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FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

**ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION**

WP 16-96-00 REGION Central Region
HWY 6 STR SITE 36-301C

Proposed Culvert Extensions
Highway 6 and Twenty Mile Creek

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FOUNDATION INVESTIGATION REPORT

FOR

Proposed Culvert Extensions

Highway 6/Twenty Road and Twenty Mile Creek

WP 16-96-00

Central Region

1.0 INTRODUCTION

This report summarizes the results of a foundation investigation conducted for proposed extensions to two culverts at the intersection of Highway 6 and Twenty Road. The existing culverts transmit the waters of Twenty Mile Creek beneath Twenty Road and Highway 6. The investigation was carried out at the request of the Central Region Structural Section. This report is applicable to the limits of the proposed culvert extensions and adjoining retaining structures and the immediate approaches (up to 25m) beyond the culvert structures.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 Site Description

The site is located at the intersection of Twenty Road and Highway 6, within the Township of Glanbrook, Regional Municipality of Hamilton-Wentworth. Two separate culverts transmit the waters of Twenty Mile Creek beneath Twenty Road and Highway 6, respectively. The first culvert spans Twenty Mile Creek at Twenty Road, approximately 36 m west of Highway 6 centre line (Culvert #1). The second culvert spans Twenty Mile Creek at Hwy 6, just south of Twenty Road (Culvert #2). The proposed retaining wall is to be located on the east end of Culvert # 2.

Land use in the area is a combination of industrial, agricultural and residential. A detailed description of each site is given in this section of the report.

Culvert #1 - Twenty Mile Creek at Twenty Road

The existing culvert, approximately 8.2 metres in length, channels south flowing Twenty Mile Creek under Twenty Road. The culvert has an arched roof and is believed to be of the open footing type (there are no plans available for the existing structure or foundations). The culvert has a span of approximately 5.5 metres and a height of 1.9 metres (from the creek bed to the top of the arch). The concrete walls are approximately 0.4 metres in thickness and the roof thickness is approximately 0.2 metres (at the top of the arch).

At the north end of the culvert, the creek has a width of approximately 5.5 to 9 metres that meanders through a low lying, flat marshy area. Twenty Road appears to have been built up above this low area, with fill material. There is approximately 0.3 metres of fill on the roof of the structure. Fill slopes at Twenty Road, adjacent to the north end of the culvert, are approximately 2H:1V. The slope at the north west corner of the culvert is eroding due to drainage from the structure deck.

At the south end of the culvert, Twenty Mile Creek widens into a small pond with an approximate width of 16 m and a length of 29 m. The slopes directly adjacent to the structure are at approximately 2H:1V. On the south side of the pond the existing slopes are approximately 3H:1V. The land on the south side of the pond is covered with mature deciduous trees.

A ditch on the south side of Twenty Road, discharges into the pond near the south-west corner of the structure. The depth of water at the time of investigation was approximately 0.4 metres. The elevation of the creek surface was approximately 217 metres.

Culvert #2 - Twenty Mile Creek at Highway 6

The existing culvert, approximately 38 metres in length, channels Twenty Mile Creek under Highway 6. It is believed that the present culvert is a rigid frame with an open type footing (there are no plans available for the existing culvert). The existing culvert opening is approximately 2.6 metres in height and has a span of approximately 5.5 metres. The roof is approximately 0.4 metres thick. The south side of the culvert has wingwalls at both ends.

Approximately 0.5 metres of fill material has been placed on top of the culvert roof. There is no evidence of pavement settlement above the existing culvert.

The inlet to the culvert is from a widening of Twenty Mile Creek (small pond), on the west end of the structure. The depth of water, in the small pond, at the time of investigation was approximately 0.5 metres and the flow was quite slow. On the east or outlet end of culvert #2 the creek narrows considerably and the flow increases.

Due to the close proximity of Twenty Road and Twenty Mile Creek at both the inlet and the outlet locations, gabion walls have been constructed to retain the embankment fills along Twenty Road.

The gabion walls extend from the ends of the culvert for 9 m on the west and 2 m on the east: the gabion walls consist of three rows. Beyond the gabion walls, Twenty Road embankments are at approximately 2H:1V. Beyond the end of the wingwall located at the southwest quadrant, the slope rises up to Hwy 6 at approximately 4H:1V. On the east end a significant amount of fill has been placed against the front face of the wingwall. Beyond the end of the wingwall the slope rises up to Hwy 6 at approximately 2.5H:1V.

2.2 Site Geology

Physiographically, the site is located within the region known as the "Haldiman Clay Plain". The Haldiman Clay Plain is the product of the advance and retreat of the Wisconsin ice sheet which covered the area during the Pleistocene Epoch (over 12,000 years ago).

This area of the Haldiman Clay Plain is characterized by a series of recessional moraines, built by a glacial ice lobe. The site itself lies in a trough between two moraines. These troughs are typically composed of lacustrine silt or clay. The depth to bedrock in this area is typically less than 15 metres.

Twenty Mile Creek is one of the principal drains for this area. Ultimately, Twenty Mile Creek drains into Lake Ontario.

3.0 INVESTIGATION PROCEDURE

3.1 General

The subsurface conditions at the site were determined by conducting a field investigation and by

carrying out laboratory testing on selected representative samples. Details of the field investigation and laboratory testing program are discussed in this section of the report.

3.2 Field Investigation

The field investigation, which consisted of a total of 5 boreholes, was carried out inclusively from July 22 to July 25, 1996. One borehole was advanced at each culvert extension location (i.e. one at each end of both culverts). One additional borehole was advanced behind the gabion wall to the west of Culvert #2, to determine the composition of the fill material at this location.

The boreholes were advanced using a track mounted diesel drilling unit, to depths ranging from 6.6 metres to 12.0 metres. Hollow stem augers were used to advance the boreholes through the overburden. Conventional rock coring techniques employing NW casing and a NX core barrel was used to retrieve rock core samples up to 1.5 metres in depth.

Subsoil samples were generally retrieved at 0.76 metre intervals in accordance with the Standard Penetration Test (ASTM D1586). All subsoil samples were identified and then sealed in plastic containers in the field to preserve natural moisture contents. The samples were then transported to the laboratory where additional visual classifications and pertinent laboratory tests were conducted.

In situ vane tests were carried out to determine the undrained shear strength of the upper cohesive, clayey silt, soil of weaker consistency. The test was carried out in accordance with ASTM D2573 employing the standard MTO 'N' vane. Remoulded shear strengths were also obtained where possible

to determine the soil sensitivity.

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 recoveries. All rock cores were placed in standard rock core boxes and carefully transported to the laboratory for detailed rock core logging by our resident geologist.

Groundwater levels were obtained by measuring the levels in the open boreholes during the investigation.

All boreholes were backfilled with the native soil cuttings, upon completion of the fieldwork.

Survey information related to the location and the ground surface elevation of the boreholes was provided by Central Region Surveys and Plans, following borehole advancement.

3.3 Laboratory Analyses

All subsoil samples were carefully examined in accordance with the procedures outlined in the Visual Method described in Chapter 2 of the MTO Soil Classification Manual. Laboratory testing on selected representative subsoil samples consisted of routine physical property testing. This testing consisted of natural moisture content determination, Atterberg Limit Testing, Particle Size Analyses and unit weight determination. Laboratory test results are shown on the individual borehole logs and figures in the Appendix.

All rock cores were carefully examined and rock core descriptions that include grain size, bedding thicknesses, jointing characteristics, strength and colour have been summarized in a Rock Core Description Report included in the Appendix of this report. Rock core recoveries and rock quality designations are also summarized in the Rock Core Description Report and also on the individual borehole logs.

4.0 SUBSURFACE CONDITIONS

4.1 General

The ground surface elevation at the boreholes advanced at the site range from 217.6 m (BH#2) to 217.4 m (BH#4) at Culvert #1, from 217.3 m (BH#1) to 220.0 m (BH#5) at Culvert #2 and 219.3 m (BH#3) behind the gabion wall.

In general, the native subsoils are uniform across the site. There is a surficial native stratum of a clayey silt with random zones of silt, that extends to bedrock. The thickness of this stratum varies from 5.2 m to 10.2 m.

The surficial 4 to 6 metres of the stratum is of a stiff to very stiff consistency. This material is underlain by material of a hard consistency. Silt layers with thicknesses up to 1.2 metres were encountered within the stratum. The denseness of the silt layers is generally compact.

Fill material used in the construction of the roadway and also as backfill to the structures, is present overlying the native clayey silt with random zones of silt. The fill material consists of an irregular

mixture of clayey silt, sand and gravel with random zones of silt, sand and gravel at some locations. The thickness of the fill material explored ranged from 4.6 to 5.0 metres.

The overburden at the site is underlain by dolostone bedrock of the Lockport Formation. This rock is characteristically very light grey to medium dark grey in colour, unweathered to slightly weathered, and strong to very strong. The elevation of the bedrock ranges from approximately 207.4 m to 209.8 metres.

Groundwater elevations measured at the time of the investigation ranged from 216.8 m to 217.3 m, coinciding generally with the water level in the pond and creek.

A plan of the two sites illustrating the locations and elevations of the boreholes is shown on Dwg No. 169600-A. Stratigraphical sections illustrating the subsurface conditions at the site are also provided on Dwg 169600-A. The boundaries between the various soil types, in situ and laboratory test results and groundwater levels established at the time of the investigation are shown on the stratigraphical sections and also on the individual record of borehole sheets.

4.2 Irregular Mixture of Clayey Silt, Sand and Gravel (Fill Material)

An irregular mixture of clayey silt, sand and gravel appears to have been placed as fill material to raise the grade at this intersection, and has also been used as backfill to the culverts. Random zones of silt, sand and gravel and traces of organics are also present within the fill material. The fill material is generally brown to mottled grey/brown.

A grain size distribution envelope produced by mechanical sieve and hydrometer analysis for the fill material is illustrated in Figure 1 in the Appendix. The envelope reveals a broadly graded material with particle sizes ranging from gravels to clays. The clay fraction ranges from 8 to 20%. Silt percentages range from 38 to 56%. The combined silt and clay percentages range from 53% to 79%. In the MTO soil classification system, soils which have a fine grained portion exceeding 50% are categorized according to their plasticity and behaviour as discussed below.

Atterberg Limit Tests were carried out on the fine grained portion of the fill material (material less than 75 micrometres) to define the behaviour and plasticity of the material. The results are plotted on the Plasticity Chart illustrated in Figure 2 in the Appendix and summarized in Table 1 below. Natural moisture contents and bulk unit weights are also summarized below.

Table 1 - Irregular Mixture of Clayey Silt, Sand and Gravel(Fill Material)

Physical Property	Range	# of Tests
Natural Moisture Content(w %)	17-23	4
Liquid Limit(w_L %)	24-31	4
Plastic Limit(w_p %)	15-19	4
Plasticity Index(I_p %)	9-12	4
Bulk Unit Weight (kN/m^3)	20.3-21.3	4

The test results reveal that the fine grained portion of the fill material is of low plasticity and hence is categorized as a clayey silt. One sample plots as a borderline clayey silt/organic silt.

Standard Penetration Tests (SPT) carried out within the fill material revealed 'N' values ranging from 1 blow/0.3 m to 10 blows/0.3m. In general, 'N' values are within the 3 to 7 blows/0.3 m range revealing a soft to firm consistency, for the cohesive clayey silt, sand and gravel and a very loose to loose denseness for the silt sand and gravel zones.

4.3 Clayey Silt with Random Zones of Silt

The predominant native soil at both sites either underlying the fill material or present surficially at an Elevation ranging from approximately 217.4 m to 217.6 m at Culvert #1 and approximately 217.3 m at Culvert #2 is a Clayey Silt of glacialacustrine origin. The thickness of this deposit ranges from approximately 5.2 metres to 10.2 metres. Random zones of silt ranging in thicknesses up to 1.2 m are also present within this deposit.

The deposit has been oxidized for the surficial 2.8 m to 3 m and is brown in colour within this zone. Beneath the brown zone, the material is grey in colour.

A Grain Size Distribution Envelope for the deposit as produced by mechanical sieve and hydrometer analysis, is shown in Figure 3 in the Appendix. The envelope illustrates a gradation containing primarily silt percentages ranging from 62% to 96% (typically 80%-90%) and clay fractions generally ranging from 9% to 30% (typically 14%-26%). The silt layers are illustrated separately as individual grain size distribution curves in Figure 3.

A boulder was also encountered at BH#2, at a depth of approximately 5.4 m below the ground

surface. The boulder, which was cored, was approximately 300 mm in thickness. The boulder was of dolostone with physical and mechanical properties similar to the bedrock.

Atterberg limits were carried out on some selected samples to determine the plasticity of the fine grained portion of the deposit. The results are plotted on the Plasticity Chart illustrated in Figure 4 of the Appendix, and are summarized in Table 2 below. Natural moisture contents and bulk unit weights are also summarized below.

Table 2 - Clayey Silt with Random Zones of Silt

Physical Properties	Range	No. Of Tests
Natural Moisture Content (w %)	13 - 35	12
Liquid Limit (w_L %)	23 - 35	11
Plastic Limit (w_p %)	16 - 22	11
Plasticity Index (I_p %)	2 - 13	11
Liquidity Index (I_L)*	0.9 - 2.1	7
Bulk Unit Weight (kN/m^3)	18.6 - 21.9	9

*Only calculated for upper, weaker, zone of stratum

The test results indicate that the material is of low plasticity and hence can be categorized as a clayey silt (CL) to a plastic silt (ML).

Natural moisture contents are generally larger within the weaker upper zone of the stratum. The natural moisture contents range from 13% to 35%, liquidity indices generally exceed unity in this upper zone. Figure 5 in the Appendix illustrates the larger moisture contents within the surficial 4

to 6 metres of the stratum. Standard Penetration Tests carried out in this stratum revealed 'N' values ranging from 1 blow/0.3 m to 49 blows/0.15 m. Lower Penetration resistance values were measured within the surficial, weaker, 4 to 6 metres of the stratum. The 'N' values ranged from 1 blow/0.3 m to 25 blows/ 0.3 m but were generally less than 10 blows/0.3 m. Within the lower thickness of the deposit, the 'N' values ranged from 30 blows/0.3 m to 49 blows/0.3 m.

In situ vane tests were conducted within the surficial 4 to 6 metres of the stratum to determine the undrained shear strength of the soil. In most cases, the vane could not be torqued. Typically, vanes cannot be torqued in soils with undrained shear strengths (C_u) exceeding 120 kPa. At this site, it is suspected that the presence of the silt layers may have contributed to increasing the soil resistance. Therefore, these vane test results may not be accurately measuring the undrained shear strength of the cohesive soils. Vanes that could be torqued, measured undrained shear strengths (c_u) ranging from 60 kPa to 90 kPa.

Based on the Standard Penetration Test results and the insitu vane test results, it is concluded that the surficial 4 to 6 metres of this stratum has a stiff to very stiff consistency with compact layers of silt. This upper zone is underlain by a clayey silt material of hard consistency, with zones of dense silt.

4.4 Bedrock

The clayey silt with random zones of silt is underlain by Dolostone bedrock of the Lockport Formation. The bedrock was both augered (up to approximately 0.3 metres at some locations) and

also cored. Bedrock core was retrieved in NX size for a depth of approximately 1.6 metres.

Detailed rock core descriptions are given in a "Rock Core Description Report" included in the Appendix to this report. The dolostone bedrock is generally grey with stylolites and abundant small vugs. The rock is unweathered to slightly weathered. The rock is horizontally bedded and contains wide to very close spaced fractures that are generally dipping to flat, planar to undulating and smooth to rough.

Core recoveries and Rock Quality Designations (RQD's) were determined in situ to evaluate the competence and integrity of the rock. Core recoveries ranged from 91% to 100% and RQD's ranged from 62% to 98%. This data reveals that the bedrock quality ranges from fair to excellent.

Based on index strength property identification, the rock strength is described as strong to medium strong.

5.0 GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in the open boreholes during the investigation. At the borehole locations where drilling water was used to facilitate the rock coring process, it is expected that the water levels were not stabilized.

Groundwater levels measured across the site ranged from approximately Elevation 217.3 m to 216.8 metres.

The water level in the creek at the time of the investigation varied from Elevation 217 m (adjacent to BH#2) to approximately 216.8 m (adjacent to BH#1). Typically, groundwater levels adjacent to the creek are equal or slightly higher than the creek water elevation.

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

6.0 DISCUSSION AND RECOMMENDATIONS

In conjunction with the proposed widening of the Highway 6/Twenty Road intersection, it has been proposed to extend the two existing culverts at the site and also to construct a retaining wall at the outlet of the culvert transmitting the water of Twenty Mile Creek beneath Highway 6. It has been proposed to extend the Twenty Mile Creek/Twenty Road culvert (Culvert #1) at both the inlet and outlet by eight (8) metres. The Twenty Mile Creek/Highway 6 culvert (Culvert #2) is to be lengthened by five (5) metres at the inlet and ten (10) metres at the outlet. At the outlet of Culvert #2, a retaining wall of six (6) metre length has also been proposed, immediately beyond the culvert, to retain the Twenty Road embankment fill at this location. Both culverts were constructed several years ago.

It is understood that the culvert dimensions are to be similar to the existing culvert dimensions. Both culverts are to have spans of 5.5 metres. The Culvert #1 extension is to have a height of approximately 1.9 metres and the Culvert #2 extension is to have a height of approximately 2.6 metres. The height of the retaining wall is expected to be a maximum of 2.6 metres.

It is understood that no or very small grade changes are proposed for Highway 6. The grade of Twenty Road may be raised by as much as 0.6 to 0.9 metres (over Culvert #1). Currently, Culvert #1 has approximately 0.3 metres of fill cover and Culvert #2 has approximately 0.5 m.

A plan illustrating the two culvert locations is shown on Drawing 169600-A in the Appendix. A

stratigraphical section produced at each culvert location is also shown on Drawing 169600-A.

Recommendations pertaining to the following foundation and geotechnical considerations are included in the purview of this report.

1. Structure Foundations
2. Culvert Inlet/Outlet
3. Backfill to Structure
4. Approach Embankments
- and 5. Construction Considerations

6.1 STRUCTURE FOUNDATIONS

6.1.1 General

It is believed that the existing culverts are both open footing type culverts, although this is not known for certain because foundation drawings are unavailable for both culverts.

The surficial native clayey silt with random zones of silt at the site is not considered a particularly suitable soil for the support of conventional shallow foundations. Settlements and possible bearing capacity failure could result from excessive applied loadings.

Subexcavation to depths of 3 to 4 metres would be required to transfer bearing pressures to more competent soil; this is not considered practical nor economically feasible. Consequently, bearing

capacities for a box type culvert, at a practical invert elevation, have been given for a spread footing option.

The depth to bedrock throughout the site is relatively shallow (approximately 10 m), making the use of deep foundations a suitable alternative. Steel H-piles driven to bedrock or concrete caissons installed in drilled shafts augered to the bedrock, are two deep foundations that can be considered.

The Structural Engineer shall undertake a cost risk analysis associated with the foundation options given. Deep foundations present less risk from a foundation performance point of view, but are more costly. Shallow foundations should be less costly to construct, but may present more of a foundation performance risk. The alternative that is considered to be more practical and economical at the site shall be chosen.

6.1.2 Shallow Foundation

6.1.2.1 Foundation Design

The culvert extensions and the retaining wall can be founded on a shallow foundation within the surficial native clayey silt with random zones of silt stratum at or below the founding elevations given in Table 3. It is recommended that the culvert extensions be founded on a closed type slab on grade, box type spread footing, rather than an open footing type culvert. The culvert extension can be either a cast-in-place or precast structure.

Shallow foundations shall be designed in accordance with Section 6-8 of the 3rd Edition of the

O.H.B.D.C. For purposes of the O.H.B.D.C., the net bearing resistances tabulated in Table 3 can be used in the design of the shallow foundations. The bearing resistance for the retaining wall is based on a minimum footing width (B) of 1 metre.

Settlements developed as a result of the applied pressures tabulated in Table 3 are expected to be within 25 mm and because the deformations will be recompression in nature, should occur immediately during construction. It is recommended that the culvert extensions be articulated from the existing culverts with an "isolation joint" to accommodate any differential settlements between the existing and the new culverts.

Table 3 - Shallow Foundation Bearing Resistance

Structure	Factored Resistance at ULS (kPa)	Factored Resistance at SLS (kPa)	Foundation Elevation (m)
Culvert #1			
-Outlet (South)	175	100	216
-Inlet (North)	175	100	215
Culvert #2			
-Outlet (East)	175	100	215
-Inlet (West)	175	100	215
Retaining Wall	175	100	215

All foundation shall be protected against frost penetration by providing a minimum of 1.2 m of earth cover or equivalent frost protection. The bearing resistances in Table 3 shall be reduced to account for the effects of inclined loadings. Reduction factors shall be computed in accordance with Section 6-8.4.2 of the OHBDC.

For the retaining wall, shallow foundations, the horizontal resistance of the footing is also a design consideration. The horizontal resistance of the spread footing can be computed using an angle of friction of 30° (δ), between the base of the concrete footing and the founding soil. The horizontal resistance of the shallow foundation shall be computed in accordance with Section 6-8.4.3 of the OHBDC.

Any additional loadings imposed on existing utilities shall be reviewed during the design of the shallow foundation. Similarly, the influence of any existing utilities on the foundation design shall also be carefully reviewed, particularly at the outlet of Culvert #2 where several utilities are known to exist. Any fill material present within a 1H:1V zone of influence of the new foundations, shall be subexcavated and replaced with a Granular 'A' material. The Granular 'A' material shall be placed and compacted in accordance with OPSS 501 series to achieve 100% of the maximum dry density.

6.1.2.2 Foundation Construction

All softened/loosened and/or organic material encountered at the founding elevation beneath the plan limits of the footings shall be removed and replaced with Granular 'A' or mass concrete. In addition any fill material previously placed in conjunction with the existing utilities or other construction, shall also be subexcavated and replaced with Granular 'A' or mass concrete. It is recommended that all utilities present within the proposed footing extension zone of influence, such as at the outlet of Culvert #2, be located and investigated.

Foundation excavation and construction adjacent to the existing culvert foundations shall be carried

out such that undermining of the existing foundations is avoided. It is recommended that the existing foundations be located and exposed prior to the construction of the new foundations for the extensions.

During the construction of the culvert extensions, it is recommended that a working slab be placed to protect the founding soil from disturbance that can be caused by weathering and/or construction related activities. The working slab can consist of mass concrete or Granular 'A' and shall be placed within 4 hours of exposure. It is recommended that a concrete working slab of 100 mm thickness or alternately a 150 mm minimum thickness of Granular 'A' be used.

6.1.3 Deep Foundations

6.1.3.1 Driven Steel H-Piles

6.1.3.1.1 Foundation Design

The structure foundations for all four culvert extensions and the retaining wall can be founded on steel H-piles driven to bedrock, at tip elevations as tabulated in Table 4. For the purposes of the OHBDC, the steel H-piles can be designed employing the axial capacities summarized below.

Table 4 - Driven Steel H-Piles: Vertical Axial Capacity

Pile Type	Factored Axial Capacity at ULS (kN)	Axial Capacity at SLS (kN)	Culvert #1 Pile Tip Elevation (m)	Culvert #2 Pile Tip Elevation (m)
HP 310X110	Structural Capacity	Structural Capacity	Inlet (North) 207.4± Outlet (South) 207.4±	Inlet(West) 208.2± Outlet(East) 209.8±

Since the piles will be driven to competent bedrock it is the structural capacities that will govern the design rather than the foundation capacity. The piles can be assumed to be fully laterally supported for the entire pile embedment lengths. The structural engineer shall calculate the structural capacities of the pile to ensure that the structural capacity is not exceeded.

Geotechnical resistance at SLS represents the capacity at which the applied load will yield a settlement that does not exceed 25mm. However, it is anticipated that the settlement of the piles on the bedrock will be less than 25mm. The structural engineer shall calculate any settlement due to axial shortening of the piles.

Resistance to lateral loads can be achieved by inclining the piles at a batter. The lateral resistance of the battered piles shall be computed in accordance with Section 6-9.8 of the OHBDC and shall be taken as the horizontal component of the factored axial capacity of the inclined pile.

Alternatively, the lateral capacity of vertical piles can be computed by modelling techniques. Soil parameters to facilitate the design of the horizontal capacities of the vertical piles using the spring constant method are given in Table 5.

The method of lateral capacity computation and the magnitude of lateral pile capacity calculated shall be reviewed by our office prior to design finalization. It is requested that the Structural Engineer contact our office following the lateral capacity calculation.

Pile spacing shall conform with Section 6-11 of the OHBDC. All pile caps shall be protected against frost penetration by providing a minimum of 1.2m of earth cover or equivalent frost protection.

6.1.3.1.2 Foundation Construction

All piles are to be driven to the bedrock surface and a note should be included on the contract drawings accordingly.

When installing piles adjacent to an existing structure/utility it is essential to avoid damaging the existing foundation/utility as a result of excessive vibrations. At the site, penetration resistance through the surficial 4 to 6 metres is not anticipated to be significant and consequently, disturbance due to vibration is not expected. However, it is recommended that the Contractor be instructed to control the intensity of ground vibrations during pile installation. Ground vibrations generated by pile driving shall be restricted to a maximum peak velocity of 100 mm/s in three mutually perpendicular directions. The Contractor shall monitor the vibrations at the closest existing structure location and maintain detailed records of the monitoring during the pile installation.

To facilitate the installation of the steel H-piles, it is recommended that the piles be equipped with reinforced tips as illustrated on MTO standard drawing DD-3301 or driving shoes. Should the steel H-piles be spliced, splicing shall be in accordance with OPSS 903.07.01.03 and as shown on the above mentioned drawing (DD-3301).

Table 5 - Horizontal Resistance Design Parameters

Structure	Soil	Elevation (m)	Undrained Shear Strength c_u (kPa)	*Bulk Unit Weight γ (kN/m³)	**Coefficient of Horizontal Subgrade Reaction (kN/m³)
Culvert #1 Outlet (South End)	Clayey Silt with Random Zones of Silt	217 - 213	60	20	13,400
		213 - 207	200		45,000
Culvert #1 Inlet (North End)	Clayey Silt with Random Zones of Silt	217 - 213	80	20	17,500
		213 - 207	200		45,000
Culvert #2 Inlet (West End)	Clayey Silt with Random Zones of Silt	217 - 212	80	20	17,500
		212 - 208	200		45,000
Culvert #2 Outlet (East End)	Clayey Silt with Random Zones of Silt	217 - 211	80	20	17,500
		211 - 210	200		45,000

* Buoyant unit weights are to be used for soils submerged below the groundwater table.

** For driven HP 310X110 piles

The Contractor shall also be alerted that random boulders may be present within the deposit of clayey silt with zones of silt.

Pilecap excavation and construction adjacent to the existing culvert foundations shall be carried out such that undermining of the existing foundations is avoided.

6.1.3.2 Caisson Piles

6.1.3.2.1 Foundation Design

Alternatively, the new structure foundations can be founded on end bearing concrete caissons founded on bedrock. The tip elevations of the caissons shall be as tabulated in Table 4.

The load carrying axial capacity of concrete caissons founded on the dolostone bedrock can be determined using a vertical factored capacity at ULS equivalent to 4000 kPa. Due to the unyielding nature of the bedrock, the Serviceability Limit State (SLS) will not govern the design because the stresses required to induce detrimental settlements at SLS, will exceed the factored capacity at ULS.

The Structural Engineer can use the axial capacity provided to select the size of the caisson and the respective ultimate load capacity. For instance, a 0.9 metre diameter caisson will yield a load capacity equivalent to 2550 kN at ULS. The lateral resistance of caisson piles shall be computed in accordance with Section 6-9.8 of the OHBDC, similar to the design of the lateral resistance of driven piles described in the previous section of this report. It is cautioned that:

- (1) battered caissons are limited to a maximum 1H:4V due to construction limitations
- and (2) that the coefficients of horizontal subgrade reaction in Table 5 are applicable to driven HP310x110 piles.

Coefficients of horizontal subgrade reaction for caisson piles can be calculated using the following method:

$$K_h = \frac{67c_u}{d}$$

where K_h = coefficient of horizontal subgrade reaction;
 c_u = undrained shear strength of soil;
 d = caisson pile diameter.

Undrained shear strength values of the soil are given in Table 5.

The method of lateral capacity computation and the magnitude of lateral pile capacity computed, shall be reviewed by our office prior to design finalization. It is requested the Structural Engineer contact our office following the lateral capacity calculation.

6.1.3.2.2 Caisson Pile Construction

Caisson piles shall be constructed in accordance with OPSS 903 and a NSSP for caisson piles, that can be obtained from our office. In addition, it is recommended that a NSSP be included to alert the Contractor that the random zones of silt present within the native stratum at the site below the groundwater table are susceptible to cave-in and sloughing. In addition, the Contractor shall be alerted that random boulders may be present within the deposit of clayey silt with zones of silt.

The Contractor shall be responsible for maintaining sidewall and basal stability during drilling and concreting. This can be achieved by using conventional liner and /or mud drilling techniques, as required.

6.2 CULVERT INLET/OUTLET

The culvert/retaining wall foundations shall be protected from the scouring forces of the creek water at both the inlet and outlet of the culverts. The design of the scour protection shall consider the applicable hydrological and hydraulic parameters and the native subsoil conditions at the site. Scour protection can be achieved by aprons and rip rap. Typically, 600 mm thick rip rap is placed a minimum 0.5 metres above the high water table. The rip rap shall be placed to a minimum extent beyond the culvert, to ensure that the foundations are not undermined.

To inhibit flow within the backfill adjacent to the culvert wall, it is recommended that headwalls or an impervious clay liner or equivalent be used at the culvert inlet. The impervious clay blanket shall be 1 metre thick and consist of material as specified in OPSS 1205 and placed as specified in OPSS 902.07.04.02.

At the culvert outlet, it is recommended that a 1 metre thick blanket of Granular 'A' material be placed as a filter, behind a 600 mm layer of rip rap.

6.3 BACKFILL TO STRUCTURE

6.3.1 Material

It is recommended that Granular 'A' or Granular 'B' (conforming to Special Provision SP109F03) material be placed behind the culvert walls as illustrated in the applicable OPSD 803 drawings, and behind the retaining wall as specified in Section 6-7 of the OHBDC. The use of a granular material combined with weep holes in the culvert walls or pipe subdrains to drain any accumulation of water

in the backfill will prevent hydrostatic pressure build-up.

Design parameters of the soil are given in Table 6 below. Computations of lateral earth pressure shall be in accordance with Section 6-7 of the OHBDC.

Table 6 - Structure Backfill Properties

Backfill Property	Granular 'A'	Granular 'B'
Angle of Internal Friction (ϕ)	35°	30°
Unit Weight (kN/m ³)	22.8	21.2
*Coefficient of Active Earth Pressure (K_a)	0.27	0.33
*Coefficient of Earth Pressure at Rest (K_0)	0.43	0.5

- * 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. The active condition applies for flexible walls where sufficient movement is permitted to mobilize the active pressure. Figure C6-7.1 and Table C6-7.1 in the O.H.B.D.C. commentary depict movements required to achieve the active state.

The compaction surcharge shall be calculated in accordance with Section 6-7.4.3 of the O.H.B.D.C.

The calculated lateral pressure is a function of the mass and type of compaction equipment and the material being compacted. A minimum compaction surcharge pressure of 16 kPa should be used for a vibratory compactor with a mass of approximately 400 kg.

6.3.2 Backfilling and Compaction

The backfill shall be placed in 300 mm lifts in accordance with OPSS 902 series and compacted to achieve 100 percent of the target maximum dry density as outlined in OPSS 501 series. The backfill

shall be placed simultaneously and evenly on both sides of the culverts such that the maximum difference in the fill placement height at the culvert walls does not exceed 600mm.

Heavy vibratory equipment is not permitted in the backfill construction adjacent to the structure to minimize deflection or possible damage of the wall. Vibratory equipment exceeding 6000 kg operating weight should be kept outside of a 1.5 vertical to 1 horizontal line extending upward from the base of the culvert wall footing. Hand compaction equipment shall be used within these limits.

6.4 APPROACH EMBANKMENTS

6.4.1 General

Transverse embankment fill slopes will have to be constructed to facilitate the widening of Twenty Road and Hwy 6. It is understood that no or small profile grade changes are proposed for both Twenty Road and Hwy 6.

The Twenty Road profile grade may be raised to a maximum of 0.9 metres above the existing profile grade. As a result, the maximum fill height required to facilitate the widening is the order of 4 metres.

There are two major factors that must be considered in the design of approach embankments:

- (1) Stability
- and (2) Settlement

The stability and settlement of the approach embankments with fill heights up to approximately 4 metres are discussed below.

6.4.2 Stability

There are no deep seated or global (external) slope instabilities expected for the transverse slopes constructed at 2H:1V. In addition, for the proposed slopes of up to 4 metres in height, the 2H:1V geometry slopes will also be adequate to avoid surficial slope instabilities provided that an effective erosion control protection scheme such as conventional seeding and mulching or sod be applied with a surface runoff drainage system at the toe of the slope. To ensure the internal stability within the embankment fill, it is recommended that the new fills be "benched" into the existing approach embankments in accordance with OPSD 208.01.

6.4.3 Settlement

Settlements induced as a result of the applied embankment loading will be the result of the elastic compression of the native subsoil and as a result of settlements within the fill material itself. It is anticipated that approximately 25mm of settlement, attributable to the elastic compression of the native subsoil, will be realized. These settlements will be elastic in nature and hence will occur almost immediately.

Settlements within the fill itself can amount to approximately 20 mm due to its self weight. These settlements will be immediate in nature if a granular fill material is used. For fill materials that are cohesive, settlements will be more time dependent, but should be realized within a 3 month period following the fill placement.

6.4.4 Embankment Construction

Embankment fills shall be placed and compacted as specified in OPSS 206.07.07 and OPSS 501 series. As mentioned previously, new fills constructed adjacent to existing fills shall be benched in accordance with OPSD 208.01.

6.5 CONSTRUCTION CONSIDERATIONS

6.5.1 Temporary River Diversion, Cofferdam Construction and Dewatering

A temporary creek diversion using impervious earth dikes or interlocking steel sheet piles in combination with corrugated steel pipes can be used to facilitate the construction of the culvert foundations adjacent to the existing river. Alternatively, the water can be controlled by conventional dam construction created by an interlocking steel sheet pile cofferdam or an impervious earth dike.

Environmental restrictions shall be addressed for any temporary diversion and specified in the contract documents.

Once the temporary creek diversion and/or cofferdam has been constructed, conventional sump pump techniques can be used to discharge any standing water and groundwater, to facilitate the culvert foundation construction. It is expected that the Contractor will be able to effectively dewater the foundation excavation area, in view of the the impervious nature of the clayey silt material. Greater pumping efforts will be required to dewater the zones of silt within the stratum. Disposal of water shall conform to OPSS 518.

6.5.2 Temporary Slopes

Any temporary excavation slopes within the irregular mixture of clayey silt, sand and gravel (Fill Material) or the clayey silt with random zones of silt, to facilitate the construction of the culvert, shall not be steeper than 1.5H:1V.

6.5.3 Temporary Shoring

6.5.3.1 General

Should the required temporary excavation slopes, recommended above, interfere with either Highway 6 or Twenty Road, temporary shoring walls may be required in order to facilitate the construction of the culvert extensions. It is highly probable that some form of roadway protection, will be required on the south side of Twenty Road, on both the east and west ends of Culvert #2. There are existing gabion basket retaining walls at these locations.

Viable shoring schemes that can be used at this site are a soldier pile-timber lagging wall and a sheet-pile wall.

The protection scheme at the site shall be designed and constructed by the Contractor to satisfy the performance requirements specified in OPSS 941. A Level 2 performance level is recommended at the site. The parameters to facilitate the design in accordance with OPSS 941 are given below. Additional information regarding the design of the temporary shoring is also included in this report for MTO internal purposes only. Design parameters and information, only as specified in OPSS 941, shall be included in the Contract Documents and on the Contract Drawings.

6.5.3.2 Shoring Design

The design of the shoring system shall include the appropriate earth pressures computed using an acceptable method specific to the site conditions and in accordance with Section 6-7 of the O.H.B.D.C. Loadings induced by any surcharge traffic shall be incorporated in the design. Lateral earth pressures can be computed using the soil design parameters tabulated in Table 7. Specific elevations to facilitate the roadway protection design, at the east and west ends of Culvert #2, have also been included in Table 7. The active earth pressure coefficient can be used in the design of the shoring wall.

The shoring system must be designed to satisfy earth pressure equilibrium using an appropriate restraining system. At the site, this can be achieved by either a cantilever wall, raker supported wall, or an anchored wall. An appropriate triangular, rectangular or trapezoidal stress envelope shall be chosen to accurately represent the retained soils and the restraining system.

The shoring system method that proves to be the most practical and economical shall be selected. Although the three options are described in this report, the cantilever wall appears to be the most suitable at this site.

Cantilever Wall

In the design of a cantilever wall, the depth of embedment shall be sufficient to ensure that the wall does not overturn. The active pressures exerted on the wall must be resisted by the passive pressures

below the dredge line. Triangular earth pressure distributions can be used in the calculation of the active and passive earth pressures.

Table 7 - Shoring Design Parameters

Site	Soil	Elevation (m)	Angle of Internal Friction (ϕ)	Unconfined Compressive Strength (kPa)	*Bulk Unit Weight (γ) (kN/m ³)
Culvert #2 (West)	Irregular Mixture of Clayey Silt, Sand and Gravel (Fill Material)	219.5 - 215	30°		20
	Clayey Silt with Random Zones of Silt	217-212 212-208	30° 32°		20
	Dolostone Bedrock	< 208		3000	
Culvert #2 (East)	Irregular Mixture of Clayey Silt, Sand and Gravel (Fill Material)	221-217	30°		20
	Clayey Silt with Random Zones of Silt	217-211 211-210	30° 32°		20
	Dolostone Bedrock	< 210		3000	

* Buoyant unit weights (γ') are to be used below the groundwater table.

Raker Supported Wall

Should the cantilever wall embedment depth be considered excessive, rakers propped against the wall can be used. Raker footings can be founded within the native clayey silt with random zones of silt deposit below the frost penetration depth using the bearing capacities tabulated in Table 3. Raker foundations can be founded at or below Elevation 215 m.

All organic and deleterious native soils and any fill material shall be removed prior to the construction of the raker foundations.

Anchored Wall

Alternatively, the shoring wall can be supported by rock anchors installed with the bond zone within the dolostone bedrock. Rock anchors within the bedrock can be designed with an allowable grout/rock bond stress of 400 kPa. Rock anchors can be either monobar or multistrand steel anchors installed in predrilled holes.

It is recommended that for this option the soldier piles be founded on the dolostone Bedrock.

6.5.3.3 Shoring Construction

Soldier piles can be driven or installed in pre-augered holes at the site. For soldier piles installed in pre-augered holes, an NSSP covering the requirements of soldier pile construction shall be included in the contract documents. A copy of this NSSP can be obtained from our office.

Should rock anchors be used, it is recommended that a NSSP be included in the Contract documents that specifies materials, installation and proof testings of the anchors. This NSSP can be obtained from our office.

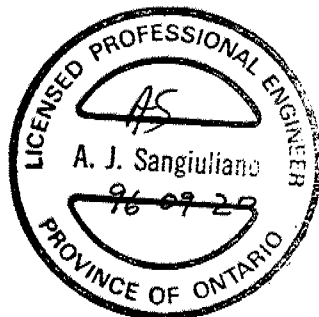
Rakers shall be installed while an earth berm remains in front of the pile. Slots should be cut into this berm to install the rakers before the supporting berm is removed. Rakers can be installed by

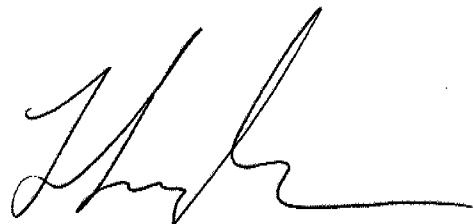
removing sections of the existing gabion wall prior to the removal of the entire wall.

7.0 MISCELLANEOUS


The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer, and J. Werner, Engineer in Training; utilizing equipment owned and operated by Malones Soil Samples.

The report was prepared by T. Sangiuliano and J. Werner and reviewed by D. Dundas, Senior Foundation Engineer.



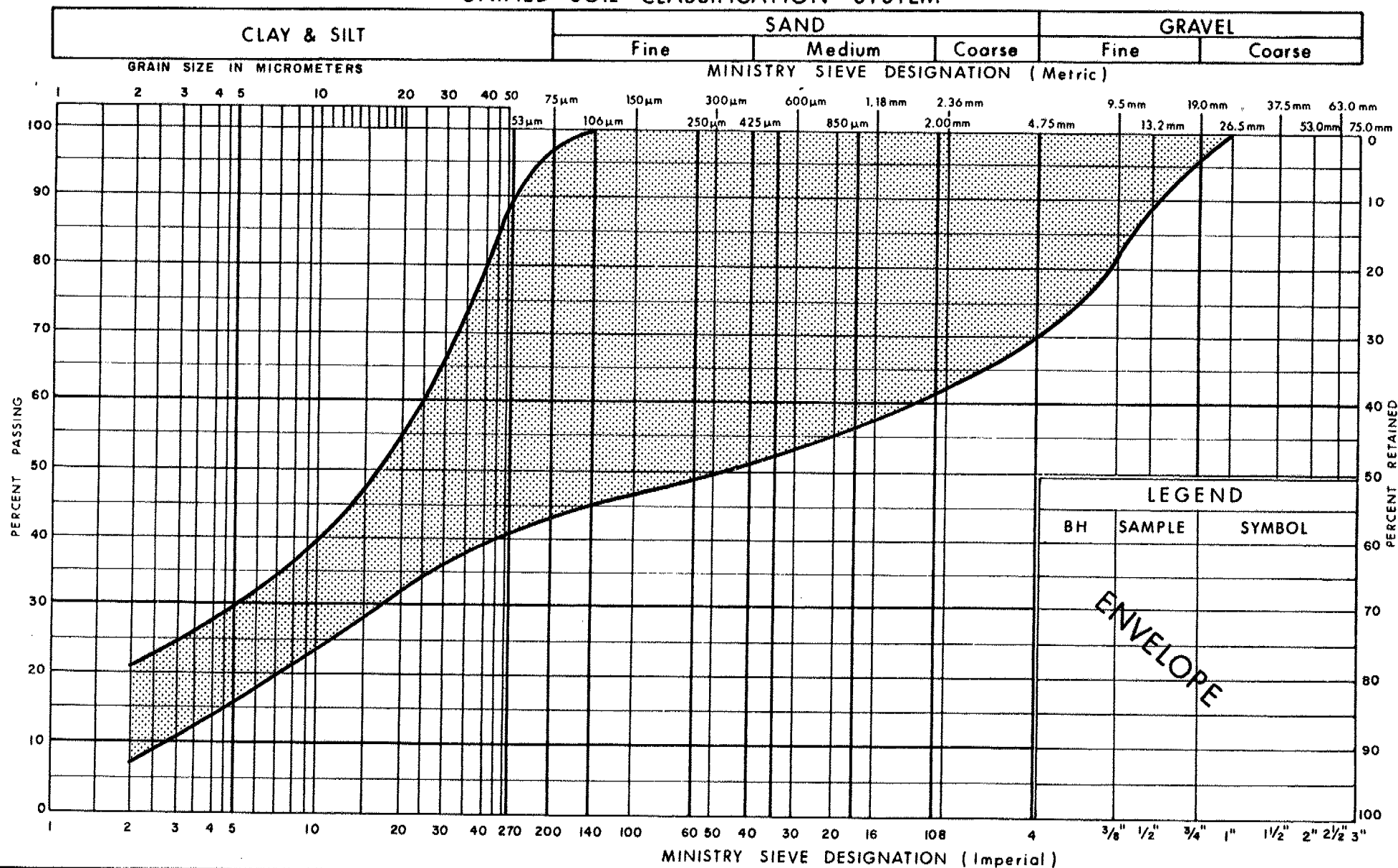

T. Sangiuliano, P. Eng.
Foundation Engineer




D. Dundas, P. Eng.
Senior Foundation Engineer

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM

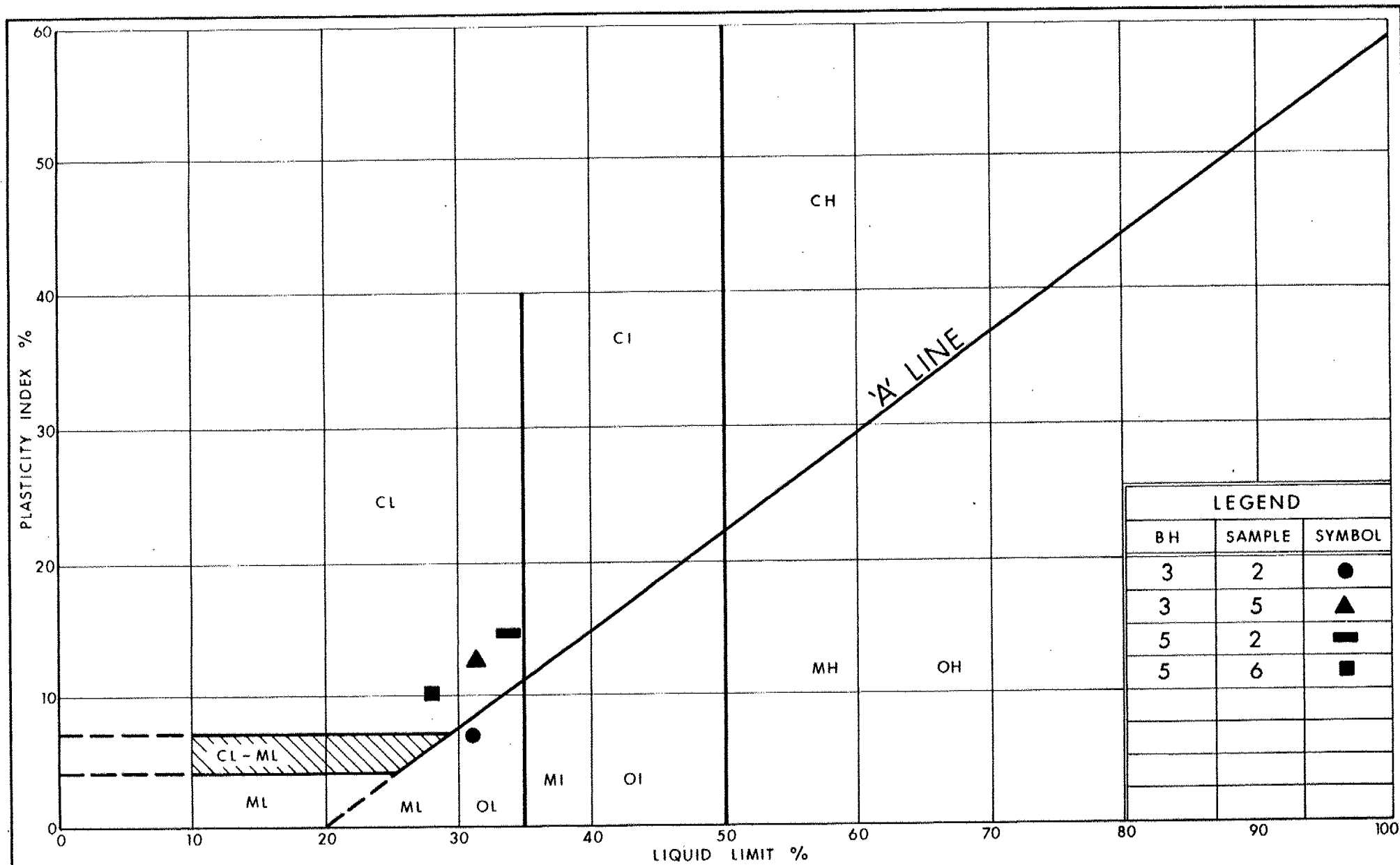


Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
 IRREGULAR MIXTURE OF CLAYEY SILT, SAND & GRAVEL
 (FILL MATERIAL)

FIG No 1

W P 16-96-00



Ontario

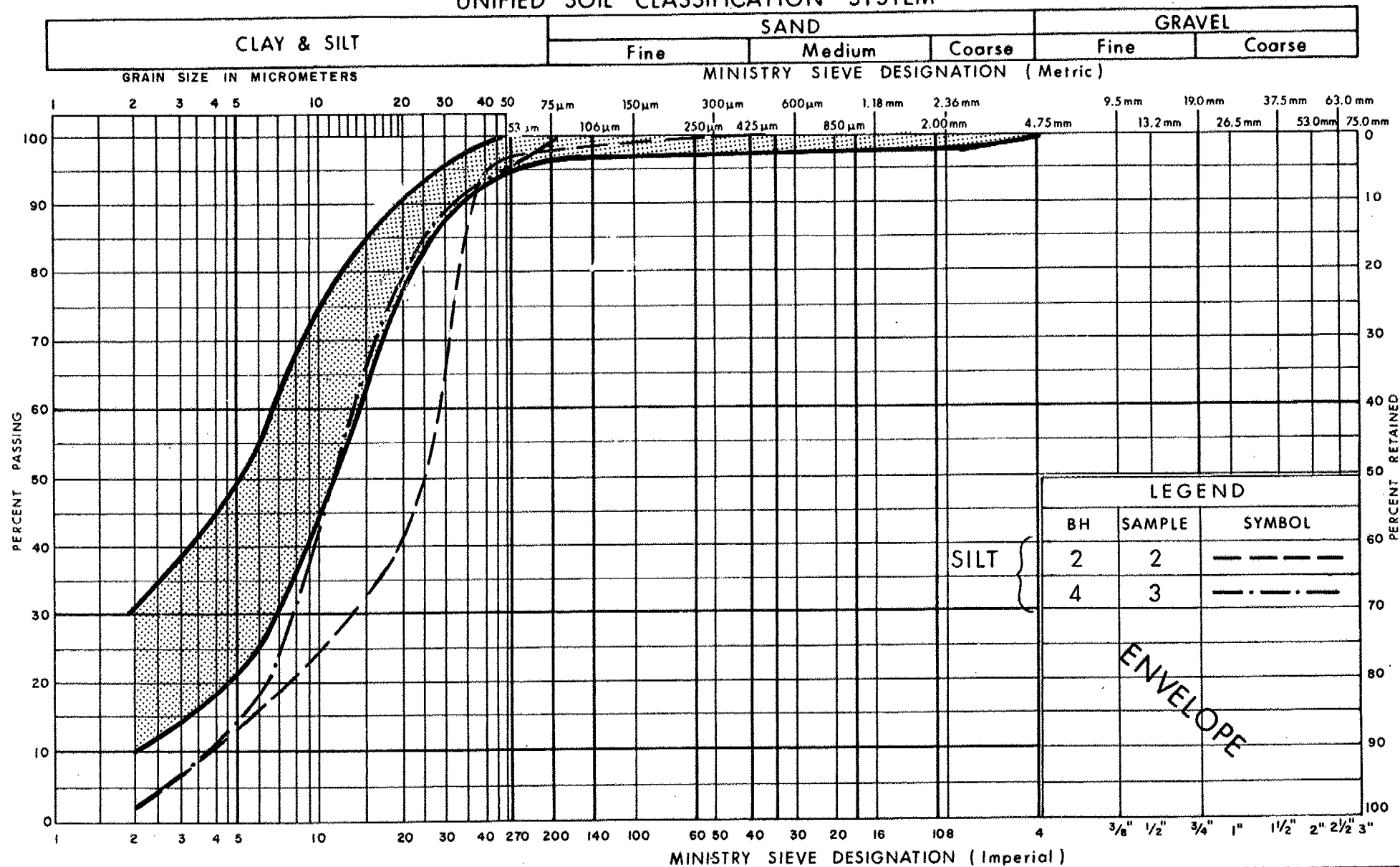
Ministry of
Transportation

PLASTICITY CHART IRREGULAR MIXTURE OF CLAYEY SILT, SAND & GRAVEL (FILL MATERIAL)

FIG No 2

W P 16-96-00

UNIFIED SOIL CLASSIFICATION SYSTEM



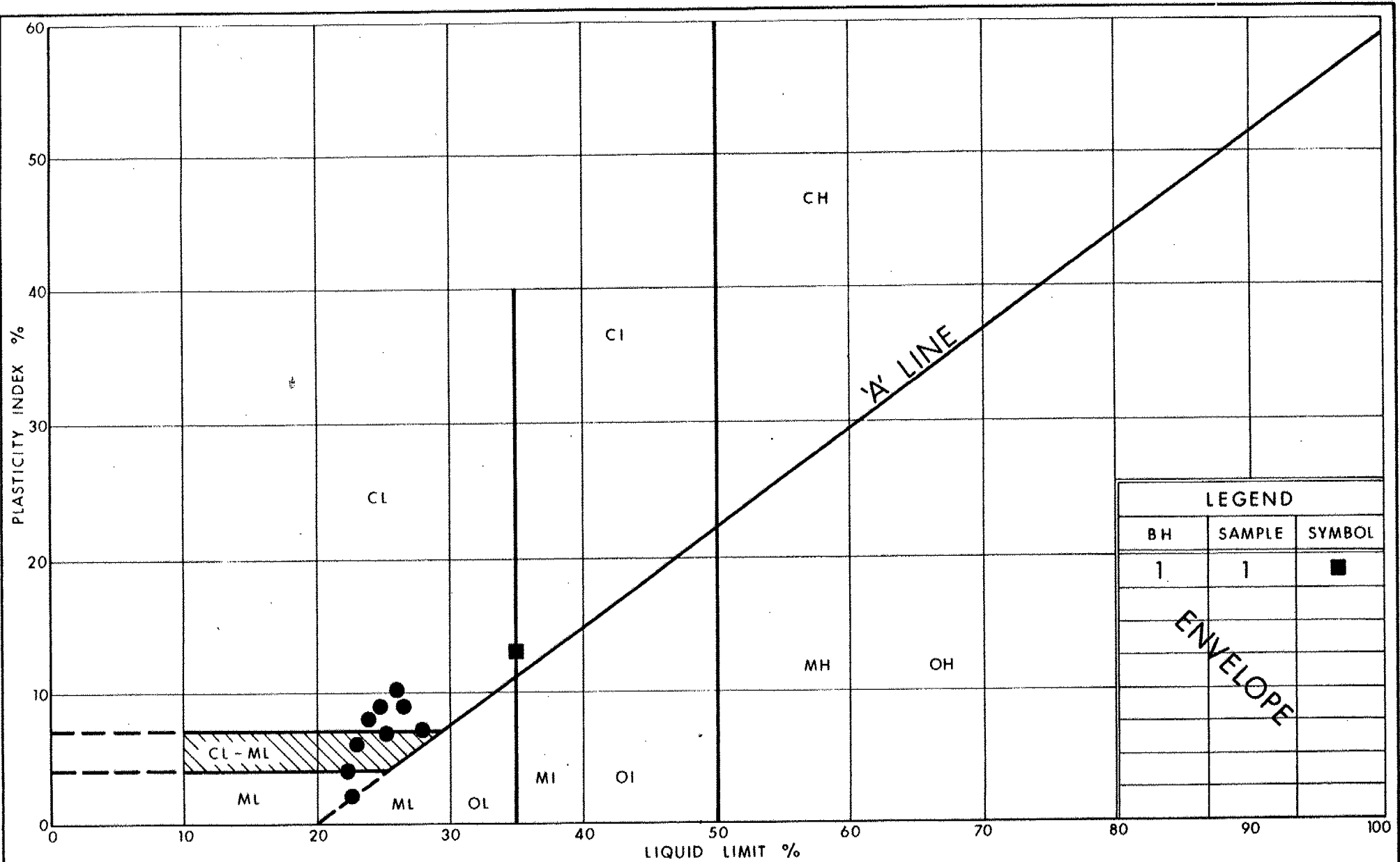
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GRAIN SIZE DISTRIBUTION

CLAYEY SILT WITH RANDOM ZONES OF SILT

FIG No 3

W P 16-96-00



Ministry of
Transportation

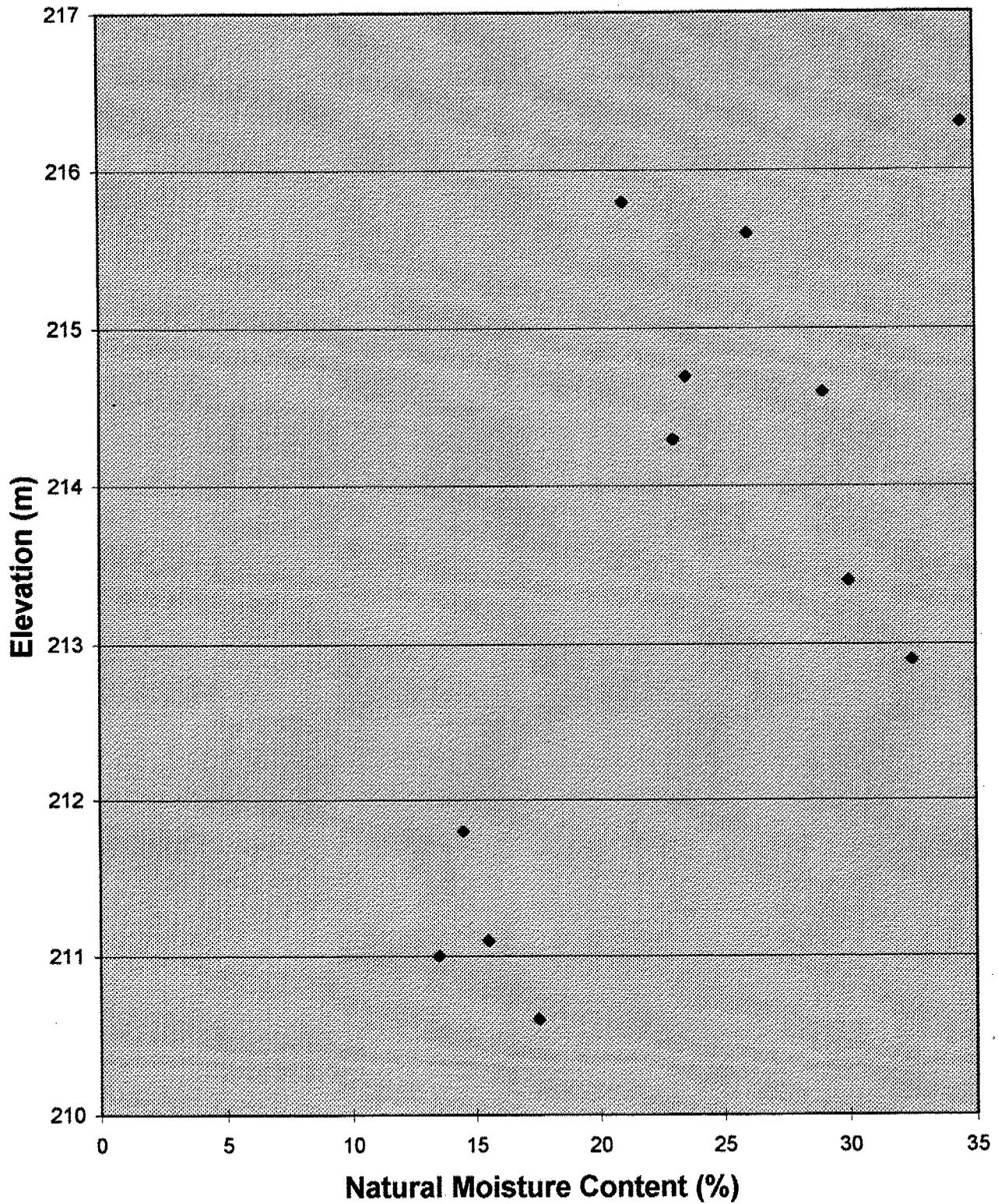
PLASTICITY CHART

CLAYEY SILT WITH RANDOM ZONES OF SILT

FIG No 4

W P 16-96-00

Figure 5 - Natural Moisture Contents versus Elevation
Clayey Silt with Random Zones of Silt



RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 16-96-00 LOCATION STA 9+981.0; o/s 15.7m RT of CL TWENTY RD ORIGINATED BY TS/JW
 DIST CR HWY 6 BOREHOLE TYPE HS AUGER, NW CASING, NX CORE COMPILED BY JW
 DATUM Geodetic DATE 96/07/22 CHECKED BY TS/DD

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					
217.3	Ground Surface																
0.0	Clayey Silt with Random Zones of Silt		1	SS	2		217									20.7	0 7 72 21
			2	SS	1		216										
	Brown		3	SS	10		215									20.3	0 0 80 20
	Grey		4	SS	4		214										
			5	SS	16		213										
	Stiff to Very Stiff		6	SS	25		212										
	Hard		7	SS	43		211									20.3	0 2 72 26
			8	SS	39		210										
208.2							209										
9.1	Dolostone Bedrock Grey, Unweathered to Slightly Weathered, Strong to Medium Strong		9	RC	REC	=100%	208										RQD =78%
206.8							207										
10.6	End of Borehole																
	* 96 07 22																

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 15-96-00 LOCATION STA 9+947.0; o/s 8.2m RT of CL TWENTY RD ORIGINATED BY JW
DIST CR HWY 6 BOREHOLE TYPE HS AUGER, NW CASING, NX CORE COMPILED BY JW
DATUM Geodetic DATE 96/07/23 CHECKED BY TS/DD

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			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100										
								20 40 60 80 100										
217.6	Ground Surface																	
0.0	Clayey Silt with Random Zones of Silt																	
			1	SS	2		217											
	Silt, Compact		2	SS	20		216					21.9	0 1 85 4					
			3	SS	24		215											
	Brown Grey		4	SS	3		214						0 2 73 25					
			5	SS	20		213											
	Stiff to Very Stiff Hard		6	SS	30		212											
	Boulder		7	RC	REC	=100%	211						0 3 67 30					
			8	SS			210											
			9	SS	38		209											
			10	SS	38		208											
			11	SS	44		207											
207.4							206											
10.2	Dolostone Bedrock Grey, Unweathered to Slightly Weathered, Strong to Medium Strong		12	RC	REC	=98%								RQD =98%				
205.9																		
11.7	End of Borehole																	
	* 96 07 23																	

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 16-96-00 LOCATION STA 9+975.0; o/s 2.9m RT of CL TWENTY RD ORIGINATED BY JW
 DIST CR HWY 6 BOREHOLE TYPE HS AUGER COMPILED BY JW
 DATUM GEODETIC DATE 96/07/24 CHECKED BY TS/DD

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
219.3	Ground Surface																
0.0	Irregular Mixture of Clayey Silt, Sand and Gravel (Fill Material) Brown Soft to Firm		1	SS	6		219										
			2	SS	2		218										0 27 65 8
			3	SS	5		217										
			4	SS	8		216										0 2 78 20
			5	SS	7		215										
			6	SS	1		214										
214.7			7	SS	3		213										
4.6	Clayey Silt with Random Zones of Silt Grey Stiff to Very Stiff		8	SS	9												
			9	SS	4												
			10	SS	10												
212.8																	
6.6	End of Borehole • 96 07 24																

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 16-96-00 LOCATION STA 9+959.0; o/s 9.7m LT of CL TWENTY RD ORIGINATED BY JW
 DIST CR HWY 5 BOREHOLE TYPE HS AUGER, NW CASING, NX CORE COMPILED BY JW
 DATUM GEODETIC DATE 96/07/24 CHECKED BY TS/DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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		
217.4	Ground Surface													
0.0	Clayey Silt with Random Zones of Silt													
			1	SS	3		217							
			2	SS	1		216						19.5	1 23 62 14
			3	SS	15		215						18.6	0 1 96 3
	Brown Grey		4	SS	8		214							
			5	SS	5		213						19.7	0 1 90 9
	Stiff to Very Stiff Hard		6	SS	30		212						21.8	0 2 83 15
			7	SS	45		211							
			8	SS	41		210							
			9	SS	36		209							
			10	SS	49		208							
207.4							207							
10.0	Dolostone Bedrock Grey, Unweathered to Slightly Weathered Strong to Medium Strong		11	RC	REC	=91%	206							RQD =85%
205.8														
11.5	End of Borehole													
	* 96 07 24													

METRIC

+3, x5: Numbers refer to Sensitivity

ROCK CORE DESCRIPTION **WP 16-96-00**

Page 1 of 1

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1	9	9.14-10.57	100	78	9.14-10.57	DOLOSTONE (with stylolites, abundant small vugs, and larger vugs up to 3 cm in diameter commonly containing calcite crystals), very light grey to medium dark grey; fine to medium grained; strong to medium strong; unweathered to slightly weathered; fractures moderate to very close spaced, dipping to flat, undulating, rough to smooth.
2	12	10.16-11.68	98	98	10.16-11.68	DOLOSTONE (with stylolites, abundant small vugs, and larger vugs up to at least 7 cm in diameter commonly containing calcite crystals), very light grey to medium dark grey; fine to medium grained; strong to medium strong; unweathered to slightly weathered; fractures wide to close spaced, dipping, undulating, rough to smooth.
4	11	10.01-11.53	91	85	10.01-11.53	DOLOSTONE (with stylolites, abundant small vugs, and larger vugs up to at least 5 cm in diameter commonly containing calcite crystals), very light grey to medium dark grey; fine to medium grained; strong to medium strong; unweathered to slightly weathered; fractures wide to close spaced, dipping to flat, undulating, rough to smooth.
5	12	10.52-12.04	98	62	10.52-12.04	DOLOSTONE (with stylolites, abundant small vugs, and larger vugs up to 2 cm in diameter commonly containing calcite crystals), very light grey to medium dark grey; fine to medium grained; strong to medium strong; unweathered to slightly weathered; fractures moderate to very close spaced, flat to dipping, 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

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 (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

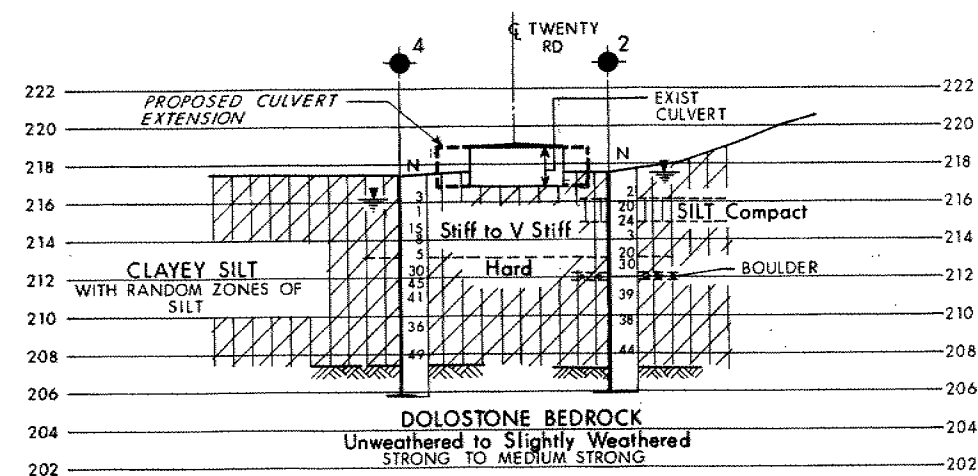
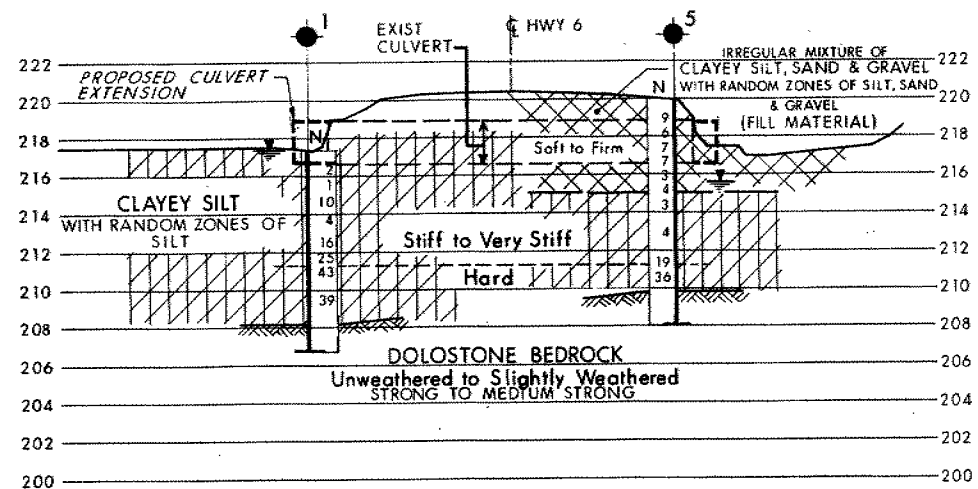
u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa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

MECHANICAL PROPERTIES OF SOIL

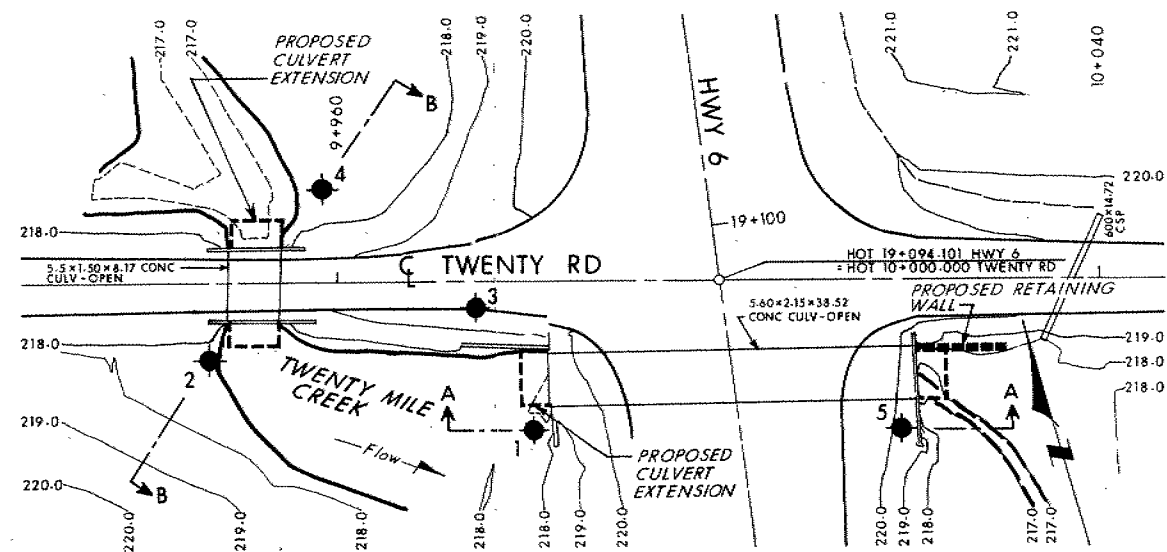
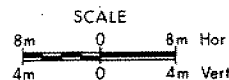
m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
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

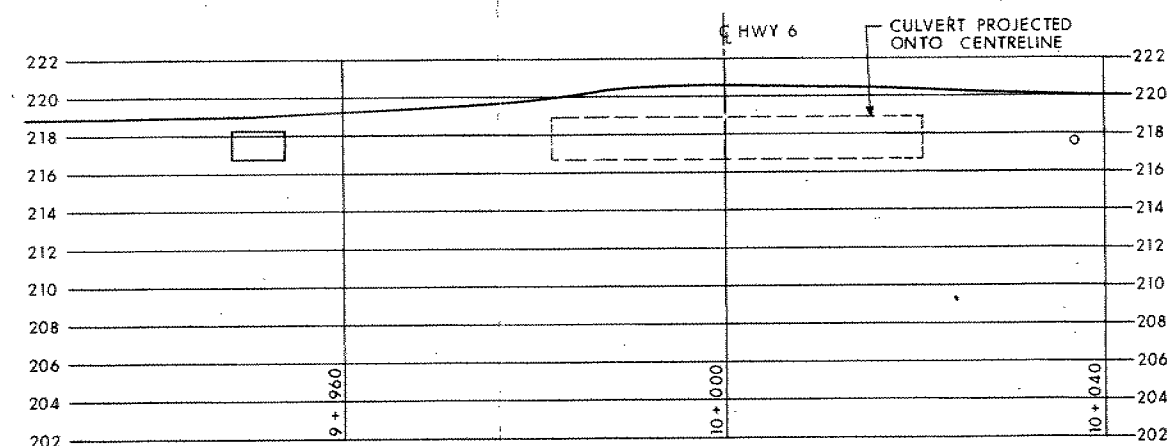
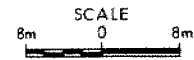
ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT 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	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						



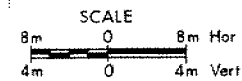
SECTIONS



PLAN



PROFILE TWENTY RD



METRIC

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

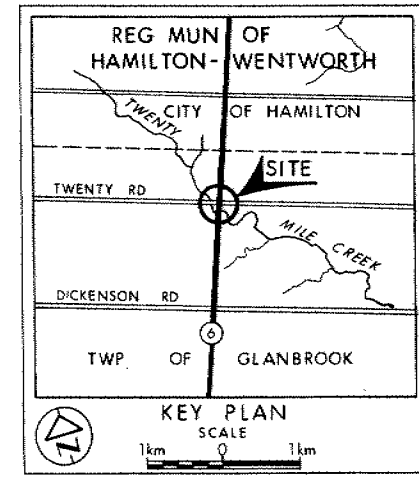
CONT No
WP No 16-96-00

TWENTY MILE CREEK

BORE HOLE LOCATIONS & SOIL STRATA



SHEET



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation 1996 07

No	ELEVATION	STATION	OFFSET
1	217.3	9+981.0	15.7m RT
2	217.6	9+947.0	8.2m RT
3	219.3	9+975.0	2.9m RT
4	217.4	9+959.0	9.7m LT
5	220.0	10+019.1	15.7m RT

NOTE

For Subsoil information of BH-3
Refer to Record of Borehole Sheet

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
1			

Geocres No 30M4-78

HWY No 6	SUBMD T5	CHECKED TS	DATE 1996 11 24	DIST CR
DRAWN DT	CHECKED TS	APPROVED		SITE 36-301C
				DWG 169600-A

