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TROW

foundation investigation and design report

Trow Ltd

Geocres No 30M12-198

GEOTECHNICAL INVESTIGATION
PROPOSED GRADE SEPARATION
CENTRE STREET AT CN RAILWAY
BRAMPTON, ONTARIO

8366

WO 86-11009

Prepared for:

CITY OF BRAMPTON
c/o PLANMAC CONSULTANTS LIMITED

M. T. C.
STRUCTURAL OFFICE
SEP 03 1986
RECEIVED
MTC DOWNSVIEW

TROW LTD

793-7800

43 Baywood Road
Rexdale, Ontario
M9V 3Y8
(416) ~~749-1290~~

Project: T 5955-G
February 6, 1985

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STRUCTURE SITE No. 24-437

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Trow Ltd

GEOTECHNICAL INVESTIGATION PROPOSED GRADE SEPARATION CENTRE STREET AT CN RAILWAY BRAMPTON, ONTARIO

SUMMARY

Trow Ltd has carried out a geotechnical investigation for an underpass which is to be constructed on Centre Street at the CN Railway in Brampton, Ontario.

The underpass will dip down to a minimum elevation of 211.7 m under the tracks and the sides will be a combination of retaining walls and cut slopes. A storm sewer will be installed under the underpass and drain southwest through the existing park.

Ten sampled boreholes ranging in depth from 3.5 to 10.7 m were undertaken for the investigation. Fill comprises the upper 0.8 to 2.1 m of subsoil over the site of the underpass except adjacent to the park which was an old garbage dump. Along and in the park the fill ranges in thickness from 4.0 to 7.0 m. Underlying the fill is clayey silt till of the Halton Formation. This till is hard in the upper levels and generally becomes firm to very stiff at 4.6 m and lower. The clayey silt till overlies dense to very dense silt till and shale till. Wet sand seams were noted in the silt till. Red shale bedrock of the Queenston Formation was interpreted at Elevation 209.3 to 206.5 m. Short-term water levels ranged from Elevation 211.1 to 214.1 m in this investigation and in our previous investigations the water levels ranged from Elevation 213.2 to 213.7 m.

Equations are given to estimate the factored bearing capacity at ultimate limit states. The serviceability limit states bearing value is 500 kPa on the hard clayey silt till and dense to very dense silt till and shale till. This value is decreased to 75 to 400 kPa on the firm to stiff clayey silt till and the bearing value on the shale is 1500 kPa.

Rock anchors may be used to tie down the retaining wall footings and these can probably be designed for a rock to concrete bond value of 0.4 MPa. Comments are made regarding rock anchor design and installation.

Excavations can be carried out by heavy hydraulic backhoes although ripping-teeth may be necessary in the bedrock. It is expected that groundwater seepage can be controlled by pumping from sumps. The excavated natural soil is suitable for use as construction backfill provided the moisture content does not increase. The fill is not recommended for use as backfill. A shoring appendix is provided. It is suggested that permanent slopes be cut at 2½ horizontal to 1 vertical. Earth pressure parameters based on the Bridge Code and a suggested pavement structure based on MTC guidelines are given. The storm sewer through the landfill should be supported by piers or the fill should be removed and replaced with well-compacted granular fill or lean concrete. Type 20, 40 or 50 cement should be used in the subsurface concrete.

INTRODUCTION

Trow Ltd was retained by the City of Brampton to undertake a geotechnical investigation for a proposed grade separation to be constructed on Centre Street at the CN Railway in Brampton, Ontario. The investigation was authorized by Mr. D.C. Batchelor, P.Eng., of Planmac Consultants Limited on behalf of the City of Brampton.

It is understood that the grade separation will consist of an underpass which will dip down to a minimum elevation of about 211.7 m under the tracks. The sides of the underpass will be a combination of retaining walls and cut slopes. A storm sewer will be installed under the underpass and drain south-west through the existing park.

A preliminary investigation was undertaken at this site in June 1983 (Project T 4460-G). At that time it was not determined whether an overpass or underpass would be constructed. This investigation provides a more extensive evaluation of the subsurface conditions for the final design of the underpass.

The purpose of this investigation was to:

(a) determine the subsoil conditions in ten additional boreholes;

(b) provide comments pertaining to the design of foundations, excavation and shoring requirements, dewatering, pavement structures, etc., and

(c) comment, from a geotechnical point-of-view, on general problems which may be anticipated during the construction phase.

This report is provided on the basis of the terms of reference presented above and on the assumption that the design will be in accordance with applicable codes and standards. If there are any changes in the design feature relevant to the geotechnical analyses, or if any questions arise concerning geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations of this office may be relied upon.

PROCEDURE

Ten boreholes ranging in depths from 3.5 to 10.7 m were drilled with continuous-flight solid stem auger equipment by a drilling subcontractor under the direction and supervision of Trow Ltd personnel. The boreholes were sampled at regular intervals with a split-spoon sampler following the procedures of the standard penetration test (A.S.T.M. D1586). The samples were logged in the field and returned to the Trow Ltd laboratory for testing and examination.

The surveying of the borehole locations and ground surface elevations were undertaken by Trow Ltd personnel. The elevations were referenced to City of Brampton Benchmark No. 28.

Water level observations were made in the open boreholes at the completion of the drilling operations and at subsequent intervals.

As well as visual examination, the laboratory testing included moisture content and natural unit weight determinations on the soil samples. One sample of groundwater from Borehole 11 was tested for pH and water soluble sulphate (SO_4).

SUBSOIL AND GROUNDWATER CONDITIONS

Detailed subsoil conditions are presented on the borehole logs, Drawings 2 to 13. These are summarized as follows.

Fill was encountered at all borehole locations with the exception of Borehole 5 (although very thin fill may also be present in the upper 0.3 m of this area). The fill thickness varies from about 0.8 to 2.1 m in Boreholes 2 to 9 and generally consists of loose to compact clayey silt which contains occasional brick fragments and pieces of wood and is partly organic-stained. In Boreholes 1, 10, 11 and 12 which are at the site of an old garbage dump, the fill thickness ranges from about 4.0 to 7.0 m and generally consists of very loose to compact clayey silt with varying amounts of wood, cinders, paper, glass and other debris.

Explosive gas measurements in the fill ranged from 0 percent to a maximum of 4 percent (on the 100 percent scale).

Underlying the fill is a clayey silt till stratum believed to be the Halton Formation. This till is generally hard except in the lower portions of Boreholes 4, 6, 7 and 8 where the till was noted to be firm to very stiff at a depth of 4.6 m and lower. Underlying the clayey silt till is a dense to very dense silt till deposit and then a shale till which consists mainly of red-brown shale fragments. Wet sand seams were noted in the silt till in Boreholes 6, 7, 8 and 9 at about Elevation 211.9 to 212.5 m.

Slightly weathered to sound shale was interpreted at Elevation 209.3 to 206.5 m. This is based on sampling refusal and on the recovery of red shale fragments in the samples. The bedrock surface appears to slope down towards the south and west.

In our previous investigation the water levels in the boreholes after nine days ranged from Elevation 213.2 to 213.7 m. In this investigation the water levels ranged from Elevation 212.6 to 214.1 m after one to two days in Boreholes 6, 7 and 10. This water is believed to be from the sand seams encountered in the silt till. Borehole 9 had a slightly higher water level at Elevation 215.1 m, however some of this may be due to water introduced during drilling. In the garbage dump area the water levels were at Elevations 212.7 to 211.1 m which is 0.8 to 0.9 m below the ground surface.

FOUNDATIONS

The proposed retaining walls for the underpass structure may be supported by normal spread footings founded on the natural clayey silt till and silt till encountered below the fill on this site.

1. Ultimate Limit States

The following formulae may be used to estimate the factored bearing capacity at ultimate limit states for footings founded on the clayey silt till and silt till.

(a) For rectangular footings with $D/B > 2.5$ placed on cohesive soils, and for all rectangular units placed on granular soils;

$$q_f = C_f N_C + 0.5 \gamma B N_\gamma$$

(b) For rectangular footings with $D/B \leq 2.5$ placed on cohesive soils;

$$q_f = \gamma D + 5C_f (1 + 0.2 (D/B)(1 + 0.2(B/L))$$

(c) For square units placed on cohesive or granular soils;

$$q_f = 1.2\ C_f N_C + \gamma\ D N_q + 0.4 \gamma\ B N_\gamma$$

where: γ = Unit weight of soil, assume a value of 22.3 kN/m^3. Below the water table the submerged unit weight should be used, assume a value of 12.5 kN/m^3

C_f = Factored shear strength of soil

= F_C x C, a factor $F_C = 0.5$ should be used.

In most cases C ≃ 250 kPa except for the firm to very stiff clayey silt till in Boreholes 4, 6, 7 and 8 where C varies from about 35 to 125 kPa. The actual value will have to be interpreted from the borehole log for each location.

A Value of C = 0 in the silt till should be assumed.

B = Width of footing

L = Length of footing

D = Depth of footing below finished grade

N_q, N_C, N_γ = bearing capacity coefficients, which depend on the effective factored angle of internal friction, ϕ_f

$$N_q = (e^{\pi \tan \phi_f}) \frac{1 + \sin\phi_f}{1 - \sin\phi_f}$$

$N_c = (N_q - 1) \cot \phi_f$ ($\phi_f \rightarrow 0$, $N_c \rightarrow 5.14$)

$N = 1.5 (N_q - 1) \tan \phi_f$

ϕ_f = factored angle of internal friction
= $\tan^{-1}(F_\phi \tan\phi)$

In the very stiff to hard clayey silt till, a value of $\phi = 0^o$ may be assumed

In the dense silt till, a value of $\phi' = 40^o$ may be assumed
? $= 35^o$ unfactored

NOTE: For more accurate values of C and ϕ additional laboratory testing must be carried out.

2. Serviceability Limit States

The spread and strip footings founded on the natural firm to hard clayey silt till and dense to very dense silt till may be designed for the bearing values given on Table 1 on the following page.

Table 1

Bearing Values

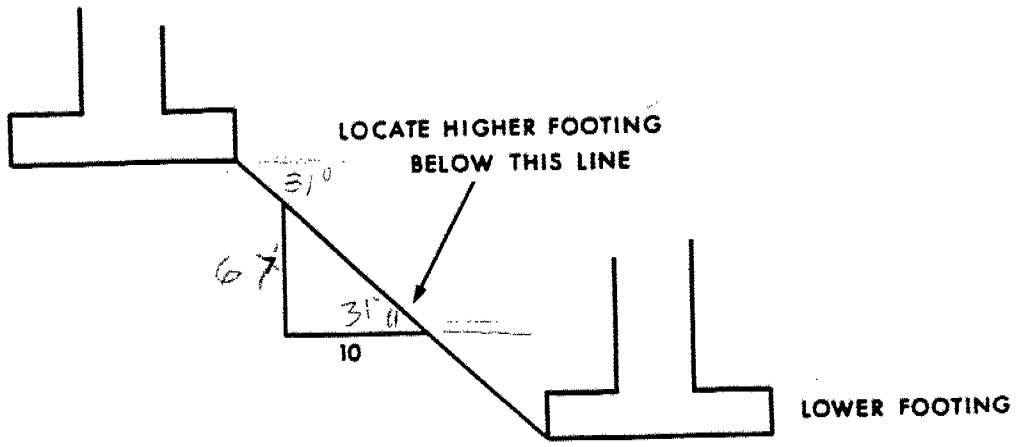
Borehole	Bearing Value (kPa)	Approximate Elevations (m)	Material
1	500	Below 214.4	Clayey Silt Till
	1500	below 207.7	Shale
2	500	Below 217.8	Clayey Silt Till
	1500	Below 208.6	
3	500	Below 217.4	Clayey Silt Till
4	500	Below 217.6	Clayey Silt Till
	400	Below 215.6	Clayey Silt Till
	200	Below 214.1	Clayey Silt Till
5	500	Below 217.2	Clayey Silt Till
6	500	Below 218.0	Clayey Silt Till
	200	Below 215.5	Clayey Silt Till
	75	Below 214.0	Clayey Silt Till
	500	Below 211.9	Silt Till
	1500	Below 208.8	Shale
7	500	Below 217.5	Clayey Silt Till
	75	Below 214.5	Clayey Silt Till
	500	Below 212.4	Silt Till
8	500	Below 216.4	Clayey Silt Till
	150	Below 213.8	Clayey Silt Till
	500	Below 211.7	Silt Till
9	500	Below 217.2	Clayey Silt Till and Silt Till
10	500	Below 211.5	Clayey Silt Till and Silt Till
	1500	Below 207.0	Shale
11	1500	Below 205.9	Shale
12	500	Below 206.0	Clayey Silt Till and Silt Till

Footings designed to the bearing values given on Table 1 are expected to settle less than 25 mm total and 19 mm differential.

All footings exposed to seasonal freezing conditions must have at least 1.2 m of soil cover for frost protection.

The recommended bearing capacities have been calculated by Trow Ltd from borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes, when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections provided by Trow Ltd to validate the information for use during the construction stage.

Where footings are stepped down, the upper footing must fall below an imaginary 7 vertical to 10 horizontal line drawn up from the lower footing as shown on the sketch below.



FOOTINGS AT DIFFERENT ELEVATIONS

The lower footing must be installed before the upper one.

ROCK ANCHORS

Rock anchors may be used to permanently tie down the footings of the retaining walls and therefore must be made completely corrosion resistant.

The rock anchors must penetrate far enough into sound bedrock so that a factor of safety of 2 against pull-out is obtained. In the Queenston Shale the use of 0.4 MPa as a design value for bond between the concrete and the shale should provide a factor of safety well in excess of 2, if the anchors are properly installed.

All holes must be cased through water-bearing deposits, such as the wet seams encountered between Elevation 211.9 to 212.5 m, so that no loss of ground and movement into the rock socket can occur.

All holes must be backfilled with a bentonite-cement mix to the bottom of the footing immediately after installation. This mix must be confirmed to be stable against shrinkage or settlement i.e., it must permanently fill the hole.

Rock anchors should be designed so that the full load is transmitted from the wires or cables to the concrete grout at the lower end of the anchor. The design bond stress between the steel wires and the concrete should incorporate a factor of safety of 2. The wires in the remaining parts above the anchor zone should be greased or sheathed to preclude bonding with the concrete. The steel to concrete bond should be in the lowest level of the anchor, such that the upper end of the concrete within the anchor zone is put into compression. If the concrete to steel bond extends to the upper end, the high concentration of tensile stresses may cause localized failures and possible progressive movement.

Each anchor must be checked to ensure that it is installed properly, i.e., that a sufficient bond has been developed. It is customary, in this phase of the testing, to apply a test load equal to 133 percent of the design load. This load must be maintained without creep for at least 30 minutes and at least 3 anchors should be tested under full load for a period of 24 hours to establish the maximum safe creep free load.

EXCAVATIONS AND BACKFILL

It is expected that excavations in the clayey silt till, silt till, shale till and fill can be carried out with a heavy hydraulic backhoe. Minor groundwater seepage can be anticipated due to the wet seams encountered in the till at approximate Elevation 211 to 213 m in Boreholes 2, 6, 7, 8 and 9. It is expected that this seepage can be collected in sumps and be removed by normal construction pumping methods. Minor slumping of the wet seams may occur and the contractor should be prepared to make slightly oversize excavations in this case.

Water seepage is also expected from the fill in the park during sewer excavation. It is expected that this water seepage can also be controlled by pumping from sumps, however slumping of the fill should be expected and oversized excavations will probably be required.

Excavation of the shale can usually be carried out with heavy hydraulic backhoes, although a ripping-tooth may be necessary. If limestone layers are encountered, it may be necessary to use jackhammers to break through the layers and then lift the slab with the backhoe.

All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act, 1978. In the hard clayey silt till the temporary excavation sides may be sloped at 60 degrees to the horizontal provided they are inspected and confirmed to be safe by a geotechnical engineer. The excavation adjacent to the railway track should not be closer than a 1 horizontal to 1 vertical cut starting approximately 1 m out from the edge of the railway ties. A shoring appendix has been included where excavations in the clayey silt till need temporary support.

The excavated clayey silt till and silt till and shale till are suitable for use as construction backfill provided the moisture content remains at its present value. In this light, the work should be undertaken during dry weather and stockpiles should be protected from moisture change by tarps or compacting the surface of the stockpile in order to minimize water infiltration. Compaction can best be achieved by sheepsfoot compactors in maximum loose lifts of

200 mm. In confined areas, imported granular fill which can readily be compacted by hand tampers is recommended.

The fill contains rubble and should not be used for backfill except possibly in the park, if long-term settlements can be tolerated.

Free-draining granular backfill must be placed behind all retaining structures as shown on Sketch 1, (Page 14).

SLOPES

Standard practice for permanent excavations is to slope the excavation face at 2 horizontal to 1 vertical. However this usually results in minor localized slumping and periodic maintenance may be required. The long-term stable slope for the clayey silt till is expected to be about 2½ horizontal to 1 vertical. At this lower angle, slumping is expected to be negligible and there is better access to the slope for lawn mowing.

EARTH PRESSURE PARAMETERS

The coefficients of the active and passive pressures on the retaining wall are dependent upon the slope and compaction of the backfill and characteristics of the wall. The coefficients of active and passive earth pressures may be calculated by utilizing the two charts (Table 2 - Pages 15 and 16).

For well-drained granular backfill the unit weight may be assumed to be = 20.5 kN/m^3 and the internal angle of friction '$\emptyset$' to be = 30 degrees.

For the case of free-draining backfill that is approximately level behind a retaining wall less than 10 m high, the following coefficients may be used.

	Ultimate Limit State	Serviceability Limit State
Active State	$(K_a)_f = 0.41$	$K_a = 0.33$
At Rest State	$(K_o)_f = 0.58$	$K_o = 0.50$
Passive State	$(K_p)_f = 2.44$	$K_p = 3.00$

If the above conditions apply, the following equivalent fluid pressures may be assumed.

(a) At Ultimate Limit States

i) Active State: 8.0 kPa/m

ii) At Rest Condition: 10.0 kPa/m

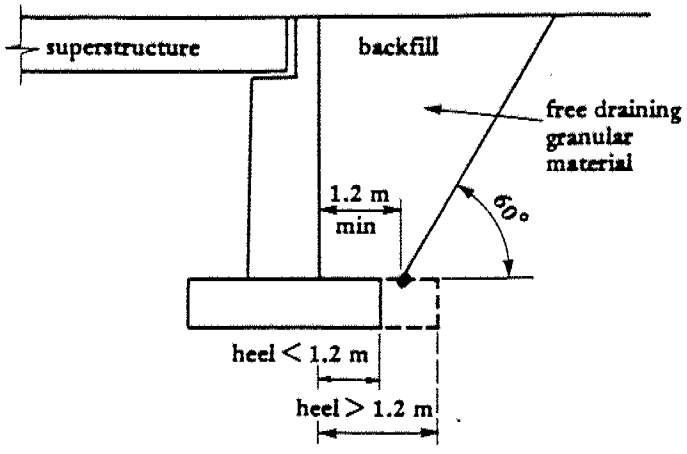
iii) Passive State: 46.0 kPa/m

(b) At Serviceability Limit States

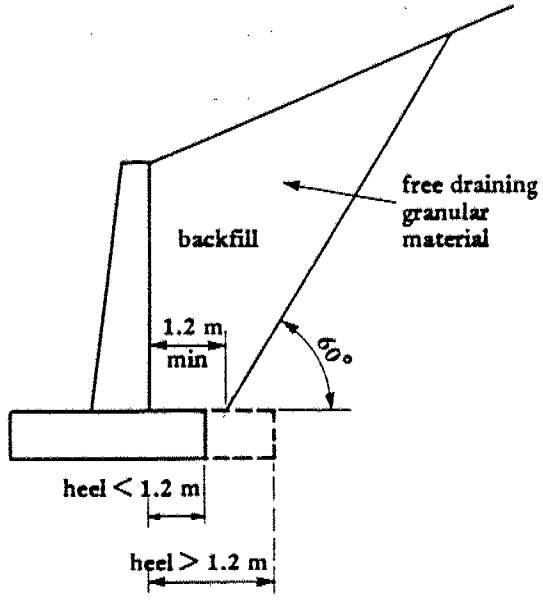
i) Active State: 6.5 kPa/m^3

ii) At Rest Condition: 8.5 kPa/m

iii) Passive State: 56.0 kPa/m



Abutments



Retaining walls

Sketch 1 – Minimum Backfill Material Requirements for Abutments and Retaining Walls

Table 2

Earth Pressure Coefficients at Ultimate Limit State
(After A.R. Jumikis, *Soil Mechanics*, Van Nostrand Company Inc.)

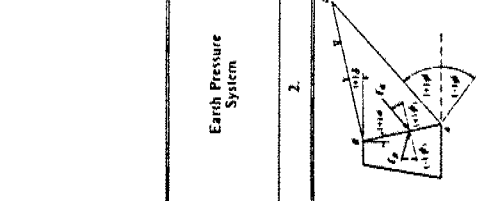
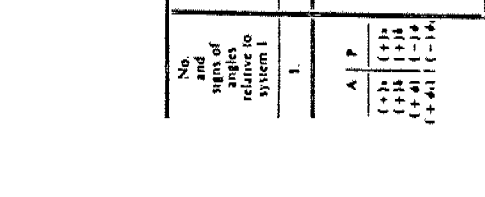
No. and signs of angles relative to system 1	Earth Pressure System	Coefficients of: Active Earth Pressure $(K_a)_f$	Coefficients of: Passive Earth Pressure (resistance) $(K_p)_f$
1.	2.	3.	4.
A | P (+)α | (+)α (+)α | (−)α (+)φ | (−)α (+)φ | (−)α		$(K_a)_f = \dfrac{\cos^2(\phi_f - \alpha)}{\cos^2\alpha\cos(\alpha+\phi_1)\left[1+\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin(\phi_f-\delta)}{\cos(\alpha+\phi_1)\cos(\alpha-\delta)}}\right]^2}$	$(K_p)_f = \dfrac{\cos^2(\phi_f + \alpha)}{\cos^2\alpha\cos(\alpha-\phi_1)\left[1-\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin(\phi_f+\delta)}{\cos(\alpha-\phi_1)\cos(\alpha-\delta)}}\right]^2}$
A | P (+)α | (+)α α = 0 | α = 0 (+)φ | (−)α (+)φ | (−)α		$(K_a)_f = \dfrac{\cos^2(\phi_f - \alpha)}{\cos^2\alpha\cos(\alpha+\phi_1)\left[1+\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin\phi_f}{\cos(\alpha+\phi_1)\cos\alpha}}\right]^2}$	$(K_p)_f = \dfrac{\cos^2(\phi_f + \alpha)}{\cos^2\alpha\cos(\alpha-\phi_1)\left[1-\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin\phi_f}{\cos(\alpha-\phi_1)\cos\alpha}}\right]^2}$
A | P (+)α | (+)α (−)α | (−)α (+)φ | (−)α (+)φ | (−)α		$(K_a)_f = \dfrac{\cos^2(\phi_f - \alpha)}{\cos^2\alpha\cos(\alpha+\phi_1)\left[1+\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin(\phi_f+\delta)}{\cos(\alpha+\phi_1)\cos(\alpha+\delta)}}\right]^2}$	$(K_p)_f = \dfrac{\cos^2(\phi_f + \alpha)}{\cos^2\alpha\cos(\alpha-\phi_1)\left[1-\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin(\phi_f-\delta)}{\cos(\alpha-\phi_1)\cos(\alpha+\delta)}}\right]^2}$
A | P α = 0 | α = 0 (+)φ | (−)α (+)φ | (−)α		$(K_a)_f = \dfrac{\cos^2\phi_f}{\cos\phi_1\left[1+\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin(\phi_f-\delta)}{\cos\phi_1\cos\delta}}\right]^2}$	$(K_p)_f = \dfrac{\cos^2\phi_f}{\cos\phi_1\left[1-\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin(\phi_f+\delta)}{\cos\phi_1\cos\delta}}\right]^2}$

Table 2 (continued)

– Earth Pressure Coefficients at Ultimate Limit State (After A.R. Jumikis, *Soil Mechanics*, Van Nostrand Company Inc.) (cont'd)

A	P		$(K_a)_f$	$(K_p)_f$
$\alpha = 0$	$\phi = 0$	$(-)\alpha$	$\dfrac{\cos^2\phi_f}{\cos\phi_1\left[1+\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin\phi_f}{\cos\phi_1}}\right]^2}$	$\dfrac{\cos^2\phi_f}{\cos\phi_1\left[1-\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin\phi_f}{\cos\phi_1}}\right]^2}$
$\alpha = 0$	$\phi = 0$	$(+)\alpha$	$\dfrac{\cos^2\phi_f}{\cos\phi_1\left[1+\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin(\phi_f+\delta)}{\cos\phi_1\cos\delta}}\right]^2}$	$\dfrac{\cos^2\phi_f}{\cos\phi_1\left[1-\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin(\phi_f-\delta)}{\cos\phi_1\cos\delta}}\right]^2}$
$(-)\alpha$	$\phi = 0$	$(+)\alpha$	$\dfrac{\cos^2(\phi_f+\alpha)}{\cos^2\alpha\cos(\alpha-\phi_1)\left[1+\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin(\phi_f-\delta)}{\cos(\alpha-\phi_1)\cos(\alpha+\delta)}}\right]^2}$	$\dfrac{\cos^2(\phi_f-\alpha)}{\cos^2\alpha\cos(\alpha+\phi_1)\left[1-\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin(\phi_f+\delta)}{\cos(\alpha+\phi_1)\cos(\alpha+\delta)}}\right]^2}$
$(-)\alpha$	$\phi = 0$	$(-)\alpha$	$\dfrac{\cos^2(\phi_f+\alpha)}{\cos^2\alpha\cos(\alpha-\phi_1)\left[1+\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin\phi_f}{\cos(\alpha-\phi_1)\cos\alpha}}\right]^2}$	$\dfrac{\cos^2(\phi_f-\alpha)}{\cos^2\alpha\cos(\alpha+\phi_1)\left[1-\sqrt{\dfrac{\sin(\phi_f+\phi_1)\sin\phi_f}{\cos(\alpha+\phi_1)\cos\alpha}}\right]^2}$
$(-)\alpha$	$\phi = 0$	$(+)\phi_1$	$\dfrac{\cos^2(\phi_f+\alpha)}{\cos^2\alpha\cos(\alpha-\phi_1)\left[1+\sqrt{\dfrac{\sin(\phi_f+\phi_2)\sin(\phi_f+\delta)}{\cos(\alpha-\phi_1)\cos(\alpha-\delta)}}\right]^2}$	$\dfrac{\cos^2(\phi_f-\alpha)}{\cos^2\alpha\cos(\alpha+\phi_1)\left[1-\sqrt{\dfrac{\sin(\phi_f+\phi_2)\sin(\phi_f-\delta)}{\cos(\alpha+\phi_1)\cos(\alpha-\delta)}}\right]^2}$

ROADWAYS

The pavement structure below which is based on MTC Structural Design Guidelines for Flexible Pavement is recommended. This is based on a four-lane highway with 10,000 AADT and a clayey silt subgrade.

Table 3

Suggested Pavement Structure

Part of Structure	Thickness	Material	Compaction Requirements
Wearing Surface	40 mm (1.5 inches) 100 mm (4.0 inches)	HL 1 asphaltic concrete HL 8 asphaltic concrete	96 to 98% Marshall
Basecourse	230 mm (9 inches)	MTC Granular 'A' or equivalent	100% SPMDD*
Sub-basecourse	400 mm (16 inches)	MTC Granular 'B'+ or equivalent	100% SPMDD*

The above pavement structure assumes that all embankment fill and trenches are adequately compacted and that the subgrade is adequately sloped to keep it in a well drained state.

STORM SEWERS

In the area of the underpass it is expected that the storm sewers will be on hard clayey silt till, silt till, shale till and shale. Excavations should be straightforward with the exception of minor water seepage and possible slumping of the wet seams in the area of Boreholes 2, 6, 7, 8 and 9 as previously stated. Class 'B' bedding should suffice to support the pipe on these materials.

* Standard Proctor Maximum Dry Density.
\+ Should have less than 5% passing No. 200 sieve.

In the area of the park the base of the trenches are expected to be on very loose to compact fill which is not considered suitable to support the sewer pipe. In this area, it is suggested that either all of the fill be removed and be replaced by well-compacted granular fill or lean concrete such as K-crete or alternatively the pipe can be placed on a grade beam supported by piers.

If it is proposed to remove the existing fill and replace it with well-compacted imported fill then the new fill should extend outward at a 45 degree slope from the base of the pipe. This will not be required if lean concrete is used i.e., only the fill directly below the pipe need be removed. As previously stated water seepage and slumping of the trench sides should be expected in the park area.

If a grade beam and piers are being considered then a bearing value of 800 kPa could be used on the hard clayey silt till and dense to very dense silt till and 2,500 kPa on the sound shale. It is suggested that the piers be installed using a caisson rig prior to excavating the trench. It may be necessary to seal a temporary liner into the till or bedrock and auger inside the liner if excessive caving and water seepage occurs during augering. A temporary liner will be required in any case while workmen are cleaning the bases of the piers and these bases are being inspected. The minimum diameter necessary to allow access by workmen is 760 mm.

Concrete having a slump of 125 to 150 mm should be poured through a short centering chute into the lined caisson. The liner must not be removed until the concrete is about 0.5 m above cut-off level. The concrete mix design must be reviewed by this office to ensure that it is suitable for the slump and dropping conditions.

All caisson bases must be inspected by this office to confirm the above bearing values.

SUBSURFACE CONCRETE REQUIREMENTS

One sample of groundwater from Borehole 2 was tested during our previous investigation and found to have a pH of 7.2 and a water soluble sulphate (SO_4) content of 910 mg/L. A sample was obtained from Borehole 11 in the landfill site and found to have a pH of 7.4 and a water soluble sulphate content of 98 mg/L.

According to CSA standard CAN 3-A23.1-M77 there is a mild potential for sulphate attack on concrete and therefore Type 20, 40 or 50 cement and a maximum water:cement ratio of 0.50 should be used in the subsurface concrete.

GENERAL COMMENTS

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

TROW LTD

H. Lohse

H. Lohse, B.Sc.

W. Trow

W.A. Trow, P. Eng.

HL/pb/30

Encls.

Dist: City of Brampton (3)
c/o Planmac Consultants Limited
3055 Lakeshore Blvd. West
Toronto, Ontario
M8W 1K8

Attention: Mr. D.C. Batchelor, P.Eng.

APPENDIX

Shoring

APPENDIX

Shoring

If there is insufficient space to permit sloping of the sides of the excavation, it is recommended that the shoring take the form of a soldier pile and lagging system supported by rakers. The following points are intended for guidance in the design of that system.

Earth Pressure

The earth pressure distribution of single support systems considered to be applicable for this site is triangular as shown on Drawing B1. The earth pressure coefficient, 'k', may vary within the following limits:

0.2 where adjacent building footings or settlement-sensitive services lie below a 45 degree line drawn up from the wall at the excavation level.

0.3 where adjacent building footings or settlement-sensitive services lie below a 60 degree line to the horizontal line drawn up from the base of the wall at excavation level.

0.4 where adjacent building footings or settlement-sensitive services lie above a 60 degree line drawn upfrom the base of the wall at excavation level.

The pressure distribution assumes that drainage is permitted between the lagging boards and no build-up of hydrostatic pressure may occur.

In areas where existing footings of adjacent buildings are to be supported by the shoring system, an estimate of the force exerted by the footings may be obtained from the formula for line loads indicated on Drawing B2.

The surcharge load from a row of concrete trucks parked adjacent to the shoring is illustrated in Drawing B3.

Soldier Piles

The soldier piles should be installed in pre-augered holes, with steps taken to exclude water from these holes. Caving of the holes may occur in the fill or in the wet sand seams in the silt till and therefore temporary liners may be necessary.

After the soldier pile is installed, the hole should be filled to excavation level with 20 MPa (3000 psi) concrete, and above that, with 1½ bag mix.

Lagging Boards

The thickness of lagging boards to be used has been determined empirically from a large number of case histories*, and the following are recommended.

Pile Spacing m (feet)	Board Thickness mm (inches)
1.8 (6.0)	50 (2)
2.6 (8.5)**	75 (3)
3.2 (10.5)	100 (4)

As an added precaution, when the excavation is deeper than 7.6 m (25 feet), the board thickness should be increased 25 mm (1 inch) below that depth.

Spaces between boards should be packed with burlap or straw, to prevent loss of ground where water-bearing sand is encountered.

All spaces behind the lagging should be filled with compacted granular fill or a 1½ bag cement grout. The former is preferable, since it allows drainage.

* Leonards, G.A., "Foundation Engineering", p. 949
** Recent experience in Toronto, indicates that 75 mm (3-inch) boards may be used over a span width of 3 m (10 feet) for the first 3 m (10 feet) of depth.

In freezing conditions, more positive means of drainage and frost protection may be necessary to prevent heaving forces against the shoring system or possible softening of soil behind soldier piles. Raker-supported soldier piles have been known to be pushed upward following softening at the soil-pile interface.

Pile Penetration

Soldier piles must penetrate below the base of the general excavation, or below the bearing level of an adjoining footing if it is lower, a distance 'd', as computed from the expression:-

$$R = NB(\tfrac{1}{2} K_p \gamma d^2 + 2cd)$$

where
- R = the passive resistance of the soil developed below excavation level, in front of the buried pile
- N = factor to account for three-dimensional resistance around the pile tip. A value $N = 2$ is considered to be available to allow for three-dimensional resistance
- B = diameter of concrete-filled hole
- γ = density of the material is estimated to be about 22 kN/m^3 (140 pcf)
- K_p = 3, is the estimated passive earth pressure coefficient
- c = cohesion in the soil below the excavation level, estimated to be 240 kPa (5000 psf)

The value for 'd' is computed by trial and error by equating moments of the earth pressure force over one soldier pile bay and the passive resistance 'R' about the lowest support point. The computed value of 'd' must be increased by 50 per cent to provide a safety factor.

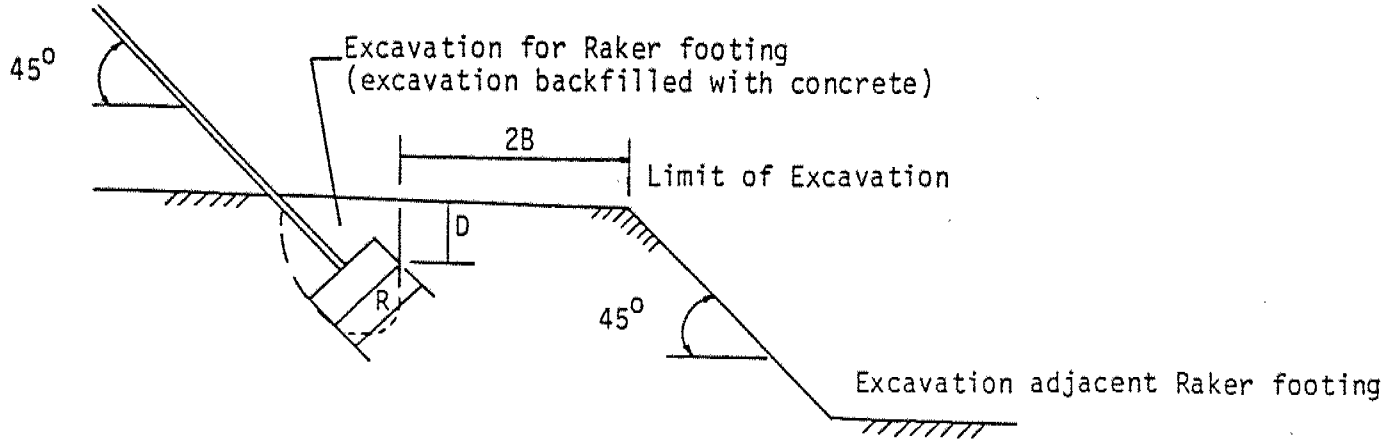
No excavation is to be made closer than 1.5 'd' in front of the pile.

Rakers

The rakers must be installed while an earth berm remains in front of the pile. This is illustrated in Drawing B4. Slots should be cut into this berm to install rakers before the supporting berm is removed. The design load must be jacked into the raker and the raker shimmed accordingly.

Raker Footings

Raker footings may be designed, assuming a bearing surface equivalent to the largest area of buried concrete at right angles and concentric to the raker, as shown on the diagram below.



For raker footings bearing in the hard clayey silt till and dense to very dense silt till, the permissible bearing value is given by the expression:

$$q = \frac{Nc}{F}$$

where c = cohesion in the soil, a value of 240 kPa (5000 psf) may be used

N = bearing capacity factor, depending on the inclination of footing and depth cover. For D/B ratio of 1.0, N = 5, with the raker at 45 degrees to the horizontal, provided any excavation is kept a distance of 2 B from the edge of the footing. For D/B = 0.5, N = 4.4

F = factor of safety = 2

Any excavation within the 2B limit should be referred to this office for comment.

General Notes

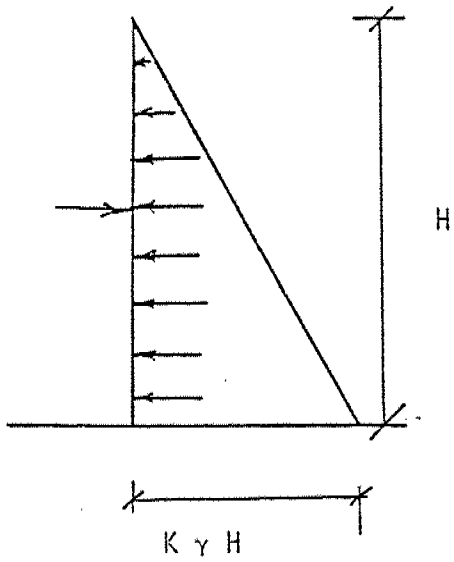
The shoring system must be designed for the worst condition that may apply. This is not necessarily when the excavation is completed.

Close monitoring of lateral movement of the shoring system will be required in settlement sensitive areas. This should include a representative number of borehole inclinometers, read in conjunction with conventional ground surveys. If movements indicate a tendency to increase above 12 mm, extra bracing is required.

This shoring system is provided as a general guideline for retaining earth for construction. If the shoring system is also intended to support adjacent structures, i.e. footings, tanks, roads, services etc. this office must be advised of this condition and the shoring system will be modified to consider the settlement criterion for the adjacent structure.

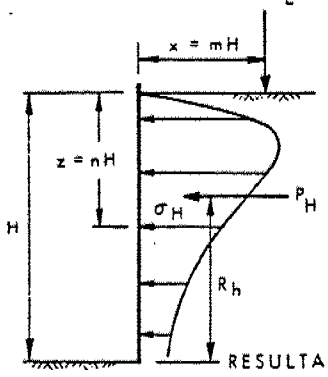
The foregoing comments are for general guidance only.

Pressure Distribution on Shoring



1. Check all Systems for Partial Excavation condition
2. If free water level above base of excavation, hydrostatic pressure must be added to above pressure distribution in sands.
3. If surcharge loading is present at or near ground surface these must be included in the lateral pressure calculation.

LINE LOAD Q_L



FOR $m \leq 0.4$:

$$\sigma_H\left(\frac{H}{Q_L}\right) = \frac{0.20n}{(0.16 + n^2)^2}$$

$$P_H = 0.55\ Q_L$$

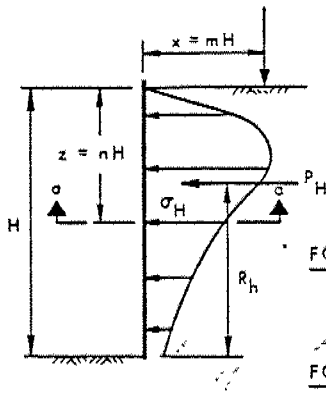
FOR $m \geq 0.4$:

$$\sigma_H\left(\frac{H}{Q_L}\right) = \frac{1.28\ m^2 n}{(m^2 + n^2)^2}$$

RESULTANT $P_H = \dfrac{0.64\ Q_L}{(m^2 + 1)}$

PRESSURES FROM LINE LOAD Q_L
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)

POINT LOAD Q_p



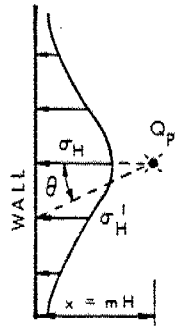
FOR $m \leq 0.4$:

$$\sigma_H\left(\frac{H^2}{Q_p}\right) = \frac{0.28n^2}{(0.16 + n^2)^3}$$

FOR $m > 0.4$:

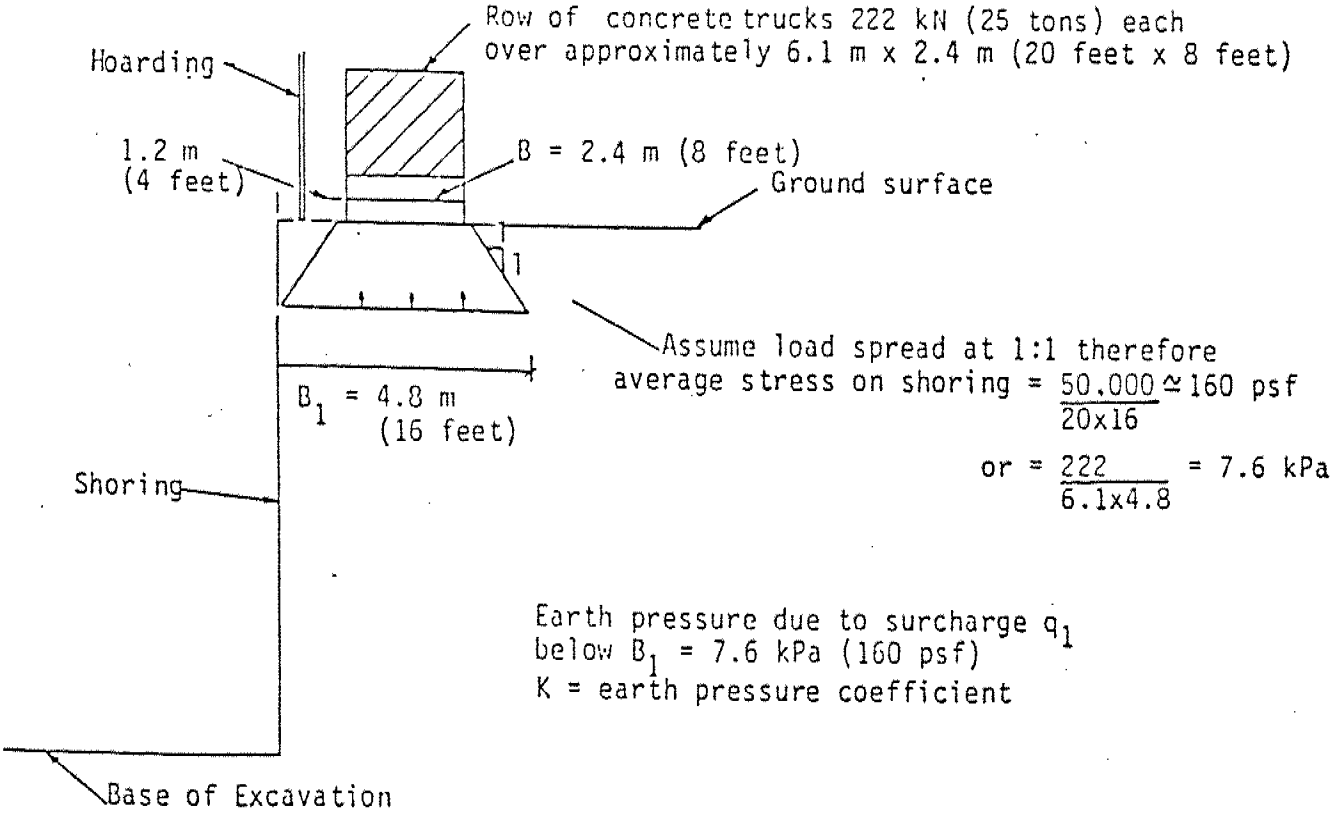
$$\sigma_H\left(\frac{H^2}{Q_p}\right) = \frac{1.77\ m^2 n^2}{(m^2 + n^2)^3}$$

$$\sigma_H^1 = \sigma_H \cos^2(1.1\ \theta)$$



SECTION a - a
PRESSURES FROM POINT LOAD Q_p
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)

HORIZONTAL PRESSURES ON WALL DUE TO SURCHARGE



SURCHARGE FROM ROW OF CONCRETE TRUCKS

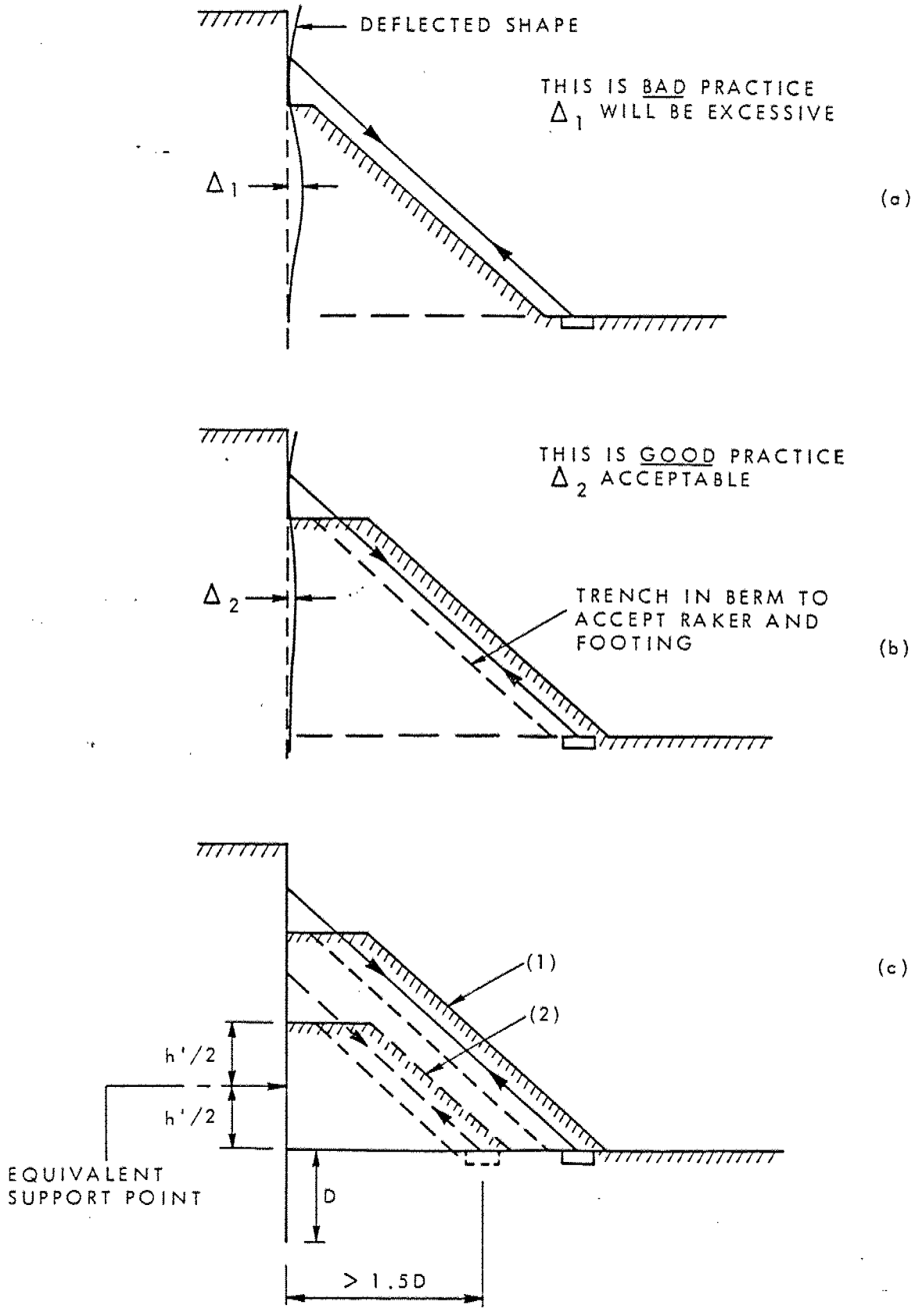


FIG 3.13

PLACEMENT OF RAKER STRUTS

Borehole Log

Trow Ltd

Auger Sample ⊠ | Natural Moisture ×
SPT (N) Value ○ ○ ▨ | Plastic and Liquid Limit ⊢—○
Dynamic Cone Test —— | Undrained Triaxial at Overburden Pressure, % Strain at Failure: 0 / 15 ⊕ 5 / 10
Shelby Tube ● ● ■ | Penetrometer ▲
Field Vane Test +s
Lab Vane Test t

Project CENTRE STREET GRADE SEPARATION WITH CNR TRACKS, BRAMPTON, ONTARIO
Dwg. No. 2
Borehole No. 1
Project No. T 5955-G
Hole location and datum see drawing No. 1

GWL	SYMBOL	Soil Description	ELEV. m (ft)	DEPTH m
	F F F F F F F	~75-100 mm Topsoil over FILL - sand, till-like, brown, slight moist, becoming silty and clayey, mottled, greenish-brown to grey with depth. Loose to compact, assorted rubble - tin cans, wire cable, rubber tires, glass, wood and brick	218.4 (716.6)	0
▼		CLAYEY SILT TILL - brown, slightly moist, dense, becoming less clayey, grey, dry, very dense with depth, some sand mixing at upper boundary. Gravel or cobbles at ~6.7 m and ~7.0 m. Coarse gravel at lower boundary.	214.4 (703.4)	4
			213.2 (699.5)	6
		SHALE TILL-red/brown, dry, very dense, high shale content	209.1 (686.0)	8
		ASSUMED SOUND SHALE BEDROCK - split-spoon bouncing at 10.6 m	208.4 (683.8)	10
		END OF BOREHOLE	207.7 (681.5)	12

N Value (ASTM) D1586-CSA A119.1: 20 40 60 80
Shear Strength: 0.1 MPa 0.2
110 / 305mm
60 / 38 mm
Natural Moisture Content and Atterberg Limits % Dry Weight: 10 20 30
Natural Unit Weight kN/m³
2 4 ksf

NOTES:
1. Borehole advanced with solid-stem augers on June 28th, 1983
2. Water Level Records:

Time	W.L. at (m)	Hole Open to (m)
On completion	7.3	10.2
After 1 day	5.3	-
After 9 days	5.2	-

TL102rD

Borehole Log

Trow Ltd

Auger Sample ⊠ | Natural Moisture ×
SPT (N) Value ○ ○ ▨ | Plastic and Liquid Limit ⊢○
Dynamic Cone Test —— | Undrained Triaxial at Overburden Pressure, % Strain at Failure: 0 / 15 / 5 / 10
Shelby Tube ● ● ■ | Penetrometer ▲
Field Vane Test +s
Lab Vane Test t

Project: CENTRE STREET GRADE SEPARATION WITH CNR TRACKS, BRAMPTON, ONTARIO
Dwg. No. 3
Borehole No. 2
Project No. T 5955-G
Hole location and datum see drawing No. 1

GWL	SYMBOL	Soil Description	ELEV. m (ft)	DEPTH m
	F F	~75-100 mm Topsoil over FILL - sand, till-like, brown	219.3 (719.5)	0
		CLAYEY SILT TILL - mottled, greenish-brown and grey, slightly moist, dense, changing to brown, moist, compact, changing to grey, dry, very dense. Some dark-brown silt pockets, sand partings at ~6.2 m, wet oxidized seams at ~6.3 m	218.4 (716.6)	2
▽			213.7 (701.1)	4
		SHALE TILL AND HEAVILY WEATHERED SHALE - red/brown, slightly moist, very dense	210.7 (691.3)	6
		ASSUMED SOUND SHALE BEDROCK split-spoon bouncing at 10.6 m	209.3 (686.7)	8
		END OF BOREHOLE	208.6 (684.4)	10

N Value (ASTM) D1586-CSA A119.1: 20 40 60 80
Shear Strength: 0.1 MPa 0.2
Natural Moisture Content and Atterberg Limits, % Dry Weight: 10 20 30
Natural Unit Weight kN/m³ (pcf)

Natural Unit Weight: 22.3 (139); 21.9 (137); 22.1 (138)

NOTES:
1. Borehole advanced with solid-stem augers on June 28th, 1983
2. Water Level Records:

Time	W.L. at (m)	Hole Open to (m)
On completion	10.0	10.5
After 1 day	5.4	-
After 9 days	5.6	-

2 ksf 4

TL102r0

Borehole Log

Troy Ltd

Auger Sample ⊠ | Natural Moisture ×
SPT (N) Value ○ ○ ▨ | Plastic and Liquid Limit ⊢○
Dynamic Cone Test —— | Undrained Triaxial at Overburden Pressure, % Strain at Failure 0 / 15 5 / 10
Shelby Tube ● ● ■ | Penetrometer ▲
Field Vane Test +s
Lab Vane Test t

Project: GRADE SEPARATION
CENTRE STREET AT CNR
BRAMPTON, ONTARIO
Hole location and datum see drawing No. 1

Dwg. No. 4
Borehole No. 3
Project No. T 5955-G

GWL	SYMBOL	Soil Description	ELEV. m (ft.)	DEPTH m
	F	FILL - clayey silt, occasional gravel sizes, organic-stained brown and grey, moist to wet, loose changes to wet fine to medium sand at 1.7 m	219.60	0
		CLAYEY SILT TILL - occasional gravel sizes and silt pockets, brown, moist hard	217.5	2
		END OF BOREHOLE	216.1	

N Value (ASTM) D1586-CSA A119.1): 20 40 60 80
Shear Strength: 0.1 0.2 MPa
Natural Moisture Content and Atterberg Limits % Dry Weight: 10 20 30
Natural Unit Weight kN/m³

NOTES:
1. Borehole advanced with solid stem augers on January 16, 1985.
2. Water Level Records:

Time	Water Level	Hole Open
On completion	1.6 m	1.9 m
Backfilled		

2 ksf 4

TL101r0

Borehole Log

Trow Ltd

Auger Sample ⊠ | Natural Moisture ×
SPT (N) Value ○ ○ ▨ | Plastic and Liquid Limit ⊢○
Dynamic Cone Test ——— | Undrained Triaxial at Overburden Pressure, % Strain at Failure: 0 / 15 ⊕ 5 / 10
Shelby Tube ● ● ■ | Penetrometer ▲
Field Vane Test +s
Lab Vane Test t

Project GRADE SEPARATION
CENTRE STREET AT CNR
BRAMPTON, ONTARIO
Hole location and datum see drawing No. 1

Dwg. No. 5
Borehole No. 4
Project No. T 5955-G

GWL	SYMBOL	Soil Description	ELEV. m (ft.)	DEPTH m
		100 mm asphaltic concrete	218.56	0
	F F F F	FILL - 600 mm road base and clayey silt with occasional gravel sizes	217.9	
		CLAYEY SILT TILL - occasional gravel sizes, mottled-brown and grey, brown at 2.5 m, moist, hard, becomes very stiff at 4.6 m		1 2 3 4
		END OF BOREHOLE	213.6	5

N Value (ASTM) D1586-CSA A119.1): 20 40 60 80
Shear Strength MPa: 0.1 0.2
Natural Moisture Content and Atterberg Limits % Dry Weight: 10 20 30
Natural Unit Weight kN/m³

NOTES:
1. Borehole advanced with solid stem augers on January 16, 1985.
2. Water Level Records:

Time	Water Level	Hole Open
On completion	dry	4.6 m
10 min.	dry	4.6 m
Backfilled		

2 ksf 4

TL10r0

Borehole Log

Trow Ltd

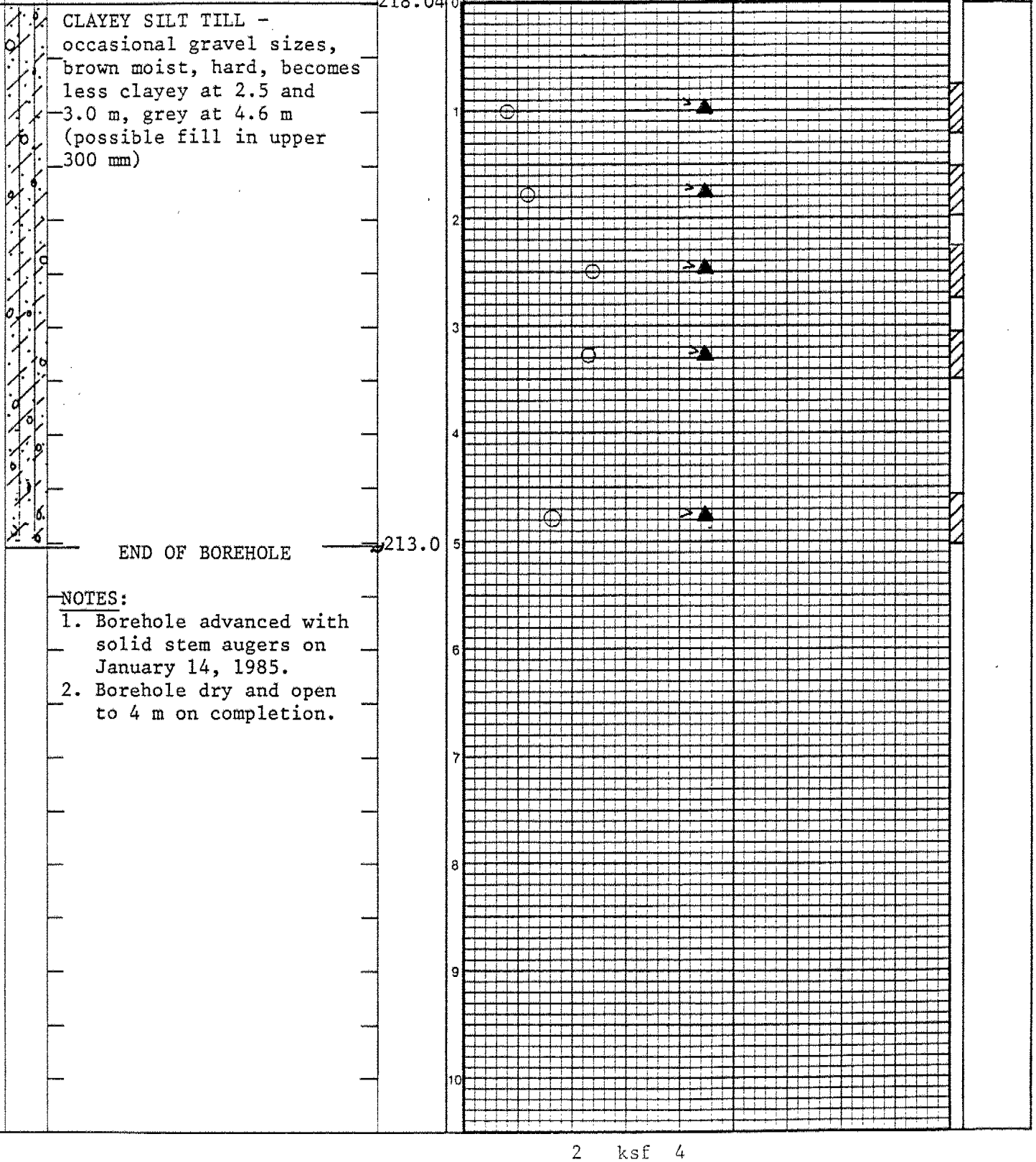
Auger Sample ⊠ | Natural Moisture ×
SPT (N) Value ○ ○ ▨ | Plastic and Liquid Limit ⊢○
Dynamic Cone Test ——— | Undrained Triaxial at Overburden Pressure 0 / 15 ⊕ 5 / 10 % Strain at Failure
Shelby Tube ● ● ■ | Penetrometer ▲
Field Vane Test +s
Lab Vane Test t

Project GRADE SEPARATION — Dwg. No. 6
CENTRE STREET AT CNR — Borehole No. 5
BRAMPTON, ONTARIO — Project No. T 5955-G
Hole location and datum see drawing No. 1

GWL	SYMBOL	Soil Description	ELEV. m (ft.)	DEPTH m
		CLAYEY SILT TILL - occasional gravel sizes, brown moist, hard, becomes less clayey at 2.5 and 3.0 m, grey at 4.6 m (possible fill in upper 300 mm)	218.04	0
		END OF BOREHOLE	213.0	5

NOTES:
1. Borehole advanced with solid stem augers on January 14, 1985.
2. Borehole dry and open to 4 m on completion.

N Value (ASTM) D1586-CSA A119.1) 20 40 60 80
Shear Strength 0.1 0.2 MPa
Natural Moisture Content and Atterberg Limits % Dry Weight 10 20 30
Natural Unit Weight kN/m³



2 ksf 4

TL101r0

Borehole Log

Trow Ltd

Auger Sample
SPT 'N' Value
Dynamic Cone Test
Shelby Tube
Field Vane Test
Lab Vane Test

Natural Moisture
Plastic and Liquid Limit
Undrained Triaxial at Overburden Pressure
% Strain at Failure
Penetrometer

Project: GRADE SEPARATION
CENTRE STREET AND CNR
BRAMPTON, ONTARIO

Dwg No 7
Borehole No 6
Project No T 5955-G

Hole location and datum see drawing No 1

USCS	Symbol	Soil Description	ELEV m (ft.)
	F	FILL - clayey silt, occasional gravel sizes, pieces of wood, minor organic-staining, brown to dark brown, moist, loose	219.52
			218.1
		CLAYEY SILT TILL - occasional gravel sizes, oxidized fissures, mottled brown and grey at 1.5 m, brown at 2.3 m, moist, hard, becomes very stiff to firm with depth	
		Gas reading at the completion of drilling in the open borehole was 0% (on 5% scale)	214.1
			212.5
		SILT TILL - occasional gravel sizes, sandy at 7.6 m, grey, moist, very dense. Borehole becomes wet at 7.6 m	211
			210.3
		SHALE TILL - red brown, slightly moist, very dense	
			209.0
		SHALE (assumed) - recovered only ground up shale and small shale fragments, red-brown, slightly moist, very dense	208.8
		END OF BOREHOLE	

N Value ASTM D1586 CSA A119.1)
20 40 60 80
Shear Strength MPa
0.1 0.2

Natural Moisture Content and Atterberg Limits % Dry Weight

Natural Unit Weight kN/m³

NOTES:
1. Borehole advanced with solid stem augers on January 15/85.
2. Water Level Records:

Time	Water Level	Hole Open
On completion	10.4 m	10.5 m
4.5 hrs.	8.8 m	9.3 m
1 day	5.4 m	7.5 m

FLOW 0

2 ksf 4

Borehole Log

Trow Ltd

Auger Sample ⊠ | Natural Moisture ×
SPT (N) Value ○ ○ ▨ | Plastic and Liquid Limit ⊢○
Dynamic Cone Test ——— | Undrained Triaxial at Overburden Pressure / % Strain at Failure 0 / 15 ⊕ 5 / 10
Shelby Tube ● ● ■ | Penetrometer ▲
Field Vane Test +s
Lab Vane Test t

Project: GRADE SEPARATION
CENTRE STREET AT CNR
BRAMPTON, ONTARIO
Hole location and datum see drawing No. 1

Dwg. No. 8
Borehole No. 7
Project No. T 5955-G

GWL	SYMBOL	Soil Description	ELEV. m (ft.)	DEPTH m
	F F F F F F F F	FILL - clayey silt, occasional gravel sizes and topsoil pockets brown, moist, hard	218.54	0
			~217.5	1
		CLAYEY SILT TILL - occasional gravel sizes and silt pockets, silty clay seam at 4.6 m, brown, grey at 4.6 m, moist		2
		hard, becomes firm to stiff at 4.6 m		3
		Gas reading in open borehole was 0% (on 5% scale)		4
		footing	214	
			213	5
			~212.7	6
		SILT TILL - occasional gravel sizes, slightly clayey and wet, fine to medium sand seam (~5 mm thick) at 6.1 m, grey, moist dense to very dense. Borehole became wet at ~6.1 m		7
				8
		END OF BOREHOLE	~210.5	
		NOTES:		9
		1. Borehole advanced with solid stem augers on January 15, 1985.		
		2. Water Level Records:		10

Time	Water Level	Hole Open
On completion	wet cave	7.6 m
2 hrs.	wet cave	7.1 m
1 day	5.9 m	6.5 m

N Value (ASTM) D1586-CSA A119.1): 20 40 60 80
Shear Strength: 0.1 MPa 0.2
2 ksf 4

Natural Moisture Content and Atterberg Limits % Dry Weight: 10 20 30

Natural Unit Weight kN/m³

TL101r0

Borehole Log

Trow Ltd

Auger Sample ⊠ | Natural Moisture ×
SPT (N) Value ○ ○ ▨ | Plastic and Liquid Limit ⊢o
Dynamic Cone Test —— | Undrained Triaxial at Overburden Pressure, % Strain at Failure 0 / 15 5 / 10
Shelby Tube ● ● ■ | Penetrometer ▲
Field Vane Test +s
Lab Vane Test t

Project GRADE SEPARATION
CENTRE STREET AT CNR
BRAMPTON, ONTARIO
Hole location and datum see drawing No. 1

Dwg. No. 9
Borehole No. 8
Project No. T 5955-G

GWL	SYMBOL	Soil Description	ELEV. m (ft.)	DEPTH m
	F F F F F	FILL - clayey silt, organic stained, pieces of wood, brown, moist	217.88	0
		CLAYEY SILT TILL - occasional gravel sizes, oxidized fissures, mottled-brown and grey to 1.5 m greyish-brown to 3 m and grey at 4.6 m, moist, hard becoming stiff to very stiff at 4.6 m	216.9	1
		SILT TILL - contains thin sand seams, grey, moist, very dense	212.3	6
		SHALE TILL - mainly re-worked shale fragments, red-brown, slightly moist, very dense	211.2	7
		END OF BOREHOLE	210.0	8

N Value (ASTM) D1586-CSA A119.1): 20 40 60 80
Shear Strength 0.1 0.2 MPa
Natural Moisture Content and Atterberg Limits % Dry Weight: 10 20 30
Natural Unit Weight kN/m³

50/150 mm

NOTES:
1. Borehole advanced with solid stem augers on January 16/84.
2. Water Level Records:

Time	Water Level	Hole Open
On completion	dry	6.7 m
Backfilled		

TL101r0

2 ksf 4

Borehole Log

Trow Ltd

Auger Sample ⊠ | Natural Moisture ×
SPT (N) Value ○ ○ ▨ | Plastic and Liquid Limit ⊢○
Dynamic Cone Test ——— | Undrained Triaxial at Overburden Pressure, % Strain at Failure 0 / 15 5 / 10
Shelby Tube ● ● ■ | Penetrometer ▲
Field Vane Test +s
Lab Vane Test t

Project: GRADE SEPARATION
CENTRE STREET AT CNR
BRAMPTON, ONTARIO

Dwg. No. 10
Borehole No. 9
Project No. T 5955-G

Hole location and datum see drawing No. 1

GWL	SYMBOL	Soil Description	ELEV. m (ft.)	DEPTH m
	F F F F F F F F	FILL - clayey silt, contains occasional gravel sizes, glass and brick fragments, organic-stained, brown to dark brown, moist, hard	218.64	0
		CLAYEY SILT TILL - occasional gravel sizes, slightly mottled-brown with grey at 1.5 m, brown below 2.3 m, moist, hard	217.2	1
				2
				3
				4
				5
		SILT TILL - occasional gravel sizes, trace of clay, wet sandy silt till pocket or seam at 6.1 m, grey, moist, very dense	213.1	6
				7
				8
		SILT TILL - occasional shale fragments, red-brown moist, very dense	210.1	9
		END OF BOREHOLE	209.5	10

N Value (ASTM) D1586-CSA A119.1: 20 40 60 80
Shear Strength MPa: 0.1 0.2

Natural Moisture Content and Atterberg Limits % Dry Weight: 10 20 30

Natural Unit Weight kN/m³

○/2.15 mm
○/125 mm
○/50 mm

NOTES:
1. Borehole advanced with solid stem augers on January 15, 1985.

2. Water Level Records:

Time	Water Level	Hole Open
On completion	wet cave	8.2
1 day	3.5	6.2

TL101r0

Borehole Log

Auger Sample
SPT N Value
Dynamic Cone Test
Shelby Tube
Field Vane Test
Lab Vane Test
Natural Moisture
Plastic and Liquid Limit
Undrained Triaxial at Overburden Pressure
% Strain at Failure
Penetrometer

Project: GRADE SEPARATION
CENTRE STREET AT CNR
BRAMPTON, ONTARIO
Hole location and datum see drawing No. 1

Dwg No 11
Borehole No 10
Project No T 5955-G

Soil Description	ELEV m (ft.)	Depth m
FILL - clayey silt, occasional gravel sizes, pieces of wood and organic-stained pockets, numerous pieces of wood at 3 m, brown grey and black, moist, loose to compact	217.65	0
Gas reading after 15 minutes in open borehole was 4% (on 100% scale) after one day gas reading was 20% (on 5% scale)		
CLAYEY SILT TILL - occasional gravel sizes and shale fragments, brown, moist, hard	212.2	5
SILT TILL - contains shale fragments, reddish-brown, slightly moist, very dense	209.2	8
SHALE - red-brown, heavily to completely weathered, very dense	207.7	9.5
END OF BOREHOLE	207.0	10.2

N Value ASTM D1586 CSA A119.1
20 40 60 80
Shear Strength 0.1 0.2 MPa

Natural Moisture Content and Atterberg Limits % Dry Weight
10 20 30

Natural Unit Weight kN/m³

NOTES:
1. Borehole advanced with solid stem augers on January 14, 1985.
2. Water Level Records:

Time	Water Level	Hole Open
On completion	9.1 m	9.6 m
1 day	4.6 m	7.1 m
2 days	4.6 m	7.1 m

1 2 ksf 4

1100-0

Borehole Log

Trow Ltd

Auger Sample ⊠ | Natural Moisture ×
SPT (N) Value ○ ○ ▨ | Plastic and Liquid Limit ⊢—○
Dynamic Cone Test —— | Undrained Triaxial at Overburden Pressure, % Strain at Failure: 0 / 15 ⊕ 5 / 10
Shelby Tube ● ● ■ | Penetrometer ▲
Field Vane Test +s
Lab Vane Test ℄

Project GRADE SEPARATION
CENTRE STREET AT CNR
BRAMPTON, ONTARIO
Hole location and datum see drawing No. 1

Dwg. No. 12
Borehole No. 11
Project No. T 5955-G

GWL	SYMBOL	Soil Description	ELEV. m (ft.)	DEPTH m
	F	FILL - clayey silt with cinders, paper, wood, glass; brown, grey and black, moist, very loose to loose	213.50	0
	F			
	F			1
	F			
	F			2
	F			
	F			3
	F	Gas reading after 4.5 hrs. in open hole was 12% (on 5% scale). No gas after one day.		
	F			4
	F			
	F	Garbage content is much less below 4.6 m		5
	F			
	F			6
	F			
	F	Hard augering at 7.0 m		7
		SHALE - red-brown, heavily to completely weathered, very dense	~206.5	
		END OF BOREHOLE	~205.9	8

N Value (ASTM) D1586-CSA A119.1): 20 40 60 80
Shear Strength MPa
75 mm

Natural Moisture Content and Atterberg Limits % Dry Weight: 10 20 30

Natural Unit Weight kN/m³

NOTES:

1. Borehole advanced with solid stem augers on January 14, 1985.
2. Water Level Records:

Time	Water Level	Hole Open
On completion	0.7 m	2.6 m
5 hrs.	0.7 m	2.6 m
1 day	0.6 m	2.0 m
2 days	0.8 m	2.1 m

TL10hrO

Borehole Log

Trow Ltd

Auger Sample ⊠ | Natural Moisture ×
SPT (N) Value ○ ○ ▨ | Plastic and Liquid Limit ⊢○
Dynamic Cone Test ——— | Undrained Triaxial at Overburden Pressure % Strain at Failure 0 / 15 5 / 10
Shelby Tube ● ● ■ | Penetrometer ▲
Field Vane Test +s
Lab Vane Test t

Project GRADE SEPARATION
CENTRE STREET AT CNR
BRAMPTON, ONTARIO
Hole location and datum see drawing No. 1

Dwg. No. 13
Borehole No. 12
Project No. T 5955-G

GWL	SYMBOL	Soil Description	ELEV. (ft.)	DEPTH (m)
	F	FILL - clayey silt with occasional gravel sizes, pieces of wood, brick fragment and topsoil pockets, brown, grey and black, moist, compact to very dense	212.06	0
	F	Gas reading after 6 hours in open hole was 2% (on 5% scale)		2
	F	No gas after one day		3
		CLAYEY SILT TILL - occasional gravel sizes and shale fragment, brown moist, hard	~206.1	6
		SILT TILL - numerous shale fragments red-brown to greenish grey moist, very dense	~205.1	7
		END OF BOREHOLE	~204.3	

N Value (ASTM) D1586-CSA A119.1): 20 40 60 80
Shear Strength MPa: 0.1 0.2
Natural Moisture Content and Atterberg Limits % Dry Weight: 10 20 30
Natural Unit Weight kN/m³

NOTES:
1. Borehole advanced with solid stem augers on January 14, 1985.
2. Water Level Records:

Time	Water Level	Hole Open
On completion	1.1 m	6.0 m
5.5 hrs.	0.7 m	5.8 m
1 day	0.8 m	3.5 m
2 days	0.9 m	3.5 m

50/150 mm

TL101r0

2 ksf 4

PLANMAC CONSULTANTS LIMITED
CONSULTING ENGINEERS & PLANNERS
3055 Lakeshore Blvd. West,
Toronto, Ontario M8V 1K6
Tel. (416) 259-5468

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MAR - 2 1987
FOUNDATION DESIGN SECTION

February 18th, 1987

Ministry of Transportation and Communications
Structural Office
4th Floor, 3501 Dufferin Street
Downsview, Ontario
M3K 1N6

Attention: Mr. A.A. Witecki, P.Eng.
Municipal Engineer, Approvals Section

Re: Centre Street Grade Separation
City of Brampton
Contract 86-118
MTC Site No. 24-437
Our E.O. 8316

Dear Sir:

We acknowledge receipt of your letter of February 9th, 1987 concerning the above project.

We wish to advise that we have re-adjusted and/or checked the design of the retaining walls using amended geotechnical parameters recommended to us by Trow Limited and have revised the drawings accordingly. We are therefore enclosing herewith four (4) sets of the revised drawings as you have requested. In order to facilitate your review of the design and also to ensure that the design is consistent with the recommendations of Trow Limited, we have summarized below the geotechnical design parameters used, particularly with respect to the "north-west" retaining wall:

- Soil Parameters: As per Trow letter dated Jan. 28/87 (copy attached).

- Coefficients of Active and Passive Pressure (ka and kp):

  Calculated in accordance with the RANKINE SINE formula; a performance factor of 0.8 applied when computing passive resistance in the ULS state.

- Sliding resistance for north-west wall:
  (refer also to Trow letter dated December 29th, 1986)

  i) Sliding considered as occurring along a horizontal plane at the level of the bottom of the caissons founded in shale. Full block of soil in front of the caissons considered as being mobilized for passive resistance.

  ii) Materials below the underside of the footing considered as being submerged (γ=12.5 kN/m^3). Weight of earth block from the underside of the footing to the plane of sliding added to vertical loads when computing frictional resistance.

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iii) Passive resistance of the soil block from the underside of the footing to the bottom of the caissons considered. Soil in front of footing face not considered as contributing to passive resistance but included as a surcharge load.

As outlined above, we believe that the geotechnical parameters used in the revised design are in accordance with the most recent recommendations from Trow Limited. By copy of this letter we are requesting Trow Limited to confirm this.

We trust that the enclosed documents are satisfactory and that your review can now proceed. Should you have any questions however, or wish to discuss any aspect of this matter, please contact us.

We thank you for your continued co-operation on this matter and remain,

Yours truly,

PLANMAC CONSULTANTS LIMITED

D.C. Batchelor

D.C. Batchelor, P.Eng.

DKH/nb
Encl.
cc: Mr. M. Devata - MTC
Mr. M. Lostracco - City of Brampton
Dr. K. Peaker - Trow Limited

Trow Ltd
Consulting Engineers

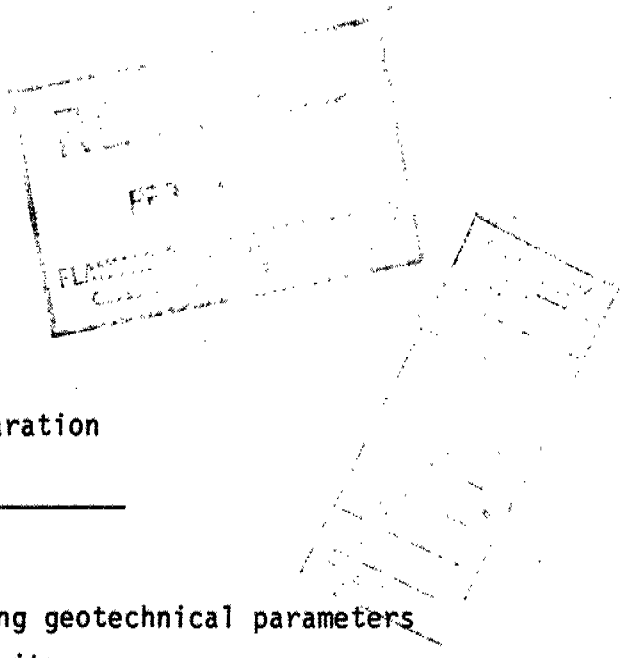
Head Office: 1595 Clark Boulevard
Brampton, Ontario, Canada
L6T 4V1
Telephone: (416) 793-9800
1-800-387-2437
Telex: 06-97802

Project: T 5955-G

January 28, 1987

City of Brampton
c/o Planmac Consultants Limited
3055 Lakeshore Blvd. West
Toronto, Ontario
M8W 1K8

Attention: Mr. D.C. Batchelor, P.Eng.



Dear Sir:

Soil Parameters
Centre Street Grade Separation
Brampton, Ontario

Further to our discussions, the following geotechnical parameters are considered applicable to the subsoil at this site.

Subsoil*	**Geotechnical Parameters**
Clayey Silt Till Down to el. 213.0	γ_o = 20.5 kN/m³ ϕ = 20° c = 20 kPa
Dense to Very Dense Silt Till Down to U/S ftg.	γ 1 = 22 kN/m³ ϕ 1 = 30° c = 0
Shale Till Down to el. 208.5 (submerged)	γ 2 = 12.5 kN/m³ ϕ = 25° c = 0
Sound Shale Below el. 208.5 (submerged)	γ 3 = 12.5 kN/m³ ϕ = 22½° c = 0

* refer to boreholes 8 and 9

Trow

Should you have any questions on the above, please do not hesitate contact this office.

Yours very truly,
TROW GEOTECHNICAL LTD.

H. Lohse, B.Sc.,

K.R. Peaker, P.Eng.

Distribution:
City of Brampton
c/o Planmac Consultants Limited (3)

Trow Consulting Engineers

By Courier

Trow Geotechnical Ltd.
1595 Clark Boulevard, Brampton
Ontario, Canada. L6T 4V1
Telephone: (416) 793-9800
Telex: 06-97802

Project: T5955-G

December 29, 1986

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JAN 5
PLANMAC CONSULTANTS LIMITED
Consulting Engineers & Planners

Planmac Consultants Limited
3055 Lakeshore Boulevard West
Toronto, Ontario
M8V 1K6

Attention: Mr. D. Horgan, P. Eng.

Dear Sirs:

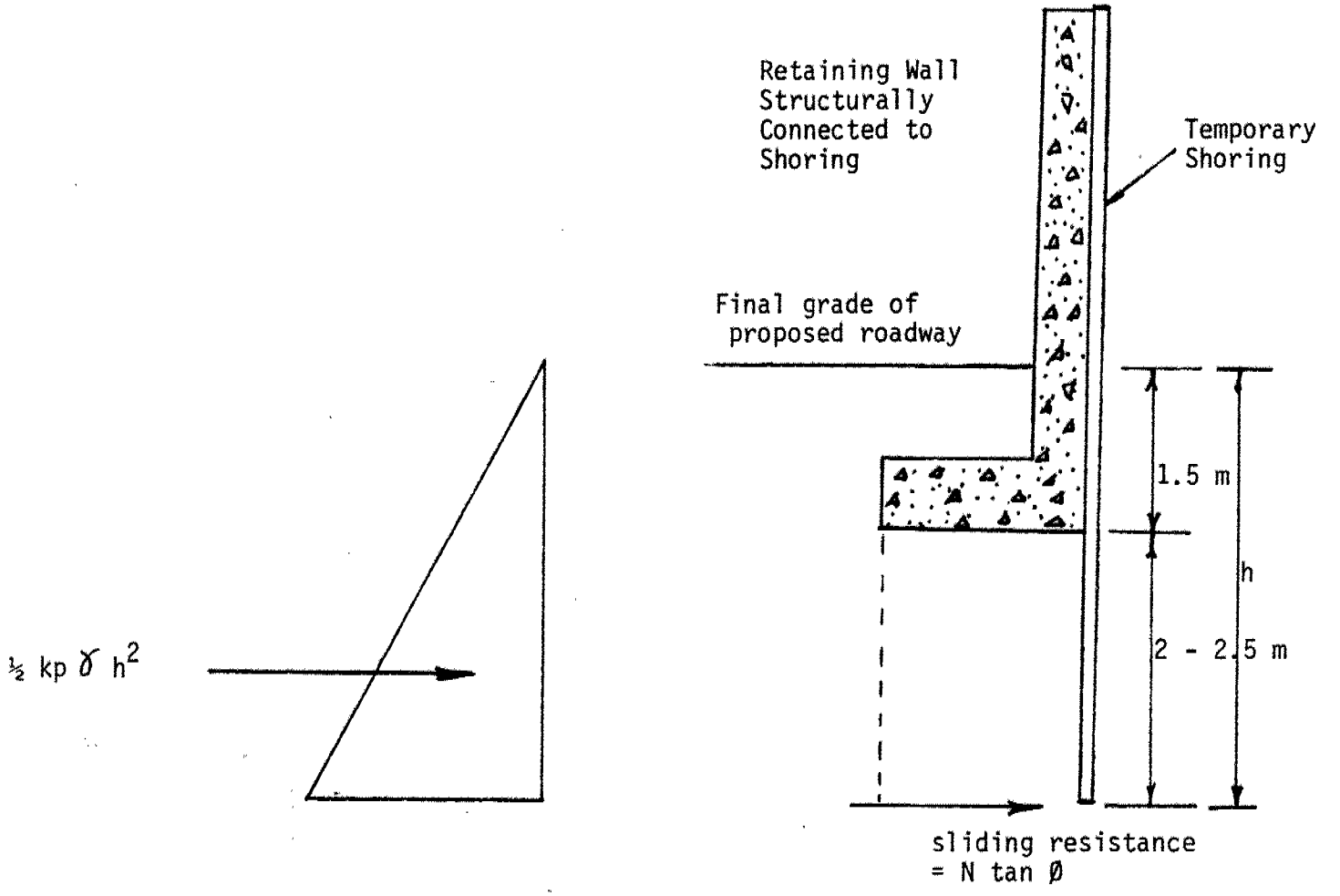
Design Comments on Retaining Wall
Centre Street Grade Separation
Brampton, Ontario

Further to our telephone conversation, we would like to confirm our comments regarding sliding resistance of the proposed temporary shoring and retaining wall.

It is understood that the temporary shoring will consist of soldier piles and lagging with soldier pile spacing at 2.4 metres (8 feet) centres. The soldier piles will penetrate about 3.6 metres (12 feet) to 4 metres (13 feet) below the proposed final grade of the road. The retaining wall will be L-shaped and constructed immediately adjacent to the temporary shoring and structurally connected to it. The footing for the retaining wall will be found at about 1.5 metres (5 feet) below the final grade of the proposed roadway.

The passive and sliding resistance are presented on the following sketch. The parameters have been given to you previously. In compacting the sliding resistance, it can be assumed that the sliding resistance will act at the base of the temporary shoring, which will allow the weight of the block of rock and/or soil to be taken into account.

cont'd.....



We trust that the preceding comments are satisfactory for your needs. Should you have any queries, please do not hesitate to contact this office.

Yours very truly,

TROW GEOTECHNICAL LTD.

Shaheen A. Ahmad, M.A.Sc., P. Eng.
Manager
Toronto Geotechnical Division

SAA/ls

PLANMAC CONSULTANTS LIMITED
CONSULTING ENGINEERS & PLANNERS
3055 Lakeshore Blvd. West,
Toronto, Ontario M8V 1K6
Tel. (416) 259-5468

RECEIVED
FEB 20 1987
Foundation Design Section

February 18th, 1987

Ministry of Transportation and Communications
Room 315, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Attention: Mr. M. Devata, P.Eng.
Chief Foundations Engineer (East)

Re: Centre Street Grade Separation
City of Brampton
Contract 86-118
F.D.S. W.O. 86-11009
Our E.O. 8316

Dear Mr. Devata:

We acknowledge receipt of your letter of February 2nd, 1987 concerning the above project. We wish to confirm that our understanding of the purpose for our meeting on December 30th was not to obtain specific foundation recommendations but rather to obtain your comments and approval/disapproval of geotechnical parameters which we proposed to use in the design. We note that the parameters which were proposed at the meeting had previously been discussed with and recommended to us by Trow Limited. (Reference Trow Ltd. dated December 29th, 1986, copy attached).

Our meeting was very useful in getting your input on the parameters which enabled us to proceed with the design.

Further and more detailed geotechnical parameters, dealing with the particular soil types present at the site, were subsequently recommended to us by Trow in their letter of January 28th, 1987 (copy attached).

We trust that this will clarify this matter and again we thank you for your considerable input on this project. We remain,

Yours truly,

PLANMAC CONSULTANTS LIMITED

D.C. Batchelor.
D.C. Batchelor, P.Eng.

DKH/nb
Encl.
cc: Mr. A.A. Witecki, MTC

Ontario

Ministry of
Transportation and
Communications

(Tel: 416 235-3731)

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

1987 02 02

Planmac Consultants Limited
3055 Lakeshore Blvd. West
Toronto, Ontario
M8V 1K6

Attention: Mr. D.C. Batchelor, P. Eng.

RE: Centre Street Grade Separation
City of Brampton
Contract 86-118
F.D.S. W.O. 86-11009

Dear Sir,

This will acknowledge receipt of your letter to our Mr. M. Devata, dated January 14, 1987 and the subsequent Drawing Sheet 217. These documents were received in our office on January 26 and 27 respectively.

In your letter you have documented a number of recommendations pertaining to foundation design for your project and we are concerned that you may have misinterpreted the purpose of our December 30, 1986 meeting and our role in this project. Hopefully this letter will clarify our position and enable you to proceed with your design responsibilities.

As you will recall our December 30, 1986 meeting was at your request and its purpose was to provide you with some basic guidelines, and definitely not to provide you with specific foundation recommendations for your project.

If your require foundation design recommendations, it is your responsibility to obtain them for your geotechnical consultant. Our role in municipal projects is to review, at the request of the M.T.C. Structural Office, the foundation component of the design.

Yours truly,

D. H. Dundas

D.H. Dundas, P. Eng.
Sr. Foundations Engineer

for

M. Devata, P. Eng.
Chief Foundations Engineer
(East)

DHD/mmj

c.c. - K. Kleinsteiber
A. Lee

PLANMAC CONSULTANTS LIMITED
CONSULTING ENGINEERS & PLANNERS
3055 Lakeshore Blvd. West,
Toronto, Ontario M8V 1K6
Tel. (416) 259-5468

RECEIVED
Foundation Design Section

January 14, 1987

Ministry of Transportation and Communications
3rd Floor, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Attention: Mr. M. Devata, P.Eng.
Chief Foundations Engineer (East)
Foundation Design Section

Re: Centre Street Grade Separation
City of Brampton
Contract 86-118
Our E.O. 8316

Dear Mr. Devata:

I wish to thank you and Dave Dundas for meeting with Diarmuid Horgan and myself on December 30th, 1986 to discuss geotechnical aspects relating to the design of the retaining walls on the above noted project, particularly the retaining wall in front of the Cemetery as shown on Drawing No. 217 (copy attached.

I presented a number of design criteria and assumptions for your consideration. Our record of your comments and advice on these matters is as follows:

1. Sliding can be considered as occurring along a horizontal plane at the level of the bottom of the caissons (in shale) provided the caissons are spaced at approximately 2.5 m c/c and the caisson diameter is 750 mm.

   In this case, the full block of soil in front of the caissons can be considered as being mobilized for passive resistance.

2. Materials below the underside of the footing can be considered as being submerged (γs = 12.5 KN/m^3). The weight of the earth block from the underside of the footing to the plane of sliding can be added to the vertical loads when computing friction resistance.

3. The coefficients of active and passive pressure (Ka & Kp) should be calculated in accordance with the Rankine Sine formula. A performance factor of 0.8 should be applied when computing passive resistance in the ULS state.

4. Passive resistance of the soil block from the underside of the footing to the bottom of the caissons can be considered. The soil in front of the footings will not be considered as contributing to passive resistance due to the possible effects of frost. However it can be considered as a surcharge load.

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CONSULTING ENGINEERS & PLANNERS

5. In the eventuality that an excavation is carried out at some future date in front of the footing, cohesion can be considered as being mobilized, due to the short term nature of this condition.

We trust that the above record accurately reflects our discussion. If you disagree with any of these items, please advise us as soon as possible.

We appreciate your assistance and cooperation in this matter and remain,

Yours truly,

PLANMAC CONSULTANTS LIMITED

D.C. Batchelor

D.C. Batchelor, P.Eng.

DKH/nb

cc: Mr. M. Lostracco, P.Eng. - City of Brampton
Dr. K. Peaker, P.Eng. - Trow Limited
Mr. A. Lee, P.Eng. - MTC

(1)

Memo

M. Devata
Chief Foundation Engineer East
Foundation Design Section
Engineering Materials Office

Attn: D. H. Dundas

Re: W.O. 86-11009 (Retaining Walls)
Grade Separation - CNR @ Centre St
City of Brampton
MTC site No. 24-437

This will acknowledge the reciept of your memo dated Dec. 11, 1986 and further recommendations given to us during the follow-up discussion held in your office on Dec. 18, 1986.

May we have your comments or confirmation on the following expressions which are formulated based on the recommendations motioned above:

1. Passive resistance in front of the Soldier piles which are spaced @ 2.4m centres.
   - Coefficients of passive earth pressure:
     For silt till SLS $K_p = 3.7$
     For Shale $K_p = 4.5$
   - Due to the size of the footing, the passive resistance can be developed along the full length of the retaining wall. Assume the footing is on shale, embedment of pile into shale is 2.0 m, and the surcharge measured from bottom of footing @ toe is 1.5 m, then; at ULS
   - The total sliding resistance per metre length of wall, take panel NW3 for example, will be.

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Total Factored Sliding Resistance per metre (R^f)

= Factored Frictional + Factored Passive

$= (A' \cdot C_f + V \tan\phi_f) + P^f [\gamma^f_{sur} \times h_s \times D + \frac{1}{2} \gamma^f_{shale} \times D^2] \cdot K^f_p$

Where. $C_f = 0$ for shale

V = Wt. of the wall and soil above the horizontal surface at the bottom of the soldier piles in submerged condition

$= 420^{KN} + 6.3 \times 1.0 \times 2.0 \times (22 - 9.8) \times \alpha_{D\,min.}$

$= 543^{KN}$ (0.8)

$\tan\phi_f = \tan 16.3^\circ = 0.293$ for the shale.

$P^f = 0.8$, Performance factor, OHBDC 6-6.1.2.2

h_{s_f} = Height of surcharge measured from bottom of footing.

$\gamma_{sur} = \alpha_D \cdot \gamma_{surcharge} = 0.8 \times (22 - 9.8)$

$= 9.8^{KN/m^3}$

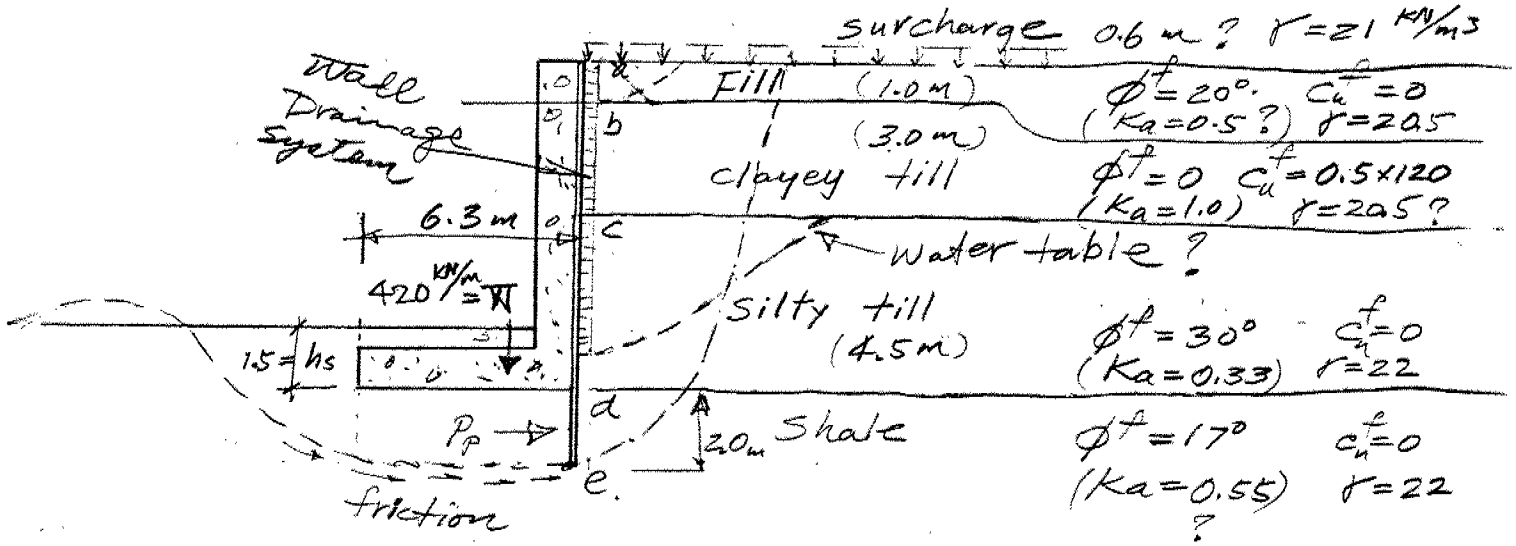
D = Depth of pile penetration $= 2.0m$

$\gamma^f_{shale} = 9.8^{KN/m^3}$ in submerged condition

$K^f_p = 3.5$ for shale.

$\therefore R^f = 543 \times 0.293 + 0.8\left(9.8 \times 1.5 \times 2 + \frac{1}{2} \times 9.8 \times 4\right) \times 3.5$

$= 159 + 137 = 296^{KN}$



2. In situ Factored Lateral Earth Pressures:

@ a. $E_a = 0.6\times21\times0.5\times\alpha_E = 6.3\times1.25 = 7.8$ kPa

b. (in fill) $E_b = (0.6\times21 + 1\times20.5)\,0.5\times1.25 = 20.7$ kPa

(in clay) $E_b = -2\,C_u^f\times\alpha_E = -120\times0.8 = -96$ kPa

c. (in clay) $E_c = 3.0\times20.5\times1.25 - 96 = -19.$ kPa

(in silt) $E_c = (0.6\times21 + 4\times20.5)\times0.33\times1.25$

$= 31.2\times1.25 = 39.0$ kPa

d. (in Silt) $E_d = (0.6\times21 + 4\times20.5 + 4.5\times22)\times0.33\times1.25$

$= 63.9\times1.25 = 80$ kPa

(in shale) $E_d = (0.6\times21 + 4\times20.5 + 4.5\times22)\times0.55\times1.25$

$= 133.3$ kPa.

e. (in Shale) $E_e = (0.6\times21 + 4\times20.5 + 4.5\times22 + 2\times22)\times0.55\times1.25$

$= 130.7\times1.25 = 163.4$ kPa.

∴ Total Lateral Force:

$\frac{1}{2}(7.8 + 20.7)\times1 = 31$ KN — in Fill

$\frac{1}{2}(-96 - 19)\times3 = -173$ KN — in Clay

$\frac{1}{2}(39 + 80)\times4.5 = 268$ KN — in Silt

$\frac{1}{2}(133.3 + 163.4)\times2 = 267$ KN — in Shale

443 KN

(4)

(3) Earth backfill on top of the heel of the footing, as shown in section (C) on DWG. # G4-8-87, is not recommended. Granular backfill should be used as per OHBDC 6-9.6.1

A. J. Lee
for AAW

memorandum

Ontario

Tel: 3282

To: A.A. Witecki
Municipal Engineer
Approvals Section
Structural Office
3501 Dufferin Street

Date: 1986 12 11

Atten: A.J. Lee

From: Engineering Materials Office
Foundation Design Section
Room 315, Central Building

RE: W.O. 86-11009 (Foundation I.D.#)
Grade Separation, Structure Site No. 24-437
Centre Street at CN Railway - Brampton
District 6, Toronto

As you requested, we have reviewed the Trow Ltd. geotechnical investigation report prepared for this project. The report was authorized by Planmac Consultants Ltd., the structural design consultant, on behalf of the City of Brampton.

Concerns regarding footing design and sliding resistance had arisen during your review of the structural design. Our review of the Trow report focuses on this aspect but also includes general comments.

1) Soils are identified and described by assessing their behaviour and their grain size characteristics. The Trow Ltd. report is incomplete in this respect as no Atterberg Limit or grain size test results have been included, and it does not appear as though bedrock core samples were even collected. This information is the primary component for any foundation report and without these essential details it is not possible to adequately assess the foundation recommendations. Also, MTC requires accurate descriptions of subsurface materials for the contractor's information.

2) Many of the recommendations in the Trow Ltd. report are heavily qualified and often require their site inspection during construction. More positive foundation recommendations are required if the structural design is to be finalized.

3) Complete recommendations for sliding resistance, and earth pressure parameters for design of retaining walls using earth backfill (eg. Sheet No. 205 of the Contract Drawings) are not included in the Trow Limited report. These omissions were discussed with representatives of the structural and geotechnical consultants at our meeting of Oct. 23/86. Subsequently, Trow Ltd. submitted a letter to the City of Brampton, dated Oct. 24/86, providing the following angles of friction for calculation of sliding resistance.

- stiff to hard clayey silt till $\phi = 20°$
- dense to very dense silt till $\phi = 40°$
- rough shale $\phi = 15°$

No parameters for calculation of earth pressures for earth backfill were submitted.

Based on the available subsurface information our recommendations for calculation of sliding resistance are as follows:

$$A'_{c_f} + V\tan\phi_f > H$$

where:
A' = effective bearing area
V = sum of factored vertical loadings
H = sum of factored horizontal loadings
c_f = factored cohesion ($c_f = F_c c$ where $F_c = 0.5$)
ϕ_f = modified angle of internal friction ($\tan\phi_f = F_\phi \tan\phi$ where $F_\phi = 0.8$)

- stiff to hard clayey sill till, c = 240 kPa
  (This value is suggested on page 6 of the Trow Ltd. report).

- dense to very dense silt till, $\phi = 30°$
  (In our meeting of Nov. 27/86, Dr. Peeker of Trow Ltd. agreed that ϕ for the silt till would be less than ϕ for granular backfill. On page 12 of their report, granular backfill has been assigned a ϕ of 30°. However, this value could possibly be increased to 35° if the material properties such as Atterberg Limits and representative grain size distribution were available).

- shale and zone 1 metre above shale, $\phi = 20°$
  (Upon review of the Trow Ltd. report it was determined that there is insufficient data in the report to describe the quality or degree of weathering of the bedrock. Hence, a conservative ϕ value has been recommended.)

Regarding design of walls with earth backfill, it is recommended that the earth pressure parameters should reflect the properties of the earth material. Based on the limited information in the Trow Ltd. report the following parameters are recommended.

	c (kPa)	ϕ (o)	γ (kN/m^3)
- stiff to hard clayey silt till	240	0	20.5
- dense to very dense silt till	0	30	22.0

If there are any questions, please contact this office.

D.H. Dundas

D.H. Dundas, P. Eng.
Sr. Foundations Engineer

for

M. Devata, P. Eng.
Chief Foundations Engineer
East

DHD/MD/mmj

PLANMAC CONSULTANTS LIMITED
CONSULTING ENGINEERS & PLANNERS
3055 Lakeshore Blvd. West,
Toronto, Ontario M8V 1K6
Tel. (416) 259-5468

December 3, 1986

Ministry of Transportation and Communications,
Structural Office,
3501 Dufferin St., 4th Floor
Downsview, Ontario
M3K 1N6

Attention: Mr. A.J. Lee, P.Eng.

Re: Retaining Walls
Centre Street Grade Separation
City of Brampton
Our E.O. 8316

Dear Mr. Lee:

This will acknowledge receipt of your letter of November 20, 1986.

At our meeting on November 27, 1986, the above matter was discussed in further detail and our record of items discussed is attached. We understand from our meeting that the representatives of your Foundation Design Section were agreeable to the following angles of friction for this project: stiff to hard, clayey silt till: $\emptyset = 20^{o}$; shale $\emptyset = 20^{o}$. There was no agreement reached at the meeting (between K. Peaker of Trow and representatives of M.T.C. Foundation Design Section) regarding the angle of friction to use for the "dense to very dense silt till." However, K. Peaker did note that, if the footings were located within 1.0 - 1.2m of the shale interface, the angle of friction for shale (20^{o}) should be used.

In view of the above, and considering the need to expedite the resolution of this matter, we propose to check the design of the retaining walls using a value of 20^{o} for the three soil classifications as noted above. As discussed at the meeting, your Foundation Design Section was to review this matter further and report to you accordingly. We would appreciate it therefore if you would advise us if your Foundation Design Section is recommending any values for the angle of friction different from the above.

Yours sincerely,

PLANMAC CONSULTANTS LIMITED

D.C. Batchelor, P.Eng.

/nb
cc: Mr. M.Devata, Foundation Design Section
Mr. D. Dundas, Foundation Design Section
Mr. A.A. Witecki, Municipal Engineer, Approvals Section
Dr. K. Peaker, Trow Limited
Mr. M. Lostracco, City of Brampton

PLANMAC CONSULTANTS LIMITED
CONSULTING ENGINEERS & PLANNERS

Minutes of Meeting
E.O. 8316
November 27, 1986

ITEM	DESCRIPTION	ACTION REQUIRED BY

1.0 GENERAL

This meeting was requested by Planmac Consultants, as a follow-up to an earlier meeting and correspondence, to resolve the issue of soil friction values to be used in the design of the retaining walls. The following is a record of the items discussed.

2.0 SOIL PARAMETERS RELATING TO DESIGN OF RETAINING WALLS

2.1 Dr. Peaker reaffirmed the values of the soil parameters given in the soils report prepared by Trow Limited and in subsequent correspondence, and commented that the use of angles of sliding resistance different to those recommended by Trow are at the discretion of the M.T.C. and Planmac. For shale, Dr. Peaker advised that an angle of friction of 20^{0} would be acceptable. For the dense to very dense silt till (including materials below the water table), an angle of 40 degrees was recommended by Trow. The representatives from M.T.C. Foundation Design Section indicated that this latter figure was too high.

M.T.C. Foundation Design Section will review Trow's recommendations and will report back to M.T.C. Structural Section who will then review Planmac's design based on the Foundation Design Section's comments. M.T.C.

2.2 Dr. Peaker stated that the use of the soil parameter for the dense to very dense silt-till pre-supposes a cover of 1.0 m - 1.2 m between this material and the underlying shale bedrock; otherwise the values of the shale must be used.

2.3 The effect of the water table must be included in earth pressure calculations for the design of the earth retaining structures. For excavations back filled with granular material, the elevation of the water table is to be taken at the bottom of the free draining material. M.T.C. representatives (Foundation Design Section) expressed some reservations regarding the use of "Miradrain" for the release of water pressure behind the cemetary wall

ITEM	DESCRIPTION	ACTION REQUIRED BY
2.3	(ie., where shoring will be used). Dr. Peaker commented that is is used extensively in shored excavations on building projects and that he was satisified that it would perform satisfactorily. This matter will be reviewed further by M.T.C.	M.T.C.
2.4	For increased sliding resistance of the walls, the key in the footing will be extended as required. D. Batchelor noted that it was desirable, at this stage, not to change the the concept of the design (e.g. by using rock anchors).	
3.0	Geotechnical Review of Shoring Design for Railway Diversion	
3.1	Dr. Peaker advised that the depth of the soldier piles should be at least 1.5 m. The top 600mm of soil must not be used in the calculation of the passive resistance (ie. affected by frost).	
3.2	Dr. Peaker advised that the soil resistance for raker footing and soldier piles is to be calculated by usingthe more conservative of one of either the shear strength (Cu = 240 kPa) or the angle of sliding resistance (ϕ). For shale Cu = 0 kPa.	

ISSUED BY:

J. Schaffner, P.Eng.

DATE:

November 30, 1986

Structural Office
4th Floor
3501 Dufferin St.
Downsview, Ontario
M3K 1N6

Telephone: 248-3769

November 20, 1986

Planmac Consultants Ltd.
Consulting Engineers and Planners
3055 Lakeshore Blvd. West
Toronto, Ontario
M8V 1K6

Attn: Mr. D.C. Batchelor, P. Eng.

Re: Retaining Walls
Centre Street Grade Separation
City of Brampton

Dear Sir:

This is to acknowledge the receipt of your letter dated November 11th, 1986, with the attached copy of a letter from Trow Ltd.

The contents of the letter from Trow Ltd. dated October 24, 1986 have been reviewed by our Foundation Design Section. The values for angles of friction given in the letter from Trow Ltd. are, according to Mr. M. Devata, Chief Foundations Engineer, not acceptable.

Please take further action to have this matter resolved.

Yours truly

AJL/AAW/kd

A. J. Lee

for: A. A. Witecki
Municipal Engineer
Approvals Section

cc. Mr. M. Devata, Foundation Design Section
Mr. D. Dundas, Foundation Design Section ✓

PLANMAC CONSULTANTS LIMITED
CONSULTING ENGINEERS & PLANNERS
3055 Lakeshore Blvd. West,
Toronto, Ontario M8V 1K6
Tel. (416) 259-5468

RECEIVED
NOV 18 1986
Foundation Design Section

RECEIVED
1986
Foundation Section

November 11th, 1986.

Ministry of Transportation and Communications
Structural Office
4th Floor
3501 Dufferin Street
Downsview, Ontario
M3K 1N6

Attention: Mr. A. Lee, P.Eng.

Re: Centre Street Grade Separation
City of Brampton
Our E.O. 8316

Dear Sir:

At our meeting of October 23rd, 1986, you outlined your concern in regard to the sliding resistance of the retaining walls and shear capacity of the retaining wall footings. The acceptability of the geotechnical factors relating to the sliding resistance as given in the report by Trow Ltd. was also questioned. Subsequent to this meeting, we have had several discussions with Trow Ltd. and have also investigated the sliding resistance and shear capacity of the footing.

Included is a copy of a letter that we received from Trow Ltd., which recommends an angle of friction of 15° at the shale/concrete interface. After further discussion with Trow Ltd., this figure was revised to 20° and it is this latter figure that we used in our calculations. The angle of friction between concrete and till was taken as 40°.

Our findings are as follows:

1) Sliding Resistance:

From the borehole data we have concluded that all the retaining walls are founded above the shale bedrock. However, in investigating a 'worst-case' scenario, we have considered a typical wall panel in the north-west retaining wall as being founded either on the shale or on the dense silt till. Wall panel NW3 was considered for the sliding check. The soldier pile and raker footings required for the temporary excavation support will be cast into the final retaining wall footing and a positive attachment will be made with shear connectors and reinforcing steel.

PLANMAC CONSULTANTS LIMITED
CONSULTING ENGINEERS & PLANNERS

1) Sliding Resistance: (cont'd)

The sliding force against the wall panel was found to be 414 kN/m. For the case of the footing founded on the shale bedrock, the resisting force considered is provided by frictional resistance and passive resistance in front of the two raker footings, soldier pile footings and key. The passive resistance to the soldier-piles includes the surcharge effect of the cover to the underside of toe. By increasing the penetration of the soldier-pile encasement to 2.0m below the footing, the resisting force was calculated as 430 kN/m.

For the case of the footing founded on the dense till, the resistance force provided by frictional resistance without the additional benefit of the soldier piles was found adequate to counterbalance the sliding force.

For the remainder of the retaining walls, sliding due to the above 'worst-case' of footing founded on shale bedrock can be prevented by relocating the footing relative to the stem i.e. increasing the heel dimension. These walls are founded above the shale and have been found adequate to resist sliding, based on a angle of friction of 40^{o}.

2) Shear Resistance:

Shear stresses in the heel of the footing have been re-evaluated. The critical shear plane was located flush with the stem. The shear capacity of the first three panels (those closet to the bridge abutments) of all retaining walls was exceeded and hence shear reinforcement will be provided.

As discussed, we are submitting the letter from Trow Ltd. for review by your Foundation Design Section. If the revised values as recommended by Trow Ltd. are acceptable, we propose to revise our drawings to incorporate the changes as outlined above.

We would appreciate it if you would review this data at your earliest convenience. If you have any questions or require further information please contact us. We look forward to hearing from you soon in this matter and remain.

Yours sincerely,

PLANMAC CONSULTANTS LIMITED

J. Schaffer

D.C. Batchelor, P.Eng.
/bg
Encls.
cc: D. Dundas, Foundation Design Section

Trow Ltd
Consulting Engineers

Head Office: 1595 Clark Boulevard
Brampton, Ontario, Canada
L6T 4V1
Telephone: (416) 793-9800
Telex: 06-97802

Project: T 5955-G

October 24, 1986

City of Brampton
c/o Planmac Consultants Limited
3055 Lakeshore Blvd., West
Toronto, Ontario
M8W 1K8

Attention: Mr. Dave Batchelor

Dear Sirs:

Sliding Resistance of Retaining Wall
Centre Street Grade Separation
Brampton, Ontario

The following angles of friction are considered applicable to the sub-soil and bedrock at this site:

stiff to hard, clayey silt till $\emptyset = 20°$
dense to very dense silt till $\emptyset = 40°$
rough shale $\emptyset = 15°$

If the above friction values are not adequate for the footing size, then consideration should be given to installing vertical rock anchors. These were discussed on page 10 of our report T 5955-G dated February 6, 1986.

The capacity of raker footings can be calculated from the expression given on page (iv) of the appendix in our previous report. Only the horizontal component of the capacity should be used.

The horizontal capacity of the soldier piles is equal to the value 'R' given in the equation for determining the pile penetration on page (iii) of the appendix in our previous report.

Should you have any questions on the above, please do not hesitate to contact this office.

Yours very truly,
TROW LTD

H. Lohse

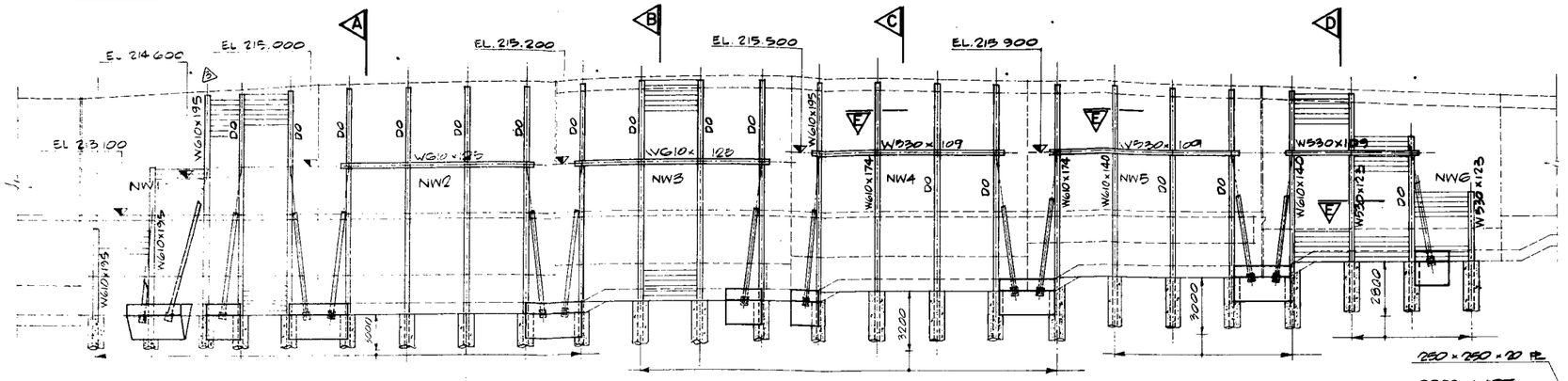
Holger Lohse, B.Sc.

Shaheen Ahmad

S.A. Ahmad, M.A.Sc., P.Eng.

HL:jw

Dist.: City of Brampton
c/o Planmac Consultants Limited (3)



ELEVATION
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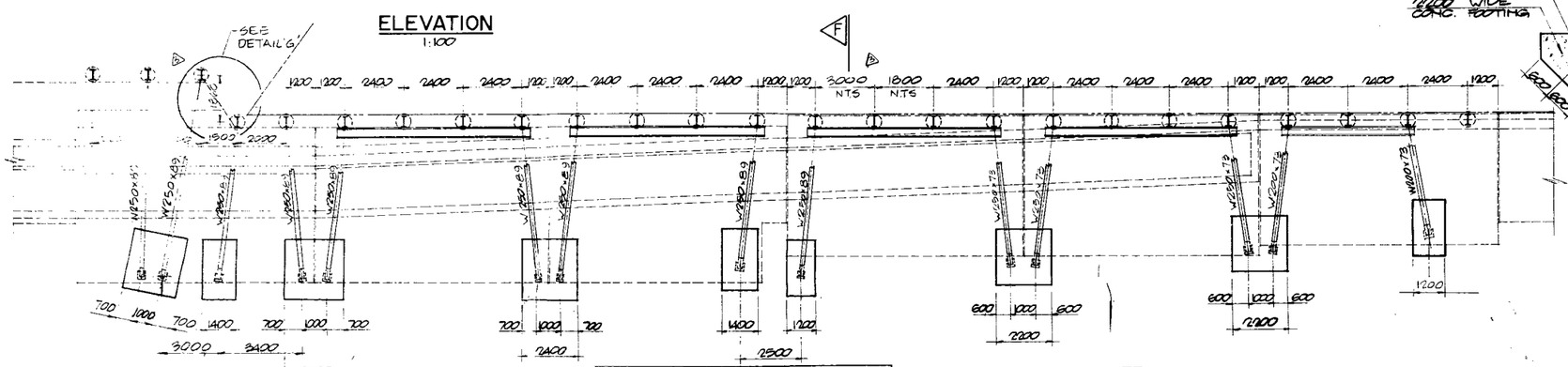

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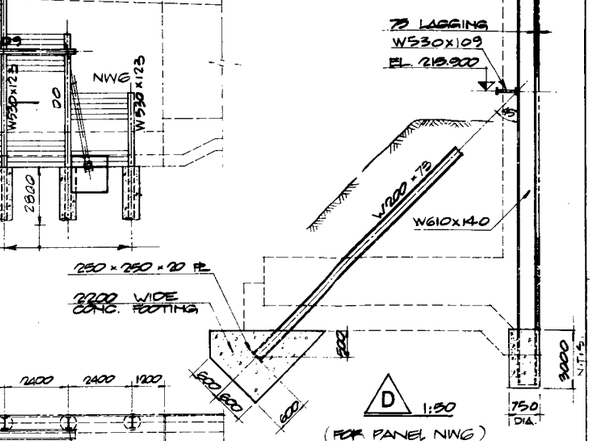
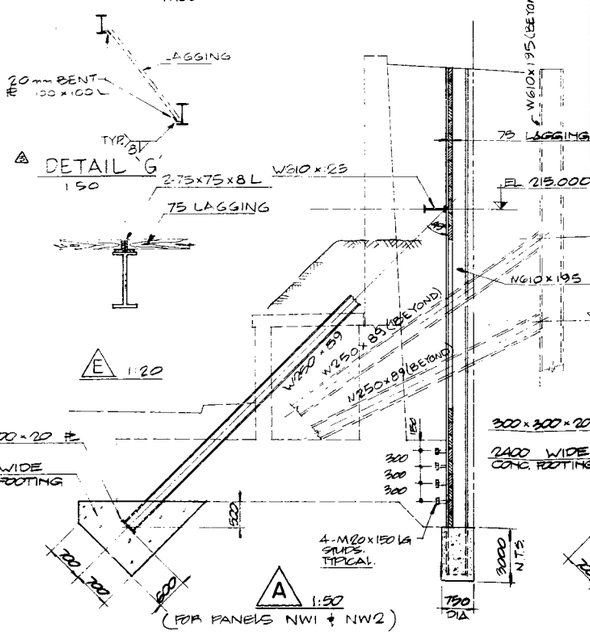
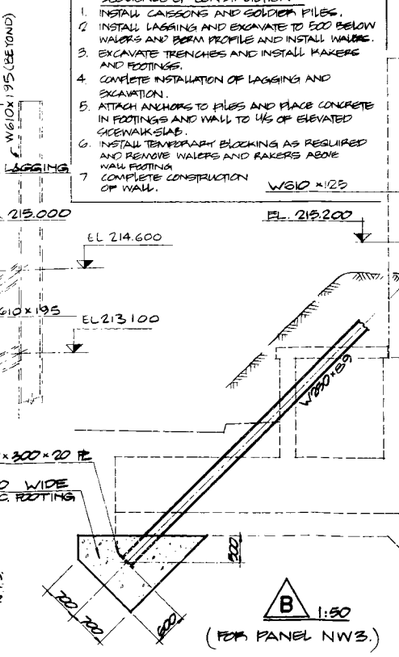
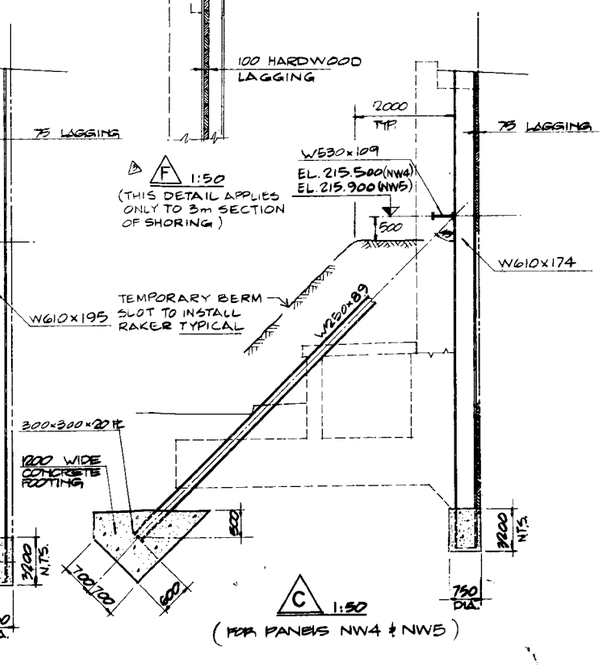


D 1:50
(FOR PANEL NW6)



SEQUENCE OF CONSTRUCTION
1. INSTALL CAISSONS AND SOLDIER PILES.
2. INSTALL LAGGING AND EXCAVATE TO 500 BELOW WALERS AND BERM PROFILE AND INSTALL WALERS.
3. EXCAVATE TRENCHES AND INSTALL RAKERS AND FOOTINGS.
4. COMPLETE INSTALLATION OF LAGGING AND EXCAVATION.
5. ATTACH ANCHORS TO PILES AND PLACE CONCRETE IN FOOTINGS AND WALL TO UG OF ELEVATED SIDEWALK SLAB.
6. INSTALL TEMPORARY BLOCKING AS REQUIRED AND REMOVE WALERS AND RAKERS ABOVE WALL FOOTING.
7. COMPLETE CONSTRUCTION OF WALL.





PLANMAC CONSULTANTS LTD.
CONSULTING ENGINEERS & PLANNERS
TORONTO CANADA

Ref. - Transmittal Letter
Dated Jan. 26/87

ISSUED
JAN 26 1987
PLANMAC CONSULTANTS LTD.
Consulting Engineers & Planners

No.	By	Date	Revision	Checked
5	PCD	87.01.26	REVISED SHORING PANEL NW1 & NW4 & W	
4	DCB	86.12.15	GENERAL REVISION	WF
3	DCB	86.11.26	ADDED SOUTH ABT.	AHM

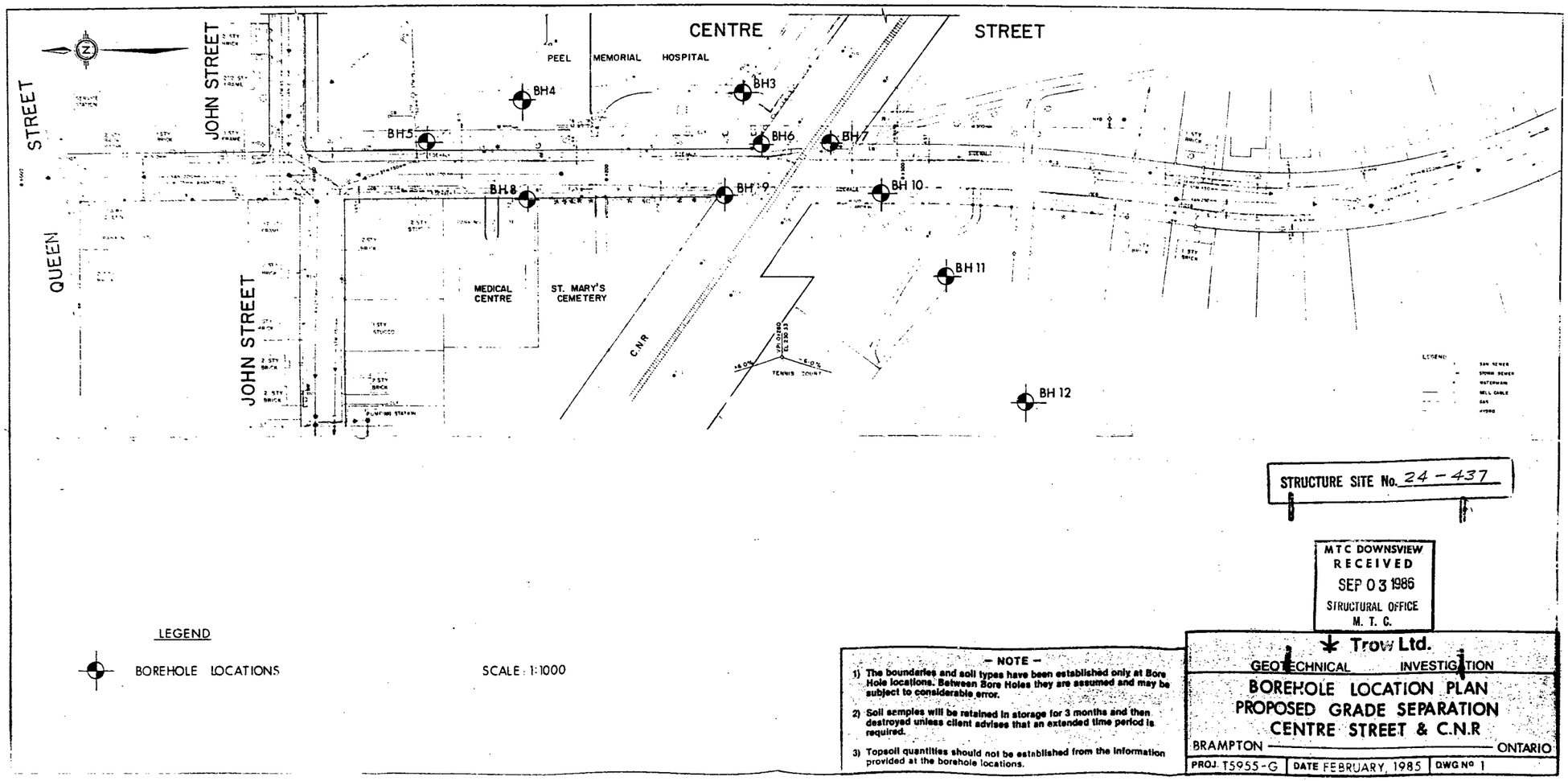
CITY OF BRAMPTON
PUBLIC WORKS DEPARTMENT
COMMISSIONER OF PUBLIC WORKS L.T. KOEHLE P.Eng.

CIVIL
R.C. BUECHLER

CENTRE STREET C.N. RAIL SUBWAY
MILEAGE 14.78 - HALTON SUBDIVISION

NORTH-WEST RETAINING WALL SHORING DETAILS

Surveyed By	Date	Contract No. 86-118	
Drawn By Y.B.	Checked By D.C.B.	Drawing No.	Sheet No.
Designed By D.C.B.	Checked By	G4-8-103	217
Scale: -	Date AUG. 1986		



STRUCTURE SITE No. 24 - 437

MTC DOWNSVIEW
RECEIVED
SEP 03 1986
STRUCTURAL OFFICE
M. T. C.

LEGEND

BOREHOLE LOCATIONS

SCALE : 1:1000

— NOTE —

1) The bounderies and soil types have been established only at Bore Hole locations. Between Bore Holes they are assumed and may be subject to considerable error.

2) Soil samples will be retained in storage for 3 months and then destroyed unless client advises that an extended time period is required.

3) Topsoil quantities should not be established from the information provided at the borehole locations.

Trow Ltd.

GEOTECHNICAL INVESTIGATION

BOREHOLE LOCATION PLAN
PROPOSED GRADE SEPARATION
CENTRE STREET & C.N.R

BRAMPTON — ONTARIO

PROJ. T5955-G	DATE FEBRUARY, 1985	DWG No 1