

ENGINEERING MATERIALS OFFICE  
FOUNDATION DESIGN SECTION

*CONT 9A-55*

WP 65-67-05 DIST 4  
HWY 403 STR SITE 36-261

Hwy. 403 and Alberton Road Underpass

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FOUNDATION INVESTIGATION REPORT  
For  
Hwy. 403 and Alberton Road Underpass  
W.P. 65-67-05, Site 36-261  
District 4, Burlington

INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the abovementioned site. It is proposed to construct a two span structure that will carry the existing Alberton Road along its present alignment over the proposed Hwy. 403.

SITE DESCRIPTION AND GEOLOGY

The site is located along the existing Alberton Road approximately 1 km north of Hwy. 2 and 3 km west of Hwy. 52 within the Town of Ancaster, Regional Municipality of Hamilton-Wentworth.

The existing Alberton Road is a two lane asphaltic roadway with narrow gravel shoulders. The shoulders, in turn, are adjoined by a flat lying grassland. The gravel shoulders were at some locations also covered with grass and weeds.

The terrain at the site has a gently rolling and hilly topography. Ground surface elevations hence vary significantly. Elevations immediately west of the proposed structure are approximately 1 to 2 m higher and similarly elevations east of the structure are approximately 1 m lower in elevation. Land use in the area is predominantly agricultural farmland and both corn and barley were being cultivated at the time of the investigation. In addition, a milk holstein, evidence, of dairy farming is located approximately 0.5 km north of the site and some forestland exists approximately 1 km east of the site beyond the abovementioned agricultural farmland.

Physiographically, the site is located within the geological domain known as the Haldimand clay plain. The Haldimand clay plain occupies the area lying between the Niagara Escarpment and Lake Erie. The entire area was submerged in Lake Warren, a glacial lake formed during the retreat of the Wisconsin glaciation (approximately 12,000 years ago). Lacustrine clays and silts were deposited as the lake gradually receded due to the deposition of sediments during isostatic land rebound.

Drainage of this belt is controlled by the Grand River which has cut a deep valley in the clay and silt. Consequently, there has been much dissection by tributary drainage.

The underlying bedrock at the site consists of weak to medium strong dolostones of the Paleozoic era. At the site, the overburden has a thickness of approximately 28 to 29 m.

#### INVESTIGATION PROCEDURES

Soil and rock data and inherent properties were obtained by conducting both an in situ field investigation and laboratory analyses. Details of the field investigation and laboratory testing program are discussed below.

##### Field Investigation

The fieldwork for this project was carried out under two separate stages. The initial stage, was implemented between 76 03 24-25 inclusive and consisted of one sampled borehole (BH 5, formerly BH 12) advanced to a depth of 29.7 m and one dynamic cone penetration test advanced to a depth of 12.8 m. This initial first stage provided information to facilitate the planning and design of the proposed Hwy. 403 route between the Towns of Ancaster and Brantford.

The second stage of the fieldwork was carried out between 91 06 24-27 inclusive and consisted of a total of four sampled boreholes reflecting the greater extent of this subsequent fieldwork. The scope of the second stage of the fieldwork was to provide detailed soils/rock data to facilitate the design and construction of the proposed structure and related earthworks. Two (2)

boreholes were advanced at the proposed abutment structure foundation locations and one borehole was advanced at either proposed approach fills north and south of the structure. The boreholes at the structures were advanced to depths of 31.2 m (BH 1) and 30.8 m (BH 2) and were accompanied by dynamic cone penetration tests driven to depths ranging from 7.9 m to 9.8 m. The approach fill boreholes were advanced to depths ranging from 12.6 m to 20.7 m.

The boreholes were advanced through the overburden using track mounted boring units employing conventional continuous flight hollow stem augering techniques. Conventional rock coring techniques were used to retrieve 3 m of rock core at BH 1 and 1.5 m at BH 2.

In general, subsoil samples were retrieved at 0.7 m intervals for the surficial 4.6 m to 6.0 m, at 1.5 m intervals beyond this surficial depth to approximately 15 m and at 3 m intervals thereafter. Subsoil samples were taken at 1.5 m intervals at the approach locations.

Subsoil samples were generally retrieved in accordance with the Standard Penetration Test (ASTM D1586) and hence were disturbed during the testing. Relatively undisturbed subsoil samples were randomly retrieved in cohesive soils using a thin wall sampler in accordance with procedures outlined in ASTM D 1587. All subsoil samples were identified in the field and then properly sealed to preserve natural moisture contents. Disturbed samples were placed in plastic containers and the thin wall samples were capped and waxed at the ends of the Shelby tubes. The samples were then transported to the laboratory where additional visual classifications were carried out and pertinent laboratory tests were conducted as described in the next section below.

Rock core samples were also identified in the field and rock recoveries and Rock Quality Designations (RQD) measured. The rock cores were then transported to the laboratory in conventional core boxes and detailed rock core logs were produced by an in-house geologist.

In situ vane tests were also carried out to determine the undrained shear strength of the weaker cohesive soils at the site. The test was carried out in accordance with ASTM D2573 using the Standard MTO 'N' vane. Remoulded shear strengths were also obtained where applicable.

Groundwater levels were obtained by monitoring the levels in the open boreholes throughout the duration of the field investigation. All open boreholes were backfilled at the completion of the fieldwork.

Survey information related to the location and elevation of boreholes was provided by Central Region Surveys and Plans.

### Laboratory Analyses

To determine the pertinent physical properties and behaviour of the soil, various laboratory tests were conducted. These tests included:

- 1) Natural Moisture Contents
- 2) Atterberg Limits
- 3) Grain Size Analyses
- 4) Bulk Unit Weights
- 5) Consolidation Tests

Laboratory test results are shown on the borehole logs and corresponding figures attached in the Appendix. These results have been summarized in the subsequent section of this report entitled "Subsurface Conditions".

### SUBSURFACE CONDITIONS

#### General

The ground surface elevation at the boreholes advanced at the site ranged from 221.3 m to 222.1 m.

The subsurface conditions are generally uniform across the site and underlying a sand and gravel roadway base fill material approximately 0.8 m in thickness consists of three (3) distinct soils strata. The surficial native deposit extending to depths ranging from 5 to 6.6 m below the ground surface consists of a stiff to very stiff clayey silt to silty clay material with occasional seams of silt. This stratum is underlain by an extensive deposit of plastic silt interbedded with layers of clayey silt. The clayey silt interbeds range in

thickness from approximately 25 mm to 2 m. The silt material is in a loose to compact state of denseness and the clayey silt layers are generally stiff to very stiff in consistency. This deposit has a thickness ranging from 19.1 m to 20.9 m. A third soil stratum consisting of a clayey silt with interbedded layers of silt underlies the silt deposit with interbeds of clayey silt and extends to the bedrock surface. The thickness of this stratum ranges from 3.4 m to 3.8 m. This stratum has a stiff to very stiff consistency. The bedrock surface exists at an elevation ranging from 191.7 m to 193.1 m.

A plan of the site illustrating the locations and elevations of the boreholes is shown on Dwg. 656705-A in the Appendix. A subsoil stratigraphical profile illustrating the subsurface conditions at the site is 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 also on the individual Record of Borehole sheets in the Appendix.

#### Sand and Gravel (Fill Material)

A cohesionless sand and gravel exists as the base of the existing Alberton Roadway and has a thickness in the order of 0.8 m. This bedding material is brown in colour and appears to be in a compact state of denseness.

#### Clayey Silt to Silty Clay with random seams of Silt

The surficial native stratum at the site consists of a cohesive clayey silt to silty clay material that contains random seams of interbedded silt. The stratum which is unstructured and without any distinct layering has a thickness ranging 4.2 to 5.8 m.

The stratum has been oxidized to a penetration depth of approximately 3 to 4.6 m beneath the existing ground surface and hence is brown in colour within this depth. Beneath this depth, the stratum is unoxidized and grey in colour.

A grain size distribution envelope produced by mechanical sieve and hydrometer analyses is shown on Figure 1 in the Appendix. The envelope clearly illustrates that the stratum is composed of grain sizes smaller than 75 micrometres.

The grain size distribution envelope for this material illustrates large percentages of silt, ranging from 55% to 82% and clay percentages ranging from 15% to 45%. A grain size distribution curve representing the gradation of a typical silt seam is also shown on Figure 1.

In view of the fact that more than 80% of the material is finer than 75 micrometres, the soil is categorized according to its behaviour in accordance with the MTO Soil Classification Manual. Atterberg Limit Tests were hence conducted to define the behaviour and plasticity of the soil. The results are plotted on Figure 2 in the Appendix and summarized on Table 1 below. Natural moisture contents and the bulk unit weight of the soil have also been included in the table.

Table 1 - Atterberg Limit Test Results

	<u>Range</u>	<u># of Tests</u>
Natural Moisture Content (w%)	22-27	8
Liquid Limit ( $w_L$ %)	24-40	8
Plastic Limit ( $w_p$ %)	17-21	8
Plasticity Index ( $I_p$ %)	5-21	8
Bulk Unit Weight ( $kN/m^3$ )	19.6-20.3	2

The test results clearly reveal that the soil has a plasticity ranging from low to intermediate and hence can be classified as a clayey silt to silty clay with seams of plastic silt. Natural Moisture Contents are generally within the plastic and liquid limits of the soil and hence the soil is in a plastic state.

The consistency and undrained shear strength of the soil was determined based on the evaluation of the 'N' values derived from the Standard Penetration Tests and by conducting in situ vane tests. The 'N' values ranged from 7 blows/0.3 m to 24 blows/0.3 m and averaged 15 blows/0.3 m based on twenty-two (22) Standard Penetration Tests. In situ vane tests were also attempted within the weaker zones of the stratum. In generally, the vanes could not be torqued within the limitations of the test and hence it can be concluded from this observation that the material has undrained shear strengths exceeding 100 kPa and hence the soil has a stiff to very stiff consistency.

### Silt with interbedded layers of Clayey Silt

An extensive cohesionless deposit of plastic silt interbedded with layers of clayey silt underlies the surficial native clayey silt to silty clay with interbedded seams stratum at the site. The surface of this deposit is present at an elevation ranging from 214.5 m to 217.1 m and the deposit has a thickness ranging from 19.1 m to 20.9 m.

The deposit has a distinct layered structure with predominant silt thickness interbedded with layers or seams of cohesive clayey silt ranging in thickness from 25 mm to 2 m. The silt material is light grey in colour whilst the clayey silt is dark grey in colour. The silt material exhibits a quick dilatancy. Grain size distribution envelopes representing the gradation of the silt material as well as the clayey silt interlayers are shown on Figure 3 in the Appendix.

Atterberg Limit Tests were also carried out to determine the behaviour and plasticity of the cohesive interbeds and also to verify any plasticity in the silty material. The results of these tests are shown on Figure 4 in the Appendix. The results reveal that this silt deposit is a plastic material and the cohesive interbeds are of low plasticity. These results are also summarized in Table 2 below.

Table 2 - Atterberg Limits

<u>Silt</u>		
	<u>Range</u>	<u># of Tests</u>
Natural Moisture Content (w%)	26-28	8
Liquid Limit (w <sub>L</sub> %)	20-26	8
Plastic Limit (w <sub>p</sub> %)	16-20	8
Plasticity Index (I <sub>p</sub> %)	1-6	8
<u>Clayey Silt</u>		
Natural Moisture Content (w%)	20-30	6
Liquid Limit (w <sub>L</sub> %)	23-35	6
Plastic Limit (w <sub>p</sub> %)	14-19	6
Plasticity Index (I <sub>p</sub> %)	7-21	6



The compressibility characteristics of the clayey silt interbeds was determined by conducting a one dimensional consolidation test on a representative sample. The results of the test (e-log curve) is illustrated on Figure 5 in the Appendix. The results reveal that the material has been preconsolidated in the past to an effective pressure approximately 500 kPa in excess of the existing effective overburden pressure. The compression index of the material ( $C_c$ ) was of small magnitude and equivalent to approximately 0.4.

Some borehole sloughing was observed during the penetration and sampling within this deposit. This soil cave-in can be attributed to unbalanced hydrostatic head.

The denseness of the silt material was determined by evaluating the 'N' values derived by the Standard Penetration Test. Based on 'N' values ranging from 7 blows/0.3 m to 30 blows/0.3 m, the silt material has a denseness ranging from loose to compact. In view of the influence of unbalanced hydrostatic head conditions in producing lower 'N' values, the denseness of the silt material can be described as compact.

The consistency of the clayey silt interbeds was determined by attempting to conduct in situ vane tests. In many cases, the presence of the silt material confining the clayey silt layers inhibited the execution of the test. However, vane tests were successfully executed in the thicker layers of clayey silt. Undrained shear strengths measured ranged from approximately 60 kPa to in excess of 120 kPa. Based on this information, the cohesive clayey silt can be described as having a stiff to very stiff consistency.

#### Clayey Silt with layers of Silt

Underlying the silt with interbedded layers of clayey silt stratum a deposit consisting of a clayey silt with layers of silt extending to the bedrock surface exists. The surface of the deposit exists at an elevation ranging from 195.3 m to 196.9 m and the deposit has a thickness in the order of magnitude of 3.4 m to 3.8 m. The silt layers are typically 50 mm to 100 mm in thickness.

A grain size distribution curve illustrating the gradation of a representative sample of this material is shown on Figure 6 in the Appendix. The curve illustrates that the material is fine grained with grain sizes less than 75 micrometres.

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 7 and summarized in Table 3 below. Natural Moisture Contents are also included in the table below.

Table 3 - Clayey Silt to Silty Clay

	<u>Range</u>	<u># of Tests</u>
Natural Moisture Content (w%)	28-30	2
Liquid Limit (w <sub>L</sub> %)	28-29	2
Plastic Limit (w <sub>p</sub> %)	18-19	2
Plasticity Index (I <sub>p</sub> %)	10-11	2

The test results reveal that the soil has a low plasticity and hence can be classified as a clayey silt. Natural moisture contents are approximately equivalent to the liquid limit of the soil revealing liquidity indices (I<sub>L</sub>) exceeding unity.

The 'N' values as derived from the Standard Penetration Test reveal values ranging from 8 blows/0.3 m to 14 blows/0.3 m. Based on these 'N' values the consistency of the soil is estimated as stiff to very stiff.

#### Bedrock

The bedrock consists of dolostone of the Amabel Formation and underlies the clayey silt with layers of silt deposit at an elevation of approximately 191.7 m to 193.1 m. The bedrock was cored in BQ and BXL size up to 3 m in depth at the proposed structure foundation locations.

The dolostone bedrock is a fine to medium grained chemical sedimentary rock that typically is composed of magnesium carbonate compounds and calcite crystals. The rock is unweathered that is featured by a porous "vug" texture and stylolites. The rock is light grey to medium dark grey in colour and contains thin horizontal beds and wide spaced vertical fractures. Detailed descriptions of the bedrock are attached in the Appendix entitled "Description of Rock Core".

An assessment of the quality and strength of the rock was carried out by measuring core recoveries and Rock Quality Designations (RQD) in the field and hardness testing in the laboratory. Recoveries ranged from 87% to 100% and RQD's ranged from 70% to 83% indicating that the rock is of good to excellent quality. Rock strengths can be described as medium strong.

#### GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water levels in the open boreholes throughout the duration of the field investigation. Groundwater levels determined at the time of the recent investigation were approximately 3 m below the ground surface (elevation 218.9 m to 219.1 m).

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

## DISCUSSION AND RECOMMENDATIONS

It is proposed to construct a two (2) span underpass structure (equal spans of approximately 29 m) that will carry the existing Alberton Road at its present alignment over the new Hwy. 403. The new Hwy. 403 will initially be a four lane median divided highway with ultimate widening plans. The Alberton Road structure is one of many structures proposed in conjunction with the Hwy. 403 connection between Hwy. 2 in Ancaster to Brantford.

The proposed deck is comprised of two 3.25 m roadway lanes and two 1.0 m shoulders. The width of the Hwy. 403 at the underpass is 40 m.

The proposed profile grade of Alberton Road is approximately 229.6 m whilst the proposed grade of Hwy. 403 is approximately 223.6 m. The existing ground surface at the site is approximately 221.3 m to 222.1 m. Consequently, approach fills in the order of magnitude of 8 m will be required for the Alberton Road approaches and approximately 1.5 m to 2 m of fill material for the Hwy. 403 at the underpass location will be required.

A plan illustrating the roadway and highway alignments and the proposed structure foundation locations is attached in the Appendix. The proposed profile grades have also been superimposed on the stratigraphical sectional profile also illustrated on Dwg. 656705-A in the Appendix.

To facilitate the design and construction of the proposed structure foundations and related earthworks, the following foundation and geotechnical recommendations are provided in the scope of this report.

- 1) Structure Foundations
- 2) Reinforced Earth Walls
- 3) Approach Embankments
- 4) Construction Considerations

### 1) Structure Foundations

The surficial clayey silt with random interbedded silt seams is considered unsuitable for the support of economical conventional spread footings at the

site. Consequently, it is recommended that all structure foundations be supported on deep foundation units. The bedrock present at this site provides an end-bearing stratum and consequently, structure foundations can be supported on end-bearing steel H-piles driven to the bedrock surface. Table 4 below provides recommended axial bearing capacities for steel H-piles driven to bedrock.

The capacities, however, apply to vertical piles and appropriate considerations shall be given to inclined piles. Any reduction to account for inclined loadings shall be carried out in conformance with the factors given in Section 6-8.3.4.3 of the O.H.B.D.C.

Table 4 - Axial Capacities

<u>Structure Foundation</u>	<u>Pile Type</u>	<u>Factored Capacity at U.L.S. (kN)</u>	<u>Bearing Capacity at S.L.S. Type II (kN)</u>	<u>Estimated Pile Tip El. (m)</u>
South Abutment	HP310x110	1600	1150	193.1±
	HP310x79	1150	890	
Pier	HP310x110	1600	1150	191.7±
	HP310x79	1150	890	
North Abutment	HP310x110	1600	1150	192.3±
	HP310x79	1150	890	

Consideration can be given to "perching" the pile caps within the embankment fill material. Should this alternative be selected, it is recommended that the fill material be limited to a maximum particle size of 75 mm to avoid any potential pile impediment during pile driving.

Resistance to lateral loads can be achieved by inclining the piles at a batter. The lateral resistance for vertical and battered piles shall be computed in accordance with Section 6-8.3.8 of the O.H.B.D.C.

Pile spacing shall conform with Section 6-8.3.10 of the O.H.B.D.C. Adjacent piles should be checked for heaving during pile installation. For centrally loaded piles equal load sharing on the deep foundation units can be assumed. The design of eccentric loaded deep foundation units shall comply with Section 6.8.3.4.2 of the O.H.B.D.C.

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

All piles shall be driven to the bedrock surface. To facilitate the installation of the steel H-piles, it is recommended that the piles be equipped with reinforced tips as illustrated on MTO standard Dwg. DD-3301. The steel H-piles shall be spliced in accordance with OPSS 903.07.01.03 and as shown on the abovementioned Dwg. DD-3301.

## 2) Reinforced Earth Walls

Should the application of reinforced earth walls prove to be an economical alternative, consideration can be given to employing reinforced earth abutment and/or retaining walls. The levelling pads can be founded on the native surficial clayey silt to silty clay stratum or alternatively "perched" on compacted Granular 'A' material. Pertinent bearing capacities are summarized in Table 5 below for the various alternatives.

Table 5 - Reinforced Earth Wall Foundation Capacities

<u>Structure</u>	<u>Description of Founding Soil</u>	<u>Founding El. (m)</u>	<u>Bearing Capacity at S.L.S. Type II (kPa)</u>	<u>Factored Capacity at U.L.S. (kPa)</u>
North/South Abutment	Surficial Native Clayey Silt to Silty Clay	Below Frost Penetration (<220.5 m)	175	250
	Granular 'A' Pad	2 m thickness	200	300
	Granular 'B' Pad	4 m thickness	250	375

Settlements induced as a result of the applied pressures are anticipated to be within 25 mm.

The Granular 'A' pad shall be placed and compacted in accordance with OPSS 501 series. All softened and/or organic material shall be removed prior to the placement of this granular pad. The native soil shall be proof-rolled prior to the granular pad placement.

A Non-Standard Special Provision (NSSP) shall be included in the contract documents that addresses the supply and installation of the reinforced earth wall.

### 3) Approach Embankments

Approach fills in the order of magnitude of 8 m will be required for structure approach embankments. Discussion of the stability, settlement, lateral earth pressures on the structure and construction of the approach embankments are provided below.

#### (a) Stability

Stability computations were carried out to determine both the overall (global) stability and surficial stability of the proposed embankment fill heights. The analyses was carried out in terms of total stress applying the Bishop's Modified Method of slices using an in-house mainframe application software package. Static loading conditions representing the embankment fills were employed in the analyses and circular failure slip surfaces were examined. A granular fill material with an angle of internal friction of  $30^\circ$  and a unit weight of  $20 \text{ kN/m}^3$  was used in the analyses. Should the actual fill material differ from this presumed material, this office should be contacted for a further assessment of the embankment stability. The slope geometry and subsurface soil parameters and conditions applied in the analyses are illustrated on Figure 8 in the Appendix.

The results of the analyses reveal that no deep seated slope failures are anticipated for embankments constructed to 8 m in height with 2H:1V slopes. However, to avoid surficial slope instabilities, it is recommended that for fill heights exceeding 8 m, the embankments be constructed at 2H:1V slopes with a nominal 2 m midheight berm sloped at a 2% gradient to facilitate surface runoff away from the slope. The midheight berm pertains to slopes both in the longitudinal and transverse directions.

All exposed slopes should be protected from erosional forces by providing an effective erosional control protection scheme.

(b) Settlement

Magnitude

Settlements induced as a result of the applied embankment loading will be of a magnitude equivalent to the summation of the elastic recompression of the native subsoils and settlements induced within the fill material itself. The total settlement predicted is in the order of 100 mm comprised of the individual settlements as discussed below.

In the computation of the settlements of the native subsoils overlying bedrock, the increase in vertical stress due to the embankment loading was determined by employing the Osterberg (1957) method. Elastic settlements as a result of the induced loading were determined using Steinbrenner's (1934) procedure and applying the compressibility curve determined by the consolidation testing executed in the laboratory for the clayey silt layers.

As discussed previously in this report, cohesive clayey silt seams and layers are present within the silt deposit located at a depth of 5 m to 6.6 m and extending to a depth of 24.4 m to 25.9 m. Based on layers ranging from 25 mm to 2 m, with most common thicknesses in the order of 50 mm, it is estimated that approximately 20% of the deposit (approximately 4 m) consists of this material. It is therefore predicted that the clayey silt layers will undergo a recompression settlement in the order of 25 mm. The total computed elastic settlement is therefore estimated at approximately 50 mm.

Settlements within the fill itself can also be expected due to self weight of the fill material. It is estimated that approximately 25 mm to 50 mm of settlement within the fill itself can be realized.

Time Rate

The elastic settlements within the native subsoil are immediate in nature and hence will be realized almost immediately subsequent to load application. It is



anticipated that the elastic settlement will occur within three months of embankment loading application. Settlements within the fill itself should be realized within three (3) months if a cohesive fill material is used and will be immediate in nature and hence occur during construction if the fill material is granular.

In view of the fact that most of the settlements will be realized within a three (3) month post construction time period, it is recommended that the approach fills be constructed as far in advance of the structure foundations as scheduling, construction feasibility and economics permit and that the final paving be delayed to account for this time period.

(c) Backfill to Structure

It is recommended that Granular 'A' or Granular 'B' be used within a wedge behind the abutments and retaining walls bounded by a plane rising at 60° to the horizontal as shown in Figure 6-9.6.1 of the O.H.B.D.C. The application of granular material combined with weep holes in the abutment walls to drain any accumulation of water in the backfill will prevent hydrostatic pressure build-up. Design parameters of the soil are given in Table 6 below.

Table 6 - Backfill Properties

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction ( $\phi$ ) (Unfactored)	35°	30°
Unit Weight (kN/m <sup>3</sup> ) $\gamma$	22.8	21.2
*Coefficient of Active Earth Pressure ( $K_a$ )		
- S.L.S.	0.27	0.33
- U.L.S.	0.33	0.4
*Coefficient of Earth Pressure at Rest ( $K_o$ )		
- S.L.S.	0.43	0.5
- U.L.S.	0.5	0.58

\*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.

(d) Embankment Construction

In the construction of the embankment fills, all softened and/or organic material should be excavated for their full depth within the plan limits prior to fill placement.

Embankment fills shall be placed and compacted as specified in OPSS 206.07.07 and OPSS 501 series.

Heavy compaction equipment should not be used behind the abutment/retaining walls within a lateral distance equal to the current height of fill above the wall footing in order to avoid imposing damage or deflection to the wall during the fill placement.

4) Construction Considerations

Dewatering

There are no dewatering problems anticipated during foundation excavation and construction due to the relatively impervious nature of the surficial clayey silt to silty clay material at the site. Any minor seepage that may develop from the occasional silt seams or any surface run-off can be readily discharged by employing conventional sump pump techniques.

Temporary Slopes

Any temporary excavation slope within the surficial clayey silt to silty clay with random seams of silt deposit shall be  $1\frac{1}{2}$ H:1V or flatter.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer, and L. Dametto, Engineering Student, utilizing equipment owned and operated by Atcost Soil Drilling Ltd. Logging of rock core in the laboratory was carried out by D. Williams, Petrographer. The project was carried out by T. Sangiuliano under the general supervision of P. Payer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed by P. Payer and approved by M. Devata, Chief Foundation Engineer.



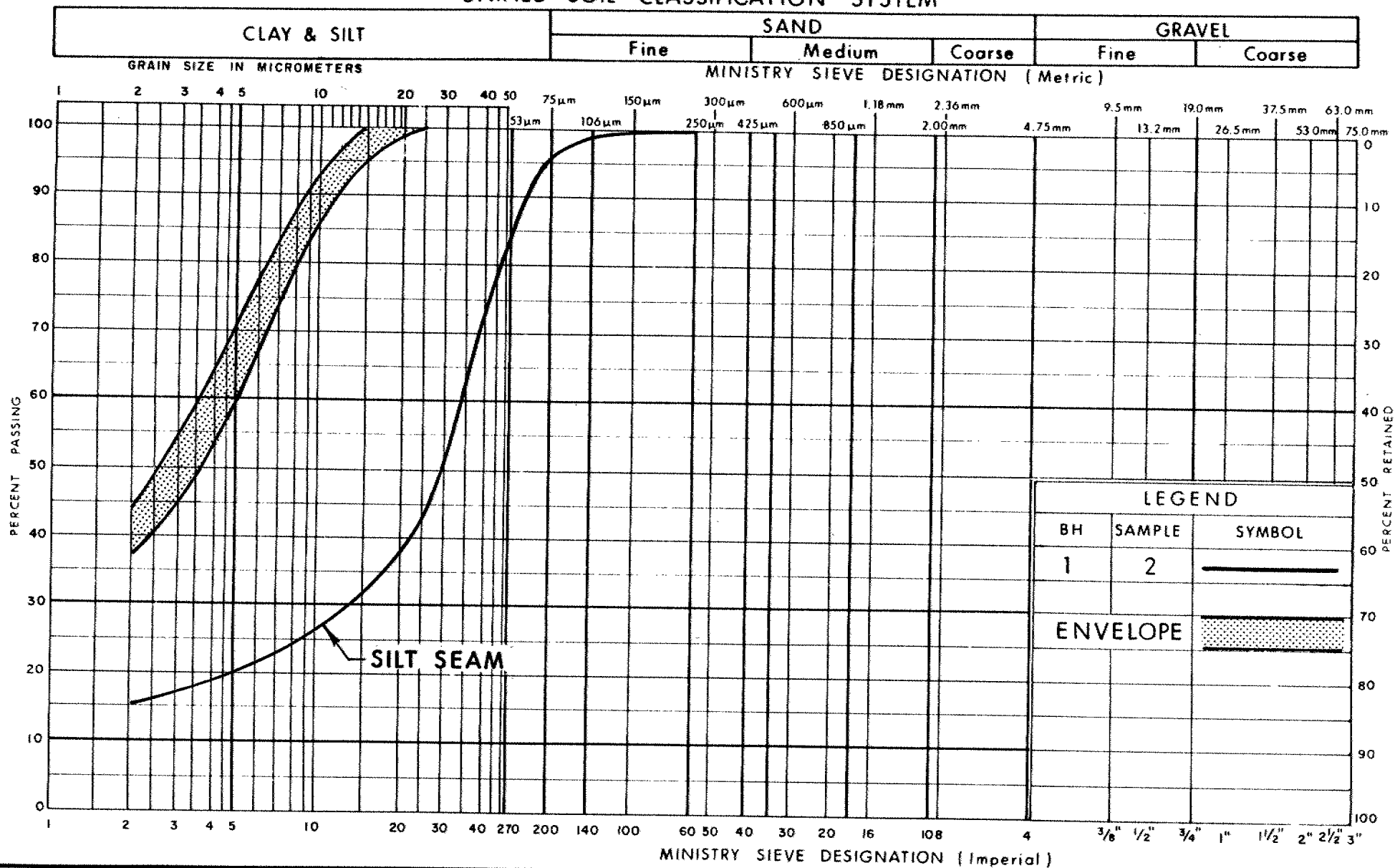
T. Sangiuliano, P.Eng.  
Foundation Engineer

A handwritten signature in black ink, appearing to read 'M. Devata'.

M. Devata, P.Eng.  
Chief Foundation Engineer

## APPENDIX

## UNIFIED SOIL CLASSIFICATION SYSTEM

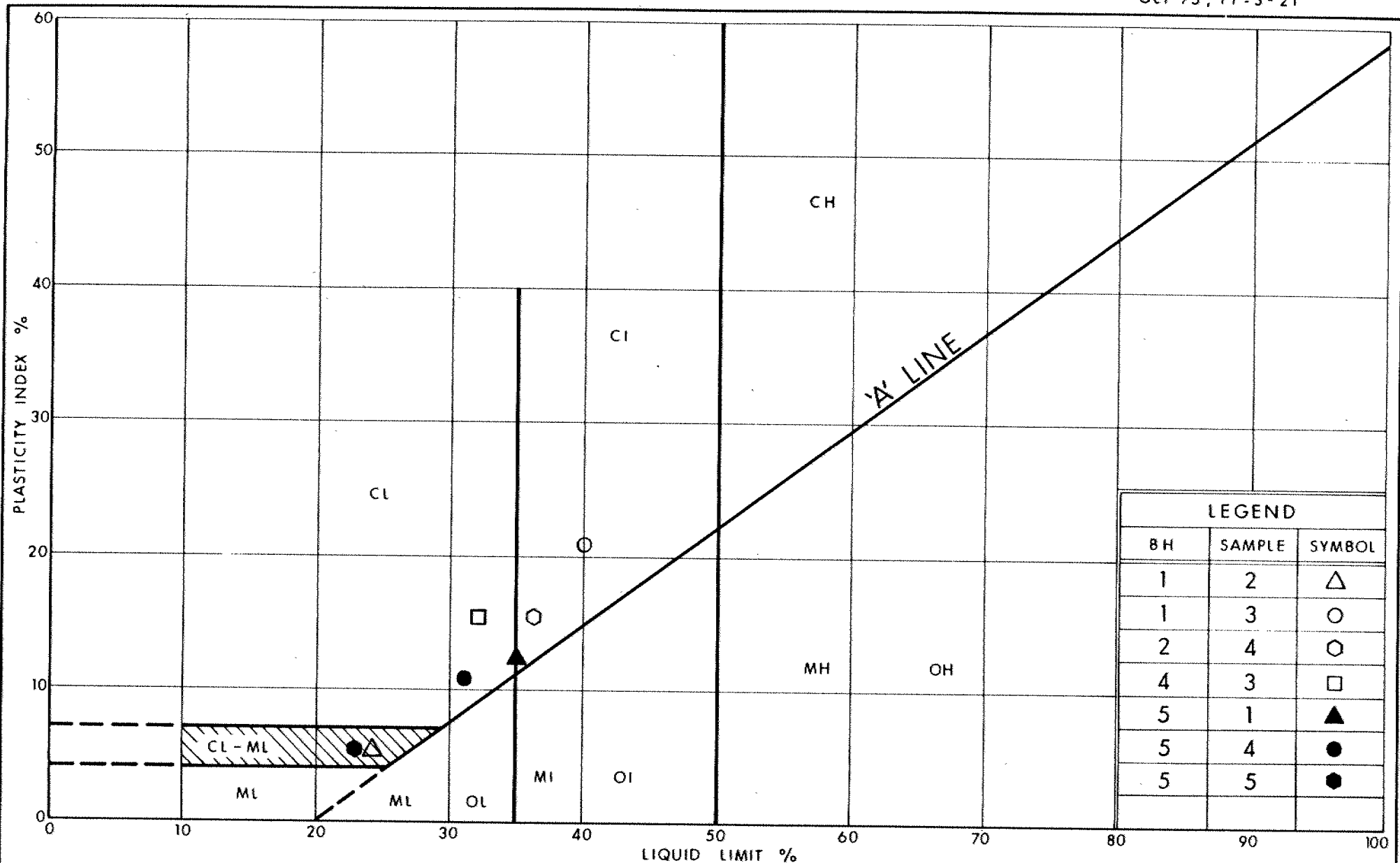


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GRAIN SIZE DISTRIBUTION  
CLAYEY SILT TO SILTY CLAY  
WITH RANDOM SEAMS OF SILT

FIG No 1

W P 65-67-05



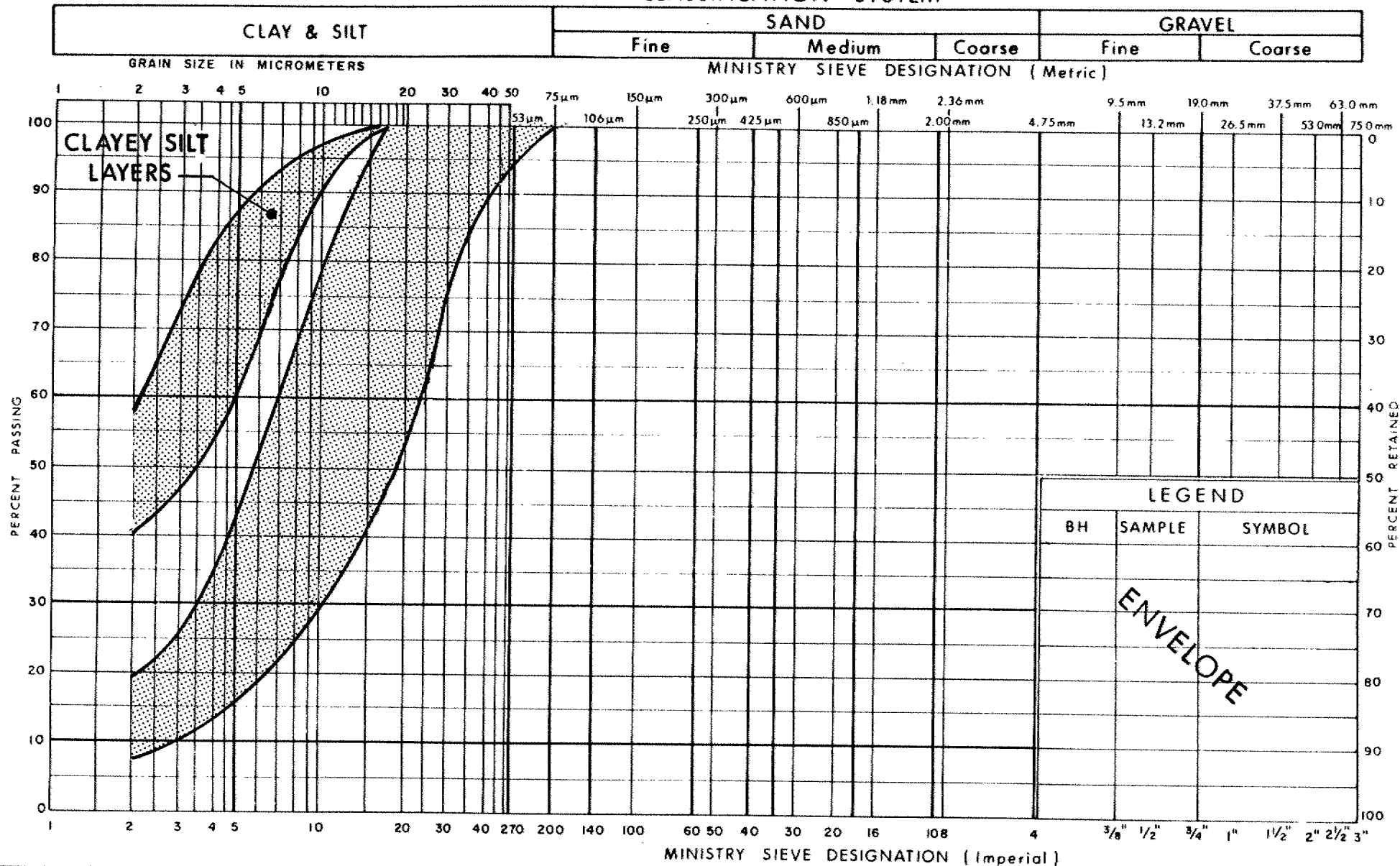
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Ontario

# PLASTICITY CHART CLAYEY SILT TO SILTY CLAY WITH RANDOM SEAMS OF SILT

FIG No 2

W P 65-67-05

## UNIFIED SOIL CLASSIFICATION SYSTEM



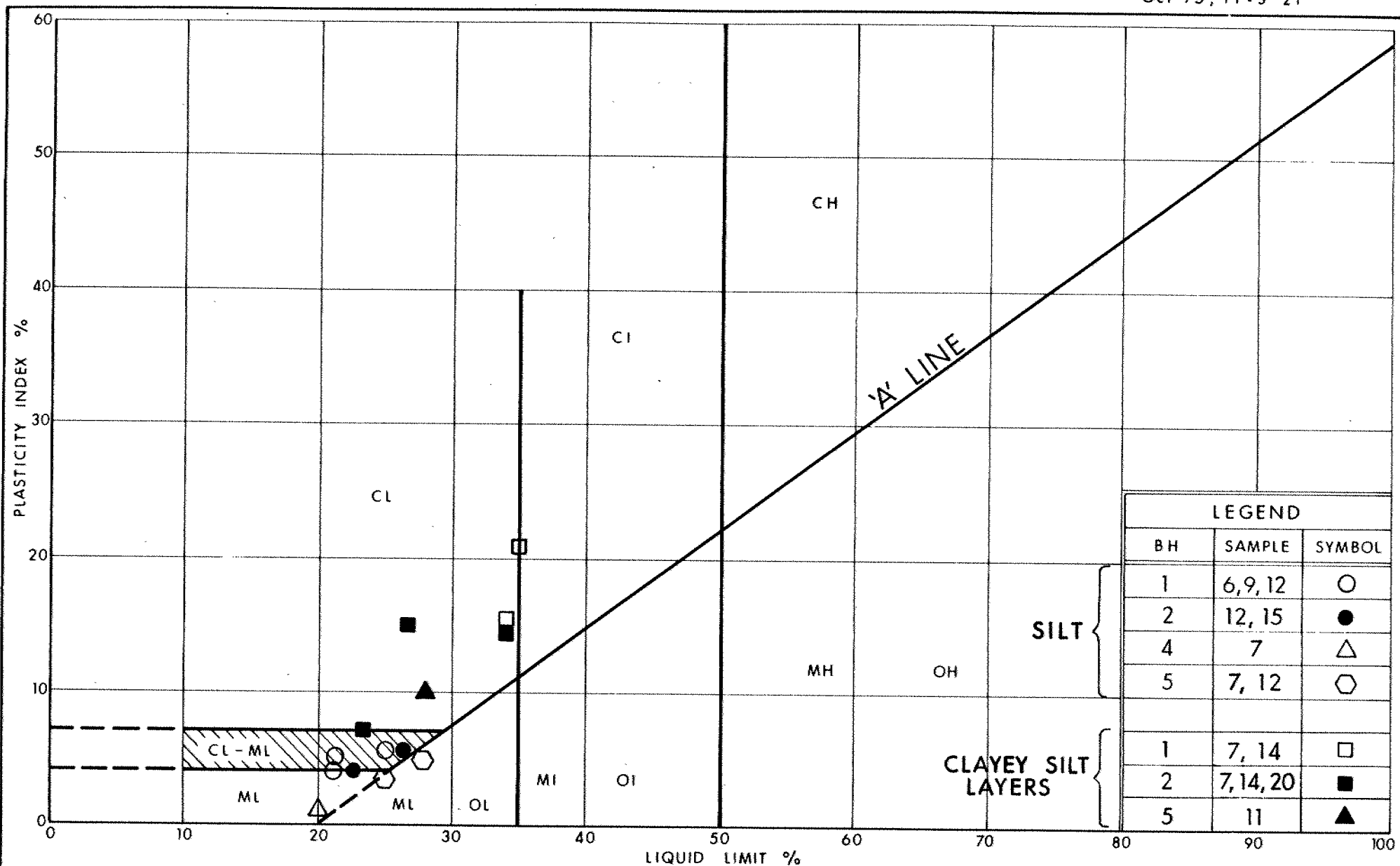
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## GRAIN SIZE DISTRIBUTION

### SILT, WITH INTERBEDDED LAYERS OF CLAYEY SILT

FIG No 3

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# PLASTICITY CHART SILT, WITH INTERBEDDED LAYERS OF CLAYEY SILT

FIG No 4

W P 65 - 67 - 05



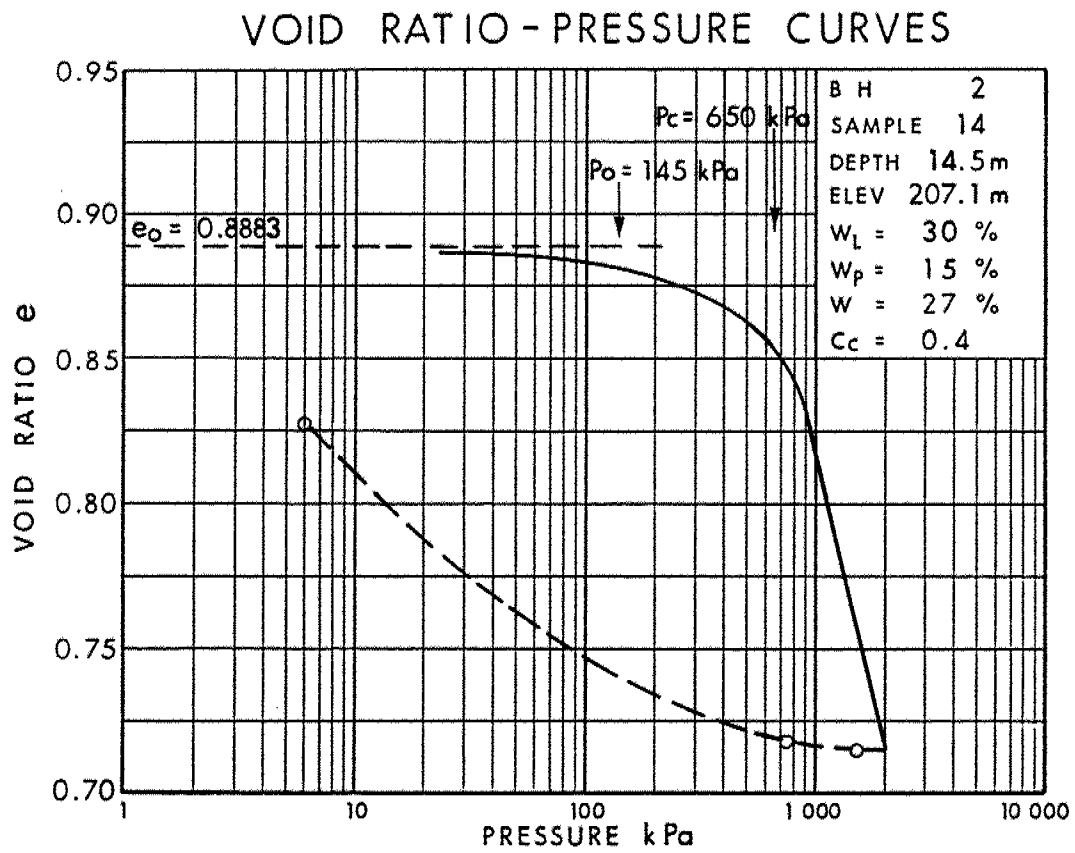
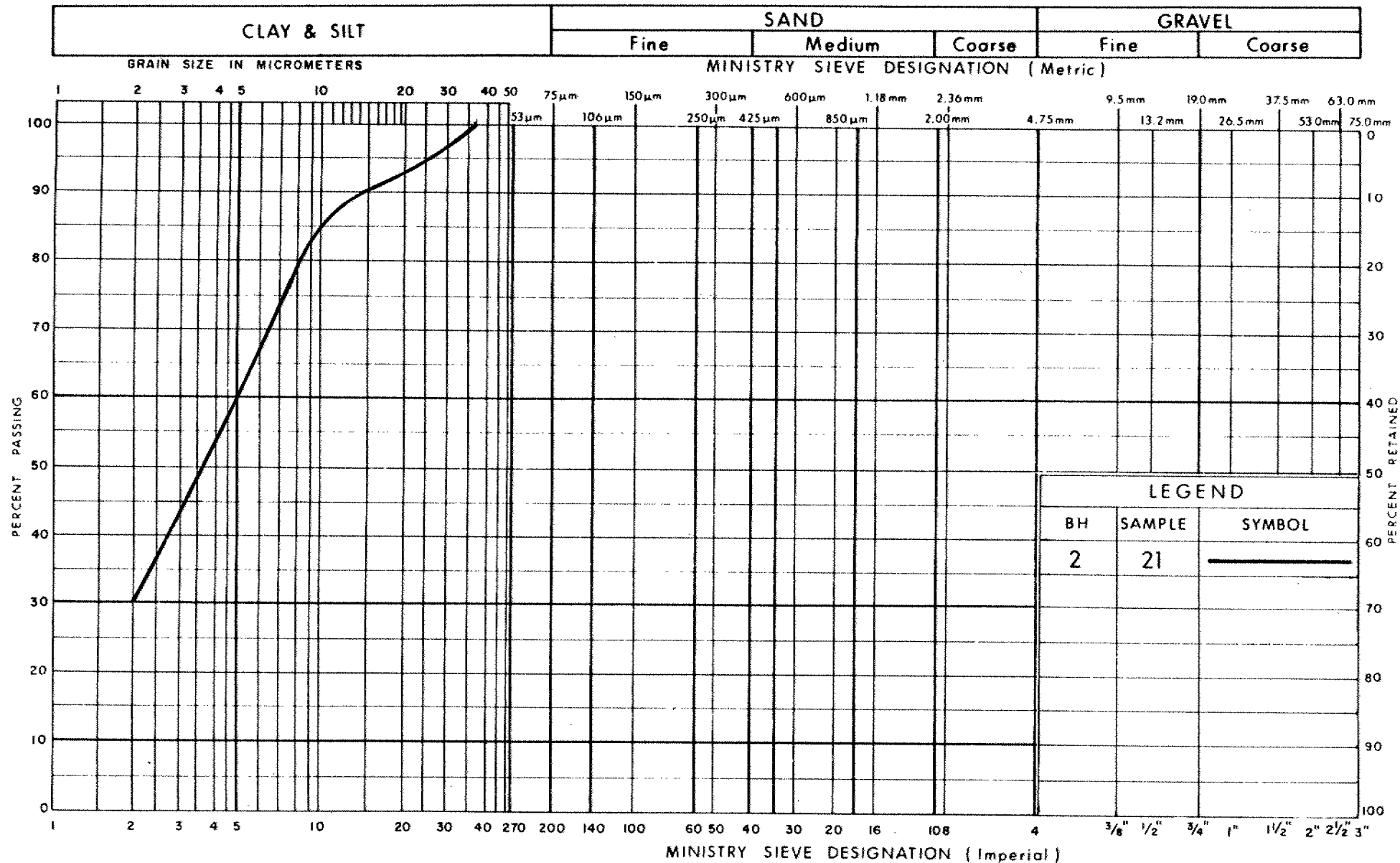


Fig 5

WP 65-67-05

## UNIFIED SOIL CLASSIFICATION SYSTEM



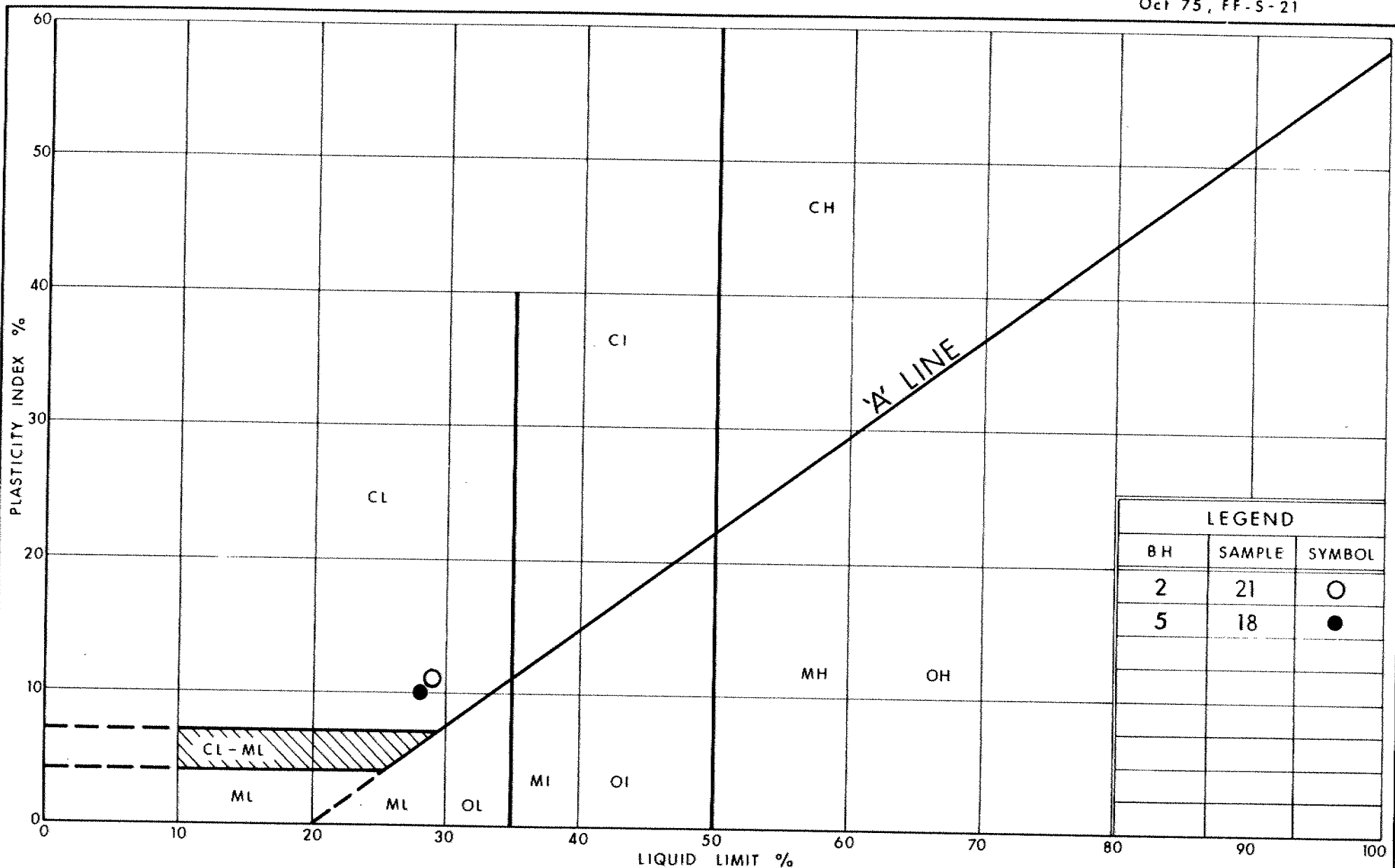
Ontario

Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT, WITH LAYERS OF SILT

FIG No 6

W P 65-67-05

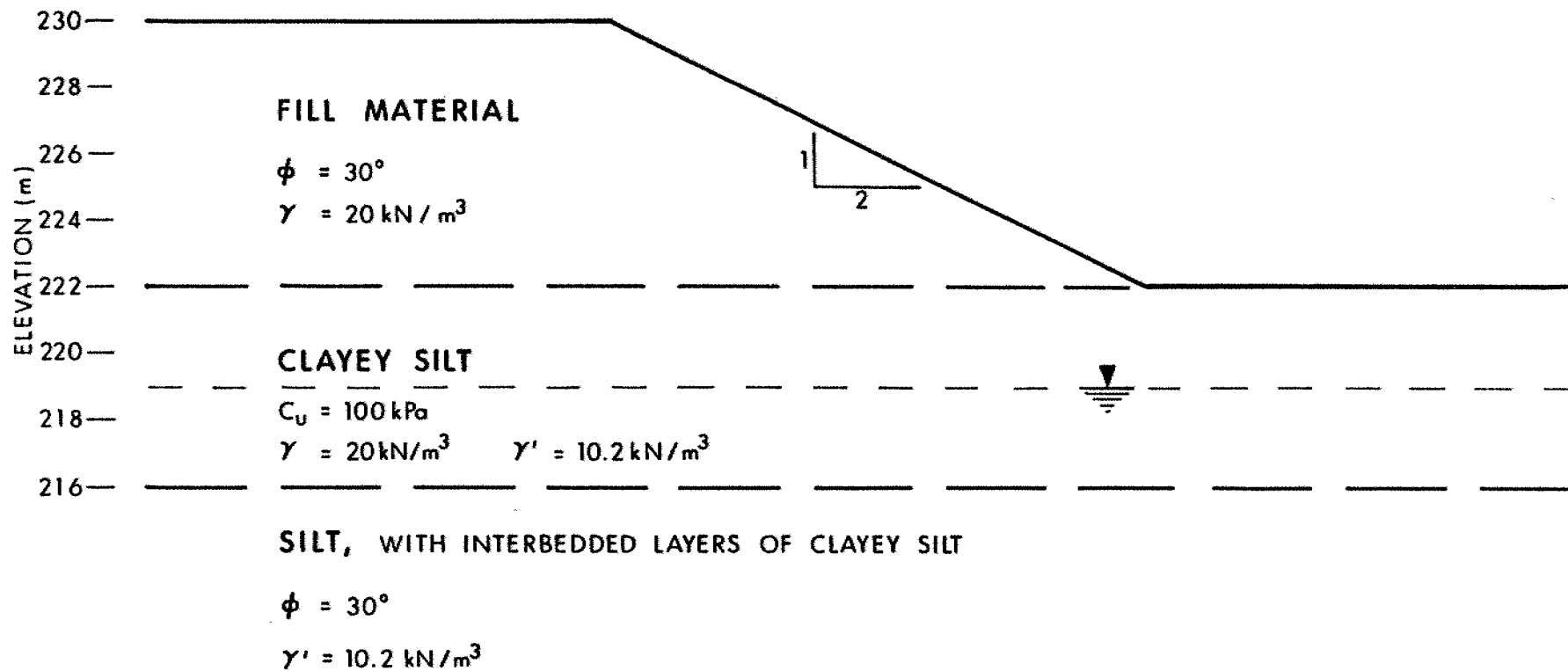


LEGEND		
BH	SAMPLE	SYMBOL
2	21	○
5	18	●



PLASTICITY CHART  
CLAYEY SILT, WITH LAYERS OF SILT

FIG No 7  
W P 65-67-05



## APPROACH EMBANKMENT STABILITY ANALYSES

FIG No 8

WP 65-67-05

## EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

**JOINTING AND BEDDING:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

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

### MECHANICAL PROPERTIES OF SOIL

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

### PHYSICAL PROPERTIES OF SOIL

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

# RECORD OF BOREHOLE No 1

1 OF 2 METRIC

W.P. 65-67-05 LOCATION Co-ords: N 4 783 630.4; E 258 154.8 ORIGINATED BY TS  
 DIST 4 HWY 403 BOREHOLE TYPE HS Auger, BW Casing, Washboring, Rock Coring COMPILED BY LD  
 DATUM Geodetic DATE 91 06 24 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			20 40 60 80 100	20 40 60 80 100					
221.3	Ground Surface													
220.5	Sand and Gravel (Fill Material) Brown, Compact					*								
0.8	Clayey Silt with random seams of Silt Very Stiff	Brown Grey	1	SS	10		220						20.3	0 3 82 15
			2	SS	8		218							0 0 55 45
			3	SS	13		216							0 0 84 16
216.0			4	SS	16		214							0 0 43 57
5.3			5	SS	22		212							0 0 88 12
			6	SS	14		210							0 0 76 24
			7	SS	11		208							
	Silt with interbedded layers of Clayey Silt Grey, Loose to Compact		8	SS	18		206							
			9	SS	12		204							
			10	SS	9		202							
			11	SS	10		200							0 0 60 40
			12	SS	18		198							
			13	SS	8		196							
			14	SS	12		194							
			15	TW	PH		192							
196.9			16	SS	14									
24.4	Clayey Silt with layers of Silt Grey, Stiff to Very Stiff		17	SS	11									
193.1			18	RC	REC 95%									RQD = 77%
28.2	Bedrock - Dolostone Light Grey, Unweathered Medium Strong		19	RC	REC 100%									RQD = 83%
190.8														

30.5

Continued

+3, x3: Numbers refer to  
Sensitivity

20  
15-5 (%) STRAIN AT FAILURE  
10

Continued

RECORD OF BOREHOLE No 1

2 OF 2

METRIC

W.P. 65-67-05 LOCATION Co-ords: N 4 783 630.4; E 258 154.8 ORIGINATED BY TS  
 DIST 4 HWY 403 BOREHOLE TYPE H5 Auger, BW Casing, Washboring, Rock Coring COMPILED BY LD  
 DATUM Geodetic DATE 91 06 24 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 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	w <sub>p</sub>	w			w <sub>L</sub>
190.8	Continued		19	RC	REC 100%												
30.5 190.1																	
31.2	End of Borehole • GWL not established																

# RECORD OF BOREHOLE No 2

1 OF 2 METRIC

W.P. 65-67-05 LOCATION Co-ords: N 4 783 686.8; E 258 142.0 ORIGINATED BY TS  
DIST 4 HWY 403 BOREHOLE TYPE HS Auger, BW Casing, Rock Coring COMPILED BY TS  
DATUM Geodetic DATE 91 06 26-27 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m <sup>3</sup>	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>	10 20 30			
221.6	Ground Surface													
0.0	Sand and Gravel (Fill Material)													
220.8	Brown, Compact													
0.8	Clayey Silt to Silty Clay with random seams of Silt		1	SS	9		220							
			2	SS	18									
			3	SS	16									
	Very Stiff Brown		4	SS	24		218					19.6	0 0 63 37	
	Grey		5	TW	PH									
216.6			6	SS	23									
5.0			7	SS	30		216						0 0 79 21	
			8	SS	29									
			9	SS	10		214							
			10	SS	24								0 0 92 8	
			11	SS	10		212							
	Silt		12	SS	8									
	with interbedded layers of		13	SS	9		210						0 0 85 15	
	Clayey Silt		14	TW	PH									
	Grey, Loose to Compact		15	SS	8		208						0 0 55 45	
			16	SS	8									
			17	SS	6		206						0 0 82 18	
			18	SS	11									
			19	SS	10		204							
			20	TW	PH									
195.7			21	SS	10		202							
25.9	Clayey Silt with layers of Silt													
	Grey, Very Stiff						200							
192.3			22	SS	3		198							
29.3	Bedrock - Dolostone													
191.1	Light Grey, Medium Strong, Unweathered		23	RC	REC	87%	196							
							194							
							192							RQD = 70%

30.5

Continued

+3, x5: Numbers refer to  
Sensitivity

20  
15-5 (%) STRAIN AT FAILURE  
10

Continued



# RECORD OF BOREHOLE No 2

2 OF 2

METRIC

W.P. 65-67-05 LOCATION Co-ords: N 4 783 686.8; E 258 142.0 ORIGINATED BY TS  
 DIST 4 HWY 403 BOREHOLE TYPE HS Auger, BW Casing, Rock Coring COMPILED BY TS  
 DATUM Geodetic DATE 91 06 26-27 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
191.1	Continued																
190.8																	
30.8	End of Borehole		23	RC	REC	= 87%											
	• Sampler Bouncing																

# RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 85-67-05 LOCATION Co-ords: N 4 783 606.6; E 258 167.6 ORIGINATED BY TS  
DIST 4 HWY 403 BOREHOLE TYPE HS Auger COMPILED BY TS  
DATUM Geodetic DATE 91 06 26 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 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					
222.1	Ground Surface																
0.0 221.3	Sand and Gravel (Fill Material) Brown, Compact																
0.8	Clayey Silt with random seams of Silt Brown Grey Stiff to Very Stiff		1	SS	16		220										
			2	SS	10		218			+2							
217.1			3	SS	7												
5.0			4	SS	9		216										
			5	SS	7		214				>100						
			6	SS	12		212			2							
			7	SS	18						>100						
			8	SS	8		210										
			9	SS	10		208										
			10	SS	8		206										
			11	TW	PH		204										
			12	SS	18					+3							
201.4			13	SS	15		202										
20.7	End of Borehole																

# RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 65-67-05 LOCATION Co-ords: N 4 783 714; E 258 145.2 ORIGINATED BY TS  
 DIST 4 HWY 403 BOREHOLE TYPE HS Auger COMPILED BY TS  
 DATUM Geodetic DATE 91 06 26 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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		
221.9	Ground Surface													
221.1	Sand and Gravel (Fill Material)													
0.8	Cloyey Silt with random seams of Silt Brown Grey Very Stiff		1	SS	11		220							
			2	SS	24		218							
			3	SS	13		216							
215.3			4	SS	22		214							
6.6	Silt with interbedded layers of Cloyey Silt Grey, Compact		5	SS	15		212							
			6	SS	21		210							
			7	SS	13									
209.3			8	SS	10									
12.6	End of Borehole													

# RECORD OF BOREHOLE No 5 \*

1 OF 1

METRIC

W.P. 65-67-05 LOCATION Co-ords: N 4 783 662.2; E 258 159.5 ORIGINATED BY BW  
DIST 4 HWY 403 BOREHOLE TYPE HS Auger, BW Casing, Rock Coring, Cone Test COMPILED BY BW  
DATUM Geodetic DATE 76 03 24-25 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	20 40 60 80 100					
218.7	Ground Surface													
0.0	Cloyey Silt with random seams of Silt  trace of organics, Brown Grey		1	SS	23									
			2	SS	12									
			3	SS	14									
			4	SS	16									
214.5	Stiff to Very Stiff		5	SS	17									0 0 92 8
5.2			6	SS	7									
			7	TW	PH								18.9	
			8	TW	PH									
			9	SS	7									
			10	SS	7									0 0 93 7
	Silt with interbedded layers of Cloyey Silt  Grey, Loose to Compact		11	SS	9									
			12	TW	PH								19.0	
			13	SS	8									
			14	SS	9									
			15	SS	10									
			16	SS	9									
195.3														
24.4	Cloyey Silt with layers of Silt  Stiff to Very Stiff		17	SS	10									
191.7			18	SS	8									0 1 80 19
28.0	Bedrock - Dolostone													
190.0	Light Grey, Medium Strong, Unweathered		19	RC	REC 100%									
29.7	End of Borehole * Formerly BH 12 (WP 65-67-01)													

# **ROCK CORE DESCRIPTION** **WP 65-67-05**

Page 1 of 1

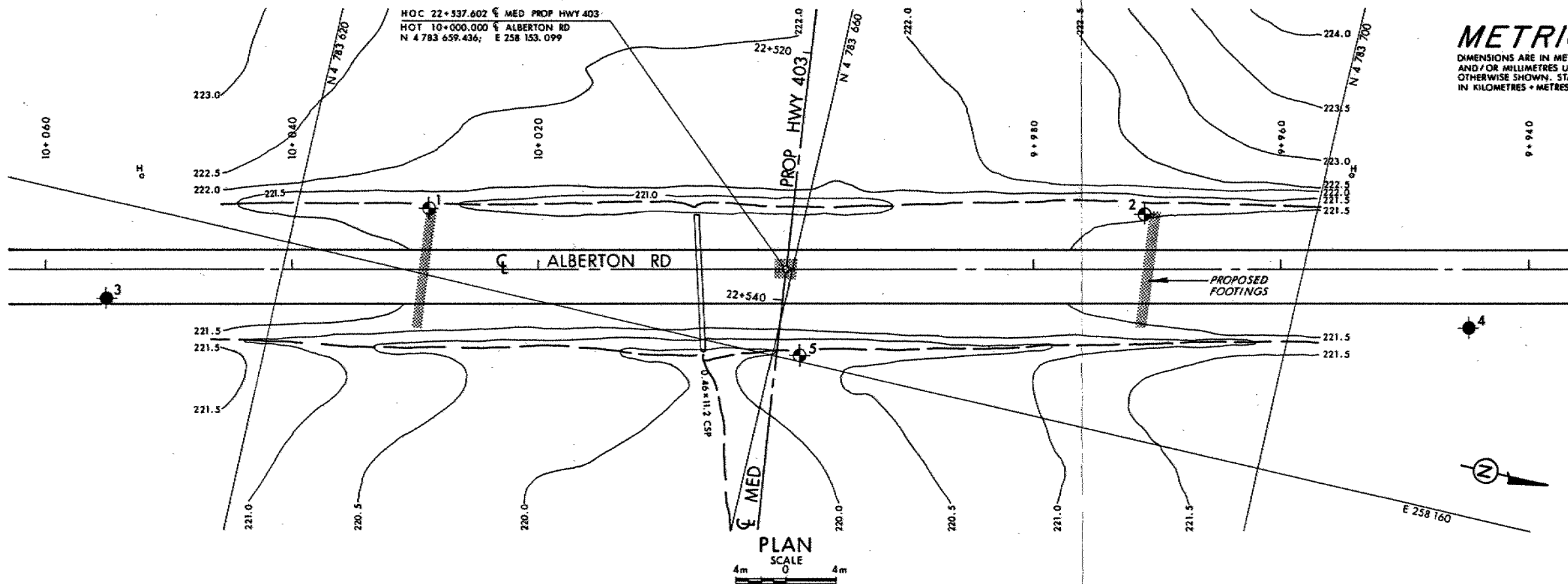
CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1	18	28.19-29.72	95	77	28.19-31.24	DOLOSTONE with stylolites and abundant vugs containing calcite crystals, light grey to medium dark grey; fine to medium grained; medium strong; unweathered to slightly weathered; fractures moderately close to very close spaced, flat to near vertical, undulating to planar, smooth to rough.
	19	29.72-31.24	100	83		
2	23	29.26-30.78	87	70	29.26-30.78	DOLOSTONE with stylolites and abundant vugs containing calcite crystals, light grey to medium dark grey; fine to medium grained; medium strong; unweathered to slightly weathered; fractures moderately close to very close spaced, flat to near vertical, undulating to planar, smooth to rough.

\*CR = CORE RECOVERY

\*RQD = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section



**METRIC**  
DIMENSIONS ARE IN METRES  
AND / OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES.

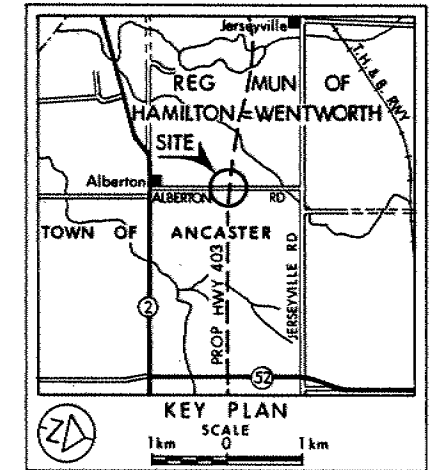
CONT No  
WP No 65-67-05

ALBERTON ROAD

BORE HOLE LOCATIONS & SOIL STRATA



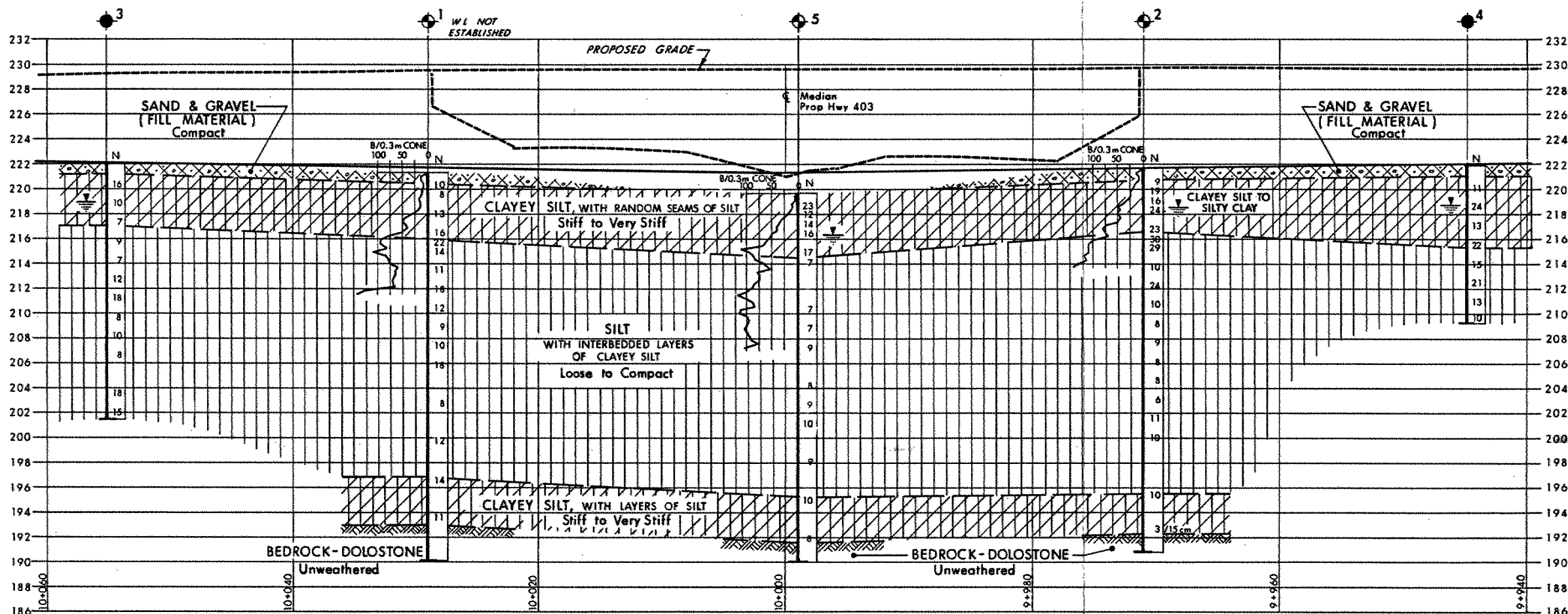
SHEET



# 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)
- W L at time of investigation 1976 03 and 1991 06

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	221.3	4 783 630.4	258 154.8
2	221.6	4 783 686.8	258 142.0
3	222.1	4 783 606.6	258 167.6
4	221.9	4 783 714.0	258 145.2
5	219.7	4 783 662.2	258 159.5



PROFILE ALBERTON RD

SCALE  
0 4m

# 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 102-2 of Form 100.

REV	DATE	BY	DESCRIPTION
1			

Geocres No 40P1-86

HWY No 403	SUBWD T 5	CHECKED 5	DATE 1991 11 12	DIST 4
DRAWN R 5	CHECKED 5	APPROVED		SITE 36-261
				DWG 656705-A

\* FUTURE LANE

## METRIC

**DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN**

DIST. 4

CONT No

WP No 65-67-05

ALBERTON ROAD UNDERPASS  
GENERAL ARRANGEMENT



**SHEET**

NOTES :

CLASS OF CONCRETE

PIER AND DECK	35 MPa
REMAINDER	30 MPa

## REINFORCING STEEL

GRADE 400 UNLESS OTHERWISE SPECIFIED.

BAR MARKS WITH SUFFIX 'C' DENOTES  
COATED BARS.

CLEAR COVER TO REINFORCING STEEL

FOOTINGS 100 ± 25

ABUTMENTS { WINGWALLS  
FRONT FACE - 80 ± 20  
BACK FACE - 70 ± 20

PIER	80 ± 20
DECK	70 ± 20
TOP AND ENDS	70 ± 20
BOTTOM AND SIDES	50 ± 10

REMAINDER ————— 70 ± 20  
UNLESS OTHERWISE NOTED

### CONSTRUCTION NOTES

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

## LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BORE HOLE LOCATIONS & SOIL STRATA
3. FOOTING DETAILS
4. ABUTMENTS & WINGWALLS
5. PIER DETAILS
6. DECK DETAILS
7. LONGITUDINAL TENDON DETAILS I
8. LONGITUDINAL TENDON DETAILS II
9. TRANSVERSE TENDON DETAILS
10. DECK REINFORCING I
11. DECK REINFORCING II
12. BARRIER WALL
13. 6000 mm APPROACH SLAB
14. DETAILS OF CONCRETE SLOPE PAVING
15. JOINT ANCHORAGE AND ARMOURING
16. AS. CONSTRUCTED ELEV. & DIM.
17. STANDARD DETAILS
18. ELECTRICAL EMBEDDED WORK
19. QUANTITIES STRUCTURE I
20. QUANTITIES STRUCTURE II

APPLICABLE STANDARD DRAWING

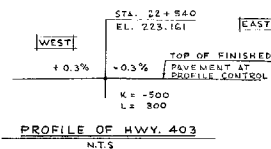
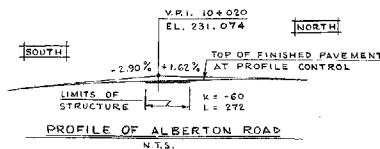
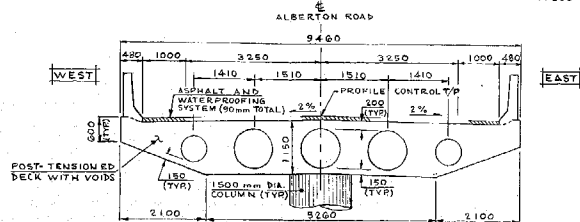
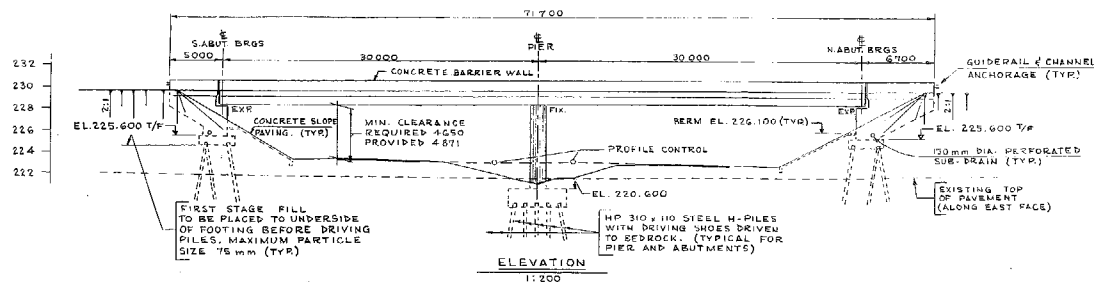
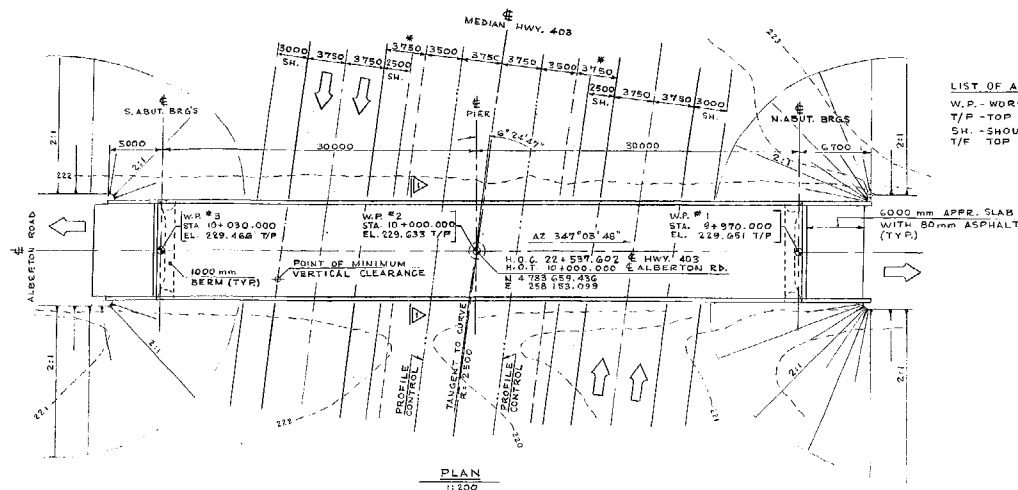
QPSD-3501.00 GRANULAR BACKFILL REQUIREMENTS

## LIST OF ABBREVIATIONS

W.P. - WORKING POINT  
T/P - TOP OF PAVEMENT  
SH. - SHOULDER  
T/F TOP OF FOOTING

NOTE

SLOPE PAVING IS NOT  
PART OF THIS CONTRACT.



B.M. 225.764  
N & W SE ROOT 0.3 m MAPLE  
134 m LT  $\phi$  22+531 HWY. 403

DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

REVISION	DATE	BY	DESCRIPTION
DESIGN	J.H.	CHK	CODE CHBDC-83 (LOAD CLAS-A)
DRAWN	G.F.	CHK	SITE 3G-2G1 STRUCT SCHEME DWG.