensitivity. Two boreholes (301B and 302B) were advanced strictly to conduct in situ vane tests.

Rock core was also retrieved at the proposed structure foundation locations using conventional rock coring techniques and a NXL core barrel. The rock core was identified in the field and physical index properties were determined by visual examination and also by measurement of rock quality designations (RQD's) and core recovery. All rock core were placed in standard rock core boxes and carefully transported to the laboratory for detailed rock logging (see Laboratory Analyses).

Groundwater levels were determined by monitoring the water levels in the open boreholes throughout the duration of the field investigation. All boreholes were backfilled upon completion of the fieldwork.

The survey related to the location and elevation of the individual boreholes was provided by Central Region Survey and Plans and MTO Construction staff.

### **Laboratory Analyses**

All subsoil samples were carefully visually examined in the laboratory in accordance with the procedures outlined in the Visual Method described in Chapter 2 of the MTO Soil Classification Manual. The behaviour, gradation and other pertinent physical properties and the mechanical properties of the soil (strength, compressibility) were determined by conducting the appropriate laboratory tests on representative samples. These tests are tabulated in Table 2 below.

Table 2 - Physical/ Mechanical Property Testing	
Physical Property Tests	Mechanical Property Tests
1) Atterberg Limit Tests	1) Consolidation Test
2) Particle Size Analysis	2) Unconfined Compression
3) Natural Moisture Contents	
4) Bulk Unit Weights	
5) Organic Content	



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Sample preparation and laboratory tests were conducted in accordance with the respective procedures outlined in the MTO Laboratory Testing Manual and as described in Chapter 3 of the MTO Soil Classification Manual.

As mentioned earlier, detailed rock core logs were produced by an in-house resident geologist and "Rock Core Descriptions" for all rock core retrieved are contained in the Appendix to this report. The descriptions include rock colour, strength, jointing, bedding and composition.

Laboratory test results on subsoil samples have been summarized below in the subsequent section of this report entitled "Subsurface Conditions", and are illustrated on the corresponding boreholes and figures included in the Appendix of this report. Rock core recoveries and rock quality designations are summarized both in the Rock Core Descriptions and on individual borehole logs.

## **SUBSURFACE CONDITIONS**

### **General**

Within the proposed structure foundation area and adjacent approaches, the subsurface conditions consist of a surficial fill material comprised of an irregular mixture of a silty sand with gravel of thickness ranging from 0.6 m to 4.3 m. The fill material, which also contains random zones of clayey silt is underlain by subsoils consisting of an uppermost deposit of sand to silty sand with traces to some gravel. This deposit is brown to grey in colour

and has a thickness ranging from 3.5 m to 10.7 m but generally is within approximately 5.5 to 8 m. The deposit has a very loose to dense range of denseness but is generally loose to compact.

The cohesionless sand to silty sand with gravel deposit is underlain by an organic silty clay to clay deposit. The organic silty clay to organic clay has a thickness ranging up to 11.8 m with thicknesses increasing in an easterly direction. As shown on Dwg. No. 3848901-B, however, the organic silty clay to organic clay stratum eventually disappears in both easterly and westerly directions (at stations 11+960 and 11+440 respectively). Based on in situ and laboratory testing of the organic silty clay to clay, this material can be categorized as having a firm to very stiff consistency.

The organic silty clay to clay is underlain by glacial till deposits consisting of either a heterogeneous mixture of silt, sand and gravel or a heterogeneous mixture of clayey silt, sand and gravel or at some locations both. These deposits are of a thickness ranging from 1.1 m to 4.9 m. The consistency/ denseness of these deposits is generally hard and very dense respectively.

The glacial till deposits are underlain by bedrock of the Queenston Shale Formation. This bedrock consists of greyish-red shale with interbedded grey siltstone. The bedrock, which is generally a very weak to weak rock, exists at depths ranging from 20.3 m to 7.8 m or equivalently at elevations ranging from 56 m to 67.9 m .

Boreholes 105 to 109 inclusive and 110 to 114 inclusive reveal that the subsurface conditions change at a distance beyond the west and east abutment respectively. At both locations, the thickness of the organic silty clay to organic clay deposit decreases and eventually disappears as mentioned earlier. At the west approach location, the sand to silty sand deposit is overlain by a cohesive clayey silt stratum ranging in thickness from 1.9 m to 3.6 m. The clayey silt is grey and of firm to stiff consistency. It appears, however, that both the clayey silt and sand to silty sand deposit also disappear in a westerly direction. At BH 105, the surficial native deposit is the heterogeneous mixture of clayey silt, sand and gravel.

At the east approach, the sand to silty sand deposit is overlain by a silt deposit that contains random layers of clayey silt. The thickness of this stratum ranges from 5 to 5.7 metres and the clayey silt layers have a thickness up to approximately 0.45 metres. The cohesionless silt can be described as having a very loose to compact state of denseness.

A plan of the site illustrating the locations and elevations of the boreholes is shown on Dwg. Nos. 3848901-A&B in the Appendix. A subsoil stratigraphical profile and stratigraphical sections at the structure foundation locations that illustrate the subsurface conditions at the site are also provided. The boundaries between the various soil types, in situ and laboratory test results as well as groundwater levels established at the

time of investigation are shown on the stratigraphical profile and sections and also on the individual Record of Borehole sheets in the Appendix. A detailed description of the subsurface conditions is given below.

#### **Irregular Mixture of Silt, Sand and Gravel (Fill Material)**

Fill material consisting of silty sand with/ traces of gravel has been placed across the existing QEW corridor at the site in conjunction with the roadway construction. Random zones of clayey silt are also present within the fill material. The material is brown to grey in colour and of thickness ranging from 0.6 m to 4.3 m. The fill material is generally in a compact state of denseness.

#### **Clayey Silt**

A stratum of grey clayey silt with random zones of silt underlies the fill material in the area of BH's 106 to 109 inclusive. The thickness of this stratum ranges from 1.9 m to 3.6 m. In addition, the stratum also occurs surficially for thicknesses upto 11.6 metres at both the western and eastern extremes of the site. Atterberg Limit tests conducted on representative samples of the soil revealed liquid limits ( $W_L\%$ ) ranging from 19% to 31% and plasticity indices ( $I_p\%$ ) ranging from 4% to 16%. The results reveal that the soil has a low plasticity and can be categorized as a clayey silt with random zones of silt.

The 'N' values derived from the SPT conducted in this stratum ranged from as low as 3 blows/0.3 m to as high as 125 blows/0.15 m. Based on these 'N' values, the soil can be described as having a firm to hard consistency.

#### **Silt with random layers of Clayey Silt**

A deposit of silt with random layers of clayey silt exists surficially at the extreme eastern limits of the site. The deposit has a thickness ranging from 5.0 to 5.8 metres with clayey silt layers upto 0.45 metres in thickness. The stratum is brown to grey in colour and based on 'N' values derived from the SPT ranging from 2 blows/0.3 m to 15 blows/0.3 m can be described as having a very loose to compact state of denseness.

#### **Sand to Silty Sand, trace/ some Gravel**

Underlying the fill material, the clayey silt stratum; the silt with random layers of clayey silt where these deposits exist and present surficially elsewhere at the site exists a cohesionless sand to silty sand with traces to some gravel. The thickness of this deposit ranges from 3.5 m to 10.7 m, but generally the deposit has a thickness ranging from 5.5 m to approximately 8 m. The deposit has been oxidized to varying depths ranging from being completely unoxidized and hence completely grey in colour to partially oxidized and brown to depths ranging from approximately 1.5 m to 6 m.

Figure 1 in the Appendix illustrates a grain size distribution envelope of the deposit derived from mechanical sieve analysis of representative samples across the site. As the envelope reveals, the main component of the deposit is the sand that ranges from fine to coarse. The envelope also illustrates traces/ some fine to coarse gravel sizes ranging from approximately 1 to 28% of the deposit. Silt percentages range from approximately 6% (traces) to 50% (silty sand to sandy silt).

This cohesionless deposit is for the most part submerged below the groundwater table and hence during the drilling and sampling process was subjected to conditions of unbalanced hydrostatic head. To prevent soil cave-in and sloughing at the base of the borehole, a constant hydrostatic force was required. This was achieved by supplying a constant head of water using pumps and hoses.

The 'N' values derived from the SPT conducted in this deposit ranged from 2 blows/ 0.3 m to 80 m blows/ 0.3 m indicating a very loose to a very dense state of denseness. However, in general, 'N' values were in the 5 blows/ 0.3 m to 25 blows/ 0.3 m suggesting a loose to compact state of denseness. The larger 'N' values may have been a product of the coarser gravel sizes in the deposits.

### **Organic Silty Clay to Organic Clay**

The surficial native sand to silty sand deposit is underlain by an organic silty clay to organic clay stratum which is present across most of the site. The organic silty clay to organic clay stratum extent was defined between approximate stations 11+960 and 11+440. The surface of this stratum was encountered at an elevation ranging from 66.7 m to 71 m and its thickness ranged from 2.9 m to 11.8 m within the structure location and immediate vicinity. The stratigraphical profile shown on Dwg. No. 3848901-B illustrates the diminishing thickness of the organic silty clay to clay stratum beyond the proposed structure.

A grain size distribution envelope produced by mechanical sieve and hydrometer analysis for this stratum is shown on Figure 2 in the Appendix. The envelope clearly illustrates that the material is fine grained with particle sizes less than 75 micrometres. The clay fraction ranges from approximately 10% to 20% but generally, the clay fraction ranges between 10% and 15%. Silt percentages range from approximately 78% to 90%.

Occasional layers of peat and organic inclusions of partially decomposed timber are also present within the soil matrix of this stratum. The organic content varies between 6.3 and 14.3% by weight. The organic inclusions and soil material are dark grey to blackish grey in colour.



In accordance with the MTO Soil Classification system, a deposit with gradations of this nature is categorized by its behaviour and hence Atterberg Limit Tests were conducted to evaluate the plasticity of the soil. The results of these tests are illustrated on Figure 3 and summarized in Table 3 below. Natural moisture contents and unit weights of the soil are also included in Table 3 below. Natural moisture contents were determined by oven drying whereas Atterberg Limits were determined by air drying the samples prior to testing. Table 3 also includes the test results conducted on representative samples of the organic silty clay to clay retrieved from the NSR/Twenty Mile Creek project (WP 325-89-01) immediately north of the site.

Table 3 - Organic Silty Clay to Clay		
	Range	# of Tests
Plastic Limit ( $w_p\%$ )	32-45	27
Liquid Limit ( $w_L\%$ )	36-71	27
Plasticity Index ( $I_p\%$ )	6-30	27
Liquidity Index ( $I_L\%$ )	0.8-1.4	27
Natural Moisture Content ( $w\%$ )	45-117	37
Unit Weight ( $kN/m^3$ )	13-19.5	14

Table 4 below provides the results of Atterberg Limit Tests in which sample air drying was compared to sample oven drying.

Table 4 - Air Drying vs. Oven Drying						
Sample	Plastic Limit ( $w_p$ %)		Liquid Limit ( $w_L$ %)		Plasticity Index ( $I_p$ %)	
	Air	Oven	Air	Oven	Air	Oven
BH 301A, TW5	50	44	80	65	30	21
BH 302A, TW5	60	52	85	70	25	18

The results tabulated in Table 3 and 4 and illustrated on Figure 3 clearly reveal that the soil exhibits an intermediate to high plasticity and behaves as an organic silty clay to an organic clay. Atterberg Limits were smaller for the oven dried samples than for the air dried samples confirming the presence and influence of the organic material in the soil.

The liquidity index for the soil ranges from 0.8 to 1.4 but in general, the liquidity index exceeds unity. This indicates that the natural moisture content of the soil exceeds the liquid limit. Natural moisture contents range from 45% to 117%.

The unit weight of the soil ranges from  $13 \text{ kN/m}^3$  to  $19.5 \text{ kN/m}^3$ , but generally the unit weights are less than  $16 \text{ kN/m}^3$ .

The undrained shear strength of the organic silty clay to organic clay was determined by in situ vane tests and laboratory unconfined compression tests and the results are tabulated in Table 5. Test results on representative samples of this deposit obtained for the NSR/ Twenty Mile Creek project (WP 325-89-01) have also been included in Table 5.

Table 5 - Undrained Shear Strength ( $c_u$ ) (kPa)		
	Range	# of Tests
Vane Test	70->100	50
Unconfined Compression Test	30-58	14

The results reveal an undrained shear strength ranging from 30 kPa to in excess of 100 kPa. It is believed that the lower undrained shear strength values observed in the unconfined compression test may be a reflection of sample disturbance induced. An undrained shear strength of 60 kPa to 100 kPa is considered as an accurate representative range of the undrained shear strength of this material and consequently the material can be described as having a stiff to very stiff consistency.

The sensitivity of the soil ranged from 2 to 3 indicating a soil of low sensitivity.

SPT 'N' values recorded in this material ranged from 1 blow/ 0.3 m to 14 blows/ 0.3 m. However, in general, there was very little resistance offered by the soil to the split spoon sampler penetration and 'N' values were usually less than 5 blows/ 0.3 m.

The compressibility characteristics of the organic silty clay to organic clay stratum were determined by conducting one dimensional consolidation tests on representative samples of the material. The samples tested were retrieved from boreholes BH 101, BH 103 and BH 301A which were advanced in conjunction with the NSR/ Twenty Mile Creek project but applicable to the QEW/ Twenty Mile Creek structures. Figures 4a and 4b illustrate the results of oedometer tests in which samples were subjected to an external loading with a load increment ratio of one and double drainage. The consolidation curves are plotted on semi-logarithmic paper with the void ratio ( $e$ ) plotted against the applied load ( $\log p$ ). This form of plotting the load-deformation properties of the soil has the advantage of enabling the determination of the preconsolidation pressure ( $p_c$ ) which is defined as the maximum pressure that the soil has experienced in its stress history. Considerable consolidation settlements can occur once the threshold preconsolidation pressure is exceeded.

The consolidation curves reveal preconsolidation pressures ranging from 128 kPa to 157 kPa. The effective overburden pressures of the samples tested ranged from approximately 57 kPa to 67 kPa. It can therefore be concluded that the soil has been

preconsolidated in the past to an effective pressure approximately 60 to 85 kPa in excess of the existing overburden pressure. Compression indices of the material ranged from approximately 0.5 to 1.5.

Attempts were made to compute the coefficient of consolidation ( $c_v$ ) using Taylor's Root time method and oedometer dial gauge readings. However, this method produced irregular and inconsistent results. The results have therefore been considered as unacceptable for application in engineering calculations.

### **Deposits of Glacial Till Origin**

#### **General**

The organic silty clay to clay is underlain by deposits of glacial till origin, namely a heterogeneous mixture of clayey silt, sand and gravel or a cohesionless heterogeneous mixture of silt, sand and gravel. Both deposits are irregular and unstratified and occur randomly across the site, generally overlying the bedrock. Detailed descriptions of these deposits are given below.

#### **Heterogeneous Mixture of Clayey Silt Sand and Gravel (Glacial Till)**

The heterogeneous mixture of clayey silt, sand and gravel has a thickness ranging from 0.2 m to 13.8 m but generally has a thickness ranging from approximately 4 to 7 metres. The colour of the soil varies from brown to grey to red. Figure 5 in the Appendix illustrates a grain size distribution envelope of this deposit revealing that the deposit is

broadly graded containing a wide range of particle sizes. The main component of this unsorted deposit is the clayey silt material. The envelope reveals that the fine grained portions (less than 75 micrometres) contribute up to 85% of the deposit. This material essentially binds the coarser sands and gravels within the deposit. Boulders and cobbles, although not encountered, are characteristic components of glacial till deposits and hence can occur in this deposit.

Atterberg Limit Tests were carried out to define the behaviour and plasticity of the fine grained portion of the soil (less than 425 micrometres) and the results are plotted on Figure 6. A summary of the indices is provided in Table 6 below. Natural Moisture Contents have also been included in the table.

Table 6 - Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till)		
	Range %	# of Tests
Natural Moisture Content (w%)	10-25	6
Liquid Limit ( $w_L$ %)	20-37	6
Plastic Limit ( $w_p$ %)	3-21	6
Plastic Index ( $I_p$ %)	4-16	6

The test results reveal that the fine grained portion of the deposit is predominantly of low plasticity and hence is classified as clayey silt. The test results also reveal that zones of heterogeneous mixture of silt, sand and gravel are also present within the deposit. Natural moisture contents are generally less than or equivalent to the plastic limit of the soil indicating that the soil is in a plastic to semi-solid state.

Standard Penetration Tests (SPT) carried out in this deposit revealed 'N' values ranging from 8 blows/ 0.3 m to 131 blows/ 0.15 and hence the material can be categorized as having a stiff to hard consistency.

#### **Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till)**

The heterogeneous mixture of silt, sand and gravel has a thickness of approximately 1.9 m and generally is red in colour. The deposit is unsorted, unstratified and broadly graded containing a wide range of particle sizes ranging from silt to gravel. Boulders and cobbles although not encountered during the investigation are characteristic components of glacial till deposits and hence can exist in this deposit. This cohesionless deposit is generally underlain by bedrock.

Standard Penetration Tests (SPT) carried out in this deposit revealed 'N' values ranging from 95 blows/ 0.3 m to 100 blows/ 0.05 m indicating a very dense state of denseness.

**Bedrock**

The overburden at the site is underlain by bedrock of the Queenston Shale Formation at an Elevation ranging from 56 m to 67.9 m. The bedrock surface appears to be irregular and varies across the site.

The bedrock consists of a greyish red shale with interbedded greenish grey siltstone. In general, the surficial metre or so has been slightly to moderately weathered. Occasional clay seams of 50 to 100 mm thickness were present within the weathered zone. The weathered rock is underlain by sound, competent and unweathered rock.

Samples of the rock were retrieved by conventional rock coring techniques using an NXL core barrel. Up to 3.5 metres of rock core was retrieved at various borehole locations across the site. Split spoon samples were also retrieved albeit with considerable penetration resistance at some locations where augering techniques were used to penetrate the weathered rock.

Physical and mechanical properties of the rock were determined by physical examination and by core recovery and rock quality designation (RQD) measurement conducted in situ. Detailed rock core designations produced by our resident geologist provide a summary of these properties and are located within the Appendix under the heading "Rock Core Descriptions".



The shale bedrock with interbedded siltstone is very fine grained and contains very thin to thin horizontal bedding. The rock contains moderately close to extremely close spaced fractures that are flat, dipping to near vertical, planar to undulating and smooth. The rock is an extremely friable material with a very low slaking durability. Rock strength as determined by index property examination in the laboratory is generally very weak to weak.

Rock core recoveries in the slightly to moderately weathered rock ranged from 85% to 100% and RQD's ranged from 7% to 77%. In the unweathered rock, recoveries were generally 100% and RQD's ranged from 33% to 77%. The 7% RQD reflects the weathered state of the bedrock. Based on these observations, it can be concluded that the weathered rock is of very poor quality and the unweathered rock ranges from poor to good quality. In general, however, the unweathered rock can be considered to have a fair quality.

### **GROUNDWATER CONDITIONS**

Observation of the groundwater level was carried out by measuring the water level in the open boreholes throughout the duration of the field investigation. Particular attention was given to avoiding non-representative water levels produced by the drilling water.

At the time of the most recent investigation, the groundwater elevation ranged from approximately 73 m to 75 m which for all practical purposes can be assumed to be approximately equal to the lake level.

Groundwater levels, in general, are subject to seasonal fluctuations and hence can vary from the values given in this report.

## **DISCUSSION AND RECOMMENDATIONS**

### **General**

It is proposed to replace the existing QEW WB and EB-Twenty Mile Creek structures with two new three(3) span structures. The proposed WB and EB structures have a similar span arrangement of W 27.5 m - 28.5 m - 27.5 m E. The centre span obviously spans the Twenty Mile Creek. The west and east piers are at stations 11+681.75 and 11+710.25 respectively and the west and east abutments are at stations 11+654.25 and 11+737.75 respectively. The bridge structures will be approximately 17.75 m in width and will accommodate three 3.75 m lanes and shoulders. The structure width is sufficient to enable a future planned widening of the QEW. A horizontal and vertical navigation clearance of 18 m and 2.75 m respectively has been designed.

Encroachment into the Jordan Harbour will be required to facilitate the construction of the EB approach embankment. The proposed profile grade varies from 79.6 m to 80.3 m within and immediately adjacent to the structure. The original ground surface at the site varies from approximately 75.6 m to 79.6 m. The elevation of the channel bed and the Jordan Harbour bed is approximately 74 m. Therefore fill heights upto approximately 4 to 5 metres will be required at the approaches to the structures, for most of the area, but fills upto approximately 6.5 metres will be required for the widening adjacent to the Jordan Harbour.

A plan illustrating the proposed structure including the proposed structure foundation locations is shown on Dwg. No. 3848901-A in the Appendix. The proposed profile grade superimposed on a longitudinal stratigraphical section across the site is also included on the drawing.

The subsurface conditions at the site as described earlier consist of native clayey silts, silts, sands to silty sands with traces/ some gravel underlain by organic silty clays to organic clays, which in turn are underlain by deposits of glacial till origin. Overburden at the site is underlain by shale bedrock of the Queenston Formation. The organic silty clays to organic clays are materials that control the slope stability and settlement requirements at the site. In view of the significant influence that slope stability and settlements have on the arrangement and positioning of the structure, recommendations pertaining to these two geotechnical considerations are given firstly in this report. Recommendations pertaining to the design and construction of structure foundations, and backfill to the structure are then provided.

## **SETTLEMENT OF APPROACH EMBANKMENT**

### **General**

Settlements of the native subsoils at the site will occur as the result of the proposed embankment loading. Soil displacement can occur as a result of elastic compression, and/ or primary consolidation and/ or secondary compression. All soils experience elastic settlements. These settlements, however, are instantaneous and usually occur during the

construction period. Consequently, elastic settlements do not induce maintenance problems. Primary consolidation and secondary compression, on the other hand, are time dependent, and hence soils susceptible to these types of settlements must be engineered to avoid excessive post construction settlements and the maintenance problems that can result. At the site, the organic silty clay to organic clay stratum will settle as a result of primary consolidation and secondary compression. A prediction of the settlements induced in the organic silty clay to clay as well as the other native subsoils at the site are discussed below.

Settlements within the embankment fill proper itself can also be expected in addition to the settlement of the native subsoils. The settlement characteristics of different fill materials are described below.

The two important aspects of settlement that must be addressed are magnitude and time rate. The magnitude of settlement is a function of the compressibility characteristics of the soil, the soil thickness and the magnitude of the embankment loading. The time rate is a function of the drainage conditions, the permeability of the soil, the soil compressibility characteristics and the soil thickness.

All the settlement considerations, with particular emphasis on the organic silty clay to organic clay are discussed below. Recommendations are then given to solve the settlement problem.

## **Settlement Considerations**

### **Primary Consolidation and Secondary Compression**

The organic silty clay to organic clay at the site will undergo primary consolidation and secondary compression settlements as a result of embankment loadings exceeding the preconsolidation pressure of the soil which is as low as approximately 60 kPa in excess of the overburden pressure at the site. With fill heights exceeding 3 metres at the site, and using a normal weight fill material (unit weight  $\geq 20 \text{ kN/m}^3$ ), primary consolidation and secondary compression settlements can be expected.

Figure 7 in the Appendix illustrates anticipated primary consolidation settlements for different fill heights. One-dimensional consolidation theory was used to compute the settlements. The load deformation curves illustrated on Figures 4a and 4b were used to determine the void ratio change (deformation) as a result of the proposed loadings and Osterberg's (1957) stress distribution curves were used to determine the influence of the embankment loading on the native subsoil. Settlements in the order of 175 mm and 250 mm have been predicted for a four metre embankment at the west and east approach embankment respectively. For an 8 metre embankment, settlements in the order of 500 mm and 700 mm have been predicted at the west and east approach embankment respectively.

The time rate of primary consolidation of the organic silty clay to organic clay stratum was predicted using Terzaghi's theory of one dimensional consolidation and the results are shown on Figures 8a and 8b. Double drainage to the overlying sands and underlying deposits of glacial till origin was assumed. The coefficient of consolidation ( $c_v = 3\text{m}^2/\text{yr}$ ) that was used in the calculations was based on values previously determined for similar subsoils for the specified loadings.

The results of the time-rate calculations reveal that it takes approximately 42 months to achieve 90% degree of consolidation. It takes approximately 10 months to achieve 50% degree of consolidation. Therefore, for a four metre embankment, after 10 months of embankment loading, 125 mm and approximately 85 mm of primary consolidation settlement would be outstanding at the east approach and west approach respectively.

Secondary compression is thought to be due to the gradual readjustment of soil mineral particles into a more stable configuration. Hence, as the pore pressures generated as a result of embankment loading dissipate, the particles rearrange themselves closer together decreasing the void ratio in the process. The magnitude of secondary compression is difficult to predict but it is not expected to be of significant magnitude (less than 5% of the primary consolidation). Furthermore, it is presumed that primary consolidation and secondary compression proceed simultaneously from the time of loading and consequently, some of the secondary compression can occur during the primary consolidation period.

### **Elastic Settlements of Native Subsoils**

The elastic settlements of the native subsoils at the site were predicted using Steinbrenner's solution (1934). Moduli of elasticity for the sand, organic silty clay to clay and glacial tills assumed in the calculations are given below in Table 7 below. Stress distribution influence factors were taken as a function of the embankment geometry and subsoil thickness.

<b>Table 7 - Moduli of Elasticity</b>	
<b>Soil</b>	<b>Modulus of Elasticity (<math>E_s</math>) (MPa)</b>
<b>Sands</b>	<b>40</b>
<b>Organic Silty Clay to Clay</b>	<b>75</b>
<b>Glacial Tills</b>	<b>250</b>

Based on the calculations, up to 50 mm of elastic settlement of the native subsoils can be realized as a result of embankment loadings produced by fill heights ranging from 3 to 8 metres. These settlements, however, are expected to be instantaneous and should occur during the construction period.



**Settlements within Embankment Fill Proper**

Settlements within the embankment fill material itself are also anticipated as the result of internal stresses induced by the self weight of the material. The magnitude and time rate of this settlement is a function of the embankment height and the composition of the fill. Settlements expected within cohesive and non-cohesive earth fills, and also rock fills are discussed below.

For earth fills up to approximately 7 metres, the total settlement within the embankment proper is expected to be approximately 0.5% of the fill height. Therefore, for fill heights ranging from 3 metres to 7 metres, total settlements in the order of magnitude of 15 mm to 35 mm can be expected. For earth fills in the 7 metre to 10 metre height range, total settlements in the embankment proper is expected to be approximately 0.75% of the fill height.

For rock fills, on the other hand, slightly greater settlements can be expected as a result of particle breakage caused by contact forces and particle reorientation within the rock fill. In general, rock fill embankments have shown settlement equivalent to approximately 1% of the fill heights. Therefore, for fill heights ranging from 3 metres to 8 metres, total settlements in the order of 30 mm to 80 mm can be expected.

Settlements within the earth fill should occur almost instantaneously and hence should occur during or immediately following construction for a granular material. Settlements

of cohesive fill embankments will be more time dependent and anticipated to be realized within a three (3) month time period following placement.

Settlements within the rock fill material will be time dependent. However, for the fill thickness proposed at the site, it is expected that post construction settlements will not be significant.

### **Settlement Recommendations**

#### **General**

In consideration of the predicted settlements, various alternatives are provided to minimize post construction settlement problems. The options discussed below include:

- (1) Surcharging and Preloading
- (2) Lightweight Fill Material
- (3) Wick Drains
- (4) Spanning the Organic Silty Clay to Clay Deposit

The option that proves to be the most technically feasible, cost effective and environmentally viable that also satisfies construction scheduling constraints shall be chosen. All factors considered, the wick drain option is perhaps the most applicable at the site.

1) Surcharge and Preload

If construction scheduling permits, post construction settlements can be minimized by preloading and surcharging. In general, for example, it is predicted that an embankment surcharge of 2 metres in thickness and assuming a unit weight of fill of  $20 \text{ kN/m}^3$ , allowed to preload for six months can accelerate the consolidation settlements to approximately 70 to 75% of the total anticipated settlements. For a nine month, 2 metre surcharge preload, approximately 75% to 90% of the total anticipated settlement can be realized. In addition within the six or nine month preloading period, it is expected that all elastic settlements of the native subsoils and fill materials will be realized.

Figure 9 in the Appendix illustrates graphically the procedure used to determine the rate of acceleration. As an example, a four metre embankment is surcharged with two metres of additional fill for nine months. Initially, the progress of settlement will occur along the curve for the surcharge (6 m fill height). After the surcharge is removed the progress of settlement will follow the time-rate pattern predicted for the original 4 metre fill height (refer to match points X and Y on Figure 9). The figure illustrates that the settlement has been accelerated by approximately 16 months (75% consolidation). Therefore an outstanding consolidation settlement of approximately 50 mm remains.

When additional surcharge load is placed, the stability of the embankment must be analyzed to ensure that no slope stability problems are triggered as a result of the additional loading. Slope stability is discussed in detail in the next section of this report.

As mentioned in this section, for proposed surcharge fill heights up to 10.5 metres constructed at 1.5H:1V or flatter, no global stability problems are anticipated. Figure 10 in the Appendix illustrates a typical surcharge load geometry.

## 2) Lightweight Fill

Total settlements can be significantly reduced if a lightweight fill material is used as a replacement for normal weight fill material. As mentioned earlier, the organic silty clay to clay is the stratum that is the most problematic because of consolidation settlements that can occur. As discussed earlier, this soil has been preconsolidated approximately a minimum of 60 kPa in excess of the existing overburden pressure. Therefore the application of a lighter weight fill material will result in being able to construct embankments of greater height without consolidation settlements.

Two types of lightweight material that can be considered is blast furnace slag or expanded or extruded polystyrene. A 9.5 mm ( $\frac{3}{8}$ ") structural coarse slag can be placed and compacted at an in situ unit weight of  $11.5 \text{ kN/m}^3$  and the polystyrene has a unit weight as low as  $0.4 \text{ kN/m}^3$ . The slag material can therefore be used to construct embankments to a height of approximately five metres without any consolidation settlements and the polystyrene can be used to construct embankments of even greater heights without consolidation settlements.

A number of design and construction considerations must be addressed in employing the lightweight fill materials. Physical, chemical and mechanical properties of these materials are design considerations that need to be evaluated and include grain size, crushing characteristics and chemical composition for the slag material and compressive and flexural strength, flammability, buoyancy problems and biodegradability and wetness for the polystyrene. Construction considerations include avoiding the overcompaction of the slag material and proper subgrade preparation, drainage and installation of the polystyrene. These are all details that have been previously considered and resolved successfully on other MTO projects. This information can be obtained from our office.

Table 8 below provides some approximate costs for the supply of the slag and the polystyrene. These costs can be used to produce preliminary estimates.

Table 8 - Lightweight Fill Costs		
Material	Supply	Transportation
Water Cooled Pelletized Slag	\$25/ m <sup>3</sup>	\$0.07/ km/ tonne
Polystyrene - Extruded	\$200/ m <sup>3</sup>	(included in supply)
- Expanded	\$100/ m <sup>3</sup>	

### 3) Wick Drains

Wick drains are vertical drains manufactured from geosynthetic material that can be installed at close spacing to increase the rate of consolidation of the organic silty clay to

organic clay. The wick drains expedite the consolidation process much faster than the conventional surcharge and preloading. This is for the reason that radial drainage occurs at a much larger rate due to the shorter drainage path and the fact that the coefficient of consolidation in the horizontal direction ( $c_h$ ) is greater than the coefficient of consolidation in the vertical direction ( $c_v$ ).

Although wick drains have not been previously incorporated in MTO embankment design, wick drains have been successfully employed in highway embankment design and large dam designs across the United States and in western Canada where subsoil conditions are generally similar to the conditions present at the Jordan Harbour site. Our office, consequently, is confident that a value engineering wick drain design can be successful at the Jordan Harbour site.

Based on our calculations, it is expected that for wick drains installed to Elevation 59 m and at a 2 metre triangular grid spacing, 90% degree of consolidation can be achieved within three months with a two metre surcharge load placed on the final embankment height. A typical section that illustrates the wick drain-surcharge load combination is given in Figure 11. The figure illustrates the embankment foundation containing the wick drains, a 0.5 metre drainage blanket and working pad consisting of Granular 'A' or Granular 'B' material and the final embankment and surcharge embankment geometries. As discussed previously, embankment materials shall be placed and compacted in accordance with OPSS 501 series.

Wick drains consist of a flexible plastic core surrounded by a filter fabric jacket. They are about 100 mm wide and 2 to 6 mm thick. The core is ribbed, studded or channelled to maximize water flow capacity and the filter allows passage of groundwater into the drain core while preventing soil migration. Some proprietary wick drain products include Alidrains and Mebradrains. Our office can coordinate standard drawings, and a NSSP for inclusion in the contract documents.

Wick drains are typically installed by means of a mandrel which is pushed through the soil to the desired depth. The wick drain is shipped in rolls and threaded through a mast and into the mandrel. Consequently, as the mandrel is pushed, the wick drain is also simultaneously pushed to the design tip elevation. There are various means of driving the mandrel including a cable pull crane and hydraulic backhoes.

In view of the surficial sand to silty sand with some gravel deposit overlying the organic silty clay deposit, preaugering through this material will be required to facilitate the installation of the wick drains. In view of the cohesionless nature of these soils submerged below the groundwater table, conditions of unbalanced hydrostatic head will result and hence, the Contractor will have to control any soil cave-in or sloughing in the preaugered hole. A NSSP should be included in the contract documents that states that this condition can develop.

Typical costs for the supply and installation of the wick drains at the Jordan Harbour site were ascertained by contacting various specialized contractors and suppliers in the industry. These costs are summarized in Table 9 below.

Table 9 - Wick Drains Supply/ Installation Costs
\$25/ m <sup>2</sup> (Embankment Area)

4) Spanning the Organic Silty Clay to Clay Deposit

Alternatively, consideration may be given to eliminating fill placement entirely within the area that contains the organic silty clay to clay by spanning the deposit with a structure. The organic silty clay to organic clay extends from Station 11+960 to Station 11+440 at the site.

## **STABILITY OF APPROACH EMBANKMENTS**

### **General**

The design of approach embankments as proposed at the site must be designed to avoid instabilities in both the longitudinal direction and also the transverse direction. The procedure conventionally involves satisfying two major criteria:

- (1) Global Stability
- and (2) Internal (Surficial) Stability



Global stability calculations involve conducting two-dimensional analyses to determine whether the applied embankment loading will induce a deep seated slip failure surface within the native subsoil which will manifest itself in a "global" slip surface movement within the embankment. Many factors must be considered in this analyses including embankment geometry, height and load; external loadings; overall geometry of embankment and natural ground surface, for instance the influence of scouring at the toe of an embankment fill; the properties and behaviour of the native subsoils and the groundwater and surface water conditions at the site.

Internal stability, on the other hand, concentrates on the stability of the embankment within the embankment proper itself. Internal embankment stability involves the assessment of seepage forces; erosional forces; embankment materials, construction and geometry. At the site, any waves produced within the Jordan Harbour are a major factor in evaluating the internal stability of the approach embankments.

Specific recommendations to safeguard against global and internal instabilities of the approach embankments at the site are given below.

### **Global Stability**

The critical condition examined in the evaluation of the global stability of embankment fills as proposed at the site location is the short term (undrained) condition and consequently a total stress analysis was conducted. In all cases, stability computations were carried

out using an in house MTO-slope application software package which is based on Sarma's method of limiting equilibrium. The formulation of Sarma's method is described in a paper entitled "Stability Analysis of Embankments and Slopes", Sarma, S.K. (1973), Geotechnique 23, No. 3, pp. 423-433.

The process of stability analyses involves the selection of pertinent shear strength parameters and physical soil properties such as unit weight, inputting the subsurface and groundwater conditions and then designing a surface geometry that produces an acceptable factor of safety of 1.3 using the MTO slope program. Scouring of the channel must also be considered in the overall global stability. A channel width of 27 m and a scour depth (channel elevation) of 70.5 m was used in the analyses. The top of embankment elevation used in the analyses was 83 m which includes an additional surcharge thickness on the surface of the final QEW grade.

Figure 12 in the Appendix illustrates the overall geometry, the subsurface conditions and relevant subsoil parameters used in the stability analyses. Both circular and composite failure surfaces were evaluated and a critical slip surface was searched. The results of the analyses reveal that a 38.5 metre offset between the centreline of the channel and the abutment is required to satisfy global stability requirements. The geometry of the offset shall consist of 2H:1V slopes and a minimum four metre berm at an elevation of approximately 76.5 m.

It is prudent that the approach embankments be protected against long term channel erosion that can result from any wave action. Channel armouring shall therefore be placed for a minimum width of the structure within the channel area. It is recommended that to minimize the impact of the channel excavation on the overall embankment stability, the excavation of the channel during the construction of the channel lining be restricted to strips not exceeding five metres in width. A NSSP shall be included in the contract documents identifying this requirement.

### **Internal Stability**

A shoreline protection scheme will be required where approach embankments are located adjacent to the Jordan Harbour to preserve the internal stability of the embankment from the erosive wave action. Figure 13 in the Appendix illustrates the proposed slope treatment that consists of armour stone and a minimum 0.6 m rip rap (OPSS 1004.05.06) placed at a 1.25H:1V slope or flatter. To prevent soil migration, it is recommended as shown on the figure that minimum 300 mm thick layers of granular A or B (OPSS 1010) and 19-26 mm clear stone (OPSS 1004.05.07) be placed as filter materials between the embankment and the coarser rip rap material.

In the areas where encroachment into the lake is required, it is recommended that a bedding material consisting of rip rap or granular material be placed to facilitate the shoreline protection construction. The shoreline protection scheme is applicable for fill heights up to 10.5 metres which is in the range at the site.

Any slope that does not require shoreline protection at the site still requires internal instability protection. It is therefore recommended that any such slope be designed to satisfy the following guidelines:

### 1 - Earth Fills

- 1.1 Earth fills up to eight (8) metres in height shall be constructed at 2H:1V slopes or flatter.
- 1.2 Earth fills exceeding eight (8) metres shall be constructed at 2H:1V slopes with a nominal two (2) metre midheight berm constructed with a 2% gradient towards the toe of the embankment to promote surface runoff or alternatively at 2.5H:1V slopes.

### 2 - Rock Fills

- 2.1 Rock fills up to ten (10) metres in height shall be constructed at 1.25H:1V or flatter slopes.

Normal slope vegetation cover shall be established as per conventional MTO standards as soon as possible to provide surface erosion protection for the slopes above the rock protection and where applicable elsewhere at the site.

**Embankment Construction**

Embankment material and construction shall conform to OPSS 212 and OPSS 206 series respectively. The embankment material including any surcharge material shall be compacted as outlined in OPSS 501 series.

All softened and/or organic material should be excavated for their full depth within the plan limits prior to fill placement. Softened material can be identified by proof rolling the native soil prior to fill placement. Any new fill constructed adjacent to existing embankment fills shall be benched as shown on OPSD 208.01.

**Embankment Settlement Monitoring Instrumentation**

In view of the compressible nature of the organic silty clay to organic clay stratum at the site and to minimize post construction maintenance, confirmation of the magnitude and time rate settlement is certainly warranted. Consequently, it is recommended that an instrumentation program be arranged and coordinated by this office to monitor the magnitude and time rate settlement of the compressible organic silty clay to clay.

The selection, design and installation procedures for this instrumentation can be provided by this office.

## **STRUCTURE FOUNDATIONS**

### **General**

Boreholes 501 to 508 inclusive (8 boreholes) were advanced specifically to retrieve subsurface data to facilitate the design and construction of the structure foundations. The boreholes were planned and advanced considering practical limitations at the proposed structure foundation locations.

The surficial soils at the site and general site conditions render shallow foundations as an unsuitable option at the site. It is therefore necessary to support all bridge structure foundations and adjoining retaining walls on deep foundations founded on or in bedrock. Deep foundations can consist of either concrete caissons installed in preaugered holes or driven steel H-piles. Recommendations for the design and construction of the deep foundation units are given below. The option that proves to be the most economical and technically feasible shall be selected.

### **Design Considerations**

#### **Concrete Caissons**

All structure foundations can be supported on concrete caissons socketed into the shale bedrock with interbedded siltstone formation. It is recommended that the caissons be founded on unweathered rock at or below the elevations provided in Table 10 below. To facilitate the design of the concrete caissons, a vertical factored capacity at U.L.S. equivalent to 3500 kPa can be employed. In addition, a factored bond stress at U.L.S.

equivalent to 350 kPa can be used in the unweathered bedrock. A minimum socket depth of 0.5 m is recommended. The total capacity from the bond stress, however, shall not exceed the end bearing capacity. Due to the unyielding nature of the bedrock, the Serviceability Limit State (SLS) will not govern the design because the stresses required to induce detrimental settlements at the S.L.S. will exceed the factored capacity at U.L.S. Reductions of axial capacities for inclined loadings shall conform to factors provided in of the O.H.B.D.C.

Table 10 - Caisson Founding Elevation	
Structure	Foundation Elevation (m)
West Abutment	57.6-62.8*
West Pier	57.9-61.2*
East Pier	57.8
East Abutment	57.2

\* It is recommended that the caissons be designed at the lower founding elevations because of the varying bedrock surface elevation.

In view of the anticipated settlements at the approach embankments, downdrag forces or negative skin friction must be accounted for in the design of the caissons at the abutment locations. Downdrag forces expected for different caisson diameters at the U.L.S. are given in Table 11 below.

Table 11 - Downdrag Forces on Concrete Caissons (kN)

Caisson Diameter (m)	West/ East Abutment
0.9	1000
1.2	1350
1.5	1700
1.8	2050

The appropriate load factors are to be applied to the downdrag forces in accordance with Section 2 of the O.H.B.D.C.

The designer can use the bearing capacity provided to select the size of the caisson and the respective ultimate capacity. For instance, a 1.2 m diameter caisson will yield a load capacity equivalent to approximately 4000 kN at U.L.S. for an end bearing capacity of 3500 kPa at the piers. An additional 1320 kN capacity can be achieved with a 1 metre socket.

The lateral resistance for vertical or battered concrete caissons can be computed in accordance with Section 6-9.8 of the O.H.B.D.C. and using the data given in Table 12 below. Specific soil thickness/ depths can be obtained from the individual borehole logs. Horizontal resistance calculations employing the soil parameters used in Table 12 shall be reviewed by our office.



Caissons can be socketed into the bedrock to augment the lateral resistance. A minimum 0.5 m of socket into rock is required. Battered caissons are limited to a maximum 1H:4V due to construction limitations.

**Table 12 - Horizontal Resistance Design Parameters (Unfactored)**

Soil/ Rock	Angle of Internal Friction ( $\phi$ )	Undrained Shear Strength ( $C_u$ ) (kPa)	Unconfined Compressive Strength ( $q_u$ ) (kPa)	Bulk Unit Weight $\gamma$ (kPa)
Fill Material	30°	-	-	20
Sand to Silty Sand	30°	-	-	20
Organic Silty Clay to Clay	-	80	-	15
Cohesive Till	-	200	-	15
Cohesionless Till	35°	-	-	20
Weathered Bedrock	-	-	1,000	20
Unweathered Bedrock	-	-	10,000	22

Submerged unit weights are to be used below the groundwater table. Pile caps shall be protected against frost penetration by providing a minimum 1.2 m earth cover or equivalent frost protection.

### Driven Steel H-piles

Alternatively, all structure foundations can be founded on steel H-piles driven to the bedrock surface as identified in Table 13a below. For purposes of the O.H.B.D.C., the steel H-piles can be designed employing the axial capacities tabulated in Table 13 below.

Table 13a - Pile Tip Elevations	
Structure	Pile Tip Elevation (m)
West Abutment	57.6-62.8
West Pier	57.9-61.2
East Pier	57.8
East Abutment	57.2

Table 13b - Driven Steel H-Piles			
Structure	Pile Type	Factored Axial Capacity at U.L.S. (kN)	Axial Capacity at S.L.S. (kN)
Piers	HP310 x 110	1600	1150
	HP310 x 79	1150	890
Abutments	HP310 x 110	1600	1150
	HP310 x 79	1150	890

Downdrag forces must also be considered in the design of driven steel H-piles at the abutment locations. The magnitude of the downdrag forces on the steel H-piles is given in Table 14 below. As mentioned earlier, downdrag forces are applicable at the U.L.S. and must be factored in accordance with Section 2 of the O.H.B.D.C.

Table 14 - Downdrag Forces on Steel H-piles	
Structure	Downdrag Forces (kN)
West Abutment	600
East Abutment	600

Across the site, the cohesionless heterogeneous mixture of silt, sand and gravel or cohesive heterogeneous mixture of clayey silt, sand and gravel overlying the bedrock may prevent pile penetration and hence the piles may not reach the bedrock surface. Therefore, the pile installation shall be carefully monitored to confirm this potential occurrence. The Hiley Dynamic Formula as shown on Dwg. No. SS103-11 can be used to verify the pile capacity and acceptability of the pile driven into the overlying till deposit. The pile installation shall be controlled using an ultimate capacity of 3450 kN and 2670 kN for HP310x110 and HP310x79 piles respectively.

Axial capacities provided in Table 13 are for vertical piles only. Reductions of axial capacities for inclined loadings shall conform to factors provided in Section 6-9.3 of the O.H.B.D.C.

It is recommended that to facilitate the pile driving process, all piles be equipped with reinforced tips.

The lateral resistance for both vertical and battered piles shall be computed in accordance with Section 6-9.8 of the O.H.B.D.C. Pertinent unfactored soil parameters to facilitate the design of vertical piles are given in Table 12.

Pile caps shall be protected against frost penetration by providing a minimum 1.2 m earth cover or equivalent frost protection.

### **Construction Considerations**

#### **Caisson Construction**

Special measures will be required for caisson construction within augered holes penetrating the cohesionless sand to silty sand with some gravel submerged below the groundwater table. This is for the reason that soil sloughing and cave-in will result due to the unbalanced hydrostatic head condition produced during construction. One method of controlling this condition is to construct the caisson within a temporary steel liner installed to the full depth of the submerged cohesionless soils to prevent soil cave-ins. After the liner has been cleared out, concrete shall be placed in the dry by bailing out the water or alternatively by tremie methods.

Alternatively, mud drilling and tremie techniques can also be used to control conditions of unbalanced head conditions. In employing this technique, the quality of the bentonite slurry (density, viscosity) should be kept under constant control to ensure that it performs satisfactorily.

The proposed method of caisson installation shall be in accordance with OPSS 903.07.03 and subject to review by this office. It is prudent that the contractor submit a caisson construction installation procedure for approval as outlined in OPSS 902.04.01 at least three (3) weeks prior to construction. A NSSP specifying caisson materials, construction procedure and inspection should be included in the contract documents. This NSSP can be provided by our office.

It is also recommended that a NSSP be included in the contract documents that states that the cohesionless sand to silty sand with some gravel submerged below the prevailing groundwater is subjected to conditions of unbalanced hydrostatic head and hence can boil. In addition, a NSSP shall also be included that boulders and cobbles are characteristic components of deposits of glacial till origin and hence can be encountered during the caisson installation.

#### **Temporary Dewatering Enclosure**

Any foundation excavation/ construction within the Jordan Harbour or the Twenty Mile Creek will have to be carried out within a temporary interlocking steel sheeting enclosure

(cofferdam). Any pile cap construction within the native surficial sand to silty sand with some gravel deposit below the prevailing groundwater table will also require similar dewatering measures. The steel sheeting shall be driven to a depth below the base of the excavation equivalent to the unbalanced hydrostatic head above this level. Once the water tight enclosure is formed, water can be discharged in an environmentally accepted manner using conventional pumping methods.

A NSSP shall be included in the contract documents that specifies the temporary dewatering enclosure. The NSSP should include that it is the responsibility of the Contractor to carry out the dewatering to avoid soil disturbance in accordance with OPSS 517. The Contractor shall submit the proposed dewatering for our review at least three weeks prior to construction.

#### **Foundation Scour Protection**

The channel armouring shall be adequately extended to protect the piers located partially or entirely in the Jordan Harbour or Twenty Mile Creek.

#### **BACKFILL TO STRUCTURE**

##### **Material**

It is recommended that Granular 'A' or Granular 'B' material as specified in OPSS 1010 be placed within a wedge behind the abutments as described in Section 6-7.4.1 of the O.H.B.D.C. The application of granular material combined with weep holes in the

abutment walls or pipe subdrains to drain any accumulation of water in the backfill will prevent hydrostatic pressure build-up.

Design parameters of the soil are given in Table 15 below. Computations of lateral earth pressure shall be in accordance with Section 6-7 of the O.H.B.D.C.

Table 15 - Backfill Properties		
	Granular 'A'	Granular 'B'
Angle of Internal Friction ( $\phi$ ) Unfactored	35°	30°
Unit Weight ( $\text{kN/m}^3$ ), $\gamma$	22.8	21.2
*Coefficient of Active Earth Pressure ( $K_a$ ) - S.L.S.	0.27	0.33
*Coefficient of Earth Pressure at Rest ( $K_o$ ) - S.L.S.	0.43	0.50

\*These earth pressure coefficients apply to horizontal backfill surfaces only. The appropriate consideration shall be given to account for sloping backfill. The coefficient of earth pressure at rest shall be applied for rigid and unyielding walls. For flexible walls, the active condition can be used provided that the wall displacement satisfies minimum deflection criteria as specified in Section C 6-7.1 of the O.H.B.D.C. (2nd Edition).

### **Backfilling and Compaction**

The backfill shall be placed in 300 mm lifts in accordance with OPSS 902 series and compacted to achieve the target maximum dry density as outlined in OPSS 501 series.

Heavy vibratory equipment should be avoided in the backfill construction adjacent to the structure to minimize wall deflection and possible wall damage. Vibratory equipment exceeding 6000 kg operating weight should be kept outside of a 1.5 vertical to 1 horizontal line extending upward from the base of the abutment footing. Hand compaction equipment shall be used within the limits described.

### **MISCELLANEOUS**

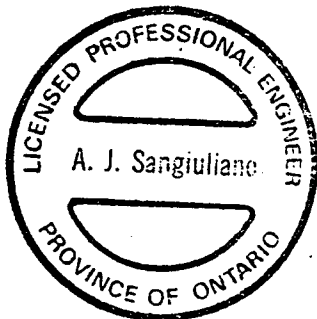
The fieldwork for this investigation was carried out under several different mobilizations as summarized below.

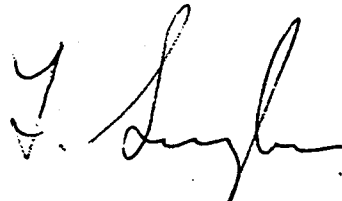
BH's	Time Period	Supervisor(s)	Contractor
1-4	November, 1965	R. Magi, F. Wang	Dominion Soil Inc.
101-104	June/ July 1992	M. Vasavithasan	Malone's Soil Samples
105-114	June 1993	M. Vasavithasan	Malone's Soil Samples
301-302	August 1993	M. Vasavithasan	Malone's Soil Samples
501-508	January/ February 1994	T. Sangiuliano, D. Rothwell	Malone's Soil Samples




Logging of rock core in the laboratory was carried out by D. Williams, Petrographer.

The project was carried out by T. Sangiuliano and M. Vasavithasan under the general supervision of P. Payer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed by P. Payer and approved by D. Dundas, Chief Foundation Engineer (acting).



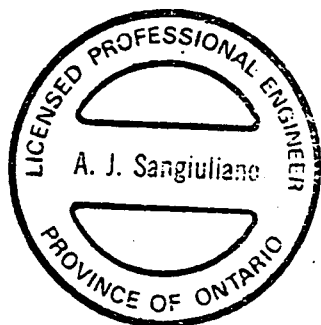
  
T. Sangiuliano P.Eng.  
Foundation Engineer



  
D. Dundas, P.Eng.  
Chief Foundation Engineer (acting)

Logging of rock core in the laboratory was carried out by D. Williams, Petrographer.

The project was carried out by T. Sangiuliano and M. Vasavithasan under the general supervision of P. Payer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed by P. Payer and approved by D. Dundas, Chief Foundation Engineer (acting).



A handwritten signature in black ink, appearing to read "T. Sangiuliano".

T. Sangiuliano, P.Eng.

Foundation Engineer



A handwritten signature in black ink, appearing to read "D. Dundas".

D. Dundas, P.Eng.

Chief Foundation Engineer (acting)

# RECORD OF BOREHOLE No 2

1 OF 1

METRIC

(Formerly BH No 2 of 65-F-113)

W.P. 384-89-01 LOCATION CO - ORDS: N 4 782 607.0 ; E 315 020.0 ORIGINATED BY R M  
 DIST 4 HWY QEW BOREHOLE TYPE WASHBORING COMPILED BY M V  
 DATUM GEODETIC DATE 65 11 02 CHECKED BY P P

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ $\text{KN/m}^3$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	20 40 60		
76.1	Ground Surface												
0.0			1	SS	37								
	SILTY SAND, Occasional Gravel, Compact to Dense		2	SS	36								
			3	SS	26								
			4	SS	38								
			5	SS	14								
70.0			6	TW	PM							19.2	
6.1	Organic Silt Clay to Clay Firm to Stiff		7	TW	PM								
			8	TW	PM								
65.4			9	SS	26								
10.7	Heterogeneous Mixture of CLAYEY SILT, SAND and GRAVEL, Very Stiff to Hard ( Glacial Till )		10	SS	95	/15cm							
			11	SS	100	/8cm							
60.5			12	SS	100	/8cm							
15.6	End of Borehole												

# RECORD OF BOREHOLE No 6

1 OF 1

METRIC

(Formerly BH No 14 of 65-F-113)

W.P. 384-89-01 LOCATION CO - ORDS: N 4 782 448.0 ; E 315 250.0 ORIGINATED BY R M  
 DIST 4 HWY QEW BOREHOLE TYPE PENNDRIILL COMPILED BY MV/TS  
 DATUM GEODETIC DATE 65 11 10 CHECKED BY PP

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		
77.4	Ground Surface												
0.0													
	SILTY SAND With Gravel, Loose to Compact  (Fill Material)		1	SS	12								
			2	SS	7								
73.4			3	SS	7								
4.0			4	SS	20								
	SILTY SAND, Occasional Gravel, Loose to Compact		5	SS	31								
			6	SS	8								
67.6													
9.8			7	TW	PM								
	ORGANIC SILTY CLAY TO CLAY  Firm to Very Stiff		8	TW	PM								
			9	TW	PM								
			10	TW	PM								
			11	TW	PM								
59.4													
18.0	Heterogeneous Mixture of GRAVEL, SAND and SILT, ( Glacial Till )		12	SS	173	/38cm							
57.5	Very Dense					/5cm							
19.9	End of Borehole  * GWL not established.												

# RECORD OF BOREHOLE No 7

1 OF 1

METRIC

W.P. 384-89-01

LOCATION CO - ORDS: N 4 782 521.0 ; E 315 111.0

ORIGINATED BY R M

DIST 4 HWY QEW

BOREHOLE TYPE PENNDRILL

COMPILED BY MV/TS

DATUM GEODETIC

DATE 65 11 09

CHECKED BY PP

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
ELV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20 40 60 80 100	20 40 60 80 100					
75.9	Ground Surface												
0.0													
	SILTY SAND With Gravel, Some Clayey Silt, Loose to Compact (Fill Material)		1	SS	18								
			2	SS	7								1 93 (6)
72.6			3	TW	PM								0 3 88 9
4.3			4	SS	23								
	SILTY SAND, Occasional Gravel, Compact		5	SS	24								
69.1			6	TW	PM							19.5	
7.8			7	TW	PM							15.4	Org.=7.4%
			8	TW	PM							16.0	
	ORGANIC SILTY CLAY TO CLAY Firm to Very Stiff		9	TW	PM							15.7	Org.= 10.7%
			9A	TW	PM							13.7	
			10	TW	PM							16.0	Org.=6.3%
			11	TW	PM							15.9	
58.9			12	TW	PM								
18.0	Heterogeneous Mixture of GRAVEL, SAND and SILT, Very Dense ( Glacial Till )		13	SS	95								60 26 (14)
57.0			14	SS	100	/10cm							
19.9	End of Borehole												

+<sup>3</sup>, x<sup>5</sup>: Numbers refer to  
Sensitivity

20  
15-5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 501

1 OF 1

METRIC

W.P. 384-89-01 LOCATION Co-ords: N 4 782 570.0 E 315 018.4 ORIGINATED BY DR  
 DIST 4 HWY QEW BOREHOLE TYPE HS Auger, NXL Core COMPILED BY TS  
 DATUM Geodetic DATE 94 02 01 CHECKED BY PP

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
77.4	Roadway Surface																
0.0	Irregular Mixture of Clayey Silt, Sand & Gravel (Fill Material)					*											
75.4	Brown, Very Stiff		1	SS	19		76										
2.0	Sand to Silty Sand Grey, Loose to Compact		2	SS	9		74										
			3	SS	14		72										
			4	SS	5		70										
66.7	Organic Silty Clay to Clay Blackish Grey to Grey, Stiff to Very Stiff		5	SS	3		68										
10.7			6	SS	4		66										
59.1	Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till)						64										
18.3	Hard						62										
57.6	Shale Bedrock with interbedded Siltstone		7	RC	REC		60										
19.8	Very Weak to Weak, Unweathered						58										
56.1																	RQD = 70%
21.3	End of Borehole • GWL not established																

+3, x5: Numbers refer to  
Sensitivity

20  
15-5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 502

1 OF 1

METRIC

W.P. 384-89-01 LOCATION Co-ords: N 4 782 602.0, E 315 033.0  
 DIST 4 HWY QEW BOREHOLE TYPE HS Auger, NXL Core  
 DATUM Geodetic DATE 94 01 20  
 ORIGINATED BY DR  
 COMPILED BY TS  
 CHECKED BY PP

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
78.9	Ground Surface																
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material)																
74.9	Brown, Very Loose		1	SS	3		76										
2.0	Sand to Silty Sand, some Gravel						74										
	Grey, Compact		2	SS	14		72										
69.1							70										
7.8	Organic Silty Clay to Clay Grey, Stiff to Very Stiff		3	SS	2		68										
66.2																	
10.7	Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till)		4	SS	4		66										
63.0	Red, Soft						64										
13.9	Slightly to Moderately Weathered Unweathered		5	SS	60	/3cm	62										
	Shale Bedrock with interbedded Siltstone		6	RC	REC 100%												RQD = 41%
60.0	Red with interbedded Grey, Very Weak to Weak		7	RC	REC 100%												RQD = 33%
16.9	End of Borehole																
	* 94 01 21																

# RECORD OF BOREHOLE No 503

1 OF 1

METRIC

W.P. 384-89-01 LOCATION Co-ords: N 4 782 560.3, E 315 040.0 ORIGINATED BY DR  
DIST 4 HWY QEW BOREHOLE TYPE HS Auger/NXL Core COMPILED BY TS  
DATUM Geodetic DATE 94.02.01 CHECKED BY PP

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	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
77.4	Ground Surface													
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material)													
75.4	Brown, Loose		1	SS	6		76							
2.0	Sand to Silty Sand, trace/some Gravel						74							
	Grey, Loose		2	SS	7		72							
							70							
			3	SS	8		68							
66.7							66							
10.7	Organic Silty Clay to Clay		4	SS	1		64							
	Grey, Stiff to Very Stiff						62							
			5	SS	3		60							
							58							
59.1							56							
18.3	Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till)													
57.9	Hard													
19.5	Shale Bedrock with Interbedded Siltstone		7	RC	REC 95%									RQD = 58%
	Red with interbedded Grey, Very Weak to Weak, Unweathered		8	RC	REC 100%									RQD = 63%
54.8														
22.5	End of Borehole													
	• GWL not established.													



# RECORD OF BOREHOLE No 504

1 OF 1

METRIC

W.P. 384-89-01 LOCATION Co-ords: N 4 782 591.6; E 315 057.0 ORIGINATED BY DR  
 DIST 4 HWY QEW BOREHOLE TYPE HS Auger, NW Casing, NXL Core COMPILED BY TS  
 DATUM Geodetic DATE 94 01 21 CHECKED BY PP

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	20	40	60	80	100	W <sub>p</sub>	W		
77.1	Ground Surface															
0.0	Sand to Silty Sand, trace Gravel  Gray, Loose to Compact		1	SS	6											
			2	SS	12											
69.5																
7.6	Organic Silty Clay to Clay Gray, Stiff to Very Stiff		3	SS	2											
			4	SS	3											
63.4	Het Mix of Clayey Silt, Sand & Gravel (Glacial Till)		5	SS	60											
13.9	Shale Bedrock with interbedded Siltstone Red with Interbedded Gray Very Weak to Weak  Weathered ----- Unweathered		6	RC	REC 85%											RQD = 7%
60.2			7	RC	REC 100%											RQD = 65%
16.9	End of Borehole															

# RECORD OF BOREHOLE No 506

1 OF 1

METRIC

W.P. 384-89-01 LOCATION Co-ords: N 4 782 558.8 E 315 104.8 ORIGINATED BY DR  
DIST 4 HWY QEW BOREHOLE TYPE HS Auger, NW Casing, NXL Core COMPILED BY TS  
DATUM Geodetic DATE 94 01 19 CHECKED BY PP

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											
								20	40	60	80	100							
77.1	Ground Surface																		
0.0	Irregular Mixture of Clayey Silt, Sand and Gravel (Fill Material) Brown, Firm						76												
75.1			1	SS	12														
2.0	Sand to Silty Sand, trace Gravel  Gey, Very Loose		2	SS	2		74												
							72												
69.2			3	SS	4		70												
7.9	Organic Silty Clay to Clay  Grey, Stiff to Very Stiff		4	SS	4		68												
							66												
			5	SS	4		64												
59.9			6	SS	1		62												
17.2	Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till) Red, Hard		7	SS	36		60												
58.4							58												
18.7	Shale Bedrock with Interbedded Siltstone  Red with interbedded Grey, Very Weak to Weak	Weathered Unweathered	8	RC	REC 100%		56												
55.3			9	RC	REC 100%														
21.8	End of Borehole  • GWL not established.																		

# RECORD OF BOREHOLE No 508

1 OF 1

METRIC

W.P. 384-89-01 LOCATION Co-ords: N 4 782 531.5 E 315 092.5 ORIGINATED BY DR  
DIST 4 HWY QEW BOREHOLE TYPE HS Auger, NW Casing, NXL Core COMPILED BY TS  
DATUM Geodetic DATE 94 02 01 CHECKED BY PP

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
77.0	Ground Surface																
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material)																
75.0	Brown, Compact		1	SS	11												
2.0	Silt to Silty Sand Grey, Very Loose		2	SS	2												
			3	SS	2												
66.3																	
10.7	Organic Silty Clay to Clay Grey, Stiff to Very Stiff		4	SS	1												
			5	SS	2												
58.7			6	SS	3												
18.3	Met. Mixture of Clayey Silt, Sand and Gravel (Glacial Till)																
57.5																	
19.5	Weathered Unweathered Shale Bedrock with interbedded Siltstone Red with interbedded Grey, Very Weak to Weak		7	RC	REC 100%											RQD = 63%	
54.4			8	RC	REC 100%											RQD = 60%	
22.6	End of Borehole																

# RECORD OF BOREHOLE No 101

1 OF 1

METRIC

W.P. 384-89-01 LOCATION CO - ORDS: N 4 782 565.0 : E 315 178.0 ORIGINATED BY M V  
 DIST 4 HWY QEW BOREHOLE TYPE HOLLOW STEM AUGER, NW CASING & CONE TEST COMPILED BY M V  
 DATUM GEODETIC DATE 92 06 29 CHECKED BY P P

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							
75.8	Ground Surface							20 40 60 80 100		20 40 60 80 100		20 40 60 80 100			GR SA SI CL
0.0		Cobbles	1	SS	8										20 71 (9)
			2	SS	4										
			3	SS	7										
			4	SS	12										
			5	SS	12										
			6	SS	18										22 68 (10)
			7	SS	6										
		Sandy Silt	8	SS	5										
68.4															
7.2			9	SS	3										Org.= 7.2%
			10	TW	PH										Org.= 9.6%
			11	SS	3										
			12	SS	5										
			13	SS	4										
60.0			14	SS	10										
15.6			15	SS	106										
58.4															
17.2															
57.2															
18.4															

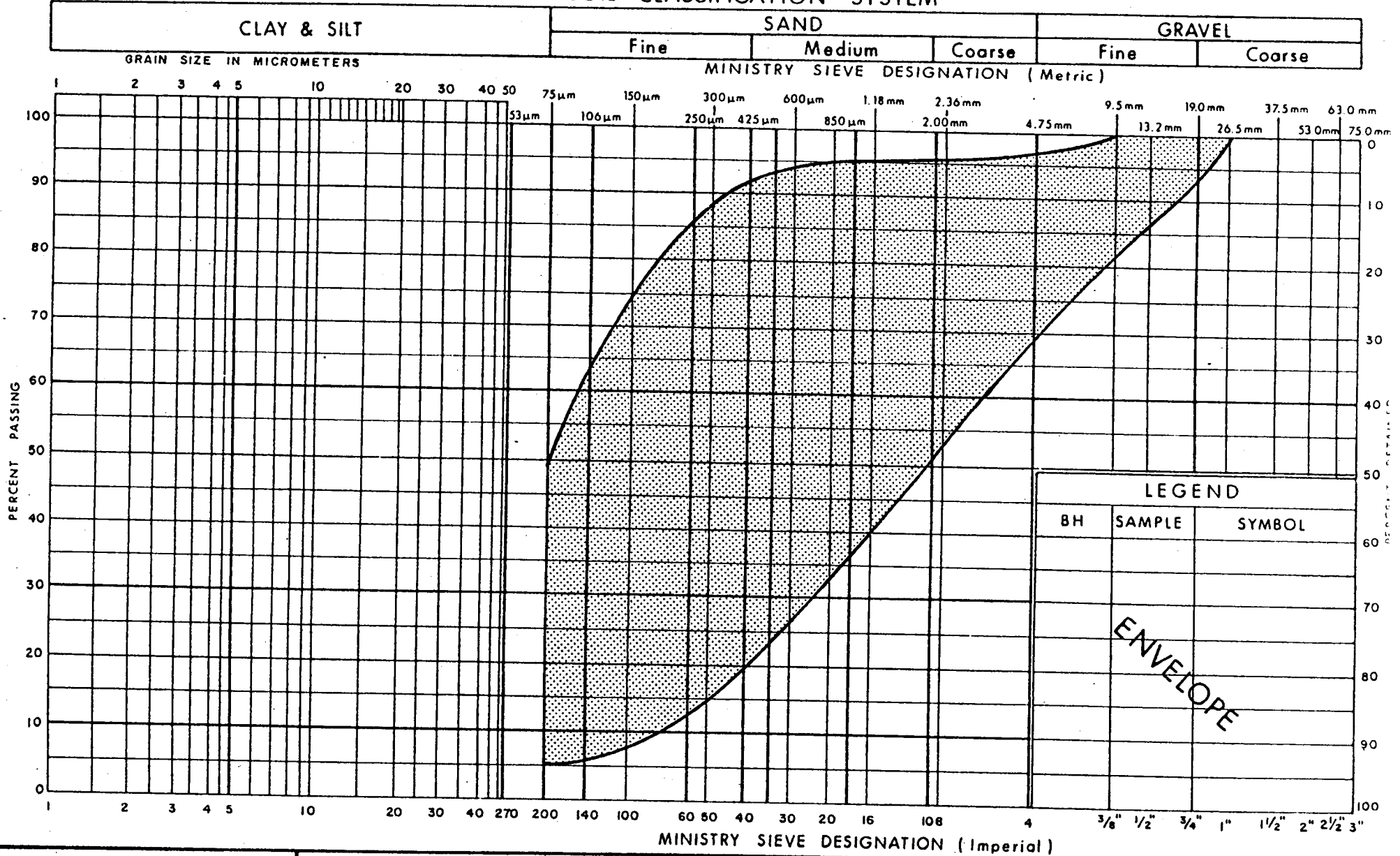
# RECORD OF BOREHOLE No 103 (Formerly BH No 103 of WP 325-89-01)

1 OF 1 METRIC

W.P. 384-89-01 LOCATION CO - ORDS: N 4 782 656.0 : E 314 942.0 ORIGINATED BY M.V.  
DIST 4 HWY QEW BOREHOLE TYPE HOLLOW STEM AUGER, NW CASING & CONE TEST COMPILED BY M.V.  
DATUM GEODETIC DATE 92 07 02 CHECKED BY P.P.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT		UNIT WEIGHT 7 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	W <sub>p</sub>	W		
76.3	Ground Surface						76						
0.0			1	SS	18		74						
	Gravelly Sand, Some Silt		2	SS	18		72						
			3	SS	10		70						
			4	SS	12		68						
	SAND to SILTY SAND, Trace of Gravel Loose to Compact		5	SS	13		66						
			6	SS	27		64						
			7	SS	28		62						
67.8			8	SS	10		60						
8.5			9	SS	3		58						
	Organic Silty Clay to Clay Occasional Layers of Peat, Decomposed Timber		10	TW	PH		56						
			11	SS	4		54						
			12	SS	4		52						
	Stiff		13	SS	3		50						
			14	SS	4		48						
			15	SS	3		46						
56.0	QUEENSTON SHALE Highly Weathered		16	SS	35		44						
20.3	End of Borehole						42						

# UNIFIED SOIL CLASSIFICATION SYSTEM



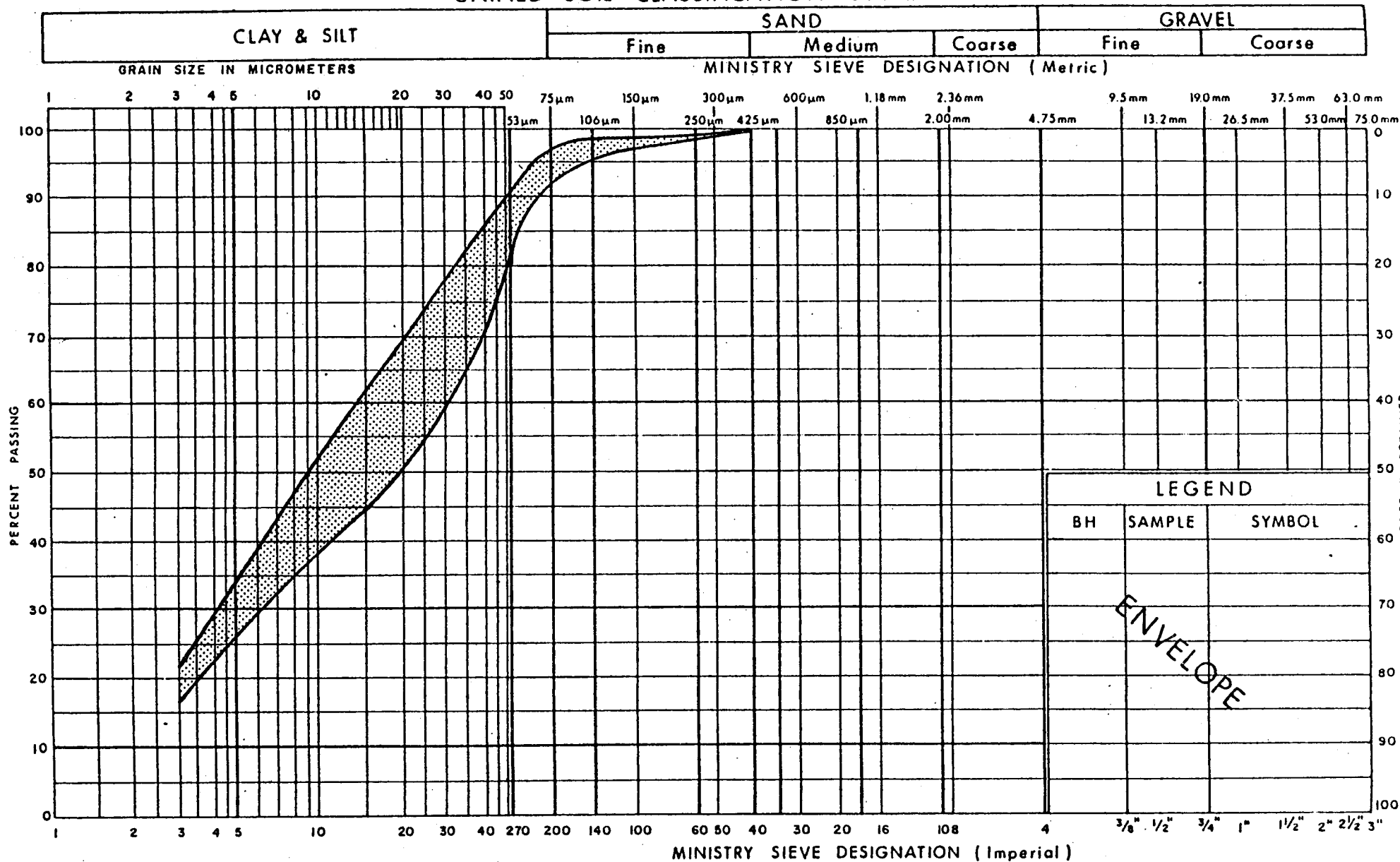
Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
SAND TO SILTY SAND, TRACE / SOME GRAVEL

FIG No 1

W P 384-89-01

## UNIFIED SOIL CLASSIFICATION SYSTEM



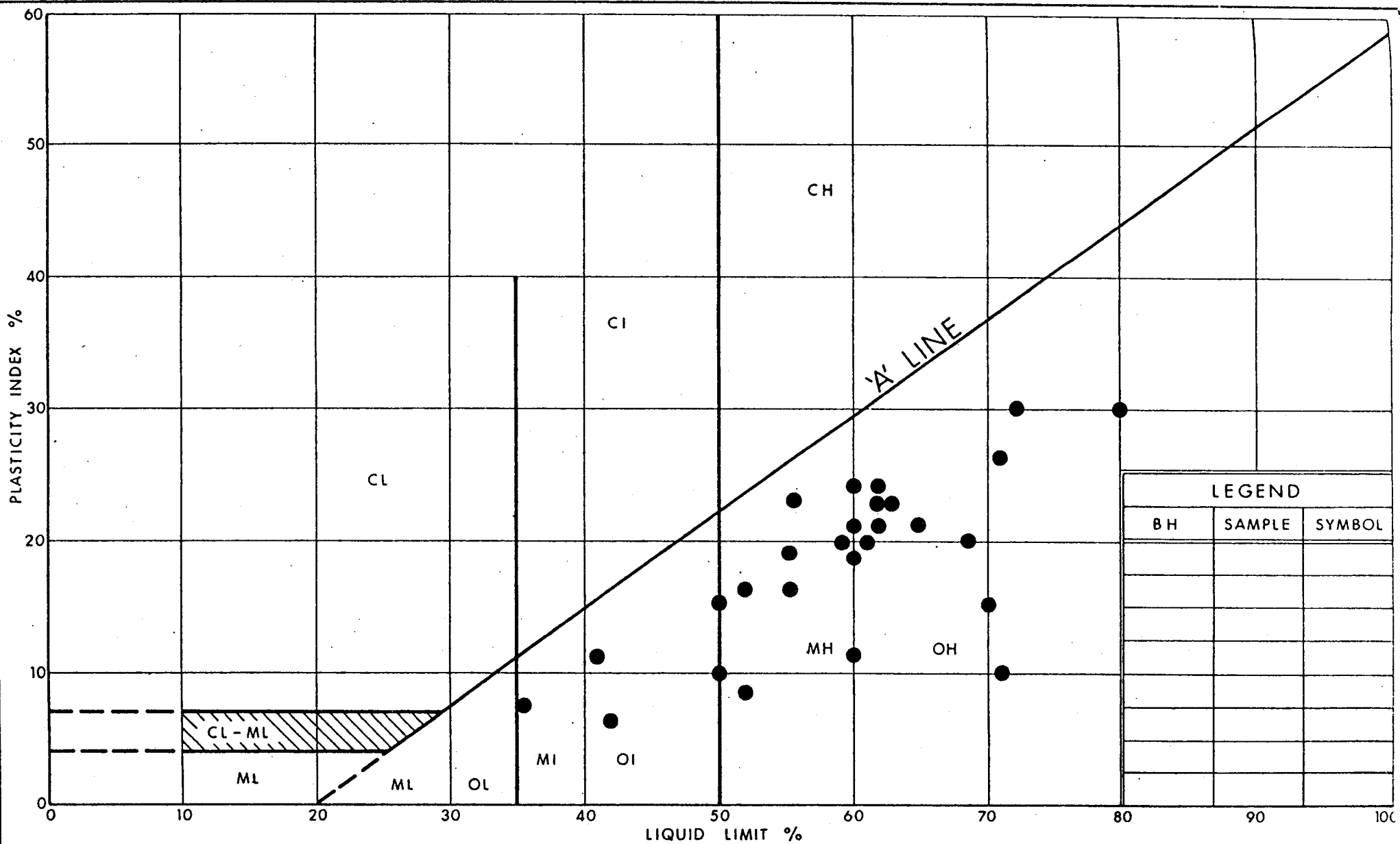
Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

### ORGANIC SILTY CLAY TO ORGANIC CLAY

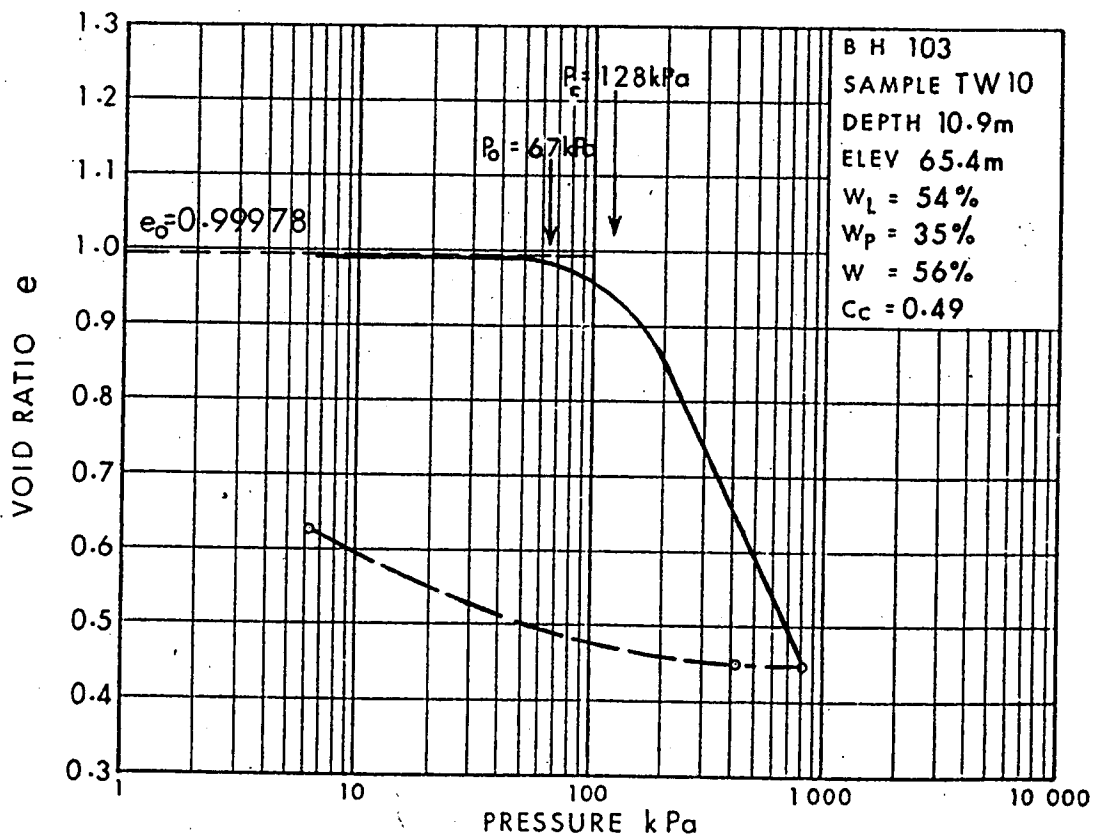
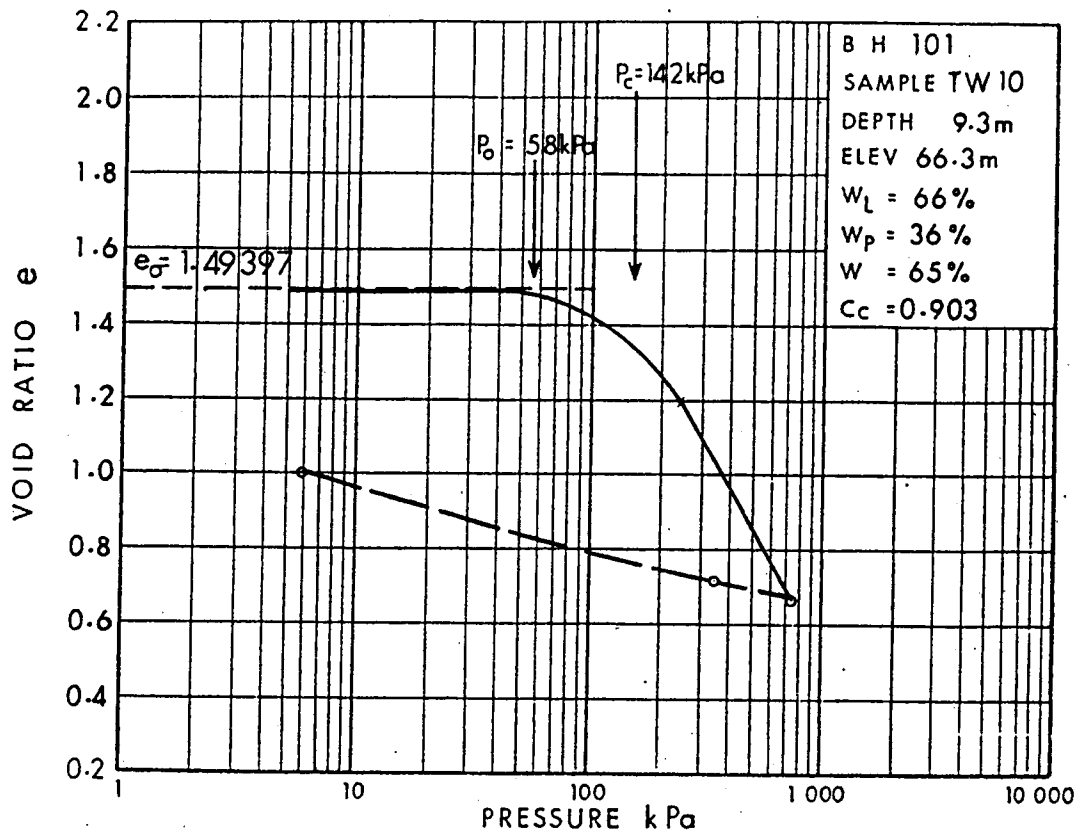
FIG No 2

W P 384-89-01





# VOID RATIO - PRESSURE CURVES



# VOID RATIO - PRESSURE CURVES

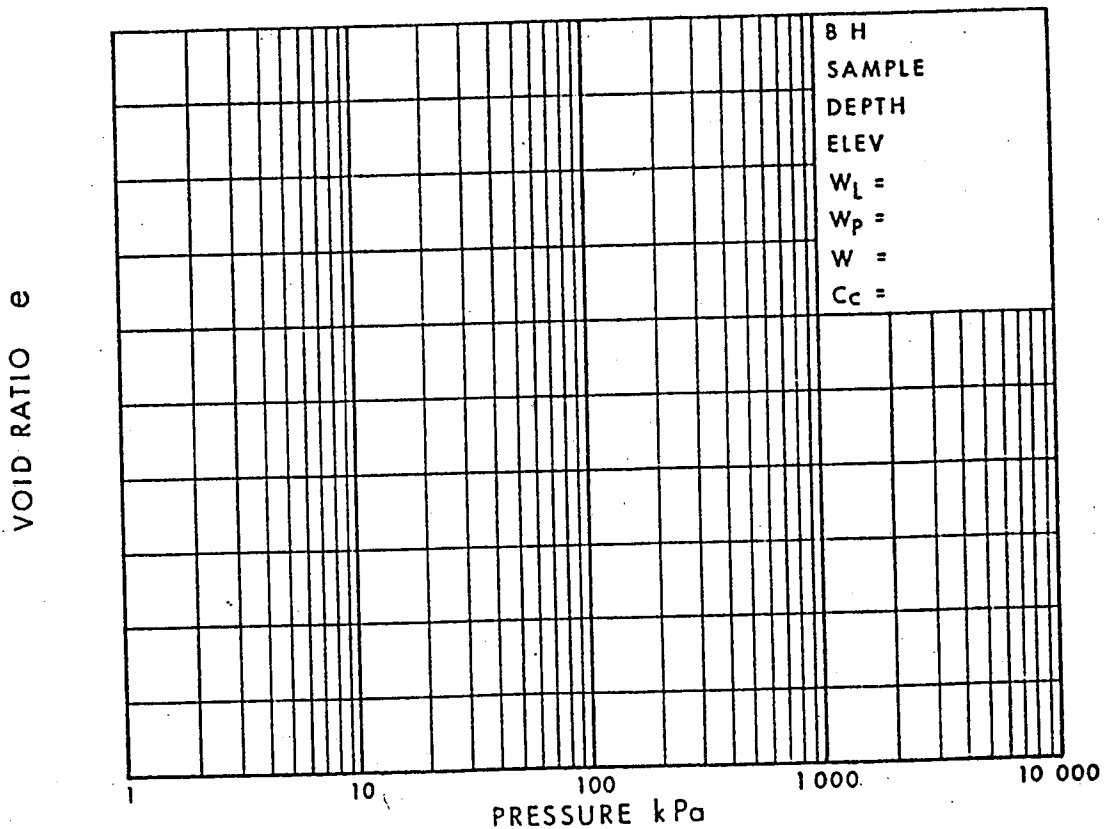
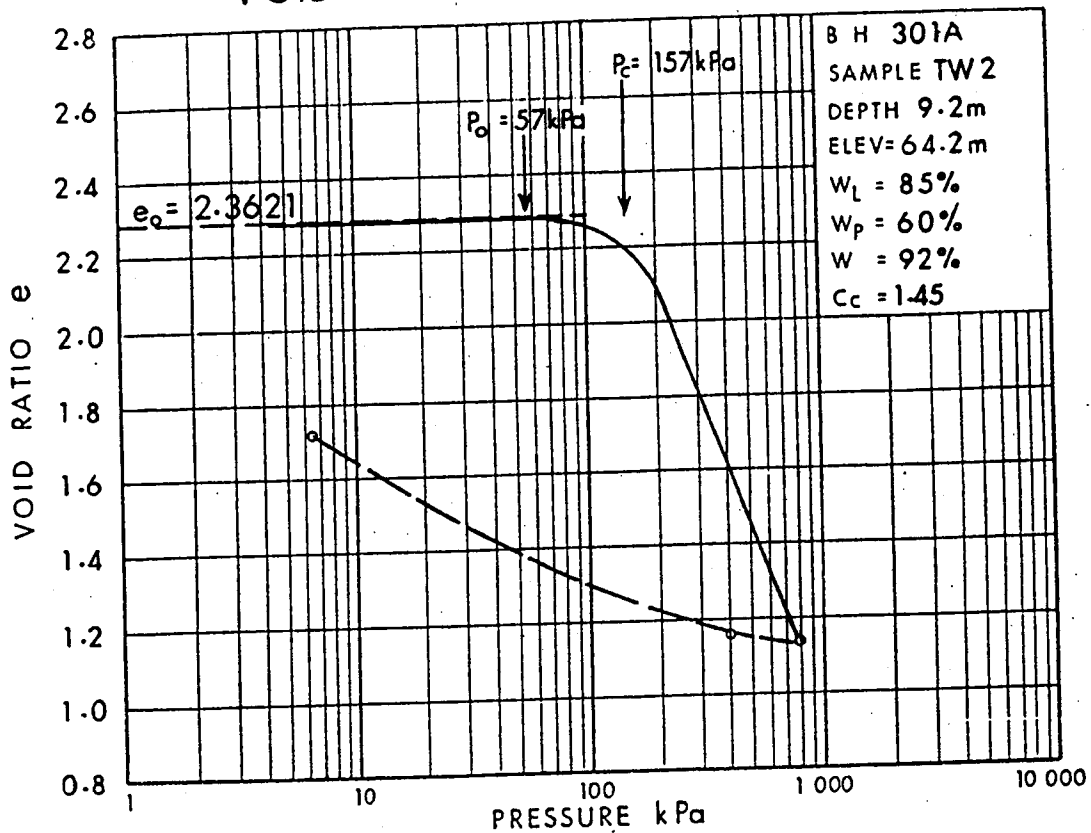


Fig 4b

W P 384-89-01

## UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY &amp; SILT

SAND

GRAVEL

Fine

Medium

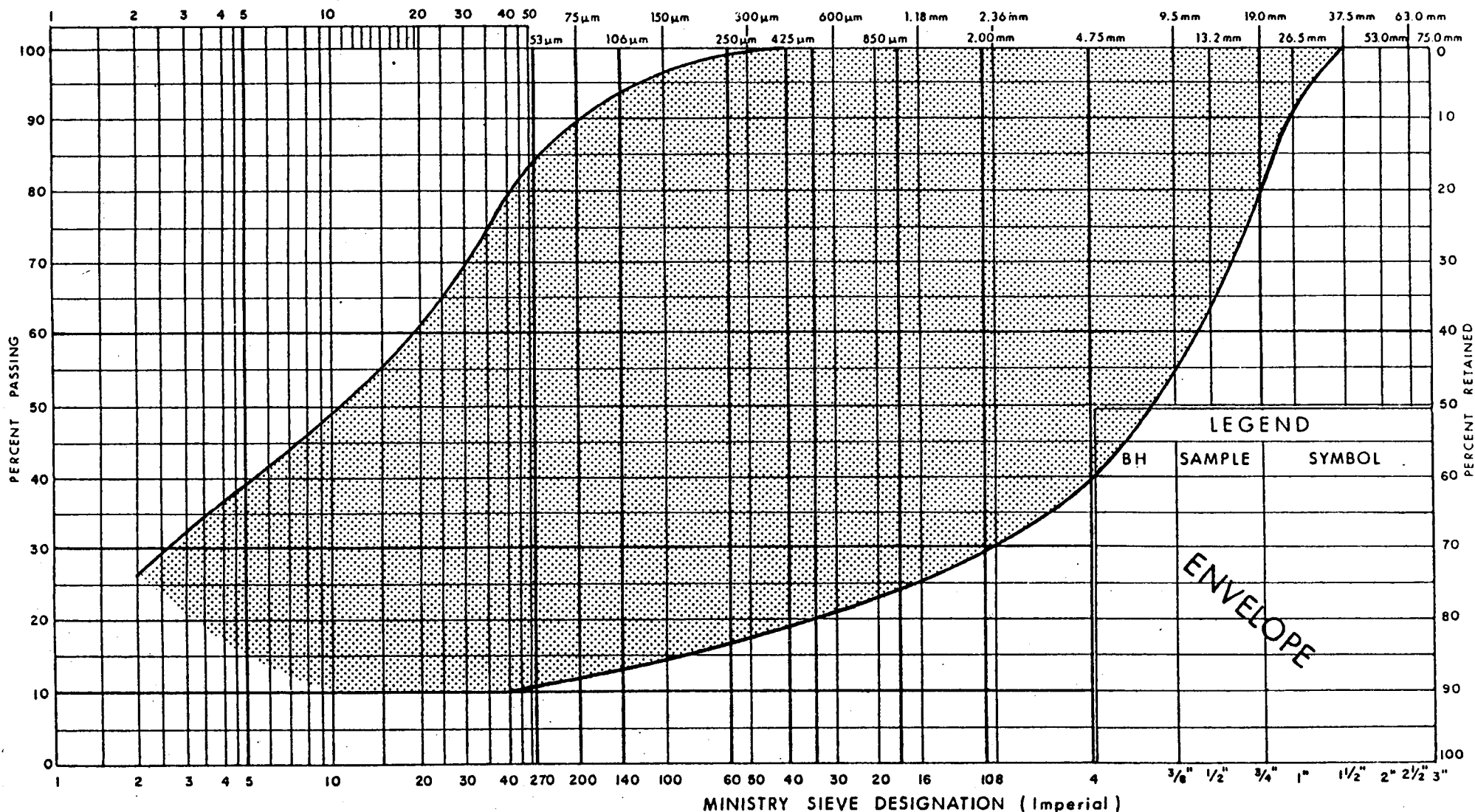
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

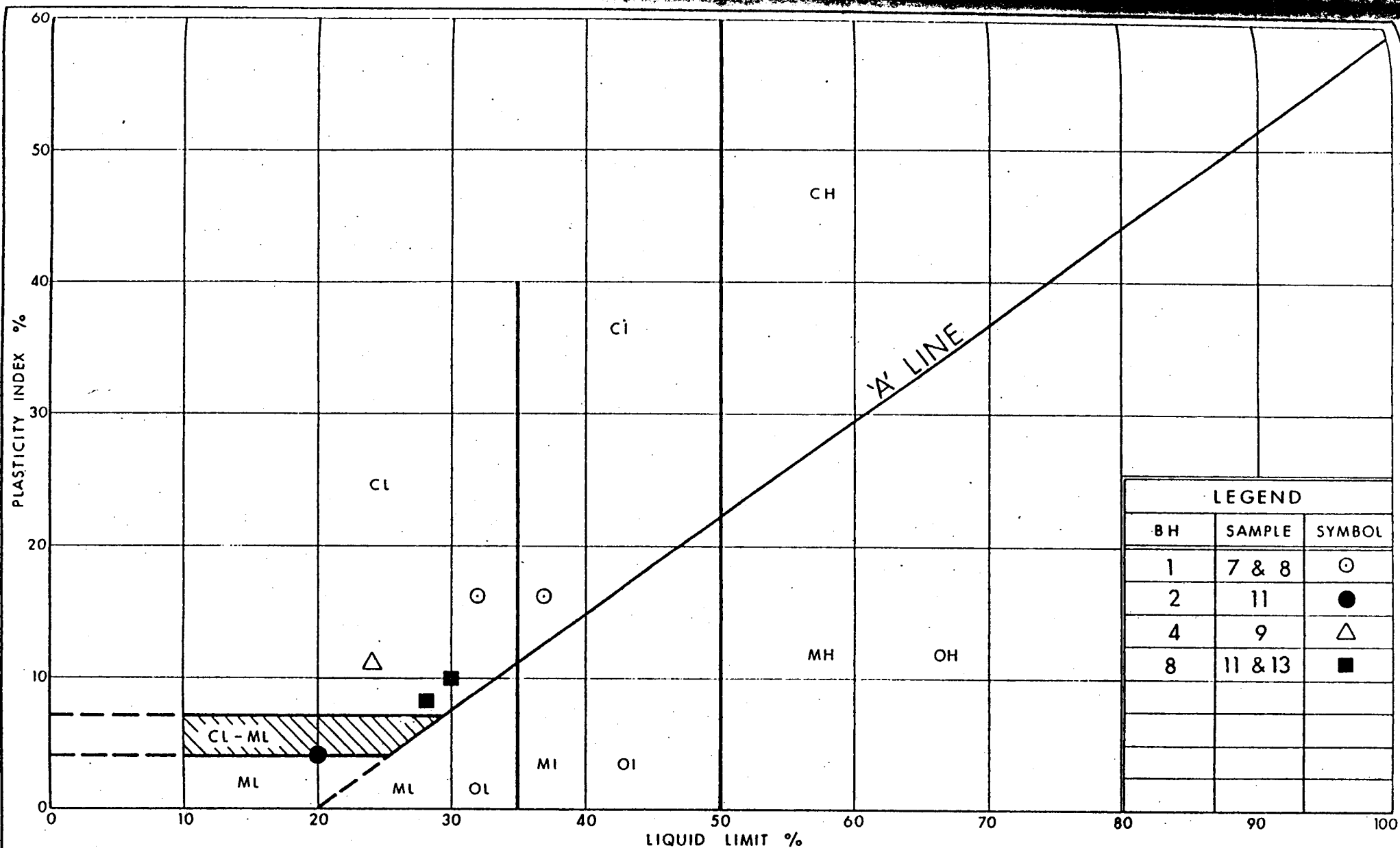


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Transportation

GRAIN SIZE DISTRIBUTION  
HET MIXTURE OF  
CLAYEY SILT, SAND & GRAVEL (Glacial Till)

FIG No 5

W P 384 -89-01



Ministry of  
Transportation

Ontario

# PLASTICITY CHART HET MIXTURE OF CLAYEY SILT, SAND & GRAVEL (Glacial Till)

FIG No 6

W P 384 - 89 - 01

**FIGURE 7**  
**APPROACH EMBANKMENT SETTLEMENT**  
**JORDAN HARBOUR(WP 384-89-01)**

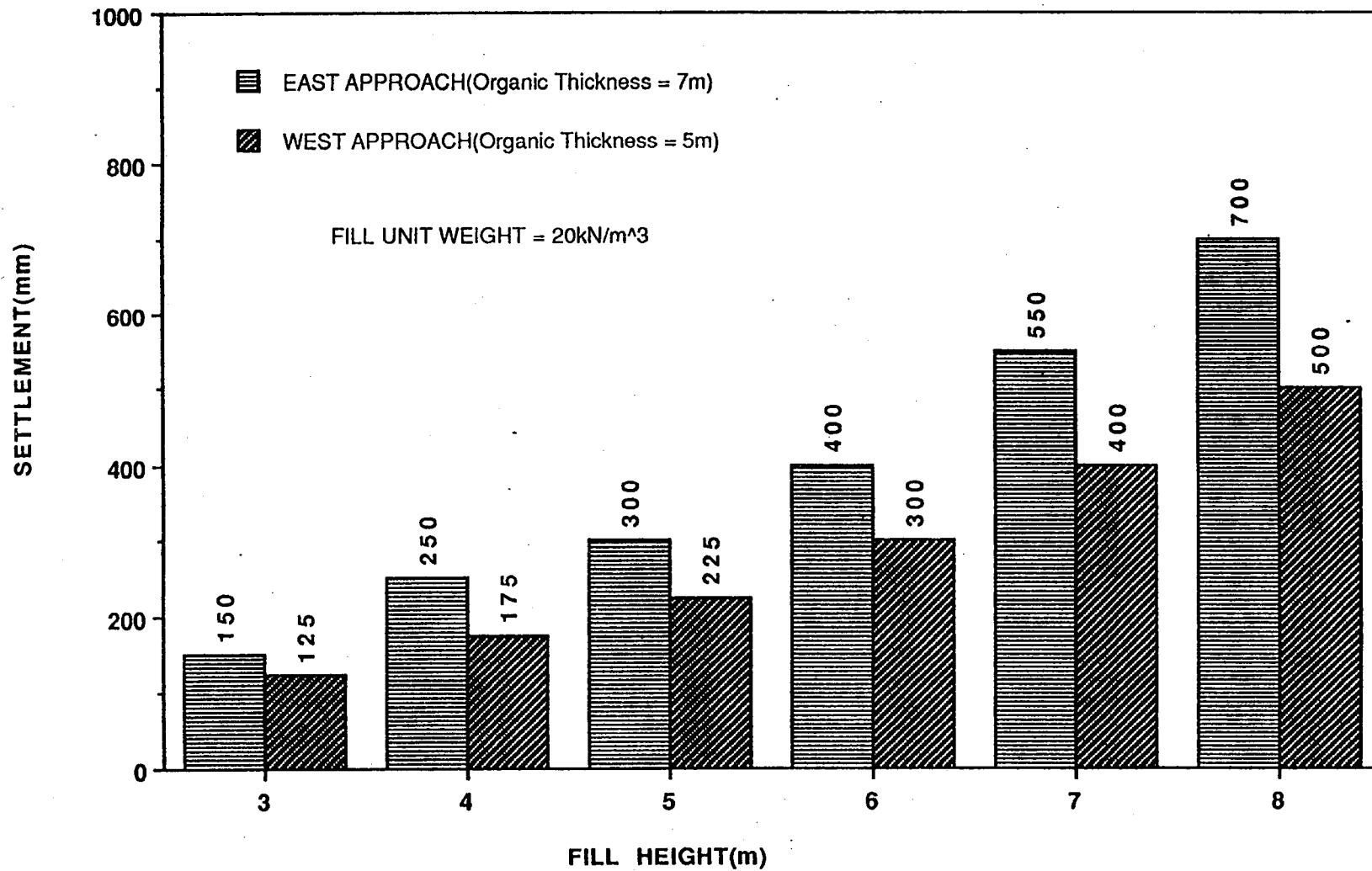
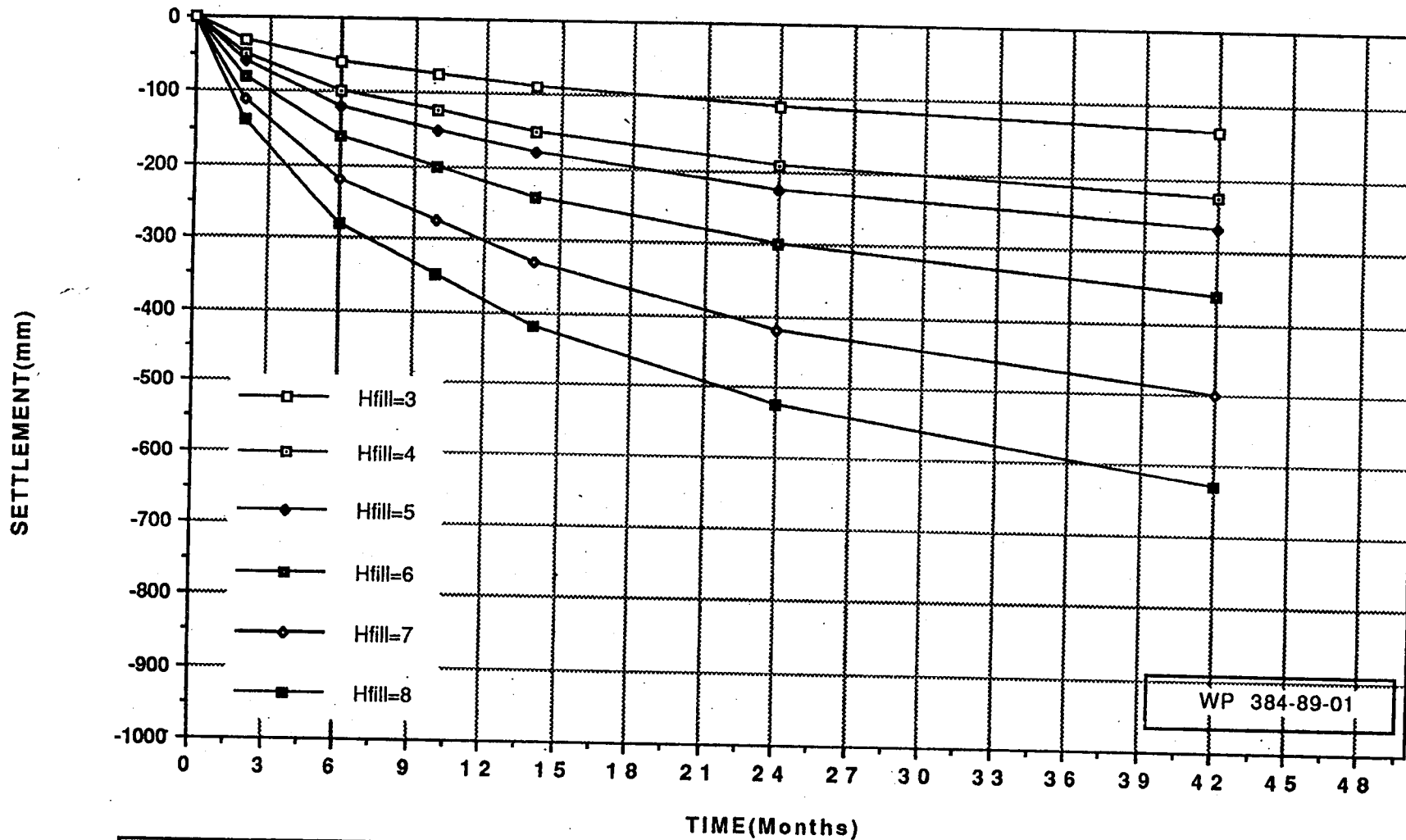
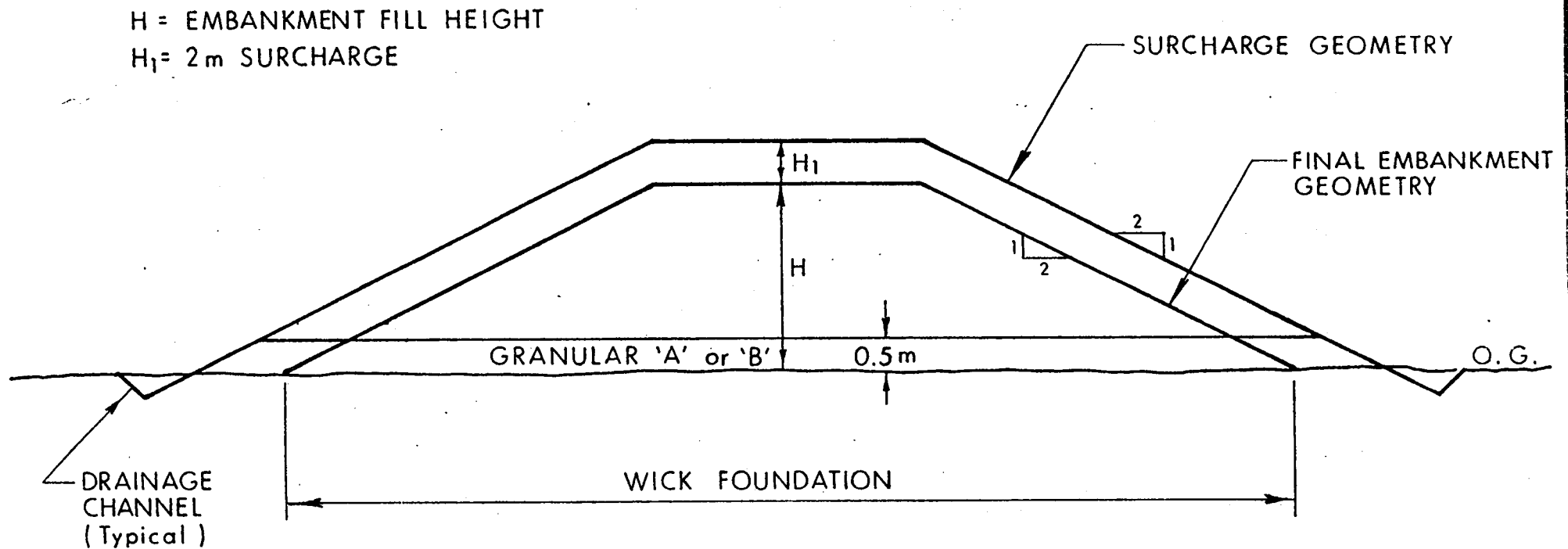


FIGURE 8a  
SETTLEMENT VS TIME - EAST APPROACH  
( $C_v = 3 \text{ m}^2/\text{yr}$ )



U = 20%    40%    50%    60%    75%    90%

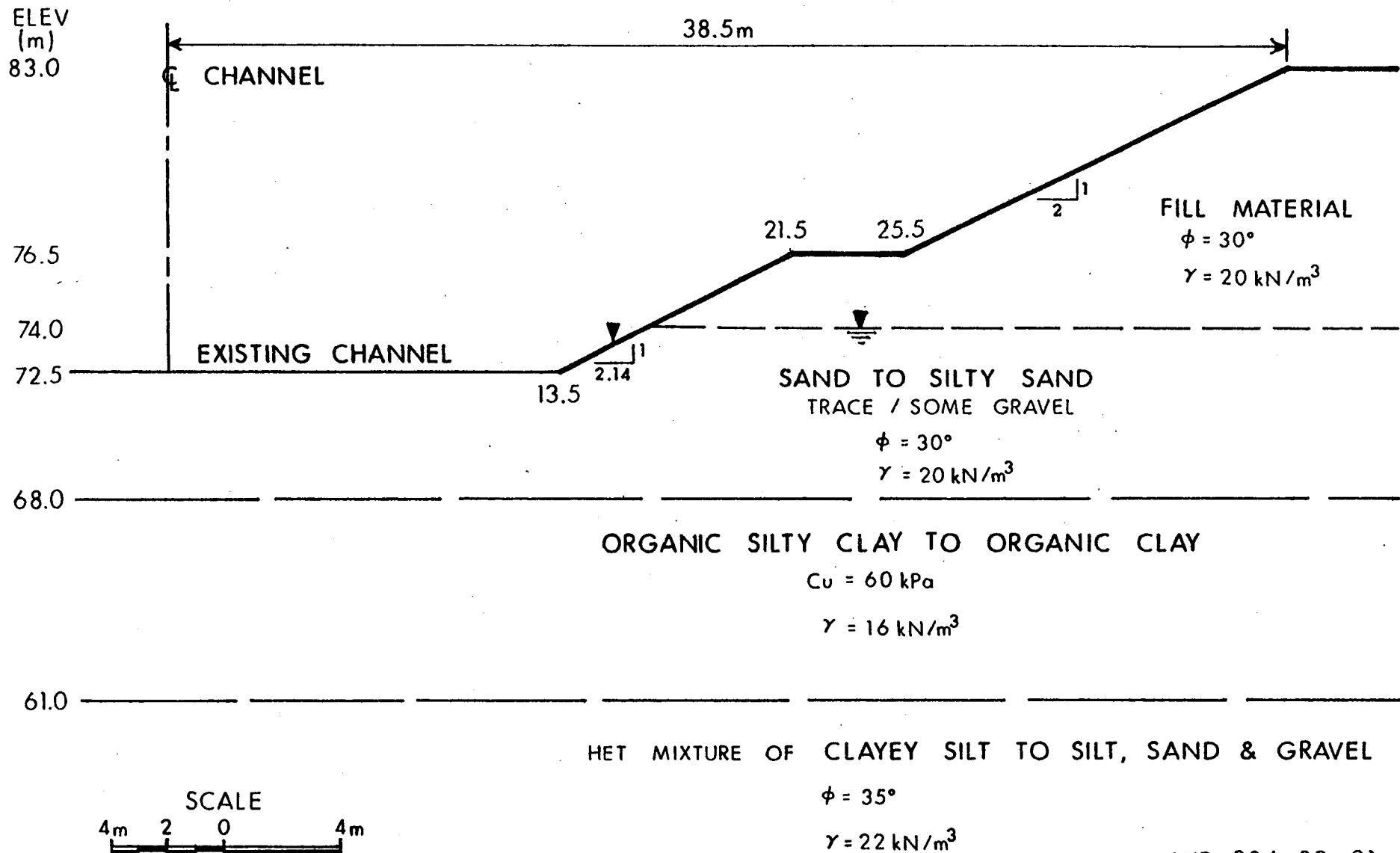
Figure 11 - WICK DRAIN DESIGN



NOT TO SCALE

WP. 384 - 89 - 01

Figure 12 - SLOPE STABILITY ANALYSES  
(APPROACH EMBANKMENTS)



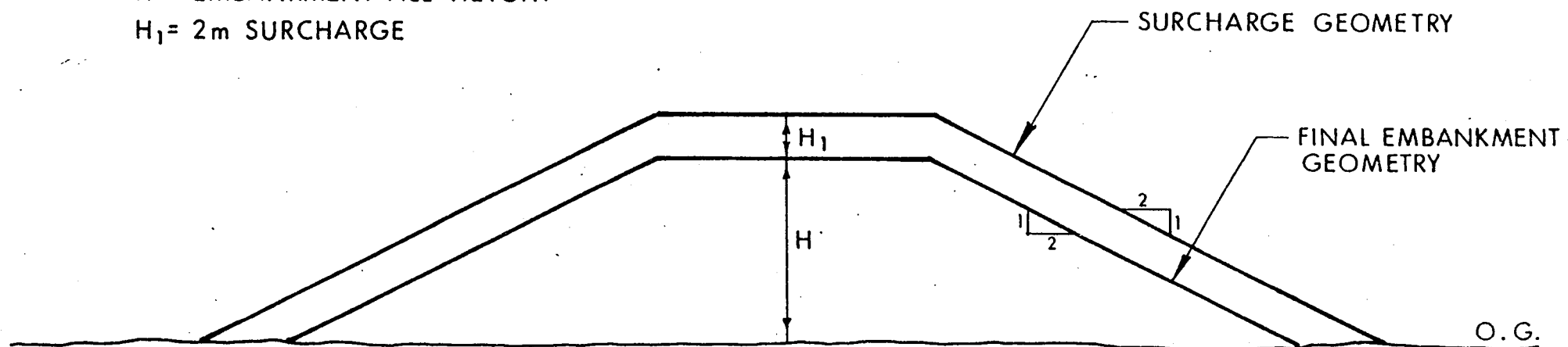
WP 384-89-01



Figure 10 - SURCHARGE LOAD GEOMETRY

H = EMBANKMENT FILL HEIGHT

$H_1 = 2\text{m}$  SURCHARGE



NOT TO SCALE

WP 384 - 89 - 01

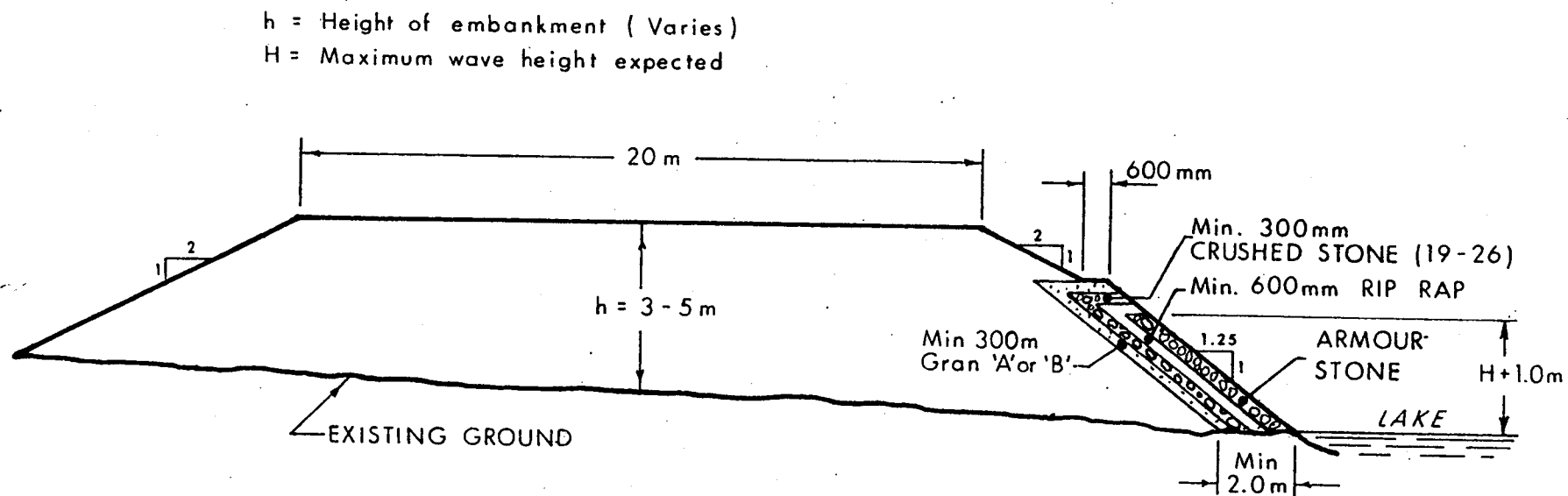
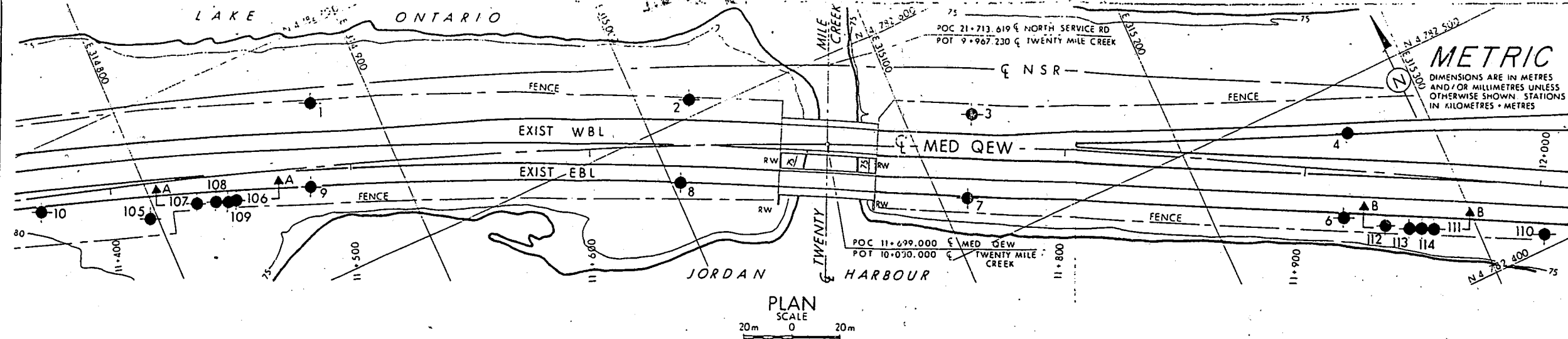


Figure 13 - SHORELINE PROTECTION SCHEME  
JORDAN HARBOUR



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MINISTRY OF TRANSPORTATION, ONTARIO P.D. 202 88 10

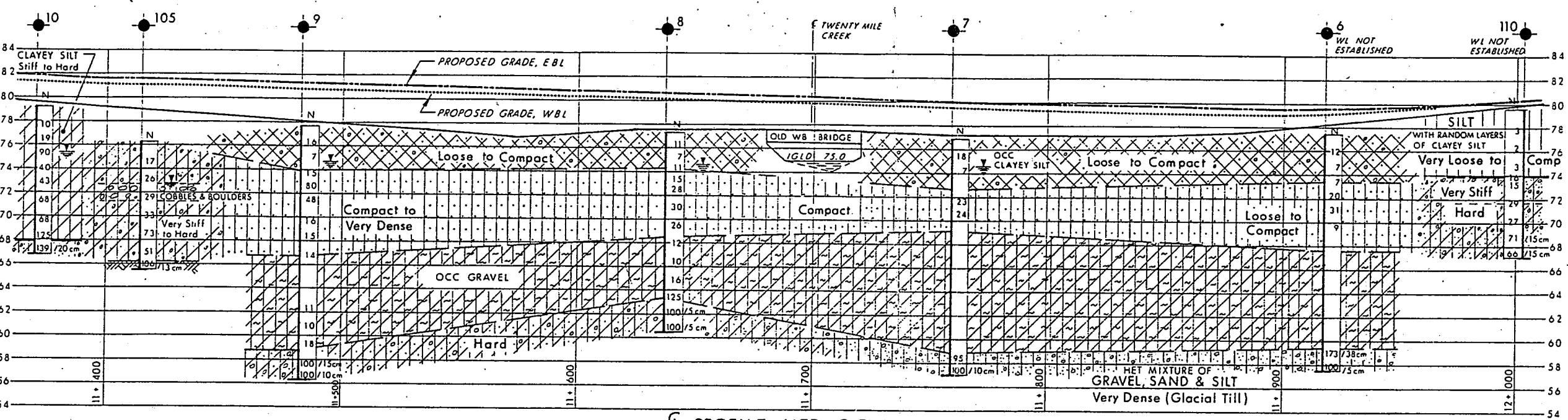
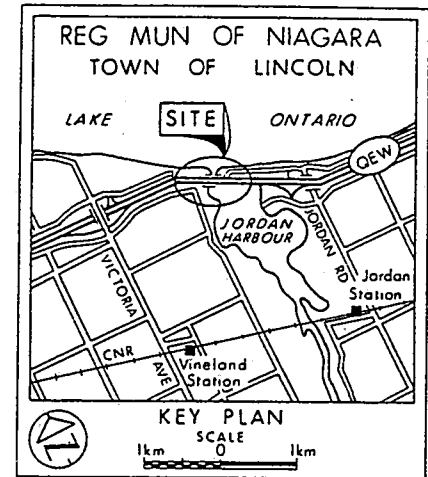


CONT No  
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JORDAN HARBOUR (TWENTY MI CR)

BORE HOLE LOCATIONS & SOIL STRATA

SHEET



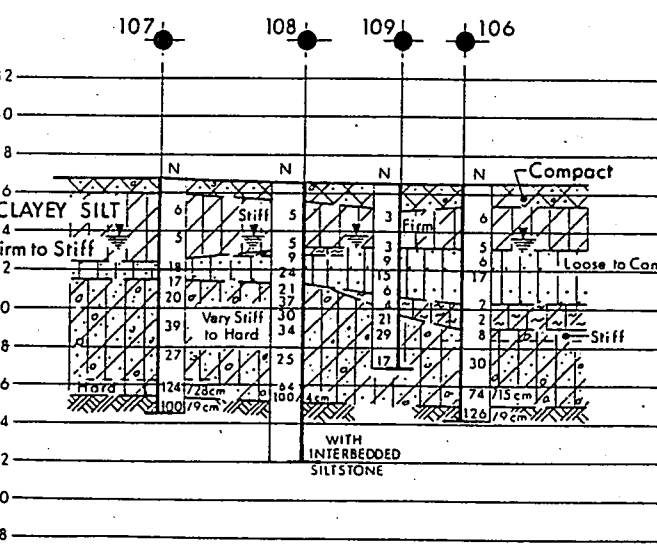
- LEGEND**
- Bore Hole
  - Dynamic Cone Penetration Test (Cone)
  - Bore Hole & Cone
  - N Blows/0.3m (Std Pen Test, 475 J/blow)
  - CONE Blows/0.3m (60° Cone, 475 J/blow)
  - WL at time of investigation
  - 1965 11, 1992 06 & 07, 1993 06 & 08

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	76.5	4 782 671.0	314 876.0
2	76.1	4 782 607.0	315 020.0
3	76.2	4 782 552.0	315 127.0
4	76.2	4 782 479.0	315 266.0
5	81.1	4 782 376.0	315 385.0
6	77.4	4 782 448.0	315 250.0
7	76.9	4 782 521.0	315 111.0
8	77.3	4 782 577.0	315 004.0
9	77.7	4 782 639.0	314 863.0
10	79.2	4 782 675.0	314 756.0
101	75.6	4 782 565.0	315 178.0
102	77.1	4 782 608.0	315 036.0
103	76.3	4 782 656.0	314 942.0
104	75.7	4 782 698.0	314 849.0
105	76.3	4 782 652.7	314 797.3
106	76.5	4 782 645.6	314 832.1
107	76.8	4 782 651.0	314 817.1
108	76.6	4 782 648.4	314 824.2
109	76.5	4 782 646.6	314 829.0
110	79.6	4 782 405.6	315 324.1
111	78.7	4 782 427.7	315 282.7
112	78.2	4 782 437.3	315 265.0
113	78.6	4 782 432.5	315 273.9
114	78.7	4 782 430.5	315 278.2
301A	75.6	4 782 565.0	315 179.5
301B	75.6	4 782 565.0	315 180.0
302A	76.3	4 782 654.0	314 946.0
302B	76.3	4 782 655.0	314 943.0

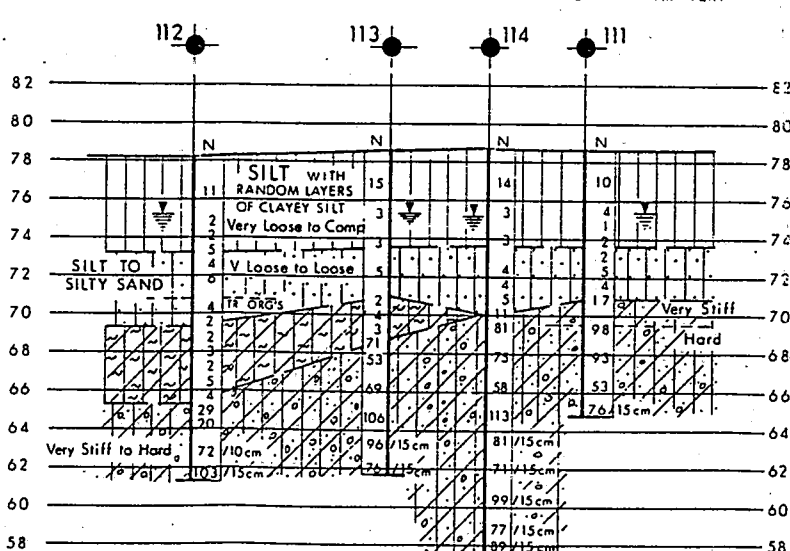
**NOTE**  
For subsoil information of BH's 1, 2, 3, 4, 5, 101, 102, 103, 104, 301A, 301B, 302A, & 302B refer to Record of Borehole Sheets.

**SOIL STRATIGRAPHY LEGEND**

- SILTY SAND TRACE WITH GRAVEL (FILL MATERIAL)
- SAND TO SILTY SAND OCCASIONAL GRAVEL
- ORGANIC SILTY CLAY TO CLAY Firm to Very Stiff
- HETEROGENEOUS MIXTURE OF CLAYEY SILT, SAND & GRAVEL (Glacial Till)
- SHALE BEDROCK Weathered



SECTION A-A



SECTION B-B

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

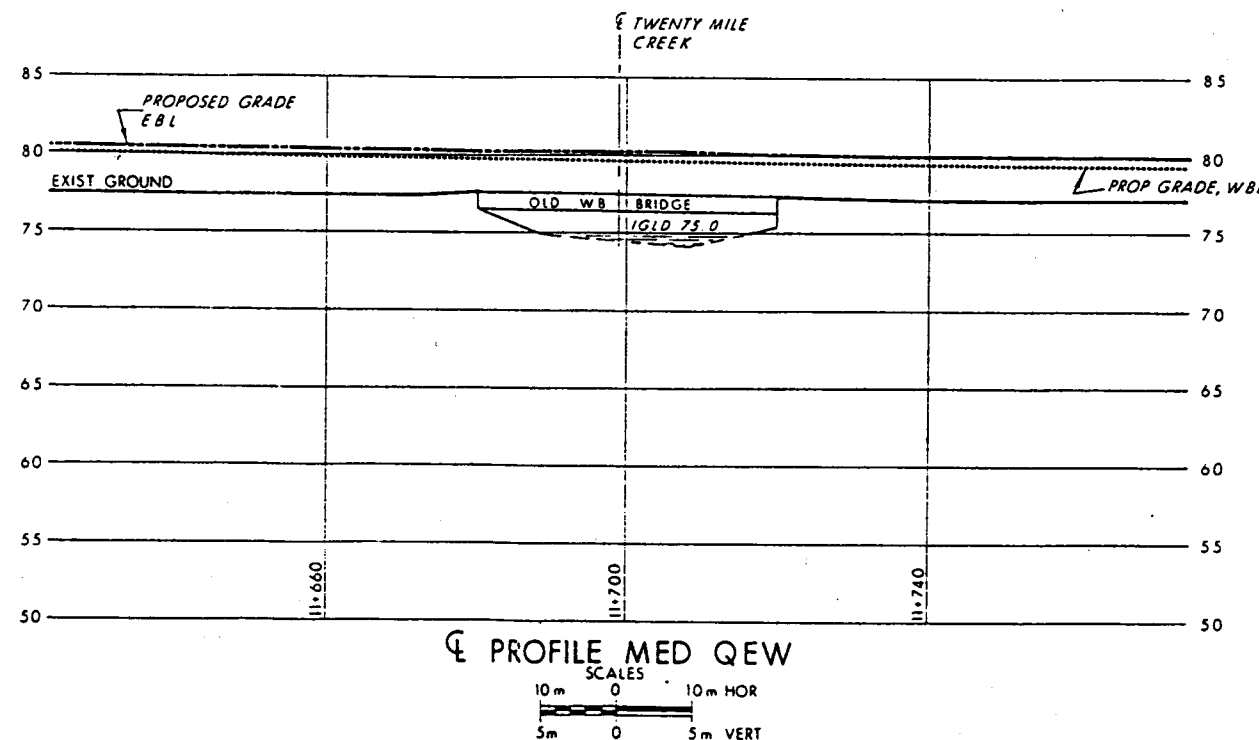
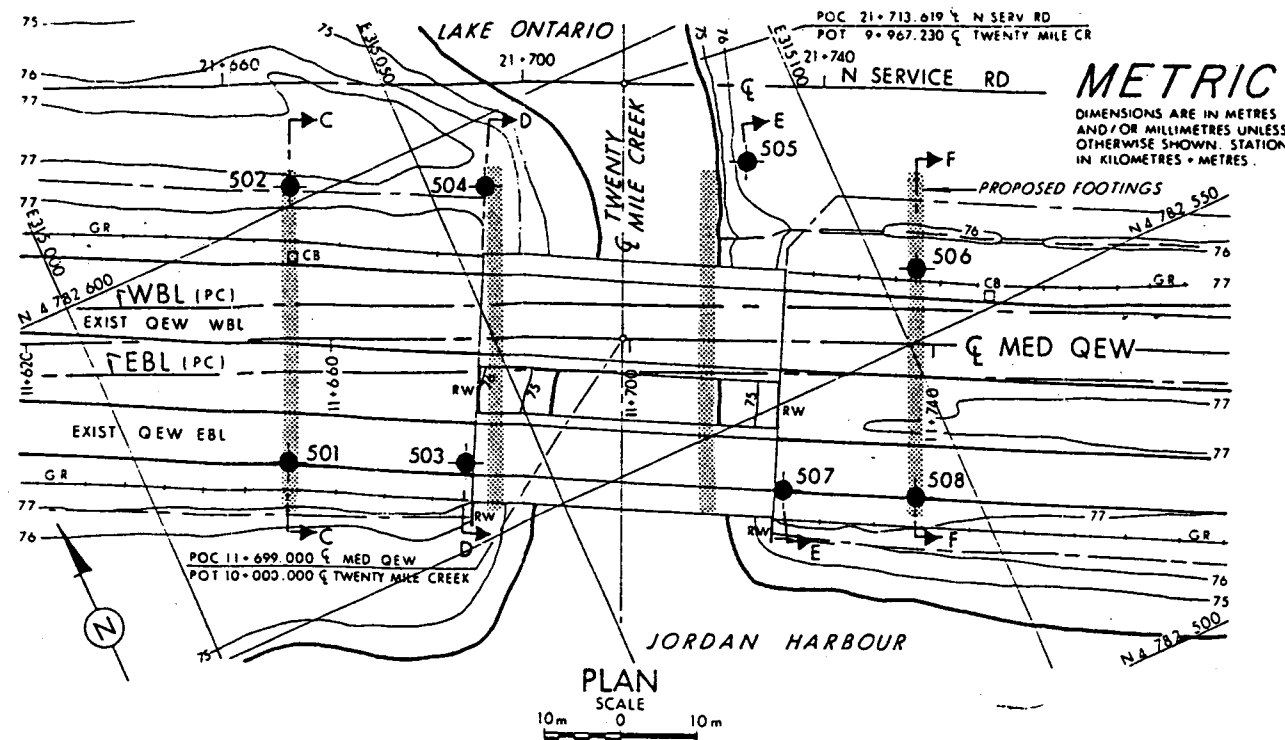
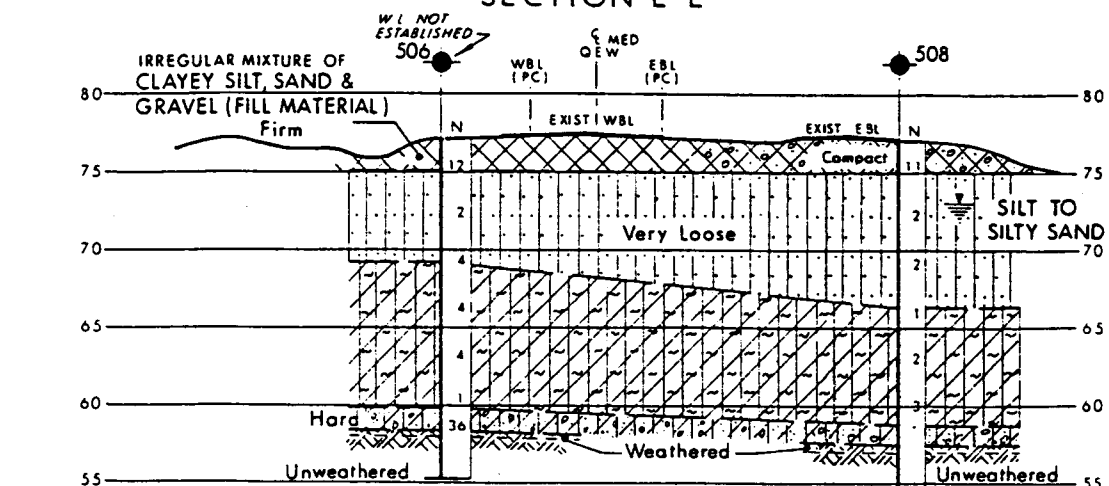
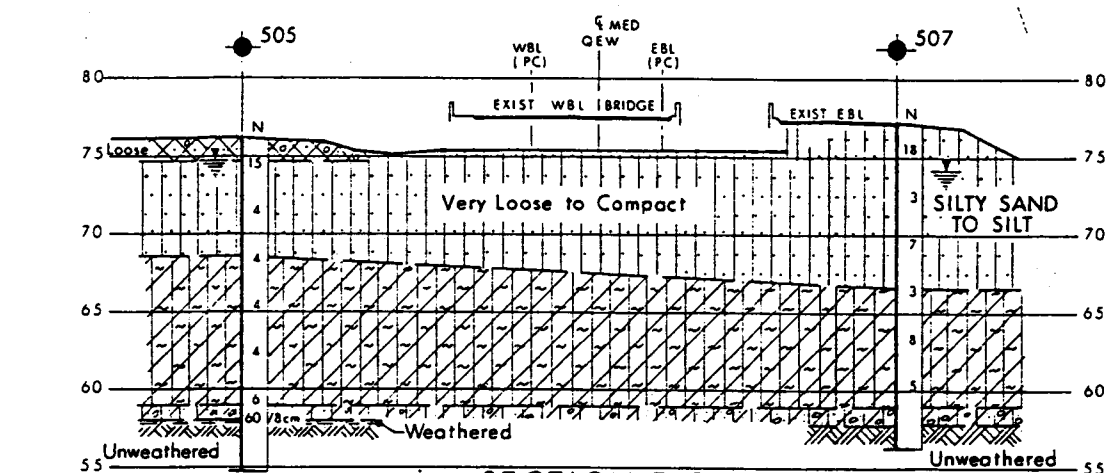
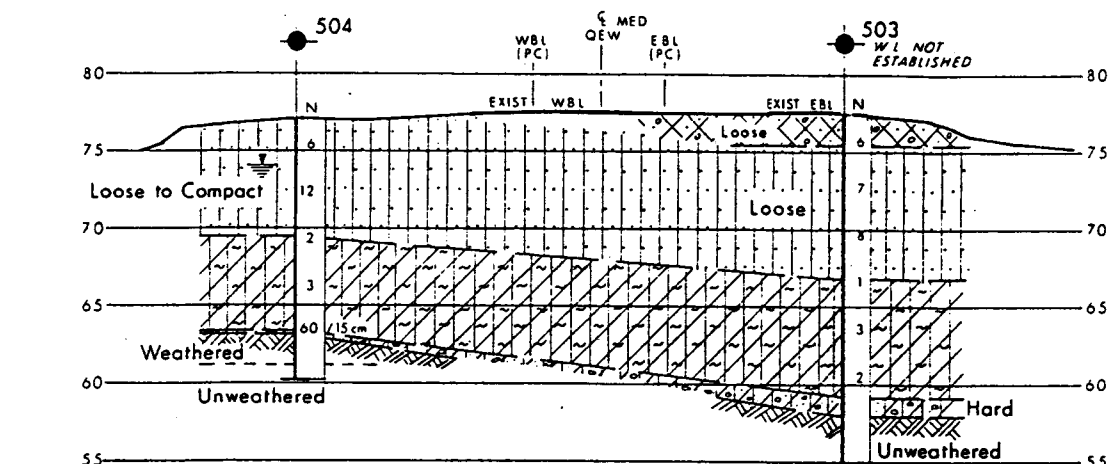
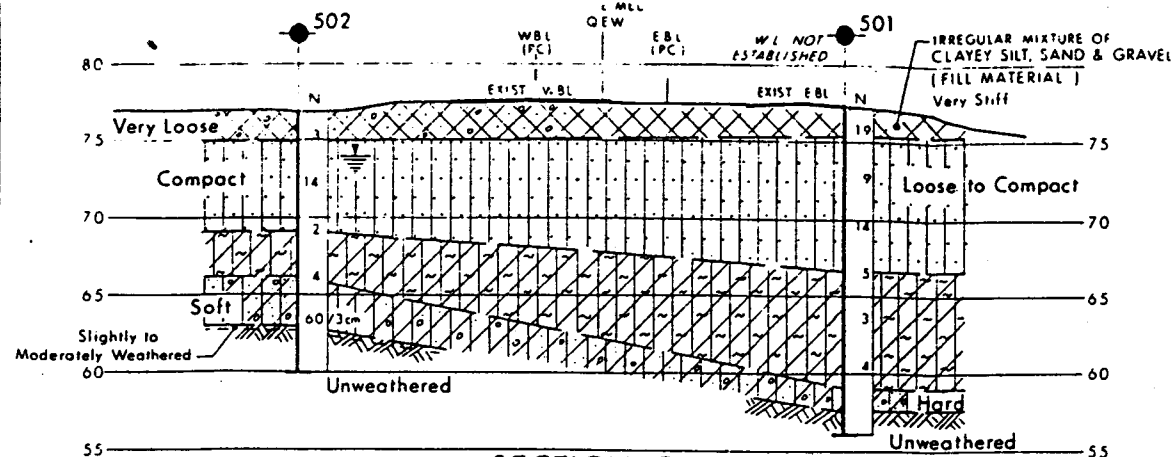
**NOTE:** The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section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

REV	DATE	BY	DESCRIPTION

Geocres No 30M3-200

HWY No QEW (WBL & EBL)	DIST 4
SUBMITTS (CHECKED) DATE 1994 08 05	SITE 18 - 19





#### SOIL STRATIGRAPHY LEGEND

- IRREGULAR MIXTURE OF SILT, SAND & GRAVEL (FILL MATERIAL)
- ORGANIC SILTY CLAY TO CLAY Stiff to Very Stiff
- SHALE BEDROCK WITH INTERBEDDED SILTSTONE
- SAND TO SILTY SAND TRACE/SOME GRAVEL
- HETEROGENEOUS MIXTURE OF CLAYEY SILT, SAND & GRAVEL (Glacial Till)

CONT No  
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JORDAN HARBOUR (TWENTY MI CR)

BORE HOLE LOCATIONS & SOIL STRATA

SHEET

SEE DWG 3848901-B

KEY PLAN  
SCALE

LEGEND			
	Bore Hole		
	Dynamic Cone Penetration Test (Cone)		
	Bore Hole & Cone		
N	Blows/0.3m (3rd Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60° Cone, 475 J/blow)		
	W.L. at time of investigation 1994 01 and 02		
No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
501	77.4	4 782 570.0	3 15 018.4
502	76.9	4 782 602.0	3 15 033.0
503	77.4	4 782 560.3	3 15 040.0
504	77.1	4 782 591.6	3 15 057.0
505	76.2	4 782 580.0	3 15 090.0
506	77.1	4 782 558.8	3 15 104.8
507	77.3	4 782 539.5	3 15 077.0
508	77.0	4 782 531.5	3 15 092.5

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

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DATE	BY	DESCRIPTION

Geocres No 30 M3-200

HWY No	QEW (WBL & EBL)	DIST
401	401	4

SUBMITTAL CHECKED: DATE 1994 08 05 SITE 18-19

DRAWN R.S. (CHECKED) DATE 1994 08 05 DWG 3848901-A