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W. O. No.

STR. SITE No. 10-481

HWY. No. 403

LOCATION Hwy 403 E.B. over
Ramp Q.E.W/E - 403/W

No. of PAGES - (Bridge #35)

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



Golder Associates Ltd.

CONSULTING ENGINEERS

REPORT

TO

MINISTRY OF TRANSPORTATION ONTARIO

GEOTECHNICAL INVESTIGATION

PROPOSED BRIDGE 35

SITE 10-481

FREEMAN INTERCHANGE

BURLINGTON, ONTARIO

W.P. 199-77-08

DISTRICT 4

CONT 93-89

GEOCRES # 30M5-179

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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. 35 to be constructed as part of the eastbound Highway 403 lanes as shown on the Key Plan, Drawing 1997708-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. 35 will be situated between the west edge of the existing QEW north 403 west ramp and the former North Service Road. The proposed Bridge 34 (Site 10-480, W.P. 199-77-07) will be situated parallel to and immediately north of Bridge 35. The existing ground surface to be traversed by the structure consists of the existing west bound QEW/403 ramp and grass covered medians. The existing ground surface elevation along the centreline of the proposed structure varies from about 105.5 to 108.7 metres. The location of the site is shown on the Key Plan, Drawing 1997708-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. 35 will carry Highway 403 eastbound traffic over the QEW north - 403 west ramp, Bridge 33 (site 10-335, W.P. 199-77-05) and the 403 east - QEW south ramp.

The proposed high level bridge will consist of a five span structure (spans of approximately 35, 45, 60, 45, and 30 metres south to north respectively) with two abutments and four piers. Approach fills of some 10 to 16 metres in height will be required.

3. INVESTIGATION PROCEDURE

The field work for this investigation was carried out between August 15 and 29, 1990 at which time two test pits were excavated and six boreholes were drilled. The locations of the boreholes and test pits are shown on Drawing 1997708-A.

The initial stage of the field work consisted of excavating two test pits (numbered 5 and 6) near the proposed south and north bridge abutments. Test pits 5 and 6 were excavated to depths of about 2.5 and 3.9 metres, respectively, 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 pits and the test pits were 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 15, 16 and 17 were advanced to bedrock, through about 3 to 6.7 metres of overburden. The bedrock encountered beneath the overburden in boreholes 15, 16 and 17 was core drilled in NQ size for about 3 metres. Boreholes numbered 13, 14 and 18 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 carried out 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 1997708-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. 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 1997708-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 1997708-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 topsoil, fill, glacial till and completely to highly weathered shale underlain 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 relatively thin layer of topsoil was encountered at the ground surface in boreholes 14, 15, 16, and 18 and in test pits 5 and 6.

4.3 Silty Clay, Clayey Silt, trace topsoil trace sand, (Fill)

Fill was encountered beneath the surficial topsoil layer in all of the boreholes and test pits and at the ground surface in borehole 13. The fill and topsoil layers were fully penetrated in the boreholes and test pits at elevations varying between about 103.8 and 105.2 metres or at depths of about 1.2 to 4.3 metres below the existing ground surface. Some 4.3 metres of fill was encountered in borehole 14 drilled immediately adjacent to the existing west bound QEW/403 ramp where the ground surface elevation is some 2 to 3 metres higher than at the other borehole locations.

The fill generally consisted of clayey silt to silty clay with traces of topsoil. A surface layer of sand and gravel fill was encountered in borehole 17 drilled adjacent to the existing 403 westbound lanes.

The clayey fill has in-situ water contents ranging from about 15 to 25 per cent with an average water content of about 20 per cent. N-values ranging from 7 to 22 blows per 0.3 metres, were determined in the standard penetration testing

carried out in the fill, indicating a generally firm to very stiff consistency.

4.4 Clayey Silt, trace to some sand, occ. Gravel (Till)

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

The N values determined in the upper till zone ranged from 27 to greater than 100 blows per 0.3 metres. The natural water contents of samples of the upper till recovered from the boreholes and test pits ranged from about 12 to 17 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 results of the penetration testing. The natural water content of the lower till typically ranged from about 7 to 14 per cent.

The corresponding liquid and plastic limits of the lower clayey silt till were about 22 and 14 based on the average of two Atterberg limit determinations. The results of the Atterberg limit determinations for two samples of the clayey silt till are presented on the plasticity chart, Figure 2.

Grain size distribution curves for samples of the lower till (obtained by a 35 millimetre I.D. sampler) are shown on Figure 1.

The till material as indicated by the gradation curves 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 locations, the overburden materials were underlain by shale bedrock of the Queenston Formation. Bedrock was encountered between about elevations 101.7 and 103 metres, or at depths of from about 3 to 6.7 metres below the existing ground surface.

The upper zone of the bedrock was generally highly to completely weathered. More competent shale was encountered in boreholes 15, 16 and 17 between about elevations 100 and 102 metres. The bedrock core recovered from boreholes 15, 16 and 17 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. The total

core recovery (TCR) ranged from about 93 to 100 per cent. The solid core recovery (SCR) ranged from about 57 to 85 per cent and the rock quality designation (RQD) ranged from about 8 to 75 percent.

4.6 Groundwater Conditions

Groundwater was not encountered during the field drilling/digging operations in boreholes 13, 14 and 18 which were terminated at practical auger refusal, or in test pits 5 and 6 which were terminated in the till. Boreholes 15, 16 and 17 were dry to the depth of practical auger refusal, corresponding to about elevations 99.5 to 102 metres.

The groundwater level was measured at about elevations 102.9 and 104.6 metres in the piezometers installed in boreholes drilled at the site of the adjacent structure Bridge 34 (Site 10-480, WP 199-77-07). These water levels, recorded about 1 month after the completion of drilling, correspond to depths of about 3.7 and 2.6 metres below the existing ground surface.

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

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, 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 118 metres at the south abutment to about 120.4 metres at the north abutment. The finished grade beneath the bridge will range from about 103.5 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 and may minimize 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, and test pits, 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+888	103.5 or lower
Pier No. 1	STN 16+923	103.7 or lower
Pier No. 2	STN 16+968	103 or lower
Pier No. 3	STN 17+028	103.5 or lower
Pier No. 4	STN 17+073	104 or lower
North Abutment	STN 17+103	104.8 or lower

The coefficients of friction between the concrete and the bearing strata may be computed by assuming an internal friction angle of 26° for the glacial till and 20° 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 four 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 ensure that the base has been

adequately prepared and is free of any softened or disturbed zones.

Both the glacial till and the shale bedrock are highly frost susceptible. Care and adequate protection during winter construction should therefore 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 4.3 metres of fill was encountered in borehole 14, drilled at the proposed south abutment for Bridge 35. Due to the depth of the existing fill, as well as the space limitations which may exist at both abutment locations, the use of deep foundations consisting of caissons socketed into the bedrock, as an alternative to spread footings, may be advantageous.

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 99 and 97 metres or lower for the south and north 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 range from about elevation 118.4 to about elevation 120.4 metres compared to the existing grades of 108.7 and 106 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.

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

exist at the abutment locations.

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 metres at the south abutment and to about elevation 105 metres 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.

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 on 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. Details regarding 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 uniformly 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 earth pressure 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 ³
ϕ	35°
Granular "B"	
Unit weight	21.2 kN/m ³
ϕ	30°

5.4 Excavations

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

Groundwater seepage from the coarser 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 protective layer 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.

5.5 Embankments

It is understood that approach fills of some 10 to 16 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 fill overlying glacial till. At the north abutment location about 1.4 metres of clayey fill overlies the till. The clayey fill generally has a firm to very stiff consistency and an in-situ water content of about 18 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 and adequately proofrolled under the direction of the geotechnical engineer. Remedial work should be carried out on any soft 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 300 to 400 millimetres for the embankment heights proposed. 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 potential for 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 can be expected between an abutment footing founded on the till below the approach embankment, and the adjacent pier also resting within the lower till deposit. Differential settlements in the range of about 20 to 40 millimetres are anticipated. 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.

GOLDER ASSOCIATES LTD.

Vince Hanemayer

V. C. Hanemayer, P. Eng.

A circular professional engineer seal for J. G. Muckle. The seal contains the text "REGISTERED PROFESSIONAL ENGINEER" around the top and "J. G. MUCKLE" in the center. A signature is written across the seal.
J. G. Muckle, P. Eng.

att.

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 (RQD), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ²	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						



METRIC

W P 199-77-07 LOCATION 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

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to Sensitivity

15 ϕ 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 14

METRIC

W P 199-77-08 LOCATION N4,799,481.2 E277,828.2 ORIGINATED BY VCH
 DIST 4 HWY QEW/403 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MHW
 DATUM Geodetic DATE August 20, 1990 CHECKED BY JGM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
108.7	Ground Surface																
	Topsoil																
0.1	Silty Clay, trace sand occasional gravel trace topsoil (Fill)		1	SS	20		108										
	Reddish Brown																
107.3	Very Stiff																
1.4	Clayey Silt trace sand trace organics occ. gravel (Fill)		2	SS	7												
			3	SS	10		106										
			4	SS	22												
			5	SS	20												
104.4	Firm to very stiff Brown																
4.3	Clayey Silt trace to some sand occ. gravel occ. shale fragments		6	SS	53		104										
			7	SS	50/75mm												
			8	SS	70/100mm												
102.0	Hard Elevation 103.5m		9	SS	50/25mm		102										
6.7	Probably shale Bedrock Highly to moderately weathered		10	SS	50/25mm												
	Reddish Brown						100										
99.6																	
9.1	End of Borehole						98										

+³, x⁵: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 15

METRIC

W P 199-77-08 LOCATION N4,799,510.4 E277,849.71 ORIGINATED BY VCH
 DIST 4 HWY 403/OEW BOREHOLE TYPE Hollow Stem Auger; NQ Rock Core COMPILED BY MHW
 DATUM Geodetic DATE August 20-21, 1990 CHECKED BY JGM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
106.1	Ground Surface																
0.1	Topsoil						106										
	Clayey Silt trace sand, tr topsoil occ. gravel (Fill)		1	SS	20												
104.6	Very Stiff Brown																
1.5	Clayey Silt trace to some sand, occasional gravel, occ. shale fragments (Till)		2	SS	27												
			3	SS	50/25mm		104										
	Very stiff to hard		4	SS	50/25mm												
	Grey becoming reddish brown at about Elev. 103.7m		5	SS	82/130mm		102										
101.7	Shale Bedrock Highly to moderately weathered		6	SS	78/130mm												
4.4			7	SS	40/0mm		100										
99.5	Reddish Brown																
6.6	Shale Bedrock moderately to slightly weathered thin to medium bedded Reddish Brown Interbedded with fine grained argillaceous limestone		8	NQ RC	TCR= 100% SCR= 85% RQD= 75%		98										
			9	NQ RC	TCR= 100% SCR= 62% RQD= 16%												
96.4																	
9.7	End of Borehole						96										
	* TCR: Total Core Recovery SCR: Solid Core Recovery RQD: Rock Quality Designation																

+³, x⁵: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16

METRIC

W P 199-77-08 LOCATION N4799544.2 E277872.3 ORIGINATED BY VCH
 DIST 4 HWY 403/QEW BOREHOLE TYPE Hollow Stem Auger; NQ Rock Core COMPILED BY MHW
 DATUM Geodetic DATE August 29, 1990 CHECKED BY JGM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
105.5	Ground Surface																
0.0	Topsoil																
0.2	Silty Clay trace sand, tr topsoil occ. gravel (Fill)		1	SS	16												
103.8	Stiff to Very Stiff Brown		2	SS	14												
1.7	Clayey Silt trace to some sand occasional gravel (Till)		3	SS	65/ 100mm												
101.8	Hard Reddish Brown		4	SS	90/130mm												
3.7	Shale Bedrock highly weathered		5	SS	50/50mm												
101.1	Reddish Brown		6	NQ RC	*TCR =93% SCR =57% RQD = 8%												
4.4	Shale Bedrock moderately to slightly weathered thinly to medium bedded Reddish Brown interbedded with fine grained argillaceous Limestone		7	NQ RC	TCR =100% SCR =78% RQD =42%												
98.0	End of Borehole																
7.5	End of Borehole																
	* TCR: Total Core Recovery SCR: Solid Core Recovery RQD: Rock Quality Designation																

+³, x⁵: Numbers refer to
Sensitivity

20
15
10
5
0
5
10
15
20
(%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 17

METRIC

W P 199-77-08 LOCATION N4,799,608.7 E277,901.4 ORIGINATED BY CB
 DIST 4 HWY 403/QEW BOREHOLE TYPE Hollow Stem Auger, NQ Rock Core COMPILED BY MRW
 DATUM Geodetic DATE August 16, 1990 CHECKED BY JGM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) 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 and gravel; granular road base (Fill) Grey																
106.0																	
0.6	Clayey Silt, trace sand, occ. gravel (Fill) Stiff Brown		1	SS	15		106										
104.9																	
1.7	Clayey silt, trace to some sand, occ. gravel		2	SS	18												
	Very stiff to Hard		3	SS	93		104										
	Reddish Brown		4	SS	50/100mm												
102.9			5	SS	50/25mm												
3.7	Shale Bedrock probably highly to completely weathered																
102.0	Reddish Brown																
4.6	Shale Bedrock, moderately to slightly weathered thin to med. bedded, Reddish Brown, interbedded with fine- grained argillaceous Limestone.		6	NQ RC	* TCR= 100% SCR= 61% RQD= 38%		102										
			7	NQ RC	* TCR= 100% SCR= 62% RQD= 16%		100										
98.9																	
7.7	End of Borehole																
	* TCR= Total Core Recovery SCR= Solid Core Recovery RQD= Rock Quality Designation						98										

+3, x5: Numbers refer to Sensitivity

15 $\frac{20}{10}$ 5 (%) STRAIN AT FAILURE

METRIC

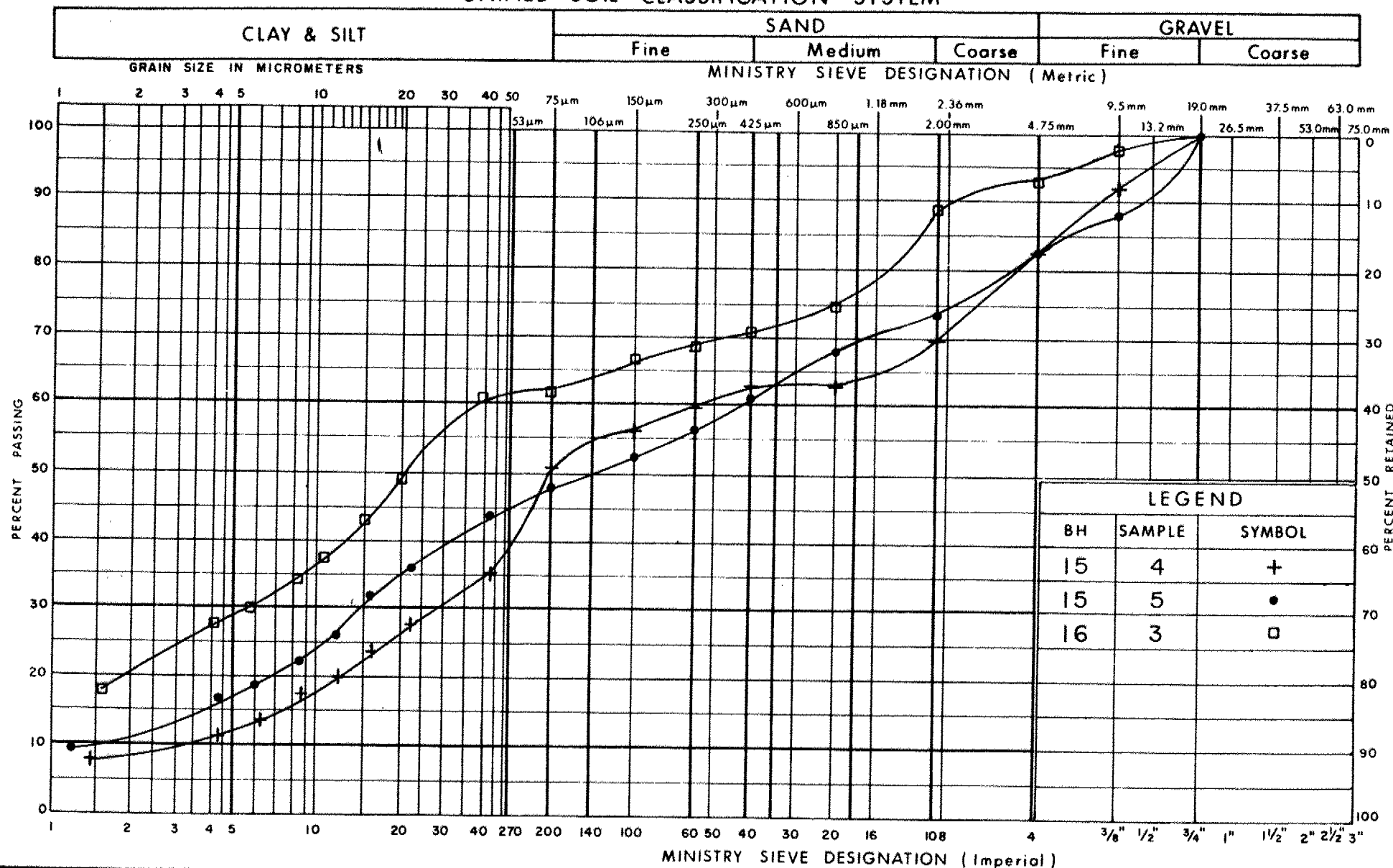
+3, x5: Numbers refer to Sensitivity

METRIC

W P 199-77-08 LOCATION N4,799,642 E277,925.5 ORIGINATED BY VCH
DIST 4 HWY 403/QEW BOREHOLE TYPE John Deere 690 - Backhoe Dug COMPILED BY MHW
DATUM Geodetic DATE August 16, 1990 CHECKED BY JGM

[illegible]

UNIFIED SOIL CLASSIFICATION SYSTEM



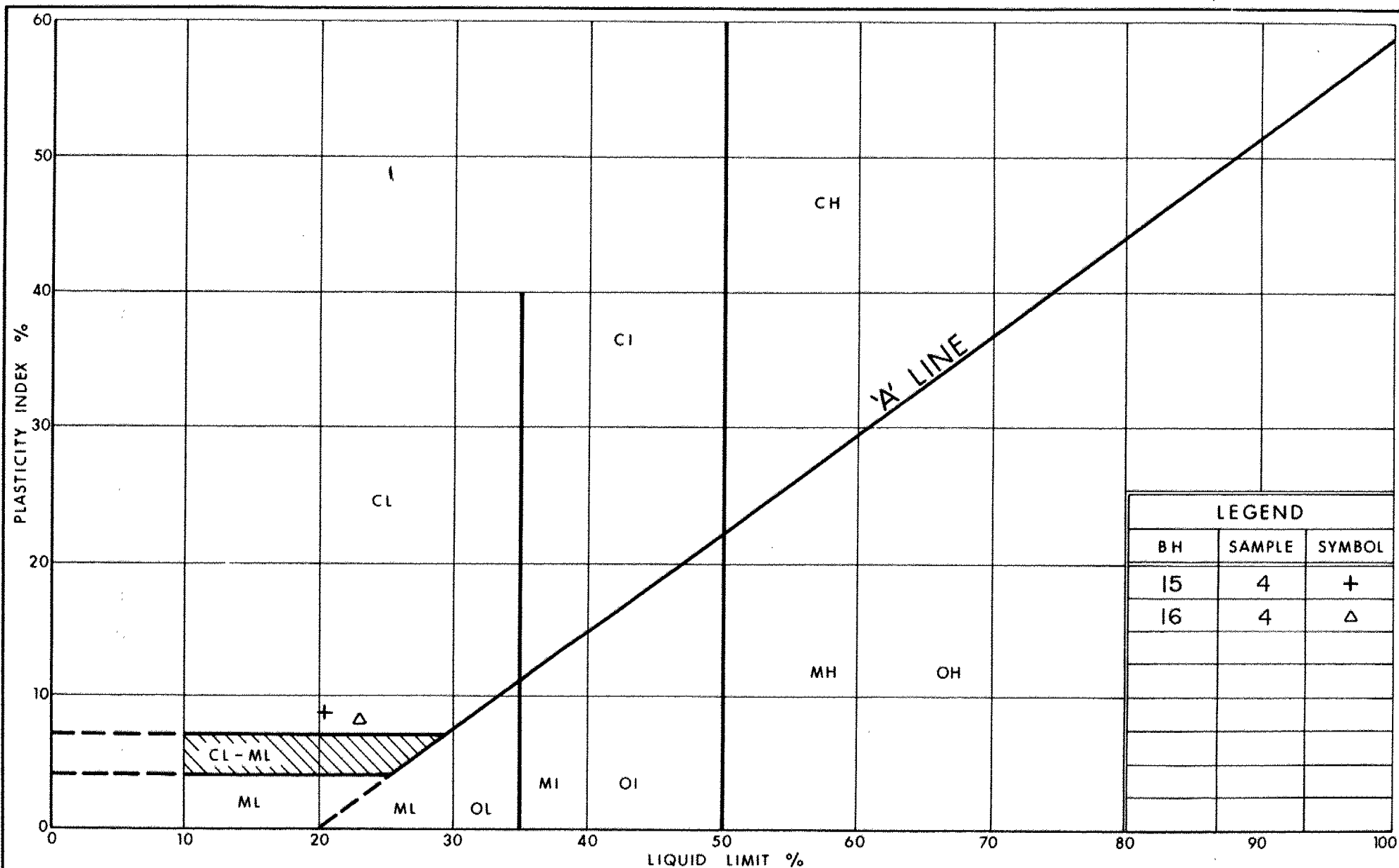
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

CLAYEY SILT TILL

FIG No 1

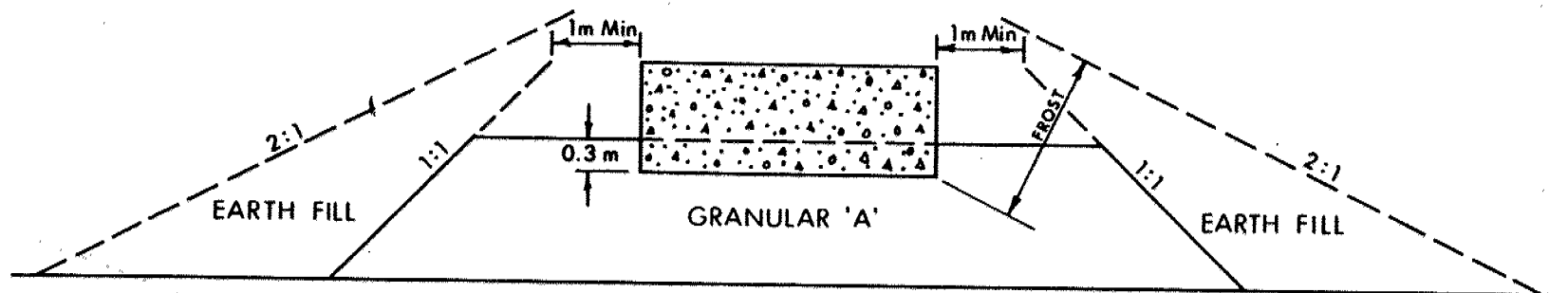
W P 199-77-08



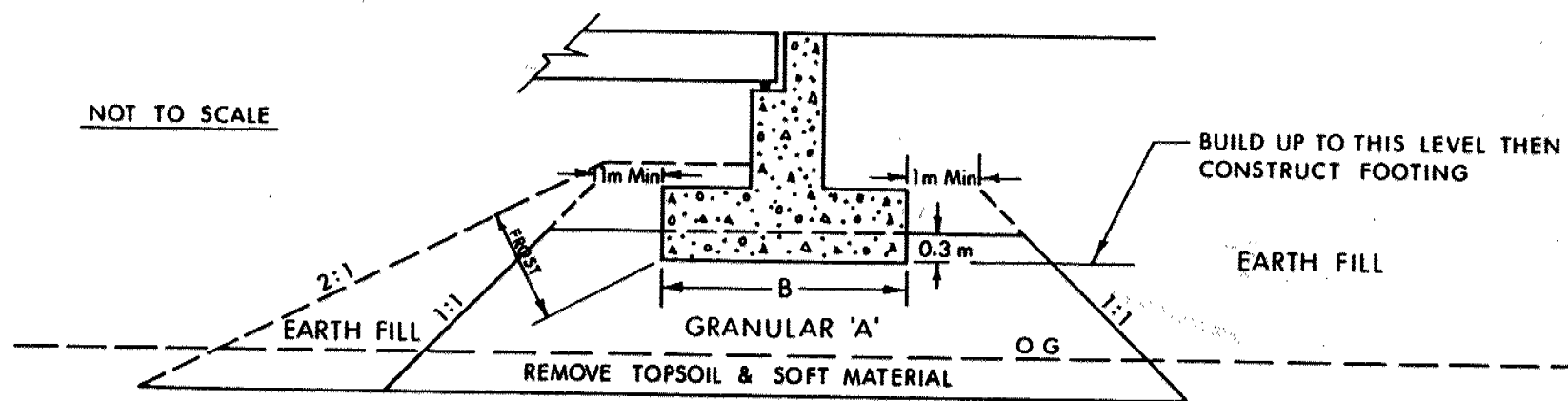
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Ontario

PLASTICITY CHART CLAYEY SILT TILL

FIG No 2
W P 199-77-08



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.



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ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

FIG No 3

W P 199-77-08