

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 30M5-178

DIST. 4 REGION

W.P. No. 199-77-07

CONT. No. 93-89

W. O. No.

STR. SITE No. 10-480

HWY. No. 403

LOCATION  Hwy 403 W.B. over  
Ramp Q.E.W. E-403 W (Bridge #34)

No of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

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**Golder Associates Ltd.**

CONSULTING ENGINEERS

REPORT

TO

MINISTRY OF TRANSPORTATION ONTARIO

**GEOTECHNICAL INVESTIGATION**

**PROPOSED BRIDGE 34**

**SITE 10-480**

**FREEMAN INTERCHANGE**

**BURLINGTON, ONTARIO**

**W.P. 199-77-07**

**DISTRICT 4**

*CONT 93-89*

*GEOCRES # 3015-178*

**Distribution:**

13 copies - Ministry of Transportation Ontario  
Downsview, Ontario

2 copies - Golder Associates Ltd.  
Hamilton, Ontario

March 1991

901-6039-3

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Record of Borehole Sheets

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Drawing WP 1997707-A

## 1. INTRODUCTION

Golder Associates Ltd. has been retained by the Ministry of Transportation Ontario (MTO) to carry out a series of site specific subsurface investigations for the design of structures for the proposed reconstruction of the Freeman Interchange in Burlington, Ontario. This report presents the results of a subsurface investigation carried out at the site of the proposed Bridge No. 34 to be constructed as part of the westbound Highway 403 lanes as shown on the Key Plan, Drawing 1997707-A.

The purpose of the investigation was to determine the subsurface conditions at the site and to provide geotechnical engineering recommendations for the design of the bridge. A proposal for carrying out the work was provided in our letter to the Ministry of Transportation Ontario (MTO) dated July 27, 1990.

## 2. SITE AND PROJECT DESCRIPTION

The proposed Bridge No. 34 will be situated between the existing QEW north - 403 west ramp and the existing North Service Road. The proposed Bridge 35 (Site 10-481, W.P. 199-77-08) will be situated parallel to and immediately to the south of Bridge 34. The existing ground surface elevation along the centreline of the proposed structure varies from about 106.5 to 109.0 metres. The location of the site is shown on the Key Plan, Drawing 1997707-A.

The site is situated within the physiographic region of

Southwestern Ontario known as the Iroquois Plain. Available geologic information indicates that the overburden in the general area of the site consists of a thin veneer of sands, glacial till and/or residual soil derived from the weathering of the underlying shale bedrock. The Queenston shale formation which comprises the bedrock in the area generally consists of thinly bedded red shale with occasional bands of grey limestone.

Bridge No. 34 will carry Highway 403 westbound traffic over Bridge No. 33, (Site 10-335, W.P. 199-77-05) the QEW east - 403 west ramp and the 403 east -QEW south ramp.

The proposed high level bridge will consist of a five span structure (spans of approximately 45, 60,60, 45 and 35 metres south to north respectively) with two abutments and 4 piers. Approach fills of some 8.5 to 9.5 metres in height will be required.

### 3. INVESTIGATION PROCEDURE

The field work for this investigation was carried out between August 16 and September 25, 1990 at which time one test pit was excavated and seven boreholes were drilled. The locations of the boreholes and test pit are shown on Drawing 1997707-A.

In addition the results of test pit 3, excavated as part of the field investigation programme for Bridge No. 33 (Site 10-335, W.P. 199-77-05) have been included in this report.

The initial stage of the field work consisted of excavating a test pit (numbered 4) near the proposed east abutment. The test pit was excavated to a depth of about 3.4 metres below the existing ground surface using a "John Deere 690" hydraulic backhoe supplied and operated by a local contractor. Chunk samples were obtained from the predominant soil strata exposed in the test pit and the test pit was loosely backfilled following sampling and logging.

The boreholes were drilled using track mounted power auger drillrigs equipped for rotary drilling supplied and operated by a specialist drilling contractor. Boreholes numbered 9, 10, 11, and 12 were advanced to bedrock, through about 2.5 to 5.2 metres of overburden, and the shale bedrock encountered beneath the overburden was core drilled in NQ size for about 3 metres in these boreholes. Borehole 9B was cored in NQ size from about 2 metres above the bedrock surface to about 3 metres below the rock surface to verify the rock/soil interface and to investigate the quality of the upper portion of the bedrock. Boreholes numbered 8 and 13 were drilled to practical auger refusal. The boreholes were advanced in the overburden and the upper portion of the bedrock using nominal 150 millimetre outside diameter hollow stem augers and nominal 100 millimetre diameter solid stem augers. Standard penetration testing and sampling was carried out within the overburden encountered in the boreholes using 35 millimetre inside diameter split spoon sampling equipment.

Samples of the overburden and the rock core recovered from the test pits and boreholes were taken to our Hamilton laboratory for examination and water content determinations.

Grain size analyses and Atterberg limit determinations were determined on selected samples of the overburden.

The soil and rock stratigraphy encountered in the boreholes and test pits are shown in detail on the Records of Boreholes and Records of Test Pits following the text of this report and on Drawing 1997707-A. The results of the field and laboratory testing are also shown on the Record of Borehole and Record of Test Pit sheets and on Figures 1 and 2.

Groundwater levels were observed in the open boreholes during drilling and in the test pits during and after excavation. Piezometers were installed in boreholes 9 and 12 as detailed on the Record of Borehole sheets. Notes pertaining to the groundwater conditions observed in the boreholes and test pits are also shown on the Record of Borehole and Test Pit sheets and on Drawing 1997707-A.

The locations and ground surface elevations at the borehole and test pit locations have been determined by Golder Associates staff with reference to site specific points and temporary bench marks provided by McCormick Rankin & Associates Limited. The final locations and ground surface elevations of the boreholes were subsequently verified by McCormick Rankin. The elevations provided are understood to be referred to geodetic datum.

#### 4. SUBSURFACE CONDITIONS

##### 4.1 General

The subsurface conditions encountered in the boreholes and test pits put down at the site are shown in detail on the Records of Borehole and Record of Test Pit sheets, and in summary form on Drawing 1997707-A. The soil boundaries and rock stratigraphy indicated, particularly for the boreholes, are inferred from non-continuous sampling and resistance to drilling advance. These boundaries typically represent a transition between one soil or rock type to another and are not intended to define an exact plane of geological change. Conditions will vary between and beyond the borehole and test pit locations.

The subsurface conditions encountered at the site generally consisted of fill, topsoil and glacial till, overlying completely to highly weathered shale and by more competent shale bedrock at depth.

The following discussion has been simplified in terms of major soil and rock strata for the purposes of geotechnical design.

It should be noted that due to the relatively soft and weathered nature of the Queenston shale formation, particularly the upper zone, together with the effects of glacial overriding at the bedrock surface, it is difficult to accurately define the bedrock surface both from a geological and a contractual standpoint.

During the course of this investigation, a stratigraphic unit directly overlying the shale bedrock, which in strictly geological terms is described as deformation till, has been encountered. This stratigraphic unit consists of an imbricate embedment of fragments of bedrock in a till matrix which has been formed from glacial overriding of the parent bedrock. This till is characterized by the presence of rounded and sub-angular clasts of the parent rock with non horizontal bedding. Based on the consistency, the relatively high penetration resistance encountered in the boreholes and the difficulty of excavation experienced in the test pits, this stratum could be contractually interpreted as a zone of the bedrock and for this reason has been referred to in this report as an upper zone of the bedrock formation.

#### 4.2 Topsoil

A layer of topsoil was encountered at the ground surface in borehole 8 and in test pits 3 and 4. A layer of topsoil was also encountered underlying the fill in borehole 10 and test pit 3.

#### 4.3 Clayey Silt, Silty Clay, occ. gravel (Fill)

Fill generally consisting of clayey silt and silty clay, was encountered in all of the boreholes and in the test pits. The fill and topsoil layers were fully penetrated in the boreholes and test pits at elevations varying between about 104.5 and 105.8 metres or at depths of about 1 to 4.4 metres below the existing ground surface. The greatest depth of fill was encountered in borehole 8. The fill at this

location probably consists of embankment fill placed in conjunction with the construction of the existing adjacent QEW south - 403 west ramp. Layers of granular fill associated with existing pavements were encountered at the ground surface in boreholes 9, 10 and 11.

The clayey fill has in-situ water contents ranging from about 14 to 31 per cent with an average in-situ water content of about 19 per cent. N-values ranging from 10 to 27 blows per 0.3 metres, were determined in standard penetration testing carried out in the clayey fill indicating a generally stiff to very stiff consistency.

4.4 Clayey Silt, trace to some sand, occ. gravel,  
occ. shale fragments (Till)

Glacial till was generally encountered beneath the fill and topsoil in all of the boreholes. In general, the till is characterized by an upper zone of very stiff to hard brown clayey silt and a lower zone of reddish brown clayey silt which is somewhat harder than the overlying till.

The N values determined in the upper till zone ranged from 19 to 54 blows per 0.3 metres. The natural water content of samples of the upper till recovered from the boreholes and test pit ranged from about 12 to 18 per cent.

Standard penetration testing was also carried out in the lower till zone, but due to its hard consistency, in most instances it was not practical to advance the sampler the entire 450 millimetres required to establish an N value. However, N values of the order of greater than 100 blows per

0.3 metres can generally be inferred from the penetration testing. The natural water content of the lower till typically ranged from about 6 to 14 per cent.

The corresponding liquid and plastic limits determined on a sample of the lower till were about 21 and 15 per cent, respectively. A grain size distribution curve for a sample (obtained by a 35 millimetre I.D. sampler) of the lower till is shown on Figure 1.

The till material as indicated by the gradation curve is relatively fine grained in nature and no major concentrations of coarse particles, such as boulders, were encountered during this investigation. This does not necessarily mean that the coarser particle sizes are not present at random or in concentrations within the deposit, since till is an inherently variable material.

#### 4.5 Shale, Completely to slightly weathered (Bedrock)

At the borehole and test pit locations, the overburden materials are underlain by shale bedrock of the Queenston Formation. Bedrock was encountered between about elevations 100.2 and 105.8 metres, or at depths of from about 2.5 to 8.7 metres below the existing ground surface.

The upper zone of the bedrock is generally highly weathered and has been described as deformation till in boreholes 8, 9, 9B, 10, 12 and 13 and in test pits 3 and 4. More competent shale was encountered in boreholes 9, 9B, 10, 11, and 12 between about elevations 100 and 104 metres, or at depths of

about 3.1 to 6.5 metres below existing ground surface. The bedrock core recovered from boreholes 9, 9B, 10, 11, and 12 generally consists of moderately to faintly weathered thinly bedded reddish-brown shale, interbedded with thinly bedded light grey, fine grained argillaceous limestone up to about 0.5 metres in thickness.

The rock core recovered from the boreholes generally exhibited a relatively high degree of fracturing. However the quality of the rock core recovered generally improved with depth. The total core recovery (TCR) ranged from about 92 to 95 per cent for the upper 1.5 metres of rock cored and 94 to 100 per cent for the lower 1.5 metres of rock. The solid core recovery (SCR) ranged from about 33 to 59 percent for the upper 1.5 metres of rock core and from about 61 to 79 percent for the lower 1.5 metres of rock core. Similarly the rock quality designation (RQD) ranged from about 12 to 43 percent in the upper 1.5 metres of rock core compared to about 34 to 75 percent for the lower 1.5 metres of rock core.

#### 4.6 Groundwater Conditions

Groundwater was not encountered during the field drilling/digging operations in borehole 13 which was terminated at auger refusal, or in test pit 4 which was terminated in the red (lower) clayey silt till. Borehole 10 was dry to the depth of practical auger refusal, corresponding to about elevation 102.3 metres. Groundwater was encountered between about elevations 103.8 and 105.3 metres, or from about 1.5 to 5.2 metres below the existing ground surface in boreholes 11 and 8 respectively. The groundwater levels were measured at

about elevations 103.1 and 105.0 metres in the piezometers installed in boreholes 9 and 12 respectively. These water levels, recorded about 1 week after the completion of drilling, correspond to depths of about 3.5 and 2.2 metres below the existing ground surface. Water levels at elevations 102.9 and 104.6 metres were measured in the piezometers in boreholes 9 and 12 respectively about 1 month after installation.

It should be noted that the piezometric groundwater level within the subsoil and underlying bedrock is subject to fluctuation not only due to precipitation conditions, but also due to seasonal variations. The water levels given above may not necessarily reflect stabilized conditions and may vary from the conditions which are encountered during construction.

## 5. DISCUSSION AND DESIGN RECOMMENDATIONS

### 5.1 General

This section of the report provides our interpretation of the factual geotechnical data obtained during the investigation. The geotechnical engineering parameters given below are intended for design purposes only. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors bidding on or undertaking the works should make their own interpretation of the subsurface information contained herein as it affects their proposed construction methods, equipment selection, scheduling and the like.

### 5.2 Foundations

#### 5.2.1 General

It is understood that the proposed finished deck grade will vary in elevation from about 117.4 metres at the south abutment to about 119.7 metres at the north abutment. The finished grade beneath the bridge will range from about 105.0 metres at the centre of the bridge to about 110.7 metres where Bridge 33 is crossed.

It is understood that the pier and abutment foundations for Bridges 33, 34, and 35 which are in close proximity to each other will be built during the same construction phase. This will minimize the impact of subsequent construction on completed foundation units as well as minimizing the

disruption to traffic.

The results of the present investigation indicate that the lower till deposit or underlying bedrock is suitable for conventional spread footing support for the proposed piers and abutments. Consideration may also be given to supporting the abutments on caissons socketed into bedrock or on spread footings placed on engineered fill.

#### 5.2.2 Spread Footings

The piers and abutments may be founded on spread footings bearing in the undisturbed hard lower glacial till, or in the underlying bedrock, using a factored bearing capacity of 750 kilopascals at Ultimate Limit States and 500 kilopascals at Serviceability Limit States. Based on the results of the boreholes, the anticipated founding elevations for the construction of spread footings as outlined above are summarized below.

FOUNDATION UNIT	LOCATION	FOUNDING ELEVATION (m)
South Abutment	STN 16+851	103 or lower
Pier No. 1	STN 16+896	104 or lower
Pie No. 2	STN 16+856	104 or lower
Pier No. 3	STN 17+016	104 or lower
Pier No. 4	STN 17+058	104.5 or lower
North Abutment	STN 17+096	104.5 or lower

The coefficient of friction between the concrete and the

bearing strata may be computed by assuming an internal friction angle of  $26^{\circ}$  for the glacial till and  $20^{\circ}$  for the shale (unfactored).

It is considered that higher design bearing pressures would be appropriate for footings constructed on rock at lower elevations than outlined above. However, construction of spread footings on the till strata is considered preferable since problems associated with the adequate control of groundwater seepage are not likely to be as severe at higher elevations.

In order to achieve the design bearing pressures, and minimize post construction settlements, it is essential that all material at the founding level which is loosened, softened or disturbed during construction excavation be removed from the base of the excavation. For the case of the footings constructed on bedrock, the shale is highly weathered and fractured and is particularly susceptible to weathering from exposure to air, to softening by water, and to any significant construction traffic. Effective control of groundwater during construction will therefore be a critical aspect to preserving the integrity of the bedrock surface.

For all spread footings it is recommended that a minimum 150 millimetre thick protective layer of lean concrete be placed within 4 hours of final excavation. To be of optimal benefit, the removal of loosened and disturbed material and the placement of the protective layer should be carried out on a simultaneous basis. The foundation excavations should

be inspected by experienced geotechnical personnel prior to any concrete placement to confirm that the base has been adequately prepared and is free of any softened or disturbed zones.

Care and adequate protection during winter construction should be provided to prevent any freezing of the foundation subgrade.

For design purposes a minimum soil cover of 1.2 metres below final grade should be provided to the footing base.

#### 5.2.3 Deep Foundations

Some 4.4 metres of fill was encountered in borehole 8, drilled at the proposed south abutment for Bridge 34. Due to the depth of the existing fill, as well as the space limitations which may exist at the abutment locations, the use of deep foundations consisting of caissons socketed into the bedrock may be considered as an alternative to spread footings.

The caissons should have a minimum length to diameter ratio of 3 within the bedrock and should be socketed at least 1 metre into the moderately weathered shale bedrock. The caissons may be designed using an end bearing factored capacity at Ultimate Limit States of 2800 kilopascals.

Serviceability limit states is not relevant to caissons founded on bedrock since the stresses required to produce detrimental settlement will be larger than the value given

for the factored bearing capacity at ULS.

The caissons must be suitably lined and should have a minimum diameter of 900 millimetres to permit 'down the hole' inspection of the socket prior to concreting.

An ultimate axial load bearing capacity of 2200 kN may be assumed for caissons founded as outlined above and with tip elevations of 102.5 and 97 metres or lower for the north and south abutments respectively.

It is anticipated that some water bearing zones within the bedrock may be encountered. In instances where significant seepage volumes are encountered, it may be necessary to place the concrete using specialized "tremie" techniques.

#### 5.2.4 Perched Abutments

The finished grade at the abutments will vary from about elevation 117.4 to about elevation 119.7 metres compared to the existing grades of 108.9 and 109.1 metres at the south and north abutments respectively. Consideration could also be given to constructing perched abutments founded on spread footings bearing on a properly engineered fill as shown schematically on Figure 3. Conceptually, the design consists of a core of well compacted granular fill which will support the foundations and a shell of compacted earth fill.

In preparation for constructing the engineered fill, it will be necessary to excavate all of the existing fill and topsoil and expose a competent native subgrade beneath areas of

"structural fill". Based on the results of the present investigation, it will probably be necessary to excavate to about elevation 104.5 metres at the south abutment and to about elevation 105.8 at the north abutment. The native subgrade should be adequately proofrolled under the direction of the geotechnical engineer. Remedial work should be carried out on any soft or loose zones identified during proofrolling.

The engineered fill should be placed and compacted in accordance with the applicable MTO directives.

Design bearing pressures of 350 KPa Serviceability Limit States and 900 KPa Ultimate Limit States may be assumed for perched abutments founded on engineered fill constructed as outlined above and to the current MTO specifications.

Abutment footings placed on engineered fill resting on the native subsoil at the site will experience settlement with time. The magnitude of the settlement is dependent on the thickness of the fill, the thickness of subsoil and the degree of compaction of the engineered fill. This has to be taken into consideration in the context of differential movements with the adjoining pier foundation and the flexibility of the bridge span to accommodate this movement.

For design purposes, differential settlements in the range of about 20 to 40 millimetres should be anticipated. The magnitude of the actual differential settlements will be affected by the bridge spans and the soil/structure interaction. In addition, poor construction techniques could

result in greater settlements.

Due to the depth of the existing fill which would have to be completely removed prior to constructing the engineered fill, this alternative would result in a relatively large scale earth moving operation which may not be feasible given the spacial constraints which are likely to exist at the abutment locations.

### 5.3 Earth Pressures for Abutments

The lateral earth loads acting on the abutments will depend on the rigidity of the structure, the type and method of placement of the backfill materials, the nature of the soils behind the backfill and the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments.

- Select free draining granular fill in accordance with OPSS Granular "A" or "B" should be used as backfill immediately behind the abutments. The minimum limits of the granular backfill are outlined in section 6 - 9.6.1 of the Ontario Highway Bridge Design Code.
- All granular fill should be placed and compacted in accordance with the current OPSS practices and MTO directives.
- Longitudinal drains should be installed to provide positive drainage of the granular backfill

- If the abutment support allows lateral yielding at its top equal to not less than 0.05 per cent of the retained height, 'active' earth pressure conditions apply. If, however, the structure is not permitted to yield by this amount, 'at rest' pressure conditions should be used. The following parameters may be used for the calculation of lateral earth pressures in accordance with the Ontario Highway Bridge Design Code:

Granular "A"		
Unit weight		22.8 kN/m <sup>3</sup>
$\phi$		35°
Granular "B"		
Unit weight		21.2 kN/m <sup>3</sup>
$\phi$		30°

#### 5.4 Excavations

Based on the results of the current investigation, excavations for the foundations will encounter fill and topsoil, glacial till, and shale bedrock.

Groundwater seepage from the sandy fill layers, from permeable layers within the till and from fractured zones within the bedrock should be anticipated.

As discussed previously, adequate control of the groundwater during construction is essential to minimize deterioration of the bearing strata during final excavation, preparation of the bearing surface for placement of the lean concrete and prior to mass foundation concreting operations. In addition, the stability of the excavation side slopes may be influenced by the adequacy of the groundwater control measures.

All excavations must conform to the current Occupational Health and Safety Act and care should be taken to direct surface runoff away from open excavations. Excavation side slopes within the overburden should not exceed an inclination of 1 horizontal to 1 vertical unless adequate temporary support is provided. Some sloughing may occur in localized zones due to groundwater seepage from relatively permeable strata within the overburden slopes. For the relatively shallow depths of cut anticipated below the bedrock surface, (<2 metres) near vertical side slopes are appropriate for temporary excavations in rock. The condition of the excavation side slopes should be monitored on a routine observation basis as some ravelling of the slopes will develop due to deterioration of the exposed rock.

#### 5.5 Embankments

It is understood that approach fills of about 8.5 to 9.5 metres will be required at the abutment locations. The subgrade encountered in the boreholes drilled at the proposed south abutment location consists of about 4.4 metres of clayey silt fill overlying the till. At the east abutment location about 3.3 metres of clayey silt fill overlies the till. The clayey silt fill had a stiff to very stiff consistency and an average in-situ water content of about 19 percent.

In the event that the embankment fill can be constructed well in advance of the pavements and some post construction deformation of the roadway can be tolerated, consideration could be given to leaving the existing fill in place. The

existing fill subgrade should be stripped of any topsoil or highly organic and adequately proofrolled under the direction of the geotechnical engineer in preparation for constructing the embankments. Remedial work should be carried out on any soft or loose zones encountered.

Following completion of the preparatory works outlined above, the embankment fill should be constructed consistent with the current OPSS practices and MTO directives.

Settlement of the embankments constructed as outlined above is estimated to be in the range of about 80 to 120 millimetres. In addition differential settlements of about 50 per cent of the above range should be anticipated due to consolidation of the existing fill beneath the proposed embankments. If settlements of this magnitude are not tolerable, consideration should be given to removing all of the existing fill prior to constructing the embankments as a means of reducing the differential settlement. Much of the existing earth fill is considered suitable for reuse in the construction of earth embankment fill.

For embankment heights of greater than 8 metres, overall side slope inclinations of 2.5 horizontal to 1 vertical or flatter are recommended. In localized areas, such as in front of abutments, where good control of runoff can be achieved, inclinations as steep as 2 horizontal to 1 vertical may be permitted. For embankment heights of less than 8 metres, 2 horizontal to 1 vertical slopes may be considered. The completed embankment side slopes should be blanketed with an appropriate vegetation cover.

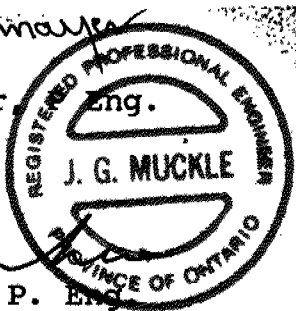
Differential settlement in the range of about 10 to 20 millimetres can be expected between an abutment footing founded on the till below the approach embankment, and the adjacent piers also resting within the lower till deposit. If the bridge structure is unable to tolerate any differential movements, consideration will have to be given to founding the abutments within the bedrock, or on caissons taken into the bedrock.

It should be brought to the attention of the contractor by way of a special provision to the contract, that the test pits put down in this investigation and in any other preconstruction investigations, should be located in the field to determine whether they are within the embankment alignment. Test pits within the embankment areas should be re-excavated and backfilled with appropriate material placed and compacted as outlined above.

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APPENDIX "A"

## EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

**JOINTING AND BEDDING:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### 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
$\phi'$	$^\circ$	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	$^\circ$	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
$u$	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
$\rho$	$\text{kg}/\text{m}^3$	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 BOREHOLE No 8

METRIC

W P 199-77-07 LOCATION Co-ordinates N4,799,471.2 E277,794.3 ORIGINATED BY VCH  
 DIST 4 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY HHW  
 DATUM Geodetic DATE August 20, 1990 CHECKED BY VCH

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					
108.9	Ground Surface																GR SA SI CL
0.1	Topsoil																
108.1	Clayey Silt (Fill) tr sand, occasional gravel Red Brown																
0.8	Silty Clay (Fill), tr sand and organics, occasional gravel		1	SS	18		108										
			2	SS	18												
			3	SS	19												
	Stiff to very stiff		4	SS	12		106										
105.2	Reddish Brown																
3.7	Clayey Silt (Fill) trace sand, occ. gravel		5	SS	27												
104.5	Very Stiff Red Brown																
4.4	Clayey Silt (Till) trace sand, occ gravel and shale fragments		6	SS	54		104										
			7	SS	130		103.8										
			8	SS	65/50mm												
			9	SS	55/25mm		102										
	Brown becoming reddish brown at about elev. 103.7		10	SS	70/50mm												
100.2	Hard																
8.7	Shale (Deformation Till)		11	SS	75/75mm		100										
99.7																	
9.2	End of Borehole																
							98										

# RECORD OF BOREHOLE No 9

METRIC

W P 199-77-07 LOCATION Co-ordinates N4,799,511.3 E277,814  
 DIST 4 HWY 403/QEW BOREHOLE TYPE Hollow Stem Augers, NQ Rock Core  
 DATUM Geodetic DATE August 20, 1990  
 ORIGINATED BY CB  
 COMPILED BY MRW  
 CHECKED BY VCH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
106.6	Ground Surface																
0.0	Sand (Fill)																
106.0	Loose Brown																
0.6	Clayey Silt (Fill) tr sand; occ. pebbles		1	SS	19		106										
105.4	Very Stiff Brown																
1.2	Clayey Silt trace to some sand occ. gravel and shale fragments. (Till)		2	SS	19												
			3	SS	118/ 250mm		104										
			4	SS	50/75mm												
			5	SS	91/150mm												
	Very stiff to Hard		6	SS	70/50mm		102										
101.4	Reddish Brown																
5.2	Shale (Deformation Till) Completely to highly weathered		7	SS	97/125mm												
			8	SS	105/125mm												
100.1	Reddish Brown						100										
6.5	Probably highly to moderately weathered shale																
99.0	Reddish Brown																
7.6	Shale Bedrock moderately to slightly weathered thinly to medium bedded Reddish Brown interbedded with fine-grained argillaceous limestone		9	NQ RC	TCR= 94% SCR= 44% RQD= 29%		98										
			10	NQ RC	TCR= 100% SCR= 68% RQD= 38%												
95.9							96										
10.7	End of Borehole																
	* TCR = Total Core Recovery SCR = Solid Core Recovery RQD = Rock Quality Designation						94										



# RECORD OF BOREHOLE No 10

METRIC

W P 199-77-07 LOCATION Co-ordinates N4,799,574 E277,841 ORIGINATED BY CB  
 DIST 4 HWY 403/QEW BOREHOLE TYPE Hollow Stem Augers, NQ Rock Core COMPILED BY MHW  
 DATUM Geodetic DATE August 17, 1990 CHECKED BY VCA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
107.2	Ground Surface																
0.0	Granular road base (Fill)																
106.5	Grey																
0.7	Clayey Silt (Fill) trace sand, occ. gravel		1	SS	15		106										
105.5	Very Stiff Brown																
1.7	Silty Topsoil		2	SS	30												
2.0	Clayey silt trace sand, occasional gravel and shale fragments. (Till)		3	SS	95/200mm												
			4	SS	50/150mm		104										
	Reddish Brown		5	SS	60/75mm												
102.9	Hard																
4.3	Shale (Deformation Till) completely to		6	SS	70/125mm												
102.3	highly weathered Reddish Brown																
4.9	Probably Shale Bedrock highly to moderately weathered						102										
101.1	Reddish Brown																
6.1	Shale Bedrock moderately to slightly weathered thinly to medium bedded Reddish brown interbedded with fine grained argillaceous Limestone		7	NQ RC	*TCR=95% *SCR=52% *RQD=42%		100										
			8	NQ RC	*TCR=97% *SCR=61% *RQD=34%												
97.9							98										
9.3	End of Borehole																
	*TCR=Total Core Recovery *SCR=Solid Core Recovery *RQD=Rock Quality Designation						96										

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 11

METRIC

W P 199-77-07 LOCATION Co-ordinates N4,799,623.5 E277,872.7 ORIGINATED BY CB  
 DIST 4 HWY 403/QEW BOREHOLE TYPE Hollow Stem Augers, NQ Rock Core COMPILED BY MHW  
 DATUM Geodetic DATE August 16, 1990 CHECKED BY VCM

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								WATER CONTENT (%)
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE								
106.8	Ground Surface															
0.0	Granular Road Base (Fill)															
106.2	Grey Sand (Fill), medium to coarse grained, trace silt, occ. gravel		1	SS	10		106									
104.9	Brown Compact Clayey silt (Till) trace to some sand occasional gravel		2	SS	19											
1.9	Reddish Brown Hard		3	SS	50/75mm		104									
103.1	Shale Bedrock, moderately to highly weathered		4	SS	50/150mm											
3.7	Shale Bedrock moderately to slightly weathered thinly to medium bedded Reddish Brown inter-bedded with fine grained argillaceous Limestone		5	SS	50/25mm		102									
102.2			6	NQ RC	TCR=95% SCR=59% RQD=43%											
4.6			7	NQ RC	TCR=94% SCR=79% RQD=75%		100									
99.0																
7.8	End of Borehole															
	*TCR=Total Core Recovery SCR=Solid Core Recovery RDQ=Rock Quality Designation						98									

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 12

METRIC

W P 199-77-07 LOCATION Co-ordinates N4,799,660.9 E277,690 ORIGINATED BY CB  
 DIST 4 HWY QEW/403 BOREHOLE TYPE Hollow Stem Augers: NO Rock Core COMPILED BY MHW  
 DATUM Geodetic DATE August 23, 1990 CHECKED BY VCA

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				
107.2	Ground Surface															GR SA SI CL
0.0	Sand (Fill)															
106.6	Brown															
0.6	Silty clay (Fill) trace sand and organics occasional gravel		1	SS	15		106									
			2	SS	10											
105.1	Stiff Dark Brown															
2.1	Clayey silt (Till) some sand															
104.7	Hard Reddish Brown		3	SS	85/125 mm											
2.5	Shale (Deformation Till) completely wethr															
104.1			4	SS	50/75 mm		104									
3.1	Shale bedrock moderately to slightly weathered thinly to medium bedded Reddish Brown Interbedded with fine grained grey argillaceous limestone		5	NQ RC	TCR=92% SCR=33% RQD=12%											
			6	NQ RC	TCR=100% SCR=68% RQD=47%		102									
101.0																
6.2	End of Borehole															
	* TCR: Total Core Recovery  SCR: Solid Core Recovery  RQD: Rock Quality Designation						100									

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 13

METRIC

W P 199-77-07 LOCATION Co-ordinates N4,799,687.6 E277,921.4 ORIGINATED BY CB  
 DIST 4 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY MW  
 DATUM Geodetic DATE August 15, 1990 CHECKED BY VCM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
107.2	Ground Surface																
0.0	Clayey Silt (Fill) trace sand and organics occasional gravel		1	SS	13		106										
105.2	Brown to Stiff Reddish Brown		2	SS	14												
2.0	Clayey silt trace to some sand, occasional gravel. (Till)		3	SS	60/150mm												
104.2	Hard Red Brown		4	SS	120		104										
3.0	Shale (Deformation Till Completely to moderately weathered Reddish Brown		5	SS	>50/25mm												
103.6	Shale Bedrock moderately to highly weathered		6	SS	>50/25mm												
102.7	Reddish Brown																
4.5	End of Borehole						102										

OFFICE REPORT ON SOIL EXPLORATION



# RECORD OF TEST PIT No. 3

METRIC

W P 199-77-05

LOCATION Co-ordinates N4,799,534 E277,833

ORIGINATED BY VCH

DIST 4 HWY 403/QEW

BOREHOLE TYPE Backhoe Dig; John Deere 690

COMPILED BY MW

DATUM Geodetic

DATE August 16, 1990

CHECKED BY VCH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	Wp	W	WL						
106.1	Ground Surface																
0.0	Topsoil						106										
0.3	Silty Clay (Fill) trace sand occ. gravel																
105.2	Brown																
0.9	Topsoil																
1.1	Clayey Silt (Till) trace sand, occ. gravel		1	CS													
	Brown to Red Brown																
102.7																	
3.4	Shale (Deformation Till)																
3.7	Bottom of Pit						102										

OFFICE REPORT ON SOIL EXPLORATION



## METRIC

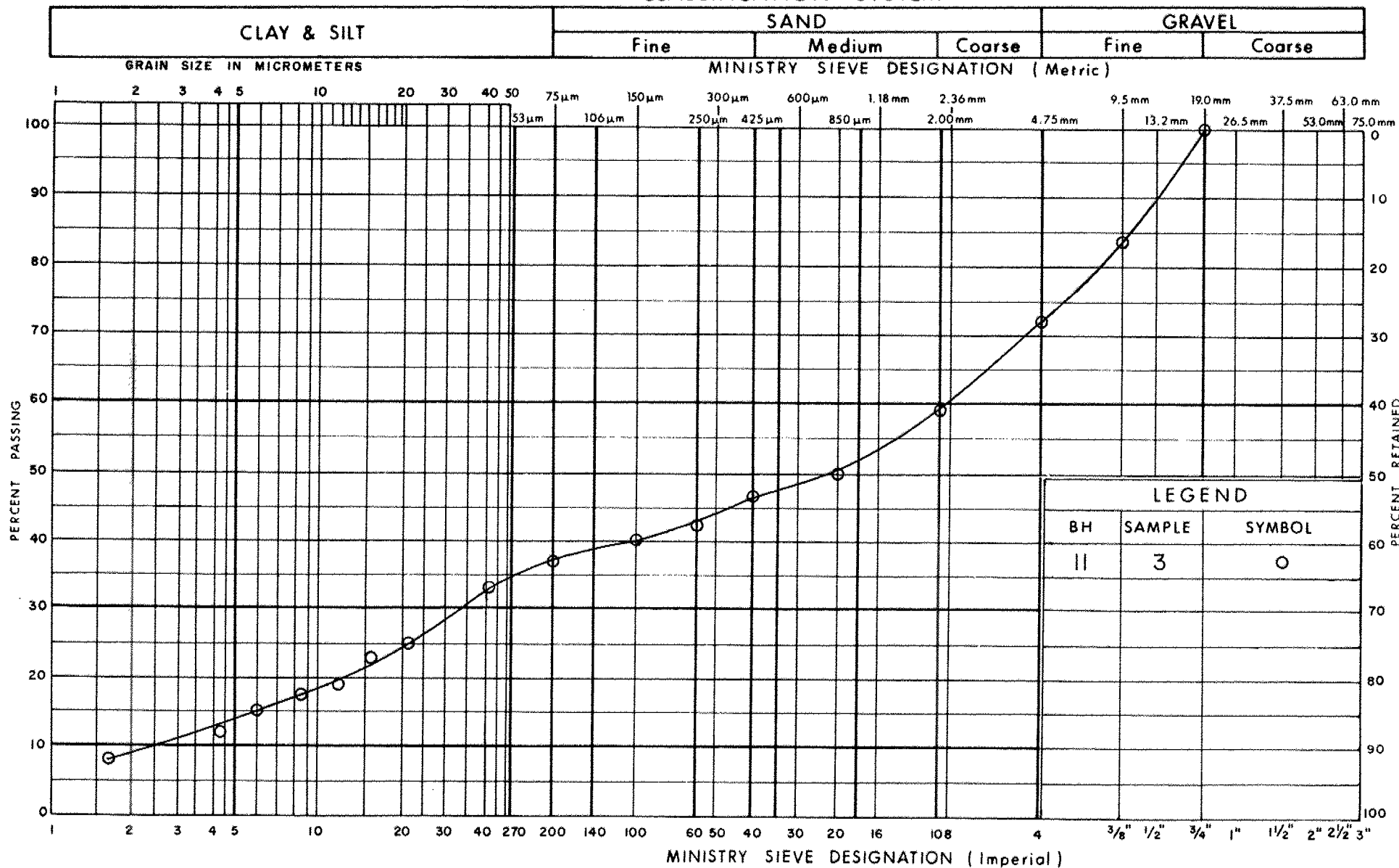
W P 199-77-07 LOCATION Co-ordinates N4,799,691 E277,898.5 ORIGINATED BY VCH  
DIST 4 HWY 403/QEW BOREHOLE TYPE Backhoe Dug; John Deere 690 COMPILED BY MHW  
DATUM Geodetic DATE August 16, 1990 CHECKED BY VCH

[illegible]

+3, x<sup>5</sup>: Numbers refer to Sensitivity

OFFICE REPORT ON SOIL EXPLORATION

## UNIFIED SOIL CLASSIFICATION SYSTEM



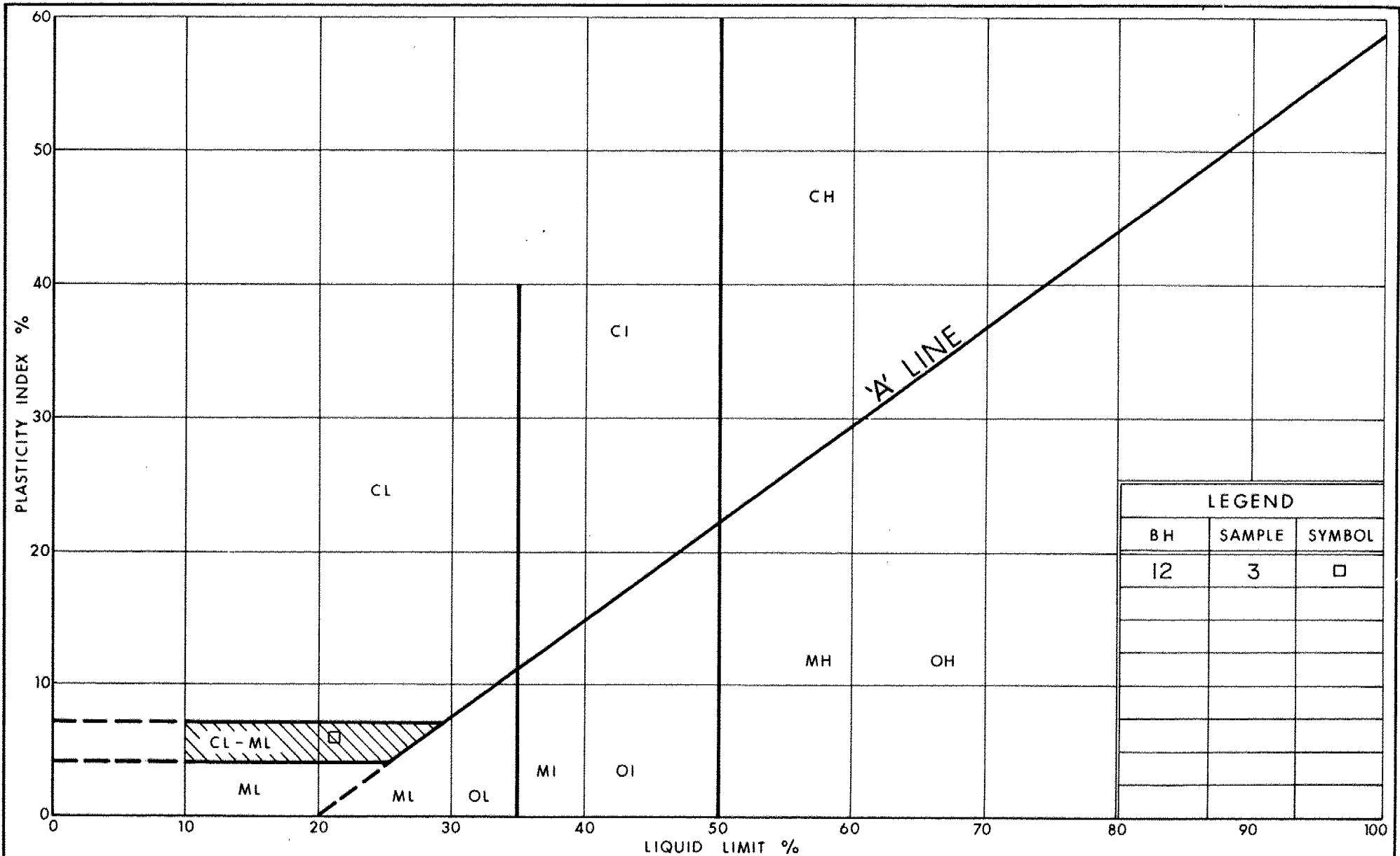
Ministry of  
Transportation

# GRAIN SIZE DISTRIBUTION

## CLAYEY SILT TILL

FIG No 1

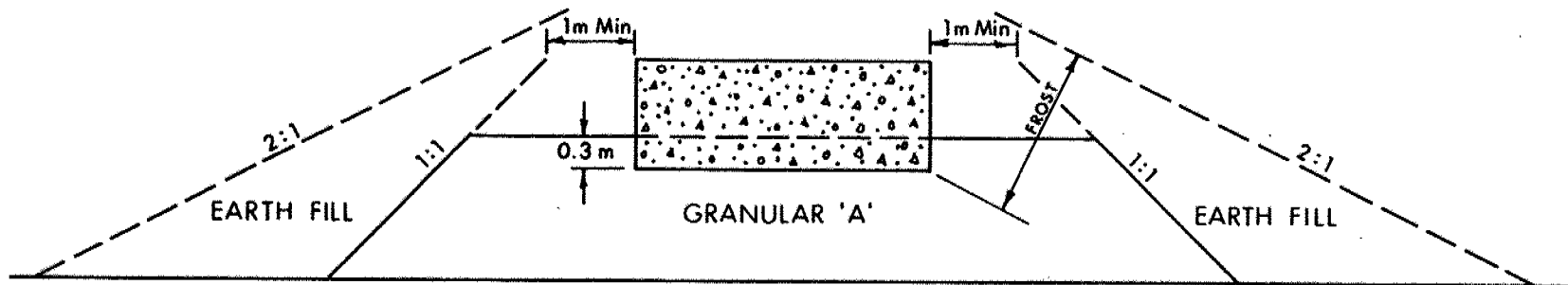
W P 199-77-07



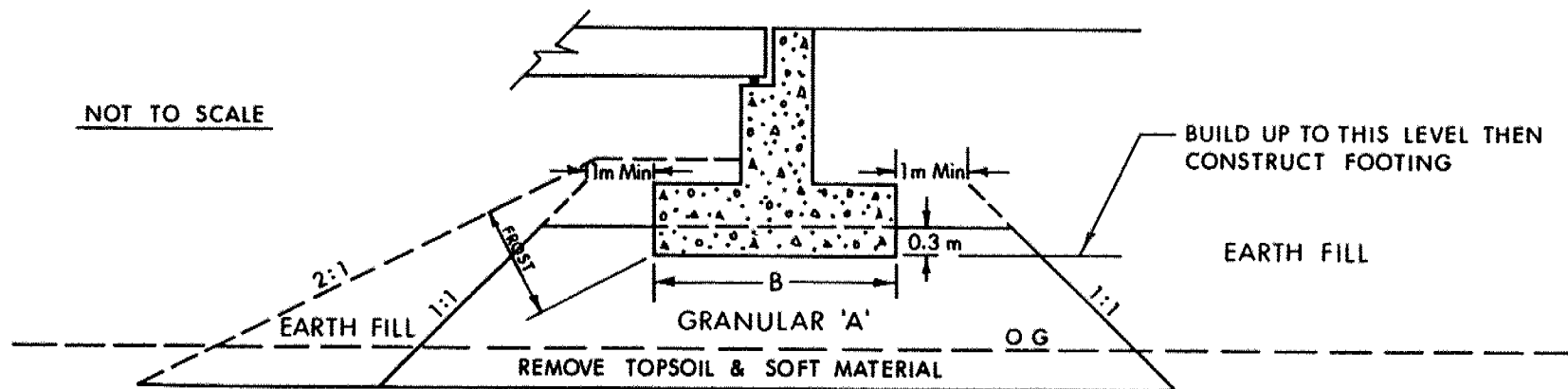
Ministry of  
Transportation

# PLASTICITY CHART CLAYEY SILT TILL

FIG No 2  
W P 199-77-07



X SECTION



LONGITUDINAL SECTION

NOTES:

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



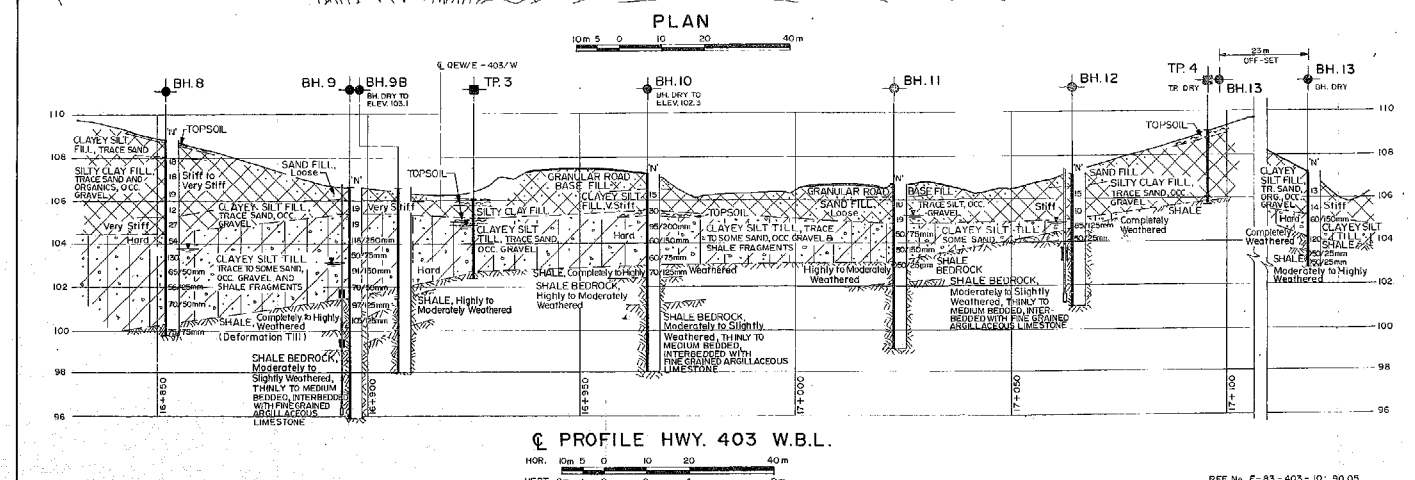
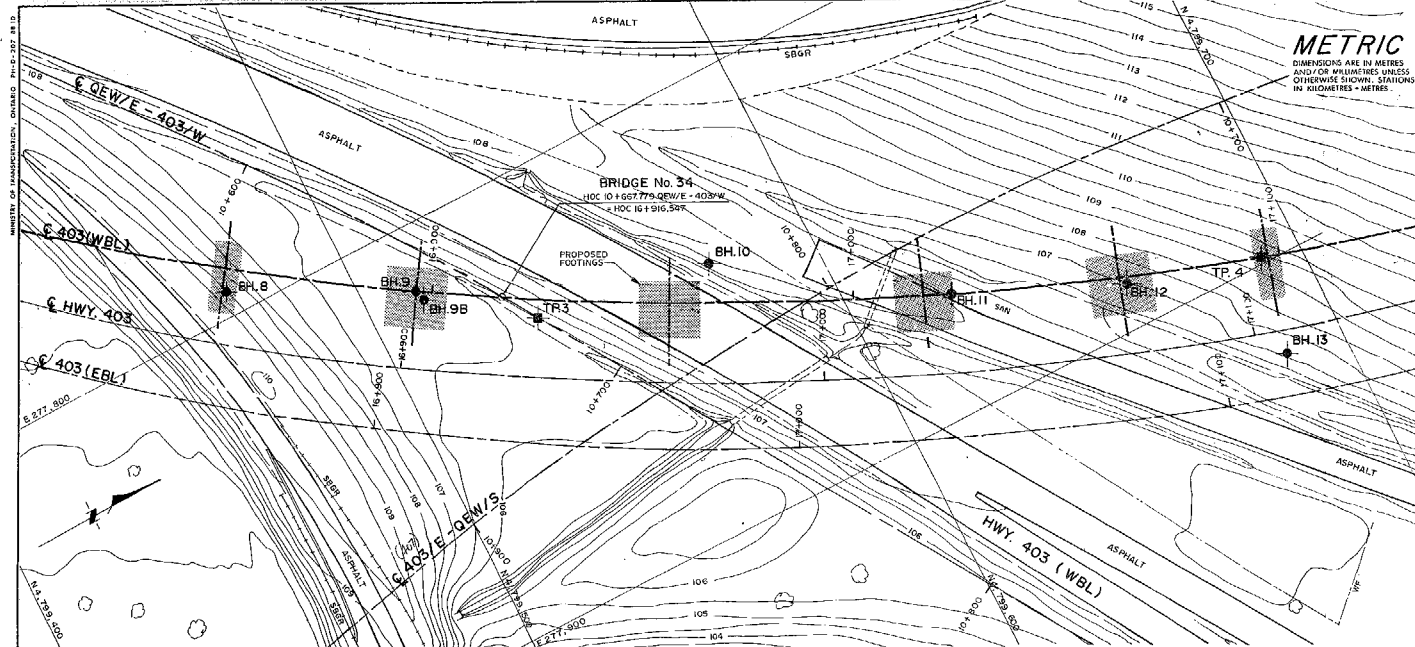
Ontario

Ministry of  
Transportation

ABUTMENT ON COMPACTED FILL  
SHOWING GRANULAR 'A' CORE

FIG No 3

W P 199 - 77 - 07



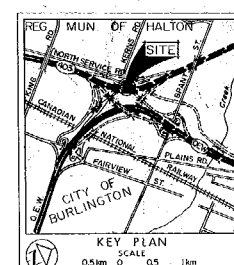
**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. DIMENSIONS  
IN KILOMETRES - METRES

CONT No  
WP No 199-77-07

BRIDGE No. 34  
HWY. 403 (WBL) OVER GEW/E - 403/W

BORE HOLE LOCATIONS & SOIL STRATA

GOLDER ASSOCIATES LTD.



**LEGEND**

- ◆ Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ◆ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CON Blows/0.3m (60° Cone, 475 J/blow)
- W/L at time of investigation August, 1990
- ⊕ Test Pit
- Piezometer

No	ELEVATION	CO-ORDINATES NORTHING	EASTING
BH.8	106.9	4,799,471.2	277,794.3
BH.9	105.6	4,799,511.3	277,814
BH.9B	106.5	4,799,511.7	277,817.2
BH.10	107.2	4,799,574	277,841
BH.11	105.8	4,799,625.5	277,872.7
BH.12	107.2	4,799,660.9	277,890
BH.13	107.2	4,799,687.6	277,921.4
TP.3	106.1	4,799,534	277,833
TP.4	105.1	4,799,691	277,898.5

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically included in accordance with the conditions of Section 102.2 of Form 100.

DATE BY DESCRIPTION

Geacres No. 30M5 - 178

HWY. 403 WB OVER RAMP GEW/E - 403/W DIST 4

SUBMIT CM CHECKED DATE JAN. 17, 1991 SITE 10-480

DRAWN MW CHECKED APPROVED DWG 1997/07-A