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STR. SITE No. 37-1019

HWY. No. 400

LOCATION Hwy 400/ Barton Rd. at
Sheppard

No of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

CONT 93-23



WP 131-85-04

30M11-188

FOUNDATION INVESTIGATION
PROPOSED SINGLE SPAN STRUCTURE
BARTOR ROAD AND SHEPPARD AVENUE WEST
NORTH YORK, ONTARIO

STRUCTURE SITE No. 37-1019

Prepared for:
BOROUGH OF NORTH YORK

D.T.C. — TORONTO
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JUN - 3 1974

STRUCTURAL
OFFICE

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December 17th, 1973

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FOUNDATION INVESTIGATION
PROPOSED SINGLE SPAN STRUCTURE
BARTOR ROAD AND SHEPPARD AVENUE WEST
NORTH YORK, ONTARIO

SUMMARY

It is proposed to construct a single span underpass structure at the crossing of Sheppard Avenue West and the realigned Bartor Road. In connection with this project a total of 12 boreholes were put down to assess the subsoil and groundwater conditions at the site. The subsoil at the site consists of a 2 feet to 7 feet of fill with thin topsoil veneer over a predominant stratum of silty clay varying in thickness from 20 feet to 26 feet. The silty clay is followed by a glacial till deposit.

Foundations for the structure can be supported either on spread footings using bearing pressures ranging from 5,000 psf to 2,000 psf at elevations noted in the report. Alternatively, the structure can be supported on end-bearing piles driven to practical refusal within the very dense silt till. Pile capacity of 55 tons per pile can be used for a 12 inch steel pipe pile.

Comments regarding settlements, design considerations, stability and construction of Bartor Road and widening of Sheppard Avenue West are also presented in this report.



INTRODUCTION

This office was requested to carry out a subsurface investigation at the junction of Sheppard Avenue West and the realigned Bartor Road in the Borough of North York, York County. The request was contained in a Borough of North York work order (No. W 20846) dated August 29th, 1973. Subsequently, an investigation was carried out by this office to determine the subsoil and groundwater conditions at the site.

The results of the investigation are presented in this report together with our recommendations for the design of the structure foundations as well as the stability and settlement considerations associated with the approach fill widening.

SITE AND GEOLOGY

The site is located some 300 feet west of Highway No. 400 at Sheppard Avenue in the Borough of North York. The terrain in the immediate vicinity is flat to gently undulating. The relatively flat nature of the terrain is interrupted by the approach fill (Sheppard Avenue) for the structure over Highway 400. The height of the fill in the proximity of the proposed structure is of the order of 18 feet. The surrounding areas have been developed for commercial and residential purposes. A large golf course exists south-east of the proposed site.

The site is located in a physiographic region known as the "South Slope"*. The predominant material in this region is a glacial till laid down during the Wisconsin Ice Age. Mixed with the till there are some areas which contain glaciofluvial deposits such as clays and silts. This is the case at this particular location. Some glacio-lacustrine deposits such as silts, sands and gravels are found north

*Chapman, L.J. and Putnam, D.F., "Physiography of Southern Ontario", University of Toronto Press 1967.



and south of the proposed site. Bedrock at the site consists of a grey fossiliferous shale belonging to the upper Ordovician period. The bedrock level at this site, as estimated from Department of Mines Map (No. 1955-7), varies from elevation 385 feet to elevation 405 feet.

FIELDWORK AND LABORATORY WORK

749-9633

Twelve sampled boreholes were placed at this site during the period of the field investigation. The boreholes, which are located as shown on Dwg. 1, were advanced by means of a continuous flight auger machine which is adapted for soil sampling purposes.

Disturbed samples were obtained at required depths in a 2-inch O.D. split-spoon barrel which was hammered into the ground with a driving energy of 350 ft.lbs./blow. Relatively undisturbed samples were obtained in the cohesive portions of the overburden by pushing a 2-inch I.D. thin-wall Shelby tube into the ground. Wherever possible, field vane tests were conducted to determine the undrained shear strength of the soil.

Surveying at the site was conducted by the personnel from William Trow Associates Limited, Toronto office. All elevations noted in this report are referenced to a geodetic datum.

All samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Following this examination, laboratory testing was carried out on selected representative samples to determine the following physical properties of the overburden.

- Natural moisture content;
- Natural bulk density;
- Undrained shear strength;
- Atterberg limits;
- Grain size distribution.



The results of these tests are plotted on the individual borehole logs and as well as on the accompanying drawings.

SUBSOIL CONDITIONS

The subsoil at the site consisted of 2 feet to 7 feet of fill consisting of a heterogeneous mixture of silt, sand and gravel with some clay and foreign inclusions such as pieces of wood, etc. Topsoil veneer was encountered above the fill. The fill is underlain by the predominant stratum of silty clay varying in thickness from 20 feet at borehole 4 to 26 feet at borehole 5. The portions of this silty clay have a desiccated crust with higher shear strengths (2200 psf to 5700 psf). Some till-like zones were encountered within this desiccated crust. Based on the undrained shear strength and the standard penetration resistance ('N' values) the crust has a consistency which ranges from very stiff to hard. The lower portions of the silty clay deposit have generally lower undrained shear strengths with a consistency ranging from stiff to very stiff.

The silty clay is followed by a glacial till. Although this deposit was not fully penetrated it was proven to extend in excess of 26 feet. The upper 7 feet to 8 feet of this glacial till appear to be cohesive and of a relatively low shear strength. The lower portions of the glacial till are a very dense, tightly packed silt till.

This sequence of subsoil stratigraphy appears to be in general agreement with those encountered at a nearby site (Sheppard Avenue West and the Canadian Pacific Railways). The results of this investigation are presented in our report J 2576, submitted October 1965.



GROUNDWATER CONDITIONS

Groundwater level observations were carried out, during the period of field investigation, by recording the water levels in the open boreholes and by using piezometers. The depths of piezometer installations together with the piezometric head are presented on the self-explanatory individual borehole logs. These observations indicate that the groundwater level ranges from elevation 472 feet to elevation 468 feet corresponding to depths below ground surface of from 12 feet to 13.5 feet. Water levels almost to the surface were noted in boreholes 4 and 5. It is believed that these high values are influenced by the recent precipitation.

Groundwater level at the neighbouring site (Sheppard Avenue West and C.P.R.) was found to be at elevation 467 (our report J 2576). Since the original ground surface at this site is some 8 feet to 13 feet lower than the proposed site, the groundwater level measured appears to agree with the previous records.

FOUNDATIONS

It is proposed to construct an underpass structure at the junction of Sheppard Avenue and the realigned Bartor Road. The proposed structure will be a single span structure with a clear span of 70 feet. Sheppard Avenue, in the vicinity of the proposed structure, is to be widened to a top width of 60 feet. The alignment of Sheppard Avenue will be shifted 2 feet to the south. The top of pavement of the proposed Bartor Road in the vicinity of the structure will be at approximate elevation 480 feet, i.e. some 2 feet to 4 feet below existing ground surface.

The proposed structure can be supported on conventional spread footings in the upper crust or founded on piles driven to



practical refusal within the very dense silt till. Because of the problems associated with determining the refusal depths (as was the case at the adjacent R.R.-crossing) the spread footing design is considered the most desirable. The two foundation schemes are discussed in detail in the following paragraphs.

1. Spread Footings

The presence of a desiccated crust in the upper portions makes it possible for the structure to be supported on conventional spread footings using a closed-type abutment scheme and safe net bearing pressures as outlined in a tabular form.

Assuming a width of 10 feet to 12 feet for the abutment footing, the following values may be used.

<u>Elevation</u>	<u>Safe Net Bearing Pressure</u>
above 478 feet	5,000 psf 240 kPa.
478 to 475 feet	3,000 psf 140 kPa.
475 to 470 feet	2,000 psf 95 kPa.

Allow 1/2"

The foregoing table indicates that safe net bearing pressures ranging from 5,000 psf to 2,000 psf are available for design purposes. In any event, minimum earth cover of 4 feet should be provided to the underside of the footings to satisfy frost protection requirements.

No major groundwater problems are anticipated, partially because of the low permeability of the subsoil and partly because the prevailing groundwater will be at or below the probable footing foundation level. Any minor seepage or surface run-off into the footing excavations can be handled readily by standard pumping techniques.



In all cases, the footings must penetrate through the fill and be founded on competent parent subsoil.

The compressible subsoil will undergo settlement because of loading induced by the footing. The settlement will be partly elastic and partly of a consolidation nature. The elastic settlement will be realized during or shortly after the construction period. The consolidation settlement will be realized some time over the future (say two years). The magnitude of the total settlement (elastic and consolidation) will be approximately 1 inch to 2 1/2 inches, approximately one half of which will be realized immediately after the application of the load.

2. End-Bearing Piles

Alternatively, the abutments may be perched within the approach fills and supported on end-bearing piles driven to practical refusal within the very dense silt till. The design load will depend on the pile section selected, but a typical value is 55 tons to 65 tons on a 12 inch pipe pile concrete filled. The piles should be driven to a final set of 12 blows/inch under 24 ft. kips energy.

A minimum earth cover of 4 feet should be provided to the underside of the pile cap for frost protection purposes. At the adjacent railway underpass, the piles penetrated beyond the predicted length, hence at this site it can be assumed that similar conditions will exist and pile lengths will be in the region of 45 feet to 55 feet in length.

DESIGN CONSIDERATIONS

In computing the sliding resistance between the base of the abutment footing and foundation subsoil, the following factors should be used:



Rough concrete and granular deposit - coefficient
of friction = 0.7

Rough concrete and clayey deposits - adhesion 2,500 psf.

A factor of safety of 2 should be applied when using these values.

If the structure is designed as a rigid frame, then a coefficient of earth pressure at rest (K_o) of 0.5 should be used for the granular fill material placed behind the wall when designing the wall section. However if some movement of the top of the wall is permitted, then a coefficient of active earth pressure (K_a) of 0.33 can be used. Coefficient of passive earth pressure (K_p) of 3.0 can be used for material in front of the abutment.

In order to relieve the build-up of any excess hydrostatic pressure behind the wall, suitable drainage measures should be provided. Backfill behind the wall should be carried out in accordance with current M.T.C. practices, specifically Standard No. SD 4-58.

CONSTRUCTION OF BARTOR ROAD AND WIDENING OF SHEPPARD AVENUE WEST

It is proposed to realign Bartor Road as shown on Dwg. 1. This realignment will necessitate maximum cuts in the order of 4 feet. In certain portions of the alignment, fills up to 18 inches will be required. The remainder of the roadway will require general regrading.

In areas where general regrading is required, the topsoil should be removed to its full depth and the ground should be thoroughly proof-rolled. Ditches should be provided to facilitate drainage along the route.



In areas where cutting operations will be required, no long term stability problems are anticipated for maximum cuts of 4 feet provided the side slopes are not steeper than 2:1.

In areas where fills are to be placed, all topsoil should be excavated and removed. No major problems are anticipated for construction of fill. The fill should be placed in shallow layers of 6 inches to 8 inches and compacted to 98 per cent standard Proctor density.

It is also proposed to widen Sheppard Avenue to a top width of 60 feet. No stability problems are anticipated for the proposed widening of the approach embankments. The settlements, induced in the foundation subsoil, will be of the order of 1 inch to 2 inches, the majority of which will be realized in a short period of time (say two to three months). It is, therefore, suggested that paving operations be delayed for a period of 3 months after the placement of fill. This will ensure that the maintenance problems with regard to pavement cracking will be reduced.

In order to have a smooth transition from the existing to the new fill sections, it is recommended that all topsoil be stripped from the existing fill and the future fill be "keyed" into the existing approaches.

Pavement sections should be as per Borough of North York requirements.

WILLIAM TROW ASSOCIATES LIMITED

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DRAWING No. 5

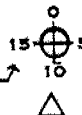
PROJECT PROPOSED STRUCTURE
LOCATION SHEPPARD AVE. AND BARTOR
NORTH YORK, ONTARIO

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
2" DIA. CONE 
PUSHED _____
VANE TEST AND SENSITIVITY (S) +5

NATURAL MOISTURE
PLASTIC AND LIQUID LIMIT

UNDRAINED TRIAXIAL AT
OVERBURDEN PRESSURE
% STRAIN AT FAILURE -
LABORATORY PENETROMETER

HOLE LOCATION AND DATUM SEE DRAWING No. 1



L.S.G.	SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FT.	PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT			NATURAL UNIT WEIGHT P.C.F.
					350 FT. LB. 20	40	BLOWS/FT. 60	80	10	
	F	FILL-sand till, gravel sizes, mixed with clay and pieces of wood	483.9		SHEAR STRENGTH		K.S.F.			
					2	4	6	8		




BOREHOLE LOG


JOB No. J7637


BOREHOLE No. 5

DRAWING No. 6

PROJECT PROPOSED STRUCTURE
LOCATION SHEPPARD AVE. AND BARTOR
NORTH YORK, ONTARIO





2" O.D. SPLIT TUBE 

2" I.D. SHELBY TUBE 

2" DIA. CONE 

PUSHED _____

VANE TEST AND SENSITIVITY (S) _____

NATURAL MOISTURE X
PLASTIC AND LIQUID LIMIT 
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
% STRAIN AT FAILURE 
LABORATORY PENETROMETER 



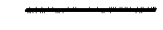

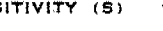
HOLE LOCATION AND DATUM SEE DRAWING No. 1






SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FT.	PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS		NATURAL UNIT WEIGHT P.C.F.
				350 FT. LB. 20	60 40 60 80	% DRY WEIGHT	10 20 30	
	FILL-brown, clayey silt with some sand and gravel sizes, pieces of wood	483.9						
	SILTY CLAY-very stiff to hard, brown, to grey, some fine sand seams, occasional trace of gravel, has a till-like appearance in the upper 8 feet, some oxidised zones and highly plastic fat clay pockets, medium plasticity, some layering at 10 feet, highly plastic between 15 to 20 feet., moist	481						135
								132
			10					126
								134
			20					130
	CLAYEY SILT TILL - hard, cohesive, some sand and gravel sizes, grey, wet, some sand seams	457						121
								122
			30					143
	SILT TILL - very dense, grey, numerous sand and gravel sizes, moist	451						
	END OF BOREHOLE	437.4						
NOTES: 1) Boring advanced by means of a continuous flight auger machine, hole uncased to full depth on November 26, 1973 2) Water level records: Time W.L. at Hole Open (ft.) to (ft.) On completion wet cave 46 Piezometer (P ₁) installed at 44 Piezometer (P ₂) installed at 10 P ₁ P ₂ After 2 hours 33.7 dry (8.8) After 1 day 19.9 3.5 After 3 days 12.6 0.5 After 4 days 11.9 0.5								

BOREHOLE LOG

JOB No. J 7637BOREHOLE No. 6DRAWING No. 7

PROJECT PROPOSED STRUCTURE
 LOCATION SHEPPARD AVE. AND BARTOR
NORTH YORK, ONTARIO

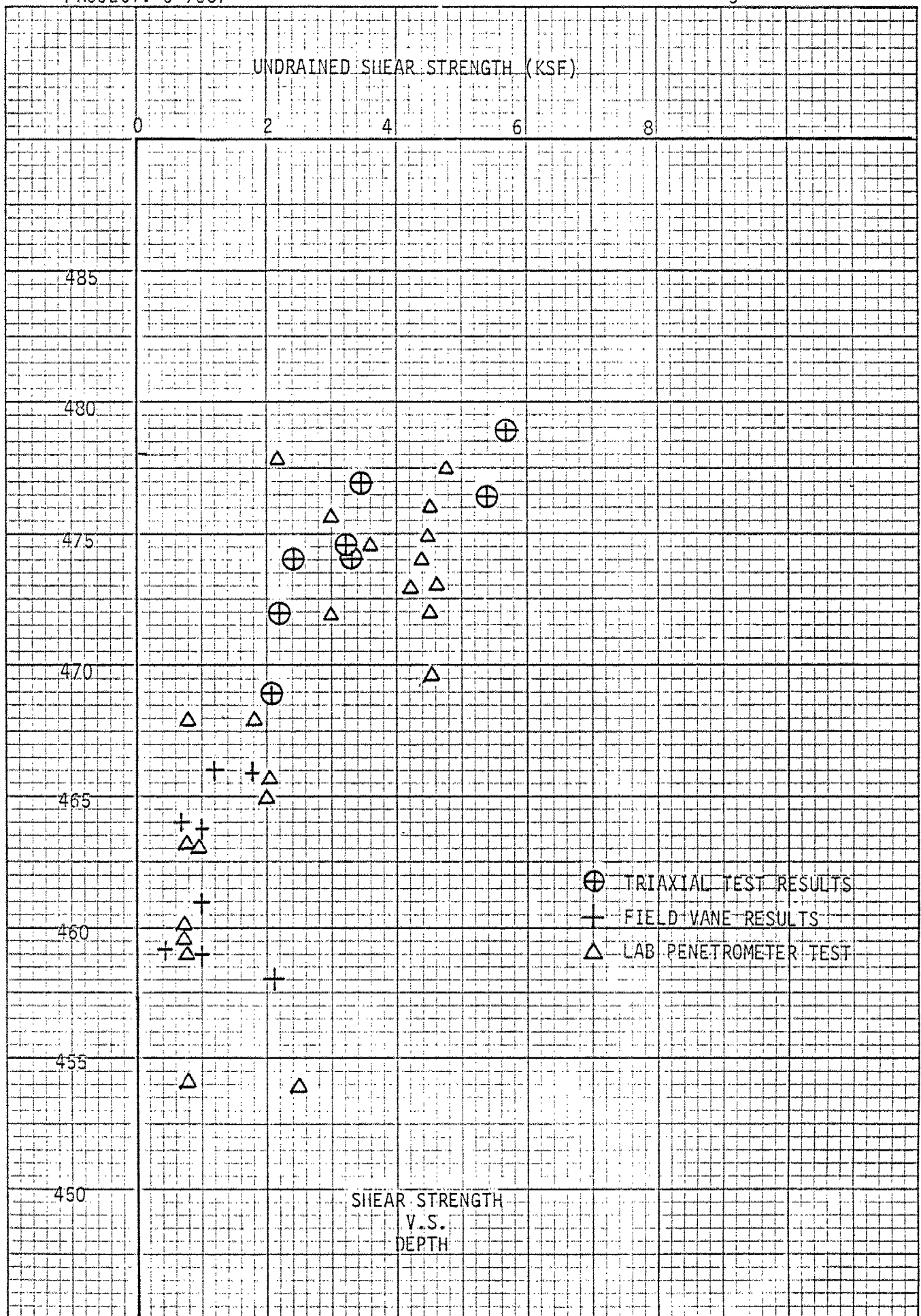
2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 2" DIA. CONE 
 PUSHED 
 VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE 
 PLASTIC AND LIQUID LIMIT 
 UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
 % STRAIN AT FAILURE 
 LABORATORY PENETROMETER 

HOLE LOCATION AND DATUM SEE DRAWING No. 1

GR F.W.	SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FT.	PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. 20 40 60 80				NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT			NATURAL UNIT WEIGHT P.C.F.
					SHEAR STRENGTH K.S.F.				10	20	30	
		FILL-clay with sand and gravel	481.9									
		SILTY CLAY - firm to hard, brown to grey, some sand and gravel sizes, some silt till zones between 7 feet to 12 feet, some oxidized zones, moist	479									
				10								132
												139
												132
												131
				20								125
												136
		SILT TILL TO CLAYEY SILT TILL - slightly cohesive, grey, numerous sand and gravel sizes, moist	457									
				30								152
		SILT TILL - very dense, grey, numerous sand and gravel sizes, slightly cohesive in the lower portions, moist	449									
				40								
				50								
		END OF BOREHOLE	430.4									
		NOTES:										
		1) Boring advanced by means of a continuous flight auger machine, hole uncased to full depth on November 29, 1973										
		2) Water level records:										
		Time W.L. at Hole Open										
		(ft.) to (ft.)										
		On completion 18 18.9										
		Piezometer installed at 45.0										
		After 7 hours 21.2										
		After 1 day 13.1										

UNDRAINED SHEAR STRENGTH (KSE)



⊕ TRIAXIAL TEST RESULTS
+ FIELD VANE RESULTS
△ LAB PENETROMETER TEST

SHEAR STRENGTH
V.S.
DEPTH

APPROX EXIST GRADE SHEPPARD AVE

APPROX EXIST GRADE SHEPPARD AVE

5 0 0

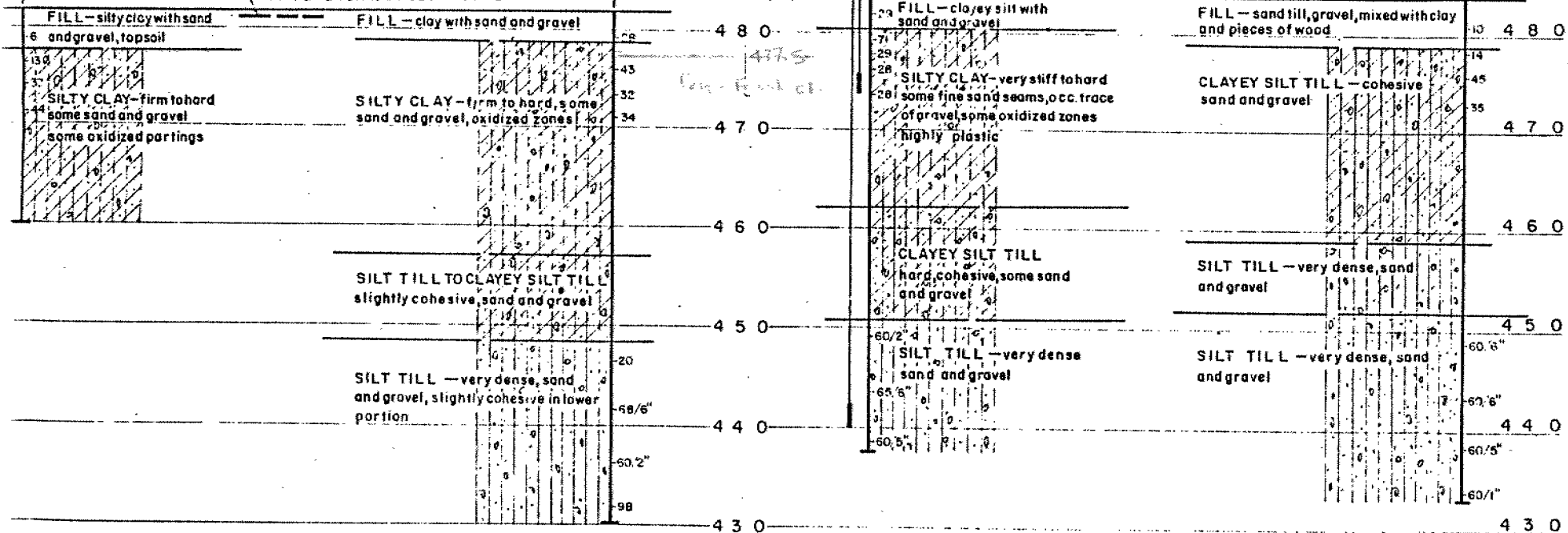
5 0 0

4 9 0

4 9 0

APPROX EXISTING GRADE BARTOR AVE

APPROX EXIST GRADE BARTOR AVE.



SECTION A - A

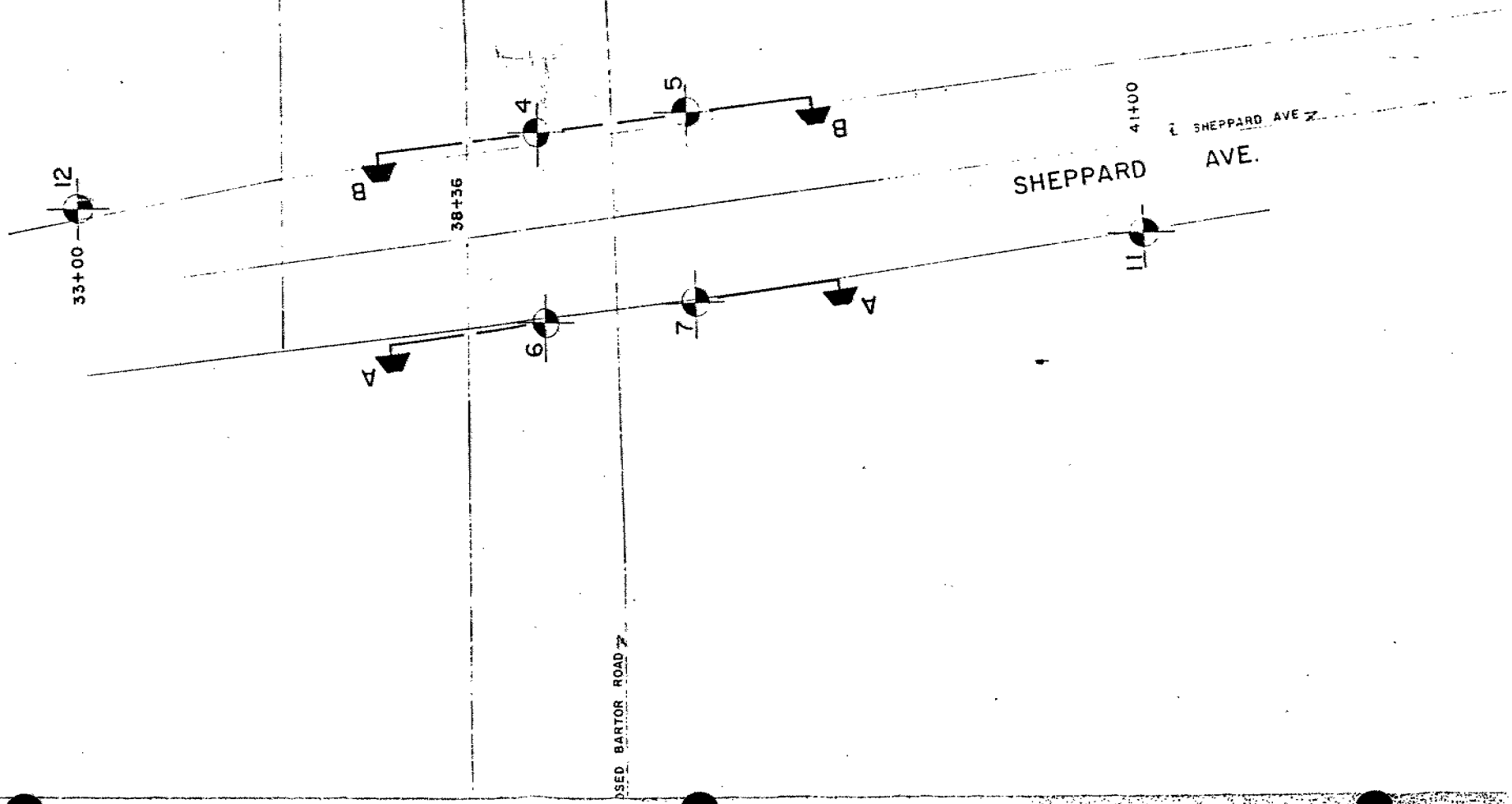
SECTION B - B

INTERPRETED

SUBSOIL

STRATIGRAPHY

HOR. 1 IN. = 10 FT.
 SCALE:
 VERT. 1 IN. = 10 FT.





Ministry
of
Transportation

FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

CONT 93-23

WP 131-85-04

DIST 6

HWY 400

STR SITE 37-1019

Sheppard Ave. and Bartor Rd. Underpass

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FOUNDATION INVESTIGATION REPORT
FOR
Sheppard Ave. and Bartor Rd. Underpass
W.P. 131-85-04, Site No. 37-1019
Hwy. 400, District 6, Toronto

INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the aforementioned site. It is proposed to widen the existing Sheppard Ave. underpass at Bartor Rd. with an extension immediately south and adjacent to the existing structure. This report is applicable to the proposed structure foundations, approaches and related earthworks.

SITE DESCRIPTION AND GEOLOGY

The site is located immediately adjacent and south of the existing Sheppard Ave. underpass at Bartor Rd., approximately 100 m west of the Sheppard Ave. underpass at Hwy. 400 in the City of North York. The existing structure is a rigid frame structure with reinforced concrete girders and counterfort abutments and wing walls. The structure presently carries four lanes of traffic and pedestrian sidewalks on Sheppard Ave. over the two lane Bartor Rd.

Land use in the area is multi-purpose consisting of commercial, residential, recreational and industrial developments. A large golf course exists south-east of the proposed site.

Physiographically, the site is located in the region known as the "Peel Plain". This region is characterized by a level-to-undulating "till or boulder clay" deposited by the glacier during the Wisconsinan period. Glaciofluvial and glaciolacustrine deposits are also founded randomly interbedded in the soil stratigraphy. Bedrock in the region consists of shale or limestone of the Georgian Bay Shale Formation of the Ordovician period.

FIELD INVESTIGATION

The field work was carried out between 89 01 16 and 89 01 24 and consisted of five sampled boreholes and two dynamic cone penetration tests. Continuous

flight hollow stem augering techniques were used to advance the boreholes through the overburden. Bedrock was proven at the proposed west abutment location utilizing conventional rock coring methods.

Subsoil samples were retrieved at selected depths by means of a split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586) and by a Shelby tube sampler. In situ vane tests were carried out wherever applicable, in order to determine the undrained shear strength of the cohesive material. The samples were identified in the field and then transported to the laboratory for further visual examination and testing on selected samples.

Water levels were obtained in the open boreholes and monitored until stabilized levels were achieved.

Survey information related to location and elevation of boreholes was provided by Central Region Surveys and Plans.

SUBSURFACE CONDITIONS

General

Subsoil conditions are generally uniform across the site. The native surficial layer consists of a soil that varies randomly from a clayey silt to silty clay that appears to be of glaciolacustrine origin. This stratum extends to a maximum thickness of 8.4 metres and consists of an upper dessicated crust underlain by a weaker, more compressible soil. Underlying this layer is a deposit of a heterogeneous mixture of clayey silt/silt, sand and gravel (glacial till) interbedded with a layer of silt with sand. This deposit was explored to a maximum thickness of 8.1 metres. The till deposit is underlain by shale bedrock.

Fill material consisting of an irregular mixture of sand, gravel and clayey silt compromises the approach material for the existing overpass structure and overlies the native surficial layer of clayey silt to silty clay at the site. Up to 6.4 metres of the fill material was identified at the selected sampling locations.

The boundaries between the various soil types, in situ and laboratory test results, as well as stabilized groundwater levels, are shown on the attached Record of Borehole Sheets provided in the Appendix. A plan of the site illustrating the locations and elevations of the boreholes and subsoil stratigraphical sections are provided on Dwg. 1318504-A.

Irregular Mixture of Clayey Silt, Sand and Gravel (Fill)

The approach material for the existing Sheppard Rd. overpass structure explored in the investigation consists of an irregular mixture of clayey silt, sand gravel (fill) that generally exhibits a cohesive behaviour. Figure 1 illustrates a typical grain size distribution for the material. The cohesive fill matrix that extends for a thickness of approximately 5.4 metres, underlies the granular roadway base that is approximately 1 metre in thickness.

The fill material is generally very stiff indicating an equivalent compact state of condition achieved during placement and subsequent loading.

Clayey Silt to Silty Clay

The native surficial deposit spread across the site consists of a soil that varies randomly in behaviour from a clayey silt to a silty clay. This layer ranges in thickness from 7.0 m to 8.4 m and can be subdivided into an upper dessicated, oxidized crust of thickness ranging from 2.7 m to 4.8 m overlying a grey saturated, weaker layer. Organic inclusions and occasional sand seams exist in the upper dessicated layer and "till-like" zones also exist randomly throughout the entire layer.

Atterberg Limits were obtained to evaluate the behaviour of this surficial layer and the results are plotted in Figure 2. A summary of the indices is provided in Tables 1 & 2 below.

Table 1 - Clayey Silt

	<u>Range (%)</u>	<u>Average (%)</u>
Moisture Content (w)	13-18	15
Liquid Limit (w_L)	23-32	28
Plastic Limit (w_p)	14-18	15
Plasticity Index (I_p)	9-14	13

Table 2 - Silty Clay

	<u>Range (%)</u>	<u>Average (%)</u>
Moisture Content (w)	22-34	26
Liquid Limit (w_L)	37-50	42
Plastic Limit (w_p)	19-23	21
Plasticity Index (I_p)	18-27	21

The results reveal that the deposit varies from a clayey silt of low plasticity to silty clay of intermediate plasticity.

Grain size distribution envelopes for the material are plotted in Figure 3.

Undrained shear strengths and associated consistencies of the deposit were determined by interpretation of 'N' values of the Standard Penetration Test, in situ vane tests and laboratory unconfined compression tests. The results, plotted on the boreholes in the Appendix and summarized on Table 3 below reveal that a generally very stiff to hard dessicated upper crust extending in thickness ranging from 2.7 m to 4.8 m overlies a weaker layer of generally firm to stiff consistency of thickness ranging from 2.3 m to 4.3 m.

Table 3 - Undrained Shear Strength
(Cu) (kPa)

<u>Layer</u>	<u>'N' Value</u>		
	<u>Blows/0.3 m</u>	<u>Field Vane</u>	<u>Lab Test</u>
Upper dessicated crust	13-46	N/A	N/A
Lower unoxidized	4-9	40-70	40-50

The results of a consolidation test (e - log p curve) of a representative sample of the lower weaker layer is shown in Figure 4. The test indicates that this layer has been preconsolidated in the past to an effective pressure some 375 kPa in excess of the existing effective overburden pressure.

Clayey Silt/Silt, some sand, trace gravel (Glacial Till)

Underlying the native surficial clayey silt to silty clay stratum exists a till deposit of glacial origin that consists of a heterogeneous mixture of clayey silt to silt, some sand, trace gravel interbedded with a layer of silt with sand layers. The deposit was explored for a maximum thickness of 6.8 m and was founded to overly bedrock.

The main component of the till deposit varies randomly from a low plasticity silt [ML] to a low plasticity clayey silt [CL]. Figure 5 illustrates the Atterberg Limits attained from the testing of the behaviour of this main component.

Based on 'N' values obtained from the Standard Penetration Test, the deposit can be considered as very dense (for the non-cohesive zones) or hard (for the cohesive zones).

Within the deposit, a layer of silt with sand is present. This layer is generally found interbedded between the surficial clayey silt to silty clay and the clayey silt/silt till deposit and ranges in thickness from 1.5 to 5.5 m. The denseness of this layer as determined by 'N' values of the Standard Penetration Test varies from dense to very dense but is generally very dense.

Although not encountered during the field investigation, boulders and cobbles are a characteristic component of glacial till deposits and consequently may exist in the deposit.

Bedrock

The glacial till deposit is underlain by bedrock which was proven at the proposed west abutment extension location by obtaining 3.5 metres of BQ size rock core samples. The bedrock surface exists at elevation 132.5 m.

The bedrock is a grey shale that is generally moderately to highly weathered and interbedded with clay seams and limestone. Detailed descriptions of the bedrock are attached in Table 4 in the Appendix, entitled "Description of Rock Core".

Core recoveries and rock quality designations (RQD) were determined in situ to evaluate the competence and integrity of the rock. Recoveries ranged from 58 to 94% and all RQD's were 0 (zero) %. It can be concluded from these results and visual examination that the shale is weak to very weak rock.

Groundwater Conditions

Observation of the groundwater level was carried out by measuring the water level in the open boreholes. At the time of the field investigations, the stabilized groundwater elevations varied from 141.2 m to 142.3 m which is equivalent to depths ranging from 4.0 to 6.5 m below the natural ground surface. The stabilized groundwater level appears to coincide with the dessicated layer - weaker soil interface in the native surficial clayey silt to silty clay.

DISCUSSION AND RECOMMENDATIONS

It is proposed to extend the existing Bartor Road/Sheppard Avenue underpass structure to facilitate the widening of Sheppard Avenue. The existing rigid frame structure with a clear span of approximately 18 m was constructed with counterfort abutments and wing walls and is founded on shallow foundations within the upper dessicated crust of the clayey silt to silty clay deposit. Shear keys were constructed as an integral part of the footing to augment further the lateral resistance to horizontal loading.

The extension is located adjacent and immediately south of the existing structure and is approximately 16 m in width. Removal of the existing southern wing walls will be required removal to facilitate the construction of the extension. Approach fills in the order of magnitude of 7 metres will be required.

Recommendations pertaining to the following geotechnical considerations are provided.

- 1) Structure Foundations
- 2) Backfill to Structures
- 3) Approach Fills
- 4) Temporary Shoring

1) Structure Foundations

It is recommended that the structure foundations be selected from one of the following three (3) alternatives.

- 1) End-Bearing Steel H-Piles
- 2) Spread Footings
- 3) End-Bearing Concrete Caissons

The alternative that proves to be the most practical and cost-effective should be selected for design. Details of each alternative are provided below.

Alternative 1 - End-Bearing Steel H-Piles

The structure foundations can be supported on end-bearing steel H-piles driven into the clayey silt/silt till deposit or driven to the underlying bedrock. For a steel 'H' section pile equipped with standard reinforced flange tip plates, the following design parameters are recommended.

<u>Pile Type</u>	<u>Capacity at S.L.S. Type II (kN)</u>	<u>Factored Capacity at U.L.S. (kN)</u>	<u>Estimated Pile Tip (m)</u>
HP 310 x 79	890	1150	133
HP 310 x 110	1150	1600	133

Pile driving in the field should be controlled by employing the Hiley Dynamic Pile Driving Formula driven in accordance with MTO standards SS103-10 or SS103-11 assuming an ultimate capacity as follows:

<u>Pile Type</u>	<u>Ultimate Capacity (kN)</u>
HP310x79	2670
HP310x110	3450

Piles within 5 m of the existing structure foundation shall be installed in holes preaugered to 5 m to avoid disturbance of the foundation soil of the existing abutments.

The pile caps should be protected against frost protection by providing a minimum 1.2 m earth cover. In view of the impervious nature of the native surficial clayey silt to silty clay conventional pumping techniques will suffice in discharging any localized seepage.

Resistance to lateral load shall be computed in accordance with Section 6.8.3.8 of the O.H.B.D.C.

Alternative 2 - Spread Footings

The abutment foundations can be supported on conventional spread footings at or above elevation 144.9 m within the native surficial clayey silt to silty clay, which corresponds to the elevation of the existing abutment footings. The footing must have a minimum earth cover of 1.2 m to protect the footing base against the effects of frost penetration. For purposes of the O.H.B.D.C., and based on a footing width equivalent to the existing abutment footing, the following design values are recommended.

Factored Bearing Capacity at U.L.S. = 210 kPa

Bearing Capacity at S.L.S. Type II = 140 kPa

Total and differential settlements anticipated for the aforementioned specified parameters is 35 mm and consequently the superstructure should be designed to accommodate these movements. The design of the footing connection to the existing structure must also consider these anticipated settlements.

Sliding resistance between the concrete and the foundation soil should be calculated in accordance with Section 6.7.3.3.2 of the O.H.B.D.C. assuming an unfactored adhesion value of 75 kPa or an effective angle of friction of 26°. Sliding resistance can be supplemented by constructing shear keys in the founding soil below the base of the footing.

Any organic or softened material present at the footing founding elevation shall be subexcavated and replaced with a granular fill material or mass concrete. In addition, to protect the founding soil from the effects of weathering, it is recommended that a concrete working slab be placed on the soil within 4 hours of exposure. Footing excavation shall be carefully controlled and monitored to avoid potential undermining of the existing structure foundations.

In view of the cohesive nature of the native surficial clayey silt to silty clay and associated relatively low impermeability, conventional pumping techniques will suffice in discharging any localized seepage.

Alternative 3 - Concrete Caissons

The structure may be supported on reinforced concrete caissons founded in the clayey silt/silt till or on bedrock. For purposes of O.H.B.D.C., the following design values are recommended.

Caisson Diameter (m)	Pile Tip Elev. (m)	Capacity at S.L.S. Type II (kN)	Factored Capacity at U.L.S. (kN)
0.76	133	1500	2250

Resistance to lateral load shall be computed in accordance with Section 6.8.3.8 of the O.H.B.D.C.

In view of the elevation of the groundwater table and the fact that the clayey silt/silt till deposit with predominant interbedded silt with sand seams is submerged, the shaft of the preaugered holes must be protected against caving during the installation of the concrete caissons. One method of achieving this is by installing a steel liner and constructing the caisson within the steel liner. After the liner has been cleaned out and the required reinforcing installed, the concrete should be placed in the dry. Alternatively mud drilling and tremie techniques may be required. The method of installation shall be based on the discretion of the contractor in accordance with OPSS 903.07.03 and subject to review by this office.

2) Lateral Earth Pressure on Structure

Free draining material such as Granular 'A' or Granular 'B' is recommended as appropriate backfill to the abutments to prevent hydrostatic pressure build-up. Design parameters of the soil are given below:

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction (ϕ)	35°	30°
Unit Weight (kN/m^3)	22.8	21.2
Coefficient of Active Earth Pressure (K_a)	0.27	0.33
Coefficient of Earth Pressure at Rest (K_0)	0.43	0.5

The earth pressure coefficient at rest is to be used in design if the abutment walls are rigid and unyielding. Weep holes in the abutment walls should be designed to drain any accumulation of water in the backfill.

The backfill should be constructed in 300 mm lifts on alternating sides of the rigid frame structure so that the maximum differential in backfill heights at no time exceeds 300 mm. O.P.S.D. 803 series illustrates the applicable backfill standards and specifications.

3) Approach Fills

Stability

Stability computations were carried out to determine the overall stability of of the approach fills both in the longitudinal and transverse directions using

Bishop's total stress analysis and a minimum factor of safety of 1.3. The properties of the fill material and subsoil, as well as the surface geometry used in the computations are shown in Figures 6a and 6b. Based on the analyses it can be concluded that no stability problems are anticipated for fill heights up to 8 metres constructed with 2H:1V slopes.

Settlements

Settlement computations were carried out to determine the magnitude and time-rate elastic and consolidation settlement of the native surficial clayey silt to silty clay layer, specifically the stratum underlying the dessicated upper crust. In view of the preconsolidated nature of the subsoil, settlements in the order of magnitude of 50 mm is anticipated. Approximately 25 mm of settlement can be expected to be realized during or immediately following construction of the approach embankments. In addition, settlements in the order of magnitude of 50 mm can also be expected within the approach fills under its own weight for a cohesive material. Consequently it is recommended that the approach embankments be placed as far in advance of roadway asphaltic paving as scheduling and economics permit, to minimize the differential settlements that may result with respect to the existing consolidated fills. A three month time period is recommended.

Construction

All softened and/or organic material should be excavated for their full depth within the planned limits prior to fill placement. In addition, to further supplement resistance to settlement and slope instability, the new approach fills are to be "benched" to the existing fills in accordance with MTO standards (OPSD-208.01).

4) Temporary Shoring

To facilitate construction of the abutment extension and simultaneously maintain traffic protection on Sheppard Avenue, a temporary shoring system may be required. A cantilever soldier pile lagging system is one method of shoring recommended. For purposes of design of the shoring, the following soil parameters are provided.

<u>Soil</u>	<u>Elevation</u>	<u>Unit Weight</u>	<u>Shear Strength Parameters</u>	
	<u>(m)</u>	<u>(kN/m³)</u>	<u>Cu (kPa)</u>	<u>ϕ°</u>
Clayey Silt to Silty Clay				
(a) upper dessicated zone	147-143	19	100	0
(b) lower saturated zone	143-139	20	50	0
Glacial Till	<139	20	0	30

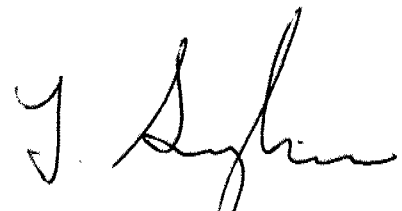
Earth pressures shall be computed in accordance with section 6.6.1.2 of the O.H.B.D.C. The effects of the groundwater table located at approximate elevation 143.2 shall be incorporated in the computation.

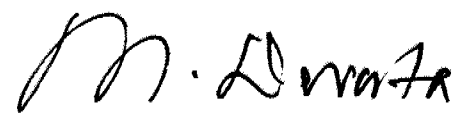
The soldier piles can be installed in preaugered holes or driven using conventional pile driving methods. Soil or rock anchors may be installed to augment the lateral resistance of the shoring wall. Pertinent pull-out capacities can be provided by this office if anchors are selected in the design.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of F. Pinder, Engineer Trainee, utilizing equipment owned and operated by Master Soil Investigation. This report was written by F. Pinder and T. Sangiuliano and reviewed by Mr. M.S. Devata, Chief Foundation Engineer.

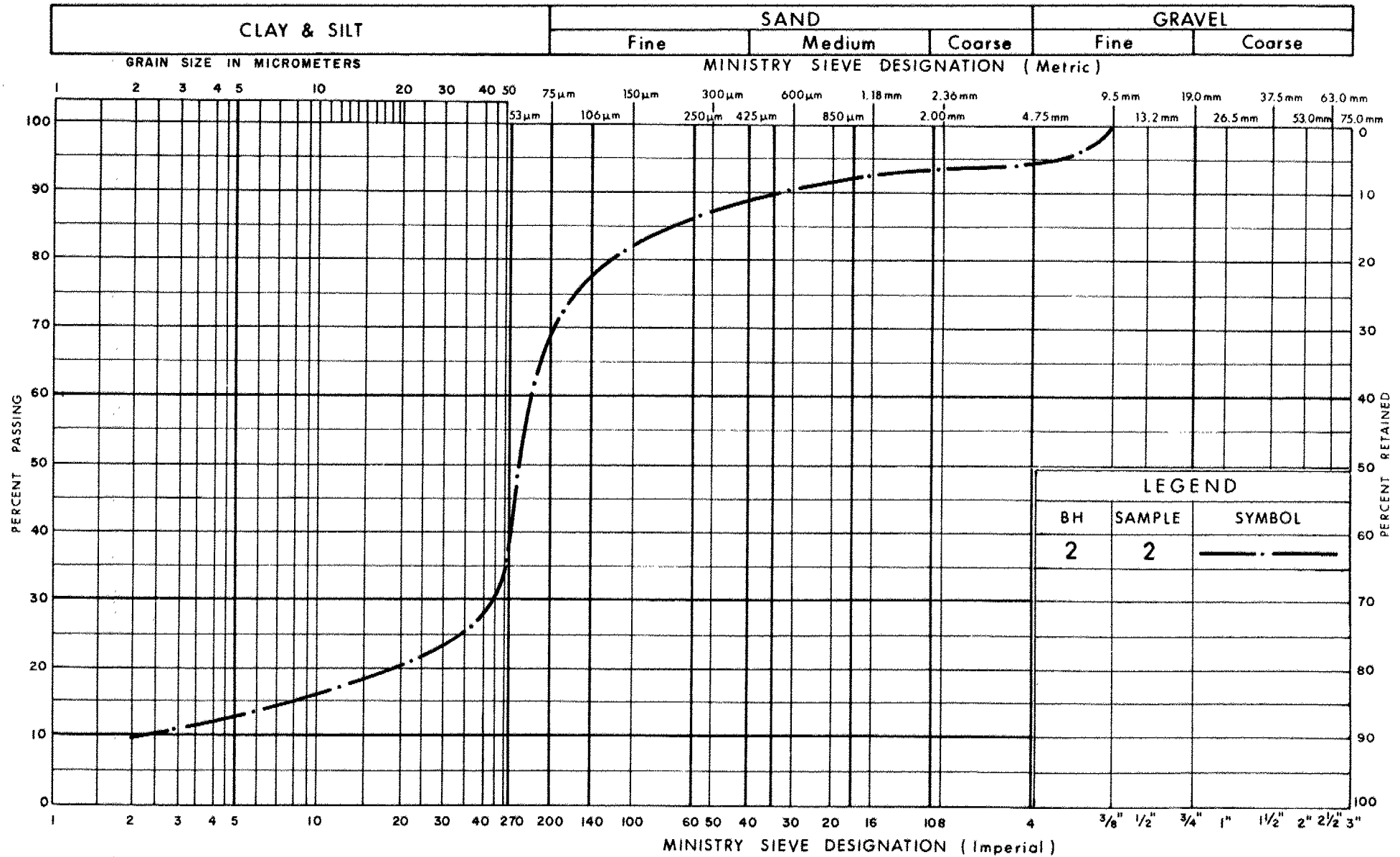



T. Sangiuliano, P.Eng.
Foundation Engineer


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Chief Foundation Engineer

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM

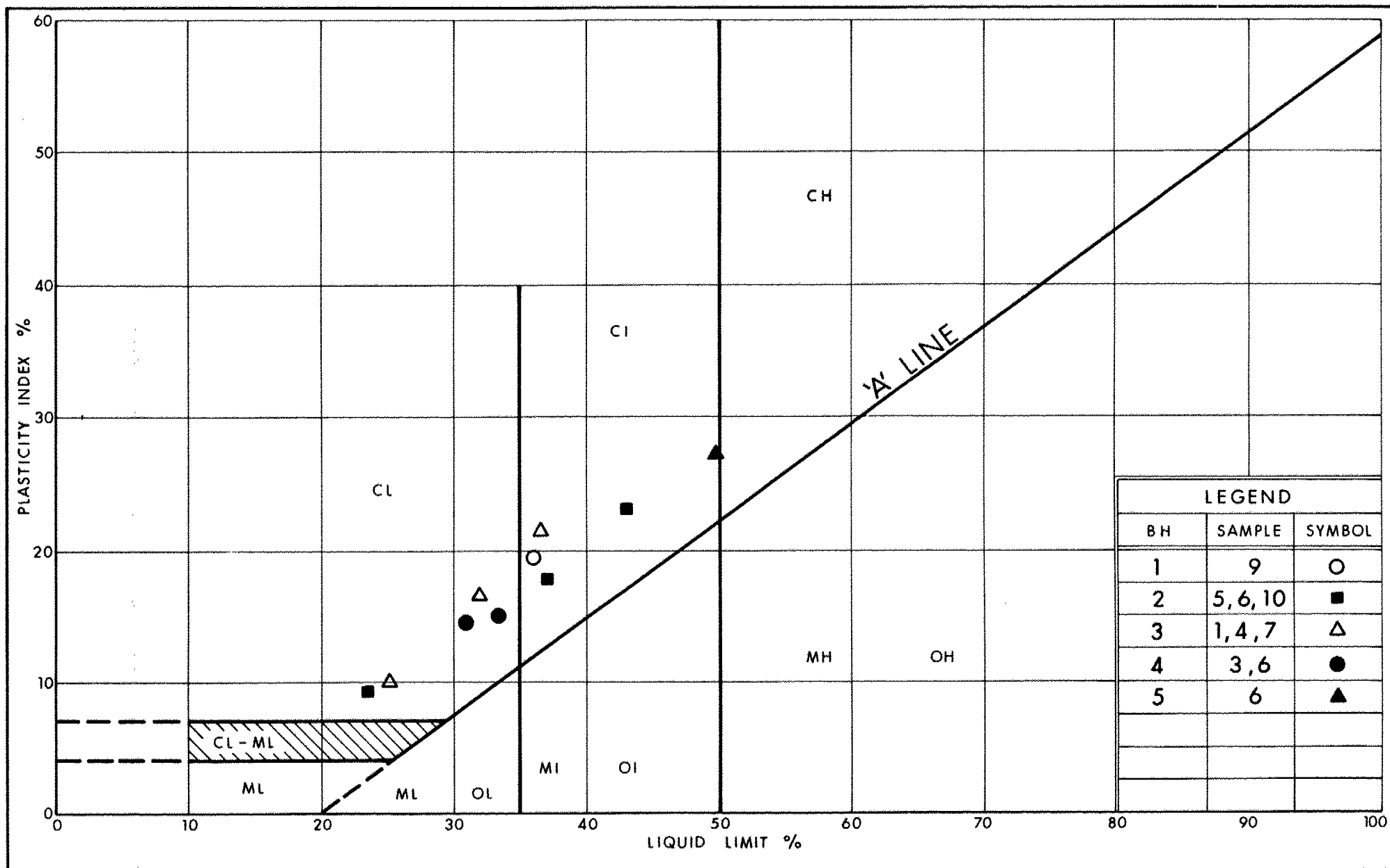


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GRAIN SIZE DISTRIBUTION
IRREGULAR MIXTURE OF
CLAYEY SILT, SAND & GRAVEL (Fill)

FIG No 1

W P 131-85-04



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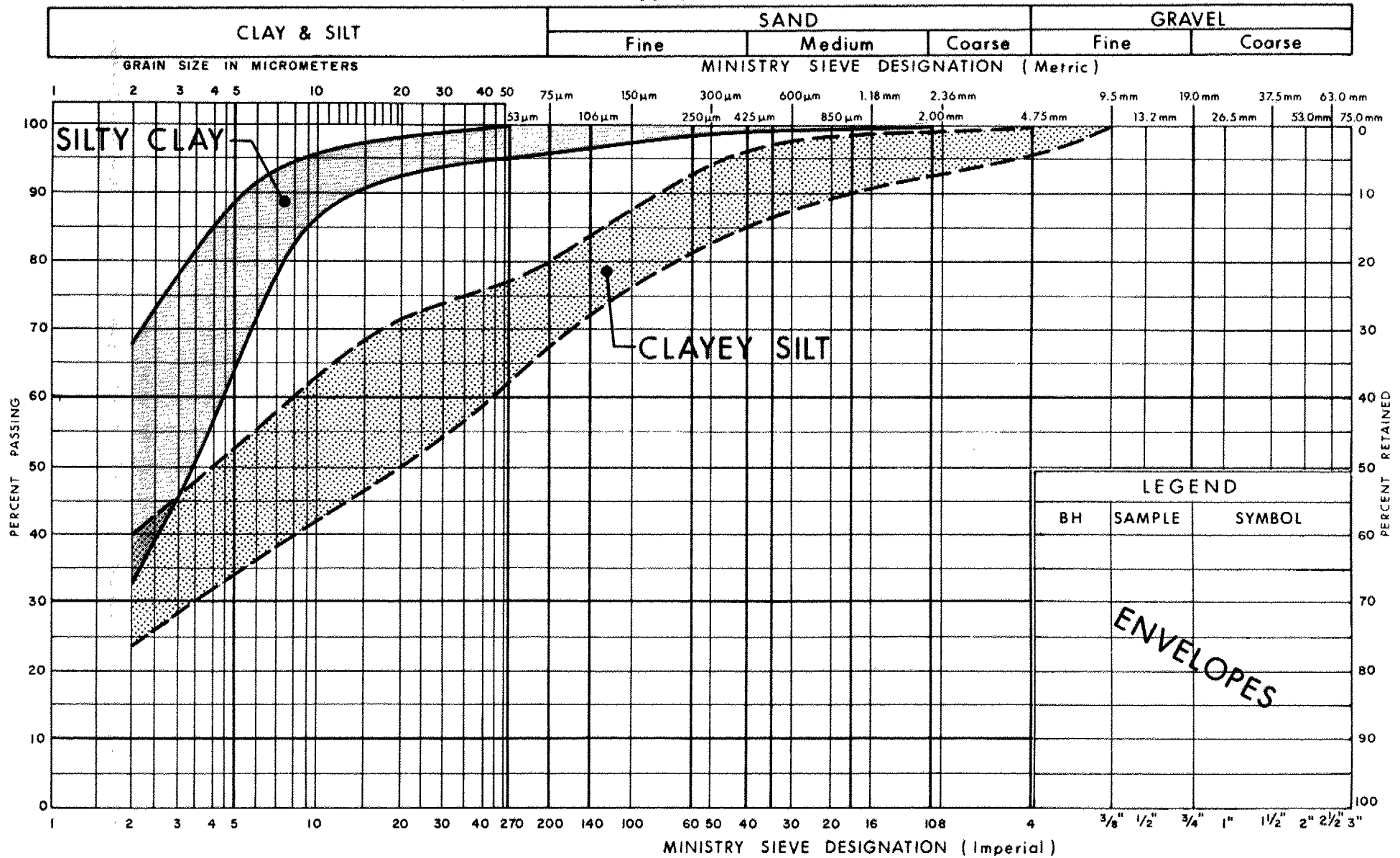
Ontario

PLASTICITY CHART CLAYEY SILT TO SILTY CLAY

FIG No 2

W P 131-85-04

UNIFIED SOIL CLASSIFICATION SYSTEM



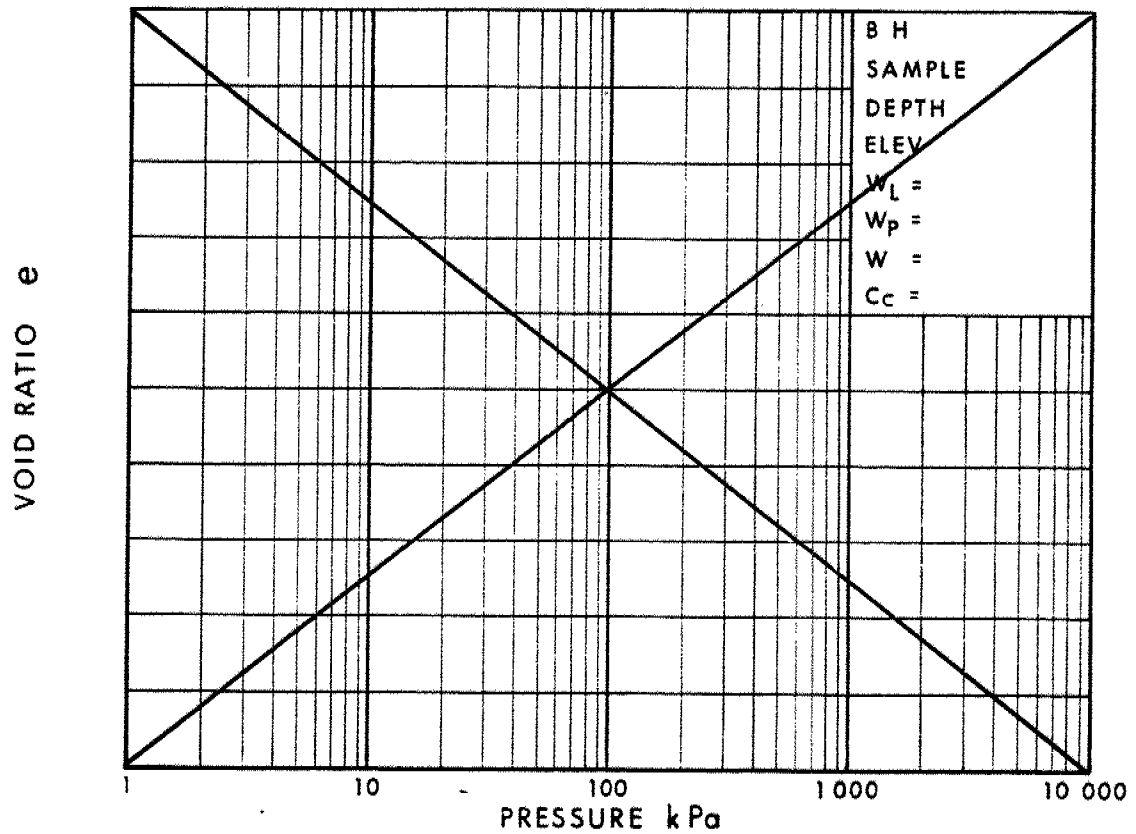
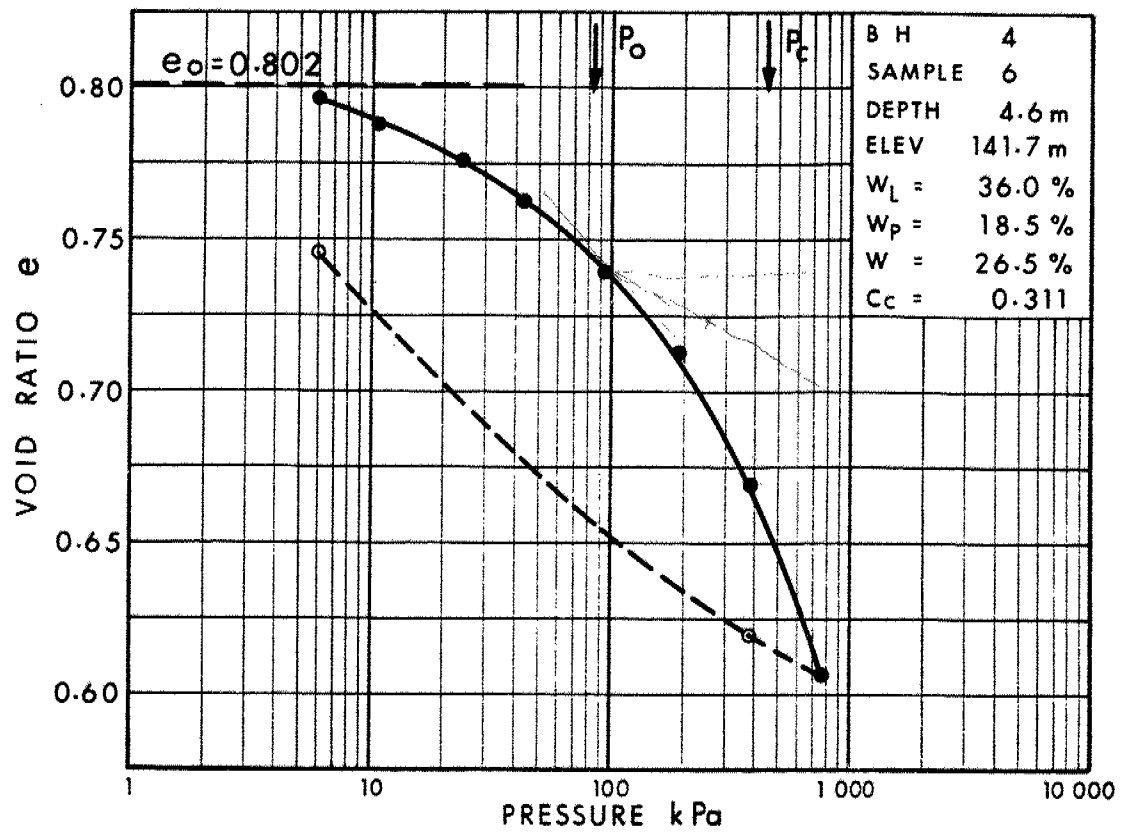
Ministry of
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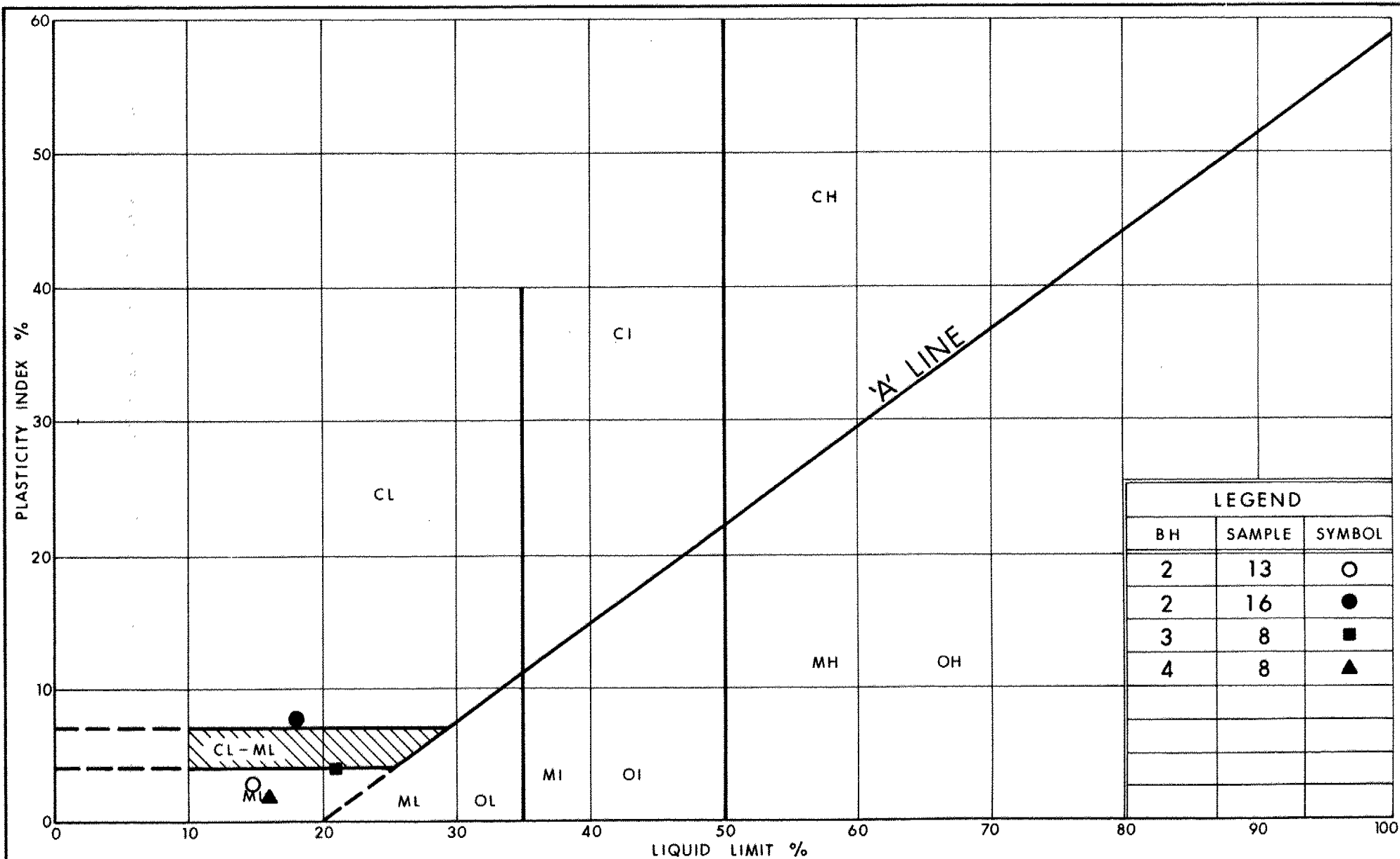
GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY

FIG No 3

W P 131-85-04

VOID RATIO - PRESSURE CURVES



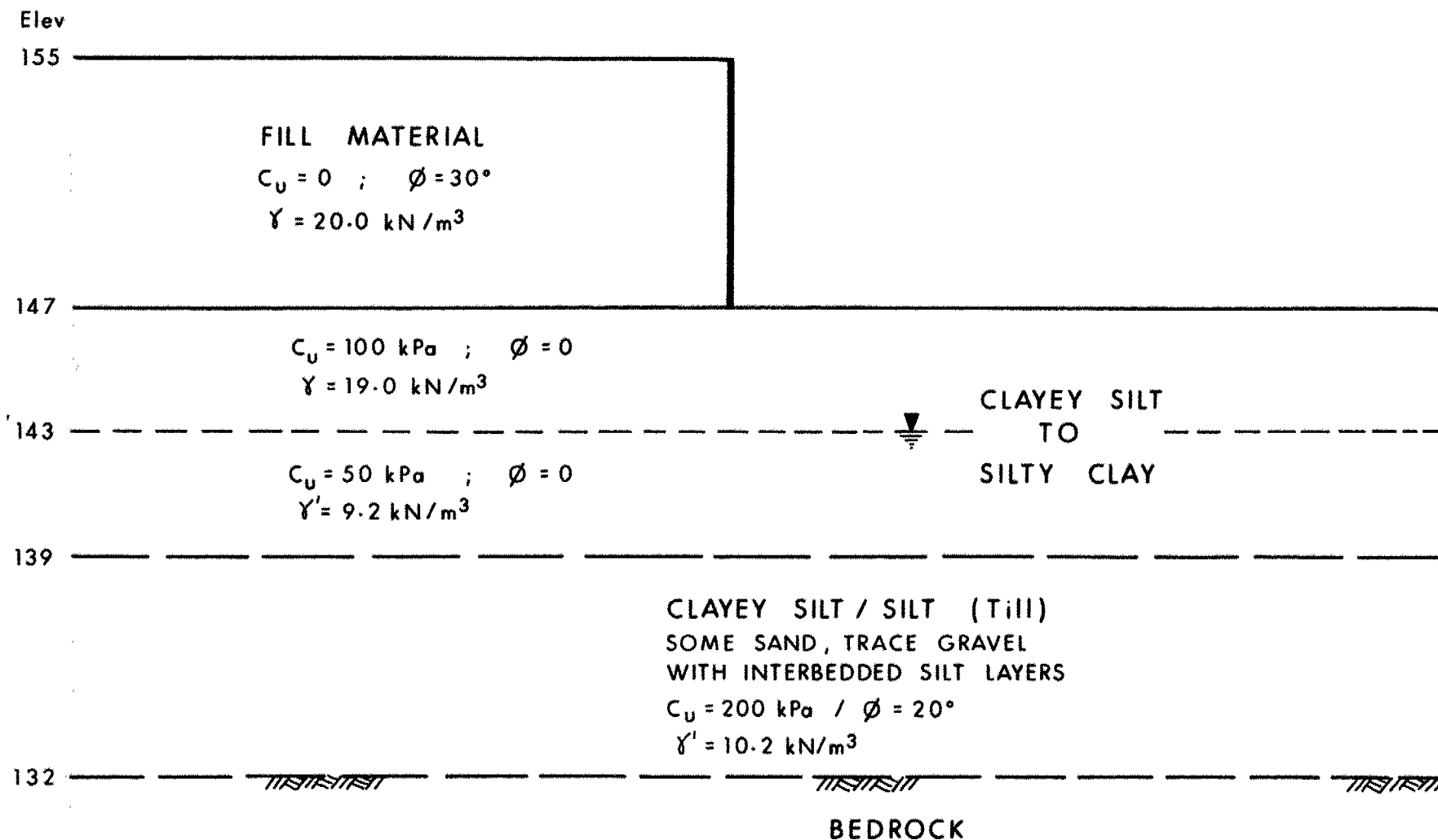


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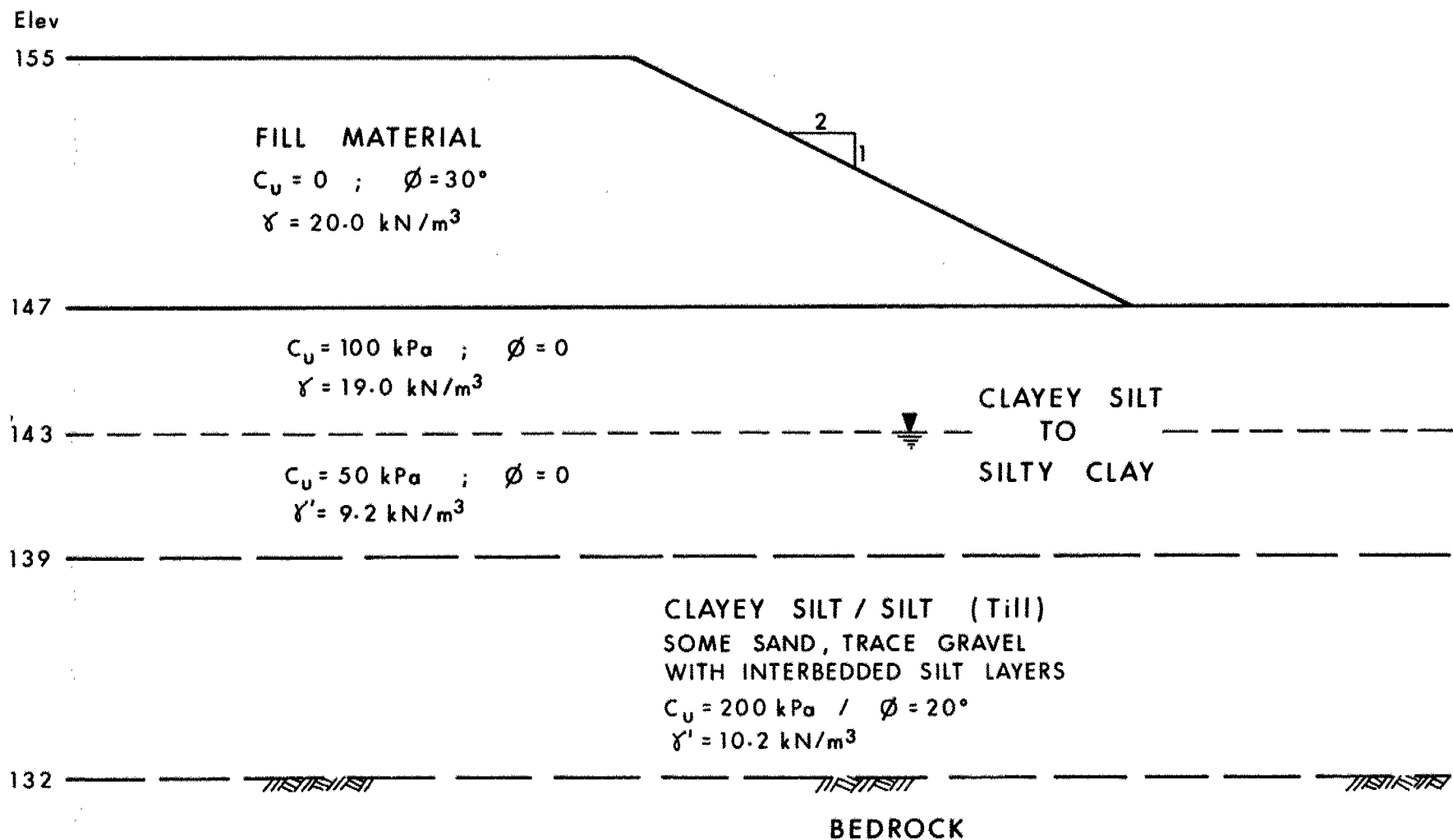
PLASTICITY CHART
CLAYEY SILT / SILT (Glacial Till)
SOME SAND, TRACE GRAVEL

FIG No 5

W P 131-85-04



APPROACH EMBANKMENT STABILITY ANALYSIS-LONGITUDINAL DIRECTION



APPROACH EMBANKMENT STABILITY ANALYSIS – TRANSVERSE DIRECTION

Fig 6b

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EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

MECHANICAL PROPERTIES OF SOIL

m_v	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 = $\frac{c_u}{\tau_r}$

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

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^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

Table 4 - ROCK CORE DESCRIPTION
WP 131-85-04

1../1

		CORE RECOVERY			CORE DESCRIPTION	
BH - RC # #		DEPTH (m)	CR* (%)	RQD* (%)	DEPTH (m)	DESCRIPTION
4	13	13.82-14.30	58	0	13.82-17.35	SHALE, medium grey, very thinly laminated; very fine grained; weak to very weak rock; moderately to highly weathered, completely weathered in sections, several clay seams 3-7 cm (thick) are also present; very close to extremely close spaced fractures: bedding joints, horizontal. Single bed of LIMESTONE from 14.30-14.40.
	14	14.30-15.37	55	0		
	15	15.37-16.92	80	0		
	16	16.92-17.35	94	0		

Logged by: S. A. Senior, Soils and Aggregates Section.

*CR = CORE RECOVERY (NOTE: Depths are approximated in zones of poor core recovery.)

*RQD = ROCK QUALITY DESIGNATION

RECORD OF BOREHOLE No 1

METRIC

W P 131-85-04 LOCATION Co-ords: N 4 843 787.3; E 302 573.4 ORIGINATED BY FP
 DIST 6 HWY 400 BOREHOLE TYPE Cone Test, Hollow-stem Auger COMPILED BY FP
 DATUM Geodetic DATE 89 01 16 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L		
152.9	Ground Surface												
0.0						*	152						
	Irregular Mixture of Clayey Silt, Sand and Gravel		1	SS	13								
	(Fill)		2	SS	13		150						
	Brown, Stiff		3	SS	11		148						
146.5			4	SS	23								
6.4			5	SS	35		146						
	Trace Organics		6	SS	46								
			7	SS	32		144						
143.1	Hard		8	SS	30								
9.8	Firm to Stiff		9	SS	11		142						
	Clayey Silt to Silty Clay occ Sand Seams		10	SS	13								
			11	SS	9		140						
			12	SS	7								
138.9			13	SS	8		138						
14.0			14	SS	21								
	Silt with Sand V. Dense		15	SS	105		136						
	Clayey Silt / Silt Some Sand, Trace Gravel Grey, Hard / V. Dense		16	SS	60 / 13 cm		134						
132.8	(Glacial Till)		17	SS	60 / 10 cm								
20.1	End of Borehole												
	* groundwater elevation not determined												

RECORD OF BOREHOLE No 2

METRIC

W P 131-85-04 LOCATION Co-ords: N 4 843 798.2; E 302 606.1 ORIGINATED BY FP
 DIST 6 HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY FP
 DATUM Geodetic DATE 89 01 17 CHECKED BY DD

SOIL PROFILE		STRAT PLOT	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		NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
153.9	Ground Surface																GR SA SI CL
0.0																	
	Irregular Mixture of Sand, Gravel and Clayey Silt Brown, Stiff to Hard		1	SS	15		152										6 25 60 9
	(Fill)		2	SS	34		150										
			3	SS	19		148										
147.7			4	SS	29		146										0 25 52 23
6.2	Trace Organics		5	SS	27		144										0 10 67 23
			6	SS	24		142										
			7	SS	42		140										
			8	SS	40		138										
142.9	V. Stiff to Hard		9	SS	24		136										0 3 47 50
11.0	Firm to Stiff		10	SS	20		134										
	Brown Grey		11	SS	11		132										
	Clayey Silt to Silty Clay		12	TW	PH												
139.3			13	SS	10												
14.6	Silt with Sand Compact to Dense		14	SS	37												0 39 53 8
			15	SS	91												
	Clayey Silt / Silt Some Sand, Trace Gravel Grey, Hard / V. Dense		16	SS	95												
	(Glacial Till)		17	SS	58												
131.2			18	SS	60	0 cm											
22.7	End of Borehole																
	Probable Bedrock																

RECORD OF BOREHOLE No 3

METRIC

W P 131-85-04 LOCATION Co-ords: N 4 843 783.8; E 302 601.8 ORIGINATED BY FP
 DIST 6 HWY 400 BOREHOLE TYPE Cone Test, Hollow Stem Auger COMPILED BY FP
 DATUM Geodetic DATE 89 01 18 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
146.7	Ground Surface															GR SA SI CL
0.0	Clayey Silt to Silty Clay		1	SS	13	*	146									6 18 41 35
			2	SS	13											
			3	SS	26		144									1 20 39 40
			4	SS	29											
			5	SS	14		142									
141.7	Stiff to V. Stiff		6	SS	14											
5.0	Firm to Stiff		7	TW	PH		140								21.2	4 25 49 22
139.4			8	SS	11											1 19 65 15
7.3	Silt with occasional Sand and Gravel Seams		9	SS	89 /	28 cm	138									
			10	SS	60 /	10 cm	136									
133.9	Clayey Silt / Silt		11	SS	83		134									
12.8	Some Sand, Trace Gravel Hard / V. Dense															
132.9	(Glacial Till)		12	SS	60 /	13 cm										
13.8	End of Borehole															
	* groundwater elevation not determined															

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 4

METRIC

W P 131-85-04 LOCATION Co-ords: N 4 843 782.8; E 302 584.6 ORIGINATED BY FP
DIST 6 HWY 400 BOREHOLE TYPE Hollow Stem Auger, BQ Rock Core COMPILED BY FP
DATUM Geodetic DATE 89 01 22 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
146.3	Ground Surface						146							
0.0	Trace Organics		1	SS	20		146							
			2	SS	21		144							
143.6	Very Stiff		3	SS	27		144							5 27 39 29
2.7	Firm to Stiff	Brown Grey	4	SS	13		142							
	Clayey Silt to Silty Clay occ. Sand Seams		5	SS	9		142							
			6	TW	PH		142							
			7	SS	5		142							
139.3							140							
7.0			8	SS	5		138							
	Loose Silt with Sand V. Dense		9	SS	94		138							3 32 60 5
135.8			10	SS	57		136							
10.5	Clayey Silt / Silt Some Sand, Trace Gravel Grey, Hard / V. Dense (Glacial Till)		11	SS	57		134							
132.5			12	SS	60	10 cm	132							
13.8	Bedrock Shale with Interbedded Clay Seams		13	RC	Rec 58%		132							RQD = 0
			14	RC	55%		130							RQD = 0
			15	BQ	Rec 80%		130							RQD = 0
129.0	Weathered Unweathered		16	RC	Rec 94%		130							RQD = 0
17.3	End of Borehole													

+³, x⁵: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 5

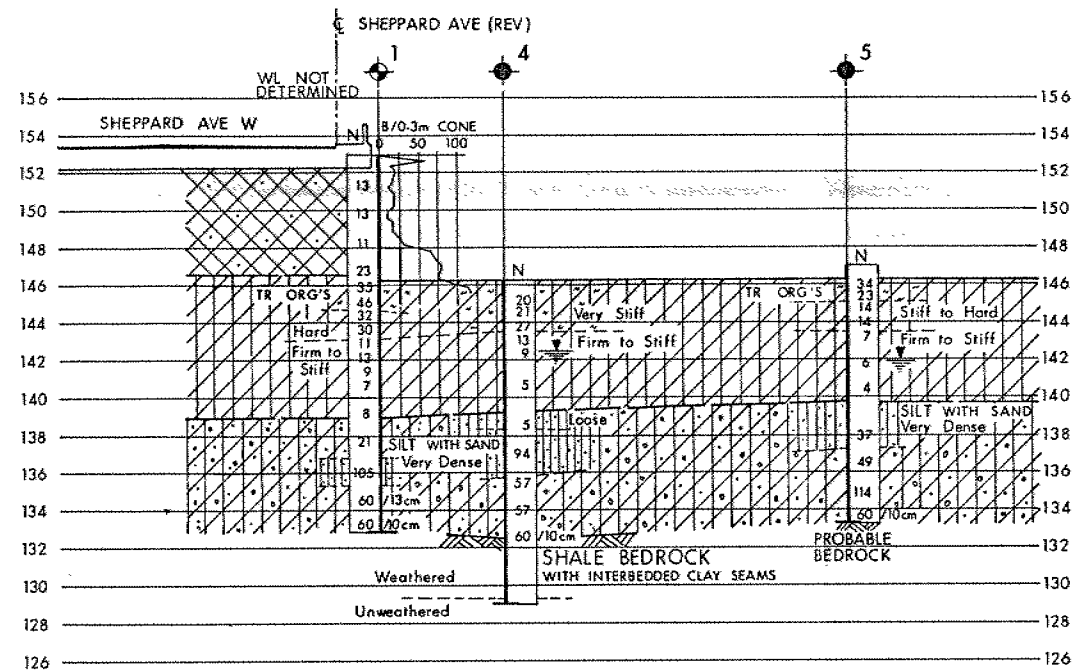
METRIC

W P 131-85-04 LOCATION Co-ords: N 4 843 761.3; E 302 570.0 ORIGINATED BY FP
DIST 6 HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY FP
DATUM Geodetic DATE 89 01 24 CHECKED BY DD

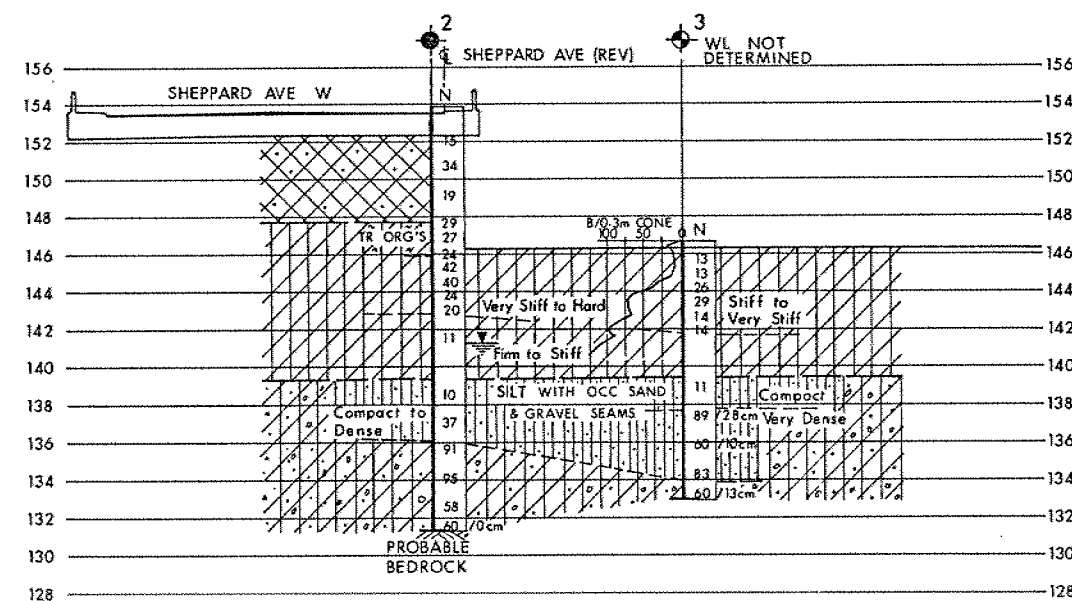
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
147.0	Ground Surface																
0.0																	
			1	SS	34		146										
	Trace Organics		2	SS	23												
			3	SS	14												
	Stiff to Hard	Brown Grey	4	SS	14		144										
143.5	Firm to Stiff		5	SS	7												
3.5			6	SS	6		142										
	Clayey Silt to Silty Clay		7	TW	PH												
			8	SS	4		140										
139.7																	
7.3	Silt with Sand V. Dense		9	SS	37		138										
	Clayey Silt / Silt		10	SS	49		136										
	Some Sand, Trace Gravel																
	Grey, Hard / V. Dense		11	SS	114		134										
	(Glacial Till)																
133.3			12	SS	60	10 cm											
13.7	End of Borehole																
	(Probable Bedrock)																

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



A-A



B-B

SECTIONS

SCALE
4m 2 0 4m

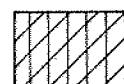
SOIL STRATIGRAPHY LEGEND



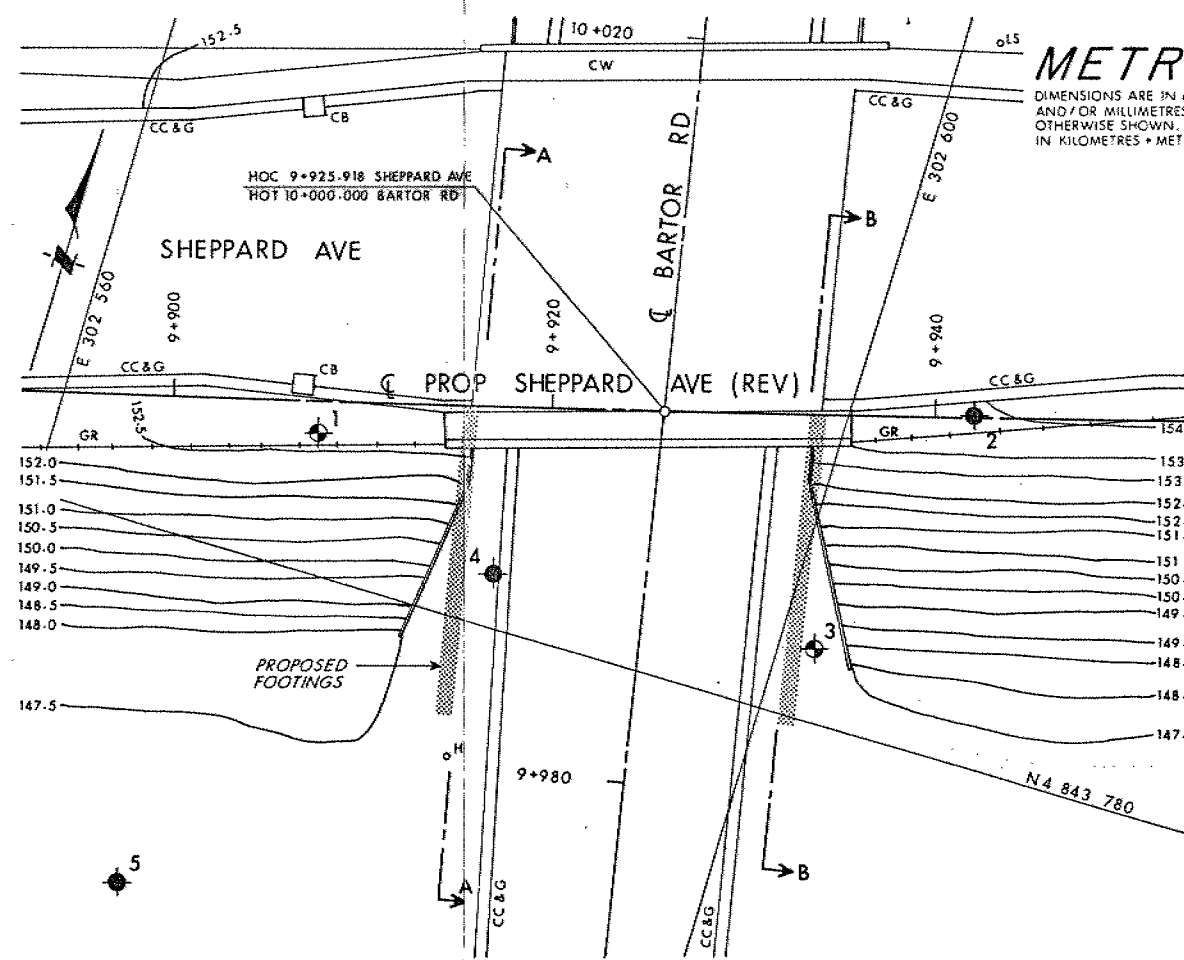
IRREGULAR MIXTURE OF
CLAYEY SILT, SAND & GRAVEL
Stiff to Hard
(FILL)



CLAYEY SILT/SILT
SOME SAND, TRACE GRAVEL
(GLACIAL TILL)
Hard/Very Dense

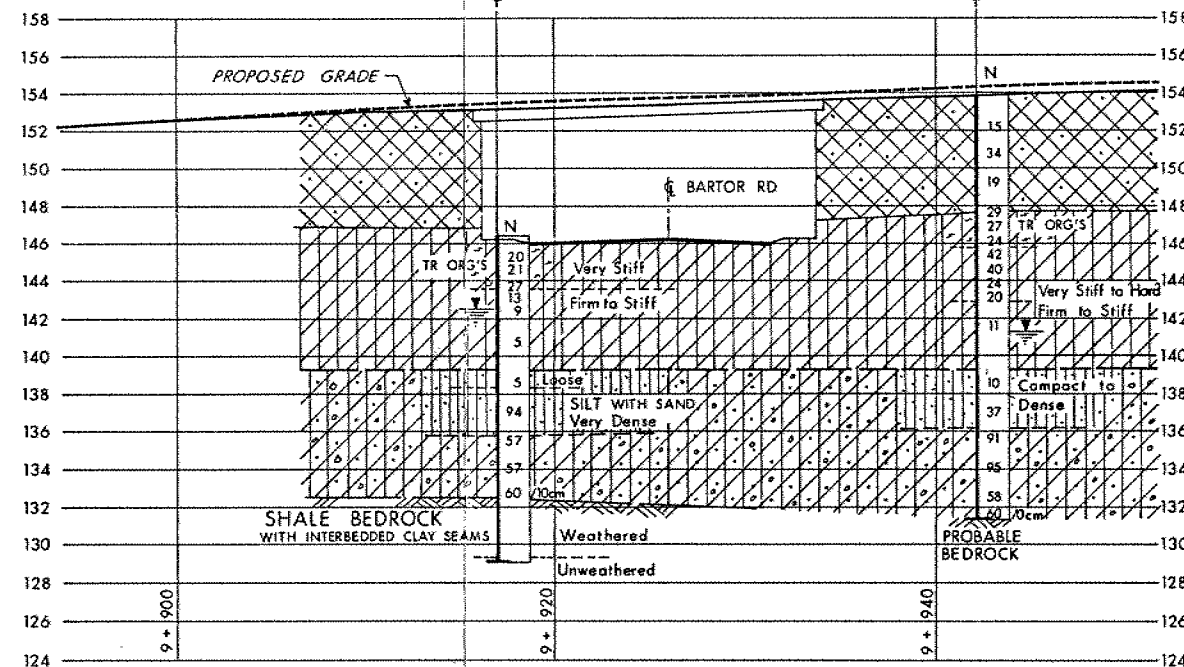


CLAYEY SILT TO SILTY CLAY
OCCASIONAL SAND SEAMS
Firm to Hard



PLAN

SCALE
4m 2 0 4m



PROFILE SHEPPARD AVE (REV)

SCALE
4m 2 0 4m

METRIC

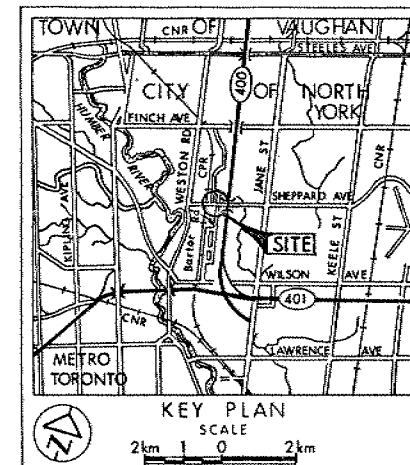
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 131-85-04

SHEPPARD AVE & BORTOR RD
UNDERPASS
BORE HOLE LOCATIONS & SOIL STRATA



SHEET



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- Wt at time of investigation 89 01

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	152.9	4 843 787.3	302 573.4
2	153.9	4 843 798.2	302 606.1
3	146.7	4 843 783.8	302 601.8
4	146.3	4 843 782.8	302 584.6
5	147.0	4 843 761.3	302 570.0

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 102-2 of Form 100.

DATE	BY	DESCRIPTION
89 04 28	TS	DATE
89 04 28	TS	DATE
89 04 28	TS	DATE

Geocres No 30M11-188

HWY No	DIST 6
SUBMD TS	CHECKED
DATE	89 04 28
SITE	37-1019
DRAWN DT	CHECKED
APPROVED	DWG 1318504-A

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

SHEET 1

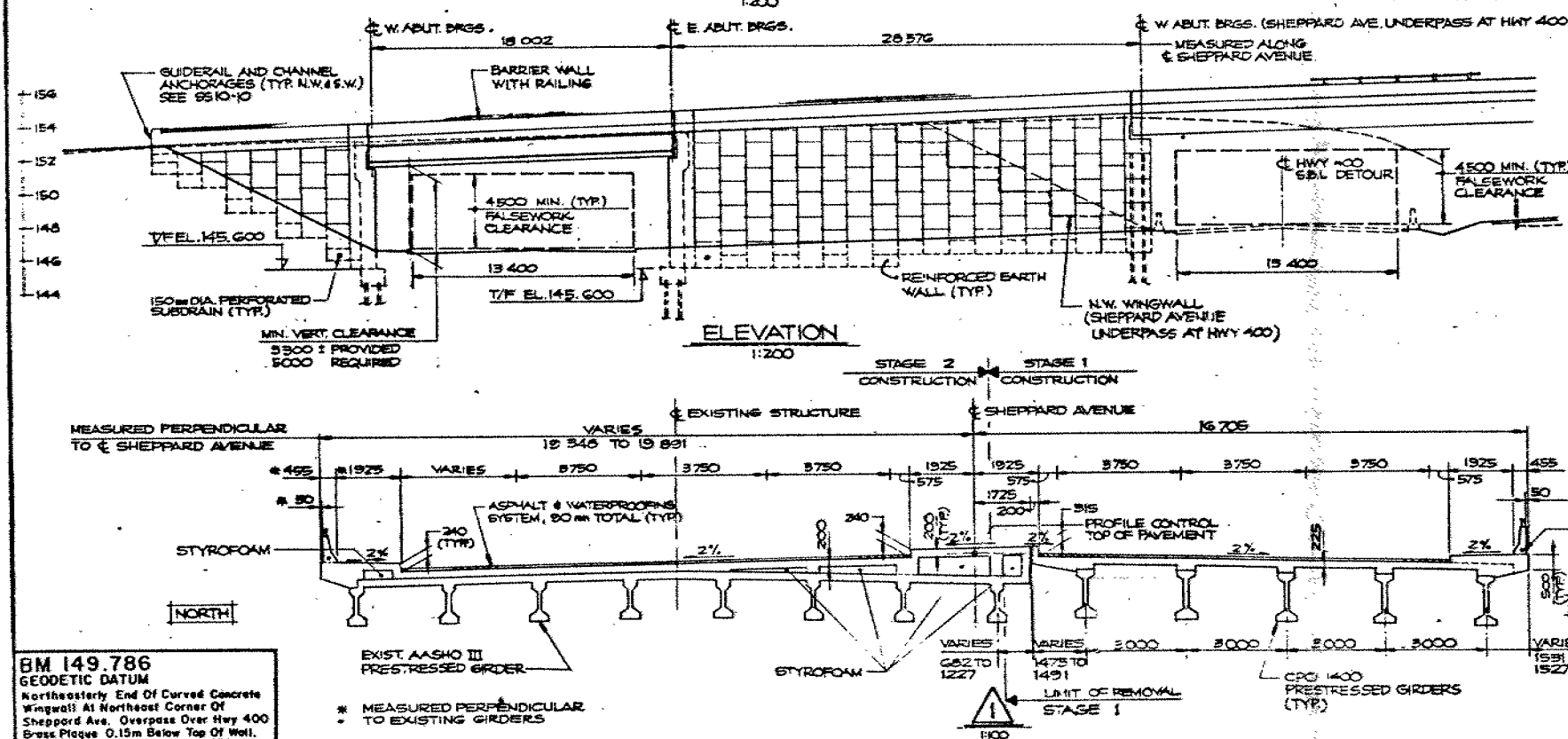
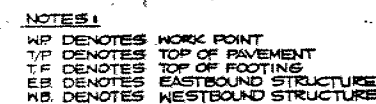
GENERAL NOTES

- ### LIST OF DRAWINGS

- APPLICABLE STANDARD DRAWINGS**



REVISIONS							
	DATE	BY	DESCRIPTION				
DESIGN	H.C.	CHK	K.A.	CODE OHBDC - 83	LOAD CLASS A	DATE	MAR. 92
DRAWN	F.M.	CHK	J.Y.	SITE 37-1019	STRUCT 2	SCHEME	DWG. 1



BM 149.786
GEODETIC DATUM
Northeasterly End Of Curved Concrete
Wingwall At Northeast Corner Of
Sheppard Ave. Overpass Over Hwy 400
Bress Plaque 0.15m Below Top Of Wall.
22.20 R. 15 + 362.8 Hwy 400

* MEASURED PERPENDICULAR
* TO EXISTING GIRDERS

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

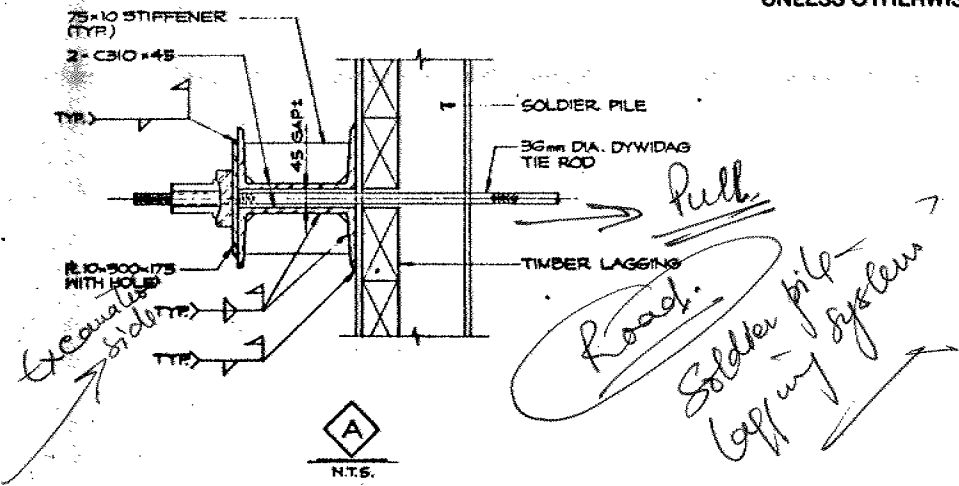
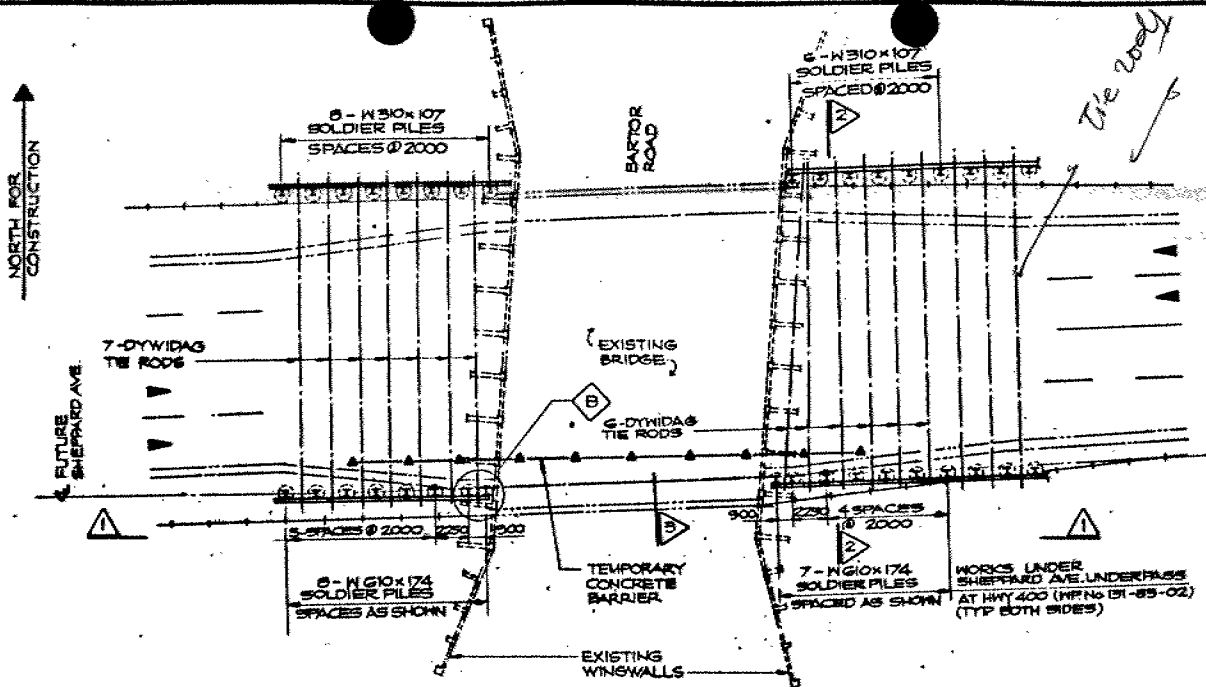
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 131-85-04

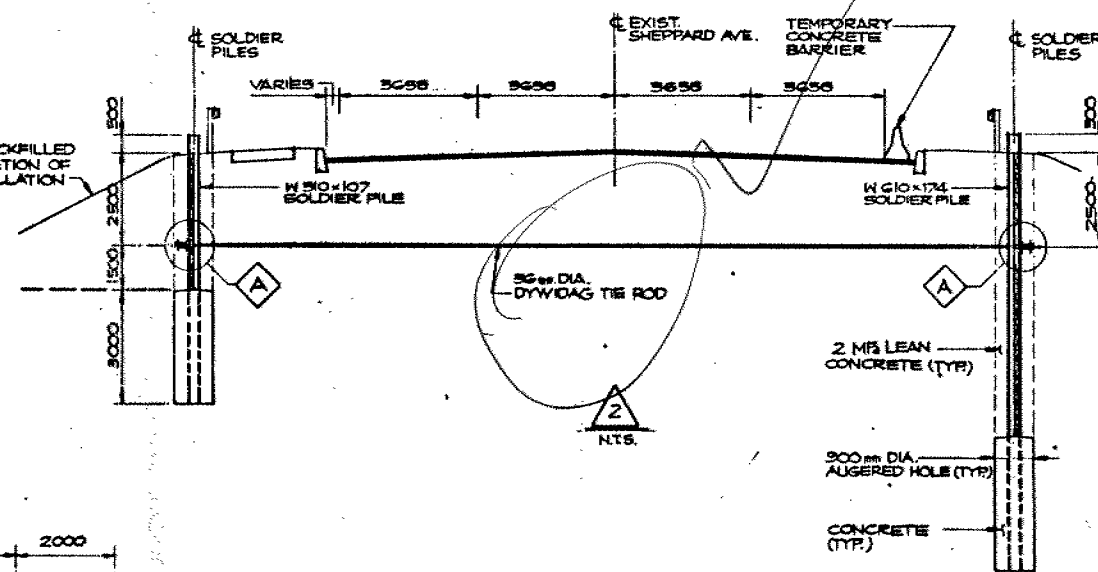
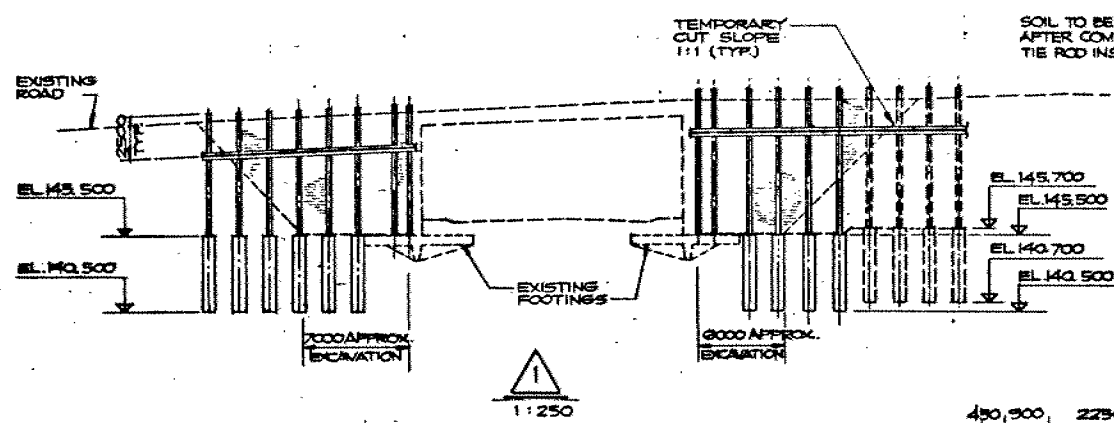
SHEPPARD AVENUE STRUCTURE
OVER BARTOR ROAD
ROADWAY PROTECTION



UMA UMA Engineering Ltd.
Engineers & Planners

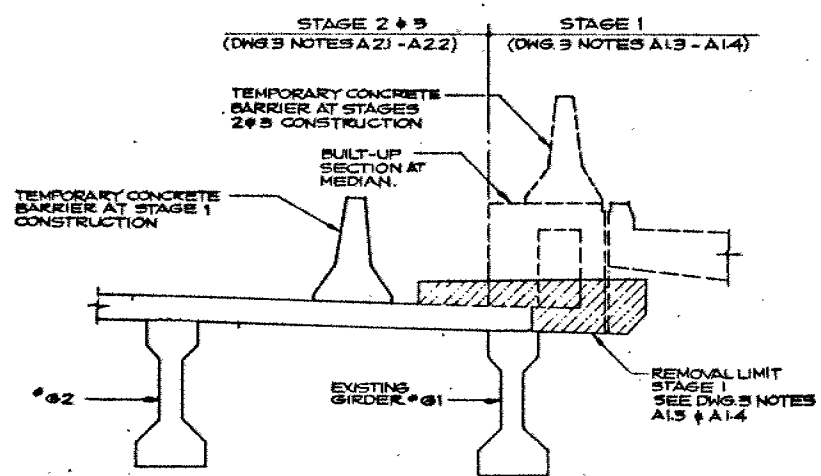
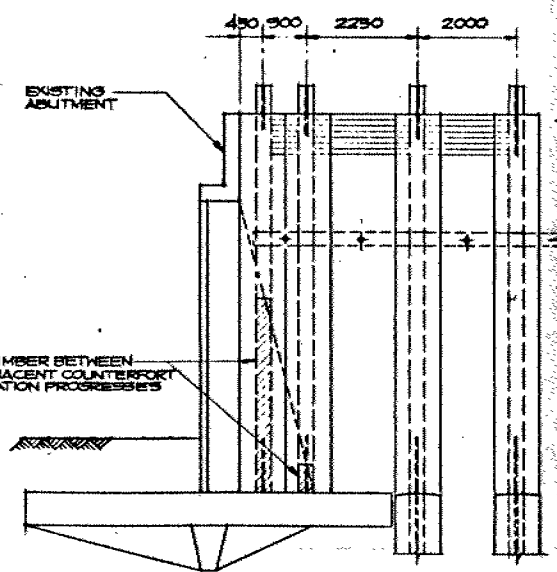
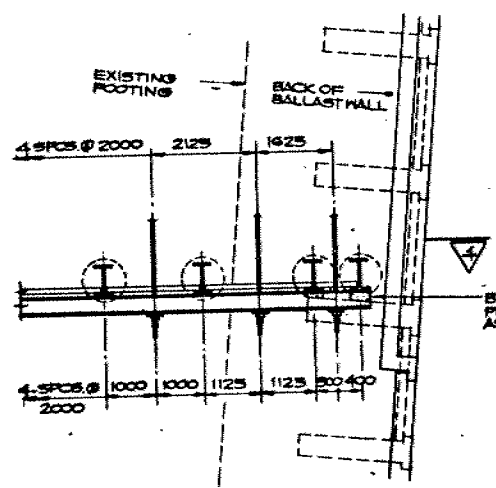


- NOTES**
- THE CONTRACTOR SHALL SUBMIT DETAILED CONSTRUCTION DRAWINGS SEALED & SIGNED BY A PROFESSIONAL ENGINEER FOR ANY ROADWAY PROTECTION WORKS. ALTERNATIVE ROADWAY PROTECTION SCHEMES WILL BE CONSIDERED SUBJECT TO THE ENGINEER'S APPROVAL. ALTERNATIVE SCHEMES WILL NOT BE ALLOWED TO ENCRUSH INTO THE EXISTING ROAD.
 - STRUCTURAL STEEL FOR ROADWAY PROTECTION SHALL BE CAN/CSA C40.21-M87, GRADE 300W.
 - WELDING OF STRUCTURAL STEEL SHALL BE IN ACCORDANCE WITH CSA-W59-M1989.
 - THE CONTRACTOR SHALL VERIFY THE EXISTING GRADE ELEVATIONS AND REPORT ANY DISCREPANCIES TO THE ENGINEER BEFORE PROCEEDING WITH THE WORKS.
 - UNLESS OTHERWISE NOTED, THE MINIMUM FILLET WELD SIZES SHALL BE AS FOLLOWS:
- | METAL THICKNESS (mm) | WELD SIZE (mm) |
|----------------------|----------------|
| 12 OR LESS | 5 |
| OVER 12 TO 20 | 6 |
| OVER 20 TO 40 | 8 |
- TIMBER LAGGING SHALL BE HARDWOOD LUMBER AND SHALL CONFORM TO CAN/CSA-M84.
 - LAGGING SHALL BE INSTALLED IN LIFTS NOT EXCEEDING 1.0 m AS EXCAVATION PROCEEDS.
 - PILES SHALL BE PLACED INTO PRE-DRILLED HOLES AND HOLES SHALL BE FILLED WITH 30 MPa CONCRETE BELOW BASE OF EXCAVATION OR AS STATED OTHERWISE AND WITH 5.0 MPa LEAN CONCRETE UP TO GRADE LEVEL.
 - TIE RODS SHALL BE DYWIDAG THREAD BARS GRADE 1030 MPa CONFORMING TO THE REQUIREMENTS OF ASTM DESIGNATION A722.



**EAST AND WEST ABUTMENT WALLS
ROADWAY PROTECTION CONSTRUCTION SEQUENCE**

- INSTALL TEMPORARY CONCRETE BARRIERS
- INSTALL SOLDIER PILES
- EXCAVATE AND INSTALL TIMBER LAGGING TO ELEVATION 0.3 m BELOW WATER LEVEL
- INSTALL TIE RODS AND WALERS
- STRESS TIE RODS TO 150 kN
- EXCAVATE TO ELEVATIONS AS SHOWN
- CONSTRUCT EAST AND WEST ABUTMENTS INCLUDING REINFORCED EARTH WALLS
- REMOVE SHORINGS 1.0 m BELOW FINISHED GRADE



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	H.C. CHK	K.A.	CODE QH80C - 83 [LOAD CLASS A] DATE MAR. 92
DRAWN	E.M. CHK	J.Y.	SITE 37-1019 [STRUCT] SCHEME DWG. 4



NOTES

1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH DWOS 1 & 7

ABUTMENT WALLS CONSTRUCTION SEQUENCE

1. CONSTRUCT ROADWAY PROTECTION AND EXCAVATE DOWN TO EL.145.600
2. REMOVE EXISTING WING WALL STRUCTURES AND COUNTERFORTS
3. DRIVE STEEL H-PILES
4. CONSTRUCT ABUTMENT PILE CAPS UP TO EL.145.600
5. CONSTRUCT TERRACE WALLS UP TO EL.153.690 (E) AND 152.690 (W)
6. CONSTRUCT ABUTMENT WALLS

PILE DATA				
LOCATION	PILE NUMBER	BATTER	LENGTH (W)	PILE CAP. ELEVATION
E. ABUT.	P1, P2	VERT.	13.0	145.85
	P3-P6	VERT.	12.0	144.63
W. ABUT.	P1, P2	VERT.	13.0	145.85
	P3-P6	VERT.	12.0	144.63

1. PILES SHALL BE 10" 310 x 110
2. PILES SHALL HAVE DRIVING SHOES
3. PILES SHALL BE DRIVEN IN ACCORDANCE WITH STANDARD SS103-11 USING AN ULTIMATE CAPACITY 3450 kN
4. PILE SPACING IS MEASURED AT UNDERSIDE OF FOOTINGS
5. PILE LENGTHS SHOWN ARE THEORETICAL LENGTH BELOW CUT-OFF
6. PILES WITHIN 5 m OF THE EXISTING STRUCTURE FOOTINGS SHALL BE INSTALLED IN A 500 mm DIAMETER HOLE PRE- AUGERED TO 3 m BELOW THE EXISTING FOOTINGS AND BACKFILL WITH GRANULAR MATERIAL AFTER PILE DRIVING
7. PILE DESIGN DATA

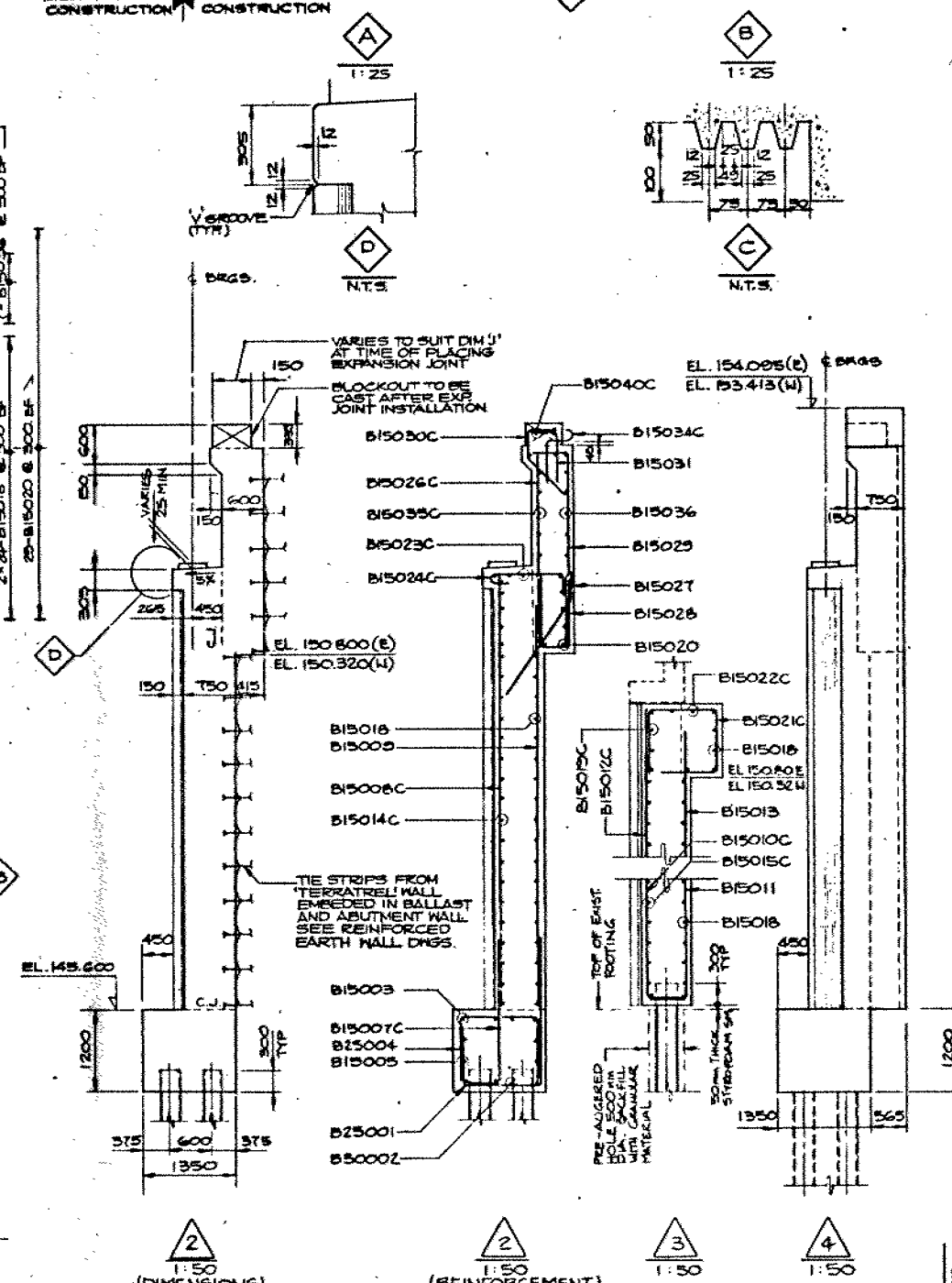
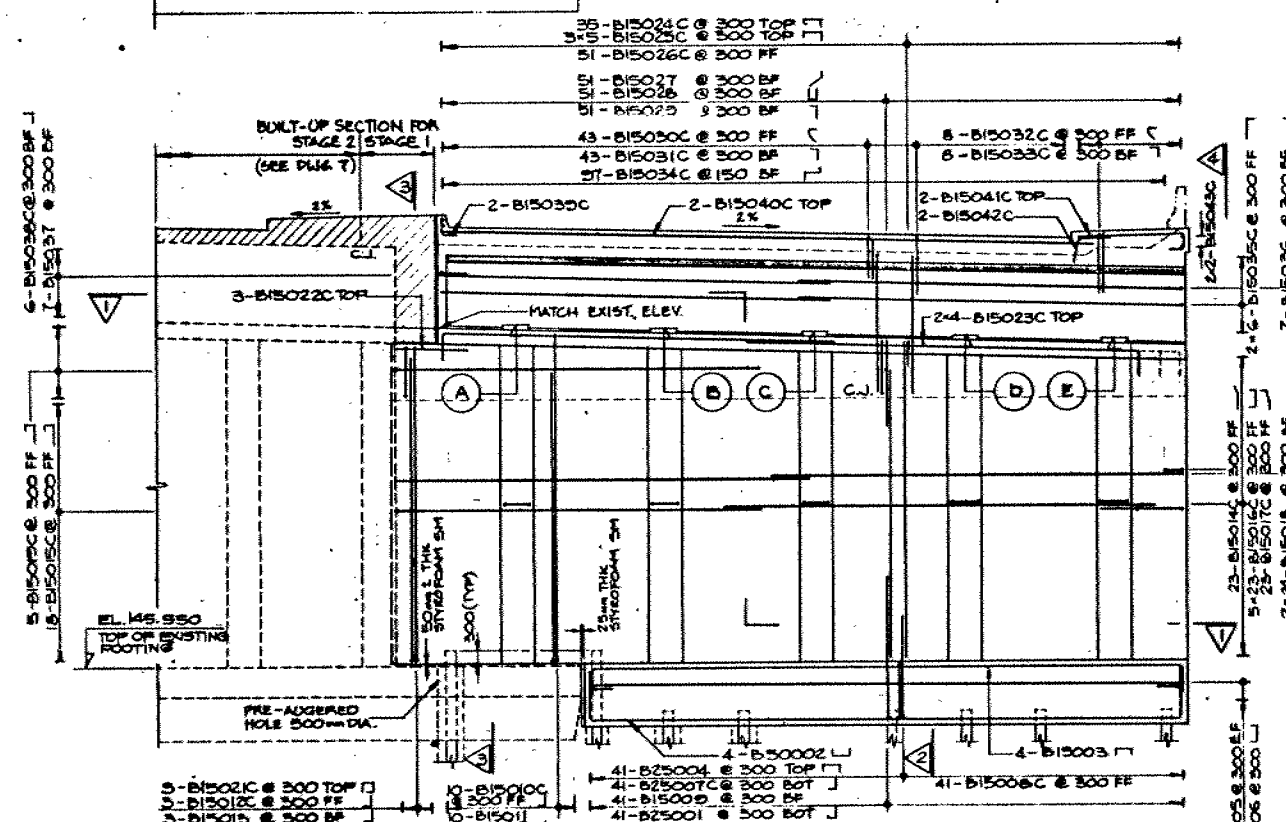
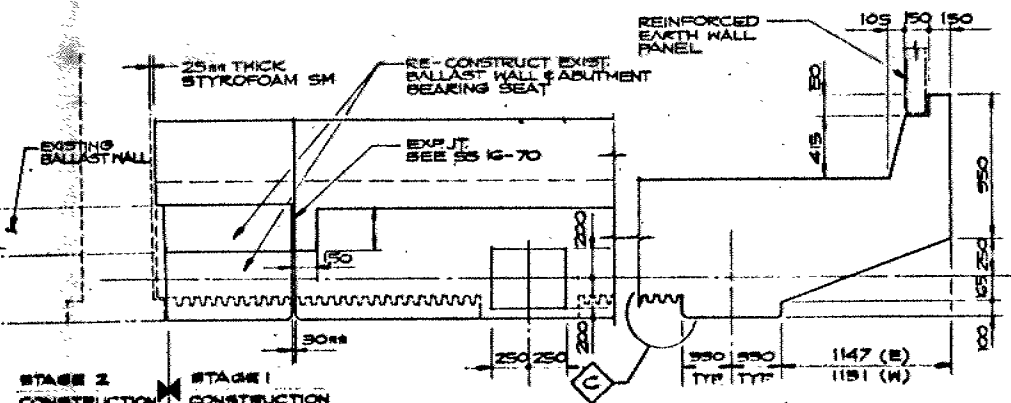
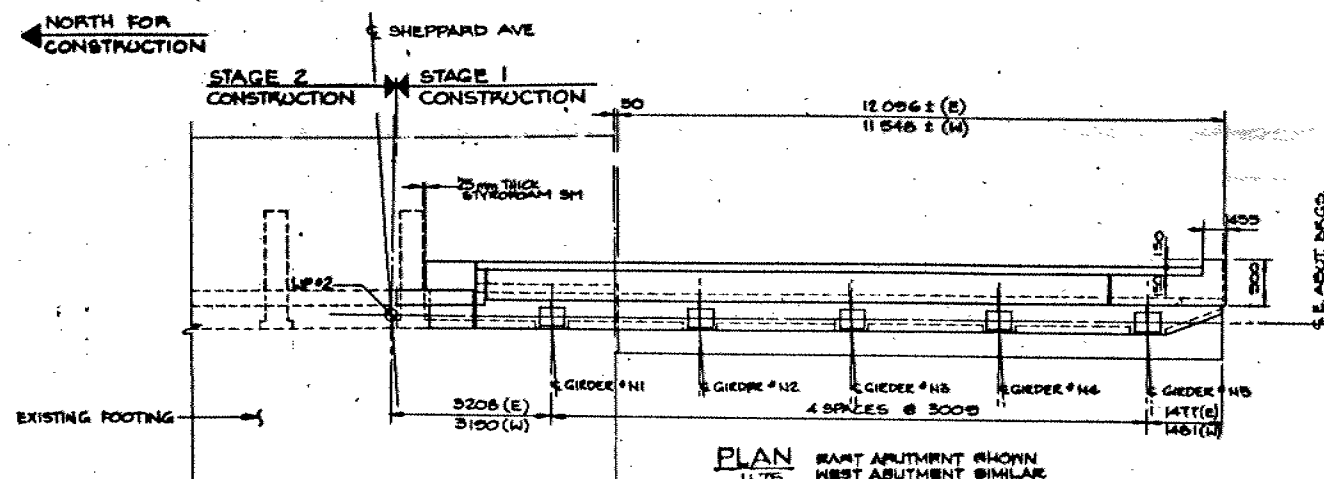
CAPACITY AT SLS II	1150 kN
FACTORED CAPACITY AT ULS	1600 kN

APPLICABLE STANDARD DRAWINGS

DD-3001 SPlice AND DRIVING SHOE DETAILS FOR STEEL HP PILES

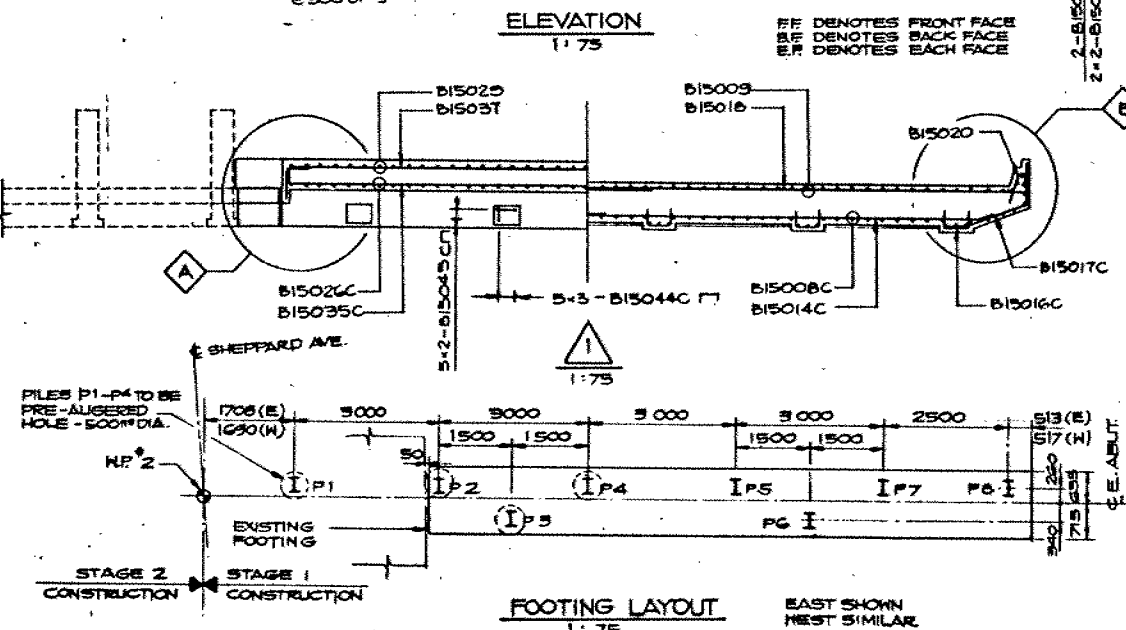


REVISIONS									
	DATE	BY				DESCRIPTION			
	DESIGN	H.C.	CHK	K.A.	CODE	OHBOC - 83	LOAD CLASS	A	DATE MAR 92
	DB&WM	R.M.	CHK	E.K.	SITE	37-3000	STRUCT	IS	SCHEME TDWG



* BEARING SEAT ELEVATIONS		
E L E V A T I O N	LOCATION	
	EAST ABUT.	WEST ABUT.
(A)	152.318	151.657
(B)	152.249	151.588
(C)	152.180	151.518
(D)	152.112	151.449
(E)	152.043	151.379

* FOR CONSTRUCTION
NOTES SEE DWG. # 1

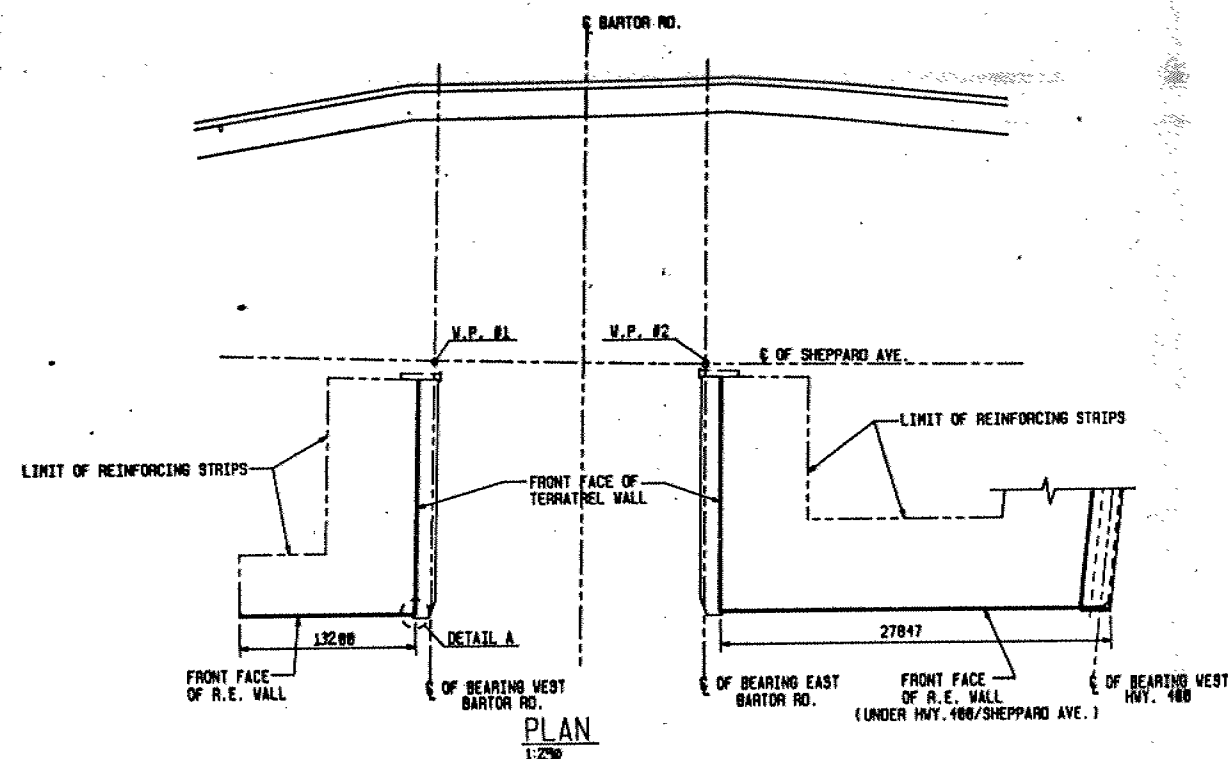


DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REINFORCED EARTH COMPANY LTD.
190 ATTVELL DRIVE, SUITE 501
REXDALE, ONTARIO M9W 6H8

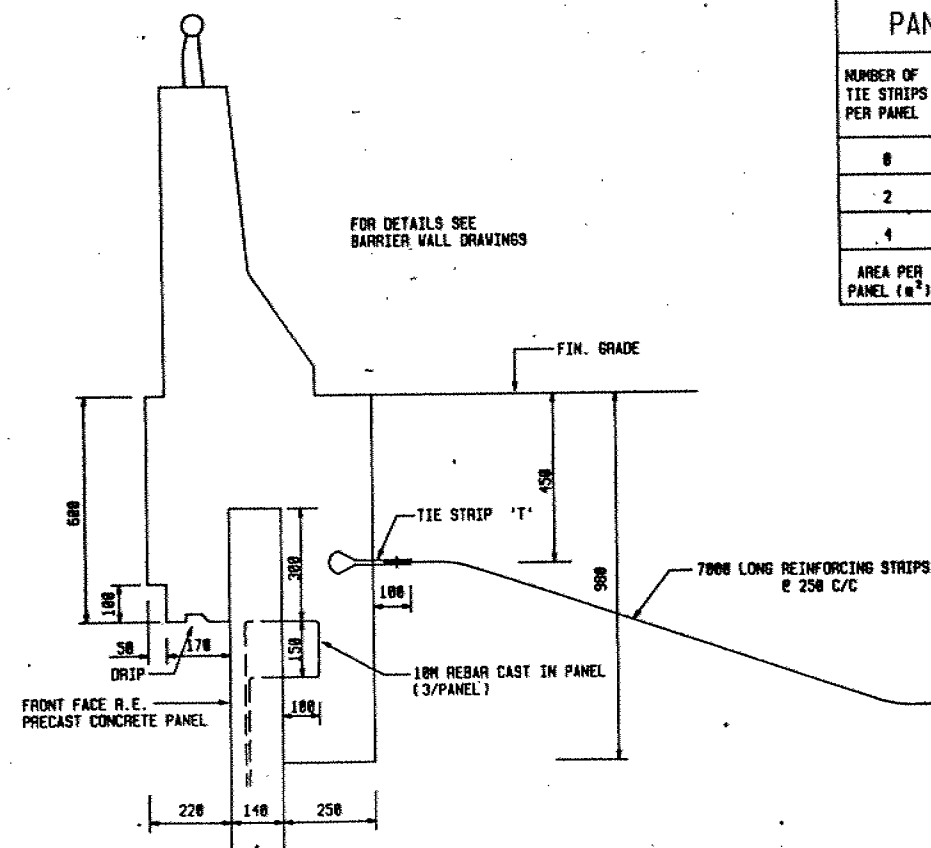
NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETRES; ELEVATIONS ARE IN METRES.
2. ITEMS SUPPLIED BY 'REINFORCED EARTH COMPANY LTD.':
 - i) PRECAST CONCRETE PANELS AND ALL EMBEDDED HARDWARE.
 - ii) GALVANIZED REINFORCING STRIPS AND BOLTS.
 - iii) JOINT FILLER MATERIAL (ie: RUBBER PADS AND FILTER FABRIC).
 - iv) 'TERRATREL' FACING ELEMENTS
 - v) GALVANIZED TIE STRIPS FOR USE IN 'TERRATREL' WALL AND CAST IN PLACE BARRIER WALL.
3. TERRATREL WALL AREA = 226,533 m²

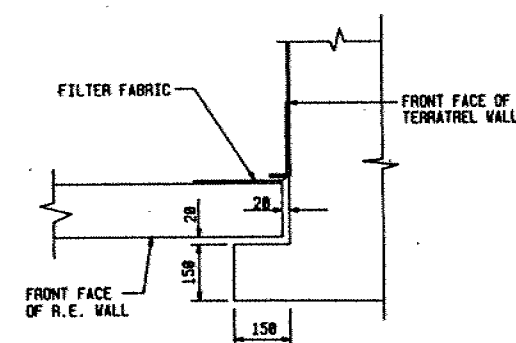


PANEL SCHEDULE				TOTAL AREA = 59.814 m ²						
NUMBER OF TIE STRIPS PER PANEL	PANEL TYPE									
	A	B	CK	EX	FX	KX	LX	AR	DXL	GKR
			1						1	
		5		1	2					1
	13					2	1	4		
AREA PER PANEL (m ²)	2.250	1.125	0.817	1.388	1.658	2.585	2.782	2.025	0.986	1.74

REINFORCING STRIP SCHEDULE						
SIZE (mm x mm)	LENGTH (mm)					
	1000	5000	6000	7000		
30 x 5	62	36	552	54		



C.I.P. BARRIER WALL DETAIL

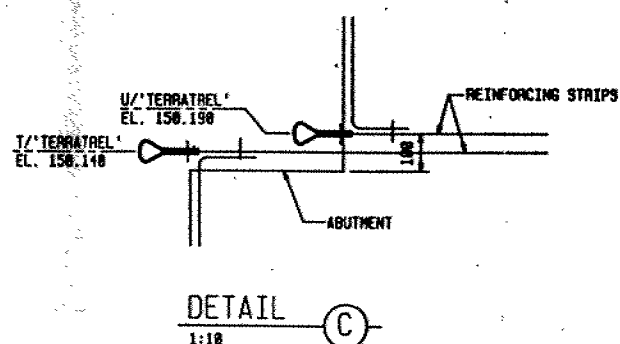
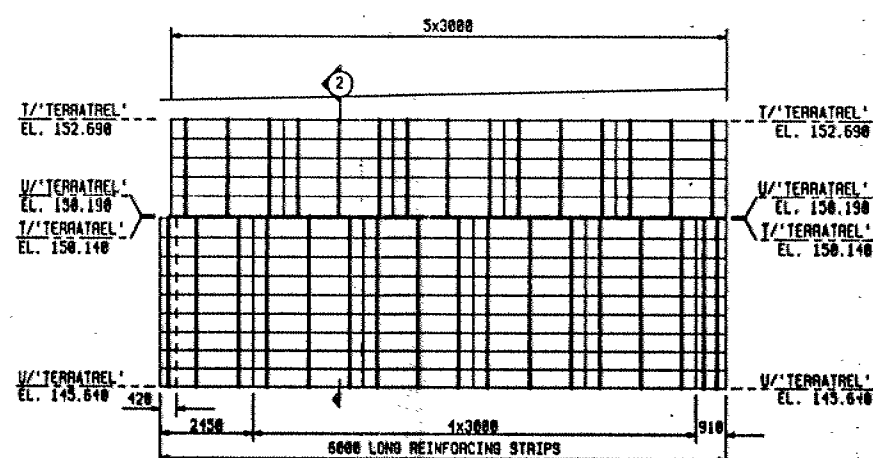
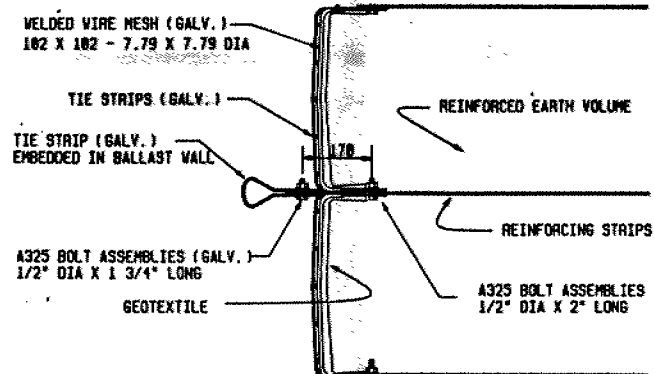
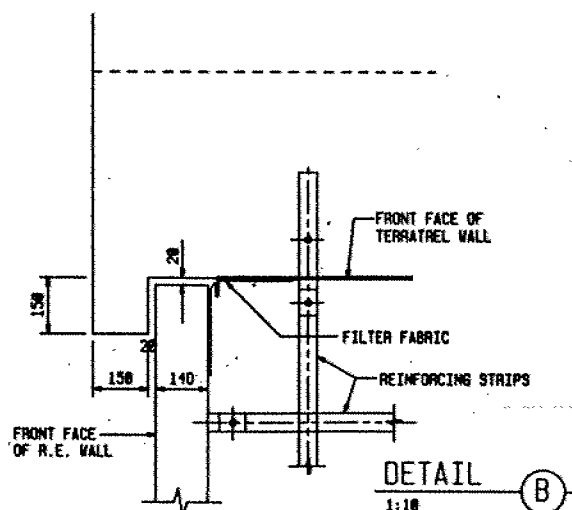
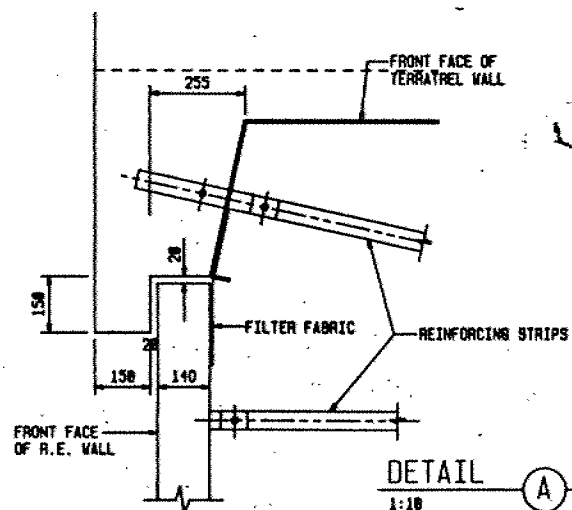
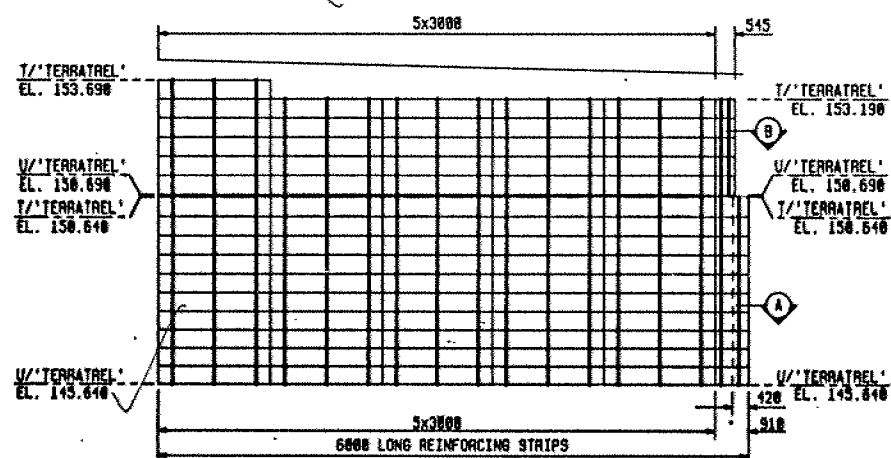
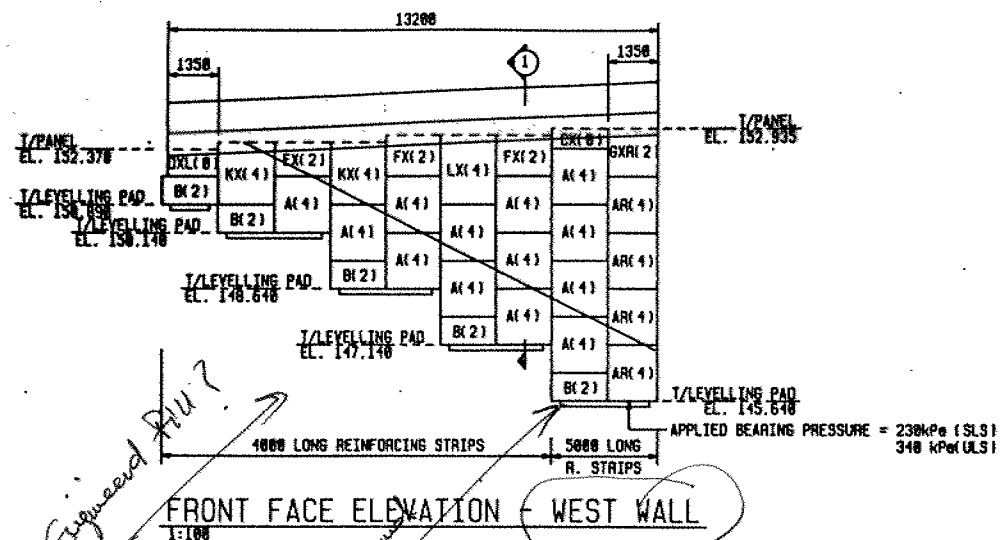


DETAIL A

THIS DRAWING CONTAINS INFORMATION PROPRIETARY TO REINFORCED EARTH COMPANY LTD. AND IS FURNISHED EXCLUSIVELY FOR THE USE OF WTO IN CONNECTION WITH THIS PROJECT. EXCEPT AS SPECIFICALLY AUTHORIZED IN WRITING BY REINFORCED EARTH COMPANY LTD., POSSESSION OF THIS DRAWING DOES NOT AUTHORIZE USE OF ITS CONTENTS FOR OTHER THAN THE EXPRESS PURPOSE FOR WHICH IT HAS BEEN PROVIDED NOR TRANSMISSION OF THIS DRAWING OR ITS CONTENTS TO THIRD PARTIES.

DRAWING NOT TO BE SCALED
100 ** ON ORIGINAL DRAWING

REVISIONS								
				DESCRIPTION				
DESIGN	PT	CHK	BB	CODE 090C-83	LOAD CLASS 'A'	DATE	MAR. 97	
DRAWN	PK	CHK	PK	SITE 37-1013	STRUCT	SCHEME	QWG 8	
RECD NO. 91839-6								



THIS DRAWING CONTAINS INFORMATION PROPRIETARY TO REINFORCED EARTH COMPANY LTD. AND IS FURNISHED EXCLUSIVELY FOR THE USE OF MTO IN CONNECTION WITH THIS PROJECT. EXCEPT AS SPECIFICALLY AUTHORIZED IN WRITING BY REINFORCED EARTH COMPANY LTD., POSSESSION OF THIS DRAWING DOES NOT AUTHORIZE USE OF ITS CONTENTS FOR OTHER THAN THE EXPRESS PURPOSE FOR WHICH IT HAS BEEN PROVIDED NOR TRANSMISSION OF THIS DRAWING OR ITS CONTENTS TO THIRD PARTIES.

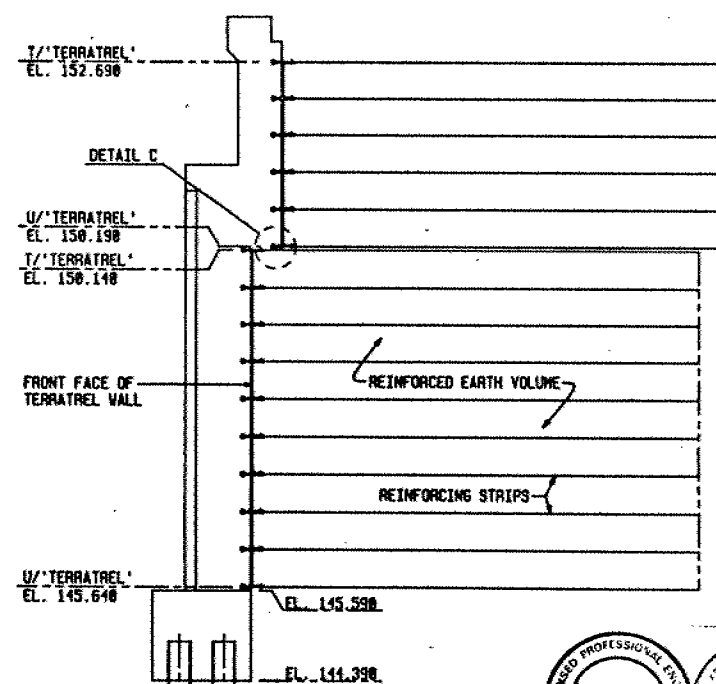
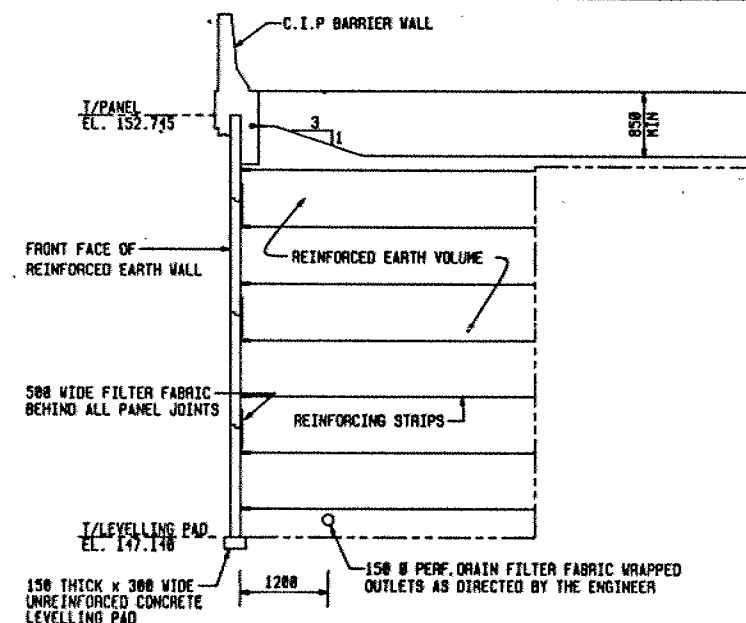
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 131-85-04

SHEPPARD AVENUE STRUCTURE
OVER BARTON ROAD
REINFORCED EARTH RETAINING WALL
ELEVATIONS, SECTIONS & DETAILS

REINFORCED EARTH COMPANY LTD.
190 ATTWELL DRIVE, SUITE 501
REXDALE, ONTARIO M9V 6H8

SHEET



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

SECTION 2
1:50

REVISIONS	DESCRIPTION
DESIGN PY	CHK BK
DRAWN MK	CHK MK
CODE 0400-83	LOAD CLASS 'A' DATE MAR 92
SITE 37-1819	STRUCT
SCHEME	DWG 9
REC'D NO. 911895-7	

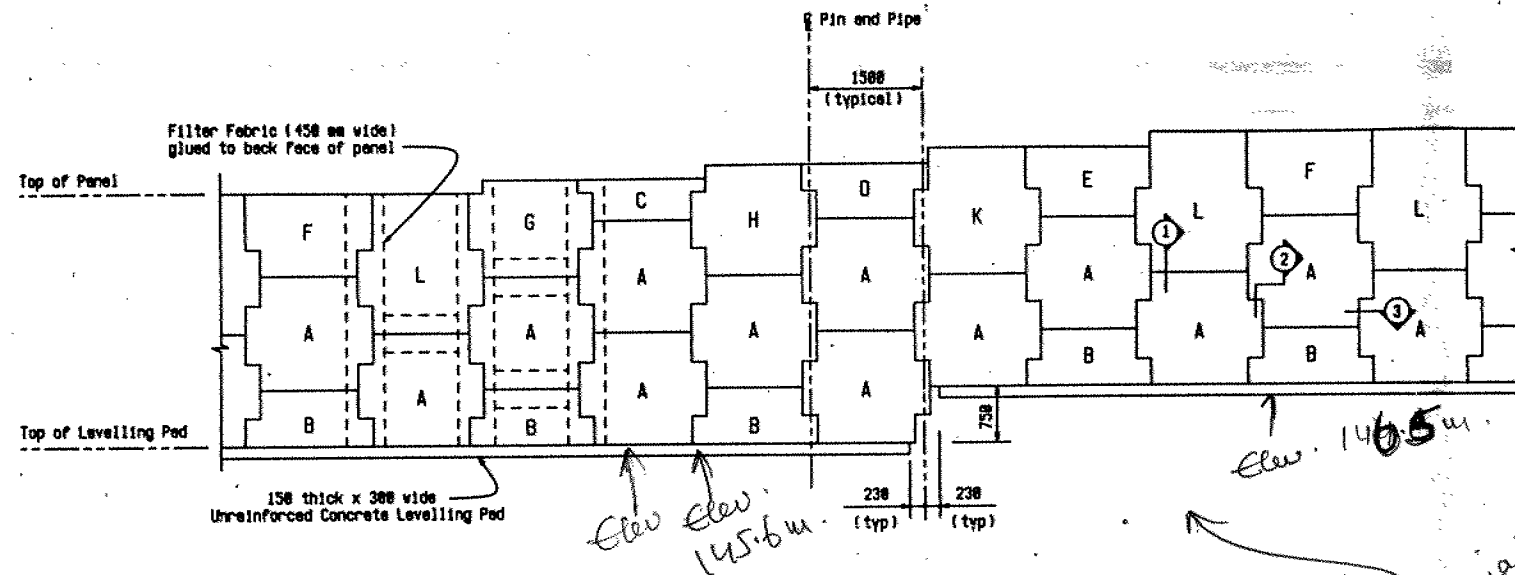
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 131-85-04

SHEPPARD AVENUE STRUCTURE
OVER BARTON RD.
REINFORCED EARTH RETAINING WALL
TYPICAL DETAILS

SHEET

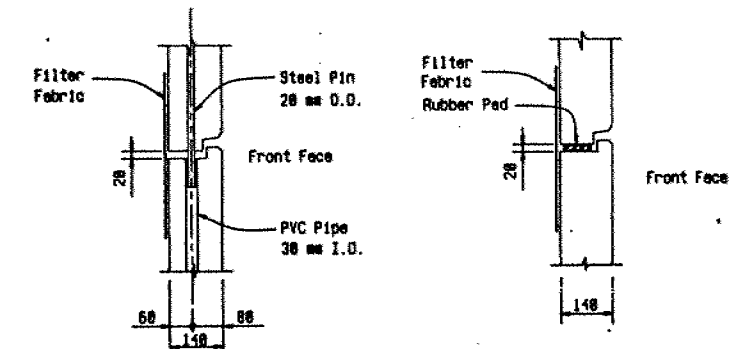
REINFORCED EARTH COMPANY LTD.
190 ATTWELL DRIVE, SUITE 501
REXDALE, ONTARIO M9W 6H8



Panel Type	A	B	C	D	E	F	G	H	K	L
Effective Height (mm)	1500	750	545	730	920	1185	1295	1480	1670	1855

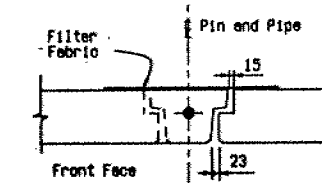
Typical Elevation (Front Face)

1 : 50

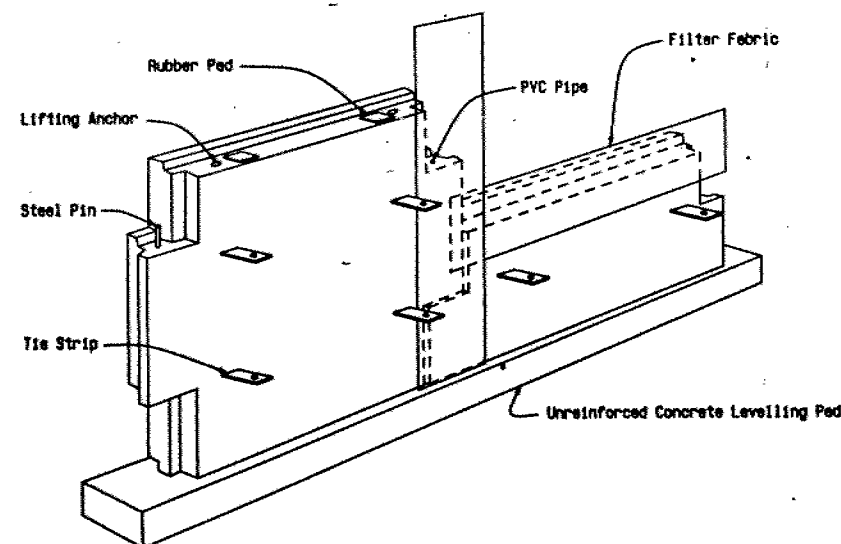


Section 2

Section 1

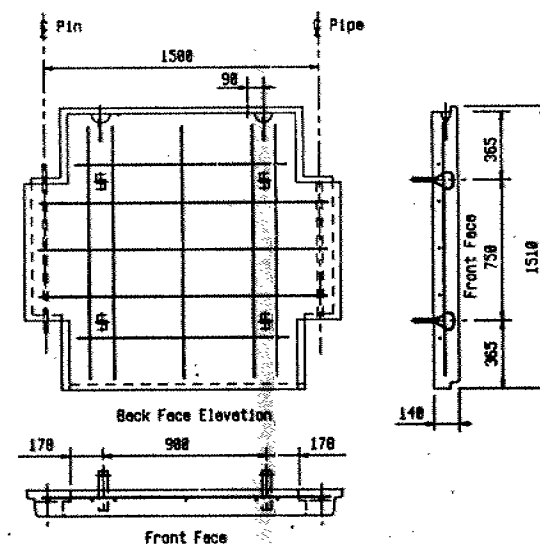


Section 3



Typical General Arrangement

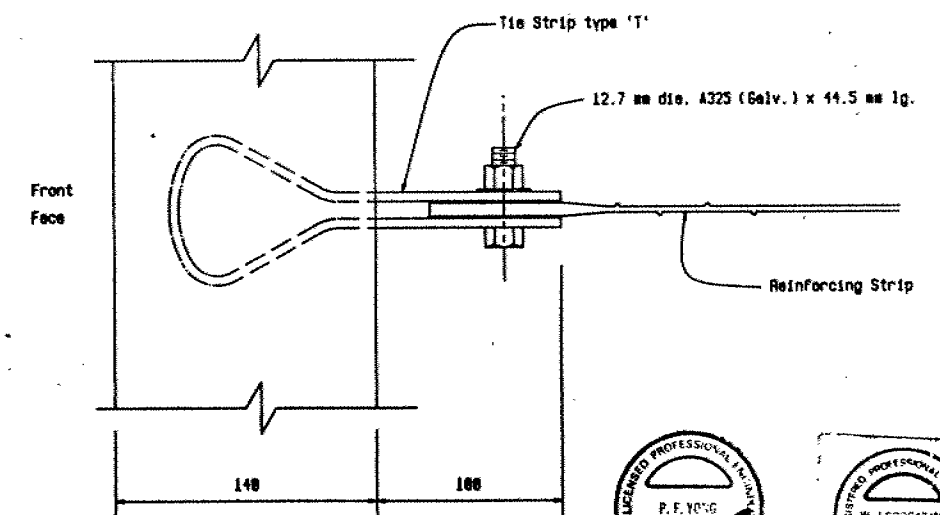
Not to scale



Typical Panel

1 : 20

- Notes:
- Concrete compressive strength (f'_c) shall be 38 MPa at 28 days.
 - Cover to reinforcement shall be 38 mm min.
 - All reinforcing bars shown are 10M, $f_y = 480$ MPa, conforming to CSA 630.12-M1977 (EPOXY COATED).
 - Clearance between reinforcing bars and galvanized tie strips shall be 38 mm minimum.



Tie Strip / Connection Detail

1 : 2

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DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DESCRIPTION
DESIGN BY	CHK
DRAWN BY	CHK
CODE	080C-03
LOAD CLASS	'A' DATE MAR. 92
SITE	37-1019
STRUCT	SCHEME
DMG	IO
REC'D NO.	911899-8