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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



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FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

CONT 92-78

WP 632-89-01 DIST 6
HWY 400 STR SITE 37-127

Proposed Crossing at
Rutherford Road and Highway 400

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GEOCRES

30M13-109

DATE JUL 30 1990

FOUNDATION INVESTIGATION REPORT

For

Proposed Crossing at
Rutherford Road and Highway 400
W.P. 632-89-01, Site 37-127
Highway 400, District 6, Toronto

INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the aforementioned site. It is proposed to reconstruct the entire Hwy. 400-Rutherford Interchange immediately north of the existing interchange. A two span structure is proposed.

The proposed profile grade of Rutherford Road is approximately 232.5 m to 233.5 m with an elevation ascension occurring in the westerly direction. The grade of Hwy. 400 will remain unchanged at approximately 225.5 m. The natural ground surface is generally at El. 223 to 224 and hence up to 10.5 m of approach fill will be required to the structure.

SITE DESCRIPTION AND GEOLOGY

The site is located immediately north and adjacent to the existing Hwy. 400-Rutherford Road Overpass approximately 0.5 km south of Canada's Wonderland which is bounded by Major Mackenzie Drive to the north and Hwy. 400 and Jane Street to the west and east respectively in the Town of Vaughan, Regional Municipality of York.

Two structures adjoin each other at the site. One structure carries Rutherford Road over the existing Hwy. 400 and the second westerly structure carries Rutherford over the existing EW-S ramp. Both structures are reinforced concrete rigid frame structures with spans of approximately 29 m and 8.2 m for the Hwy. 400 and ramp structure respectively. The Hwy. 400 structure was constructed in the early 1950's and the ramp structure extension was built under Contract #79-100.

The aging of the older structure is evidenced by the spalling and delamination of the concrete. This is particularly pronounced at the bridge deck soffit

where exposed corroded reinforcing steel is present. Despite the concrete deterioration, no signs of distress caused by foundation subsidence or earth pressure exists. No signs of structural distress or deterioration are present for the newer ramp structure.

Approach fills for the structure and the EW-S ramp contour the generally flat to gently undulating terrain at the site. Approach fills to the structures are in the order of 8 m and embankment fills for the existing EW-S ramp is in the order of 4 m.

Corrugated steel plate culverts and shallow surface drainage channels also occupy the site as part of the surface runoff drainage regime.

The land surrounding the site consists mainly of grassland and forestland.

Physiographically, the site lies in the geological domain known as the Bolton Area, an area that covers approximately 1200 square kilometres located at the northwestern border of the Municipality of Metropolitan Toronto. The Bolton area has drumlins, till plains, moraines and numerous other features associated with deglaciation. The area was covered with the Wisconsin glacier of the Pleistocene period that advanced into the region approximately 50,000 years ago and retreated approximately 15,000 years ago.

The overburden deposits at the site consist of moraine tills of the Halton Till Formation underlain by glaciolacustrine sediments deposited by Lake Peel, a body of water impounded between lobes of projecting ice. The Halton Till is primarily a clayey silt till composed of varying percentages of clay, silt, sand and gravel. The glaciolacustrine deposits generally consist of stratified silt, clayey silt and/or silty clay.

The surficial deposits of the Cenozoic era are underlain by bedrock of the Paleozoic era. Bedrock consists of grey, thinly bedded shales with interbedded limestone from the Dundas Meaford Formation. Bedrock topographical maps reveal

that the bedrock exists at depths approximately 70-80 m below the natural ground surface at the site location.

INVESTIGATION PROCEDURES

Field Investigation

The fieldwork for the investigation was carried out between 90 03 27 to 90 04 11 and consisted of 7 sampled boreholes advanced to depths ranging from 12.6 m to 41.6 m below the ground surface. The elevation of the ground surface at the borehole locations range from 223.2 to 225.2. Four boreholes (BH's 8-11 inclusive) previously advanced between 78 11 24 and 78 12 04 in conjunction with the existing structures have also been included in this report.

Track mounted CME 55 equipment employing hollow stem augering techniques and also washboring/casing methods was used to advance the boreholes in the overburden. In general, disturbed subsoil samples were retrieved at 0.7 m for the surficial 6 m and 1.5 m thereafter. Sample retrieval was conducted in accordance with the Standard Penetration Test (ASTM D1586). All samples were identified in the field and then returned to the laboratory for applicable testing.

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

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

Laboratory Analyses

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

- 1) Atterberg Limits
- 2) Grain Size Distributions
- 3) Unit Weights
- 4) Natural Moisture Contents

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

SUBSURFACE CONDITIONS

The soil stratigraphy at the site consists of fill material that comprises the approach embankments to the existing structure. This fill material consists of an irregular mixture of clayey silt to sandy silt. The height of the embankment fill material is in the order of 8 m.

Underlying the fill material and present surficially adjacent to the fill at all proposed structure foundation locations and the east approach location, exists a deposit of clayey silt to silty clay with some sand and a trace of gravel. The thickness of this till deposit varies across the site ranging from 2.3 m to 4.5 m at the proposed east abutment, 6.6 m at the proposed pier location and 3.1 m to 6.1 m at the proposed west abutment. The consistency of the deposit also varies ranging from firm to hard.

At the west approach (BH 4), the clayey silt to silty clay deposit is overlain by a cohesionless deposit of sand to silty sand. The thickness of this deposit is approximately 6.6 m and its state of denseness varies from loose to compact.

Underlying the clayey silt to silty clay till deposit, a cohesionless deposit of varying percentages of silt and sand exists. The extent of this deposit was not determined at the proposed east abutment and pier locations. At the west abutment location, the thickness of this deposit varied from approximately 1.5 to 6.1 m. The deposit is generally in a compact to dense state of condition.

At the west abutment location, the cohesionless silt and sand deposit is underlain by a second till deposit consisting of a clayey silt with some sand and a trace of gravel. This deposit also contains random interbeds of layered clayey silt of glaciolacustrine origin. The thickness of this deposit, determined at BH 7, is equivalent to 10.7 m.

A deposit of sand and gravel underlies the lower clayey silt deposit at the west abutment location. This deposit extends to a depth of approximately 33.5 m below the existing natural ground surface (EL. 190 m) and is approximately 14.4 m in thickness. The deposit has a very dense state of denseness.

The sand and gravel deposit is further underlain by a cohesive stratum consisting of a clayey silt. This stratum has a thickness, as determined at BH 7, of approximately 4.6 m and has a hard consistency.

The cohesive clayey silt stratum is underlain by a very dense silt deposit that contains traces of sand. The extent of this deposit was not determined in the investigation.

The boundaries between the various soil types, in situ and laboratory test results as well as groundwater levels established at the time of investigation, are shown on the attached Record of Borehole sheets in the Appendix. A plan of the site illustrating the locations and elevations of the boreholes and subsoil stratigraphical sections are provided on Dwg. 6328901-A.

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

Clayey Silt to Sandy Silt (Fill Material)

The existing Rutherford Road approach embankments consist of an irregular mixture of clayey silt to sandy silt and hence the material varies in behaviour from cohesive to cohesionless respectively. Traces of gravel and occasional shale fragments are also present within the fill. Traces of organics and topsoil were found at the bottom of the embankment fill immediately overlying

the natural subgrade. Figure 1 in the Appendix illustrates grain size distribution curves for representative samples of the fill material.

The material is brown in colour and contains natural moisture contents ranging from 10 to 20%. The unit weight of the material ranges from 21.1 to 22.1 kN/m³.

Based on 'N' values obtained from the Standard Penetration Test ranging from 7 blows/0.3 m to 43 blows/0.3 m, the fill material is in a firm to hard/loose to dense state. In general, however, the fill material can be categorized as very stiff/compact.

Sand to Silty Sand

At the proposed west approach embankment (BH 4), a surficial cohesionless deposit of brown sand to silty sand exists. This deposit is approximately 6.6 m in thickness and based on 'N' values obtained from the Standard Penetration Test ranging from 8 blows/0.3 m to 28 blows/0.3 m, the deposit can be categorized as having a loose to compact denseness.

Clayey Silt to Silty Clay, some Sand, trace of Gravel (Glacial Till) (Upper)

The native surficial deposit across the site, except at the west approach embankment, is composed of a clayey silt to silty clay with some sand and also traces of gravel. The deposit is a till deposit of glacial origin and hence, although not encountered during the field investigation, boulders and cobbles are characteristic components of these deposits and hence can exist. Figure 2 in the Appendix provides a grain size distribution envelope illustrating the gradation of this deposit. The envelope reveals that a significant portion of the deposit is comprised of clay and silt (71 to 90%) and sand composition ranges from 10 to 25%.

Traces of black organics are also present within the surficial metre or so of the deposit. This deposit has been completely oxidized and hence is brown in colour.

The thickness of the deposit varies across the site. At the proposed east abutment, the thickness ranges from 2.3 to 4.5 m, whilst at the proposed pier and west abutment the thickness is 6.6 m and 3.1 to 6.1 m respectively.

Atterberg Limit tests were carried out to define the behaviour and plasticity of the fine grained portion of the soil and the results are plotted in Figure 3 in the Appendix. A summary of the indices is provided in Table 1 below.

Table 1 - Clayey Silt to Silty Clay

	<u>Range</u>	<u># of Tests</u>
Natural Moisture Content (w%)	11-16	8
Liquid Limit (w _L %)	21-45	8
Plasticity Index (I _p %)	8-25	8
Unit Weight (kN/m ³)	19.5-22.4	9

The results reveal that the fine grained portion of the deposit varies randomly in plasticity ranging from low (clayey silt) to intermediate (silty clay). The fine grained portion of the deposit constitutes the main component of the deposit and hence the behaviour of the overall deposit is governed by this fine grained portion.

Standard Penetration Tests carried out in this deposit revealed 'N' values ranging from 6 blows/0.3 m to 74 blows/0.3 m. Lower 'N' values were obtained at the proposed west abutment location where values between 6 to 20 were more frequent at varying depths. Aside from these localized lower 'N' values, 'N' values are generally in the 25 to 30 range. Based on these 'N' values, the material can be described as having a firm to hard consistency.

Silt, trace/some Sand

Underlying the clayey silt to silty clay till deposit exists a cohesionless deposit composed of a silt with traces to some sand. The extent of this deposit was not established at the proposed east abutment and pier but at the west abutment the thickness of the deposit varied between 1.5 and 6.1 m. Minor traces of gravel are also present within this deposit.

A grain size distribution envelope for this deposit is provided in Figure 4 in the Appendix.

The deposit has experienced varying degrees of oxidization and hence varies in colour with depth from brown to grey.

Standard Penetration Tests carried out in this deposit revealed 'N' values ranging from 9 blows/0.3 m to 108 blows/0.3 m indicating that the deposit ranges in denseness from loose to very dense. In general, 'N' values are in the 25 to 40 range and consequently, the deposit can be categorized as having a compact to dense denseness.

Clayey Silt, some Sand, trace Gravel (Glacial Till) (Lower)

At the proposed west abutment location, a second till deposit consisting of a clayey silt with some sand and a trace of gravel underlies the silt deposit. The thickness of this deposit was determined at BH 7 only and is equivalent to 10.7 m.

The deposit also contains random interbeds of clayey silt that is layered and of glaciolacustrine origin. Boulders and cobbles are generally characteristic components of till deposits and although not encountered in the investigation can exist. Grain size distribution curves derived from representative samples of this material is illustrated in Figure 5 in the Appendix.

Atterberg Limit Tests were carried out to evaluate the behaviour and plasticity of the fine grained portion of the soil and the results are plotted in Figure 6 in the Appendix and summarized in Table 2 below.

Table 2 - Clayey Silt

	<u>Range</u>	<u># of Tests</u>
Natural Moisture Content (w%)	11-26	5
Liquid Limit (w _L %)	19-35	5
Plasticity Index (I _p %)	6-18	5

The test results reveal that the deposit is predominantly of low plasticity.

Standard Penetration Tests carried out in this deposit revealed 'N' values ranging from 2 blows/0.3 m to 60 blows/0.3 m. Localized weaker zones that reflect in the lower 'N' value exist randomly in the deposit, particularly at BH 3 where 'N' values ranged from 2 blows/0.3 m to 9 blows/0.3 m. In general, the deposit has a very stiff to hard consistency with localized very soft to firm zones.

Sand and Gravel

At the west abutment location, the clayey silt lower till deposit is underlain by a cohesionless sand and gravel deposit that is approximately 14.4 m in thickness. Figure 6 in the Appendix illustrates the varying grain size distributions of the sand and gravel.

The sand and gravel deposit is water bearing and consequently, when the deposit was penetrated in the open borehole, soil cave-in resulted due to unbalanced hydrostatic head. Washboring and casing techniques were required to facilitate borehole advancement through this deposit.

Standard Penetration Tests carried out in this deposit revealed 'N' values ranging from 34 blows/0.3 m to 75 blows/0.3 m, indicating a denseness of dense to very dense. In general, 'N' values exceeded 50 blows/0.3 m and hence the deposit can be categorized as very dense.

Clayey Silt

The cohesionless sand and gravel deposit is underlain by a stratum of clayey silt that is cohesive and of low plasticity. Its thickness at BH 7 was equivalent to 4.6 m and based on 'N' values of 70 blows/0.3 m and 95 blows/0.3 m, the clayey silt stratum can be categorized as having a hard consistency.

Silt

The clayey silt stratum is further underlain by a cohesionless deposit of silt with traces to some sand. This stratum exists at elevation 185.4 m which is approximately 38.1 m below the natural ground surface at the proposed west

abutment location. The extent of this deposit was not determined during the investigation. Figure 7 in the Appendix illustrates a grain size distribution curve for this soil.

Based on 'N' values ranging from 100 blows/0.15 m to 165 blows/0.3 m, the soil can be categorized as very dense. The soil did not require protection against cave-in during borehole advancement using conventional washboring techniques.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in the open boreholes. In general, the water levels obtained at the time of the investigation were below or close to the depth of the borings that were advanced except at BH 7. The relatively shallow boreholes were advanced to depths ranging from 12.6 m to 14.2 m or Elevations ranging from approximately 209 to 212.5 m. At the deeper BH 7, the elevation of the groundwater level was approximately 216 m, approximately 7.5 m below the natural ground surface. This latter groundwater level reflects the subartesian water condition in the lower sand and gravel deposit.

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 replace the existing Rutherford Road-Highway 400 Interchange with an interchange located adjacent and immediately north of its existing location. The interchange will include a two span structure (equal span of approximately 44 m) and associated W-N and E-S ramps. At the time of the foundation request, consideration had been given to employing either a steel box girder deck or a cast in place post-tensioned deck.

The width of the structure will be designed to carry four lanes of traffic on Rutherford over the Hwy. 400. The maximum total width of the structure including the shoulders and tapered ramps is 33.40 m. The structure is also being designed for a minimum vertical clearance of 5.15 m.

The natural ground surface in the approach areas is approximately at Elevation 223 to 223.5 m. The proposed profile grade of Rutherford Road varies from approximately 232.5 m to 233.5 m where the elevation increases in a westerly direction. Consequently, approach fills in the order of magnitude of 9.5 to 10.5 m will be required.

Two bridges and a EW-S ramp constitute the existing Rutherford Road-Hwy. 400 Interchange. One of the the structures carries Rutherford Road over the Hwy. 400 whilst the most westerly structure carries Rutherford Road over the EW-S ramp. The two bridges are adjoined by an approach embankment located west of Hwy. 400 and east of the EW-S ramp. Both structures are reinforced concrete rigid frame structures supported on shallow foundations within the surficial till deposit. The spread footings are founded at El. 223.6 m and 224.2 m for the Hwy. 400 and EW-S ramp structure respectively. Although the Hwy. 400 structure has experienced some deck concrete spalling and delamination, the structures show no visible signs of distress as a result of any foundation and/or geotechnical displacements.

Approach fills at the site are in the order of magnitude of 8 m. The approach fills at the Hwy. 400 structure are supported by toe walls approximately 1 m in height. There appears to be no visible signs of instabilities in the approach embankments.

Recommendations pertaining to the following foundation and geotechnical components of the design and construction of the new interchange are included in the scope of this report.

- 1) Structure Foundations
- 2) Approach Embankments
- 3) Construction Considerations

1) Structure Foundations

East Abutment

In view of the competent nature of the surficial soils at the proposed east abutment location, the structure can be supported on a shallow foundation. The conventional spread footings can be designed and constructed as discussed below.

a) Compacted Granular 'A' Pad

Structure foundations can be "perched" on a compacted Granular 'A' pad placed on the existing native soil. For purposes of the O.H.B.D.C., the following bearing capacities are recommended.

Table 3 - Perched Abutment on Granular 'A'

Bearing Capacity at S.L.S. Type II	350 kPa
Factored Capacity at U.L.S.	900 kPa

The settlements induced as a result of the applied footing pressure will be elastic in nature and consequently is expected to take place during or immediately following the construction period. The magnitude of this settlement is anticipated to be within 25 mm, provided the subsoil/granular material is not softened/loosened by construction or related activities and the granular pad is constructed to the specifications described below.

Figure 8 in the Appendix illustrates the recommended geometry for the design of the perched abutment foundation. The thickness of the pad shall be a minimum 2 m above the existing natural ground surface and a minimum edge distance of 1 m from the edge of the footing to the crest of the granular 'A' pad shall be provided. The figure also illustrates that frost protection is required to protect the founding soil. All footings must have 1.2 m of earth cover.

The Granular 'A' material must be placed and compacted to achieve 100% of the Proctor maximum dry density as outlined in OPSS 501.08.02 (Method A). Quality control in the form of material inspection and field density measurements shall be conducted. Any softened and/or organic material present within the natural subgrade must be removed prior to the placement of the granular pad. In addition, any existing embankment fill material shall be completely subexcavated.

The computation of the sliding resistance of the foundation shall be computed in accordance with Section 6-7.3.3.2 of the O.H.B.D.C. An unfactored friction angle of 35° can be used between the concrete footing and the Granular 'A' material. If additional sliding resistance is required, consideration can be given to employing shear keys beneath the footing. The passive resistance developed by the shear key can be computed using the parameters of the granular material tabulated in Table 8 (see "Lateral Earth Pressures on Structures" in subsequent section of the report).

b) Spread Footings on Native Soil

Alternatively, east abutment structure foundations can be founded on the native clayey silt to silty clay till deposit at an elevation of 222.3 m. For purposes of the O.H.B.D.C., the bearing capacities tabulated in Table 4 are recommended.

Table 4 - Spread Footings on Native Soil

Bearing Capacity at S.L.S. Type II	300 kPa
Factored Capacity at U.L.S.	450 kPa

Settlement of the foundation subsoil as a result of the applied footing pressure will be elastic and hence will develop during or immediately following construction. The total and differential settlement is expected to be within 25 mm.

An unfactored friction angle of 30° can be used between the concrete footing and the native soil in the computation of the sliding resistance of the foundation.

All loosened/softened, foreign and/or organic material present at the footing founding elevation shall be removed and replaced with a granular material and/or mass concrete. In addition, to preserve the founding soil during construction, it is recommended that a concrete working slab be placed in advance of the concrete footing construction.

All footings must be provided with adequate frost protection. Hence, the foundation must be designed with 1.2 m earth cover or equivalent frost protection.

No dewatering difficulties are anticipated during foundation construction because excavation will not intercept the groundwater table which is significantly lower than the footing founding elevation and because of the impervious nature of the subsoil. Conventional sump pumping techniques will suffice in discharging any 'perched' or surface runoff water. Temporary slopes to facilitate the footing construction can be constructed no steeper than $1\frac{1}{2}H:1V$.

West Abutment

In view of localized weaker material within the surficial soils at the proposed abutment location and the fact that a deep foundation unit would require embedment lengths in the order of 38 m to provide any economical capacity, it is recommended that the west abutment foundation be founded on a compacted Granular 'A' pad. For purposes of the O.H.B.D.C., the following bearing capacities are recommended.

Table 5 - Perched Abutment on Granular 'A'

Bearing Capacity at S.L.S. Type II	350 kPa
Factored Capacity at U.L.S.	900 kPa

The granular pad shall be a minimum 3 m thickness.

All other design and construction considerations discussed previously in conjunction with the Granular 'A' pad at the east abutment are also applicable to the west abutment foundation.

Pier

The centre pier structure foundations can also be founded on conventional spread footings within the surficial till deposit consisting of a clayey silt with some sand and a trace of gravel. In view of the presence of some weaker material within the surficial 2.5 to 3 m of this deposit, it is recommended that this material be subexcavated to an elevation of 222.2 m and replaced with a Granular 'A' material. The spread footing foundation can then be placed on the Granular 'A' material. For purposes of the O.H.B.D.C., the following bearing capacities are given provided that the Granular 'A' pad is a minimum 1.5 m thickness and a minimum edge distance of 1 m beyond the footings included for its entire thickness is given.

Table 6 - Pier Foundation on Granular 'A'

Bearing Capacity at S.L.S. Type II	350 kPa
Factored Capacity at U.L.S.	800 kPa

The selection of the founding elevation of the pier foundation on the Granular 'A' material must include the frost penetration depth criteria of 1.2 m.

The settlements induced as a result of the applied footing pressure will be elastic in nature and consequently is expected to take place during or immediately following the construction period. The magnitude of this settlement is anticipated to be within 25 mm provided the subsoil/granular material is not softened/loosened by construction or related activities. Any softened/loosened material at the founding elevation shall be removed.

As previously discussed, the sliding resistance can be computed using an unfactored friction angle of 35° between the concrete footing and the Granular 'A' material.

Alternatively, the pier foundation can be founded on spread footings within the surficial clayey silt till deposit at an elevation of 222.2 m. For purposes of the O.H.B.D.C., the capacities tabulated in Table 7 are recommended.

Table 7 - Pier Foundation on Native Soil

Bearing Capacity at S.L.S. Type II	300 kPa
Factored Capacity at U.L.S.	450 kPa

All other design and construction considerations previously discussed for the spread footing on native soil alternative at the proposed east abutment are equally applicable at the pier footing.

No dewatering problems are anticipated for either foundation alternative because excavation will not intercept the groundwater table and because of the impervious nature of the subsoil. Conventional sump pumping techniques will suffice in discharging any 'perched' or surface runoff water. Temporary slopes to facilitate the footing construction can be constructed no steeper than $\frac{1}{2}H:1V$.

2) Approach Embankments

Approach fills in the order of magnitude of 8.5 m at the east approach and 10.5 m at the west approach will be required for the structure approach embankments. Discussion of the lateral earth pressures on the structure, stability, settlement and construction of the approach embankments are provided below.

Lateral Earth Pressure on Structure

Free draining material such as Granular 'A' or Granular 'B' shall be used within a wedge behind the abutments and retaining walls bounded by a plane rising at 60° to the horizontal as shown in Figure 6-9.6.1 in the O.H.B.D.C. The application of granular material combined with weep holes in the abutment walls to drain any accumulation of water in the backfill will prevent hydrostatic pressure build-up. Design parameters of the soil are given in Table 8 below.

Table 8 - Backfill Properties

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction (ϕ) (unfactored)	35°	30°
Unit Weight (kN/m^3)	22.8	21.2
Coefficient of Active Earth Pressure (K_a)*	0.33	0.4
Coefficient of Earth Pressure at Rest (K_0)*	0.5	0.58

*These earth pressure coefficients apply to horizontal backfill surfaces only. The appropriate consideration shall be given to account for sloping surface backfill. (Factored angles of internal friction were employed in the computation).

The earth pressure coefficient at rest is to be used in design if the abutment/retaining walls are rigid and unyielding.

Stability/Settlement

In view of the competent nature of the subsoil, no deep-seated stability problems are anticipated for the proposed embankment fill heights. However to ascertain the internal (surficial) stability of the embankment fill, it is recommended that for embankment fill heights exceeding 8 metres, the approach embankment be constructed with a stabilizing midheight berm of 2 m width. The berm should be constructed at a nominal 2% gradient to facilitate surface runoff. In addition, an effective erosion control protection scheme, such as sodding, should be provided to protect the exposed slopes.

Settlements in the order of magnitude of 50 mm attributable to the elastic recompression of the native subsoil and settlement within the fills under its own weight are anticipated. It is predicted that the majority of the settlements will be realized during or immediately following the construction of the embankment.

Embankment Construction

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

Any new fill placement shall be connected to existing fills by "benching" the earth slopes in accordance with MTO Standards (OPSD 208.01).

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

3) Construction Considerations

In order to maintain traffic on both Hwy. 400 and Rutherford Road throughout the construction of the new structure and demolition of the existing structure, a combination of a roadway protection scheme and detour staging has been proposed. Recommendations for the design and construction of the roadway protection scheme is discussed below.

Temporary Roadway Protection

To facilitate the construction of the structure foundations for the proposed structure whilst maintaining traffic on Rutherford Road, a temporary shoring system will be required to support the existing approach embankments at the site. A timber lagging-soldier pile shoring wall is recommended at the site.

The design of the shoring system shall include the appropriate earth pressures computed in accordance with Section 6.6.1.2 of the O.H.B.D.C. The loadings induced by the surcharge traffic and adjustment for any sloping surfaces shall be incorporated in the design. Soil design parameters to facilitate the shoring wall design are summarized in Table 9 below.

Table 9 - Shoring Design Soil Parameters

<u>Soil Type</u>	<u>Saturated Unit Weight (kN/m³)</u>	<u>Unfactored Effective Shear Strength Parameters (0°)</u>
Fill	22	30
Clayey Silt/silty Clay (Till) (Upper and Lower)	21	35
Silt trace/some Sand	20	32

The shoring system must be designed to ascertain earth pressure equilibrium. This equilibrium can be achieved by a cantilevered wall or raker supported wall. The feasibility of a cantilever wall is contingent on the practicality of the soldier pile embedment length.

If the embedment length is found to be excessive, a more economical design may involve the application of rakers. Rakers can be installed in front of the wall with footings founded in the surficial clayey silt to silty clay till deposit. The rakers must be installed while an earth berm remains in front of the soldier pile. Slots should be cut into this berm to install rakers before the supporting berm is removed.

Soldier piles can be installed employing conventional pile driving techniques.

A temporary roadway protection may also be required for the construction of the centre pier. A shoring system consisting of a cantilevered timber lagging-soldier pile wall or a braced timber lagging-soldier pile wall can be considered.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer and F. Reynolds, Construction Technician, utilizing equipment owned and operated by Malone's Soil Samples Ltd.

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



A handwritten signature in cursive script, appearing to read 'T. Sangiuliano'.

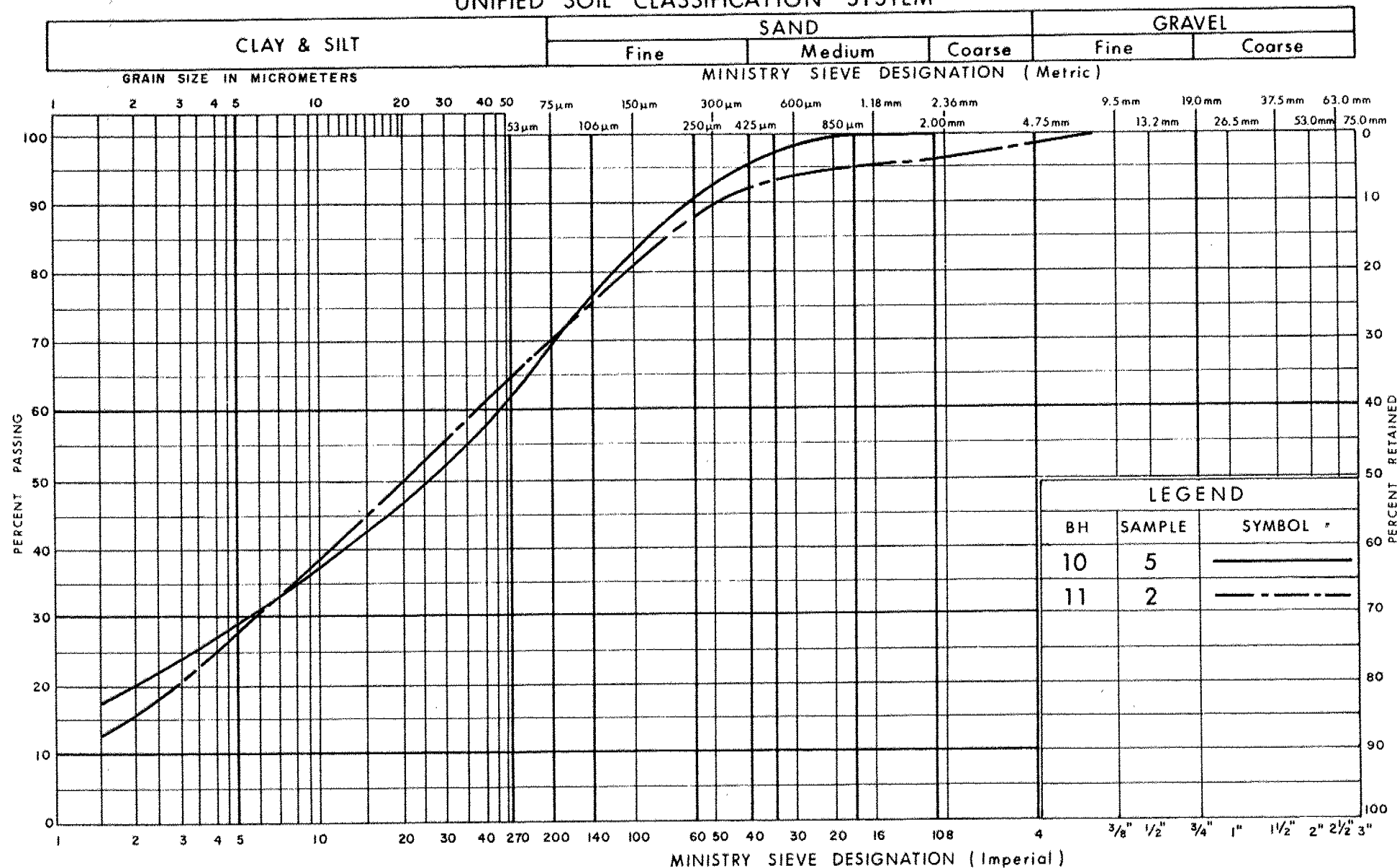
T. Sangiuliano, P.Eng.
Foundation Engineer

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

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM



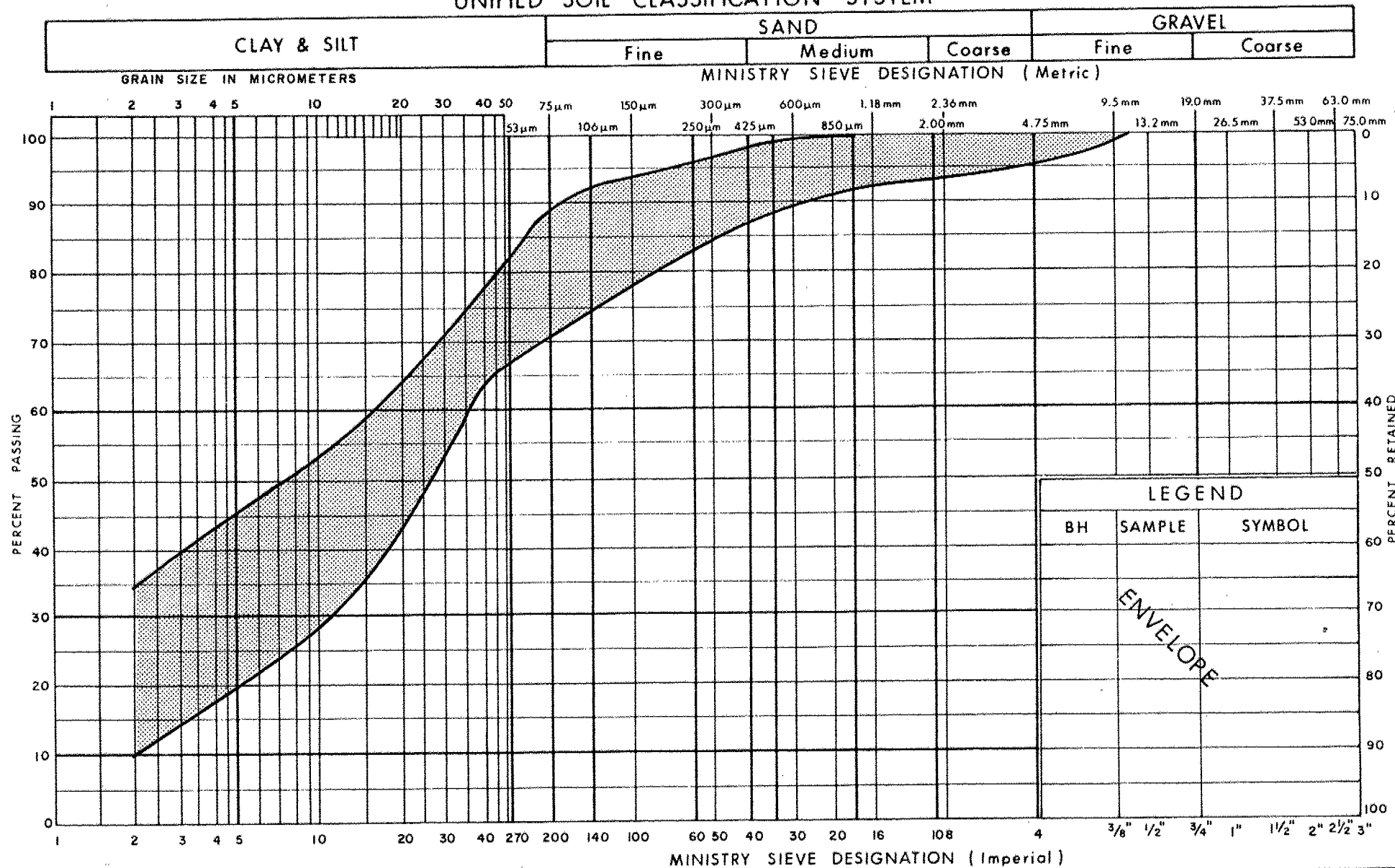
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SANDY SILT

FIG No 1

W P 632-89-01

UNIFIED SOIL CLASSIFICATION SYSTEM



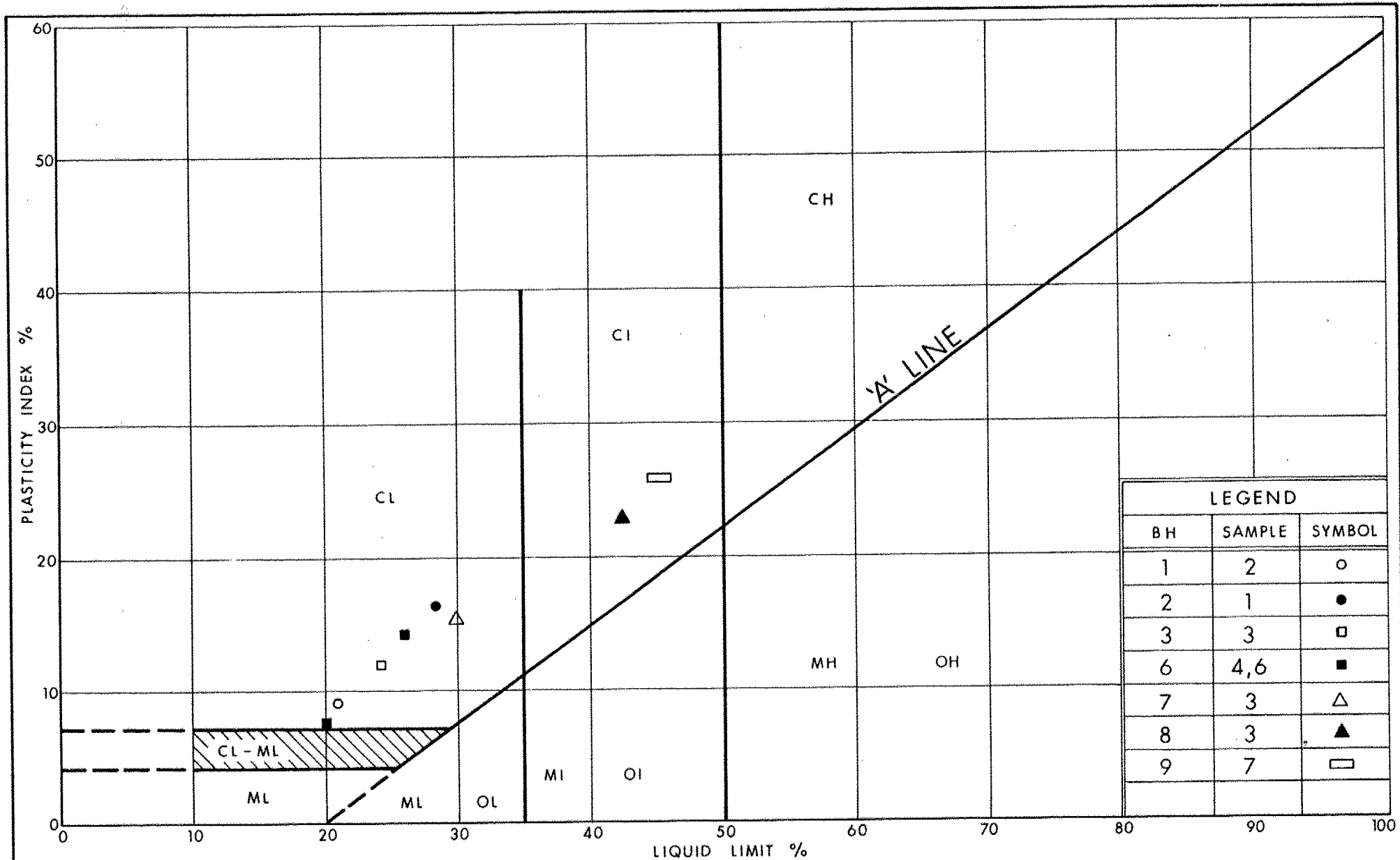
Ontario

Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY SOME SAND, TR GRAVEL
 (GLACIAL TILL) UPPER

FIG No 2

W P 632-89-01



Ministry of
Transportation

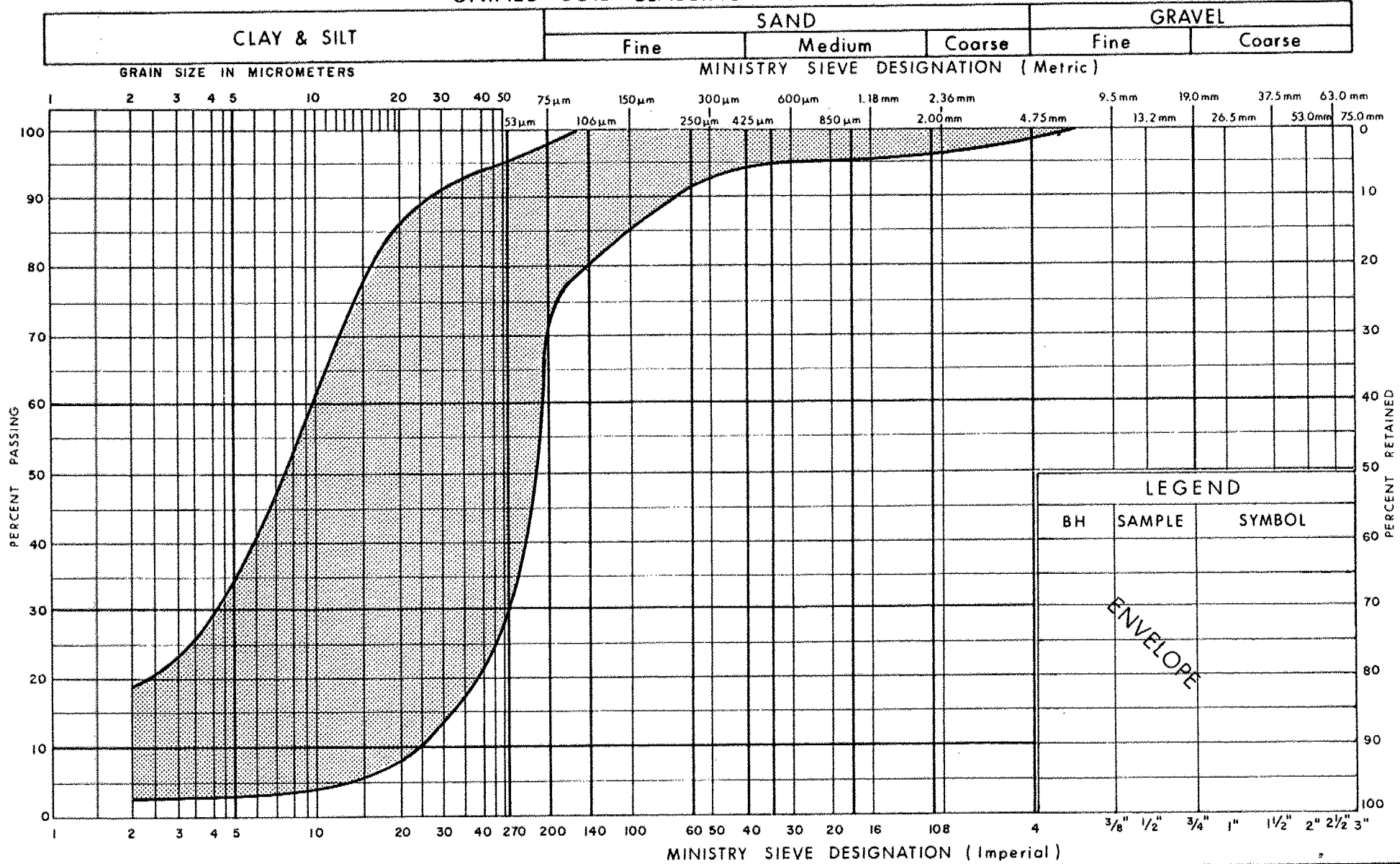
Ontario

PLASTICITY CHART
CLAYEY SILT, SOME SAND, TR GRAVEL
(GLACIAL TILL) UPPER

FIG No 3

W P 632-89-01

UNIFIED SOIL CLASSIFICATION SYSTEM



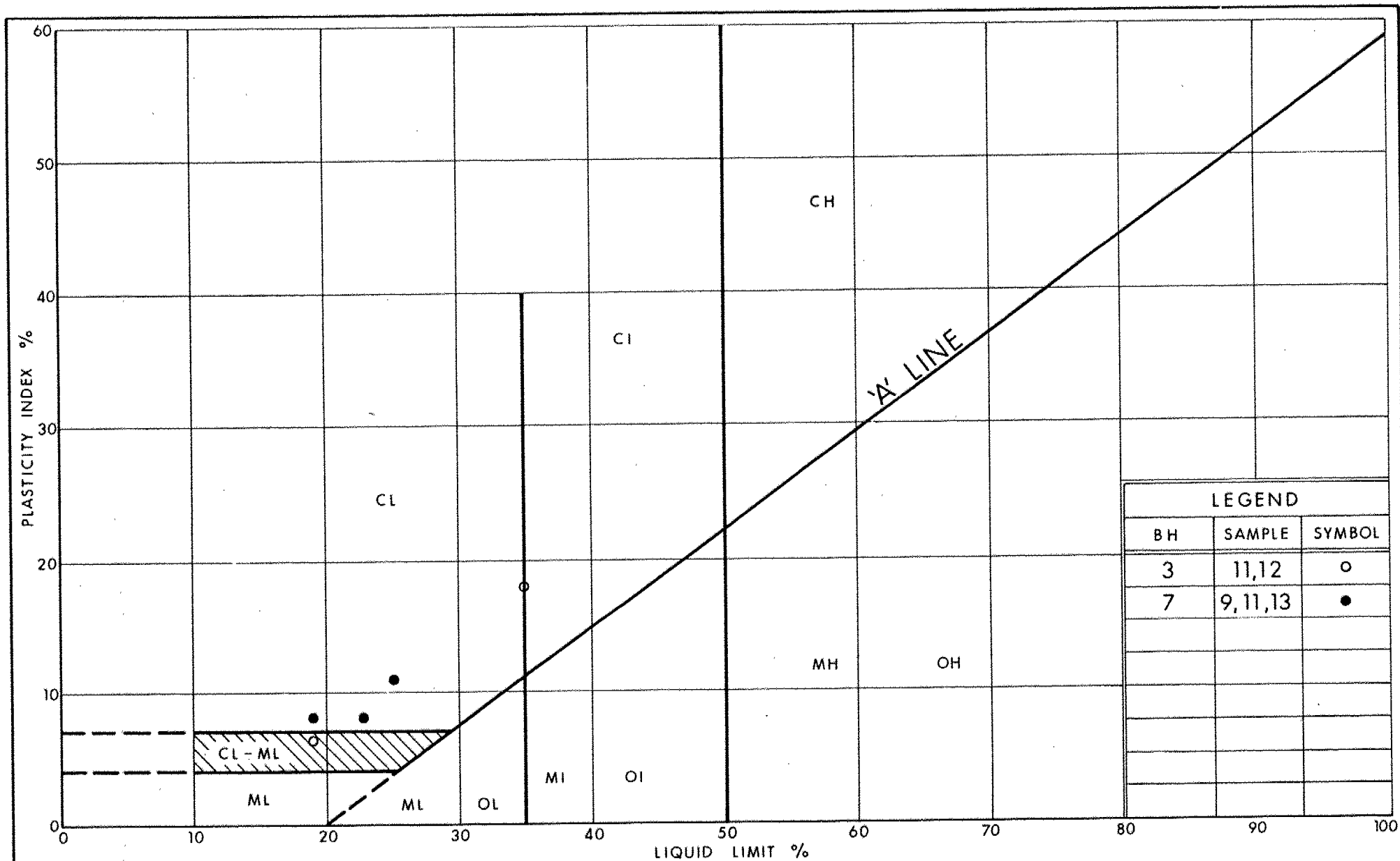
Ministry of
Transportation

Ontario

GRAIN SIZE DISTRIBUTION
SILT, TRACE / SOME SAND

FIG No 4

W P 632-89-01



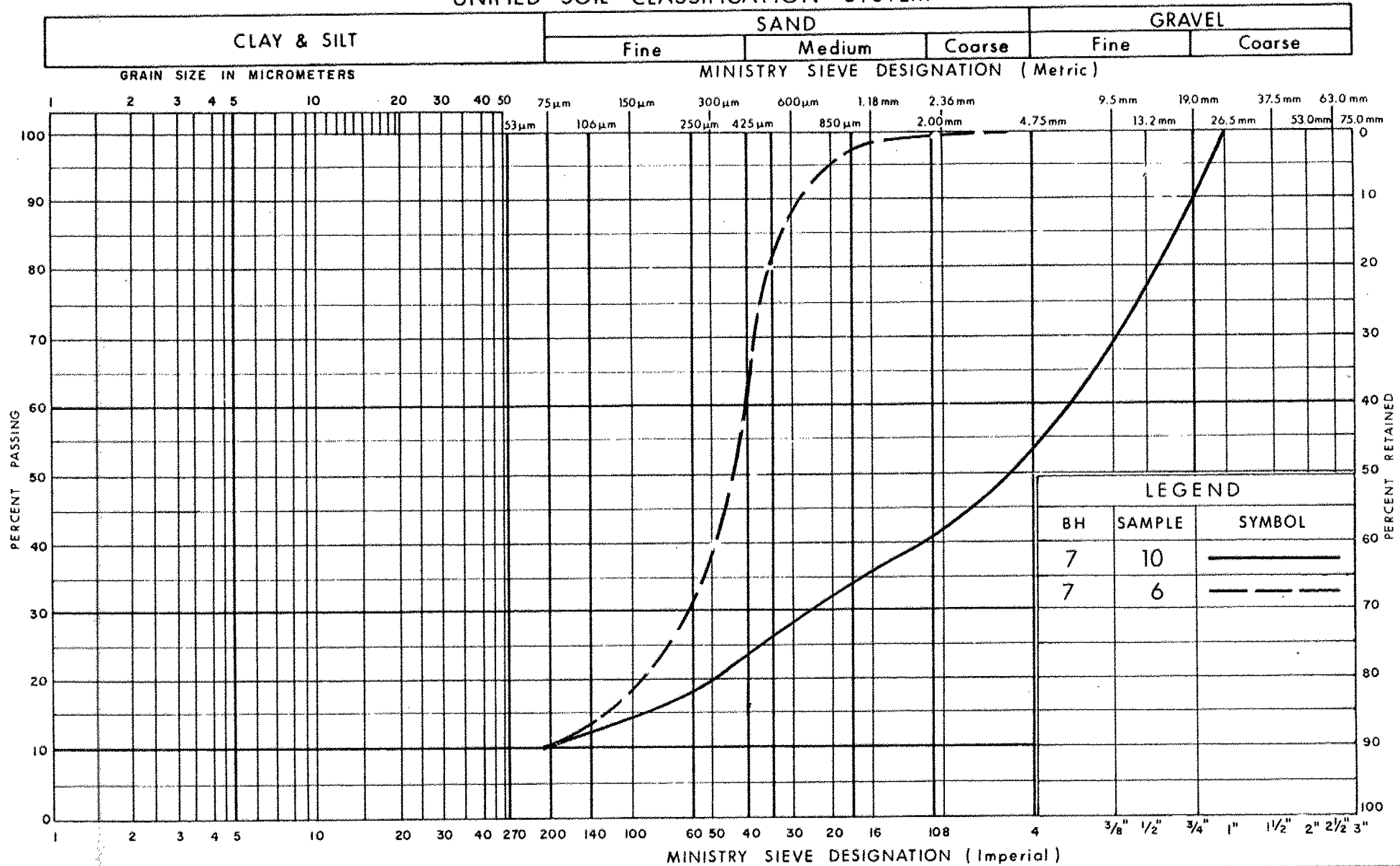
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Transportation
Ontario

PLASTICITY CHART
CLAYEY SILT, SOME SAND, TR GRAVEL
(GLACIAL TILL)

FIG No 5

W P 632-89-01

UNIFIED SOIL CLASSIFICATION SYSTEM



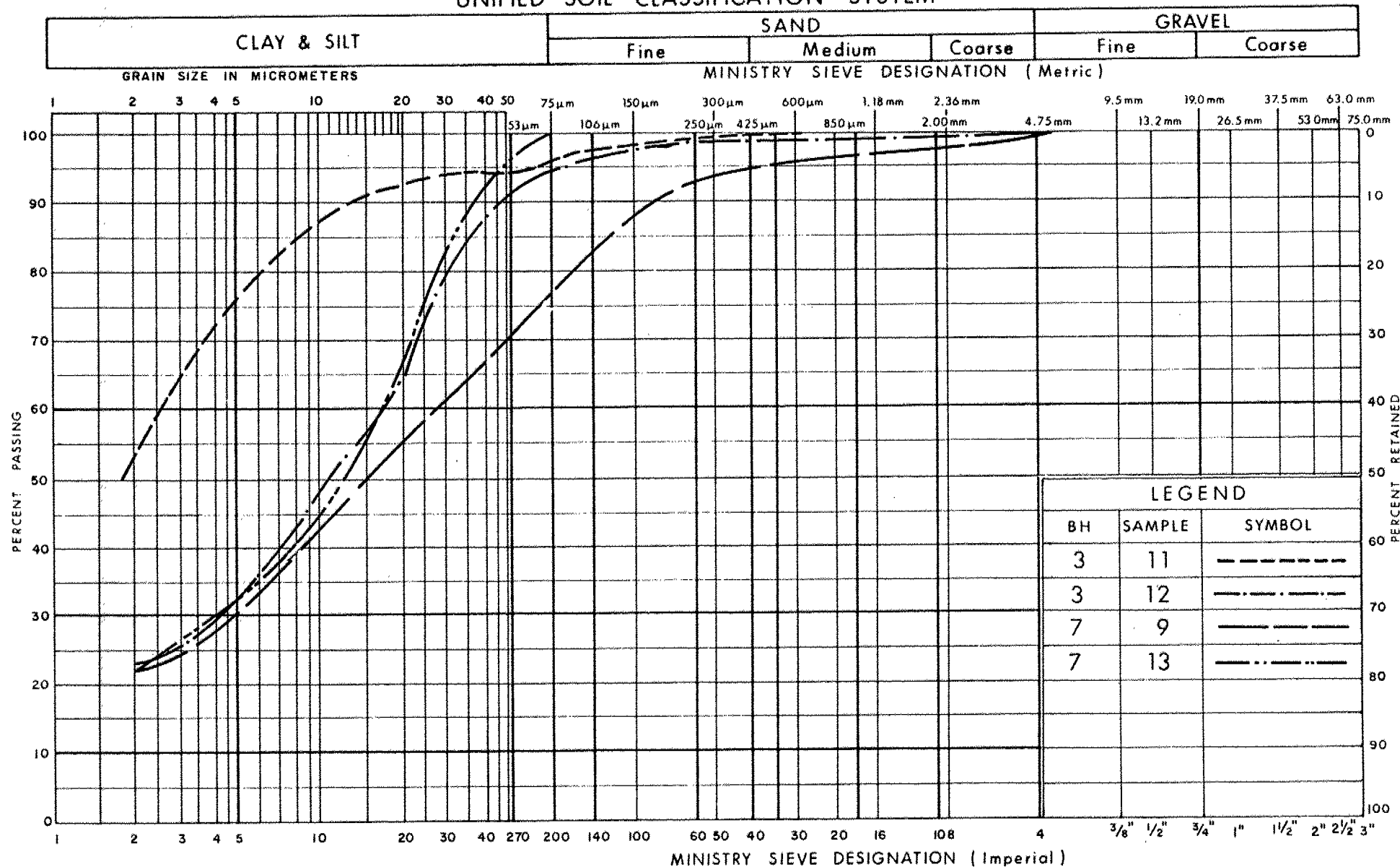
Ontario

Ministry of
TransportationGRAIN SIZE DISTRIBUTION
SAND & GRAVEL

FIG No 6

W P 632 - 89 - 01

UNIFIED SOIL CLASSIFICATION SYSTEM

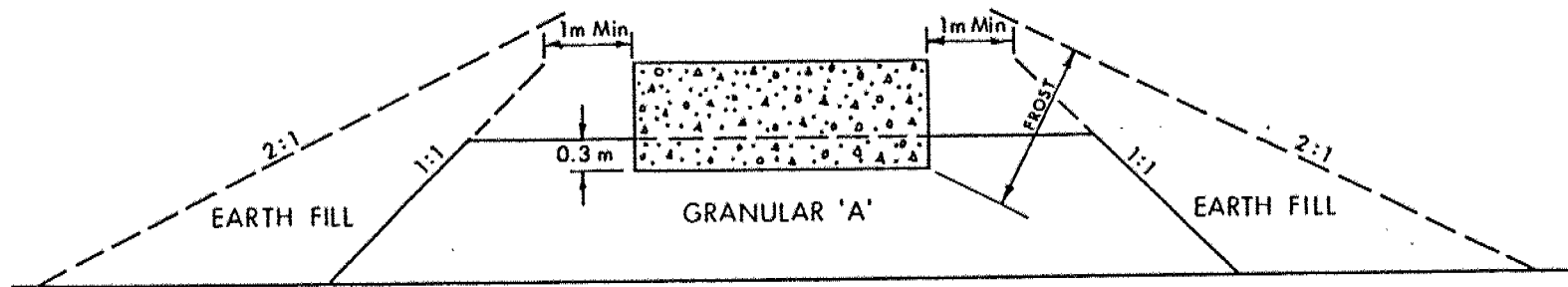


Ministry of
Transportation

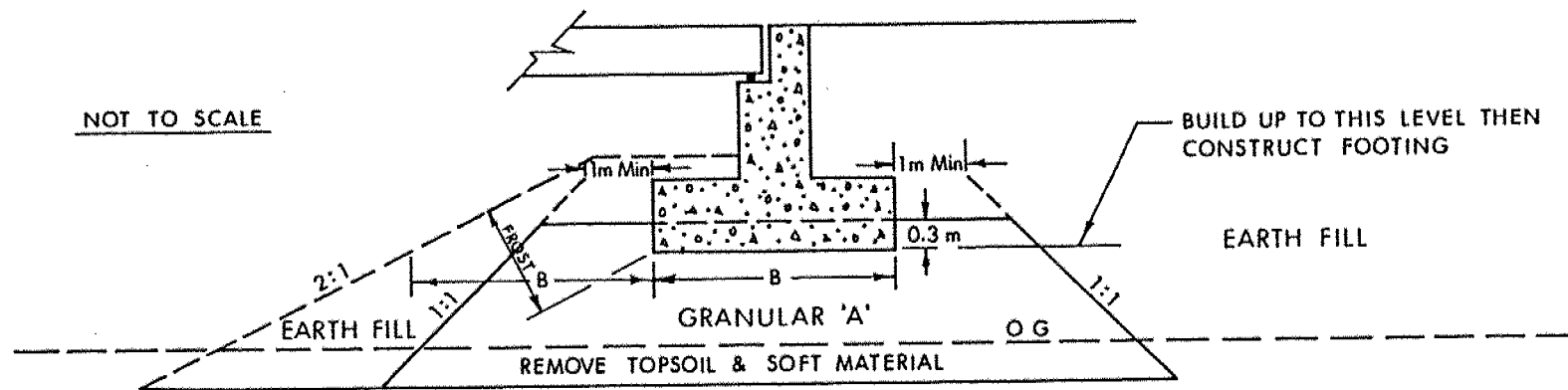
GRAIN SIZE DISTRIBUTION
CLAYEY SILT, SOME SAND, TR GRAVEL
(GLACIAL TILL) (LOWER)

FIG No 7

W P 632-89-01



X SECTION



LONGITUDINAL SECTION

NOTES:

- 1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.
- 3 - CONSTRUCT CONCRETE FOOTING.
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.



Ministry of
Transportation

ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

FIG No 8

W P 632-89-01

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'_{v0}	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. 632-89-01 LOCATION Co-ords: N 4 854 048.5 ; E 301 100 ORIGINATED BY FLR
DIST 5 HWY 400 BOREHOLE TYPE HS Auger COMPILED BY FLR
DATUM Geodetic DATE 90 03 27 CHECKED BY TS

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	20 40 60 80 100	20 40 60 80 100					
223.7	Ground Surface														
0.0	Clayey Silt, Some Sand, Trace Gravel (Glacial Till)		1	SS	11		222							22.4	0 19 64 17
221.4	Brown, Stiff to Hard		2	SS	26										
2.3			3	SS	36										
			4	SS	36										
			5	SS	29										
			6	SS	22										
			7	SS	17										
			8	SS	108										
			9	SS	66										
			10	SS	14										
211.1			11	SS	15										
12.6	End of Borehole														

RECORD OF BOREHOLE No 2

1 OF 1 METRIC

W.P. 632-89-01 LOCATION Co-ords N. 4 854 047.5 ; E 301 132 ORIGINATED BY FLR
 DIST 6 HWY 400 BOREHOLE TYPE HS Auger COMPILED BY FLR
 DATUM Geodetic DATE 90 03 27-28 CHECKED BY TS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT 7 kN/m 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	WATER CONTENT (%) 10 20 30			
223.5	Ground Surface													
0.0	Clayey Silt, Some Sand, Trace Gravel (Glacial Till)		1	SS	21		222						0 15 55 30	
221.3	Brown, Very Stiff		2	SS	28		220						21.0	0 10 80 10
2.3	Brown		3	SS	28		220							
	Grey		4	SS	37		220							
			5	SS	22		220							
			6	SS	33		220							
							218							0 3 82 15
				7	SS		22	218						
								216						
				8	SS		21	216						
								214						
	Silt, Trace Sand Compact to Very Dense	9	SS	60	214									
		10	SS	39		212								
210.9			11	SS	18									
12.6	End of Borehole													

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 632-89-01 LOCATION Co-ords: N 4 854 036 ; E 301 020.5 ORIGINATED BY FLR
DIST 6 HWY 400 BOREHOLE TYPE HS Auger COMPILED BY FLR
DATUM Cedoretic DATE 90 03 29 CHECKED BY TS

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
223.2	Ground Surface																
0.0	Clayey Silt, Some Sand, Trace Gravel (Glacial Till) Brown, Firm to Very Stiff		1	SS	27	*	222										4 25 40 31
			2	SS	27												
			3	SS	16												
219.4			4	SS	24		220										
3.8			5	SS	6												
			6	SS	19												
	Silt, Some Sand Grey, Loose to Compact		7	SS	9		218										2 22 56 20
			8	SS	10		216										
213.3			9	SS	17		214										
9.9	Clayey Silt (LAC) Very Soft		10	SS	2		212										
	Clayey Silt, Trace Sand (Glacial Till)		11	SS	9		210										0 4 44 52
209.0	Grey, Firm to Hard		12	SS	74												0 4 79 17
14.2	End of Borehole																
	* GWL - Dry upon completion																

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 532-89-01 LOCATION Co-ords: N 4 854 016 ; E 300 987 ORIGINATED BY TS
 DIST 5 HWY 400 BOREHOLE TYPE HS Auger COMPILED BY TS
 DATUM Geodetic DATE 90 04 11 CHECKED BY TS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W _p	W	W _L		
223.4	Ground Surface																
0.0	Sand to Silty Sand Brown, Loose to Compact	.	1	SS	8												
			2	SS	10												
			3	SS	20												
			4	SS	15												
			5	SS	14												
			6	SS	28												
216.8	Clayey Silt, Same Sand, Trace of Gravel (Glacial Till) Grey, Very Stiff to Hard	/	7	SS	21												
6.5			8	SS	30												
			9	SS	19												
			10	SS	39												
			11	SS	32												
210.8																	
12.6	End of Borehole																
	• GWL - Dry upon completion																

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

W.P. 632-89-01 LOCATION Co-ords: N 4 854 025 : E 301 061 ORIGINATED BY FLR
 DIST 5 HWY 400 BOREHOLE TYPE HS Auger COMPILED BY FLR
 DATUM Geodetic DATE 90 03 29 CHECKED BY TS

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT 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	W _p	W			W _L
225.2	Ground Surface																
0.0	Clayey Silt, Some Sand, Trace Gravel (Glacial Till) Brown, Stiff to Hard		1	SS	11												
			2	SS	15												
			3	SS	16												
			4	SS	25												
			5	SS	27												
			6	SS	52												
218.6			7	SS	19												
6.5	Silt, Some Sand Compact to Dense, Grey		8	SS	25												
			9	SS	26												
			10	SS	44												
			11	SS	43												
212.5																	
12.6	End of Borehole																
	* GWL - Dry upon completion																

RECORD OF BOREHOLE No 7

1 OF 2 METRIC

W.P. 632-89-01 LOCATION Co-ords: N 4 854 013.5 ; E 301 027 ORIGINATED BY TS
DIST 6 HWY 400 BOREHOLE TYPE HS Auger, NW Casing, Washboring COMPILED BY TS
DATUM Geodetic DATE 90 04 02 CHECKED BY TS

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
223.5														
0.0	Trace Organics		1	SS	8		222						21.5	2 24 39 35
	Clayey Silt, Some Sand, Trace Gravel (Glacial Till) Brown, Firm to Hard		2	SS	40		220							
			3	SS	28									
			4	SS	28									
218.8			5	SS	25									
4.7			6	SS	48									
	Silt, Some Sand Brown, Compact		7	SS	25		218							0 17 79 4
			8	SS	25		216							
215.1			9	SS	28		214							1 22 56 21
8.4			10	SS	27		212							
	Clayey Silt, Some Sand, Trace Gravel (Glacial Till) Grey, Firm to Hard		11	SS	13		210						20.7	
			12	SS	55		208							0 0 78 22
			13	SS	25		206							
			14	SS	58		204							0 89 (11)
204.5			15	SS	60		202							
19.1			16	SS	75		200							
	Sand and Gravel Grey, Very Dense		17	SS	50		198							46 42 (12)
			18	SS	50		196							
			19	SS	52		194							
			20	SS	60									
			21	SS	54									
			22	SS	55									

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (x) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 7

2 OF 2

METRIC

W.P. 632-89-01 LOCATION Co-ords: N 4 854 013.5 ; E 301 027 ORIGINATED BY TS
 DIST 5 HWY 400 BOREHOLE TYPE HS Auger, NW Casing, Washboring COMPILED BY TS
 DATUM Geodetic DATE 90 04 02 CHECKED BY TS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 KN/m 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	20 40 60 80 100	10 20 30	10 20 30					
30.5	Continued Sand and Gravel Grey Very Dense		23	SS	34		192										
190.0								190									
33.5	Clayey Silt Grey, Hard		24	SS	95		188										
185.4			25	SS	70		186										
38.1			26	SS	165		184										
181.9	Silt, Some Sand Grey, Very Dense		27	SS	120	/25cm										2 13 80 5	
41.8			28	SS	100	/15cm	182										
41.8	End of Borehole																

RECORD OF BOREHOLE No 7A

1 OF 1

METRIC

W.P. 632-89-01 LOCATION Co-ords: N 4 854 024.5 ; E 301 024.5 ORIGINATED BY TS
 DIST 6 HWY 400 BOREHOLE TYPE HS Auger COMPILED BY FLR
 DATUM Geodetic DATE 90 04 11 CHECKED BY TS

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					
223.5	Ground Surface																
0.0	Clayey Silt, Trace Sand, Trace Gravel (Glacial Till)		1	SS	28		222										
			2	SS	74												
220.5	Brown, Very Stiff to Hard		3	SS	29												
3.0	Silt, Trace Sand Brown, Compact		4	SS	32		220										
			5	SS	23												
218.2			6	SS	16												
5.3			7	SS	15		218										
	Clayey Silt, Same Sand, Trace Gravel (Glacial Till)		8	SS	33	*	216										
	Grey, Stiff to Hard		9	SS	18		214										
			10	SS	41												
210.8	Clayey Silt (LAC)		11	SS	11		212										
12.8	End of Borehole																
	* Cave-in at El. 216 m																

RECORD OF BOREHOLE No 8 (96-78-02) 1 OF 1 METRIC

W.P. 632-89-01 LOCATION Co-ords: N 4 854 004 ; E 301 019 ORIGINATED BY _____
 DIST 6 HWY RUTHERFORD RD. BOREHOLE TYPE Solid Stem Auger COMPILED BY _____
 DATUM Geodetic DATE November 24, 1978 CHECKED BY _____

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
223.4																	
0.0	Topsoil - Organics, 150 mm		1	SS	32		222							o		21.4	
			2	SS	25									o		21.5	
			3	SS	28		221								o	19.8	
	Silty Clay, Traces of Sand and Gravel, Sand layers		4	SS	31		220							o			
	Brown, Very Stiff to Hard		5	SS	20		219								o		
217.3							218										
5.1 216.9			6	SS	70		217							o		22.9	1 25 56 18
6.5	End of Borehole																
	• Sandy Silt to Clayey Silt, Traces of Gravel Very Dense																
	Water level at 6.0 m depth and borehole open to 6.1 m depth upon completion.																

RECORD OF BOREHOLE No 9(96-78-02) 1 OF 1

METRIC

W.P. 632-89-01 LOCATION Co-ords: N 4 853 986 : E 301 020 ORIGINATED BY _____
 DIST 5 HWY RUTHERFORD RD. BOREHOLE TYPE Solid Stem Auger COMPILED BY _____
 DATUM Geodetic DATE November 28, 1978 CHECKED BY _____

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					
230.6																	
0.0	Gravel - 150 mm		1	SS	20											21.9	
			2	SS	11		229									21.2	
	Fill - Sandy Silt, Trace to Some Clay Brown, Compact		3	SS	12		228									21.1	
			4	SS	14		227										
			5	SS	26		226										
			6	SS	12		225										
	Occasional Organic Materials at 7.6 m depth. (Possible Original Topsail)						224										
223.0			7	SS	19		223										
7.6	Silty Clay Trace to Some Sand Stiff to Very Stiff, Brown, Moist		8	SS	12		222										
							221										
219.9			9	SS	59		220										
10.7	Sand Very Dense, Fine Grained, Traces of Silt, Silty Layers, Wet at 12.2 m depth						219										
217.9			10	SS	56		218										
12.7	End of Borehole Borehole dry and open to 11.6 m depth upon completion.																

RECORD OF BOREHOLE No 10(96-78-02) OF 1 METRIC

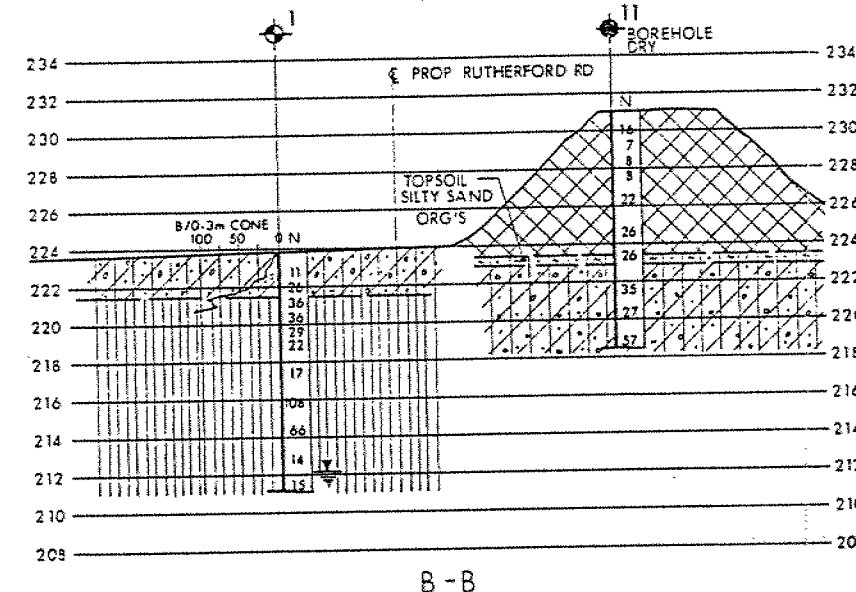
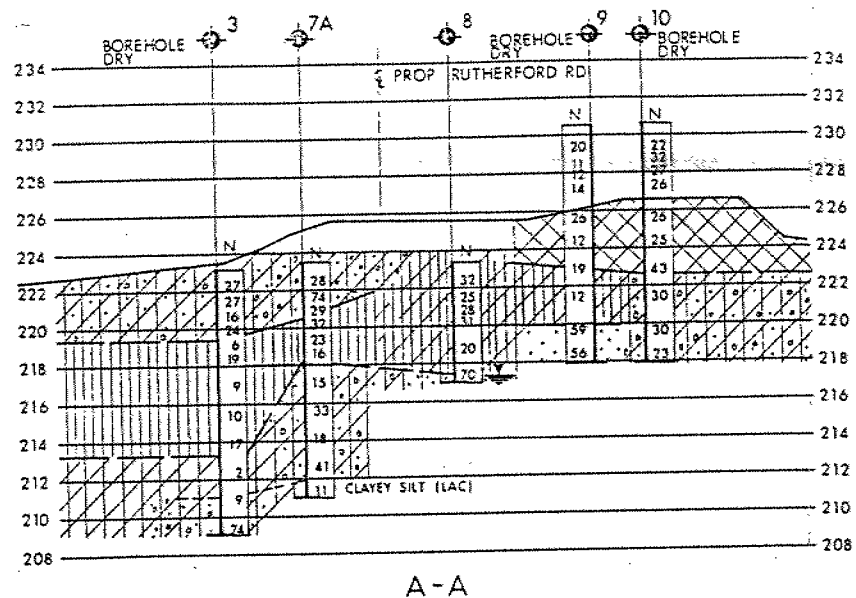
W.P. 832-89-01 LOCATION Co-ords: N 4 853 979 ; E 301 027 ORIGINATED BY _____
 DIST 6 HWY RUTHERFORD RD. BOREHOLE TYPE Solid Stem Auger COMPILED BY _____
 DATUM Geodetic DATE November 24, 1978 CHECKED BY _____

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					
230.6																	
0.0	Gravel - Some Sand, 150 mm		1	SS	22												
	Fill - Sandy Silt to Clayey Silt, Traces of Gravel, Occasional Shale Fragments, Same Sand		2	SS	32		229										
	Compact to Dense, Brown, Moist, Slightly Cohesive		3	SS	27		228										
			4	SS	26		227										
			5	SS	26		226										
							225										
	Organic Materials at 8.0 m depth (Possible Original Topsoil)		6	SS	25		224										
222.6			7	SS	43		223									21.3	
8.0							222										
	Clayey Silt, Some Sand, Traces of Gravel (Glacial Till) Brown, Very Stiff		8	SS	30		221										3 20 49 28
			9	SS	30		220										
							219										
217.9			10	SS	23		218										
12.7	End of Borehole Borehole dry and open to 11.5 m depth upon completion.																

RECORD OF BOREHOLE No 11(96-78-02) 1 OF 1 METRIC

W.P. 632-89-01 LOCATION Co-ords: N 4 854 006 ; E 301 088 ORIGINATED BY _____
 DIST 6 HWY RUTHERFORD RD. BOREHOLE TYPE Solid Stem Auger COMPILED BY _____
 DATUM Geodetic DATE December 4, 1978 CHECKED BY _____

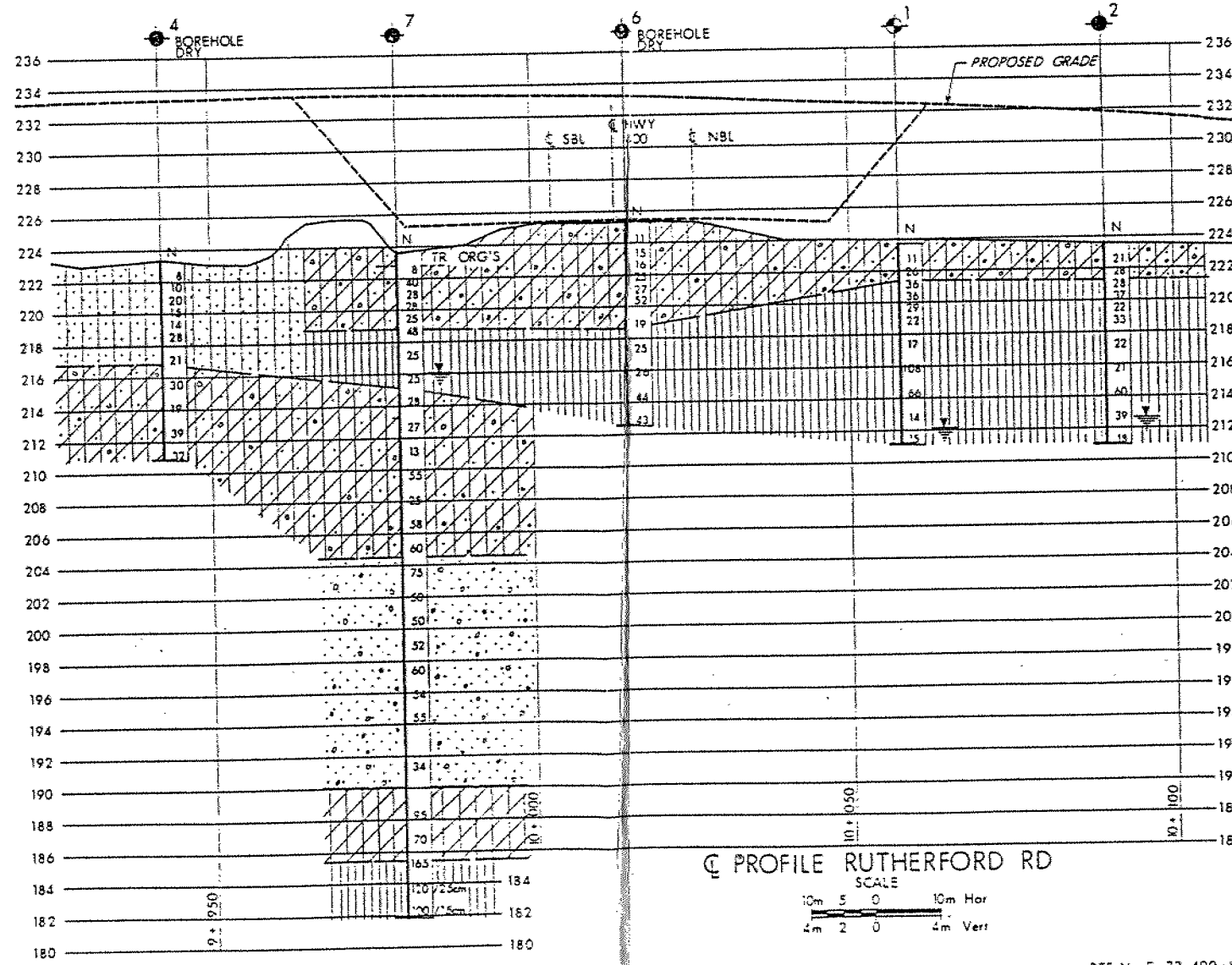
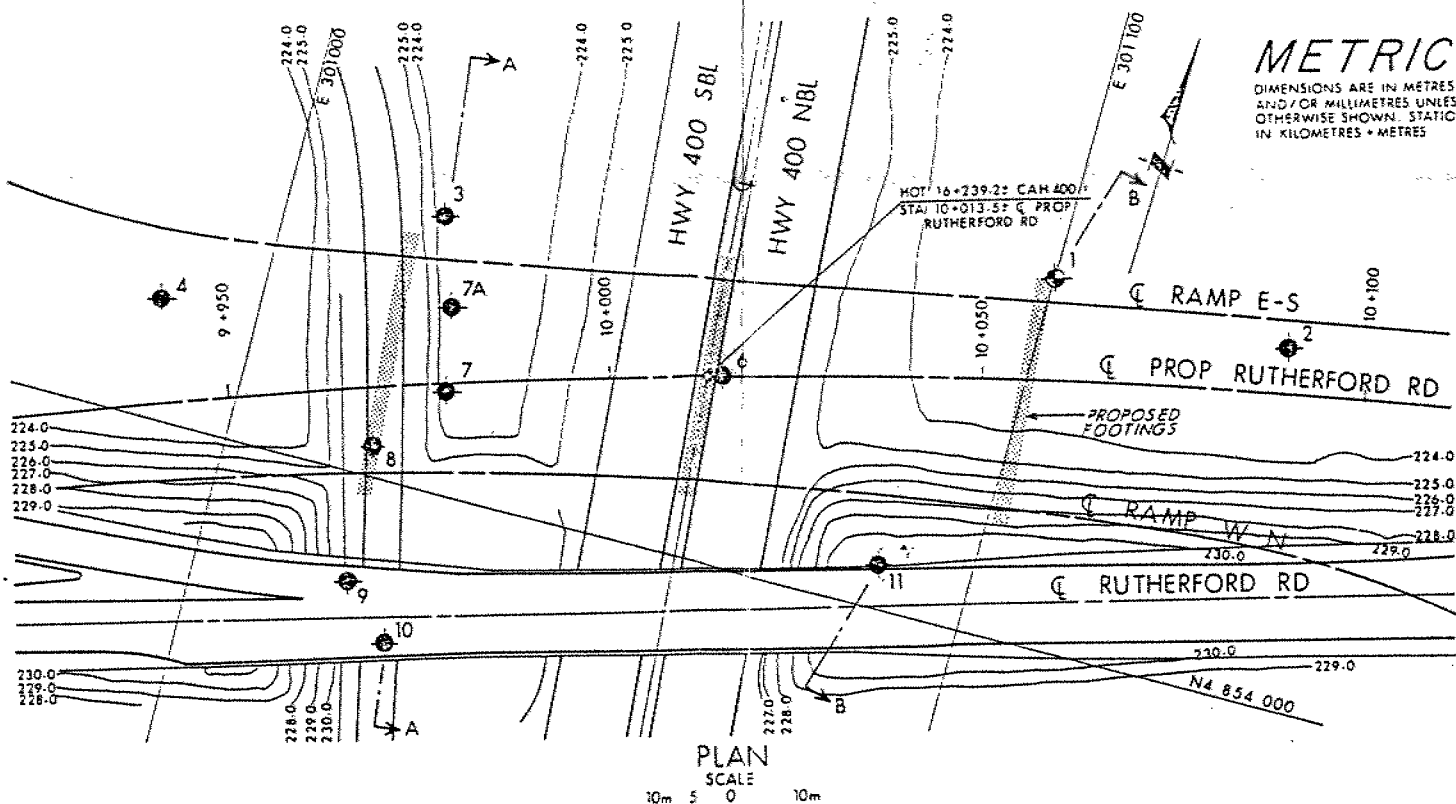
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
231.0																	
0.0	Gravel - Some Sand, 150 mm																
			1	SS	16		230									22.1	
			2	SS	7		229										2 27 53 18
			3	SS	8		228										
			4	SS	8		227										
			5	SS	22		226										
			6	SS	26		225									22.1	
							224										
223.4							223										
7.6 222.9	Topsoil - Silty Sand, Black, Organics		7	SS	26		222										
8.1			8	SS	35		221									21.9	4 23 44 29
			9	SS	27		220										
			10	SS	57		219										
218.4																	
12.6	End of Borehole																
	Borehole Dry and open to 8.2 m depth upon completion.																



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

SOIL STRATIGRAPHY LEGEND

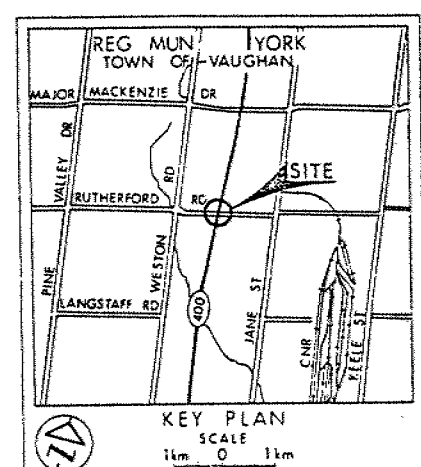
	SANDY SILT TO CLAYEY SILT TRACE TO SOME CLAY FRAGMENTS OF GRAVEL, OCC SHALE FRAGMENTS, OCC ORG MATERIALS AT 7.6m DEPTH (POSSIBLE ORG TOPSOIL) MOIST, SLIGHTLY COHESIVE Loose to Dense (FILL)		SAND TO SILTY SAND Loose to Compact
	CLAYEY SILT TRACE/SOME SAND, TRACE GRAVEL Firm to Hard (GLACIAL TILL)		SAND TRACES OF SILT, SILTY LAYERS WET AT 12.2m DEPTH Very Dense FINE GRAINED
	SILT TRACE/SOME SAND Loose to Very Dense		SANDY SILT TO CLAYEY SILT TRACE OF GRAVEL Very Dense
	SILTY CLAY TRACES OF SAND & GRAVEL SAND LAYERS Very Stiff to Hard		SAND & GRAVEL Very Dense



PROFILE RUTHERFORD RD
SCALE
10m 5 0 10m Hor
4m 2 0 4m Vert

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

CONT No
WP No 632-89-01
RUTHERFORD RD
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
78 11, 78 12, 90 03 and 90 04

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	223.7	4 854 048.5	301 100.0
2	223.5	4 854 047.5	301 132.0
3	223.2	4 854 036.0	301 020.5
4	223.4	4 854 016.0	300 987.0
6	225.2	4 854 025.0	301 061.0
7	223.5	4 854 013.5	301 027.0
7A	223.5	4 854 024.5	301 024.5
8	223.4	4 854 004.0	301 019.0
9	230.6	4 853 986.0	301 020.0
10	230.6	4 853 979.0	301 027.0
11	231.0	4 854 006.0	301 088.0

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

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

REV	DATE	BY	DESCRIPTION
1			

Geocres No 30M13-109
HWY No 400 DIST 6
SUBMITTS CHECKED DATE 90 07 17 SITE 37-104
DRAWN DT CHECKED DATE 90 07 17 SITE 37-104
REF No E-73-400-8-9001

HIGHWAY 400

GENERAL ARRANGEMENT

DILLON
Consulting Engineers • Planners
Environmental Scientists

GENERAL NOTES

CLASS OF CONCRETE

• ALL CONCRETE (UNLESS OTHERWISE SPECIFIED) ——— 30MPa

REINFORCING STEEL

- REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED.

* BAR MARKS WITH SUFFIX 'C' DENOTE COATED BARS.

CLEAR COVER TO REINFORCING STEEL

• FOOTINGS 100±25

- 38TMENTS. LINGWALLS — FRONT FACE — 80±20

AND RETAINING WALL BACK FACE—70320

* PIER 80120

• DECK TOP 70 ± 30

BOTTOM — 40 ± 10

CONSTRUCTION NOTE

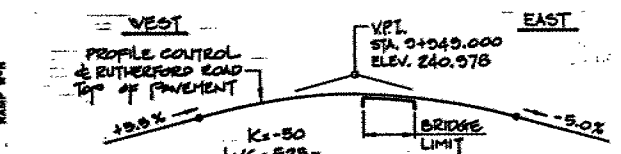
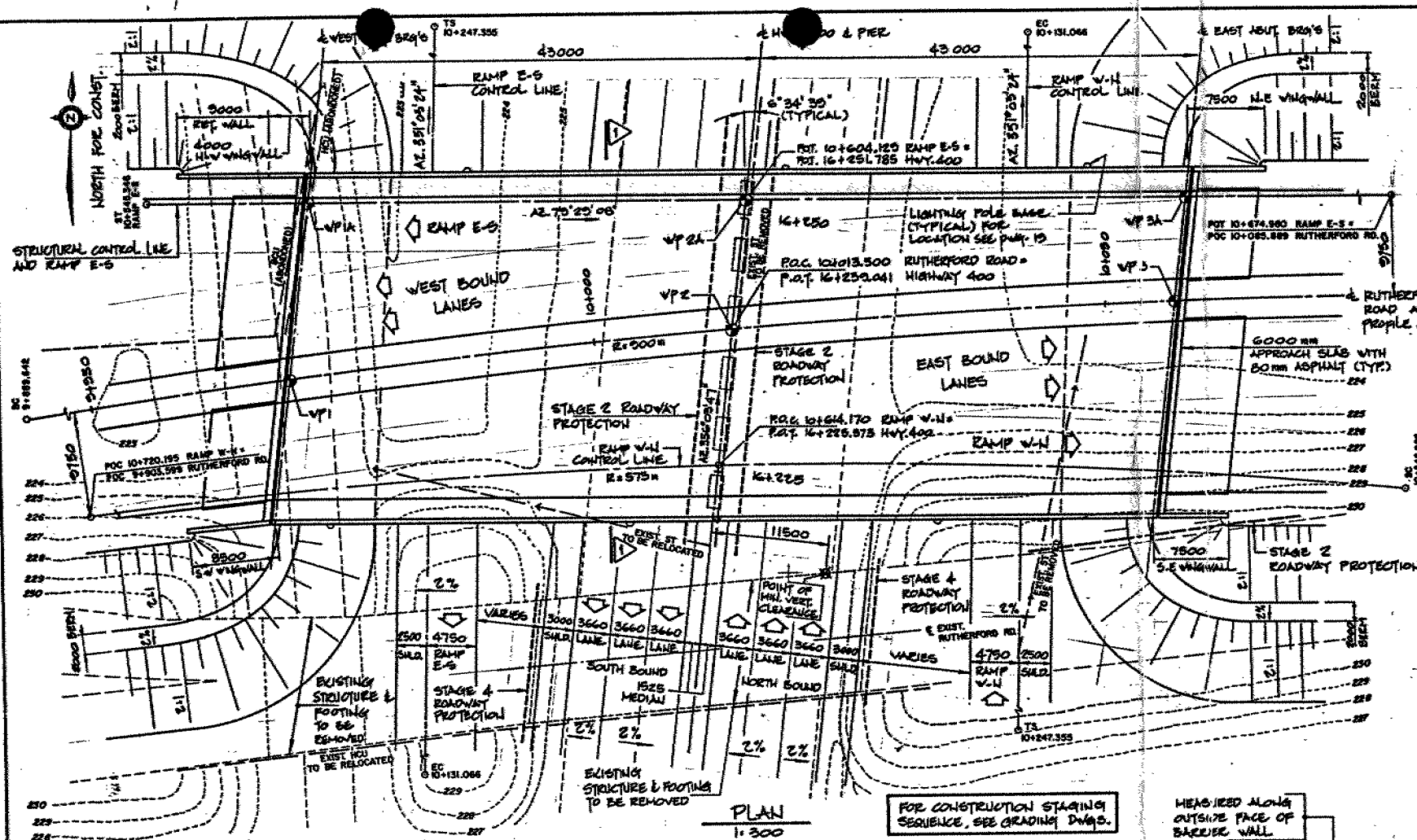
- IF THE ACTUAL BEARING HEIGHTS ARE DIFFERENT FROM THE ASSUMED BEARING HEIGHTS GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE BEARING SEAT ELEVATIONS AND THE REINFORCING STEEL TO SUIT THE ACTUAL HEIGHTS.

LIST OF DRAWINGS

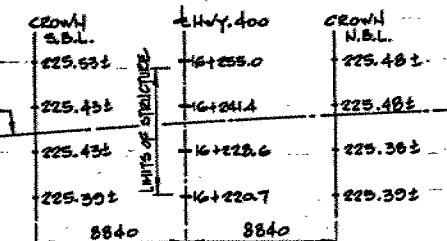
1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATION AND SOIL STRATA
3. ROADWAY PROTECTION I
4. ROADWAY PROTECTION II
5. ROADWAY PROTECTION III
6. FOOTING LAYOUT AND REINFORCEMENT
7. WEST ABUTMENT LAYOUT AND REINFORCEMENT
8. EAST ABUTMENT LAYOUT AND REINFORCEMENT
9. ABUTMENT DETAILS
10. N/W & S/W VINGWALL DETAILS AND REINFORCING
11. N/E & S/E VINGWALL DETAILS AND REINFORCING
12. RETAINING WALL DETAILS AND REINFORCING
13. PIER DETAILS AND REINFORCING
14. BEARING DETAILS
15. STRUCTURAL STEEL I
16. STRUCTURAL STEEL II
17. STRUCTURAL STEEL III
18. STRUCTURAL STEEL IV
19. DECK LAYOUT
20. DECK REINFORCEMENT
21. NORTH BARRIER WALL ON SIDEWALK
22. SOUTH BARRIER WALL ON SIDEWALK
23. RAILING FOR BARRIER WALL
24. JOINT ANCHORAGE AND ARMOURING
25. 6000 mm APPROACH SLAB
26. DETAILS OF CONCRETE SLOPE PAVING
27. AS CONSTRUCTED ELEV. & DIM.
28. STANDARD DETAILS I
29. STANDARD DETAILS II
30. ELECTRICAL EMBEDDED WORK
31. ELECTRICAL EMBEDDED DETAILS
32. QUANTITIES - STRUCTURE I
33. QUANTITIES - STRUCTURE II

APPLICABLE STANDARD DRAWING

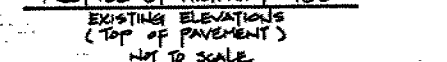
DD 3503 MINIMUM GRANULAR BACKFILL REQUIREMENTS
DD 4604 FALSEWORK CLEARANCES



PROFILE OF RUTHERFORD RD.
NOT TO SCALE

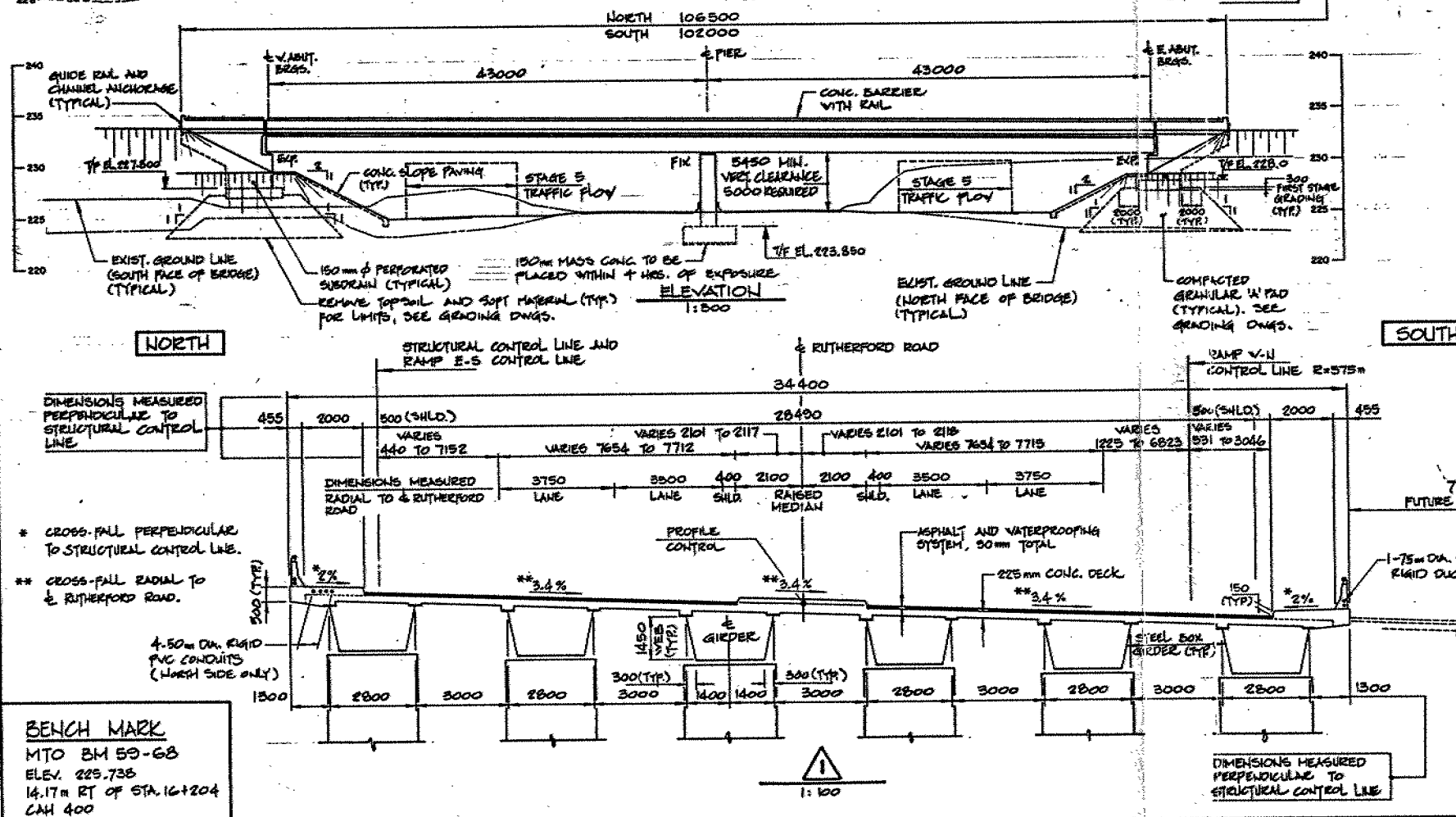


PROFILE OF HIGHWAY 420



LIST OF ABBREVIATIONS

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



BENCH MARK
MTO BM 59-68
ELEV. 225.738
14.17m RT OF STA. 16+204
CAN 400

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS							
	DATE	BY	DESCRIPTION				
DESIGN M.R.	CHK D.C.	CODE OHBDC-83	LOAD CLASS-A	DATE MAY 91			
DRAWN AKI	CHK M.R.	SITE 37-127	STRUCT	SCHEME	DWG. 1		



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

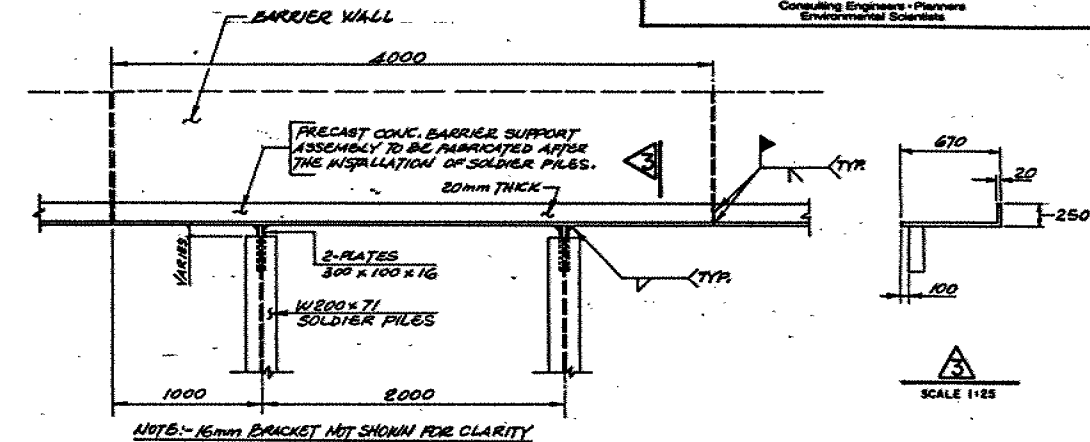
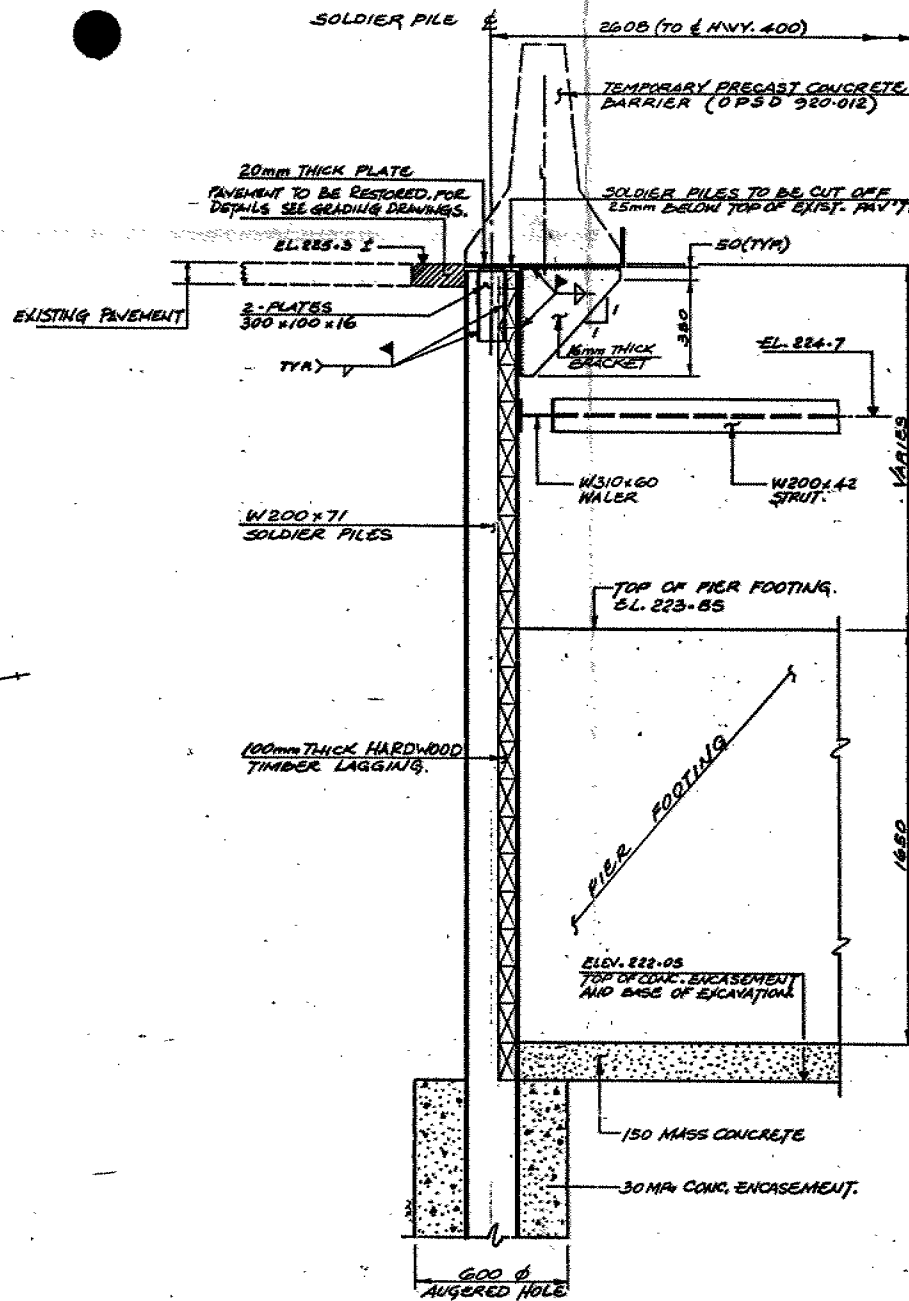
CONT No
WP - No 632-89-01

HIGHWAY 400
RUTHERFORD RD. UNDERPASS
ROADWAY PROTECTION - I

DILLON
Consulting Engineers - Planners
Environmental Scientists



SHEET



CENTRAL PIER CONSTRUCTION SEQUENCE

1. INSTALL SOLDIER PILES.
2. INSTALL TIMBER LAGGING AS EXCAVATION PROCEEDS TO ELEVATION 224.1 (APPROX. 12m BELOW EXISTING GROUND).
3. INSTALL STRUT AND WALER.
4. INSTALL BRACKETS AND TEMPORARY BARRIER (EACH 4.0m LONG).
5. EXCAVATE TO ELEV. 222.05.
6. PLACE CONCRETE FOOTING.
7. CONSTRUCT PIER.
8. REMOVE SHORING 1000mm BELOW FINISHED GRADE.

NOTES

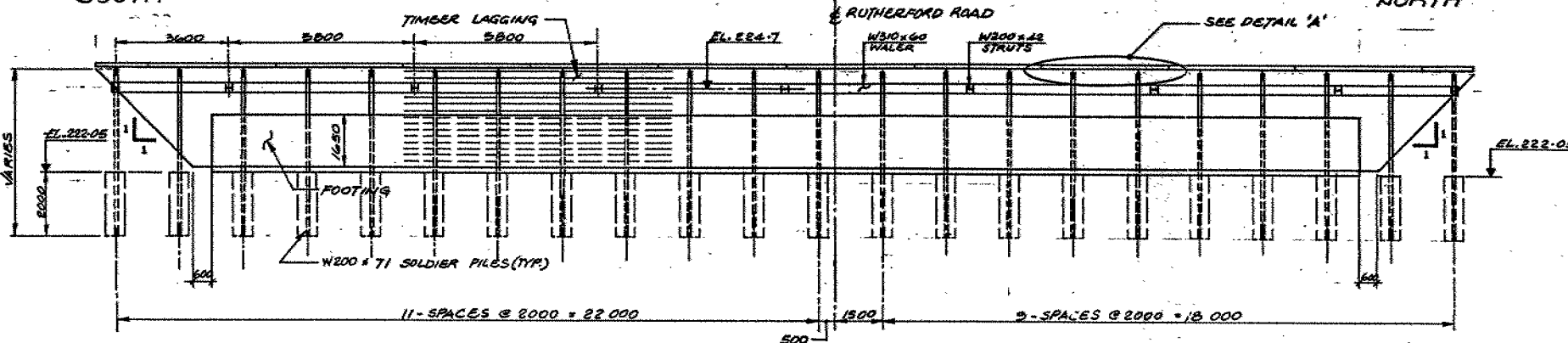
1. ALTERNATE ROADWAY PROTECTION SCHEMES WILL BE CONSIDERED SUBJECT TO THE ENGINEER'S APPROVAL. ALTERNATE SCHEMES WILL NOT BE ALLOWED TO ENCRoACH ON THE PAVED PORTIONS OF HWY. 400, BEYOND THE LIMITS SHOWN ON THIS DRAWING. THE CONTRACTOR SHALL SUBMIT DETAILED CONSTRUCTION DRAWINGS FOR ANY ROADWAY PROTECTION SEALED AND SIGNED BY A PROFESSIONAL ENGINEER.
2. STRUCTURAL STEEL FOR ROADWAY PROTECTION: CAN/CSA - G40.21 - MBT, GRADE 300W.
3. WELDING OF STRUCTURAL STEEL: CSA W59.
4. CONTRACTOR TO VERIFY EXISTING GRADE ELEVATIONS AND REPORT ANY DISCREPANCIES TO THE ENGINEER BEFORE PROCEEDING WITH THE WORK.
5. UNLESS OTHERWISE NOTED WELD SIZES FOR STRUCTURAL STEEL SHALL BE IN ACCORDANCE WITH STRUCTURAL STEEL NOTES ON DWG. No. 15.

PLAN

SCALE 1:100

SOUTH

NORTH



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

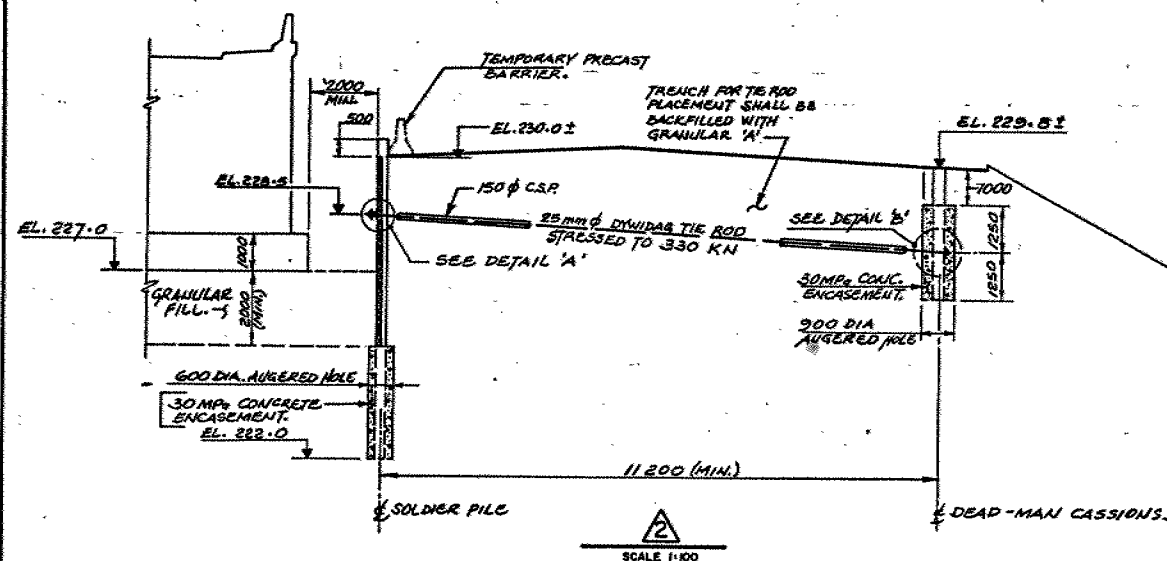
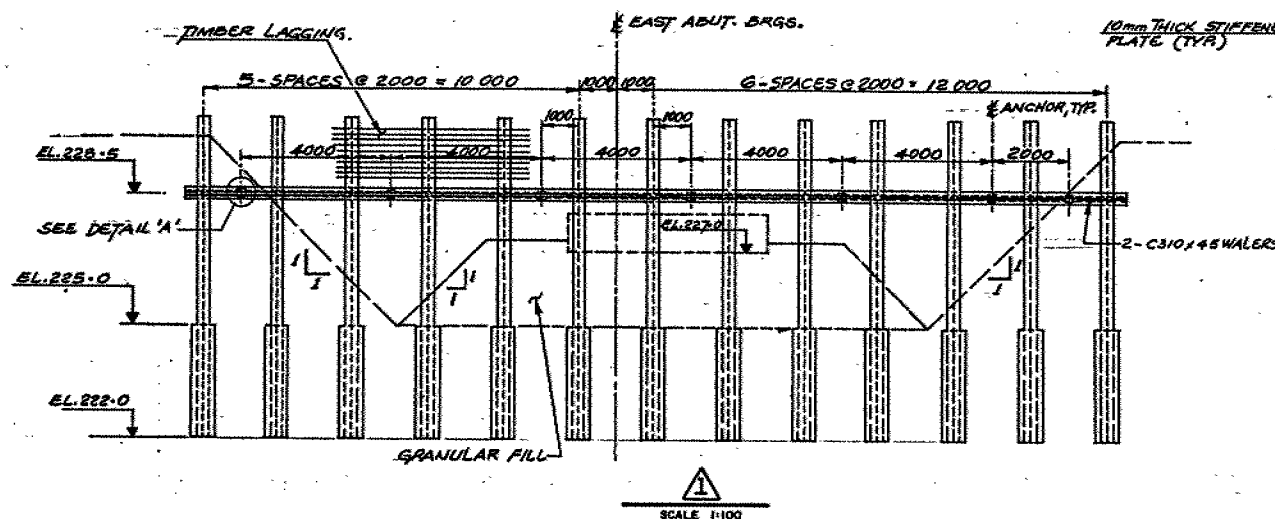
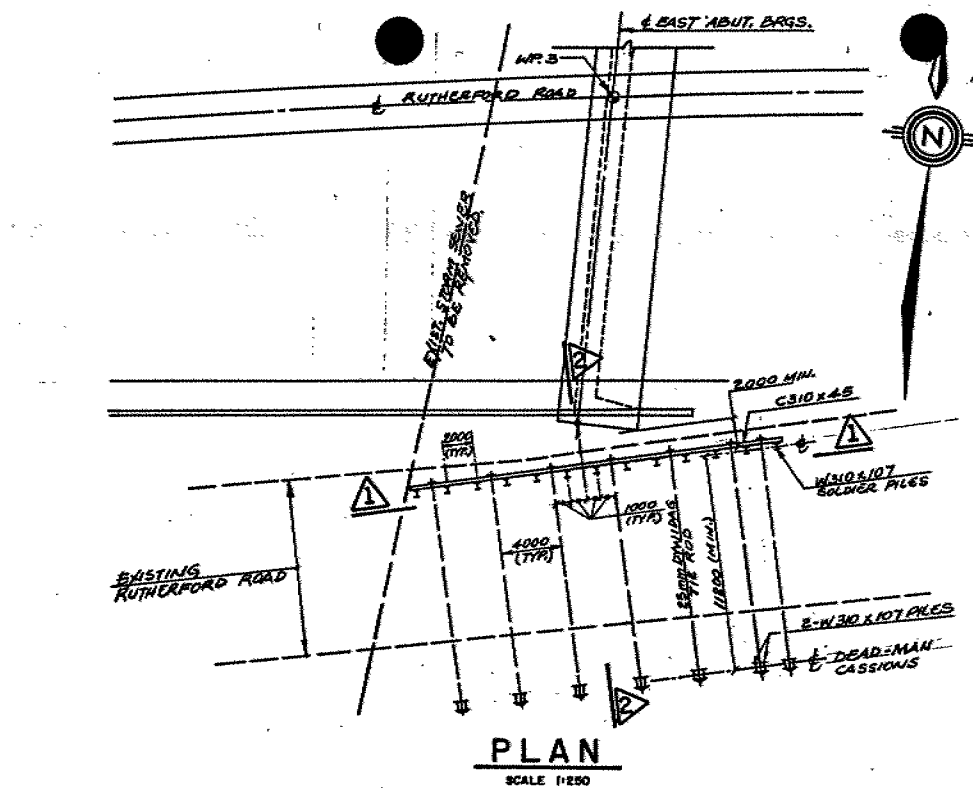
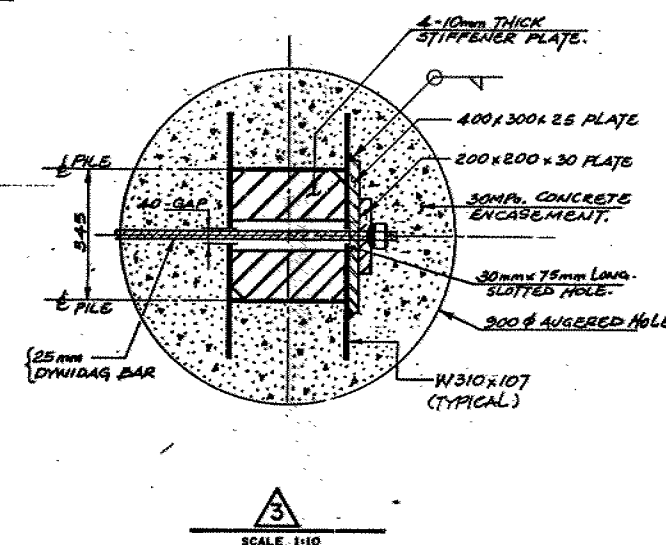
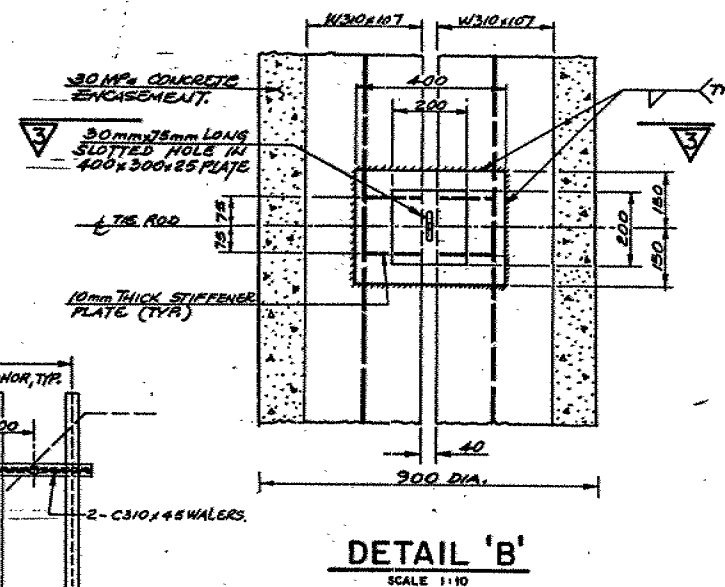
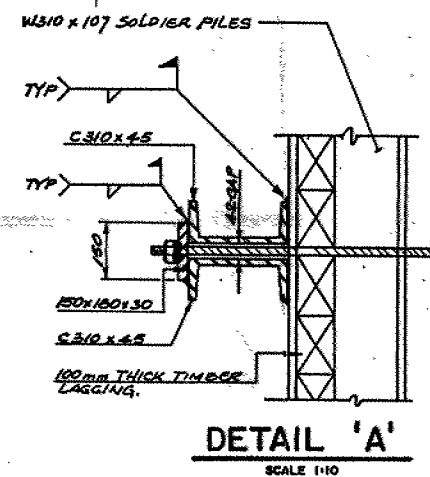
REVISIONS	DATE	BY	DESCRIPTION
DESIGN	D.C.	CHK R.T.	CODE QH60C-83 LOAD CLASS-A DATE MAY 91
DRAWN	D.R.S.	CHK D.C.	SITE 37-127 STRUCT SCHEME DWG. 3

EAST ABUTMENT ROAD PROTECTION CONSTRUCTION SEQUENCE

1. INSTALL SOLDIER PILES
2. INSTALL TIMBER LAGGING TO ELEVATION 227.0 m.
3. TRENCH FOR THE RODS AND INSTALL IN 150mm DIAMETER C.S.F.
4. INSTALL WALERS AND TIE RODS.
5. DRILL DEAD-MAN CASSIONS AND INSTALL DOUBLE PILE SECTION.
6. PLACE DEAD-MAN CONCRETE, WITH THE ROD ENGAGED.
7. PROOF TEST THE RODS TO 430 KN. AND REDUCE TO LOCK-IN LOAD AS NOTED.
8. EXCAVATE TO ELEVATION 225.0 m
9. PLACE AND COMPACT GRANULAR FILL TO 227.0 m
10. PLACE EAST ABUTMENT FOOTING AND ABUTMENT WALL.
11. REMOVE SHORING TO 1.0 m BELOW FINISHED GRADE.

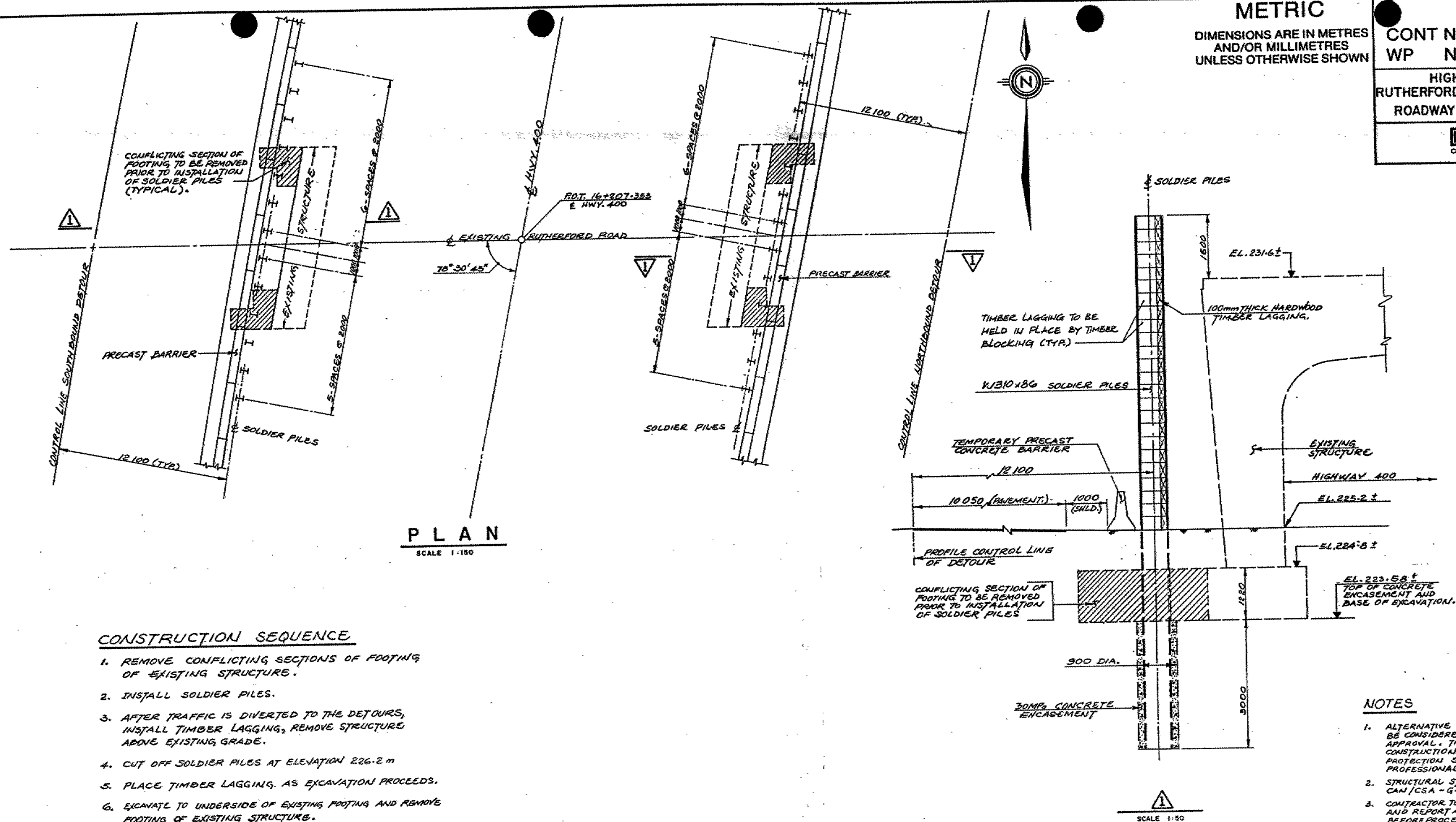
NOTES :

1. ALTERNATE ROADWAY PROTECTION SCHEMES WILL BE CONSIDERED SUBJECT TO THE ENGINEER'S APPROVAL. ALTERNATE SCHEMES WILL NOT BE ALLOWED TO ENCROACH INTO THE GRANULAR FILL SUPPORTING THE EAST ABUTMENT. THE CONTRACTOR SHALL SUBMIT DETAILED CONSTRUCTION DRAWINGS FOR ANY ROADWAY PROTECTION SEALED AND SIGNED BY A PROFESSIONAL ENGINEER.
2. STRUCTURAL STEEL FOR ROADWAY PROTECTION CAN/CSA - G40.21 - M87, GRADE 300W.
3. WELDING OF STRUCTURAL STEEL : C.S.A. W53 -
4. CONTRACTOR TO VERIFY EXISTING GRADE ELEVATIONS AND REPORT ANY DISCREPANCIES TO THE ENGINEER BEFORE PROCEEDING WITH THE WORK.
5. UNLESS OTHERWISE NOTED WELD SIZES SHALL BE IN ACCORDANCE WITH STRUCTURAL STEEL NOTES ON DWG. NO 15
6. TIE RODS SHALL BE DWIDAG THREADBARS, GRADE 1030 MPa, CONFORMING TO THE REQUIREMENTS OF ASTM DESIGNATION A722.
7. TIE RODS TO BE INSTALLED BY TRENCHING ACROSS ROADWAY OR BY HORIZONTALLY BORING ACROSS ROADWAY.



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

[illegible]



1. REMOVE CONFLICTING SECTIONS OF FOOTING OF EXISTING STRUCTURE.
2. INSTALL SOLDIER PILES.
3. AFTER TRAFFIC IS DIVERTED TO THE DETOURS, INSTALL TIMBER LAGGING, REMOVE STRUCTURE ABOVE EXISTING GRADE.
4. CUT OFF SOLDIER PILES AT ELEVATION 226.2 m
5. PLACE TIMBER LAGGING, AS EXCAVATION PROCEEDS.
6. EXCAVATE TO UNDERSIDE OF EXISTING FOOTING AND REMOVE FOOTING OF EXISTING STRUCTURE.
7. CUT OFF AND REMOVE ALL ROADWAY PROTECTION 1.2 m BELOW FINISHED GRADE.

1. ALTERNATIVE ROADWAY PROTECTION SCHEMES WILL BE CONSIDERED SUBJECT TO THE ENGINEER'S APPROVAL. THE CONTRACTOR SHALL SUBMIT DETAILED CONSTRUCTION DRAWINGS FOR ANY ROADWAY PROTECTION SEALED AND SIGNED BY A PROFESSIONAL ENGINEER.
2. STRUCTURAL STEEL FOR ROADWAY PROTECTION CAN / CSA - G40-21 - MBT, GRADE 300W.
3. CONTRACTOR TO VERIFY EXISTING GRADE ELEVATIONS AND REPORT ANY DISCREPANCIES TO THE ENGINEER BEFORE PROCEEDING WITH THE WORK.



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS										
	DATE	BY				DESCRIPTION				
	DESIGN	D.C.	CHK	P.T.	CODE	0H80C - #3	LOAD	CLASS-A	DATE	MAY 91
	DRAWN	D.R.S.	CHK	D.C.	SITE	37-127	STRUCT	SCHEME		DWG. 5

memorandum



To: K. Bassi
Head, Structural Office
7th Floor, Atrium Tower

Attn: G. Al-Bazi
Design Engineer

From: Foundation Design Section
Room 315, Central Bldg.

Re: General Arrangement Drawing Review
Hwy. 400 - Rutherford Rd. U'Pass
W.P. 632-89-01, Site 37-127
District 6, Toronto

Date: 1991 01 14

The General Arrangement Drawing 37-127-P1 for the aforementioned structure has been reviewed by this office, and the following comments are provided.

1) Construction Staging Sequence Drawings

An illustration and description of the various construction stages that defines the sequence of construction activities is required so that a thorough review can be conducted. These drawings should be submitted to our office.

2) Roadway Protection

Details of the roadway protection scheme should also be forwarded to our office for pertinent review of the selected system including type of shoring, depths of penetration, method of lateral support and method of installation and removal. The design and construction of the roadway protection at the proposed east abutment in particular, requires careful scrutiny and review.

Stage 2 roadway protection enables the construction of the new structure whilst maintaining traffic on Hwy. 400 and Rutherford Rd. Roadway protection is presently illustrated to facilitate the proposed pier and east abutment construction. However, at the west abutment, roadway protection is not indicated to facilitate the construction of the proposed west abutment. It is therefore assumed that E-S movement will not interfere with the proposed west abutment/embankment construction.

As documented in item 11a in progress meeting No. 5, the limit of working area to install the roadway protection for the pier foundation should be ascertained and the appropriate traffic control to facilitate this installation should be organized.

3) Embankment Construction

a) Surficial

Approach embankments in the order of magnitude of 10 metres in height are required at the abutment locations. As stated in the original

foundation report, (W.P. 632-89-01, pg. 17), embankments exceeding 8 metres in height are to be constructed with midheight stabilizing berms. Although the General Arrangement Drawing depicts this berm in the transverse direction, forward slopes in the longitudinal direction shall also include the nominal 2 metre midheight berm.

b) West Abutment Construction

The Granular "A" pad supporting the west abutment intersects the existing E-S ramp embankment. The granular "A" pad MUST have a minimum 1.0 metre thickness. Consequently, subexcavation of the existing embankment is required. Furthermore, beneath this 1.0 metre subexcavation that facilitates the Granular "A" pad placement, the subgrade should be proof rolled prior to fill placement. All weak or soft material should be subexcavated and backfilled with suitable compacted fill material. These comments should be reflected on the drawing.

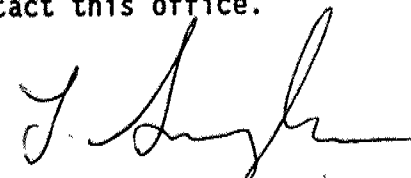
The removal of topsoil and soft material as indicated on the drawing still applies to the limits beyond the existing E-S ramp embankment.

c) East Abutment Construction

Temporary Retaining Wall

The proposed east approach embankment will be constructed to an elevation of 232.6m. The existing east approach exists at an elevation of 231m. Consequently, a temporary retaining wall to support the new approach fills will be required due to the elevation difference in the old and new Rutherford Rd. profiles. Details of this temporary retaining wall should be submitted to our office for pertinent review.

If you have any questions or queries regarding the above comments or require additional information, please do not hesitate to contact this office.



T. Sangiuliano, P. Eng.
Foundation Engineer

TS/jb

cc: N. Garland - Structural Section
(Central Region)

To: File

From: Tony Sanginliano

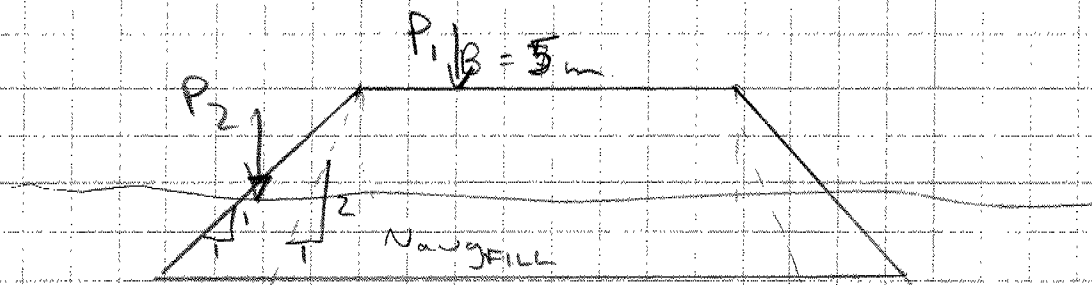
RE: Excavation of Existing Fill at West Abutment

Attached is a theoretical computation indicating the depth of subexcavation to distribute 350 KPa so that the applied loading does not overstress 150 KPa bearing capacity. The result reveals a depth of 7m which of course is totally unrealistic in any case.

In this case, the thickness of the existing embankment is 2 metres. Based on discussion with Balu Iyer (91 01 09), it was decided that the minimum thickness of the pad be 1 metre. Hence, 1 metre of subexcavation is required.

Depth of Subexcavation

WP 632-89-01



$$N_{avg \text{ Fill}} = 12 \text{ blows/Ft}$$

assume granular fill

∴ Bearing Capacity at S.L.S. Type II = 150 kPa

$$P_f = 350 \text{ kPa} \times 5 \text{ m} = 1750 \text{ kN}$$

$$B_2 = 1750 \text{ kN} / 150 = 12$$

$$B_2 = B_1 + 2\left(\frac{1}{2}D\right)$$

$$17 = 5 + 0$$

0 = 7

SEND TO

N. Garland
Structural Section, Central Region
Atrium Tower

FROM

T. Sangiuliano Foundation Design

DEPT.

DATE

90 06 08

SUBJECT

Hwy 400 - Rutherford Rd U'Pass

Site 37-127, WP 632-89-01

In response to your memo dated 90-05-28

1. Soil anchors are not considered feasible at the site because of the presence of the silt material and the underlying sand and gravel deposit, the latter submerged under groundwater. These conditions render anchor installation unfavourable.

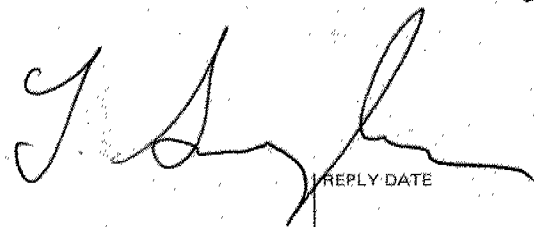
2. Mass concrete can be used instead of

REPLY

a granular pad provided that the pier Foundation is designed applying the bearing capacities applicable to the native (Table 5 in Preliminary Recommendations).

REPLY FROM

REPLY DATE



memorandum



235-3731

To: V. Boehnke
Head, Structural Section
Central Region

Date: 1990 05 23

Attn: N. Garland
Inter. Structural Engineer

From: Foundation Design Section
Room 315, Central Building

Subject: Preliminary Foundation Report
Hwy. 400 Underpass at Rutherford Road
W.P. 632-89-01, Site 37-127
District 6, Toronto

This memorandum summarizes the results of a foundation investigation conducted at the aforementioned site and provides preliminary recommendations pertaining to the structure foundations and related earthworks. The final report will be submitted by July 3, 1990 in accordance with the project schedule.

Proposed Structure

It is proposed to construct a two span structure immediately adjacent and north of the existing structure. The new structure will facilitate the widening of Rutherford Road to four lanes and also allow for the future widening of Hwy. 400 in this vicinity. Approach embankment fill heights in the order of 8.5 and 10.5 metres at the east and west approaches respectively are proposed.

Subsurface Conditions

The natural ground surface at the site varies from 223.2 to 225.2 m. Existing approach embankments in advance of the structure are in the order of 8 metres in height.

The subsoil stratigraphy at all locations across the site except at the west approach area, consists of a surficial deposit of a clayey silt to silty clay with some sand and a trace of gravel. This till deposit varies in thickness across the site ranging from 2.3 m to 6.1 m. Its consistency ranges from firm to hard with localized weaker material present at the proposed west abutment location. At the west approach, the till deposit is overlain by a cohesionless deposit of sand to silty sand, approximately 6.6 m in thickness. The state of denseness of this deposit varies from loose to compact.

.../2

Underlying the clayey silt to silty clay till deposit, a cohesionless deposit of varying percentages of silt and sand exists. The extent of this deposit was not determined at the proposed east abutment and pier locations. At the west abutment location, the thickness of this deposit varied from approximately 1.5 to 6.1 metres. The deposit is generally in a compact to dense state of condition.

At the west abutment location, the cohesionless silt and sand deposit is underlain by a second till deposit consisting of a clayey silt with some sand and a trace of gravel. This deposit also contains random interbeds of layered clayey silt of glaciolacustrine origin. The thickness of this deposit is equivalent to 10.7 metres.

A deposit of sand and gravel underlies the lower clayey silt deposit at the west abutment location. This deposit extends to a depth of approximately 33.5 m below the existing natural ground surface (Elevation 190 m) and is approximately 14.4 metres in thickness. The deposit has a very dense state of denseness.

The sand and gravel deposit is further underlain by a cohesive stratum consisting of a clayey silt. This stratum has a thickness of approximately 4.6 metres and has a hard consistency.

The cohesive clayey silt stratum is underlain by a very dense silt deposit that contains traces of sand. The extent of this deposit was not determined in the investigation.

Groundwater levels obtained at the time of the investigation revealed levels below or close to the depth of the boreholes advanced. The relatively shallow boreholes were advanced to depths ranging from 12.6 m to 14.2 m below the natural ground surface (Elevation 209 to 212.5 m). At the one deep borehole that was advanced to a depth of 41.6 m, the groundwater was at an elevation of approximately 216 m or 7.5 m below the natural ground surface. This latter groundwater level reflects the subartesian water condition in the lower sand and gravel deposit.

DISCUSSION AND RECOMMENDATIONS

The following foundation/geotechnical items are hereby discussed.

- 1) Structure Foundations
- 2) Approach Embankments
- 3) Construction Considerations

1) Structure Foundations

(a) East Abutment

In view of the competent nature of the surficial soils at the proposed east abutment location, the structure can be supported

on a shallow foundation. The conventional spread footings can be designed and constructed as discussed below.

i) Compacted Granular 'A'

Structure foundations can be "perched" on a compacted Granular 'A' pad placed on the existing native soil. For purposes of the O.H.B.D.C., the following bearing capacities are recommended.

Table 1

Bearing Capacity at S.L.S.. Type II (kPa)	350
Factored Capacity at U.L.S. (kPa)	900

The settlements induced as a result of the applied footing pressure will be elastic in nature and consequently is expected to take place during or immediately following the construction period. The magnitude of this settlement is anticipated to be within 25 mm, provided the subsoil/granular material is not softened/loosened by construction or related activities and the granular pad is constructed to the specifications described below.

The figure attached to this memorandum illustrates the recommended geometry for the design of the perched abutment foundation. The thickness of the pad shall be a minimum 2 metres above the existing natural ground surface and a minimum edge distance of 1 metre from the edge of the footing to the crest of the granular 'A' pad shall be provided. The figure also illustrates that frost penetration is required to protect the founding soil. All footings must have 1.2 m of earth cover.

The computation of the sliding resistance of the foundation shall be computed in accordance with Section 6-7.3.3.2 of the O.H.B.D.C. An unfactored friction angle of 35° can be used between the concrete footing and the Granular 'A' material.

ii) Spread Footings on Native Soil

Alternatively, east abutment structure foundations can be founded on the native clayey silt to silty clay till deposit at an elevation of 222.3 m. For purposes of the O.H.B.D.C., the bearing capacities tabulated in Table 2 are recommended.

Table 2 - Spread Footings on Native Soil

Bearing Capacity at S.L.S. Type II (kPa)	300
Factored Capacity at U.L.S. (kPa)	450

Settlement of the foundation subsoil as a result of the applied footing pressure will be elastic and hence will develop during or immediately following construction. The total and differential settlement is expected to be within 25 mm.

An unfactored friction angle of 30° can be used between the concrete footing and the native soil in the computation of the sliding resistance of the foundation.

All footings must be provided with adequate frost protection. Hence, the foundation must be designed with 1.2 m earth cover or equivalent frost protection.

No dewatering difficulties are anticipated during foundation construction because excavation will not intercept the groundwater table which is significantly lower than the footing founding elevation. Temporary slopes to facilitate the footing construction can be constructed no steeper than $\frac{1}{2}H:1V$.

b) West Abutment

In view of localized weaker material within the surficial soils at the proposed west abutment location and the fact that a deep foundation unit would require embedment lengths in the order of 38 metres to provide any economical capacity, it is recommended that the west abutment foundation be founded on a compacted Granular 'A' pad. For purposes of the O.H.B.D.C., the following bearing capacities are recommended.

Table 3 - Perched Abutment or Granular 'A'

Bearing Capacity at S.L.S. Type II	(kPa)	350
Factored Capacity at U.L.S.	(kPa)	900

The granular pad shall be a minimum 3 metres thickness.

All other design and construction considerations discussed previously in conjunction with the granular 'A' pad at the east abutment are also applicable to the west abutment foundation.

c) Pier

The centre pier structure foundations can also be founded on conventional spread footings within the surficial till deposit consisting of a clayey silt with some sand and a trace of gravel. In view of the presence of some weaker material within the surficial 2.5 to 3 metres of this deposit, it is recommended that the material be subexcavated to an elevation of 222.2 m and

replaced with a granular 'A' material. The spread footing foundation can then be placed on the granular 'A' material. For purposes of the O.H.B.D.C., the following bearing capacities are given provided that the granular 'A' pad is designed with a minimum 1.5 m thickness and a minimum edge distance of 1 m beyond the footings is included for its entire thickness.

Table 4 - Pier Foundation on Granular 'A'

Bearing Capacity at S.L.S. Type II (kPa)	350
Factored Capacity at U.L.S. (kPa)	800

The selection of the founding elevation of the pier foundation on the granular 'A' material must include the frost penetration depth criteria of 1.2 metres.

As previously discussed, the sliding resistance can be computed using an unfactored friction angle of 35° between the concrete footing and the Granular 'A' material.

Alternatively, the pier foundation can be founded on spread footings within the surficial clayey silt till deposit at an elevation of 222.2 metres. For purposes of the O.H.B.D.C., the capacities tabulated in Table 5 are recommended.

Table 5 - Pier Foundation on Native Soil

Bearing Capacity at S.L.S. Type II (kPa)	300
Factored Capacity at U.L.S. (kPa)	450

All other design and construction considerations previously discussed for the spread footing on native soil alternative at the proposed east abutment are equally applicable at the pier footing.

No dewatering problems are anticipated for either foundation alternative because excavation will not intercept the groundwater table. Temporary slopes to facilitate the footing construction can be constructed no steeper than $\frac{1}{2}H:1V$.

2) Approach Embankments

Approach fills in the order of magnitude of 8.5 m at the east approach and 10.5 m at the west approach will be required for the structure approach embankments. In view of the competent nature of the subsoil, no deep-seated stability problems are anticipated. However, to ascertain the internal (surficial) stability of the embankment fill, it is recommended that for

embankment fill heights exceeding 8 metres, the approach embankment be constructed with a stabilizing midheight berm of 2 m width.

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

Table 6 - Backfill Properties

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction (ϕ)	35°	30°
Unit Weight (kN/m^3)	22.8	21.2
Coefficient of Active Earth Pressure (K_a)	0.33	0.4
Coefficient of Earth Pressure at Rest (K_o)	0.5	0.58

These earth pressure coefficients apply to horizontal backfill surfaces only. The appropriate consideration shall be given to account for sloping surface backfill. (Factored angles of internal friction were employed in the computation of earth pressure coefficients).

The earth pressure coefficient at rest is to be used in design if the abutment walls are rigid and unyielding.

Settlements induced as a result of the applied embankment loading on the native soil are expected to be due to the elastic recompression of the soil and hence immediate in nature. Settlements within the embankment fill under its own weight can also be expected. The combined total magnitude of settlement is expected to be in the order of 75-100 mm. The embankment fill should be placed as far in advance of paving as scheduling permits.

3. Construction Considerations

In order to maintain traffic on both Hwy. 400 and Rutherford Road throughout the construction of the new structure and demolition of the existing structure, a combination of a roadway protection scheme and detour staging has been proposed. Recommendations for the design and construction of the roadway protection scheme is discussed below.

Temporary Roadway Protection

To facilitate the construction of the structure foundations for the proposed structure whilst maintaining traffic on Rutherford Road, a temporary shoring system will be required to support the existing approach embankments at the site. A timber lagging-soldier pile shoring wall is recommended at the site.

The design of the shoring system shall include the appropriate earth pressures computed in accordance with Section 6.6.1.2 of the O.H.B.D.C. The loadings induced by the surcharge traffic and adjustment for any sloping surfaces shall be incorporated in the design. Soil design parameters to facilitate the shoring wall design are summarized in Table 7 below.

Table 7 - Shoring Design Soil Parameters

<u>Soil Type</u>	<u>Saturated Unit Weight (kN/m³)</u>	<u>Unfactored Effective Shear Strength Parameters (0°)</u>
Fill	22	30
Clayey Silt/Silty Clay (Till) (Upper and Lower)	21	35
Silt, trace/some Sand	20	32

The shoring system must be designed to ascertain earth pressure equilibrium. This equilibrium can be achieved by a cantilevered wall or raker supported wall. The feasibility of a cantilever wall is contingent on the practicality of the soldier pile embedment length.

If the embedment length is found to be excessive, a more economical design may involve the application of rakers. Rakers can be installed in front of the wall with footings founded in the surficial clayey silt to silty clay till deposit.

A temporary roadway protection may also be required for the construction of the centre pier. A shoring system consisting of a cantilevered timber lagging-soldier pile wall or a braced timber lagging-soldier pile wall can be considered.

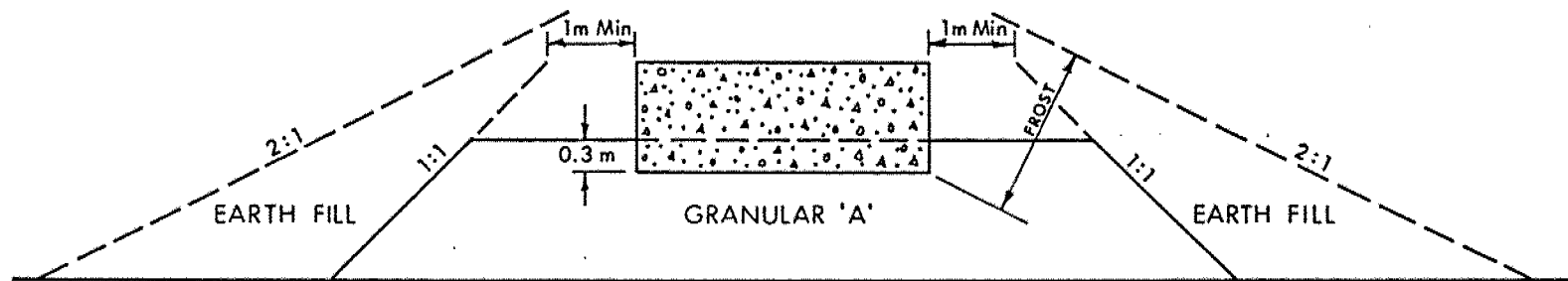
If you have any queries regarding the above comments or require additional information, please do not hesitate to contact this office.



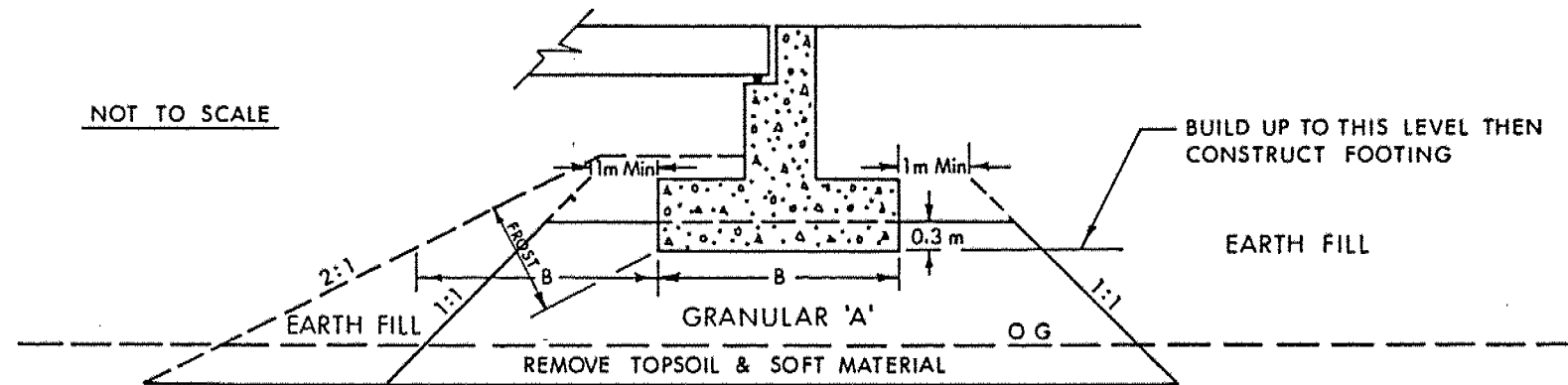
T. Sangiuliano, P.Eng.
Foundation Engineer

for

Dr. B. Iyer, P.Eng.
Sr. Foundation Engineer



X SECTION



LONGITUDINAL SECTION

NOTES:

- 1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.
- 3 - CONSTRUCT CONCRETE FOOTING.
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.



Ministry of
Transportation

ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

FIG No

W P