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CONT. No.

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STR. SITE No. 10-478

HWY. No. 403

LOCATION HWY403 & CNR OVERHEAD

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**Ministry of Transportation  
Province of Ontario**

**Foundation Investigation for  
Proposed Highway 403/  
CNR Subway  
District #4, Burlington  
WP 408-85-01, Site 10-478**

**March 1991**

**Acres International Limited  
Niagara Falls, Ontario**

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# **1 Introduction**

Acres International Limited (Acres) was retained by the Ministry of Transportation of Ontario (MTO) to undertake a foundation investigation for the structures at the proposed Highway 403/CNR subway, District No. 4, Burlington, WP 408-85-01, Site 10-478. The work was authorized by Agreement No. 4240-9190-188 dated September 12, 1990.

Drilling and sampling operations were performed by Master Soil Investigations Ltd. under the supervision and direction of Acres geotechnical staff. The field work was carried out between November 14 and 21, 1990. Additional site visits were made on November 29 and December 19, 1990 and January 9, 1991 to monitor water levels in the piezometers.

Plans of the bridge structure and approach cut showing the borehole locations together with the stratigraphic profiles are shown on Drawings 4088501-A and 4088501-B respectively. For the purposes of simplifying location descriptions in the text, it has been assumed that Highway 403 will run east/west and the CNR tracks run northeast/southwest.

Details of the investigation program are outlined in Section 2 with the site conditions being described in Section 3 of this report.

All of the data obtained during this investigation have been evaluated and recommendations regarding the geotechnical aspects of the design and construction of the proposed bridge and associated works are presented in Section 4.

## 2 Exploratory Work

Seven boreholes, numbered BH-101 to 107 inclusive, were drilled using two Type III auger drills (MTO Boring Contract - Item 5.3(c)) equipped with solid stem augers and diamond core drilling equipment. The total depth of drilling was 101.1 m including 10.6 m drilled as NQ rock core. Prior to drilling, borehole locations were surveyed and staked in the field by the Surveys and Plans Branch of the MTO. Some modification was made to the original scope of the investigations and borehole locations were adjusted in the field. Final positioning of the boreholes is shown on Drawings 4088501-A and 4088501-B.

Fieldwork commenced with one drill on November 14, 1990. A second drill was mobilized to the site November 20 and the two drills operated until completion of the fieldwork on November 21, 1990. A Bombardier tractor, equipped with a 500-gal tank, was used during the field program as a water carrier. A pond located near Chainage 27+175 was used as the water source for drilling operations.

Prior to commencement of the drilling, CNR personnel gave Acres verbal clearance that the borehole locations would not interfere with buried CNR services.

A flagman, supplied by CNR, was present during all drilling hours to authorize passage of rail traffic while the drill was working near the tracks and to supervise all crossings of the tracks by the drills and support vehicles.

The majority of the overburden was drilled using solid stem augers; however, an N tricone bit and wash-bored casing was also used in BH-101 to 103 for the deeper portion of the overburden drilling. Due to the hard or very dense soil encountered, water was added to boreholes to aid drilling and promote return of auger cuttings to surface. Sampling of overburden was undertaken using split-barrel samplers in accordance with the Standard Penetration Test (SPT) procedure. In the majority (83%) of the sampling attempts, the sampler met refusal ( $N > 100$ ) and could not be driven the full depth. The bedrock surface was inferred from observations of the drill action while advancing the hole using the wash-bored casing method. No samples were obtained in the upper 0.4 to 0.5 m of the bedrock due to the seating of the casing into the rock and the subsequent cleaning of the cased section of the hole with a tricone bit. NQ-size coring was used to sample the 'sound' bedrock and a total of 10.6 m of diamond drilling was performed.

Table 1 presents a summary of all the borehole physical data.



**Table 1**

**Summary of Borehole Physical Data**

Borehole Number	Ground Surface Elevation (m)	Highway 403 Centerline		Coordinates		Bedrock				Bottom of Borehole	
						Assumed Rock Surface		Start of NQ Coring			
		Chainage & (m)	Offset (m)	Northing	Easting	Depth (m)	Elev (m)	Depth (m)	Elev (m)	Depth (m)	Elev (m)
BH-101	160.5	26+911	24.0 LT	4 808 966	279 447	13.0	147.5	13.5	147.0	16.9	143.6
BH-102	160.3	26+909	24.0 RT	4 808 933	279 482	13.5	146.8	13.9	146.4	16.9	143.4
BH-103	160.4	26+919	CL	4 808 956	279 470	13.4	147.0	13.8	146.6	18.0	142.4
BH-104	163.1	27+000	CL	4 809 018	279 522	N/E	N/E	N/E	N/E	13.8	149.3
BH-105	163.5	27+100	CL	4 809 095	279 587	N/E	N/E	N/E	N/E	10.8	152.7
BH-106	163.4	27+189	1.7 LT	4 809 164	279 643	N/E	N/E	N/E	N/E	7.7	155.7
BH-107	164.6	26+883	CL	4 808 928	279 447	N/E	N/E	N/E	N/E	17.0	147.6

N/E - Not encountered.

Total Overburden Drilled (m) 89.2  
 Total Bedrock Drilled (m) 11.9 (10.6 m of NQ core)

Total Drilling (m) 101.1

Bedrock surface is inferred from drill action observations while advancing the hole by the wash-bored casing method. No samples were obtained in the upper 0.4 to 0.5 m of the bedrock due to seating of the casing into the rock and the subsequent cleaning of the cased section with a tricone bit. The top of the NQ-cored rock was found to be sound.

Piezometers were installed in BH-103 and BH-104. Details of piezometer installations are presented in Table 2.

All soil and rock samples were returned to Acres geotechnical laboratory in Niagara Falls for detailed logging and testing.

The laboratory testing program included the following types of tests, the results of which are presented and discussed in subsequent sections of the report.

- Natural moisture content
- Liquid limit
- Plastic limit
- Sieve and hydrometer analyses
- Sulfate content analysis.

**Table 2**

**Summary of Piezometer Installations  
and Observations**

Borehole Number	Ground Surface Elev (m)	Piezometer Tip Elev (m)	Bentonite Seal		Date Installed	Water Level Elev	
			Top Elev (m)	Bottom Elev (m)		(m)	(date)
BH-103	160.4	142.7	148.4	143.7	Nov 21/90	160.0	Nov 29/90
						159.7	Dec 19/90
						159.9	Jan 9/91
BH-104	163.1	150.1	154.6	152.8	Nov 20/90	162.4	Nov 29/90
						162.9	Dec 19/90
						Frozen at 163.1 on Jan 9/91	

### **3 Site Conditions**

#### **3.1 Physiography and Topography**

As outlined in the publication "The Physiography of Southern Ontario", Third Edition, by the Ministry of Natural Resources, the Highway 403/CNR Subway area is located within the physiographic region referred to as the 'South Slope', being the south slope of the Oak Ridges Moraine. The surface deposit at the site is the Trafalgar Moraine (till moraine-glacial till) which overlies reddish mudstone (shale) of the Queenston Formation.

The site is situated in a generally level area with a ground surface elevation ranging from approximately 163.0 to 164.5 m. The double-track CNR line is located in a cut approximately 3 to 4 m deep to elev 160.5 m approximately, crossing the site from northeast to southwest. The fields on either side of the CNR tracks are presently cultivated and growing forage crops. On the eastern end of the investigated area, there is a small stand of mixed deciduous trees and scrub bush.

A small pond, possibly intermittent, exists on the Highway 403 centerline near Chainage 27+175. Drainage on the farm fields is poor due to the impervious nature of the soil with surface water being pooled in low areas for several days following rain. No distinct drainage courses cross the site, although the CNR Subway ravine is about 250 m to the west of the bridge site area. Ditches are present on both sides of the CNR tracks and drain southward toward the Bronte Creek valley.

Topography and geotechnical conditions are shown on Drawings 4088501-A and 4088501-B. Details of the geotechnical conditions encountered at the borehole locations, together with laboratory test results, are summarized in the Record of Borehole forms. Grain size distributions are shown in Figures 1 and 2. Plasticity charts showing results of consistency limits testing are presented in Figures 3 and 4.

#### **3.2 Overburden Conditions**

The overburden at the site is generally of glacial origin. The major portion of the materials encountered are indicative of sediments carried or deposited by a glacier (glacial till). However, some pockets or lenses showed signs of bedding while others consisted almost completely of silt sizes. These latter deposits may be of glaciolacustrine origin representing intermittent ponding between glacial advances. The glacial deposits generally consist of a heterogeneous mixture of sand, gravel, silt and clay sizes with

occasional cobbles and boulders. While the larger particles were not encountered in the boreholes, several cobbles and small boulders were observed on the exposed soil surface in the CNR cut. The percentage composition of the various constituent particles varies throughout the deposit. The major soil types encountered on this site are described below.

Most of the site is covered by a layer of cultivated topsoil having a thickness of approximately 0.4 m.

### **3.2.1 Heterogeneous Mixture of Clayey Silt and Sand, Some Gravel (Glacial Till)**

The heterogeneous mixture of clayey silt and sand, some gravel comprises the major portion of the overburden deposits on the project site. It was encountered in all boreholes with its measured thickness ranging from 2.7 m in BH-106 to 15 m in BH-107, with the average thickness being 10 m. The color generally grades from brown near the ground surface to reddish brown progressing down toward the bedrock.

A typical grain size distribution curve for this material is presented in Figure 1, which shows a well graded soil containing 62% silt and clay sizes, 28% sand and 10% gravel.

The results of Atterberg limits tests yielded average liquid and plastic limits of 23% and 13% respectively. As shown in Figure 3, it can be classified as a clayey silt of low plasticity (CL). Natural moisture contents ranged from approximately 7% to 12% and averaged 8%, being well below the plastic limit.

SPTs generally met refusal ( $N > 100$ ) except in the upper 1 to 4 m where 'N' values between 38 and 92 were measured. The consistency of this deposit can be described as 'hard'.

### **3.2.2 Sandy Silt to Silty Sand, Some Gravel, Trace Clay (Glacial Till)**

Within and near the base of the deposit described in Section 3.2.1, there are pockets, lenses or layers which grade from nonplastic combinations of silt, sand and gravel (GM-SM) to low plastic mixtures of sandy silt, trace clay, trace gravel (ML). On the stratigraphic profiles shown on Drawings 4088501-A and 4088501-B, these deposits

are generally described as sandy silt to silty sand, some gravel, trace clay (Glacial Till). Such materials were encountered in all boreholes except BH-104 and ranged in thickness from approximately 1.5 to 3.7 m.

Five grain size distribution curves which are typical of these materials are shown in Figure 2. Two sets of Atterberg limits tests on the low plastic portions yielded liquid and plastic limit values of approximately 16 and 13% respectively as shown in Figure 4.

All SPTs performed in these soils met refusal indicating that the deposit exists in a very dense state.

### **3.2.3 Silt and Interbedded Silt and Fine Sand, Some Gravel**

Several thin layers, pockets or lenses of dense to very dense silt or interbedded silt and fine sand, some gravel were encountered in BH-102, 103, 104, 106 and 107. They ranged in thickness from a few millimetres in BH-104 and 107 to more than a metre in BH-106.

A silt sample from BH-106 was tested and found to be nonplastic with a natural moisture content of 17%.

These thin deposits may be of glaciolacustrine origin.

## **3.3 Bedrock Conditions**

The bedrock in the area is a mudstone of the Queenston Formation frequently referred to as the Queenston Shale. Bedrock surface elevations were inferred, on the basis of drill performance, to be between elev 146.8 and 147.5 in the CNR bridge structure area. As indicated in Section 2, coring did not commence until a depth of 0.4 to 0.5 m below the inferred bedrock surface. Some of the rock in this surface zone is possibly weathered.

For the purposes of this report, 'sound' bedrock has been used to describe those zones where a core recovery of at least 90% and a Rock Quality Designation (RQD) of at least 30% are consistently achieved. Bedrock with these qualities has been assumed to require typical rock excavation techniques.

Using the above definition, all the cored bedrock was generally found to be 'sound'. The rock mass was faintly weathered and contained only occasional thin bands of moderately weathered rock material in the upper 2 m. Core recovery in BH-101, 102 and 103 was 94% or greater and the RQD ranged from 64 to 100%.

BH-103 encountered a 50-mm thick band of moderately to highly weathered mudstone at approximately elev 146 m. Infrequent brecciated bands ranging from 5 to 130 mm thick were also identified in the upper 2 m.

The rock in general was thinly bedded but ranged from very thinly bedded to medium bedded.

### **3.4 Groundwater Conditions**

Groundwater level observations from the two piezometers, including their installation details, are summarized in Table 2. In BH-104, located east of the CNR tracks, the groundwater level in the overburden appears to be at or near ground surface. On January 9, 1991, ice was encountered at the ground surface at elev 163.1. In BH-103, located in the CNR cut, the piezometric level in the bedrock also appears to be within 1 m of ground surface in the cut area, but approximately 3 m lower (elev 159.9 m) than the water level measured in the overburden, indicating a downward flow of groundwater.

Measurements in a piezometer about 100 m west of the CNR cut and located near the bedrock surface indicated a groundwater level at elev 161.6 m on January 9, 1991.

## **4 Geotechnical Design and Construction Considerations**

### **4.1 General Configuration of Structures**

It is understood that the work proposed at the subject site involves

- relocating the existing CNR tracks on a detour around the west side of the site at such a distance from the proposed bridge to permit the bridge to be constructed
- constructing a new railway bridge
- carrying out the excavation required to pass the Highway 403 under the railway bridge, and construction of approach cuts to the west and east of the bridge structure.

The locations (chainages) for the various bridge elements, as suggested by the MTO, are as follows.

North Abutment	26+910 to 26+926	o/s	22.5 to 25.5 Lt.	€
Center Pier	26+899 to 26+917	€		
South Abutment	26+890 to 26+906	o/s	22.5 to 25.5 Rt.	€

These locations are shown by hatching on Drawings 4088501-A and 4088501-B.

Profile control beneath the bridge structure will be approximately elev 153.8 m.

The various geotechnical aspects to be considered in the design and construction of these works are discussed in the following sections, together with recommendations.

### **4.2 Bridge Foundations**

#### **4.2.1 Bridge Pier**

At the location of the bridge for the CNR tracks, the proposed Highway 403 road grade is approximately elev 154.0 m. Therefore, the base of the footing for the bridge pier is estimated to be at approximately elev 152.5 m. The pier could be founded on



spread footings within the overburden or, alternatively, deep foundations could be provided to transmit the loads to 'sound' bedrock at about elev 145.0 m.

If spread footings located within the glacial soils are adopted, a factored bearing capacity of 900 kPa at 'Ultimate Limit State' (ULS) is recommended. The bearing capacity at 'Serviceability Limit State (SLS) Type II' can be assumed as 500 kPa.

When the foundation excavation reaches the design grade, it is recommended that the undisturbed soil be covered, within 4 hours of exposure, with a 75-mm thick concrete mud slab to prevent any softening or deterioration of the soil conditions.

A composite soil sample from the general footing or pile cap zone was tested for sulfate content. The concentration of water-soluble sulfate was sufficiently low to cause negligible damage to normal Portland cement concrete.

Deep foundations for the bridge pier would need to be about 6.5 m long to transfer loads below the moderately weathered layer within the bedrock observed at elev 146.0 m. Driven 310×110 'H' piles are recommended. However, due to the hard nature of the overburden, preaugering a 375-mm dia hole will likely be necessary to ensure the piles reach the required depth. Typically, 310×110 'H' piles driven to refusal on or into a nondeteriorating rock can be assumed to have a factored axial capacity of 1600 kN at ULS. However, due to the lower strength and slaking nature of the Queenston Shale, it is recommended that a factored axial capacity of 1400 kN at ULS be assumed for piles driven into the 'sound' bedrock. Since the settlement of 'H' piles driven into the 'sound' bedrock will consist essentially of the elastic shortening of the pile plus a very small amount due to the consolidation of any weaker shale layers within the zone of influence of the pile tip, the axial capacity at SLS Type II should not govern. However, a axial capacity at SLS Type II of 1000 kN can be assumed.

If piles driven into an augered hole are used, it is recommended that a layer of wet concrete, a minimum of 1 m thick, be placed in the bottom of the hole and have the pile driven through it. The concrete should seal the bedrock surface and prevent the deterioration of the bedrock near the pile tip. Consideration might also be given to grouting up the voids around the pile, depending on the requirement for lateral pile resistance.

Temporary excavations required for construction of the bridge pier footing should be formed with cut slopes no steeper than 1H:1V. BH-103 drilled at the site of the

bridge pier encountered the clayey silt and sand, some gravel down to elev 151.1 m, which is relatively impervious so dewatering excavations in this material should be able to be controlled by conventional sumping and pumping. However, below elev 151.1 m, the soils were silt and fine sand and some gravel, generally cohesionless. If such soils are encountered in the footing excavation, special dewatering techniques such as well points may be required to control the water and prevent the loosening and flow of soil.

#### **4.2.2 Bridge Abutments**

Plan E-83-403-7 indicates that the faces of the bridge abutments will be located just outside the roadway shoulders. At these locations, the abutment footings could be founded at approximately elev 152.5 m within the overburden. As discussed above for the bridge pier, the abutments can be founded either on spread footings in the overburden or abutment loads can be carried by deep foundations down to the bedrock.

For spread footings within the overburden at approximately elev 152.5 m, a factored bearing capacity of 900 kPa at ULS is recommended with a bearing capacity at SLS Type II of 500 kPa.

In considering the sliding resistance of a concrete footing on the dense soil, an unfactored  $\tan \phi$  of 0.45 can be assumed.

Consideration could be given to the use of spread footings perched above the base of the cut. Such an arrangement will result in longer spans but reduce the height of the abutments. Using the 2H:1V slope configuration recommended, the span will increase 2 m for every 1-m rise in footing level up to the area of the 3-m wide bench at elev 157.0 m where the span will increase by 3 m with no reduction in abutment height.

If the footing elevation is raised to about elev 155.5 m, the factored bearing capacity will reduce to 450 kPa at ULS and 250 kPa at SLS Type II. Between elev 152.5 and 155.5 m, the bearing capacities can be assumed to reduce linearly between the two limits.

Steel 310×110 'H' piles driven in a preaugered hole can be used as deep foundations. These should be taken to elev 145.0 m. A factored axial capacity of 1400 kN

at ULS and an axial capacity of 1000 kN at SLS Type II can be assumed. The piles should be driven into a minimum 1-m thick layer of wet concrete.

Backfill of the abutment retaining wall structures should be with free-draining granular materials in accordance with OPSS - Special Provision 109 F03.

All backfill material should be placed and compacted in layers and drained by perforated pipes, weep holes or equivalent in accordance with MTO standards. The lateral earth pressure on the abutment structure will depend on the type of granular backfill material used and the rigidity of the wall. If the abutment loads are carried to 'sound' bedrock and the structure is considered to be rigid and nonyielding, the at-rest pressure would be applicable. However, if the abutments are free to rotate, the active pressure could be used. The following parameters are recommended for use in the design.

	Granular "A"	Granular "B"
Unit weight ( $\text{kN/m}^3$ )	22.8	21.2
Friction angle ( $^\circ$ )	35	30
Active earth pressure coefficient ( $K_a$ )	0.27	0.33
At-rest earth pressure coefficient ( $K_o$ )	0.43	0.5

#### **4.2.3 Frost Protection**

All footings or pile caps should be placed at a minimum depth of 1.2 m below finished grade to provide adequate protection against frost action.

### **4.3 CNR Detour during Construction**

Construction of the proposed bridge requires the temporary relocation of the twin CNR tracks around the west side of the bridge construction area. It is understood that, during the period of construction, the railway lines must be operational at or near the same grade without any interruption in service. According to information provided by the MTO, the centerline of the detour will be approximately 23 m west of the centerline of the existing tracks, measuring perpendicularly from the tracks. To maintain the same track grades, it will be necessary to place the relocated tracks in a temporary cut about 3 to 4 m deep.

While the details of the configuration of the detoured track and the size and location of the bridge foundations are not fixed at the time of preparing this report, it appears unlikely that space will permit construction of the bridge in an excavation with a sloping face extending up toward the detour.

Several alternatives could be considered. The first would involve moving the location of the detoured track further west to obtain the required space. If this is not feasible, then an arrangement incorporating either a full-height vertical tied-back wall or a combination of an excavated slope and a vertical wall of lesser height could be used. The following parameters which are shown in Figure 5 are suggested for laying out this latter arrangement. First, establish the length of the bridge footing and the space required on the west side of the bridge foundation for construction purposes. In Figure 5, the footing size is as shown on MTO sketches and a working space of 1.5 m has been assumed on the west side of the footing. This will define the alignment of the vertical wall. It is recommended that the crest of the excavation be located no closer than 7 m from the centerline of the nearest set of detoured tracks. If an excavation slope (2H:1V) is extended from track grade down to intersect the line of the wall, this will define the wall height. It is sometimes desirable to have a level area behind the wall for access and drainage control, however, the layout shown does not provide for such a zone because of space limitations.

Assuming that the excavation extends down to approximately elev 152.0 m, a single, full height wall would be about 9 m high. Using a partially sloped excavation will reduce the height of the wall only slightly. The dimensions assumed in preparing the sketch may not be consistent with the actual condition but the parameters indicated can be used in any other layout.

The most feasible type of retaining structure would probably consist of steel 'H' soldier piles and timber lagging in combination with grouted soil anchors. Because of the density of the soil in the area, it is not considered feasible to drive steel sheet piles on this site. The soldier piles would be placed in predrilled vertical holes about 600-mm dia. If water-bearing zones or pockets of cohesionless soil are encountered, it may become necessary to keep the walls of the augered holes stable using drilling mud or casing. The toes of the piles would be set in concrete up to excavation grade. Where drilling mud is used, it will be necessary to place the concrete by tremie methods. Timber lagging with a thickness of 75 to 100 mm should provide the necessary soil support between piles spaced at about 2-m centers.

If the wall is 9 m high, it will probably be necessary to use two rows of anchors; however, if it can be reduced to only 5 m high, a single row of anchors may be sufficient.

The soils to be supported will generally consist of low plastic, hard, clayey silt and sand, some gravel. There will also be pockets or lenses of generally cohesionless combinations of sand, silt and gravel which are very dense. These latter deposits may be water bearing.

For the purposes of designing the tie-back wall, it is recommended that a uniform pressure distribution be assumed equal to

$0.4 \gamma H K_o$ , where

$\gamma$  = 22 kN/m<sup>3</sup> above GWT (groundwater table)

$\gamma_s$  = 12 kN/m<sup>3</sup> below GWT

H = height of wall

$K_o$  = 0.46 (based on an assumed angle of shearing resistance of 33° for the glacial till as measured on another project with similar soil).

Assume groundwater level at elev 160.0 m.

When the configuration of the wall and excavation are resolved, further information can be provided on the spacing, length and inclination of the required tie-back anchors. However, for preliminary considerations, anchors placed in 300-mm dia holes and having a bond length of about 6 m can be assumed to have an ultimate capacity of approximately 1000 kN.

#### **4.4 Approach Cuts and Subway Excavation**

The depth of excavations required for the proposed Highway 403 grade range from about 11 m on either side of the new CNR bridge to a minimum of 4 m at the eastern end of the investigated area. The cuts will generally be made in the hard clayey silt and sand, some gravel (clayey silt till) except at the eastern end which will encounter both the clayey silt till and the generally cohesionless glacial till deposits. East of Chainage 27+000 approximately, a cohesionless zone of sandy silt to silty sand, some gravel, trace clay lies close to the road grade. Slopes of 2H:1V are recommended for the excavations; however, due to the drainage considerations described below, benches and toe drains will be required in some areas.

Preliminary measurements indicate that the groundwater level is in the elevation range of 161.0 to 162.0 m just west of the CNR bridge, and between elev 162.0 and 163.0 m to the east. Therefore, unless suitable drainage measures are provided, seepage will emerge from the cut slopes and cause softening and sloughing of the soil. To the west of Chainage 27+000, it is recommended that drainage trenches be provided both at road grade level and in a bench at elev 157.0 m to depress the groundwater level and prevent it intersecting the cut slope. Commencing in the vicinity of Chainage 26+980 and extending to the east, it is recommended that the bench and drainage trench be sloped up to elev 160.0 m. The benches should be 3 m wide to facilitate installation of the slope drains and provide access for future maintenance. Typical details to the west and east of Chainage 27+000 are shown in Figures 6, 7 and 8 respectively. The drains should be constructed with an adequate grade to promote flow toward the drainage discharge point. It is anticipated that the quantity of seepage will be small because of the relatively impervious nature of the upper glacial soils. To the east of Chainage 27+000, due to the proximity of the sandy silt to silty sand, some gravel, trace clay, to the road grade and the possibility of these materials flowing into the cut under seepage forces, it is recommended that a granular toe drain be placed in the lower part of the slope. The toe drain should extend a minimum of 2.5 m above the road grade elevation. Typical details are shown in Figure 8.

By cutting the toe drain into the slope, it is possible to minimize the increase in the overall width of the approach cut. However, to reduce the possibility of slope instability by cutting away the toe, it is recommended that the work be completed in lengths not greater than 15 m.

All cut slopes must be vegetated to protect them against runoff erosion. Also, interceptor ditches must be provided at the top of each cut slope.

Seepage inflows during construction should be able to be controlled by sumping and pumping, although deep ditches may be required for pumping in some areas where inflows are larger. For the long term, gravity drainage facilities are considered adequate if properly installed and maintained.



A handwritten signature in cursive script, reading "C. S. Bodimeade", positioned above a horizontal line.

C. S. Bodimeade  
Senior Geotechnical Engineer



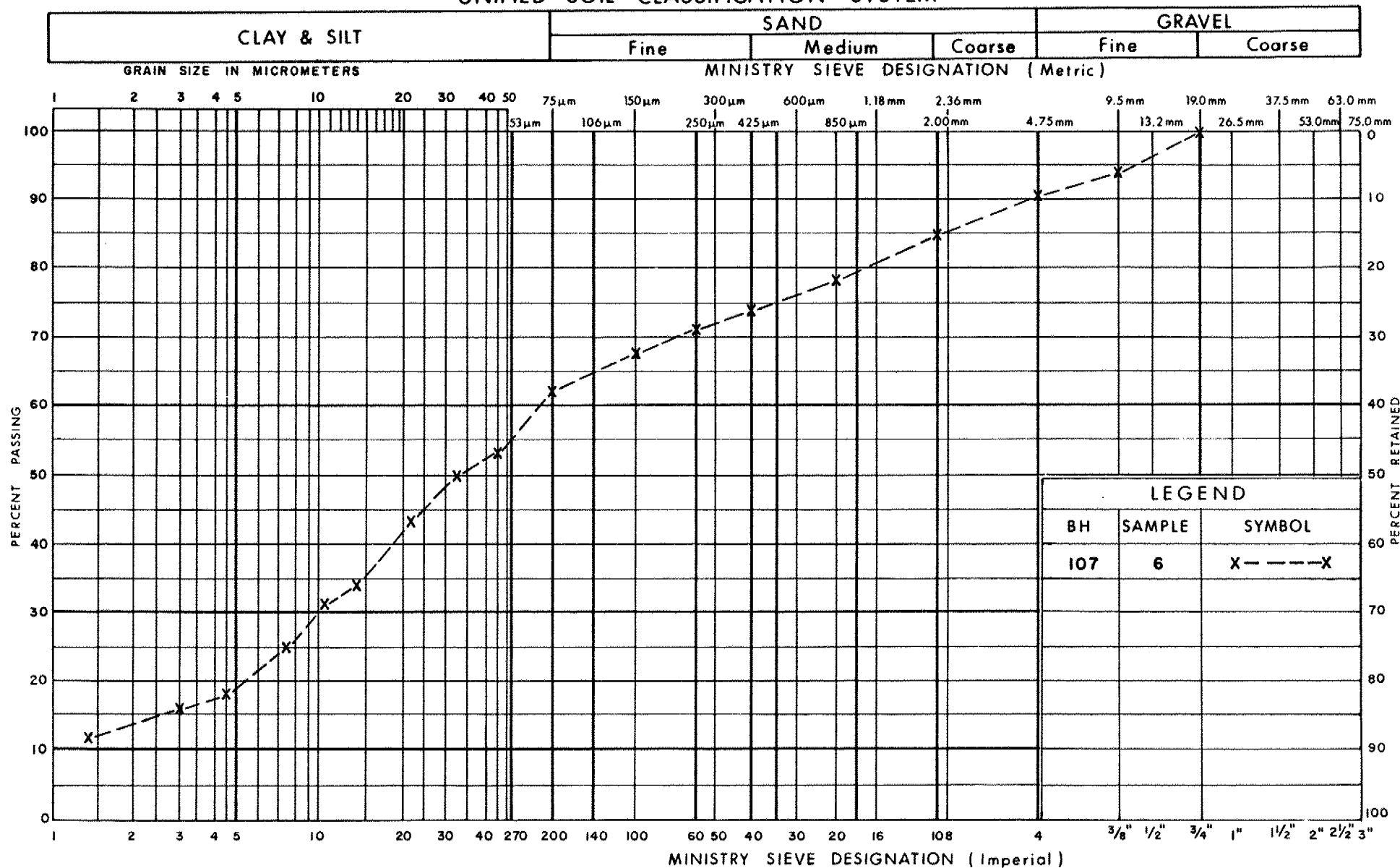
A handwritten signature in cursive script, reading "T. J. Bradshaw", positioned above a horizontal line.

T. J. Bradshaw  
Deputy Head, Geotechnical Department

## Figures



## UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

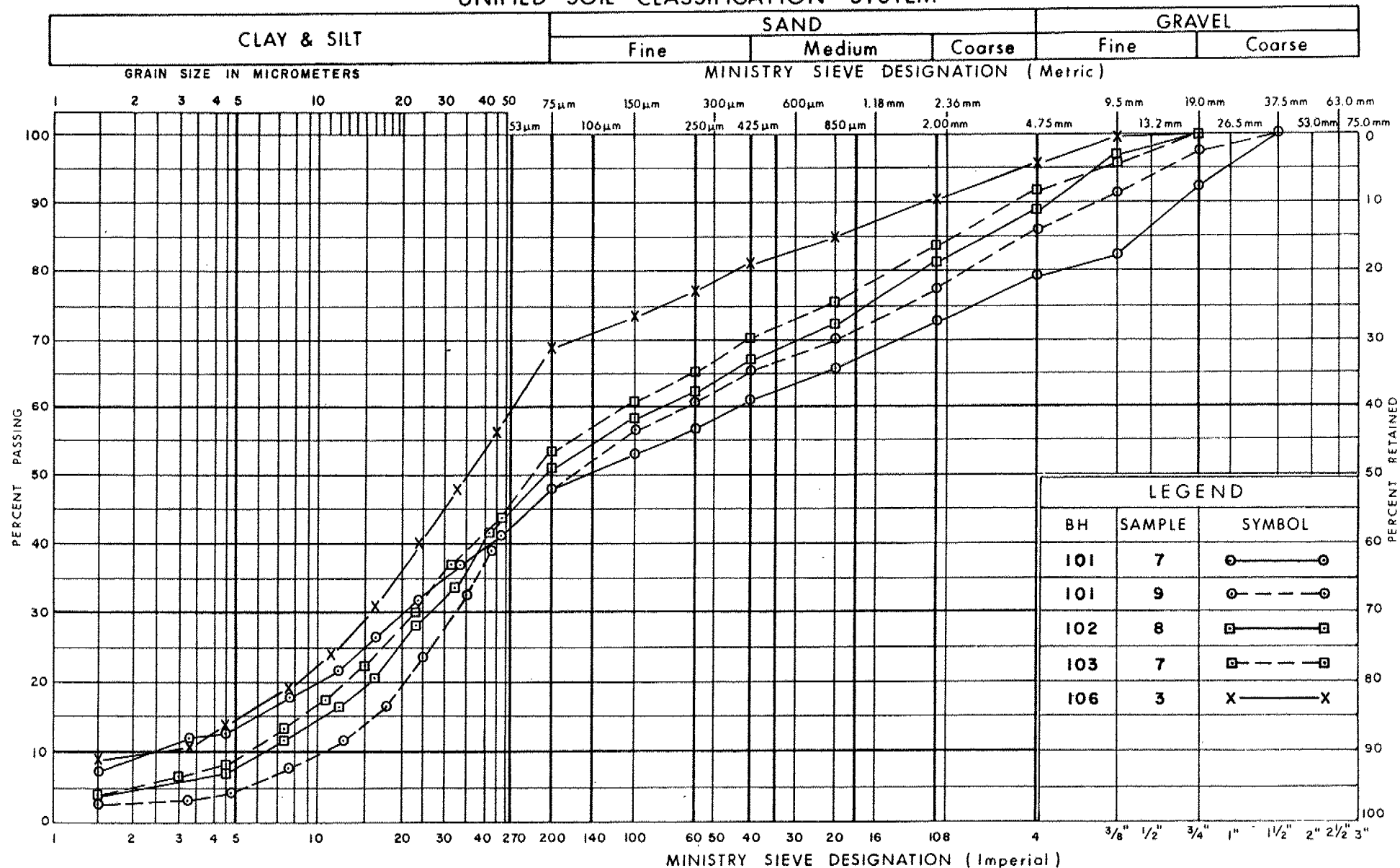
Ministry of  
Transportation

**GRAIN SIZE DISTRIBUTION**  
**HETEROGENEOUS MIXTURE OF CLAYEY SILT**  
**AND SAND, SOME GRAVEL**  
 ( GLACIAL TILL )

FIG No 1

W P 408-85-01

## UNIFIED SOIL CLASSIFICATION SYSTEM

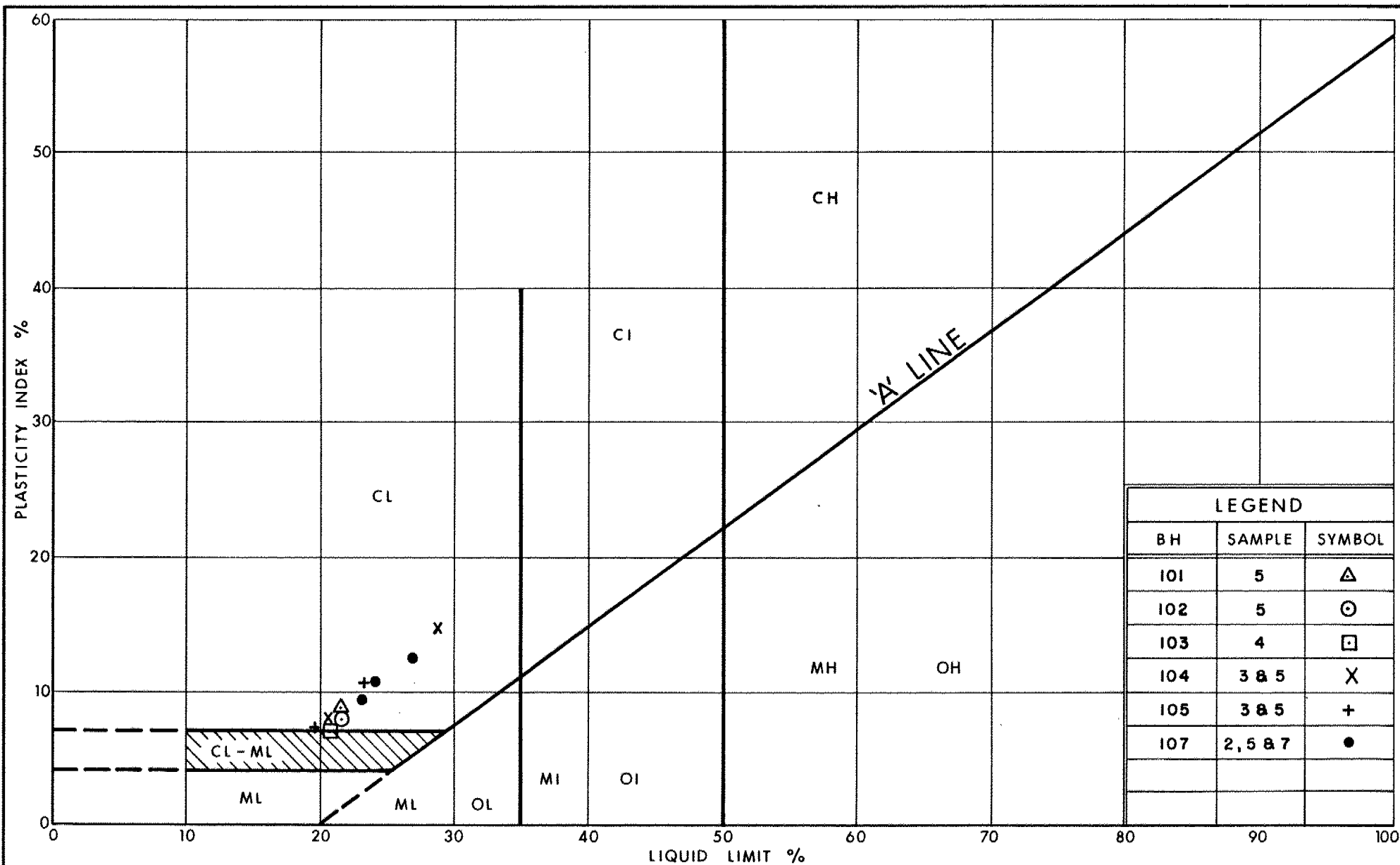


Ministry of  
Transportation

**GRAIN SIZE DISTRIBUTION**  
SANDY SILT TO SILTY SAND,  
SOME GRAVEL, TRACE CLAY  
(GLACIAL TILL)

FIG No 2

W P 408-85-01

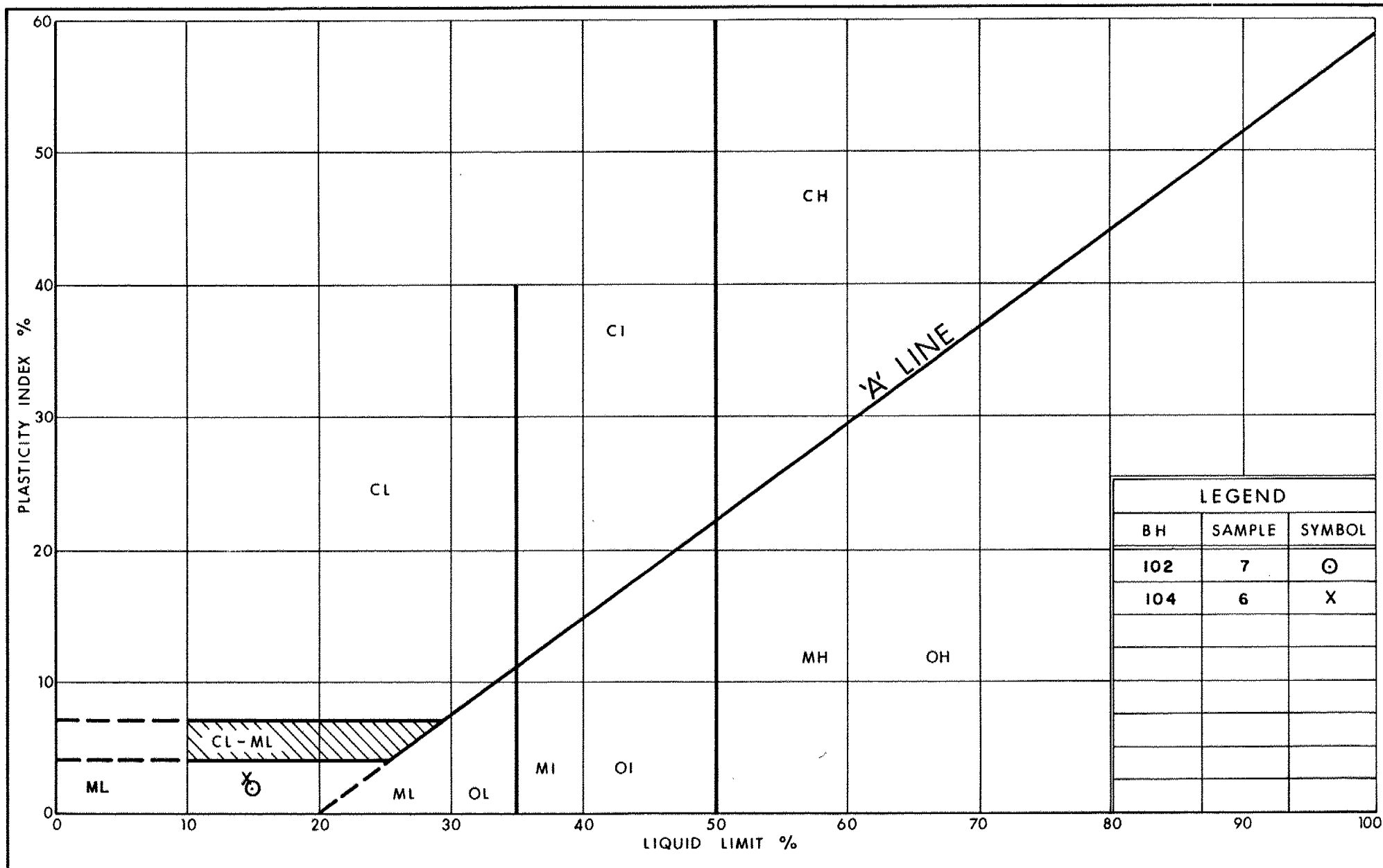


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Transportation  
Ontario

PLASTICITY CHART  
HETEROGENEOUS MIXTURE OF CLAYEY SILT AND  
SAND, SOME GRAVEL  
(GLACIAL TILL)

FIG No 3

W P 408-85-OI

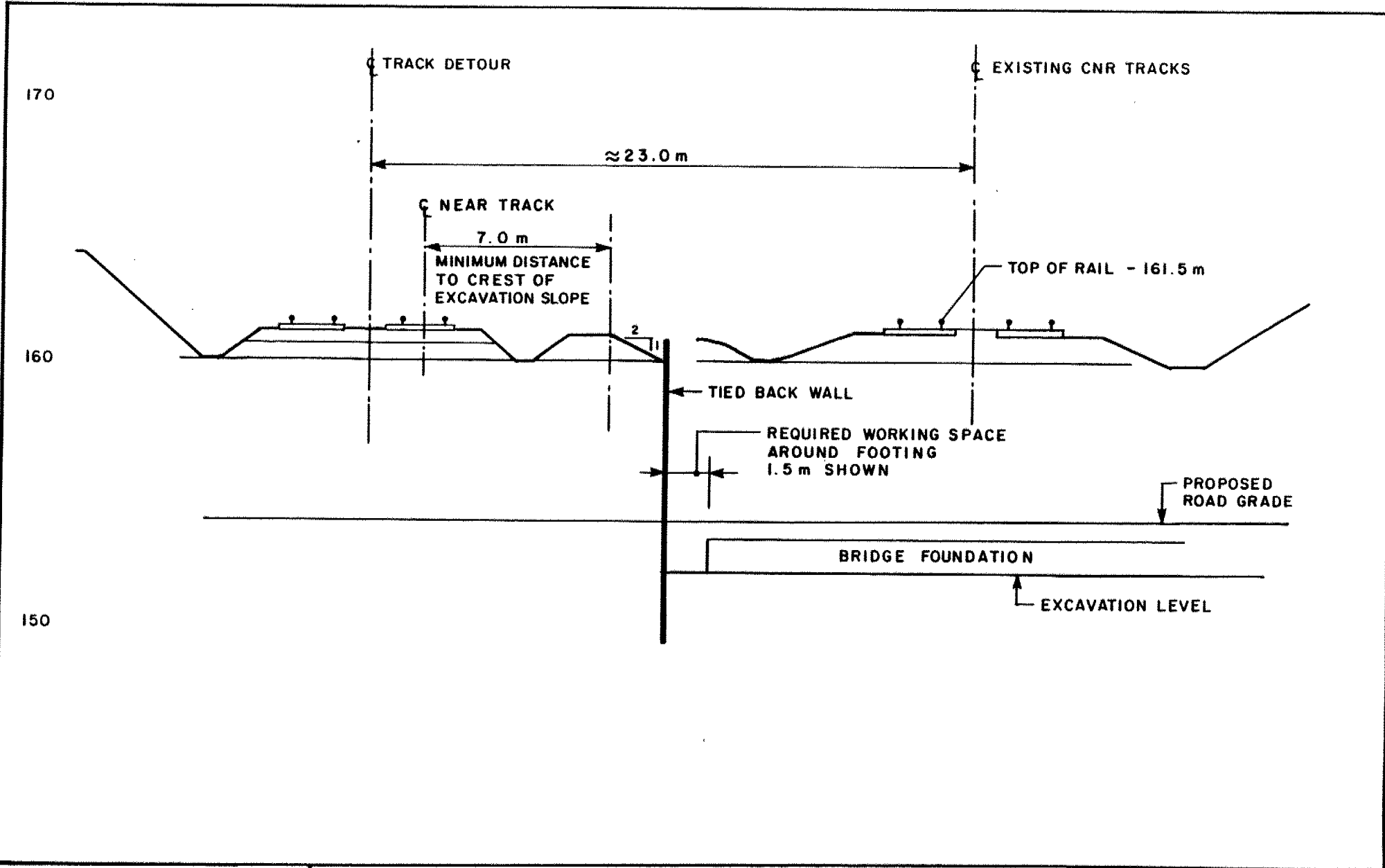


Ministry of  
Transportation  
Ontario

PLASTICITY CHART  
SANDY SILT TO SILTY SAND,  
SOME GRAVEL, TRACE CLAY  
(GLACIAL TILL)

FIG No 4

W P 408-85-01

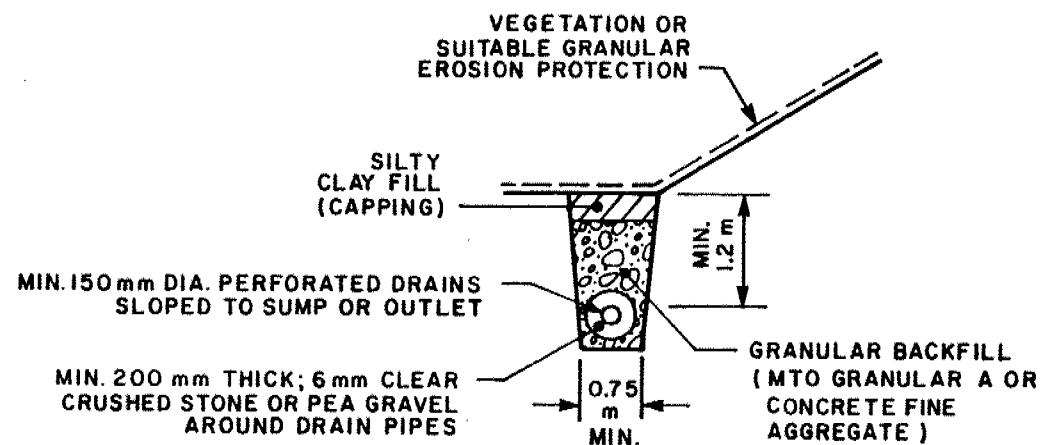
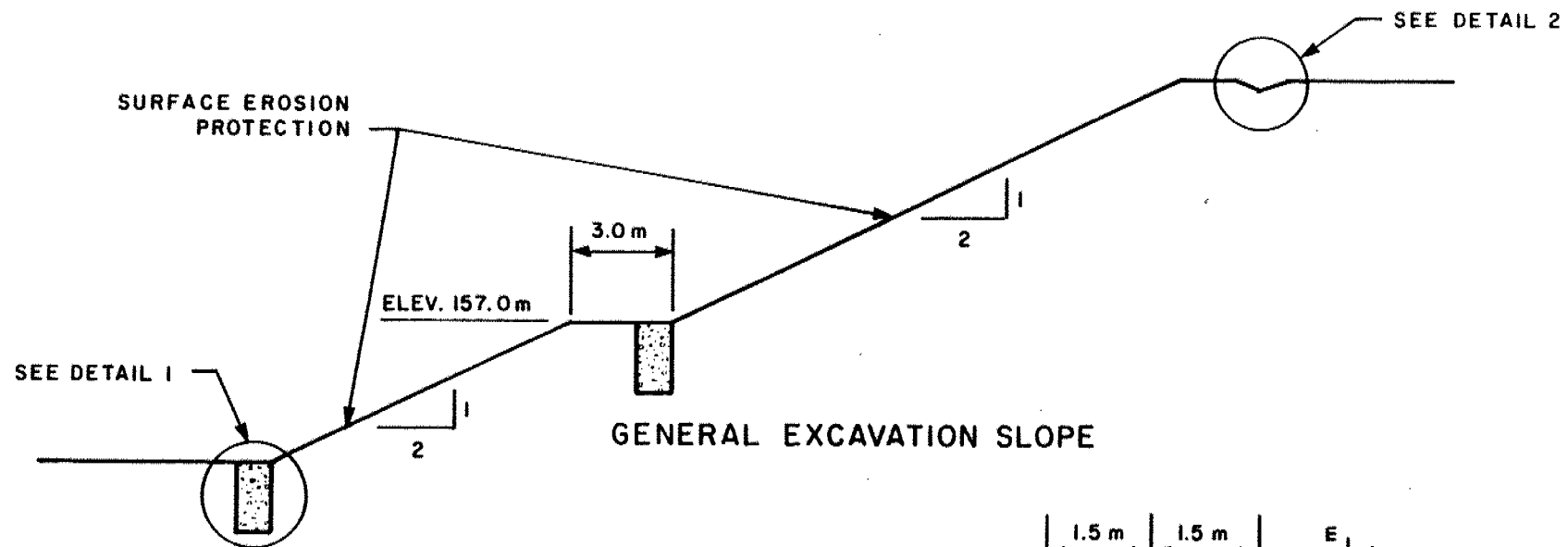


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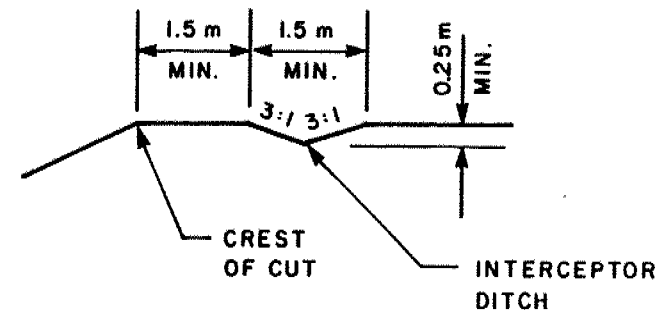
# LAYOUT CRITERIA FOR TIED BACK WALL ADJACENT TO RAILWAY TRACK DETOUR

FIG No 5

W P 408-85-01



DETAIL 1



DETAIL 2

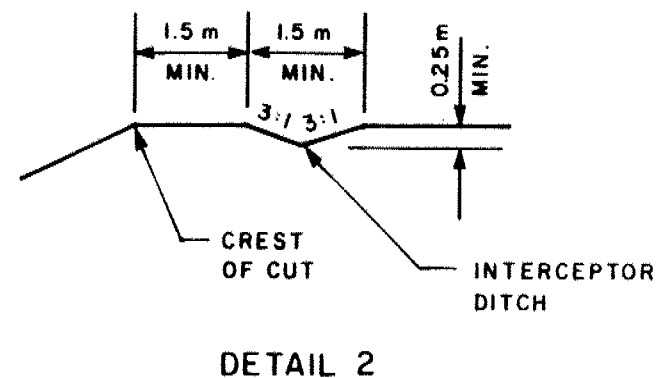
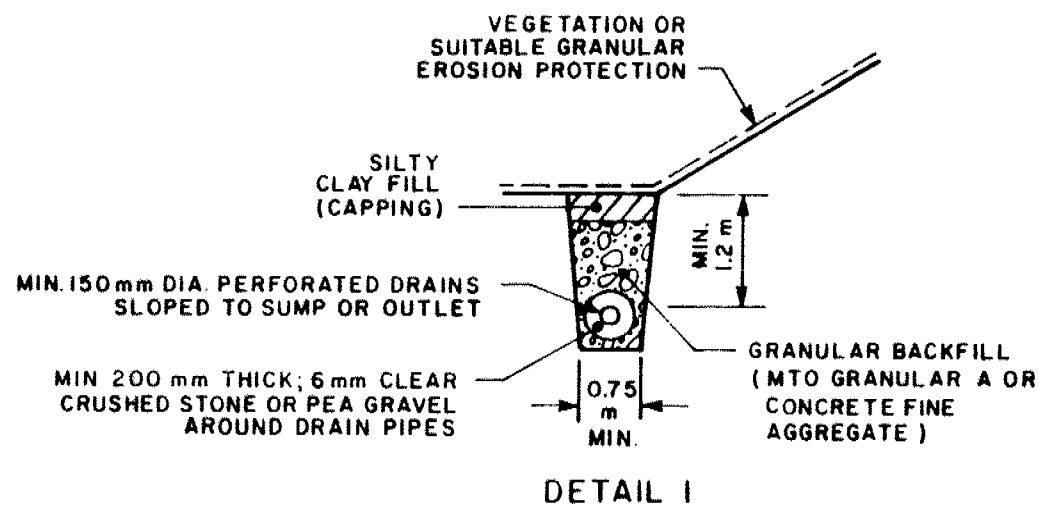
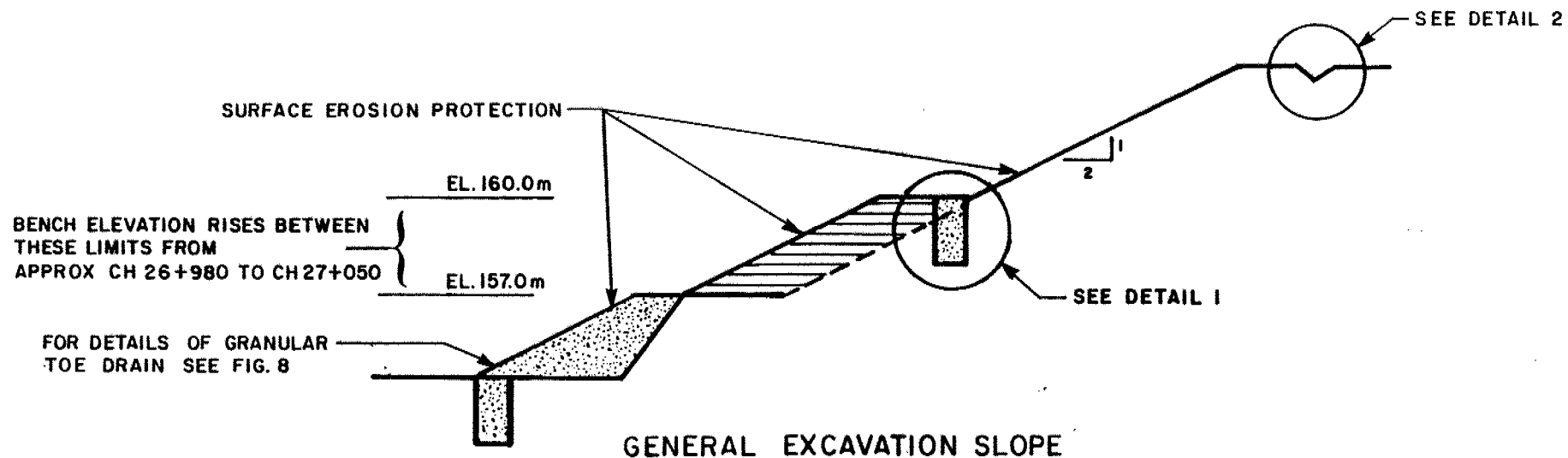


Ministry of  
Transportation

# RECOMMENDED DRAINAGE SYSTEM WEST OF CHAINAGE 27+000 (APPROX.)

FIG No 6

W P 408-85-01

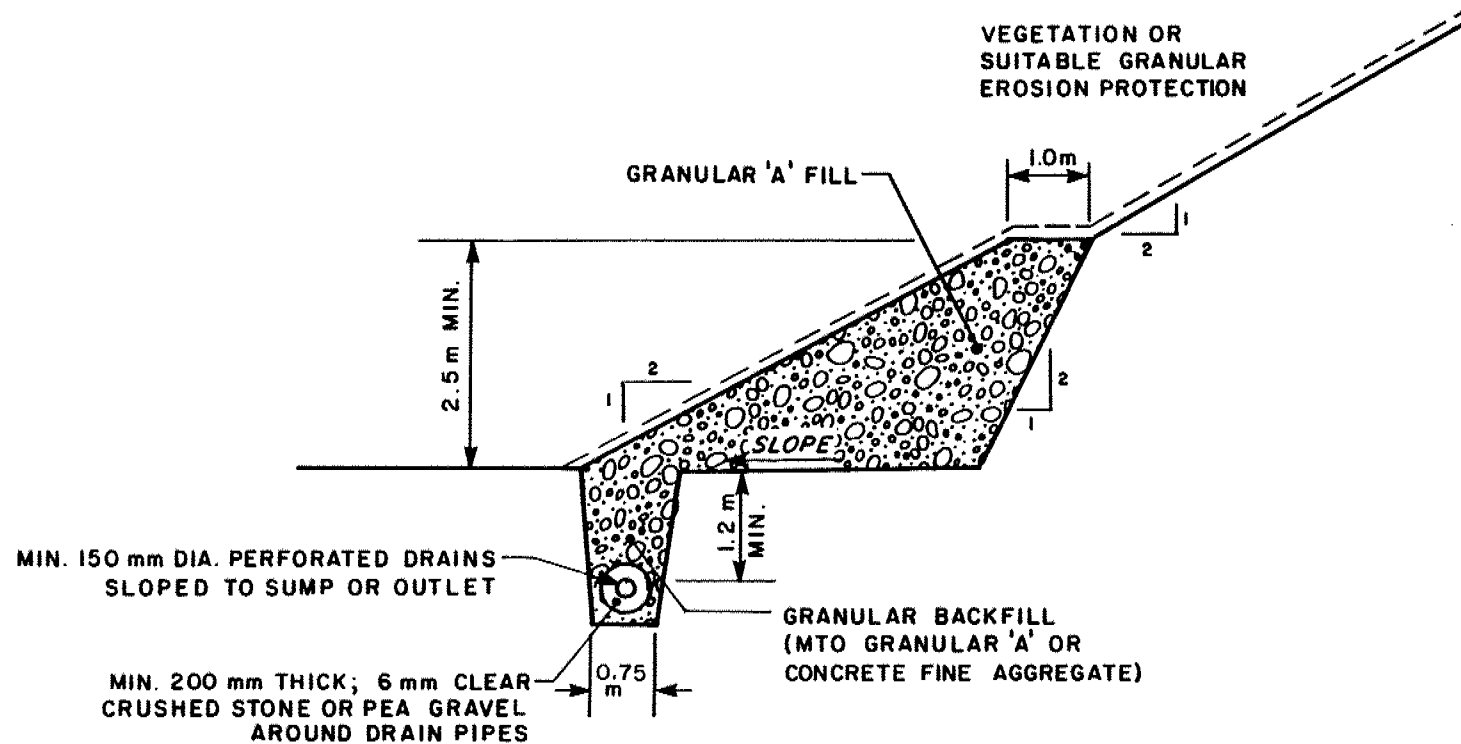


Ministry of  
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# **RECOMMENDED DRAINAGE SYSTEM** **VICINITY OF CHAINAGE 27+000 AND EASTWARD**

FIG No 7

W P 408-85-01



Ministry of  
Transportation

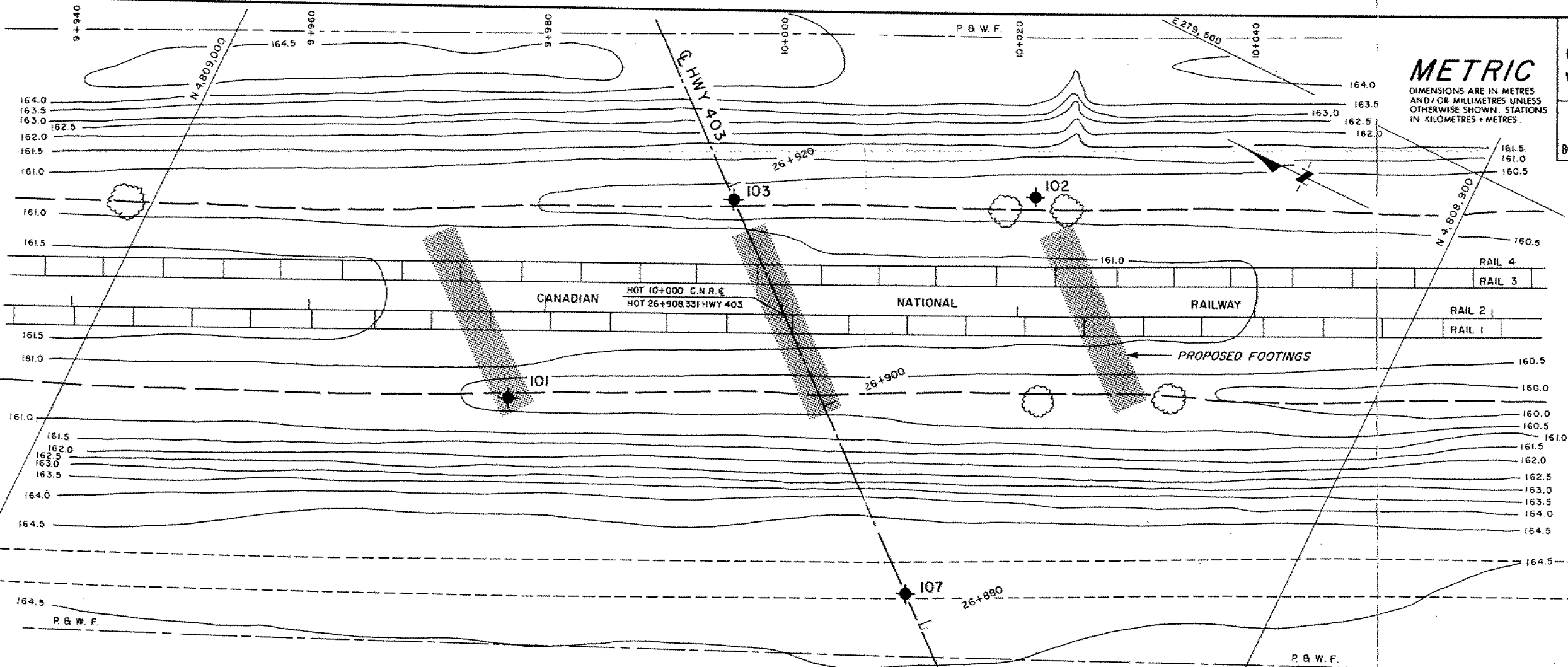
# GRANULAR TOE DRAIN AND DRAINAGE TRENCH EAST OF CHAINAGE 27+000 (APPROX.)

FIG No 8

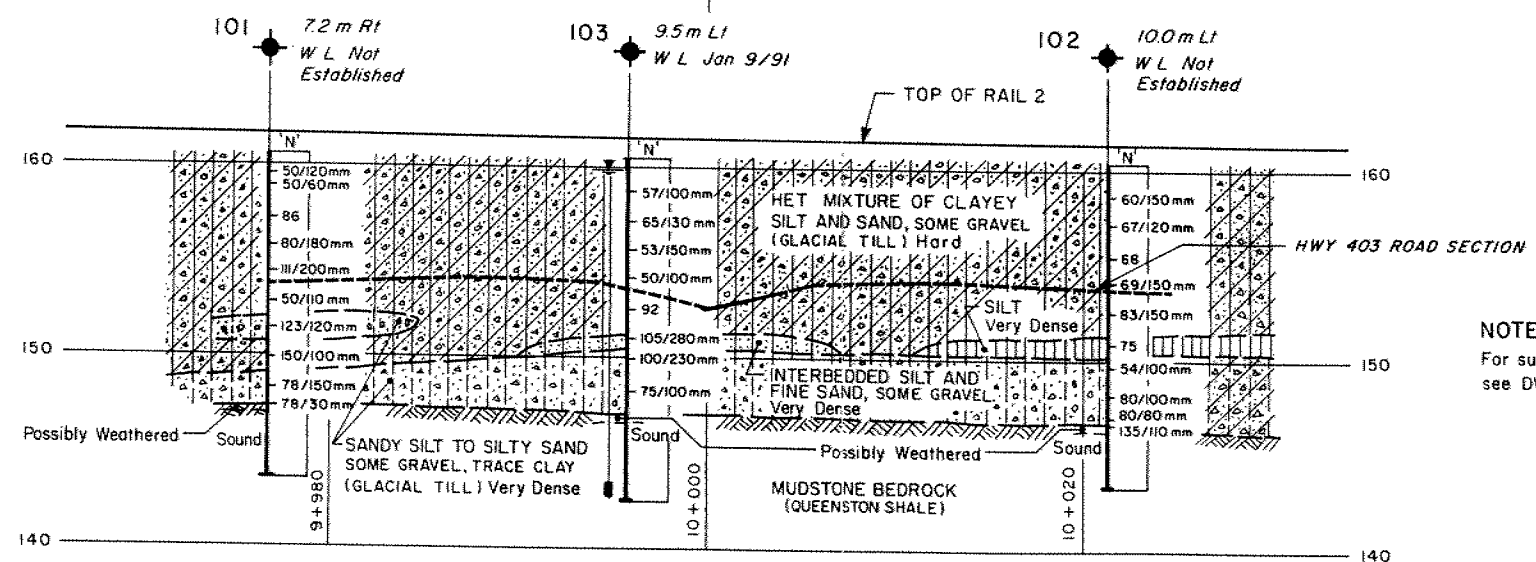
W P 408-85-01



**Drawings**



PLAN  
SCALE  
5m 0 5 10m



PROFILE OF C.N.R.  
TOP OF WEST RAIL  
SCALE  
5m 0 5 10m

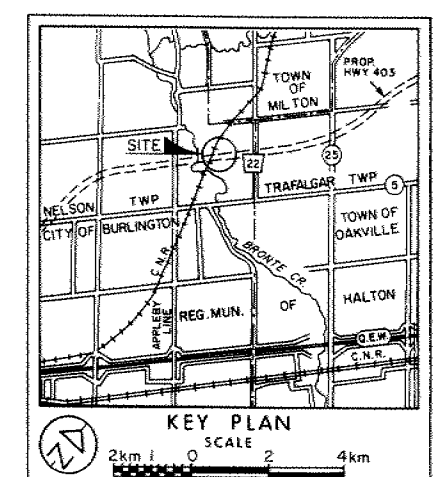
**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES.

CONT No  
WP No 408-85-01

C.N.R. SUBWAY  
BORE HOLE LOCATIONS & SOIL STRATA



ACRES INTERNATIONAL LIMITED



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
- Piezometer Tip
- Rt. Offset Right of Rail 2
- Lt. Offset Left of Rail 2

No	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
101	160.5	4 808 966	279 447
102	160.3	4 808 933	279 482
103	160.4	4 808 956	279 470

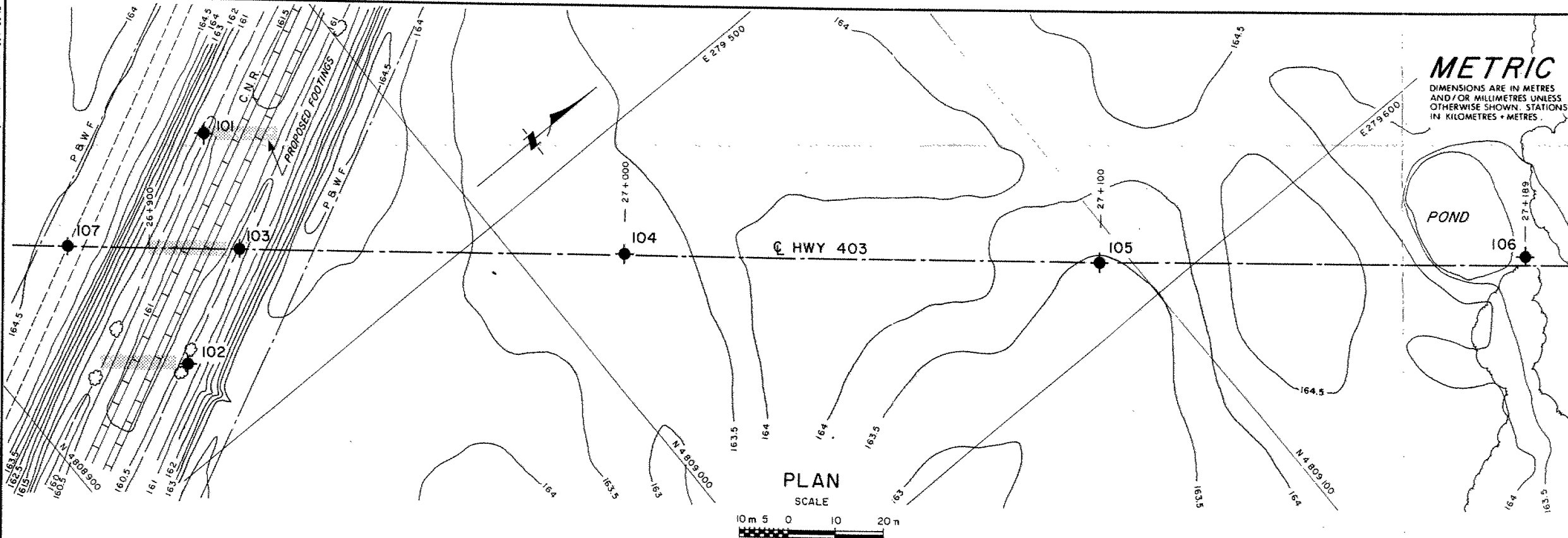
NOTE  
For subsurface information for BH-107,  
see DWG 4888501-B

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			
2			
3			
4			
5			
6			
7			
8			
9			
10			

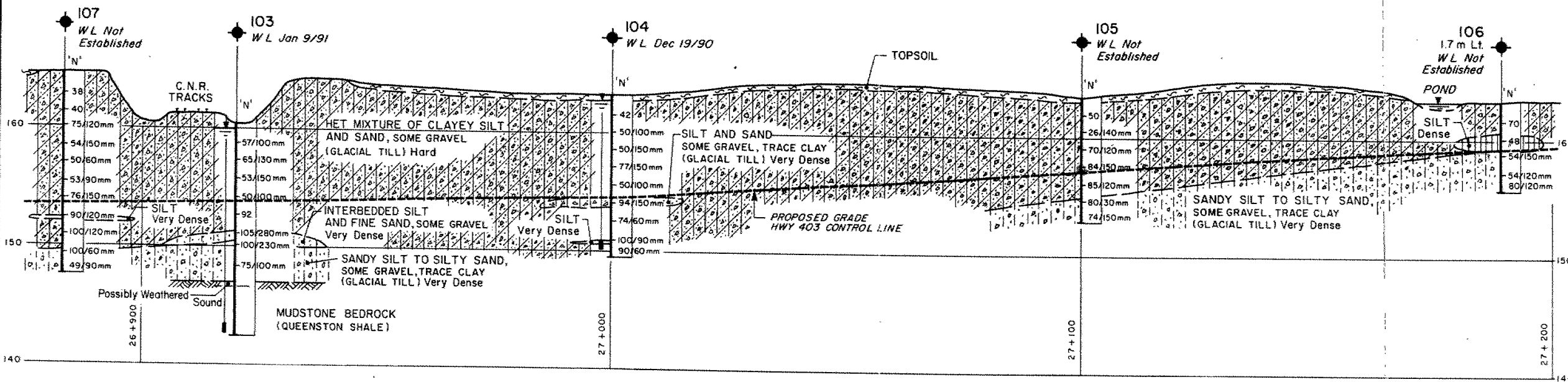
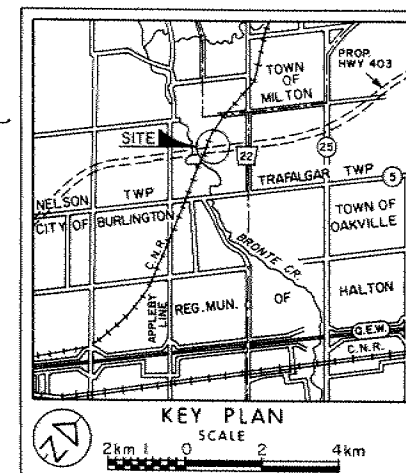


**CONT No**  
WP No 408-85-01

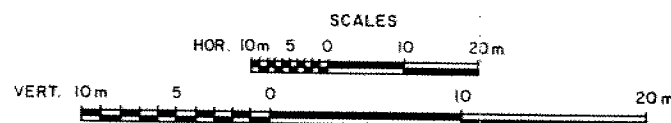
**C.N.R. SUBWAY**  
BORE HOLE LOCATIONS & SOIL STRATA

**SHEET**

**ACRES INTERNATIONAL LIMITED**



**PROFILE HWY 403**



**NOTE**

For subsurface information for BH-101 and BH-102 see DWG 4088501-A

**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)
- Wt at time of investigation
- Piezometer Tip
- Rt. Offset Right of C
- Lt. Offset Left of C

No	ELEVATION	CO-ORDINATES NORTHING	EASTING
101	160.5	4 808 966	279 447
102	160.3	4 808 933	279 482
103	160.4	4 808 956	279 470
104	163.1	4 809 018	279 522
105	163.5	4 809 095	279 587
106	163.4	4 809 164	279 643
107	164.6	4 808 928	279 447

**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			

Geacres No 30M5-170

HWY No 403	DIST 4
SUBMD 20	CHECKED 40
DATE JAN 1991	SITE 10-478
DRAWN T.T.	CHECKED 40
APPROVED 40	DWG 4088501-B

## **Explanation of Terms Used in Report**

## 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:

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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	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
$C_\alpha$	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
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_f$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	$kg/m^3$	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	$kn/m^3$	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	$kg/m^3$	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	$kn/m^3$	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	$kg/m^3$	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	$kn/m^3$	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	$kg/m^3$	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	$m^3/s$	RATE OF DISCHARGE
$\gamma_d$	$kn/m^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	$kg/m^3$	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	$kn/m^3$	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	$kg/m^3$	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	$kn/m^3$	SEEPAGE FORCE
$\gamma'$	$kn/m^3$	UNIT WEIGHT OF SUBMERGED SOIL						

## Broken Zone

Zone of full diameter core of very low RQD which may include some drill-induced fractures.

## Fragmented Zone

Zone where core is less than full diameter and RQD = 0.

## Strength

Term	Description	Unconfined Compressive Strength	
		(MPa)	(psi)
Extremely weak rock	Indented by thumbnail	0.25-1.0	36-145
Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0-5.0	145-725
Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0-25	725-3625
Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25-50	3625-7250
Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50-100	7250-14500
Very strong rock	Specimen requires many blows of geological hammer to fracture it	100-250	14500-36250
Extremely strong rock	Specimen can only be chipped with geological hammer	>250	>36250

## Weathering

Term	Description
Fresh	No visible sign of rock material weathering.
Faintly weathered	Discoloration on major discontinuity surfaces.
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than in its fresh condition.
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Highly weathered	More than half of the rock is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely weathered	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.
Residual	All rock material is converted to soil. The mass structure and material soil fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

## Soil Description

Term	Example	%
Trace	Trace sand	1 - 10
Some	Some sand	10 - 20
Adjective	Sandy	20 - 35
And	And sand	>35
Noun	Sand	>50

## **Record of Boreholes**



# RECORD OF BOREHOLE No 101

METRIC

W P 408-85-01 LOCATION Co-ords. 4 808 966 N: 279 447 E ORIGINATED BY REC  
DIST 4 HWY 403 BOREHOLE TYPE Solid Stem Auger, NO Rock Core COMPILED BY REC  
DATUM Geodetic DATE 90 11 14 to 15 CHECKED BY TJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	120	140	160	180		
160.5	Ground Level													GR SA SI CL
0.0	Heterogeneous mixture of clayey silt and sand some gravel, dry, low plasticity		1	SS	50/	120 mm	160							
	CL (Glacial Till)		2	SS	50/	60 mm								
	Hard Brown becoming reddish brown		3	SS	86		158							
			4	SS	80/	180 mm	156							
			5	SS	111/	200 mm	154							
			6	SS	50/	110 mm								
	Silt and sand, some gravel, trace clay) Very Dense		7	SS	123/	120 mm	152							20 32 39 9
			8	SS	150/	100 mm	150							
149.0														
11.5	Sandy silt, some gravel, trace clay, nonplastic to low plasticity (Glacial Till)		9	SS	78/	150 mm	148							15 36 46 3
147.5	Very Dense Reddish Brown		10	SS	78/	30 mm								
13.0	Mudstone Bedrock (Queenston Shale) Possibly weathered			NQ										RQD = 64%
	Sound		11	RC	REC 94%		146							
	Reddish brown with occasional gray bands, medium strong faintly weathered, thinly bedded		12	NQ	REC 100%									RQD = 85%
	14.28-14.50: Breccia- ated		13	NQ	REC 100%		144							RQD = 100%
143.6				RC										
16.9	End of borehole													
	Bedrock surface in- ferred from drill action.													
	Groundwater elevation not established. Surface water flowing over borehole collar.													

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 102

METRIC

W P 408-85-01 LOCATION Sta. 26 + 909 o/s 24.0 Rt. of Hwy. 403  
 DIST 4 HWY 403 BOREHOLE TYPE Solid Stem Auger, NQ Rock Core  
 DATUM Geodetic DATE 90 11 16  
 ORIGINATED BY REC  
 COMPILED BY REC  
 CHECKED BY TJB

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100				
160.3	Ground Level														GR SA SI CL
0.0	Heterogeneous mixture of clayey silt and sand some gravel, dry, low plasticity		1	SS	60	150 mm									
	CL (Glacial Till)		2	SS	67	120 mm									
	Hard Brown becoming reddish brown		3	SS	68										
			4	SS	69	150 mm									
			5	SS	83	150 mm									
151.3			6 <sup>a</sup>	SS	75										
9.0	Silt and clay be- coming uniform silt		6 <sup>b</sup>	SS	75										
150.3	Very Dense Brown		7	SS	54	100 mm									
10.0	Silty sand to sandy silt, some gravel, trace clay, dry to wet, nonplastic to low plas- ticity		8	SS	80	100 mm									
	ML (Glacial Till)		9	SS	80	80 mm									
146.8	Very Dense Reddish Brown		10	SS	135	110 mm									
13.5	Mudstone Bedrock (Queenston Shale) Possibly weathered		11	NQ											
	Sound Reddish brown with occasional gray bands, medium strong faintly weathered, thinly bedded		12	RC											
	14.07-14.14: Breccia- ated			NQ											
	14.91: Brecciated			RC											
	15.67-15.79: Frag- mented			RC											
143.4															
16.9	End of borehole														
	Bedrock surface inferred by drill action.														
	Groundwater elevation not established. Surface water flowing over borehole collar.														

\*3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 103

METRIC

W P 408-85-01 LOCATION Sta. 26 + 919 @ Hwy. 403  
Co-ords. 4 808 956 N; 279 470 E  
DIST 4 HWY 403 BOREHOLE TYPE Solid Stem Auger, NQ Rock Core  
DATUM Geodetic DATE 90 11 19 to 21  
ORIGINATED BY REC  
COMPILED BY REC  
CHECKED BY TJB

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	
160.4	Ground Level							W <sub>p</sub>	W	W <sub>L</sub>	GR SA SI CL
0.0	Heterogeneous mixture of clayey silt and sand some gravel, dry, low plasticity		1	SS	57/	100 mm					
	CL (Glacial Till)		2	SS	65/	130 mm					
	Hard Brown becoming reddish brown		3	SS	53/	150 mm					
			4	SS	50/	100 mm					
			5	SS	92						
151.1	Silt and fine sand, horizontal layers, trace to some gravel		6	SS	105/	280 mm					
9.3	Very Dense Reddish Brown										
150.4	Silt and sand, some gravel, trace clay, moist to wet, nonplas- tic to low plasticity		7	SS	100/	230 mm					9 38 48 5
10.0	(Glacial Till)		8	SS	75/	100 mm					
	Very Dense Reddish Brown		9	NQ	REC 102						RQD = 0%
147.0	Mudstone Bedrock (Queenston Shale) Possibly weathered			NQ							
13.4	Sound Reddish brown with occasional gray bands, medium strong, faintly weathered, thinly bedded		10	RC	REC 100%						RQD = 83%
	14.39-14.44: Clay seam		11	RC	REC 100%						RQD = 69%
	14.60-14.61: Clay seam			NQ							
	14.67-14.70: Brecci- ated		12	RC	REC 100%						RQD = 63%
142.4											
18.0	End of borehole										
	Bedrock surface inferred from drill action. Groundwater elevation 159.9 on January 9, 1991.										

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 104

METRIC

W P 408-85-01 LOCATION Sta. 27 + 000 @ Hwy. 403  
Co-ords. 4 809 018 N; 279 522 E  
DIST 4 HWY 403 BOREHOLE TYPE Solid Stem Auger  
DATUM Geodetic DATE 90 11 20  
ORIGINATED BY REC  
COMPILED BY REC  
CHECKED BY TJB

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	10 20 30					
163.1	Ground Level													
0.0	<u>Topsail*</u> Heterogeneous mixture of clayey silt and sand some gravel, dry, low plasticity		1	SS	42		162							
	CL (Glacial Till)		2	SS	50	100 mm	160							
	Hard Brown becoming Reddish Brown		3	SS	50	150 mm	158							
			4	SS	77	150 mm	156							
			5	SS	50	100 mm	154							
	Silt and sand, some gravel, trace clay (Glacial Till) ML		6	SS	94	150 mm	152							
			7	SS	74	60 mm								
	Silt, light brown		8	SS	100	90 mm								
						Piezometer Tip	150							
149.3			9	SS	90	60 mm								
13.8	End of borehole													
	*Ground surface is cultivated to 0.4 m depth. Presently growing forage crops.													
	Groundwater elevation 162.9 on December 19, 1991, pipe frozen at 163.1 on January 9, 1991.													

\*3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10



# RECORD OF BOREHOLE No 105

METRIC

W P 408-85-01

LOCATION

Sta. 27 + 100 @ Hwy. 403  
Co-ords. 4 809 095 N; 279 587 E

ORIGINATED BY REC

DIST 4 HWY 403

BOREHOLE TYPE Solid Stem Auger

COMPILED BY REC

DATUM Geodetic

DATE 90 11 20

CHECKED BY TJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
163.5	Ground Level													
0.0	Topsoil*													
	Heterogeneous mixture of clayey silt and sand, some gravel, dry, low plasticity		1	SS	50		162							
	CL (Glacial Till)		2	SS	26	140 mm	160							
	Hard Brown becoming reddish brown		3	SS	70	120 mm	158							
			4	SS	84	150 mm								
			5	SS	85	120 mm	156							
155.0														
8.5	Silty sand to sand and silt, some gravel, trace clay		6	SS	80	30 mm	154							
	(Glacial Till)													
152.7	Very Dense Reddish Brown		7	SS	74	150 mm								
10.8	End of borehole													
	*Ground surface is cultivated to 0.4 m depth. Presently growing forage crops.													
	Borehole dry on com- pletion of drilling. Groundwater level not established.													

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 106

METRIC

W P 408-85-01 LOCATION Sta. 27 + 189 o/s 1.7 Lt. 0 Hwy. 403  
Co-ords. 4 809 164 N; 279 643 E  
DIST 4 HWY 403 BOREHOLE TYPE Solid Stem Auger  
DATUM Geodetic DATE 90 11 20  
ORIGINATED BY REC  
COMPILED BY REC  
CHECKED BY TJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					
163.4	Ground Level													
0.0	Topsoil													
	Heterogeneous mixture of clayey silt and sand, some gravel, dry, low plasticity (Glacial Till)		1	SS	70		162							
160.7	Hard Brown													
2.7	Silt, uniform except for one clayey silt layer, moist, non- plastic		2	SS	48		160							
159.4	Dense Brown													
4.0	Sandy silt, trace to some gravel, trace clay, dry to moist, nonplastic to low plas- ticity (Glacial Till)		3	SS	54/-	150 mm								5 26 60 9
			4	SS	54/-	120 mm	158							
155.7	Very Dense Brown		5	SS	80/-	120 mm	156							
7.7	End of borehole													
	Borehole dry on com- pletion of drilling. Groundwater level not established.													

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 107

METRIC

W P 408-85-01 LOCATION Sta. 26 + 883 @ Hwy. 403 Co-ords. 4 808 928 N; 279 447 E ORIGINATED BY REC  
DIST 4 HWY 403 BOREHOLE TYPE Solid Stem Auger COMPILED BY REC  
DATUM Geodetic DATE 90 11 21 CHECKED BY TJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
164.6	Ground Level																
0.0	Heterogeneous mixture of clayey silt and sand some gravel, dry, low plasticity						164										
	CL (Glacial Till)		1	SS	38												
							162										
			2	SS	40												
	Hard Brown becoming reddish brown		3	SS	75/-	120 mm	160										
	Several fine sand partings		4	SS	54/-	150 mm											
							158										
			5	SS	50/-	60 mm											
							156										
			6	SS	53/-	90 mm											
			7	SS	76/-	150 mm	154										
	Silt, light brown		8	SS	90/-	120 mm	152										
			9	SS	100/-	120 mm											
							150										
149.6																	
15.0	Silt and sand, some gravel, trace clay, dry, nonplastic to low plasticity (Glacial Till)		10	SS	100/-	60 mm											
147.6	Very Dense Reddish Brown		11	SS	49/-	90 mm	148										
17.0	End of borehole																
	Borehole dry on com- pletion of drilling. Groundwater level not established.																

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to  
Sensitivity

20  
15 ÷ 5 (%) STRAIN AT FAILURE  
10

164.6  
146.6  
118.0

C.N.R. SUBWAY AT HWY 403  
W.P. : 408-85-01 & 02      SITE : 10-478  
HWY. 403, DISTRICT 4



LOOKING NORTH



LOOKING SOUTH POLE AT 10+000 CHAINAGE



File No. 42.80 HALTON Drawing Number AA 899-42.80-1.1