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FOUNDATION DESIGN SECTION

foundation investigation and design report

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

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WP 141-87-00C DIST 6

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Jersey Creek Concrete Culvert
at
Hwy. 407/CPR Subway

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FOUNDATION INVESTIGATION REPORT

For

Jersey Creek Concrete Culvert

at

Hwy. 407/CPR Subway

W.P. 141-87-00C

District 6, Toronto

INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the aforementioned site. The Jersey Creek, a tributary of the Humber River, flows through an existing concrete culvert located beneath the CPR embankment at the site. The creek and culvert are currently aligned at a skew to the proposed Hwy. 407. Hence, it is proposed to construct a new culvert that will intercept the creek along a realignment located approximately 35 metres north of the Hwy. 407 right- of-way. The 4.27 m x 2.44 m x 110 m concrete culvert will direct the flow of the waters of Jersey Creek to the Humber River located approximately 50 metres west of the proposed culvert outlet.

This report describes the subsurface conditions present at the site of the proposed structure and provides geotechnical recommendations for the design and construction of the new culvert. In addition, recommendations on the treatment of the existing culvert are provided.

SITE DESCRIPTION AND GEOLOGY

The site is located along and adjacent to the existing CPR track approximately 0.3 km northwest of the CPR-Islington Avenue level crossing in the Town of Vaughan, Regional Municipality of York. The site is characterized by a meandering valley that supports side slopes of approximately 2.5H:1V and trends in a general southwesterly direction. The crest of the valley is approximately 200 m in width and the valley depth is approximately 20 m. The valley houses the Jersey Creek that runs its course at the valley floor and is approximately 2 m in width and normally flows at 1 m depths. The Jersey Creek flows into the Humber River located in a floodplain immediately west of the site.

The valley slopes are densely covered with trees, brush, tall grasses and shrubs. There is no evidence of slope creep or displacement indicating that the valley slopes appear to be stable at its present geometry.

The existing CPR track at the site is supported by an earth embankment spanning the valley crest width. The railroad embankment, supposedly constructed in the early 1900's supports side slopes approximately 1.5H:1V. Trees and low lying shrubs and grassland cover the existing constructed slopes. There appears to be no evidence of slope stability other than a localized area at the northeastern portion of the embankment. Rip-rap and armour stone was placed on the slope to retard surficial erosion at this location.

A concrete culvert is located at the base of the constructed embankment to facilitate the Jersey Creek outflow beneath the embankment. Again, no visible signs of distress in the culvert were apparent.

Land use surrounding the site consists of residential lots located east of the site, a hydro corridor consisting of transmission towers just north of the site and forestland elsewhere. A CPR two span structure is located approximately 0.3 km north of the site along the same track alignment. The structure spans of the Humber River at this location. In addition, a CNR rigid frame overhead exists approximately 0.2 km south of the site to facilitate CN Rail traffic in a east-west direction over the CPR track.

Physiographically, the site lies within the region known as the South Slope (Chapman and Putnam, 1984). The south Slope Formation at the site consists of a ground moraine, scoured at intervals by valleys tributary to the Humber River systems. The valleys accentuate the hilly moraine topography. The glacial landforms and deposits were formed by the advance and retreat of the Wisconsinan ice sheet that covered the area during the Pleistocene epoch (over 5000 years ago).

The overburden is underlain by the grey shales of the Georgian Bay Formation of the Ordovician period.

FIELD INVESTIGATION

The fieldwork for the investigation was coordinated with the field investigation for the proposed CPR Subway and the associated detour. The fieldwork was implemented between 89 10 21 and 89 11 30 and consisted of a total of 14 sampled boreholes. Four of these boreholes were advanced along the proposed concrete culvert alignment.

Five of the fourteen boreholes were advanced through the overburden using hollow stem augering techniques to the depths of the lower sand to silty deposit (approximately 39 metres). Beyond that depth, the boreholes were advanced using conventional diamond drilling techniques (casing and washboring) to overcome torquing restriction imposed on the hollow stem augers. The NW casing used was advanced by both driven and rotary methods. The drilling equipment used was a track mounted CME 55.

In consideration of the importance of establishing the composition of the CPR embankment fill, a total of four boreholes were advanced in the existing embankment fill. Two of the boreholes were advanced from the crest of the embankment using the track-mounted CME. The other two boreholes were advanced at mid-slope, on the west side of the embankment (BH's 4A, 6A). These boreholes were advanced using conventional diamond drilling techniques via a tripod apparatus.

In general, subsoil samples were retrieved at 0.7 m intervals within a significant depth of 5 to 6 m beneath the proposed culvert invert elevations and 1.5 m intervals elsewhere. Disturbed subsoil samples were retrieved by a split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586). Relatively undisturbed samples were also randomly retrieved in the surficial till deposit using a shelby tube sampler in accordance with standard practice (ASTM D1587). In situ vane tests were also conducted in the cohesive surficial deposit, generally at 1.5 m intervals, to determine the undisturbed and remoulded undrained shear strengths of the soil. The test was conducted employing the standard MTO 'N' vane in accordance with ASTM D2573.

Bedrock was cored at five of the fourteen boreholes advanced at the overall site including BH C-3 located along the proposed culvert alignment. Bedrock was cored using conventional rock coring methods in NQ size.

All subsoil samples and rock core were identified in the field and then returned to the laboratory for further examination and applicable testing.

Water levels were obtained in the open boreholes and also in a sealed piezometer installed at BH D-8. Groundwater levels were monitored throughout the duration of the investigation. All boreholes were backfilled at the completion of the fieldwork.

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

LABORATORY ANALYSES

To identify the behaviour, gradation and pertinent properties and characteristics of the soil, various laboratory tests were performed. These tests included:

1) Atterberg Limits
2) Grain Size Distributions
3) Unit Weights
4) Natural Moisture Contents
5) Unconfined Compression Tests
6) Unconsolidated Undrained Tests
7) Multi-stage consolidated undrained tests with pore pressure measurements
8) Consolidation Test

In view of the general uniformity of soil types found in the general site area, including the proposed CPR Subway structure and the proposed detour, all laboratory results for similar soil strata have been integrated from the different structures. Laboratory test results have been summarized in the subsequent section of this report and are illustrated on corresponding figures and boreholes included in the attached Appendix.

SUBSURFACE CONDITIONS

The native subsoil of the original valley at the site consists of a surficial deposit composed of a clayey silt to silty clay with occasional sand seams and traces of gravel. The stratum is a till deposit of glacial origin and extends to a maximum thickness of 13.7 m at the crest of the valley. The thickness of this deposit decreases down the valley slope and does not exist at the valley floor and the floodplain located at the base of the existing CPR embankment (BH D-3) and at the culvert outlet (BH C-1). The consistency of this deposit ranges from firm to hard.

Underlying the upper till deposit and located as a surficial stratum in the floodplain exists a deposit of clayey silt that extends for a considerable thickness ranging from 18.3 m to 28 m below the upper till and from 10.7 m to 11.1 m in the floodplain area. This stratum also contains random interbeds of sand and gravel.

The clayey silt deposit is in turn underlain by a cohesionless deposit of sands and silts. The deposit varies randomly in silt and sand percentages, varying from sand with some silt to sandy silt. Random zones of silt also exist in the soil matrix. Gravel, boulders and cobbles are also components of the lower depths of the deposit. The thickness of this deposit ranges from 4.2 m to 15.8 m with an average thickness of approximately 10 m. The denseness of this deposit varies from compact to very dense. This cohesionless deposit overlies shale bedrock of the Georgian Bay shale formation.

Two types of fill material was used to construct the CPR embankment. Surficially and within a zone above and immediately adjacent to the existing concrete culvert, a cohesionless backfill material consisting of a sand with some silt to sandy silt was used. Beneath the surficial sand material and beyond the culvert backfill wedge zone, the embankment fill material consists of a clayey silt with interbedded layers of sand. The thickness of the surficial cohesionless fill material which also exists on the embankment slopes, ranges from 2.0 to 4.6 m. The maximum depth of the embankment fill explored was 13.9 m at BH D-6, located at the proposed CPR Subway pier location. At BH's D-1, D-2, located in the area of the south abutment, only 1.5 to 2.4 m of granular fill was encountered, confirming the valley crest location. At BH D-4, the location of the proposed north abutment, 12.2 m of clayey silt fill material with interbedded layers of sand exists.

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 attached Record of Borehole sheets in the Appendix. A plan of the site illustrating the locations and elevations of the boreholes and subsoil stratigraphical sections are provided on Dwgs. 1418700C-1 & 1418700C-2.

A detailed description of the subsurface conditions encountered is given below.

<u>Sand, some Silt (Fill)</u>

As previously mentioned, the surficial embankment fill material and backfill material to the existing concrete culvert consists generally of a brown sand with some silt. Occasional layers of sandy silt and clayey silt are also present in the fill material and traces of fine gravel are also randomly intermixed. A grain size distribution envelope illustrating the gradation of the fill is provided in Figure 1 in the Appendix. The surficial thickness of the fill material varies from 1.5 to 2.4 metres and the maximum thickness explored was 16.2 metres at which depth the existing concrete culvert roof was encountered (see BH D-7).

Standard Penetration tests carried out in the cohesionless fill material revealed 'N' values ranging from 2 blows/0.3 m to 21 blows/0.3 m indicating a very loose to compact state of condition.

<u>Clayey Silt (Fill)</u>

Beneath the surficial cohesionless fill and beyond the culvert cohesionless backfill material, the CPR embankment fill consists of a brown, cohesive clayey silt. The maximum thickness of the clayey silt fill encountered was 12.2 metres at the proposed north abutment location. Interbedded layers of fine sand ranging in thickness from 50 mm to 150 mm are also present randomly in the cohesive matrix. A grain size distribution envelope for this material as determined by mechanical sieve and hydrometer analysis is given in Figure 2.

Atterberg Limits were obtained to evaluate the behaviour and plasticity of the soil and the results are plotted in Figure 3. A summary of the indices is provided in Table 1 below. Unit weights are also included.

Table 1 - Clayey Silt (Fill)

	Range	# of Tests
Natural Moisture Content (w%)	15-24	8
Liquid Limit (w_L%)	21-32	8
Plastic Limit (w_P%)	13-19	8
Unit Weight (kN/m^3)	19.2-20.2	4
Undrained Shear Strength (cu) (kPa)	80->120	5

The test results reveal that the cohesive fill material is of low plasticity and hence can be categorized as clayey silt.

Undrained shear strength measurements (cu) were obtained in situ by conducting field vane tests. Results are plotted on the Record of Borehole sheets in the Appendix and summarized in Table 1 above. However, in consideration of the interbedded layers of sand, consistencies ranging from stiff to very stiff which is representive of the determined shear strength values cannot be implicity assumed.

Silty Clay to Clayey Silt (Glacial Till)

The native surficial deposit present at the site consists of a cohesive silty clay to clayey silt with traces of sand and gravel and occasional random interbedded sand seams. The thickness of the deposit explored in the investigation ranges from 11.3 to 13.7 and the interbedded sand seams are generally 50 to 100 mm in thickness. At BH D-4, the approximate location of the proposed north abutment, this deposit does not exist indicating that the deposit decreases in thickness from the crest of the valley to the valley floor. The deposit is generally oxidized (brown) for the upper 1.5 to 3.5 metres and unoxidized (grey) for its lower thickness. The deposit is a till of glacial origin.

A grain size distribution envelope for this deposit as determined by mechanical sieve and hydrometer analysis is given in Figure 4. The envelope illustrates that clay and silt percentages in the deposit range from 25-61% and 35-61% respectively, confirming the range in behaviour of the fine grained portion of the deposit.

Although not encountered during this investigation, boulders and cobbles are characteristic components of till deposits and consequently may be encountered in this deposit.

Atterberg Limit tests were carried out to define the behaviour and plasticity of the soil and the results are plotted in Figure 5. A summary of the indices is provided in Table 2. Unit weights are also included.

<u>Table 2 - Silty Clay to Clayey Silt</u>

	Range	# of Tests
Natural Moisture Content (w%)	15-29	14
Liquid Limit (w_L%)	22-47	14
Plastic Limit (wp%)	12-20	14
Unit Weight (kN/m^3)	18.8-20.3	9
Undrained Shear Strength (cu) (kPa)		
- Field Vane	35->120	28
- Laboratory*	41-82	28
Sensitivity	2-3	28

*Unconfined Compression Tests
*Unconsolidated Undrained Tests

The test results reveal that the deposit varies randomly in plasticity ranging from low (clayey silt) to intermediate (silty clay).

Undrained shear strength measurements (cu) of the soil were obtained both by in situ vane tests and by laboratory tests, namely unconfined compression tests and unconsolidated undrained tests (quick triaxial). Results are plotted on the Record of Borehole sheets in the Appendix and summarized in Table 2 above. A Shear Strength vs Elevation profile is also provided in Figure 6. Based on shear strength values ranging from 35->120 kPa, it is considered that the soil has a firm to very stiff consistency.

The sensitivity of the soil as defined by the ratio of the undrained strength in the undisturbed state to the undrained strength, at the same water content, in the remoulded state was also determined by the field vane test and the results are tabulated in Table 2 and identified on the Record of Borehole sheets. Sensitivity values range from 2 to 3 indicating that the soil has a low sensitivity.

Consolidated undrained multi-stage triaxial tests with pore pressure measurements were conducted in the laboratory to determine the effective strength parameters of the material. The effective shear strength parameters determined from the test are summarized in Table 3.

Table 3 - Effective Shear Strength Parameters

Sample	BH D-1, TW5
Elevation (m)	147.0
Liquid Limit	47
Plastic Limit	20
Natural Moisture Content (w%)	26
Effective Angle of Internal Friction (Ø°)	29.5
Effective Shear Strength Intercept (c') (kPa)	10

For design purposes, a reduced angle of internal friction (Ø°) of 26° and a shear strength intercept of 5 kPa was selected to account for the fact that the sample test was not saturated.

In conjunction with the proposed detour, (BH D-5, WP 141-87-00D) located immediately west of the proposed CPR Subway, a consolidation test was conducted to evaluate the compressibility characteristics of this same deposit. The results (e-log p curve) of the test are illustrated in Figure 7 in the Appendix. The results reveal that this cohesive stratum has been preconsolidated in the past to an effective pressure 200 kPa in excess of the existing overburden pressure.

The coefficient of consolidation (cv) used to determine the time rate of consolidation settlement was computed using Taylor's Method (1948). The results reveal values ranging from 0.004 m^2/day to 0.005 m^2/day for loadings ranging from 100 to 200 kPa.

Standard Penetration tests carried out in this deposit revealed 'N' values ranging from 2 blows/0.3 m to 15 blows/0.3.

Clayey Silt

Underlying the surficial clayey silt to silty clay deposit at a depth ranging from 10.7 m to 13.7 m below the ground surface (Elevation 140.0 to 135.2) and extending for a maximum thickness of 18.3 m, and present at the the surface of the floodplain and extending to a maximum thickness of 11.1 m, a cohesive deposit of clayey silt exists. This stratum also contains traces of sand and random zones of silt. In the floodplain, a trace of organics was encountered in the surficial 2 m of the deposit and interbedded layers of sand and gravel approximately 100 mm in thickness also exist.

A grain size distribution envelope for this deposit as determined by mechanical sieve and hydrometer analysis is given in Figure 8. The envelope illustrates that clay and silt percentages in the deposit range from 12-34% and 60-88% respectively.

Atterberg Limit tests were carried out to define the behaviour and plasticity of the soil and the results are plotted in Figure 9. A summary of the indices is provided in Table 4. Unit weights are also included.

Table 4 - Clayey Silt

	Range	# of Tests
Natural Moisture Content (w%)	14-35	17
Liquid Limit (w_L%)	26-29	17
Plastic Limit (w_P%)	14-18	17
Unit Weight (kN/m^3)	20-22	8

The test results reveal that the fine grained portion of the deposit is of low plasticity and hence can be categorized as a clayey silt.

Standard Penetration tests carried out in this stratum revealed 'N' values ranging from 5 blows/0.3 m to 76 blows/0.3 m indicating that the deposit ranges in consistency from firm to hard. In general, in the upper 10 m or so, 'N' values ranged from 20 blows/0.3 m to 30 blows/0.3 m (although lower 'N' values were obtained in the floodplain area because of the presence of organics), indicating a very stiff consistency. In the lower depths of the deposit, 'N' values ranged from 10 blows/0.3 m to 20 blows/0.3 m and the soil can be categorized as having a stiff consistency.

Sand and Silt

Underlying the clayey silt deposit and extending to bedrock a cohesionless deposit of sand and silt exists. The deposit is predominantly composed of sand with some silt but random zones of sandy silt to silt are also present within this deposit. In addition, gravel, boulders and cobbles exist as a heterogeneous mixture in the deposit at the lower depths immediately above the bedrock. At BH's D-4 and C-3, approximately 2.5 m of the coarser grained gravel, boulders and cobbles was encountered. The thickness of the entire deposit ranges from 4.2 m to 15.8 m, but is generally in the order of 10 metres. A grain size distribution envelope for this deposit is provided in Figure 10 in the Appendix.

This cohesionless deposit is water bearing and consequently, when the deposit was penetrated in the open borehole, soil cave-in resulted due to unbalanced hydrostatic head.

Standard Penetration tests carried out in this deposit revealed 'N' values ranging from 10 blows/0.3 m to 120 blows/0.8 m indicating that the deposit ranges in denseness from compact to very dense. In the floodplain area of the site, 'N' values obtained below the upper 3 m thickness were generally representative of very dense material. The deposit is predominantly compact to dense at the other locations of the site.

Bedrock

The cohesionless sand with some silt deposit is directly underlain by shale bedrock of the Georgian Bay shale formation. The bedrock surface is generally flat with surface elevations ranging from 105.9 m to 107.7 m. The bedrock was cored by NQ size up to 2.8 metres in thickness.

The shale bedrock is grey in colour and is very fine grained and thinly laminated. The rock is generally slightly to moderately weathered and contains occasional clay seams, approximately 50 to 100 mm in thickness. Minor beds of argillaceous limestone are also present in the rock formation. Detailed descriptions of the bedrock are attached in the Appendix entitled "Description of Rock Core".

Core recoveries and Rock Quality Designations (RQD) were determined in situ and also in the laboratory to evaluate the competence and integrity of the rock. Rock recoveries varied between 60 and 100% while RQD's varied between 0 and 15%. The shale bedrock is weak to very weak rock.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in the open boreholes and monitoring the level in a piezometer installed at BH D-8 (CPR Subway pier location). The piezometer was installed in the clayey silt deposit with bentonite seals above and below the piezometer tip.

The water levels measured at the time of investigation along the proposed culvert alignment varied which reflects the changing topography from the culvert inlet to the culvert outlet. At the culvert inlet (BH C-4) and at BH C-2, the water level was encountered at elevations 135.2 m to 135.0 m respectively. At BH C-3, a location representing the crest of the valley along the proposed alignment, the water level was at a higher elevation of 148.2 m.

Artesian conditions were encountered in the lower gravels, boulders and cobbles present at the lower depths of the sands and silts deposit within the floodplain. Up to 3 m of piezometric head above the natural ground surface was observed.

In all cases, boreholes advanced in the embankment fill were dry and groundwater was not observed.

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

DISCUSSION AND RECOMMENDATIONS

The existing Jersey Creek, a tributary to the Humber River, intersects the proposed Hwy. 407 at the site location. The creek flows beneath the existing CPR embankment present at the site in a westerly direction through an existing concrete culvert. To facilitate advancement of the proposed highway at the site, it is proposed to intercept the creek approximately 35 m north of the highway right-of-way and direct the flow through a proposed 4.27 m x 2.44 m x 110 m concrete box culvert. An open channel will connect the flow from the culvert outlet to the Humber River.

The invert elevation at the culvert inlet and outlet is proposed at 135.5 m and 135.17 respectively. Consequently, excavation depths ranging from approximately 1.5 m to 17.5 m will be required to facilitate the construction of the culvert applying the conventional cut and cover method. The CPR rail tracks are to be placed at an approximate elevation of 153 m above the proposed culvert and consequently replacement fills in the order of magnitude of 15 m will be required using a cut and cover procedure.

Recommendations pertaining to the following geotechnial considerations for the design and construction of the concrete culvert are contained in the scope of this report.

1) Structure Foundations
2) Channel Outlet
3) Treatment of Existing Culvert
4) Construction Considerations

1. Structure Foundations

The proposed concrete culvert structure may be founded on spread footings located within the clayey silt deposit at an elevation of 135.5 m to 135.0 m. For purposes of the O.H.B.D.C., the following values are recommended:

Table 5 - Bearing Capacity - Shallow Foundation

Bearing Capacity at S.L.S. Type II (kPa)	200
Factored Capacity at U.L.S. (kPa)	300

Settlement of the foundation subsoil as a result of the applied footing pressure will be a result of the recompression of the soil and hence immediate in nature - i.e. take place during or immediately following the construction period. The magnitude of differential and total settlements induced within the proposed footing is anticipated to be less than 25 mm, provided that any organics and softened material present naturally or induced during construction is removed and replaced with mass concrete or a granular pad. It is recommended that a working slab be placed to protect the footing founding soil.

The underside of all footings should be provided with a minimum 1.2 m of earth cover for frost protection. In addition, to protect the footings against scour, a properly designed rip-rap, meeting the hydrological requirements at the site should be placed at the culvert channel inlet.

In general, no major dewatering difficulties are anticipated for footing excavations in consideration of the relatively low permeability of the silty clay to clayey silt till deposit and the underlying clayey silt stratum. Any localized seepage or surficial run-off can be discharged using conventional sump pumping techniques.

2. Channel Outlet

Excavation into the native silty clay to clay deposit and underlying clayey silt deposit will be required for the construction of the channel outlet that stretches from the culvert outlet to the Humber River, a total distance of approximately 50 m. The proposed stream bed elevation and profile gradient shall conform to hydrological requirements at the site. Based on these requirements, it is anticipated that excavation cuts up to 5 m will be required. The slopes can be excavated at 2H:1V provided that slope protection (rip-rap) conforming to hydrological data at the site is included.

3. Treatment of Existing Culvert

The presence of the existing culvert at its existing location and elevation should not hinder the performance of the highway if it is abandoned and left in place. Consequently, if the culvert does not impede or interfere with any other construction activity, it is recommended that the culvert be plugged at both ends using suitable backfill material acceptable as subgrade material.

4. Construction Considerations

The construction of the concrete culvert must be coordinated and scheduled with the other activities at the site, including the construction of the CPR detour, construction of the CPR Subway structure and maintaining train traffic throughout the construction process. To facilitate the integration of these activities at a selected sequence, various methods of construction schemes may be considered. Recommended methods that are feasible from a geotechnical point of view are discussed below. The method or combination of methods that proves to be most economical should be selected.

Tunnelling

Tunnelling procedures can be employed to advance the proposed culvert adjacent and beneath the existing CPR tracks. At the proposed culvert invert elevation, the tunnel face and crown would be advanced through the till deposit and the invert through the underlying clayey silt. In view of the stiff to hard consistencies of these soils, the soil should behave elastically when excavated and consequently remain stable until temporarily supported by a liner.

Conventional jacking and boring and/or "forepoling" (advanced crown protection) combined with hand excavation are methods that can be considered in tunnel advancement. The contractor should also be prepared for large boulder sizes that are characteristic components of till deposits as alluded previously in this report.

No dewatering problems are anticipated in the tunnel construction in view of the impervious nature of the host material. Conventional pumping techniques will suffice in discharging any localized seepage.

In an attempt to optimize the tunnelling procedure, consideration can be given in revising the culvert geometry to a circular concrete pipe rather than a box culvert. Circular tunnel liners are common in current tunnelling procedures.

The design and construction of the tunnel including liners, shafts and other tunnel appurtenances shall conform to corresponding sections of the O.H.B.D.C. and O.P.S.S. 415-416.

Cut and Cover Method

The construction of the culvert applying the conventional cut and cover method can be advanced by incorporating excavation slopes and/or a temporary shoring scheme. However, the considerable volumes of excavation, shoring requirements and placement and settlement of fills above the culvert are components that make this alternative less favourable. Specifics concerning this method are discussed below.

Excavation cuts up to 17.5 m will be required in soil consisting of varying thickness of fill, silty clay to clayey silt till and clayey silt as described in previous sections of this report. An effective stress stability analysis was carried out using Bishop's method on an in-house mainframe program incorporating a factor of safety of 1.3. The properties of the subsoil used in the analysis is summarized in Figure 11 in the Appendix.

The results of the analysis are summarized in Table 6 below.

Table 6 - Temporary Excavation Cuts

Depth (m)	Recommended Geometry
0-5 incl.	2H:1V slopes
>5-8 incl.	2 m mid-depth bench, 2H:1V slopes
>8-9 incl.	4 m mid-depth bench, 2H:1V slopes
>9-10 incl.	5 m mid-depth bench, 2H:1V slopes
>10-12 incl.	3 m double bench at 1/3 depths, 2H:1V slopes
>12-14 incl.	4 m double bench at 1/3 depths, 2H:1V slopes
15	5 m double bench at 1/3 depths, 2H:1V slopes
>15-17.5 incl.	5 m triple bench (see Figure 11)

It is apparent from the results that a considerable volume of material excavation is required to satisfy the indicated geometries.

Alternatively, excavation for culvert foundations can be facilitated by incorporating a temporary shoring system parallel to the proposed culvert alignment. A shoring systems that can be considered is a braced timber lagging-soldier pile wall. Liaison with this office should be coordinated during the shoring design process.

The design of the shoring system shall include the appropriate earth pressure distribution computed in accordance with Section 6.6.1.2 of the O.H.B.D.C. Preliminary parameters of the soil to be supported are summarized in Table 7 below.

Table 7 - Shoring Design Soil Parameters

Soil Type	Elevation (m)	Saturated Unit Weight (kN/m^3)	Effective Shear Strength Parameters (Ø°)
Fill	152.5-150-5	20	30°
Silty Clay to Clayey Silt (Till)	150.5-138.5	19.6	26°
Clayey Silt	>138.5	20	30°

Appropriate consideration for sloping soil surfaces shall be included in the design. Buoyant unit weights of soil shall be applied below the prevailing groundwater table.

Basal heave of the braced excavation is not anticipated. However, it is recommended that this be reviewed by this Office for confirmation in the preliminary design process should this scheme be selected.

Depending on the construction scheme selected, a temporary shoring system may also be required to support the train tracks during construction. It is recommended that if a staged construction scheme is selected, an anchored soldier pile timber lagging wall be designed to maintain the train traffic during construction. The parameters tabulated in Table 7 can be used in the design of the shoring system and the applicable earth pressure distribution shall be applied. Surcharge train traffic loadings shall be properly represented in the design.

Soil anchors can be installed in the native clayey silt material present at the site to resist the induced loadings. A bond stress of 500 kPa can be used for design. Further information pertaining to the design of soil anchors and test loading procedures can be obtained from this office.

Alternatively, the CPR track shoring system can be supported with rakers installed in front of the wall. Rakers must be installed while an earth berm remains in front of the pile. Slots should be cut into this berm to install rakers before the supporting berm is removed. Raker footings can be founded in the clayey silt deposit in accordance with the parameters tabulated in Table 5.

The aforementioned shoring systems can be installed by employing conventional augering or driving techniques. The construction of the shoring shall conform to O.P.S.S. 538 and O.P.S.S 539 series.

Free draining material such as Granular 'A' or Granular 'B' is recommended as appropriate backfill to the culvert to prevent hydrostatic pressure build-up. Design parameters of the soil are given in Table 8 below.

Table 8 - Backfill Soil Parameters

	Granular 'A'	Granular 'B'
Angle of Internal Friction (Ø)	35°	30°
Unit Weight (kN/m^3)	22.8	21.2
Coefficient of Earth Pressure at Rest (K_o)	0.43	0.5

Lateral earth pressures should be computed in accordance with Section 6.6.1.2.1 of the O.H.B.D.C. Weep holes should be designed to drain accumulation of water in the backfill. The tabulated earth pressure coefficients apply to horizontal surfaces only. Adjustment for sloping surfaces shall be implemented in the computation of lateral pressures.

Backfill to the culvert should be constructed in accordance with appropriate O.P.S.D. Standards (O.P.S.D. 803 series). The backfill should be constructed in 300 mm lifts on alternating sides of the culvert so that the maximum differential in backfill at any time does not exceed 300 mm.

Compaction of the backfill material shall be carried out in accordance with O.P.S.S. 501 series. Excessive vibratory equipment loadings should be prevented from inducing undue lateral pressure on the culvert walls during the compaction procedure.

Settlement of the fill above the culvert can be expected under its own weight. In general, the settlements within the fill are a function of its height as tabulated in Table 9 below.

Table 9 - Settlements within Fill

Height of Fill (H_{FILL}) (m)	Total Settlement (S_T)
0-7	0.5% x H_{FILL}
7-10	0.75% x H_{FILL}
10-12	1% x H_{FILL}

Settlements of granular fills are immediate in nature and should be realized during or immediately following construction. Settlements of cohesive fills, however, are time dependent and should be realized during or immediately following construction.

Temporary Diversion

Temporary diversion of the Jersey Creek at the culvert inlet may be required to facilitate construction in that area. This can be achieved by constructing an impervious earth dam.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer and Bill Cung, Engineer Trainee, utilizing equipment owned and operated by Marathon Drilling. The description of bedrock core samples was carried out by S. Senior, Geological Engineer.

The project was carried out by T. Sangiuliano under the general supervision of Dr. B. Iyer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed by Dr. B. Iyer and approved by Mr. M.S. Devata, Chief Foundation Engineer.

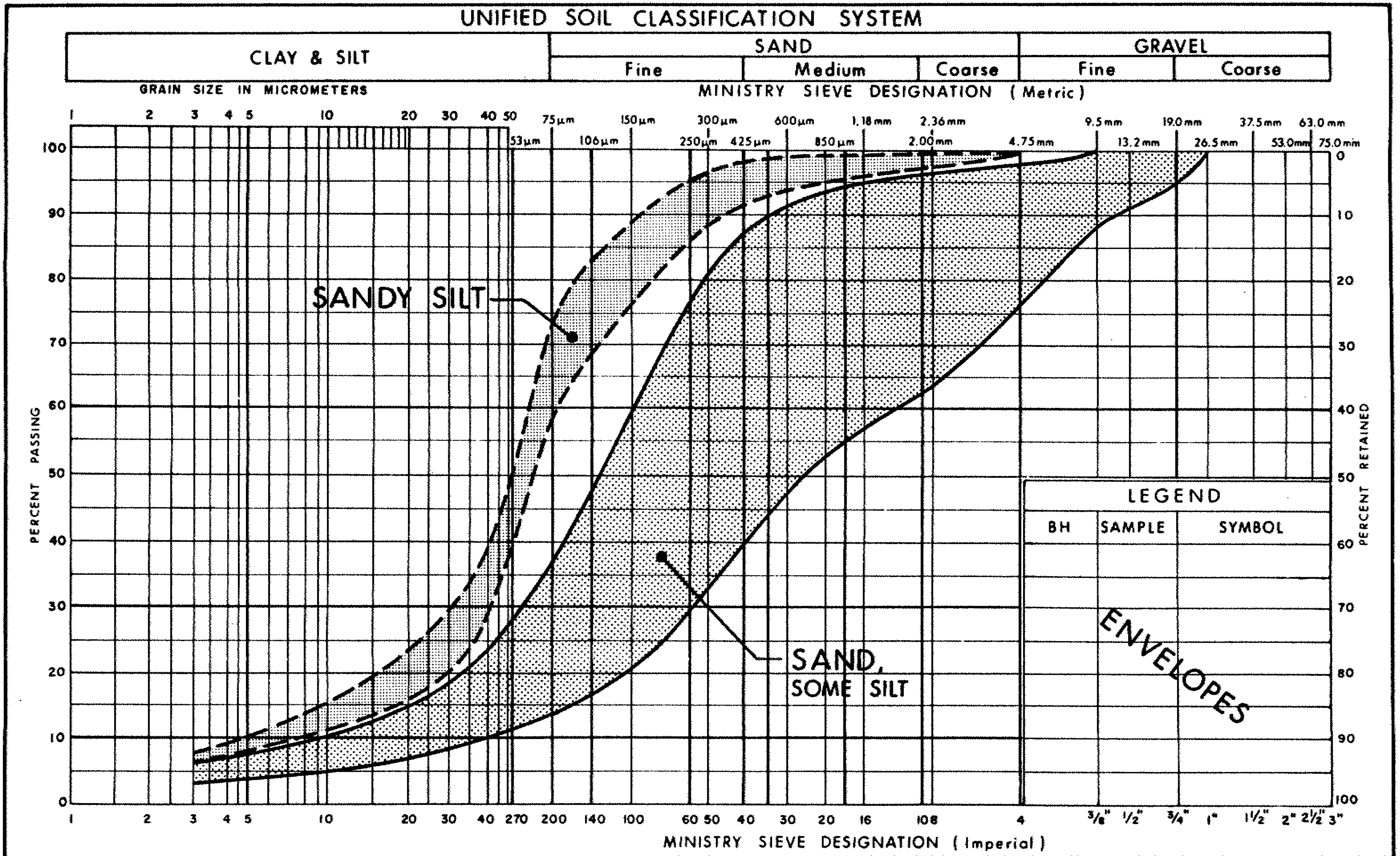
T. Sangiuliano, P.Eng.
Foundation Engineer

LICENSED PROFESSIONAL ENGINEER
A. J. Sangiuliano
PROVINCE OF ONTARIO

M.S. Devata, P.Eng.
Chief Foundation Engineer

APPENDIX

78 12 M



Ministry of Transportation
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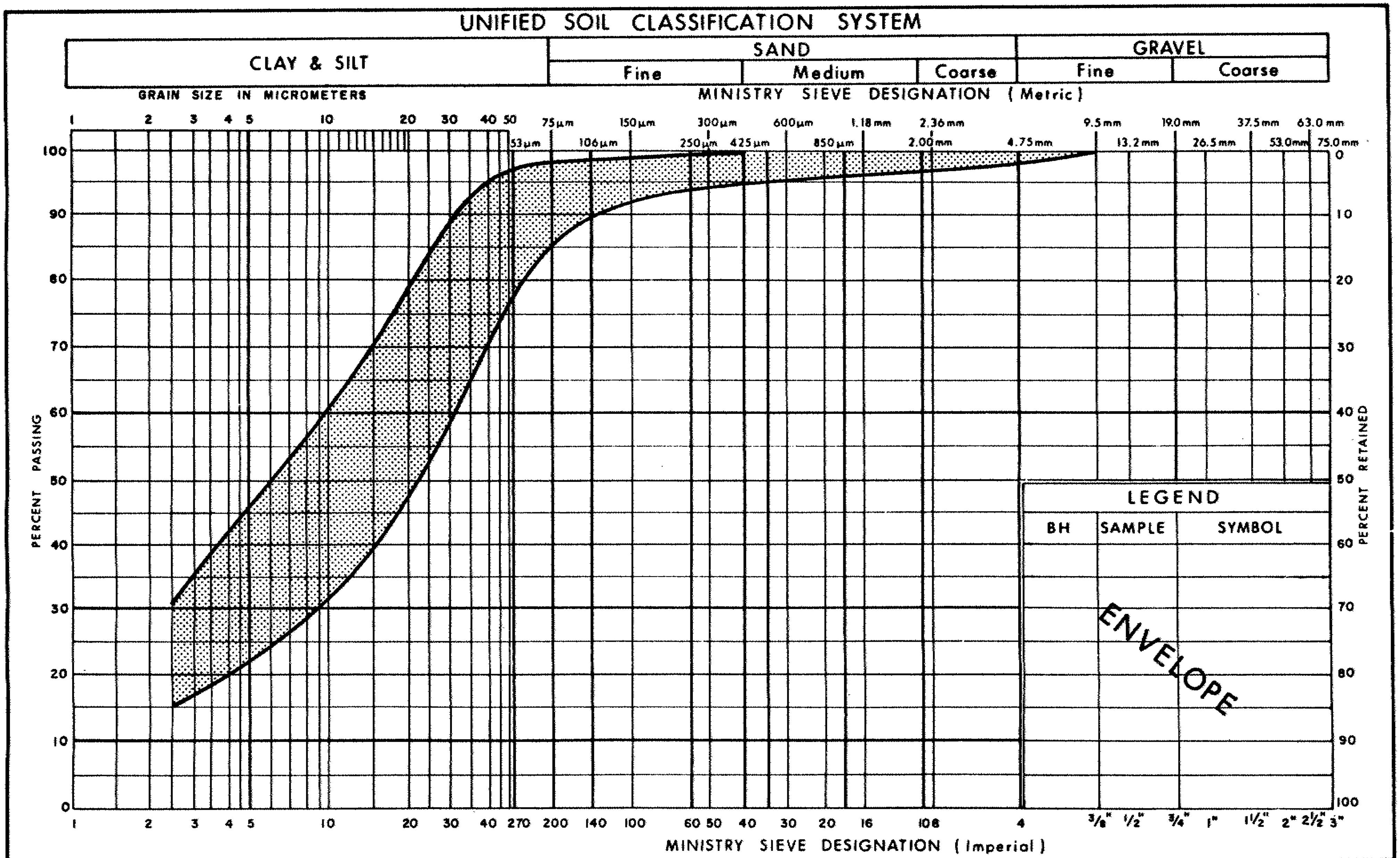
GRAIN SIZE DISTRIBUTION
(FILL MATERIAL)

FIG No 1
WP 141-87-00C

78 12 M



Ministry of Transportation
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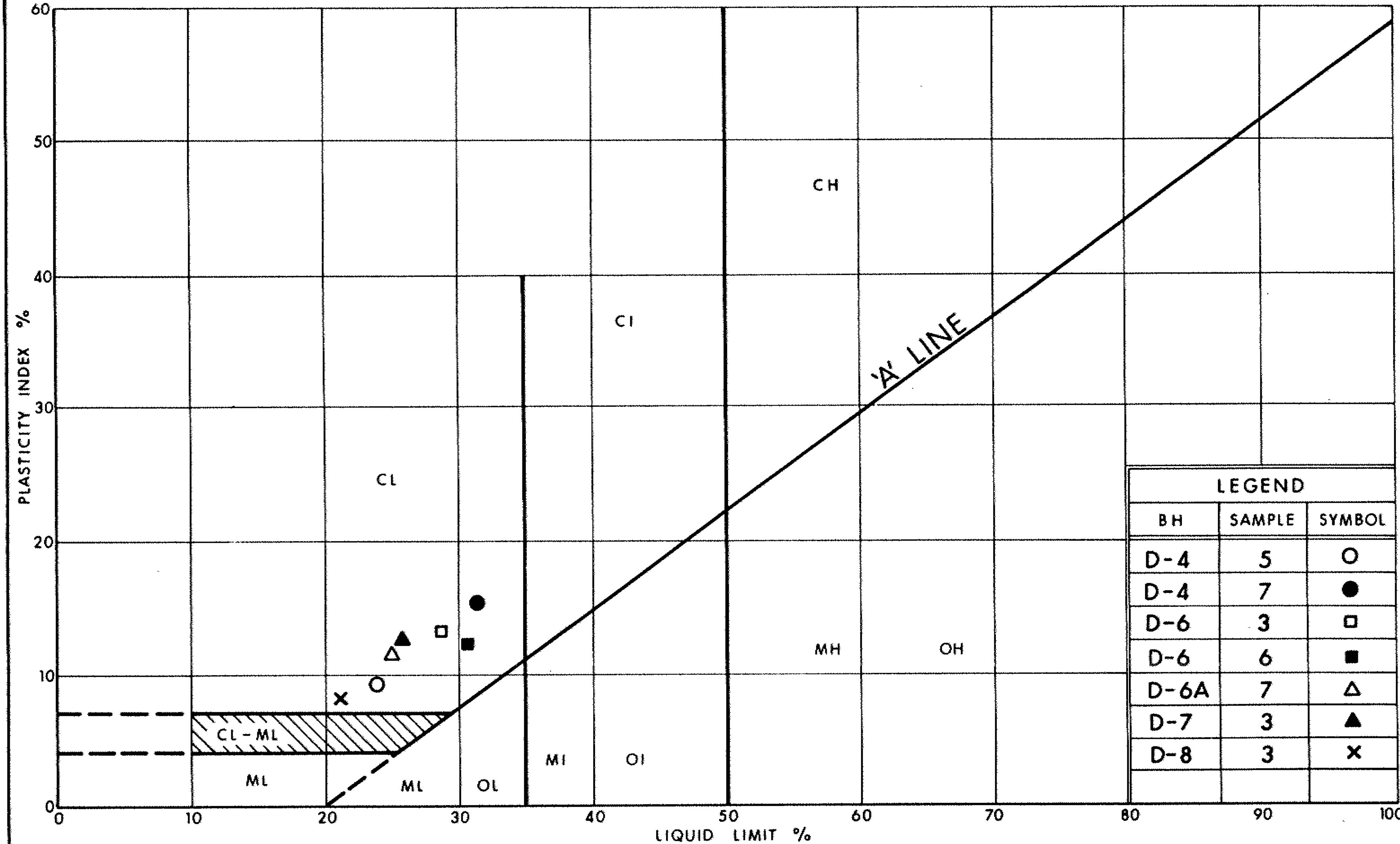
GRAIN SIZE DISTRIBUTION
CLAYEY SILT (FILL)

FIG No 2

WP 141-87-00C

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
D-4	5	○
D-4	7	●
D-6	3	□
D-6	6	■
D-6A	7	△
D-7	3	▲
D-8	3	×

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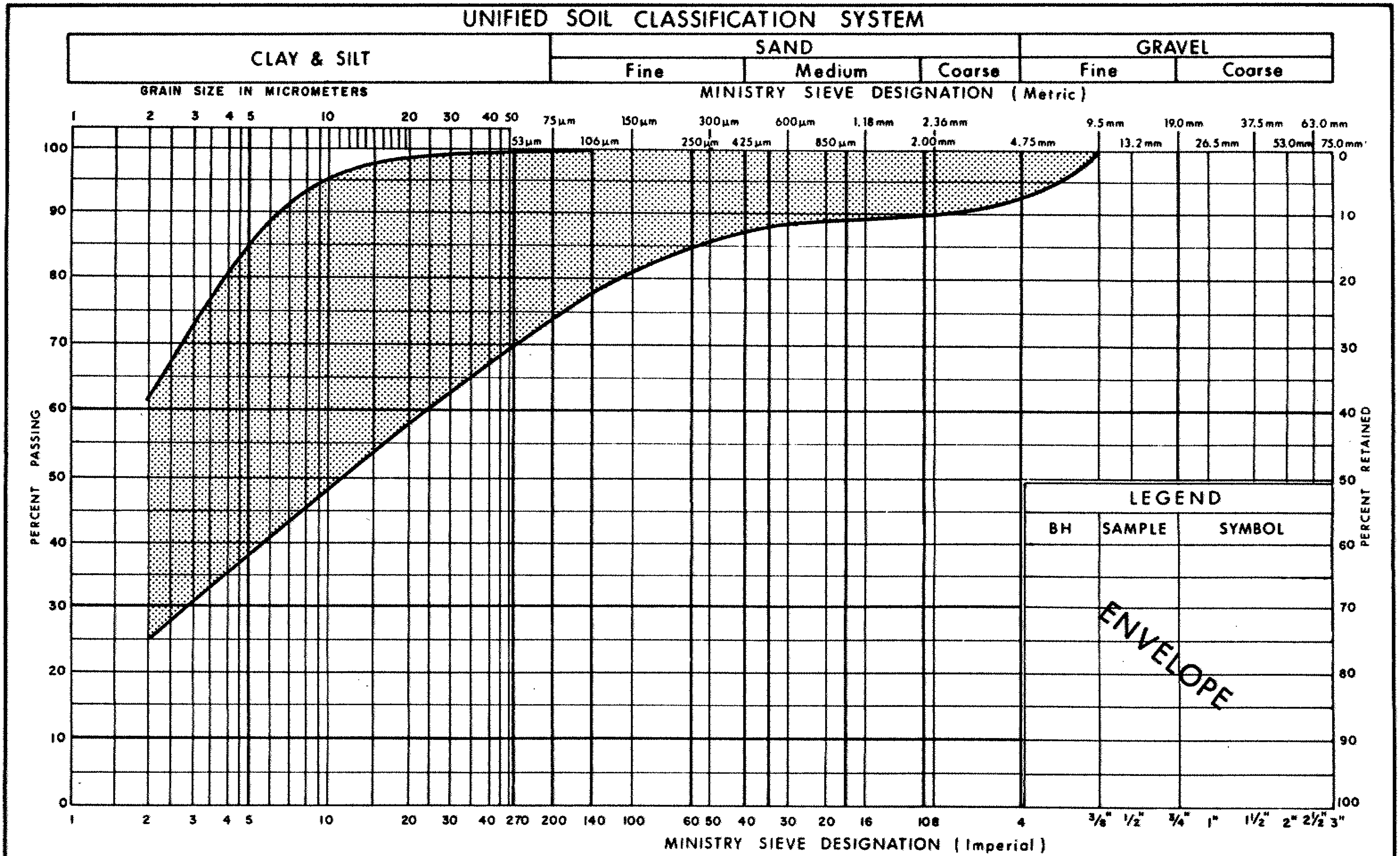
PLASTICITY CHART
CLAYEY SILT (FILL)

FIG No 3
W P 141-87-00C

78 12 M



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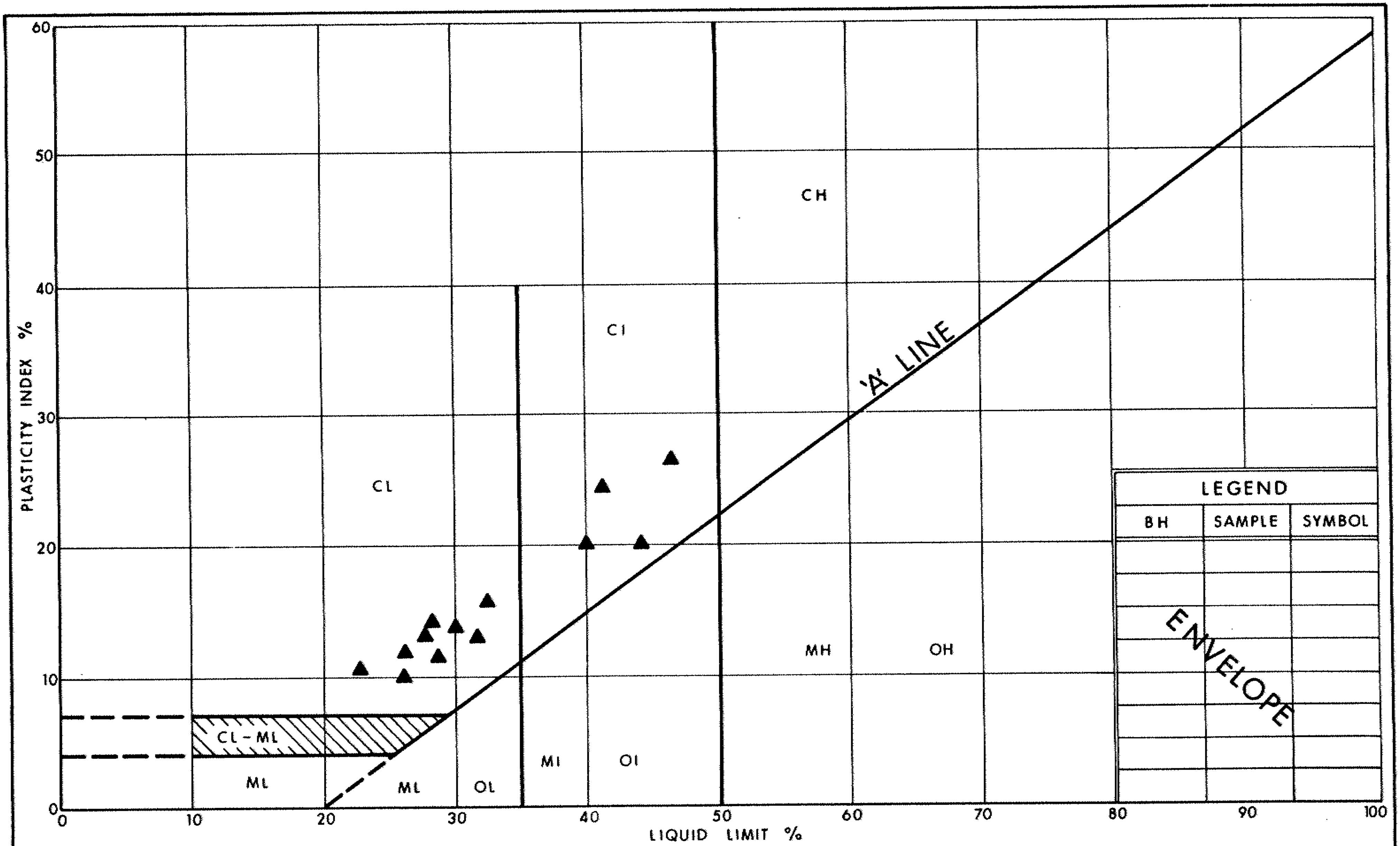
GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAYEY SILT
(Glacial Till)

FIG No 4

W P 141-87-00C

Oct 75, FF-S-21



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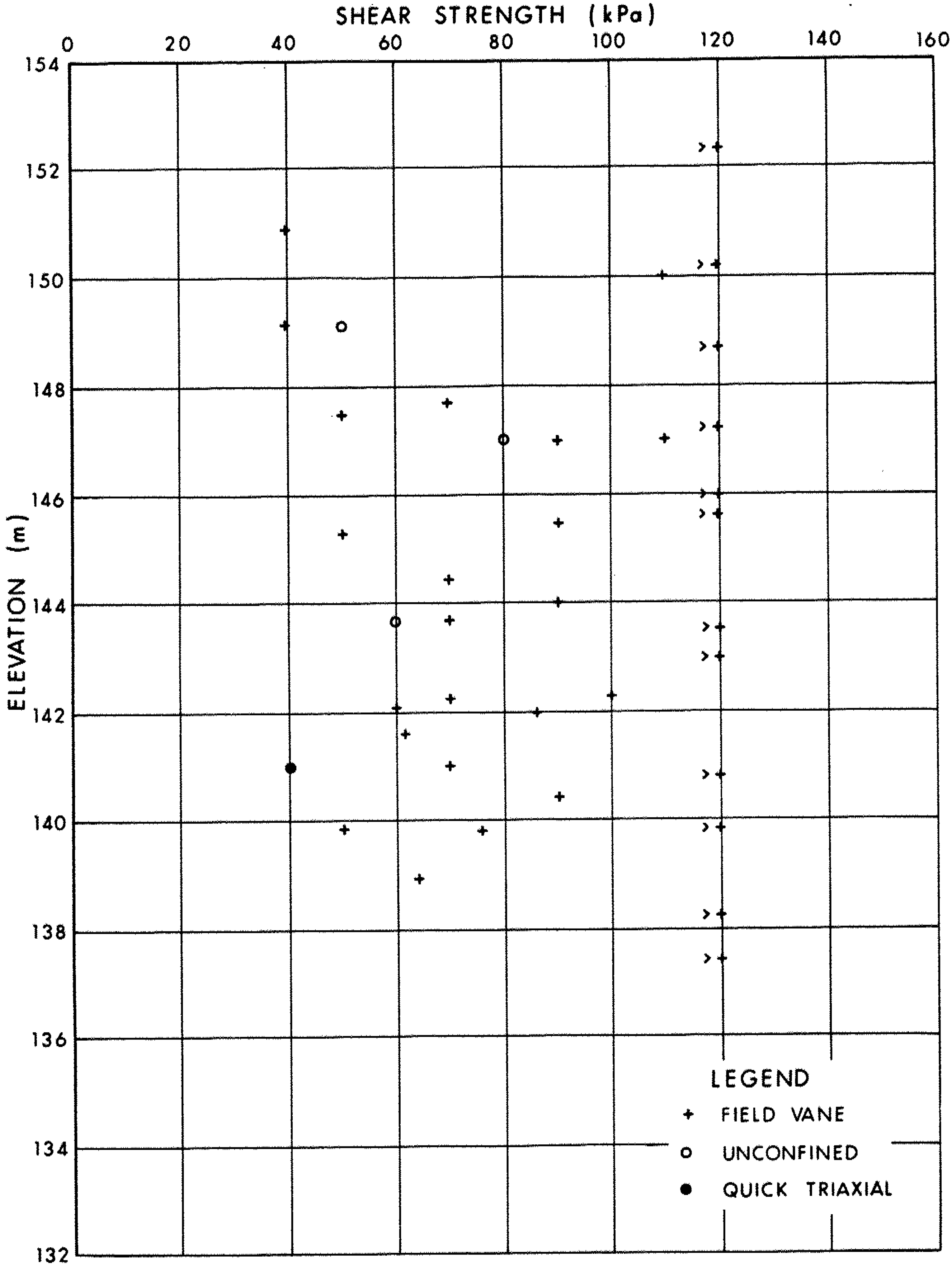
PLASTICITY CHART
SILTY CLAY TO CLAYEY SILT
(Glacial Till)

FIG No 5
W P 141-87-00C

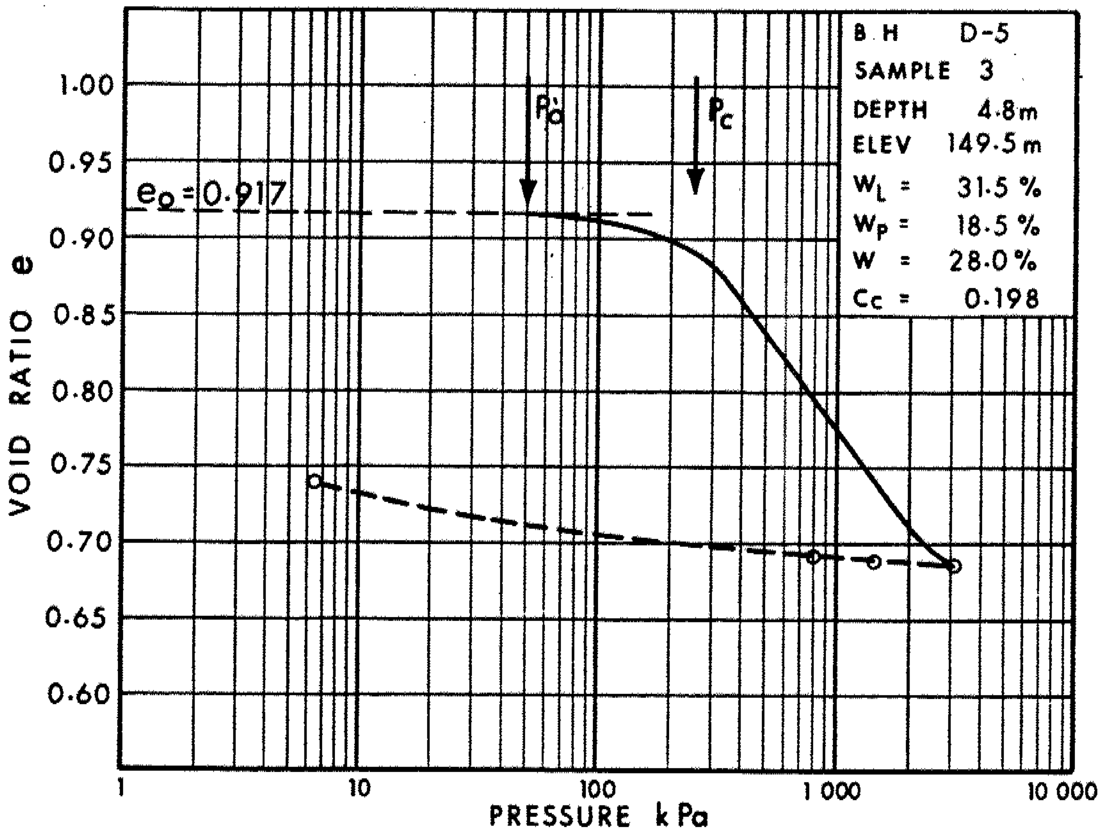
UNDRAINED SHEAR STRENGTH Vs ELEVATION



W P 141-87-00C

Fig 6

VOID RATIO - PRESSURE CURVES



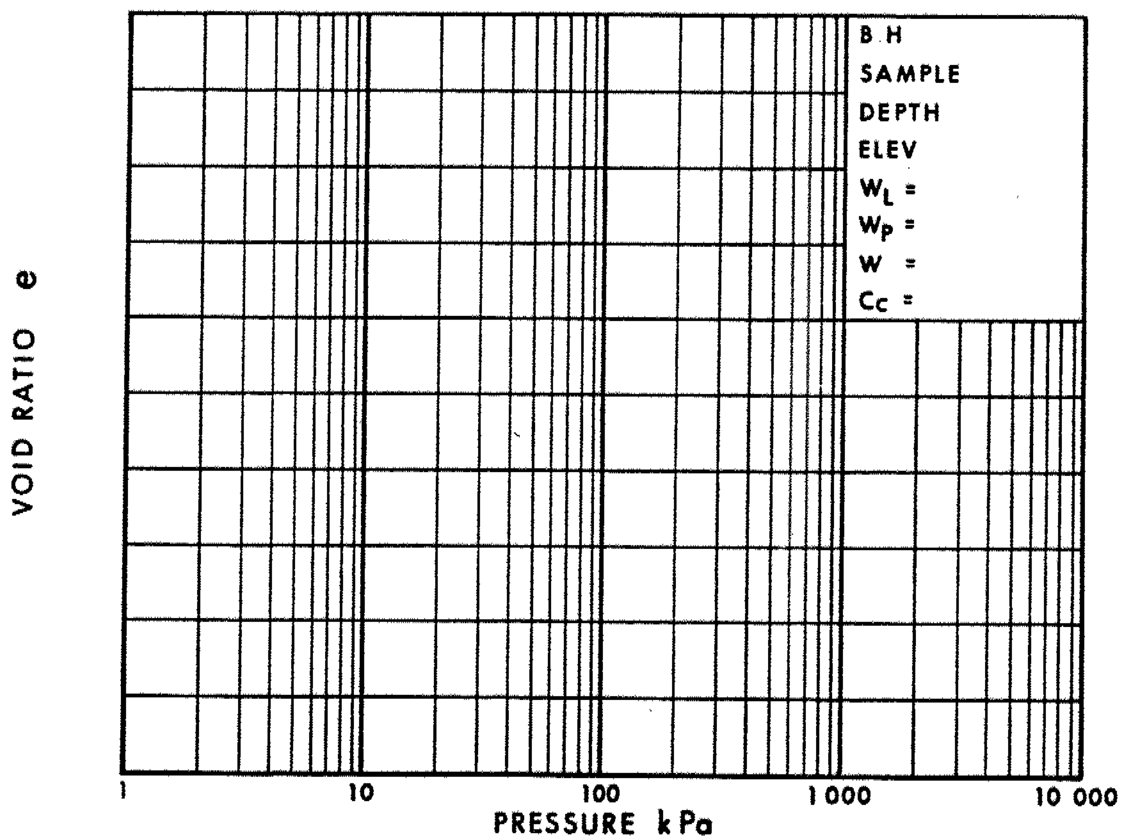
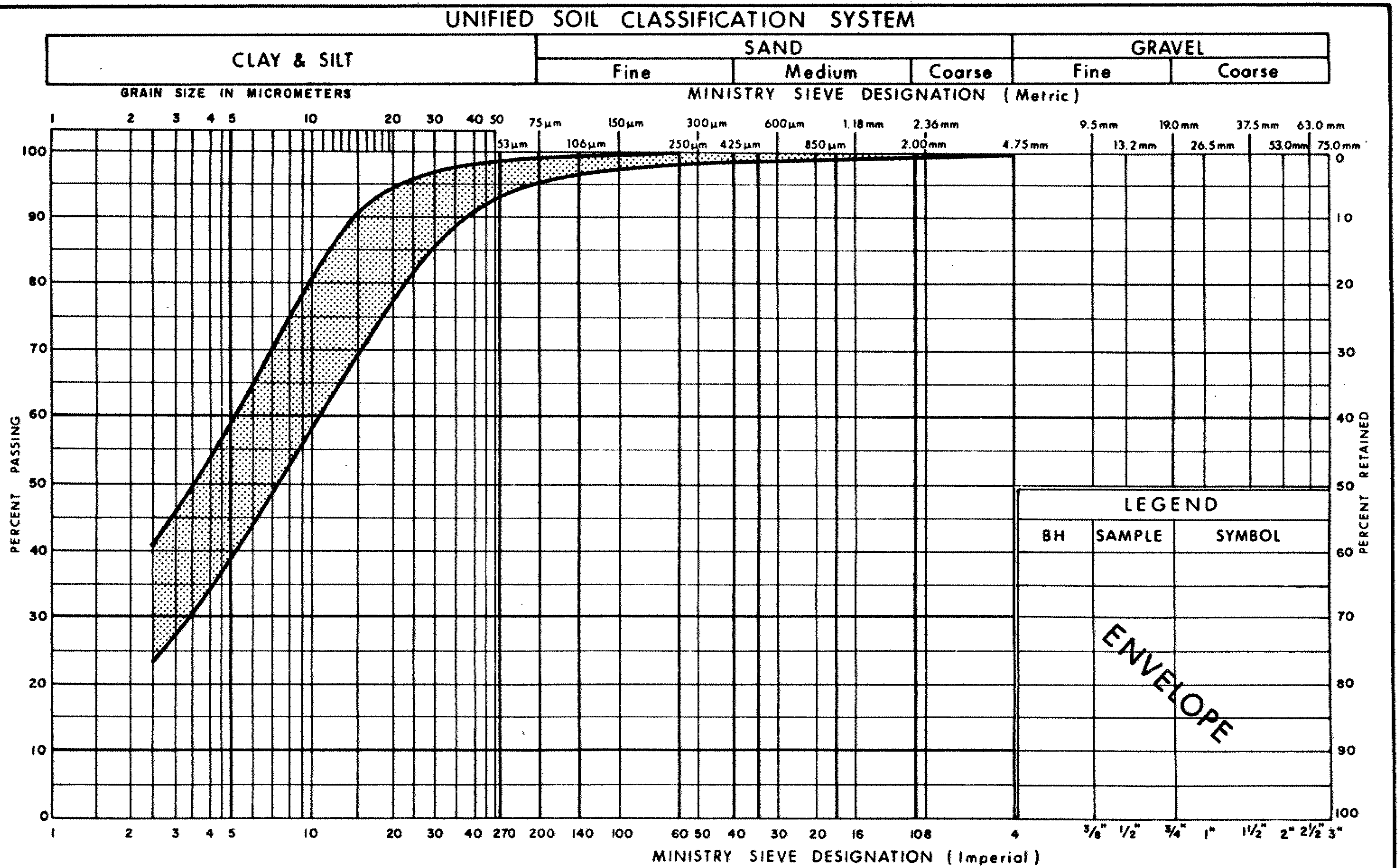


Fig 7

WP 141-87-00C

78 12 M



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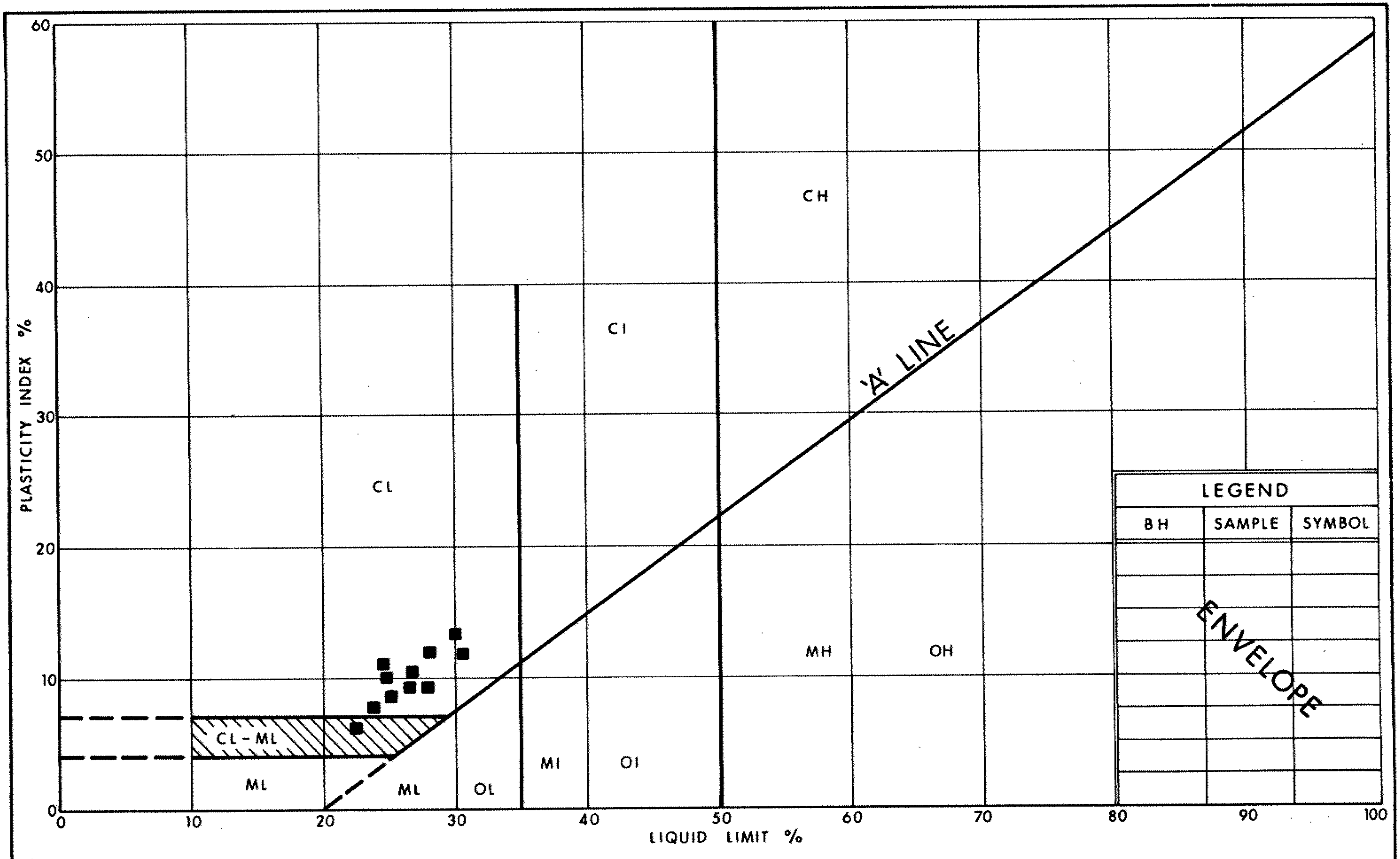
Ontario

GRAIN SIZE DISTRIBUTION
CLAYEY SILT

FIG No 8

W P 141-87-00C

Oct 75, FF-S-21



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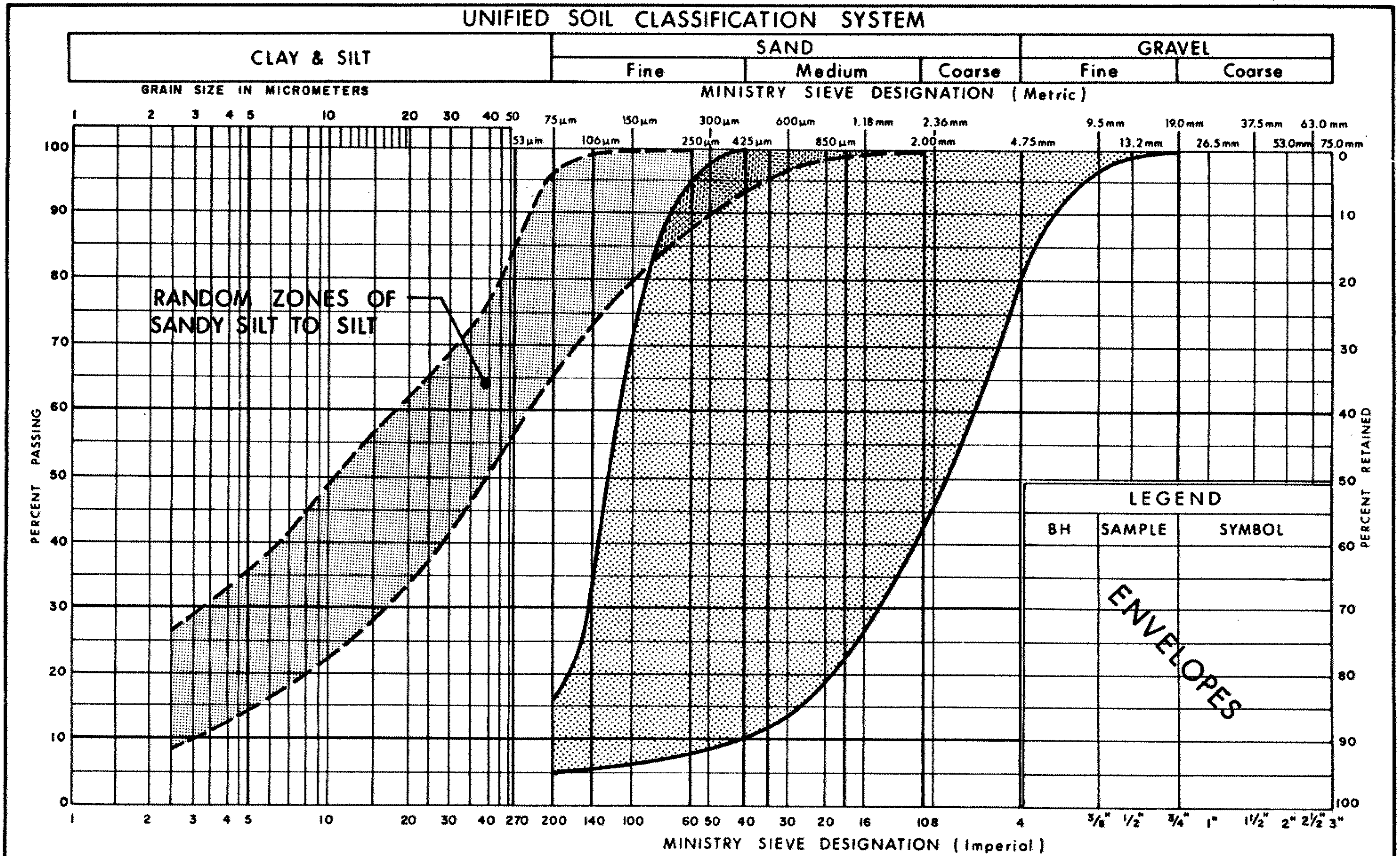
PLASTICITY CHART
CLAYEY SILT

FIG No 9
W P 141-87-00C

78 12 M



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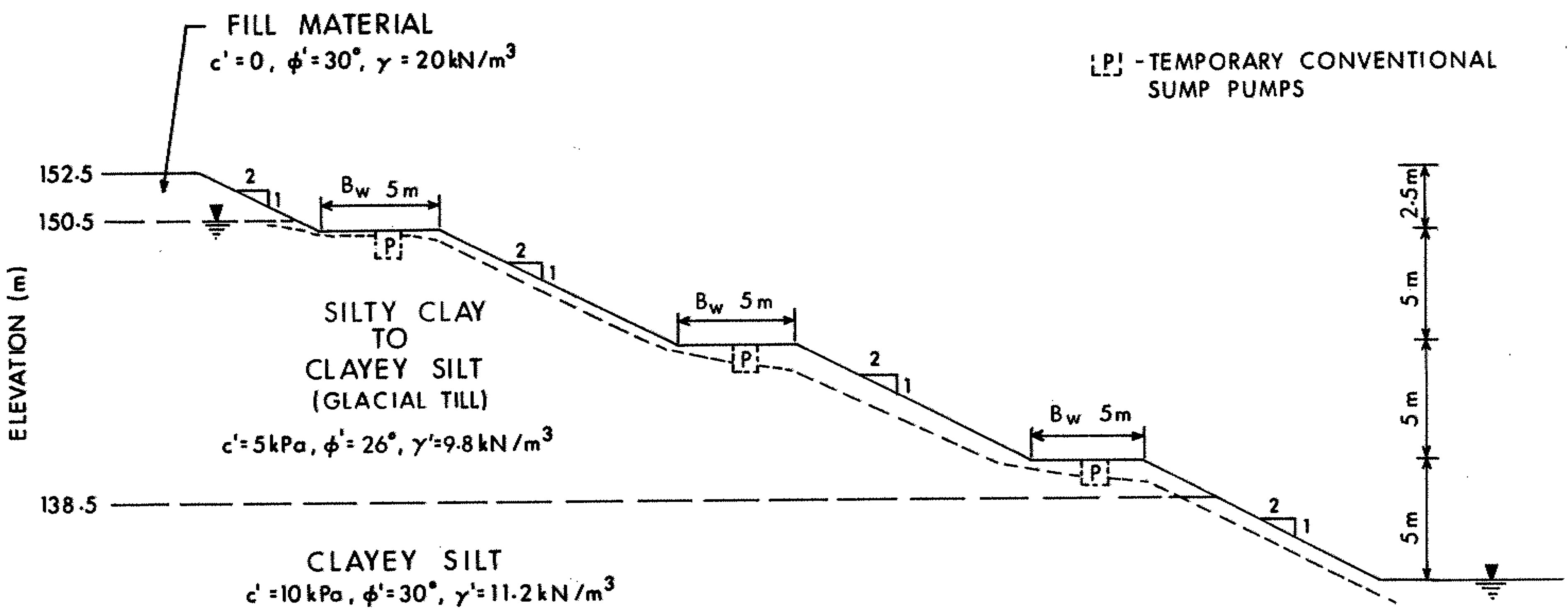
GRAIN SIZE DISTRIBUTION

SAND, SOME SILT

FIG No 10

W P 141-87-00C



TEMPORARY EXCAVATION CUT STABILITY ANALYSIS (DEPTH OF CUT = 17.5 m)

FIG 11

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 (R Q D), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE
e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

ROCK CORE DESCRIPTION
WP 141-87-00

	CORE RECOVERY				CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
C-3	27	45.67-47.19	60	10	45.67-46.28	**OVERBURDEN**, gravel, cobbles, weathered bedrock.
					46.28-47.19	**SHALE**, medium grey to medium dark grey; very fine grained, very thinly laminated; weak to medium strong rock; slightly to medium weathered; extremely close spaced fractures.

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated in zones of poor core recovery)

Logged by: SAS, Soils and Aggregates Section.

ROCK CORE DESCRIPTION

WP 88-78-16

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
D-1	34	46.10-47.22	73	0	46.10-48.90	**SHALE**, medium grey to medium dark grey; very fine grained; very thinly laminated; weak to very weak rock; slightly weathered to moderately weathered; very close to extremely close spaced fractures. Minor interbeds of fine grained argillaceous limestone (5%).
	35	47.22-48.90	100	0		
D-2	22	47.55-49.07	92	7	47.55-49.07	**SHALE**, medium grey to medium dark grey; very fine grained; very thinly laminated; weak to very weak rock; slightly weathered to moderately weathered, intensely weathered sections at 47.60m and 48.18m; very close to extremely close spaced fractures. Minor interbeds of fine grained argillaceous limestone (8%).
D-4	22	44.81-46.33	60	8	44.81-44.98	**OVERBURDEN**, cobbles, weathered, bedrock.
					44.98-46.33	**SHALE**, medium grey to medium dark grey; very fine grained; very thinly laminated; weak to very weak rock; moderately weathered to highly weathered; very close to extremely close spaced fractures. Minor interbeds of fine grained argillaceous limestone (20%).
D-8	25	42.98-44.65	100	15	42.98-44.65	**SHALE**, medium grey to medium dark grey; very fine grained; very thinly laminated; weak to very weak rock; slightly weathered to moderately weathered; very close to extremely close spaced fractures. Minor interbeds of fine grained argillaceous limestone (11%).

***CR** = CORE RECOVERY

***RQD** = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated in zones of poor core recovery)

Logged by: SAS, Soils and Aggregates Section.

Ministry of Transportation Ontario

RECORD OF BOREHOLE No C-1

METRIC

W P 141-87-00C | LOCATION Co-ords: N 4 847 519.6; E 298 137.8 | ORIGINATED BY TS

DIST 6 HWY 407 | BOREHOLE TYPE H.S. Auger | COMPILED BY TS

DATUM Geodetic | DATE 1989 11 24 | CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
136.5	Ground Surface							
0.0						136		
	Trace Organics	1	SS	7				
		2	SS	9				
	Firm to Stiff, Brown	3	SS	11		134		
	Stiff to Hard, Grey	4	SS	22				0 2 73 25
		5	SS	30			21.6	
		6	SS	32		132		
	Clayey Silt							
	Trace of Sand	7	SS	28		130		
		8	SS	30		128	21.3	0 5 65 30
		9	SS	12				
125.8						126		
10.7	Sandy Silt	10	SS	12				0 1 84 15
		11	SS	18		124		0 36 60 4
	Compact, V. Dense	12	SS	120		122		28 32 34 6
	Occ. Gravel Seams	13	SS	120/15cm		120		
		14	SS	90		118		
						116		
		15	SS	94		114		
	Gravel, Boulders and Cobbles	16	AS	–		112		
111.8								
24.7	End of Borehole				Artesian Condition Encountered			
	*Artesian Head 3.0m Above Ground Surface							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No C-2

METRIC

W P 141-87-00C LOCATION Co-ords: N 4 847 540.1; E 298 173.9 ORIGINATED BY BC

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger COMPILED BY BC

DATUM Geodetic DATE 1989 11 28 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
147.4	Ground Surface				
0.0	Sand, Trace Silt Trace Gravel (Fill) Brown, V. Loose to Loose	1	SS	5	
		2	SS	4	10 81 (9)
142.4		3	SS	7	
5.0	Silty Clay to Clayey Silt Trace Gravel Occ. Sand Seams Firm to V. Stiff (Glacial Till)	4	SS	5	
		5	SS	10	
		6	TW	PH	
135.2		7	TW	PH	
12.2	Clayey Silt Trace Sand Grey	8	SS	9	
		9	SS	15	
		10	SS	14	0 4 66 30
		11	SS	15	
		12	SS	13	
		13	SS	11	
		14	SS	12	
		15	SS	13	
	Stiff Firm	16	SS	10	
124.5		17	SS	7	
22.9	Sand Some Silt Grey, Loose	18	SS	5	
122.6		19	SS	8	
24.8	End of Borehole				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (146, 144, 142, 140, 138, 136, 134, 132, 130, 128, 126, 124); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ.

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No C-3

METRIC

W P 141-87-00C | LOCATION Co-ords: N 4 847 561.0; E 298 190.8 | ORIGINATED BY TS

DIST 6 HWY 407 | BOREHOLE TYPE H.S. Auger, NW Casing, Washbore, NQ Core | COMPILED BY TS

DATUM Geodetic | DATE 89 11 22 - 25 | CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

Soil Profile

ELEV / DEPTH	DESCRIPTION
152.2 / 0.0	Ground Surface
	Clayey Silt, Trace Sand, Trace Gravel; Brown / Grey; Firm to V. Stiff; Occ. Sand Seams (Glacial Till); Silt, Tr. Sand
140.0 / 12.2	Clayey Silt, Trace Sand, Grey, Stiff to V. Stiff
122.0 / 30.2	Continued

Samples

NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	8		
2	SS	9		
3	SS	2		
4	TW	PH		
5	SS	12		0 4 61 35
6	SS	5	20.2	
7	SS	12		0 10 85 5
8	SS	16		
9	SS	23		
10	SS	24		
11	SS	26		
12	SS	26	21.7	
13	SS	27		0 8 69 23
14	SS	29		
15	SS	24		
16	SS	21		
17	SS	15		
18	SS	36		
19	SS	14		
20	SS	14		
21	SS	11		
22	SS	14		

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (152 to 124); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30.

Continued

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

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

Continued

Ministry of Transportation Ontario

RECORD OF BOREHOLE No C-3 Cont'd METRIC

W P 141-87-00C LOCATION Co-ords: N 4 847 561.0; E 298 190.8 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger, NW Casing, Washbore, NQ Core COMPILED BY TS

DATUM Geodetic DATE 89 11 22 - 25 CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
122.0	Continued					
121.7 30.5	Clayey Silt					
	Sand Some Silt Grey, Compact to Dense	23	SS	10	120	
		24	SS	39	118	
		25	SS	28	114	0 62 30 8
		26	SS	46	110	
	Occ. Cobbles Boulders and Gravel				108	
105.9 46.3	Bedrock Shale	27	RC	REC 60%	106	RQD = 10%
105.0 47.2	End of Borehole					

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; GROUND WATER CONDITIONS; UNIT WEIGHT γ; STRAT PLOT

OFFICE REPORT ON SOIL EXPLORATION

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

15 -○- 5 (20 / 10) (%) STRAIN AT FAILURE

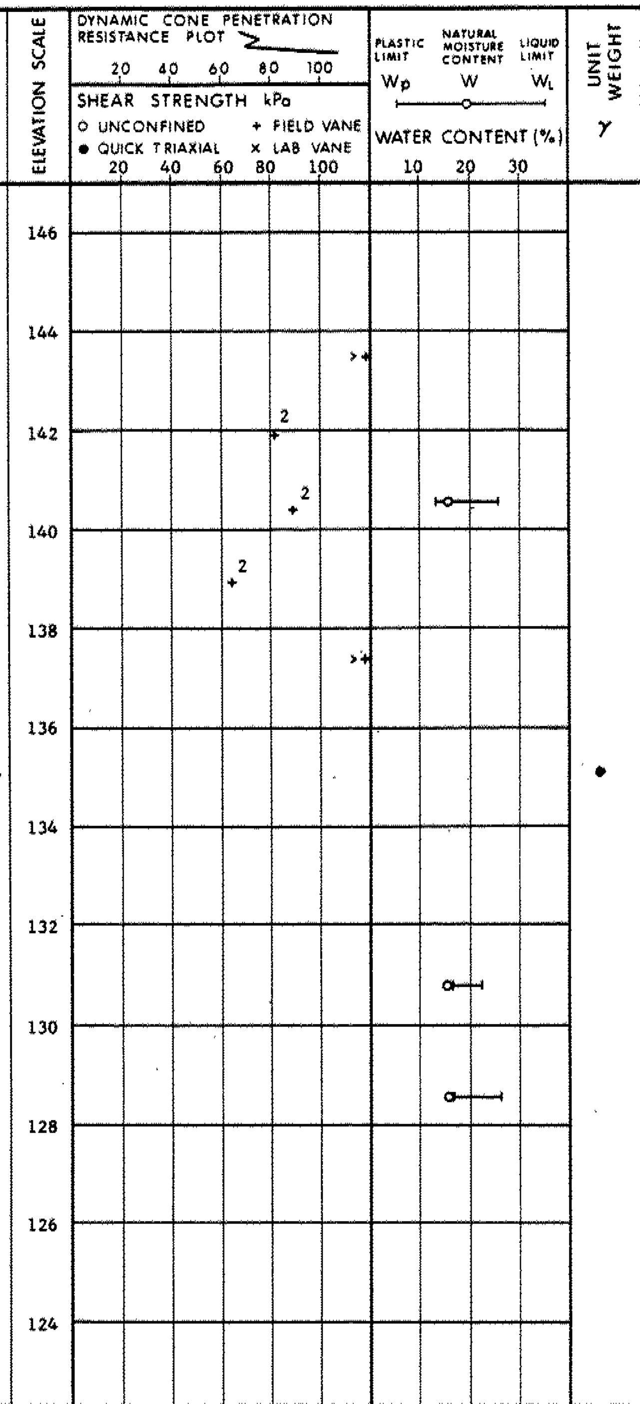
Ministry of Transportation
Ontario

RECORD OF BOREHOLE No C-4

METRIC

W P 141-87-00C | LOCATION Co-ords: N 4 847 580.0; E 298 204.5 | ORIGINATED BY BC
DIST 6 HWY 407 | BOREHOLE TYPE H.S. Auger | COMPILED BY BC
DATUM Geodetic | DATE 1989 11 27 | CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
147.0	Ground Surface					
0.0	Brown / Grey Clayey Silt Some Sand, Tr. Gravel Stiff to V. Stiff Occ. Sand Seams (Glacial Till)	1	SS	12		
		2	SS	6		
		3	SS	7		8 17 47 28
		4	SS	5		
		5	SS	5		
136.3 10.7	Clayey Silt Grey, Stiff to Hard Random Zones of Silt	6	SS	13		
		7	SS	22	Water level	
		8	SS	13		
		9	SS	34		
		10	SS	39		0 1 73 26
		11	SS	25		
		12	SS	23		
		13	SS	19		0 3 67 30
		14	SS	18		
		15	SS	15		
		16	SS	14		
		17	SS	10		
122.2		18	SS	11		
24.8	End of Borehole					



+3, ×5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-1

METRIC

W P 88-78-16	LOCATION Co-ords: N 4 847 491.9; E 298 279.3	ORIGINATED BY TS
DIST 6 HWY 407	BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, BXL Rock Core & Cone Test	COMPILED BY TS
DATUM Geodetic	DATE 89 10 21-30	CHECKED BY

SOIL PROFILE

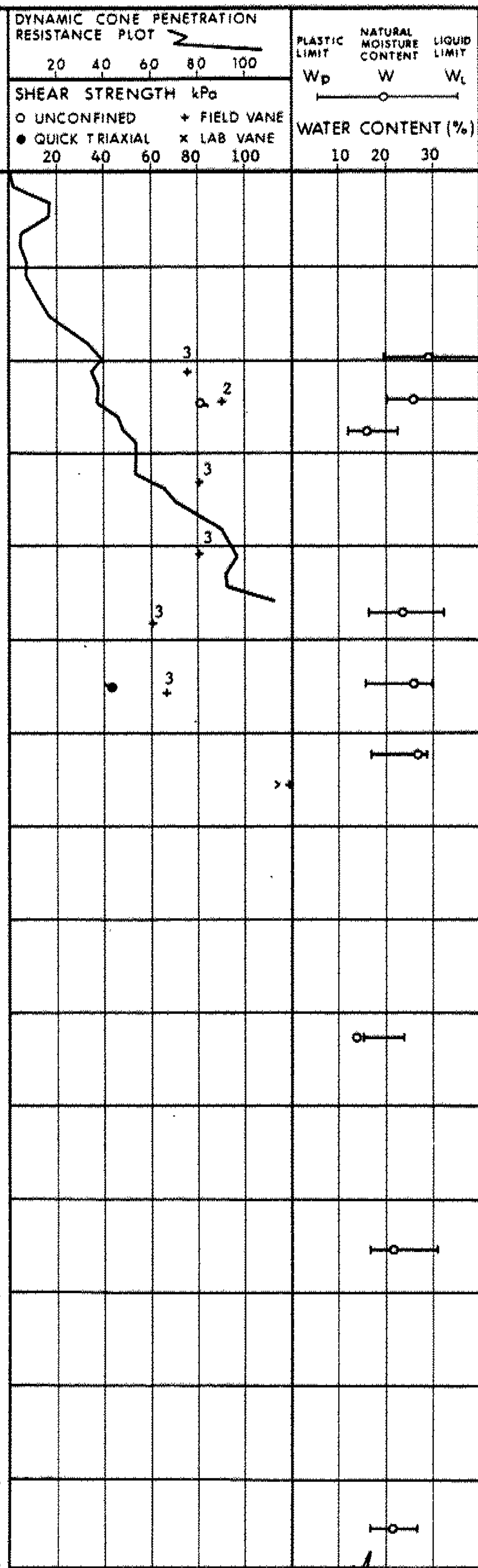
ELEV DEPTH	DESCRIPTION
152.0	Ground Surface
0.0	Sand (Fill) Brown, Compact
150.5 1.5	Silty Clay to Clayey Silt, Brown / Grey, Some Sand, Stiff, Occ. Sand Seams (Glacial Till)
138.3 13.7	Clayey Silt V. Stiff to Hard
121.8 30.2	Continued

SAMPLES

NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AS	-		
2	SS	10		
3	SS	10		
4	SS	6	19.1	0 0 44 56
5	TW	PH	18.8	0 1 38 61
6	SS	9		2 24 49 25
7	SS	9		
8	SS	9		
9	SS	7	20.3	0 3 54 43
10	TW	PH	19.5	0 13 40 45
11	SS	4		
12	SS	21		
13	SS	21		
14	SS	32		
15	SS	30	20.8	0 2 73 25
16	SS	26		
17	SS	32		
18	SS	19		
19	SS	18		
20	SS	20		
21	SS	16		
22	SS	18		0 2 78 20

GROUND WATER CONDITIONS: water level at approx. elev. 150

ELEVATION SCALE: 150 – 122



Continued

+³, ×⁵ : Numbers refer to Sensitivity

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

Continued

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No D-1 Cont'd METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 491.9; E 298 279.3 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, BXL Rock Core & Cone Test COMPILED BY TS

DATUM Geodetic DATE 89 10 21-30 CHECKED BY

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
121.8	Continued					
30.2	Clayey Silt Very Stiff to Hard	23	SS	16		
		24	SS	16	120	
		25	SS	76	118	
		26	SS	45	116	
115.4 / 36.6	Sand Tr. Silt Compact to V. Dense	27	SS	15		0 86 (14)
		28	SS	59	114	
		29	SS	58	112	
		30	SS	65	110	1 89 (10)
		31	SS	33		
	Tr. Gravel	32	SS	44	108	8 85 (7)
105.9		33	SS	129/23cm	106	
46.1	Bedrock Shale Weak to Very Weak	34	BXL RC	REC 73%		RQD = 34%
		35	BXL RC	REC 100%	104	RQD = 0%
103.1 / 48.9	End of Borehole					

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

+3, ×5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-2

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 472.6; E 298 274.6 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, NQ Core COMPILED BY TS

DATUM Geodetic DATE 1989 11 08-11 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	SHEAR STRENGTH kPa (field vane, sensitivity)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL (%)
153.0	Ground Surface								
0.0	Irregular Mixture of Silt, Sand, Slag Ballast (Fill) Brown-Black, Compact	1	SS	11					
150.6 / 2.4	Silty Clay to Clayey Silt, Some Sand, Trace Gravel, Firm to V. Stiff, Occ. Sand Seams (Glacial Till); Brown / Grey	2	SS	7		+2			
		3	SS	7		+2			
		4	SS	8		+2			
		5	TW	PH		o, +2	o	21.0	1 13 58 28
		6	TW	PH		+2			
		7	SS	4		+3	o		4 13 35 48
		8	SS	4					
139.3 / 13.7	Clayey Silt, Firm to V. Stiff	9	SS	22			o		0 4 79 17
		10	SS	20					
		11	SS	23					
		12	SS	12					
		13	SS	20					
		14	SS	18					
		15	SS	13			o		
122.8 / 30.2									

Continued

Continued

+3, x5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-2 Cont'd

METRIC

W P 88-78-16 | LOCATION Co-ords: N 4 847 472.6; E 298 274.6 | ORIGINATED BY TS

DIST 6 HWY 407 | BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, NQ Core | COMPILED BY TS

DATUM Geodetic | DATE 1989 11 08-11 | CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
122.8	Continued							
30.2	Clayey Silt Firm to Very Stiff	16	SS	9		122		
		17	SS	5		120, 118		
116.1 / 36.9	Sand Tr. Silt Compact to V. Dense Occ. Gravelly Seams	18	SS	50		116, 114		
		19	SS	20		112	~22	17 78 (5)
		20	SS	56		110, 108		
		21	SS	50		106		
105.5 / 47.5	Bedrock Shale Weak to Very Weak	22	RC	REC 92%		104		RQD = 20%
103.9 / 49.1	End of Borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-3

METRIC

W P 141-87-00D | LOCATION Co-ords: N 4 847 504.0; E 298 204.0 | ORIGINATED BY TS
DIST 6 HWY 407 | BOREHOLE TYPE H.S. Auger | COMPILED BY TS
DATUM Geodetic | DATE 1989 11 27 | CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
136.0	Ground Surface									
0.0	Interbedded Layers of Sand and Gravel Tr. Organics; Brown / Grey	1	SS	4	▼					
		2	SS	2		134	+2			
		3	SS	14						
		4	SS	12						
	Clayey Silt, Tr. Sand, Tr. Gravel, Stiff to Hard	5	SS	22		132				0 5 61 34
		6	SS	19						
						130				
		7	SS	27						
		8	SS	20		128				0 0 77 23
		9	SS	20						
						126				
124.9		10	SS	15						
11.1	Silt, Tr. Clay, Tr. Sand, Loose / V. Dense					124				
		11	SS	7						
		12	SS	85		122				
		13	SS	100/15cm						0 5 85 10
						120				
119.1		14	SS	120/10cm						
16.9	End of Borehole									

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE
PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L

+3, ×5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No D-4

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 539.4; E 298 210.1 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger, NW Casing, Washbore, NQ Rock Core COMPILED BY TS

DATUM Geodetic DATE 89 11 13-21 CHECKED BY

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
152.1	Ground Surface
0.0	Clayey Silt With Interbedded Layers of Sand (Fill) Brown to Grey V. Soft to Stiff
139.9 / 12.2	Clayey Silt Grey, Stiff to Hard (with Sandy Silt layer)
121.9 / 30.2	Continued

SAMPLES

NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	SHEAR STRENGTH kPa	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	2				
2	SS	3				
3	SS	3				0 32 64 4
4	SS	6				
5	SS	6		+2	20.0	0 2 78 20
6	SS	8				
7	SS	15			20.2	4 22 49 25
8	SS	20				
9	SS	25				0 1 79 20
10	SS	30			22.0	
11	SS	53	▼			
12	SS	32				
13	SS	17			21.2	
14	SS	11				
15	SS	13				0 0 88 12

ELEVATION SCALE: 152, 150, 148, 146, 144, 142, 140, 138, 136, 134, 132, 130, 128, 126, 124, 122

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

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

Continued Continued

+3, ×5 : Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-4 Cont'd METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 539.4; E 298 210.1 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger, NW Casing, Washbore, NQ Rock Core COMPILED BY TS

DATUM Geodetic DATE 89 11 13 CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
121.9	Continued										
30.2	Clayey Silt Grey Stiff to Hard		16	SS	10		120				
118.6 / 33.5	Sand Some Silt Grey, Compact to V. Dense		17	SS	45		118				
			18	SS	20		116, 114				
			19	SS	72		112		o (approx. 10)		0 85 14 1
	Occ. Cobbles Boulders and Gravel		20	SS	120/8 cm		110				
107.1			21	SS	65		108				
45.0	Bedrock Shale		22	NQ RC	REC 60%		106				RQD = 8%
105.8 / 46.3	End of Borehole										

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No D-4A

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 520.0; E 298 212.0 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE B-Casing, Washbore COMPILED BY TS

DATUM Geodetic DATE 1989 11 30 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L / WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
143.5	Ground Surface										
0.0	Sand, Tr. Gravel (Fill) Brown, Loose to Compact					*					
			1	SS	6		142				
			2	SS	21		140				
138.9 / 4.6	Clayey Silt With Interbedded Layers of Sand (Fill) Brown, Stiff to Hard		3	SS	37		138				
			4	SS	22						
136.9 / 6.6	End of Borehole *Borehole Dry										

OFFICE REPORT ON SOIL EXPLORATION

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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-5

METRIC

W P 141-87-00D
LOCATION Co-ords: N 4 847 605.0; E 298 115.0
ORIGINATED BY TS

DIST 6 HWY 407
BOREHOLE TYPE H.S. Auger
COMPILED BY TS

DATUM Geodetic
DATE 1989 11 16-17
CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
154.3	Ground Surface										
0.0	Silty Clay to Clayey Silt Trace Gravel Grey, Firm to V. Stiff Occ. Sand Seams (Glacial Till)					*	154				
			1	SS	7		152				
			2	SS	3						
			3	TW	PH		150			19.5	0 4 61 35
			4	TW	PH		148				
			5	SS	11		146				
			6	SS	8		144				
			7	SS	10					20.1	6 7 60 27
			8	SS	5		142				
			9	SS	10		140				
139.1 / 15.2	Clayey Silt Grey, Stiff to V. Stiff		10	SS	12		138				
			11	SS	22					20.0	0 2 74 24
			12	SS	32		136				
134.0			13	SS	22						
20.3	End of Borehole *Borehole Dry										

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-6

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 511.1; E 298 240.4 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger COMPILED BY TS

DATUM Geodetic DATE 89 11 20 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR SA SI CL (%)
152.5	Ground Surface							
0.0	Sand, Tr. Silt (Fill) Brown, V. Loose	1	SS	4	*	152		
150.5 / 2.0	Clayey Silt With Interbedded Layers of Sand (Fill) Brown, Firm	2	SS	6		150		
		3	SS	6		148		0 5 73 22
		4	SS	5		146		
		5	SS	7			19.4	0 2 84 14
		6	SS	6		144	19.2	0 14 64 22
141.8 / 10.7	Sand, Some Silt (Fill) Brown, Very Loose to Loose	7	SS	4		142		1 79 15 5
		8	SS	6		140		
138.6 / 13.9	Clayey Silt, Tr. Gravel Tr. Organics Grey, Firm to Stiff	9	SS	7				6 72 18 4
		10	SS	14		138		
136.8 / 15.7	End of Borehole							
	*Borehole Dry							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30

OFFICE REPORT ON SOIL EXPLORATION

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No D-6A

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 504.0; E 298 229.0 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE B-Casing, Washbore COMPILED BY TS

DATUM Geodetic DATE 1989 11 29 CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
143.5	Ground Surface										
0.0	Sand, Some Gravel, Trace Silt (Fill) Brown, V. Loose to Compact		1	SS	2	*					
			2	SS	6						20 77 (3)
			3	SS	8		142				
			4	SS	9						
			5	SS	12		140				26 59 (15)
138.9 / 4.6	Clayey Silt (Fill) Brown, V. Stiff		6	SS	19		138				
137.3 / 6.2	Clayey Silt Grey, Tr. Organics		7	SS	29						4 6 65 25
136.9 / 6.6	End of Borehole										
	* Borehole Dry										

OFFICE REPORT ON SOIL EXPLORATION

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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-7

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 530.8; E 298 232.1 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger COMPILED BY TS

DATUM Geodetic DATE 89 11 20 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
152.5	Ground Surface								
0.0	Silty Sand to Sandy Silt (Fill) V. Loose to Loose				*				
		1	SS	2		11	62	23	4
		2	SS	3					
	Clayey Silt Brown, Firm	3	SS	8		0	12	66	22
		4	SS	4					
		5	SS	5		2	41	54	3
		6	SS	5					
		7	SS	8					
		8	SS	9		0	30	66	4
		9	SS	9		1	63	31	5
		10	SS	8		4	73	19	4
136.3 / 16.2	End of Borehole Auger Refusal Probable Culvert Roof								

*Borehole Dry

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (152, 150, 148, 146, 144, 142, 140, 138); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30); UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No D-8

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 516.1; E 298 252.3 ORIGINATED BY TS

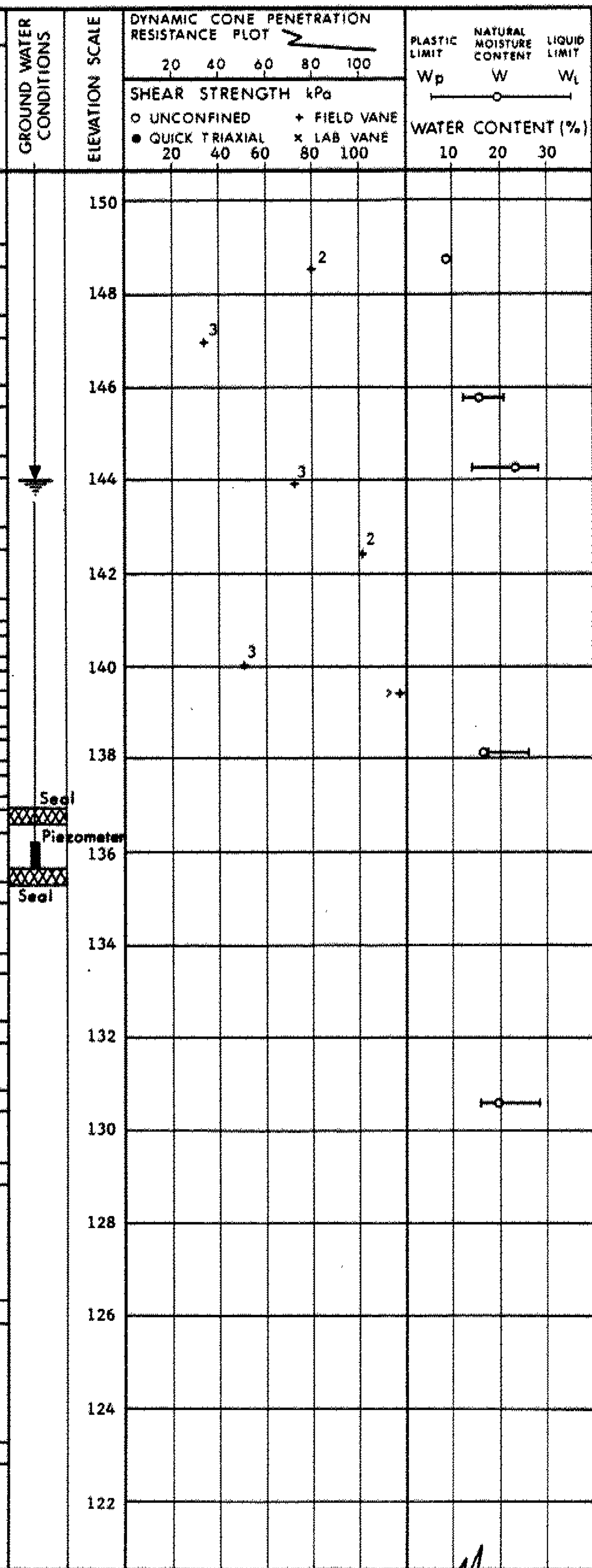
DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, NQ Rock Core COMPILED BY TS

DATUM Geodetic DATE 89 11 02-08 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
150.6	Ground Surface					
0.0	Sand, Some Silt With Interbedded Layers of Clayey Silt Brown, Loose (Fill)	1	SS	9		14 56 29 1
		2	SS	6		
		3	SS	4		0 13 75 12
144.5 / 6.1	Clayey Silt Grey, Firm to Stiff Occ. Sand Seams (Glacial Till)	4	SS	4		0 7 52 41
		5	SS	8	20.2	
		6	SS	12		
		7	SS	4		
139.9 / 10.7	Clayey Silt Grey, Firm to Hard	8	SS	17		
		9	SS	22		
		10	SS	19		2 1 72 25
		11	SS	22	20.3	
		12	SS	28		
		13	SS	25	21.6	
		14	SS	23		
		15	SS	19		
		16	SS	16		0 5 60 35
		17	SS	14		
		18	SS	18		
		19	SS	10		
120.4 / 30.2	Continued					

GROUND WATER CONDITIONS: Seal, Piezometer, Seal



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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-8 Cont'd — METRIC

W P 88-78-16 | LOCATION Co-ords: N 4 847 516.1; E 298 252.3 | ORIGINATED BY TS

DIST 6 HWY 407 | BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, NQ Rock Core | COMPILED BY TS

DATUM Geodetic | DATE 89 11 02-08 | CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
120.4	Continued								
30.2		20	SS	9		120			
						118			
	Silt, Tr. Sand V. Dense	21	SS	107			plotted		0 7 79 14
						116			
	Clayey Silt Grey Firm to Hard	22	SS	36		114			
111.9						112			
38.7	Sand With Silt Grey, Compact	23	SS	12		110			
107.7	Some Gravel	24	SS	120/15 cm		108	plotted		17 38 31 14
42.9	Bedrock Shale Weak to Very Weak	25	NQ RC	REC 100%					RQD = 15%
105.9						106			
44.7	End of Borehole								

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

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

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

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10

METRIC

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

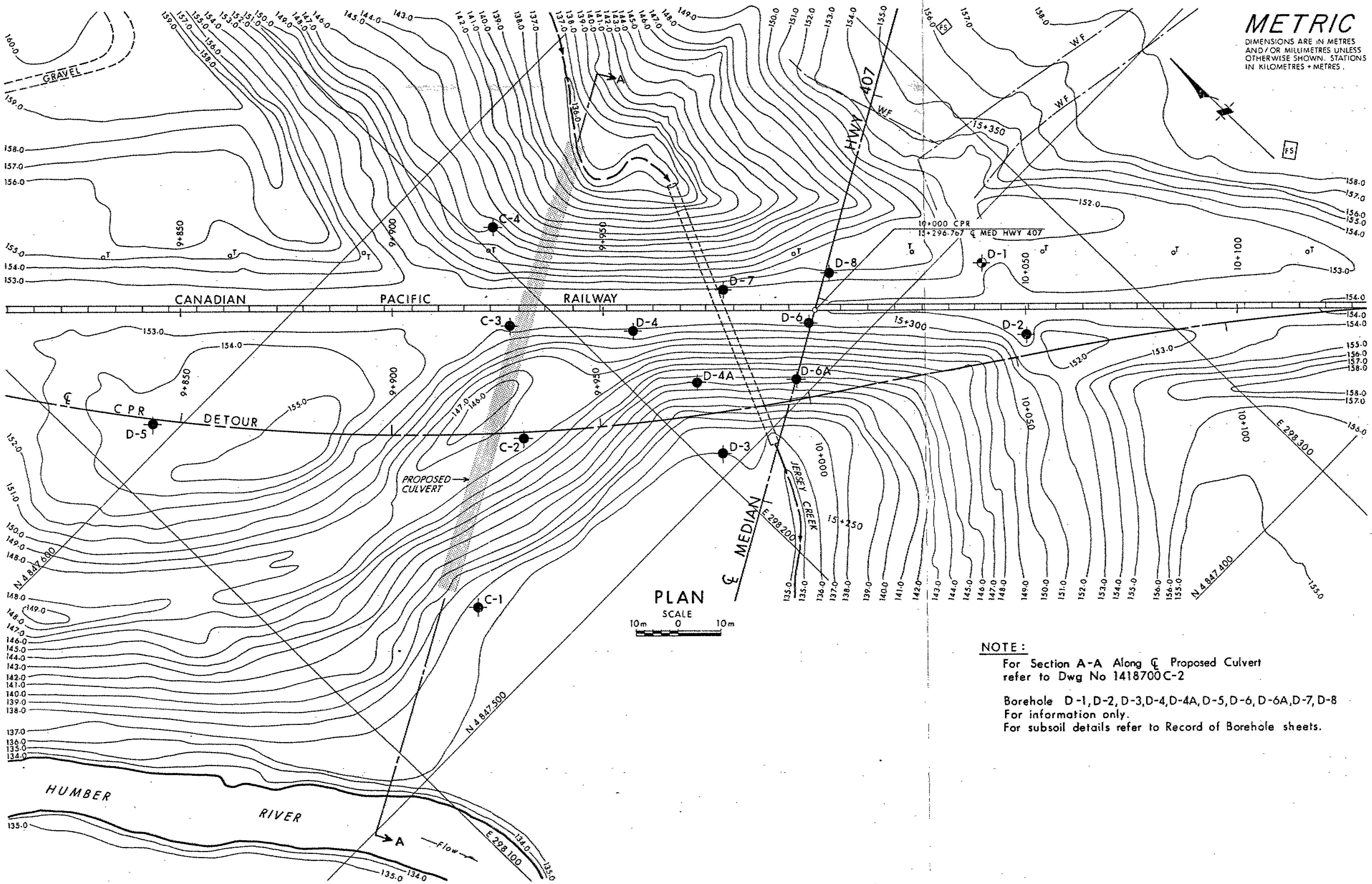
CONT No
WP No 141-87-00C

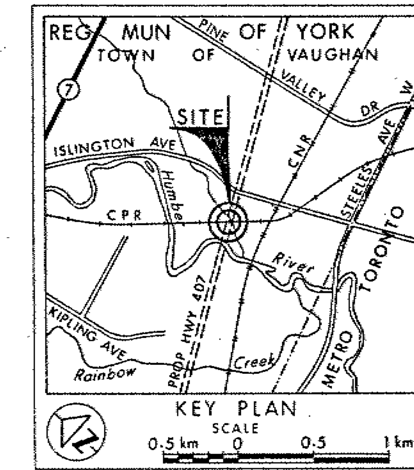
SHEET

JERSEY CREEK CULVERT

AT HWY 407/CPR SUBWAY

BORE HOLE LOCATIONS & SOIL STRATA





NOTE:
For Section A-A Along ℄ Proposed Culvert refer to Dwg No 1418700C-2

Borehole D-1, D-2, D-3, D-4, D-4A, D-5, D-6, D-6A, D-7, D-8
For information only.
For subsoil details refer to Record of Borehole sheets.

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

No	ELEVATION	CO-ORDINATES NORTH	EAST
C-1	136.5	4 847 519.6	298 137.8
C-2	147.4	4 847 540.1	298 173.9
C-3	152.2	4 847 561.0	298 190.8
C-4	147.0	4 847 580.0	298 204.5
D-1	152.0	4 847 491.9	298 279.3
D-2	153.0	4 847 472.6	298 274.6
D-3	136.0	4 847 504.0	298 204.0
D-4	152.1	4 847 539.4	298 210.1
D-4A	143.5	4 847 520.0	298 212.0
D-5	154.3	4 847 605.0	298 115.0
D-6	152.5	4 847 511.2	298 240.4
D-6A	143.5	4 847 504.0	298 229.0
D-7	152.5	4 847 530.8	298 232.1
D-8	150.6	4 847 516.1	298 252.3

=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

Geocres No 30M13-96

HWY No 407			DIST 6
SUBM'D TS	CHECKED YL	DATE 90 03 14	SITE
DRAWN GP	CHECKED	APPROVED	DWG 1418700C-1

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10

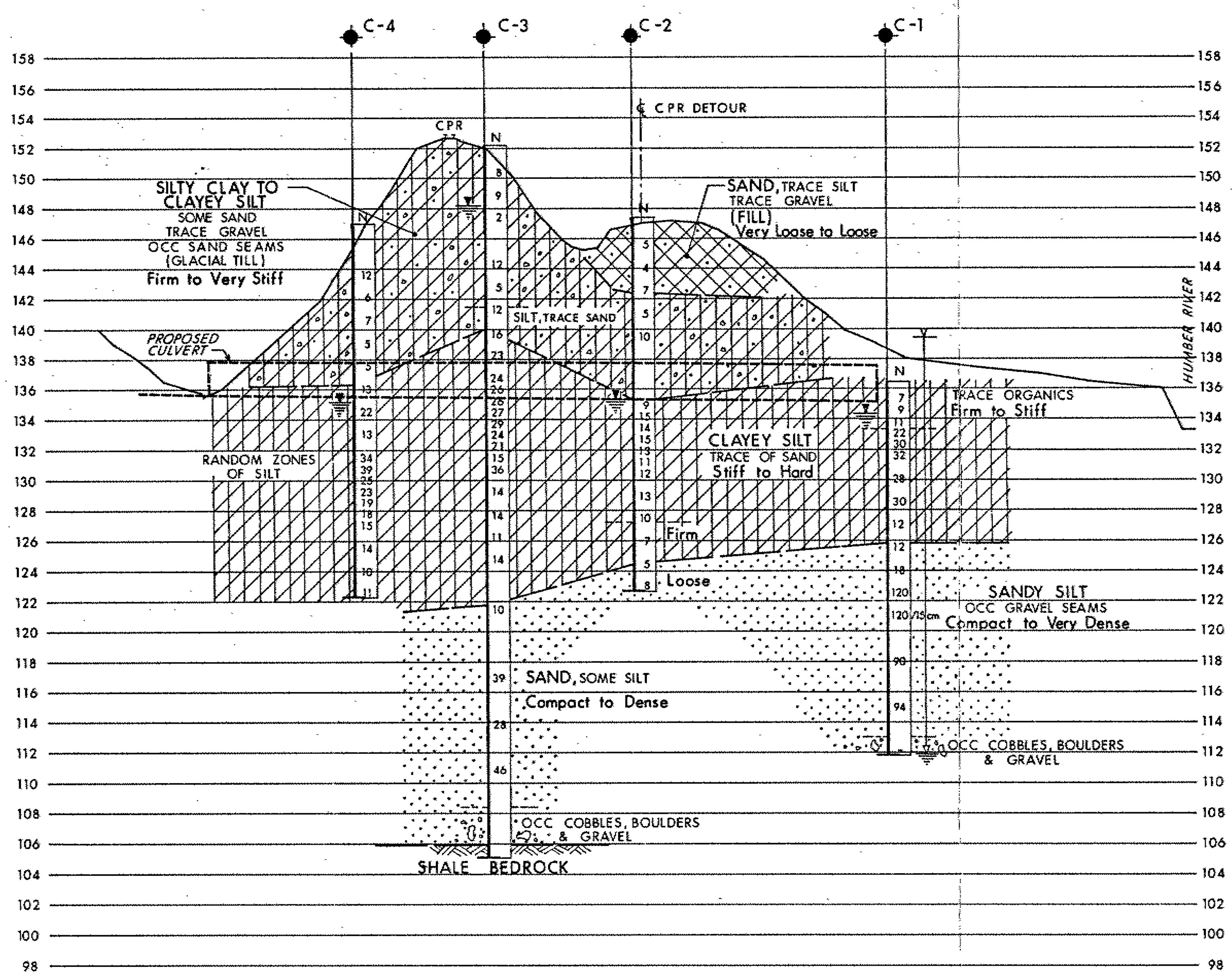
METRIC

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

CONT No
WP No 141-87-00C

JERSEY CREEK CULVERT
AT HWY 407/CPR SUBWAY
BORE HOLE LOCATIONS & SOIL STRATA

SHEET



SECTION A-A ALONG ℄ PROPOSED CULVERT

SCALE
10m 5 0 10m Hor
4m 2 0 4m Vert

NOTE:
For Plan refer to Dwg No 1418700C-1

SEE DWG No 1418700C-1

KEY PLAN
SCALE

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 89 11
	Head ARTESIAN WATER Encountered

No	ELEVATION	CO-ORDINATES NORTH	EAST
C-1	136.5	4 847 519.6	298 137.8
C-2	147.4	4 847 540.1	298 173.9
C-3	152.2	4 847 561.0	298 190.8
C-4	147.0	4 847 580.0	298 204.5

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

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

Geocres No 30M13-96

HWY No 407			DIST 6
SUBM'D TS	CHECKED	DATE 90 03 12	SITE
DRAWN DT	CHECKED	APPROVED	DWG 1418700C-2

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

CONT. 92-40

WP 141-87-00D DIST 6

HWY 407 STR SITE N/A

CPR Detour at
Hwy. 407/CPR Subway
(Between Islington Avenue and Humber River)

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GEOCRES 30M13-97 DATE MAR 19 1990

FOUNDATION INVESTIGATION REPORT
For
CPR Detour
at
Hwy. 407/CPR Subway
(Between Islington Avenue and Humber River)
W.P. 141-87-00D
District 6, Toronto

INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the aforementioned site. In order to maintain train traffic throughout the construction of the Hwy. 407/CPR Subway which is proposed along the existing alignment, it is proposed to construct a detour for the CPR at a location west of the existing track. The proposed alignment is illustrated on Drawing 1418700D-1 in the Appendix and reflects radii of curvature required by the CPR to facilitate train traffic at speeds requested by the railroad. The realignment will stretch from Sta. 9+621.5 to Sta. 10+127 through a range of surface topographies that will dictate excavation cuts in some locations and traversion of low lying areas (by earth fills or trestle structure) at other locations.

This report describes the subsurface conditions within the proposed realignment and provides discussion and recommendations pertaining to:

1) stability of any associated excavation cut or embankment fill placement
2) settlement considerations along the proposed alignment
3) foundations of a possible detour trestle
4) construction considerations associated with proposed adjacent structures (CPR Subway, Jersey Creek concrete culvert)

SITE DESCRIPTION AND GEOLOGY

The site is located immediately west of the existing CPR track approximately 0.3 km northwest of the CPR-Islington Avenue level crossing in the Town of Vaughan, Regional Municipality of York. The site is characterized by a pronounced valley that supports side slopes of approximately 2.5H:1V and trends in a general southwesterly direction. The crest of the valley is approximately 200 m in

200 m in width and the valley depth is approximately 20 m. The valley houses the Jersey Creek that runs its course at the valley floor and is approximately 2 m in width and normally flows at 1 m depths. The Jersey Creek flows into the Humber River located in a flood plain immediately west of the site.

The valley slopes are densely covered with trees, brush, tall grasses and shrubs. There is no evidence of slope creep or displacement indicating that the valley slopes appear to be stable at its present geometry.

The existing CPR track at the site is advanced partly in a excavation cut in the native subsoil and partly on embankment fills. An earth embankment spans the valley at the proposed Hwy. 407 right-of-way. This railroad embankment, supposedly constructed in the early 1900's supports side slopes approximately 1.5H:1V. Trees and low lying shrubs and grassland cover the existing constructed slopes. There appears to be no evidence of slope stability other than a localized area at the northeastern portion of the embankment. Rip-rap and armour stone was placed on the slope to retard surficial erosion at this location.

A concrete culvert is located at the base of the constructed embankment to facilitate the Jersey Creek outflow beneath the embankment. Again, no visible signs of distress in the culvert were apparent.

Land use surrounding the site consists of residential lots located east of the site, a hydro corridor consisting of transmission towers just north of the site and forestland elsewhere. A CPR two span structure is located approximately 0.3 km north of the site along the same track alignment. The structure spans the Humber River at this location. In addition, a CNR rigid frame overhead exists approximately 0.2 km south of the site to facilitate CN Rail traffic in a east-west direction over the CPR track.

Physiographically, the site lies within the region known as the South Slope (Chapman and Putnam, 1984). The South Slope Formation at the site consists of a ground moraine, scoured at intervals by valleys tributary to the Humber River systems. The valleys accentuate the hilly moraine topography. The glacial landforms and deposits were formed by the advance and retreat of the Wisconsinan ice sheet that covered the area during the Pleistocene epoch (over 5000 years ago).

The overburden is underlain by the grey shales of the Georgian Bay Formation of the Ordovician period.

FIELD INVESTIGATION

The fieldwork for the investigation was coordinated with the field investigation for the proposed CPR Subway and the proposed Jersey Creek concrete culvert. The fieldwork was conducted between 89 10 21 and 89 11 30 and consisted of a total of 14 sampled boreholes, including the 8 sampled boreholes advanced in conjunction with the CPR Subway and 3 sampled boreholes advanced in conjunction with the concrete culvert.

Five of the twelve boreholes were advanced through the overburden using hollow stem augering techniques to the depths of the lower sand to silty deposit (approximately 39 metres). Beyond that depth, the boreholes were advanced using conventional diamond drilling techniques (casing and washboring) to overcome torquing restriction imposed on the hollow stem augers. The NW casing used was advanced by both driven and rotary methods. The drilling equipment used was a track mounted CME 55.

In consideration of the importance of establishing the composition of the CPR embankment fill, a total of four boreholes were advanced in the existing embankment fill. Two of the boreholes were advanced from the crest of the embankment using the track-mounted CME. The other two boreholes were advanced at mid-slope, on the west side of the embankment (BH's 4A, 6A). These boreholes were advanced using conventional diamond drilling techniques via a tripod apparatus.

Three of the boreholes were advanced along the proposed realignment until a "competent" subsoil was encountered and ascertained where "competent subsoil" is defined as a compact to dense cohesionless deposit, a hard cohesive deposit or bedrock. Hollow stem auger equipment was used to advance the boreholes in the overburden. At the site, a very dense cohesionless deposit was encountered in the floodplain area of the proposed detour alignment and a hard cohesive deposit was located at the crest of the valley and hence the boreholes were terminated in these deposits.

Two additional boreholes were advanced in conjunction with the concrete culvert using hollow stem augering techniques.

In general, subsoil samples were retrieved at 1.5 m intervals for the upper 27-30 m and at 3.0 m intervals thereafter. Disturbed subsoil samples were retrieved by a split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586). Relatively undisturbed samples were also randomly retrieved in the surficial silty clay to clayey silt till deposit using a shelby tube samples in accordance with standard practice (ASTM D1587). In situ vane tests were also conducted in the cohesive silty clay to clayey silt till deposit, generally at 1.5 m intervals, to determine the undisturbed and remoulded undrained shear strengths of the soil. The test was conducted employing the Standard MTO 'N' values in accordance with ASTM 2573.

Bedrock was cored at five boreholes using conventional rock coring methods in NQ size.

All subsoil samples and rock core were identified in the field and returned to the laboratory for further examination and applicable testing.

Water levels were obtained in the open boreholes and also in a sealed piezometer installed at BH D-8. Groundwater levels were monitored throughout the duration of the investigation. All 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 identify the behaviour, gradation and pertinent properties and characteristics of the soil, various laboratory tests were performed. These tests included:

1) Atterberg Limits
2) Grain Size Distributions
3) Unit Weights
4) Natural Moisture Contents
5) Unconfined Compression Tests
6) Unconsolidated Undrained Tests
7) Multi-stage consolidated undrained tests with pore pressure measurements
8) Consolidation Test

In view of the general uniformity of soil types found in the general site area, all laboratory results for similar soil strata have been integrated from the different structures (CPR Subway structure, Jersey Creek concrete culvert). Laboratory test results have been summarized in the subsequent section of this report and are illustrated on corresponding figures and boreholes included in the attached Appendix.

SUBSURFACE CONDITIONS

The native subsoil of the original valley at the site consists of a surficial deposit composed of a clayey silt to silty clay with occasional sand seams and traces of gravel. The stratum is a till deposit of glacial origin and extends to a maximum thickness of 13.7 m at the crest of the valley. The thickness of this deposit decreases down the valley slope and does not exist at the valley floor and the floodplain located at the base of the existing CPR embankment (BH D-3) and at the culvert outlet (BH C-1). The consistency of this deposit ranges from firm to hard.

Underlying the upper till deposit and located as a surficial stratum in the floodplain exists a deposit of clayey silt that extends for a considerable thickness ranging from 18.3 m to 28 m below the upper till and from 10.7 m to 11.1 m in the floodplain area. This stratum also contains random interbeds of sand and gravel.

The clayey silt deposit is in turn underlain by a cohesionless deposit of sands and silts. The deposit varies randomly in silt and sand percentages, varying from sand with some silt to sandy silt. Random zones of silt also exist in the soil matrix. Gravel, boulders and cobbles are also components of the lower depths of the deposit. The thickness of this deposit ranges from 4.2 m to 15.8 m with an average thickness of approximately 10 m. The denseness of this deposit varies from compact to very dense. This cohesionless deposit overlies shale bedrock of the Georgian Bay shale formation.

Two types of fill material was used to construct the CPR embankment. Surficially and within a zone above and immediately adjacent to the existing concrete culvert, a cohesionless backfill material consisting of a sand with some silt to sandy silt was used. Beneath the surficial sand material and beyond the culvert backfill wedge zone, the embankment fill material consists of a clayey silt with interbedded layers of sand. The thickness of the surficial cohesionless fill material which also exists on the embankment slopes, ranges from 2.0 to 4.6 m. The maximum depth of the embankment fill explored was 13.9 m at BH D-6, located at the proposed CPR Subway pier location. At BH's D-1, D-2, located in the area of the south abutment, only 1.5 to 2.4 m of granular fill was encountered, confirming the valley crest location. At BH D-4, the location of the proposed north abutment, 12.2 m of clayey silt fill material with interbedded layers of sand exists.

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 attached Record of Borehole sheets in the Appendix. A plan of the site illustrating the locations and elevations of the boreholes and subsoil stratigraphical sections are provided on Dwgs. 1418700D-1 & 1418700D-2.

A detailed description of the subsurface conditions encountered is given below.

<u>Sand, some Silt (Fill)</u>

As previously mentioned, the surficial embankment fill material and backfill material to the existing concrete culvert consists generally of a brown sand with some silt. Occasional layers of sandy silt and clayey silt are also present in the fill material and traces of fine gravel are also randomly intermixed. A grain size distribution envelope illustrating the gradation of the fill is provided in Figure 1 in the Appendix. The surficial thickness of the fill material varies from 1.5 to 2.4 metres and the maximum thickness explored was 16.2 metres at which depth the existing concrete culvert roof was encountered (see BH D-7).

Standard Penetration tests carried out in the cohesionless fill material revealed 'N' values ranging from 2 blows/0.3 m to 21 blows/0.3 m indicating a very loose to compact state of condition.

Clayey Silt (Fill)

Beneath the surficial cohesionless fill and beyond the culvert cohesionless backfill material, the CPR embankment fill consists of a brown, cohesive clayey silt. The maximum thickness of the clayey silt fill encountered was 12.2 metres at the proposed north abutment location. Interbedded layers of fine sand ranging in thickness from 50 mm to 150 mm are also present randomly in the cohesive matrix. A grain size distribution envelope for this material as determined by mechanical sieve and hydrometer analysis is given in Figure 2.

Atterberg Limits were obtained to evaluate the behaviour and plasticity of the soil and the results are plotted in Figure 3. A summary of the indices is provided in Table 1 below. Unit weights are also included.

Table 1 - Clayey Silt (Fill)

	Range	# of Tests
Natural Moisture Content (w%)	15-24	8
Liquid Limit (w_L%)	21-32	8
Plastic Limit (wp%)	13-19	8
Unit Weight (kN/m^3)	19.2-20.2	4
Undrained Shear Strength (cu) (kPa)	80->120	5

The test results reveal that the cohesive fill material is of low plasticity and hence can be categorized as clayey silt.

Undrained shear strength measurements (cu) were obtained in situ by conducting field vane tests. Results are plotted on the Record of Borehole sheets in the Appendix and summarized in Table 1 above. However, in consideration of the interbedded layers of sand, consistencies ranging from stiff to very stiff which is representive of the determined shear strength values cannot be implicity assumed.

Silty Clay to Clayey Silt (Glacial Till)

The native surficial deposit present at the site consists of a cohesive silty clay to clayey silt with traces of sand and gravel and occasional random interbedded sand seams. The thickness of the deposit explored in the investigation ranges from 11.3 to 13.7 and the interbedded sand seams are generally 50 to 100 mm in thickness. At BH D-4, the approximate location of the proposed north abutment, this deposit does not exist indicating that the deposit decreases in thickness from the crest of the valley to the valley floor. The deposit is generally oxidized (brown) for the upper 1.5 to 3.5 metres and unoxidized (grey) for its lower thickness. The deposit is a till of glacial origin.

A grain size distribution envelope for this deposit as determined by mechanical sieve and hydrometer analysis is given in Figure 4. The envelope illustrates that clay and silt percentages in the deposit range from 25-61% and 35-61% respectively, confirming the range in behaviour of the fine grained portion of the deposit.

Although not encountered during this investigation, boulders and cobbles are characteristic components of till deposits and consequently may be encountered in this deposit.

Atterberg Limit tests were carried out to define the behaviour and plasticity of the soil and the results are plotted in Figure 5. A summary of the indices is provided in Table 2. Unit weights are also included.

Table 2 - Silty Clay to Clayey Silt

	Range	# of Tests
Natural Moisture Content (w%)	15-29	14
Liquid Limit (w_L%)	22-47	14
Plastic Limit (wp%)	12-20	14
Unit Weight (kN/m^3)	18.8-20.3	9
Undrained Shear Strength (cu) (kPa)		
- Field Vane	35->120	28
- Laboratory*	41-82	4
Sensitivity	2-3	28

*Unconfined Compression Tests
*Unconsolidated Undrained Tests

The test results reveal that the deposit varies randomly in plasticity ranging from low (clayey silt) to intermediate (silty clay).

Undrained shear strength measurements (cu) of the soil were obtained both by in situ vane tests and by laboratory tests, namely unconfined compression tests and unconsolidated undrained tests (quick triaxial). Results are plotted on the Record of Borehole sheets in the Appendix and summarized in Table 2 above. A Shear Strength vs Elevation profile is also provided in Figure 6. Based on shear strength values ranging from 35->120 kPa, it is considered that the soil has a firm to very stiff consistency.

The sensitivity of the soil as defined by the ratio of the undrained strength in the undisturbed state to the undrained strength, at the same water content, in the remoulded state was also determined by the field vane test and the results are tabulated in Table 2 and identified on the Record of Borehole sheets. Sensitivity values range from 2 to 3 indicating that the soil has a low sensitivity.

Consolidated undrained multi-stage triaxial tests with pore pressure measurements were conducted in the laboratory to determine the effective strength parameters of the material. The effective shear strength parameters determined from the test are summarized in Table 3.

Table 3 - Effective Shear Strength Parameters

Sample	BH D-1, TW5
Elevation (m)	147.0
Liquid Limit	47
Plastic Limit	20
Natural Moisture Content (w%)	26
Effective Angle of Internal Friction (Ø°)	29.5
Effective Shear Strength Intercept (C')	10

For design purposes, a reduced angle of internal friction (Ø°) of 26° and a shear strenth intercept of 5 kPa was selected to account for the fact that the sample test was not saturated.

A consolidation test was conducted to evaluate the compressibility characteristics of this deposit. The results (e-log p curve) of the test are illustrated in Figure 7 in the Appendix. The results reveal that this cohesive stratum has been preconsolidated in the past to an effective pressure 200 kPa in excess of the existing overburden pressure.

The coefficient of consolidation (cv) used to determine the time rate of consolidation settlement was computed using Taylor's Method (1948). The results reveal values ranging from 0.004 m^2/day to 0.005 m^2/day for loadings ranging from 100 to 200 kPa.

Standard Penetration tests carried out in this deposit revealed 'N' values ranging from 2 blows/0.3 m to 15 blows/0.3 m.

Clayey Silt

Underlying the surficial clayey silt to silty clay deposit at a depth ranging from 10.7 m to 13.7 m below the ground surface (Elevation 140.0 to 135.2) and extending for a maximum thickness of 18.3 m, and present at the the surface of the floodplain and extending to a maximum thickness of 11.1 m, a cohesive deposit of clayey silt exists. This stratum also contains traces of sand and random zones of silt. In the floodplain, a trace of organics was encountered in the surficial 2 m of the deposit and interbedded layers of sand and gravel approximately 100 mm in thickness also exist.

A grain size distribution envelope for this deposit as determined by mechanical sieve and hydrometer analysis is given in Figure 8. The envelope illustrates that clay and silt percentages in the deposit range from 12-34% and 60-88% respectively.

Atterberg Limit tests were carried out to define the behaviour and plasticity of the soil and the results are plotted in Figure 9. A summary of the indices is provided in Table 4. Unit weights are also included.

Table 4 - Clayey Silt

	Range	# of Tests
Natural Moisture Content (w%)	14-35	17
Liquid Limit (w_L%)	26-29	17
Plastic Limit (w_P%)	14-18	17
Unit Weight (kN/m^3)	20-22	8

The test results reveal that the fine grained portion of the deposit is of low plasticity and hence can be categorized as a clayey silt.

Standard Penetration tests carried out in this stratum revealed 'N' values ranging from 5 blows/0.3 m to 76 blows/0.3 m indicating that the deposit ranges in consistency from firm to hard. In general, in the upper 10 m or so, 'N' values ranged from 20 blows/0.3 m to 30 blows/0.3 m (although lower 'N' values were obtained in the floodplain area because of the presence of organics), indicating a very stiff consistency. In the lower depths of the deposit, 'N' values ranged from 10 blows/0.3 m to 20 blows/0.3 m and the soil can be categorized as having a stiff consistency.

Sand and Silt

Underlying the clayey silt deposit and extending to bedrock a cohesionless deposit of sand and silt exists. The deposit is predominantly composed of sand with some silt but random zones of sandy silt to silt are also present within this deposit. In addition, gravel, boulders and cobbles exist as a heterogeneous mixture in the deposit at the lower depths immediately above the bedrock. At BH's D-4 and C-3, approximately 2.5 m of the coarser grained gravel, boulders and cobbles was encountered. The thickness of the entire deposit ranges from 4.2 m to 15.8 m, but is generally in the order of 10 metres. A grain size distribution envelope for this deposit is provided in Figure 10 in the Appendix.

This cohesionless deposit is water bearing and consequently, when the deposit was penetrated in the open borehole, soil cave-in resulted due to unbalanced hydrostatic head.

Standard Penetration tests carried out in this deposit revealed 'N' values ranging from 10 blows/0.3 m to 120 blows/0.8 m indicating that the deposit ranges in denseness from compact to very dense. In the floodplain area of the site, 'N' values obtained below the upper 3 m thickness were generally representative of very dense material. The deposit is predominantly compact to dense at the other locations of the site.

Bedrock

The cohesionless sand with some silt deposit is directly underlain by shale bedrock of the Georgian Bay shale formation. The bedrock surface is generally flat with surface elevations ranging from 105.9 m to 107.7 m. The bedrock was cored by NQ size up to 2.8 metres in thickness.

The shale bedrock is grey in colour and is very fine grained and thinly laminated. The rock is generally slightly to moderately weathered and contains occasional clay seams, approximately 50 to 100 mm in thickness. Minor beds of argillaceous limestone are also present in the rock formation. Detailed descriptions of the bedrock are attached in the Appendix entitled "Description of Rock Core".

Core recoveries and Rock Quality Designations (RQD) were determined in situ and also in the laboratory to evaluate the competence and integrity of the rock. Rock recoveries varied between 60 and 100% while RQD's varied between 0 and 15%. The shale bedrock is weak to very weak rock.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in the open boreholes and monitoring the level in a piezometer installed at BH D-8 (CPR Subway pier location). The piezometer was installed in the clayey silt deposit with bentonite seals above and below the piezometer tip. Measurements obtained at the time of the investigation revealed levels as tabulated in Table 5 below.

Table 5 - Groundwater Levels

Depths (m)	Elevations (m)	BH's	Location
2.0-4.0 m	148.2-150.6	D-1 D-2 D-3	Valley Crest (South Abutment, 40 m north of North Abutment)
6.2	144.4	D-8	Pier
16	136.1	D-4	North Abutment (12.2 m of embankment fill).

Artesian conditions were encountered in the gravels, boulders and cobbles present at the lower depths of the sands and silts deposit within the floodplain. Up to 3 m of piezometric head above the natural ground surface was observed.

In all cases, the groundwater level was not found in the boreholes advanced in the embankment fill.

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 Hwy. 407/CPR Subway along the existing CPR alignment. To render continuity in the raidroad service during the construction of this structure, it is proposed to construct a detour along an alignment immediately west of the existing track. The alignment, illustrated on Dwg. 1418700D-1 in the Appendix was determined incorporating various design parameters, including radii of curvature to satisfy train velocity requirements and boundary conditions induced by the Humber River bridge to the north and the CNR overhead to the south.

Although a profile of the proposed detour was unavailable at the time of the foundation request, it is hereby assumed that the elevation grade is similar to the existing CPR track grade equivalent to approximately 152.0 to 152.5 m. The integration of this grade with the existing topography along the proposed detour alignment reveals that both fill and cut sections will be required. The sections have been identified in Table 7 below.

Table 7 - Cut and Fill Sections

Location	Section	Cut (m)	Fill (m)*	Comments
Sta. 9+682 to Sta. 9+813	1	-	0 to 11	
Sta. 9+813 to Sta. 9+892	2	3	-	
Sta. 9+892 to Sta. 9+955	3	-	0 to 17	
Sta. 9+955 to Sta. 10+127	4	-	0 to 17	Possible Trestle

*effective fill heights measured from embankment fill crest to toe of embankment fill.

Geotechnical recommendations for the design and construction of the detour at the aforementioned subsection segments is discussed below.

Stability

General

Stability computations were carried out to determine both the overall (global) stability and internal stability of the fills and cuts associated with each section of the proposed detour. The analyses was carried in terms of total stress for the fill application and effective stress for the proposed excavation cut. The Bishop's Modified Method was applied using an inhouse mainframe program incorporating a factor of safety of 1.3. Static loading conditions and an external live load of 120 kN/m representing the train traffic (American Railroad Engineering Association) was incorporated in the design. Circular slip surfaces were analyzed for both the total and effective stress analysis. The soil parameters, geometries and the results of the analyses for each designated section along the proposed alignment are described in this report.

In the total stress analysis, total stress parameters were applied for the native soil and a granular fill material having an angle of internal friction ($\emptyset$) equivalent to 30° and unit weight (γ) of 20 kN/m^3 was used. In all cases, new fills shall be benched to existing fills in accordance with OPSD 208.01. Any loose and/or organic material in the native subsoil shall be removed prior to the placement of fill.

The critial condition examined in the evaluation of the excavation cuts proposed is the drained condition and consequently an effective stress analysis was conducted. Effective stress parameters of the soil were employed in the calculations. Drained stability analyses of the slopes are very sensitive to groundwater levels and pore pressures that can develop in the slope. Therefore slope protection and drainage measures will be required to preserve surficial stability. By employing a 1.2 m thick granular blanket consisting of free draining materials such as Granular 'A' material, softening of material due to freeze-thaw cycles and development of excess pore water pressures can be prevented. The granular blankets should be designed in conjunction with a temporary drainage system that will discharge drained water from the slope.

SETTLEMENT

General

The surficial silty clay to clayey silt till deposit is the only compressible susceptible material at the site. This deposit, however, is an overconsolidated material and based on a consolidation test conducted on a representative sample of the deposit, the soil has been preconsolidated to an effective pressure approximately 200 kPa in excess of the existing overburden pressure. In the sections of the alignment where this deposit exists, additional loadings induced as a result of fill placement will generally not exceed the preconsolidation pressure and consequently most of the settlement will be attributable to the recompression of the soil and hence will develop almost immediately subsequent to load application (during or shortly after construction).

Primary consolidation of the silty clay to clayey silt deposit will occur if the the preconsolidation pressure is exceeded. For instance in section 3, 11 m of embankment fill material required will produce an additional vertical stress of approximately 220 kPa (assuming a unit weight of fill of 20 kN/m^3). Primary consolidation settlement in this case was predicted using one- dimensional consolidation theory and the load-deformation curve obtained by laboratory testing. The increase in vertical stress and corresponding stress distribution induced by the placement of the fill was determined using Osterberg (1957) solution.

The time rate of primary consolidation of the silty clay to clayey silt was predicted using the Terzaghi consolidation theory. Single drainage to the embankment fill at the surface was assumed in the deposit. The coefficient of consolidation (Cv) derived from Taylor's Method and equal to 0.0045 m^2/day was used in the computation.

Elastic settlements induced within the fill itself as a result of self loading can also be expected. As in the case of elastic settlements induced as a result of the recompression of the native soil, settlements within the fill that is properly compacted and placed will also be immediate (elastic) in nature. The magnitude of the settlements induced within the fill is predicted as a percentage of the fill height and is summarized in Table 8 below.

Table 8 - Settlements within Fill

Height of Fill (H_{FILL}) (m)	Total Settlement (S_T)
0-7	0.5% x H_{FILL}
7-10	0.75% x H_{FILL}
10-15	1% x H_{FILL}

CPR DETOUR SUBSECTIONS

1. Sta. 9+682 to Sta. 9+813 - Section 1

In this detour subsection, "effective" fill heights up to 11 m in depth will be required to be placed. Effective fill height refers to the total distance between the crest and toe of the slope. In view of the natural sloping surfaces within this subsection, the height of fill that is required varies from the centreline to the edge of the additional fill. Figure 11 illustrates a typical section of the superimposition of the new embankment.

The results of a total stress stability analyses applying the soil parameters identified in Figure 12 and varying geometries that integrate the existing slopes in this section, indicate fills up to an effective height of 8 m will remain stable constructed at 2H:1V. For effective fill heights exceeding 8 m, it is recommended that the slopes be constructed with a midheight stabilizing berm of 3 m width and 2H:1V slopes. The nominal berm is required to prevent shallow surficial slope failures. No deep-seated failures are anticipated.

Predicted settlements induced as a result of the additional embankment loading are in the order of magnitude of 25-50 mm. These settlements are attributable to the recompression of the native subsoil and settlements of the fill under self load. Consequently, the settlements should be realized during or shortly after construction. It is therefore recommended that the embankment be constructed as far in advance of the track placement as scheduling permits. It is expected that any settlement that does occur subsequent to the track installation can be easily corrected by routine maintenance methods.

2. Sta. 9+813 to 9+892 - Section 2

In section 2, excavation cuts up to 3 m in depth will be required. Based on stability analyses using the effective stress parameters tabulated in Table 9 for the various soils, it can be concluded that excavated slopes will remain stable at 2H:1V. Slope protection and drainage should be included in the design as previously discussed.

Table 9 - Effective Stress Parameters

Material	Effective Strength c' (kPa)	Parameters Ø' (°)	Unit Weight kN/m^3
Fill (Granular)	0	30	20
Silty Clay to Clayey Silt	0	26	19.6
Clayey Silt	5	26	21

It is expected that settlements induced as a result of the dynamic train loading will be negligible and less than 25 mm.

3. Sta. 9+892 to Sta. 9+955 - Section 3

Effective embankment fills ranging from 0 to 17 m will be required in Section 3 of the proposed detour. Stability analyses based on total stress parameters and integration of embankment fills on the prevalent topography at site as illustrated on Figure 12 indicate that internal (surficial) stability of the fills dictate the design. Consequently to ascertain internal stability of the fills, it is recommended that the embankment fills be constructed with stabilizing berms and 2H:1V slopes. Any slope exceeding 8 m in height should be constructed with a midheight stabilizing berm of 3 m width. Accordingly, for fill heights exceeding 16 m, a double berm at one-third fill heights should be constructed.

The total predicted settlement induced as the result of the additional embankment loading is the summation of the elastic recompression of the native subsoil, elastic settlements within the fill and primary consolidation of the silty clay to clayey silt deposit. Assuming that the train traffic will be detoured for a period of two years, approximately 100 mm of settlement can be anticipated. Approximately 50 to 75 mm of this settlement will be of the immediate nature and should be realized during or immediately following construction. It is therefore recommended that the embankment fills be constructed as far in advance of final track installation as scheduling permits. It is expected that any subsequent settlement that does occur can be readily corrected using routine maintenance methods.

4. Sta. 9+955 to 10 +127 - Section 4

This section of the proposed detour is located immediately west of the proposed CPR Subway structure. A distance of approximately 12 m (centre to centre) separates the proposed structures at the CPR Subway south abutment and 25 m at the north abutment. The proposed alignment intersects the toe of the existing railroad embankment fill north of the existing culvert and hence an elevation difference of approximately 15 m exists from the existing slope to the proposed top of railroad embankment. South of the existing culvert, the alignment intersects the existing surface topography at increasingly higher elevations in a southward direction. Hence, the elevation difference varies from approximately 13 m to 0 m at the crest of the original valley.

The design of this section of the detour shall be sensitive to be construction of the proposed CPR structure because of their close proximity. The following types of structure are therefore recommended to facilitate construction of the CPR Subway structure.

1) earth embankment structure with shoring scheme
2) trestle structure

The method or combination thereof that proves to be most economical and technically feasible should be selected for design.

Earth Embankment Structure

In view of the competent nature of the clayey silt beneath the existing embankment fill and the competent clayey silt deposit in the floodplain adjacent to the proposed detour, no deep seated slope failures are anticipated for the embankment fills constructed at 2H:1V, north of the existing culvert. Stability analyses was also implemented applying the total stress parameters and subsoils identified in Figure 12 with the varying existing topographical geometries located south of the existing culvert to the valley crest. The results also reveal that no deep seated external slope instabilities are anticipated. However, as previously discussed to avoid internal instabilities within the fill it is recommended that fills exceeding 8 m in effective height be constructed with a midheight stabilizing berm of 3 m width and 2H:1V slopes. For fills up to 8 m internal stability can be satisfied with 2H:1V slopes.

In consideration of the close proximity of the proposed detour and CPR Subway, it appears that the east slopes of the detour embankment will be required to be supported by a temporary shoring wall to facilitate construction of the CPR Subway. The height of the wall will vary as a function of the distance between the two structures. Temporary excavation cuts to construct the proposed CPR Subway structure shall be no steeper than 1.5H:1V and this restriction should be integrated in the selection of the location of the temporary shoring.

Shoring systems that can be considered include an anchored steel sheet pile wall or an anchored timber lagging-soldier pile wall. Cantilevered walls can be considered provided that earth pressure equilibrium is ascertained and the design is practically feasible. Liaison with this office should be coordinated in the temporary shoring selection process.

The design of the shoring system shall include the appropriate earth pressures computed in accordance with Section 6.6.1.2 of the O.H.B.D.C. The loadings induced by the surcharge train traffic and consideration of sloping surfaces shall be incorporated in the design. Preliminary design parameters of the soil to be supported are summarized in Table 10 below.

Table 10 - Shoring Design Soil Parameters

Soil Type	Saturated Unit Weight (kN/m^3)	Effective Shear Strength Parameter (Ø°)
Fill Material	20	30
Silty Clay to Clayey Silt (Till)	19.6	26
Clayey Silt	20	30

Buoyant unit weights of soil shall be applied below the prevailing groundwater table.

Soil anchors can be installed in the native clayey silt material present at the site to resist the induced loadings. A bond stress of 500 kPa can be used for design. Further information pertaining to the design of soil anchors and test loadings procedures can be obtained from this office.

Alternatively the shoring wall can be supported with rakers installed in front of the wall. Rakers must be installed while an earth berm remains in front of the pile. Slots should be cut into this berm to install rakers before the supporting berm is removed. Raker footings can be founded in the clayey silt deposit underlying the surficial silty clay to clayey silt deposit at an elevation ranging approximately from 137 to 140 m. An allowable bearing value of 200 kPa at S.L.S. Type II and 300 kPa at U.L.S. can be used for the raker footing design.

The depth of penetration of the shoring system must also be designed to ascertain overall stability of the temporary detour embankment. Generally, an anchored system will not produce instabilities provided that the anchors are installed below the level of base of the shoring wall and sufficient earth cover exists in front of the shoring wall. In view of the relative competent strength of the clayey silt deposit, it is recommended that the shoring wall be installed within this deposit.

To reduce the extent of shoring, consideration may be given to constructing the temporary detour embankment using polymer geogrid reinforcement. The advantage of the application of geogrid reinforced embankments is that it enables steeper embankment slopes. Additional information can be obtained from this office should this alternative be considered.

Trestle Structure

Alternatively, constructing this section of the detour on a temporary trestle structure would eliminate the requirement of a shoring system. Similar to the recommendations for the foundations of the CPR Subway, it is recommended that the temporary trestle structure also be supported on deep foundation units.
The deep foundations can be supported using either end bearing or friction piles as discussed below. The design that proves to be most enconomical and feasible shall be selected.

End-Bearing Piles

All foundations can be founded on end-bearing steel H-piles driven to the bedrock surface. To facilitate pile penetration, particularly through the gravel, boulders and cobbles, it is recommended that the steel H-piles be equipped with reinforced tips. In view of the considerable pile length, splicing of the piles shall conform to pertinent MTO Standards (OPSS 903.07.01.03). The piles, nontheless, can be designed as fully laterally supported columns. The design parameters recommended for vertical steel H-piles, are summarized in Table 11 below.

Table 11 - Trestle Foundation Design

Structure*	Pile Type	Factored Capacity at U.L.S. (kN)	Bearing Capacity at S.L.S. Type II (kN)	Bedrock Surface El. (m)
North Abutment	HP310x79	890	1150	107.1±
	HP310x110	1150	1600	
Pier	HP310x79	890	1150	107.7±
	HP310x110	1150	1600	
South Abutment	HP310x79	890	1150	105.5±
	HP310x110	1150	1600	

*Locations referred to proposed CPR/Hwy. 407 Subway strucuture.

Driving of piles shall be carefully monitored and controlled employing the Hiley Dynamic Pile Driving Formula driven in accordance with MTO Standards SS103-10 or SS103-11 assuming an ultimate capacity as follows:

Table 12 - Dynamic Formula Capacities

Pile Type	Ultimate Capacity (kN)
HP310x79	2670
HP310x110	3450

Friction Piles

Alternatively, all structure foundations can be founded on friction piles that derive their supporting strength primarily as a result of shaft resistance produced in the surficial till deposit and underlying clayey silt stratum.
For purposes of the O.H.B.D.C., the design capacities for various pile types are summarized below.

Table 13 - Friction Pile Capacities

Pile Type	Factored Capacity at U.L.S. (kN)	Bearing Capacity at S.L.S. Type II (kN)	Founding Elevation (m)
HP310x110	990	660	115
324 mm 0 x 9.5 thick steel tube pile A252 (concrete filled)	990	660	115
Reinforced Precast Concrete (305 mm x 305 mm)	1600	1150	115
Treated Timber (size 36)	450	300	15 m embedment length

It is recommended that the design capacities and load/deformation behaviour of the piles be verified by a full scale load test conducted at the site.

Driving of piles shall be carefully monitored and controlled employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standards SS103-10 or SS103-11 assuming an ultimate capacity as follows:

Pile Type	Ultimate Capacity (kN)
HP310x110	1980
steel pipe	1980
reinforced concrete	3450

Design/ Construction Criteria for Trestle Structure

Regardless of the type of deep foundation unit selected, the following criteria and comments are applicable.

1) Reduction of axial capacities for inclined loadings shall conform to factors provided in Section 6.8.3.4.3 of the O.H.B.D.C.

2) The lateral resistance for both vertical and battered piles shall be computed in accordance with Section 6.8.3.8 of the O.H.B.D.C.

3) Pile spacing shall conform to Section 6.8.3.10 of the O.H.B.D.C. Adjacent pile should be checked for heaving during pile installation.

4) All pile caps shall be protected against frost protection by providing a minimum 1.2 m of earth over. No dewatering problems are anticipated for the construction of pile caps within the surficial native silty clay to clayey silt or underlaying clayey silt deposits in view of the impervious nature of the material. Pile cap construction within the embankment fill can also be conducted without major dewatering difficulty. Any localized seepage can be readily discharged using conventional sump pumping techniques.

5) At the north approach of the proposed trestle structure up to 15 m of fill material will be required. Longitudinal slopes were analyzed using the total stress parameters illustrated in Figure 12 and integrating the fills with the existing surface topography. Based on the analyses, it can be concluded that no deep seated failures are anticipated for slopes constructed at 2H:1V. However, to maintain the internal stability of the fills, it is recommended that fills beyond 8 m in height be constructed with a nominal 3 m width midheight berm and 2H:1V slopes.

6) Free draining material such as Granular 'A' or Granular 'B' is recommended as appropriate backfill to the structure to prevent hydrostatic pressure build-up. Design parameters of the soil are given in Table 13 below.

Table 13 - Approach Fill Soil Parameters

	Granular 'A'	Granular 'B'
Angle of Internal Friction (ϕ)	35°	30°
Unit Weight (kN/m^3)	22.8	21.2
Coefficient of Active Earth Pressure (K_a)	0.27	0.33
Coefficient of Earth Pressure at Rest (K_o)	0.43	0.5

Lateral earth pressures should be computed in accordance with Section 6.6.1.2.1 of the O.H.B.D.C. The active condition (K_a) will govern earth pressure design if the structure is yielding while the at-rest condition (K_o) will govern for an unyielding structure. The tabulated earth pressure coefficient apply to horizontal surfaces only. Adjustment for any sloped surfaces must be implemented. Weep holes should be designed to drain accumulation of water in the backfill.

7) At the south abutment, excavation cuts up to approximately 8-9 m in depth may be required for the foundation construction of the proposed CPR south abutment. Based on the analyses conducted for the proposed structure in terms of effective stress (see WP 88-78-16), slopes excavated 8 m in the surficial silty clay to clayey silt shall be constructed with a mid-depth stabilizing bench equivalent to 2 m in width. However, for practical construction considerations a 3 m bench may be desirable. Table 12 below summarizes the required slope geometry for various depths of cut in the surficial silty clay to clayey silt deposit.

Table 12

Depth of Cut (m)	Slope Geometry
0-5 incl	2H:1V slopes
>5-8 incl	2 m bench, 2H:1V slopes
>8-9 incl	4 m bench, 2H:1V slopes
>9-10 incl	5 m bench, 2H:1V slopes

Existing slopes west of the proposed trestle detour structure are at approximately 2.5H:1V slopes or flatter and do not require modificiation.

8) Settlement at the north abutment of the trestle as discussed in conjunction with section 3 of the detour is anticipated to be approximately 100 mm to be realized over a period of two years. As previously discussed approximately 50 to 75 mm of this settlement is expected to be immediate in nature and should be realized during or immediately following construction. It is therefore reiterated that the embankment fills be constructed as far in advance of the structure as scheduling permits. It is expected that any subsequent settlement that does occur can be readily corrected using routine maintenance methods.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer, utilizing equipment owned and operated by Marathon Drilling. The description of bedrock core samples was carried out by S. Senior, Geological Engineer.

The project was carried out by T. Sangiuliano under the general supervision of Dr. B. Iyer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed by Dr. B. Iyer and approved by Mr. M. Devata, Chief Foundation Engineer.

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A. J. Sangiuliano
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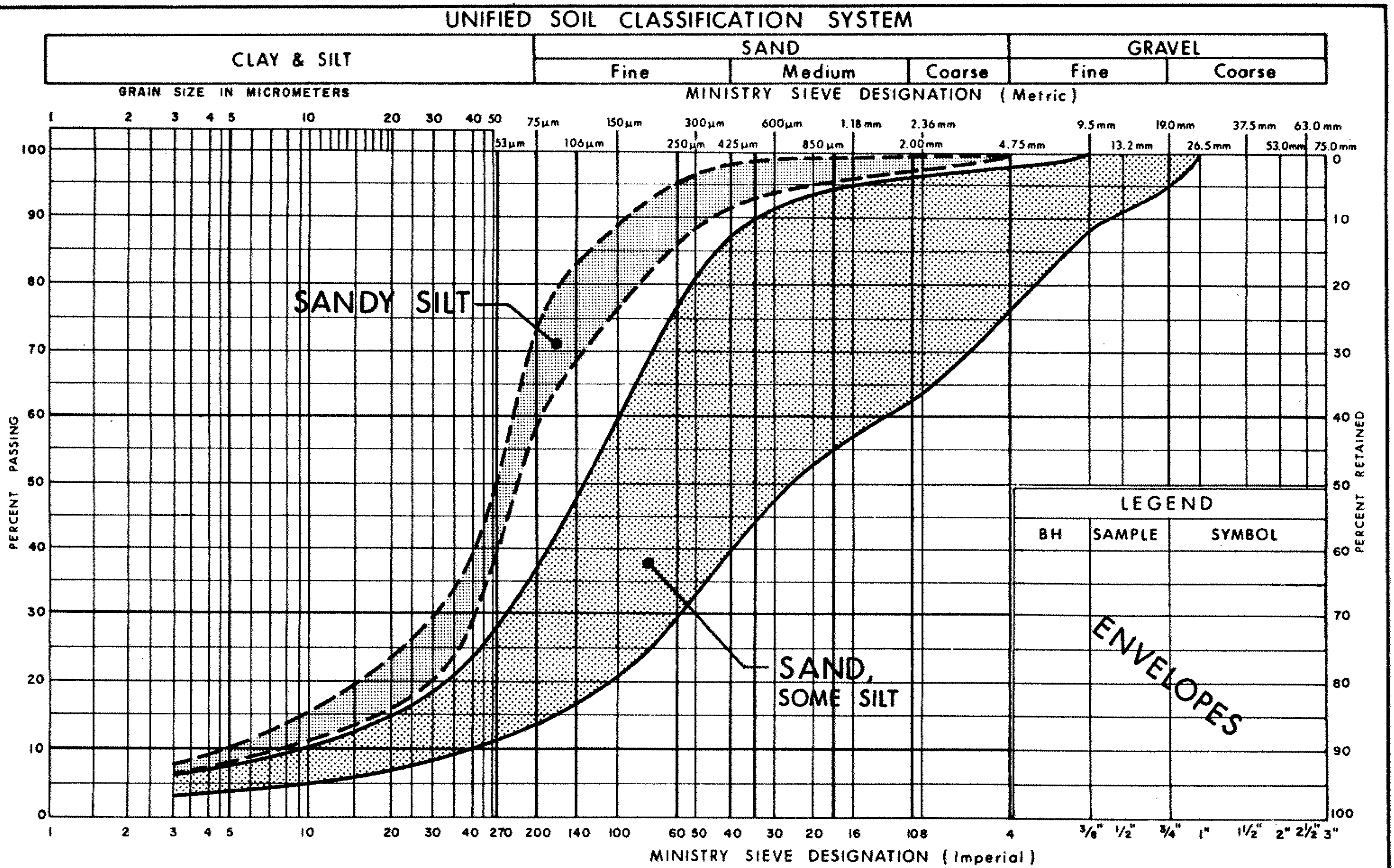
APPENDIX

78 12 M



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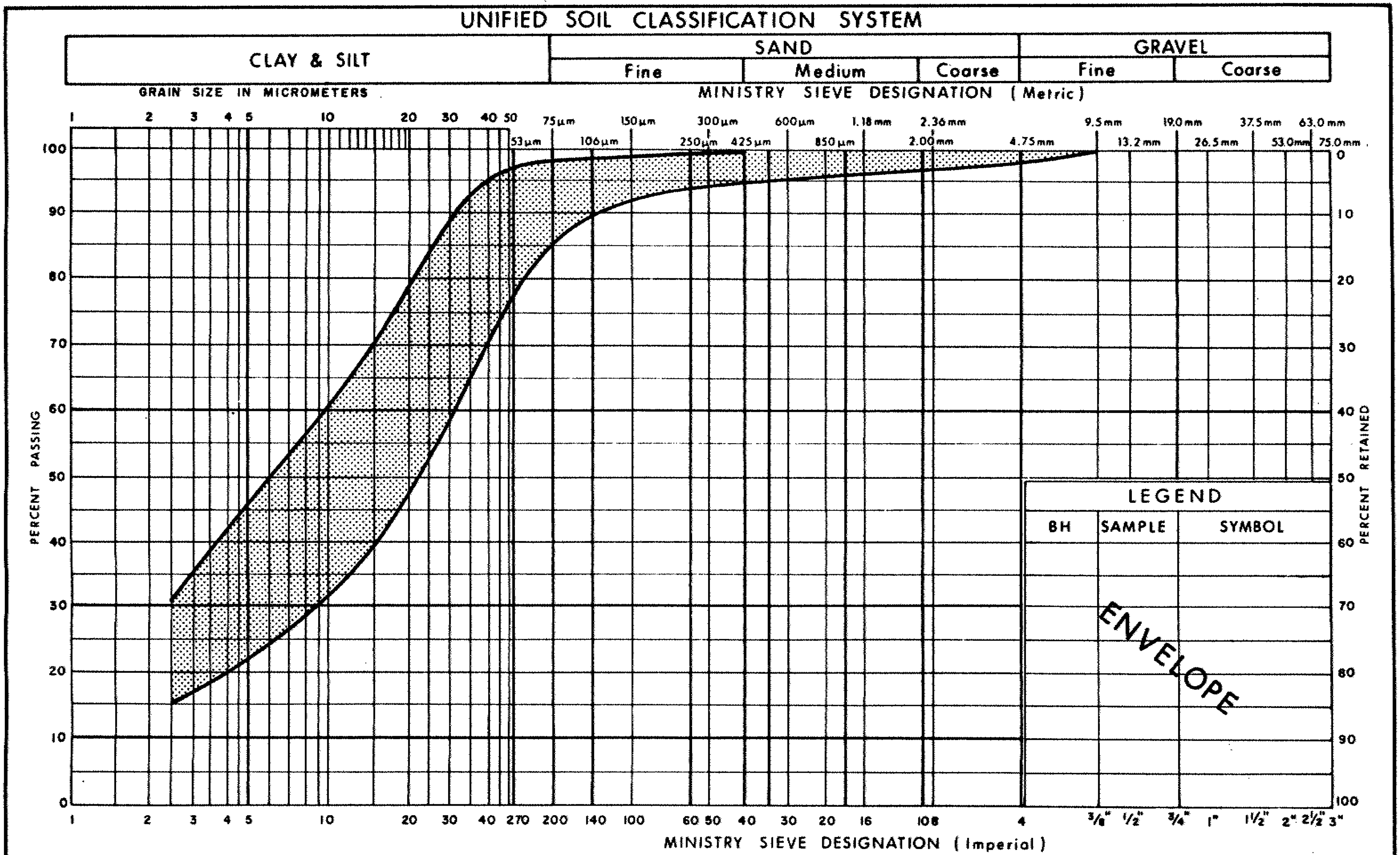
GRAIN SIZE DISTRIBUTION
(FILL MATERIAL)

FIG No 1

W P 141-87-00 D

78 12 M



Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION
CLAYEY SILT (FILL)

FIG No 2

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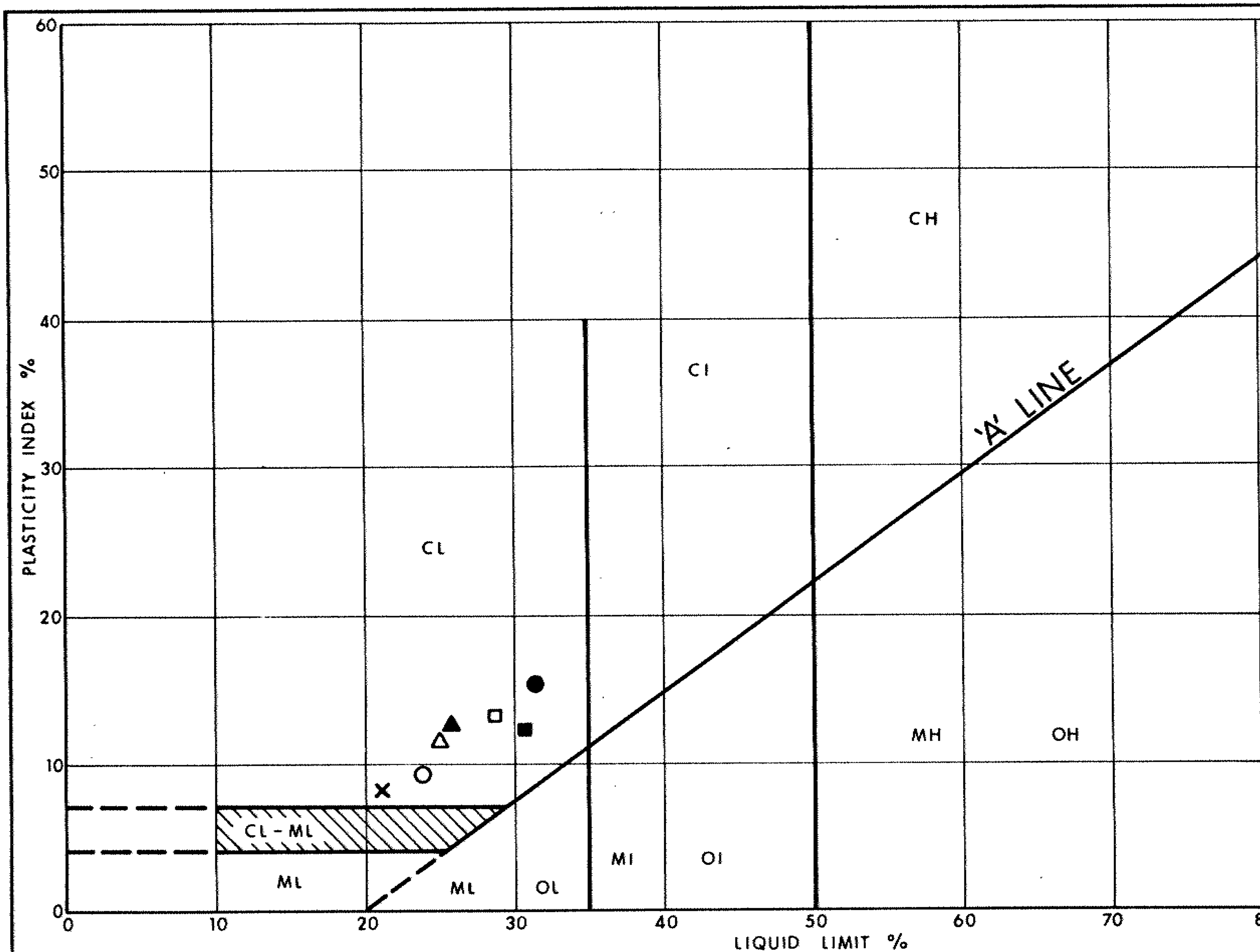

LEGEND

BH	SAMPLE	SYMBOL
D-4	5	○
D-4	7	●
D-6	3	□
D-6	6	■
D-6A	7	△
D-7	3	▲
D-8	3	×

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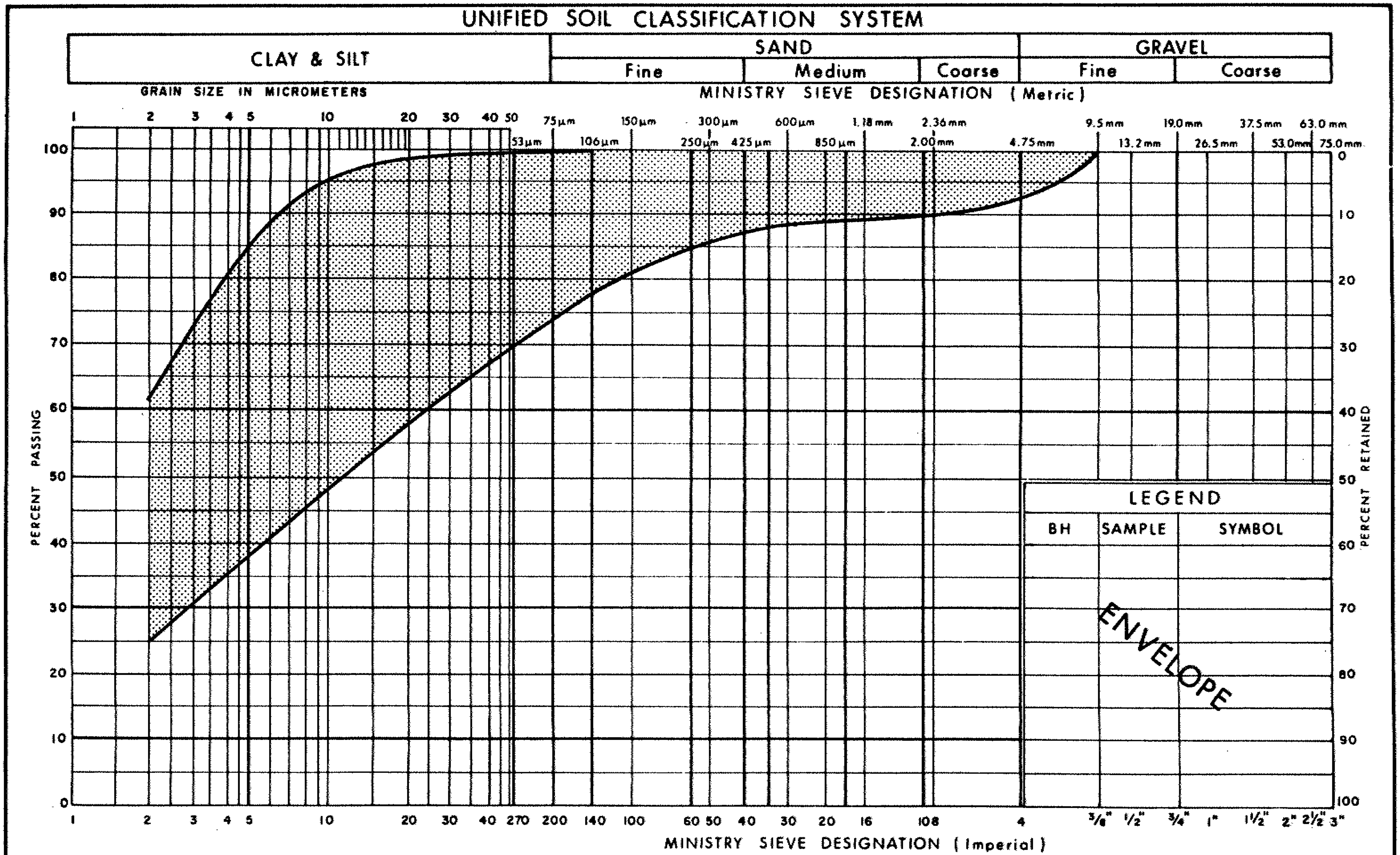
PLASTICITY CHART
CLAYEY SILT (FILL)

FIG No 3
W P 141-87-00D

78 12 M



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GRAIN SIZE DISTRIBUTION

SILTY CLAY TO CLAYEY SILT
(Glacial Till)

FIG No 4

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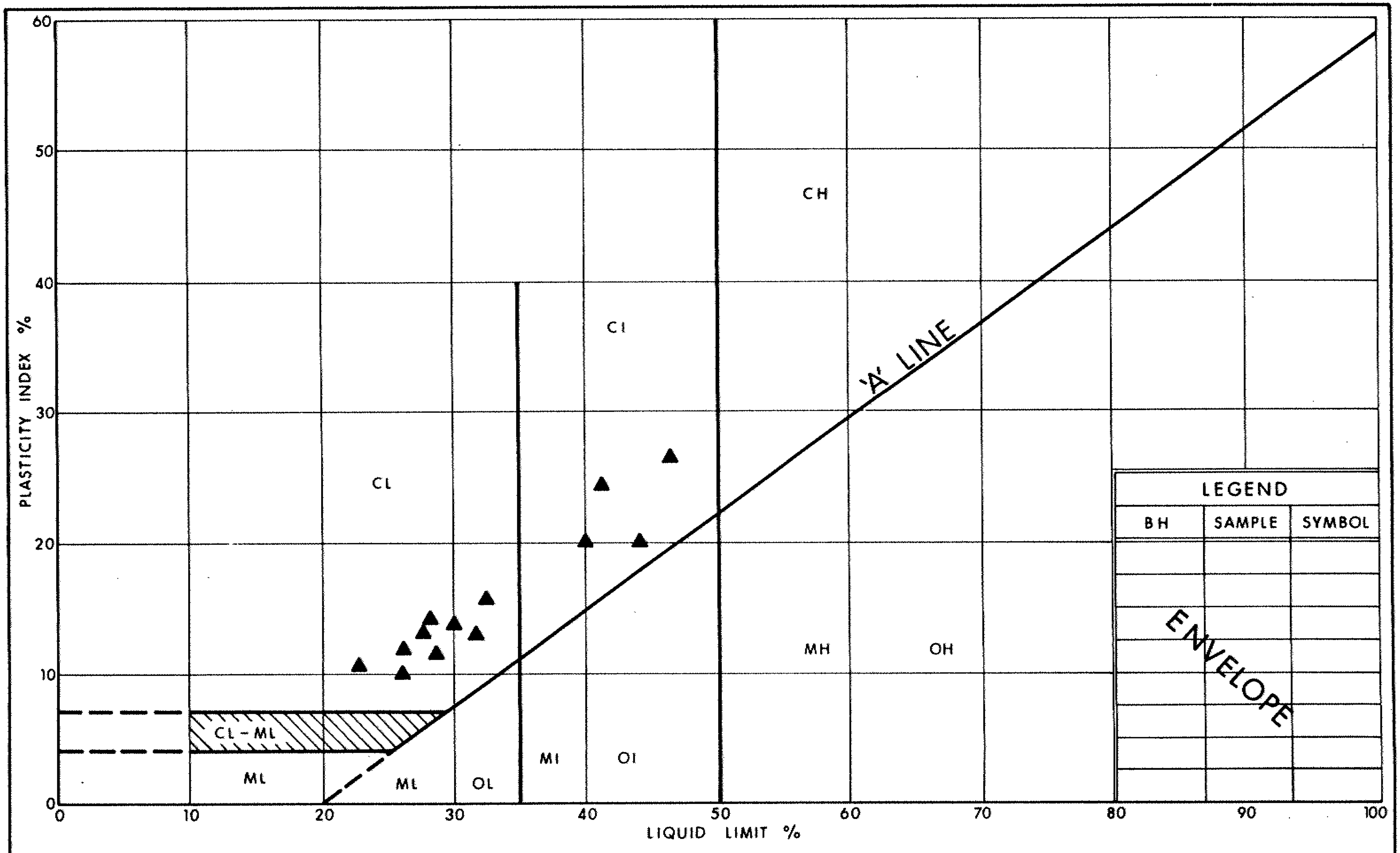
Oct 75, FF-S-21



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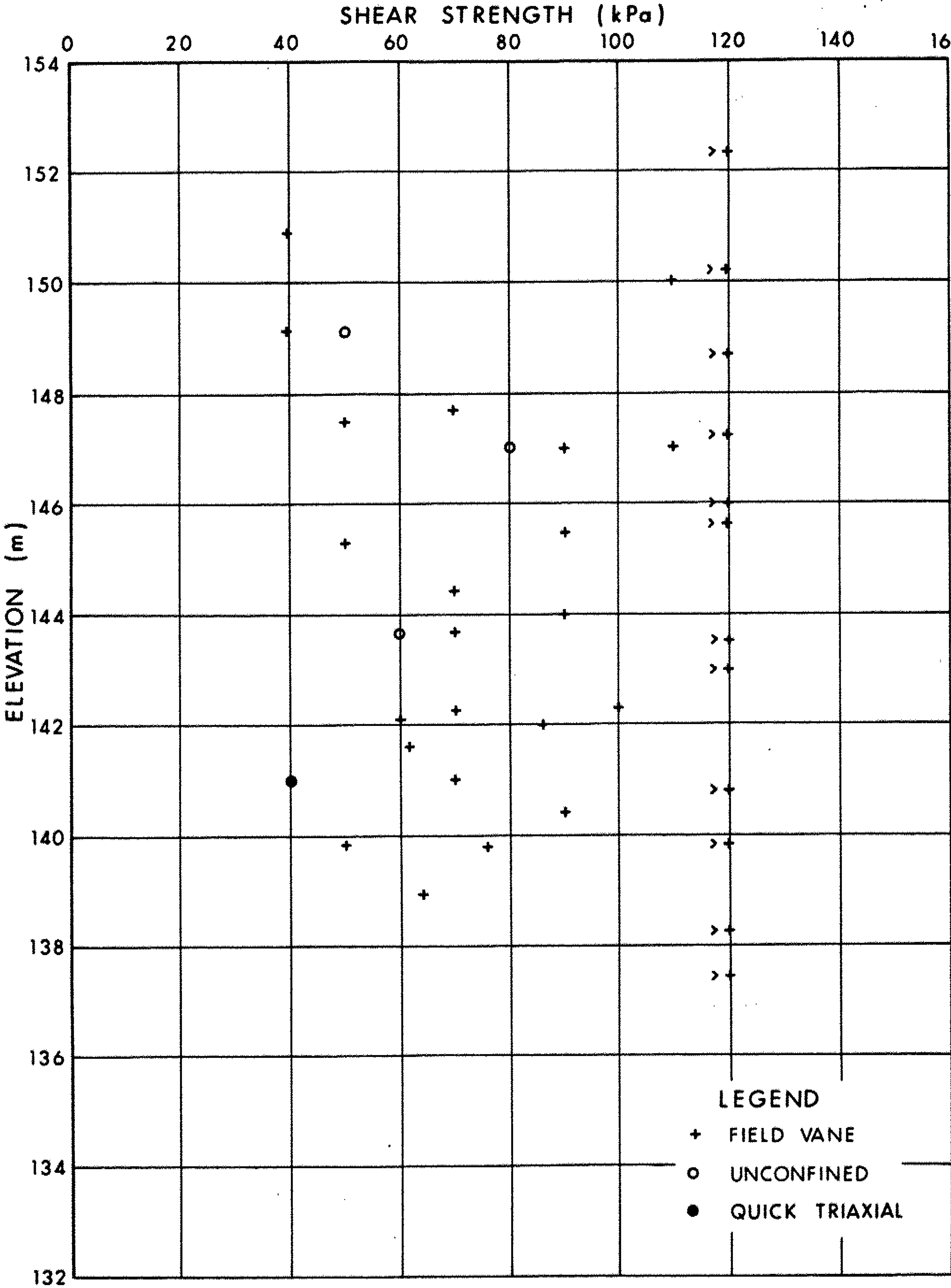
PLASTICITY CHART
SILTY CLAY TO CLAYEY SILT
(Glacial Till)

FIG No 5
W P 141-87-00D

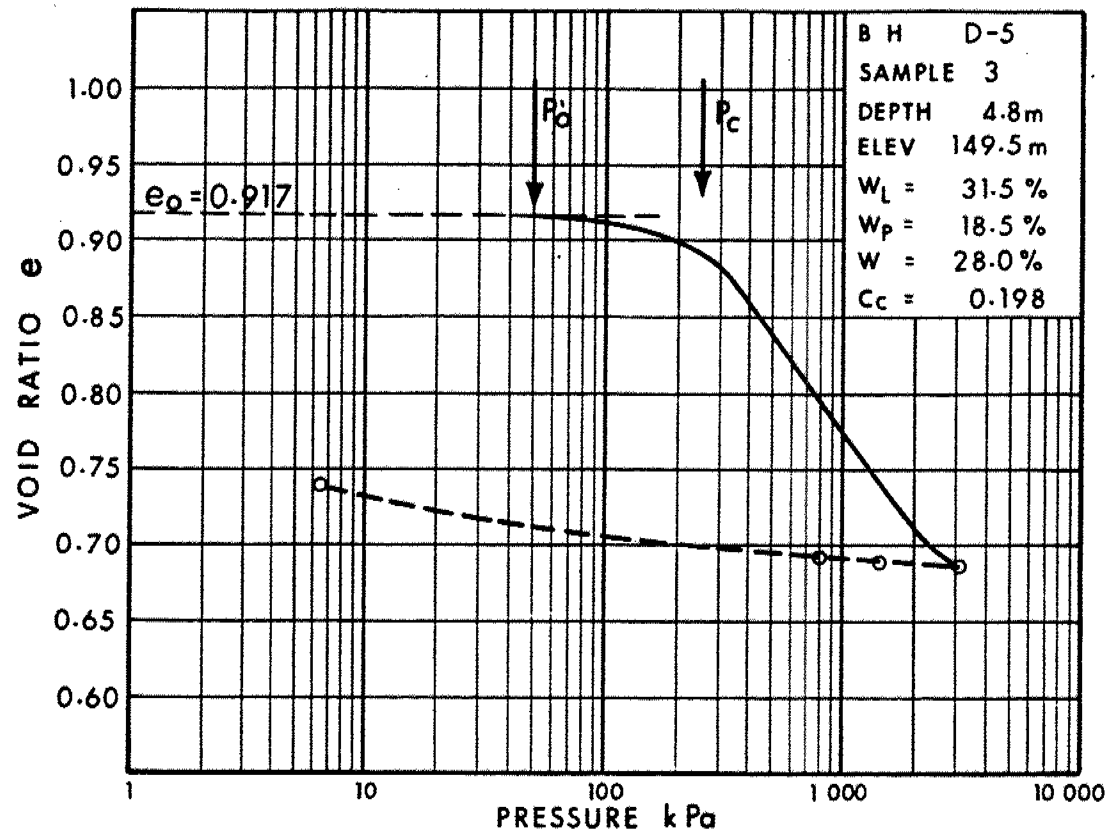
UNDRAINED SHEAR STRENGTH Vs ELEVATION



W P 141-87-00D

Fig 6

VOID RATIO - PRESSURE CURVES



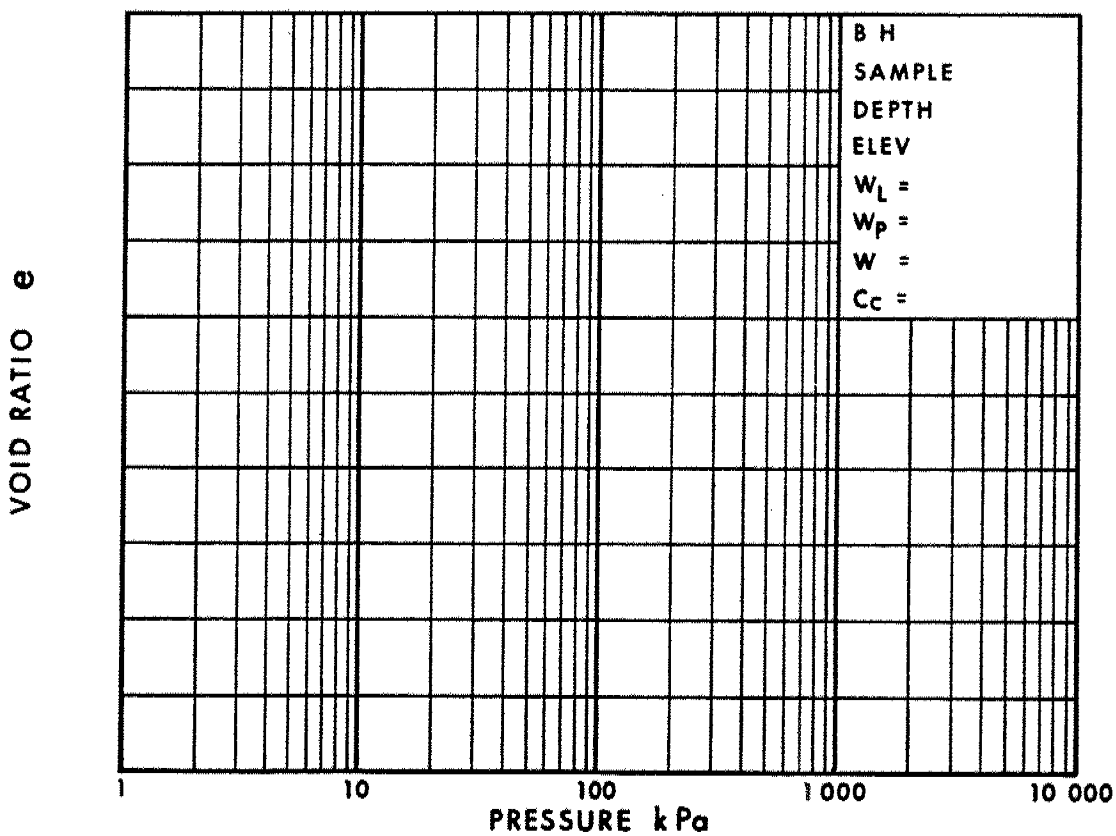
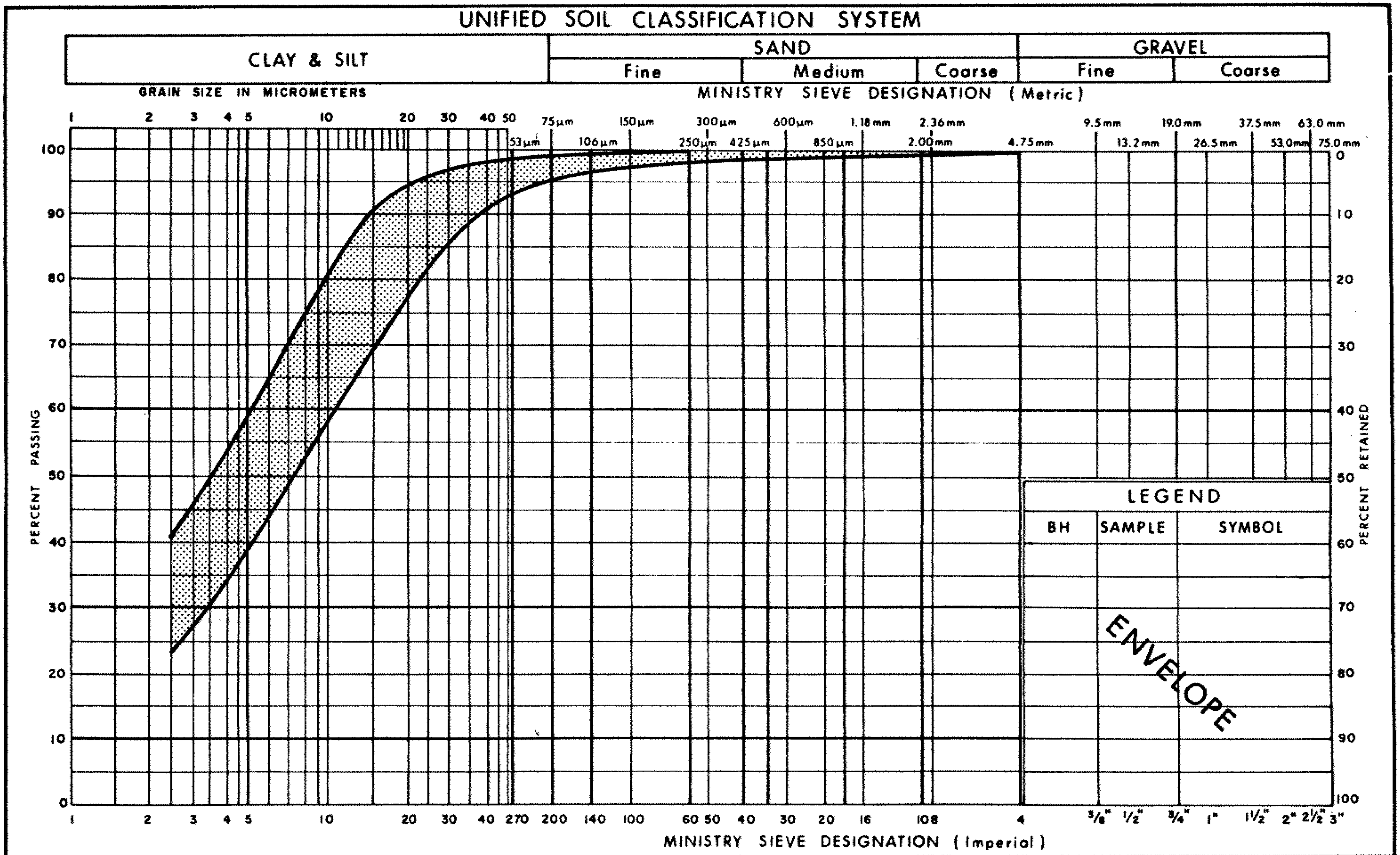


Fig 7

78 12 M



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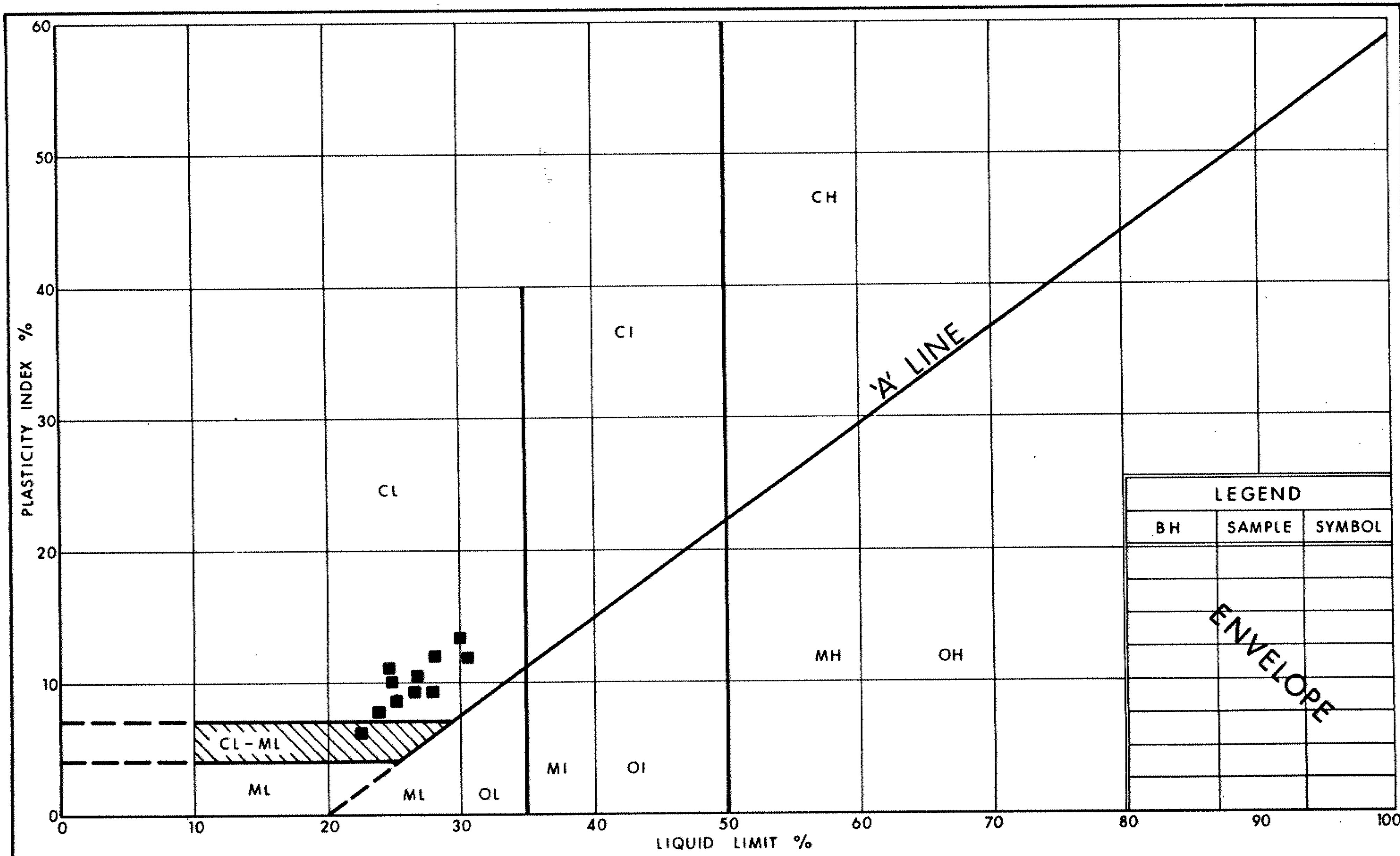
GRAIN SIZE DISTRIBUTION
CLAYEY SILT

FIG No 8

WP 141-87-00D

Oct 75, FF-S-21



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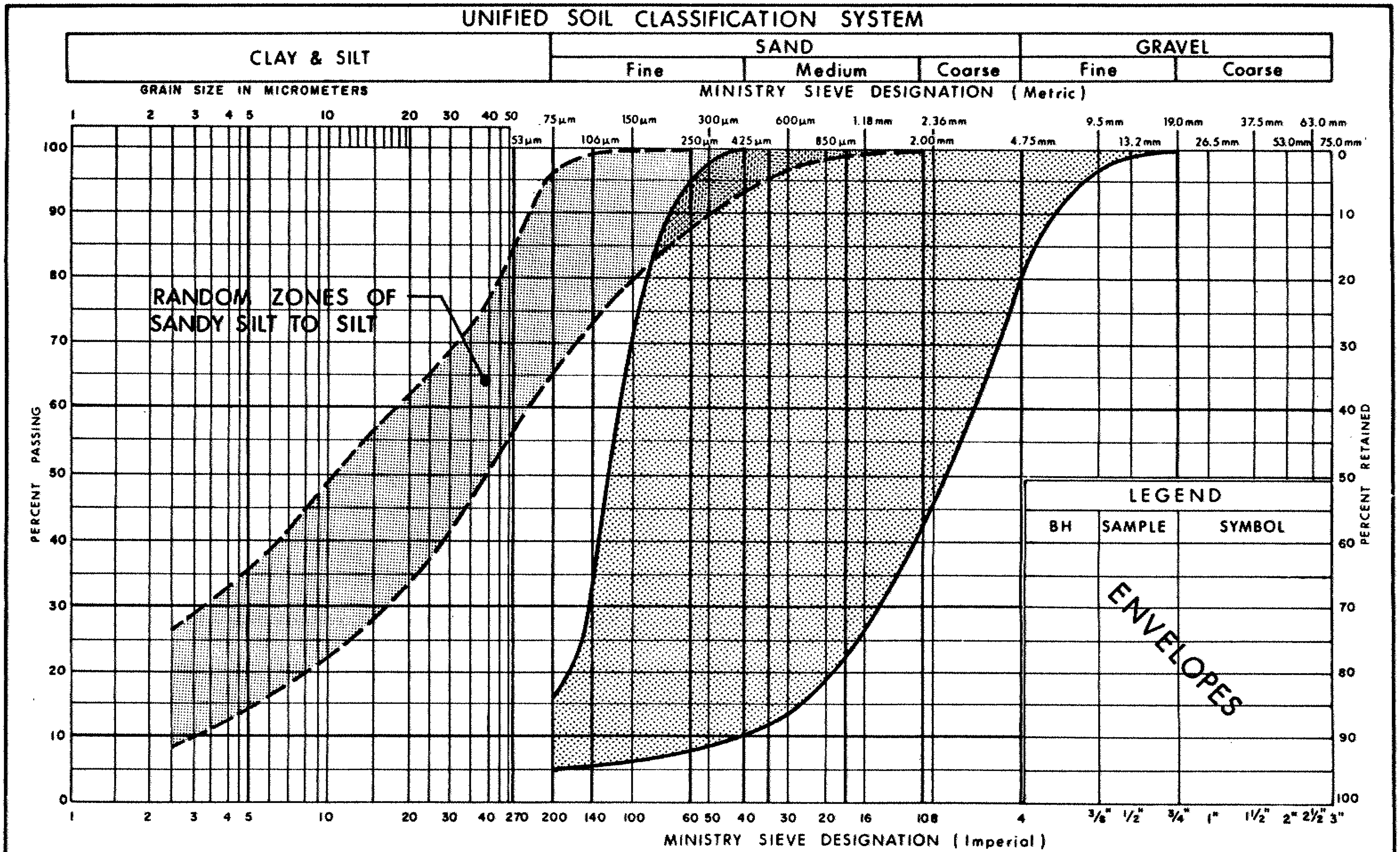
PLASTICITY CHART
CLAYEY SILT

FIG No 9

W P 141-87-00 D

78 12 M



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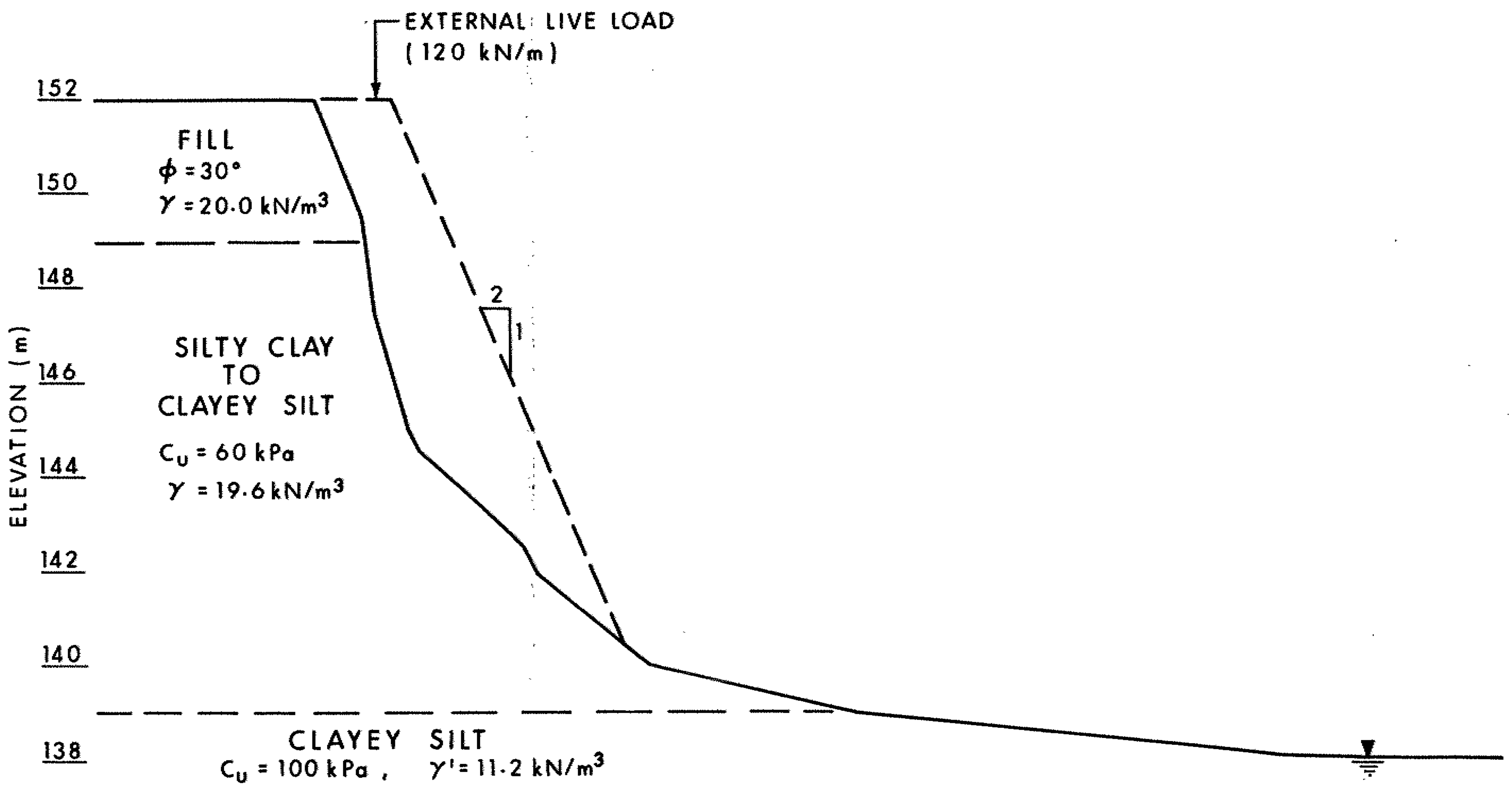
GRAIN SIZE DISTRIBUTION

SAND, SOME SILT

FIG No 10

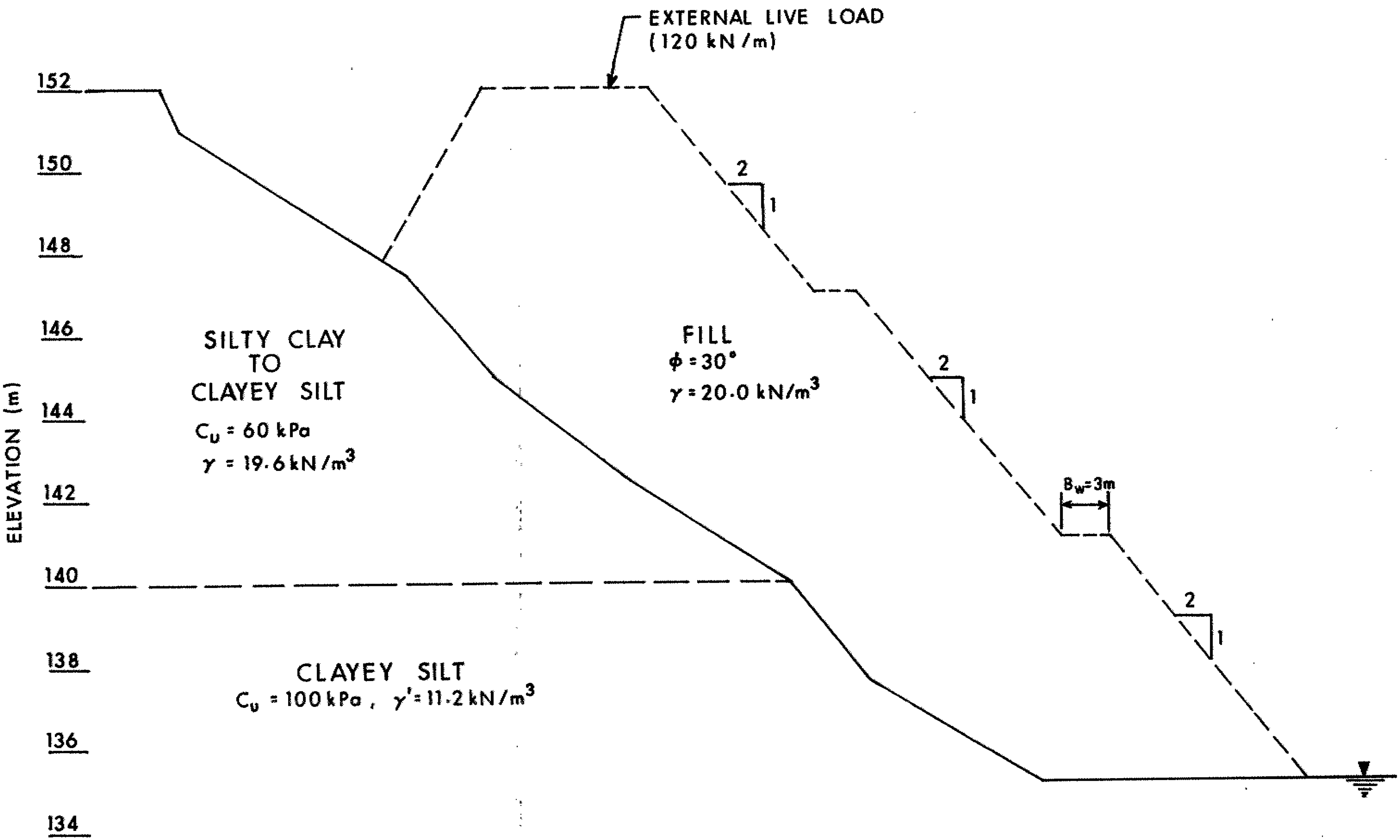
W P 141-87-00D



STABILITY ANALYSIS

FIG 11

WP 141-87-00D



STABILITY ANALYSIS

FIG 12

WP 141-87-00D

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 (R Q D), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_p	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_p	%	PLASTICITY INDEX $= w_L - w_p$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE
e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

ROCK CORE DESCRIPTION
WP 141-87-00

	CORE RECOVERY				CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
C-3	27	45.67-47.19	60	10	45.67-46.28	**OVERBURDEN**, gravel, cobbles, weathered bedrock.
					46.28-47.19	**SHALE**, medium grey to medium dark grey; very fine grained, very thinly laminated; weak to medium strong rock; slightly to medium weathered; extremely close spaced fractures.

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated in zones of poor core recovery)

Logged by: SAS, Soils and Aggregates Section.

ROCK CORE DESCRIPTION
WP 88-78-16

	CORE RECOVERY				CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
D-1	34 35	46.10-47.22 47.22-48.90	73 100	0 0	46.10-48.90	**SHALE**, medium grey to medium dark grey; very fine grained; very thinly laminated; weak to very weak rock; slightly weathered to moderately weathered; very close to extremely close spaced fractures. Minor interbeds of fine grained argillaceous limestone (5%).
D-2	22	47.55-49.07	92	7	47.55-49.07	**SHALE**, medium grey to medium dark grey; very fine grained; very thinly laminated; weak to very weak rock; slightly weathered to moderately weathered, intensely weathered sections at 47.60m and 48.18m; very close to extremely close spaced fractures. Minor interbeds of fine grained argillaceous limestone (8%).
D-4	22	44.81-46.33	60	8	44.81-44.98 44.98-46.33	**OVERBURDEN**, cobbles, weathered, bedrock. **SHALE**, medium grey to medium dark grey; very fine grained; very thinly laminated; weak to very weak rock; moderately weathered to highly weathered; very close to extremely close spaced fractures. Minor interbeds of fine grained argillaceous limestone (20%).
D-8	25	42.98-44.65	100	15	42.98-44.65	**SHALE**, medium grey to medium dark grey; very fine grained; very thinly laminated; weak to very weak rock; slightly weathered to moderately weathered; very close to extremely close spaced fractures. Minor interbeds of fine grained argillaceous limestone (11%).

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated in zones of poor core recovery)

Logged by: SAS, Soils and Aggregates Section.

Ministry of Transportation Ontario

RECORD OF BOREHOLE No C-1

METRIC

W P 141-87-00C | LOCATION Co-ords: N 4 847 519.6; E 298 137.8 | ORIGINATED BY TS

DIST 6 HWY 407 | BOREHOLE TYPE H.S. Auger | COMPILED BY TS

DATUM Geodetic | DATE 1989 11 24 | CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) / PLASTIC LIMIT Wp, NATURAL MOISTURE CONTENT W, LIQUID LIMIT WL	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
136.5	Ground Surface								
0.0	Trace Organics	1	SS	7		136			
	Firm to Stiff / Stiff to Hard; Brown / Grey	2	SS	9	▽				
		3	SS	11		134			
		4	SS	22			o—	21.6	0 2 73 25
		5	SS	30					
	Clayey Silt	6	SS	32		132			
	Trace of Sand	7	SS	28		130			
		8	SS	30		128	⊢o⊣	21.3	0 5 65 30
		9	SS	12					
125.8 / 10.7	Sandy Silt	10	SS	12		126	⊢⊣ o		0 1 84 15
		11	SS	18		124			0 36 60 4
	Compact / V. Dense	12	SS	120			o ⊢⊣		28 32 34 6
	Occ. Gravel Seams	13	SS	120/15cm		122			
						120			
		14	SS	90		118			
						116			
		15	SS	94		114			
111.8 / 24.7	Gravel, Boulders and Cobbles	16	AS	–	▽	112			
	End of Borehole				Artesian Condition Encountered				

*Artesian Head 3.0m Above Ground Surface

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

WATER CONTENT (%) 10 20 30

+3, ×5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No C-2

METRIC

W P 141-87-00C LOCATION Co-ords: N 4 847 540.1; E 298 173.9 ORIGINATED BY BC

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger COMPILED BY BC

DATUM Geodetic DATE 1989 11 28 CHECKED BY

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
147.4	Ground Surface					
0.0	Sand, Trace Silt Trace Gravel (Fill) Brown, V. Loose to Loose	1	SS	5		
		2	SS	4		10 81 (9)
142.4		3	SS	7		
5.0	Silty Clay to Clayey Silt Trace Gravel Occ. Sand Seams Firm to V. Stiff (Glacial Till)	4	SS	5		
		5	SS	10		
		6	TW	PH		
135.2		7	TW	PH		
12.2	Clayey Silt Trace Sand Grey	8	SS	9	Water level ~135.2	
		9	SS	15		
		10	SS	14		0 4 66 30
		11	SS	15		
		12	SS	13		
		13	SS	11		
		14	SS	12		
		15	SS	13		
	Stiff / Firm	16	SS	10		
124.5		17	SS	7		
22.9	Sand Some Silt Grey, Loose	18	SS	5		
122.6		19	SS	8		
24.8	End of Borehole					

ELEVATION SCALE: 146, 144, 142, 140, 138, 136, 134, 132, 130, 128, 126, 124

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

+3, ×5 : Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No C-3

METRIC

W P 141-87-00C — LOCATION Co-ords: N 4 847 561.0; E 298 190.8 — ORIGINATED BY TS

DIST 6 HWY 407 — BOREHOLE TYPE H.S. Auger, NW Casing, Washbore, NQ Core — COMPILED BY TS

DATUM Geodetic — DATE 89 11 22 - 25 — CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE:

ELEV / DEPTH	DESCRIPTION
152.2 / 0.0	Ground Surface
	Clayey Silt, Trace Sand, Trace Gravel
	Brown / Grey
	Firm to V. Stiff, Occ. Sand Seams (Glacial Till)
	Silt, Tr. Sand
140.0 / 12.2	Clayey Silt, Trace Sand, Grey, Stiff to V. Stiff
122.0 / 30.2	

SAMPLES:

NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	GR SA SI CL
1	SS	8		
2	SS	9		
3	SS	2		
4	TW	PH		
5	SS	12		0 4 61 35
6	SS	5	20.2	
7	SS	12		0 10 85 5
8	SS	16		
9	SS	23		
10	SS	24		
11	SS	26	21.7	
12	SS	26		
13	SS	27		0 8 69 23
14	SS	29		
15	SS	24		
16	SS	21		
17	SS	15		
18	SS	36		
19	SS	14		
20	SS	14		
21	SS	11		
22	SS	14		

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (152 to 124); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued

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

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

Continued

Ministry of Transportation Ontario

RECORD OF BOREHOLE No C-3 Cont'd   METRIC

W P 141-87-00C   LOCATION Co-ords: N 4 847 561.0; E 298 190.8   ORIGINATED BY TS

DIST 6 HWY 407   BOREHOLE TYPE H.S. Auger, NW Casing, Washbore, NQ Core   COMPILED BY TS

DATUM Geodetic   DATE 89 11 22 - 25   CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
122.0	Continued										
121.7 / 30.5	Clayey Silt										
	Sand, Some Silt, Grey, Compact to Dense		23	SS	10						
							120				
							118				
			24	SS	39						
							116				
			25	SS	28		114				0 62 30 8
							112				
			26	SS	46						
							110				
	Occ. Cobbles Boulders and Gravel						108				
105.9 / 46.3	Bedrock Shale		27	RC	REC 60%		106				RQD = 10%
105.0 / 47.2	End of Borehole										

$+^3$, $\times^5$: Numbers refer to Sensitivity   15 ⟡ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No C-4

METRIC

W P 141-87-00C
LOCATION Co-ords: N 4 847 580.0; E 298 204.5
ORIGINATED BY BC

DIST 6 HWY 407
BOREHOLE TYPE H.S. Auger
COMPILED BY BC

DATUM Geodetic
DATE 1989 11 27
CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
147.0	Ground Surface				
0.0	Clayey Silt, Some Sand, Tr. Gravel, Stiff to V. Stiff, Occ. Sand Seams (Glacial Till); Brown / Grey				
		1	SS	12	
		2	SS	6	
		3	SS	7	8 17 47 28
		4	SS	5	
		5	SS	5	
136.3 / 10.7	Clayey Silt, Grey, Stiff to Hard, Random Zones of Silt				
		6	SS	13	
		7	SS	22	
		8	SS	13	
		9	SS	34	
		10	SS	39	0 1 73 26
		11	SS	25	
		12	SS	23	
		13	SS	19	0 3 67 30
		14	SS	18	
		15	SS	15	
		16	SS	14	
		17	SS	10	
		18	SS	11	
122.2 / 24.8	End of Borehole				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (146, 144, 142, 140, 138, 136, 134, 132, 130, 128, 126, 124); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-1

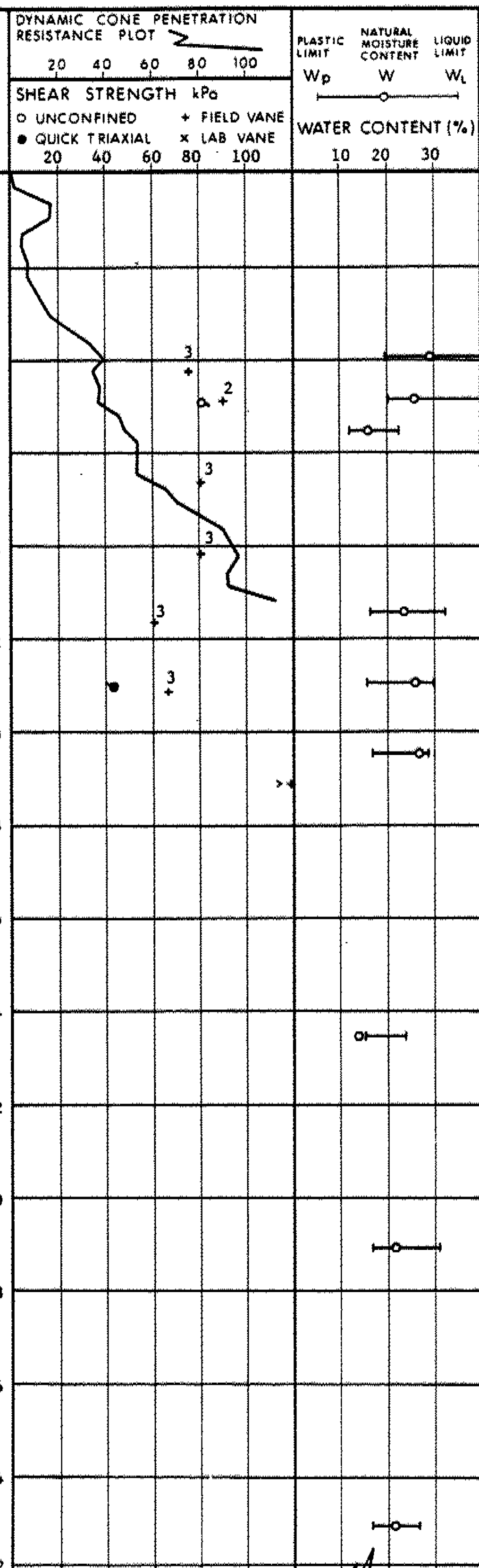
METRIC

W P 88-78-16 | LOCATION Co-ords: N 4 847 491.9; E 298 279.3 | ORIGINATED BY TS

DIST 6 HWY 407 | BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, BXL Rock Core & Cone Test | COMPILED BY TS

DATUM Geodetic | DATE 89 10 21-30 | CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
152.0	Ground Surface							
0.0	Sand (Fill) Brown, Compact	1	AS	-				
150.5 / 1.5	Silty Clay to Clayey Silt, Brown / Grey, Some Sand, Stiff, Occ. Sand Seams (Glacial Till)	2	SS	10	▽	150		
		3	SS	10			19.1	0 0 44 56
		4	SS	6		148		
		5	TW	PH			18.8	0 1 38 61
		6	SS	9				2 24 49 25
		7	SS	9		146		
		8	SS	9		144		
		9	SS	7			20.3	0 3 54 43
		10	TW	PH		142	19.5	0 13 40 45
		11	SS	4		140		
138.3 / 13.7	Clayey Silt, V. Stiff to Hard	12	SS	21		138		
		13	SS	21		136		
		14	SS	32		134		
		15	SS	30			20.8	0 2 73 25
		16	SS	26		132		
		17	SS	32		130		
		18	SS	19		128		
		19	SS	18				
		20	SS	20		126		
		21	SS	16		124		
		22	SS	18				0 2 78 20
121.8 / 30.2	Continued					122		



Continued

+3, ×5 : Numbers refer to Sensitivity

20 15 ◇ 5 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-1 Cont'd — METRIC

W P 88-78-16 — LOCATION Co-ords: N 4 847 491.9; E 298 279.3 — ORIGINATED BY TS

DIST 6 — HWY 407 — BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, BXL Rock Core & Cone Test — COMPILED BY TS

DATUM Geodetic — DATE 89 10 21-30 — CHECKED BY

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, x LAB VANE

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

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

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS (GR SA SI CL)
121.8	Continued					
30.2	Clayey Silt, Very Stiff to Hard	23	SS	16		
		24	SS	16		
		25	SS	76		
		26	SS	45		
115.4 / 36.6	Sand, Tr. Silt, Compact to V. Dense	27	SS	15		0 86 (14)
		28	SS	59		
		29	SS	58		
		30	SS	65		1 89 (10)
		31	SS	33		
	Tr. Gravel	32	SS	44		8 85 (7)
105.9		33	SS	129/23cm		
46.1	Bedrock, Shale, Weak to Very Weak	34	BXL RC	REC 73%		RQD = 34%
		35	BXL RC	REC 100%		RQD = 0%
103.1 / 48.9	End of Borehole					

ELEVATION SCALE: 120, 118, 116, 114, 112, 110, 108, 106, 104

$+^3$, x^5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No. D-2

METRIC

W P 88-78-16 | LOCATION Co-ords: N 4 847 472.6; E 298 274.6 | ORIGINATED BY TS

DIST 6 HWY 407 | BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, NQ Core | COMPILED BY TS

DATUM Geodetic | DATE 1989 11 08-11 | CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL
153.0	Ground Surface									
0.0	Irregular Mixture of Silt, Sand, Slag Ballast (Fill) Brown-Black, Compact	1	SS	11	Water level approx. 150.8	152				
150.6 2.4	Silty Clay to Clayey Silt, Some Sand, Trace Gravel, Firm to V. Stiff, Occ. Sand Seams (Glacial Till); Brown / Grey	2	SS	7		150	+2			
		3	SS	7		148	+2			
		4	SS	8		146	+2			
		5	TW	PH			o, +2	w ≈ 19	21.0	1 13 58 28
		6	TW	PH		144	+2			
		7	SS	4		142	+3	w ≈ 25		4 13 35 48
		8	SS	4		140				
139.3 13.7	Clayey Silt, Firm to V. Stiff	9	SS	22				w ≈ 14		0 4 79 17
		10	SS	20		138				
		11	SS	23		136				
		12	SS	12		134				
		13	SS	20		132, 130				
		14	SS	18		128				
		15	SS	13		126		w ≈ 20		
122.8 30.2	Continued					124				

Dynamic cone penetration resistance plot: 20 40 60 80 100. Shear strength kPa: o Unconfined, + Field Vane, ● Quick Triaxial, × Lab Vane. Plastic limit W_p, natural moisture content W, liquid limit W_L.

Continued

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-2 Cont'd — METRIC

W P 88-78-16 | LOCATION Co-ords: N 4 847 472.6; E 298 274.6 | ORIGINATED BY TS

DIST 6 HWY 407 | BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, NQ Core | COMPILED BY TS

DATUM Geodetic | DATE 1989 11 08-11 | CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
122.8 / 30.2	Continued							
	Clayey Silt, Firm to Very Stiff	16	SS	9		122		
						120		
		17	SS	5		118		
116.1 / 36.9	Sand, Tr. Silt, Compact to V. Dense, Occ. Gravelly Seams	18	SS	50		116		
						114		
		19	SS	20		112	o	17 78 (5)
		20	SS	56		110		
						108		
		21	SS	50		106		
105.5 / 47.5	Bedrock Shale Weak to Very Weak	22	RC	REC 92%		104		RQD = 20%
103.9 / 49.1	End of Borehole							

Soil profile / Samples / Dynamic cone penetration resistance plot (20 40 60 80 100) / Shear strength kPa: O Unconfined, + Field vane, ● Quick triaxial, × Lab vane / Plastic limit W_p, Natural moisture content W, Liquid limit W_L / Water content (%) 10 20 30 / Unit weight γ

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-3

METRIC

W P 141-87-00D LOCATION Co-ords: N 4 847 504.0; E 298 204.0 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger COMPILED BY TS

DATUM Geodetic DATE 1989 11 27 CHECKED BY

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
136.0	Ground Surface									
0.0	Interbedded Layers of Sand and Gravel Tr. Organics, Brown / Grey	1	SS	4	▼					
		2	SS	2		134	+2			
		3	SS	14						
		4	SS	12						
	Clayey Silt Tr. Sand, Tr. Gravel Stiff to Hard	5	SS	22		132		~20		0 5 61 34
		6	SS	19						
		7	SS	27		130				
		8	SS	20		128		~20		0 0 77 23
		9	SS	20		126				
124.9		10	SS	15						
11.1	Silt Tr. Clay, Tr. Sand, Loose / V. Dense	11	SS	7		124				
		12	SS	85		122				
		13	SS	100/ 15cm						0 5 85 10
119.1		14	SS	120/ 10cm		120				
16.9	End of Borehole									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-4 METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 539.4; E 298 210.1 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger, NW Casing, Washbore, NQ Rock Core COMPILED BY TS

DATUM Geodetic DATE 89 11 13-21 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

ELEV / DEPTH	DESCRIPTION
152.1	Ground Surface
0.0	Clayey Silt With Interbedded Layers of Sand (Fill) Brown to Grey V. Soft to Stiff
139.9 / 12.2	Clayey Silt Grey, Stiff to Hard
	Sandy Silt
121.9 / 30.2	Continued

NUMBER	TYPE	'N' VALUES	SHEAR STRENGTH	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	2			
2	SS	3			
3	SS	3			0 32 64 4
4	SS	6			
5	SS	6	+2	20.0	0 2 78 20
6	SS	8			
7	SS	15	>+	20.2	4 22 49 25
8	SS	20			
9	SS	25			0 1 79 20
10	SS	30		22.0	
11	SS	53			
12	SS	32			
13	SS	17		21.2	
14	SS	11			
15	SS	13			0 0 88 12

Elevation scale: 152, 150, 148, 146, 144, 142, 140, 138, 136, 134, 132, 130, 128, 126, 124, 122

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

Continued

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

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

Continued

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-4 Cont'd METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 539.4; E 298 210.1 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger, NW Casing, Washbore, NQ Rock Core COMPILED BY TS

DATUM Geodetic DATE 89 11 13 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	WATER CONTENT (%) Wp W WL (10 20 30)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
121.9	Continued										
30.2	Clayey Silt Grey Stiff to Hard		16	SS	10		120				
118.6 / 33.5	Sand Some Silt Grey, Compact to V. Dense		17	SS	45		118				
			18	SS	20		116, 114				
			19	SS	72		112, 110		o (~11)		0 85 14 1
	Occ. Cobbles Boulders and Gravel		20	SS	120/8 cm						
107.1			21	SS	65		108				
45.0	Bedrock Shale		22	NQ RC	REC 60%						RQD = 8%
105.8 / 46.3	End of Borehole						106				

OFFICE REPORT ON SOIL EXPLORATION

+3, x5 : Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-4A

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 520.0; E 298 212.0 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE B-Casing, Washbore COMPILED BY TS

DATUM Geodetic DATE 1989 11 30 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
143.5	Ground Surface										
0.0	Sand, Tr. Gravel (Fill) Brown, Loose to Compact					*					
			1	SS	6		142				
			2	SS	21		140				
138.9 / 4.6	Clayey Silt With Interbedded Layers of Sand (Fill) Brown, Stiff to Hard		3	SS	37		138				
136.9			4	SS	22						
6.6	End of Borehole *Borehole Dry										

OFFICE REPORT ON SOIL EXPLORATION

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No D-5

METRIC

W P 141-87-00D
LOCATION Co-ords: N 4 847 605.0; E 298 115.0
ORIGINATED BY TS

DIST 6 HWY 407
BOREHOLE TYPE H.S. Auger
COMPILED BY TS

DATUM Geodetic
DATE 1989 11 16-17
CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
154.3	Ground Surface						
0.0	Silty Clay to Clayey Silt Trace Gravel Grey, Firm to V. Stiff Occ. Sand Seams (Glacial Till)				*		
		1	SS	7			
		2	SS	3			
		3	TW	PH		19.5	0 4 61 35
		4	TW	PH			
		5	SS	11			
		6	SS	8			
		7	SS	10		20.1	6 7 60 27
		8	SS	5			
		9	SS	10			
139.1 / 15.2	Clayey Silt Grey, Stiff to V. Stiff	10	SS	12			
		11	SS	22		20.0	0 2 74 24
		12	SS	32			
134.0 / 20.3	End of Borehole *Borehole Dry	13	SS	22			

Elevation scale: 154, 152, 150, 148, 146, 144, 142, 140, 138, 136

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

+3, ×5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No D-6

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 511.1; E 298 240.4 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger COMPILED BY TS

DATUM Geodetic DATE 89 11 20 CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR SA SI CL
152.5	Ground Surface								
0.0	Sand, Tr. Silt (Fill) Brown, V. Loose		1	SS	4	*	152		
150.5 2.0	Clayey Silt With Interbedded Layers of Sand (Fill) Brown, Firm		2	SS	6		150		
			3	SS	6		148		0 5 73 22
			4	SS	5		146		
			5	SS	7			19.4	0 2 84 14
			6	SS	6		144	19.2	0 14 64 22
141.8 10.7	Sand, Some Silt (Fill) Brown, Very Loose to Loose		7	SS	4		142		1 79 15 5
			8	SS	6		140		
138.6 13.9	Clayey Silt, Tr. Gravel Tr. Organics Grey, Firm to Stiff		9	SS	7		138		6 72 18 4
136.8			10	SS	14				
15.7	End of Borehole *Borehole Dry								

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100
PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 10 20 30
REMARKS & GRAIN SIZE DISTRIBUTION (%)

+³, ×⁵ : Numbers refer to Sensitivity

20
15 ⌖ 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No D-6A

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 504.0; E 298 229.0 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE B-Casing, Washbore COMPILED BY TS

DATUM Geodetic DATE 1989 11 29 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
143.5	Ground Surface						
0.0	Sand, Some Gravel, Trace Silt (Fill) Brown, V. Loose to Compact	1	SS	2	*		
		2	SS	6			20 77 (3)
		3	SS	8		142	
		4	SS	9			
		5	SS	12		140	26 59 (15)
138.9 / 4.6	Clayey Silt (Fill) Brown, V. Stiff	6	SS	19		138	
137.3 / 6.2	Clayey Silt Grey, Tr. Organics	7	SS	29			4 6 65 25
136.9 / 6.6	End of Borehole						
	* Borehole Dry						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

$+^3$, x^5 : Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-7

METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 530.8; E 298 232.1 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger COMPILED BY TS

DATUM Geodetic DATE 89 11 20 CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
152.5	Ground Surface					
0.0	Silty Sand to Sandy Silt (Fill) V. Loose to Loose				*	
		1	SS	2		11 62 23 4
		2	SS	3		
	Clayey Silt Brown, Firm	3	SS	8		0 12 66 22
		4	SS	4		
		5	SS	5		2 41 54 3
		6	SS	5		
		7	SS	8		
		8	SS	9		0 30 66 4
		9	SS	9		1 63 31 5
		10	SS	8		4 73 19 4
136.3 16.2	End of Borehole Auger Refusal Probable Culvert Roof *Borehole Dry					

Elevation scale: 152, 150, 148, 146, 144, 142, 140, 138

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

OFFICE REPORT ON SOIL EXPLORATION

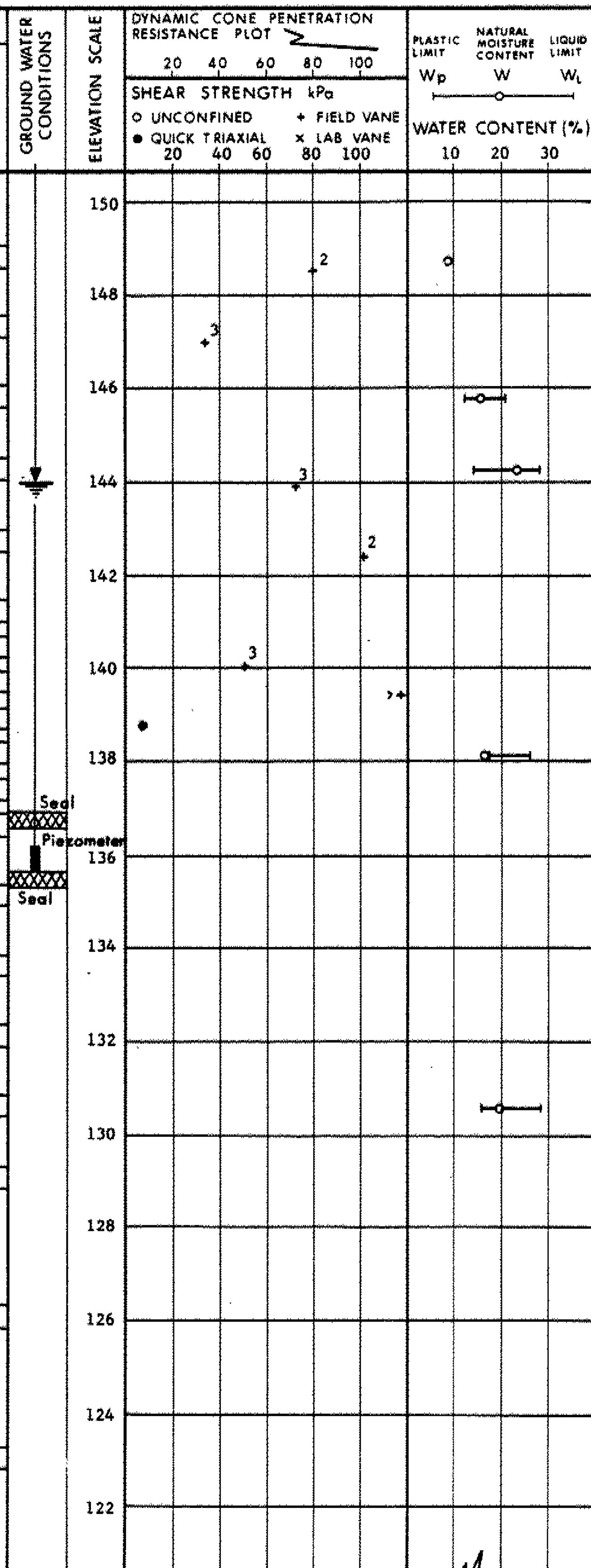
RECORD OF BOREHOLE No D-8 METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 516.1; E 298 252.3 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, NQ Rock Core COMPILED BY TS

DATUM Geodetic DATE 89 11 02-08 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	GR SA SI CL
150.6	Ground Surface					
0.0	Sand, Some Silt With Interbedded Layers of Clayey Silt Brown, Loose (Fill)	1	SS	9		14 56 29 1
		2	SS	6		
		3	SS	4		0 13 75 12
144.5 / 6.1	Clayey Silt Grey, Firm to Stiff Occ. Sand Seams (Glacial Till)	4	SS	4		0 7 52 41
		5	SS	8	20.2	
		6	SS	12		
		7	SS	4		
139.9 / 10.7	Clayey Silt Grey, Firm to Hard	8	SS	17		
		9	SS	22		
		10	SS	19		2 1 72 25
		11	SS	22	20.3	
		12	SS	28		
		13	SS	25	21.6	
		14	SS	23		
		15	SS	19		
		16	SS	16		0 5 60 35
		17	SS	14		
		18	SS	18		
		19	SS	10		
120.4 / 30.2	Continued					



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

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

Continued

Ministry of Transportation Ontario

RECORD OF BOREHOLE No D-8 Cont'd METRIC

W P 88-78-16 LOCATION Co-ords: N 4 847 516.1; E 298 252.3 ORIGINATED BY TS

DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger, BW Casing, Washbore, NQ Rock Core COMPILED BY TS

DATUM Geodetic DATE 89 11 02-08 CHECKED BY

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
120.4	Continued							
30.2		20	SS	9		120		
	Silt, Tr Sand V. Dense	21	SS	107		118 / 116		0 7 79 14
	Clayey Silt Grey Firm to Hard	22	SS	36		114		
111.9 / 38.7	Sand With Silt Grey, Compact	23	SS	12		112 / 110		
107.7	Some Gravel	24	SS	120	15 cm	108		17 38 31 14
42.9	Bedrock Shale Weak to Very Weak	25	NQ RC	REC 100%				RQD = 15%
105.9 / 44.7	End of Borehole					106		

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

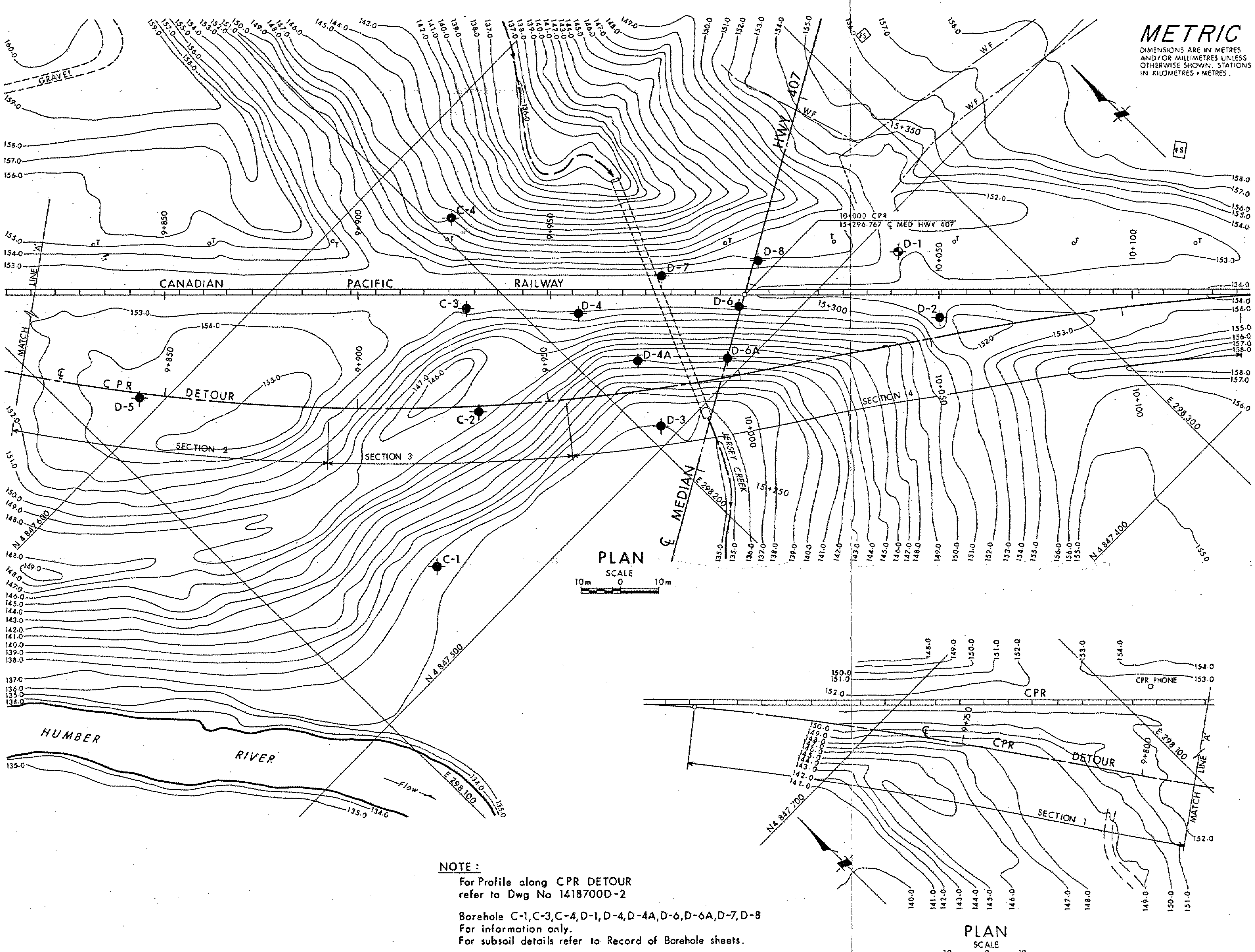
PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

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

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

OFFICE REPORT ON SOIL EXPLORATION



METRIC

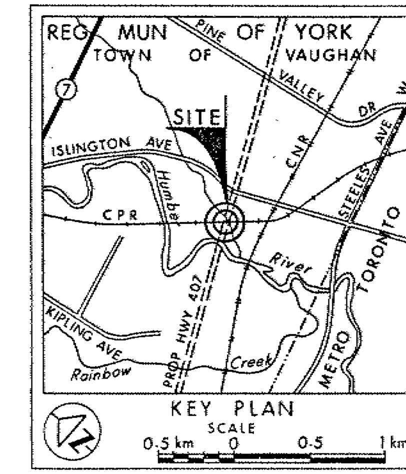
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No
WP No 141-87-00D

HWY 407/CPR DETOUR

SHEET

BORE HOLE LOCATIONS & SOIL STRATA



KEY PLAN
SCALE
0.5 km 0 0.5 1 km

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

No	ELEVATION	CO-ORDINATES NORTH	EAST
C-1	136.5	4 847 519.6	298 137.8
C-2	147.4	4 847 540.1	298 173.9
C-3	152.2	4 847 561.0	298 190.8
C-4	147.0	4 847 580.0	298 204.5
D-1	152.0	4 847 491.9	298 279.3
D-2	153.0	4 847 472.6	298 274.6
D-3	136.0	4 847 504.0	298 204.0
D-4	152.1	4 847 539.4	298 210.1
D-4A	143.5	4 847 520.0	298 212.0
D-5	154.3	4 847 605.0	298 115.0
D-6	152.5	4 847 511.2	298 240.4
D-6A	143.5	4 847 504.0	298 229.0
D-7	152.5	4 847 530.8	298 232.1
D-8	150.6	4 847 516.1	298 252.3

NOTE:

For Profile along CPR DETOUR refer to Dwg No 1418700D-2

Borehole C-1, C-3, C-4, D-1, D-4, D-4A, D-6, D-6A, D-7, D-8
For information only.
For subsoil details refer to Record of Borehole sheets.

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

Geocres No 30M13-97

HWY No 407			DIST 6
SUBM'D TS	CHECKED	DATE 90 03 14	SITE
DRAWN	CHECKED	APPROVED	DWG 1418700D-1

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10

METRIC

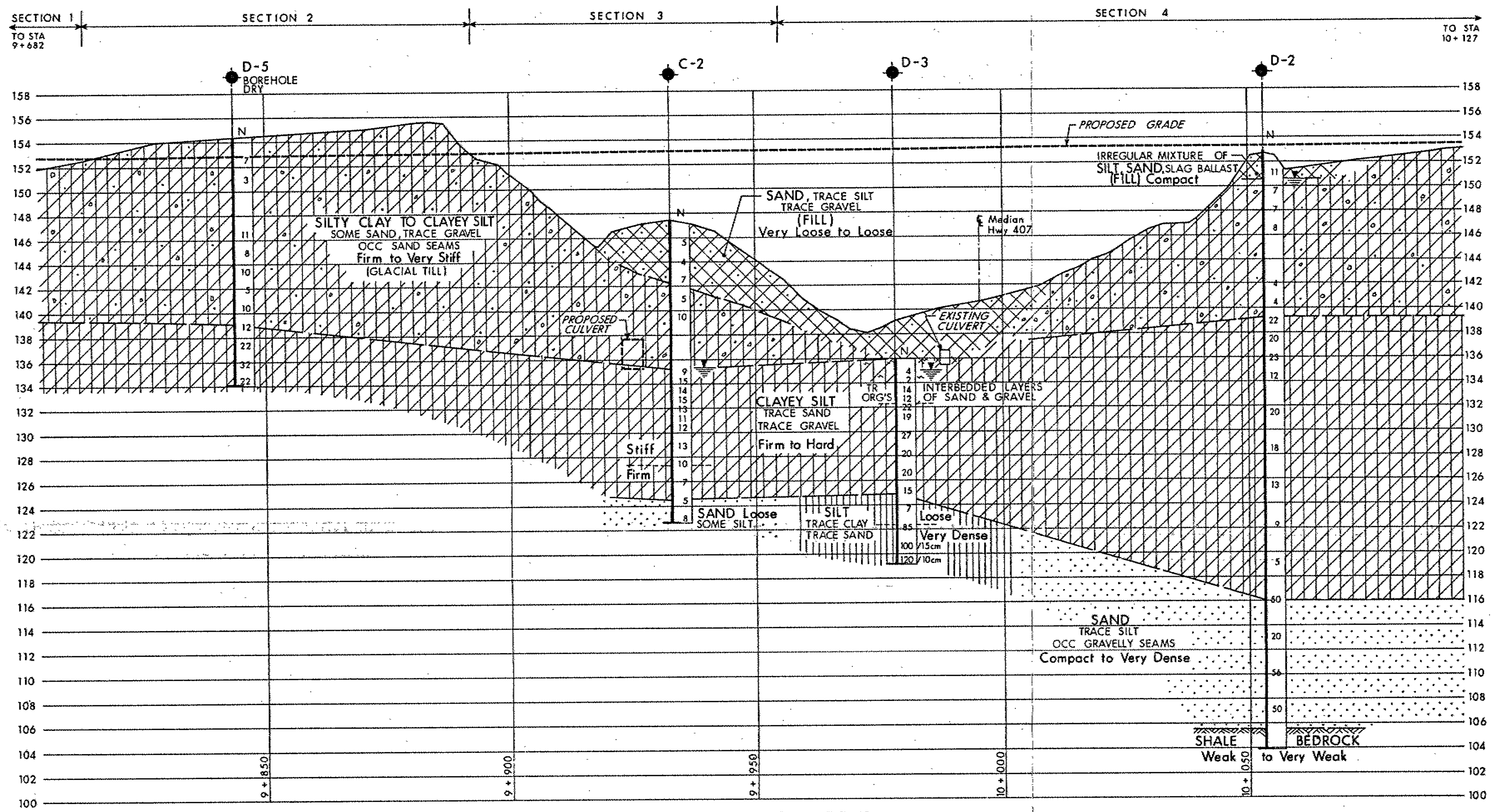
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No
WP No 141-87-00D

HWY 407/CPR DETOUR

SHEET

BORE HOLE LOCATIONS & SOIL STRATA



℄ PROFILE CPR DETOUR

SCALE
10m 5 0 10m Hor
4m 2 0 4m Vert

NOTE:
For Plan refer to Dwg No 1418700D-1

SEE DWG No 1418700D-1

KEY PLAN
SCALE

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 89 11

No	ELEVATION	CO-ORDINATES NORTH	EAST
C-2	147.4	4 847 540.1	298 173.9
D-2	153.0	4 847 472.6	298 274.6
D-3	136.0	4 847 504.0	298 204.0
D-5	154.3	4 847 605.0	298 115.0

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV	DATE	BY	DESCRIPTION

Geocres No 30M13-97

HWY No 407			DIST 6
SUBM'D TS	CHECKED	DATE 90 03 01	SITE
DRAWN DT	CHECKED	APPROVED	DWG 1418700D-2

memorandum

Ontario

To:	R. Middleton Senior Project Supervisor Contract 92-40	Date: 1993 08 23
From:	Foundation Design Section Room 315, Central Building	
Subject:	CPR Detour Embankment Cracks along the west slope	

Based on visual observations made during the site visit of 1993 08 19 by the undersigned and a review of the cross sections forwarded to our office today, we are providing the following comments:

1. The cracks along the west slope of the CPR detour embankment are probably associated with differential settlements along this slope, rather than a deep seated movement.

2. It is possible that, if water enters these cracks, this may trigger surficial sloughing/sliding. To avoid this possibility, the cracks should be sealed using impervious materials such as clay, cement grout etc. Alternatively, consideration may also be given to covering the cracks using an impervious geomembrane anchored near the crest.

3. The above treatment should be carried out as soon as possible to avoid further deterioration of the slope due to rain and associated surface run off.

4. We recommend that visual monitoring of the slope should continue even after the above treatment.

5. As part of monitoring, install two or three stakes along the slope at the three stations. Measure the slope distance between the stakes periodically to check if any movement is taking place along the slope.

We trust that the recommendations given herein are sufficient for your needs. Please call this office if you need further assistance on this project.

Biyer

Balu Iyer, P.Eng.
Sr. Foundation Engineer

SNC•LAVALIN INC.
FENCO ENGINEERS INC.
MacLAREN ENGINEERS (1991) INC.
MacLAREN PLANSEARCH (1991) INC.

Telephone: 416-756-2300
Telephone: 416-756-3400
Telephone: 416-756-3700
Telephone: 416-756-2009

SNC•LAVALIN

2235 Sheppard Avenue East
Willowdale, Ontario M2J 5A6
FAX: 416-756-2266

1 Lansing Square, Suite 200
Willowdale, Ontario M2J 4P8
FAX: 416-491-0953

DATE: MAR 11 93

TO:
Telecopier: 235 5240
Name: DR B. IVOR
Company: M.T.O
Location: 401 BOWNSVIEW

CONTRACT 92-40

FROM:
Name: B. [illegible]
Company: F[illegible]

MESSAGE:

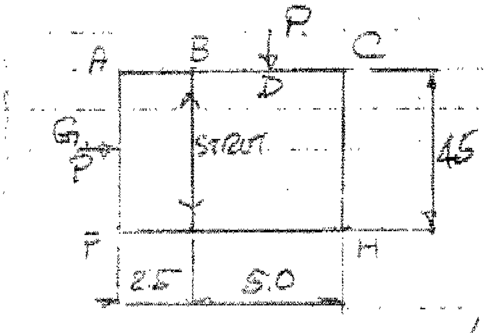
- Discussed attached calcs w. Bob Jeffries on 1993 03 18 PN.
- If Fenco's calcs indicate that a couple of walers are to be strengthened, they can propose that.
- Use of 0.35, with min. (>1.0) FS.
- use of 0.2 may work, provided there is an arching effect.
- Acc to Bob. J., the shaft has already been constructed!! He is going to the site to verify this.

BB 93 03 18

TOTAL NUMBER OF PAGES: 2 (Including Cover Sheet) ivalin Engineers

DESIGN BRIEF

PROJECT TITLE	JERSEY CREEK CULVERT	Project No. 05114	Act	Sub. 1 Sub. 2 Sub. 3	Design No.	
SUBJECT	ACCESS SHAFT				Page	of
DESIGNED BY:	B.T.P 93 03 03	CHECKED BY:			Y M D	



PLAN

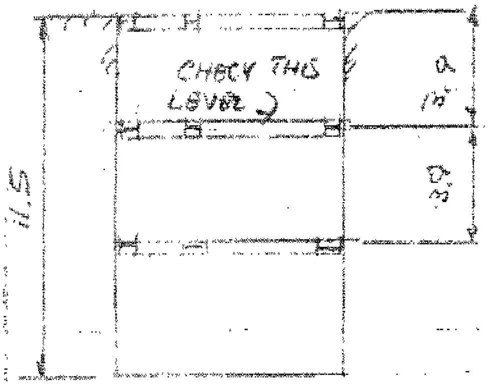
ASSUME RECTANGULAR PRESSURE DIAGRAM
ASSUME NO MOMENT CONNECTIONS AT A, G, F, E, H.

$K_a = 0.35$ (AS USED IN STAGE I)

PRESSURE $= 0.65 \times 0.35 \times 20 \times 11.5 = 52.3$ kPa

U.D.L ON WALERS $= 52.3 \times 3.9 = 204$ kN/m

P.L P $= 204 \times 2.5 = \underline{510\ \text{kN}}$



SECTION

FOR WALER AC

$$M_B = \frac{WL}{8} = \frac{510 \times 5}{8} = \underline{320}\ \text{kNm}$$

$$M_{BC} = \frac{510 \times 5}{4} - \frac{320}{2} = \underline{478}\ \text{kNm}$$

FOR WALER AF

$$M_{AF} = \frac{510 \times 4.5}{4} = \underline{574}\ \text{kNm}$$

ALLOWABLE MOMENT $= .67 F_y \times 1520$

$= .67 \times 300 \times 1520 = \underline{310\ \text{kNm}}$

WHICH IS < THAN ACTUAL MOMENTS

LEI ENG 001 E 90/02

☐ SNC•LAVALIN INC. Telephone: 416-756-2300
☐ FENCO ENGINEERS INC. Telephone: 416-756-3400
☐ MacLAREN ENGINEERS (1991) INC. Telephone: 416-756-3700
☐ MacLAREN PLANSEARCH (1991) INC. Telephone: 416-756-2009

SNC•LAVALIN

☐ 2235 Sheppard Avenue East, Willowdale, Ontario M2J 5A6, FAX: 416-756-22(·)
☐ 2 Lansing Square, Suite 200, Willowdale, Ontario M2J 4P8, FAX: 416-491-0953

DATE: March 3, 1993

TO:
Telecopier: 235-4382
Name: Mr. R. Jeffries
Company: MTO
Location: Downsview
cc: Dr. B. Iyer (235-5240)

FROM:
Name: B. I. Phelp
Company: Fenco MacLaren

MESSAGE:

JERSEY CREEK ACCESS SHAFT STAGE II SHORING

Please find enclosed a copy of Isherwood Associates calculations using a K = 0.2 for the soil in designing the above access shaft.

We feel this value should be agreed to by the MTO Foundation Group, before accepting the Stage II shoring, because in Stage I a K value of 0.35 was used.

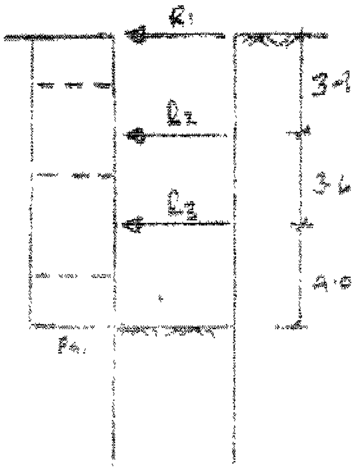
TOTAL NUMBER OF PAGES: 2 (Including Cover Sheet)

RECEIVED
11.15 a.m.
MAR 3 1993
... BRIDGE DEPT

STAGE II

SHORING CALCULATIONS
JERSEY CREEK CULVERT – SHAFT

SURFACE 146.0 BRACES: 146.0
EXC: 134.5 H = 11.5 m 142.1 – 3.9 m.
138.5 – 7.5 m



NO INFLUENCE OF EMBANKMENT OR RAILWAY LOADING.

SOIL ARCHING AROUND SHAFT.

USE $K = 0.2$, $\gamma = 20$, $q = 12$

$p_a = 0.65 \times 0.2(11.5 \times 20 + 12) = 31.5$ kPa.

$R_3 = 31.5 \times 3.8 = 120$ kN/m.

PILE: $M = 31.5 \times 2.5 \times 4^2/10 = 126$ kN m
HP 310×110 RM = 305 kN.m ✓

STRUT: $P = 120 \times 3.75 = 450$ kN
$L = 4.2$ m $P_{allow} = 2057$ kN ✓

WALER: WORST CASE:
$M = 120 \times \dfrac{4.85^2}{2 \times 6} = 235$ kN m.
$f_b = 235/1.54 = 153$, $F_b = 198$, $\dfrac{f_b}{F_b} = 0.77$.

$P = 120 \times 2.4 = 288$ kN.
$f_a = 288/14.1 = 20.4$, $F_a = 180$, $\dfrac{f_a}{F_a} = \dfrac{0.11}{0.88}$ ✓

REGISTERED PROFESSIONAL ENGINEER
BRIAN ISHERWOOD
PROVINCE OF ONTARIO

ISHERWOOD ASSOCIATES
Geotechnical and Construction Consultants
3100 Ridgeway Dr., #3
Mississauga, Ontario
Canada
L5L 5M5
Fax: (416) 820-3492
Tel: (416) 820-3480

Title	Ref.	Date
M.O.T. CONTRACT 92-40 HIGHWAY 407 SHORING CALCULATIONS	92.6	93.03.03
	Scale	Drg.No. 4

7540-1021

SEND TO V. BOEHNKE
Head, Structural Section,
Central Regn. Attn: R. JEFFRIES

FROM B. IYER **DEPT.** Foundn Design **DATE** 1992 12 01

SUBJECT SHORING DETAILS - 407 / CP DETOUR, WP 141-87-00, CONT 92-40

The latest shoring drawing SHIB Rev. 2 dated 92 11 23 is acceptable for stage 1 excavation construction. Further excavations would require additional design and review.

B Iyer

REPLY

REPLY FROM | REPLY DATE

TO WRITE: HANDWRITE OR TYPE. REMOVE AND RETAIN YELLOW COPY. FORWARD BALANCE OF SET.

TO REPLY: WRITE REPLY IN BOTTOM AREA. SNAP SET APART.

Ontario

FOLD AT MARKS FOR USE IN #9 OR #10 WINDOW ENVELOPE

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memorandum

MINISTRY OF TRANSPORTATION
Structural Section
1201 Wilson Avenue
Atrium Tower, 4th Floor
Downsview, Ontario, M3M 1J8
Telephone: 235-5515

DATE: November 16, 1992

TO: J. Cullen
Manager
Construction Office

RE: Contract 92 - 40
Hwy. 407 - Jersey Creek Culvert
Temporary Shoring Submission

A temporary shoring scheme, designed by Isherwood Associates was submitted for the Jersey Creek Culvert at the C.P. detour embankment. The consultant met with our office and the Foundation Office to review the submission and make changes and additions to the design. A second revised submission was then submitted for approval. The details of the shoring scheme have basically been approved however, the concept for the shoring arrangement is not stated and the drawing with a 'unspecified' earth slope line is unacceptable.

The 'unspecified' slope from the end of the shoring to the bottom of the culvert 'appears' to be a 1:1 slope, which is unacceptable. The parameters for slopes regarding the C.P. embankment and the C.P. detour embankment are contained in the corresponding Foundation Reports. The proposed scheme cannot function as a construction access for the construction of the remainder of the Jersey Creek Culvert as indicated on the submission. The submission was discussed at a meeting on November 10, 1992, with M. Devata, B. Iyer, V. Boehnke, K. Pilgrim and myself.

The shoring scheme is unacceptable as it neither provides adequate protection for the C.P. detour embankment or protection for the existing C.P. embankment. Without a proposal for construction of the remainder of the Jersey Creek Culvert, any scheme for shoring at this location appears to be premature.

Three sets of the shop drawing stamped 'Rejected' are returned, for your action in this matter. This is the second submission, plus one revision, returned unapproved.

R. A. Jeffries

R. A. Jeffries
Structural Supervisor
for:
V. F. Boehnke
Head, Structural Section

:rj
Attach:

cc. M. Devata
B. Iyer
R. Temple (Fenco)
B. Isherwood (Isherwood Assoc.)

B.I.

ISHERWOOD ASSOCIATES

3100 Ridgeway Drive, Unit 3
Mississauga, Ontario L5L 5M5
Telephone: (416) 820-3480
Fax: (416) 820-3492

Mr. Larry Ng
Fenco Engineers Inc.
2235 Sheppard Avenue East
Willowdale, Ontario
M2J 5A6

92.26

3 November, 1992

Dear Sir:

Jersey Creek Culvert
MTO CONTRACT 92-40

Thank you for your Notes of Meeting. I confirm our telephone conversation today when I suggested the following corrections:

A.1. second sentence "The shoring scheme...." revise to:

"The shoring scheme is prepared to allow maximum flexibility for the choice of method".

A.1. last sentence "It was agreed...." revise to:

"It was agreed that the shoring drawings should not be interpreted as approval of the side slopes of the future excavation. Approval is required now for stage 1 as defined on SH1. Stage 2 is submitted for review of structural aspects only, and is subject to approval of the method of culvert construction before excavation is taken below elevation 145".

A.3. Delete "inspection" wherever it occurs, replace with "review".

B.3. Second sentence - replace "... for end bearing alone" with "...for toe bearing alone".

C.4. Delete "30 MPa" replace with "30 KPa".

I trust these agree with your understanding of the meeting.

Yours truly,
ISHERWOOD ASSOCIATES

S. Wright
for. Brian Isherwood, P. Eng.

cc. All present
Dave Weltz, Graham Contr.

Foundation and Geotechnical Construction Consultants

ISHERWOOD ASSOCIATES

3100 Ridgeway Drive, Unit 3
Mississauga, Ontario L5L 5M5
Telephone: (416) 820-3480
Fax: (416) 820-3492

Mr. Dave Weltz
Graham Construction Limited
290 Clarence Street
Brampton, Ontario
L6W 1Y4

92.26

2 November, 1992

Jersey Creek Culvert
MOT CONTRACT 92.40
Highway 407

This letter reports on my meeting at MTO Offices on Friday afternoon, October 30, 1992 at 1 pm. I have underlined the undertakings I made at the meeting.

Present:	Bob Jeffries	MTO Structural
	Ken Pilgrim	MTO Structural
	Balu Iyer	MTO Foundations
	Ron Temple	Fenco
	Larry Ng	Fenco
	Brian Isherwood	Isherwood Associates

Agenda was provided by Fenco, copy attached.
Each Item was discussed and was responded to as follows:

A. 1. Drawing is not meant to define method of future culvert construction, as Graham Construction is still investigating several options. Although the 1:1 slope indicated on the drawing conforms to Ontario Regulation 234 for type 3 soil under the Occupational Health and Safety Act, it may not be stable at this particular site. The drawing is not defining this as the cut slope for future work. The 1:1 line is the influence line of the future excavation used for design of the shoring beyond which the shoring would require re-design. Isherwood to modify note on drawing to clarify this.

2. Size and grade of tendons to be clarified by providing a schedule.

3. Isherwood has commitment from Graham to carry out necessary Field Review of shoring installation and will be doing this.

4. Lift-off requirement on the shoring drawing is to put the contractor on notice that such tests may be required.- it is more a question of adjusting the loads to control deflection. Isherwood to decide as part of Field Review.

5. Isherwood standard procedure to be followed for testing tie-backs - <u>to be distributed for review before tests are commenced.</u>

B. 1. Isherwood has taken 14 m as maximum height for calculation of soil pressures. This represents height to centre line of pipe, and takes account of the rigid concrete pipe already in place behind the shoring. Isherwood has analyzed each section, not just the ones shown on the drawings.

2. <u>Attached are calculations for lateral resistance of the toes as requested.</u>

3. Isherwood advised that a friction angle of 10° between the piles and the supported soil is sufficient to account for the full vertical component of the tie loads, and considering the flat angles involved was of the opinion that this load would not reach the toes. The maximum toe shown is 3.5 m, and this has a factor of safety of 1.27 against the vertical load if the wall friction is ignored, and assuming bearing of the pile in cohesive soil.

Iyer was of the opinion that a more conservative approach was appropriate, and considered zero friction at the back of the wall, and bearing capacity based on no cohesion and submerged soil weights <u>Isherwood undertook to extend the toes of piles 8 and 10 to allow for this approach.</u>

4. Same as item A-2.

5. Tributary area perpendicular to face. Component parallel to face distributed through walers to all the piles.

6. Isherwood stated that drawing should have shown double W460 x 74 walers. <u>Isherwood to revise.</u>

C. 1. Isherwood advised that trenches for deadmen should stand, considering cohesive nature of fill and previous experience with similar installations. <u>Will be reviewed in the field.</u>

2. Same points as A-1.

3. Meeting agreed that design parameters were appropriate.

4. Bond stress subject to advance load tests.

5. Isherwood outlined his design assumptions, and showed drawings of previous project (Skydome) with similar geometry that performed very successfully. Influence lines not so critical as with a long continuous face of shoring, since future excavation will be narrowly confined to the pipe installation and will be very short term.

6. Potential sliding surface discussed - no change required to drawings - anchor lengths subject to load tests.

Following the meeting, Bob Jeffreys and the writer visited the site and confirmed that the embankment fill had been built up to 147 approximately at the line of shoring so that exposed height of shoring in near future will be confined to upper tie-rods portion.

Yours truly,
ISHERWOOD ASSOCIATED.

Brian Isherwood

Brian Isherwood, P. Eng.

cc. All present.
Peter Meuendyk (Site)

SOLDIER PILE TOE.

MAXIMUM LOAD = $74 \times 3.25 \times 1.5$ = 361 kN.

TOE :- ORIGINAL DRAWING. 3.5 m DEEP.

SOIL :- STIFF TO HARD CLAYEY SILT, N=15.

T&P (347) N=15, CLAY → C = 2.0ksf (100kPa)

$Res = B.N.(K_p \gamma d^2/2 + 2cd)$

USE : $N=2$, $K_p=1$, $\gamma=10$, $c=100$.

$R = 2(122+700)$ 1645kN F= 4.5.

TOE :- BALU MYER :- USE $C=0$, $K_p=4$.

$Res = 2\times1(4\times10\times\frac{d^2}{2}) = 40d^2$.

for $F=2$, $d = 4.25$ m.

PROVIDE 6m TOE F= 4. ✓

REGISTERED PROFESSIONAL ENGINEER
BRIAN ISHERWOOD
PROVINCE OF ONTARIO

ISHERWOOD ASSOCIATES
Geotechnical and Construction Consultants
3100 Ridgeway Dr., #3
Mississauga, Ontario
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L5L 5M5
Fax: (416) 820-3492
Tel: (416) 820-3480

Title	Ref.	Date
M.T.O. CONTRACT 92-40. HIGHWAY 407 SHORING CALCULATIONS	92.26	92.11.02
	Scale	Drg.No. 3

1992 10 30

CONTRACT 92-40 W.P. 141-87-00C

HWY. 407 - JERSEY CREEK CULVERT

MEETING TO DISCUSS SHOP DRAWINGS SH1 AND SH2 FOR EXCAVATION SHORING

October 30, 1992

AGENDA

A. GENERAL

Extent of excavation shown on SH2 is not in conformity of the excavation slopes given in the Foundation report.

1. Clarify sequence and method for culvert construction.

2. Clarify size and grade of tie-rods and tiebacks (soil anchors) to be used.

3. Design Engineer responsible for the excavation shoring design shall inspect the shoring work installation to ensure work is carried out in accordance with the design assumptions and method.

4. Clarify procedures of lift-off test on the tie-rods.

5. Clarify procedures of testing the tiebacks in advance.

B. STRUCTURAL

1. The critical location for pile and tiebacks design is at pile 8 as presented in design calculations. Cross sections in shoring drawing do not show the critical case. Actual height of soil retained at pile 8 appears to be more than 14 m as used in design calculations.

2. Provide design calculations for resisting the lateral force at bottom of piles.

3. Clarify how vertical load in pile resulting from tie-rod and tiebacks is resisted.

4. Specify size and grade of tie-rods and tiebacks.

5. Check adequacy of maximum load in tiebacks for piles 8 and 10 due to wider tributary load area.

6. Waler W460 x 74 spans approximately 4.8 m between piles 8 and 10. Provide design calculations for member.

W.P. 141-87-00

C. GEOTECHNICAL

1. Clarify whether compacted fill material can maintain vertical face for deadman trench.

2. Justify whether excavation slopes can be maintained at 1.5:1 and 1:1.

3. Clarify soil parameters used in shoring design calculations. Soil Report recommends $\phi = 26°$ ($Ka = 0.39$) for silty clay material (EL 150.5 - 138.5). Design calculations used $Ka = 0.35$.

4. Check bond stress used for design of soil anchors.

5. Clarify the location of deadman for development of full capacity.

6. Anchorage zone of tiebacks should start beyond the theoretical sliding surface of the soil. Check adequacy of anchorage length provided.

005114

Ontario

memorandum

MINISTRY OF TRANSPORTATION
Structural Section
1201 Wilson Avenue
Atrium Tower, 4th Floor
Downsview, Ontario, M3M 1J8
Telephone: 235-5515

DATE: October 26, 1992

TO: J. Cullen
Manager
Construction Office

RE: Contract 92 - 40
Hwy. 407 - Jersey Creek Culvert
C.P.R. Embankment Stability

A field meeting was held on Friday September 11, 1992, to observe the C.P. embankment at the Jersey Creek Culvert, at which you where present. Construction was to inform the contractor of our immediate concern for the embankment stability and to contact C.P. to inform them of our concern. Please confirm the action taken with C.P. and the name of their contact person.

R. A. Jeffries
Structural Supervisor
for:
V. F. Boehnke
Head, Structural Section

:rj

cc. B. Iyer
M. Devata

B.I.

(RETAIN THIS COPY FOR FOLLOW-UP)

7540-1021

SEND TO: R. TEMPLE
FENCO

FROM: BOB JEFFRIES

DEPT.: STR. SECT.

DATE: 92-10-26

SUBJECT: CONTRACT 92-40 - JERSEY CK. / C.P.R. DETOUR

- PLEASE FIND ATTACHED, THE CONTRACTOR'S SECOND SUBMISSION FOR TEMPORARY SHORING OF THE C.P.R. DETOUR EMBANKMENT, FOR APPROVAL
- ONE SET IS FORWARDED TO THE FOUNDATION SECTION FOR COMMENTS. DIRECT ANY INQUIRY TO THE SPECIFIC PERSON FOR EXPEDITIOUS APPROVAL OR COMMENTS.
- 4 COPIES OF DWGS SH1 & SH2 AND 2 SHEETS OF SHORING CALCS ARE ATTACH.
- AS YOU KNOW A REVIEW A.S.A.P. IS APPRECIATED

REPLY

BOB

c.c. B. IYER.

RECEIVED
OCT 26 1992
FOUNDATION DESIGN SECTION

REPLY FROM

REPLY DATE

TO WRITE: HANDWRITE OR TYPE. REMOVE AND RETAIN YELLOW COPY. FORWARD BALANCE OF SET.

TO REPLY: WRITE REPLY IN BOTTOM AREA. SNAP SET APART.

Ontario

▸ FOLD AT MARKS FOR USE IN #9 OR #10 WINDOW ENVELOPE ◂

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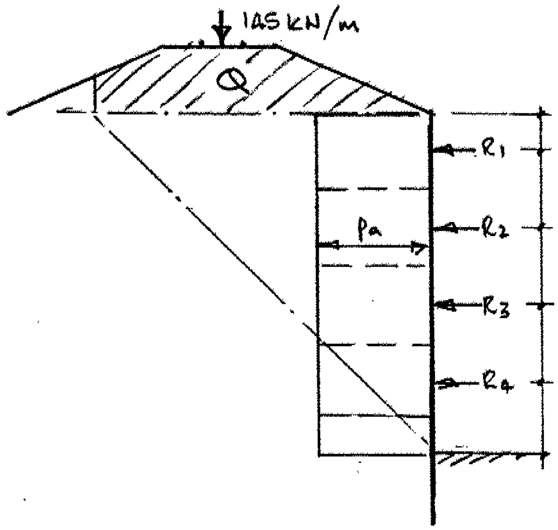
SHORING CALCULATIONS

JERSEY CREEK CULVERT – CPR DIVERSION

SOIL: SILT TILL FILL – COMPACTED TO >95% PROCTOR
BOREHOLES C2 & C3.
$\gamma = 20\ kN/m^3$, $K_A = 0.35$.
$p_a = 0.65\ K_a (\gamma H + q)$

q = RAILWAY LOADING = 145 kN/m + OVERBURDEN SOIL WT



PILE SPACING 2.4 m.

$R_1 = R_2 = R_3 = R_4 = 2.4 \times 3.0\ p_a$.

AT PILES 8+10. USE 3.25 m.
($R = 74 \times 3.25 \times 3 = 721\ kN$).

SEE SCHEDULE.

DEADMEN:

DEPTH TO ℄ DEADMAN = 6m SAY.

$\phi = 32.5°$ $K_A = 0.3$, $K_P = 3.3$

RESISTANCE: $(K_p - K_a)\dfrac{\gamma H}{F} = \dfrac{3.0 \times 20 \times 6}{1.5} = \underline{240\ kPa}$.

SEE SCHEDULE (P2)

SOIL ANCHORS:

CLAYEY SILT N=15, OR COMPACTED FILL
TAKE c = 100 kPa. WORKING STRESS $\dfrac{4.5 \times 100}{1.5}$ SAY = $\underline{30\ kPa}$.

SUBJECT TO TEST.

REGISTERED PROFESSIONAL ENGINEER
BRIAN ISHERWOOD
PROVINCE OF ONTARIO

ISHERWOOD ASSOCIATES
Geotechnical and Construction Consultants
3100 Ridgeway Dr., #3
Mississauga, Ontario
Canada
L5L 5M5
Fax: (416) 820-3492
Tel: (416) 820-3480

Title	Ref.	Date
M.O.T. CONTRACT 92-40 HIGHWAY 407. SHORING CALCULATIONS	92.26	92.10.23
	Scale	Drg.No. 1 of 2.

SECTION:		1 (pile 8)	2 (pile 5)	3 (pile 13)
H	m.	14.0	9.0	7.0
Q	kN/m	145 + 484	360	210
q	kPa	45	40	30.
pa	kPa	74	50	39
R	kN	532	360	281
	@ 15°	550	373	291
	USE:—	650	400	320
DEADMEN	TOTAL LOAD:	3,300	1,850	1,160.
AREA REQUIRED: m^2		13.75	7.71	4.83
	USE	6 x 2.5	5 x 2	4 x 2.
SOIL ANCHOR	LOAD	650	400	320
AREA REQUIRED: m^2		21.7	13.3	10.7
600 DIA.	LENGTH m	11.5	7.1	5.7
	PROVIDE m.	12.0	8.0	8.0
PILE	M. kN.m.	160	108	84
HP310x110	R.M.	305	305	305

✓

DEADMAN DESIGN:

VERT. REINF. 2.5m DEEP $M = \frac{240 \times 1.2^2}{2} = 173$ kN.m.

900 WIDE $A_s = \frac{173 \times 1000}{.75 \times .6 \times 165} = 2330 \text{ mm}^2$ 25M @ 200

2.0m x 750 WIDE $A_s = 1945 \text{ mm}^2$ 25 M @ 250

HORIZ BEAM. $\frac{3,300}{6} = 550$ kN/m $\frac{wl^2}{10}$, M = 220 kN.m

2/W200x59.

BRIAN ISHERWOOD — REGISTERED PROFESSIONAL ENGINEER — PROVINCE OF ONTARIO	**ISHERWOOD ASSOCIATES** Geotechnical and Construction Consultants	3100 Ridgeway Dr., #3 Mississauga, Ontario Canada L5L 5M5	Fax: (416) 820-3492 Tel: (416) 820-3480
	Title: M.O.T. CONTRACT 92-40 HIGHWAY 407 SHORING CALCULATIONS	Ref. 92.26 / Scale	Date 92.10.23 / Drg.No. 2 of 2

Minutes of Meeting
407 - Jersey Ck. Culvert - C.P. Embankment

Date: Friday Sept. 11, 1992 11 am

Present: R. Jeffries, J. Cullen, B. Kant

Purpose: To discuss the method of construction of the culvert and the resulting affect on the C.P. embankment that I had observed on Thursday Sept. 10 4 pm site visit.

Discussion:

Construction was informed that the culvert had been constructed by 'open cut' method rather than tunnelling as designed, in the area of the C.P. embankment. It was agreed that the contractor had removed considerable amount of existing earth in this area during his clearing operation. The amount of earth removed was not agreed upon with my opinion of the entire 'plateau' of over 6 m in depth having been removed was later observed during the afternoon field visit.

The stability of the C.P. embankment was discussed with our position that any removal of earth from this area would create the possibility of slope failure of the C.P. embankment.

The meeting was informed that we were attending a meeting at 11:30 am with the Foundation Office to discuss the associated problems and arrange a immediate field visit.

Also discussed was the previous 'proposal' from the contractor to trench this very same area to install the pipes without tunnelling which we had rejected as unsafe to the C. P. embankment.

R. A. Jeffries
Structural Supervisor
for:
V. F. Boehnke
Head, Structural Section

rj:

Minutes Of Meeting
407 - Jersey Ck. Culvert / C.P. Embankment

Date: Friday Sept. 11, 1992   11:30 am

Present: R. Jeffries, K. Pilgrim, V. Boehnke, M. Devata, B. Iyer

Purpose: To discuss what action our offices should recommend due to the construction activities carried out affecting the C. P. embankment and to arrange an immediate site visit to verify the slope configuration, condition and our concern based on my observations from a field visit on Thursday 10, 1992, 4 pm.

Discussion:

M. Devata discussed his concerns with the procedures that have or have not taken place, during construction, with submissions and approvals related to the culvert and C.P. embankment. The ongoing involvement of his office with this project and the concern with the implications with deviations from the design and recommendations that have been provided was expressed.

From my description of the field situation the Foundation Office restated its serious concern with the stability of the C.P. embankment because of the excavation at the C.P. embankment.

Agreement to meet with our office and construction immediately in the field to confirm my observations and recommend action to construction. The planned recommendation to construction would be to require the contractor to rectify the situation and provide a proposal for approval.

A field meeting was confirmed for 1:30 pm the same day.

R. A. Jeffries
Structural Supervisor
for:
V. F. Boehnke
Head, Structural Section

rj:

Site Visit - 407 Jersey Ck. Culvert/C.P. Embankment

Date: Thursday Sept. 10 , 1992 3:30 pm

Present: R. Jeffries

Purpose: After receiving a proposal for bedding and backfill to precast pipe installation from construction and a subsequent request for more details from the Foundation Office, I made a field visit to see the construction area.

Observations:

Approximately one half of the designed tunnel length portion of the culvert had been constructed in a trench (to the top of the pipe) as the original ground had previously been excavated, now producing a flat grade from the river to the C.P. embankment. It appears that the 'plateau' of approximately 8 m high had been removed and the interface with the C.P. embankment left to more or less parallel the C.P. tracks in a similar configuration to the C.P. embankment to the south, across the river valley.

The contractor was now cutting further into this new face of the embankment to install the last two or three sections of pipe, producing a almost vertical face about 5 m high. (This latest excavation was covered with a grey clay material packed against the face before the friday site visit).

There was about four slope failure cracks evident between the tracks and the excavation. The most severe had a drop of about .7 m . Water is leaking from a old wooden culvert (1' x 1') which had been uncovered during one of the excavation operations.

R. A. Jeffries
Structural Supervisor
for:
V. F. Boehnke
Head, Structural Section

rj:

Minutes of Site Meeting
407 - Jersey Ck. Culvert / C.P. Embankment

Date: Friday 11, 1992, 1:30 pm

Present: R. Jeffries, K. Pilgrim, V. Boehnke, B. Iyer, J. Cullen, B. Kant, A. Snider

Purpose: To observe the existing situation and provide construction with direction regarding the stability of the C.P. embankment in the area of the culvert construction.

Discussion and Observations:

It was agreed that the contractor had cut into the C. P. embankment during his various construction activities. About 8 m of earth had been removed alongside the projected C. P. embankment alignment in the area of the culvert. Some minor amount of fill had been placed against the almost vertical face excavated on Thursday for the pipe installation.

Surface failure cracks were evident between the tracks and the excavation (four or more, with one larger surface failure where the earth had dropped about .7 m).

It was agreed that construction would issue a letter to the contractor to inform him of the slope instability and requiring a proposal for immediate action to stabilize the C.P. embankment.

Construction would notify C.P.R. of our concern on the stability of the embankment due to construction activities not included in the designed contract work.

R. A. Jeffries
Structural Supervisor
for:
V. F. Boehnke
Head, Structural Section

rj:

7540-1021

SEND TO R. TEMPLE / B. PHELP
FENCO

FROM BOB JEFFRIES

DEPT. STR. SECT.

DATE 92-10-9

SUBJECT 407 - CONSTR. LIAISON - JERSEY CK. / C.P.R. SHORING PROPOSAL.

AS DISCUSSED ON ~~TUES. 6~~ WED. 7TH, THE CONTRACTOR'S PROPOSAL FOR TEMPORARY SHORING (DWG # C-414) WAS GIVEN TO B.PHELP AT A MTG. ON ~~W~~ THURS 8TH. BRIAN WAS INSTRUCTED TO REVIEW THE PROPOSAL FOR APPROVAL, BASED ON PROTECTION OF THE C.P.R. DETOUR FILL ONLY. HE WAS ADVISED THAT THE M.T.O. WAS NOT SATISFIED WITH THE OVERALL SCHEME, MAINLY THE LACK OF PROTECTION FOR THE EXISTING C.P.R. EMBANKMENT. AS THE CONTRACTOR IS PROCEEDING

REPLY

TO WORK IN THIS AREA, THE REVIEW IS REQUIRED A.S.A.P.

C.C. B. IYER

BOB

REPLY FROM

REPLY DATE

TO WRITE: HANDWRITE OR TYPE. REMOVE AND RETAIN YELLOW COPY. FORWARD BALANCE OF SET.

TO REPLY: WRITE REPLY IN BOTTOM AREA. SNAP SET APART.

Ontario

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OVERSIZE DRAWING

MEMORANDUM

GEOTECHNICAL SECTION, CENTRAL REGION, TELEPHONE: 235-5433

TO: Mr. T. Zander
Area Construction Engineer
Construction Office
2nd Floor, Atrium Tower
Central Region

DATE: 92-09-21

RECEIVED
SEP 2[illegible]
FOUNDATION DESIGN

RE: CONTRACT 92-40, HIGHWAY 407
HUMBER RIVER TO PINE VALLEY

A site meeting was held on Sept. 16/92 between Mr. Russ Middleton, Project Supervisor, Dr. Balu Iyer, Foundation Design and myself to review the need for placement of granular blanket on specified slopes.

After reviewing the field conditions, it was decided to:

1.0 Delete the granular blanket between Sta. 15+725 and 15+890 on the north slope from the bench downward.

2.0 From Sta. 15+890 eastward to Sta. 16+250, the need for granular blanket to be reviewed by Mr. R. Middleton. A granular blanket is to be placed within the area if seepage zones are encountered.

3.0 **Placement of Granular Blanket from Islington Ave. Westward**

3.1 North Outside Slope

Granular Blanket to be placed on the slope from the bench upward to top of the slope.

Granular blanket to be deleted on the slope from the bench downward to the toe of slope.

3.2 South Outside Slope

Mr. R. Middleton to review, during excavation of the slope, if and where the granular blanket is required.

L. Crowder
P.D.E.O.
for:
G. Cautillo
Head, Geotechnical Section

LC/GC/rb

c.c. B. Kant
R. Middleton
B. Iyer

Disk File: c:\contract\92-40hwy.407

M. DEVATA
B. IYER
FOUNDATIONS

(RETAIN THIS COPY FOR FOLLOW-UP)

RECEIVED
SEP 18 1992
FOUNDATION DESIGN SECTION
B.I

7540-1021

SEND TO R. MIDDLETON
CONTRACT 92-40

FROM BOB JEFFRIES
DEPT. STR. SECT
DATE 92-09-16

SUBJECT 407- JERSEY CK. CULVERT / C.P.R. EMBANKMENT

DURING A SITE VISIT ON WED. SEPT. 16/92 IT WAS OBSERVED THAT NO ADDITIONAL WORK HAS BEEN ACCOMPLISHED SINCE FRI. 11/92 REGARDING STABILIZING THE C.P.R. EMBANKMENT IN THE AREA OF THE JERSEY CK. CULVERT.

THE MINOR GRADING ACCOMPLISHED ON FRI. FROM THE ℄ OF CULVERT TO THE SOUTH HAS LITTLE IF ANY VALUE IN STABILIZING THE

REPLY

EXISTING EMBANKMENT. WE HAVE BEEN UNDER THE IMPRESSION THAT MATERIAL WAS BEING PLACED TO REPLACE THE MATERIAL REMOVED PREVIOUSLY. THAT IS NOT BEING DONE IN THE AREA OF CONCERN. PLEASE PROVIDE A CROSS-SECTION SQ. TO TRACKS TO INTERSECT WITH 2×4 LUMBER MARKING END OF PIPE, PLUS ONE SECT. 5M. EACH SIDE.

c. B. KANT
J. CULLEN.
BOB

REPLY FROM
REPLY DATE

NOTE
AS OF 3:30 P.M. THURS. 17, 1992 EARTH WAS BEING PLACED AND COMPACTED IN THE CULVERT/SLOPE AREA
BOB

CONTRACT 92-40

1992 09 11

Hwy 407 - Jersey Creek Culvert
Contract 92-40.

Called and discussed attached matter w. Bob Jeffries & V. Bodnute and arranged a meeting w. Monty Devatz at 11:30am.

Ministry of Transportation
Ontario

Action Memo

Time

Date 92, 09, 10

To R. JEFFRIES

From B. IYER, Foundation Section

- [ ] Phoned
- [ ] On Hold
- [ ] Please Call
- [ ] Returned Your Call
- [ ] Will Call Back
- [ ] Wishes Appointment

Telephone No

- [ ] Waiting in Person
- [ ] Was Here
- [ ] Will Return

Message Taken By

- [ ] Note and File
- [ ] For Your Signature
- [ ] Take Appropriate Action
- [ ] Per Discussion
- [ ] Please Answer
- [ ] Return With Comments
- [ ] Note and See Me
- [ ] Per Your Request
- [ ] Prepare Reply For My Signature
- [ ] Investigate and Report
- [ ] Note and Return to Me
- [ ] Returned With Thanks
- [ ] For Your Approval
- [x] Return With More Details
- [ ] For Your Information
- [ ] ______

Comments

The information given in the attached is not in sufficient detail. Please have the Contractor show the extent of trench excavation, location of 'shaft' etc. on plan/ Longitudinal profile. We will be able to review the construction scheme only after we have received all pertinent data.

B Iyer

ADM-G-45 81-05

- [ ] Over

Ontario

Action Memo

Time

Date Year 92 | Month 9 | Day 10

To BALU MURTY

From (Name and City) BOB JEFFRIES

I.C.N. No.	Area Code	Telephone No	Ext	Message Taken By
		5515		

- [ ] Phoned
- [ ] On Hold
- [ ] Please Call
- [x] Returned Your Call
- [ ] Will Call Back
- [ ] Wishes Appointment
- [ ] Waiting ~~in Person~~
- [x] Was Here
- [ ] Will Return

ASAP

☐ File	☐ Draft Reply For My Signature	☐ Provide More Details	☐ For Your Information
☐ Type Draft	☐ For Your Approval and Signature	☐ Keep Me Informed	☐ Per Discussion
☐ Type Final	☐ Circulate, Initial and Return	☐ Take Appropriate Action	☐ Per Your Request
☐ Make Copies ____	☐ Return With Comments	☐ Note and See Me	☐ Returned With Thanks
☐ Please Answer	☐ Investigate and Report	☐ Note and Return	☐

Comments

407 - JERSEY CK. CULVERT (TUNNEL) - PROPOSAL ON INSTALLING PIPE IN AREA WITH NO COVER - BEFORE TUNNELLING OPERATION. - BASED ON 'BEDDING STD'

7540-1037 (Rev. 6/87)

☐ Over

7540-1021

7268115

SEND TO: MTO
re:-92-40
ATTENTION: MR. R. MIDDLETON

FROM: GRAHAM BROS CONST^N DEPT. DAVE WELTZ DATE 9 SEPT/92

SUBJECT: JERSEY CR CULVERT.

KINDLY BE ADVISED THAT GRAHAM BROS PROPOSES THE FOLLOWING BEDDING & BACKFILL IN AREAS WITH LESS THAN 2 M COVER OVER THE PIPE.

BEDDING :- 300 MM UNDER PIPE & UP TO SPRING LINE = HC 6 STONE

B'FILL :- SPRING LINE OF PIPE & ABOVE = NATIVE FILL TO 95% COMPACTION

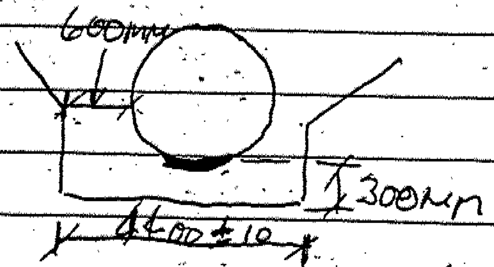
NB NATIVE B'FILL TO BE STONE FREE WITHIN 2M OF PIPE

WORK TO COMMENCE AT DOWNSTREAM END & PROCEED TO AN AREA BETWEEN ℄'S OF

PLY CPR MAINLINE & DETOUR AT SHAFT LOCATION.

- PIPE TO BE USED IS 100-D AS SHOWN ON DWG^S

- DETAILS OF TRENCH :-



2700 mm PIPE. WALL THICKNESS = 255mm

Regards
Dave

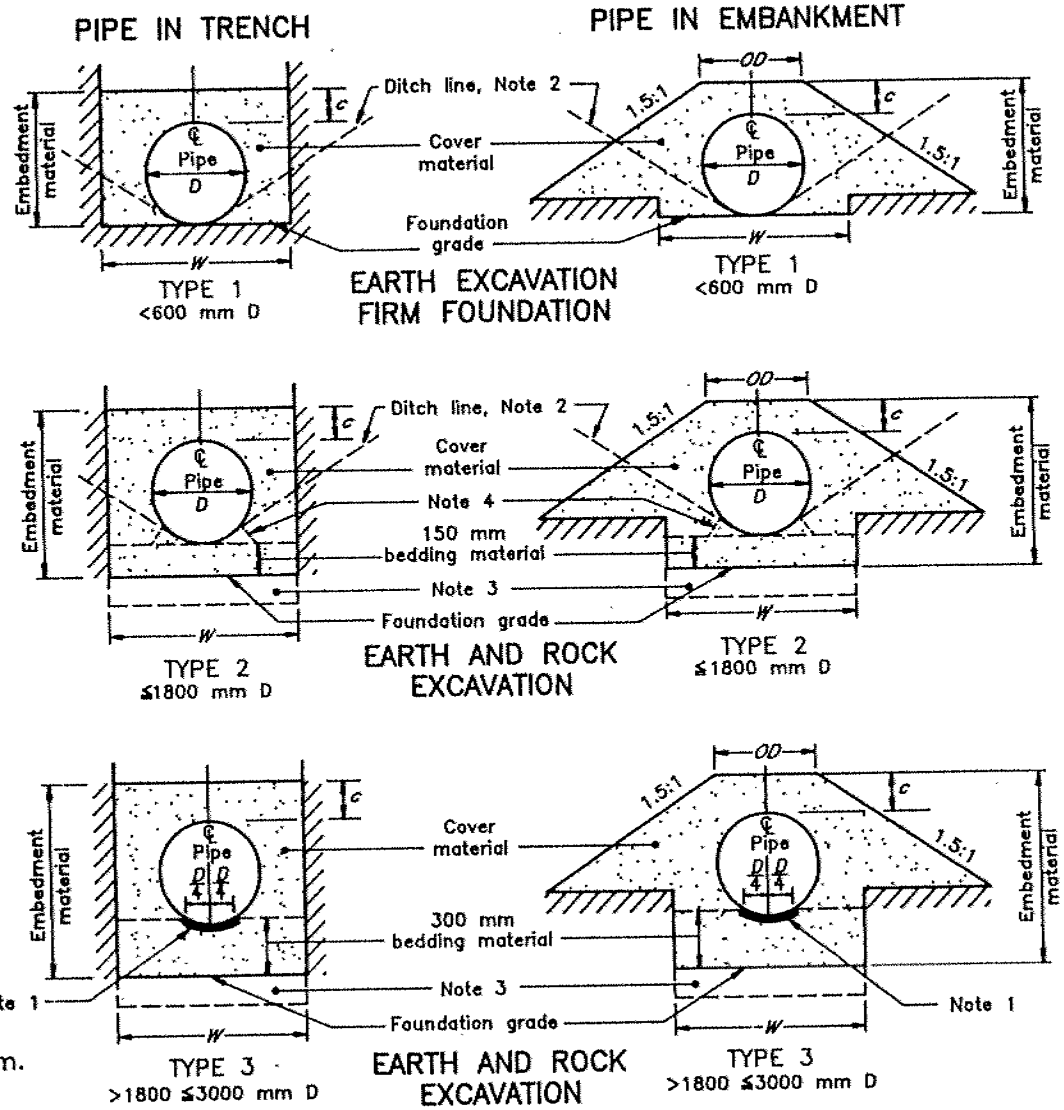
REPLY FROM R Middleton

REPLY DATE

TO WRITE: HANDWRITE OR TYPE. REM
YELLOW COPY. FORWARD

FOLD AT MARKS FOR USE IN # 9 OR # 10 WIN

SEE OPSD 801
802
803



NOTES:

1 For Type 3, pipe bedding to be shaped to receive the pipe for a width of D/2.

2 Installation in regular ditch cross-section is allowed at private entrances only.

3 Foundation when required will be specified in the contract.

4 Haunches must be backfilled and compacted prior to continued placing of embedment material.

A Protection against Heavy Construction Equipment according to OPSD-808.01.

B Backfill according to OPSD-803.04.

C All dimensions are in millimetres unless otherwise shown.

LEGEND:

OD – Outside diameter

D – Inside diameter

W – Minimum width of bedding:
D + 800 mm for $D \leq 600$ mm.
1.67 x D for $D > 600$ mm & < 1800 mm.
D + 1200 mm for $D \geq 1800$ mm.

c – Pipe diameter < 600 mm
c = 300 mm,
Pipe diameter ≥ 600 mm
$c = \frac{D}{4} + 300$ mm

ONTARIO PROVINCIAL STANDARD DRAWING	Date	1989 05 01	Rev	2
BEDDING FOR CIRCULAR STEEL PIPE STORM SEWERS AND CULVERTS	Date ______			
	OPSD – 802.01			

memorandum

Ontario

To: R. Middleton
Senior Project Supervisor
Contract 92-40

Date: 1992 09 18

From: Foundation Design Section
Room 315, Central Building

Subject: Excavation - CPR Embankment adjacent to Jersey Creek Culvert, Contract 92-40, Hwy 407, District 6, Toronto

---

The letter from Graham Construction Limited dated 1992 09 14 regarding the reference subject was received at our office today.

The letter implies that, following a meeting between Cullen, Kant and the Contractor at 3 pm on 1992 09 11, the Contractor "pushed" fill against the CPR embankment "immediately" to increase the stability of the embankment.

Even though the site was not visited by this section, based on information contained in the memo from R. Jeffries to R. Middleton dated 1992 09 16/17, we consider that the fill placed at the toe of the excavation may not be adequate to reduce our concern regarding instability of the CPR embankment in its excavated condition.

Please refer to our recommendations given in our report(s) on CPR Structure and CPR Detour, where we have addressed the safe design slopes for cuts and embankments in this area.

Please call us if we can be of assistance.

B Iyer

Balu Iyer, P. Eng.
Sr. Foundation Engineer

cc: V. Boehnke / R. Jeffries
T. Zander

B:\CONT9240.4

memorandum

Ontario

To:

R. Middleton
Senior Project Supervisor
Contract 92-40

Date: 1992 09 17

From: Foundation Design Section
Room 315, Central Building

Subject: Accuracy of Location of Driven piles
Contract 92-40,
District 6, Toronto

We received the information related to the abutment piles which are out of plumb by more than 2 % at Hwy 407 - Islington Avenue Structure. This information was forwarded to the Central Region Structural Section and the Structural Office for their information, review and recommendations. Such measurements should be carried out at other structure locations as well.

As discussed with you yesterday, the accuracy of location of the piles driven to-date at different structure locations should be checked and this information should be forwarded to this office with copies to the Central Region Structural Section and the Structural Office for review.

Please call us if we can be of assistance.

Balu Iyer, P. Eng.
Sr. Foundation Engineer

cc: V. Boehnke / R. Jeffries
K.G. Bassi / G. Al-Bazi

B:\CONT9240.3

RECEIVED
MAY 8 1992
FOUNDATION DESIGN SECTION

B.I. → Files

Tel. (416) 235-5515

Structural Section
1201 Wilson Avenue
Atrium Tower, 4th Floor
Downsview, Ontario
M3M 1J8

7 May 1992

Mr. R. Temple
Chief Bridge Engineer
Fenco Engineers Inc.

Dear Sir:

RE: Hwy. 407 Jersey Creek Culvert
Culvert Extension
Contract 92-40 W.P. 141-87-00
District 6, Toronto

In response to your letter of April 15, 1992, the following comments are provided after an internal meeting with our office and M. Devata and B. Iyer of the Foundation Design Section to review our previous direction given to you and your staff. Discussions were also held with Bob Kant of the Construction Office.

The original request to provide a permanent extension to the culvert, specifically was to extend the tunnel design section to the new total length required by grading concerns. Your letter stated that your office proposed that the extension be constructed by open cut and cover and the design was revised to incorporated this method of construction. I am unaware of any such proposal when or after the revision work was assigned.

In reply to the last paragraph in your letter, this concept decision <u>was</u> made with adequate consideration of the engineering feasibility and with previous consultation with the construction and foundation personnel mentioned above. We would have appreciated any concerns that you may have had for this concept being forwarded to our office before producing an alternate design which later had to be revised.

Contract drawings were marked up by us showing the proposed changes required to extend the tunnel design section and submitted to your office. Your office was asked to review the proposed revision from the design aspect and to make the changes to the contract package.

In order to best suit the contract staging and to provide the C.P.R. with one consistent design, the requested scheme of simply extending the tunnel section, was the preferred method of permanently extending the culvert.

Summarizing, construction's, foundation's and our own position on extending the tunnel design section is as follows:

- contract payment will be under the tunnelling item.

- method of construction for the extension portion will depend on the contractor - tunnelling in fill, cut and cover, and construct and fill.

- normal construction concerns and requirements for tunnelling will be present, both for the existing material and for the fill placed under this contract.

- proposals for tunnelling work will be submitted to the Construction Office for approval.

Yours truly,

R. A. Jeffries
Senior Structural Technician
for;
V. F. Boehnke
Head, Structural Section

rj
cc. M. Devata
B. Iyer
B. Kant

→ FAX: B. IYER, FDN
\# 5240

FENCO

RECEIVED
APR 23 1992
CENTRAL REGION
STRUCTURAL SECTION

VIA FAX

April 15, 1992

Fenco Engineers Inc.
Atria North Phase II
2235 Sheppard Avenue E.
Willowdale, Ontario
Canada M2J 5A6

Telephone: (416) 756-3400
Fax: (416) 756-2266

Mr. V. F. Boehnke
Head, Structural Section
Ministry of Transportation of Ontario
4th Floor, Atrium Tower
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Dear Sir:

SUBJECT: HWY 407 - C.P.R. MILEAGE 10.34
JERSEY CREEK CULVERT
W.P. 141-87-00, DISTRICT 6, TORONTO

Further to our telephone discussions on April 8, 1992 we should be pleased if you would have the M.T.O. Foundation Section review the feasibility of tunnelling the proposed additional length of culvert that is outside the limits of the existing railroad embankment.

We originally proposed that this length of culvert should be constructed by open cut and cover, and drawings and specifications were revised to incorporate this method of construction. Granular backfill limits were specified around the primary liner.

Subsequently, the Ministry requested us to revise the drawings and specifications to suit tunnel construction, since this speeded up the staged construction of the contract and allowed tunnelling to start from this end of the culvert.

..... Page 2

RECEIVED
APR 24 1992
FOUNDATION DESIGN SECTION

In retrospect, it would appear that this decision was made without adequate consideration of the engineering feasibility. We are concerned that tunnelling may not be feasible since the earth cover is minimal, and the recently placed ordinary material may not be sufficiently compacted or consolidated to avoid collapses, and/or distortion of the primary liner. Further, we believe tunnelling from this end would still be feasible even if the first part was constructed as cut and cover as the new embankment is placed.

Your follow up would be appreciated.

Yours very truly,
FENCO ENGINEERS INC.

RT Ralph for

R. Temple, P. Eng.
Chief Bridge Engineer

RT:jl

15780

memorandum

Ontario

To: G. Cautillo
Head, Geotechnical SECTION
2nd Floor, Atrium Tower

Date: 91/06/28

Att: K. Ganesh

Frm: Foundation Design Section
RM 315, Central Building

Re: Islington - Pinevalley/ Hwy 407 Detention Pond
WP 141-87-00
District 6, Toronto

Due to difficulties in gaining clearance to enter the property. The soils investigation for the retention pond and surrounding excavation cuts has been delayed, to resume shortly on July 8, 1991.

However we are providing the following slope stability recommendations at the location of the detention pond based on previous information available in the area.

Information was requested by June 18th from Giffels concerning final elevations for the invert of the detention pond, Hwy 407, ditches and any planned structures such as the transitway in the area. Subsequently a cross-section and plan was received June 20th. Details regarding the transitway was not shown on the cross-section. As this could affect the design specifications for the slopes on this side we tried to contact Giffels to clarify this issue. As of this memo we have had no response, thus we went ahead with the slope stability analysis based on the cross-section provided. (without the transitway)

Subsurface conditions found just north of the pond, along the centreline of Hwy 407 consisted of:

	Fi	c	G	Depth
Silt and Sand	30	0	20	4.5m
Clayey Silt t o Silty Clay	26	0	19	4m,*10.25m
Clayey Silt(TILL)	30	5	20	17m,*10.75m

*Two alternative thicknesses of the two lower materials were used due to varying stratigraphical profiles within this area.

Effective stress analysis was implemented using Bishops method on an in-house desktop computer using the Sarma program. The analysis was carried out employing static loading conditions with a circular slip surface. A static surcharge load was placed on Hwy 407 but no load effects due to the transitway was included.

The results of the analysis are summarized in the table below:

SLOPE GEOMETRIES

Location	Depth of cut	Recommended Geometry
South Side	15m	3.75H:1V
North Side	6m(north of Hwy407)	3H:1V
	9m(south of Hwy407)	3H:1V, with a 3m midheight bench

The above specifications will be confirmed by the upcoming soils investigation. If any discrepancies are found due to any unexpected subsoil conditions the above could be subject to change.

If you have any questions please do not hesitate to contact this office.

M. Michalek
Jr. Foundation Engineer
For:
Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

cc W. Lankinen (Planning and Design Section)
R. Jeffries (Structural Section)
W. Lachmaniuk (Giffels)

memorandum

March 4, 1991

To : Mr. F. Conforti
Utility Coordinator, Dist. 6, Toronto
Atrium Tower, 1st Floor

Attn.: Mr. H. Wendland

From : Foundation Design Section
Central Building, Room 315

Re : Proposed Ontario Hydro 500 kv Tower Line crossing Hwy. 407 and Pinevalley, W.P. 141-87-00 & W.P. 144-87-00
District 6, Toronto

I refer to your memorandum dated Feb.15, 1991, requesting our comments on slope stability for the above projects.

The drawings provided did not include a cross section showing the exact location and dimensions of the footings. Ontario Hydro's Plan No.0247 is an orthophoto showing Hwy. 407 and Pinevalley drive and is not adequate enough for our review.

Please provide us with a drawing that shows the required details of the footings.

If you have any questions, please contact us.

Dr. B. Iyer, P.Eng
Sr. Foundation Engineer

cc: W.R. Lankinen

7540-1478 (Rev. 10/89)

Re W.P. 1441-87-00

Temporary excavation slope – interface between natural soil and reinforced earth

Assume $\varphi' \simeq 24^\circ$ to 26°

max. permissible slope, say 2.25:1 to 2.05:1

W.P. 1441-87-00

memorandum

Ontario

To: G. Cautillo
Head, Geotechnical Section
Central Region
2nd Floor, Atrium Tower

Date: 1990 11 26

Attn: K. Ganesh

From: Foundation Design Section
Room 315, Central Building

Re: Slope Erosion Protection
Embankment across Humber Flood Plain
W.P. 141-87-00
District 6, Toronto

As per information received from you, the source of (suitable) fill which will be used in the construction of the embankment across the Humber flood plain is shown in blue in attached Figure 3. The gradation envelope of this material is shown on Figure 1. It is not clear whether sufficient material could be generated from the excavation between CPR tracks and Pine Valley Drive for the construction of the embankment across the Humber flood plain to at least 1 m above the design flood water level.

It is recommended that no "special" slope erosion provision would be required, if this material is used for the embankment construction across the Humber flood plain, to at least 1 m above the design flood water level.

It is understood from you that the material shown in orange colour is not suitable for use on this project (W.P. 141-87-00), and that this material would be stockpiled and used at another site.

We also understand that the CPR embankement fill material (shown in green colour in attached Figure 3) whose gradation is shown on Figure 2, would not be used at the Humber flood plain area.

Please call this office if you require additional information on the above subject.

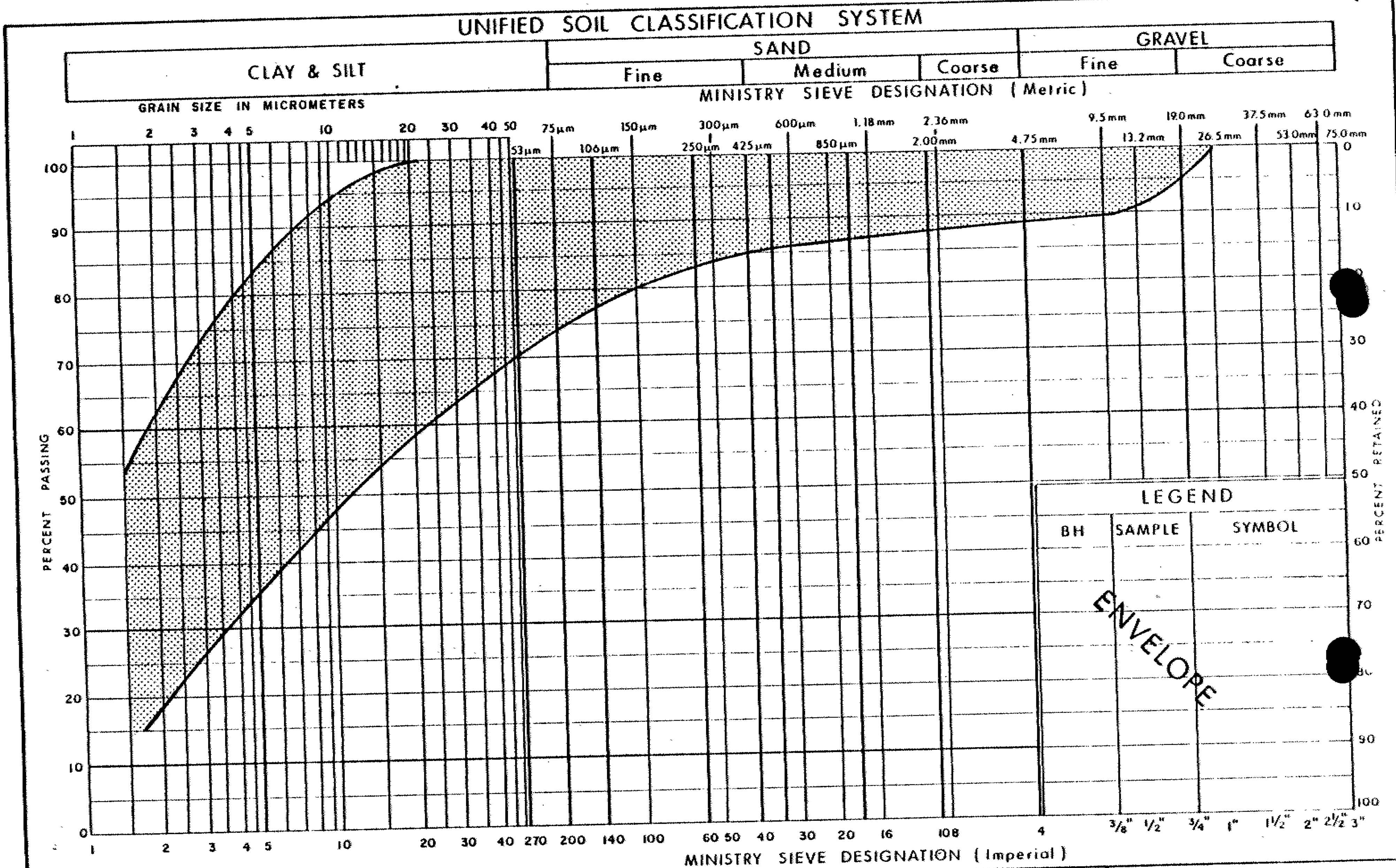
Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

BI/jb
BI\draft

cc: W. Lankinen
R. Jeffries

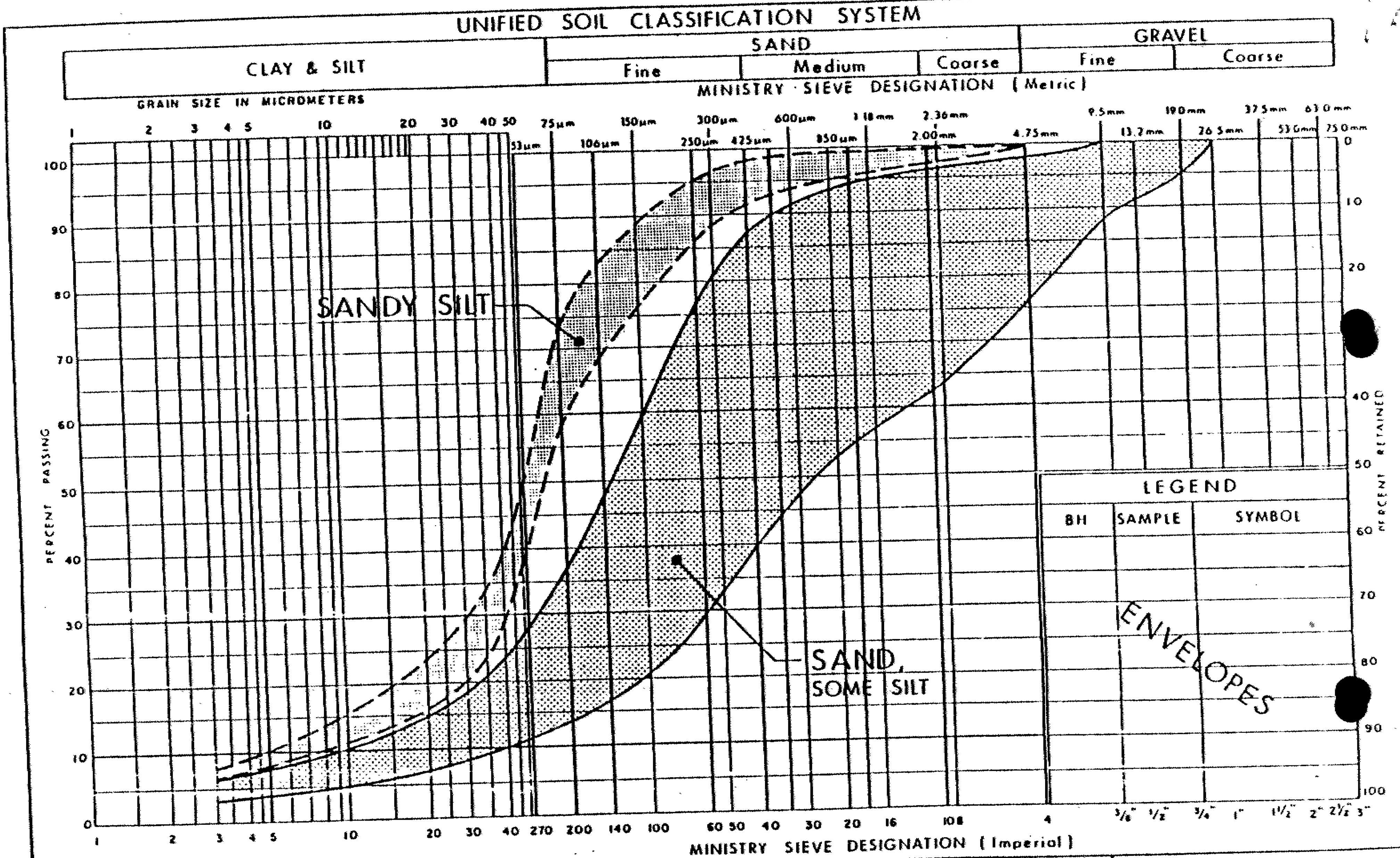
7540-1478 (Rev. 10/89)

78 12 M



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION SUITABLE FILL MATERIAL	FIG No 1 W P 141-87-00

78 12 M



Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION
CPR EMBANKMENT
(FILL MATERIAL)

FIG No 2

W P 141-87-00

[redacted]

7540-1021

SEND TO Balu Iyer
Senior Foundation Engineer
Foundation Section

Urgent

1

-5434

FROM Karam GANESH **DEPT.** Geotechnical Section **DATE** Nov 8/90

SUBJECT WP 141-87-00, Hwy 407, From Humber River E'ly to Pine Valley, Humber River Embankment Fill

Bill Lankmei from PED advise me that at Nov 08/90 Meeting with Giffels, that under the Giffels Job, the material for the Humber River embankment fill for station 15+030 to 15+060 and from 15+190 to 15+230, will be coming from Cuts east of the Humber River.

The attached profile shows material suitable to be placed as fill upon being excavated, and material, because of its high moisture content,

REPLY to be used on a later contract.

My interpretation of suitability of material between CPR Structure and Islington Avenue, is based upon BH information that were done for structures at these two location.

To assist you in determining the suitability of this material for the embankment fill,

REPLY FROM

REPLY DATE

7540-1021

SEND TO Balu Iyer.

(2)

FROM

DEPT.

DATE

SUBJECT WP 141-87-00, Hwy 407

Please refer to the following Foundation Reports
for:- WP 141-87-00D
WP 141-87-00C
WP 88-78-16
and Golder's Report 88-78-18
Also see Foundation Report 141-87-00A and
WP 88-78-20
for material between Islington and Pine Valley
to be used for embankment fill
East of Pine Valley, the only information

REPLY we have indicates that the material
is a firm till below the topsoil.
I am also attaching the above
mentioned reports for your convience.
Please return them when finish with same.
Thank You
Karan.

REPLY FROM

REPLY DATE

memorandum

Ontario

To: H.A. McNeely
Supv., Environmental Unit
5th Floor, Atrium Tower

Date: 1990 11 05

From: Foundation Design Section
Room 315, Central Building

Re: Possible Groundwater Lowering
Humber River to Pine Valley Drive
W.P. 141-87-00
District 6, Toronto

Further to your memo of 90 10 17, we have reviewed the data related to the subsurface soil strata and groundwater condition in the subject area. No soils and groundwater data are available for the area to the west of Islington Ave., since permission to enter this site is still pending. Thus, the comments given below are based on the data from Pine Valley to Islington Ave., where cuts to a maximum of about 10 m would be involved in connection with the Hwy. 407 construction.

The simplified subsurface soil conditions consist of 2 to 3 m thick surficial layer of sand underlain by relatively less previous strata consisting of silt, clayey silt to silty clay, clayey silt till etc. The average ground level is at elevation 160 m. Two water levels have been identified, one a perched water level at about elevation 158, within the upper sand layer and a lower groundwater level at elevation 141 to 142 m within the silty clay stratum.

A 10 m deep excavation, to about elevation 150 m, would not influence the lower groundwater level. From a review of the geometry of the cut in relation to the subsurface soil and groundwater data, it is considered that the influence of the cut on the perched water level would be negligible within about 30 m away from the cut; closer to the cut, the perched water level would be lowered slightly. It is considered that, with proper sprinkling operations, the perched water could be maintained at a level which would not cause any significant influence on the market gardening operations.

.../2

We trust that this memo is in sufficient details for your immediate needs. Please do not hesitate to contact us if you need additional input on this subject.

BIyer

Dr. Balu Iyer, P. Eng.
Sr. Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer

MD/BI/jb

cc: W. Lankinen, P & D

Planning and Design
Environmental Unit
Central Region
Atrium Tower, 5th Floor
235-5541

MINISTRY OF TRANSPORTATION

MEMORANDUM

RECEIVED
OCT 19 19[illegible]
FOUND'TIO'! DESIGN SECTION

TO: Murry Devata
Chief Foundation Engineer
Foundation Design Section

DATE: 90-10-17

RE: W.P. 141-87-00, HWY 407 HUMBER RIVER TO PINE VALLEY DRIVE, ADVANCED STRUCTURES

The area of Hwy 407 at Islington Avenue has been subject to a number of past concerns regarding the impact of construction on the groundwater table and the implications for the market garden operations there. A search through our files indicate that the opinions have varied from time to time on whether there would be impacts.

In July, 1984 your Section concluded that "there should not be any significant influence on the market gardening operations due to drawdown of the water table." Our understanding is that since that time additional bore holes have been dug. Does any of this additional information lead you to differ from your earlier assessment ? Would you please provide a written rational for the conclusions reached.

A hearing of necessity is being held in early December at which this issue may be raised. Bill Lankinen, the project manager, has requested the Environmental Unit to obtain from your Section a statement regarding the impact of construction on the groundwater and how it may impact the market garden operations between the CPR structure and Pine Valley Drive.

Your speedy reply to this request would be appreciated.

H. A. McNeely
Supervisor, Environmental Unit
Central Region

HAM/GJI/gji
grwater.407

cc. Bill Lankinen, P&D
Wally Lachmaniuk, Giffels

File
141-87-00

memorandum

Ontario

To: G. Cautillo
Head, Geotechnical Section
2nd Floor, Atrium Tower

Date: 1990 05 08

Atten: K. Ganesh

From: Foundation Design Section
Room 315, Central Building

Re: Hwy. 407 Deep Excavation Cuts Between
Pine Valley and CPR Tracks
W.P. 141-87-00
District 6, Toronto

In response to your memo dated 1990 04 27 regarding a request for;

1) recommendations for excavation cut slope geometries between Islington Ave. and the CPR along the proposed Hwy. 407 (Station 15 + 530 ± to 15 + 290 ±)

and,

2) an evaluation of applying 3H:IV slopes as an alternative slope geometry to previously submitted slope geometry recommendations between Islington Ave. and Pine Valley Drive,

the following comments are provided;

1) Excavation Cuts Between Islington and CPR

The difficulty in accessing this property for a required field investigation is well known and documented by all parties associated with the project. In the absence of site specific data, recommendations for cut geometries are based entirely on extrapolation of soils data and information in conjunction with the Hwy. 407 - Islington Ave. Underpass and the Hwy. 407 - CPR subway projects. Hence, the recommendations provided are preliminary and subject to modification once site specific data is procured.

It is suggested that the slope geometry recommended at the Islington Ave. structure, summarized in a memorandum dated 1990 03 12, be extended to Station 15 + 400. Similarly, it is suggested that the slope geometry recommended at the CPR Subway structure be extended to 15 + 350. The area between the defined limits will be a transition zone between the two different geometries.

Depths of cut in the area bounded by Islington Ave. and the CPR are in the order of 8 to 10 metres. The following table summarizes the proposed geometries within the area.

..../2

Table 1

Location	Depth of Cut (m)	Bench Width (m) 2.5H:IV Slopes	3H:IV* Slopes
Stn. 15 + 530 ± to 15 + 400 +	8	10	8
	9	11	9
	10	12	10
Stn. 15 + 290 ± to 15 + 350 ±	See CPR/Hwy 407	Table 6, page 18 in Foundation Report	
Stn. 15 + 330 ± to 15 + to 400 ±	Transition Zone		

*alternate 3H:IV slopes also applicable at Islington Ave. U'Pass (W.P. 88-78-18)

2) 3H:IV Slope Alternative - Excavation Cuts Between Pine Valley Drive and Islington Avenue

To facilitate any landscaping and associated maintenance on the excavation cut slopes, a request to investigate the application of 3H:IV slopes has been suggested for the area bounded by Islington Ave. and Pine Valley Dr. A subsequent slope stability analyses was implemented incorporating 3H:IV slopes and applying the identical procedures and subsurface conditions described in the original report. The recommended geometries are summarized below.

Table 2

Depth of Cut (m)	Recommended Geometry
> 4 - 8 inclusive	3H:IV
> 8 - 11 inclusive	3H:IV, 2 m Bench

As indicated in the memorandum dated 1990 03 12, the recommended geometry at the Islington Ave. structure shall be extended for a distance of 30 m beyond the east side of the structure.

If you have any queries regarding the above comments, please do not hesitate to contact this office.

T. Sangiuliano, P. Eng.
Foundation Engineer
for
Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

BI/TS/jb