

Series I



ACRES

**Ministry of Transportation
Province of Ontario**

**Foundation Investigation for Bridge Structure
Proposed Highway 416 and CNR Subway
District No. 9, Ottawa
WP 126-87-01, Site 3-544**

GEOCREP # 3165-173

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Niagara Falls, Ontario**

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1 Introduction

Acres International Limited (AIL) was retained by the Ministry of Transportation of Ontario (MTO) to undertake a foundation investigation for a proposed CNR bridge structure over Highway 416, District No. 9, Ottawa, WP 126-87-01, Site 3-544. The work was authorized by Agreement 4238-9089-238 dated August 28, 1989.

The location, site plan and section of the proposed bridge are shown on MTO Plan E-52-416-

Drilling and sampling operations were performed by Marathon Drilling Co. Ltd., under the full-time supervision and direction of an Acres geotechnical engineer. Fieldwork commenced on November 2, 1989 and was completed on November 11, 1989. A plan of the site, showing the borehole locations, together with stratigraphic profiles are shown on Drawings No. 1268701-A and 1268701-B.

All soil samples and bedrock cores were returned to Acres geotechnical laboratory in Niagara Falls for detailed examination, logging and testing.

The results of the field and laboratory investigations are presented in this report, together with an interpretation of the data obtained and recommendations concerning the geotechnical aspects of the design and construction of the proposed bridge and associated works.

2 Exploratory Work

The exploratory work consisted of a total of 10 boreholes (BH-101 to BH-110 inclusive) drilled to depths ranging from 11.9 m to 16.3 m in both overburden and bedrock. Initially, the MTO staked the borehole locations and determined their ground surface elevations based on coordinates submitted by Acres. Three of the boreholes (BH-102, BH-107 and BH-109 inclusive) were located in the vicinity of the centerline of the west and east piers of the proposed CNR bridge. BH-101, 105 and 106, inclusive, were located in the vicinity of the west abutment, and BH-103, 109 and 110, inclusive, in the east abutment. BH-104 was located south of the proposed bridge, and was intended to determine the subsurface conditions for the southern part of the project site.

These borehole locations were subsequently modified slightly in the field to allow access and/or clearance from existing structures, etc. The final borehole elevations and their coordinates, as shown on Drawing No. 1268701-A, were measured and tied into the initially proposed locations, and are considered to be accurate to within 0.1 m.

Drilling was performed using a CME-55 auger drill rig mounted on an all-terrain tracked vehicle. Hollow stem augers were used to advance the drilling in the overburden, and diamond core drilling with a BX core barrel in the bedrock. A total of 116.4 m of overburden and 20.0 m of bedrock were drilled in the 10 boreholes. A summary for the physical data for each borehole is provided in Table 1.

Table 1

Summary of Borehole Physical Data

Borehole Number	Ground Surface Elevation (m)	Coordinates	Overburden/Bedrock Contact		Bottom of Borehole	
			Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
101	86.6	E 358 878.3 N 5021 267.4	11.3	75.3	14.3	72.3
102	86.4	E 358 914.9 N 5021 272.9	12.2	74.2	14.9	71.5
103	86.4	E 358 994.0 N 5021 283.0	13.8	72.6	16.3	70.1
104	86.2	E 358 912.0 N 5021 251.0	12.0	74.2	13.6	72.6
105	86.3	E 358 840.0 N 5021 286.4	10.3	76.0	11.9	74.4
106	85.5	E 358 871.9 N 5021 291.1	10.0	75.5	12.2	73.3
107	85.1	E 358 904.4 N 5021 296.9	10.6	74.5	12.1	73.0
108	85.0	E 358 935.3 N 5021 301.1	11.5	73.5	13.9	71.1
109	84.5	E 358 964.3 N 5021 305.8	11.9	72.6	14.4	70.1
110	85.7	E 358 990.4 N 5021 316.4	12.8	72.9	12.8	72.9

With the exception of BH-104, 106 and 110, attempts were made to sample the overburden materials at intervals between 1.2 to 1.5 m, using a 51-mm OD split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure or using a 73-mm OD thin-walled tube sampler (Shelby) as appropriate to the soil type. In these boreholes, field vane shear tests were performed using an Acker field vane tester on the undisturbed cohesive silty clay materials between each consecutive thin-walled tube sample. In BH-104, 106 and 110, almost continuous sampling was performed in the cohesive silty clay materials using a 73-mm OD thin-walled tube piston sampler.

A total of 16 piezometers were installed in the 10 boreholes. At least one piezometer was installed in each borehole. Two piezometers were installed in BH-101, 103, 105, 106, 109 and 110; one at a depth between 3.4 and 7.1 m and another at a depth between 8.5 and 12.3 m. In BH-102, 107 and 108, piezometers were installed in the bedrock.

All soil and rock samples were returned to Acres geotechnical laboratory for more detailed logging and testing. The laboratory testing included natural moisture content determinations, Atterberg limits, grain size analyses including hydrometer tests, unconsolidated-undrained triaxial tests, consolidated-undrained triaxial tests with pore pressure measurements, and unconfined compressive strength tests on rock core specimens.

The results of these tests, together with the details of drilling and sampling, are summarized in the Record of Borehole sheets following the report text.

3 Site Conditions

3.1 General Description

The crossing of the CNR tracks and the proposed Highway 416 is located in the vicinity of Bells Corners, in the city of Nepean, south of the existing Highway 417. A key plan of the area is shown on Drawing No. 1268701-A.

The site is located between Cedarview Road to the west, a vacant lot belonging to the National Capital Commission (NCC) to the north, and a green belt area to the east and south. Existing CNR railtracks run approximately east-west. The natural topography of the site is generally flat up to the NCC lot, and then gently sloping towards the north near Baseline Road. The existing railtracks are constructed on a single embankment, with a ditch on each side. The existing ground surface is generally grass covered, and a few small bushes exist on the site. A CN service road runs parallel to and to the south of the railtracks. BH-101, 102 and 103 were located at the northern edge of the service road and south of the railtracks. BH-104 is located south of the service road. BH-105 is located east of Cedarview Road, on top of the railway embankment. BH-106, 107, 108 and 109 were all located at approximately the bottom of the north side ditch. The majority of the ditch, except for the eastern part, was dry. It appeared that water runs from the east end of the ditch and turns to the north at a ditch junction located just east of BH-109. BH-110 was located just outside the CNR right-of-way boundary and above the north side ditch.

3.2 Soil Conditions

3.2.1 Topsoil and Possible Fill Deposits

Dark brown silty sand with some gravel was encountered in BH-101 to 104, beneath a veneer of grass-covered topsoil. The thickness of this material varies from 0.6 to 1.3 m and it exists in a loose condition. In BH-101, 102 and 103, this material may be a fill material because of its proximity to the edge of the service road located south of the railtrack embankment.

In BH-106 to 110 inclusive, dark gray to dark brown silty clay with some gravel extends to depths between 0.1 and 0.4 m, and is considered as part of the topsoil layer. In BH-105, 1.9 m of similar material was encountered with some black mottling and oxidation staining. The consistency of this silty clay material ranges from soft to stiff.

3.2.2 Silty Clay (Marine Deposit)

A gray silty clay deposit was encountered beneath the surficial silty sand to silty clay with some gravel. The thickness of this deposit varies from west to east, with a minimum thickness being located in the center and western sides of the area under investigation, i.e., around BH-101, 105 and 106. In this area, its thickness ranges from 6.1 to 6.9 m. At the east end of the site, i.e., in the vicinity of BH-103, 109 and 110, the thickness ranges from 10 to 12.8 m. The silty clay deposit contains occasional horizontal to subhorizontal layers or lenses of silty fine sand to silt, the thickness of which vary from 2 to 10 mm. Continuous samples obtained from BH-104, 106 and 107 revealed that these layers and lenses generally exist between el 80 to 83 m. On the eastern side of the site where the deposit is thicker, silty sand and silt layers are also concentrated between el 76.5 and 78.1 m, and below el 75.8 m. In the zones where the thin layers or lenses of silty sand and silt exist, the

deposit also contains occasional pieces of round fine gravel. Pockets of silty clay or silt were also encountered in these zones.

The average natural moisture content of the marine deposit based on over 70 sample tests is 44%, with a range between 16 and 61%. The natural unit weight ranges from 16.0 to 17.8 kN/m³, with an average of 16.9 kN/m³. The index properties as determined by Atterberg limits tests indicate Liquid Limit ranges from 49 to 68% and Plastic Limit ranges from 18 to 32%. The average Liquid Limit, Plastic Limit and Plasticity Index are 57, 22 and 35% respectively indicating a material which is highly plastic.

The measured undrained shear strength of this material ranges from 14 to 113 kPa, as determined by field vane tests. A plot of elevation versus undrained shear strength indicates a desiccated crust down to approximately el 82.0 m. The laboratory unconsolidated-undrained triaxial tests provided an undrained shear strength range from 26 to 88 kPa. Figure 1 shows a plot of elevation versus undrained shear strength for this deposit. The sensitivity of the marine deposit, as measured by the field vane, ranges from 2 to 15, with an average of approximately 8, thereby classifying the clay as sensitive to extra sensitive.

Three sets of consolidated-undrained triaxial tests with pore pressure measurements were performed on piston samples taken from BH-104, 106 and 110. The Mohr circles from the tests are plotted in Figure 2, along with a failure envelope. The characteristics of this deposit are rather typical of Champlain clays reported in the literature (e.g., Crawford (1963), Mitchell (1970), Lo (1972)), and are similar to marine clay encountered by Acres at Arnprior, 50 km west of Ottawa (Peggs (1982)). The presence of cementation bonds is reflected in the nonlinear failure envelope shown, especially in the lower stress ranges. Depending on the interpretation used, failure envelopes indicating an effective angle of shearing resistance ranging between 23° and 28° can be drawn together with cohesion intercepts which vary from approximately 28 to 35 kPa.

3.2.3 Sand and Silt with Some Gravel and Some Clay (Till)

A till deposit, consisting of sand and silt with some gravel and some clay and occasional cobbles, was encountered in all boreholes between the marine clay deposit and the bedrock. The thickness of this deposit varies across the site.

The maximum till thickness is located near the center of the area, i.e., around BH-102 and 107, where it varies from 4.5 to 5.2 m. The investigation also reveals that the thickness of the material decreases towards the west and east. In BH-103, 109 and 110, the thickness varies from 0.9 to 1.9 m. At the western side, the thickness is approximately 1.8 m.

The 'N' value of this material, as determined by the SPT, ranges from 1 to 29, with an average of 5, indicating a relative density from very loose to compact, but generally loose.

The natural moisture content of the till ranges from 11 to 33%, with an average of 18%. During the drilling and sampling, the obtained samples were generally wet,

and caving occurred during the retraction of the auger casings. Two samples were tested for their plasticity, and the results indicated that this till material is either nonplastic or has very low plasticity. Results from the grain size analyses exhibit a well-graded material, a typical gradation for till. The clay size content ranges between 7 and 10%. The density of this material, as tested by the SPTs, is relatively low in comparison to typical glacial till deposits. On the basis of its low density, it is expected that this material is a waterlain till, as described in some research papers (McGown et al, 1977, and Dreimanis, 1977).

The existence of this material has also been reported in the Ottawa area by other researchers (Adams, 1960), and the term 'soft till' has been used in their discussion.

The presence of gravel-size particles and possible cobbles made undisturbed samples difficult to obtain. Attempts were made to use thin-walled Shelby tubes to sample this material, but the tip of the tube was always damaged. The paper by Adams (1960) suggested that the unit weight of this material is in the order of 20.4 kN/m^3 .

3.2.4 Summary of Soil Classification Tests

The results of laboratory tests to determine the natural moisture content, the unit weight and the index properties of the soil deposits described in this section are summarized in Table 2. The grain size distribution curves for possible fill materials, marine silty clay and the till are presented in Figures 3, 4 and 5 respectively. Figure 6 contains the plasticity chart, showing Liquid Limits and Plasticity Indexes of the marine silty clay samples.

Table 2

Soil Properties

	Number of Test	Minimum	Maximum	Average
(a) Topsoil and Possible Fill Deposit				
Natural Moisture Content (%)	4	6	30	17
(b) Silty Clay (Marine Deposit)				
Natural Moisture Content (%)	78	16	61	44
Liquid Limit (%)	9	49	68	57
Plastic Limit (%)	9	18	32	22
Unit Weight (kN/m^3)	17	16.0	17.8	16.9
(c) Sand and Silt Till				
Natural Moisture Content (%)	14	11	33	17
Liquid Limit (%)	2	14	15	14
Plastic Limit (%)	2	12	15	13

3.3 Bedrock Conditions

The bedrock consists of light to medium gray dolomitic limestone with frequent dolomite beddings. The rock is part of the Oxford Formation, deposited during the lower Ordovician period. The dolomite has a fine crystalline texture, but often grades into a medium to coarse sandy clastic dolomite with some sections showing elongated and subrounded intraclasts up to 1.5 cm in length. Occasional shaly partings are present in all boreholes. One 100-mm section of dark gray microcrystalline dolomite was encountered in boreholes BH-101 and BH-102, approximately 2.5 m from top of rock. Up to 1-cm diameter vugs filled with pinkish-white calcite are present in some boreholes.

Bedding in all holes is subhorizontal, and except for the shaly partings, is gradational. Spacing between the shaly partings averages approximately 0.3 to 0.4 m for all holes. In general, the partings are smooth, undulating and intact. Prominently closely spaced partings are present in both BH-101 and BH-102 at approximately 0.5 m from top of rock.

Subvertical jointing was encountered in BH-104 and BH-108. The joints are rough, irregular and show development of calcite crystals and minor mineralization.

In general, the rock is very strong and fresh. Core recoveries ranged from 94 to 100% with an average of 99.5%. RQD values ranged from 40 to 100% with an average of 83%. The average unit weight, based on three core samples, is 27.2 kN/m³.

Unconfined compressive strengths obtained from three selected rock core samples ranged from 216.3 and 245.2 MPa, and averaged 230.8 MPa.

3.4 Groundwater Conditions

Groundwater level observations from the 16 piezometers, including their detail installations, are summarized in Table 3. The last readings were taken on December 28, 1989, approximately 6 weeks after the completion of the field drilling program.

The water level in the piezometers installed within the marine clay deposit ranges from el 82.0 to 84.8 m, with an average at el 83.6 m. Generally, the lower part of the deposit exhibits a lower piezometric level than the upper part.

The water level in the piezometers installed within the till deposit is generally the same or slightly lower than the level in the marine clay. It ranges from el 81.8 to 84.8 m, with an average at el 83.1 m.

The piezometric level in the bedrock was encountered approximately 3.5 m below the level in the marine clay. The average level based on three piezometric readings is at el 79.9 m.

Within the same elevation zone, the piezometric levels are generally flat. Groundwater observations as indicated by piezometric readings provided above show that there is no potential artesian condition at the area investigated.

Table 3

**Summary of Piezometer Installation
Details and Observations**

Borehole	Ground Surface Elevation (m)	Elevation of Tip of Piezometer (m)	Elevation of Bentonite Seal		Date of Installation	Water Elevation on Dec 28/89 (m)	Remarks
			Top (m)	Bottom (m)			
101U	86.6	81.7	83.1	81.6	Nov 2/89	84.8	Marine clay
103U	86.4	79.4	80.1	78.7	Nov 4/89	84.2	Marine clay
103L	86.4	74.4	75.1	74.0	Nov 4/89	82.2	Marine clay
105U	86.3	82.3	82.7	81.4	Nov 7/89	83.3	Marine clay
106U	85.5	82.1	82.7	81.6	Nov 8/89	84.4	Marine clay
109U	84.5	80.2	81.0	80.0	Nov 10/89	83.8	Marine clay
109L	84.5	74.8	75.5	74.6	Nov 10/89	82.0	Marine clay
110U	85.7	80.2	80.7	79.8	Nov 11/89	84.2	Marine clay
101L	86.6	76.3	76.8	76.0	Nov 3/89	84.8	Till
104	86.2	75.7	76.1	75.4	Nov 6/89	82.5	Till
105L	86.3	77.0	77.8	76.5	Nov 7/89	83.1	Till
106L	85.5	77.0	77.5	76.6	Nov 7/89	83.4	Till
110L	85.7	73.5	74.0	-	Nov 11/89	81.8	Till
102	86.4	71.9	72.7	-	Nov 3/89	79.3	Bedrock
107	85.1	73.6	74.2	-	Nov 8/89	80.4	Bedrock
108	85.0	71.5	72.3	-	Nov 9/89	80.0	Bedrock

Legend: U - upper piezometer
L - lower piezometer

4 Geotechnical Design and Construction Considerations

4.1 General

In summary, the subsurface conditions in the vicinity of the proposed Highway 416 and CNR subway consist of a surficial deposit of sensitive marine clay with a thickness varying from 6 to 13 m. This deposit is generally composed of a desiccated crust about 3 m thick, of firm to stiff consistency, underlain by softer materials below approximately el 82.0 m. The marine clay is underlain by a loose, sand and silt till with a thickness varying from approximately 1 to 5 m. This till was deposited on the limestone-dolomite bedrock which gently slopes down from west to east. The bedrock is fresh, very strong, has frequent subhorizontal bedding planes and is generally of good to excellent quality. Groundwater level measurements, as obtained from piezometers installed in the marine clay and till, were within the marine clay deposit. The piezometric level in the bedrock is substantially lower, indicating a downward flow of groundwater.

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

- relocating the existing CNR tracks around the north side of the site at such a distance from the proposed bridge to permit the bridge to be constructed
- constructing the new railway bridge
- carrying out the excavation required to pass the proposed Highway 416 and the relocated Cedarview Road under the railway bridge. This excavation will extend from grade at some distance north of the bridge, reach its maximum depth under the bridge and return to grade to the south of the bridge. Excavation of the approach cuts is generally beyond the scope of this investigation.

The major geotechnical design and construction considerations for the above works relate to the excavation or penetration of the sensitive to extra sensitive marine clay. Cuts in these deposits to the depth required for this project are not common. With regard to the bridge foundations, all structure loads must be carried down to the bedrock. Whether this is done by constructing the foundations in open excavations to bedrock or by extending them down by driven piles or caisson-type piles, the properties of the clay will have a significant influence on the design and construction procedures to be selected.

The structure layout will also be controlled to a major extent by the configuration of stable slopes in the marine clay. These slopes will also dictate the distance that the railway detour must be located away from the bridge centerline.

Because of the relatively weak and sensitive nature of the marine clay, it will be necessary to adopt certain precautions in carrying out the excavation work.

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

4.2 CNR Bridge Structure

It is assumed that the CNR bridge structure consists of a 3-span bridge with two intermediate pier locations and west and east abutments, as indicated on Drawing No. 1268701-A.

4.2.1 Bridge Piers

BH-102 and 107, which are located close to the west pier, encountered bedrock at el 74.2 and 74.5 m, respectively. BH-108, which is located north of the east pier, encountered bedrock at el 73.5 m. The bedrock elevations at the actual pier locations should be relatively close to these elevations.

It is assumed that excavation in the area for construction of the roadway will extend down to approximately el 76 m, which is within the loose till deposit. Since the deposit is not suitable for the support of such a structure, it is recommended that the bridge piers be founded on the bedrock. For the design of the footings on bedrock, a factored bearing capacity of 3000 kPa at 'Ultimate Limit State' is recommended. The bearing capacity at 'Serviceability Limit State' will not govern since the bedrock is assumed to be an unyielding foundation base.

4.2.2 Bridge Abutments

The west abutment will be located in the vicinity of BH-101 and 106, where the elevation of the bedrock is at 75.3 and 75.5 m respectively. The east abutment will be in the vicinity of BH-103 and 109, where the bedrock surface was measured at el 72.6 and 72.7 m respectively.

As recommended for the bridge piers, the abutment loads should be carried down to the bedrock. If this achieved by constructing the abutment concrete directly on the bedrock surface, a factored bearing capacity of 3000 kPa is recommended at the 'Ultimate Limit State'. The bearing capacity at the 'Serviceability Limit State' will not govern since the bedrock is assumed to be an unyielding foundation base.

Alternatively, consideration could be given to founding the abutment concrete at a higher level and transferring the loads down to bedrock using driven piles or caisson-type piles. However, the installation of such foundation elements raises some problems regarding the disturbance of the marine clay due to their installation methods. Of the various driven pile options available, 'H' piles probably cause the least disturbance. The clay disturbance results in a significant loss of strength which could, in turn, affect the stability and configuration of the excavation slopes. This may require special measures to be taken in the abutment areas to ensure the stability of slopes. The loss of strength may mean that the piles should be designed as long columns rather than short, as is normally the situation.

There are various options which could be studied in more detail if piling appears to be an attractive solution to the abutment foundations. One would be the driving of the piles from the existing ground level and waiting for the remolded soil around the piles to regain much of its strength loss before undertaking the excavation in the vicinity of the abutments. Other areas of the excavation could proceed in the meantime. When the excavation gets down to the concrete abutment founding level in the abutment areas, the piles would be cut off, a granular mat placed and the

abutment built as part of this investigation, the rate of strength regain has not been studied. This aspect should probably be reviewed as part of the design.

A similar clay remolding problem would exist with the installation of caisson-type piles. With this option, it is recommended that they be built by leaving the steel casings in the ground to avoid the potential problem of soil intrusion into the concrete column on casing withdrawal.

Typically, 310 x 110 'H' piles can support loads of 1600 kN at 'Ultimate Limit State' and 1150 kN at the 'Serviceability Limit State'. To prevent damage of the pile tips on contacting bedrock, it is recommended that the tips be reinforced. If batter piles are to be used, a type of pile shoe similar to the 'APF Hardbite', which can chisel into the bedrock, is recommended to avoid skidding along the bedrock surface. Such shoes should be used on piles driven on any batter flatter than 6V:1H.

Caisson-type piles could be designed as end-bearing columns using a factored bearing pressure of 3000 kPa at 'Ultimate Limit State'.

Concrete abutments founded directly on the bedrock surface could be constructed within sheeted and braced excavations and caissons. The walls of these caissons could consist of steel sheet piles or steel 'H' piles and timber lagging. Such structures should be designed to resist soil pressures as shown in Figure 8. Because of the disturbance and remolding of the sensitive marine clay during pile driving, the strength assumed in determining the soil pressure on the caisson has been assumed to be the remolded value using a minimum average undrained shear strength of 21.5 kPa and an average sensitivity of approximately 8. Further details regarding the undrained shear strength are presented in Section 4.3.2.

To avoid a base heave of the soil within the caisson, all sheeting should extend to bedrock.

Further details regarding the design of such caissons can be provided at a time when the construction concepts are better defined.

4.2.3 Sliding Stability

Both the west and east abutments will experience some horizontal loadings. Where the abutments are constructed directly on the bedrock surface, it is recommended that a sliding friction angle of 30° be assumed for the contact between the bedrock and concrete, and also along bedding planes within the bedrock. Even though shale partings are present, they are intact and undulate significantly. This angle represents the ultimate sliding resistance and must, therefore, be factored to provide an adequate margin of safety. If the value given above cannot adequately provide the sliding resistance, installation of corrosion protected rock bolts may be required to increase the sliding stability between bedrock and the concrete. It is also important that the exposed bedrock surface be clean from water, clay debris or clay layers from the excavation work prior to concreting of the abutment footings.

4.3 Stability of Cut Slopes

4.3.1 General

As noted in Section 4.1, the stability of the cut slopes in the sensitive marine clay is the major geotechnical design and construction problem on this site. Some of the significant factors involved in the resolution of this matter are soil strength, effect of soil disturbance, excavation techniques, groundwater conditions, depth of cut and special loading conditions, such as the detoured railway tracks and earthquakes. These factors are discussed below.

The critical stability section is along the eastern side of the site, i.e., in the east abutment area where the marine clay deposit is thickest, approximately 13 m. Most of the numerical stability analyses were performed on the configuration of conditions in this area.

The stability analyses were carried out using the PCSTABL5 computer program based on the Modified Bishop Method.

Two types of analyses have been undertaken; 'total stress' to assess the short-term temporary construction conditions and 'effective stress' to represent the long-term stability. These analyses are described below.

4.3.2 Total Stress Analyses

Total stress analyses were undertaken to assess the steepest slope that would have adequate stability for temporary short-term conditions. The marine clay strength used in these analyses was the undrained shear strength obtained from both field vane shear testing and laboratory unconsolidated undrained triaxial compression tests. A correction factor, which depends on the plasticity index of the clay, was used to adjust the field vane strengths for comparison with the values determined by the laboratory triaxial tests (Chandler, 1988).

Table 4 provides a summary of the calculations of the design undrained strength parameters for the marine clay, together with the other parameters used in the analyses. For the clay, a factor of 0.6 was introduced to allow for anisotropy, strain rate, progressive failure effects as well as possible softening of the clay during the excavation process (Benson et al, 1975).

Using the above parameters, various slope arrangements were analyzed to obtain an overall factor of safety of 1.3. The configuration which yielded this value was a uniform slope of 3.5H:1V without any berms or granular fill zones. It was assumed that the groundwater level follows the exposed surface as the excavation proceeds.

Table 4

Design Parameters for Total Stress Analyses

Marine Clay

Elevation Range (m)	Undrained Shear Strength (kPa)			Minimum Value of (2) and (3)	Strength for Analyses (Correction Factor 0.6)	Unit Weight (kN/m ³)
	(1) Uncorrected Average Field Vane Results	(2) Corrected (Chandler, 1988) Avg PI=35	(3) Average U-U Triaxial Test Results			
Above 82.0	61	76	58	58	33.5	16.8
79.5 - 82.0	30	38	36	36	21.5	16.8
Below 79.5	29	36	64	36	21.5	16.8
Till						
c' = 0	$\phi' = 28^\circ$					20.4

4.3.3 Effective Stress Analyses

The long-term stability of the cut slopes was assessed on the basis of two sets of effective stress parameters for the marine clay in combination with various other loading assumptions described below.

It is the general belief in the soil mechanics community that the effective cohesion, c' , component of a lacustrine clay strength reduces with time and approaches zero in the long term. Marine clays, however, are somewhat different in that a small amount of cohesion or tensile strength remains as a result of the particle bonding.

The results of the consolidated undrained tests performed as part of this investigation are presented in terms of Mohr circles and envelope in Figure 2. The tests performed at stress levels comparable to those which will exist in the cut at the subject site define a nonlinear failure envelope. However, at higher stress levels, the more traditional failure envelope becomes apparent. The change in envelope has been explained as being the limit of the strength due to bonding and the point where the friction component becomes effective.

A similar type of envelope was obtained by Acres as a result of fairly extensive testing on a similar clay near Arnprior, Ontario, about 50 km from the site.

The effective angle of shearing resistance defined by the tests on the Ottawa site is approximately 23° with an effective cohesion intercept of approximately 30 kPa.

On the basis of these test results, it has been assumed that the lower bound long-term strength of the marine clay on the subject site would be defined by an effective angle of shearing resistance of 23° and an effective cohesion of zero. Considering

the conservative nature of this strength assumption, a lower-than-normal factor of safety against slope failure appears to be warranted with a value of 1.3 being the one used in considering various slope configurations.

A somewhat less conservative set of strength assumptions was also considered for unusual or dynamic loading conditions. The parameters selected were $\phi' = 23^\circ$ and $c' = 5$ kPa. The final configuration developed, together with the stratigraphy and parameters used, are shown in Figure 7. They represent the conditions which apply to the eastern half of the site where the marine clay is thickest. The excavation is defined by three segments cut at a slope of 3H:1V. The segments are separated by two 3-m wide berms at el 81 m and 83.5 m respectively. A granular fill zone, placed over the lower slope, has a surface slope of 3.5H:1V and a top width of 2 m at el 81 m. This zone acts as a weighting berm, prevents seepage from exiting the lower portion of the slope and provides protection against frost penetration into the marine clay in this critical zone of the slope.

It was assumed that the piezometric groundwater levels at the start of excavation would be the same as the average level indicated in Table 3 for each of the three strata. It will be necessary to install an effective drainage system in the slope as excavation proceeds and monitor the lowering in piezometers to ensure that the design assumptions are being met. This is discussed in more detail in a subsequent section of the report. The lowered phreatic surface shown in Figure 7 is the one assumed for long-term stability. The overall slope defined by this configuration is approximately 4H:1V which is very similar to the average slope of 3.5H:1V defined by the 'total stress' analyses.

Other factors taken into account in analyzing the slopes were the loading due to the detoured railroad and those resulting from earthquakes.

Since it is uncertain as to the duration that the railroad detour will be in operation, its effect was considered in the effective stress stability analyses. A loading equivalent to Cooper E85 on a single line track, the centerline of which is located a minimum of 13 m from the crest of the cut slope, was included. It was assumed that the railtracks will be built on a 2-m high granular embankment, 7 m wide at the base, and the impact loading will be equivalent to 55% of the railway live load resulting in a load intensity at the ground surface of 75 kPa.

The results of the stability analyses are shown in Figure 7 and tabulated below for the two sets of assumed strength parameters.

ϕ'	c'	Factor of Safety
23°	0 kPa	1.28
23°	5 kPa	1.58

These factors of safety are considered reasonable.

The effect of earthquake loading was also considered using a seismic coefficient equal to 0.1g in a pseudostatic analysis. Because of the unusual and dynamic nature of such loading, strength parameters of $\phi' = 23^\circ$ and $c' = 5$ kPa were used giving a factor of safety of 1.06.

This factor of safety is relatively low, but is considered to be adequate taking into account the relatively conservative nature of the strength parameters and the inherent conservatism of the pseudostatic analysis coupled with the shallow soil deposit.

On the basis of these analyses, it is recommended that the excavation outline shown in Figure 7 be used for the slopes along the eastern side of the excavation and around the north side to the centerline of the cut. Beyond this point, towards the west and along the west side, the upper berm could be deleted because of the less severe soil conditions.

The excavation slope around the south side of the site is discussed further in Section 4.3.4.

4.3.4 Construction

Because of the generally weak and sensitive nature of the marine clay, special attention must be paid to the excavation techniques used and the necessity to minimize disturbance of the clay particularly near the finished slopes.

The maximum depth of the cut in the marine clay is about 10 m which consists of about 3 m of desiccated crust overlying the weaker unweathered portion of the deposit.

Consideration should be given to undertaking the excavation work in two lifts. The upper one would involve the crust in which conventional equipment, such as dozers and scrapers, might be appropriate. While the depth of material to be excavated below the crust is within the capacity of the larger backhoes, the slope in front on the machine would be too steep to be stable. For this reason, it will probably be necessary to use a dragline for the lower lift.

If the first lift is taken too deep, it may become necessary to operate the equipment involved in the second lift from a granular mat placed over at least a part of the area to provide a suitable working surface.

It will be important to ensure that the cut slopes in the second lift are maintained relatively flat to prevent the triggering of a slide within the cut or near the finished slopes.

Final trimming of the slopes can probably be done with a small dozer. This work is preferably carried out after several weeks of dry weather during which the near surface clay has dried out and increased in strength. Rainfall or freezing and thawing will tend to reduce the strengths. It will be necessary to place some erosion protection on the cut slopes to prevent damage due to precipitation and runoff and establish a surface drainage system since the clay erodes rapidly if left exposed.

The excavation slope to the south of the bridge can be cut much flatter than required for stability since this material must be removed as part of the overall road construction. The contractor may wish to use this area to construct his access road down into the bridge foundation site.

4.3.5 Location of the CNR Detour

The CNR tracks must be detoured around the bridge construction site on a new embankment. The offset distance between the two lines must be sufficient to allow (a) adequate working space around the bridge foundations, (b) the construction of a stable slope, and (c) sufficient setback from the crest of the excavation to ensure slope stability. Based on the analyses undertaken, the distance from the toe of the excavation slope to the centerline of the relocated railway embankment should be a minimum of 52 m. If a working space of 15 m from the toe of the excavation slope to the centerline of the new bridge is considered adequate, the total offset of the detour centerline should be a minimum of 67 m.

It will be very important that once this distance is established that the toe of the slope is not cut away or the slope oversteepened.

4.4 Other Construction Considerations

4.4.1 Control of Groundwater Conditions

The existing piezometric data indicate that it will be necessary to lower the groundwater levels in the bedrock and the till to avoid boiling and 'blowing up' of the thin overburden layer which will be left over the bedrock as the excavation approaches final grade. Depressurization of the bedrock may also be required to lower the phreatic surface in the overburden slopes to improve their stability. A pumping test conducted prior to construction, using the existing piezometers set in the 10 boreholes drilled for this investigation, would provide useful information required for the design of the depressurizing system. The construction of an effective drainage system at the toe of each berm, together with the depressurization system and a detailed monitoring system, will be important aspects in the success of the excavation at the site.

The design of the depressurizing system is considered to be beyond the scope of this investigation.

The drainage system in each berm and at the toe of the slope is essential for the long-term stability of the slope. Typical details of such a system are shown in Figure 9.

To prevent freezing of the drains, they must be placed a minimum of 1.8 m below the berms. In excavating these drain trenches, it is essential that they be opened in short sections and backfilled with compacted materials before moving on to the next section. Failure to do this in short sections may result in a slope failure.

It is important that water flowing across the area be diverted away from the cut to avoid any flow over the crest and down the slope. This can be done by creating small mounds back from the slope crest to divert the water away from the cut. This is preferable to cutting swales or drainage ditches which may pond water near the crest of the slope and have an adverse effect on the slope stability.

4.4.2 Performance Monitoring

Cuts of the depth proposed for this project are not common anywhere in Canada. It is important therefore that the design assumptions be monitored to confirm satisfactory performance. Physical stability of the cuts may be verified with

- slope indicators
- shear strips
- surface pins.

The groundwater history and performance of the drainage system should also be monitored by piezometers and wiers to check drainage pipe flows. Typically, failures of marine clay slopes occur in the spring under conditions of high groundwater.



A handwritten signature in black ink, appearing to read "T. J. Bradshaw".

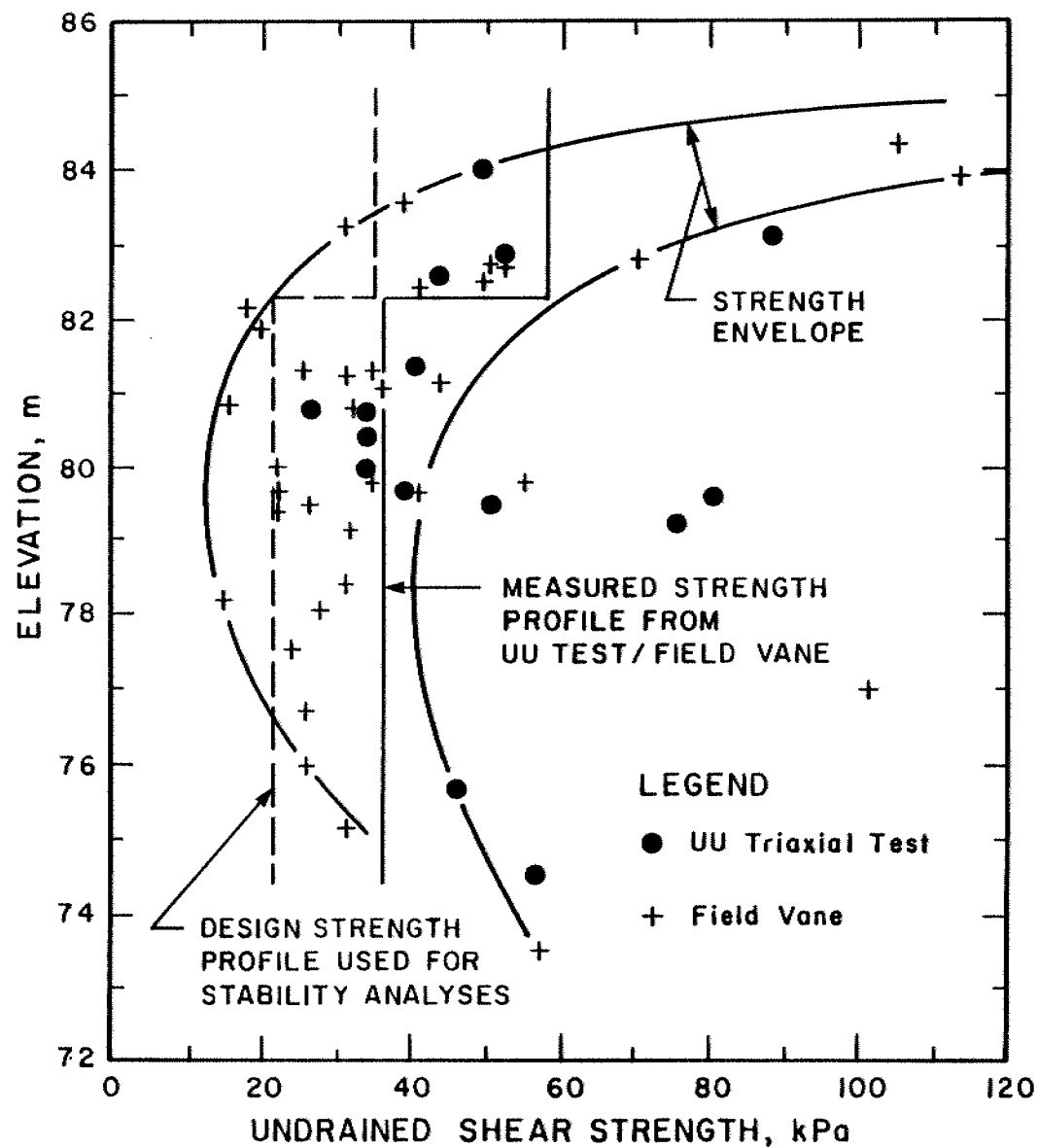
T. J. Bradshaw
Deputy Head, Geotechnical Department

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List of References

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Figures

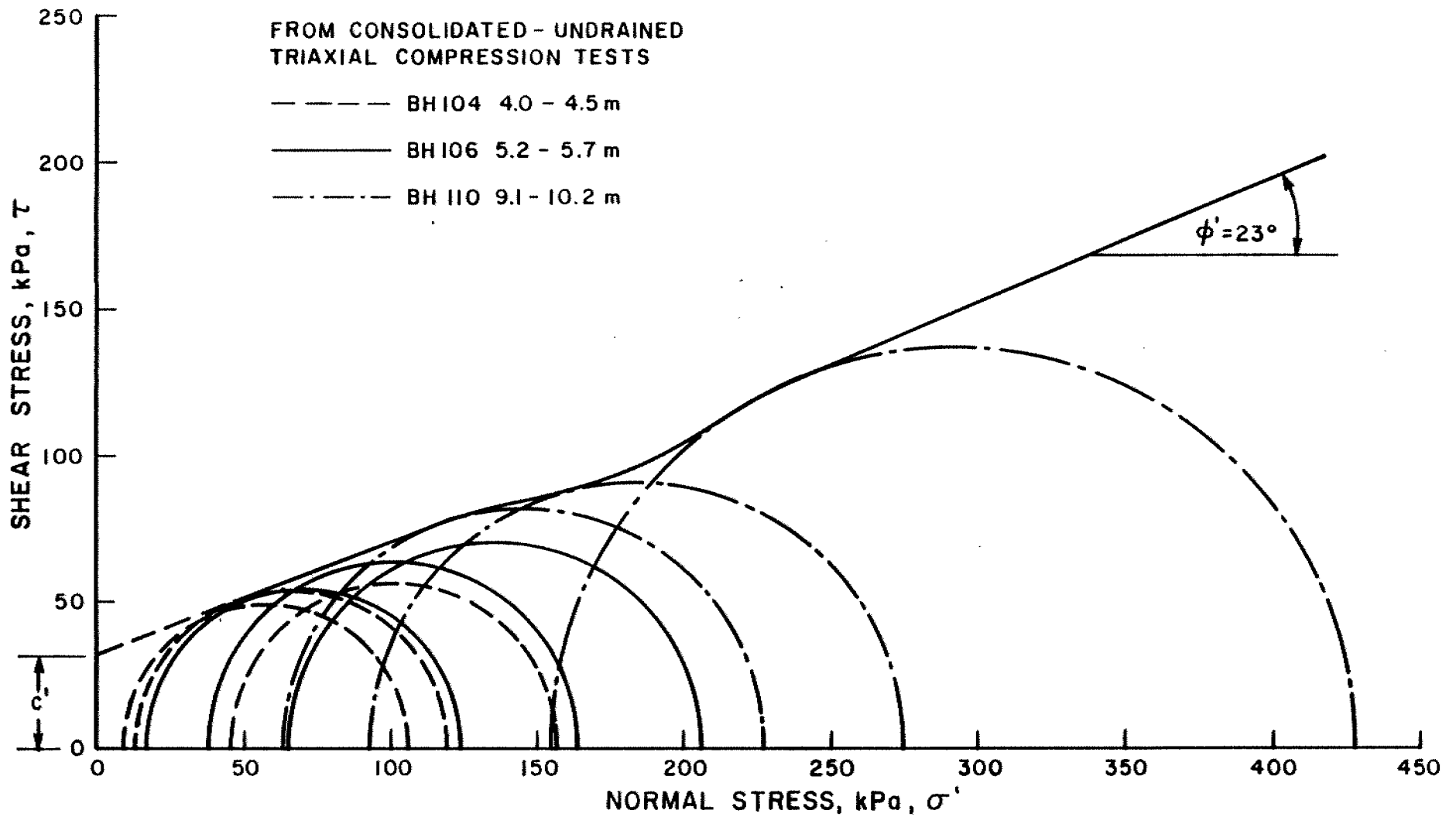


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UNDRAINED SHEAR STRENGTH PROFILE OF MARINE CLAY

FIG No 1

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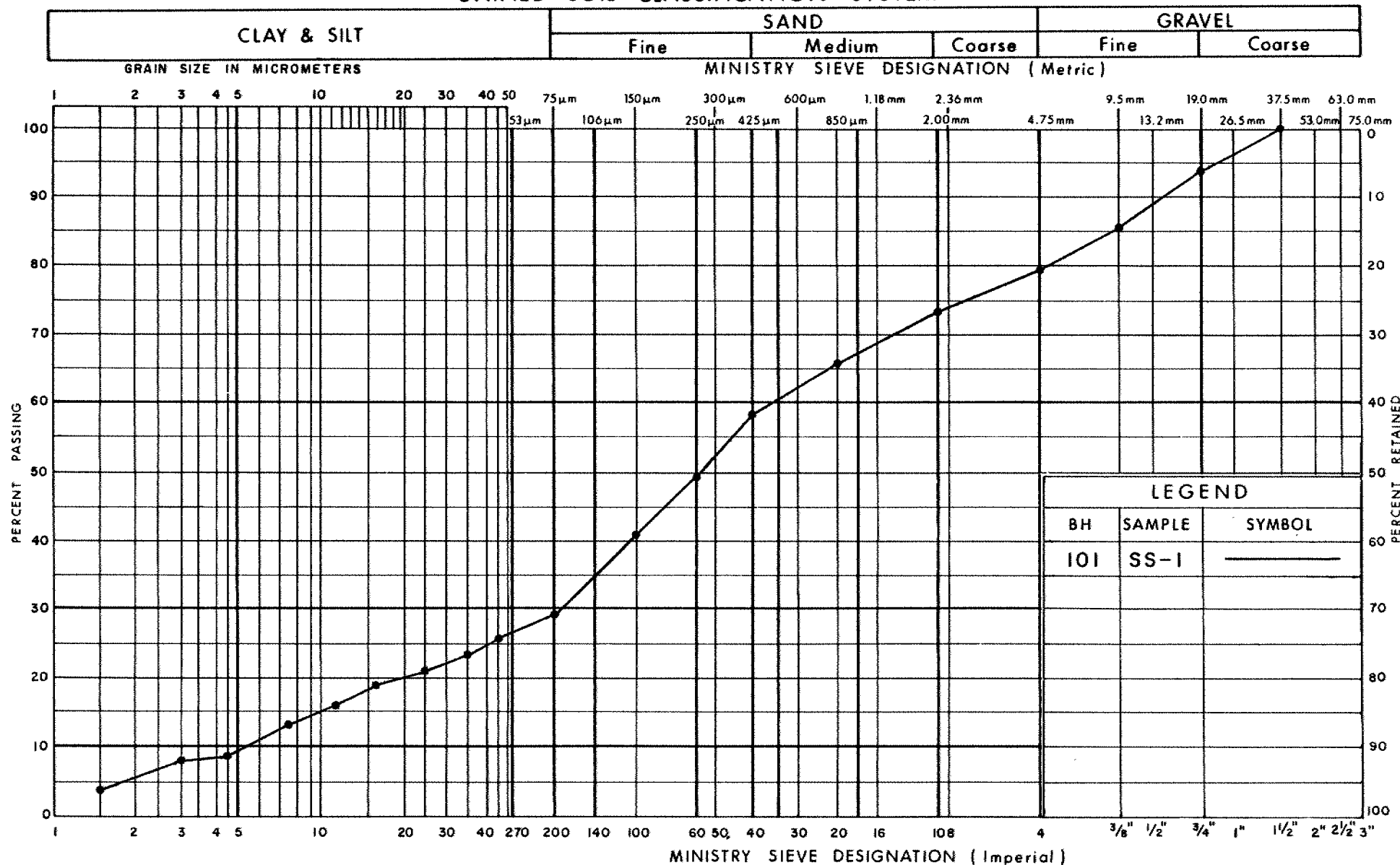
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MOHR CIRCLES AND ENVELOPE-MARINE CLAY

FIG No 2

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UNIFIED SOIL CLASSIFICATION SYSTEM



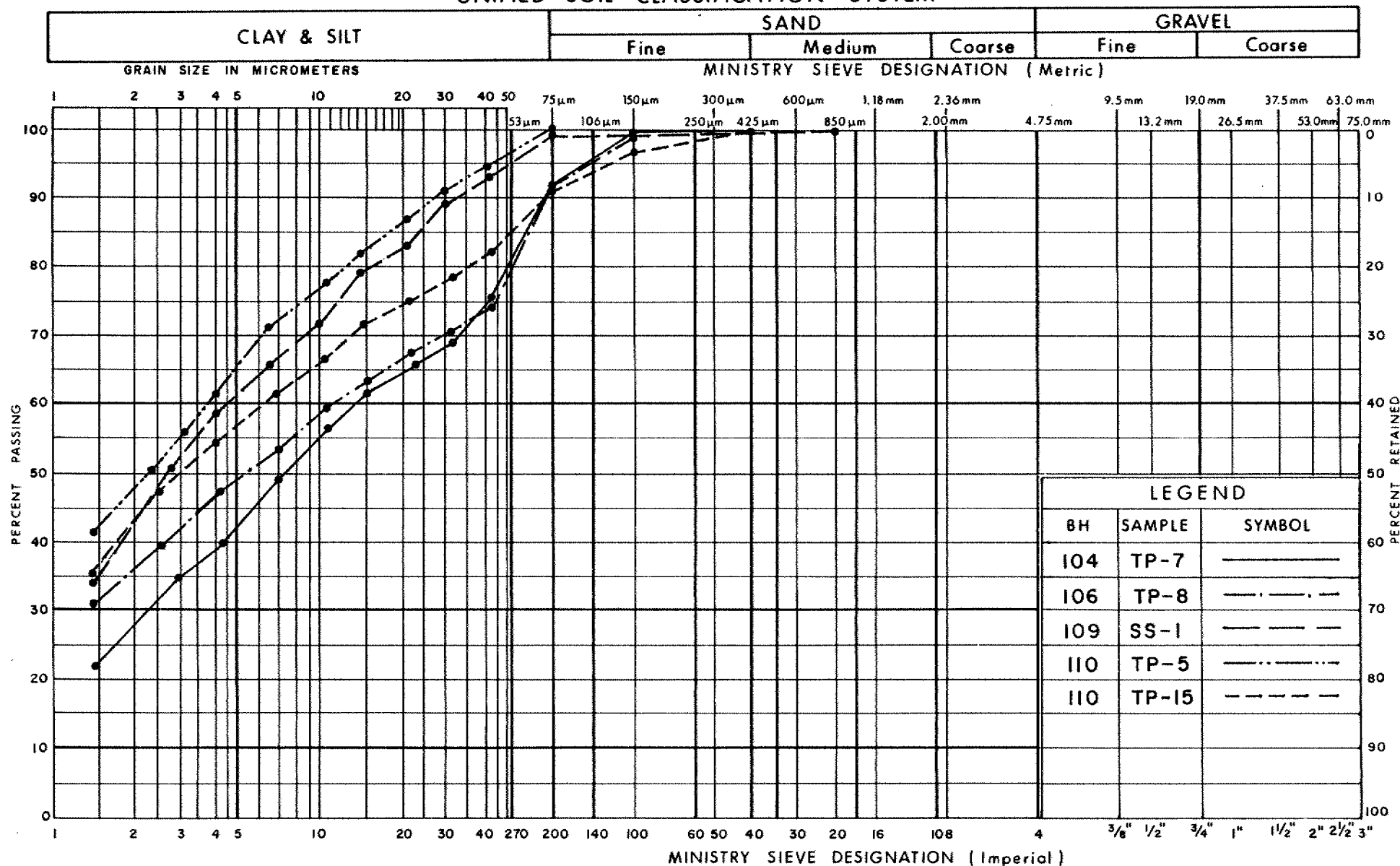
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**GRAIN SIZE DISTRIBUTION
GRAVELLY SILTY SAND WITH TRACE OF CLAY
(POSSIBLE FILL)**

FIG No 3

W P 126-87-01

UNIFIED SOIL CLASSIFICATION SYSTEM



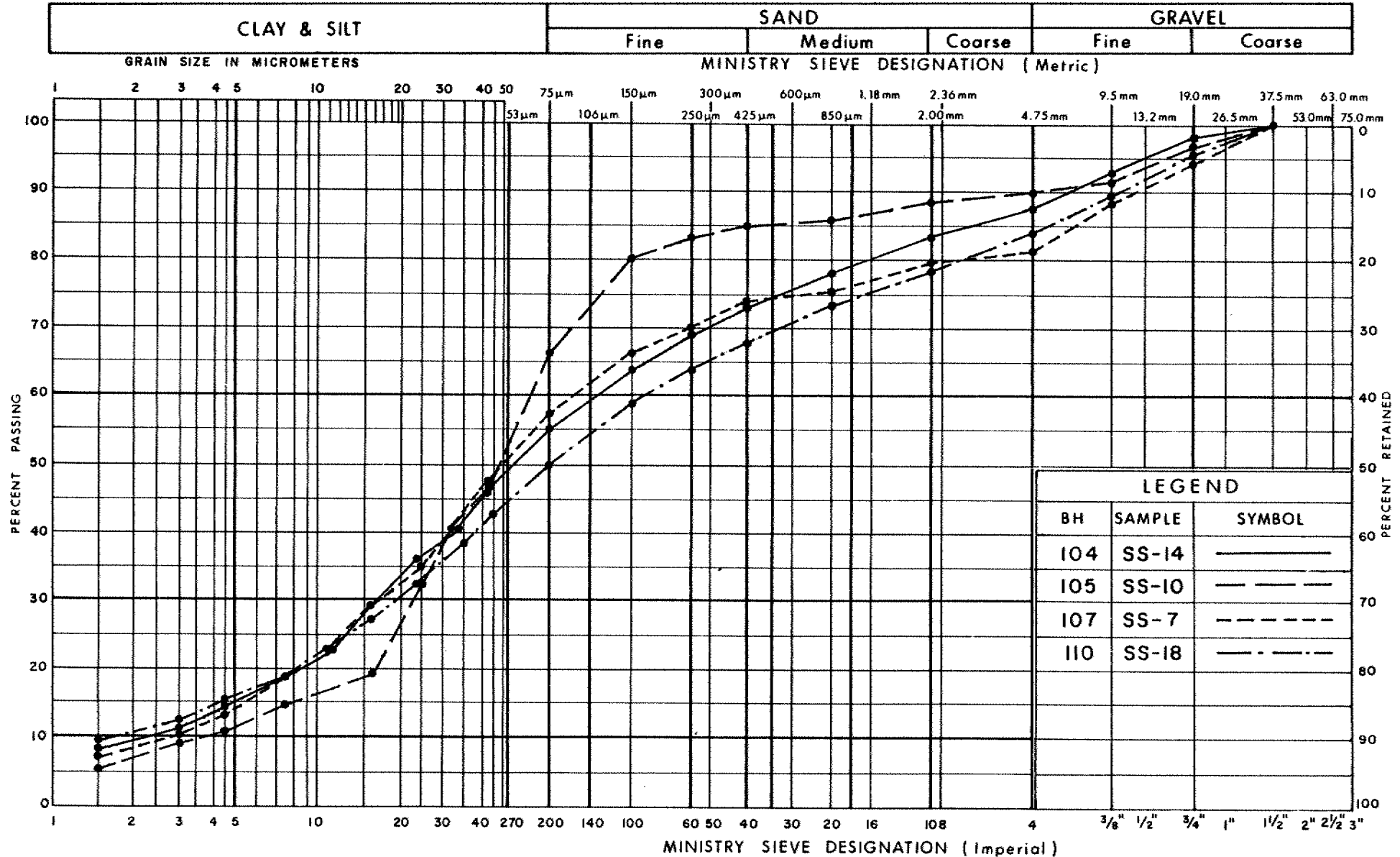
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GRAIN SIZE DISTRIBUTION SILTY CLAY (MARINE DEPOSIT)

FIG No 4

W P 126-87-01

UNIFIED SOIL CLASSIFICATION SYSTEM

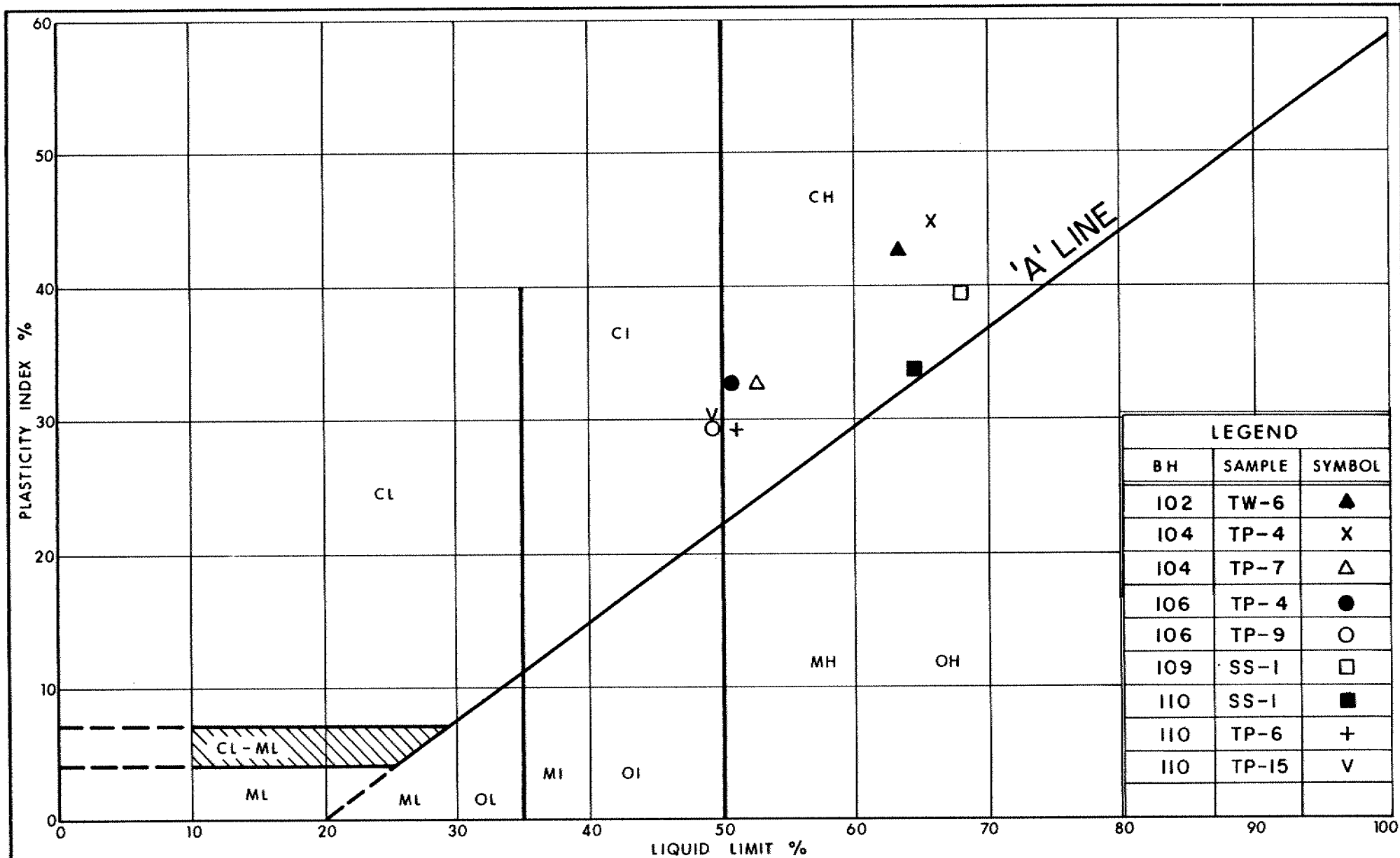


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GRAIN SIZE DISTRIBUTION
SAND AND SILT WITH
SOME GRAVEL AND SOME CLAY (TILL)

FIG No 5

W P 126-87-01



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Ontario

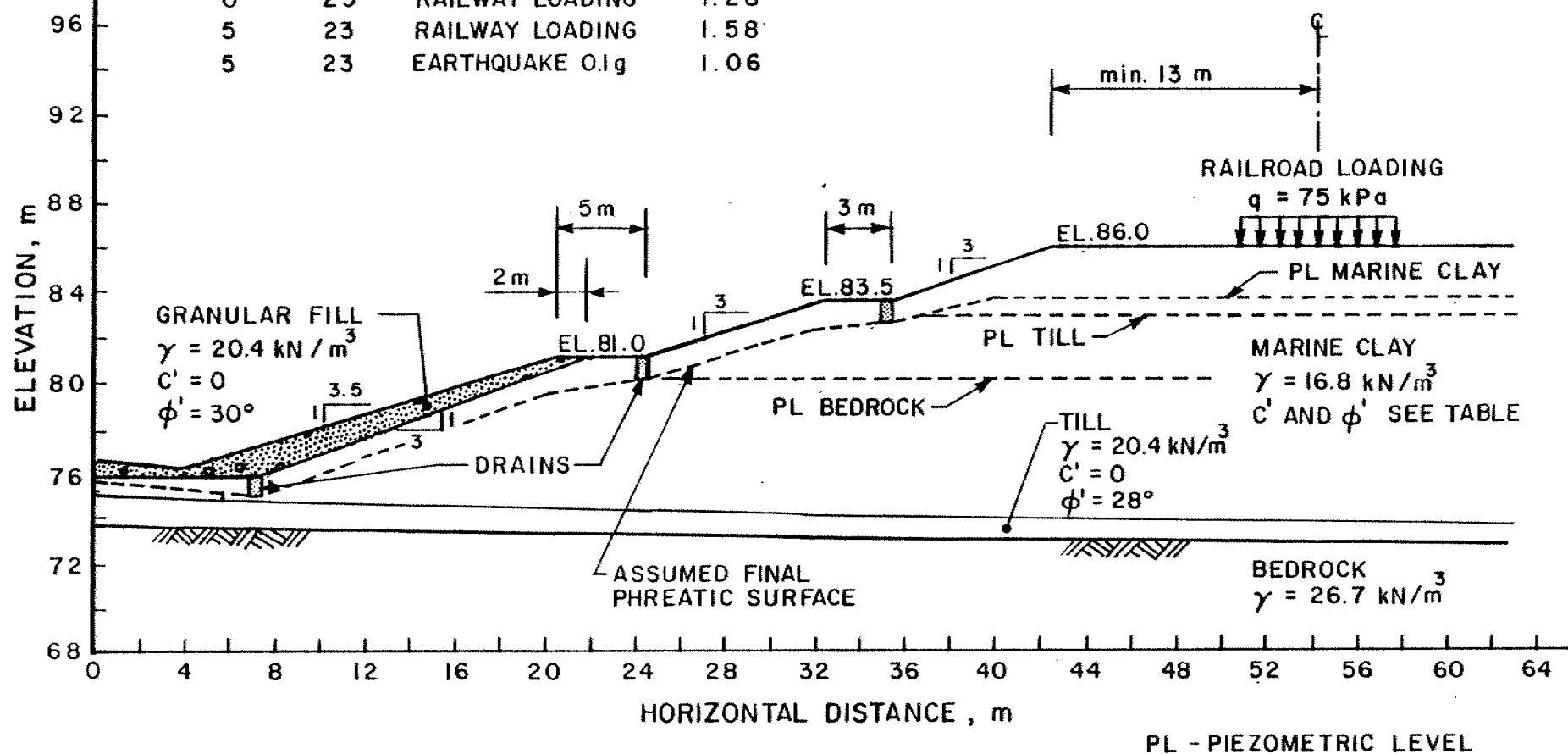
PLASTICITY CHART SILTY CLAY (MARINE DEPOSIT)

FIG No 6

W P 126-87-01

STABILITY ANALYSES - MODIFIED BISHOP METHOD

MARINE CLAY C' kPa	ϕ' °	CONDITION	MINIMUM F.S.
0	23	RAILWAY LOADING	1.28
5	23	RAILWAY LOADING	1.58
5	23	EARTHQUAKE 0.1g	1.06

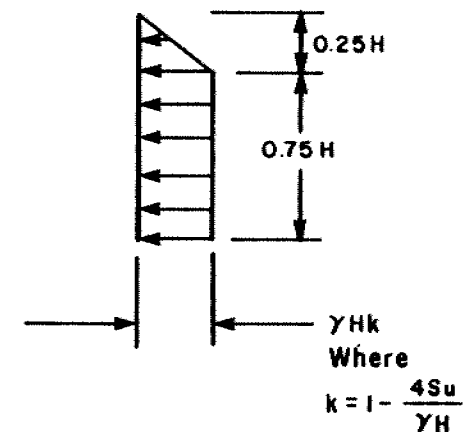
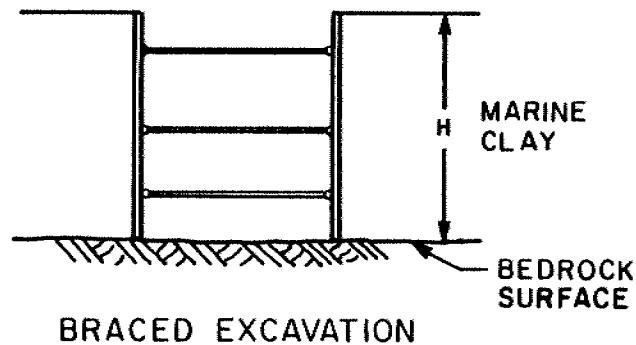


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SLOPE STABILITY ANALYSES EFFECTIVE STRESS ANALYSES EAST SIDE CONDITIONS

FIG No 7

W P 126-87-01



EARTH PRESSURE DIAGRAM
 (SEE NOTES 1 AND 2)

NOTES

1- Soil Parameters for Marine Clay

$S_u = 2.5 \text{ kPa}$ (Based on Remolded Strength)

$\gamma = 16.8 \text{ kN/m}^3$

$H =$ Height of Excavation

2- Design Must Also Include Surcharge Loading from Nearby Slopes and Equipment Loads Near the Excavation Area, etc.

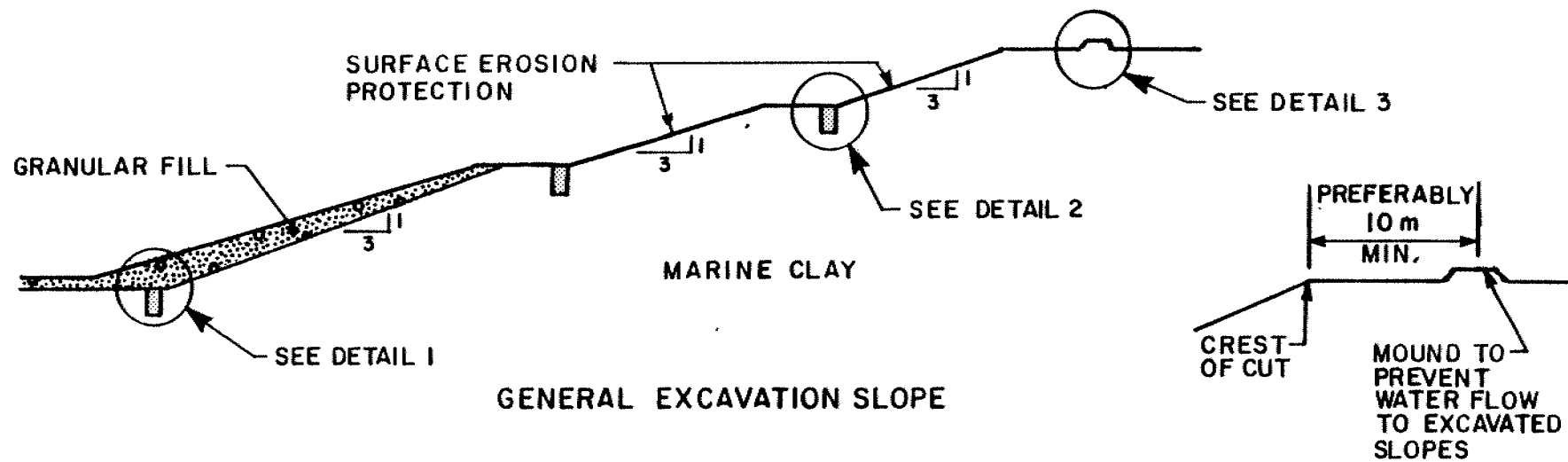


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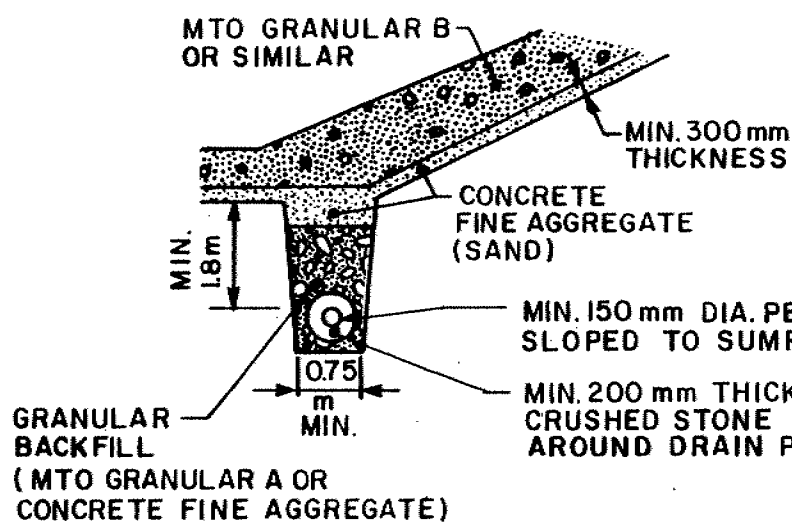
LATERAL EARTH PRESSURE DIAGRAM
 FOR BRACED EXCAVATION

FIG No 8

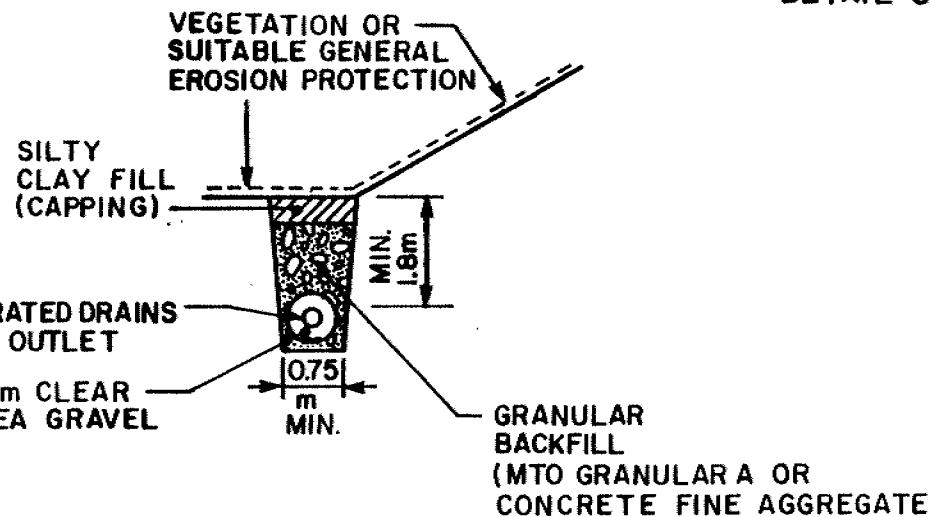
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DETAIL 3



DETAIL 1



DETAIL 2

NOT TO SCALE



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RECOMMENDED DRAINAGE SYSTEM

FIG No 9

W P 126-87-01

**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 - 3	3 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{VO}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_r	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 ²	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

Rock Weathering

Fresh	- No discoloration or loss of strength.
Slightly Weathered	- Some discoloration on discontinuities, no loss of strength.
Moderately Weathered	- Rock is discolored, discontinuities may be open, alteration starting to penetrate, rock is weaker than the fresh rock.
Highly Weathered	- Rock is discolored, discontinuities may be open, alteration penetrates deeply, loss of strength.
Completely Weathered	- Rock is discolored, completely altered but original fabric is preserved. A few core stones may be present. Properties still partly dependent on parent rock.
Residual Soil	- Rock is completely changed to a soil in which original fabric is absent. There is a large change in volume.

Rock Strength

	<u>Unconfined Compressive Strength</u>	
	<u>MPa</u>	<u>lb/in.</u>
Extremely strong	>200	>29000
Very strong	100 - 200	14500 - 29000
Strong	50 - 100	7750 - 14500
Moderately strong	12.5 - 50	1800 - 7750
Moderately weak	5 - 12.5	725 - 1800
Weak	1.25 - 5	180 - 725
Very weak	<1.25	<180

Fragmented Core - Fractured core where the average fracture spacing is less than 25 mm and the core pieces are less than full core diameter.

Very Closely Broken Core - Fracture core where the average fracture spacing is less than or equal to 50 mm and the core pieces are full core diameter.

Rock Soundness - The term 'sound rock' has been applied where RQD values are consistently greater than 75%.

Record of Boreholes

RECORD OF BOREHOLE No 101

METRIC

W P 126-87-01 LOCATION Coords N 5 021 267.4 ; E 358 878.3 ORIGINATED BY RH
DIST 9 HWY 416 BOREHOLE TYPE Hollow stem auger, BX rock core COMPILED BY RH
DATUM Geodetic DATE November 2, 3, 1989 CHECKED BY JLB

OFFICE REPORT ON SOIL EXPLORATION

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	* VALUES			20	40	60	80	100					
86.6	Ground Level					28/12/89											
0.0	Gravelly silty sand																
85.7	Possible Fill Brown		1	SS	9		86										21 50 23 6
0.9	Silty Clay, occasional thin layers of silty fine sand to fine sand, moist to wet, medium to high plasticity CI-CH (Marine Deposit)		2	SS	6		85										
			3	TW	PH		84										
			4	TW	PH		83										
			5	TW	PH		82										
			6	TW	PH		81										
			7	TW	PH		80										
79.6	Gray		8	TW	PH		79										
7.0	Sand and silt, with some gravel and some clay; wet, low plasticity to non-plastic, rapid dilatancy SM-NL (Till)		9	SS	5		78										
			10	TW	PH		77										
			11	SS	5		76										
75.3	Loose Dark Gray						75										
11.3	Limestone bedrock with frequent dolomite beddings, occasional calcite-filled vugs and shaly partings, very strong, fresh		12	RC BXL	REC 100%		74										RQD = 86%
72.3	Gray		13	RC BXL	REC 100%		73										RQD = 95%
14.3	End of borehole																

*For RC samples, numbers represent Core Recovery.

*3, x5: Numbers refer to Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



METRIC

W P 126-87-01 LOCATION Coords N 5 021 272.9 ; E 358 914.9 ORIGINATED BY RH
DIST 9 HWY 416 BOREHOLE TYPE Hollow stem auger, BX rock core COMPILED BY RH
DATUM Geodetic DATE November 3, 1989 CHECKED BY JOB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS 28/12/89	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT	Liquid Limit	UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE			*'N' VALUES		SHEAR STRENGTH *P _a ○ UNCONFINED • FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%) 20 40 60		
86.4	Ground Level											
0.0	Gravelly silty sand											
85.5	(Possible Fill) Brown		1	SS	7							
0.9	Silty clay, occasional thin layers of silty fine sand to fine sand, moist to wet, medium to high plasticity CI-CH (Marine Deposit)		2	SS	9							
			3	TW	PH							
			4	TW	PH							
			5	TW	PH							
	Very stiff, becoming firm with depth		6	TW	PH							
79.4	Gray		7	TW	PH							
7.0	Sand and silt with some gravel and some clay, wet, low plasticity to non-plasticity, rapid dilatancy SM-ML (Till)		8	SS	22							
	Very loose to compact		9	SS	2							
74.2	Dark Gray		10	RC	100%							
12.2	Limestone bedrock with frequent dolomite beddings, occasional calcite-filled vugs and shaly partings, very strong, fresh		11	BXL	100%							RQD = 67%
			12	RC BXL	100%						27.3	RQD = 84%
71.5	Gray		13	RC BXL	100%							RQD = 87%
14.9	End of borehole											

*For RC samples, numbers represent Core Recovery.

OFFICE REPORT ON SOIL EXPLORATION

*3, x5: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 103

METRIC

W P 126-87-01 LOCATION Coords N 5 021 283.0 ; E 358 994.0
DIST 9 HWY 416 BOREHOLE TYPE Hollow stem auger, BX rock core
DATUM Geodetic DATE November 3, 4, 1989
ORIGINATED BY RH
COMPILED BY RH
CHECKED BY *[Signature]*

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W _n	WATER CONTENT (%) W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES *			20 40 60 80 100	20 40 60 80 100					
86.4	Ground Level				8/12/89	86							
0.0	Gravelly silty sand	1	SS	8		86							
85.1	Possible fill Brown	2	SS	4		85							
1.3	Silty clay, some black mottling	3	SS	5		85							
84.3	Gray	4	TW	PH		84							
2.1	Silty clay, occasional thin layers of silty fine sand to fine sand, moist to wet, medium to high plasticity	5	TW	PH		83							
	CI-CH (Marine Deposit)	6	TW	PM		82							
		7	TW	PM		81							
		8	TW	PM		80							
		9	TW	PM		79							
		10	TW	PH		78							
		11	TW	PH		77							
73.6	Soft to firm Gray					76							
12.8	Sand and silt with some gravel and some clay	12	SS	11		75							
72.6	SM-ML (Till)					74							
13.8	Compact Dark Gray					73							
	Limestone bedrock with frequent dolomite beddings, occasional calcite-filled vugs and shaly partings, very strong, fresh	13	RC	REC		72							RQD = 98%
70.1	Gray	14	RC	REC		71							RQD = 100%
16.3	End of borehole												

*For RC samples, numbers represent Core Recovery.

RECORD OF BOREHOLE No 104

METRIC

W P 126-87-01 LOCATION Coords N 5 021 251.0 ; E 358 912.0 ORIGINATED BY RH
 DIST 9 HWY 416 BOREHOLE TYPE Hollow stem auger, BX rock core COMPILED BY RH
 DATUM Geodetic DATE November 6, 1989 CHECKED BY JMB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT Y KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	* VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	Wp	W	W _L		
86.2	Ground Level					28/12/89	86									
85.6	Silty clay with some gravel Dark Brown		1	SS	15		85									
0.6	Silty clay, occasional thin layers of silty fine sand to fine sand, moist to wet, medium to high plasticity		2	SS	9		84									
			3	SS	4		83									
			4	TP	PH		82									
			5	TP	PH		81									
	CI-CH (Marine Deposit)		6	TP	PM		80									
			7	TP	PH		79									
			8	TP	PH		78									
			9	TP	PM		77									
			10	TP	PH		76									
79.5	Firm to stiff Gray		11	TP	PH		75									
6.7	Sand and silt with some gravel and some clay, wet, low plasticity to nonplastic, rapid dilatancy		12	SS	5		74									
			13	SS	3		73									
	SM-ML (Till)		14	SS	3											
			15	SS	4											
			16	SS	5											
	Very loose to compact Dark Gray		17	SS	18											
74.2			18	SS	5											
12.0	Limestone bedrock with frequent dolomite beddings, occasional shaly partings, very strong, fresh		19	RC	REC											
72.6				BXL	98%											
13.6	End of borehole															

OFFICE REPORT ON SOIL EXPLORATION

*For RC samples, numbers represent Core Recovery.

+³, x⁵: Numbers refer to Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 105

METRIC

W P 126-87-01 LOCATION Coords N 5 021 286.4 ; E 358 840.0 ORIGINATED BY RH
 DIST 9 HWY 416 BOREHOLE TYPE Hollow stem auger, BX rock core COMPILED BY RH
 DATUM Geodetic DATE November 6, 7, 1989 CHECKED BY JAB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		UNIT WEIGHT Y 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	Wp	N		
36.3	Ground Level					28/12/89							
0.0	Silty clay with some gravel, black mottling and oxidation staining		1	SS	5		86						
			2	SS	10		85						
84.4	Gray		3	SS	10		84						
1.9	Silty clay, occasional thin layers of silty fine sand to fine sand, moist to wet, medium to high plasticity		4	TW	PH		83						
			5	TW	PH		82						
			6	TW	PH		81						
	CI-CH (Marine Deposit)		7	TW	PH		80						
			8	TW	PH		79						
77.8	Firm to stiff Gray		9	TW	PH		78					16.5	
8.5	Sand and silt with some gravel and some clay, wet, nonplastic SM-ML (Till)		10	SS	4		77						10 23 60 7
76.0	Loose to compact Dark Gray		11	SS	29		76						
10.3	Limestone bedrock with frequent dolomite bed- dings, occasional shaly partings, very strong fresh		12	RC BXL	REC 100%		75						RQD = 97%
74.4													
11.9	End of borehole												

OFFICE REPORT ON SOIL EXPLORATION

*For RC samples,
numbers represent
Core Recovery.

RECORD OF BOREHOLE No 106

METRIC

W P 126-87-01 LOCATION Coords N 5 021 291.1 ; E 358 871.9 ORIGINATED BY RH
DIST 9 HWY 416 BOREHOLE TYPE Hollow stem auger, BX rock core COMPILED BY RH
DATUM Geodetic DATE November 7, 8, 1989 CHECKED BY JAB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT Wp	NATURAL MOISTURE CONTENT W _p	LIQUID LIMIT WL	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	VALUES*			20 40 60 80 100	20 40 60 80 100					
85.5	Ground Level					08/12/89								
0.0	Silty clay, occasional thin layers of silty fine sand to fine sand, moist to wet, medium to high plasticity CI-CH (Marine Deposit)		1	SS	7		85						17.3	CU Triaxial Test See Fig. 2 0 8 55 37
			2	TP	PH		84							
			3	TP	PH		83							
			4	TP	PH		82							
			5	TP	PH		81							
			6	TP	PH		80							
			7	TP	PH		79							
			8	TP	PH		78							
			9	TP	PH		77							
			10	TP	PM		76							
78.6	Gray		11	TP	PH		75							RQD = 84%
6.9	Sand and silt with some gravel and some clay, wet, low plasticity to nonplastic SM-ML (Till)		12	SS	3		74							
			13	SS	7									
			14	SS	7									
			15	SS	29									
75.5	Very loose to compact Dark Gray		16	RC BXL	REC 100%									
10.0	Limestone bedrock with frequent dolomite beddings, occasional shaly partings, very strong, fresh		17	RC BXL	REC 100%									RQD = 100%
73.3	Gray													
12.2	End of borehole													

OFFICE REPORT ON SOIL EXPLORATION

*For RC samples, numbers represent Core Recovery.

RECORD OF BOREHOLE No 107

METRIC

W P 126-87-01 LOCATION Coords N 5 021 296.9 : E 358 904.4 ORIGINATED BY RH
 DIST 9 HWY 416 BOREHOLE TYPE Hollow stem auger, BX rock core COMPILED BY RH
 DATUM Geodetic DATE November 8, 1989 CHECKED BY JGP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	* VALUES			20 40 60 80 100	Wp	Wp	Wp		
85.1	Ground Level				28/12/89							
0.0	Silty clay, occasional thin layers of silty fine sand to fine sand, moist to wet, medium to high plasticity	1	SS	3		85					16.5	GR SA SI CL
		2	TW	PH		84						
						83						
	CI-CH (Marine Deposit)	3	TW	PH		82						
		4	TW	PH		81						
						80						
						79						
						78						
78.9	Firm Gray	5	TW	PH		77					18 24 49 9	
6.2	Sand and silt with some gravel and some clay, wet, low plasticity to nonplastic, rapid dilatancy	6	SS	5		76						
	SM-ML (Till)	7	SS	5		75						
74.5	Loose Dark Gray	8	SS	100		74						
10.6	Limestone bedrock with frequent dolomite beddings, occasional shaly partings, very strong, fresh	9	RC	REC							27.0	RQD = 96%
73.0												
12.1	End of borehole											

OFFICE REPORT ON SOIL EXPLORATION

*For RC samples, numbers represent Core Recovery.

METRIC

W P 126-87-01 LOCATION Coords N 5 021 301.1; E 358 935.3 ORIGINATED BY RH
DIST 9 HWY 416 BOREHOLE TYPE Hollow stem auger, BX rock core COMPILED BY RH
DATUM Geodetic DATE November 8, 9, 1989 CHECKED BY 2003

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		UNIT WEIGHT Y KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES*			20	40	60	80	100	W _p			W
85.0	Ground Level					28/12/89										
0.0	Silty clay, occasional thin layers of silty fine sand to fine sand, moist to wet, medium to high plasticity		1	SS	8											
			2	TW	PH											
	CI-CH (Marine Deposit)		3	TW	PH											
	Firm, becoming soft with depth		4	TW	PH											
			5	TW	PH											
77.4	Gray															
7.6	Sand and silt with some gravel and some clay, wet, low plasticity to nonplastic, rapid dilatancy SM-MI (Till)		6	TW	PH											
			7	SS	3											
73.5	Very loose to loose Dark Gray		8	SS	9											
11.5	Limestone bedrock with frequent dolomite beddings, very strong, fresh, vertical joints at approx. 13-m depth, occasional shaly partings		9	RC BXL	REC 100%											
71.1	Gray		10	RC BXL	REC 94%											
13.9	End of borehole															

*For RC samples, numbers represent Core Recovery.

RECORD OF BOREHOLE No 109

METRIC

N.P. 126-87-01 LOCATION Coords N 5 021 305.8 ; E 358 964.3 ORIGINATED BY RH
DIST 9 HWY 416 BOREHOLE TYPE Hollow stem auger, BX rock core COMPILED BY RH
DATUM Geodetic DATE November 9, 10, 1989 CHECKED BY TJB

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	NUMBER	TYPE			20 40 60 80 100	20 40 60 80 100					
84.5	Ground Level											
0.0	Silty clay; occasional thin layers of silty fine sand to fine sand, moist to wet, medium to high plasticity	1	SS 2		84							0 1 56 43
		2	TW PH		83							
		3	TW PH		82		+11					
	CI-CH (Marine Deposit)	4	TW PH		81		+10				17.2	
	Stiff, becoming soft to firm with depth	5	TW PM		80		+8				17.2	
		6	TW PH		79		+9					
		7	TW PH		78		+13					
		8	SS 1		76		+9					
74.5	Gray	9	TW PM		75							
10.0	Sand and silt with some gravel and some clay, wet, low plasticity to non-plastic	10	SS 1		74							
72.6	SM-ML (Till)	11	SS 1		73							
11.9	Very loose Dark Gray	12	RC REC		72							RQD = 92%
	Limestone bedrock with frequent dolomite beddings, occasional shaly partings, very strong, fresh	13	RC REC		71							RQD = 100%
70.1	Gray											
14.4	End of borehole											

OFFICE REPORT ON SOIL EXPLORATION

*For RC samples, numbers represent Core Recovery.

RECORD OF BOREHOLE No 110

METRIC

W P 126-87-01 LOCATION Coords N 5 021 316.4 ; E 358 990.4 ORIGINATED BY RH
 DIST 9 HWY 416 BOREHOLE TYPE Hollow stem auger, BX rock core COMPILED BY RH
 DATUM Geodetic DATE November 10, 11, 1989 CHECKED BY JAB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES*			20 40 60 80 100	W _p W _L	20 40 60	KN/m ³			
85.7	Ground Level					28/12/89								
0.0	Silty clay, occasional thin layers of silty fine sand to fine sand, moist to wet, medium to high plasticity		1	SS	8		85							
			2	SS	10		84							
			3	TP	PH		84							
			4	TP	PH		83							
			5	TP	PH		83							
	CI-CH (Marine Deposit)		6	TP	PM		82							
			7	TP	PH		82							
	Firm to stiff		8	TP	PM		81							
			9	TP	PH		80							
			10	TP	PH		80							
			11	TP	PM		79							
			12	TP	PM		78							
			13	TP	PM		77							
			14	TP	PH		76							
			15	TP	PM		75							
			16	TW	PH		75							
73.8	Gray		17	TW	PH		74							
11.9	Sand and silt with some clay, wet, low plasticity to non-plastic		18	SS	2		73							
72.9														
12.8	SM-ML (Till) Very loose Dark Gray End of borehole Refusal to auger (probable bedrock)													

OFFICE REPORT ON SOIL EXPLORATION

Drawings

OVERSIZE DRAWING