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CONT 94-55

WP 114-87-00B DIST 4

HWY 403 STR SITE

Proposed Embankment at the
Hwy. 403 - Dunmark Lake West Crossing

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FOUNDATION INVESTIGATION REPORT

FOR

Proposed Embankment at the

Hwy 403 - Dunmark Lake West Crossing

WP 114-87-00B

District 4, Burlington

INTRODUCTION

This report summarizes the results of a foundation investigation conducted in conjunction with the embankment fills proposed at the Dunmark Lake West - Hwy 403 crossing. Embankment fills up to approximately seven (7) metres have been proposed at the specific site location. At the time of the investigation, the valley at the site location contained approximately up to 1.5 metres of standing water.

SITE DESCRIPTION AND GEOLOGY

The site is located within the limits of the valley containing the Dunmark Lake (referred to Dunmark Lake West in this report). The site is situated approximately 0.5km. east of Sunnyridge Rd, approximately 1 to 2 km. south of Jerseyville Rd. within the Regional Municipality of Hamilton-Wentworth. The valley is confined by earth ridges up to fourteen (14) metres in height to the west and up to approximately seven (7) metres in height to the east. The existing valley slopes are relatively flat and are estimated to range from 4H:1V to 5H:1V. These slopes reveal no signs of instability. The slopes are generally covered by both tall and short coniferous trees.

The existing valley at the proposed Hwy. 403 crossing is contained within the private ownership of the Heron Links Golf Course. The golf course is located immediately south of the proposed highway.

A natural drainage valley is located north of the Dunmark Lake West. It appears that drainage from Sunnyridge Rd. and the general catchment area occurs within this existing drainage valley.

The area north-east of the site consists of agricultural farmland. Residential homes are located north-west of the site location.

The waters of the Dunmark Lake are murky and unclear. The depth of the water was approximately 1 metre at the time of the investigation, although this water level has been known to fluctuate throughout the year. In fact, the water level has been completely drawn down during past summers as a result of irrigation conducted at the Heron Links golf course. Wild grass and weed are present within the northern limits of the lake.

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

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

The underlying bedrock at the site consists of hard dolomites of the Paleozoic era. At the site, the overburden has a thickness ranging from approximately thirty (30) metres within the valley to thirty-eight (38) metres at the crest of the valley.

INVESTIGATION PROCEDURE

GENERAL

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

Field Investigation

The fieldwork was carried out in two separate stages and hence within two different site mobilizations. In the initial stage, one borehole accompanied by two (2) dynamic cone penetration tests were advanced as part of a preliminary type investigation that occurred between 91 02 18-22. The borehole (BH1) was advanced to a depth of 38.3m, whilst the dynamic cone penetration tests were advanced to depths of 6.1m and 15.5m. The borehole and dynamic cone penetration test were advanced on land west of the existing Dunmark Lake West employing a track mounted CME 75 equivalent drilling unit.

A more detailed investigation was conducted between 92 07 09 and 92 07 22 that included a total of five (5) additional boreholes. The boreholes were advanced to depths ranging from 4.8m to

31.9m. Two (2) of the boreholes were advanced on land using conventional track mounted drilling units equivalent to Central Mining Equipment (CME) 55's. Both boreholes were advanced to auger refusal encountered at the probable bedrock surface.

Three (3) boreholes were advanced within the waters of the Dunmark Lake using a raft and a more portable, lighter diamond drill unit. The diamond drill used was a skid mounted Boyles Bros. No. 1 unit that had a weight of approximately 900kg.

Conventional hollow stem augering techniques were used to advance the boreholes on land. Offshore, conventional diamond drilling techniques were used.

Both disturbed and undisturbed samples were retrieved in the overburden both on land and offshore. Subsoil samples were retrieved at both 0.7 metre and 1.5 metre intervals on land. Offshore, surface samples were taken and then sampling proceeded at both 0.7 metre and 1.5 metre intervals. Disturbed subsoil samples were retrieved using a 50mm diameter split spoon sampler driven in accordance with the Standard Penetration Test (SPT - ASTM D1586). Relatively undisturbed samples were also retrieved within the weaker cohesive materials using a 57mm diameter thin wall sampler. The thin wall sampler was pushed manually into the soil offshore and hydraulically on land in accordance with procedures outlined in ASTM D1587. Wash samples were also retrieved at the boreholes advanced offshore.

All subsoil samples were identified in the field and then properly sealed to preserve natural moisture contents in the soil. Disturbed samples were placed in sealed plastic containers and thin wall samples were capped and waxed. The samples were then transported to the laboratory where additional visual classifications were carried out and pertinent laboratory tests were conducted as described in the next section below.

In situ vane tests were also carried out to determine the undrained shear strength at selected intervals between the subsoil sample retrieval. The test was carried out in accordance with ASTM D2573 employing the standard MTO 'N' vane. Remoulded shear strengths were also obtained allowing the determination of soil sensitivity.

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

The survey related to the location and elevation of the individual boreholes was provided by Central Region Surveys and Plans. A boat was required to enable the borehole layout offshore. Long steel rods were used to stake the boreholes in the waters of Dunmark Lake.

Laboratory Analyses

All subsoil samples were carefully visually examined in the laboratory in accordance with the procedures outlined in the Visual Method described in Chapter 2 of the MTO Soil Classification Manual. The behaviour, gradation and other pertinent physical and mechanical properties of the soil were determined by conducting the appropriate laboratory tests on representative samples. These tests are tabulated in Table 1 below:

Table 1 - Physical/Mechanical Property Testing

Physical Properties	Mechanical Properties
1) Atterberg Limit Tests	1) Consolidation Test
2) Particle Size Analysis	
3) Natural Moisture Contents	
4) Bulk Unit Weights	

Sample preparation and testing were conducted in accordance with the MTO Laboratory Testing Manual.

Laboratory test results have been summarized below in the subsequent section of this report entitled "Subsurface Conditions" and are illustrated on the corresponding boreholes and figures included in the Appendix to this report.

SUBSURFACE CONDITIONS

GENERAL

Subsurface conditions across the site are uniform and generally consists of extensive stratifications of silt and clayey silt to silty clay. The surficial deposit which underlies the standing waters of Dunmark Lake (up to 1.5 metres) consists of a cohesive clayey silt to silty clay with random layers of silt. The thickness of this deposit ranges from 2.2 metres to 9.1 metres. The stratum generally has a very stiff consistency.

The surficial cohesive clayey silt deposit is underlain by a plastic silt that has a loose to compact denseness and contains random interbedded seams/layers of stiff to very stiff clayey silt to silty clay. The thickness of this stratum ranges from 10.5m to 15.7m.

The plastic silt stratum is in turn underlain by a cohesive clayey silt deposit which has a thickness ranging from 6.1m to 8.7m. This deposit also contains random layers of plastic silt. The cohesive material has a stiff to very stiff consistency.

A lower silt deposit with interbedded layers of clayey silt to silty clay underlies the lower cohesive clayey silt. This deposit has a thickness of 4.8 to 6.1 metres and overlies the probable bedrock surface as determined by auger refusal.

A plan of the site illustrating the locations and elevations of the boreholes is shown on Dwg. No. 1148700B-A in the Appendix. A subsoil stratigraphical profile illustrating the subsurface conditions at the site is also provided. The boundaries between the various soil types, in situ and laboratory test results as wall as groundwater levels established at the time of investigation are shown on the stratigraphical profile and also on the individual Record of Boreholes sheets in the Appendix.

Water

Approximately up to 1.5m of standing water was present in the Dunmark Lake West at the time of the investigation. The water was murky and calm at the time of the investigation.

Clayey Silt to Silty Clay with random layers of Silt

The surficial stratum at the site consists of a cohesive clayey silt to silty clay that extends for a thickness ranging from 2.2 metres to 9.1 metres below the ground surface. Within the valley, the thickness of the deposit is within 4.6 metres, but at the crest of the slope, the deposit thickens to 9.1 metres. The deposit also contains traces of organics within the surficial three (3) metres and also contains random interbedded layers of silt of thickness ranging up to 100mm.

The colour of the deposit is primarily brown to mottled grey-brown but changes to grey at a depth as shallow as 3.0 metres at some locations. This varying colour is indicative of different depths of oxidation at the site.

A grain size distribution envelope produced by mechanical sieve and hydrometer analysis for this material is shown in Figure 1 in the Appendix. The envelope clearly illustrates that the stratum is composed of grain sizes smaller than 75 micrometres. The grain size distribution envelope illustrates silt percentages ranging from 35% to 62% and clay percentages ranging from 34% to 65%. In view of the fact that more than 50% of the material is finer than 75 micrometres, the soil is categorized according to its behaviour in accordance with the MTO Soil Classification Manual. Atterberg Limit Tests were hence conducted to define the behaviour and plasticity of the soil as discussed below. Atterberg Limit Tests were carried out on the fine grained soil and the results are plotted on Figure 2 in the Appendix and summarized on Table 2 below. Natural Moisture Contents and the Bulk Unit Weight of the soil have also been included in the table.

Table 2 - Atterberg Limit Test Results - Clayey Silt

	Range	# of Tests
Natural Moisture Content (w%)	29 - 37	7
Liquid Limit (w_L%)	34 - 49	7
Plastic Limit (w_P%)	18 - 26	7
Plasticity Index (Ip%)	13 - 27	7
Bulk Unit Weight (kN/m^3)	18.4 - 18.7	1

The test results clearly reveal that the soil has a plasticity that ranges from low to intermediate and hence can be categorized as a clayey silt (CL) to silty clay (CI). Natural moisture contents are generally within the plastic and liquid limits of the soil and hence the soil is in a plastic state.

The consistency and undrained shear strength of the soil were determined by conducting in situ vane tests. In general, the material exhibited sufficient undrained shear strengths that inhibited the torquing of the vane. Hence, undrained shear strengths are generally in excess of 120 kPa.

The 'N' values as determined by the Standard Penetration Test ranged from 2 blows/0.3m to 30 blows/0.3m. Within the lake, the 'N' values were lower and generally less than 10 blows/0.3m, for the surficial metre and a half or so. The larger 'N' values, (values exceeding 10 blows/0.3m) were retrieved offshore.

Silt with Interbedded seams/layers of Clayey Silt to Silty Clay

A cohesionless silt of quick dilatancy interbedded with random layers of cohesive clayey silt to silty clay exists below the clayey silt to silty clay surficial deposit and extends to depths ranging from 15.0m to 24.8m below the ground or water surface. The thickness of this stratum ranges from 10.5m to 15.7m and the cohesive interbedded seams or layers are approximately 25mm to 150mm in thickness. The cohesive interbedded layers are distinct and easily recognized and determined by visual index property identification tests. The layers have a darker grey colour, low plasticity, medium toughness, stickiness, shine and medium to high dry strength.

Grain size distribution envelopes as determined by mechanical sieve and hydrometer analysis for both the cohesionless host silt and cohesive layers of clayey silt to silty clay are shown on Figure 3 in the Appendix. The envelopes illustrate silt percentages ranging up to 92% with traces of clay for the silt material and for the cohesive layers, silt percentages range from 67% to 86% and the clay fraction ranges from 14% to 42%. In accordance with the MTO Soil Classification system, materials with gradations of this nature are categorized by its behaviour and hence Atterberg Limit Tests were conducted to evaluate the plasticity of the soil. The results of these tests are illustrated on Figure 4 and summarized in Table 3 below. Natural Moisture Contents are also included in the Table below.

Table 3 - Atterberg Limit Test Results
Silt with interbedded seams/layers of
Clayey Silt to Silty Clay

a) Silt

	Range	# of Tests
Natural Moisture Content (w%)	24 - 33	6
Liquid Limit (w_L%)	18 - 23	6
Plastic Limit (w_P%)	16 - 19	6
Plasticity Index (Ip%)	1 - 4	6

b) Clayey Silt to Silty Clay

	Range	# of Tests
Natural Moisture Content (w%)	28 - 35	3
Liquid Limit (w_L%)	25 - 37	3
Plastic Limit (w_P%)	17 - 18	3
Plasticity Index (Ip%)	7 - 19	3

The test results reveal that the main component of the deposit behaves as a plastic silt (ML). The interbedded cohesive layers have a low to intermediate plasticity (CL to CI).

In situ vane tests conducted within this deposit revealed undrained shear strength values ranging from 70 kPa to in excess of 120 kPa. It can therefore be concluded that the cohesive layers have a stiff to very stiff consistency, although the larger undrained shear strength values may be attributable to the presence of the silt material.

The sensitivity of the cohesive soil also determined by the vane test ranged from 2 to 5 indicating a low to moderately sensitive material.

The 'N' values as derived from the Standard Penetration Test range from 3 blows/0.3m to 16 blows/0.3m. Based on these 'N' values it can be stated that the deposit has a loose to compact denseness.

Clayey Silt with Random Layers of Silt

Underlying the plastic silt with interbedded seams/layers of clayey silt to silty clay, a second cohesive deposit consisting of clayey silt exists. This deposit also contains random layers of plastic silt ranging in thickness from 50mm to 150mm. The thickness of the entire stratum ranges from 6.1m to 8.7m.

A grain size distribution envelope illustrating the gradation of the cohesive material of this deposit and the interbedded silt layers is shown on Figure 5 in the Appendix. The envelope clearly illustrates that the material is fine grained with grain sizes less than 75 micrometres. Typically, clay percentages range from 22% to 45% and silt percentages range from 55% to 78%, within the cohesive material.

In accordance with the MTO Soil Classification system, a deposit with gradations of this nature is categorized by its behaviour and hence Atterberg Limit Tests were conducted to evaluate the plasticity of the soil. The results of these tests are illustrated on Figure 6 and summarized in Table 4 below. Natural Moisture Contents are also included in the Table below.

Table 4 - Clayey Silt with random layers of Silt

	Range	# of Tests
Natural Moisture Content (w%)	25 - 34	8
Liquid Limit (w_L%)	25 - 34	8
Plastic Limit (w_P%)	16 - 19	8
Plasticity Index (Ip%)	6 - 17	8

The test results reveal that the cohesive soil has a low plasticity and hence can be classified as a clayey silt (CL). Natural moisture contents are generally similar to the liquid limit of the soil.

The consistency and undrained shear strength of the soil was determined by conducting in situ vane tests and interpretation of SPT 'N' values. Undrained shear strengths ranged from 50 kPa to 100 kPa indicating a stiff to very stiff consistency. The 'N' values as determined by the SPT ranged from 6 blows/0.3m to 20 blows/0.3m confirming the stiff to very stiff consistency.

The compressibility characteristics of the clayey silt stratum was determined by conducting a one dimensional consolidation test on a representative sample of the material. The results of the test is shown graphically on Figure 7 in the Appendix. The consolidation curve is plotted on semi-logarithmic paper with the void ratio (e) plotted against the applied load (log p). This form of plotting the load-deformation properties of the soil has the advantage of enabling the determination of the preconsolidation pressure (p_c) which is defined as the maximum pressure that the soil has experienced in its stress history. Considerable consolidation settlements can occur once the threshold preconsolidation pressure is exceeded.

The consolidation curve reveals a preconsolidation pressure of 460 kPa. The effective overburden pressure of the sample tested is approximately 180 kPa. Therefore, the soil has been preconsolidated in the past to an effective pressure approximately 280 kPa in excess of the existing effective overburden pressure. The compression index of the material (C_c) is of small magnitude and equivalent to 0.4.

Silt with interbedded layers of Clayey Silt to Silty Clay

A second cohesionless deposit of plastic silt exists beneath the lower cohesive deposit of clayey silt. This deposit has a thickness ranging from 4.8m to 6.1m. Once again, stratification is present within the deposit and random layers of cohesive clayey silt to silty clay ranging in thickness from 25mm to 100mm are present within the deposit. The host material is a plastic inorganic silt that exhibits a quick dilatancy, no dry strength and no toughness, which is characteristic of this material. An Atterberg Limit test carried out on a representative sample of this silt material (BH2, SS14) confirms that the material behaves as a plastic silt (w_L = 23%, I_P = 3%). The cohesive interbedded layers are distinct and easily recognized and determined by visual index property identification tests. The layers have a darker grey colour, low plasticity, medium toughness, stickiness, shine and medium to high dry strength.

Standard Penetration Tests conducted within this deposit reveal 'N' values ranging from 6 blows/0.3m to 18 blows/0.3m indicating a loose to compact state of denseness.

Probable Bedrock

The bedrock was inferred at three borehole location (BH's 1,2 and 3) from auger and tricone refusal. The refusal was encountered at a uniform elevation of 181.7m to 182.4m across the site. Based on data retrieved at neighbouring sites, the bedrock surface elevation appears to be uniform from the proposed Sunnyridge Rd./Hwy. 403 structure to the Big Creek/Hwy. 403 structure and it is suspected that the bedrock is "vuggy" dolostone of the Amabel formation.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water levels in the open boreholes and monitoring the lake level throughout the duration of the field investigation. The lake level at the time of the investigation was approximately elevation 211.5m. On land the water level varied from elevation 217.7 (measured on 91 02 23) or approximately 3 metres below the ground surface at the crest of the slope to Elevation 210.7m to 211.2m on shore immediately adjacent to the Dunmark Lake West.

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

DISCUSSION AND RECOMMENDATIONS

In conjunction with the proposed Hwy. 403 between Ancaster and Brantford, it is proposed to construct embankment fills that will involve the placement of fill materials within an existing valley that houses the Dunmark Lake West. The water level within the lake was approximately up to 1.5 metres in height at the time of the investigation. Dwg. 1148700B-A in the Appendix illustrates the proposed embankment fill area.

The lake bed elevation is relatively flat and approximately at an elevation of 210 metres. Beyond the lake, the ground surface elevation increases and slopes to an elevation of approximately 224 metres and 219 metres at the western and eastern crest of the slope respectively. The proposed Hwy. 403 profile grade is approximately 216.5m to 217m. As a result, approximately seven (7) metres of fill will be placed above the lake bed. The thickness of fill diminishes to zero on either side of the valley and excavation cuts are proposed beyond the fill areas.

Embankment fills as proposed at the site must be designed in consideration of regional storm events. At the site, the regional storm water levels as tabulated in Table 5 have been provided.

Table 5 - Regional Storm Water Levels

Flood Level Period (Year)	Elevation (m.)
50	212.1
100	214.6

Recommendation for the design and construction of the proposed embankment fills are contained within the scope of this report.

EMBANKMENT FILLS

General

As mentioned previously, the site is located within a depression area inundated with water and up to approximately seven (7) metres of embankment fill will be required to achieve the proposed Hwy. 403 grade at the site. The design of the embankments such as those proposed at the site must address the following geotechnical considerations:

1) Stability
2) Settlement

The selection of material and the method of construction must also satisfy environmental protocol.

A further important consideration at this site is safeguarding the embankment from the consequences of regional storm water levels. As mentioned earlier, fifty (50) and one hundred (100) year storm water levels will reach elevations of 212.1m and 214.6m respectively. Therefore, embankment material, geometry and drainage must be appropriately designed to prevent loss of material and subsequent imminent embankment failure.

Stability

Global

The critical condition examined in the evaluation of the global stability of embankment fills as proposed at the site location is the short term (undrained) condition and consequently a total stress analysis was conducted. In all cases, stability computations were carried out using an in-house MTOslope application software package which is based on Sarma's method of limiting equilibrium. The formulation of Sarma' method is described in a paper entitled "Stability Analysis of Embankments and Slopes", Sarma, S.K. (1973), Geotechnique 23, No. 3, pp 423-433.

The process of stability analyses involves the selection of pertinent shear strength parameters and physical soil properties such as unit weight, inputting the subsurface and groundwater conditions and then designing a surface geometry that produces an acceptable factor of safety of 1.3 using the MTOslope program.

Figure 8 in the Appendix illustrates the subsurface conditions and relevant subsoil parameters used in the stability analyses. Circular surfaces were evaluated and a critical slip surface was searched. The results of the analyses reveals that no **deep seated** slope instabilities are anticipated for the proposed embankment fills constructed at 1.5H:1V for the rock fill and at 2H:1V for the earth fill. Figure 9 illustrates a 2m thickness of either crushed stone, Granular 'A', or rockfill supporting the earthfill. As discussed later (see Embankment Construction/Material), different options are available in consideration of the standing water level. The options vary in cost and in technical feasibility.

Both a granular fill material and a cohesive clay material were selected in the stability analyses and the final results are similar. The advantage of the granular material however, is that pore pressures which can develop from the rise in the water level as is expected during the regional storm will dissipate quickly and hence preserve the stability of the embankment.

Internal

As discussed above, there are no deep seated failures anticipated as a result of the applied embankment loading. The following guidelines, however, shall be adhered to in designing the embankment to preserve the internal stability and to avoid surficial slope failures.

1 - Earth Fills

1.1 Earth fills up to eight (8) metres in height shall be constructed at 2H:1V slopes or flatter.

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

2 - Rock Fills

2.1 Rock fills up to ten (10) metres in height shall be constructed at 1.5H:1V slopes or flatter.

At the site, if earth fill is used for the full embankment, then guideline 1.1 above applies and hence the slopes shall be constructed at 2H:1V. Earth slopes constructed at 2H:1V will therefore satisfy both internal and global stability considerations.

The internal stability of the embankment fill must be ensured during the regional storm period. Consequently, soil migration and piping within the embankment fill must be prevented. Based on a headwater and tailwater head difference of 0.75m as given in previous correspondence (see internal memo - P. Jankowski - 92 06 19), no major seepage or piping problems are anticipated. However, it is recommended that the base material of the embankment be protected from the external water with a minimum 1 metre thick rip-rap or gabion stone material as outlined in OPSS 1004.05.06. It is further recommended that filter materials be placed between the rock protection and the base embankment material. Specifications for the gradation and thickness of the filter material are dependent on the gradation of the base embankment material. This office can provide recommendations for the filter material once the gradation of the base material is known. The rock protection and filter material shall extend to a minimum 0.3m above the high water level (100 year flood level).

Consideration can also be given to employing a cohesive clay material as the base core embankment material or as an impervious core material. An impervious core shall extend a minimum 0.3 m above the HWL. Suitable filters and rock protection are recommended for the clay material as discussed above.

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

Native slopes exist at the western and eastern limits of the site. Longitudinal embankment fill slopes shall be benched into the existing slopes as per OPSD 207.01.

Embankment Construction/Material

General

In consideration of the standing water level in the Dunmark Lake, various options as discussed below are available in the selection of the embankment material. Three (3) options are provided, as illustrated on Figure 9 and as discussed below. Each option shall be evaluated based on technical merit and cost. The most economical and technically feasible option shall be adopted.

Each option involves the selection of a material to be placed within the prevailing standing water. In all cases, the material shall be placed a minimum 0.5 metres above the water level. Granular 'A' material has been recommended as a filter/transition material. This office should be contacted to verify the suitability of the Granular 'A' filter material once the composition of the base embankment material is known. The suitability of the Granular 'A' material as a filter is a function of the host material it will be filtering.

Embankment material and construction shall conform to OPSS 212 and OPSS 206 series respectively. The embankment earth material above the water level shall be compacted as outlined in OPSS 501 series. Embankment material can be end dumped into the existing lake below the prevailing water level.

Option 1 - Granular 'A' Fill

This option involves end dumping a Granular 'A' material into the lake. The advantage of a Granular 'A' material is that it provides a suitable filter material and hence will prevent soil migration from the overlying fill material. However, some segregation of the Granular 'A' material can be expected which may lead to a loss of fines and some subsidence. However, in view of the cohesionless nature of the material, this settlement should occur almost instantaneously and hence should be realized during or shortly after construction.

The environmental consideration of the soil migration and suspended fines in the water must also be addressed. Silt curtains may be required to mitigate and control transmission of fines in the water.

Option 2 - Open Graded Crushed Rock

Open graded 19.0mm crushed rock as specified in OPSS 1004.05.07 is an alternative material that can be placed. The advantage of this material is that no segregation will occur when end dumped into the lake. However, overlying earth fill material must be protected against migration into the voids of the crushed rock. This can be achieved by the placement of a geotextile on the crushed rock surface overlain by a 1 metre thick Granular 'A' material. The geotextile shall comply with the material specifications described in OPSS 1860. The material shall be a non woven class II geotextile with a filtration opening size equivalent to 50 micrometres.

A NSSP shall be included in the contract documents that specifies the method of placement of the geotextile.

Option 3 - Rock Fill

A more elaborate scheme involves the placement of rock fill. This scheme is probably the most costly of the three options because rock fill may not be readily available in the general area and because a geotextile/natural soil filter is recommended above the rock fill.

The rock fill shall be placed such that the voids on the top of the rock embankment are chinked with the smaller rock fragments and spalls as indicated in OPSS 206.07.08. This is usually achieved by placing the coarser particles firstly and then blading the smaller rock fill sizes over the coarser materials. A geotextile overlain by a Granular 'A' filter transition material of one (1) metre thickness shall then be placed on the rock fill surface. The geotextile shall be of a material as described in Option 2. A NSSP shall be included in the contract documents that specifies the method of placement of the geotextile.

Embankment Settlement

The overburden materials at the site are weaker in nature and hence will experience settlement as a result of the applied loadings. In view of the preconsolidation nature of the cohesive clayey silt to silty clay layers and the predominant cohesionless silt deposits and silt interlayers it is expected that approximately 75mm of settlement will be realized as a result of the compression of the native soil. This settlement is expected to be elastic in nature and hence should be realized during or shortly after the construction period.

Settlements within the embankment fill material are also anticipated as the result of internal stresses induced by the self weight of the material. It is anticipated that approximately 50mm to 75mm of settlement will occur within a seven (7) metre earth fill embankment including the effects of particle segregation and migration beneath the water table. Approximately 20 to 25mm of settlement will be realized for a two (2) metre rock fill thickness due to particle breakage caused by contact forces and particle reorientation within the rock fill. In general, rock fill embankments have shown settlement equivalent to approximately 1% times the embankment height. Therefore, for a seven (7) metre embankment consisting of a combination earth and rock fill approximately 50mm to 75mm of settlement can also be realized.

Settlements within the earth fill should occur almost instantaneously and hence should occur during or immediately following construction for a granular material. Settlements of cohesive fill embankments will be more time dependent and anticipated to be realized within a three (3) month time period following placement.

Settlements within the rock fill material will be more time dependent. For rock fill thicknesses up to 2 metres, the magnitude of settlement is not significant and any post construction settlement can be easily remedied using conventional post construction maintenance.

It is recommended that in view of the cumulative embankment settlements, that the final surface paving be delayed as far as scheduling permits in order to minimize post construction maintenance. Most of the settlements should be realized within a three (3) month period following construction.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer, P. Martin and L. Dametto, Student Engineers utilizing equipment owned and operated by Atcost Soil Drilling.

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

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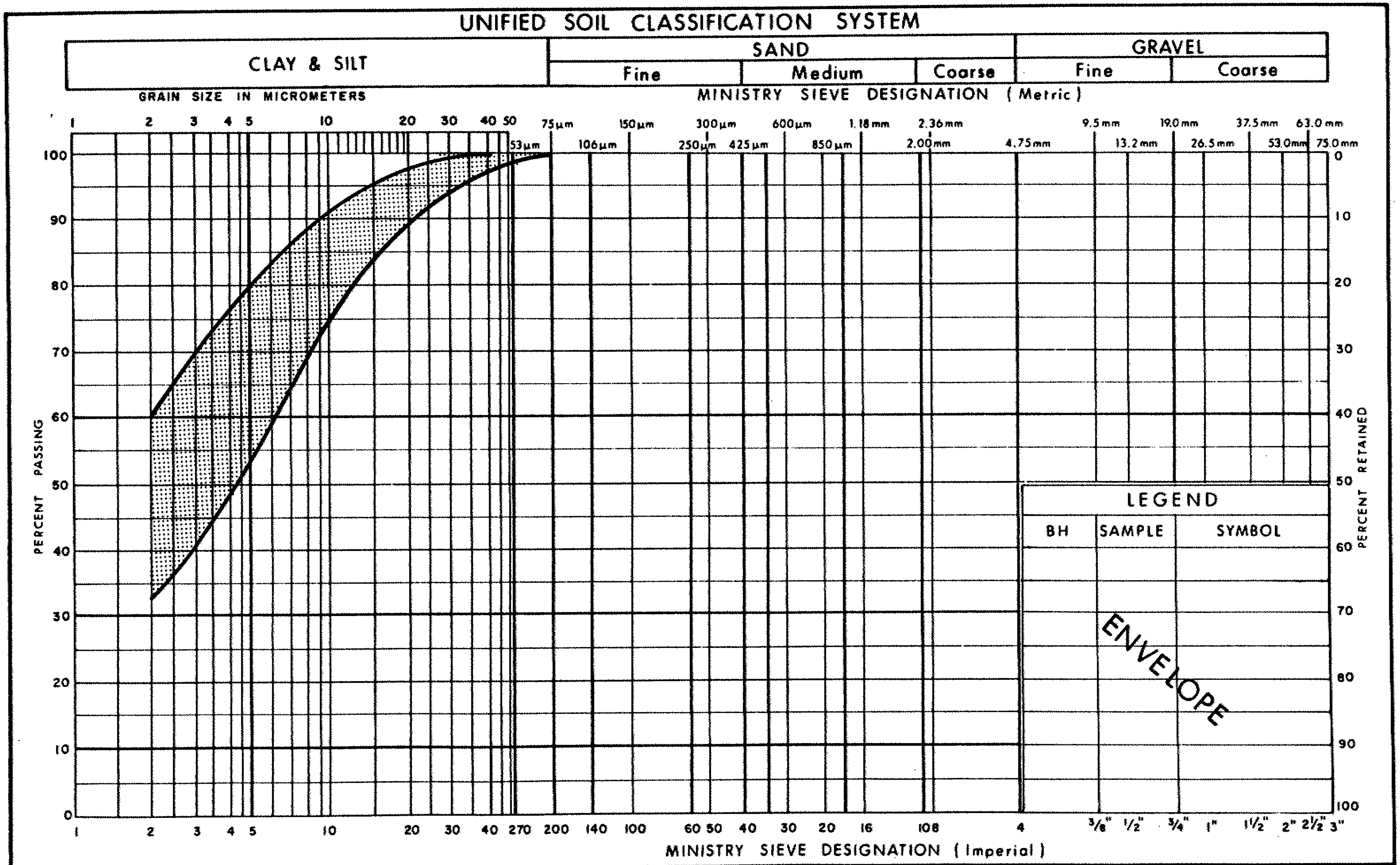
APPENDIX

78 12 M



Ministry of Transportation

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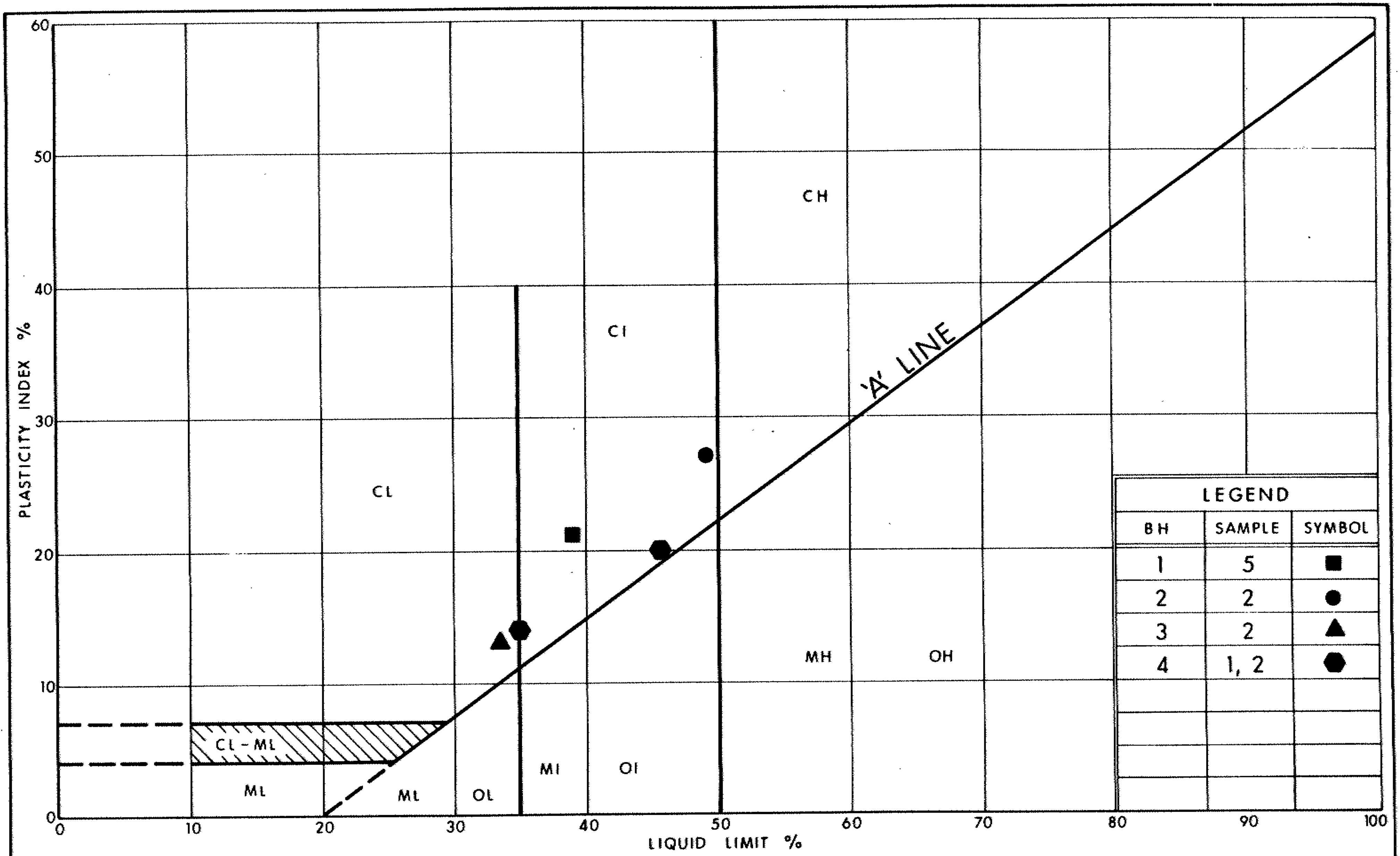
GRAIN SIZE DISTRIBUTION

CLAYEY SILT TO SILTY CLAY
WITH RANDOM LAYERS OF SILT

FIG No 1

W P 114 -87-00 B

Oct 75, FF-S-21



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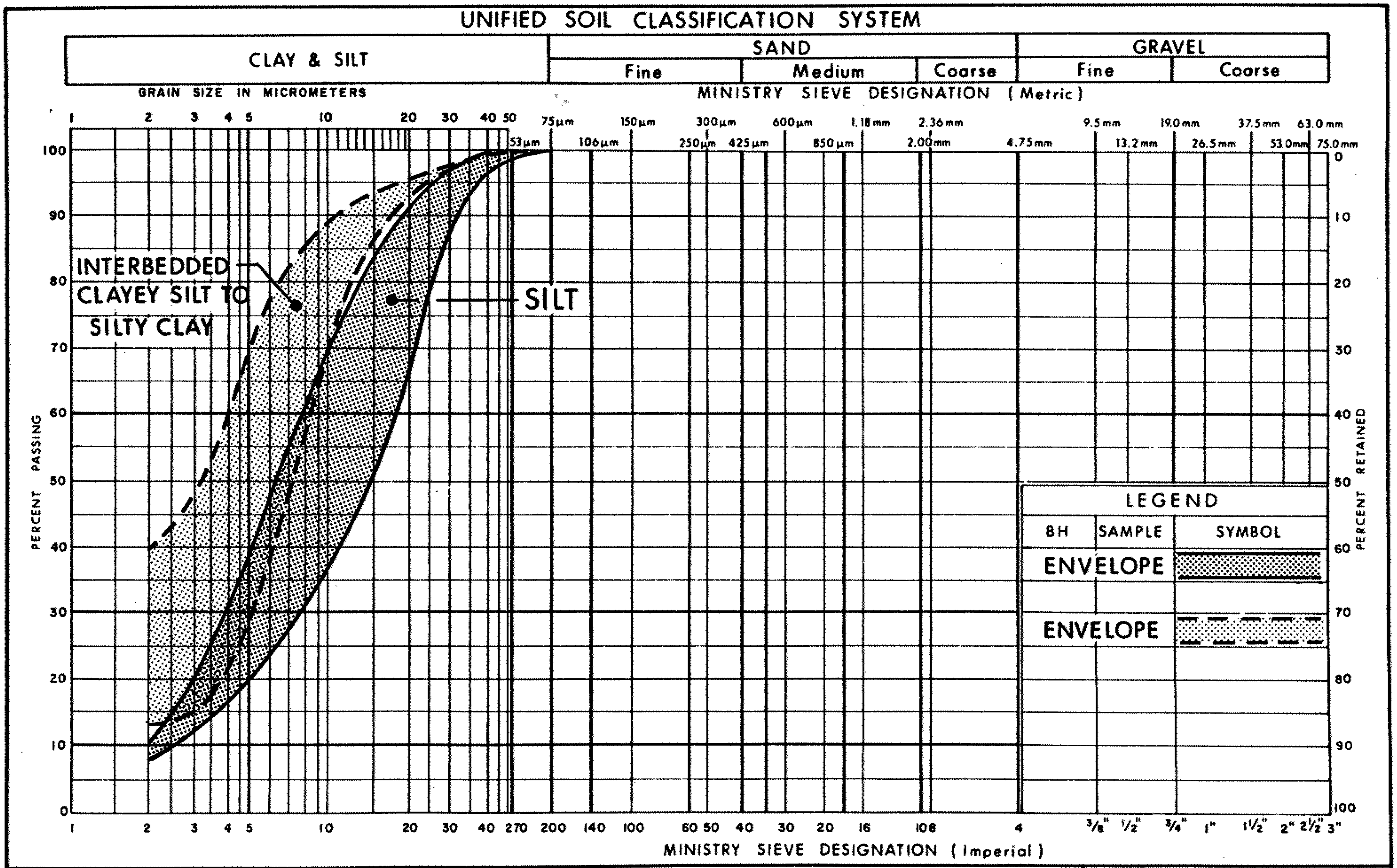
PLASTICITY CHART
CLAYEY SILT TO SILTY CLAY
WITH RANDOM LAYERS OF SILT

FIG No 2

W P 114-87-00B

78 12 M



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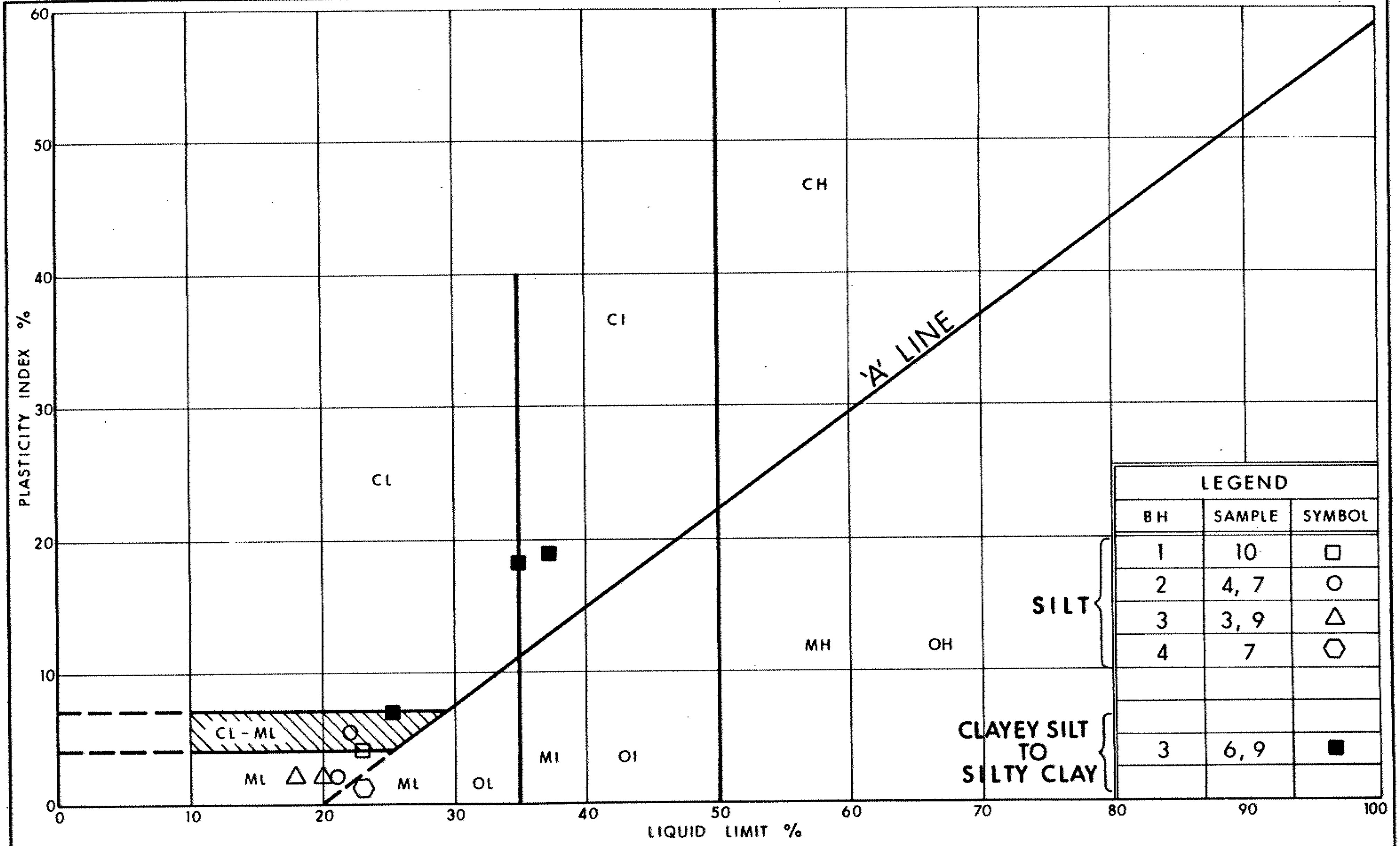
GRAIN SIZE DISTRIBUTION

SILT, WITH INTERBEDDED SEAMS/LAYERS OF CLAYEY SILT TO SILTY CLAY

FIG No 3

WP 114 - 87 - 00B

Oct 75, FF-S-21



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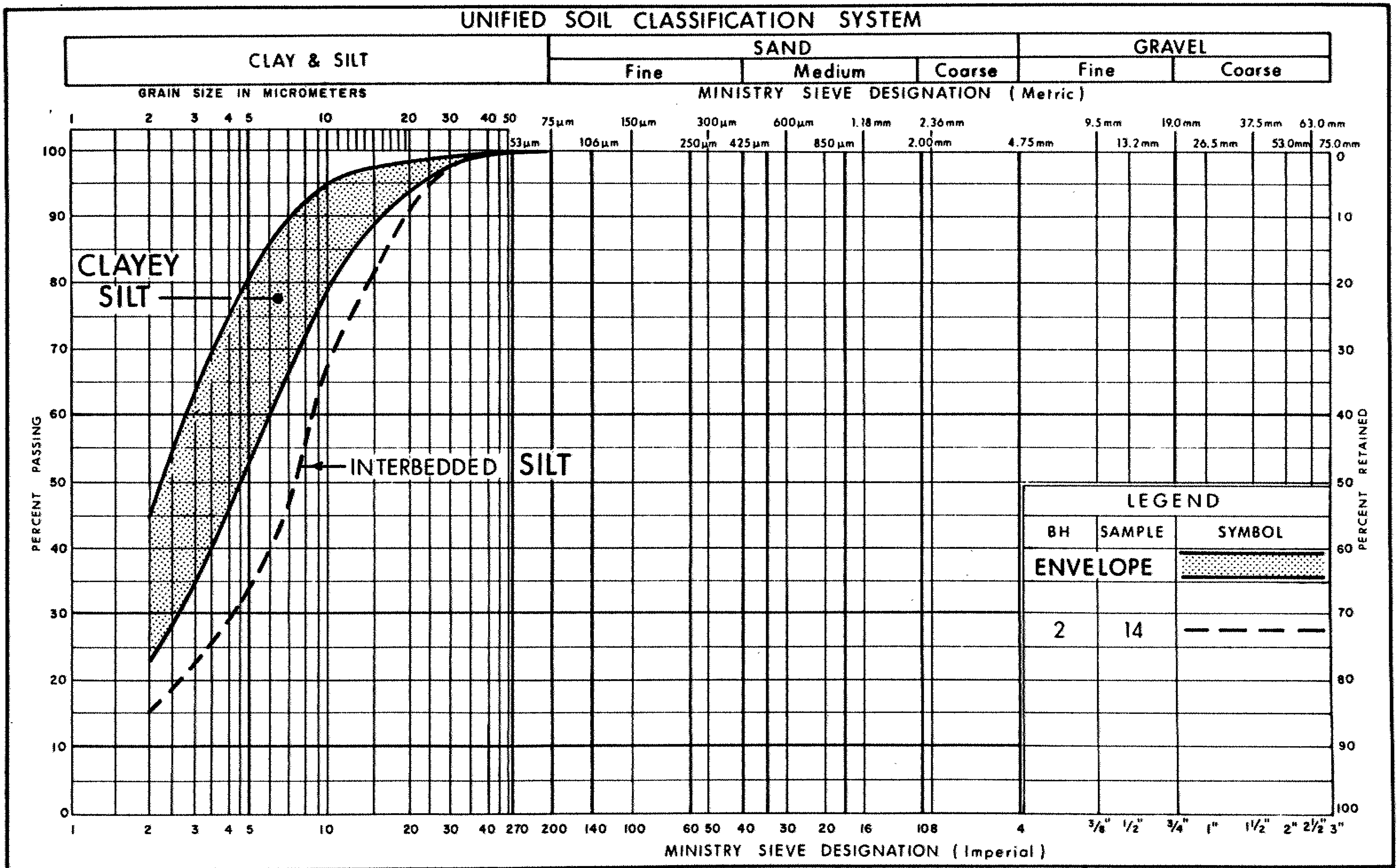
PLASTICITY CHART

SILT, WITH INTERBEDDED SEAMS/LAYERS OF CLAYEY SILT TO SILTY CLAY

FIG No 4

W P 114 - 87 - 00B

78 12 M



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GRAIN SIZE DISTRIBUTION

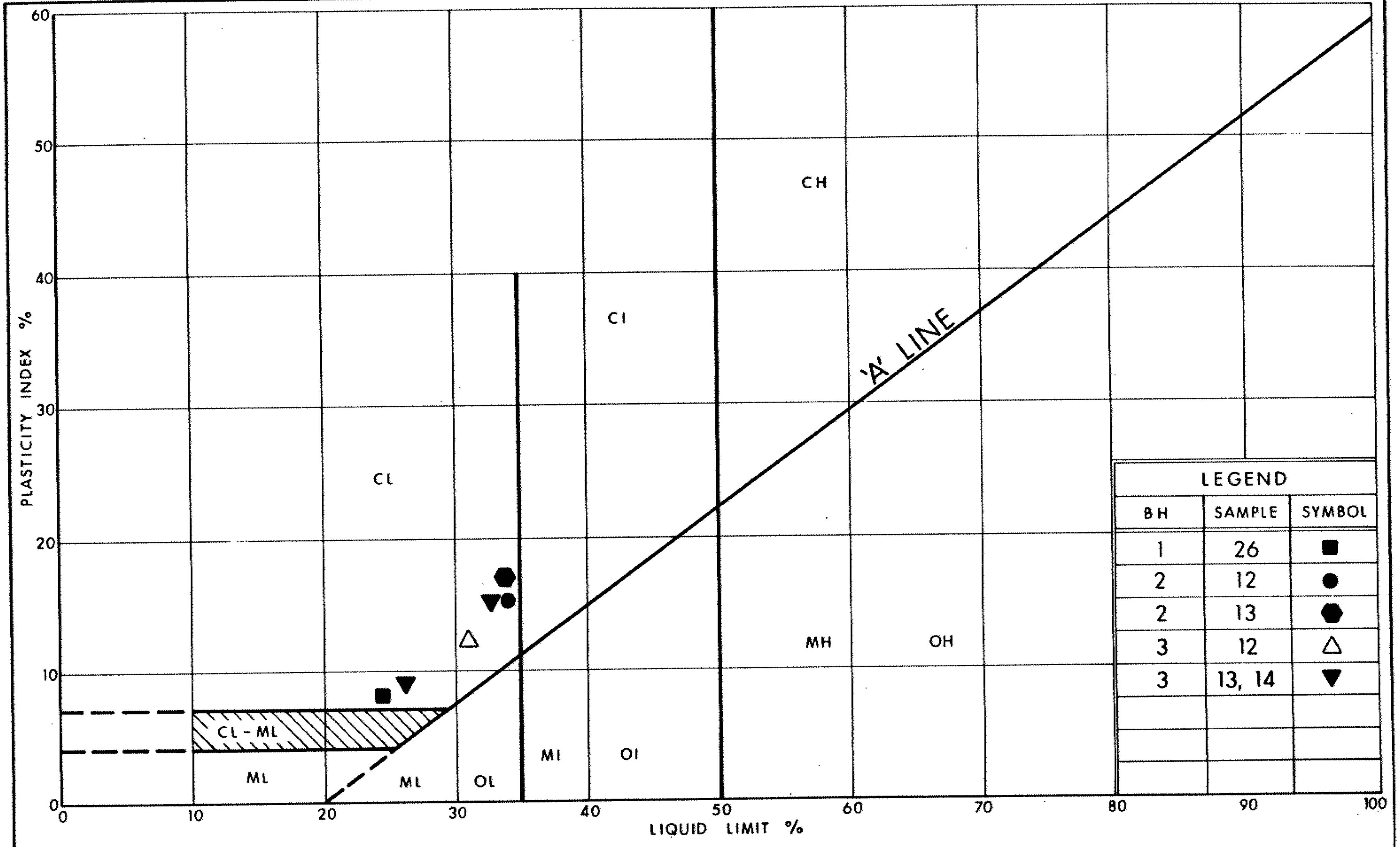
CLAYEY SILT

WITH RANDOM LAYERS OF SILT

FIG No 5

W P 114 - 87 - 00 B

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
1	26	■
2	12	●
2	13	⬢
3	12	△
3	13, 14	▼

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Ontario

PLASTICITY CHART

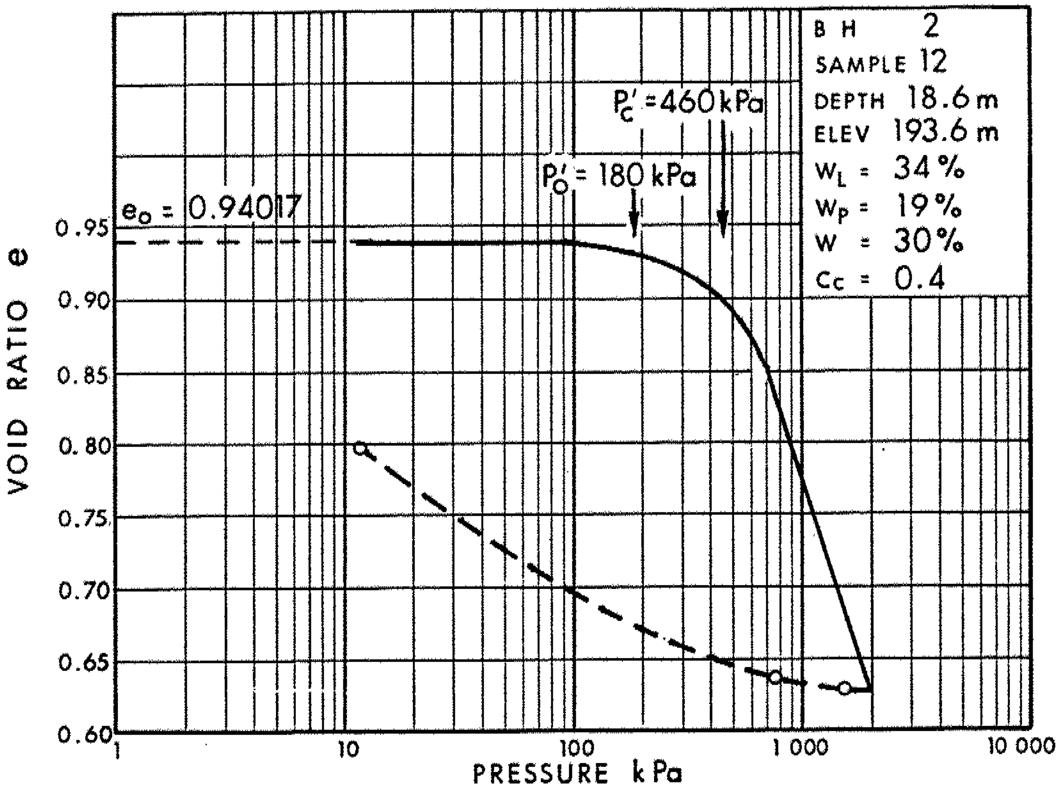
CLAYEY SILT

WITH RANDOM LAYERS OF SILT

FIG No 6

W P 114-87-00 B

VOID RATIO - PRESSURE CURVES



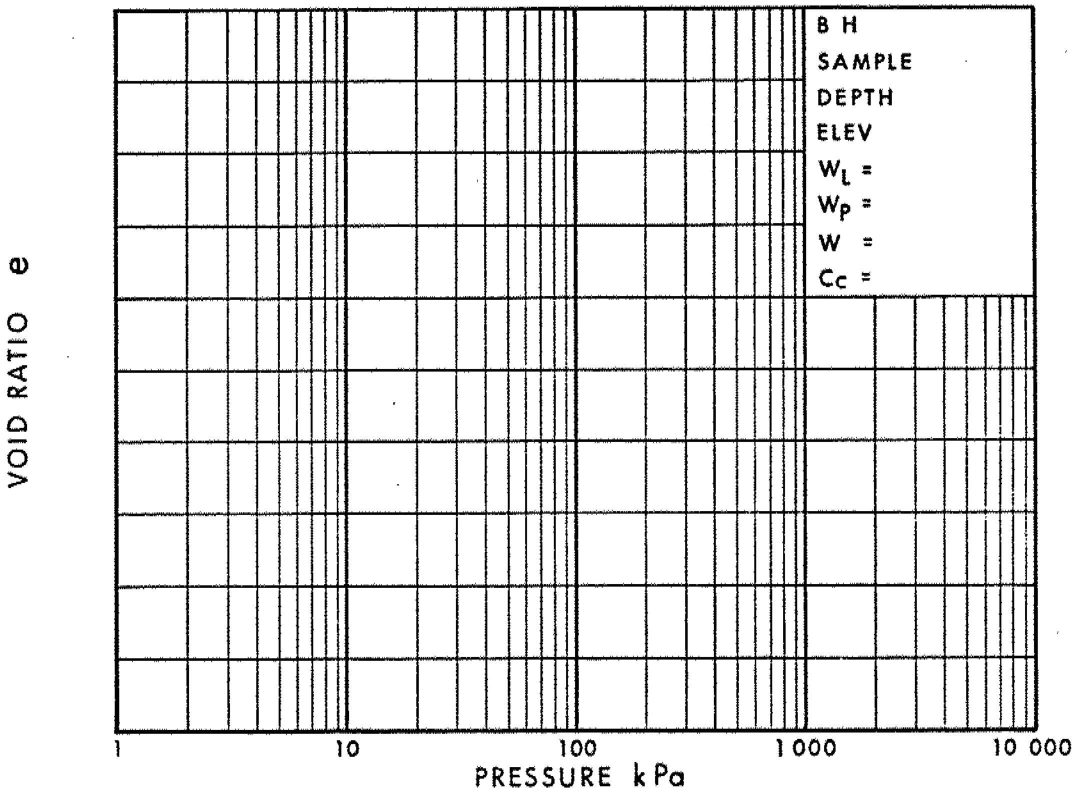
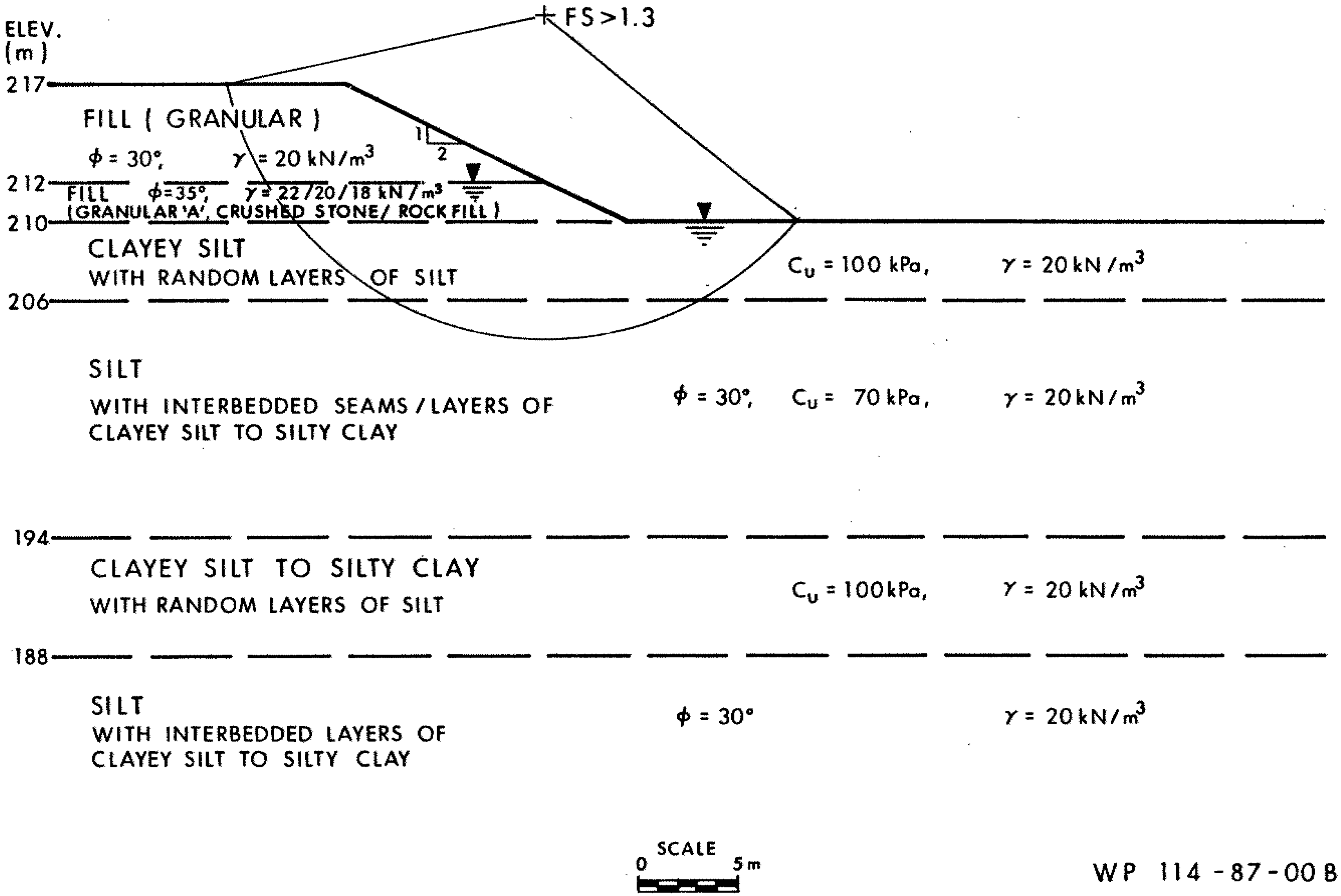


Fig 7

Figure 8 - Slope Stability Analyses

(HWY 403 & DUNMARK LAKE WEST CROSSING)

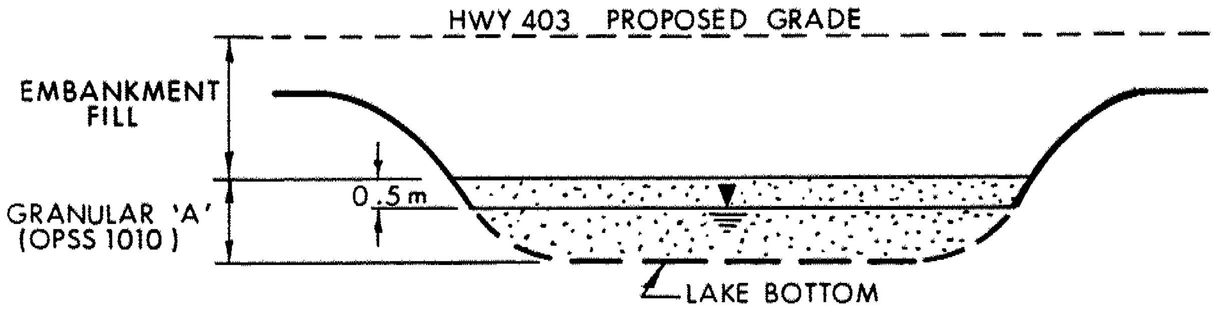


WP 114 - 87 - 00 B

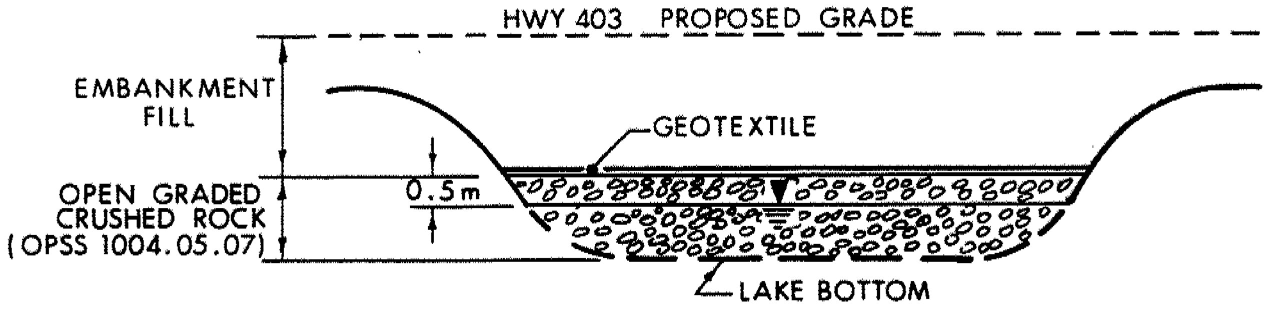
Figure 9 - Embankment Material / Construction

(HWY 403 & DUNMARK LAKE WEST CROSSING)

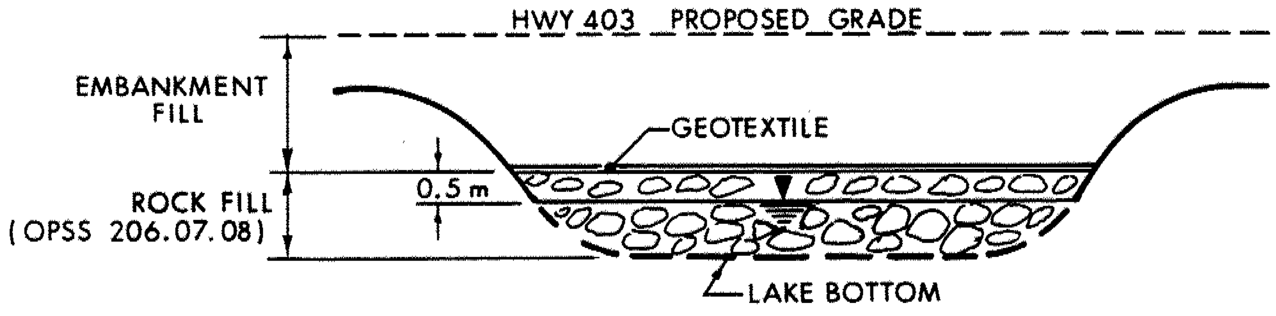
OPTION 1 - GRANULAR 'A'



OPTION 2 - OPEN GRADED CRUSHED ROCK



OPTION 3 - ROCK FILL



LEGEND:

▼ Lake Water Level

NOT TO SCALE

WP 114-87-00B

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 $\overline{N}$.

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:

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

RECORD OF BOREHOLE No 1

(Formerly BH No 1 WP114-87-00)

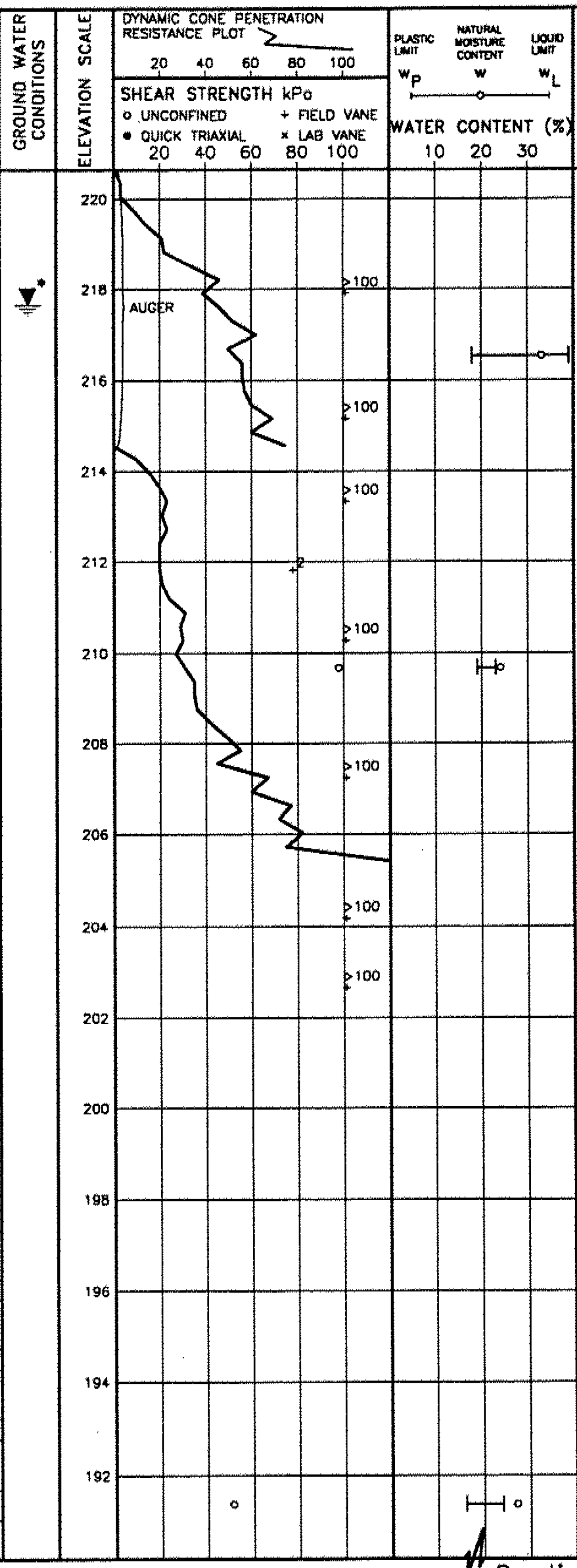
1 OF 2 METRIC

W.P. 114-87-008 LOCATION Co-Ords: N 4 783 438.0; E 255 868.0 ORIGINATED BY MP

DIST 4 HWY 403 BOREHOLE TYPE HS Auger & Dynamic Cone Test COMPILED BY LD

DATUM Geodetic DATE 91 02 18-22 CHECKED BY PP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	'N' VALUES	WATER CONTENT	UNIT WEIGHT γ kN/m³	GR SA SI CL
220.7	Ground Surface						
0.0	Clayey Silt to Silty Clay with random layers of Silt Stiff to Very Stiff Brown Grey	1	SS	3			
		2	SS	7			
		3	SS	10			
		4	SS	17			
		5	TW	PH	wP–w–wL	18.5	0 0 35 65
		6	SS	7			
		7	SS	12			
		8	SS	7			
211.6 / 9.1	Silt with interbedded seams/layers of Clayey Silt to Silty Clay Grey, Loose to Compact/ Stiff to Very Stiff	9	SS	7			
		10	TW	PH	wP–w	19.4	0 0 90 10
		11	SS	15			
		12	SS	6			
		13	SS	9			
		14	TW	PH			
		15	SS	7			
		16	SS	7			
		17	SS	9			
		18	SS	9			
		19	SS	8			
		20	SS	9			
		21	SS	11			
		22	SS	10			
195.9 / 24.8	Clayey Silt with random layers of Silt Grey, Stiff to Very Stiff	23	SS	9			
		24	SS	13			
		25	SS	11			
		26	TW	PH	wP–w	20.1	0 0 70 30
190.2 / 30.5	Continued						



+3, x5: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 1

2 OF 2 METRIC

W.P. 114-87-008
LOCATION Co-Ords: N 4 783 438.0; E 255 868.0
ORIGINATED BY MP
DIST 4 HWY 403
BOREHOLE TYPE HS Auger & Dynamic Cone Test
COMPILED BY LD
DATUM Geodetic
DATE 91 02 18-22
CHECKED BY PP

SOIL PROFILE			SAMPLES			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 (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						GR	SA	SI	CL
190.2	Continued													
30.5			27	SS	15		190							
			28	SS	18		188							
187.2 33.5	Silt with interbedded layers of Clayey Silt to Silty Clay Grey, Compact		29	SS	11		186							
			30	SS	11									
			31	SS	12		184							
182.4			32	SS	25	/3cm					5	5	75	15
38.3	End of Borehole (Auger Refusal – Probable Bedrock) * 91 02 23													

+3, x5 : Numbers refer to Sensitivity

20 15-◇-5 10 (%) STRAIN AT FAILURE

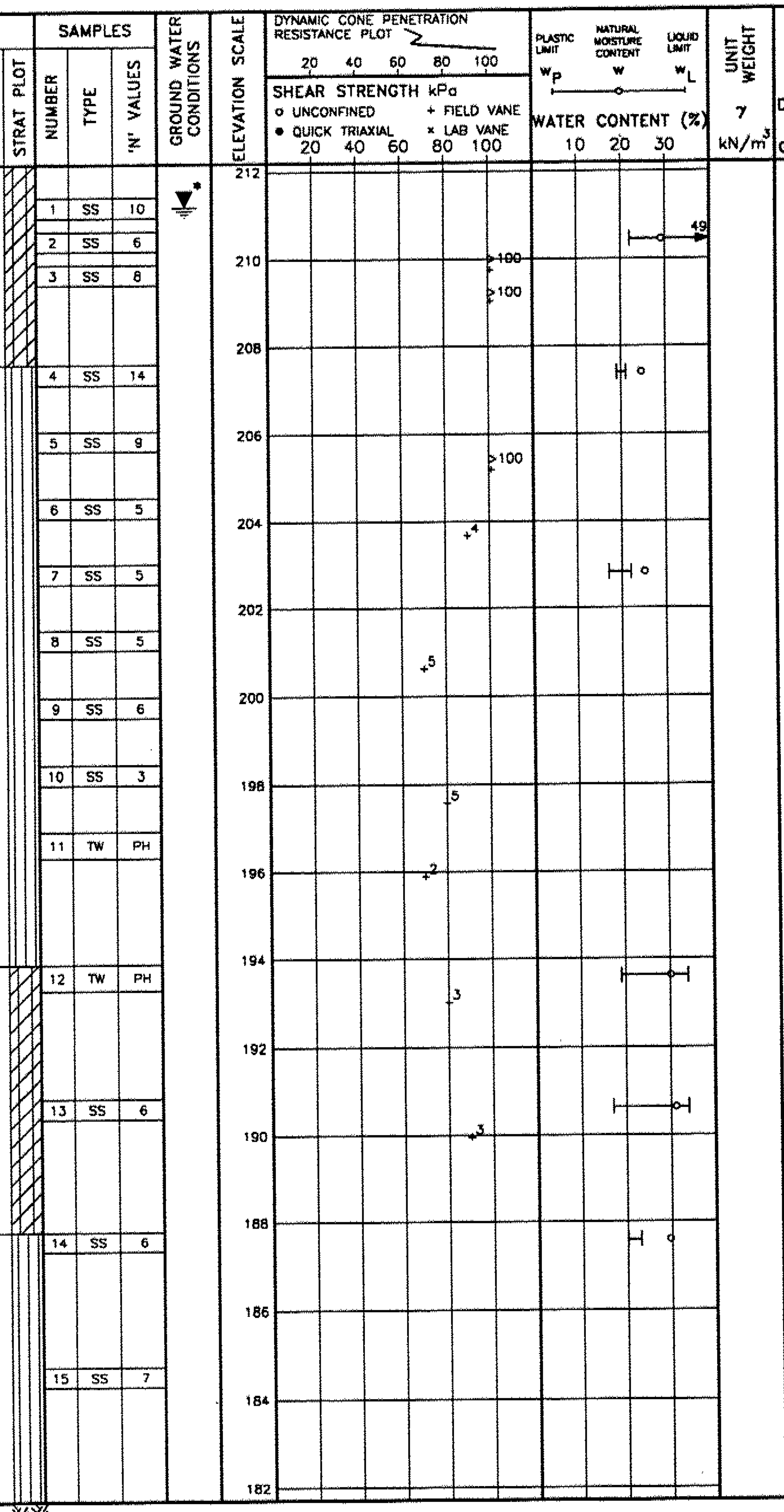
RECORD OF BOREHOLE No 2

1 OF 1 METRIC

W.P. 114-87-00B
LOCATION Co-Ords: N 4 783 449.3; E 255 908.1
ORIGINATED BY LD
DIST 4 HWY 403
BOREHOLE TYPE HS Auger
COMPILED BY LD
DATUM Geodetic
DATE 92 07 10
CHECKED BY PP

ELEV DEPTH	DESCRIPTION
212.2	Groud surface
0.0	Clayey Silt to Silty Clay with random layers of Silt Brown, Very Stiff
207.6 4.6	Silt with interbedded seams/layers of Clayey Silt to Silty Clay Grey, Very Loose to Compact/ Stiff to Very Stiff
193.9 18.3	Clayey Silt with random layers of Silt Grey, Stiff to Very Stiff
187.8 24.4	Silt with interbedded layers of Clayey Silt to Silty Clay Loose
181.7 30.5	End of Borehole * 92 07 10 (Auger Refusal – Probable Bedrock)

SAMPLES NUMBER	TYPE	'N' VALUES	GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	10				
2	SS	6	0	0	45	55
3	SS	8				
4	SS	14	0	0	92	8
5	SS	9				
6	SS	5				
7	SS	5	0	0	88	12
8	SS	5				
9	SS	6				
10	SS	3				
11	TW	PH				
12	TW	PH	0	0	57	43
13	SS	6	0	0	55	45
14	SS	6	0	0	81	19
15	SS	7				



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

$15\overset{20}{\underset{10}{\diamond}}5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

1 OF 2 METRIC

W.P. 114-87-008
DIST 4 HWY 403
DATUM Geodetic
LOCATION Co-Ords: N 4 783 494.0; E 255 936.7
BOREHOLE TYPE HS Auger, Washbore - Tricone
DATE 92 07 09-10
ORIGINATED BY TS
COMPILED BY LD
CHECKED BY PP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION
213.7	Ground Surface
0.0	Clayey Silt with random layers of Silt Brown, Very Stiff
209.1 4.6	Silt with interbedded seams/layers of Clayey Silt to Silty Clay Grey, Very Loose to Compact/ Stiff to Very Stiff seams layers
195.4 18.3	Clayey Silt with random layers of Silt Grey, Stiff to Very Stiff
187.8 25.9	Silt with interbedded layers of Clayey Silt to Silty Clay Compact
183.2 30.5	Continued

SAMPLES NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS	20					
2	SS	25		0	0	50	50
3	SS	9		0	0	89	11
4	SS	7					
5	SS	6					
6	SS	10		0 0	0 0	86 58	14 42
7	SS	10					
8	SS	7					
9	SS	7		0 0	0 0	89 67	11 33
10	SS	11					
11	SS	8					
12	TW	PH	19.2	0	3	72	25
13	SS	17		0	0	58	42
14	SS	20		0	0	78	22
15	SS	16					
16	SS	18					

GROUND WATER CONDITIONS

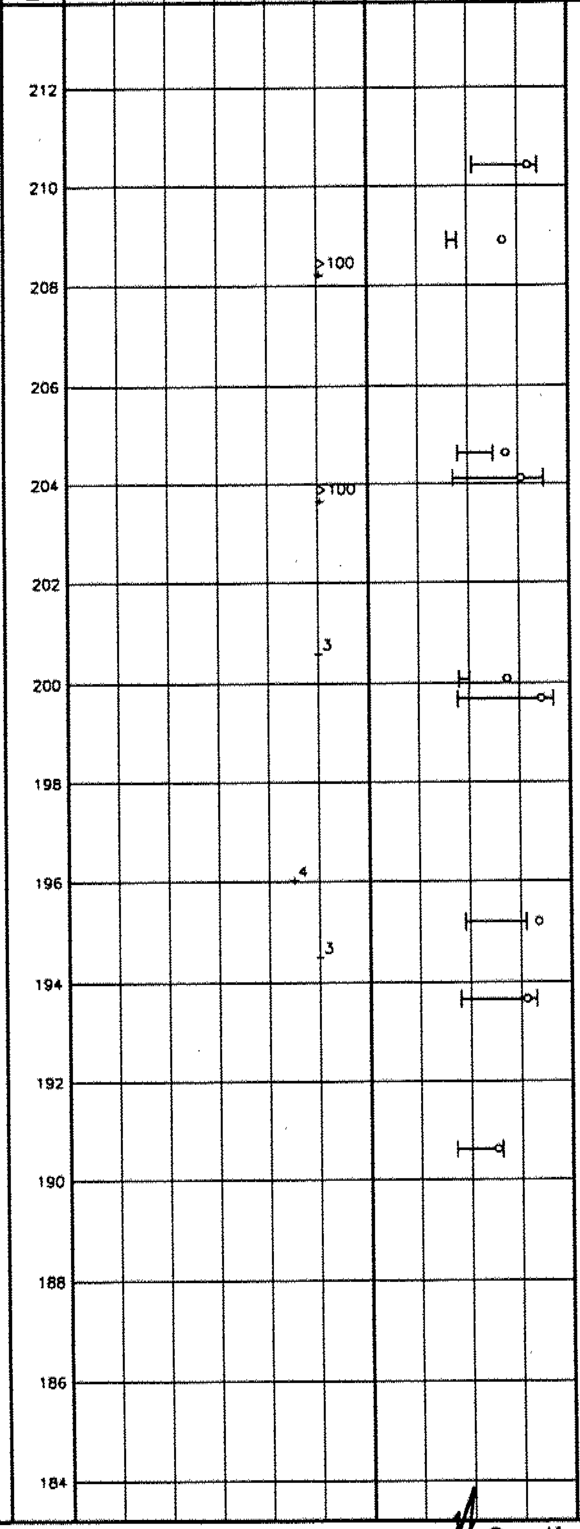
ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30



Continued

+3, x5: Numbers refer to Sensitivity

$15\overset{20}{\underset{10}{\oplus}}5$ (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 3 2 OF 2 METRIC

W.P. 114-87-008
LOCATION Co-Ords: N 4 783 494.0; E 255 936.7
ORIGINATED BY TS
DIST 4 HWY 403
BOREHOLE TYPE HS Auger, Washbore – Tricone
COMPILED BY LD
DATUM Geodetic
DATE 92 07 09-10
CHECKED BY PP

SOIL PROFILE			SAMPLES			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
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
183.2 30.5	Continued										
181.8 31.9	End of Borehole (Auger Refusal – Probable Bedrock) * 92 07 09						182				

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

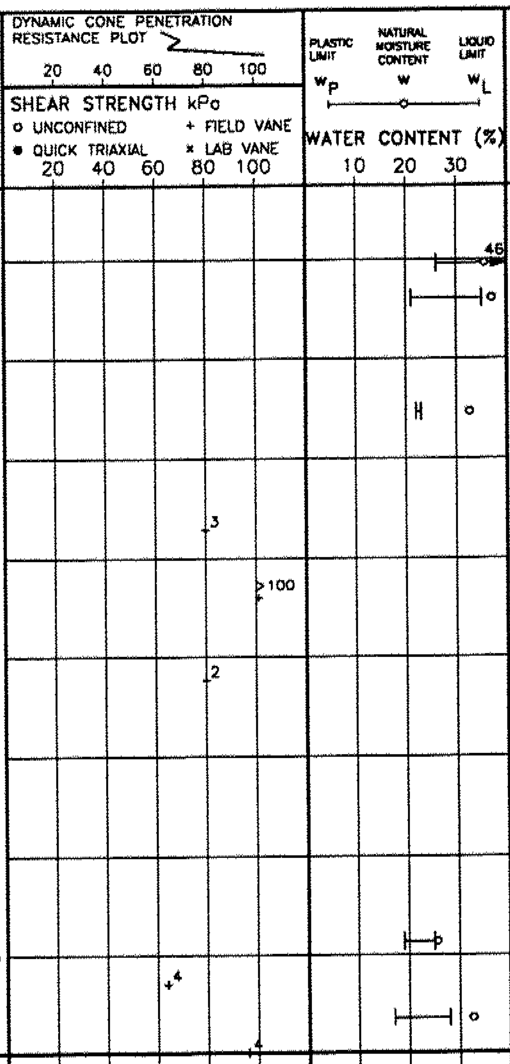
$15\overset{20}{\underset{10}{\oplus}}5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

1 OF 1 METRIC

W.P. 114-87-008	LOCATION Co-Ords: N 4 783 473.5; E 255 921.6	ORIGINATED BY PM
DIST 4 HWY 403	BOREHOLE TYPE Washboring, N Casing	COMPILED BY LD
DATUM Geodetic	DATE 92 07 20-21	CHECKED BY PP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
211.5	Water Surface								
0.0	Water								
210.0 1.5	Clayey Silt to Silty Clay with random layers of Silt		1	SS	2		210	18.5	Org. 1.5%
			2	SS	3			Org. 2.4%	0 0 40 60
	trace Organics		3	WS	–				
	Brown, Very Stiff		4	TW	PH		208		
			5	SS	15				
207.0			7	SS	13				0 0 92 8
4.5			8	WS	–		206		
	Brown Grey		9	SS	12				
			10	WS	–				
			11	SS	7		204		
			12	WS	–				
			13	SS	9				
	Silt with interbedded seams/layers of Clayey Silt		14	WS	–		202		
	Grey, Loose to Compact/ Stiff to Very Stiff		15	TW	PH				
			16	WS	–		200		
			17	SS	10				
			18	WS	–		198		
196.5			19	WS	–				
15.0	Clayey Silt with random layers of Silt		20	SS	9				
			21	WS	–		196		
194.1	Grey, Stiff to Very Stiff		22	SS	11				
17.4	End of Borehole * Sample 6, Wash Sample								



+3, x5: Numbers refer to Sensitivity

20
15-◇-5 (%) STRAIN AT FAILURE
10

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Ontario

Foundation Design

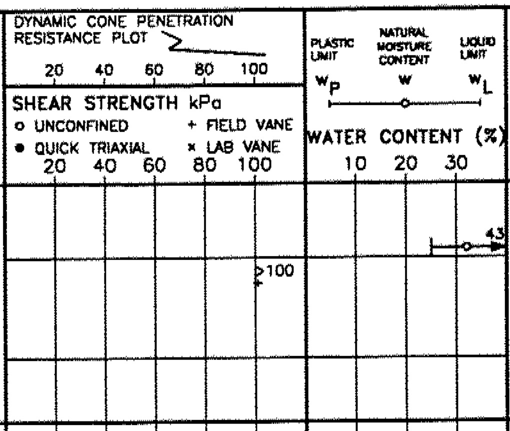
RECORD OF BOREHOLE No 5

1 OF 1

METRIC

W.P. 114-87-008
LOCATION Co-Ords: N 4 783 452.1; E 255 912.2
ORIGINATED BY PM
DIST 4 HWY 403
BOREHOLE TYPE Washboring, N Casing
COMPILED BY LD
DATUM Geodetic
DATE 92 07 22
CHECKED BY PP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
211.5	Water Surface								
0.0	Water								
0.3	Clayey Silt to Silty Clay with random layers of Silt Grey – Brown, trace Organics, Very Stiff		1	SS	8		210	18.7	0 0 66 34 Org. 1.9%
			2	WS	–				
			3	SS	10				
			4	WS	–				
			5	TW	PH				
208.0			6	SS	23		208		
3.5	Silt, with interbedded seams/layers of Clayey Silt, Compact/Stiff to Very Stiff		7	WS	–				
206.7			8	SS	15				
4.8	End of Borehole								



+3, ×5 : Numbers refer to Sensitivity

20
15◇5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 6

1 OF 1 METRIC

W.P. 114-87-008	LOCATION Co-Ords: N 4 783 488.4; E 255 928.5	ORIGINATED BY PM
DIST 4 HWY 403	BOREHOLE TYPE Washboring, N Casing	COMPILED BY LD
DATUM Geodetic	DATE 92 07 22	CHECKED BY PP

SOIL PROFILE			SAMPLES			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 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 (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						GR	SA	SI	CL
211.5	Water Surface													
0.0 210.3	Water													
1.2	Clayey Silt to Silty Clay with random layers of Silt Very Stiff Brown, trace Organics		1	SS	4		210				0	0	62	38
			2	WS	–									
208.1	Grey		3	TW	PH									
3.4	Silt, with interbedded seams/layers of Clayey Silt		4	SS	30		208							
			5	WS	–									
206.7	Compact/ Stiff to Very Stiff		6	SS	16									
4.8	End of Borehole													

+³, ×⁵ : Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\diamond}} 5$ (%) STRAIN AT FAILURE

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

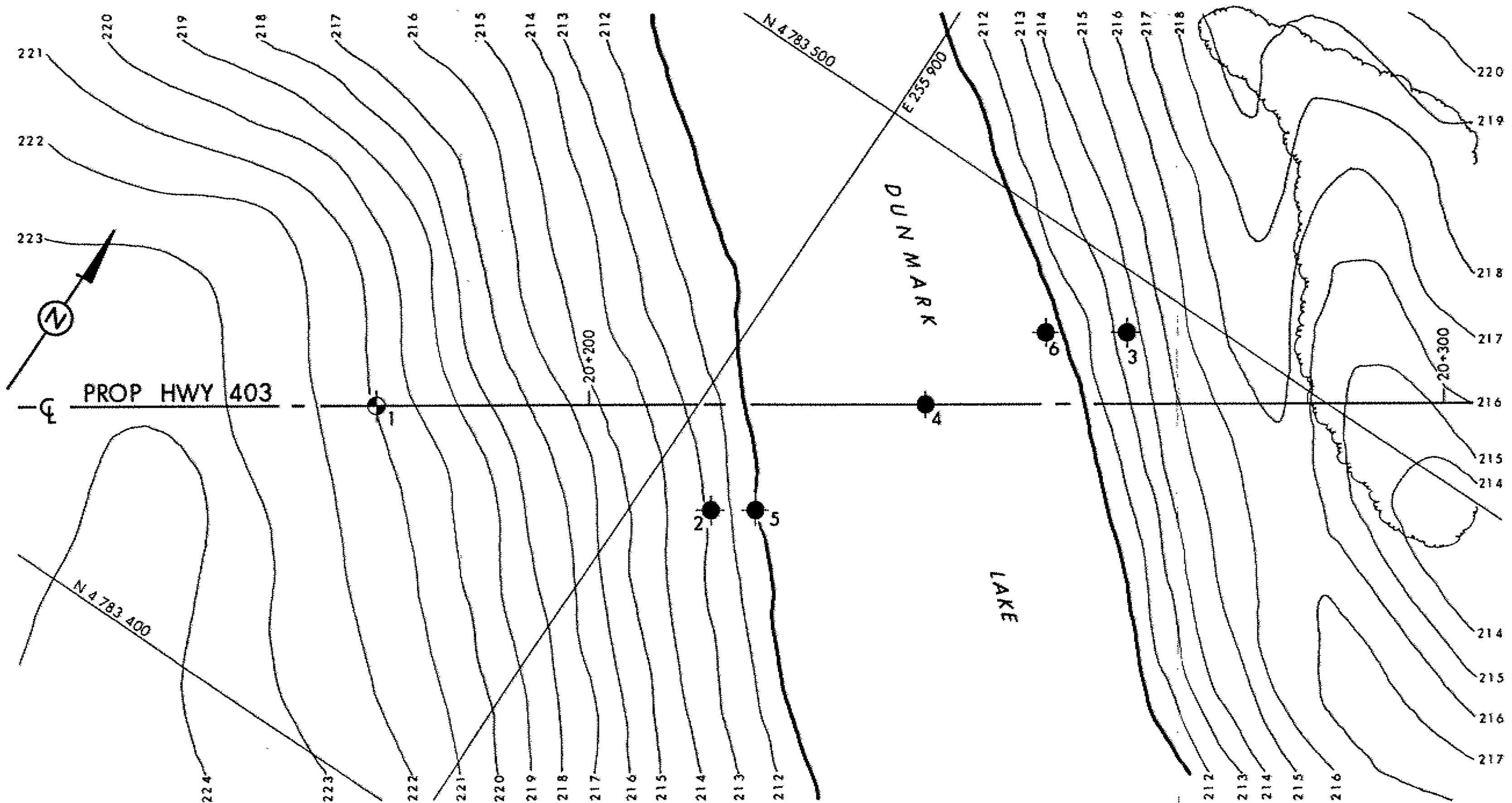
METRIC

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

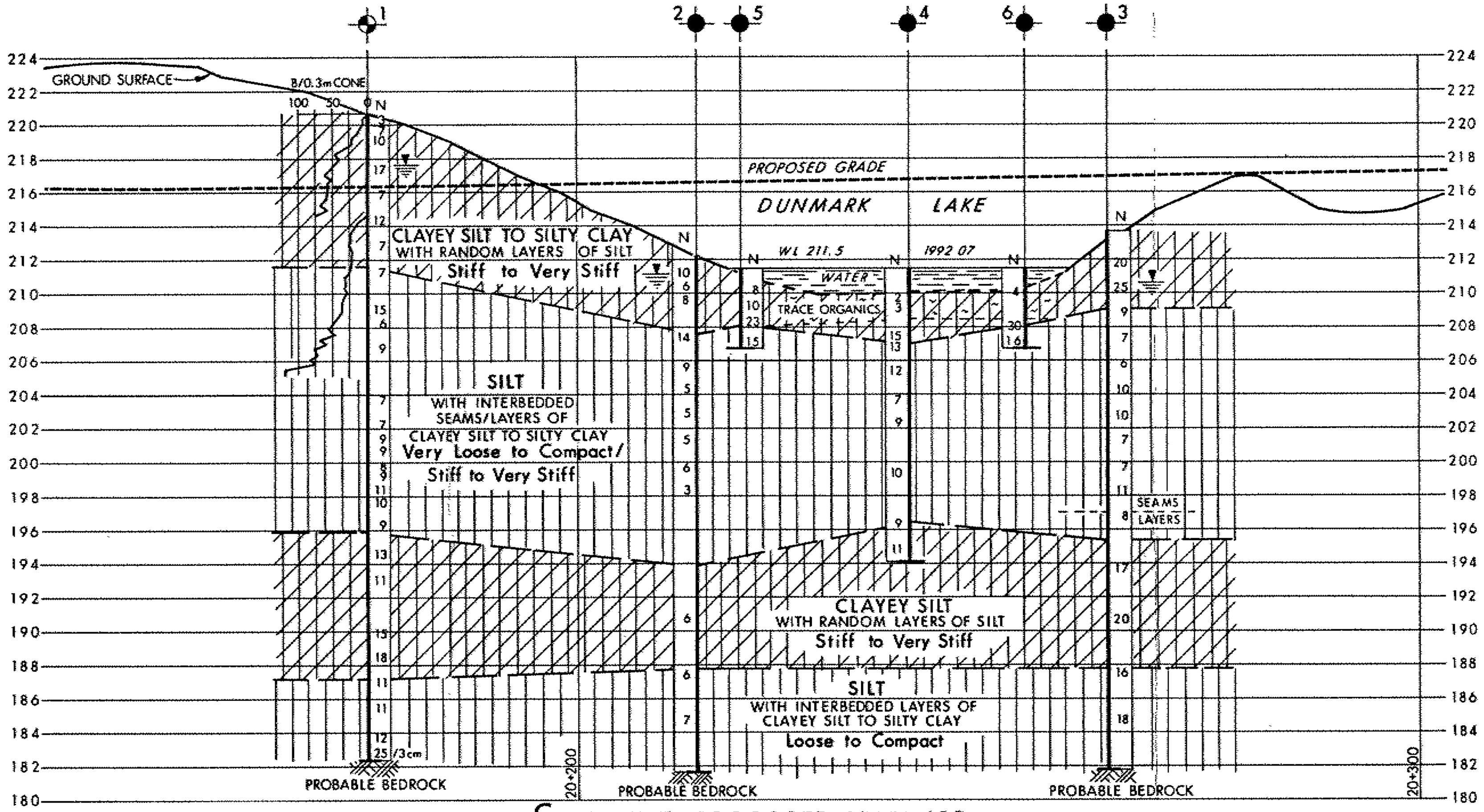
CONT No
WP No 114-87-00 B

PROPOSED EMBANKMENT
DUNMARK LAKE WEST CROSSING
BORE HOLE LOCATIONS & SOIL STRATA

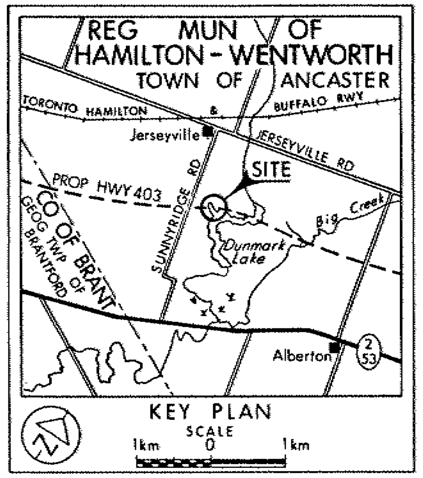
SHEET



PLAN
SCALE
8m 4 0 8m



℄ PROFILE PROPOSED HWY 403
SCALES
8m 4 0 8m HOR
4m 2 0 4m VERT



KEY PLAN
SCALE
1km 0 1km

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 1991 02 and 1992 07

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	220.7	4 783 438.0	255 868.0
2	212.2	4 783 449.3	255 908.1
3	213.7	4 783 494.0	255 936.7
4	211.5	4 783 473.5	255 921.6
5	211.5	4 783 452.1	255 912.2
6	211.5	4 783 488.4	255 928.5

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 GC 2.01 of OPS Gen.Cond.

REV.	DATE	BY	DESCRIPTION

Geocres No 40P1-90

HWY No 403			DIST 4
SUBM'D TS	CHECKED TS	DATE 1993 02 15	SITE
DRAWN RS	CHECKED	APPROVED	DWG 1148700B-A