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HWY. No. 403

LOCATION Ramp 403/W - Q.E.W./S over  
Ramp 403/E - Fairview St.

No of PAGES - (Bridge #42)

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



# **Golder Associates Ltd.**

CONSULTING ENGINEERS

## **REPORT**

**TO**

**MINISTRY OF TRANSPORTATION ONTARIO**

### **GEOTECHNICAL INVESTIGATION**

**PROPOSED BRIDGE 42**

**SITE 10-334**

**FREEMAN INTERCHANGE**

**BURLINGTON, ONTARIO**

**W.P. 199-77-04**

**DISTRICT 4**

*CONT 93-89*

*GEOCRES # 3045-176*

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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. 42. As presently proposed, Bridge 42 will carry the 403 west/QEW south traffic over the 403 east/Fairview Street ramp as shown on the Key Plan, Drawing 1997704-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 rigid frame 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 site of the proposed Bridge 42 is located in a relatively flat, grassed area, situated between the existing 403-QEW south and, the 403 east-QEW south ramps of the Freeman Interchange in Burlington, Ontario. The location of the site is shown on the Key Plan, Drawing 1997704-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 layers of grey limestone.

The bridge will carry 403 east/QEW south traffic over the 403 south/Fairview Street ramp.

The proposed bridge will consist of a 9.5 metre span rigid frame, 50 metre in length. Two sections of retaining wall, 25 metres and 40 metres in length are required at the northwest and southeast corners, of the proposed structure. Approach fills of some 7 metres in height will also be required.

### 3. INVESTIGATION PROCEDURE

The field work for this investigation was carried out on August 17 and 22, 1990 at which time two test pits were excavated and four boreholes were drilled. The locations of the boreholes and test pits are shown on Drawing 1997704-A.

The initial stage of the field work consisted of excavating two test pits (numbered 9 and 10) at the west and east limits of the proposed bridge. The test pits were excavated to a depth of about 3.8 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 pits and

the test pits were loosely backfilled following sampling and logging.

The boreholes were drilled using a track mounted power auger equipped for rotary drilling, which was supplied and operated by a specialist drilling contractor. Boreholes numbered 29 and 30 were advanced to bedrock, through about 5.2 to 5.8 metres of overburden, and the shale bedrock encountered beneath the overburden was core drilled in NQ size for 3 metres in these boreholes. Boreholes numbered 28 and 31 were drilled to practical auger refusal. The boreholes were advanced in the overburden using nominal 150 millimetre diameter hollow 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. The grain size distribution and Atterberg limits 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 1997704-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. A piezometer was installed in Borehole 29 as detailed on the Record of Borehole sheet. Notes pertaining to the groundwater conditions observed in the boreholes and test pits are also shown on the Records of Boreholes and Test Pits and on Drawing 1997704-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 at 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 Record of Borehole and Record of Test Pit sheets, and in summary form on Drawing 1997704-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

Topsoil was encountered at the ground surface in the boreholes and the test pits put down at the site. The thickness of the topsoil layer ranged from about 0.1 to 0.2 metres at the locations investigated.

#### 4.3 Clayey Silt, Silty Clay, Sand and Gravel (Fill)

A layer of fill was encountered overlying the glacial till in all of the boreholes and test pits. The thickness of the fill material ranged from about 0.6 metres to about 1.8 metres. The fill generally consisted of brown clayey silt with trace sand and organics and occasional gravel. A layer of sand and gravel fill was encountered below the topsoil in borehole 29.

The clayey silt fill has in-situ water contents ranging from about 8 to 37 per cent. The N-values determined in standard penetration testing carried out in the clayey silt fill ranged from 15 to 22 blows per 0.3 metres indicating a very stiff consistency.

#### 4.4 Clayey Silt trace sand, occasional gravel and cobbles (Till)

Glacial till was encountered in all of the boreholes, and in the test pits, and was generally characterized by an upper zone of very stiff to hard brown to grey 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 46 to 120 blows per 0.3 metres. The natural water content of samples of the upper till recovered from the boreholes and test pits ranged from about 8 to 14 per cent. The corresponding plastic and liquid limits of the upper till are about 14 and 27 respectively based on a single Atterberg limit determination. A grain size distribution curve for a sample of the clayey silt till (recovered from a 35 millimetre I.D split spoon sampler) is shown on Figure 1 and a corresponding plasticity chart is shown on Figure 2.

Test pits 9 and 10 were terminated in the upper till deposit at a depth of about 3.8 metres below the existing ground surface. The upper till deposit was fully penetrated in the boreholes at depths of about 4 to 5 metres below the existing ground surface, or between about elevations 96 and 98 metres.

Beneath the grey-brown clayey silt till in boreholes 28, 29 and 30, a reddish brown clayey silt till deposit was encountered.

Standard penetration testing was carried out in the lower till zone, but due to its hard consistency 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 10 per cent. The lower till deposit tended to be slightly coarser than the overlying till.

The till deposits encountered were generally fine grained in nature and no major concentrations 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 95.6 and 96.6 metres, or at depths of from about 5.0 to 5.8 metres below the existing ground surface.

The upper zone of the bedrock is generally highly weathered and has been described as deformation till in the boreholes. More competent shale was encountered in boreholes 29, and 30 between about elevations 94.6 and 95.3 metres, or at depths of about 6 metres below existing ground surface. The bedrock core recovered from boreholes 29 and 30 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 90 to 100 per cent and the solid core recovery (SCR) ranged from about 52 to 97 per cent. The rock quality designation (RQD) ranged from about

32 to 55 per cent.

#### 4.6 Groundwater Conditions

Groundwater was not encountered during the field drilling/digging operations in boreholes 28 and 31 which were terminated at auger refusal, or in test pits 9 and 10 which were terminated in the grey brown (upper) clayey silt till. Boreholes 29 and 30 were dry to the depth of practical auger refusal, corresponding to about elevations 95.3 and 94.6 metres respectively. The groundwater level was measured at about elevation 101.1 metres or at a depth of about 0.3 metres below the existing ground surface, in the piezometer in Borehole 29, about 2 weeks after the completion of drilling and at about elevation 100.5 metres some 3 months after drilling.

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.

## 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 in the following discussion 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 provided 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 Bridge No. 42 will consist of a 9.5 metre wide single span rigid frame 50 metres in length. Two sections of retaining walls 25 metres and 40 metres in length will also be required adjacent to the north west and southeast corners of the proposed structure.

The proposed finished deck grade will be at about elevation 108 metres and the proposed pavement beneath the bridge will be at about elevation 102 metres.

The results of the present investigation indicate that the

glacial till encountered at a relatively shallow depth is suitable for conventional spread footing support for the proposed abutments and retaining walls. Although other alternatives such as deep foundations and perched foundations on engineered fill are technically feasible, spread footings founded in the undisturbed glacial till are considered to be the most appropriate alternative for this site.

#### 5.2.2 Spread Footings

Based on the results of the present investigation it is considered that the bridge abutments and retaining walls may be founded on conventional spread footings bearing in the upper grey-brown till at or below elevation 99.5 metres. Footings founding as outlined above may be designed using bearing pressures of 500 kilopascals at Ultimate Limit States and 350 kilopascals at Serviceability Limit States. The coefficient of friction between the concrete and the till may be computed by assuming an internal friction angle ( $\phi$ ) of 26° for the till.

In order to achieve the design bearing pressures, and minimize post construction settlement, 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. Effective control of groundwater during construction is a critical aspect to preserving the integrity of the bearing strata.

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, is free of any softened or disturbed zones, and is within the natural till deposit.

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 for frost protection.

#### 5.2.3 Perched Foundations

From a geotechnical standpoint, the support of the structure foundations on spread footings founded in the till, is the preferred alternative. The support of the foundations at higher elevations on engineered granular fill as shown schematically on Figure 3 is feasible, however the potential for differential settlements is greater than for the preferred alternative. Conceptually, the design consists of a core well compacted Granular "A" material 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 to expose a competent native subgrade beneath areas of "structural fill". Based on the results of the present investigation, a suitable native subgrade will probably be encountered between about elevations 99.5 and 101 metres. 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 and as shown on Figure 3.

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

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 and the flexibility of the rigid frame to accommodate these movements.

For design purposes, differential settlements in the order of about 25 millimetres should be anticipated. The magnitude of the actual differential settlements will be affected by the bridge span and soil/structure interaction. In addition,



poor construction techniques could result in greater settlements.

#### 5.2.4 Alternative Retaining Structures

A variety of proprietary earth retaining, gravity type structures could be considered as an alternative to conventional reinforced concrete retaining walls. Deep foundations are typically not required for foundations for this type of wall and the walls are generally able to tolerate larger settlements than conventional walls can. Suitable subgrade preparation and an adequate degree of compaction of the backfill materials are key elements to the performance of these wall systems.

It is understood that consideration is being given to the use of such earth retaining wall systems for the retaining walls which are required adjacent to the rigid frame. As presently proposed the walls would be founded on the existing earth fill subgrade at about elevation 100.8 metres. This will result in the walls being founded on existing earth fill materials which are variable in nature and consistency. Some post constructing settlement of the walls will therefore occur.

As a minimum, preparation for the construction of the walls should consist of stripping all of the topsoil and any highly organic or soft fill beneath the walls. The exposed subgrade should be adequately proofrolled to identify any areas of the subgrade requiring remedial work. Any over excavated areas should be backfilled with good quality earth fill and

uniformly compacted to at least 98 per cent of standard Proctor maximum dry density.

The final design details of any alternative wall systems should be submitted to the MTO for review by the geotechnical engineer.

### 5.3 Earth Pressures for Abutments and Retaining Walls

The lateral earth loads acting on the abutments and retaining walls will depend on the rigidity of the structure, on the type and method of placement of the backfill materials, on 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 and retaining walls.

- Select free draining granular fill in accordance with OPSS Granular "A" or "B" should be used as backfill immediately behind the abutments and retaining walls. 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 should apply. If, however, the structure is not permitted to yield by this amount, 'at rest' pressure conditions should be used. The following earth 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 glacial till.

Groundwater seepage from the granular fill and from permeable layers within the till 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 and preparation of the bearing surface 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 about 7 metres in height will be required at the abutment locations. The subgrade encountered in the boreholes drilled at the proposed abutment locations consists of about 1.0 to 1.5 metres of clayey silt and granular fill over glacial till. The clayey silt fill had a very stiff consistency and an average in-situ water content of about 17 per cent. However, a sample of the fill from test pit 9 had an in-situ water content of about 37 per cent.

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 some of the existing fill in place beneath the proposed embankment. Prior to constructing the embankments the existing fill subgrade should be stripped of any topsoil and any highly organic fill materials and proofrolled under the direction of the geotechnical engineer. Remedial work should be carried out on any soft or loose zones encountered.

Following completion of the preparatory work as outlined above, the embankment fill should be constructed consistent with 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 inconsistencies within 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 embankments of 7 metres in height, overall side slope inclinations of 2 horizontal to 1 vertical may be considered provided that good control of surface water runoff can be achieved. The completed embankment side slopes should be blanketed with an appropriate vegetation cover.

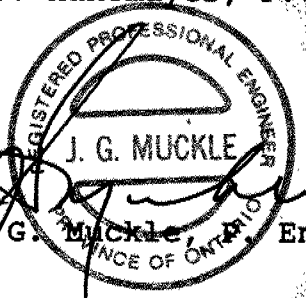
Some differential settlement can be expected between the abutment footings founded on the till below the approach embankments. For design purposes differential settlement in the order of 10 to 20 millimetres should be considered. If the bridge structure is unable to tolerate any differential movements, consideration will have to be given to founding the abutments within the lower till 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 all of the test pits put down in this investigation and in any other preconstruction investigations, are to be located in the field by the contractor to determine whether they are within the bridge or embankment alignments. Test pits within the embankment areas should be re-excavated and backfilled with appropriate material placed and compacted as outlined above. All foundation excavations should fully penetrate any disturbed material.

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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 (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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /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
$S_i$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

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



# RECORD OF BOREHOLE No 28

METRIC

W P 199-77-04 LOCATION Co-ordinates N4,799,237.5 E277,943  
 DIST 4 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers  
 DATUM Geodetic DATE August 22, 1990  
 ORIGINATED BY VCH  
 COMPILED BY MHW  
 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					
101.6	Ground Surface																
0.1	Topsoil																
101.0	Clayey Silt (Fill) occ. gravel Brown																
0.6	Clayey Silt (Till) trace sand, occasional gravel		1	SS	57												
			2	SS	66		100										
	Grey brown to red brown at about elevation 97.8		3	SS	64												
			4	SS	57		98										
			5	SS	60/75mm												
			6	SS	70/100mm												
96.6	Hard																
5.0	Shale (Deformation Till) comp. weathered																
96.2	Shale Bedrock Highly weathered		7	SS	65/150mm		96										
5.4			8	SS	60/25mm												
95.5	End of Borehole																
6.1							94										

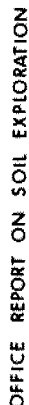
OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 29

METRIC

W P 199-77-04 LOCATION Co-ordinates N4,799,218 E277,961 ORIGINATED BY VCH  
 DIST 4 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers: NO Rock Core COMPILED BY MHW  
 DATUM Geodetic DATE August 22, 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					
101.4	Ground Surface																
0.1	Topsoil																
100.6	Sand and Gravel (Fill)																
0.8	Brown																
0.8	Clayey Silt (Fill)		1	SS	22												
	trace sand and topsoil																
99.9	occ. gravel																
	Very Stiff Brown																
1.5	Clayey Silt, trace		2	SS	53												
	sand, occ. gravel and																
	rock fragments (Till)																
			3	SS	65												
			4	SS	59												
			5	SS	58												
	Hard brown becoming		6	SS	60/	150mm											
	reddish brown at about																
	elevation 97.0																
			7	SS	60/	100mm											
95.6																	
5.8	Shale (Deformation Till)																
95.3	Completely weathered																
6.1	Shale Bedrock		8	NQ RC	*TCR= 98% SCR= 63% RQD= 32%												
	moderately to slightly																
	weathered thinly to																
	medium bedded Reddish																
	Brown																
	interbedded with grey																
	fine grained argil- laceous limestone		9	NQ RC	TCR= 100% SCR= 97% RQD= 55%												
92.3																	
9.1	End of Borehole																
	* TCR: Total Core Recovery SCR: Solid Core Recovery RQD: Rock Quality Designation																



## METRIC

+3, x5: Numbers refer to Sensitivity

## METRIC

W P 199-77-04 LOCATION Co-ordinates N4,799,158 E277,988 ORIGINATED BY CB  
DIST 4 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY MHW  
DATUM Geodetic DATE August 22, 1990 CHECKED BY VCM

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

+3, x5 : Numbers refer to Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10

# RECORD OF TEST PIT No. 9

METRIC

W P 199-77-04 LOCATION Co-ordinates N4,799,213.5 E277,950 ORIGINATED BY VCH  
 DIST 4 HWY 403/QEW BOREHOLE TYPE Bachoe Dug "John Deere 690" 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					UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>			
101.0	Ground Surface													
100.8	Topsoil													
0.2	Clayey Silt (Fill) with zones of gravel and sand		1	CS										
99.8	Brown													
1.2	Clayey Silt (Till) tr. sand, occasional gravel and cobbles		2	CS										
97.3	Grey Brown													
3.7	Bottom of Pit													
				</										

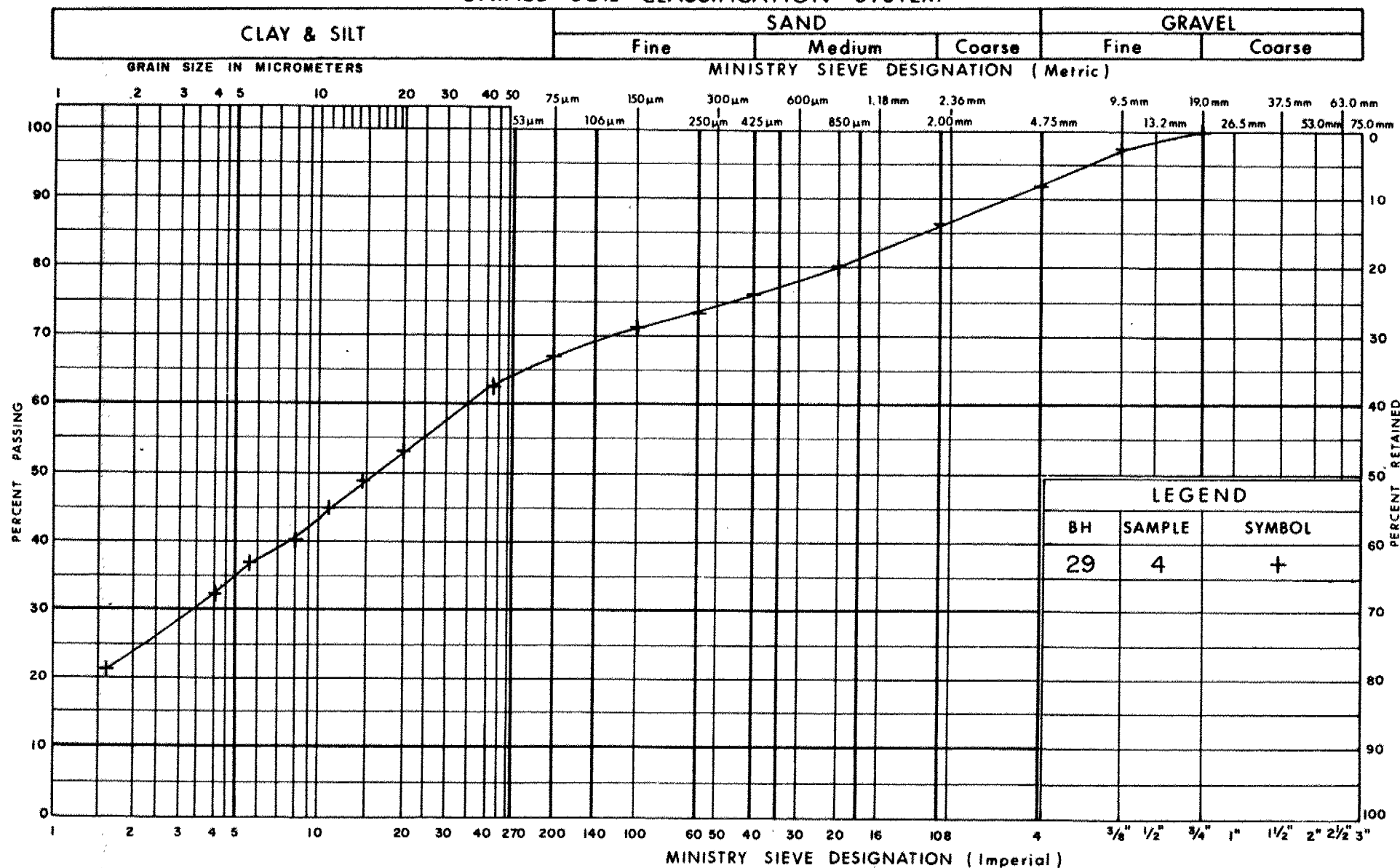
OFFICE REPORT ON SOIL EXPLORATION

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

20  
15  
10  
5 (% ) STRAIN AT FAILURE



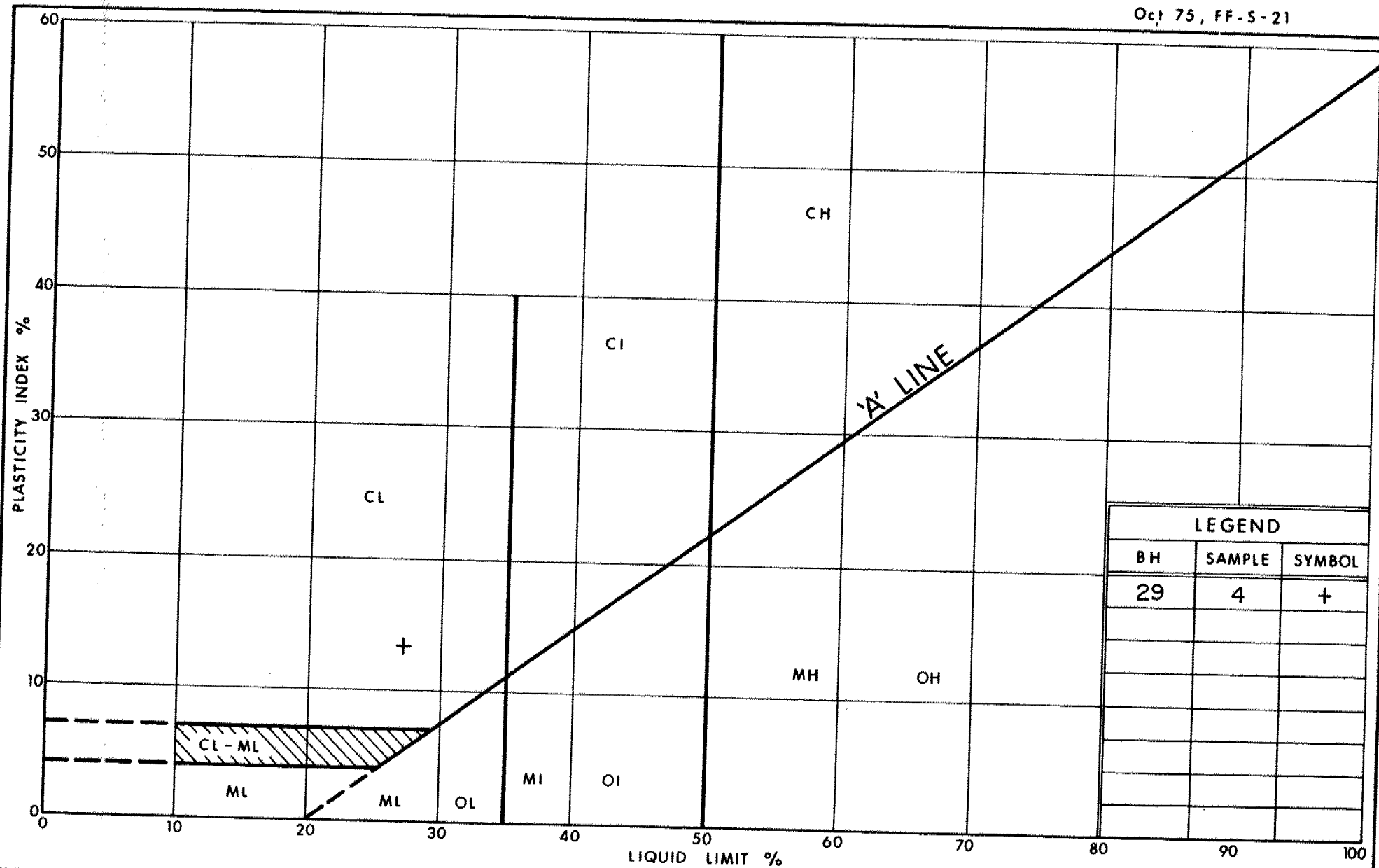
## UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
GREY BROWN CLAYEY SILT TILL

FIG No 1  
W P 199-77-04

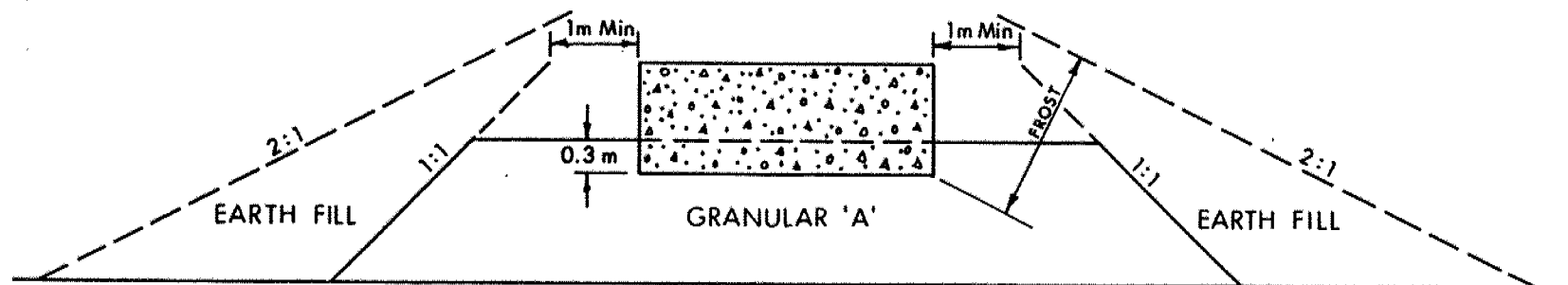


Ministry of  
Transportation

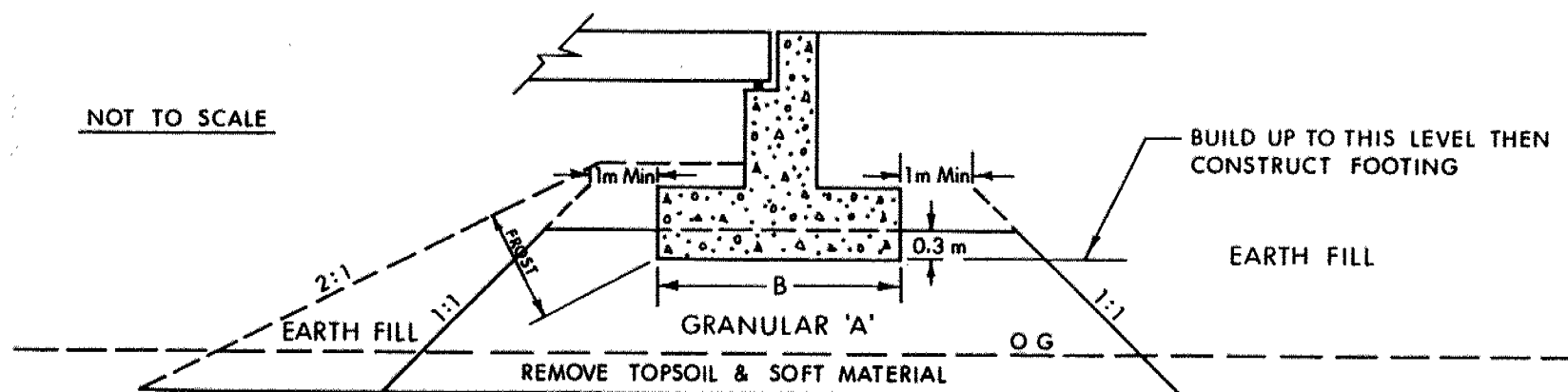
# PLASTICITY CHART GREY BROWN CLAYEY SILT TILL

FIG No 2  
W P 199-77-04





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 199-77-04

DIST No 4  
 CONT No  
 WP No 199-77-04



RAMP 403/W - Q.E.W./S  
OVER RAMP 403/E-FAIRVIEW ST.  
(BRIDGE #42)  
GENERAL ARRANGEMENT

**R.E. Winter & Associates Ltd.**  
Consulting Engineers, Architects, Planners and Landscape Architects  
200 UNIVERSITY AVE., SUITE 200, WILLOWDALE, ONT. M2H 3G5 TEL: (416) 491-1000

CLASS OF CONCRETE 30 MPa

CLEAR COVER TO REINFORCING STEEL

- |   |                                    |        |
|---|------------------------------------|--------|
| • | FOOTINGS                           | 100±25 |
| • | ABUTMENTS                          |        |
|   | FRONT FACE                         | 80±20  |
|   | BACK FACE                          | 70±20  |
| • | DECK                               |        |
|   | TOP                                | 70±20  |
|   | BOTTOM                             | 50±10  |
| • | REMAINDER (UNLESS OTHERWISE NOTED) | 70±20  |

## REINFORCING STEEL

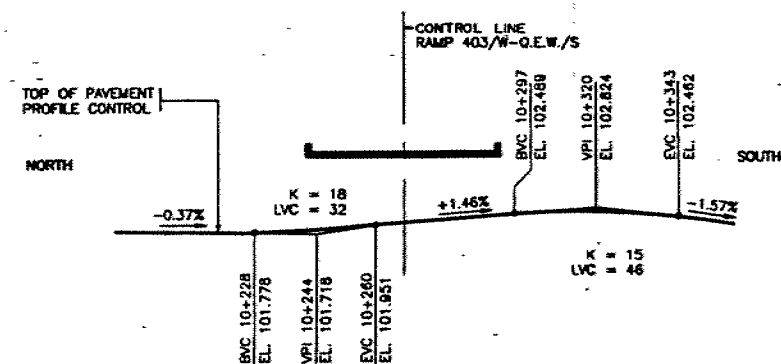
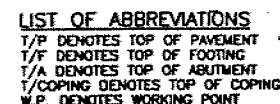
- \* REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX 'C' DENOTE COATED BARS.

CONSTRUCTION NOTES:

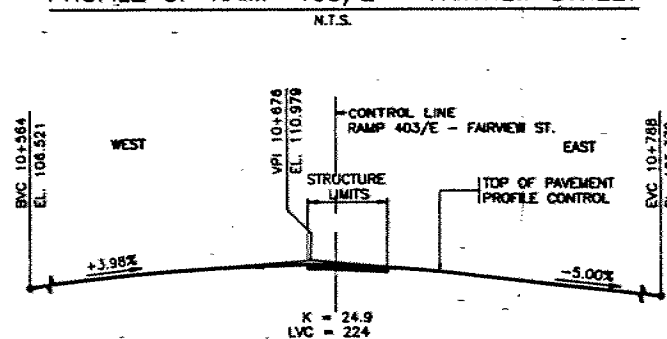
- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.

## LIST OF DRAWINGS

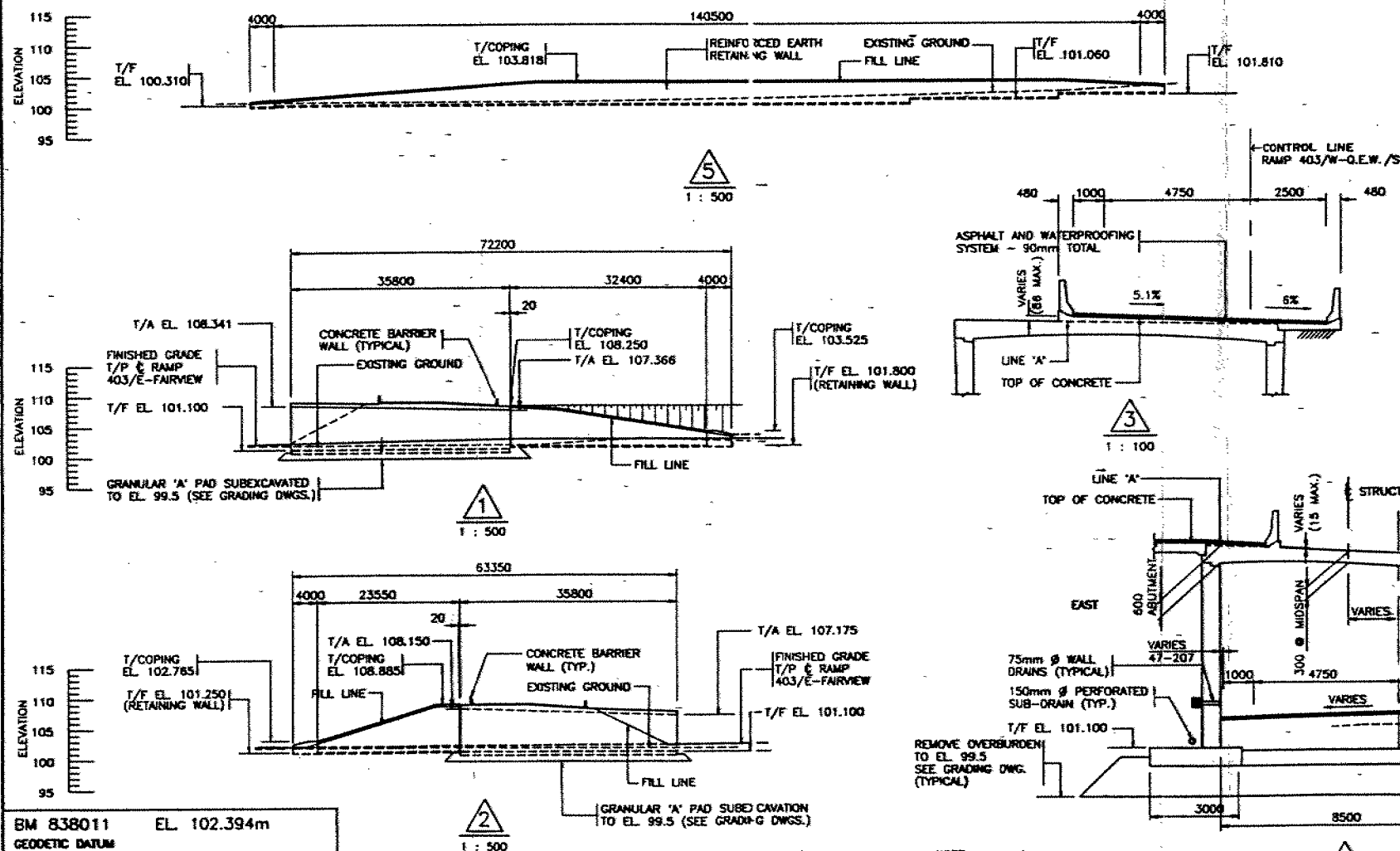
1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS & SOIL STRATA
3. ABUTMENT FOUNDATION LAYOUT
4. ABUTMENT FOUNDATION REINFORCING
5. ABUTMENT LAYOUT AND REINFORCING
6. DECK LAYOUT AND DETAILS
7. DECK REINFORCING
8. SCREED ELEVATIONS & TRANSITION CONCRETE BARRIER.
9. BARRIER WALL NORTH SIDE.
10. BARRIER WALL SOUTH SIDE.
11. 6000mm APPROACH SLABS
12. AS CONSTRUCTED ELEV. & DIM.
13. R/E RETAINING WALLS PLAN, DETAILS & SCHEDULE
14. R/E RETAINING WALLS ELEVATION & DETAILS
15. R/E RETAINING WALLS ELEVATIONS, SECTIONS AND DETAILS
16. R/E RETAINING WALLS TYPICAL DETAILS
17. STANDARD DETAILS
18. ELECTRICAL EMBEDDED WORK - LAYOUT
19. ELECTRICAL EMBEDDED WORK - DETAILS
20. QUANTITIES STRUCTURE



### PROFILE OF RAMP 403/E - FAIRVIEW STREET



PROFILE OF RAMP 403/W - Q.E.W./S



NOTE:  
LINE 'A' DENOTES CONTROL LINE FOR  
PLACEMENT OF REINFORCING.



APPLICABLE STANDARD DRAWINGS  
DD-3502 MINIMUM GRANULAR BACKFILL REQUIREMENTS.

REVISIONS						
	DATE	BY				
DESIGN	DI	CHK VZ/KT	CODE OMBDC-83	LOAD CLASS A	DATE	OCT. 1991
DRAWN	ML	CHK KT	SITE 10-334	STRUCT	SCHEME	DWG. 01

METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

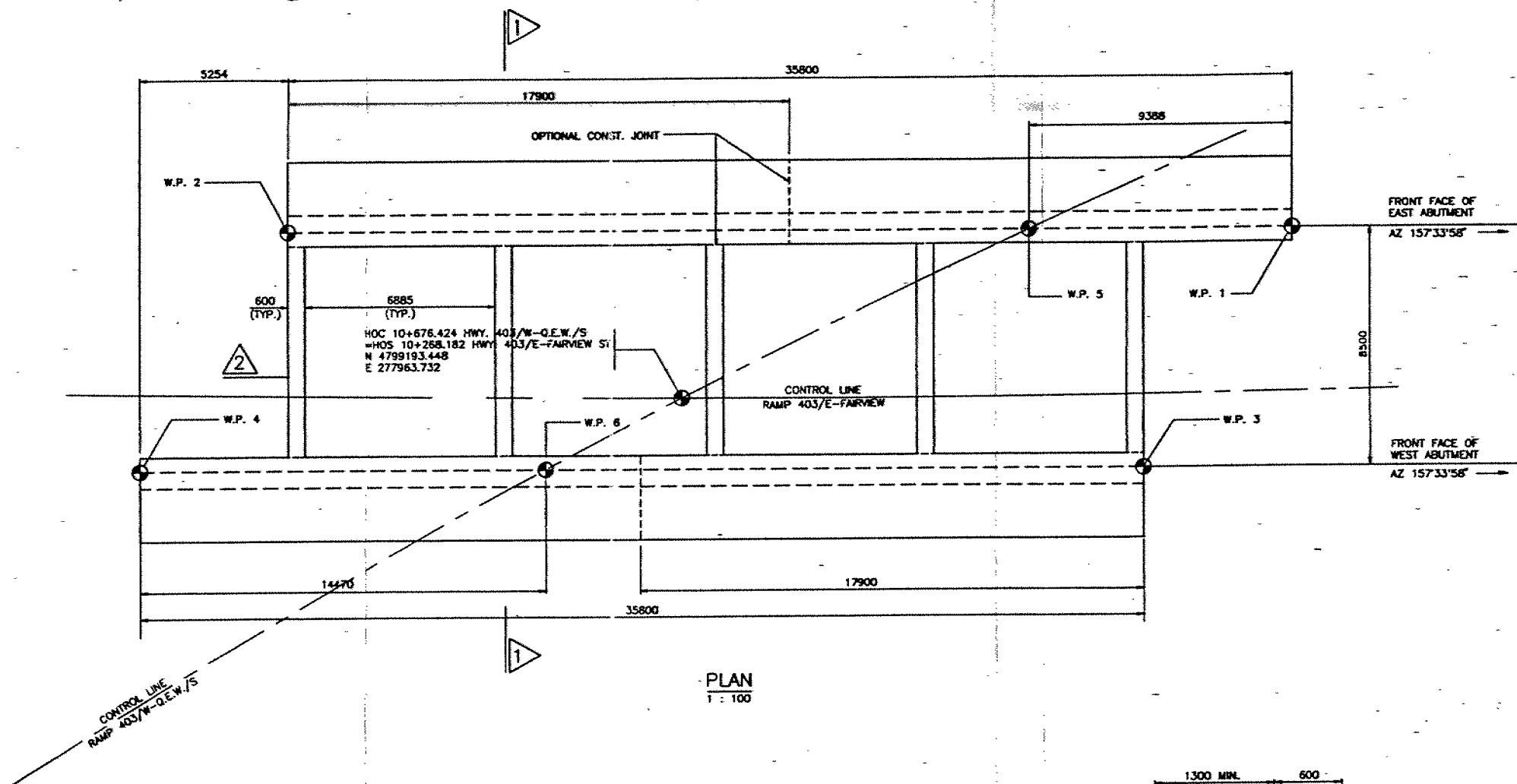
CONT No  
WP No 199-77-04



RAMP 403/W - Q.E.W./S  
OVER RAMP 403/E-FAIRVIEW ST.  
(BRIDGE #42)  
ABUTMENT FOUNDATION-LAYOUT

SHEET

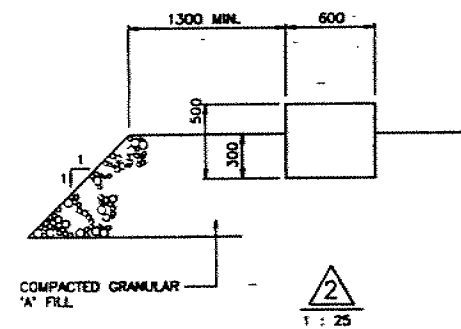
**R.E. Winter & Associates Ltd.**  
Consulting Engineers, Architects, Planners and Landscape Architects  
200 AVENUE BRIDLEWATER, MISSISSAUGA, ONT. L4V 1V1 TEL. (416) 670-8800



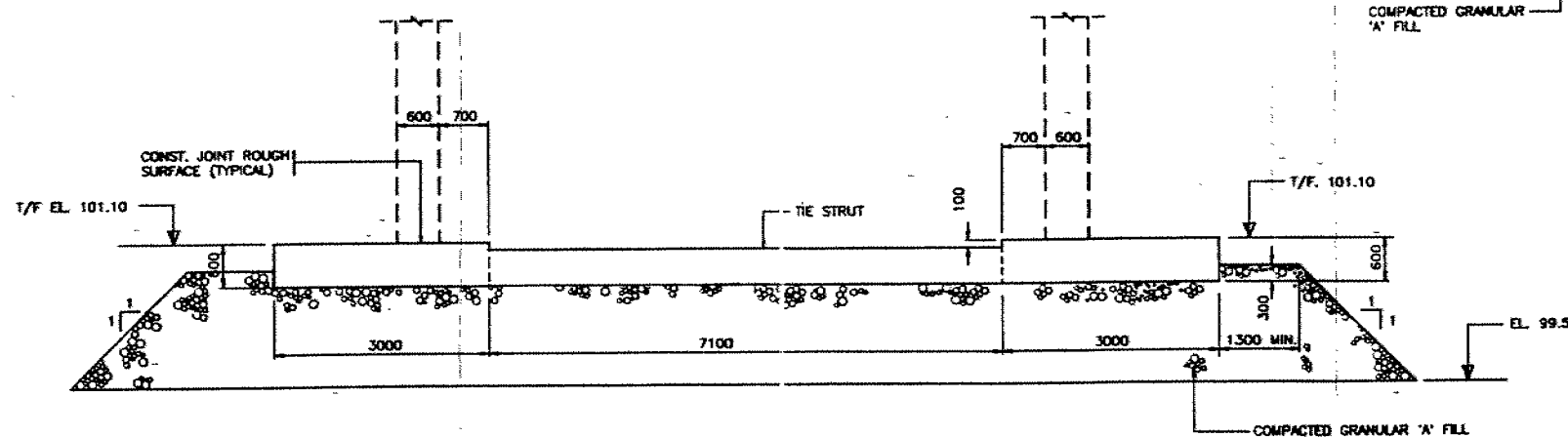
PLAN  
1 : 100

WORKING POINT DATA

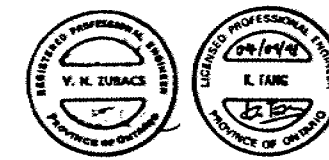
W.P.	CO-ORDINATES	
	NORTHING	EASTING
1	4799175.650	277977.571
2	4799208.741	277963.855
3	4799177.262	277967.657
4	4799210.353	277953.995
5	4799184.331	277973.933
6	4799196.978	277959.516



1 : 25



1 : 50



DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY
DESIGN DB	CHK VZ/KT	CODE OHBDC-83
DRAWN MU	CHK KT	SITE 10-334
		STRUCT
		SCHEME
		DWG. 03

LOAD CLASS A DATE OCT. 1991

# memorandum



To: K.G. Bassi  
Head, Design Section  
7th Floor, Atrium Tower

Date: 1992 02 20

Atten: G. Al-Bazi

From: Foundation Design Office  
Room 315, Central Bldg.

Re: Bearing Capacity Values  
Freeman Interchange  
W.P. 199-77-02  
W.P. 199-77-03  
W.P. 199-77-04 ✓  
W.P. 199-77-05  
W.P. 199-77-07  
W.P. 199-77-08  
District #4 (Burlington)

As per our discussion on 92 02 11 the spread footings may be placed on the undisturbed hard lower glacial till or on the underlying bedrock, using a factored bearing capacity of 1000 kPa at ultimate limit states and 500 kPa at serviceability limit states.

A handwritten signature in black ink, appearing to read "P. Payer".

P. Payer, P. Eng.  
Sr. Foundation Engineer

for

M. Devata, P. Eng.  
Chief Foundation Engineer

MD/PP/mmj



Ministry  
of  
Transportation

Ministère  
des  
Transports

Foundation Design Section  
Engineering Materials Office  
Room 315, Central Building  
1201 Wilson Avenue  
Downsview, Ontario  
M3M 1J8

Tel: (416) 235-3731

January 9, 1992

McCormick Rankin  
Consulting Engineers  
2655 North Sheridan Way  
Mississauga, Ontario  
L5K 2P8


Attn: Mr. K. Woon-Fat, P. Eng.

Dear Sir,

Re: Review of Final Drawings  
Freeman Interchange, Bridge 42  
Hwy. 403 and Q.E.W. Complex  
W.P. 199-77-04, Site 10-334  
District 4 (Burlington)

We have reviewed the Final Structural Drawings (#1 & # 3) for this project and the following comments are offered:

- a) The abutments will be supported on spread footings placed on well compacted granular material and should adhere to the geometry as outlined on the attached figure.
- b) All the dimensions should be shown on the Structural Drawings.

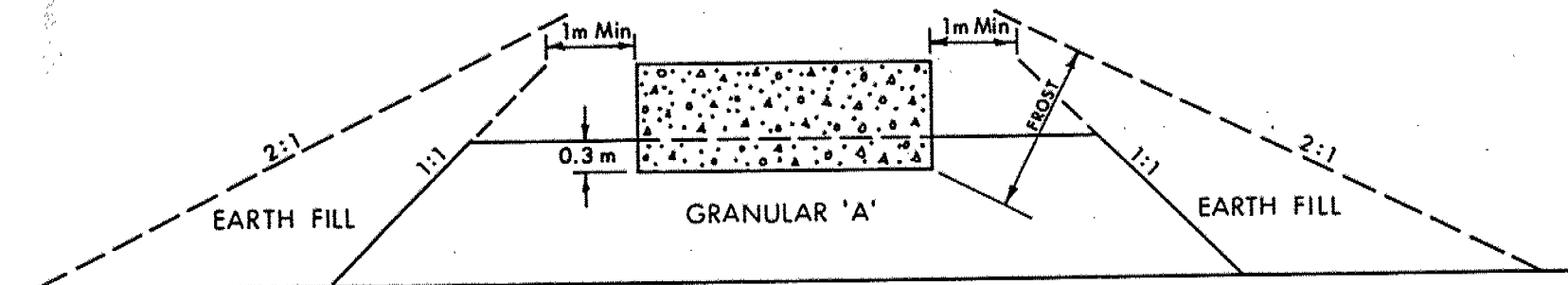
  
P. Payer, P. Eng.  
Sr. Foundation Engineer

for

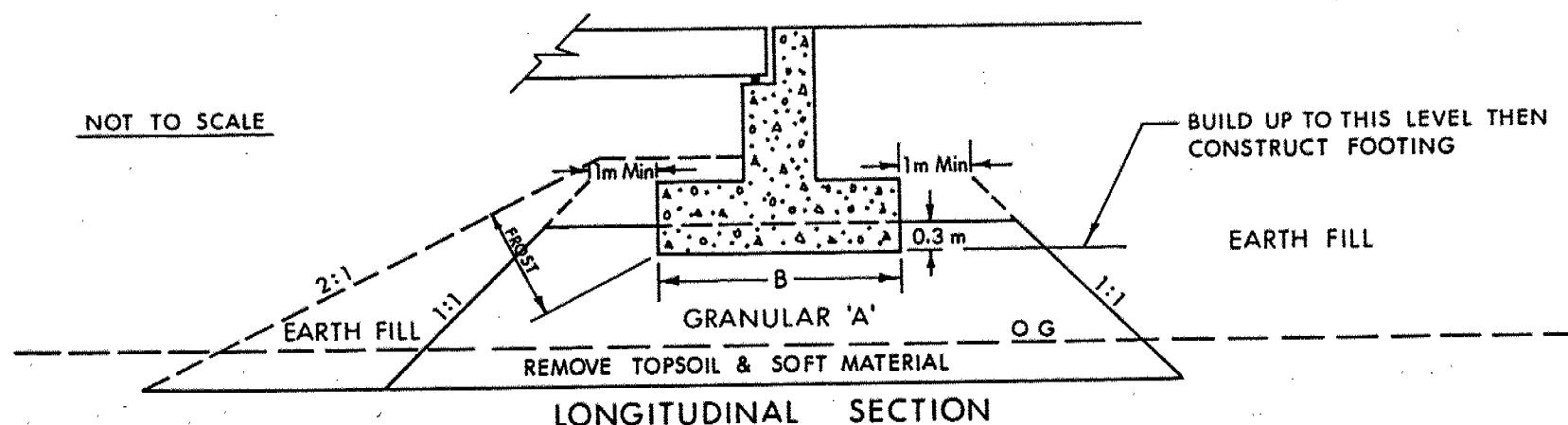
M. Devata, P. Eng.  
Chief Foundation Engineer

MD/PP/jb

cc: K. Bassi



X SECTION



LONGITUDINAL SECTION

NOTES:

- 1- REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
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Ontario

Ministry of  
Transportation

ABUTMENT ON COMPACTED FILL  
SHOWING GRANULAR 'A' CORE

FIG No

W P



**Golder Associates Ltd.**  
CONSULTING ENGINEERS

March 6, 1991

Our ref: 901-6039

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

Attention: Mr. P. Payer, P. Eng.

RE: BRIDGE NO. 42  
W.P. 199-77-04 SITE 10-334  
BURLINGTON, ONTARIO



Dear Sir:

We are in receipt of a January 31, 1991 facsimile transmission from Mr. Bendayon (Structural Section) regarding a perched abutment alternative for Bridge 42. The facsimile transmission was received in our office on March 4, 1991. This letter provides our comments as requested in the facsimile transmission.

*F.P. ✓ Could you brief Mr. Payer?*  
*Thank you*

1. PERCHED ABUTMENTS

The information provided shows the rigid frame founded on spread footings bearing in engineered fill at about elevation 100.5. The preliminary information previously provided had addressed spread footings bearing in the glacial till at about elevation 99.5 metres.

The perched abutment alternative is considered feasible from a geotechnical engineering standpoint provided it is constructed to current MTO specifications and as outlined below.

In preparation for constructing the engineered fill, it will be necessary to excavate all of the existing fill and topsoil and to expose a competent native subgrade beneath areas of structural fill. Based on the boreholes drilled at the site, a suitable native subgrade will probably be exposed at between about elevations 99.5 and 101 metres. The engineered fill should be placed and compacted in accordance with the applicable MTO directives and

as shown in the attached Figure.

Design bearing pressure of 350 kPa Serviceability Limit States and 900 kPa Ultimate Limit States may be assumed for perched abutments constructed as outlined above.

Some differential settlement can be expected between the abutment footings founded on the engineered fill. For design purposes, differential settlements in the range of about 10 to 20 millimetres should be considered. It should be noted however that poor construction techniques could result in greater settlements.

## 2. Reinforced Earth Retaining Wall

It is understood that consideration is being given to the use of a reinforced earth retaining wall adjacent to the rigid frame. It is proposed to found the reinforced earth wall at about elevation 100.8 metres. This will result in the wall being founded on existing earth fill materials which are variable in nature and consistency. Some post constructing settlement of the wall will therefore occur. Differential settlements are also anticipated, however we understand that reinforced earth walls are able to tolerate some post construction deformations.

As a minimum, preparations for the construction of the wall should consist of stripping all of the topsoil and any highly organic or soft fill beneath the walls. The exposed subgrade should be adequately proofrolled to identify any areas of the subgrade requiring remedial work. Any over excavated areas should be backfilled with good quality earth fill and uniformly compacted to at least 98 per cent of standard Proctor maximum dry density.

The final design details of the reinforced earth walls should be reviewed by the geotechnical engineer.

We trust that this letter adequately addresses the points raised by Mr. Bendayan. We will incorporate these comments into our geotechnical report which is presently being finalized. If there is any point requiring further clarification, please contact our office.

Yours truly,

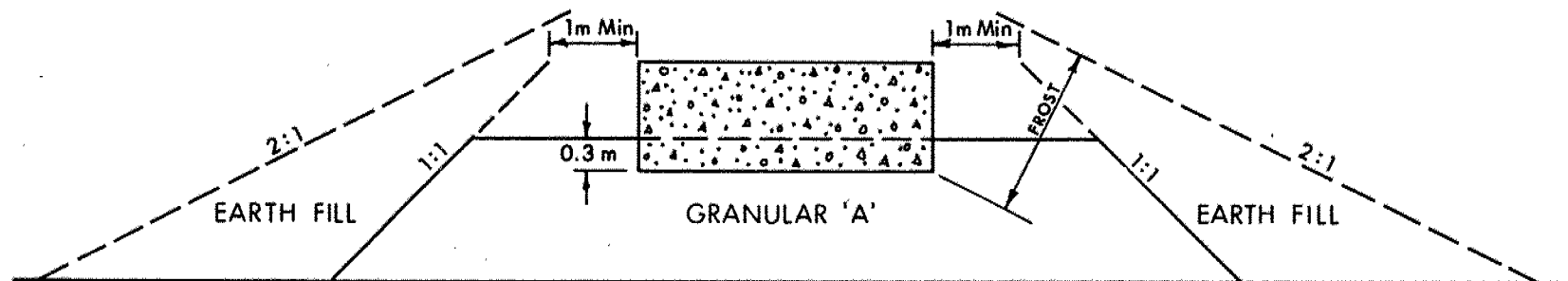
GOLDER ASSOCIATES LTD.



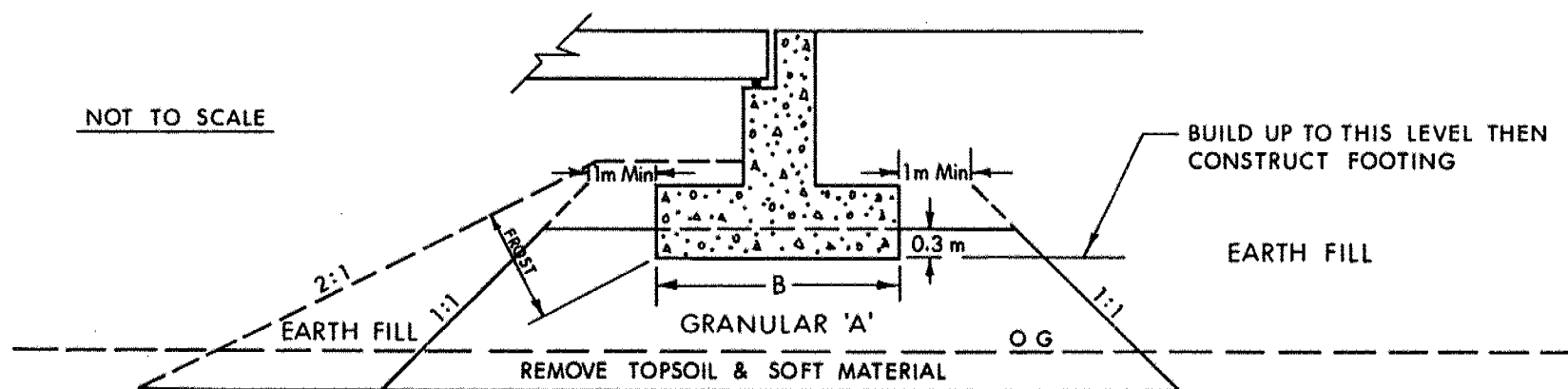
J.G. Muckle, P. Eng.

JGM/apf





X SECTION



LONGITUDINAL SECTION

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Ontario

Ministry of  
Transportation

ABUTMENT ON COMPACTED FILL  
SHOWING GRANULAR 'A' CORE

FIG No

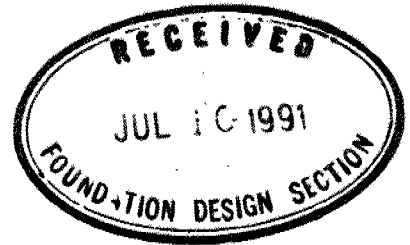
W P 199-77-04



Consulting Engineers, Architects  
Planners & Landscape Architects

200 Matheson Boulevard West  
Mississauga, Ontario, L5R 3L7

TEL: (416) 890-0110  
FAX: (416) 890-0319



## FACSIMILE TRANSMISSION SHEET

To: MINISTRY OF TRANSPORTATION  
FOUNDATION SECTION  
3/E CENTRAL BUILDING, DOWNSVIEW

Date: 10<sup>th</sup> JULY, 1991  
Re: RAMP 4025 - TEN AVENUE  
RAMP 407N - 402N (STAIRCASE 42)  
N.P. NO. 199-77-04  
SITE 10-334

Fax No: 416-524-5240

Attn: MR. PAUL PAYER

Total number of pages sent 4 including cover sheet

Dear Mr. Payer,

Further to our discussion with Mr. Morris Bandayan (Central  
Region) on the design parameter of the reinforced earth retaining  
walls of the captioned contract, we are advised to ask for your  
assistance.

Please advise on the suitable value to be used for the  
internal frictional angle at the interface between the base of the  
reinforced earth volume and native ground. Please refer to the  
enclosed sketches sk11, sk12 & sk13 for details of the walls.

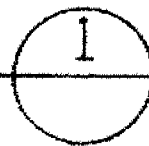
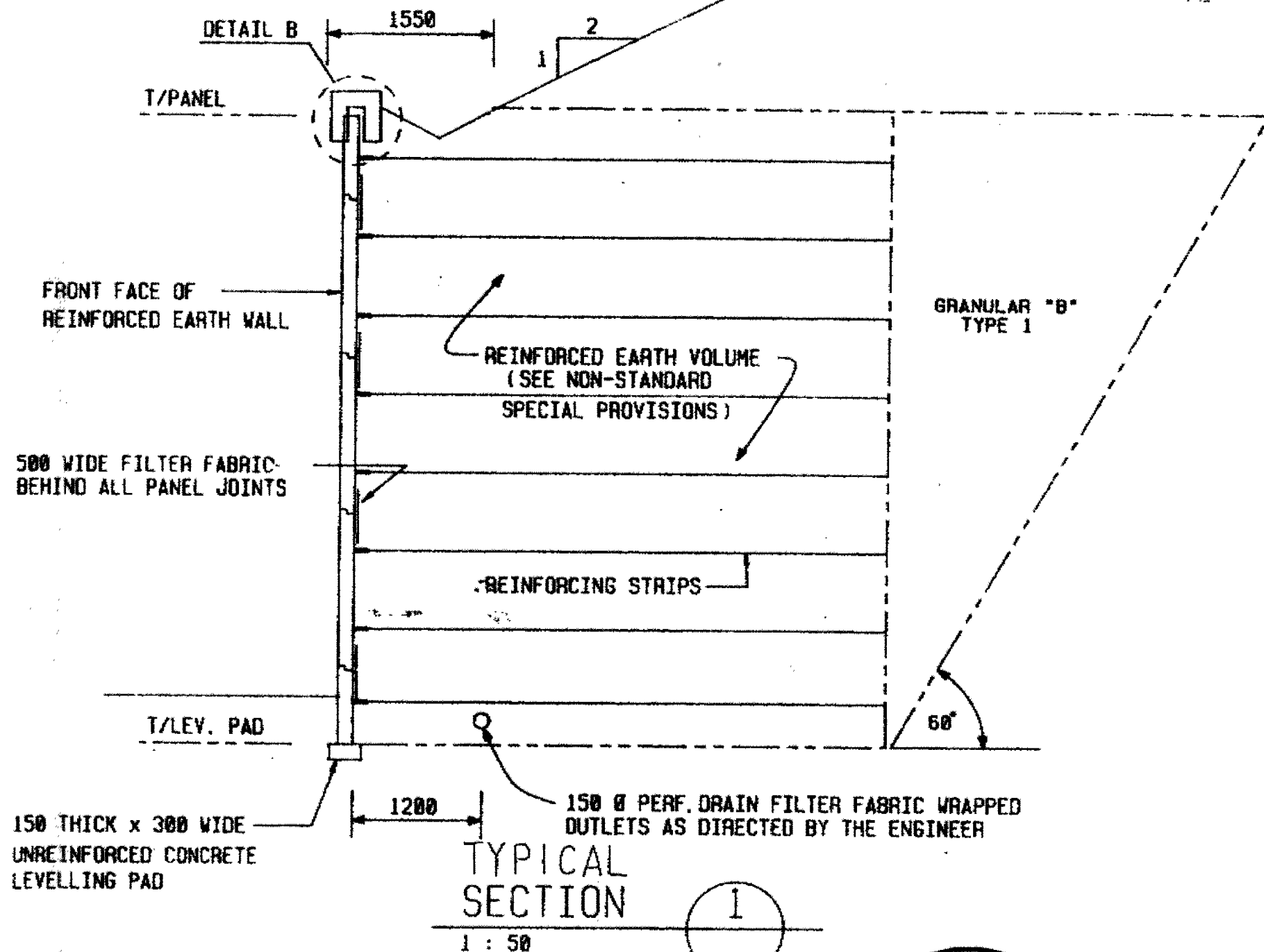
Regards,

Please advise if all pages not received

Fax sent by: DANIEL IP

SKETCH SK 13

TYPICAL SECTION OF WALL



PROFESSIONAL

REGISTERED

JUL 10 '91 10:53

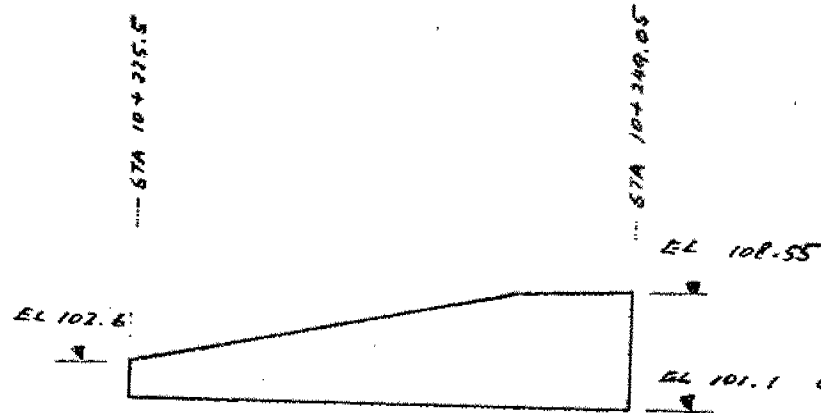
SKETCH SK 12  
DETAILS OF R/E WALLS

Applied bearing pressure:

200 kPa (SLS)

260 kPa (ULS)

Length of strips: 3m to 6m



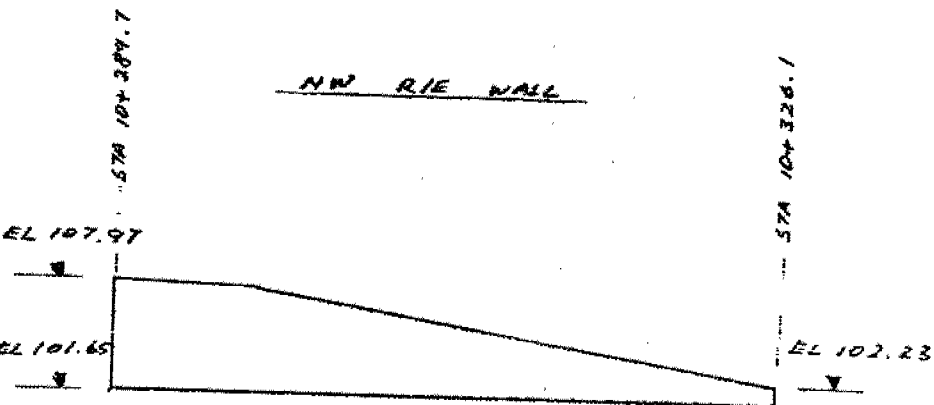
NW R/E WALL

Applied bearing pressure: EL 107.97

160 kPa (SLS)

220 kPa (ULS)

Length of strips: 3m to 5m



SE R/E WALL

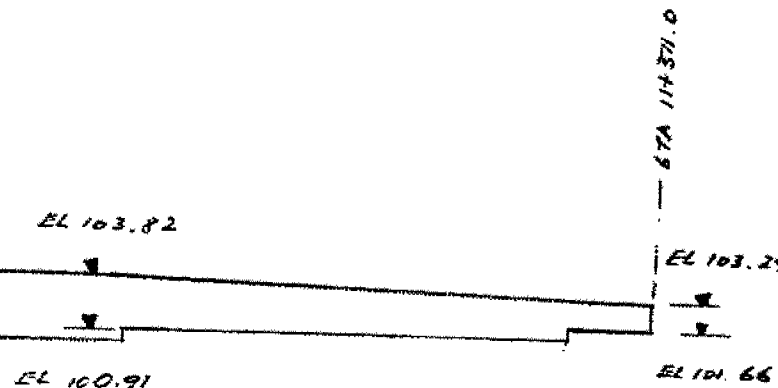


Applied bearing pressure:

100 kPa (SLS)

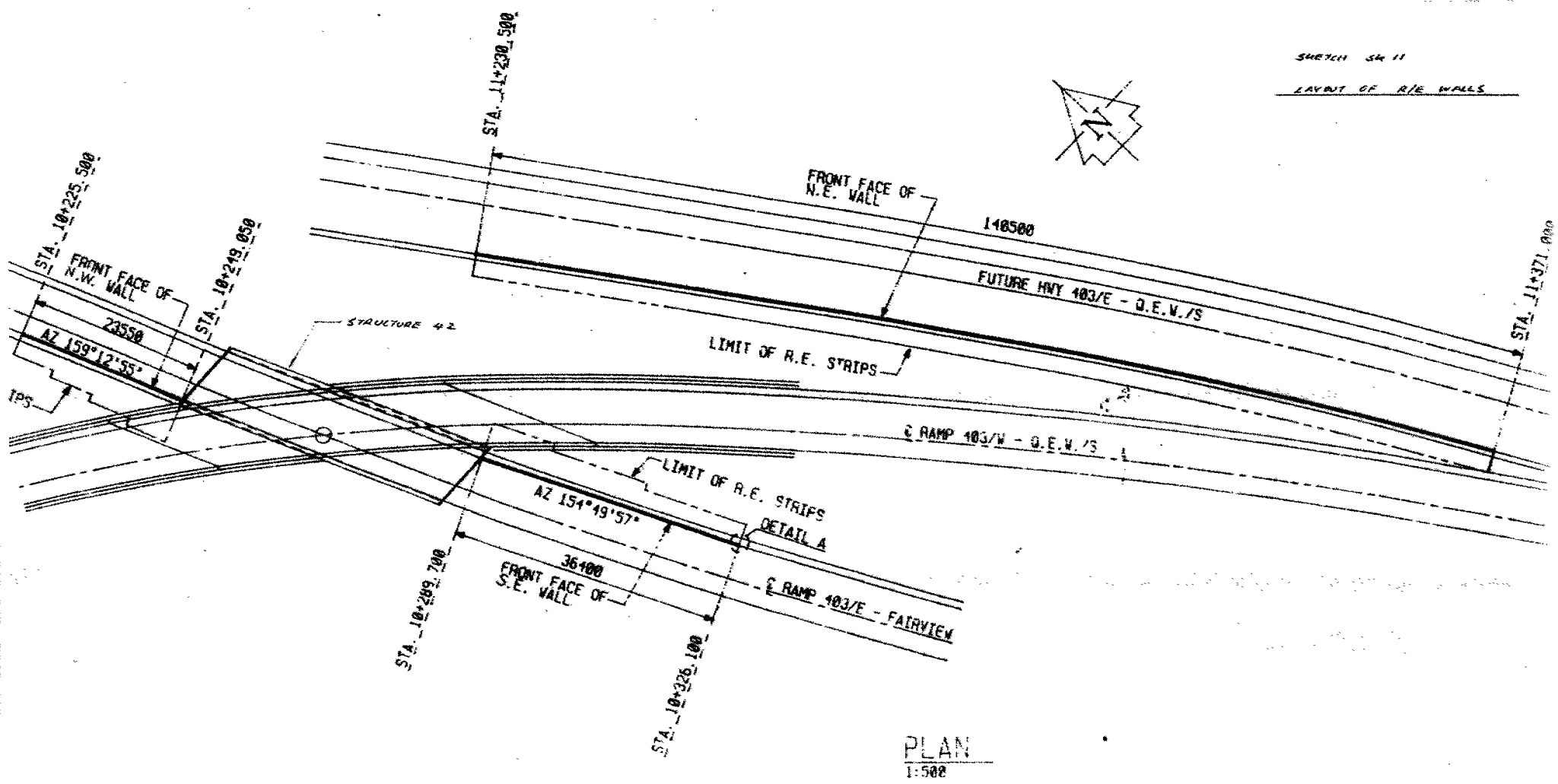
130 kPa (ULS)

Length of strips: 3m to 3.5m



NE R/E WALL

JUL 10 '91 10:52  
416 B900320 PAGE.002

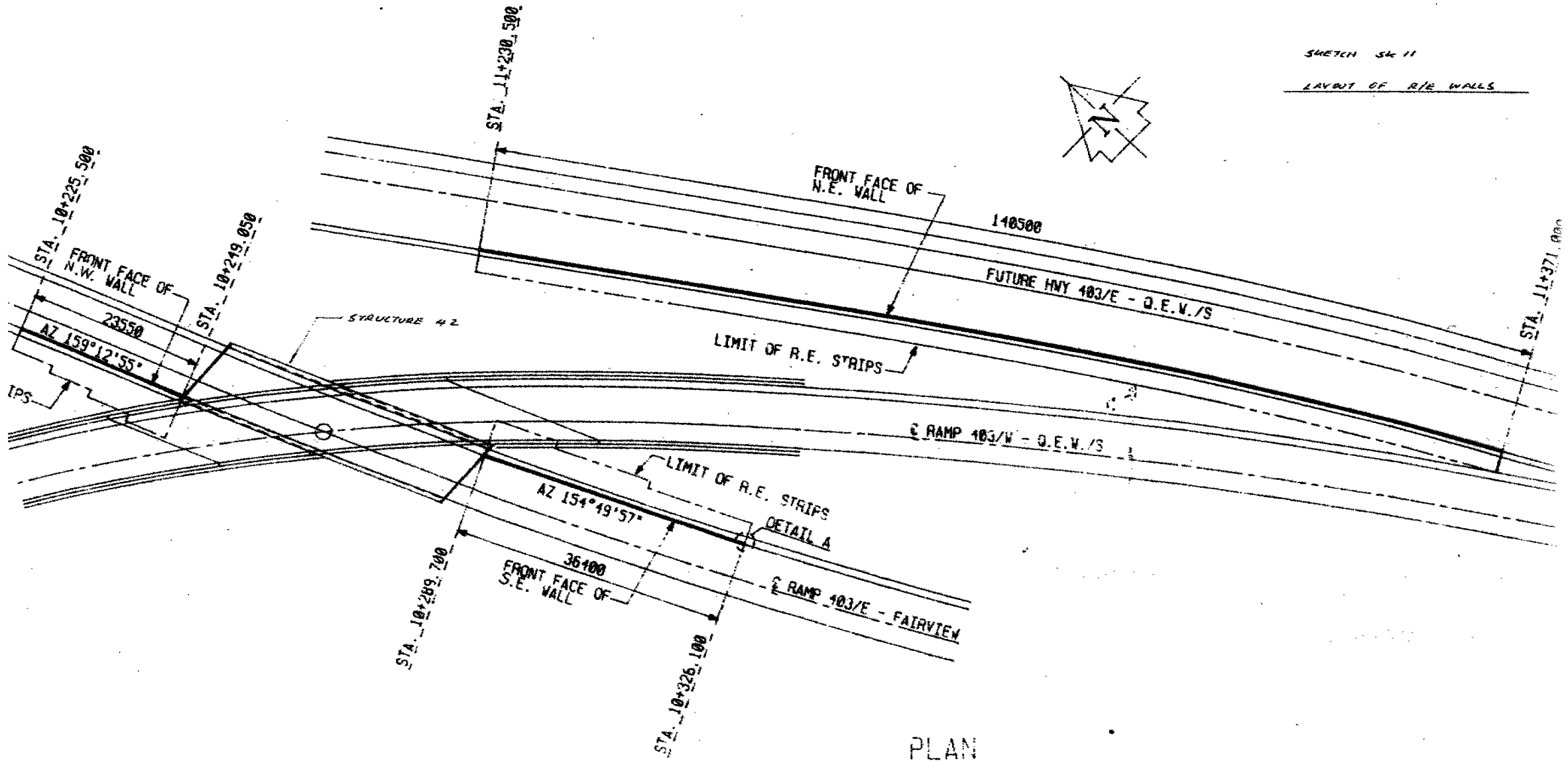


SKETCH SH 11  
LAYOUT OF R/E WALLS

PLAN  
1:500

SKETCH SK 11

LAYOUT OF R/E WALLS



PLAN  
1:500



Engineering Materials Office  
Foundation Design Section  
Room 315, Central Bldg.  
1201 Wilson Avenue  
Downsview, Ontario  
M3M 1J8

Tel: (416) 235-3731

1991 07 31

McCormick Rankin  
Consulting Engineers  
2655 North Sheridan Way  
Mississauga, Ontario  
L5K 2P8

Attn: Mr. K. Woon-Fat, P. Eng.

Re: Review of Preliminary Drawing  
Hwy. 403 Advance Structures  
Ramp 403/W - Q.E.W./S over  
Ramp 403/E - Fairview Street  
W.P. 199-77-04, Site 10-334  
District 4, Burlington  
Your File: W.O. 2204-90

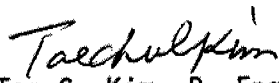
Further to your letter dated July 17, 1991, the General Arrangement Drawing P1 for the aforementioned structure has been reviewed by this office.

Based on our review, it is concluded that the design confirms to our recommendations. However, the following corrections and comments should be made on the details shown on the above mentioned drawing.

- 1) 75 mm Mass. Concrete is not required on Granular 'A' Pad.
- 2) Inconsistencies with dimensions in Cross-Section 4.
  - a) Width of footing indicated on drawing is 3000 mm.  
Actual measurement to scale measures 2000 mm
  - b) Inside dimensions of the structure indicated on drawing is 8500 mm.  
Actual measurement to scale measures 8250 mm.
- 3) Unsuitable materials underneath the proposed embankment file should be excavated before fill placement.

4) Due to the existence of Permeable layer within the fill adequate control at ground of water during construction is essential to minimize the deterioration of the founding soil.

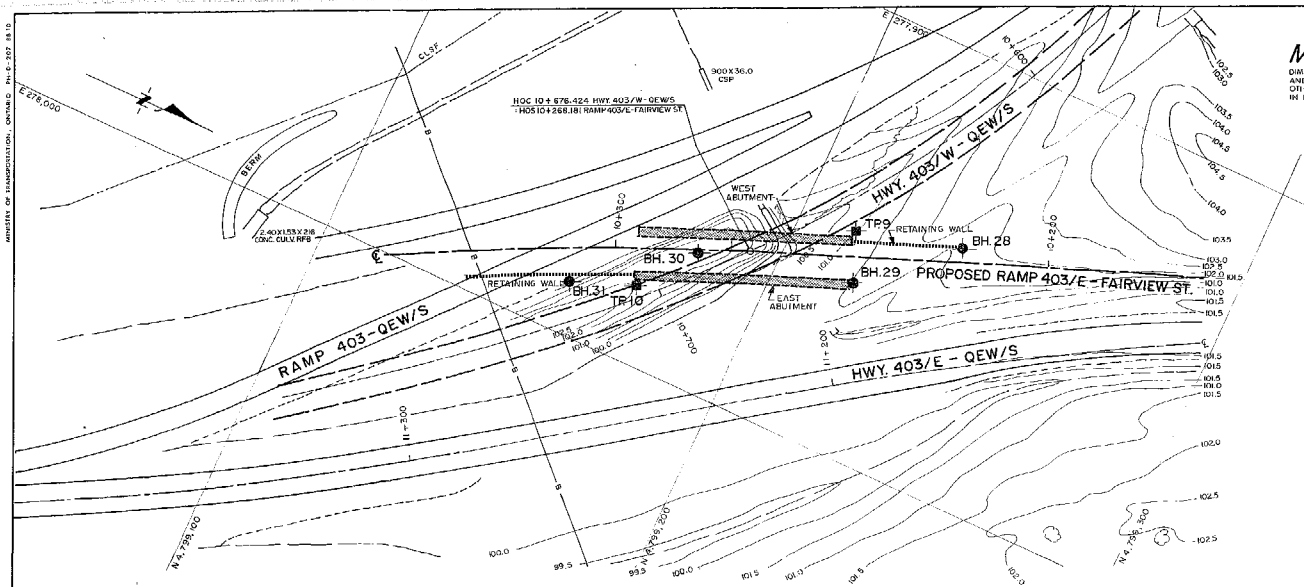
We have no further comments. If you have any questions, please contact this office.

  
Tae C. Kim, P. Eng.  
Sr. Foundation Engineer

TCK/me

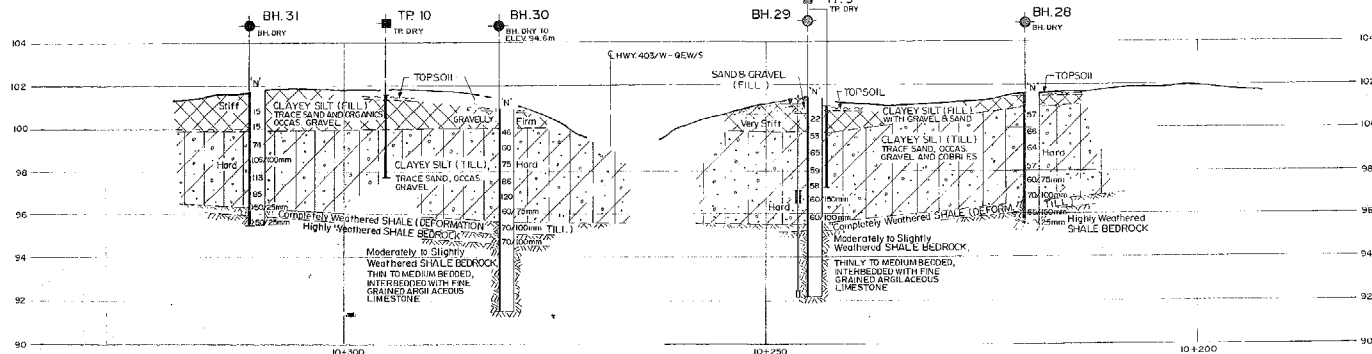
cc: M. Bendayan





PLAN

10m 5 0 10 20 40 Metres



PROFILE, RAMP 403/E - FAIRVIEW ST.

HOR. 5m 2.5 0 5 10 20 Metres  
VERT. 2m 1 0 2 4 8 Metres

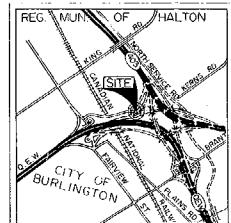
**METRIC**

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES, GRADES  
AND ELEVATIONS IN METRES  
AND/OR FEET. DIMENSIONS  
IN MILLIMETRES - 1/8 INCHES

CONT No  
WP No 199-77-04

BRIDGE No 42  
(HWY. 403/W - QEW/S OVER  
RAMP 403/E - FAIRVIEW ST.)  
BORE HOLE LOCATIONS & SOIL STRATA

GOLDER ASSOCIATES LTD.  
CONSULTING ENGINEERS



KEY PLAN  
SCALE  
0 0.5km 1km

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 August, 1990
- Test Pit
- Piezometer

No	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
BH. 28	101.6	4,795,257.5	277,943
BH. 29	101.4	4,795,218	277,961
BH. 30	100.8	4,799,182	277,970
BH. 31	101.7	4,795,158	277,988
TP. 9	101.0	4,795,215.5	277,950
TP. 10	101.5	4,795,172.5	277,982

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, University. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

DATE	BY	DESCRIPTION
Geocodes: No. 50M5-176		
HWY No. 403 / QEW	DATE DEC 28/90	SITE 10-334
SUBMIT GM	CHECKED	APPROVED
DRAWN MW	CHECKED	DWG 1997704-A