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HWY. No. 403

LOCATION Ramp 403/W-Q.E.W./E over
Q.E.W./RB & SB & over Q.E.W/S-403/E
No. of PAGES - (Bridge #41)

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



Golder Associates Ltd.

CONSULTING ENGINEERS

REPORT

TO

MINISTRY OF TRANSPORTATION ONTARIO

GEOTECHNICAL INVESTIGATION

PROPOSED BRIDGE 41

SITE 10-333

FREEMAN INTERCHANGE

BURLINGTON, ONTARIO

W.P. 199-77-03

DISTRICT 4

CONT 93-89

GEOCRES # 30M5-175

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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. 41 to be constructed as part of the 403/QEW East ramp as shown on the Key Plan, Drawing No. 1997703-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 site of the proposed Bridge 41 is located in a relatively flat, grassed area, south of the east bound 403/QEW ramp, west of the QEW north/QEW east ramp and east of the QEW north/403 west ramp in Burlington, Ontario. The location of the site is shown on the Key Plan, Drawing 1997703-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. 41 will carry 403 west/QEW east ramp traffic over the QEW East bound lanes and the QEW/403 north ramp.

The proposed bridge will consist of a three span structure (spans of approximately 45, 56 and 47 metres west to east respectively), with two abutments and two piers. Approach fills of some 11 to 13 metres in height will be required.

3. INVESTIGATION PROCEDURE

The field work for this investigation was carried out on August 18 and August 28, 1990 at which time one test pit was excavated and five boreholes were drilled. The locations of the boreholes and test pits are shown on Drawing 1997703-A, enclosed.

The initial stage of the field work consisted of excavating a test pit (numbered 8) near the east limit of the proposed bridge. The test pit was excavated to a depth of about 3.6 metres using a "John Deere 690" hydraulic backhoe supplied and operated by a local contractor. Chunk samples were obtained from the predominant soil strata exposed in the test pit and the test pit was loosely backfilled following sampling and logging.

The boreholes were drilled using track mounted power auger drillrigs supplied and operated by a specialist drilling

contractor. Boreholes numbered 24, 25 and 26 were advanced to bedrock, through about 2.5 to 3.5 metres of overburden, and the shale bedrock encountered beneath the overburden was core drilled in NQ size for about 3 metres in these boreholes. Boreholes numbered 23 and 27 were drilled to practical auger refusal. The boreholes were advanced within the overburden and the highly weathered shale using both nominal 150 millimetre 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 pit 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 pit are shown in detail on the Records of Boreholes and Record of Test Pit following the text of this report and on Drawing 1997703-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 25 as detailed on the Record of Borehole sheet. Notes pertaining to the groundwater conditions encountered in the boreholes and test pit are also shown on the Record of Borehole and Test Pit

sheets and on Drawing 1997703-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 pit 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 1997703-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, strata of sandy silt to silty sand, and completely to highly weathered shale underlain by more competent shale bedrock.

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 all of the boreholes and in the test pit put down at the site.

4.3 Clayey Silt, trace sand, trace organics
occasional gravel (Fill)

Fill generally consisting of clayey silt was encountered beneath the granular road base in test pit 8 and below the surface topsoil in boreholes 26 and 27. The thickness of the fill material ranged from about 1.6 metres in test pit 8 to about 1.9 metres in borehole 27. The fill generally consists of reddish brown to brown clayey silt to silty clay with traces of organics, sand and occasional gravel. In borehole 27 a 0.5 metre layer of black silty sand fill was encountered below the clayey silt fill.

The clayey fill had an in-situ water content of about 14 per cent. N-values of 12 and 18 blows per 0.3 metres were determined in standard penetration testing carried out in the fill.

4.4 Clayey Silt, trace to some sand,
occasional gravel (Till)

Clayey Silt till was encountered in all of the boreholes and in the test pit and was generally characterized by a brown upper zone and a lower zone of reddish brown clayey silt till which was somewhat harder and slightly coarser than the overlying till.

The N values determined in the upper till zone ranged from 28 to 112 blows per 0.3 metres. The natural water content of samples of the upper till recovered from the boreholes and test pit ranged from about 10 to 20 per cent.

Standard penetration testing was also 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 13 per cent.

The corresponding liquid and plastic limits of the lower till were about 20 and 15 per cent, respectively, based on the average of three Atterberg Limit determinations shown on the plasticity chart, Figure 2. Grain size distribution curves for samples (obtained by a 35 millimetre I.D. sampler) of the lower till are shown on Figure 1.

The till material as indicated by the gradation curve is 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 in concentrations or at random within the deposit, since till is an inherently variable material.

4.5 Sandy Silt, Silty Fine Sand

Layers of sandy silt and silty fine sand were encountered beneath the upper till in borehole 23 and below the lower till in borehole 27. These strata were about 1 and 0.4 metres thick and were very dense with inferred N values in the order of greater than 100 blows per 0.3 metres. The natural water content of these strata was about 16 per cent.

4.6 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 98.4 and 100 metres, or at depths of from about 2.4 to 4.7 metres below the existing ground surface.

The upper zone of the bedrock is generally highly weathered and has been described as deformation till in boreholes 24, 26 and 27. More competent shale was encountered in boreholes 24, 25 and 26 between about elevations 97.5 and 99 metres, or at depths of about 3 to 4.5 metres below existing ground surface. The bedrock core recovered from boreholes 24, 25 and 26 generally consists of moderately to slightly weathered thinly bedded reddish-brown shale, interbedded with thinly bedded light grey, fine grained argillaceous limestone up to about 0.5 metres in thickness.

The rock core recovered from the boreholes generally exhibited a relatively high degree of fracturing. However the quality of the rock core recovered generally improved with depth. The total core recovery (TCR) ranged from about 84 to 97 per cent for the upper 1.5 metres of rock cored and was typically 100 per cent for the lower 1.5 metres of rock. The solid core recovery (SCR) ranged from about 37 to 76 percent for the upper 1.5 metres of rock core and from about 85 to 97 percent for the lower 1.5 metres of rock core. Similarly the rock quality designation (RQD) ranged from about 28 to 45 percent in the upper 1.5 metres of rock core compared to 67 to 75 percent for the lower 1.5 metres of rock core.

4.7 Groundwater Conditions

Groundwater was not encountered during the field drilling/digging operations in boreholes 23 and 27 which were terminated at practical auger refusal, or in test pit 8 which was terminated in the red (lower) clayey silt till. Boreholes 24 and 25 were dry to the depth of practical auger refusal, corresponding to about elevations 97.7 and 98.3 metres, respectively. Groundwater was encountered at about elevation 100.8 metres, or at about 2.1 metres below the existing ground surface in borehole 26. The groundwater level was measured at about elevation 101.5, or at a depth of about 0.3 metres below the existing ground surface, in the piezometer in borehole 25, about 1 week after the completion of drilling.

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

5. DISCUSSION AND DESIGN RECOMMENDATIONS

5.1 General

This section of the report provides our interpretation of the factual geotechnical data obtained during the investigation. The geotechnical engineering parameters given 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 contained herein as it affects their proposed construction methods, equipment selection, scheduling and the like.

5.2 Foundations

5.2.1 General

It is understood that the proposed finished deck grade for Bridge 41 will vary from about elevation 113.2 metres at the west abutment, to about elevation 117.5 metres at the east abutment. The finished grade beneath the bridge will vary from about 101.1 metres on the west side to about 106.7 metres on the east side.

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 within 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 the silty sand and sandy silt strata, at or below elevation 100 metres, or in the underlying bedrock, using a factored bearing capacity of 750 kilopascals at Ultimate Limit States and 500 kilopascals at Serviceability Limit States. The coefficient of friction between concrete and the bearing strata may be computed assuming an internal friction angle of 20° for the shale bedrock and 26° for the glacial till.

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 protection 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 confirm that the base has been adequately prepared and is free of any softened or disturbed zones.

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

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

5.2.3 Deep Foundations

It is understood that the anticipated construction sequencing will result in the existing ramp and QEW east bound lanes remaining in service although detoured locally at various stages.

For this reason, the use of deep foundations consisting of caissons socketed into the bedrock as an alternative to spread footings may offer some advantages from a construction space limitation view point.

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 settlements 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 900 millimetre diameter caissons founded as outlined above and with a tip elevation at or below about 96 metres.

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 be at about elevation 113.2 to about elevation 117.5 metres compared to the existing grades of 102.3 and 103.7 metres at the west and east abutments respectively. Consideration could also be given to constructing perched abutments founded on spread footings bearing on a properly engineered fill as shown on

Figure 3. Conceptually, the design consists of a core of well compacted Granular "A" material which will support the foundations and a shell of compacted earth fill.

In preparation for constructing the engineered fill, all existing fill, topsoil and soft, loosened or disturbed soil should be removed from beneath areas of "structural" fill. Areas requiring subexcavation can best be identified by the geotechnical engineer during proofrolling. Based on the results of the present investigation, it will probably be necessary to excavate to about elevation 102 metres or lower at the west abutment and to about elevation 101.7 or lower at the east abutment.

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 these movements.

For design purposes, differential settlements in the range of about 15 to 25 and 20 to 40 millimetres should be anticipated at the west and east abutments respectively. 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.

5.3 Earth Pressures for Abutments

The lateral earth loads acting on the abutments will depend 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.

- Select free draining granular fill in accordance with OPSS Granular "A" or "B" should be used as backfill immediately behind the abutments. The minimum limits of the granular backfill are outlined in section 6 - 9.6.1 of the Ontario Highway Bridge Design Code.
- All granular fill should be placed and compacted in accordance with the current OPSS practices and MTO directives.
- Longitudinal drains should be installed to provide positive drainage of the granular backfill
- If the abutment support allows lateral yielding at its top equal to not less than 0.05 per cent of the retained

height, 'active' earth pressure conditions apply. If, however, the structure is not permitted to yield by this amount, 'at rest' pressure conditions should be used. The following parameters may be used in the calculation of lateral earth pressures:

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 encounter fill, glacial till and possibly silty sand and shale bedrock.

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

As discussed previously, adequate control of the groundwater during construction is essential to minimize deterioration of the bearing strata during final excavation preparation of the bearing surface for placement of the protection concrete 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 about 11 to 15 metres will be required at the abutment locations. The subgrade encountered in the boreholes drilled at the proposed west abutment location consists of glacial till. About 1.7 metres of clayey silt fill and silty sand fill overlie the till at the east abutment. The clayey silt fill had a very stiff consistency and an in-situ water content of about 15 percent. The silty sand fill had a compact consistency and an in-situ water content of about 15 per cent.

In order to ensure satisfactory performance of the embankments both in terms of stability and settlement, consideration should be given to removing all of the existing fill prior to constructing the embankments. The existing earth fill is considered suitable for reuse in the construction of earth embankment fill.

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. In preparation for constructing embankment fill, the 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 170 to 350 millimetres. In addition differential settlements of about 50 percent of the above range should be anticipated due to consolidation of the existing fill beneath the proposed embankments. If settlements of this magnitude are not tolerable, consideration should be given to removing all of the existing fill prior to constructing the embankments as a means of reducing the differential settlement. Much of the existing earth fill is considered suitable for reuse in the construction of earth embankment fill.

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

Differential settlement 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.

Based on the embankment heights proposed differential settlement in the range of about 10 to 20 millimetres are anticipated. If the bridge structure is unable to tolerate differential movements in this range, consideration will therefore 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 pit put down in this investigation and in any other preconstruction investigations, should be located in the field by the contractor 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.



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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
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_a	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

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

RECORD OF BOREHOLE No 23

METRIC

W P 199-77-03 LOCATION N4,799,406.8 E278,067.0 ORIGINATED BY CB
 DIST 4 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY MHW
 DATUM Geodetic DATE August 24, 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					
102.3	Ground Surface																
102.1	Topsoil																
100.2	Clayey Silt trace sand, occasional gravel (Till)		1	SS	33												
99.6	Hard Brown		2	SS	61												
99.2	Silty sand, occ. gravel Very Dense red Brown		3	SS	70/150mm												
97.7	Sandy silt, tr. clay Very Dense Red Brown		4	SS	50/25mm												
97.7	Shale Bedrock Highly to moderately weathered		5	SS	50/25mm												
97.7	Reddish Brown																
4.6	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 25

METRIC

W P 199-77-03 LOCATION N4,799,436.9 E278,120.8 ORIGINATED BY CB
DIST 4 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers; NQ Rock Core COMPILED BY MHW
DATUM Geodetic DATE August 24, 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					
							O UNCONFINED + FIELD VANE				WATER CONTENT (%)			
							● QUICK TRIAXIAL x LAB VANE				10 20 30			
101.8	Ground Surface													
0.1	Topsoil													
	Clayey Silt trace to some sand, occ. gravel (Till)		1	SS	28									
	Very stiff to hard		2	SS	78/150mm									
99.4	Brown to reddish brown													
2.4	Shale Bedrock highly weathered													
98.8														
3.0	Shale Bedrock moderately to slightly weathered thinly to medium bedded Reddish Brown inter- bedded with grey fine- grained argillaceous limestone		3	SS	90/10mm									
			4	NQ RC	TCR=84% SCR=37% RQD=28%									
			5	NQ RC	TCR=100% SCR=89% RQD=71%									
95.4														
6.4	End of Borehole													
	* TCR: Total Core Recovery SCR: Solid Core Recovery RQD: Rock Quality Designation													

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 26

METRIC

W P 199-77-03 LOCATION N4,799,456.4 E278,158.3 ORIGINATED BY VCH
 DIST 4 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers; NQ Rock Core COMPILED BY MHW
 DATUM Geodetic DATE August 23, 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					
102.9	Ground Surface																
0.1	Topsoil																
	Clayey silt (Fill) trace sand & organics occasional gravel & asphalt		1	SS	26		102										
101.0	Very Stiff Brown		2	SS	19												
1.9	Clayey silt trace sand, occ. gravel (Till)		3	SS	112												
99.9	Hard Red Brown						100										
3.0	Shale (Deformation Till) completely weathered		4	SS	50/75mm												
99.2	Shale highly weathered																
3.7	Shale highly weathered																
98.8	Shale Bedrock moderately to slightly weathered thinly to medium bedded Reddish Brown interbedded with grey fine-grained argillaceous limestone		5	SS	105/10mm												
4.1			6	NQ RC	*TCR=97% SCR=57% RQD=42%		98										
			7	NQ RC	TCR=100% SCR=85% RQD=61%												
96.0	End of Borehole						96										
6.9																	

* TCR: Total Core Recovery
 SCR: Solid Core Recovery
 RQD: Rock Quality
 Designation

RECORD OF BOREHOLE No 27

METRIC

W P 199-77-03 LOCATION N4,799,469.1 E278,169.8
 DIST 4 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers
 DATUM Geodetic DATE August 23, 1990
 ORIGINATED BY VCH
 COMPILED BY MHW
 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					
103.7	Ground Surface																
103.5	Topsoil																
0.2	Clayey silt (Fill) tr. sand, occ. gravel, tr. topsoil		1	SS	18												
102.3	Very Stiff Brown																
1.4	Silty Sand (Fill)																
101.8	Compact Black		2	SS	12		102										
1.9	Clayey silt tr. to some sand, occ. gravel (Till)		3	SS	25												
			4	SS	35												
	Very stiff to hard at about Elev. 100.0 m		5	SS	50/75	mm	100										
99.4																	
4.3	Silty fine sand																
99.0	Reddish Brown																
4.7	Shale Bedrock (Deformation Till)		6	SS	70/150	mm											
98.3	completely weathered																
5.4	End of Borehole						98										

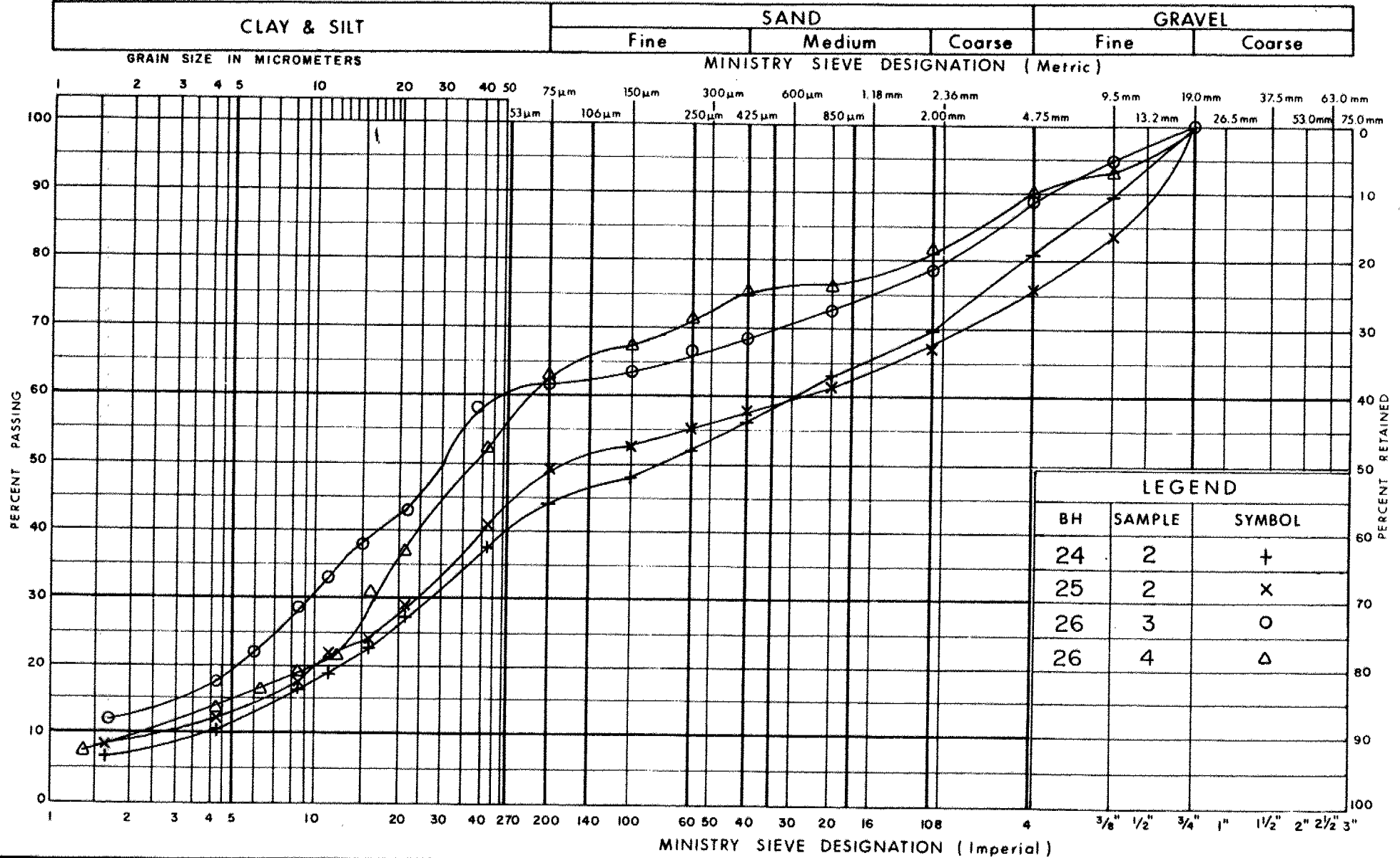
OFFICE REPORT ON SOIL EXPLORATION

METRIC

W P 199-77-03 LOCATION N4,799,458 E278,176 ORIGINATED BY VCH
DIST 4 HWY 403/QEW BOREHOLE TYPE Backhoe Dug "John Deere 690" COMPILED BY MHW
DATUM Geodetic DATE August 18, 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		SHEAR STRENGTH kPa				WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
103.8	Ground Surface													
103.5	Granular Road Base	X												
0.3	Clayey Silt, trace sand, occasional gravel (Fill)	X												
		X	1	CS										
102.2	Brown	X												
1.6	Clayey Silt, trace to some sand, occasional gravel, rootlets at top of layer (Till)	X	2	CS			102							
		X						Test pit dry following excavation						
		X	3	CS										
100.5	Brown	X												
3.3	Clayey silt trace sand (Till) Red Brown	X												
3.6	End of Test Pit						100							

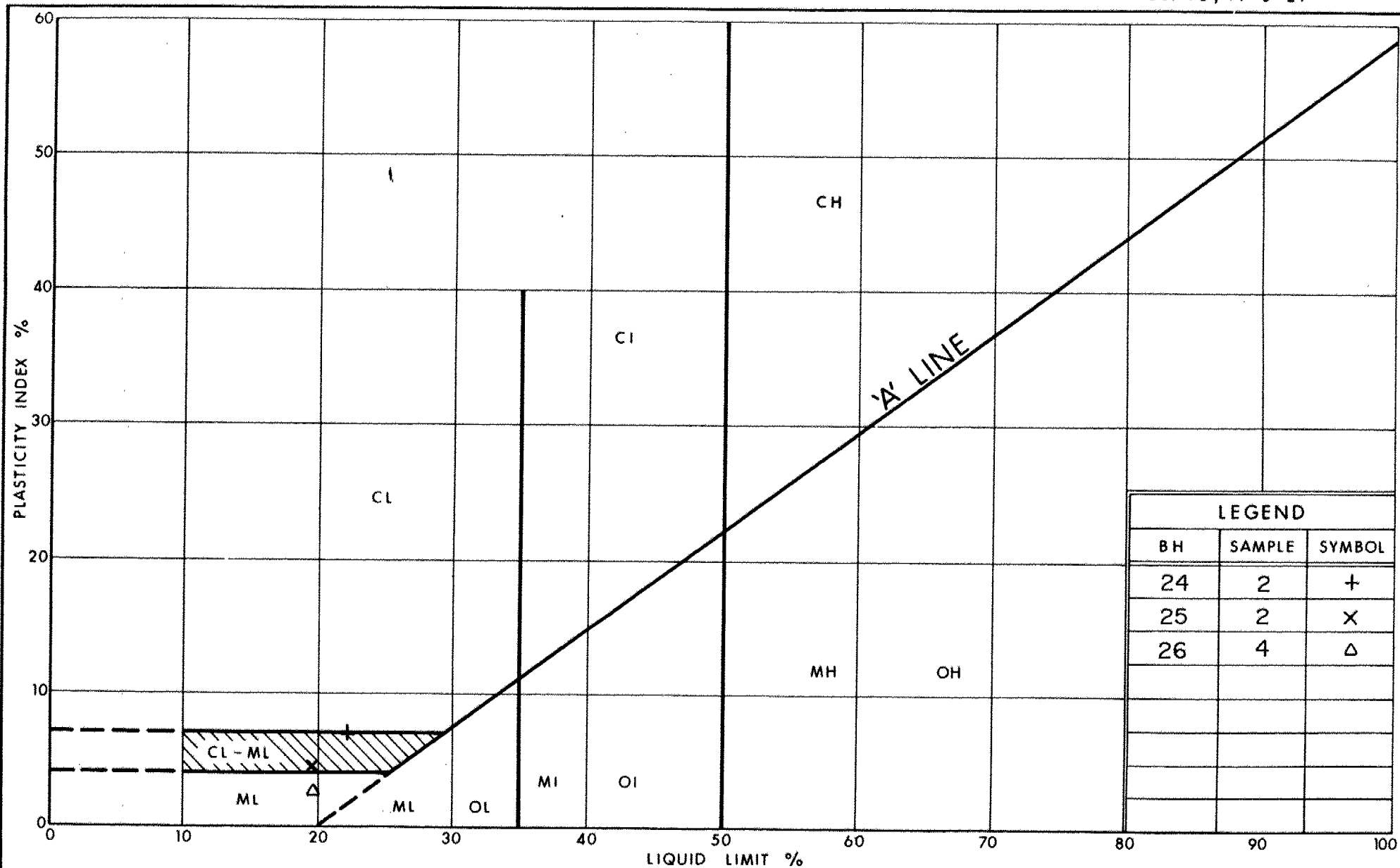
UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
CLAYEY SILT TILL

FIG No I
W P 199-77-03



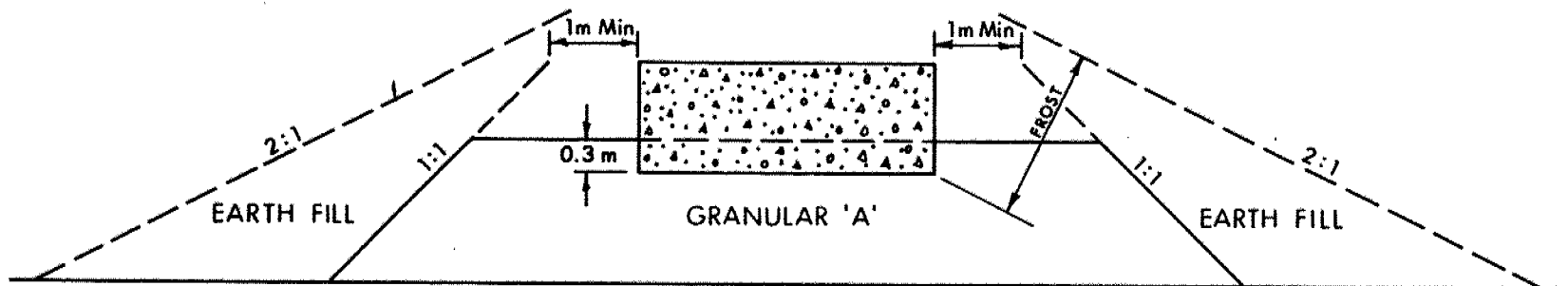
Ontario

Ministry of
Transportation

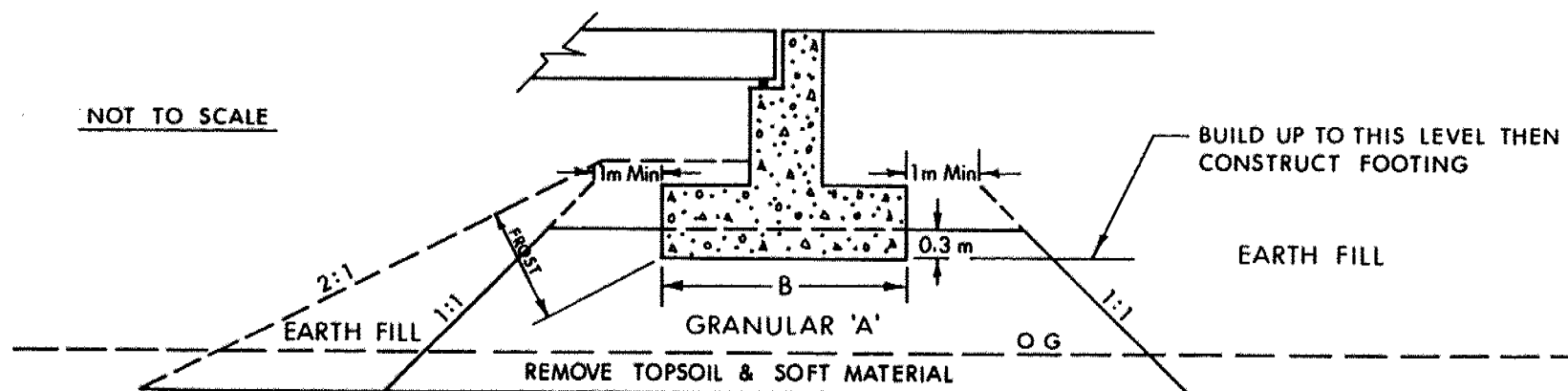
PLASTICITY CHART CLAYEY SILT TILL

FIG No 2

W P 199-77-03



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 199-77-03

SHEET

C

1. CLASS OF CONCRETE	
DECK AND PIER COLUMNS	35 MPa
REMAINDER	30 MPa

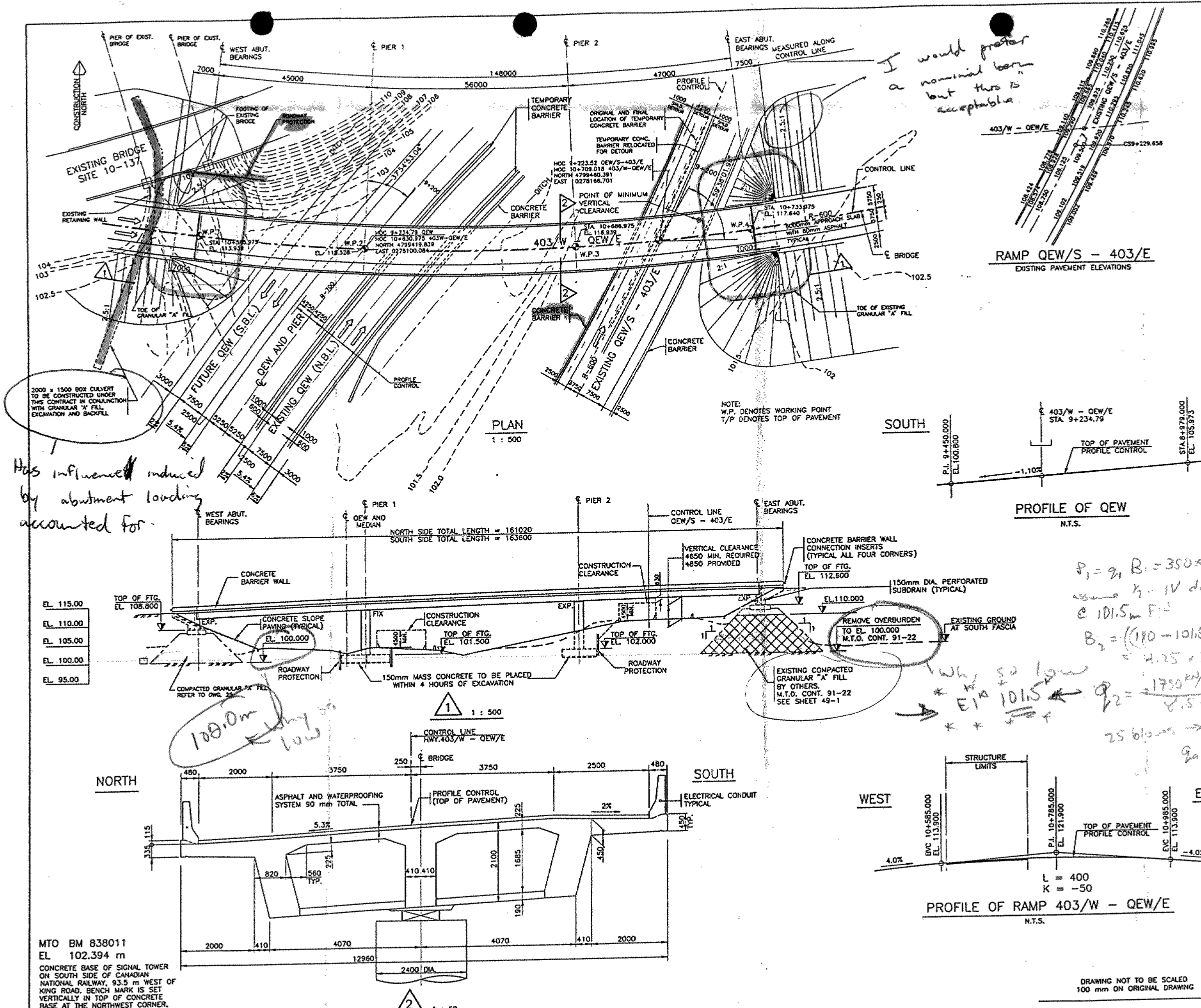
- ### LIST OF DRAWINGS

- 1 - GENERAL ARRANGEMENT
- 2 - BOREHOLE LOCATIONS AND SOIL STRATA
- 3 - ROADWAY PROTECTION
- 4 - FOOTINGS AND PIERS
- 5 - WEST ABUTMENT
- 6 - EAST ABUTMENT
- 7 - WINGWALLS
- 8 - BEARING DETAILS
- 9 - DECK DETAILS I
- 10 - DECK DETAILS II
- 11 - TRANSVERSE TENDONS I
- 12 - TRANSVERSE TENDONS II
- 13 - LONGITUDINAL TENDONS I
- 14 - LONGITUDINAL TENDONS II
- 15 - DECK REINFORCING I
- 16 - DECK REINFORCING II
- 17 - DECK REINFORCING III
- 18 - DECK REINFORCING IV
- 19 - BARRIER WALL
- 20 - JOINT ANCHORAGE & ARMOURING - WEST
- 21 - JOINT ANCHORAGE & ARMOURING - EAST
- 22 - 6000mm APPROACH SLAB
- 23 - DETAILS OF CONCRETE SLOPE PAVING
- 24 - AS CONSTRUCTED ELEV. AND DIM
- 25 - STANDARD DETAILS
- 26 - ELECTRICAL EMBEDDED WORK
- 27 - QUANTITIES STRUCTURE I
- 28 - QUANTITIES STRUCTURE II

DO-3503 - MINIMUM GRANULAR BACKFILL
REQUIREMENTS

DO-4602 - FALSEWORK CLEARANCES

REVISIONS									
	DATE	BY	DESCRIPTION						
	DESIGN M.E.S.	CHK J.T.L.	CODE - 1983-OHBDG		LOAD - CLASS A	DATE	NOV. 1991		
	DRAWN S.C.	CHK J.T.L.	SITE 10-333		STRUCT # 41	SCHEME	DWG 1		



- I would prefer
a nominal term
but this is
acceptable.

Has influence induced
by abutment loading
accounted for.

$P_1 = 21$ B: $= 350 \times 5 = 1750 \text{ kW}$
 assume k_2 IV distribution
 $\epsilon = 101.5 \text{ m F}$
 $B_2 = ((110 - 101.5) \times \frac{1}{2}) \times 2$
 $= 4.25 \times 2 = 8.5 \text{ m}$
 $\rightarrow P_2 = \frac{1750 \text{ kW}}{8.5 \text{ m}} = 205 \text{ kW}$
 $25 \text{ bl/s} \rightarrow cu = 125 \text{ kg}$
 $Q_{\text{tail}} = 2cu = 250 \text{ kW}$

MTO BM 838011
 EL 102.394 m
 CONCRETE BASE OF SIGNAL TOWER
 ON SOUTH SIDE OF CANADIAN
 NATIONAL RAILWAY, 93.5 m WEST OF
 KING ROAD. BENCH MARK IS SET
 VERTICALLY IN TOP OF CONCRETE
 BASE AT THE NORTHWEST CORNER,
 2.40 m SOUTH OF THE MOST
 SOUTHERLY RAIL OF MAIN LINE.

CONT No
WP No 199-77-03

RAMP 403/W - QEW/E (BR.41)
OVER
QEW AND RAMP QEW/S-403/E
ROADWAY PROTECTION



SHEET

Gregg and Edens Limited
consulting engineers & planners



NOTES:

ALTERNATIVE ROADWAY PROTECTION DESIGNS MAY BE CONSIDERED BY THE ENGINEER. ALTERNATIVE DESIGNS SHALL CONFORM TO THE FOLLOWING:

SOIL UNIT WEIGHT	=	20 kN/m ³
LIVE LOAD SURCHARGE	=	600mm OF FILL
ACTIVE EARTH PRESSURE COEFF.	K_a	= 0.33 AT SLS
	$(K_a)_f$	= 0.41 AT ULS
PASSIVE EARTH PRESSURE COEFF.	K_p	= 3.00 AT SLS
	$(K_p)_f$	= 2.44 AT ULS
COMPRESSIVE STRENGTH OF CONCRETE ENCASING	=	20 MPa

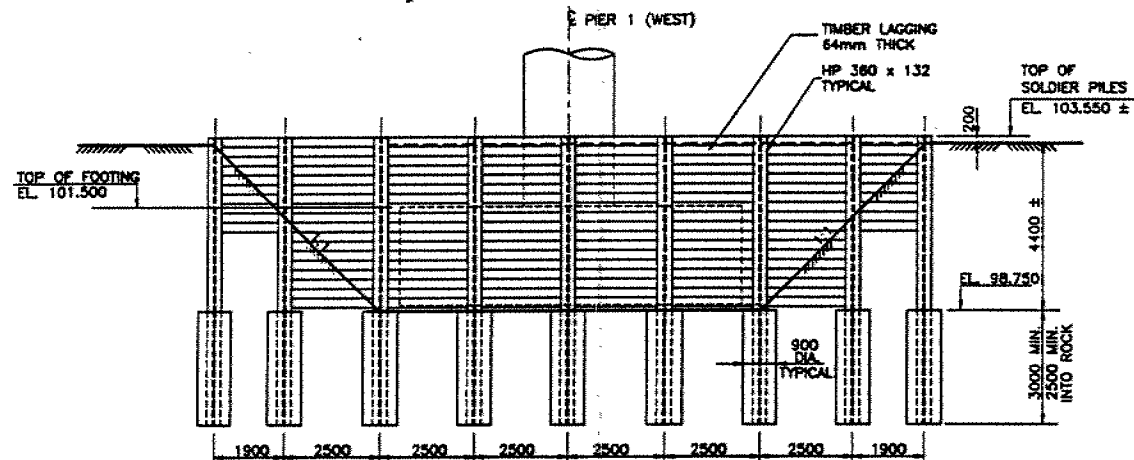
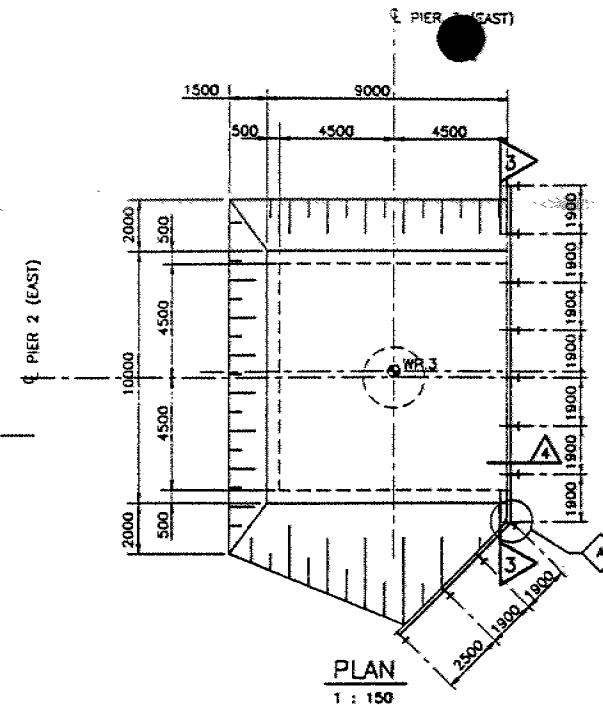
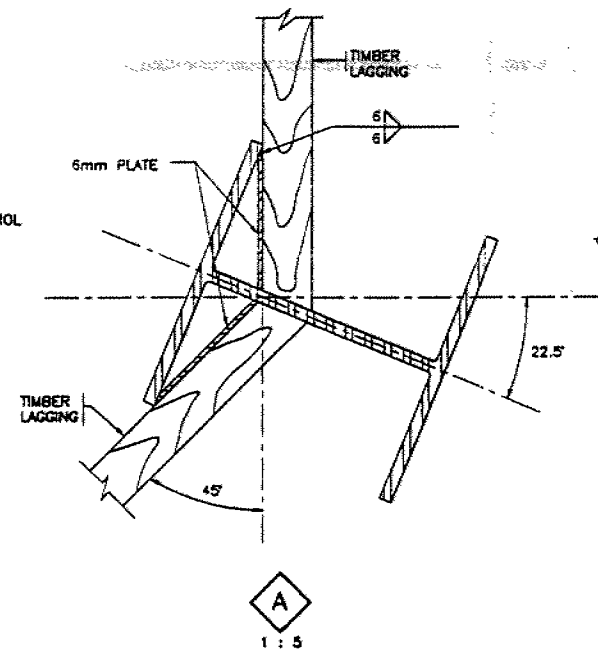
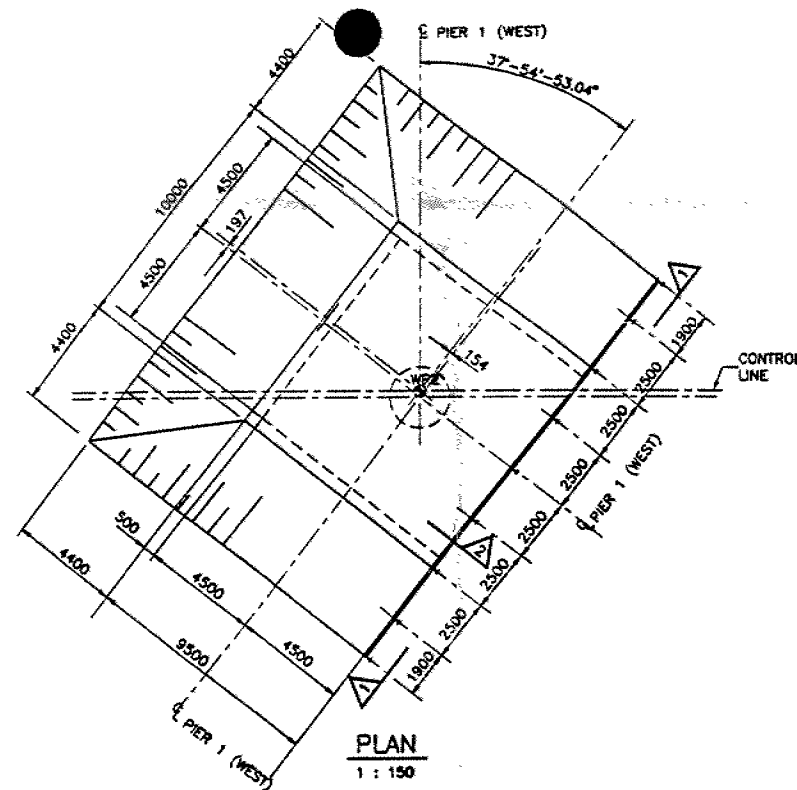
THE CONTRACTOR SHALL SUBMIT SIX COPIES OF CALCULATIONS AND DRAWINGS SHOWING FULL DETAILS OF ROADWAY PROTECTION TO THE ENGINEER FOR REVIEW PRIOR TO COMMENCING WORK. ALL CALCULATIONS AND SHOP DRAWINGS SHALL BE STAMPED AND SIGNED BY A PROFESSIONAL ENGINEER LICENCED IN ONTARIO.

1. SUGGESTED CONSTRUCTION PROCEDURE

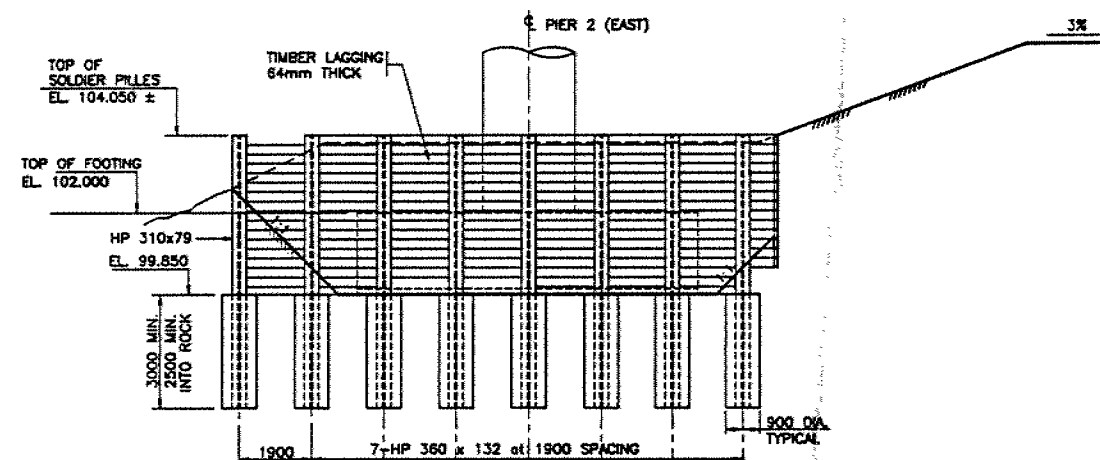
- 1.1 LOCATE THE POSITION OF THE SOLDIER PILE WALL AND WHERE NECESSARY EXCAVATE THE SOIL BETWEEN THE WALL AND THE EDGE OF THE ROADWAY TO 1:1 SLOPE DOWN TO EL. 103.85±.
- 1.2 PREDRILL HOLES FOR CONCRETE ENCASING TO DIAMETERS AND LIMITS AS SHOWN ON THE DRAWINGS.
- 1.3 SET PILES HP 360 x 132 AND HP 310 x 79 PLUMB AND TRUE TO REQUIRED ALIGNMENT IN THE PREDRILLED HOLES AND THEN ENCASE WITH 20 MPa CONCRETE ALL TO LIMITS SHOWN ON THE DRAWINGS.
- 1.4 AFTER CONCRETE HAS REACHED A STRENGTH OF 15 MPa, EXCAVATE WITHIN CONFINES OF INTENDED SHORING AND INSTALL TIMBER LAGGING SEQUENTIALLY AS EXCAVATION PROCEEDS. IN NO CASE SHALL THE EXCAVATION BE PERMITTED TO EXCEED MORE THAN 450mm AHEAD OF INSTALLED LAGGING.
- 1.5 AS EACH TIMBER LAGGING IS INSTALLED IT SHALL BE WEDGED TIGHT IN PLACE AND ANY VOID BETWEEN INSTALLED TIMBER AND RETAINED EARTH SHALL BE FILLED WITH COMPACTED MATERIAL.
- 1.6 ONCE CONSTRUCTION OF THE FOOTINGS AND PIERS ARE COMPLETED THE CONTRACTOR SHALL REMOVE THE TIMBER LAGGING AND SOLDIER PILES OR SHALL CUT OFF 900mm BELOW FINISHED GRADE AND REMOVE ALL MATERIALS ABOVE THAT ELEVATION.

2. MATERIALS

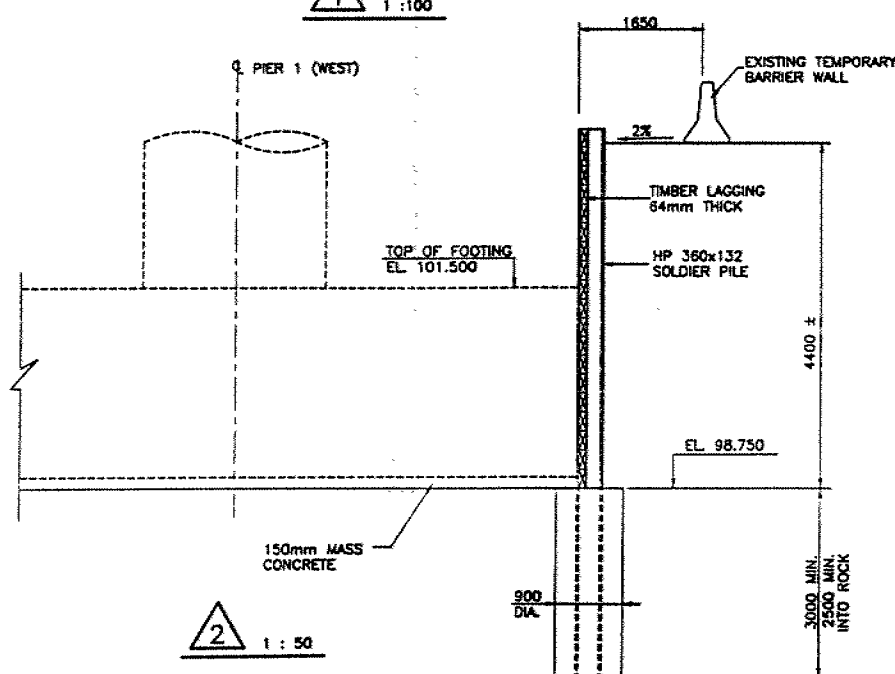
- 2.1 ALL STRUCTURAL STEEL TO BE IN ACCORDANCE WITH CSA STANDARD CSA G40.21-M1987 AND BE GRADE 300W.
- 2.2 BEFORE FABRICATION AND INSTALLATION THE CONTRACTOR SHALL SUBMIT FOR APPROVAL OF THE ENGINEER SHOP AND FIELD ASSEMBLY DRAWINGS.
- 2.3 CONCRETE ENCASING AT TOE OF SOLDIER PILES TO BE OF COMPRESSIVE STRENGTH 20 MPa.
- 2.4 TIMBER LAGGING TO BE GRADE S.P.F. No.2 ACCORDING TO CSA CAN3-086-M84.



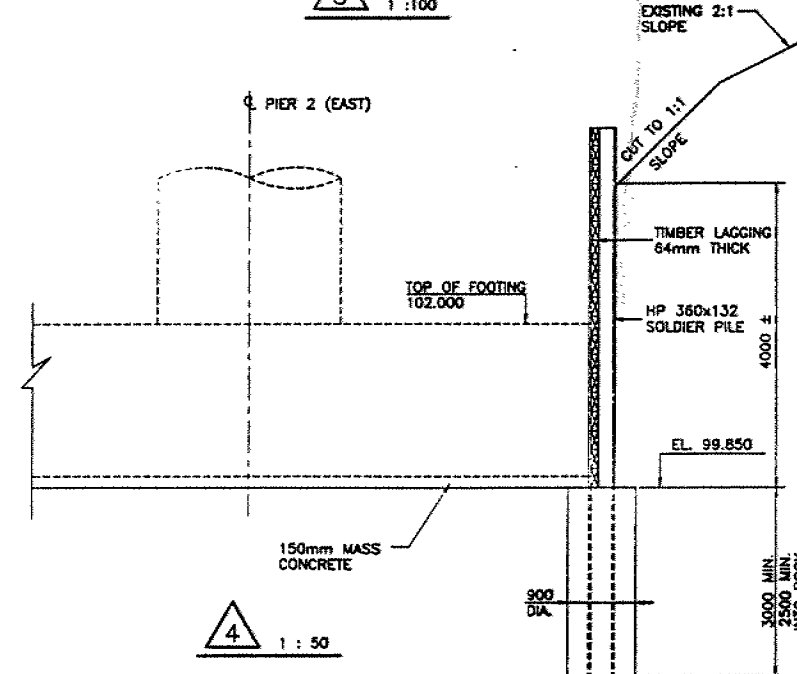
1 1:100



3 1:100




2 1:50



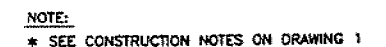
4 1:50

DATE	BY	DESCRIPTION
DESIGN	M.G.S.	CHK J.T.G. CODE - 1983-CHOC LOAD - CLASS A [DATE MAR 1992]
DRAWN	S.G.	CHK M.A.-N SITE 10-333 [SCHEME DWG 3]

CONT No
WP No 199-77-03
RAMP 403/W - QEWS/E (BR.41)
OVER
QEWS AND RAMP QEWS/S-403/E
FOOTINGS AND PIERS

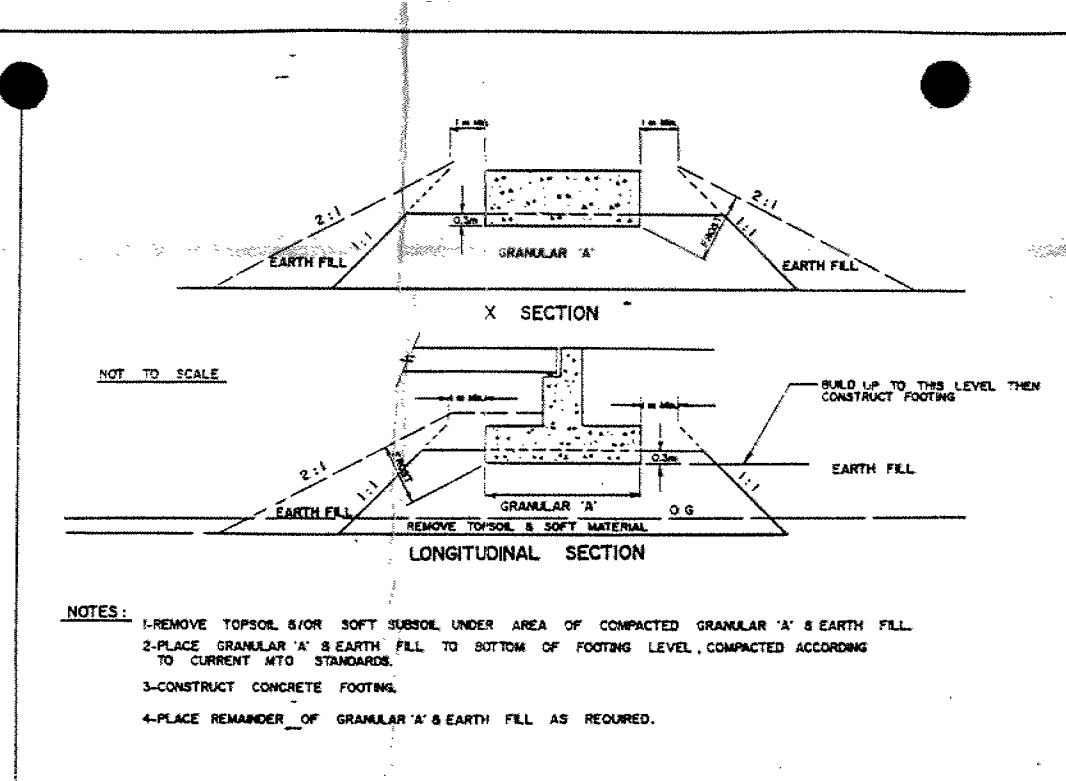
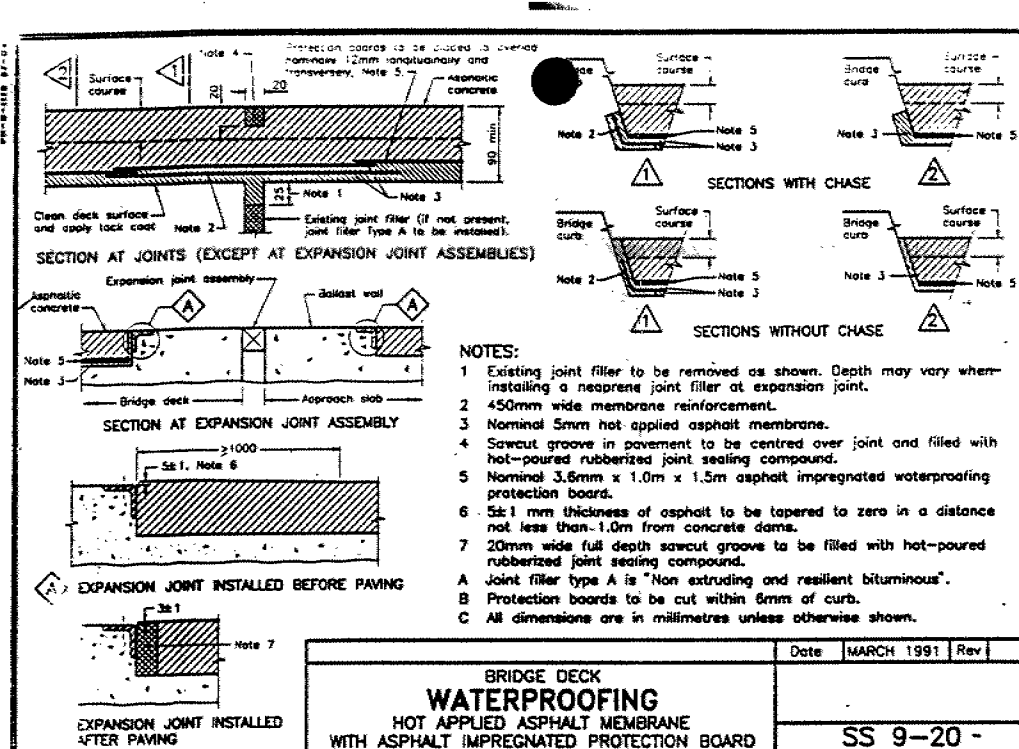


N.T.S



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS									
	DATE	BY	DESCRIPTION						
DESIGN	P.C.	CHK	J.T.G.	CODE - 1983-CHSOC LOAD - CLASS A				DATE	MAR. 1992
DESIGN	S.G.	CHK	M.A.-M	SITE 10-333				SCHEME	DWG 4



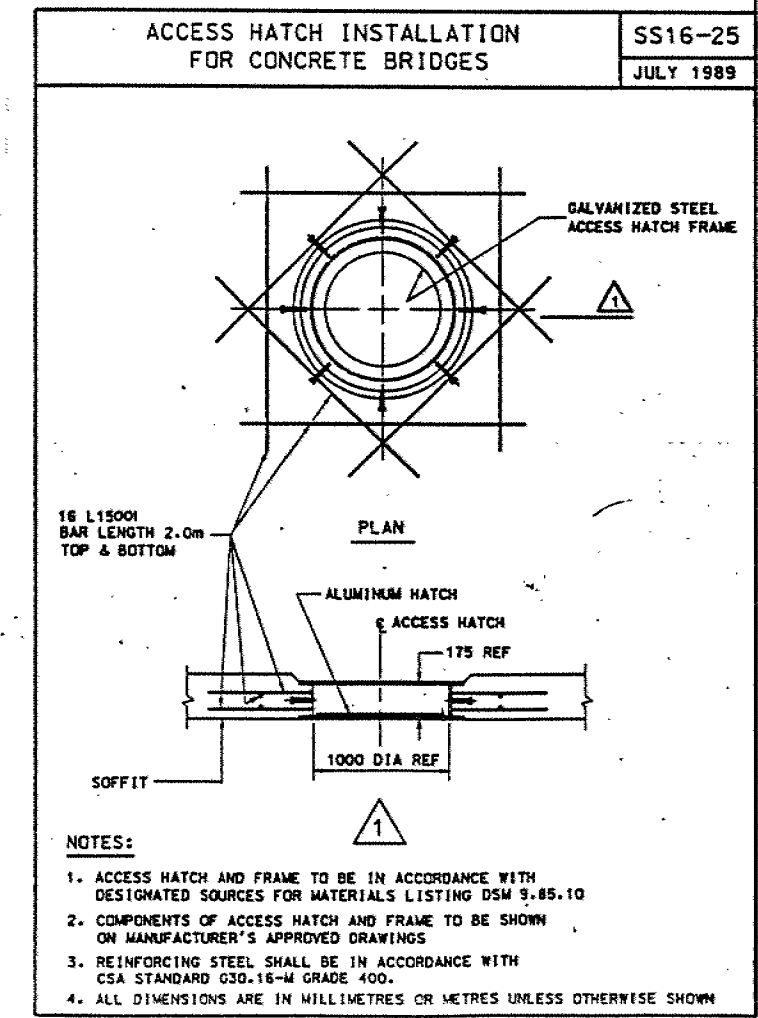
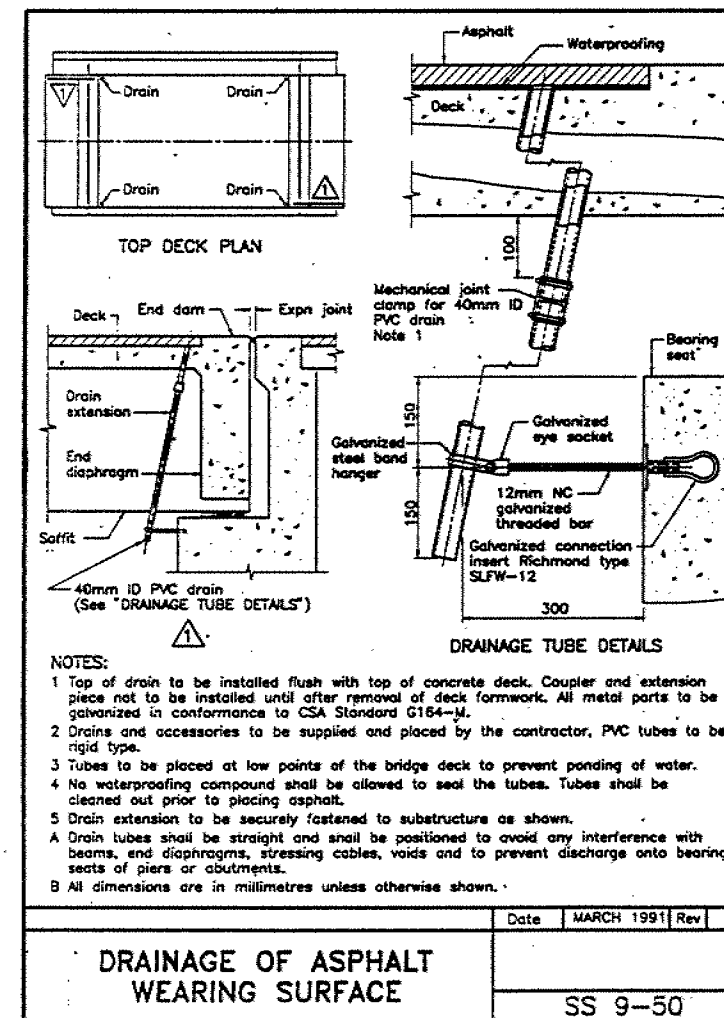
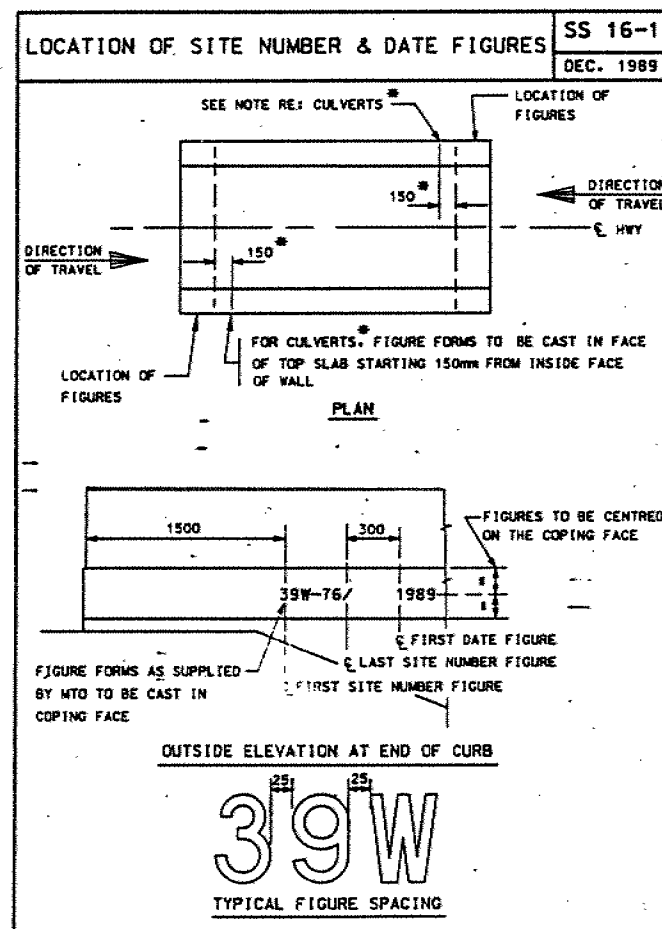
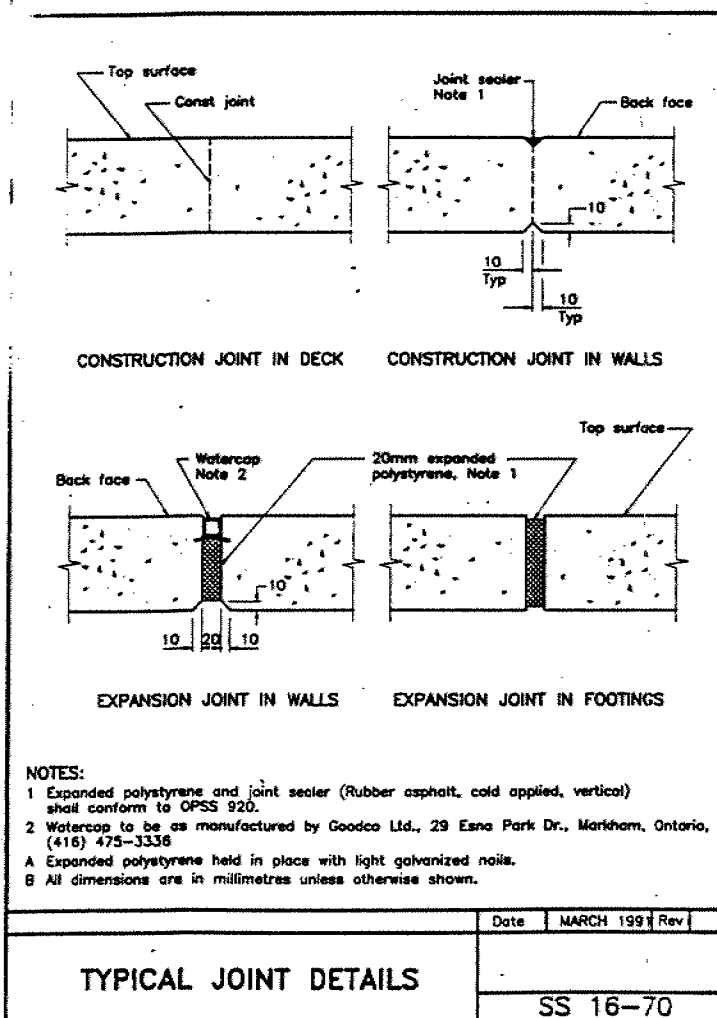
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 199-77-03

RAMP 403/W - QEW/E (BR. #41)
OVER
QEW AND RAMP QEW/S - 403/E
STANDARD DETAILS I

SHEET

Gregg and Edens Limited
consulting engineers & planners



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN STD. CHK	CODE 1983 QHBC LOAD CLASS A	DATE MAR 1992	
DRAWN J.T.L. CHKMA-N	SITE 10-333	STRUCT	SCHEME DWG. 25



Gregg
&
Edens
Limited

C O N S U L T I N G E N G I N E E R S A N D P L A N N E R S

MAR 18 1992

March 16, 1992

McCormick, Rankin & Assoc. Ltd.
Consulting Engineers
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

Attention: Dr. Roy Skelton

Gentlemen:

Re: Highway 403 Structures, Freeman Interchange
Structure No. 41, W.P. 199-77-03
Structure No. 37, W.P. 199-77-02

Further to our recent discussion concerning the design of foundations and road protection for the above two structures we wish to report as follows:

1. Bridge No. 37

1.1 Road Protection

We suggest that the design of the east abutment be retained as is, rather than revise using more expensive caissons for support. We further suggest that the roadway protection be revised, still using a soldier pile/ground anchor arrangement as indicated on the accompanying sketch. The MTO Geotechnical section may wish to review our assumed design criteria for the anchors which is:

Active Earth Pressure Coeff. $K_a = 0.33$ at SLS ✓
 $(K_a)_f = 0.41$ at ULS ✓

Passive Earth Pressure Coeff. $K_p = 3.0$ at SLS ✓
 $(K_b)_f = 2.44$ at ULS ✓

Soil Unit Weight = 20 kN/m^3 ✓

Live Load surcharge = 600 mm of Earth ✓

Anchor Allowable Bond Stress = 100 kPa ✓

Anchor Grout Strength = 35 MPa

The previously 2m wide strip excavation at 1:1 1/2 slope is now eliminated. The soldier piles would be installed in pre-augered holes with the tips encased in 20 MPa concrete. The suggested redesign ensures that all anchors are embedded in original soil or in the well compacted fill of the existing road embankment. We believe the suggested design parameters to be reasonably conservative

1.2 Pier Footings

A soil bearing capacity of 1000 kPa at ULS and 500 kPa at SLS will reduce the footing sizes for the two piers. The west pier footing would reduce from 9 x 5 x 1.5 m to 7 x 4.5 x 1.5 m, saving 25.5 m³ of concrete. In all, approximately 45.75 m³ of concrete would be saved. Assuming a unit price of \$200/per m³, the total saving in concrete alone is estimated at \$9,150.00, plus excavation savings.

We recommend that these footings be revised using the new design bearing capacities for the soil.

2. BRIDGE NO. 41

2.1 West Abutment

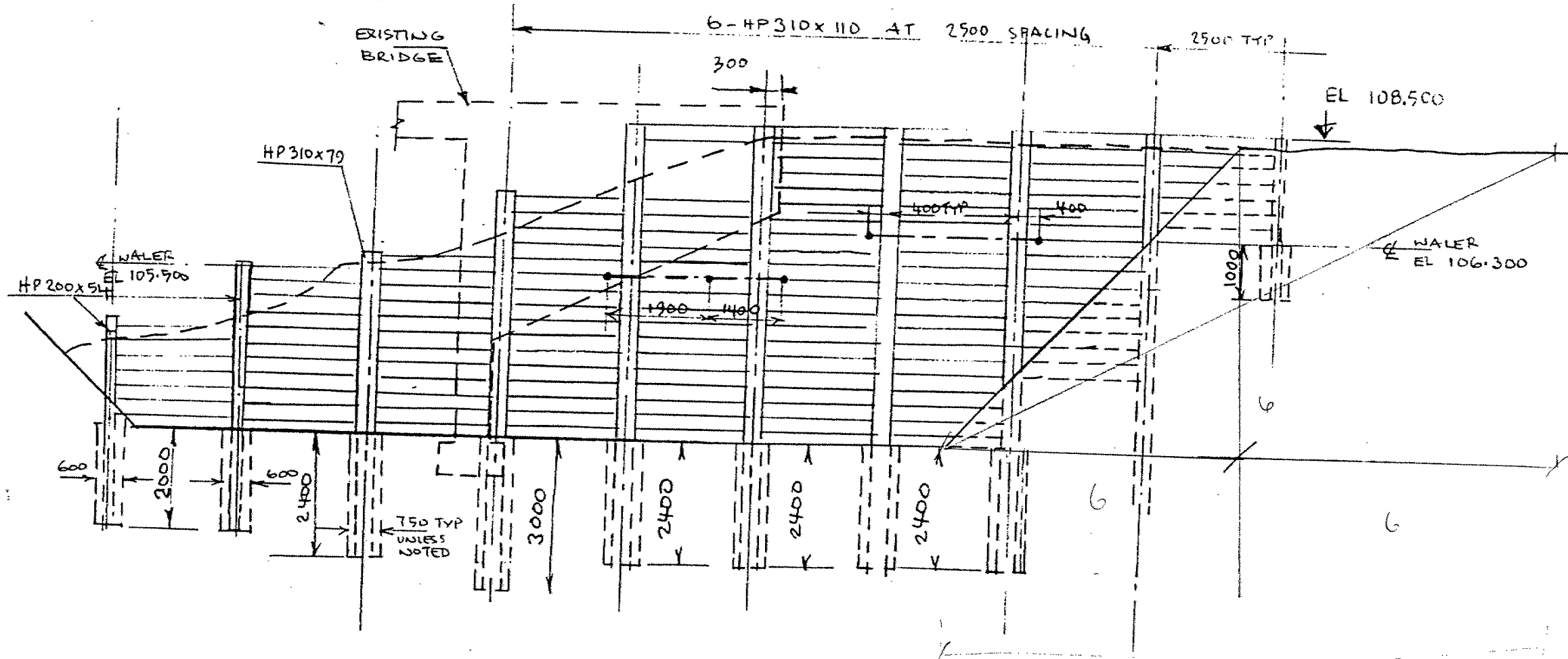
The elevation of sub-excavation for Granular 'A' at the west abutment has been raised to 102.000. This in effect reduces the extent of Granular 'A' material and eliminates the need for roadway protection at this location.

2.2 Pier Footings

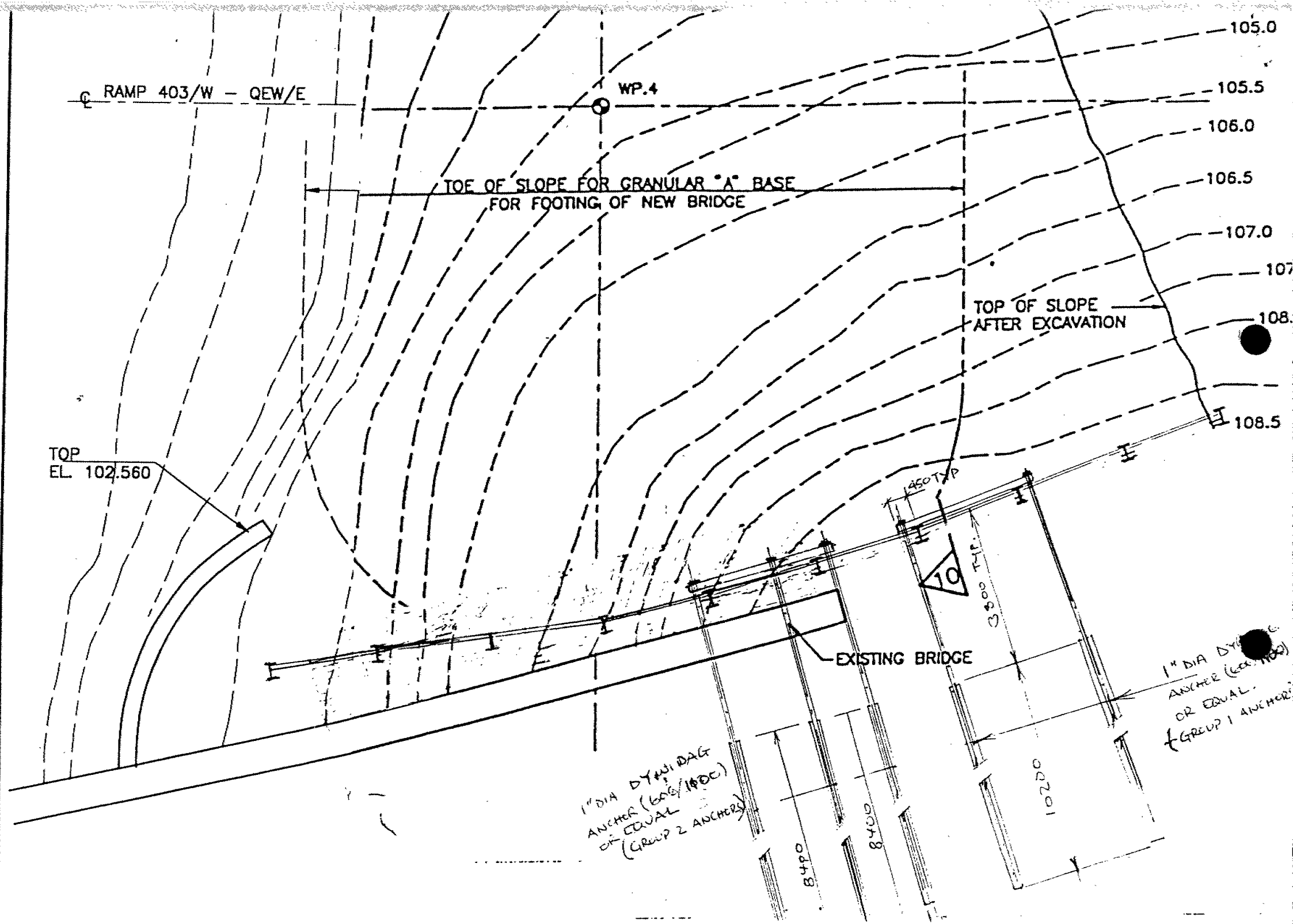
The pier footing sizes must remain as they are despite the higher calculated soil capacity of ULS. The east and west footings are the same size because different limit state conditions govern. The west pier footing size is controlled by ULS, whereas the east pier footing size is controlled by SLS and the effect of horizontal forces on the pier. No change in footing design is recommended.

2.3 Roadway Protection

A revised roadway protection scheme at the excavation for the pier footings is suggested using soldier piles and lagging. The design would be based on parameters similar to those for Bridge #37 described above.



12



RAMP 403/W - QEW/E

WP.4

TOE OF SLOPE FOR GRANULAR "A" BASE
FOR FOOTING OF NEW BRIDGE

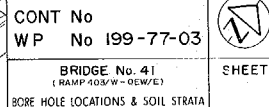
TOP OF SLOPE
AFTER EXCAVATION






TOP
EL. 102.560

EXISTING BRIDGE

1" DIA DYWIDAG
ANCHER (606/1000)
OR EQUAL
(GROUP 2 ANCHERS)

1" DIA DYWIDAG
ANCHER (606/1000)
OR EQUAL
(GROUP 1 ANCHERS)



- | | |
|---|--|
|  | Bore Hole |
|  | Dynamic Cone Penetration Test (Cone) |
|  | Bore Hole & Cone |
| N | Blows/0.3m [Std Pen Test, 475 J/blow] |
| CONE | Blows/0.3m [50° Cone, 475 J/blow] |
|  | WT at time of investigation
(August 1990) |
|  | Test Pit |
| | Piezometer |

No	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
BN123	102.3	4,799,406.8	278,067.0
BN24	102.0	4,799,420.6	278,095.4
BH25	101.9	4,799,536.9	278,120.8
BH26	102.0	4,799,455.4	278,154.3
BN27	103.7	4,799,469.1	276,169.8
TP8	103.8	4,799,458	278,176

NOTE

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 107-2 of Form 100.

REV.	DATE	BY	DESCRIPTION

Geocres No 30M5-175

HWY No 403, QEW, BRIDGE No. 41			DIST	4
SUBMITTED V.H	CHECKED G.M	DATE OCT 4/90	SITE	10-333
DESIGNED M.M	CHECKED G.M	DATE	DWG	1997703-A

REF. No. E-83-403-B, 90 05