

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 30 MS-128

DIST. 4 REGION

W.P. No. 83-74-24

CONT. No. 83-11

W. O. No.

STR. SITE No. 10

HWY. No. 1

LOCATION Hager Creek Culvert  
Extension

No. of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

GEOTECHNICAL INVESTIGATION  
C.N.R. OVERHEAD AT Q.E.W.  
SITE 10-135A, W.P. 83-74-26R  
C.N.R. OVERHEAD AT HWY. 403W-  
FAIRVIEW STREET RAMP  
SITE 10-135B, W.P. 83-74-27R  
RAMBO-HAGER CREEK CULVERT EXTENSION  
SITE 10, W.P. 83-74-24  
DISTRICT 4, BURLINGTON, ONTARIO



PETO MacCALLUM LTD.

45 BURFORD RD., HAMILTON, ONTARIO L8E 3C6  
CONSULTING ENGINEERS

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FOR

MINISTRY OF TRANSPORTATION &  
COMMUNICATIONS.

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Job No. 80 F 300

January, 1981.



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January 26, 1981

Ministry of Transportation and Communications  
Pavement and Foundation Design Section  
Room 315, Central Building  
1201 Wilson Avenue  
DOWNSVIEW, Ontario  
M3M 1J8

Attention: Mr. K. Selby, P.Eng.  
Senior Foundation Engineer

Gentlemen:

Re: Geotechnical Investigation  
C.N.R. Overhead at Q.E.W.  
Site 10-135A, W.P. 83-74-26R  
C.N.R. Overhead at Hwy. 403W -  
Fairview Street Ramp,  
Site 10-135B, W.P. 83-74-27R  
Rambo-Hager Creek Culvert Extension  
Site 10, W.P. 83-74-24  
District 4, Burlington, Ontario

We are pleased to present our final report for the geotechnical investigation carried out for the proposed grade separation structures and culvert extension referenced above, as authorized in Agreement No. 4242-9080-106.

Our preliminary comments and recommendations concerning construction of the two structures were presented in our report dated December 29, 1980.



The attached report provides complete details of the field and laboratory work carried out, the soils rock and ground-water conditions encountered at the two development sites and foundation recommendations.

The stratigraphy encountered at the bridge site generally comprises relatively thin surficial fills, topsoil and sand layers, a major hard silty clay till deposit extending to about 8.2 to 10.2 m below grade underlain by very dense sand/silt till overlying Queenston shale bedrock at approximately 11.6 to 13.6 m depth. At the Rambo-Hager Creek Extension site the stratigraphy is generally similar except the major silty clay till is interlayered with very dense sandy silt till.

Subsurface conditions are quite favourable for the use of conventional spread footings to support the proposed bridge structures. However, due to construction and other constraints, various alternative foundation schemes, which are particularly applicable to supporting the abutments are discussed. These include supporting footings on an engineered fill, drilled caissons or driven piles as summarized below:

<u>Foundation Type</u>	<u>Founding Material</u>	<u>Founding Elevation</u>	<u>Bearing Capacity</u>
Spread Footings	Silty Clay Till	96.5	400 kPa
Spread Footings	Well Compacted Granular fill	Nominal Depth	175 kPa
Drilled Caisson	Silty Clay Till	94	1200 kPa
Concrete filled steel pipe pile	Sand/Silt Till	88	*980 kN for 324 mm (12.75 in.) O.D. X 7.1 mm (0.281 in.) wall thickness  *1070 kN for 324 mm (12.75 in.) O.D. X 7.9 mm (0.312 in.) wall thickness
Steel "H" Pile	Shale Bedrock	83 to 85	Structural capacity of section

\*Capacity based on 20 MPa concrete strength. A 300 kN increase in pile capacity may be achieved by utilizing 30 MPa compressive strength concrete.

The report presents parameters for design of abutment walls and approach embankment and discusses problems that might be encountered during the bridge construction including safe construction slopes, bracing requirements and ground-water control.

At the Rambo-Hager Creek culvert extension site, two methods of extending the existing culvert are discussed and compared. It is more feasible from a geotechnical viewpoint to extend the existing structure by constructing a 3 cell box culvert and relocating the inlet at the new upstream location, as it would involve shallower excavation, a minimum of groundwater control and in general, less materials handling and relatively straightforward construction.

We trust that this report satisfies your requirements and thank you for the opportunity to be of service to the Ontario Ministry of Transportation and Communications. If you have any questions, or if any point in the report requires clarification, please do not hesitate to call our office.

Yours very truly,  
PETO MACCALLUM LTD.

  
Sol Pilch, P. Eng.,  
Chief Geotechnical Engineer.

TLB/SP/rf

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### LABORATORY TEST RESULTS

TABLE I	- Atterberg Limit Test Results
TABLE II	- Laboratory Unconfined Compresion Test Results
TABLE III	- pH Value and Sulphate Content of Soil Samples
TABLE IV	- pH Value and Sulphate Content of Water Sample

FIGURE NO'S 1 to 3 - Grain Size Distribution

RECORD OF BOREHOLE SHEETS

BOREHOLE LOCATION PLAN AND SOIL STRATA



## 1. INTRODUCTION

Peto MacCallum Ltd. was authorized by The Ministry of Transportation and Communications, Agreement No. 4242-9080-106 to carry out a geotechnical investigation at the sites for the proposed C.N.R. overhead structures at Q.E.W. and Hwy. 403W - Fairview Street Ramp and the Rambo-Hager Creek Culvert Extension in Burlington, Ontario.

The subject project constitutes part of the overall Q.E.W. reconstruction from Lockhardt Road northerly to Brant Street.

A summary of the proposed development plans and construction sequence was provided in a copy of The Ministry of Transportation and Communications internal memorandum to Mr. K. G. Selby, Senior Foundation Engineer, dated October 14, 1980, and accompanying Drawings 1123-912 and 913. The development plans were subsequently discussed in the meeting of November 10, 1980.

The proposed C.N.R. overhead at Q.E.W. will be a twin structure (one to carry north bound traffic and the other south bound traffic) with three spans of about 20 m - 30 m - 20 m. Stage construction is planned, involving firstly, construction of the east portion of the east bridge. This would allow for construction clear of the existing Q.E.W. Traffic will then be detoured over the newly constructed east deck section so that the present bridge can be removed and the remaining deck cross-section completed.

The proposed superstructure will be of steel girders and therefore no falsework will be required over the C.N.R.

We understand that both sections of the proposed twin structure will be widened in 10+ years to suit ultimate expansion of the Q.E.W.

The new C.N.R./Hwy. 403W - Fairview Street Ramp will have similar spans as the proposed twin structure. No future deck widening is anticipated for this bridge.

It is noteworthy that the site for the present C.N.R./Q.E.W. bridge was investigated in 1955 by our predecessors, Racey, MacCallum and Associates Limited, and the results presented in report No. S-500-505/55 T91-1.

Due to realignment, the north Q.E.W. lanes will be located over the present inlet structure of the Rambo-Hager Creek diversion channel. Extension of the existing culvert will be required to accommodate the new Q.E.W. and two alternatives are being considered. The first involves extending the existing structure with a 3 cell concrete box culvert and reconstructing the inlet at the new location some 30 m upstream. The other possibility consists of removing the existing drop structure, extending the 3 cell culvert and reconstructing the new drop and inlet structures at the new upstream location.

The purpose of this investigation is to determine the subsurface soils and groundwater conditions at the proposed construction sites and based on this information to comment on and provide geotechnical engineering recommendations pertinent to the design and construction of the proposed C.N.R. overhead structures at Q.E.W. and Hwy. 403W - Fairview Street Ramp and the Rambo-Hager Creek culvert extension.

A preliminary report dated December 29, 1980, was provided following completion of the initial field work, which summarizes the subsurface conditions and geotechnical recommendations for the two projects.

## 2. FIELD WORK

The results of the previous investigation by Racey, MacCallum and Associates Limited at the bridge site, indicated that a competent silty clay till layer extends over the development area, which has the capacity to support relatively heavy loads. The scope of the present investigation was established cognizant of the subsurface information available at the site.

A total of eight (8) boreholes were initially scheduled at Sites 10-135A and B, for the proposed grade separation structures. Three (3) of these holes were scheduled to be drilled through the approach embankment from above the present bridge structure. However, due to spacial limitations as well as traffic and safety constraints which would require closure of the inside Q.E.W. lanes, two (2) of these boreholes were relocated to natural grade under the existing bridge beside the C.N.R. tracks and the other deleted. Consequently, no information is available concerning the nature and engineering properties of the approach embankment fill. Therefore, a total of seven (7) boreholes, boreholes 1 to 7, were drilled at the site for the C.N.R./Q.E.W. and C.N.R./Hwy. 403W grade separation structures.

Two (2) holes, boreholes 8 and 9, were drilled at the site for the Rambo-Hager Creek culvert extension.

The field work was carried out during the period of November 18 to 22, 1980 and January 7 and 8, 1981. The

holes were drilled at the locations shown on the appended Plan. The boreholes were extended to a depth of 6.55 to 17.34 m below existing grade, using a CME-55 drillrig equipped with continuous flight solid stem and hollow stem augers and rock coring capabilities, supplied and operated by a specialist drilling contractor.

Representative samples of the overburden were obtained at frequent intervals using a conventional split spoon sampler in conjunction with standard penetration tests. Relatively undisturbed samples of the cohesive soils encountered at the site were recovered in thin walled shelby tubes. The undrained shear strength of the silty clay till was assessed by conducting pocket penetrometer tests on the recovered samples. BXL core samples of the bedrock at the bridge site were recovered.

The groundwater conditions in the open boreholes were closely monitored during and on completion of drilling. Two (2) piezometers at the proposed C.N.R./Q.E.W. bridge site, and one (1) piezometer at the proposed culvert extension site were installed and monitored to determine the stabilized groundwater conditions. Details of the piezometer installations are described on the appended, Record of Borehole No's 1, 5 and 8.

The field work was supervised throughout by a member of our engineering staff who directed the drilling and sampling operations, documented the soil and bedrock stratigraphy encountered, monitored the groundwater condition in the open boreholes, detailed the piezometer installations and cared for the recovered samples.

The location and ground surface elevation at the boreholes were established in the field by Peto MacCallum Ltd. The following geodetic benchmarks, provided by The Ministry

of Transportation and Communications were used as reference for vertical control:

BM: Cut cross on south concrete retaining wall of culvert under C.N.R. tracks. 112.0 m left of STATION 9+913.8. Approximately 100 m east of Q.E.W./C.N.R. bridge. Elevation: 99.986.

BM: Nail and washer in east root of 0.35 m  $\emptyset$  ash tree. 10.0 m left of STATION 11+190.0. Elevation: 87.746.

### 3. LABORATORY TESTING PROGRAMME

All of the recovered samples were brought to our laboratory for detailed visual examination and routine testing to confirm field visual classifications. In addition, the following tests were conducted:

- Moisture Content on all samples
- Four (4) - Atterberg Limits, Table I
- Three (3) - Unconfined Triaxial Compression, Table II
- Two (2) - pH and Sulphate Content of Soil Samples, Table III
- One (1) - pH and Sulphate Content of Ground-water Sample, Table IV
- Five (5) - Grain Size Analysis, Figure No's 1 to 3.

### 4. SITE DESCRIPTION AND GEOLOGY

The site for the proposed grade separation structures and culvert extension are generally flat and fairly well

drained. A light vegetative cover of small bushes and grass has developed over the surface.

The development areas are located near the northern edge of the Lake Iroquois Plain, which is typified by a fine to silty fine sand veneer.

Bedrock consists of red Queenston Shale. Drift thickness is generally less than 15 m thick and comprises generally glacial till deposits. Very often the basal portion of the overburden is distinctly red in color from large amounts of incorporated Queenston Shale.

## 5. SUBSURFACE SOILS AND GROUNDWATER CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the field work including soil and rock classifications, inferred stratigraphy, standard penetration "N" values, the results of field and laboratory undrained shear strength testing, groundwater observations in the open boreholes and installed piezometers, laboratory moisture content determinations and Atterberg Limit test results.

The stratigraphy at the bridge site generally comprises relatively thin surficial fills, topsoil and sand layers, a major hard silty clay till deposit extending to about 8.2 to 10.2 m below grade underlain by very dense sand/silt till overlying Queenston shale at approximately 11.6 to 13.6 m depth.

At the Rambo-Hager Creek extension site, the stratigraphy is generally similar except the major silty clay till deposit is interlayered with very dense sandy silt till.

A summarized account of the major stratigraphic units are presented below:

5.1 SITES 10-135A AND B,  
C.N.R. OVERHEADS AT Q.E.W. AND HWY. 403W -  
FAIRVIEW STREET RAMP

5.1.1 FILL AND TOPSOIL

A 1.37 m thick fill layer generally comprising firm clayey silt and silty clay was encountered near the C.N.R. tracks in boreholes 2 and 6. Local 300 mm thick silty clay fill and mixed fill deposits were noted at the surface in boreholes 3 and 7.

A topsoil layer, 130 to 300 mm in thickness was encountered surficially in boreholes 4 and 5 and under the fill layer in borehole 7.

5.1.2 SAND

A loose to compact silty sand unit was contacted generally over the site. This unit was encountered surficially in borehole 1, and under the topsoil or fill layers in boreholes 2, 3, 6 and 7. The layer is 0.33 to 1.38 m thick and was penetrated at 0.84 to 2.07 m below grade, near elevation 96.51 to 97.78.

Moisture content in the sand varies between about 8 to 18%.

5.1.3 SILTY CLAY TILL

A major, generally hard, heavily over-consolidated silty clay till deposit was contacted in all of the testholes under the topsoil or sand layer some 0.13 to 2.07 m below grade near elevation 96.51 to 98.54. Two typical gradation curves are shown in Figure No. 1, appended.

The moisture content in the clay varies between about 7 to 18% but typically ranges between approximately 11 to 14%. The liquid and plastic limits of two representative samples are 24 and 32, and 15 and 16 respectively indicating a low plasticity.

The major clay till unit was not penetrated in boreholes 2, 3, 4 and 6 at the termination depth of 6.55 m, but was penetrated in boreholes 1, 5 and 7 at the 8.23 to 10.21 m depth, elevation 88.18 on the west side of the bridge and 90.44 on the east side.

#### 5.1.4 SANDY SILT TILL AND SILTY SAND TILL

Dense to very dense reddish brown sandy silt till and silty sand till (typical gradation, Figure No. 2) were encountered below the major clay till unit in boreholes 1, 5 and 7.

In the west portion of the site, the granular till unit was contacted at about 10.1 and 10.2 m below grade, near elevation 88.2 and 88.4, in boreholes 1 and 7 respectively, while in the east section, borehole 5, contact was at about the 8.2 m depth at approximate elevation 90.4.

The moisture content is about 8 to 11%. This unit was not penetrated in borehole 7.

#### 5.1.5 BEDROCK

Red Queenston Shale bedrock was contacted under the sand/silt till unit in the two deep boreholes 1 and 5 at 13.56 and 11.58 m, elevation 84.68 and 87.09 respectively.

Based on detailed visual examination of the recovered core samples, the top approximately 1.8 and 0.9 m of the shale bedrock in boreholes 1 and 5 respectively, is weathered, becoming sound below this depth. The shale is generally of poor quality in the weathered zone and of fair to good quality in the sound zone.



#### 5.1.6 GROUNDWATER CONDITIONS

The groundwater conditions observed in the testholes and piezometers indicate a variable groundwater regime at the site. Groundwater which was encountered in the surficial pervious materials in the holes drilled within the railway right-of-way, boreholes 2 and 6, is considered to be perched.

Boreholes 1, 3 and 4, which were terminated in the silty clay till layer remained dry during and upon completion of drilling. The piezometer installed in borehole 1, has also remained dry.

Groundwater was encountered in boreholes 1A, 5, 5A and 7 which contacted the basal sand/silt till. The piezometer installed in the basal till in borehole 5, shows that the stabilized groundwater level at the time of the investigation is at elevation 94.01.

### 5.2 SITE 10, RAMBO-HAGER CREEK CULVERT EXTENSION

#### 5.2.1 SUBSOIL CONDITIONS

The surficial stratigraphy at this site comprises a thin veneer of silty clay fill (borehole 9) and compact silty sand to 1.37 and 1.52 m depth (boreholes 8 and 9 respectively). Underlying the sand layer, interlayered generally hard silty clay till (typical gradation shown on Figure No. 3, appended) and very dense sandy silt till units were encountered and was penetrated at the 7.01 m depth, near elevation 81.39 and 81.29 (boreholes 8 and 9, respectively) where layered silt and weathered shale and shale bedrock were contacted.

### 5.2.2 GROUNDWATER CONDITIONS

Borehole 9 was open and dry at the conclusion of drilling while the groundwater level in borehole 8 at the conclusion of drilling was near elevation 81.3. The piezometric level in borehole 8 is stabilizing near elevation 85.95.

## 6. ENGINEERING CONSIDERATIONS

### 6.1 C.N.R. OVERHEADS AT Q.E.W. AND HWY. 403W. - FAIRVIEW STREET RAMP

#### 6.1.1 FOUNDATION ALTERNATIVES

The soils and groundwater conditions at the site are quite favourable for the use of conventional spread footings. However, due to construction constraints, including spacial limitations, traffic considerations and the existing 10 m high fill embankment, an alternate deep foundation system may be considered, especially to support the abutments. The final support system will be dictated by economics and may comprise any one or combination of several alternate geotechnically feasible schemes described below:

##### 6.1.1.1 Spread Footings

The proposed bridge structures may be supported on conventional spread footings founded in the native hard silty clay till layer and proportioned using a net allowable bearing capacity of 400 kPa. The surface elevation of the competent bearing material varies somewhat across the site from about minimum elevation 96.5 to maximum elevation 97.5. The founding level should be established during construction cognizant of the normal requirement for frost protection (1.2 m of earth cover) and the variation in surface elevation of the bearing material. For preliminary

design purposes it may be assumed that footings are founded at elevation 96.5.

Settlement of footings founded in the silty clay till unit is expected to comprise primarily elastic compression in the clay stratum which should occur immediately following load application. Long-term consolidation settlement of the heavily over-consolidated silty clay till layer is expected to be negligible. It is estimated that the magnitude of the immediate elastic settlement will not be greater than 15 mm.

It may be possible to support the abutment footings in the existing approach fill. However, as pointed out earlier, determination of the pertinent properties of the existing approach fill material was not possible at this time as drilling would require closure of one of the Q.E.W. traffic lanes. As such, design recommendations for this alternative are not available unless additional drilling is carried out in the approach embankment.

The abutment may also be supported on relatively shallow spread footings founded within the approach embankment providing granular substitution of the existing approach fill material below the footing and provision of a structural fill in the widened portion of the embankment is carried out in accordance with standard MTC construction procedures. The footings may then be designed for an allowable bearing capacity of 175 kPa. Further comments in regards to site preparation prior to structural fill placement are presented later in the text.

Anticipated maximum settlement of footings in the engineered fill is about 25 mm. The majority of this settlement should occur during the construction period.

#### 6.1.1.2 Drilled Caissons

Should construction constraints dictate, drilled caissons may be used to support the proposed bridges. An allowable end bearing capacity of 1200 kPa may be used for design of caissons founded in the hard silty clay till layer. The caisson should be embedded to a depth equal to at least 4 times the caisson diameter, into the silty clay till layer. The anticipated founded level is near elevation 94 but depends on the diameter of the caisson. Downhole inspection is recommended to ensure the hole is properly cleaned out of all disturbed material and to verify the competency of the founding surface.

Groundwater is not expected to pose any undue problems since the augering will be within the relatively impervious clay till layer.

Additional comments regarding the caisson installation operations are presented in the following section.

#### 6.1.1.3 Driven Piles

Alternatively, either concrete filled closed end steel pipe or steel "H" piles may be used to support the bridge structure.

It is anticipated that driven steel pipe piles will reach practical refusal near elevation 87 on the west side of the bridge and 89 on the east side of the bridge in the very dense sand/silt till layer with an average founding elevation across the structure of 88. Relatively high driving resistance is expected during installation through the hard silty clay till and therefore a heavy pile section is recommended.

Working loads of 980 and 1070 kN are suggested for concrete filled pipe piles 324 mm (12.75 in.) O.D. with wall thickness of 7.1 and 7.9 mm (0.281 and 0.312 in.) respectively, filled with 20 MPa compressive strength concrete. An increase of 300 kN in pile capacity may be achieved by utilizing 30 MPa compressive strength concrete.

Steel "H" piles driven to practical refusal in the shale bedrock are suitable for developing the full structural capacity of the section.

The borehole information indicates that the bedrock surface generally dips from east to west, from about elevation 87.1 to 84.7 in boreholes 1 and 5 respectively. It is anticipated that the steel "H" piles will meet refusal between elevation 83 to 85 across the bridge structure.

In this regard, it should be pointed out that the steel "H" piles founded in the Queenston Shale are known to "relax" following driving. Therefore, it is recommended that the piles are redriven after a waiting period of at least 3 days. The indicated refusal depth takes this mechanism into consideration.

It is expected that the majority of any settlement which the deep foundation support system experiences will be limited to elastic compression of the structural member.

The characteristics of the approach fills have not been established as noted previously and consequently, additional drilling will be required in order to provide comments regarding installation of caissons/driven piles through the fill.

The "H" pile tip should be reinforced with welded steel plates or rock points to minimize damage during driving through the very dense basal till and bedrock.

A minimum spacing of 3 times the pile diameter is recommended between individual members.

Piles should be driven using a hammer with a minimum rated energy of 40 kJ/blow. The capacity of the pile should be confirmed during installation using the Hiley's Formula.

The installation operations should be inspected on a full-time basis by qualified geotechnical personnel to ensure uniformity of set, founding elevation, alignment, plumbness as well as proper splice welds.

#### 6.1.1.4 Design Considerations

Although the magnitude of settlement of the various stages of the proposed twin bridge construction would be similar (assuming equivalent loading and foundations are used to support each stage), the settlement of the initial east portion of the easterly bridge will essentially be completed prior to commencement of the second stage of this bridge. Accordingly, the planned structure should be designed to accommodate the differential settlement at the junction of the staged segments, cognizant of the type or combination of types of support systems that are adopted. Overlapping of stress bulbs from adjacent shallow foundations is not expected to affect the performance of the structure.

Differential settlement will not be of concern in the C.N.R./Hwy. 403W structure as stage construction is not programmed on this bridge.

It may be prudent to construct the foundations for the ultimate bridge widening at this time, so as to minimize the excavation, bracing requirements or duplication of work during construction of the future bridge deck. In this regard, negligible settlement should be expected in these "pre-built foundations" as the full load will not be imposed until construction of the future addition.

Further, cognizant of the favourable subsurface conditions at this site, a single span structure with conventional "closed" abutments supported on spread footings may be considered as an alternative to the proposed three span bridge structure.

### 6.1.2 ABUTMENTS

Both closed and open ended type abutments are feasible depending on spacial limitations and the bridge design chosen. Abutment walls should be designed to resist the unbalanced lateral forces acting on the wall. In this regard, provided that standard practice is followed involving the provision of free-draining granular backfill and the installation of weepholes or weeping tiles behind the wall to prevent the build-up of hydrostatic pressures as well as the mobilization of the passive pressures in front of the wall, the following design parameters are recommended:

Earth pressure coefficient at rest,  $k_0 = 0.5$   
if the wall is rigid and unyielding.

Earth pressure coefficient,  $k = 0.33$   
if some movement of the top of the wall is permitted.

Friction angle between granular fill and wall,  
 $\delta = 24^\circ$ .

Friction angle between footing and hard silty clay  
till,  $\delta = 22^\circ$ .

Friction angle between footing and well compacted  
granular fill,  $\delta = 24^\circ$ .

Passive earth pressure coefficient,  $k_p = 3.0$   
assuming granular backfill in front of wall.

Bulk density for compacted granular fill behind  
the wall,  $\gamma = 21.2 \text{ kN/m}^3$ .

### 6.1.3 APPROACH EMBANKMENT

The proposed construction will involve realignment and widening of the existing approach. Anticipated maximum height of the approach fill is some 10 m.

Prior to construction of the fill embankment (or structural fill), all topsoil, existing fills and obviously deleterious materials should be sub-excavated and the exposed surface proof-rolled to ensure at least 95% Standard Proctor maximum dry density.

We recommend longitudinal and transverse slopes of 2 horizontal to 1 vertical for the approach embankment. Provided suitable borrow material is employed and MTC standard construction procedures are observed, we do not anticipate any slope or base stability problems, as exemplified by the satisfactory performance of the existing approach embankment. Conventional slope protection involving seeding or sodding should be observed to control erosion due to surface runoff.

#### 6.1.4 CONSTRUCTION AND GROUNDWATER CONTROL

Staged construction is planned as described in the previously referenced memorandum.

Construction will be carried out within the area of the existing approach embankment as well as in areas unaffected by the present embankment. We do not expect that the different stress history in these areas will affect the overall performance of the finished structure.

No real advantage is derived from construction of the approach fill in advance of construction (i.e. surcharging the site). However, the construction sequence of the approach embankment should be designed so as to facilitate construction of the foundation type that is utilized.

Construction slopes should be cut at 1 horizontal to 1 vertical, subject to geotechnical inspection.

It will be necessary to support the existing or new approach fill slopes during the various stages of construction if excavation encroaches within a line drawn at 1 horizontal to 1 vertical through the crest of the embankment. In accordance with this requirement, construction of the engineered



fill to support the abutments of the various stages of the proposed structure will require partial removal of the existing approach embankment and bracing the remaining portion of the embankment in order to maintain the integrity of the Q.E.W.

One advantage of supporting the abutments on piles/caissons is that the approach fill widening can be constructed to abut the existing embankment, and then the piles/caissons can be installed through the fill slope. This method would involve a minimum of braced construction slopes.

Groundwater should not pose any special problems. Local nuisance seepage or surface runoff that enters the construction area should be readily handled by conventional sump pumping.

## 6.2 RAMBO-HAGER CREEK CULVERT EXTENSION

### 6.2.1 GENERAL

Two methods of extending the existing culvert are under consideration as described in the Introduction. For the purposes of this report, Alternative A, refers to the extension of the existing structure with a 3 cell concrete box culvert and relocating the inlet some 30 m upstream. Alternative B, refers to the method involving extending the present pipe culvert and reconstructing the inlet and drop structures at the new upstream location.

### 6.2.2 EXCAVATION AND GROUNDWATER CONTROL

The invert of the proposed 3 cell box culvert extension, Alternative A, is similar to the existing channel invert, i.e. near elevation 85.5. Installation will generally require relatively minor excavation in the existing channel

where construction slopes of 1 horizontal to 1 vertical are recommended, but should be flattened if concentrated seepage zones and sloughing of the slopes in the granular soils develop.

The groundwater level was stabilizing near elevation 86, near the invert level. Accordingly, if construction of Alternative A is carried out during the drier months, only minor amounts of water should be encountered and conventional sump pumping should prove adequate.

Alternative B, may be achieved by open cut methods. The trench excavation would be about 8 m deep and will be carried out for the most part within hard/very dense overburden materials and weathered shale bedrock. Large hydraulic earth moving equipment will be required to excavate expeditiously the material. Construction slopes of 1 horizontal to 1 vertical in the upper 2 m and 1 horizontal to 2 vertical in the remainder of the trench are recommended, subject to geotechnical inspection.

Excavation will be taken down below the groundwater table. Several testpits should be put down at the site prior to construction to assess the quantity of water to be handled and the most suitable means of dewatering.

Tunnelling may be used to connect the existing pipes to the new drop structure. We understand that the existing concrete pipes were installed by this method and groundwater posed somewhat of a problem. It is anticipated that tunnelling will require difficult and complex construction techniques since a variety of cohesive and granular overburden materials and weathered shale bedrock will be encountered.

Consequently, we favour the open cut method over tunnelling as we believe this method will pose less of a construction problem as well as being more economical.

#### 6.2.3 BEARING CAPACITY AND BEDDING REQUIREMENT

Support of the proposed new structures in both Alternatives A and B is not expected to be of concern, since the insitu materials below the invert level of the various components are capable of supporting a load of 400 kPa.

A levelling course of at least 150 mm of Granular "A" material compacted to 95% Standard Proctor maximum dry density is recommended under precast culvert sections. Cast-in-place concrete structures may be supported directly on the native soil.

#### 6.2.4 BACKFILL AND WALL DESIGN

All backfill under the proposed roadway should be compacted to at least 95% Standard Proctor maximum dry density in uniform thin lifts to minimize settlement that would be detrimental to the performance of the pavement structure.

The backfill behind the walls of the proposed concrete box culvert extension and inlet structure, should comprise free-draining granular material to facilitate placement and compaction operations in the relatively confined space.

Earth pressure coefficient at rest,  $k_0 = 0.5$  (assuming rigid unyielding wall) and bulk unit weight  $\gamma = 21.2 \text{ kN/m}^3$  are recommended for design of the walls. In addition, the walls should be designed to resist the full hydrostatic water pressure and the surcharge due to the soil cover over the culvert and traffic loading.

Bulk fill over the box culvert (Alternative A) and in the relatively deep open cuts in Alternative B, may comprise the excavated materials, which based on our experience and general knowledge of similar types of soils are quite suitable for reuse on this project.

#### 6.2.5 ENGINEERING DISCUSSION

Extending the existing culvert with a 3 cell concrete box culvert and relocating the inlet upstream of the existing structure is more feasible than Alternative B from a geotechnical viewpoint. This would involve relatively shallow excavation and a minimum of groundwater control. Alternative B would involve deeper excavations, more materials handling and would probably encounter more severe groundwater problems.

#### 6.3 ANCILLARY CONSIDERATIONS

At the site for the proposed bridges, the results of chemical test of one soil sample (Table III) indicate a negligible degree of soluble sulphate attack on buried concrete structure, while the results of the test on a groundwater sample (Table IV) show that a positive relative degree of attack is to be expected. It is possible that the relatively high soluble sulphate concentration measured in the groundwater sample obtained at shallow depths in Borehole 2 is a result of water seeping through the railway ballast. Additional testing should be carried out prior to final design to confirm the potential for soluble sulphate attack.

The chemical test on a soil sample from the Rambo-Hager Creek culvert extension site (Table III) also indicates a negligible degree of soluble sulphate attack.

Reference is made to The Canadian Standard Association, CSA Standard A23 and The Canadian Building Digest, CBD-136 dated April 1971 for recommendation regarding the type of cement required.

PETO MacCALLUM LTD.

*Turney Lee-Bun*  
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Project Engineer.

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TLB/DWK/rf



LABORATORY TEST RESULTS

JOB NO. 80F300

TABLE I  
ATTERBERG LIMIT TEST RESULTS

PROPOSED C.N.R. OVERHEADS AT O.E.W. AND H'Y 403W - FAIRVIEW STREET RAMP  
RAMBO-HAGER CREEK CULVERT EXTENSION  
BURLINGTON, ONTARIO.

<u>BOREHOLE NO.</u>	<u>SAMPLE NO.</u>	<u>DEPTH (m)</u>	<u>NATURAL WATER CONTENT (w) %</u>	<u>LIQUID LIMIT (<sup>w</sup><sub>L</sub>)</u>	<u>PLASTIC LIMIT (<sup>w</sup><sub>p</sub>)</u>	<u>PLASTICITY INDEX (<sup>I</sup><sub>p</sub>)</u>	<u>REMARKS</u>
5	2	1.52 - 1.98	14.4	32	16	16	Silty Clay
5	7	5.33 - 5.79	11.3	24	15	9	Silty Clay
8	2	1.52 - 1.98	6.5	29	16	13	Silty Clay
9	6	4.57 - 5.03	11.9	25	13	12	Silty Clay

TABLE II  
LABORATORY UNCONFINED COMPRESSION TEST RESULTS

PROPOSED C.N.R. OVERHEADS AT Q.E.W. AND HWY. 403W - FAIRVIEW STREET RAMP  
RAMBO-HAGER CREEK CULVERT EXTENSION  
BURLINGTON, ONTARIO.

BOREHOLE NO.	SAMPLE NO.	DEPTH (m)	NATURAL	UNIT WEIGHT		VOID RATIO (e)	DEGREE	FAILURE STRAIN ( $\epsilon_f$ ) (%)	SHEAR	REMARKS
			WATER CONTENT (w) (%)	WET ( $\gamma$ ) (t/m <sup>3</sup> )	DRY ( $\gamma_d$ ) (t/m <sup>3</sup> )		OF SATURATION ( $S_r$ ) (%)		STRENGTH ( $\tau_f$ ) (kPa)	
1	6	4.57-5.03	11.7	2.23	2.00	0.35	90	4	160	Suspect sample disturbance.
4	4	3.05-3.51	14.3	2.18	1.91	0.41	93	11	290	
7	3	2.29-2.74	13.9	2.18	1.92	0.41	91	2.5	460	

JOB NO. 80F300

TABLE III  
pH VALUE AND SULPHATE CONTENT OF SOIL SAMPLES

PROPOSED C.N.R. OVERHEADS AT Q.E.W. AND HWY 403W - FAIRVIEW STREET RAMP  
RAMBO-HAGER CREEK CULVERT EXTENSION  
BURLINGTON, ONTARIO.

<u>BOREHOLE NO.</u>	<u>SAMPLE NO.</u>	<u>DEPTH (m )</u>	<u>pH VALUE</u>	<u>SULPHATE CONTENT % as SO<sub>4</sub></u>	<u>RELATIVE DEGREE SULPHATE ATTACK ON CONCRETE</u>
6	3	2.29-2.74	7.9	0.01	Negligible
8	5	3.81-4.27	8.0	0.01	Negligible



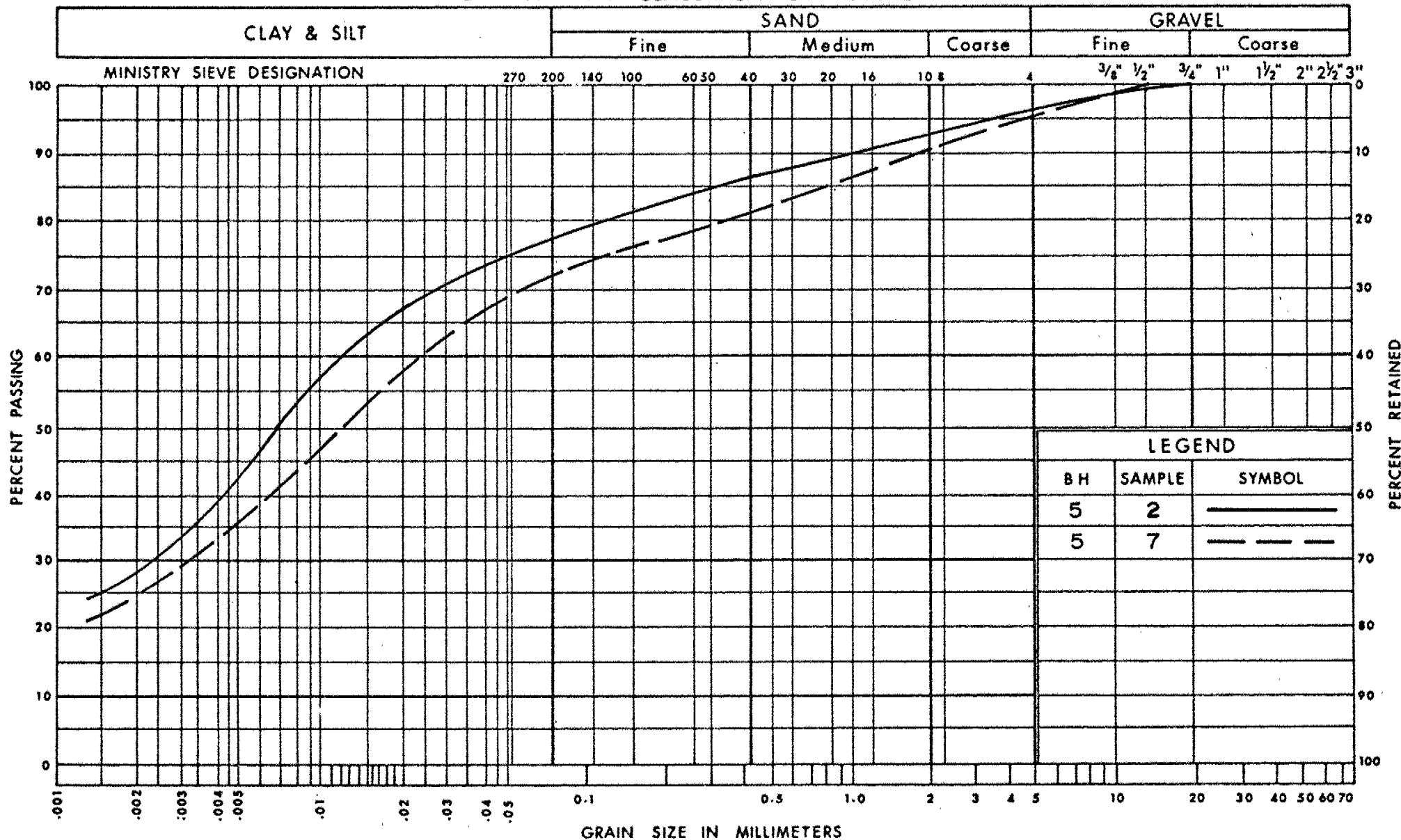
JOB NO. 80F300

TABLE IV  
pH VALUE AND SULPHATE CONTENT OF WATER SAMPLES

PROPOSED C.N.R. OVERHEADS AT Q.E.W. AND HWY. 403W - FAIRVIEW STREET RAMP  
RAMBO-HAGER CREEK CULVERT EXTENSION  
BURLINGTON, ONTARIO.

<u>BOREHOLE NO.</u>	<u>DEPTH (m)</u>	<u>pH VALUE</u>	<u>SULPHATE CONTENT ppm as SO<sub>4</sub></u>	<u>RELATIVE DEGREE SULPHATE ATTACK ON CONCRETE</u>
2	1.44	7.0	325	Positive

## UNIFIED SOIL CLASSIFICATION SYSTEM



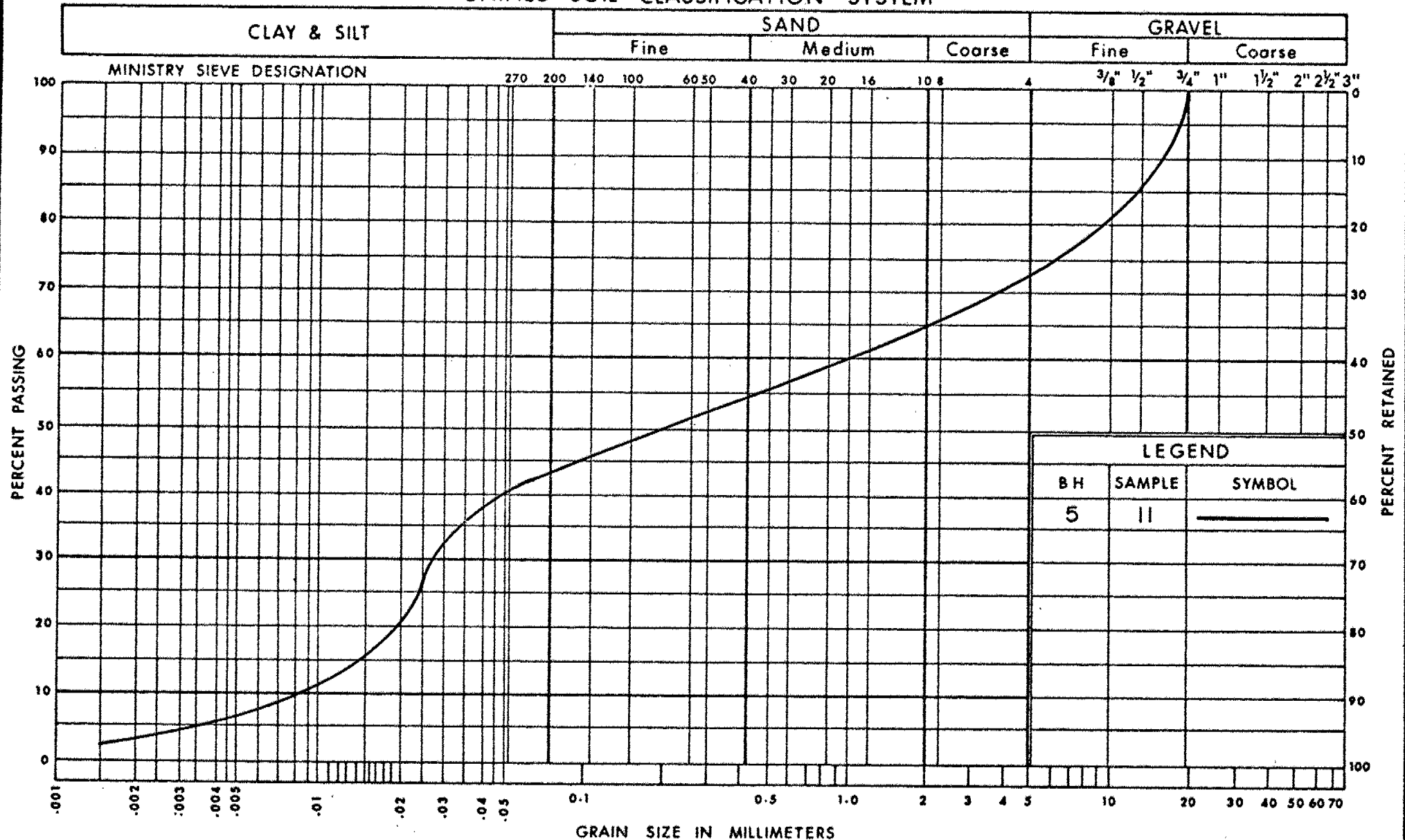
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**GRAIN SIZE DISTRIBUTION**  
**SILTY CLAY**  
SOME SAND, TRACE OF GRAVEL

FIG No 1

W P 83-74-26R &amp; 27R

## UNIFIED SOIL CLASSIFICATION SYSTEM



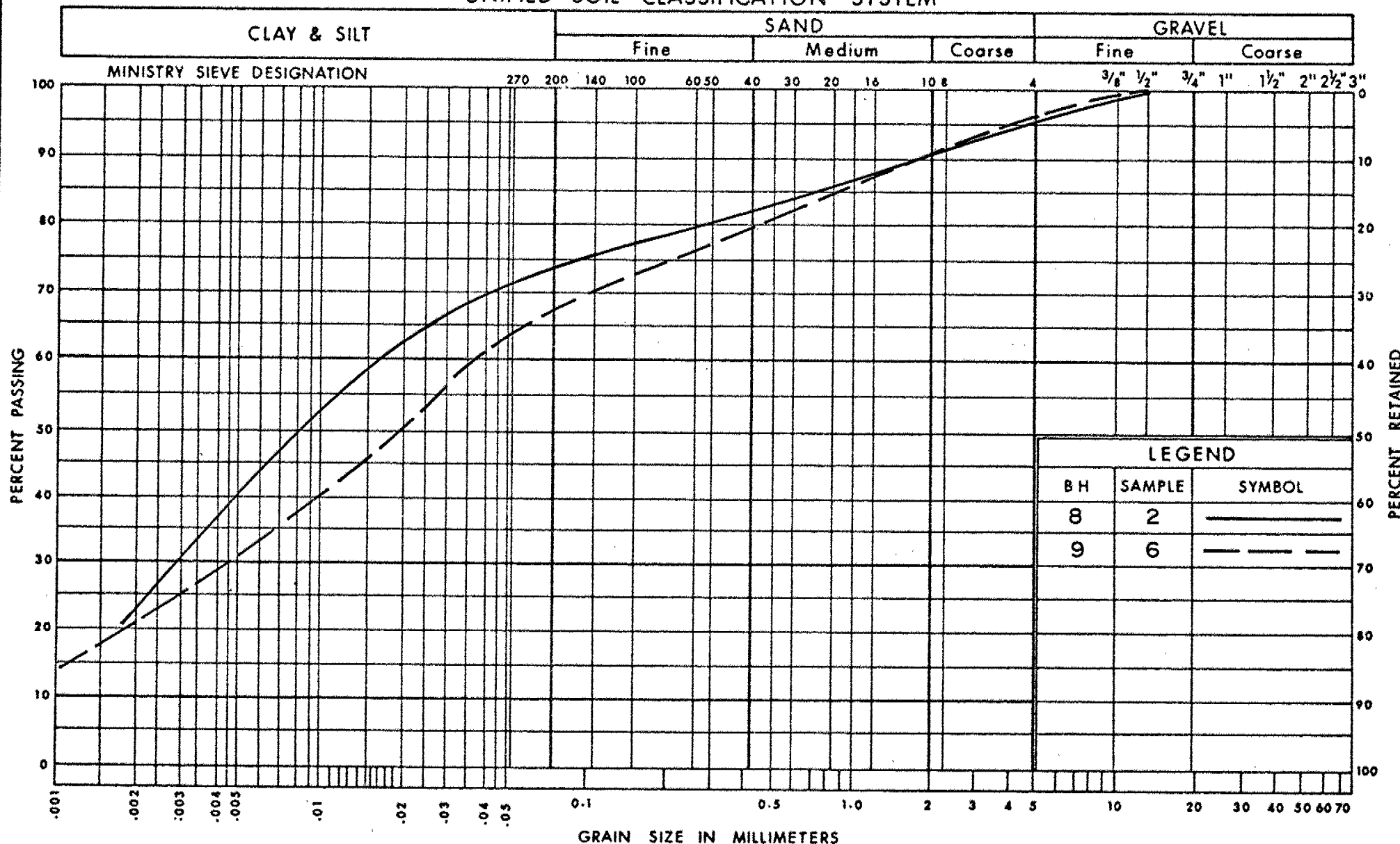
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GRAIN SIZE DISTRIBUTION  
SANDY SILT  
SOME GRAVEL, TRACE OF CLAY

FIG No 2

W P 83-74-26R & 27R

## UNIFIED SOIL CLASSIFICATION SYSTEM



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**GRAIN SIZE DISTRIBUTION**  
**SILTY CLAY**  
SOME SAND, TRACE OF GRAVEL

FIG No 3

W P 83-74-24

## EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

**JOINTING AND BEDDING:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

SS SPLIT SPOON	TP THINWALL PISTON
WS WASH SAMPLE	OS OSTERBERG SAMPLE
ST SLOTTED TUBE SAMPLE	RC ROCK CORE
BS BLOCK SAMPLE	PH TW ADVANCED HYDRAULICALLY
CS CHUNK SAMPLE	PM TW ADVANCED MANUALLY
TW THINWALL OPEN	FS FOIL SAMPLE

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$\text{kPa}^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	$\text{m}^2/\text{s}$	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
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### 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

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	$\text{kg}/\text{m}^3$	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{\min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
$\rho_w$	$\text{kg}/\text{m}^3$	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
P	$\text{kg}/\text{m}^3$	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	$\text{kg}/\text{m}^3$	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	$\text{m}^3/\text{s}$	RATE OF DISCHARGE
$\gamma_d$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{\text{sat}}$	$\text{kg}/\text{m}^3$	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{\text{sat}}$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	$\text{kg}/\text{m}^3$	DENSITY OF SUBMERGED SOIL	$e_{\max}$	1, %	VOID RATIO IN LOOSEST STATE	j	$\text{kN}/\text{m}^2$	SEEPAGE FORCE
$\gamma'$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLES



# RECORD OF BOREHOLE No 1 & 1A

Metric

W P 83-74-26R & 27R LOCATION Co-ords. 4,798,674 N; 278,215 E ORIGINATED BY M.R.  
DIST 4 HWY Q-E.W. BOREHOLE TYPE Solid Stem Auger, BXL Rock Core COMPILED BY TLB  
DATUM Geodetic DATE November 18, 1980 & January 7, 1981 CHECKED BY *OK*

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80					
98.24																
0.00	Sand, fine to medium, silty, trace of clay.		1	SS	13											
26.87	Compact Brown		2	SS	27											
1.37	Silty clay, some sand, trace of gravel, trace of clay.		3	TW	PH											
	(Glacial Till)		4	SS	36											
	Hard Brown		5	SS	29											
	Greyish Brown		6	TW	PH											
			7	SS	26											
			8	SS	55											
			9	SS	60											
			10	SS	56											
38.18																
10.06	Sand, fine to coarse, silty, some gravel.		11	SS	58											
	(Glacial Till)		12	SS	60/80 mm											
	Very dense reddish Brown															
84.68																
13.56	Bedrock Shale		13	SS	100/280 mm											
			14	BXL Rec												
	Red Weathered Sound		15	RC 100%												
30.90																
17.34	End of Borehole															
Note:																
Borehole 1			Borehole 1A													
BH 1 drilled on Nov. 18/80 was terminated at 6.55 m.			BH 1A was drilled adjacent to BH 1 on Jan. 7/81 and was terminated at 17.34 m.													
Upon completion of augering no cave, no water in open borehole.			0 to 7.62 m unsampled 1 hour after SS 13.													
Piezometer installed at elevation 92.14 Seal at elev. 95.8			Water at elevation 92.69.													
Date Water Elevation																
Nov. 22/80 Dry																
Nov. 24/80 Dry																
Dec. 1/80 Dry																



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## RECORD OF BOREHOLE No 2

Metric

W P 83-74-26R & 27R LOCATION Co-ords. 4,798,730 N; 278,248 E. ORIGINATED BY W.J.  
DIST 4 HWY O.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY TLB  
DATUM Geodetic DATE November 22, 1980 CHECKED BY AK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL ∇ Pocket Per 50 100 150 200 250	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									
99.35														
100	Fill-sandy gravel.													
97.98	Fill-clayey silt, trace of sand & gravel, occ. shale fragments. Reddish firm Brown		1	SS	7									
97.28	Sand, fine to coarse trace silt & gravel		2	SS	5									
2.07	Loose Dark Grey Silty clay, some sand, trace of gravel, occ. silt inclusions (Glacial Till)		3	SS	35									
	Hard Brown Greyish Brown		4	SS	39									
			5	SS	36									
			6	SS	38									
			7	SS	49									
92.80	red weathered shale layers and fragments below 5.2 m		8	SS	40									
6.55	End of Borehole													
	Note: Upon completion of augering cave at elevation 97.83, water at elevation 97.95. Stabilised ground- water level not established.													

+3, x5: Numbers refer to  
Sensitivity

20  
15 x 5 (%) STRAIN AT FAILURE  
10





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Ontario

# RECORD OF BOREHOLE No 3

Metric

W P 83-74-26R & 27R LOCATION Co-ords. 4,798,730 N; 278,285 E. ORIGINATED BY MR  
DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY TLB  
DATUM Geodetic DATE November 18, 1980 CHECKED BY RAK

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					
98.19																	
97.89	Silt-silty clay, trace of sand. Brown		1	SS	14												
96.51	Sand, fine to coarse silty, trace of clay & gravel. Brown		2	SS	19												
1.60	Compact. Brown		3	SS	34												
	Silty clay, some sand, trace of gravel, occ. silt inclusions. (Glacial Till)		4	SS	33												
	V. stiff to Hard Brown Greyish Brown		5	SS	25												
	occ. red weathered shale fragments below 3.7 m		6	SS	37												
91.64			7	SS	41												
6.50			8	SS	32												
	End of Borehole																
	Note: Upon completion of drilling no cave, no water in open borehole. Stabilised ground-water level not established.																

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10  
5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 4

Metric

W P 83-74-26R & 27R LOCATION Co-ords. 4,798,764 N; 278,298 E. ORIGINATED BY MR  
 DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY TLB  
 DATUM Geodetic DATE November 18, 1980 CHECKED BY JSK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
98.06																	
97.76	Topsoil																
300																	
	Silty clay, sand, trace of gravel, occasional silt inclusions.		1	SS	29												
			2	SS	24												
	(Glacial Till)		3	SS	30												
	Hard Brown		4	TW	PH												
			5	SS	36												
	Greyish Brown		6	SS	31												
			7	SS	30												
91.51			8	SS	40												
6.55	End of Borehole																
	Note: Upon completion of augering no cave, no water in open borehole. Stabilised ground- water level not established.																

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 5 & 5A

Metric

W P 83-74-26R & 27R LOCATION Co-ords 4,798,811 N; 278,285 E. ORIGINATED BY W.J.  
DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger, BXL Rock Core COMPILED BY TLB  
DATUM Geodetic DATE November 22, 1980 & January 8, 1981 CHECKED BY *AK*

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		
98.67	Topsoil							SHEAR STRENGTH						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL ▼ Pocket Per						
								WATER CONTENT (%)						
								10	20	30				
130	Silty clay, some sand, trace of gravel, occ. silt inclusions. (Glacial Till)		1	SS	21		98							4 18 50 28
			2	SS	42									
			3	SS	33									
			4	SS	32		96							
	Hard Brown		5	SS	35									
	Greyish Brown		6	SS	53		94							
			7	SS	52									
			8	SS	48									
			9	SS	65		92							
90.44	Sandy silt, some gravel, occ. cobble, trace of clay (Glacial Till)		10	SS	72/150 mm		90							
8.23	Very Reddish Dense Brown		11	SS	88		88							28 28 40 4
87.09	occasional wet sand seams		12	SS	100/25 mm		86							*no recovery
11.58	Bedrock Shale		13	BXL Rec	100%									R.Q.D.=0%
	Red Weathered Sound		14	RC	100%									R.Q.D.=68%
			15	BXL Rec	92%		84							R.Q.D.=93%
83.28	End of Borehole													
15.39	Note: Borehole 5 BH 5 drilled on Nov.22/80, was terminated at 11.13 m. After sample 10, borehole dry. Upon completion of augering cave at elevation 88.92 and water at elevation 90.07. Piezometer installed at elevation 88.92 Seal at elev. 93.79.  Date Water Elevation Nov.22/80 93.79 Nov.24/80 94.04 Dec. 1/80 94.01													
	Borehole 5A BH 5A was drilled adjacent to BH 5 on Jan.8/81 and was terminated at 15.39 m. 0 to 12.19 m unsampled. After SS 12 water at elevation 89.83.													



Ministry of  
Transportation and  
Communications  
Ontario

# RECORD OF BOREHOLE No 6

Metric

W P 83-74-26R & 27R LOCATION Co-ords. 4,798,760 N; 278,259 E. ORIGINATED BY WJ  
DIST 4 HWY O.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY TLB  
DATUM Geodetic DATE November 22, 1980 CHECKED BY WJ

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					
99.05																	
0.00	Fill-silty clay, some sand & gravel, pockets of reddish-brown soil.		1	SS	4		98										
97.68	Firm Brown		2	SS	16												
1.37	Sand, fine to coarse, silty, some gravel, trace of clay.		3	SS	41												
1.83	Compact Brown		4	SS	50												
	Silty clay, some sand, trace of gravel, occ. silt inclusions. (Glacial Till)		5	SS	42												
	Hard Greyish Brown		6	SS	44												
	occ. red weathered shale fragments		7	SS	40												
92.50	below 5.2 m		8	SS	58												
6.55	End of Borehole																
	Note: After sample 2, water at elevation 97.58. Upon completion of augering no cave, water at elevation 96.00 in open borehole.  One hour later water at elevation 98.26.  Stabilized ground-water level not established.																

+3, x5: Numbers refer to Sensitivity.

20  
15  
10  
5 (%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No 7

Metric

W P 83-74-26R & 27R LOCATION Co-ords. 4,798,729 N; 278,212 E. ORIGINATED BY WJ  
DIST 4 HWY Q.E.W BOREHOLE TYPE Solid Stem Auger COMPILED BY TLB  
DATUM Geodetic DATE November 20, 1980 CHECKED BY *WJ*

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
98.62	Fill-mixed silty clay, sand & gravel some organics. Br. and reddish brown.		1	SS	14								
300	Topsoil		2	SS	33								
510	Silty fine sand, trace of gravel. Loose to compact Brown		3	TW	PH								
840	Silty clay, some sand, trace of gravel, occ. silt inclusions. (Glacial Till) Very stiff Brown to hard Greyish Brown		4	SS	30								
			5	SS	26								
			6	SS	30								
			7	SS	43								
			8	SS	50								
	occasional red weathered shale fragments below 3.66 m		9	SS	48								
			10	SS	70								
88.41	Sand, fine to coarse silty, some gravel with coarse sand seams. (Glacial Till)												
10.21	Dense Reddish Br.		11	SS	40								
87.49													
11.13	End of Borehole												
<p>Note: After sample 10, no water in open borehole. Upon completion of augering, cave at elevation 88.26, and water at elevation 92.98. Nov. 22, 1980 water at elevation 93.18.</p>													

# RECORD OF BOREHOLE No 8

Metric

W P 83-74-24 LOCATION Co-ords. 4,797,767 N; 279,085 E. ORIGINATED BY MR  
DIST 4 HWY Q.E.W. BOREHOLE TYPE Hollow Stem Auger COMPILED BY TLB  
DATUM Geodetic DATE November 19, 1980 CHECKED BY *AK*

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20					
88.40													
0.00	Silty fine sand, trace of clay.		1	SS	27								
87.03	Compact Brown												
1.37	Silty clay, some sand, trace of gravel (Glacial Till)		2	SS	26								
85.76	V. silty, reddish to hard Brown		3	SS	55								
2.64	Sandy silt, some gravel, trace of clay.		4	SS	68								
84.74	(Glacial Till)												
3.66	V. Dense Reddish Brown		5	SS	54								
83.22	Silty clay, some sand, trace of gravel (Glacial Till)		6	SS	44								
5.18	Hard Greyish Brown		7	SS	81/280 mm								
82.61	Silt, trace of fine sand & clay.		8	SS	60/180 mm								
5.79	V. Dense Grey												
81.39	Silt, some fine sand & gravel, trace of clay.												
7.01	V. Dense Reddish Brown		9	SS	60/180 mm								
80.68													
7.72	Layered silt and weathered shale. Brown V. Dense and Red												
	End of Borehole												
Note: Upon completion of augering water at elevation 81.27 inside hollow stem augers.  Piezometer installed at elevation 80.78 Seal at elev. 81.70													
Date                      Water Elevation													
Nov. 24/80            85.95													
Dec. 1/80            85.95													



# RECORD OF BOREHOLE No 9

Metric

W P 83-74-24 LOCATION Co-ords. 4,797,740 N; 279,122 E. ORIGINATED BY MR  
DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY TLB  
DATUM Geodetic DATE November 19, 1980 CHECKED BY PLK

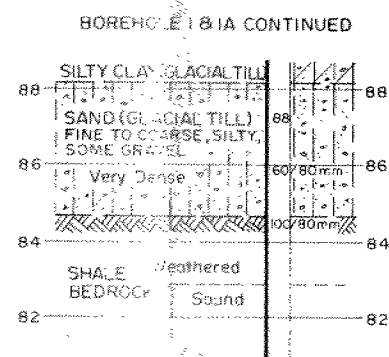
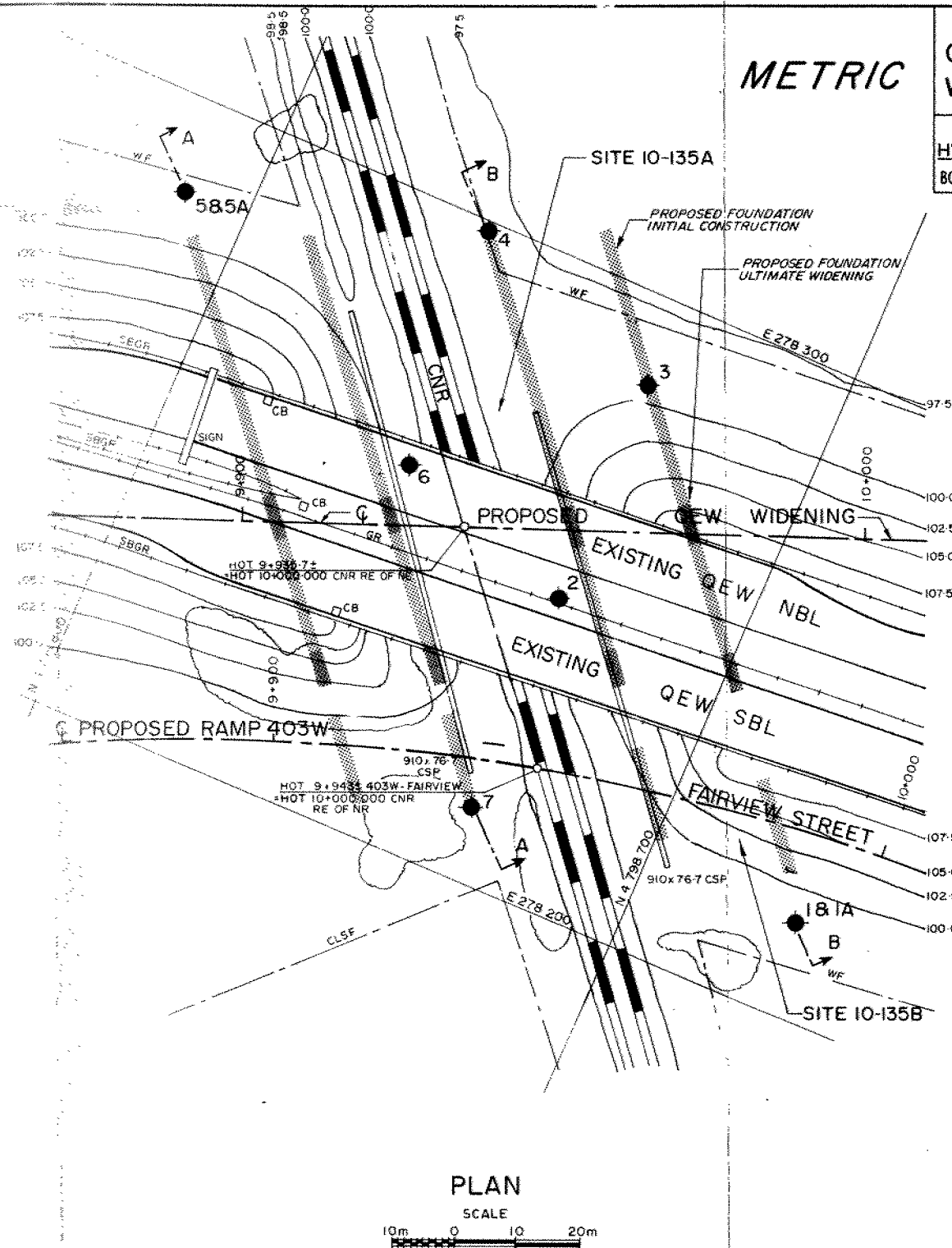
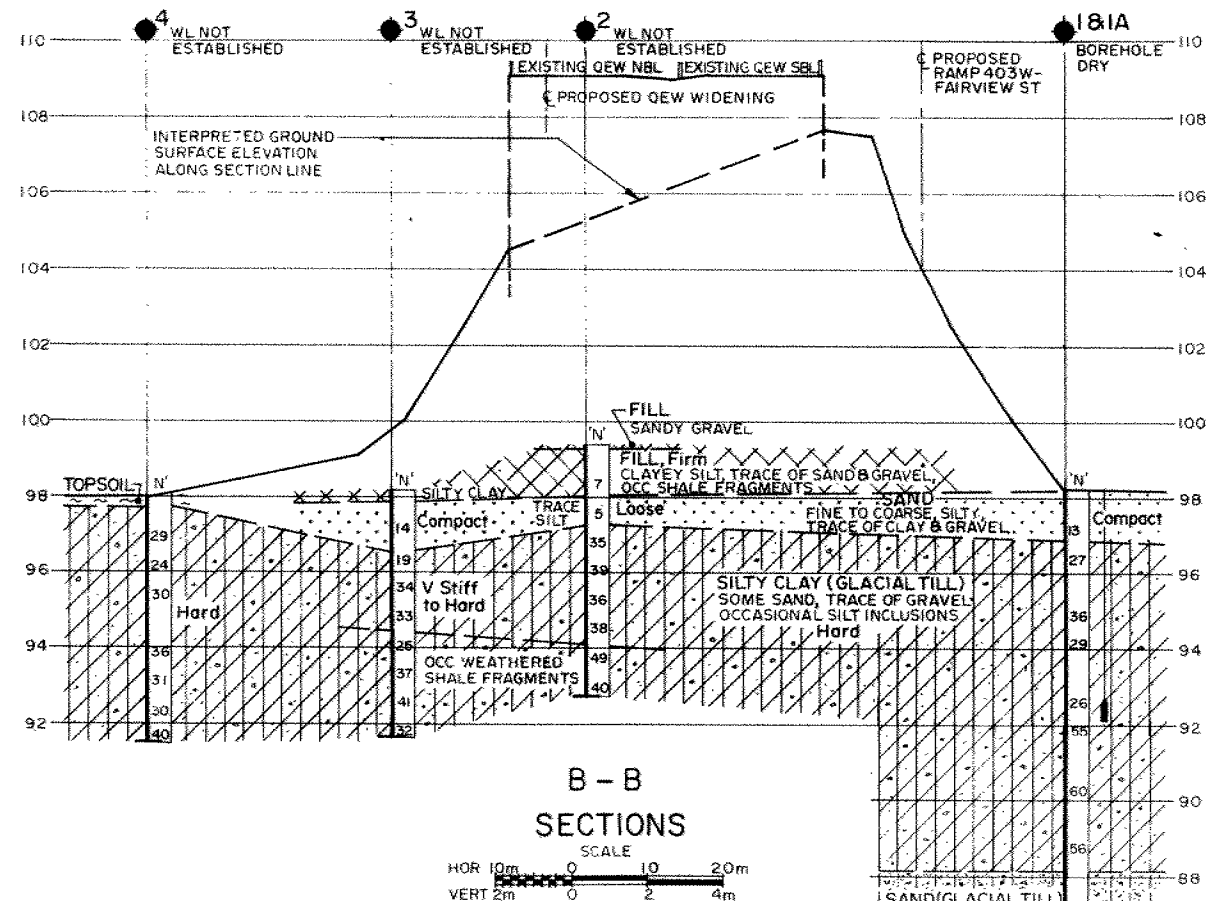
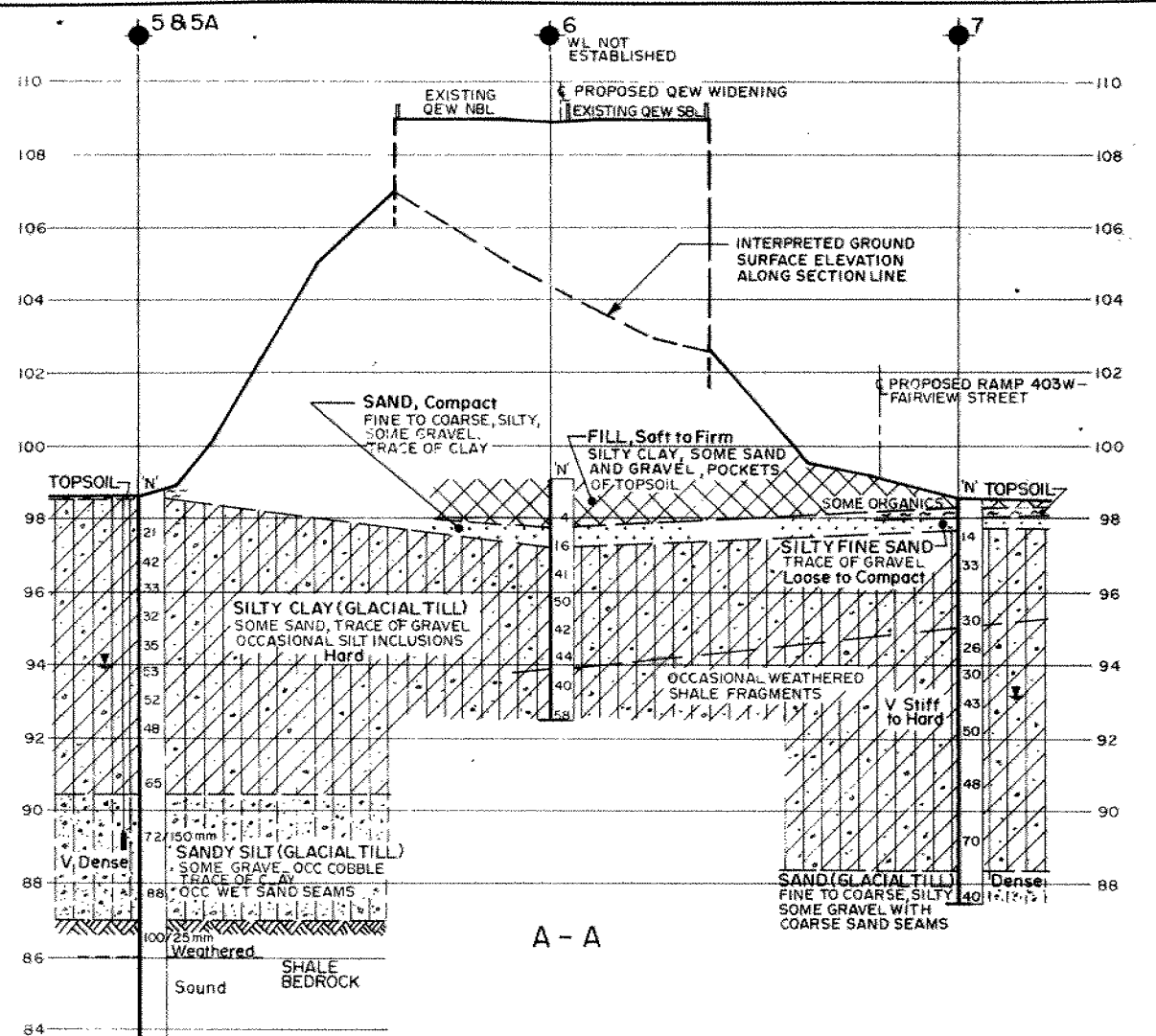
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
88.30	0.00 Till-silty clay, little sand. Brown												
300	Silty fine sand, trace of clay. Brown		1	SS	11								
86.78	Compact Brown		2	SS	21								
1.52	Silty clay, some sand, trace of gravel. (Glacial Till)		3	SS	41								
85.40	Stiff Brown to to hard Greyish Br		4	SS	62								
2.90	Sandy silt, trace of clay & gravel (Glacial Till)		5	SS	57								
84.64	Dense Reddish Br		6	SS	82								
3.66	Silty clay, some sand, trace of gravel. (Glacial Till)		7	SS	85/150 mm								
81.29	Hard Reddish Br		8	SS	50/0 mm								
7.01	Shale Bedrock		9	SS	50/0 mm								
79.13	Weathered Red												
9.17	End of Borehole												
	Note: Upon completion of augering no cave, no water in open borehole.  Stabilized ground- water level not established.												

\*3, \*5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

BOREHOLE LOCATION PLAN AND SOIL STRATA





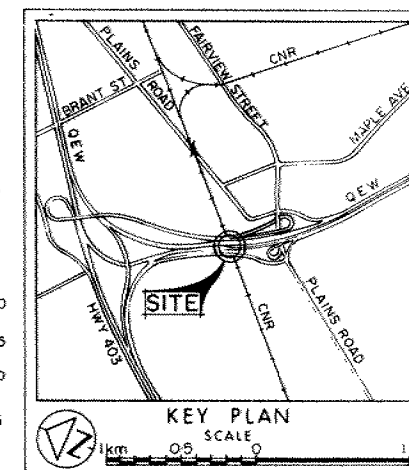
# METRIC

CONT No  
WP No 83-74-26 8






CNR OVERHEADS AT QE W &  
HWY 403 W-FAIRVIEW ST RAMP  
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

PETO MACCALLUM LTD.



### 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 NOV-DEC 1980 |
|  | PIEZOMETER                               |

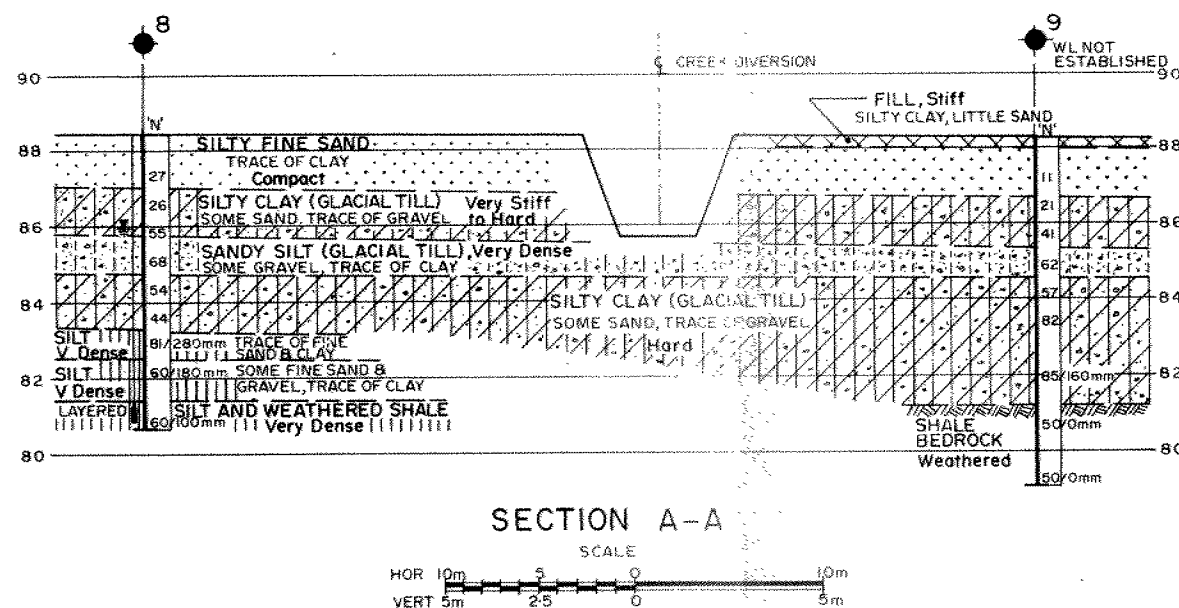
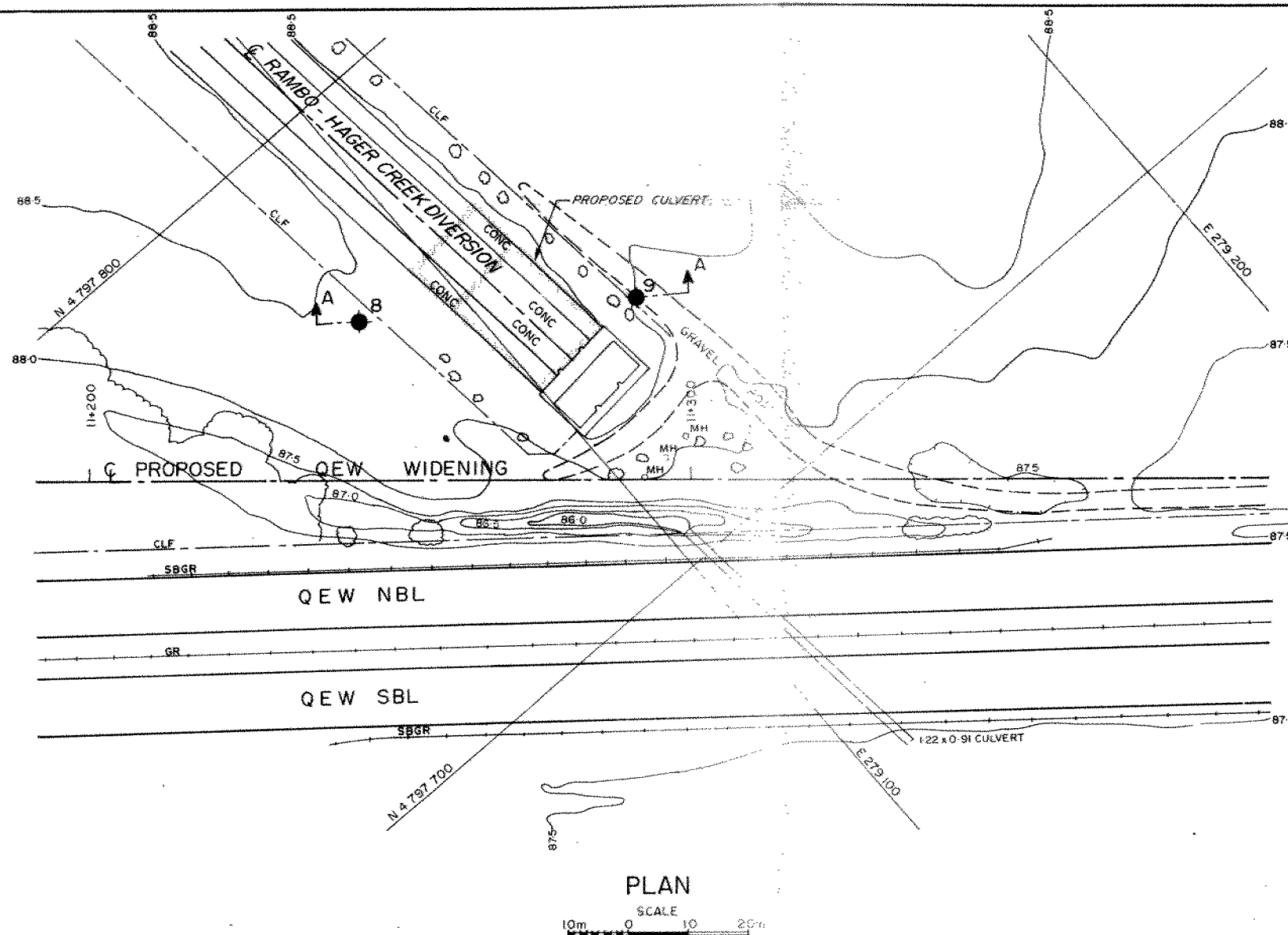
No	ELEVATION	CO ORDINATES	
		NORTH	SOUTH
1BIA	98 24	4 798 674	278 215
2	99 35	4 798 730	278 248
3	98 19	4 798 730	278 285
4	98 06	4 798 764	278 298
5B5A	98 67	4 798 811	278 285
6	99 05	4 798 760	278 259
7	98 62	4 798 729	278 212

NOTE

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

REVISIONS			
	DATE	BY	DESCRIPTION

HWY No QEW		DIST 4
SUBMIT TLB CHECKED	DATE 1981 01 16	SITE 10-35A B B
DRAWN KK CHECKED JLB	APPROVED	DWG 837426 827-



METRIC

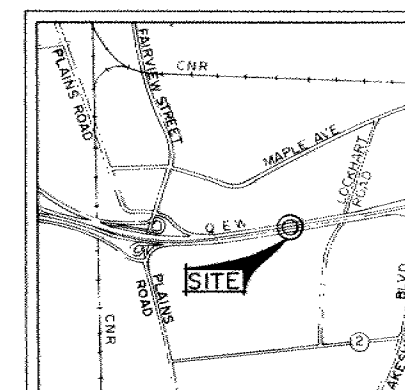
CONT No  
WP No 83-74-24

RAMBO-HAGER CREEK  
CULVERT EXTENSION  
BORE HOLE LOCATIONS & SOIL STRATA



SHEET

PETO MacCALLUM LTD.



# 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 NOV-DEC 1980
- PIEZOMETER

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
8	88.40	4 797 767	279 085
9	88.30	4 797 740	279 122

# NOTE

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

REVISIONS			
DATE	BY	DESCRIPTION	

Geocres No			
HWY No QEW		DIST 4	
SUBM'D T/LB	CHECKED	DATE 1981 01 16	SITE 10
DRAWN K.E.	CHECKED J.G.	APPROVED	DWG 837424-A