

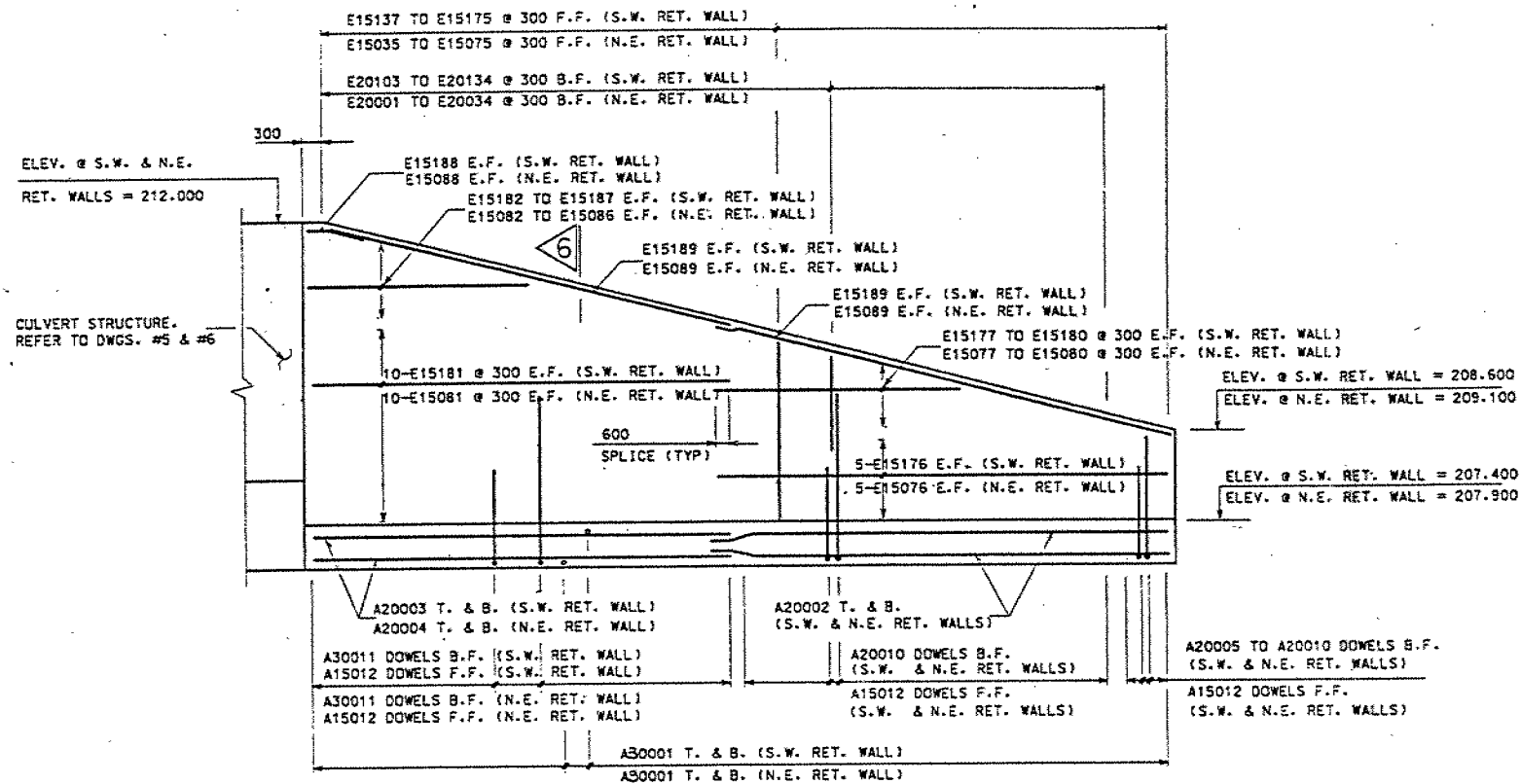
G.I.-30 SEPT. 1976

GEOCRES No. 40P1-89DIST. 4 REGION W.P. No. 114-87-00(C)CONT. No. 94-55W. O. No. STR. SITE No. HWY. No. 403LOCATION Hwy 403 - Dunmark Lake E
(Culvert)No of PAGES - =====
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

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DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

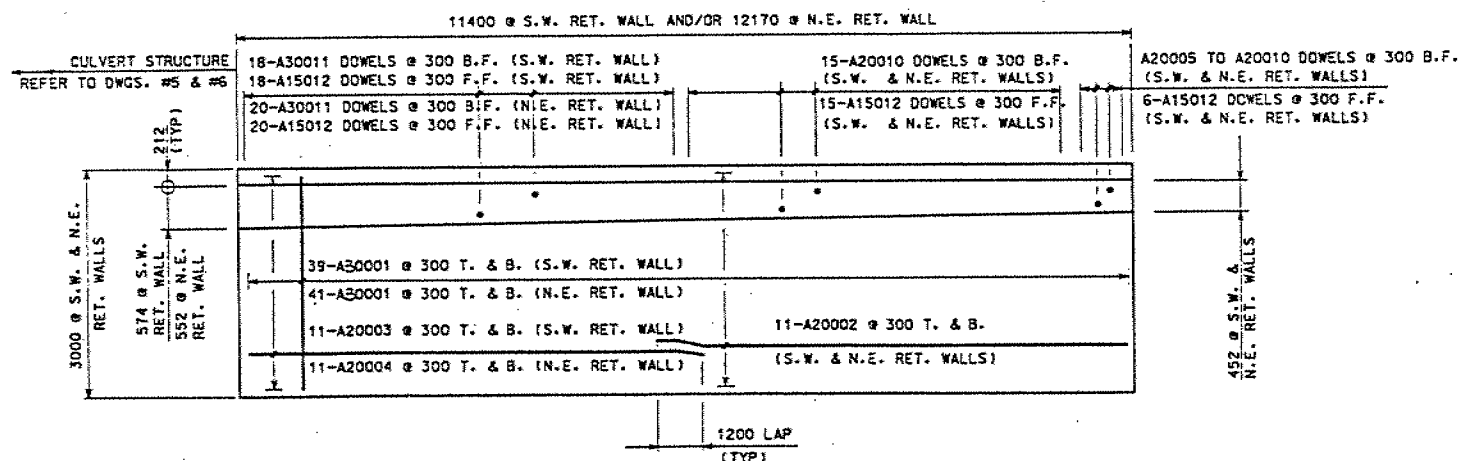
CONT No 93-87
WP No 268-92-01
HWY 403 / BIG CREEK
CULVERT @ STA. 21+643
RETAINING WALLS

SHEET
253

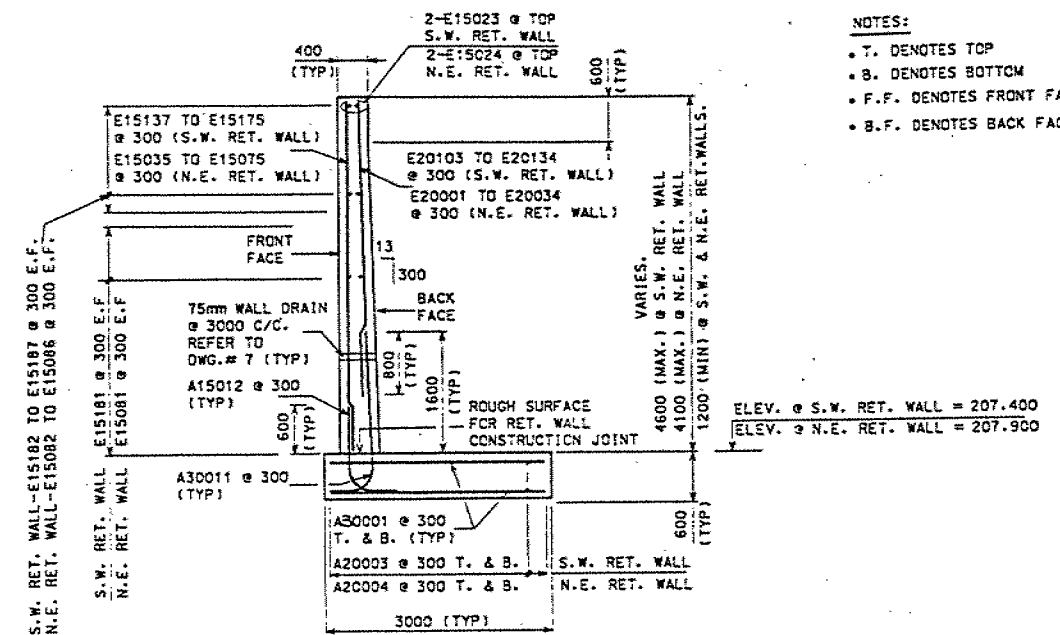


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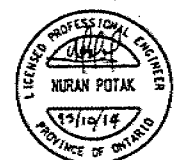
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FOOTING PLAN
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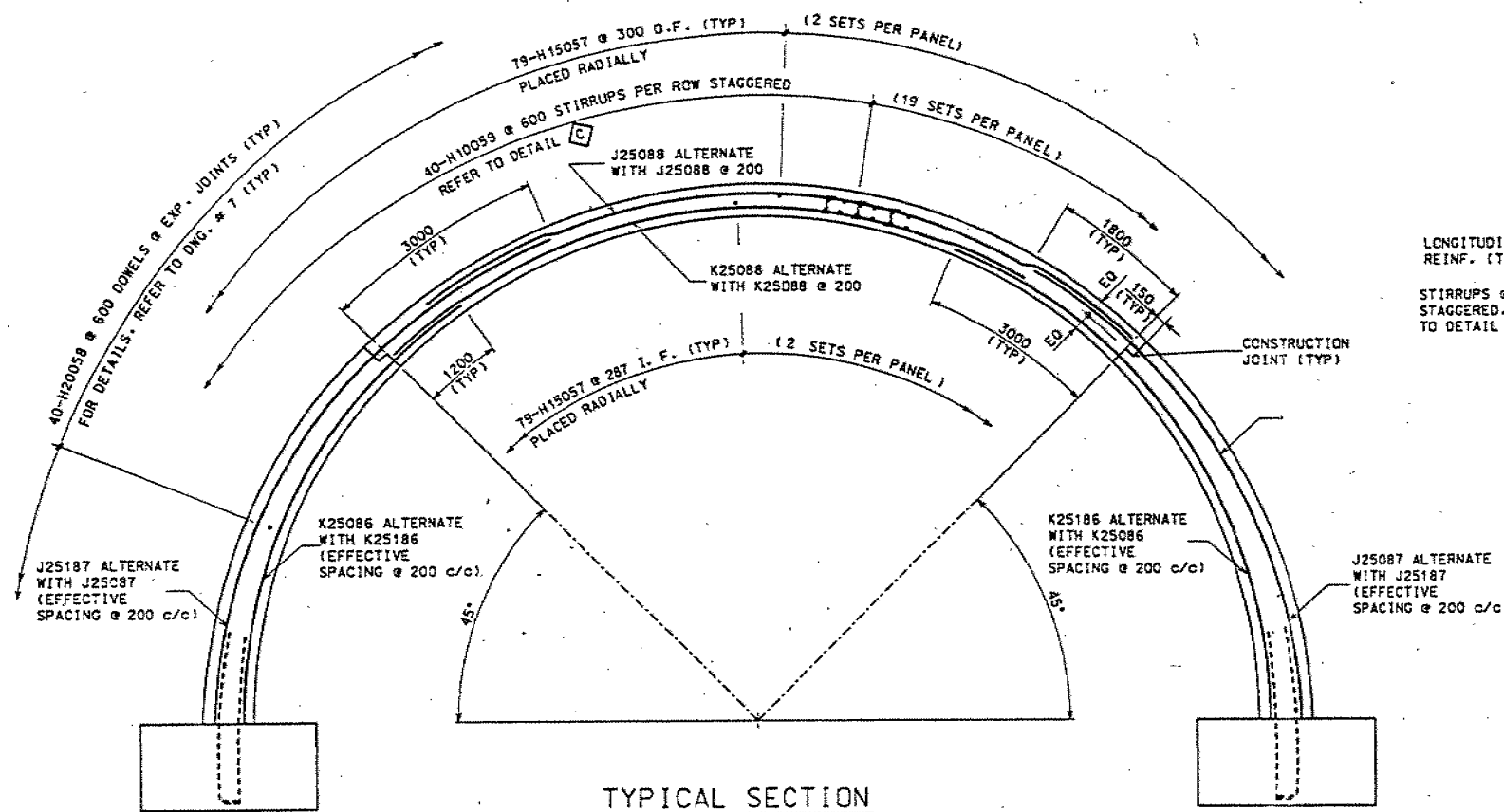
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COUNT NO 95-01
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HWY 403 / BIG CREEK
CULVERT @ STA. 21+643
CULVERT DETAILS II

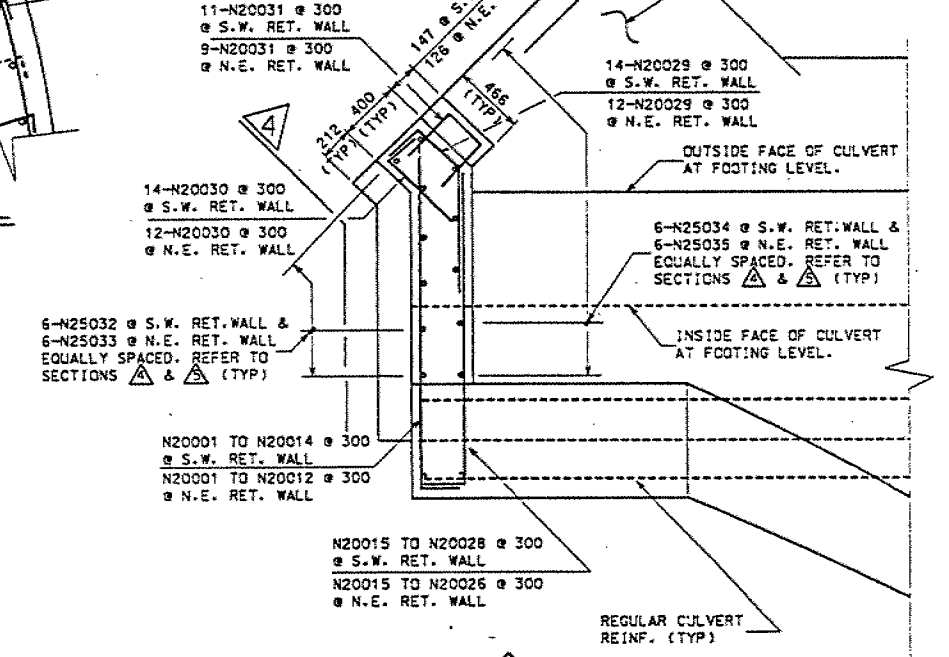
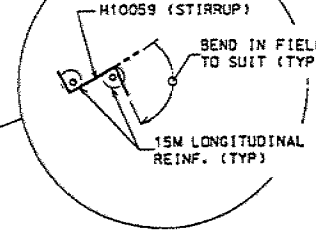
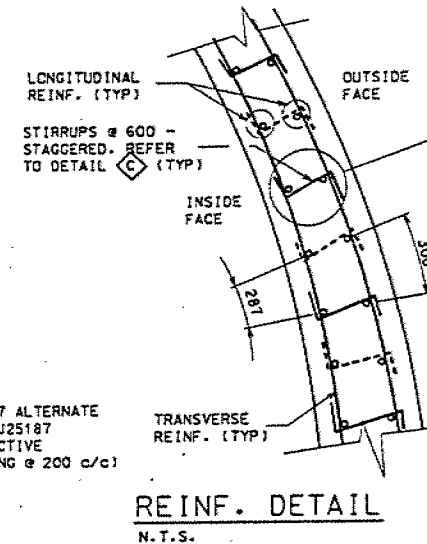
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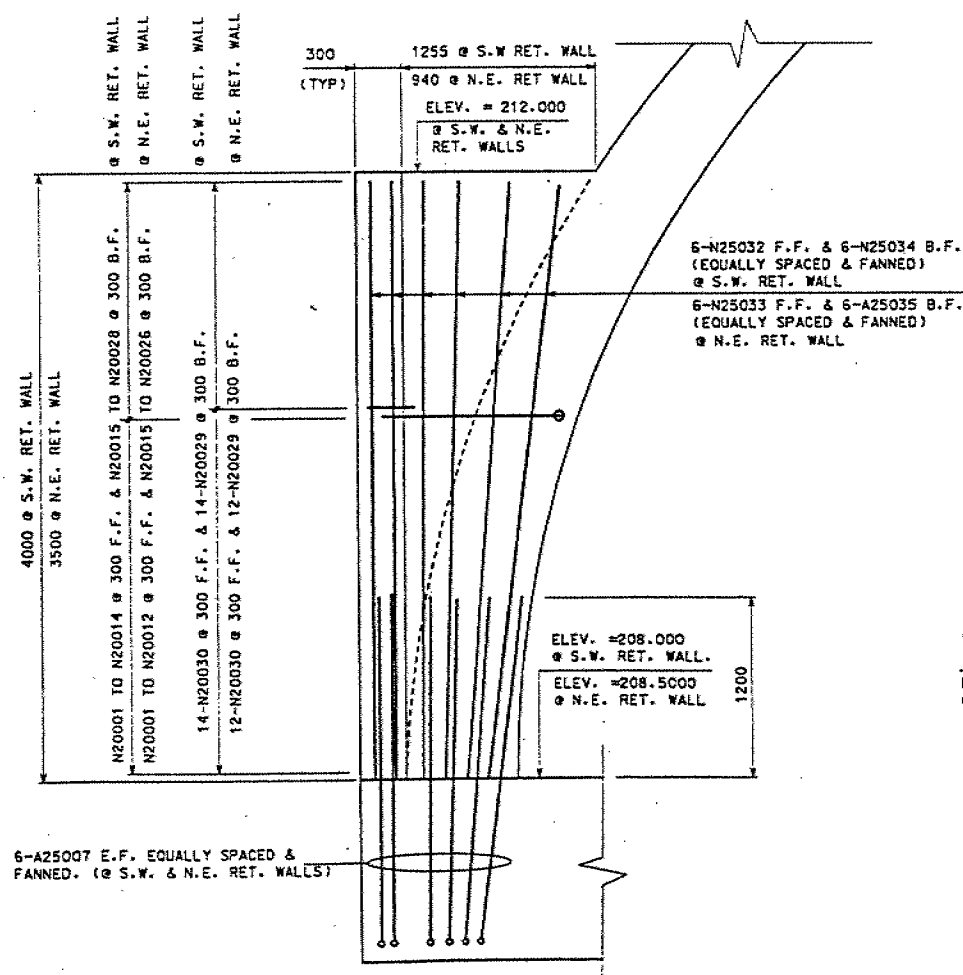
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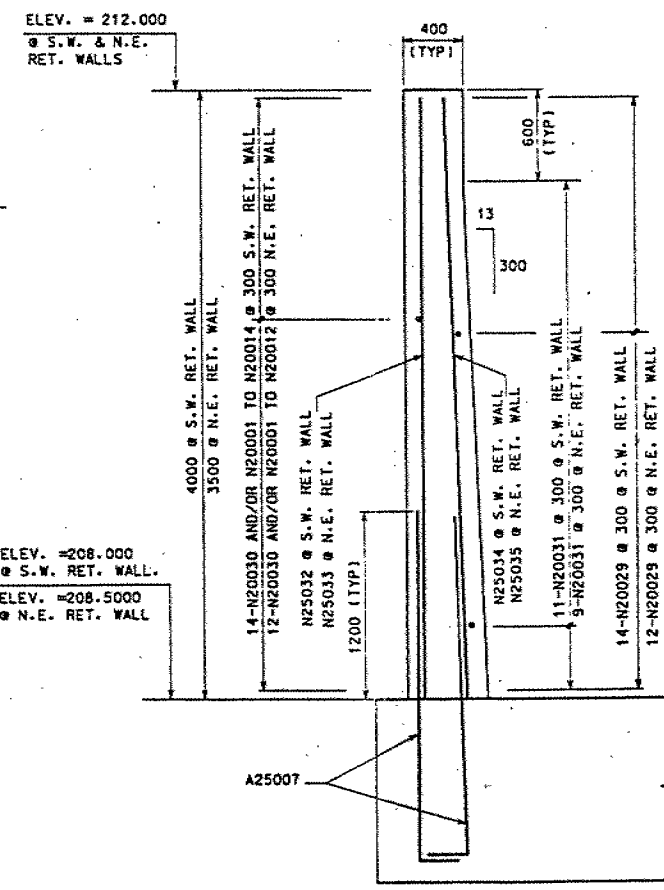
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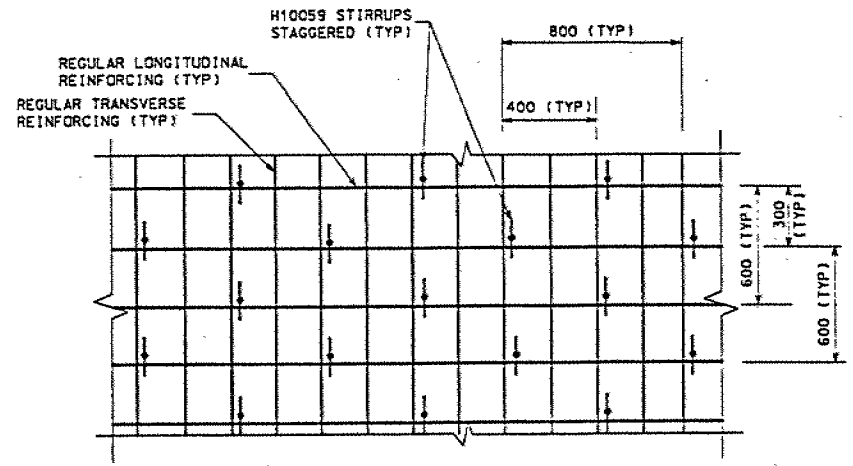
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SECTION 4
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SECTION 5
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CONT NO 93-87
WP NO 268-92-01

SHEET
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- SEGMENTS ARE NAMED FOR REINF. STEEL SCHEDULE PURPOSES ONLY.

PLAN
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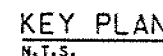
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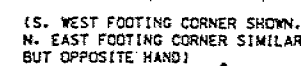
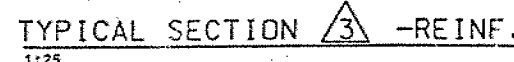
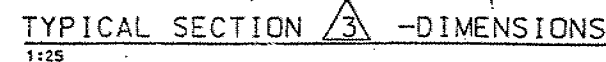
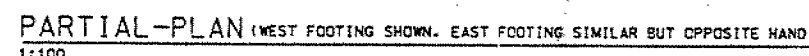
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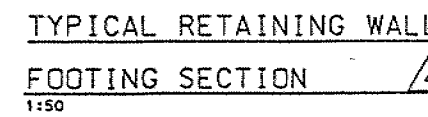
1. ALL BEARING PILES TO BE HP 310 x 110
2. PILE SPACINGS ARE MEASURED FROM UNDERSIDE OF FOOTINGS.
3. PILE LENGTHS SHOWN IN TABLE ARE THEORETICAL LENGTHS BELOW CUT-OFF LEVELS.
4. ULTIMATE DRIVING CAPACITY PER PILE SHALL BE 1600 KN.
5. PILE TIPS MUST BE DRIVEN BELOW ELEV. 183.000 FOR EAST & WEST FOOTINGS.
6. • E.F. DENOTES EACH FACE
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 - O.F. DENOTES OUTSIDE FACE



PARTIAL-PLAN (WEST FOOTING SHOWN. EAST FOOTING SIMILAR BUT OPPOSITE HAND)



DETAIL 
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APPLICABLE STANDARD DRAWINGS
1. CPSD 3301.00



(S. WEST FOOTING CORNER SHOWN.
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ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

CONT 94-55

WP 114-87-00C DIST 4

HWY 403 STR SITE -

Twin Cell Culverts at the
Hwy. 403 - Dunmark Lake East Crossing

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FOUNDATION INVESTIGATION REPORT FOR

Twin Cell Culverts at the Hwy 403 - Dunmark Lake East Crossing

WP 114-87-00C

District 4, Burlington

INTRODUCTION

This report summarizes the results of a foundation investigation conducted in conjunction with the Hwy 403 crossing at the northeastern "leg" of the Dunmark Lake. Twin cell culverts in combination with approach fills have been proposed to carry the highway over the lake. The structure is a component of the new Highway 403 that has been planned between Ancaster and Brantford.

SITE DESCRIPTION AND GEOLOGY

The site is located within and adjacent the northeastern area of the Dunmark Lake (hereafter identified as Dunmark Lake East) which is located between Dunmark Rd. to the south and Jerseyville Rd. to the north. The site, located within the Regional Municipality of Hamilton-Wentworth, is located approximately 1 kilometre east of Sunnyridge Rd.

The Dunmark Lake East is presently situated on private land bounded by agricultural farmland to the north and a recreational golf course (Heron Links) to the south. The lake is confined by tall deciduous tree forest to the north and to the east and west.

The waters of the Dunmark Lake East 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 were present within the northern limits of the lake.

A drainage channel located north of Dunmark Lake East serves to drain the farmer's fields situated in this area. Existing native slopes immediately east and west of the Dunmark Lake East are approximately 16 metres to 10 metres in height respectively. The slopes are benched with gradients of approximately 4 to 5H:1V. The slopes, which are covered with trees show no signs of instability.

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 of approximately 30m.

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 25-27. The borehole (BH5, formerly BH2, W.P. 114-87-00) was advanced to a depth of 33.4m, whilst the dynamic cone penetration test were advanced to depths of 6.1m and 18.3m. The deeper dynamic cone followed 6.1 metres of preaugering. The borehole and dynamic cone penetration test were advanced on land east of the existing Dunmark Lake employing a track mounted CME 55 equivalent drilling unit.

A more detailed investigation was conducted between 92 06 30 and 92 07 17 that included a total of four (4) additional boreholes. A dynamic cone penetration test also accompanied BH2 positioned at the proposed culvert outlet location. The boreholes were advanced to depths ranging from 15.4m to 30.2m. The two (2) boreholes advanced at the culvert inlet and outlet (BH1 and BH2) penetrated the bedrock. The two (2) other boreholes were advanced in conjunction with the proposed west approach embankment fills (BH3 and BH4) and represent the shallower boreholes. The dynamic cone penetration test accompanying BH2 was advanced to a depth of 15.5m.

The water in the Dunmark Lake at the time of the investigation necessitated 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 900 kg. Conventional diamond drilling techniques were used to advance three (3) boreholes offshore. The process involves driving the casing with a 63.5 kg. hammer and washboring within the driven casing. NW casing was driven within the overburden.

The one borehole (BH4) on land west of the proposed culvert structure was advanced using a track mounted CME 75 equivalent drilling unit. Hollow stem augering techniques were used to penetrate the overburden at this location.

Both disturbed and undisturbed samples were retrieved in the overburden at the site generally at 1.5m intervals within the surficial 15 metres or so and at 3.0 metre intervals thereafter. 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 in accordance with procedures outlined in ASTM D1587. Wash samples were also randomly taken during the washboring process.

Rock core was also retrieved at two borehole locations using conventional rock coring techniques. A BXL core barrel within BW casing was used to retrieve up to 1.6 metres of rock core.

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 samplers 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.

Rock core samples were also identified in the field and physical index properties were determined by visual examination and also by measurement of rock quality designations (RQD's) and rock core recovery. All rock core were placed in standard rock core boxes and carefully transported to the laboratory.

In situ vane tests were also carried out to determine the undrained shear strength at selected intervals between the subsoil sample retrieval. The tests were 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 was also monitored 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 East.

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 summarized in Table 1 below.

Table 1 - Physical/Mechanical Property Tests

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.

Detailed rock core logging was conducted in the laboratory by an in-house resident geologist. The rock core logging included descriptions of colour, grain size, bedding, jointing and strength.

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 (approximately 0.7m to 1.0m at the time of the investigation) of Dunmark Lake consists of a cohesive clayey silt to silty clay. The thickness of this deposit ranges from 4.4 metres to 7.6 metres. The stratum has a stiff to very stiff consistency.

The surficial cohesive clayey silt to silty clay deposit is underlain by a plastic silt that has a loose to compact denseness and contains random interbedded layers of stiff to very stiff clayey silt to silty clay. The thickness of this stratum ranges from 9.0m to 12.2m.

The plastic silt stratum is in turn underlain by a second cohesive clayey silt to silty clay deposit which has a thickness ranging from 9.1m to 10.4m. This deposit also contains random layers of plastic silt. The cohesive material has a stiff to very stiff consistency.

A lower silt deposit with random layers of clayey silt to silty clay underlies the lower cohesive clayey silt to silty clay deposit. This deposit has a relatively shallow thickness of 1.5 to 3.2 metres.

Underlying the lower cohesionless silt deposit and overlying the bedrock exists a shallow deposit consisting of a heterogeneous mixture of silt, sand and gravel. This deposit is a glacial till and has a thickness of 1m to 2.9m. This deposit has a compact denseness.

The bedrock surface is uniform across the site and exists at an elevation of 182.6m to 182.8m.

The bedrock is a medium strong dolostone.

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

Water

Approximately 0.7m to 1m of standing water was present in the Dunmark Lake East at the time of the investigation. The water was murky and calm at the time of the investigation.

Clayey Silt to Silty Clay

The surficial stratum at the site consists of a cohesive clayey silt to silty clay that extends for a thickness ranging from 4.4 metres to 7.6 metres below the ground surface. The deposit also contains traces of organics within the surficial three (3) metres. Inorganic silt extending approximately 0.6m in thickness also exists surficially at the site.

The colour of the deposit is primarily brown to mottled grey-brown but changes to grey at a depth as shallow as 3.7 metres at some locations. This varying colour is indicative of different depths of oxidation at the site and possible sedimentation in the Dunmark Lake.

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 primarily of grain sizes smaller than 75 micrometres. The grain size distribution envelope illustrates silt percentages ranging from 47% to 73% and clay percentages ranging from 24% to 53%. In general, the clay fraction is in the order of 24 to 28%. A grain size distribution curve illustrating the gradation of a representative sample of the surficial inorganic silt is also shown on Figure 1. The curve shows a large silt percentage (89%). 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 to Silty Clay

	Range	# of Tests
Natural Moisture Content (w%)	28 - 35	4
Liquid Limit (w_L %)	34 - 49	4
Plastic Limit (w_p %)	19 - 20	4
Plasticity Index (I_p %)	15 - 29	4
Bulk Unit Weight (kN/m^3)	19	2

The test results clearly reveal that the soil has a plasticity ranging from low to intermediate and hence can be categorized as 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. Figure 2 also illustrates that inorganic silt (MH) and organic clay of high plasticity was encountered within the surficial 1 to 2 metres of the deposit. The inorganic silt is perhaps an indication of some sedimentation within the lake and the organic clay is a reflection of the traces of organics in the material.

The consistency and undrained shear strength of the soil were determined by conducting in situ vane tests. In general, the material exhibited sufficient 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 4 blows/0.3m to 19 blows/0.3m. Within the lake, the 'N' values were lower and generally less than 10 blows/0.3m. The larger 'N' values above 10 blows/0.3m were retrieved on land.

The compressibility characteristics of the clayey silt to silty clay stratum were determined by conducting a one dimensional consolidation tests on a representative sample. The results of the test are shown graphically on Figure 3 in the Appendix. The consolidations 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 290 kPa. The effective overburden pressure for the sample tested is approximately 35 kPa. Therefore, the soil has been preconsolidated in the past to an effective pressure approximately 255 kPa in excess of the existing effective overburden pressure. The compression index of the material (C_c) is of small magnitude and equivalent to 0.16.

Silt with random 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 13.9m to 19.8m below the ground or water surface. The thickness of this stratum ranges from 9.0m to 12.2m 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 4 in the Appendix. The envelopes illustrate silt percentages ranging up to 85% with traces of clay for the silt material and for the cohesive layers, silt percentages range from 57% to 75%, but are generally in the 71 to 75% range with a clay fraction ranging from 20 to 43%, but generally in the 20 to 29% range. 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 5 and summarized in Table 3 below. Natural Moisture Contents are also included in the Table below.

Table 3 - Atterberg Limit Test Results

Silt with random layers of
Clayey Silt to Silty Clay

a) Silt

	Range	# of Tests
Natural Moisture Content (w%)	21 - 30	9
Liquid Limit (w_L %)	19 - 23	9
Plastic Limit (w_p %)	17 - 22	9
Plasticity Index (Ip%)	1 - 5	9

b) Clayey Silt to Silty Clay

	Range	# of Tests
Natural Moisture Content (w%)	24 - 35	8
Liquid Limit (w_L %)	23 - 34	8
Plastic Limit (w_p %)	15 - 18	8
Plasticity Index (Ip%)	6 - 21	8

The test results reveal that the main component of the deposit behaves as a plastic silt (ML to CL-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 54 kPa to in excess of 120 kPa. Most of the undrained shear strength values were within the 100 kPa range. These larger undrained shear strength values are attributable to the

significant silt percentages within the deposit. However, it can be concluded that the cohesive interlayers have a stiff to very stiff consistency.

The sensitivity of the soil also determined by the vane test ranged from 3 to 5 indicating a low to moderately sensitive cohesive material.

The 'N' values as derived from the Standard Penetration Test ranged from 5 blows/0.3m to 21 blows/0.3m. Based on these 'N' values it can be stated that the deposit has a loose to compact denseness.

Clayey Silt to Silty Clay with random layers of Silt

Underlying the plastic silt with random layers of clayey silt to silty clay, a second cohesive deposit consisting of clayey silt to silty clay 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 9.1m to 10.4m.

A grain size distribution envelope illustrating the gradation of the cohesive material of this deposit is shown on Figure 6 in the Appendix. The envelope clearly illustrates that the material is fine grained with grain sizes less than 75 micrometres. Clay percentages range from 17% to 53%, but generally between 17% and 38% and silt percentages range from 47% to 83%, but generally between 62% and 83% 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 7 and summarized in Table 4 below. Natural Moisture Contents are also included in the Table below.

Table 4 - Clayey Silt to Silty Clay with
random layers of Silt

	Range	# of Tests
Natural Moisture Content ($w\%$)	25 - 43	7
Liquid Limit ($w_L\%$)	22 - 39	7
Plastic Limit ($w_p\%$)	16 - 19	7
Plasticity Index ($I_p\%$)	6 - 20	7

The test results reveal that the cohesive soil has a low to intermediate plasticity and hence can be classified as a clayey silt to silty clay. 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 were generally equal to or greater than 100 kPa indicating a stiff to very stiff consistency. The 'N' values as determined by the SPT ranged from 10 blows/0.3m to 36 blows/0.3m confirming the stiff to very stiff consistency.

Silt with random layers of Clayey Silt to Silty Clay

A second cohesionless deposit of plastic silt exists beneath the lower cohesive deposit of clayey silt to silty clay. This deposit has a relatively shallow thickness ranging from 1.5m to 3.2m. 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 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 curves and Atterberg Limit Tests were conducted on representative samples of this material and are shown on Figures 8 and 9. The figures reveal that the host silt has large silt percentages (94%) and that the material behaves as an inorganic silt of low plasticity. The interbedded cohesive layer tested revealed a clay fraction of 69% and an intermediate plasticity (CI).

Standard Penetration Tests conducted within this deposit reveal 'N' values ranging from 7 blows/0.3m to 20 blows/0.3m indicating a loose to compact state of denseness.

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

Underlying the lower cohesionless silt deposit with random layers of clayey silt to silty clay and immediately overlying the bedrock, a deposit comprised of a heterogeneous mixture of silt, sand and gravel exists. This deposit is a glacial till and as is inherent of these types of materials, is unsorted and unstratified. The thickness of this deposit is relatively small and ranges from 1 to 2.9 metres.

The main component of this deposit is the silt material. Traces of sand and gravel are also present within the deposit. Although not encountered during the investigation boulders and cobbles are characteristic components of glacial till deposits and hence can exist in the deposit.

Bedrock

The bedrock consisting of a "vuggy" dolostone of the Amabel Formation underlies the heterogeneous mixture of silt, sand and gravel deposit at an elevation of approximately 182.6m to 182.8m. The bedrock was cored in BXL size up to 1.6m in depth.

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

An assessment of the quality and strength of the rock was carried out by measuring core recoveries and Rock Quality Designations (RQD) in the field and physical index property testing. Recoveries were in the order of 92% to 93% and RQD's were in the order of 87% to 88% indicating that the rock is of good quality. Rock strengths can be described as medium strong.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water levels in the open boreholes and monitoring the lake level throughout the duration of the field investigation. The lake level ranged from Elevation 211 metres to 211.4 metres.

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 twin cell reinforced concrete culvert structure that will carry the proposed Hwy 403 over the existing Dunmark Lake. The lake spans approximately ninety (90) metres from shore to shore at the Hwy 403 crossing.

Dwg. 1148700C-A in the Appendix illustrates a plan of the proposed culvert structure. The plan identifies the location of the culvert positioned perpendicular to the Hwy 403. The dimensions of each cell is 4.2m x 3.0m producing a cross sectional area equivalent to 25.2m². The length of the proposed culvert structure is approximately 82m.

The proposed grade elevation for the Hwy 403, which initially will be a four lane median divided highway with ultimate widening plans, is approximately 219 metres. The culvert invert elevation is 210.1m at the inlet and 209.7m at the culvert outlet indicating a bed slope gradient of approximately 0.5%. The lake bed elevation is relatively flat and approximately 210 metres. Beyond the lake, the ground surface elevation increases and slopes to an elevation of approximately 224 metres on the east side and 219 metres on the west side. Therefore, approximately 9 to 10 metres of fill will be placed above the lake bed. The thickness of fill will then diminish to zero in the westerly direction and excavation cuts will be required on the east side. Approximately six (6) metres of fill material will be placed above the roof of the culvert.

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
100	213.7

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

- 1) Structure Foundations
- 2) Erosion Protection at Culvert Inlet/Outlet
- 3) Backfill to Structure
- 4) Approach Embankments
- 5) Construction Considerations

1) STRUCTURE FOUNDATIONS

The surficial native soils at the site are not considered suitable for the support of conventional, economical shallow foundations because of the weaker nature of the material. A closed box culvert type of structure is hence not considered feasible. It is therefore recommended that the culvert be an open footing type of culvert supported on end-bearing deep foundation steel H-piles driven to the bedrock surface. For purposes of the O.H.B.D.C., the deep foundation units can be designed employing the axial capacities tabulated in Table 6 below.

Table 6 - Axial Capacities - Steel H-piles

Pile Type	Factored Capacity at U.L.S (kN)	Axial Capacity at S.L.S (kN)	Estimated Pile Tip Elevation (m)
HP 310 x 110	1600	1150	182.5
HP 310 x 79	1150	890	182.5

Axial capacities provided in Table 6 are for vertical piles only. Reductions 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.

It is recommended that to facilitate the pile driving process, all piles be equipped with reinforced tips. Driving shoe details are given on OPSD 3301.00.

In view of the pile embedment lengths required to reach bedrock, splicing of the piles will be required. Splicing shall be carried out in accordance with OPSS 903.07.01.03 and as illustrated on OPSD 3301.00.

Pile spacing shall conform with Section 6.8.3.10 of the O.H.B.D.C. For centrally loaded piles equal load sharing on the deep foundation units can be assumed. The design of eccentric loaded deep foundation units shall comply with Section 6.8.3.4.2 of the O.H.B.D.C.

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. Pertinent unfactored soil parameters to facilitate the design of vertical piles is given in Table 7 below.

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

Table 7 - Horizontal Capacity Soil Parameters

Soil	Elevation (m)	Shear Strength Parameters		Bulk Unit Weight (kN/m ³) γ
		Undrained Shear Strength (Cu) (kPa)	Angle of Internal Friction ϕ	
Clayey Silt to Silty Clay	209 - 206.5	100		19
Silt with random layers of Clayey Silt/Silty Clay	206.5 - 196.5		30°	20
Clayey Silt to Silty Clay with random layers of Silt	196.5 - 186	100		19
Silt with random layers of Clayey Silt/Silty Clay	186 - 184		30°	20
Heterogeneous mixture of silt, sand and gravel	184 - 182.5		30°	20

2) EROSION PROTECTION AT CULVERT INLET AND OUTLET

A smooth transition should be provided at the inlet and outlet of each culvert to minimize the potential for erosion caused by the scouring forces of the creek water. Aprons and rip-rap should be constructed to provide the necessary erosional resistance. Rip-rap shall be placed over the entire stream bed area in accordance with the hydrology and hydraulic requirements. These requirements include the rip-rap gradation and the height of rip-rap coverage.

To inhibit flow within the backfill adjacent to the culvert wall, it is recommended that headwalls be constructed in conjunction with the rip-rap protection. Alternatively, impervious clay liners can also be used at the culvert inlet as a sealer behind the rip-rap. The impervious clay blanket shall be 1 metre thick and consist of material as specified in OPSS 1205.

3) BACKFILL TO STRUCTURE

Material

It is recommended that Granular 'A', Granular 'B' or crushed rock be used behind the culvert structure walls placed as shown on OPSD 800 series. The application of granular material or crushed rock combined with weep holes in the culvert walls to drain any accumulation of water in the backfill will prevent hydrostatic pressure build-up. Design parameters of the soil/rock are given in Table 8 below. Computations of lateral earth/rock fill pressure shall be in accordance with Section 6-6.1.2 of the O.H.B.D.C.

Table 8 - Backfill Properties

	Granular 'A'	Granular 'B'	Crushed Rock
Angle of Internal Friction (ϕ) (unfactored)	35°	30°	35°
Unit Weight (kN/m ³), γ	22.8	21.2	18
*Coefficient of Active Earth Pressure (Ka)			
- S.L.S	0.27	0.33	0.27
- U.L.S	0.33	0.40	0.33
*Coefficient of Earth Pressure at Rest (Ko)			
- S.L.S	0.43	0.50	0.43
- U.L.S	0.50	0.58	0.50

*These earth pressure coefficients apply to horizontal backfill surfaces only. The appropriate consideration shall be given to account for sloping backfill. The coefficient of earth pressure at rest shall be applied for rigid and unyielding walls.

Backfilling and Compaction

In the placement of the backfill material, the backfill shall be placed simultaneously behind both sides of the structure walls and at no time shall the difference in elevation be greater than 500mm. The backfill shall be constructed in 300mm lifts in accordance with OPSS 902 series and applicable OPSD 800 series. Granular backfill shall be compacted to achieve the target maximum dry density as outlined in OPSS 501.070.08.

Heavy vibratory equipment should be avoided in the backfill construction adjacent to the structure. It is therefore recommended that hand compaction equipment be employed in backfilling the sides of the culvert within a lateral distance equal to the current height of fill above the wall footing, in order to minimize deflection or possible damage of the wall. Special care must be exercised in the placement of any rock fill behind the culvert wall. End dumping of rock backfill against a structure is not permitted as outlined in OPSS 206.07.09.

3) APPROACH EMBANKMENTS

General

The site is located within a depression area inundated with water. As mentioned earlier, up to approximately ten (10) 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 212m and 213.7m respectively. Therefore, embankment material, geometry and drainage must appropriately be designed to prevent loss of material and subsequent imminent embankment failure.

The embankment design considerations and recommendations are discussed below. Construction of the embankment is also discussed.

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's method is described in a paper entitled "Stability Analysis of Embankments and Slopes", Sarma, S.K. (1973), Geotechnique 23, No. 3, pp 423-433.

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

Figure 10 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 11 illustrates a 1.5m 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.

1.0 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 2.5H:1V slopes.

2.0 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 1.2 above applies and hence the slopes shall be constructed at 2H:1V slopes with a two (2) metre midheight berm or alternatively at 2.5H:1V slopes. Should rock fill be placed such that the earth embankment height is less than eight (8) metres, then the earth fills can be constructed at 2H:1V slopes and the rock fills at 1.5H:1V.

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.

Consideration can also be given to employing a cohesive predominantly 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 high water elevation. 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 11 and as discussed below. Each option shall be evaluated based on technical merit and cost. The option that proves to be the most economical and technically feasible 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 this 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.

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 be addressed. Silt curtains may be required.

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 on 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 120mm 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 75mm to 100mm of settlement will occur within a ten (10) metre earth fill 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 ten (10) metre embankment consisting of a combination earth and rock fill approximately 75mm to 100mm 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.

Any additional vertical shearing forces created by the relative movements of the embankment fill and the culvert foundation must be considered in deducing the applied loadings.

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.

CONSTRUCTION CONSIDERATIONS

Interlocking Steel Sheet Pile Wall (Cofferdam Construction)

In view of the Dunmark Lake waters, it is recommended that the construction of the twin cell culvert take place within an enclosure formed by an interlocking steel sheet pile wall. In order to prevent basal heave at the base of the excavation, it is recommended that the sheeting be driven to a depth below the base of the excavation equivalent to the unbalanced hydrostatic head above this level (see Figure 12 in the Appendix). Once the enclosure is formed, water can be discharged in an environmentally accepted manner using conventional pumping methods and the culvert construction can proceed in the dry. Sheet pile materials and installation shall conform to applicable specifications described in OPSS 903 series. The construction procedure shall incorporate the requirements as specified in OPSS 902.07.04.

Temporary Slopes

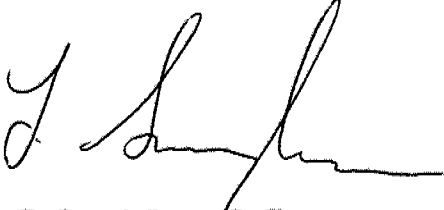
Temporary earth excavation slopes within the surficial clayey silt to silty clay shall not be steeper than 1.5H:1V.

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

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

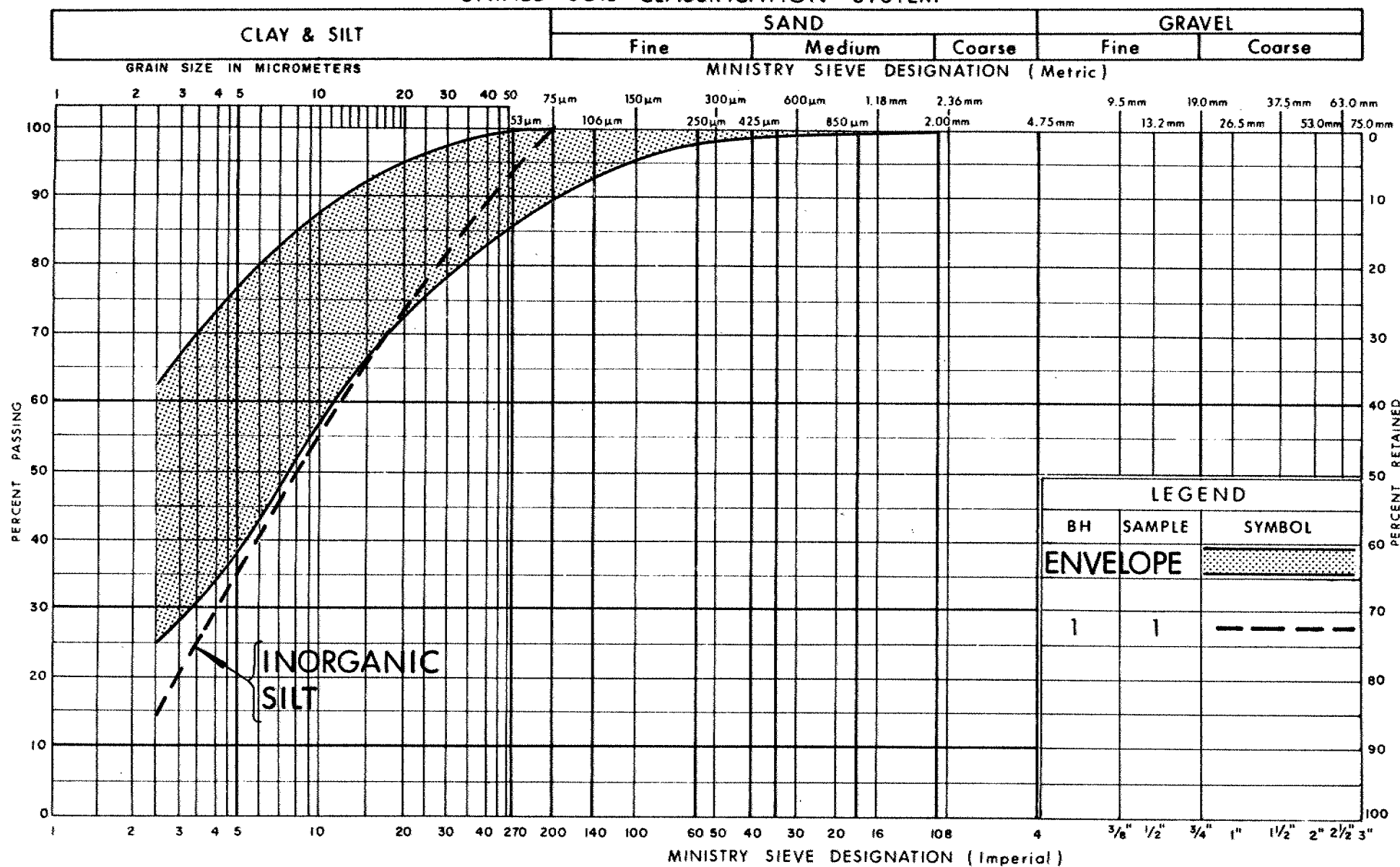



T. Sangiuliano, P. Eng
Foundation Engineer


M. Devata, P. Eng.
Chief Foundation Engineer

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM

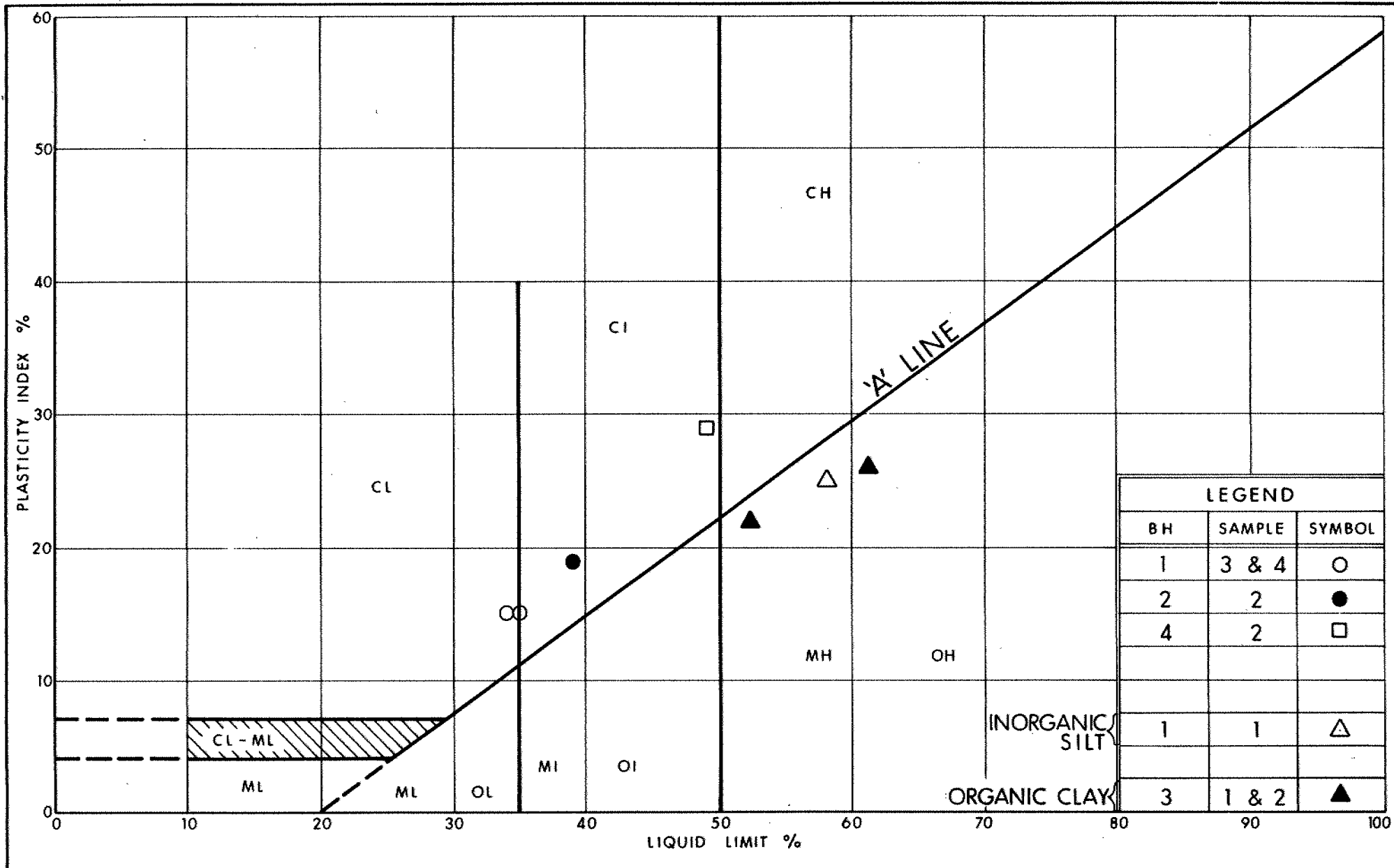


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GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY

FIG No 1

W P 114-87-00 C



Ministry of
Transportation

PLASTICITY CHART CLAYEY SILT TO SILTY CLAY

FIG No 2

W P 114 - 87-00 C

VOID RATIO - PRESSURE CURVES

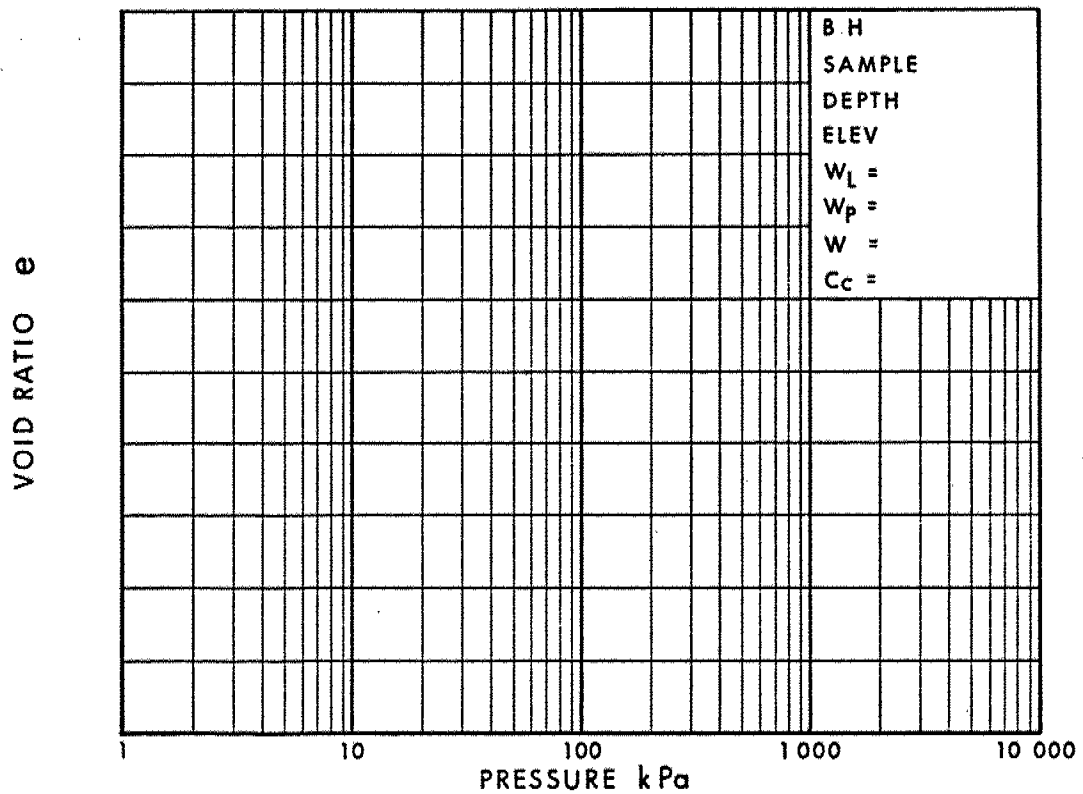
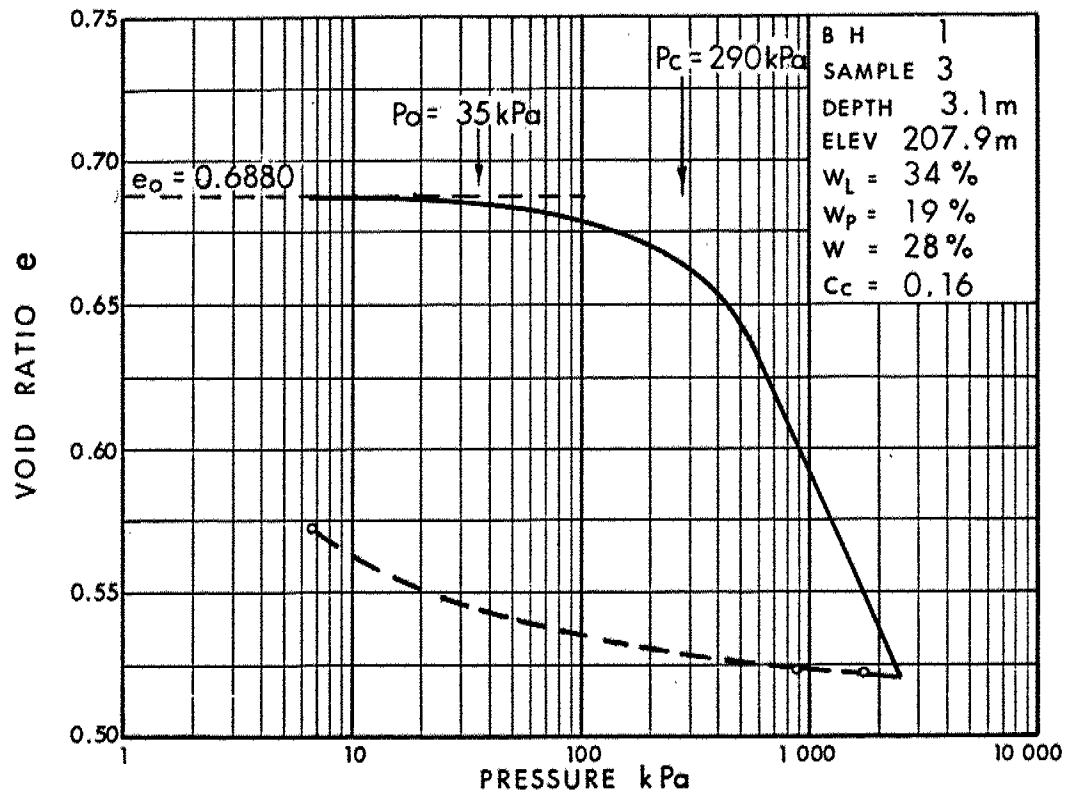
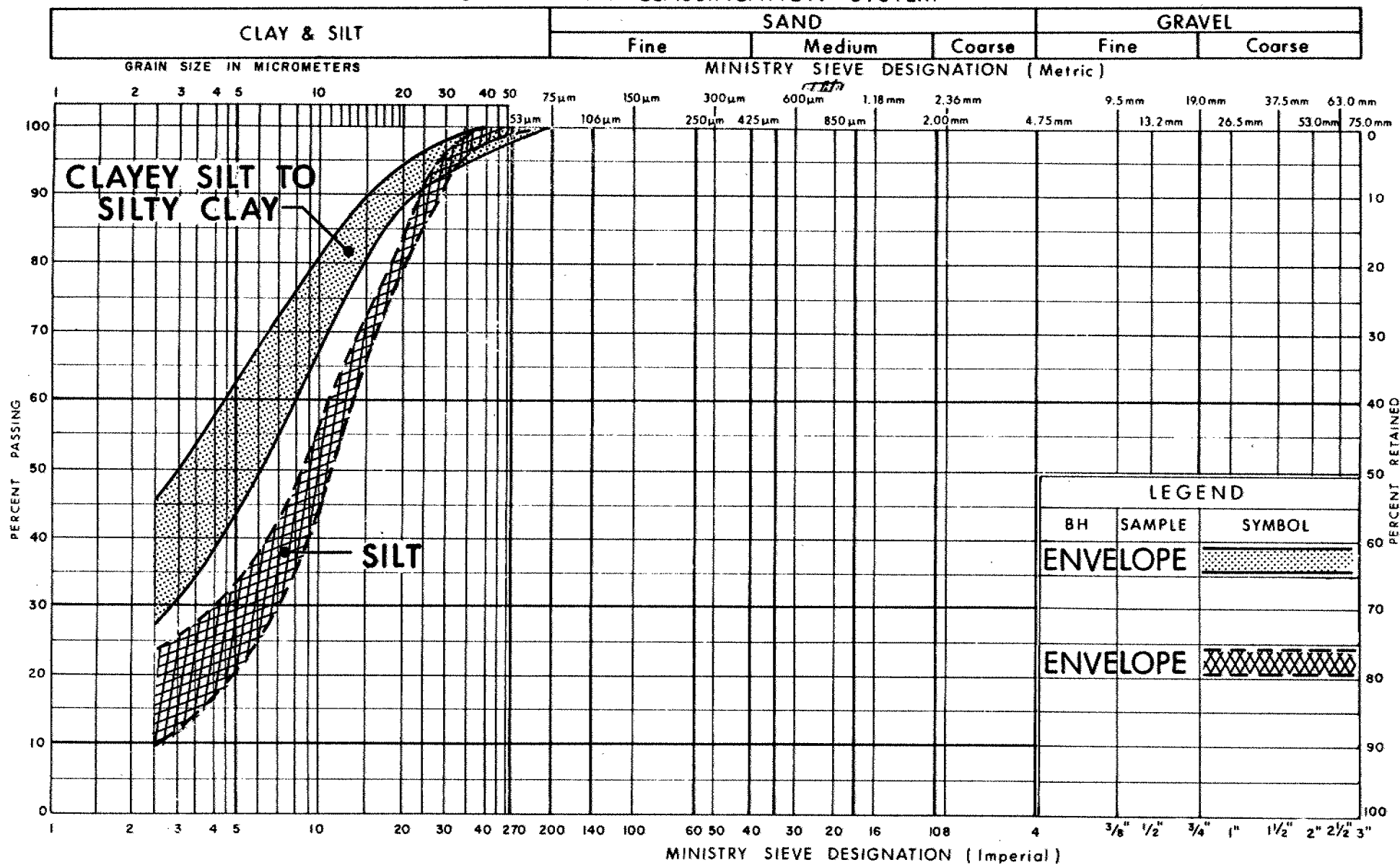


Fig 3

W P 114-87-00C

UNIFIED SOIL CLASSIFICATION SYSTEM

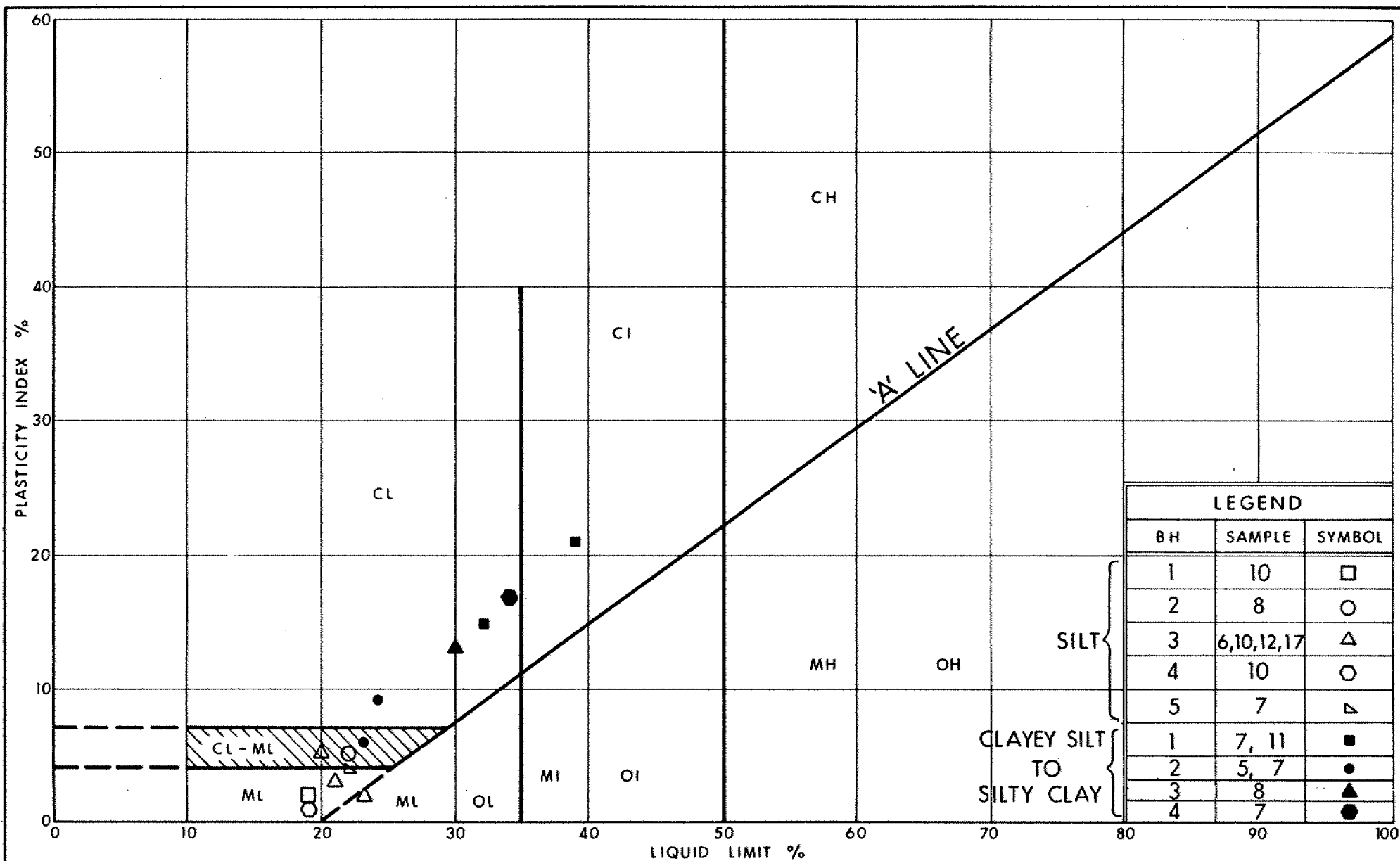


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GRAIN SIZE DISTRIBUTION
SILT, WITH RANDOM LAYERS OF
CLAYEY SILT TO SILTY CLAY

FIG No 4

W P 114-87-00 C



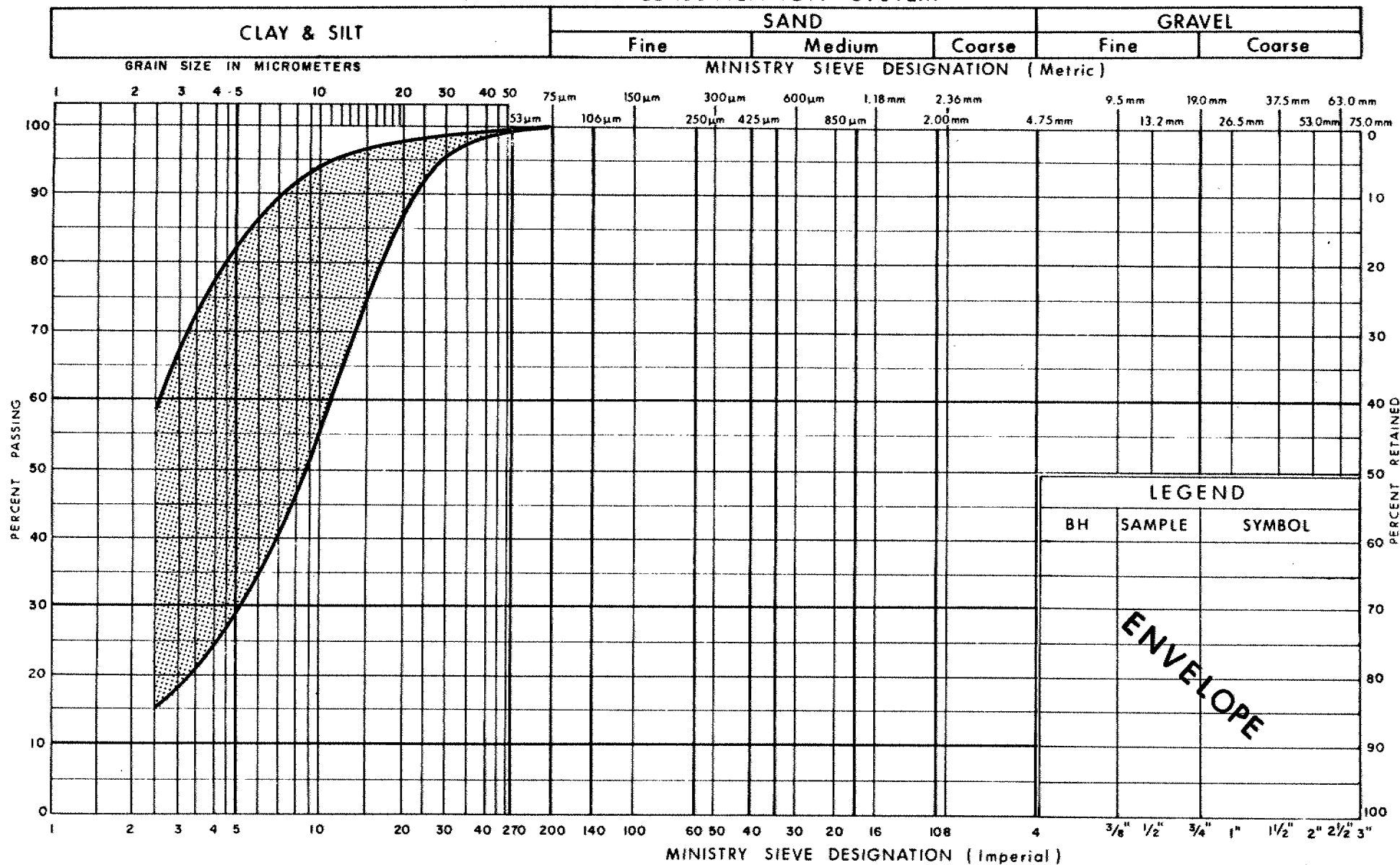
Ministry of
Transportation

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

FIG No 5

W P 114-87-00 C

UNIFIED SOIL CLASSIFICATION SYSTEM

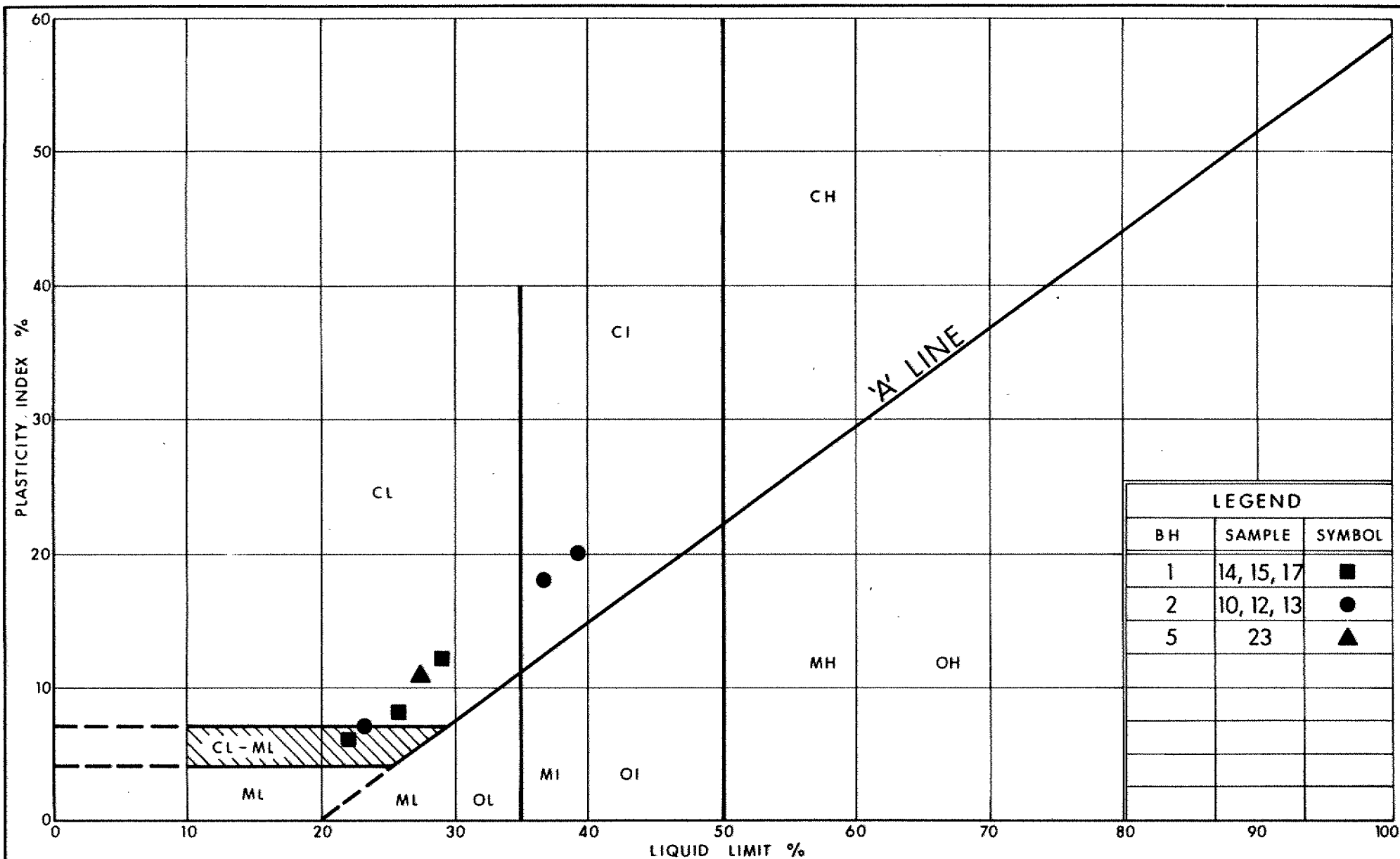


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**GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY
WITH RANDOM LAYERS OF SILT**

FIG No 6

W P 114-87-00 C



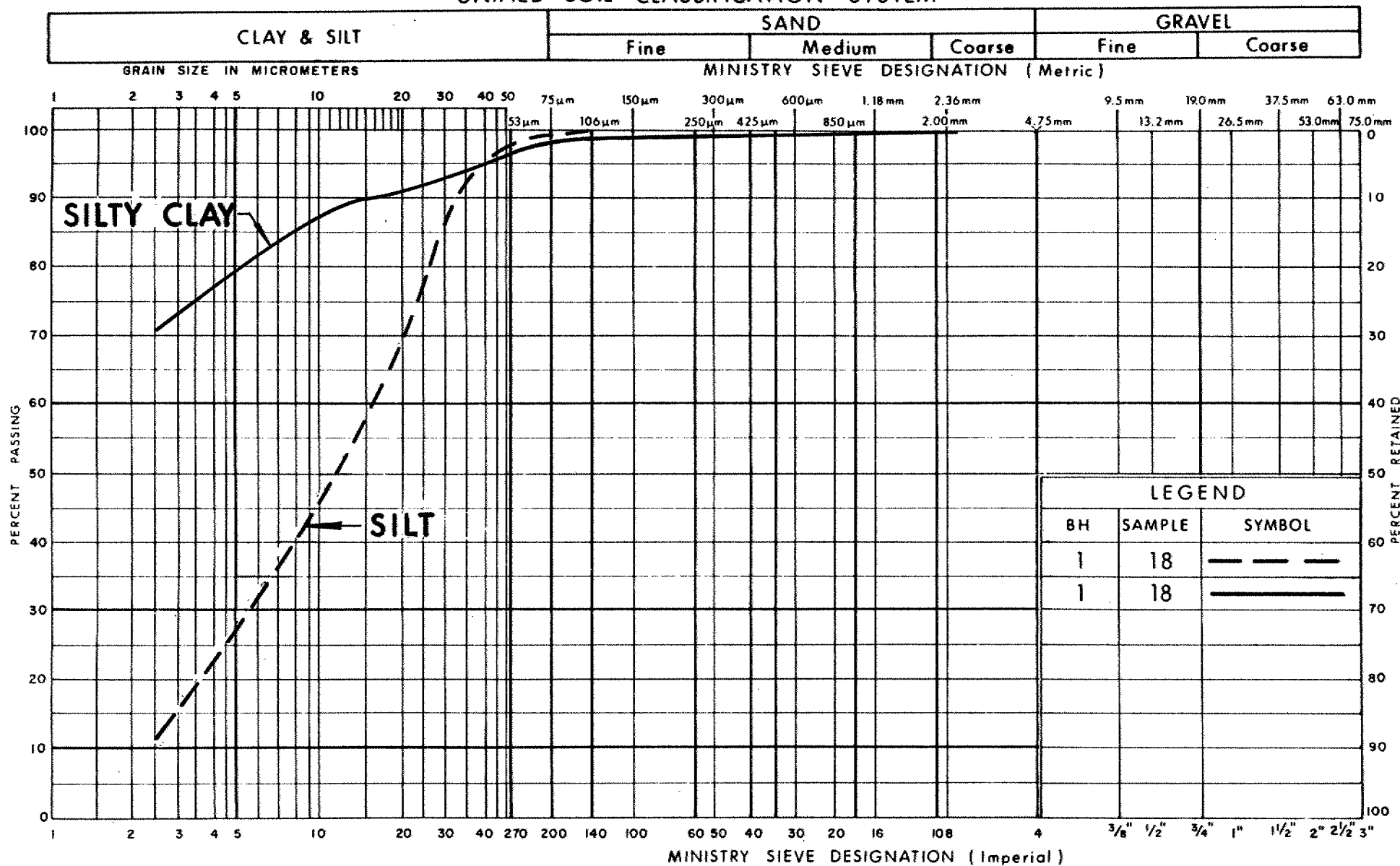
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**PLASTICITY CHART
CLAYEY SILT TO SILTY CLAY
WITH RANDOM LAYERS OF SILT**

FIG No 7

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UNIFIED SOIL CLASSIFICATION SYSTEM

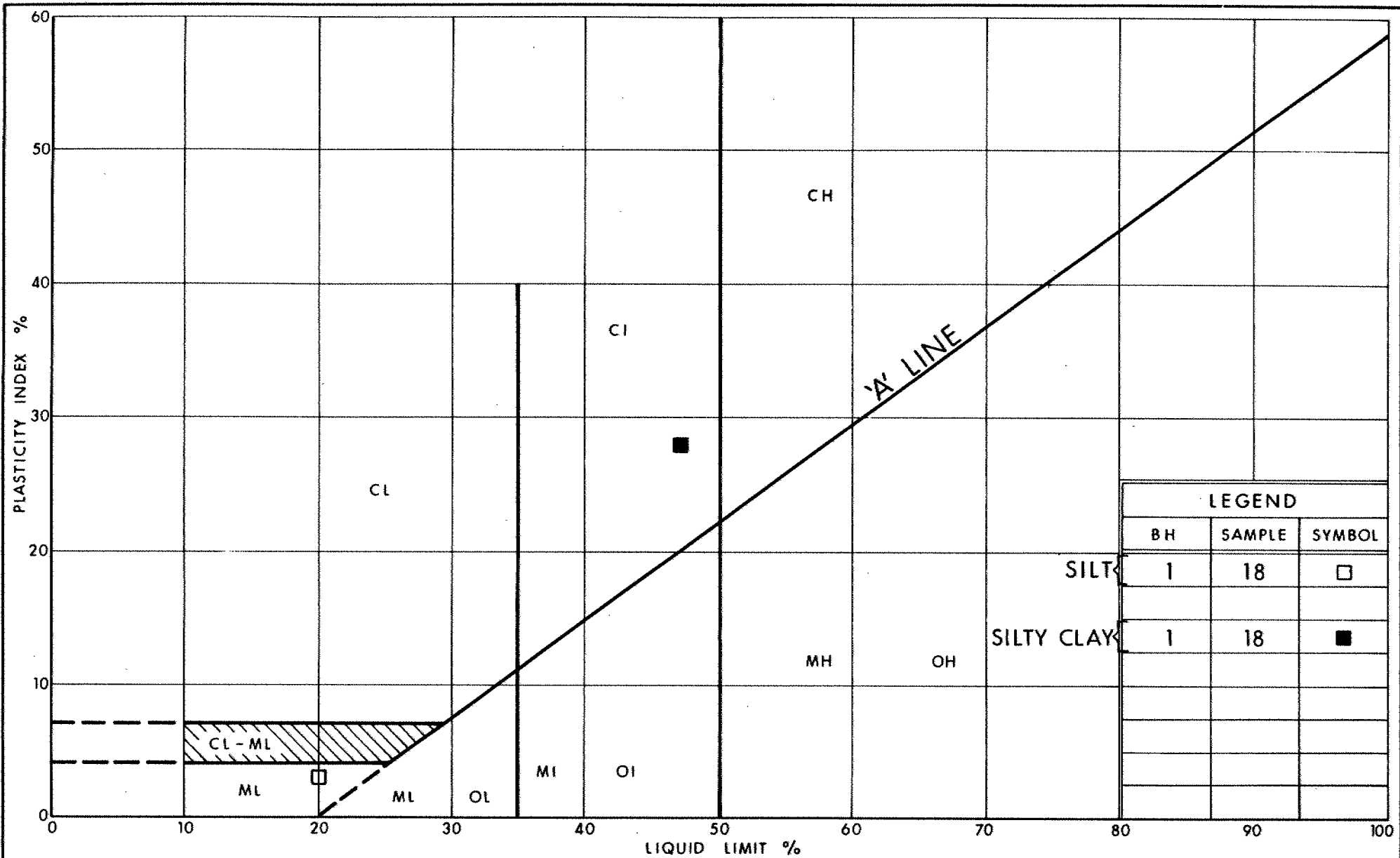


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GRAIN SIZE DISTRIBUTION
SILT, WITH RANDOM LAYERS OF
CLAYEY SILT TO SILTY CLAY

FIG No 8

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Ontario

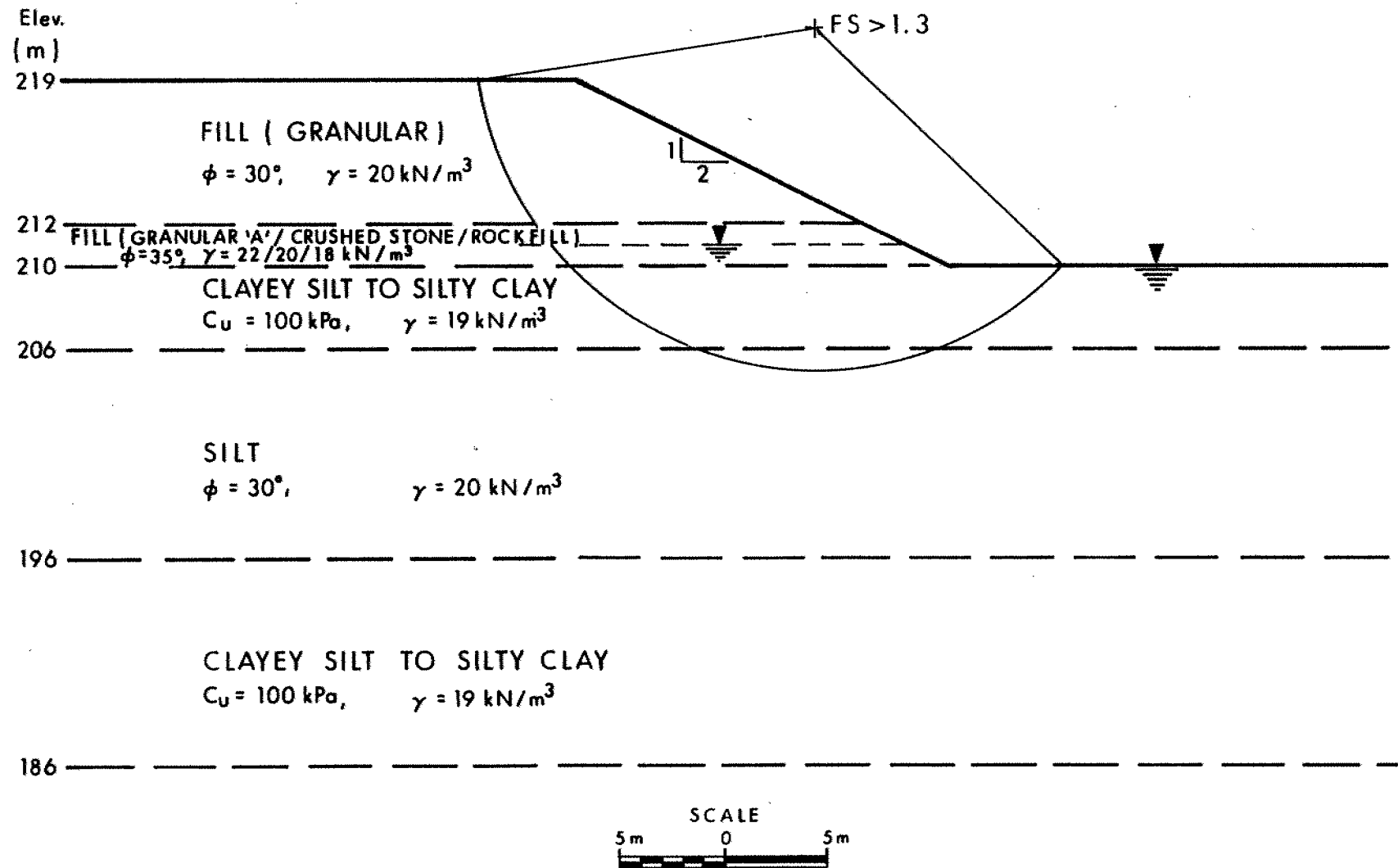
PLASTICITY CHART SILT, WITH RANDOM LAYERS OF CLAYEY SILT TO SILTY CLAY

FIG No 9

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Figure 10 - Slope Stability Analyses

(HWY 403 & DUNMARK LAKE EAST CROSSING)

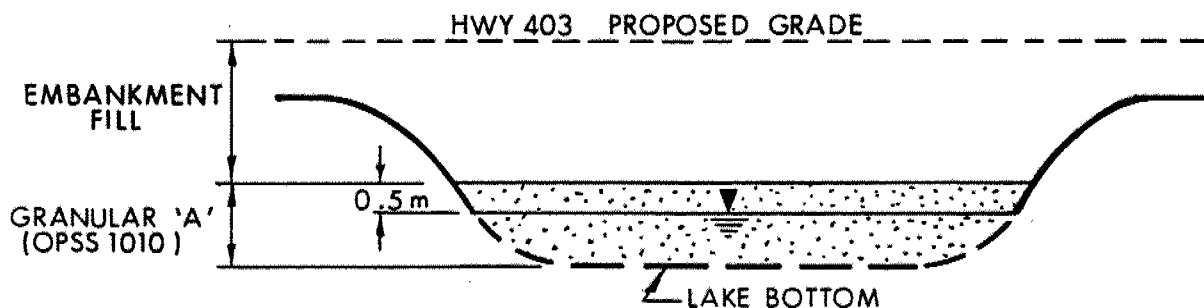


WP 114 - 87 - 00 C

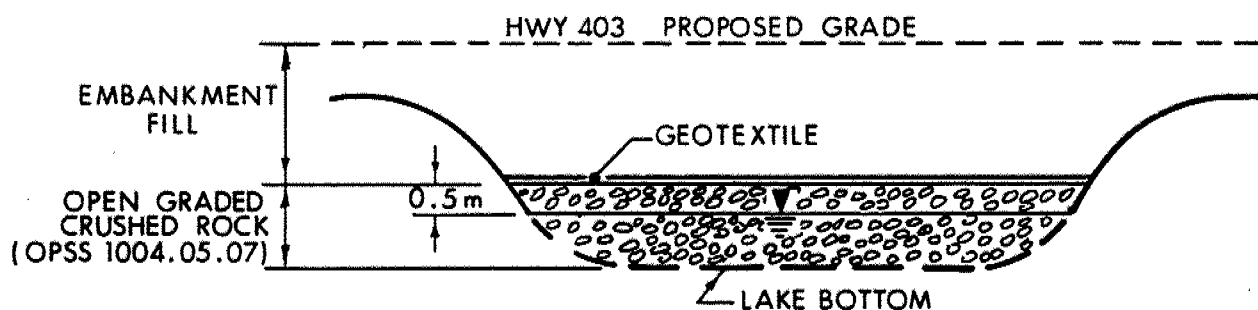
Figure 11- Embankment Material / Construction

(HWY 403 & DUNMARK LAKE EAST CROSSING)

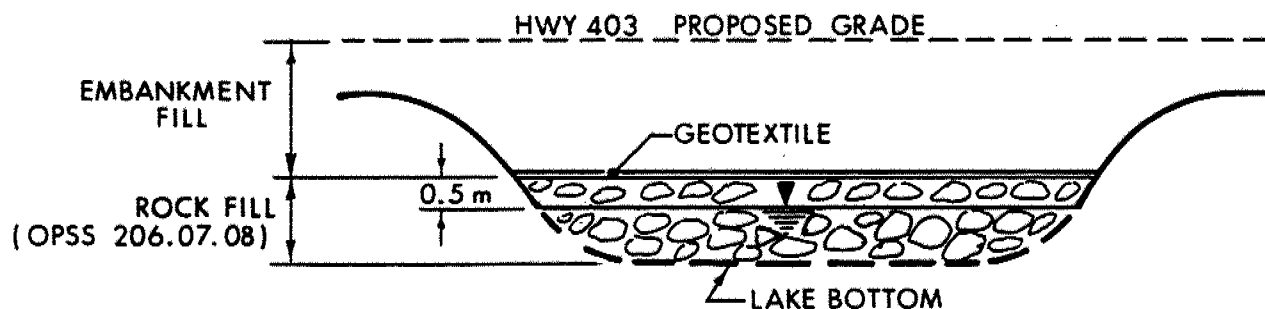
OPTION 1 - GRANULAR 'A'



OPTION 2 - OPEN GRADED CRUSHED ROCK



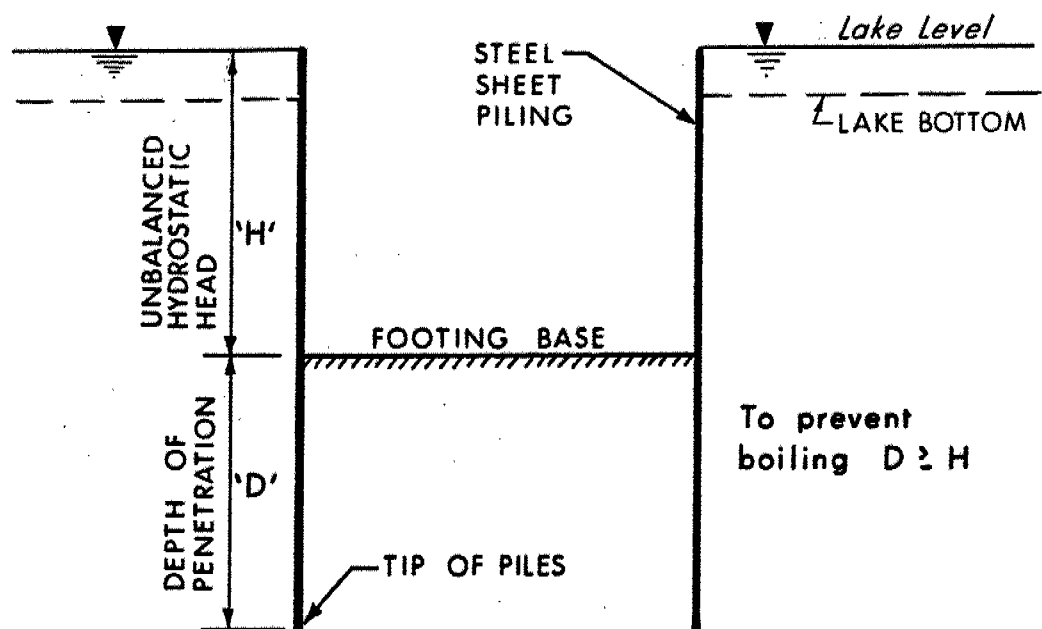
OPTION 3 - ROCK FILL



LEGEND:

 Lake Water Level

NOT TO SCALE



COFFERDAM CONSTRUCTION

W P 114-87-00 C
FIG 12

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
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	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No 1

1 OF 1 METRIC

W.P. 114-87-00C LOCATION Co-ords: N 4 783 775.8 E 256 510.8 ORIGINATED BY PM
 DIST 4 HWY 403 BOREHOLE TYPE HS Auger, NW/BW Casing, Washbore, BXL Rock Core COMPILED BY TS
 DATUM Geodetic DATE 92 07 09-15 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
211.0	Water Surface																
0.0 210.3	Water																
0.7			1	SS	5		210									Org. 4.6%	0 0 89 11
	----- Organic		2	TW	PM												
	----- Grey																
	Clayey Silt to Silty Clay																
	Stiff to Very Stiff																
	----- Brown		3	TW	PM		208									Org. 0.6%	0 12 65 23
	----- Grey		4	SS	6												0 5 67 28
206.4			5	SS	12		206										
4.6			6	TW	PH												
	Silt		7	SS	9		204										6 1 73 20
	with random layers of		8	TW	PM												0 0 75 25
	Clayey Silt to Silty Clay		9	SS	15		202										
	Grey, Loose to Compact with		10	SS	10												
	Stiff to Very Stiff Layers																
			11	SS	10		200										
			12	SS	9												
			13	WS	-		198										
187.0			14	SS	14												0 0 83 17
14.0			15	SS	14		196										0 0 80 20
	Clayey Silt to Silty Clay																
	with random layers of																
	Silt																
	Grey, Stiff to Very Stiff		16	SS	17		194										
							192										
							190										
			17	SS	12		188										
186.6																	
24.4	Silt		18	SS	14		186										0 0 94 6
	with random layers of																0 2 30 68
	Clayey Silt to Silty Clay																
183.6	Grey, Compact						184										
27.4	Het. Mixt. of Silt, Sand and		19	SS	28												
182.6	Gravel (Glacial Till), Compact																
28.4	Bedrock - Dolostone		20	RC	REC		182										
181.1	Light Grey, Unweathered				93%												RQD = 88%
	Medium Strong																
29.9	End of Borehole																

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 114-87-00C LOCATION Co-ords: N 4 783 696.2 E 256 522.6 ORIGINATED BY PM
DIST 4 HWY 403 BOREHOLE TYPE NW/BW Casing, Washbore, BXL Rock Core & Dynamic Cone Test COMPILED BY TS
DATUM Geodetic DATE 92 06 30 - 92 07 03 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W _P W W _L	10 20 30			
211.3	Water Surface													
0.0	Water													
210.3														
1.0	Clayey Silt to Silty Clay Stiff to Very Stiff Grey, Organic Brown		1	SS	4		210					19.0	0 6 69 25	
			2	SS	9		208							
			3	SS	5									
206.4			4	SS	12		206						0 0 86 14	
4.9														
	Silt with random layers of Clayey Silt to Silty Clay Grey, Loose to Compact with Stiff to Very Stiff Layers		5	SS	9		204						0 0 75 25	
			6	SS	15		202							
			7	SS	10		200						0 0 71 29	
			8	SS	11								0 0 77 23	
			9	SS	9		198							
197.4			10	SS	8		196						0 0 82 18	
13.9														
	Clayey Silt to Silty Clay with random layers of Silt Grey, Stiff to Very Stiff		11	SS	17		194						0 0 62 38	
			12	SS	10		192						0 0 47 53	
			13	SS	36		190							
			14	SS	36		188							
188.3			15	SS	12		186							
23.0														
	Silt with random layers of Clayey Silt to Silty Clay Compact		16	SS	20		184							
185.1														
26.2	Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till) Grey, Compact		17	WS	-		182							
182.7			18	SS	-									
28.6	Bedrock - Dolostone Grey, Unweathered Medium Strong		19	RC	REC 92%									
181.1														

* Sampler Bouncing

+3, x5: Numbers refer to
Sensitivity
20
15-5 (%) STRAIN AT FAILURE
10

RQD = 87%

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 114-B7-00C LOCATION Co-ords: N 4 783 749.8 E 256 483.6 ORIGINATED BY PM
 DIST 4 HWY 403 BOREHOLE TYPE NW/BW Casing, Washbore COMPILED BY TS
 DATUM Geodetic DATE 92 07 16-17 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
211.4	Water Surface																
0.0	Water																
1.0	Clayey Silt to Silty Clay Brown, Stiff to Very Stiff Trace Organics		1	SS	4		210									Org 5.0%	
			2	SS	6											3.6%	0 2 73 25
			3	WS	-												
			4	SS	9		208										
206.7			5	WS	-												
4.7	Brown Grey Silt with random layers of Clayey Silt to Silty Clay Loose to Compact with Stiff to Very Stiff Layers		6	SS	9		206										
			7	WS	-												
			8	SS	7												0 0 57 43
			9	WS	-		204										
			10	SS	12												
			11	WS	-												
			12	SS	21		202										0 0 84 16
			13	WS	-												
			14	SS	18		200										
			15	WS	-												
			16	SS	20												
197.5							198										
13.9	Clayey Silt to Silty Clay with random layers of Silt Grey, Stiff to Very Stiff		17	SS	11												0 0 85 15
195.7			18	SS	18		196										
15.7	End of Borehole																

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 114-87-00C LOCATION Co-ords: N 4 783 697.1 E 256 469.2 ORIGINATED BY LD
DIST 4 HWY 403 BOREHOLE TYPE HS Auger COMPILED BY TS
DATUM Geodetic DATE 92 07 13 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
215.5	Ground Surface																
0.0	Clayey Silt to Silty Clay Brown, Very Stiff		1	SS	10	*	214										
	trace Organics		2	SS	9		212										0 0 47 53
211.1			3	SS	11		210										
4.4	Brown Grey		4	SS	16		208										0 0 92 8
	Silt with random layers of Clayey Silt to Silty Clay		5	SS	9		206										
	Loose to Compact with Silt to Very Stiff Layers		6	SS	6		204										0 0 85 15
			7	SS	6		202										
			8	SS	7												
			9	SS	7												
200.1			10	SS	5												0 0 88 12
15.4	End of Borehole * GWL Not Established																

RECORD OF BOREHOLE No 5

1 OF 2

METRIC

(Formerly BH2, W.P. 114-87-00)

W.P. 114-87-00C LOCATION Co-ords: N 4 783 740.1 E 256 588.1 ORIGINATED BY MP
DIST 4 HWY 403 BOREHOLE TYPE HS Auger & Dynamic Cone Test COMPILED BY TS
DATUM Geodetic DATE 91 02 25-27 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT		UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W _P	W		
216.2	Ground Surface						216						
0.0	Trace Organics		1	SS	6		216						
			2	SS	14		214						
	Clayey Silt to Silty Clay		3	TW	PH		214						
	Very Stiff		4	SS	11		212						
			5	SS	17		212						
	Brown		6	SS	19		210						
	Grey		7	TW	PH		210						
208.6			8	SS	8		208					19.8	0 0 92 8
7.6			9	SS	7		208						
	Silt		10	SS	8		206						
	with random layers of		11	SS	10		206						
	Clayey Silt to Silty Clay		12	SS	6		204						
	Loose to Compact with		13	SS	10		204						
	Very Stiff Layers		14	SS	7		202						
			15	SS	9		202						
			16	TW	PH		200					19.9	
			17	SS	6		200						
196.4			18	SS	11		198						
19.8			19	SS	14		198						
	Clayey Silt to Silty Clay		20	SS	18		196						
	with random layers of		21	SS	11		196						
	Silt		22	SS	13		194						
	Grey, Very Stiff		23	TW	PH		194						
			24	SS	7		192						
187.2							192						
29.0							190						
185.7	Silt						188					19.9	0 0 67 33
	with random layers of						188						
	Clayey Silt to Silty Clay, Loose						186						

30.5 Continued

+3, x5: Numbers refer to
Sensitivity 20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 5

2 OF 2

METRIC

(Formerly BH2, W.P. 114-87-00)

W.P. 114-87-00C LOCATION Co-ords: N 4 783 740.1 E 256 588.1 ORIGINATED BY MP
 DIST 4 HWY 403 BOREHOLE TYPE HS Auger & Dynamic Cone Test COMPILED BY TS
 DATUM Geodetic DATE 91 02 25-27 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
185.7	Continued																
30.5	Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till)		25	SS	27												
			26	SS	22		184										
182.8	Grey, Compact				**												
33.4	End of Borehole • 91 02 28 ** Sampler Bouncing (Probable Bedrock)																

ROCK CORE DESCRIPTION **WP 114-87-00C**

Page 1 of 1

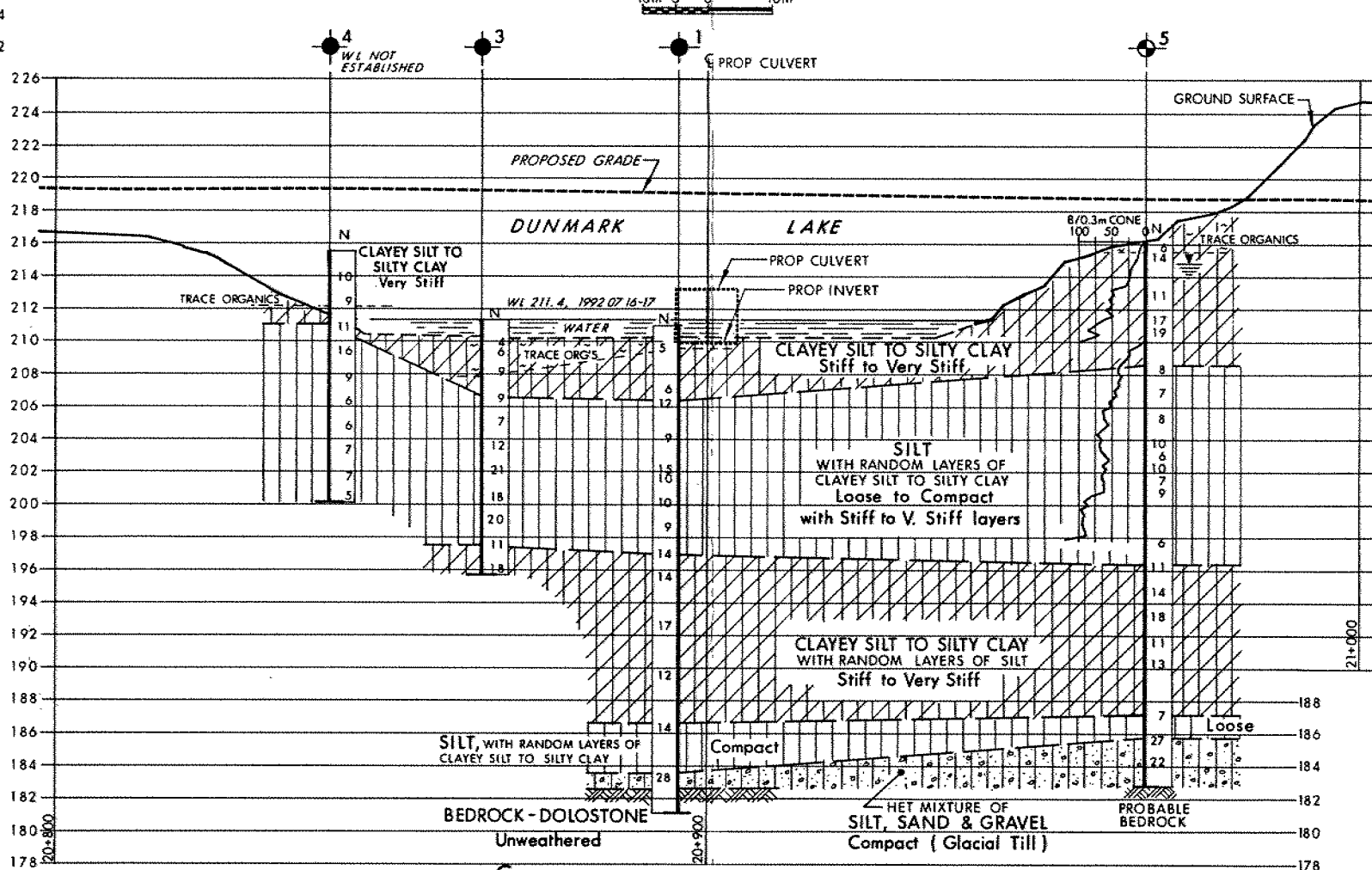
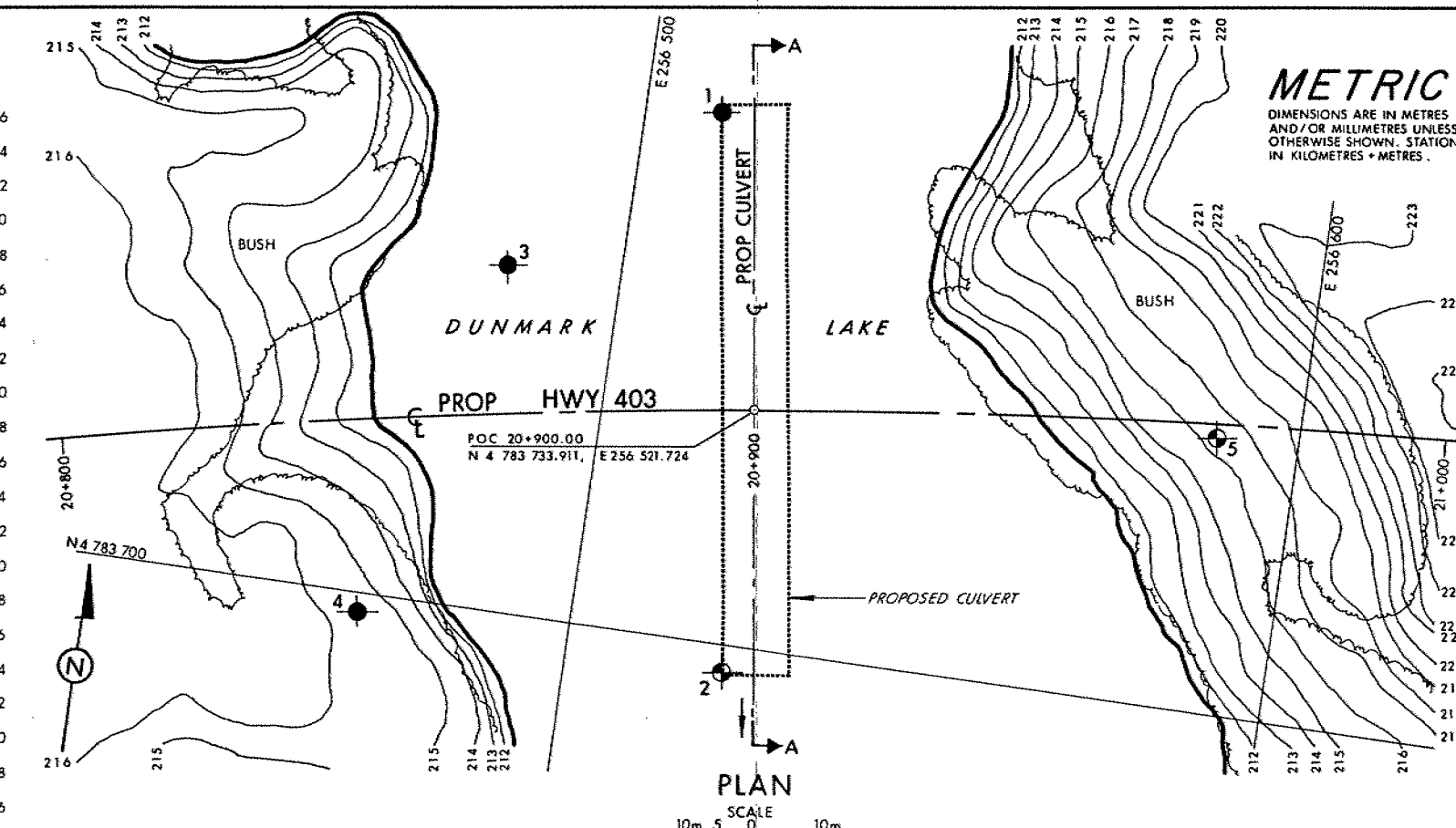
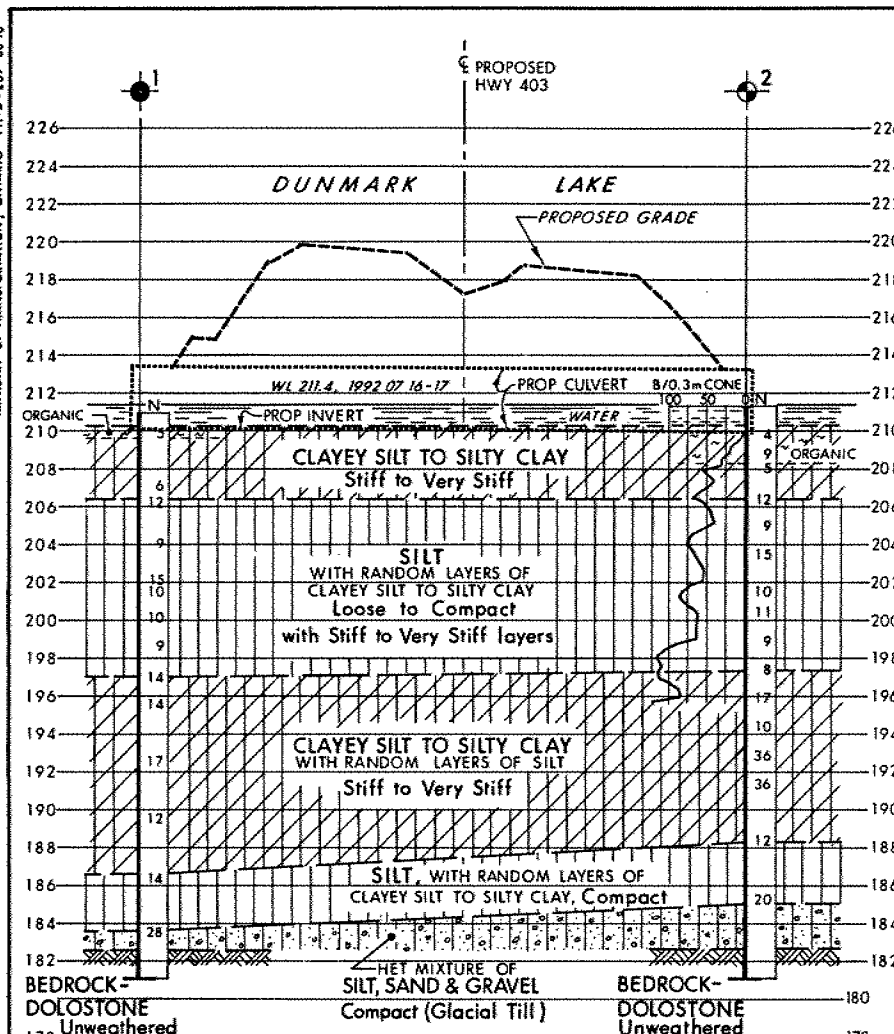
CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1	20	28.40-29.92	93	88	28.40-29.92	DOLOSTONE with stylolites and abundant vugs containing calcite crystals, light grey to medium dark grey; fine to medium grained; medium strong; unweathered to slightly weathered; fractures wide to very close spaced, flat to dipping, undulating to planar, smooth to rough.
2	19	28.63-30.23	92	87	28.63-30.23	DOLOSTONE with stylolites and abundant vugs containing calcite crystals, light grey to medium dark grey; fine to medium grained; medium strong; unweathered to slightly weathered; fractures moderate to close spaced, flat, undulating to planar, smooth to rough.

*CR = CORE RECOVERY

*RQD = ROCK QUALITY DESIGNATION

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

Logged by: DAW, Soils and Aggregates Section



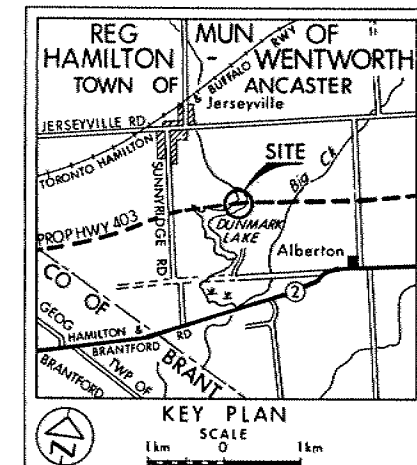
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 114-87-00 C

PROPOSED CULVERT
DUNMARK LAKE EAST CROSSING

BORE HOLE LOCATIONS & SOIL STRATA

SHEET



- LEGEND**
- Bore Hole
 - Dynamic Cone Penetration Test (Cone)
 - Bore Hole & Cone
 - N Blows/0.3m (Std Pen Test, 475 J/blow)
 - CONE Blows/0.3m (60° Cone, 475 J/blow)
 - WL at time of investigation
1991 02, 1992 06 and 07

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	211.0	4783 775.8	256 510.8
2	211.3	4783 696.2	256 522.6
3	211.4	4783 749.8	256 483.6
4	215.5	4783 697.1	256 469.2
5	216.2	4783 740.1	256 588.1

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

Geocres No 40P1-89

HWY No 403

SUBMD T5 CHECKED DATE 1993 01 27 SITE

DRAWN R5 CHECKED APPROVED TS DWG 1148700 C-A