DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 30M12-87

DIST. 6 REGION

W.P. No. 127-66-12

CONT. No. 79-77

W. O. No.

STR. SITE No. 24-319

HWY. No. 410/403

LOCATION Prop Str. #37 at Crossing of Eglinton Ave

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

EGLINTON AVE. UNDERPASS
BRIDGE 37

GENERAL ARRANGEMENT

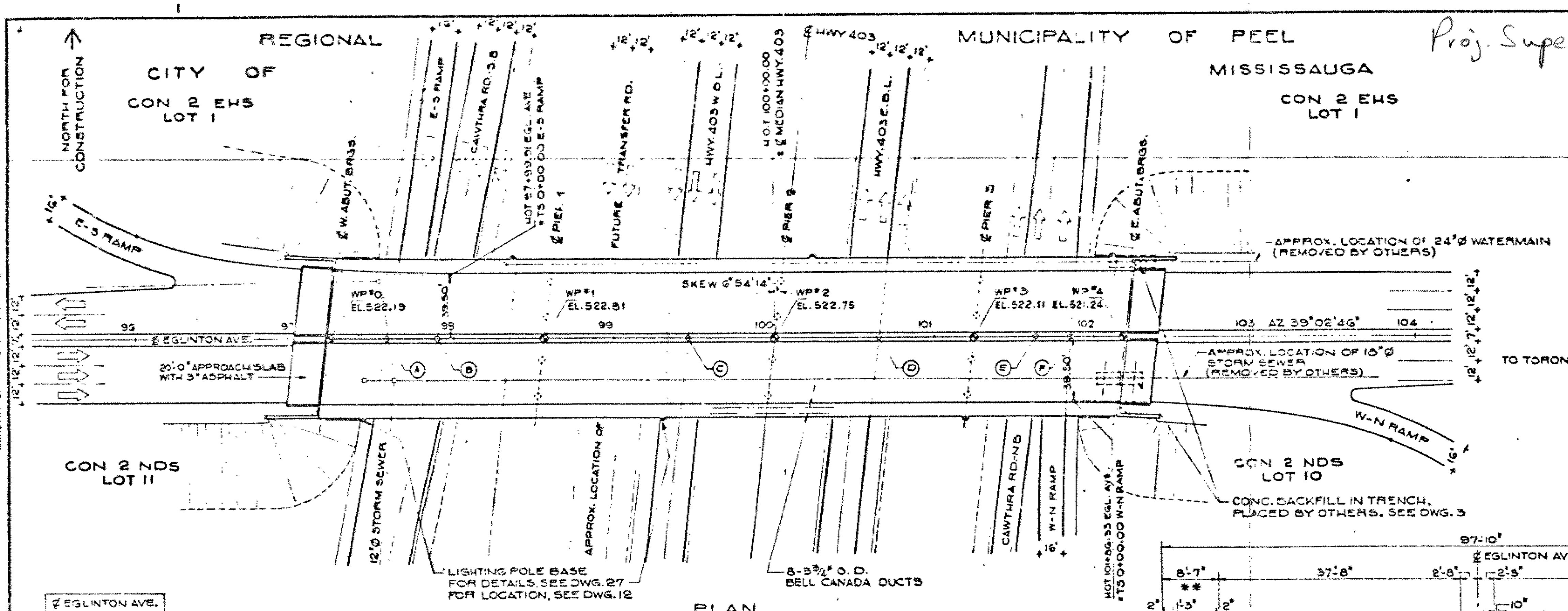
DISTRICT 6
CONT No 79-77
WP No 127-66-12

SHEET

CC PARKER & ASSOCIATES LTD.
CONSULTING ENGINEERS — HAMILTON

Proj. Super. Geary Leon
453-0936
275-6388

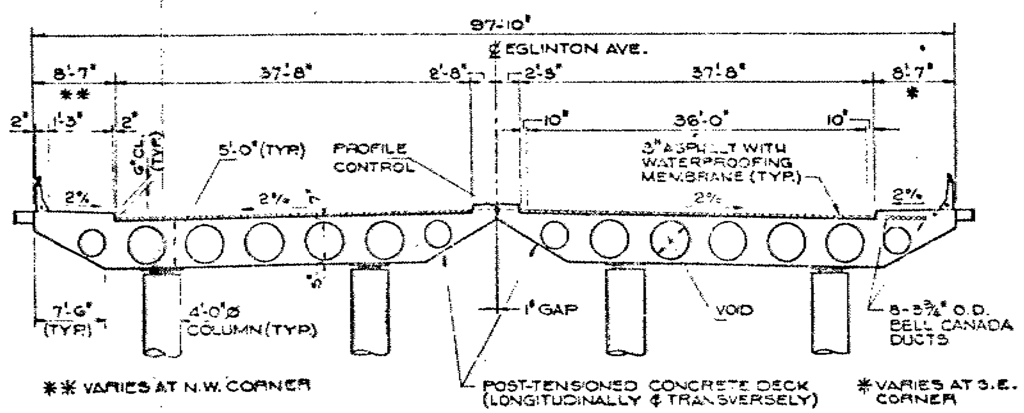
REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA



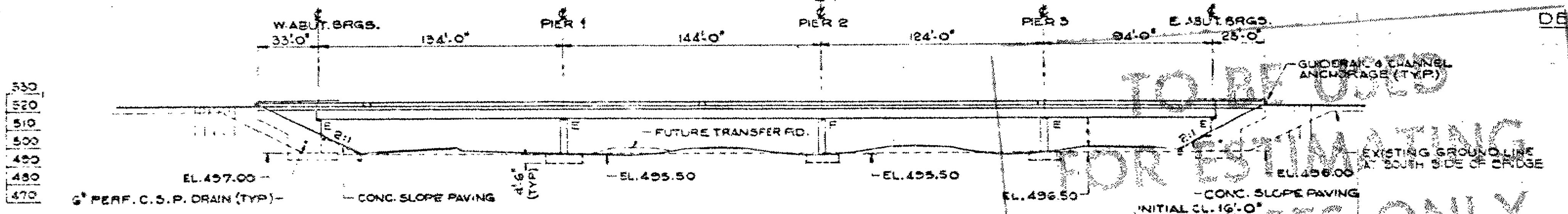
PLAN
1" = 40'-0"

NOTE
℄ PIERS AND ABUTMENTS ARE PARALLEL TO HWY. 403

℄ EGLINTON AVE.		
(A)	HOT 97+37.75	POC 12+71.05 E-S RAMP
(B)	HOT 97+89.66	POC 52+57.89 CAWTHRA RD.-SB
(C)	HOT 99+46.61	HOT 1291+41.65 HWY. 403-WBL
(D)	HOT 100+25.48	HOT 52+23.99 HWY. 403-EBL
(E)	HOT 101+22.15	HOT 53+05.70 CAWTHRA RD.-NB
(F)	HOT 101+84.36	POS 12+79.83 W-N RAMP



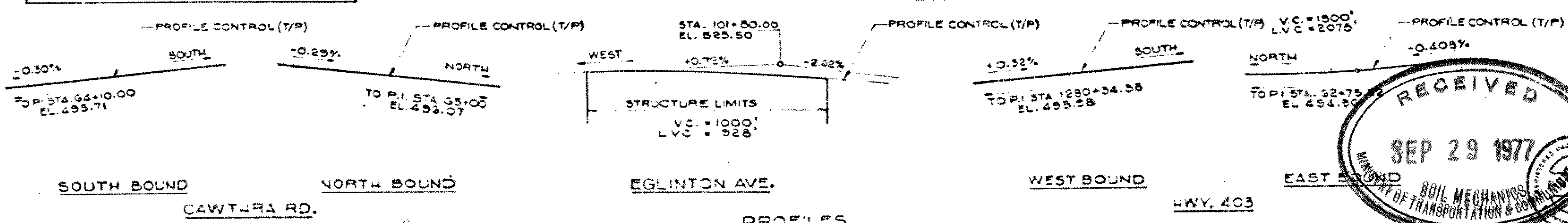
DECK SECTION
1" = 10'-0"



TO BE USED FOR ESTIMATING PURPOSES ONLY
DATE SEP 26 1977

SOUTH SIDE ELEVATION
1" = 40'-0"

PROPERTIES OF 3°54' 14" SKEW	
SINE	.12020422
COSINE	.99274919
TANGENT	.12108216



PROFILES
N.T.S.

RECEIVED
SEP 29 1977
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

GENERAL NOTES:

CLASS OF CONCRETE
COLUMNS, DECKS & BARRIER WALLS - 5,000 P.S.I.
FOOTINGS, ABUTMENTS & WING WALLS - 4,000 P.S.I.
REMAINDER - 3,000 P.S.I.

REINFORCING STEEL
GRADE OF STEEL - C.S.A. STD. G30, GRADE 50 MIN.

REINFORCING STEEL CLEAR COVER

FOOTINGS	ABUTMENTS	COLUMNS	DECK
3"	3"	2"	2" TOP 1½" BOT.

BARRIER WALLS	APPROACH SLABS
AS NOTED	2"

TO ACHIEVE THE MIN. CLEAR COVER OF 2" SPECIFIED THE TOP LAYER OF DECK REINF. SHALL BE PLACED PRIOR TO CONCRETING, WITH A CLEAR COVER OF 2½" ± ½" TOLERANCE.

CONSTRUCTION NOTES
CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± ⅛".
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.
FORMWORK BETWEEN THE END OF THE DECK AND BALLAST WALL AND UNDERSIDE OF DECK AND BEARING SEAT (E.G. EXPANDED POLYSTYRENE) SHALL BE REMOVED.

CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE CONCRETE LUMP SUM TENDER ITEMS.

CONCRETE QUANTITIES	CU. YDS.
PIERS, ABUTMENTS AND WING WALLS	917
DECK	5550
BARRIER WALLS	84
APPROACH SLABS	100
SLOPE PAVING	75

GEODETIC BENCH MARK
TOPO. 616 - EL. 594.130 (1965)
TABLET SET IN THE CENTRE OF THE SOUTH SIDEWALK OF THE SECOND LINE EAST OVERPASS OVER HWY. 401, 2.2 FT FROM CURB AND 17.9 FT. SOUTHEAST OF THE ℄ OF THE SECOND LINE ROAD 92 FT. NORTH-WEST FROM THE SOUTH END OF THE SIDEWALK.

LIST OF DRAWINGS

1 GENERAL ARRANGEMENT
2 BORE HOLE LOCATIONS & SOIL STRATA
3 FOUNDATION LAYOUT
4 FOOTING REINFORCING
5 WEST ABUTMENT DETAILS
6 WEST ABUTMENT REINFORCING
7 WEST ABUTMENT WING WALLS
8 EAST ABUTMENT DETAILS
9 EAST ABUTMENT REINFORCING
10 EAST ABUTMENT WING WALLS
11 PIER DETAILS
12 DECK LAYOUT
13 DECK DETAILS
14 LONGITUDINAL CABLE DETAILS I
15 LONGITUDINAL CABLE DETAILS II
16 TRANSVERSE CABLE DETAILS I
17 TRANSVERSE CABLE DETAILS II
18 TRANSVERSE CABLE DETAILS III
19 DECK REINFORCING WEST ABUTMENT TO PIER 1
20 DECK REINFORCING PIER 1 TO PIER 2
21 DECK REINFORCING PIER 2 TO PIER 3
22 DECK REINFORCING PIER 3 TO EAST ABUTMENT
23 DECK REINFORCING ABUTMENT BEAMS
24 DECK REINFORCING PIER 1 BEAM
25 DECK REINFORCING PIER 2 AND 3 BEAMS
26 BARRIER WALL
27 STANDARD DETAILS I
28 STANDARD DETAILS II
29 STANDARD DETAILS III
30 STEEL RAILING (SINGLE TUBE)
31 20 FT APPROACH SLAB
32 DETAILS OF CONC. SLOPE PAVING
33 AS CONSTRUCTED ELEV & DIM.
34 ELECTRICAL EMBEDDED WORK I
35 ELECTRICAL EMBEDDED WORK II

FOR REDUCED PLAN
USE SCALE BELOW
3 INCHES ON ORIGINAL PLAN

REVISIONS DATE	BY	DESCRIPTION

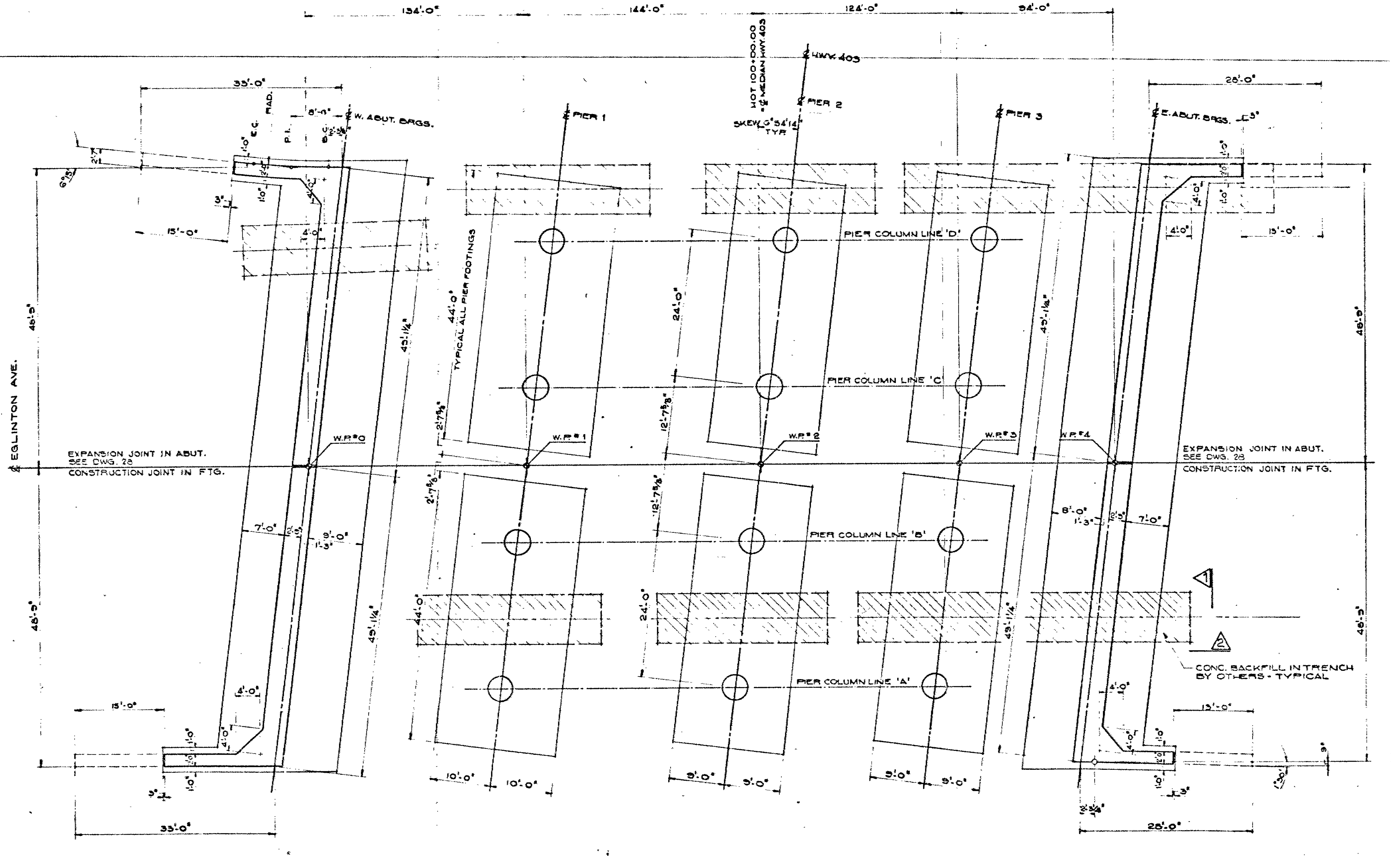
DESIGN C.M.A. CHECK S.M.A. LOAD. No. HS20-44 DATE 77.09.13
DRAWING G.I.N. CHECK S.M.A. SITE No. 24-39 DWG 1

DISTRICT 6
CONT No
WP No 127-66-12

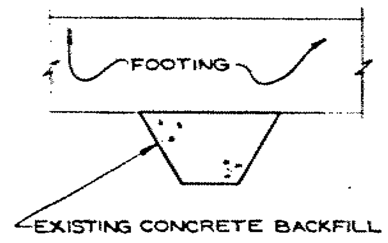
EGLINTON AVE. UNDERPASS
BRIDGE 37
FOUNDATION LAYOUT

SHEET

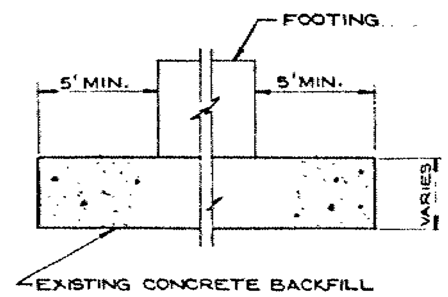
C.C. PARKER & ASSOCIATES LTD.
CONSULTING ENGINEERS — HAMILTON



FOUNDATION LAYOUT
1/8" = 1'-0"



1
NTS



2
NTS

POINT	STA.	CO-ORDINATES	
		NORTH	EAST
WP#0	97+22	849,801.712	963,676.964
WP#1	98+56	849,905.795	963,761.377
WP#2	100+00	850,017.645	963,852.089
WP#3	101+24	850,113.960	963,930.202
WP#4	102+18	850,186.974	963,989.417

FOR REDUCED PLAN
USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN

REVISIONS

DATE	BY	DESCRIPTION

DESIGN E.M.A. | CHECK E.M.A. | LOAD No HS20-44 | DATE 77-09-15
DRAWN No J.S.N. | CHECK E.M.A. | SITE No 24 3-9 | DWG 3

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ~~ONTARIO~~

30M12-87

MEMORANDUM

TO: Mr. G.C.E. Burkhardt, (3)
Regional Structural Planning Eng.,
Central Region,
3501 Dufferin St., Downsview.

ATTENTION:

OUR FILE REF.

FROM: Foundations Office,
Design Services Branch,
West Bldg., Downsview.

DATE: August 2, 1973.

IN REPLY TO AUG - 8 1973

SUBJECT:

CONT 79-77

FOUNDATION INVESTIGATION REPORT
For
The Proposed Structure No. 37
At the Crossing of
Eglinton Avenue (Base Line Road) and
Hwy. #410 & Hwy. #403, Site #24-319
Town of Mississauga, County of Peel
District #6 (Toronto)
W.O. 73-11032 -- W.P. 127-66-12

Attached we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above-mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

AGS/ao
Attch.

c.c. E. J. Orr
B. R. Davis
A. Rutka
R. S. Pillar
H. Greenland
B. J. Giroux
C. Mirza
G. A. Wrong
B. A. Singh

Foundations Files
Documents

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FOUNDATION INVESTIGATION REPORT
For
The Proposed Structure No. 37
At the Crossing of
Eglinton Avenue (Base Line Road) and
Hwy. #410 & Hwy. #403, Site #24-319
Town of Mississauga, County of Peel
District #6 (Toronto)
W.O. 73-11032 -- W.P. 127-66-12

1. *INTRODUCTION:*

The Foundations Office was requested to carry out a subsurface investigation at the site of the proposed Bridge #37 (Eglinton Avenue over Hwy. 403 and Hwy. 410) in the Town of Mississauga, County of Peel. The request was contained in a memo from Mr. G.C.E. Burkhardt, Regional Structural Planning Engineer, Central Region, dated May 7, 1973. Subsequently, an investigation was carried out by this Office to determine the subsoil and groundwater conditions in this area.

This report presents the factual information obtained from this investigation together with recommendations pertaining to the foundation design of the proposed structure and stability and settlement considerations associated with the approach embankments.

2. *DESCRIPTION OF SITE AND GEOLOGY:*

The site is located at the intersection of Eglinton Avenue (Base Line Road) and Cawthra Road, in the Town of Mississauga, County of Peel. The land is flat to gently undulating between elevations 492 and 502. In this area, the land is primarily developed for farming purposes with occasional light housing.

The site is located in the physiographic region known as the "Peel Plain." The characteristic deposit in the vicinity of the area under investigation, is composed of a cohesive glacial till whose thickness is quite variable. In this region, the Credit River, Oakville Creek and Etobicoke Creek have cut deep valleys into the overburden. There is, therefore, no large undrained depression, swamp or bog in this area, although in many of the interstream areas drainage is still imperfect. The overburden is underlain by shale bedrock of the Meaford-Dundas Formation, Ordovician Period.

3. *FIELD AND LABORATORY WORK:*

Ten sampled boreholes, each accompanied with a dynamic cone penetration test, were put down during the course of the field investigation. The borings were advanced by means of a continuous flight auger machine adapted for soil sampling purposes. In addition, the results of two boreholes put down during a previous investigation (W.O. 72-11053) were included due to their close proximity to the proposed structure site.

Samples of the overburden were obtained in a 2" O.D. split spoon sampler at required depths. The sampler was hammered into the soil with a driving energy of 350 ft.-lb. per blow in accordance with the specifications for the Standard Penetration Test. The same method was used to advance the dynamic cone penetration testing.

Groundwater level observations were carried out during the period of investigation, in the open boreholes. In addition, piezometers were installed at several boring locations where artesian water pressure was encountered. The location of the piezometers are shown on Drawing No. 73-11032A. The outflow of graoundwater from the boreholes were successfully sealed off except at B.H. #5.

The soil and groundwater conditions encountered at the boring locations are presented in the Record of Borehole sheets. The location and ground elevation of the various boreholes were surveyed in the field by construction personnel from District #6 (Toronto). The elevations in this report are referenced to a Geodetic Datum. The borehole locations (referenced to a co-ordinate system) and elevations, together with estimated stratigraphical sections, are shown on Drawing No. 73-11032A.

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

Natural Water Contents
Atterberg Limits
Grain-Size Distribution

The results of the laboratory testing were presented on the Record of Borehole sheets as well as summarized on Figures No. 1 to 3, inclusive, all of which are contained in the Appendix of this report.

4. *SUBSOIL CONDITIONS:*

4.1) *General:*

The predominant stratum across the site is composed of a heterogeneous mixture of hard clayey silt, sand and gravel (glacial till). The thickness of this deposit varies from 12 to 34 ft. The cohesive glacial till is underlain by a granular deposit of silty sand with gravel, which was not fully penetrated at any of the boring locations. At certain locations, the glacial till is overlain by fill material up to 6 feet thick.

The boundaries of the various deposits, as determined in the boreholes, are shown on the accompanying Record of Borehole sheets. The stratigraphical sections shown on Drawing No. 73-11032A have been inferred from this data. From ground surface downward, the various soil types encountered are as follows:

4.2) Fill Material:

Two boreholes (B.H.'s #1 and #10) were put down close to the existing roadway. At these locations, up to 6 feet of fill material was encountered. The fill material is composed of a mixture of clayey silt, sand and gravel, which is similar in composition to the underlying glacial till.

Standard Penetration testing carried out within the fill material gave 'N' values ranging from 7 to 21 blows per foot. These values would indicate that the fill material has been subjected to a moderate degree of compaction.

4.3) Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till):

This is the predominant stratum across the site. It is composed of a heterogeneous mixture of clayey silt, sand and gravel (glacial till). The thickness of this deposit varies from 12 feet (B.H. #5) to 34 feet (B.H. #10). Occasional silty sand layers up to 4 feet thick were found within this deposit.

Grain-size distribution curves, for samples of this cohesive deposit, are shown on Figure No. 2 in the Appendix. Atterberg Limit tests were also performed on samples of the glacial till. The results, which are shown on the Record of Borehole sheets and on the Plasticity Chart (Figure No. 1), are tabulated below:

	Range	*(Average)*
Liquid Limit (W_L) %	18 - 29	(23)
Plastic Limit (W_p) %	12 - 21	(15)
Natural Moisture Content (W) %	7 - 19	(11)

Based on the above values, it may be concluded that the glacial till has a matrix, which is inorganic and of low plasticity.

The results of Standard Penetration Tests, carried out within the glacial till, are plotted on the Record of Borehole sheets and Drawing No. 73-11032A. The testing gave 'N' values generally ranging from 21 blows per foot to 100 blows for 3 inches. In several localized zones, 'N' values are as low as

8 blows/foot.

It is estimated that the consistency of the glacial till generally varies from very stiff to hard. Localized firm to stiff pockets are encountered within this deposit at B.H.'s #4 and #10.

4.4) *Silty Sand with Gravel:*

This granular deposit was found underlying the glacial till stratum. It consists of silty sand with gravel. This stratum was not fully penetrated at any of the boring locations. Standard penetration testing was carried out within this granular deposit. The results gave 'N' values ranging from 10 blows per foot to 100 blows for 4 inches, with the higher values being dominant. The relative density of this deposit is, therefore, from compact to very dense.

5. *GROUNDWATER CONDITIONS:*

An artesian groundwater pressure head was encountered at most of the boreholes. This artesian condition was encountered once the borings penetrated through the cohesive glacial till stratum down into the lower granular deposit. Once this occurred, water rapidly filled the borehole and in most cases, water continuously flowed out of the borehole. The water flowed out of the boreholes was clear, indicating no loss of fines in the aquifer. In order to establish the groundwater table and to observe the variations in artesian pressure head at various depths, a total of five piezometers were installed at B.H.'s #2, #4 and #5. The location and relative depth of the piezometers are shown on Drawing No. 73-11032A.

Water level readings within open boreholes had been taken before the artesian condition was encountered. The results, which are plotted on the individual Record of Borehole sheets as well as on Drawing No. 73-11032A, indicate that a perched groundwater table exists within the cohesive glacial till. It was found varying in elevations from 480 to 502.

....6

Piezometric water levels were observed periodically. These observations indicate that the artesian pressure head was found to be as much as 23 feet above the ground surface, corresponding to an elevation of 522. The results of the observations are also plotted on Figure No. 4. Referring to Figure No. 4, it can be seen that the excess head due to artesian condition within the granular deposit is more or less constant. Further, the artesian head is not confined solely to the granular deposit, instead it dissipates throughout the cohesive glacial till stratum as well.

6. *DISCUSSIONS AND RECOMMENDATIONS:*

6.1) *General:*

This report will deal with the Proposed Bridge No. 37 (Eglinton Avenue over Hwy. 403 and Hwy. 410). This structure is to have five spans (68'-111'-144'-125'-116'). In the initial stage, the bridge is 103 feet wide. The ultimate scheme requires that it be widened to 192 feet. The proposed profile grade of Eglinton Avenue in the vicinity of the structure will vary from elevation 529 to elevation 530, while that of Hwy. 410 and Hwy. 403 is to vary between elevations 500 and 506. To reach these grades, approach embankments of 37 feet and 29 feet high in the transverse and longitudinal directions, respectively, will be necessary.

The subsoil consists of a 12 to 34 feet thick cohesive glacial till underlain by an extensive deposit of silty sand with gravel, which was found to be the primary source of artesian water.

In the subsections to follow the foundation support for the proposed structure together with stability and settlement considerations associated with the approach fills will be discussed.

6.2) *Foundations:*

....7

6.2.1) Piers:

The subsoil is competent, therefore, it is recommended that the piers be supported on spread footings located within the hard parent glacial till deposit. In order to fulfill the frost protection requirements, the underside of the footings should be at least 4 feet below the finished grade. Taking all these into consideration, the recommended founding elevations for the piers are as follows:

Location	*Station*	*Recommended Founding Elevation*	*Refer to B.H.'s*
Pier #1	97+45	497	#2 & #8
Pier #2	98+56	496	#3 & #9
Pier #3	100+00	491	#4 & #10
Pier #4	101+25	492	#5 & #11

An allowable bearing value of up to 3 t.s.f. may be used in designing the footings, founded as recommended. In computing the lateral resistance of the footings, an adhesion value of 2,000 p.s.f. may be used between the rough concrete surface and glacial till.

The excavations for the pier footings will be carried out within the cohesive glacial till. In view of the relatively impervious nature of the glacial till, no major dewatering problems are anticipated. As mentioned elsewhere in this report, thin silty sand layers are present within this cohesive deposit. Once these granular layers are intercepted, excess water seepage into the excavations can be expected. However, it is believed that this could be handled by employing ordinary pumping methods.

The foundation subsoil will settle due to the imposed foundation loading. The subsoil is composed of a competent cohesive glacial till, thus the settlement will be of a recompression nature. For a spread footing foundation of the size contemplated, imposing the aforementioned pressure, it is estimated that the settlement should not exceed one half of an inch, provided the subsoil is not softened by groundwater seepage or uncontrolled surface runoff. It may be advantageous to protect the cohesive glacial till, at the founding level, by covering it with a lean

.....8

concrete working slab immediately after the completion of the excavation.

6.2.2) Abutments:

The abutments for this structure may be perched within the approach fills. The presence of artesian condition precludes the use of end-bearing piles driven to the very dense granular stratum to support the abutments, since the piles may penetrate and disturb the artesian zone in the granular subsoil and consequently endanger the stability of the overall structure complex. In view of this, it is recommended that the abutments be supported on spread footings or short piles terminated within the cohesive glacial till. These two alternatives are discussed separately as follows:

i) The abutments may be supported on spread footings perched within the approach fills. The material, below the tops of the footings, should consist of well compacted Granular 'A' and should extend to a horizontal distance of at least 10 feet from the footing edges in the plane of the footing tops. This portion of the fill should be constructed with side slopes no steeper than 1:1. The remainder of the fill should be completed to about profile grade for a distance of about 50 feet behind the abutments before re-excavating for the abutment footings. An allowable bearing value of 2.5 t.s.f. may be used in footing design.

ii) The abutment footings may be perched within the approach fills and supported on short piles driven to a level at least 10 feet above the upper boundary of the granular deposit to ensure that the artesian zone is not disturbed. The piles should be terminated at the elevations given below:

Location		*Tip Elevation*	*Refer to*
West Abutment	North	485	#1
	South	490	#7
East Abutment		485	#6 & #12

12-3/4" O.D. tubular piles driven to the above elevations may be designed for a safe load of 30 tons/pile. No rock or bouldery fill should be placed in areas where piles are to be driven.

6.3) *Approach Embankments:*

As mentioned elsewhere in this report, the maximum height of the approaches will be of the order of 37 feet. Subsoil conditions are generally favourable and consequently no deep-seated rotational type of failure is anticipated provided standard 2:1 slopes are adopted.

The fill itself and the natural subsoil will settle. The magnitude of this combined settlement is estimated to be of the order of 2 inches. The majority portion of this settlement, which occur within the fill itself, should take place within two months following the placement and compaction of the fills. In view of this, it is recommended that the embankments should be constructed and left in place for a period of two months prior to the construction of the structure, if spread footings are chosen to support the abutments. If the above-mentioned scheme is adopted, the differential settlement between the spread footing supported abutment and adjacent pier should not exceed 1/2 inch.

7. *MISCELLANEOUS:*

The field work was carried out between June 1 and June 12, 1973, under the supervision of Mr. V. Korlu, Project Foundations Engineer.

The drilling equipment used was owned and operated by Canadian Longyear Co. Ltd., Toronto.

.....10

This report was prepared by Mr. C. S. Poon, Project Foundations Engineer, and reviewed by Mr. M. Devata, Supervising Foundations Engineer.

REGISTERED PROFESSIONAL ENGINEER
C. S. L. POON
PROVINCE OF ONTARIO

C. S. Poon, P. Eng.

M. Devata, P. Eng.

CSP/ao
August 2, 1973.

APPENDIX I

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS – ONTARIO

DESIGN SERVICES BRANCH FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 1 (B.H. No. 8, 72-11053)

JOB 73-11032 LOCATION Co-ords. 15,849,750N 963,600E ORIGINATED BY V.K.

W.P. 127-66-12 BORING DATE April 18, 1972 COMPILED BY C.S.P.

DATUM Geodetic BOREHOLE TYPE Auger and Dynamic Penetration Test CHECKED BY

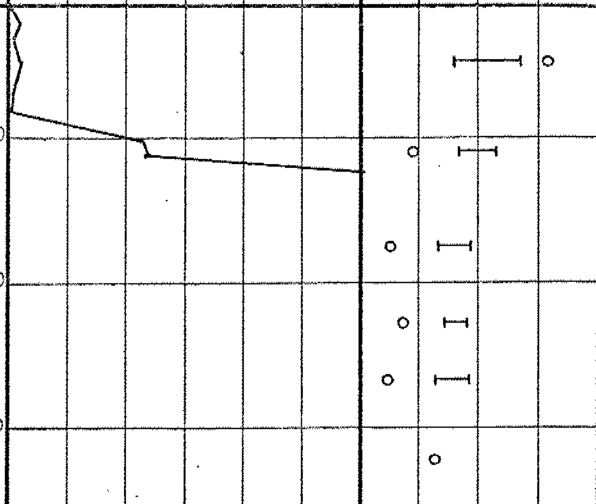
ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS (GR. SA. SI. CL.)
499.0	Ground level					516.0 ▼ / 508.4 ▼
0.0	Fill material (Clayey silt with sand, gravel)					
494.0	Firm	1	SS	7		
5.0	Brown / Grey	2	SS	68		P1(1) 3 27 59 11
	Heterogeneous mixture of clayey silt with sand and gravel	3	SS	88	490	Tip El. 491.0
	(Glacial till)	4	SS	65		P1(2) 21 31 34 4
	Hard	5	SS	62		Tip El. 483.0
		6	SS	110	480	
		7	SS	100/3"		
474.0		8	SS	100/3"		17 41 33 9
25.0	Sandy silt to silty sand with gravel, trace of clay				470	
		9	SS	40		P1(3) 4 39 40 17
	Dense to very dense	10	SS	100/6"		Tip El. 467.0
		11	SS	23	460	
		12	SS	100/6"	450	
	With boulders below elevation 439	13	SS	100/4"	440	
					430	
					420	
					410	
	(To be continued)				400	
397.0 / 102.0						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W — WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.



20 / 15 -5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE №1

JOB 73-11032
W.P. 127-66-12
DATUM Geodetic

LOCATION
BORING DATE
BOREHOLE TYPE

ORIGINATED BY V.K.
COMPILED BY C.S.P.
CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT W_L; PLASTIC LIMIT W_P; WATER CONTENT W; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT					
397.0 102.0										
						390				
						380				
374.0 125.0	End of Borehole									
						370				

OFFICE REPORT ON SOIL EXPLORATION

15 20 5 10 % STRAIN AT FAILURE

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE № 2

JOB 73-11032
W.P. 127-66-12
DATUM Geodetic

LOCATION Co-ords. 15,849,859N 963,658E
BORING DATE June 1, 1973
BOREHOLE TYPE Auger and Sample with C.M.E.

ORIGINATED BY V.K.
COMPILED BY V.K.
CHECKED BY [signature]

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS
499.1	Ground level					W.L. Elev 522.3 HEAD; GR-SA-SI-CL
0.0	Heterogeneous mixture of clayey silt, sand and gravel	1	SS	42		4-31-56-9
	Brown	2	SS	74		P2(3) TIP ELEV. 489.1
		3	SS	45	490	W.L. ELEV. 488.1
	Grey	4	SS	47		
	(Glacial till)	5	SS	71		12-33-40-15
	(Very stiff to hard)	6	SS	66	480	P2(2) TIP ELEV. 479.1
		7	SS	85		
		8	SS	25		P2(1) TIP ELEV. 471.1
		9	SS	138/10"	470	9-34-43-14
466.1 / 33.0	Silty sand and some gravel (Compact)	10	SS	12		ARTESIAN WATER ELEV. 466.1 ENCOUNTERED
462.6 / 36.5	End of Borehole				460	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100

SHEAR STRENGTH P.S.F.: o UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE

LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W

WATER CONTENT %: 10 20 30

BULK DENSITY γ P.C.F.

20 / 15 – 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

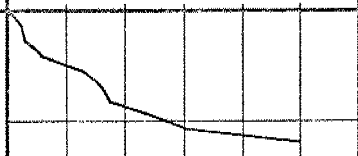
RECORD OF BOREHOLE Nº 3

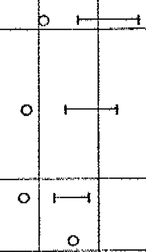
JOB 73-11032 LOCATION Co-ords. 15,849,943N 963,725E ORIGINATED BY V.K.

W.P. 127-66-12 BORING DATE June 1, 1973 COMPILED BY V.K.

DATUM Geodetic BOREHOLE TYPE Auger and Sample with C.M.E. CHECKED BY [illegible]

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS Head GR SA.SI.CL.
497.6	Ground level									
0.0	Heterogeneous mixture of clayey silt, sand and gravel Brown		1	SS	29					
			2	SS	65	490				
	Grey		3	SS	82					W.L. Elev. 486.1 6-32-43-19
	(Glacial till)		4	SS	120/9"					
	Hard		5	SS	96	480				13-32-39-1?
476.6			6	SS	130					Artesian water Encountered Elev.476.1
21.0 / 475.1	Silt, sand, some gravel		7	SS	43					11-51-46-2
22.5	End of Borehole					470				





20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 4

JOB 73-11032	LOCATION Co-ords. 15,850,054N 963,818E		ORIGINATED BY V.K.
W.P. 127-66-12	BORING DATE June 4, 1973		COMPILED BY V.K.
DATUM Geodetic	BOREHOLE TYPE Auger and Sample with C.M.E.		CHECKED BY

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS (GR.SA.SI.CL.)
494.9	Ground level					Elev. 501.8
0.0	Heterogeneous mixture of clayey silt, sand and gravel (Glacial till) (Very stiff to hard) Brown	1	SS	31	490	
		2	SS	98		23-24-37-1?
	Grey	3	SS	23		WL Elev. 482.9 Head
		4	SS	29		6-44-40-8
		5	SS	21	480	13-35-42-1?
	(With occasional silty sand layers)	6	SS	8		P4 Tip Elev. 476.9
		7	SS	21		Artesian Water Encountered Elev. 474.9 6-10-75-9
		8	SS	28	470	
		9	SS	111		7-26-44-23
461.9 / 33.0	Silty sand with gravel Grey Dense				460	
454.4		10	SS	49		24-44-32
40.5	End of Borehole				450	

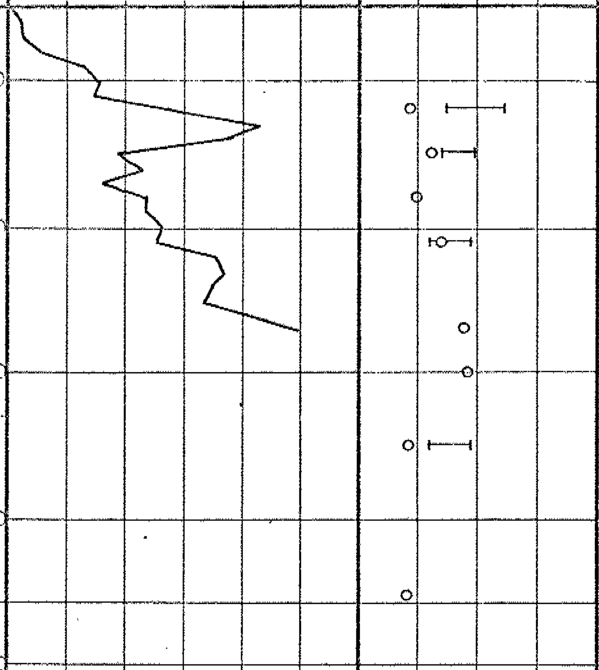
DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.



20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE № 5

JOB 73-11032 — LOCATION Co-ords. 15,850,152N 963,895E — ORIGINATED BY V.K.

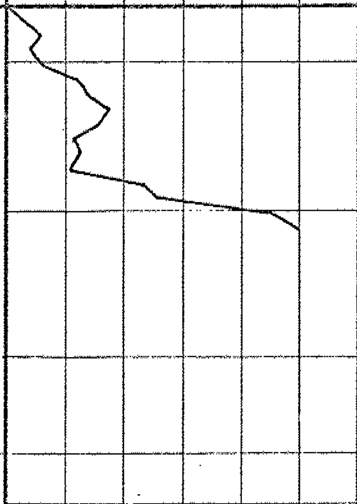
W.P. 127-66-12 — BORING DATE June 5, 1973 — COMPILED BY V.K.

DATUM Geodetic — BOREHOLE TYPE Auger and Sample with C.M.E. — CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS GR.SA.SI.CL.
493.8	Ground level					WL Elev. 507.6 Head
0.0	Heterogeneous mixture of clayey silt, sand and gravel (Glacial till) Very stiff to hard Brown	1	SS	28	490	
		2	SS	37		7-32-48-13
	Grey	3	SS	23		WL Elev. 482.8
481.8						
12.0	Silty sand with some gravel and traces of clay (Compact to very dense)	4	SS	20	480	2-37-57-4
		5	SS	67		P Tip Elev. 475.8
		6	SS	68		
		7	SS	88	470	8-52-34-6
		8	SS	151		7-32-46-15
						Artesian Water Encountered Elev. [illegible]3.8
463.3		9	SS	85		
30.5	End of Borehole				460	30-33-32-5

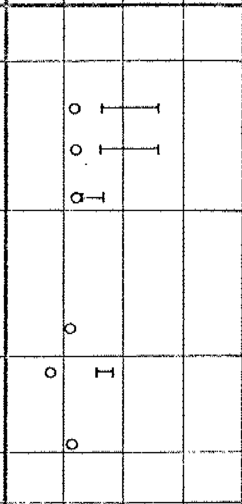
DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE



LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W

WATER CONTENT % 10 20 30



BULK DENSITY γ P.C.F.

20 15 5 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

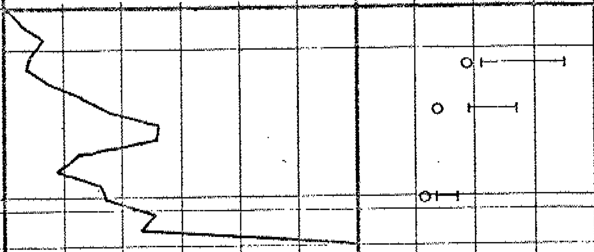
RECORD OF BOREHOLE Nº 6

JOB 73-11032 | LOCATION Co-ords. 15,850,265N 963,966E | ORIGINATED BY V.K.

W.P. 127-66-12 | BORING DATE June 5, 1973 | COMPILED BY V.K.

DATUM Geodetic | BOREHOLE TYPE Auger and Sample with C.M.E. | CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
492.8	Ground level									
0.0	Heterogeneous mixture of clayey silt, sand and gravel Glacial till Stiff to hard					490				
			1	SS	12					
			2	SS	57					5-22-62-11
			3	SS	20					WL Elev. 481.8
479.3 13.5			4	SS	27	480				8-36-46-10
476.8	End of Borehole									
16.0	End of Cone Test					470				



20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 7

JOB 73-11032 LOCATION Co-ords. 15,849,711N 963,697E ORIGINATED BY V.K.

W.P. 127-66-12 BORING DATE June 11, 1973 COMPILED BY V.K.

DATUM Geodetic BOREHOLE TYPE Auger and Sample with C.M.E. CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/FOOT	REMARKS GR.SA.SI.CL
501.7	Ground level				W.L. Elev. 501.7
0.0	Heterogeneous mixture of clayey silt, sand and gravel (Glacial till) Brown	1	SS	29	
		2	SS	72	25-19-41-16
		3	SS	45	
	Grey (Very stiff to hard)	4	SS	43	
		5	SS	38	
		6	SS	37	
480.7 / 21.0	Silt to silty sand with traces of gravel and clay (Dense)	7	SS	51	4-41-45-10
		8	SS	61	
471.7 / 30.0	End of Borehole	9	SS	133	7-47-42-4

ELEV. SCALE: 500, 490, 480, 470

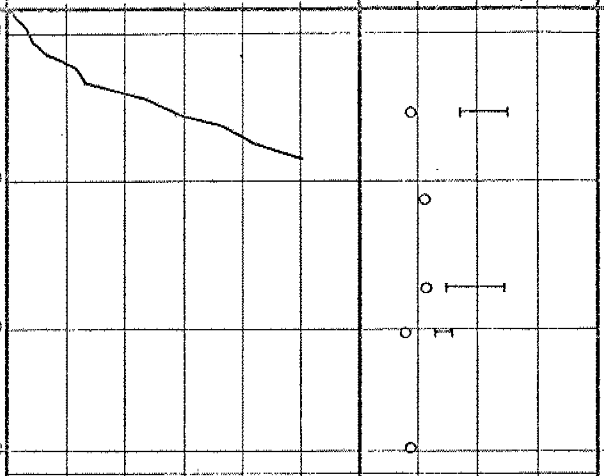
DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ⊗ QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.



15 20 5 10 % STRAIN AT FAILURE

OFFICE REPORT SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

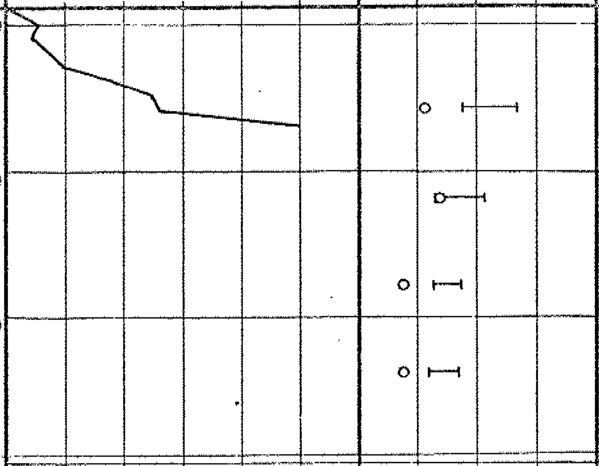
RECORD OF BOREHOLE Nº 8

JOB 73-11032 LOCATION Co-ords. 15,849,779N 963,725E ORIGINATED BY V.K.

W.P. 127-66-12 BORING DATE June 12, 1973 COMPILED BY V.K.

DATUM Geodetic BOREHOLE TYPE Auger and Sample with C.M.E. CHECKED BY [illegible]

SOIL PROFILE ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
501.1	Ground level									W.L. Elev. 501.1
0.0	Brown					500				
			1	SS	30					
	Heterogeneous mixture of clayey silt, sand and gravel		2	SS	75					3-26-54-17
			3	SS	41					
	(Glacial till)		4	SS	24	490				3-21-61-15
	Very stiff to hard		5	SS	38					
	Grey		6	SS	60					
			7	SS	65	480				
			8	SS	56					
470.6			9	SS	27					
30.5	End of Borehole					470				



20 / 15 ⌖ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 9

JOB 73-11032 — LOCATION Co-ords. 15,849,867N 963,798E — ORIGINATED BY V.K.

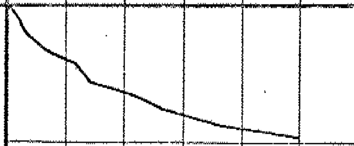
W.P. 127-66-12 — BORING DATE June 5, 1973 — COMPILED BY V.K.

DATUM Geodetic — BOREHOLE TYPE Auger and Sample with C.M.E. — CHECKED BY [illegible]

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT % 10 20 30



ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
499.2	Ground level							W.L. Elev. 499.2
0.0	Heterogeneous mixture of clayey silt, sand and gravel		1	SS	35			
	Glacial till		2	SS	70			6-23-52-19
	(Hard) Brown		3	SS	111	490		
	Grey		4	SS	56			0-14-84-2
	(With occasional silty sand layers		5	SS	81			
	below elev. 487)		6	SS	73	480		7-38-53-2
			7	SS	62			
476.2 / 23.0	Silty sand with gravel							
472.7	Compact		8	SS	10			16-59-25
26.5	End of Borehole					470		

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 10 (B.H. No. 7, 72-11053)

JOB 73-11032
W.P. 127-66-12
DATUM Geodetic

LOCATION Co-ords. 15,850,008N 963,879E
BORING DATE April 17, 1972
BOREHOLE TYPE Auger and Dynamic Penetration Test

ORIGINATED BY V.K.
COMPILED BY C.S.P.
CHECKED BY

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
497.3	Ground level
0.0	Fill material-(Clayey silt with sand and gravel)-Traces of organics. Soft to stiff
491.3 / 6.0	Brown / Grey. Clayey silt with sand and gravel (Glacial till) Hard to stiff
	Silty sand
457.3 / 40.0	Silty sand and gravel changing to sand and silt. Very dense
395.3 / 102.0	(To be continued)

SAMPLES

NUMBER	TYPE	BLOWS/FOOT
1	SS	21
2	SS	70
3	SS	49
4	SS	47
5	SS	29
6	SS	15
7	SS	13
8	SS	4
9	SS	103
10	SS	100/6"
11	SS	115
12	SS	100/6"
13	SS	100/6"

ELEV. SCALE: 490, 480, 470, 460, 450, 440, 430, 420, 410, 400

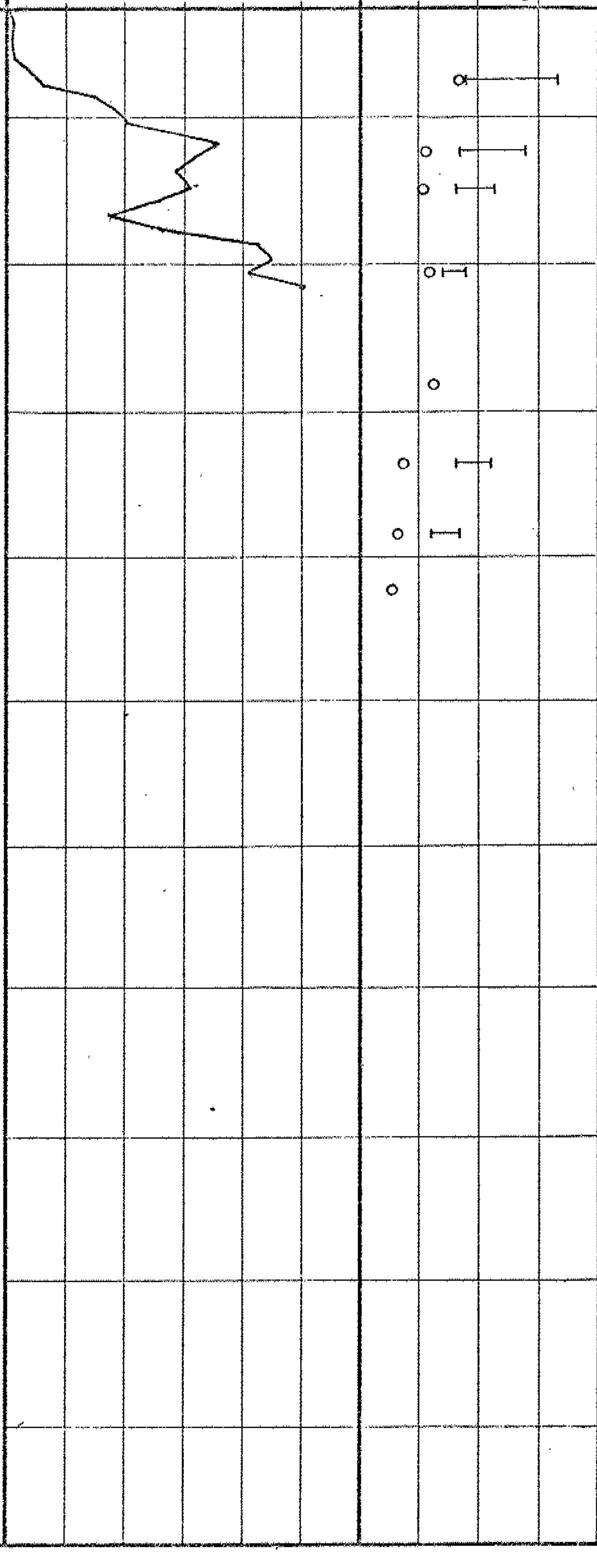
DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100

SHEAR STRENGTH P.S.F.: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE

LIQUID LIMIT W_L; PLASTIC LIMIT W_P; WATER CONTENT W

WATER CONTENT %: 10 20 30

BULK DENSITY γ P.C.F.



REMARKS

500.3

GR SA. SI. CL.

P10(1) Tip Elev 489.3

2 54 39 5

P10(2) Tip Elev. 480.3

WL Elev 480.

8 39 43 10

6 68 16 10

19 40 35 6

2 30 58 10

20 / 15 - 5 / 10 % STRAIN AT FAILURE

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 10 (CONT.)

JOB 73-11032 LOCATION Co-ords. 850,010N 963,879E ORIGINATED BY V.K.

W.P. 127-66-12 BORING DATE April 17, 1972 COMPILED BY C.S.P.

DATUM Geodetic BOREHOLE TYPE Auger and Dynamic Penetration Test CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
395.3 / 102.0										
						390				
383.3 / 114.0	End of Borehole									
						380				

15 ◇ 5 (20 above, 10 below) % STRAIN AT FAILURE

OFFICE REPORT – SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 11

JOB 73-11032
W.P. 127-66-12
DATUM Geodetic

LOCATION Co-ords. 15,850,078 N 963,969 E
BORING DATE June 8, 1973
BOREHOLE TYPE Auger and Sample with C.M.E.

ORIGINATED BY V.K.
COMPILED BY V.K.
CHECKED BY

SOIL PROFILE / SAMPLES / ELEV. SCALE / DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 / SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE / LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W / WATER CONTENT % 10 20 30 / BULK DENSITY γ P.C.F. / REMARKS GR.SA.SI.CL.

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS GR.SA.SI.CL.
495.2	Ground level						W.L. Elev. 495.2
0.0	Heterogeneous mixture of clayey silt, sand and gravel (Glacial till) Brown		1	SS	48		
			2	SS	89	490	6-27-52-15
	Grey (Hard)		3	SS	49		
481.2			4	SS	36		
14.0	Silt to silty sand with traces of gravel and clay		5	SS	23	480	7-43-42-8
			6	SS	29		
	(Compact to dense)		7	SS	23		2-48-44-6
469.7			8	SS	34	470	
25.5	End of Borehole						
						460	

20
15 ⊹ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH | FOUNDATIONS OFFICE

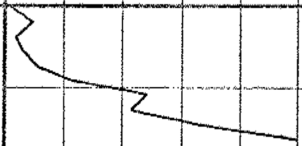
RECORD OF BOREHOLE № 12

JOB 73-11032 | LOCATION Co-ords. 15,850,162N 964,065E | ORIGINATED BY V.K.

W.P. 127-66-12 | BORING DATE June 8, 1973 | COMPILED BY V.K.

DATUM Geodetic | BOREHOLE TYPE Auger and Sample with C.M.E. | CHECKED BY

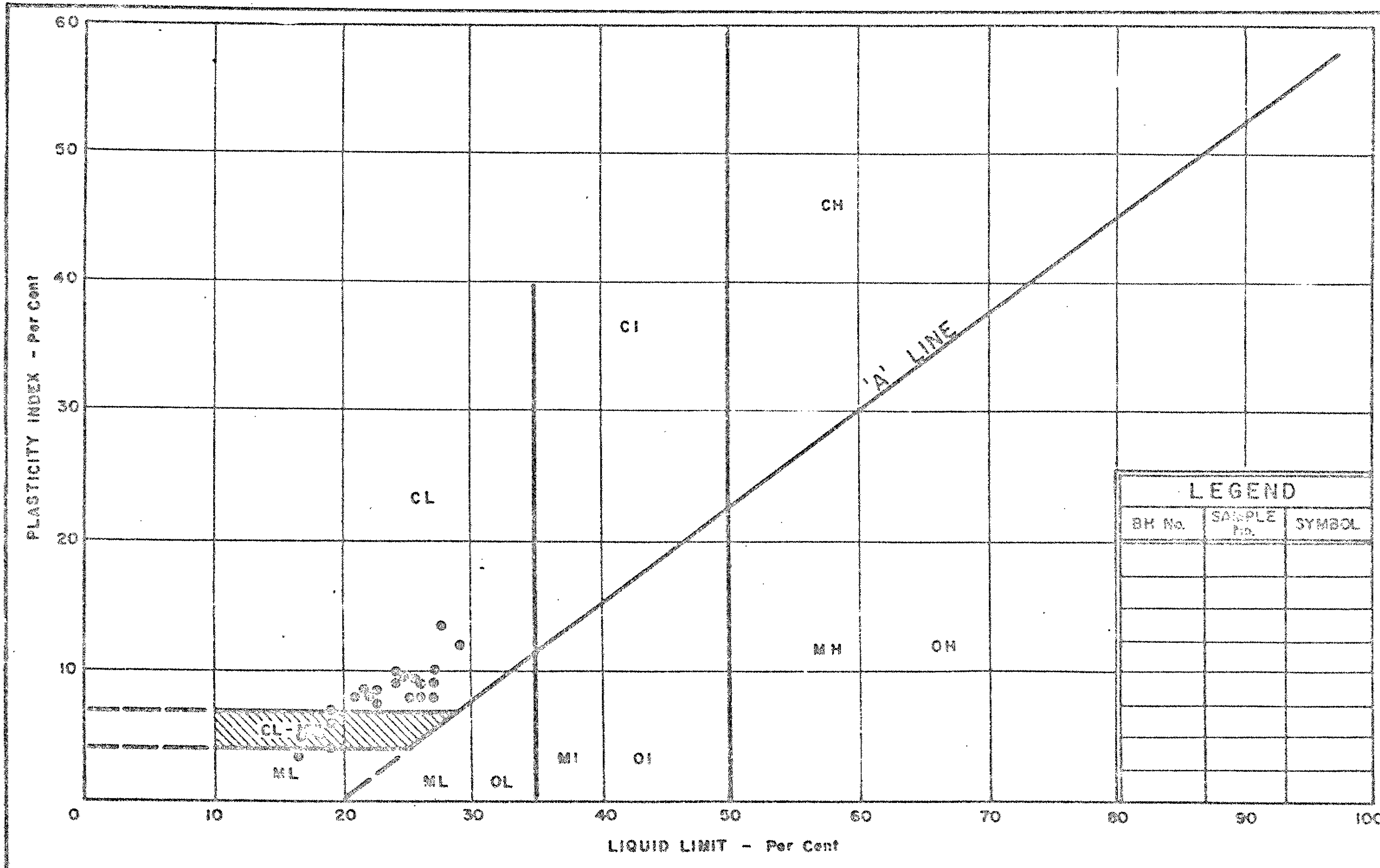
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
495.4	Ground level									
0.0	Heterogeneous mixture of clayey silt, sand and gravel (Glacial till) Brown		1	SS	22					W.L. Elev. 495.4
			2	SS	60	490				1-28-53-18
			3	SS	150/9"					
	Grey (Very stiff to hard)		4	SS	74					4-27-49-20
478.9			5	SS	112	480				
16.5	End of Borehole									
						470				



OFFICE REPORT ON SOIL EXPLORATION

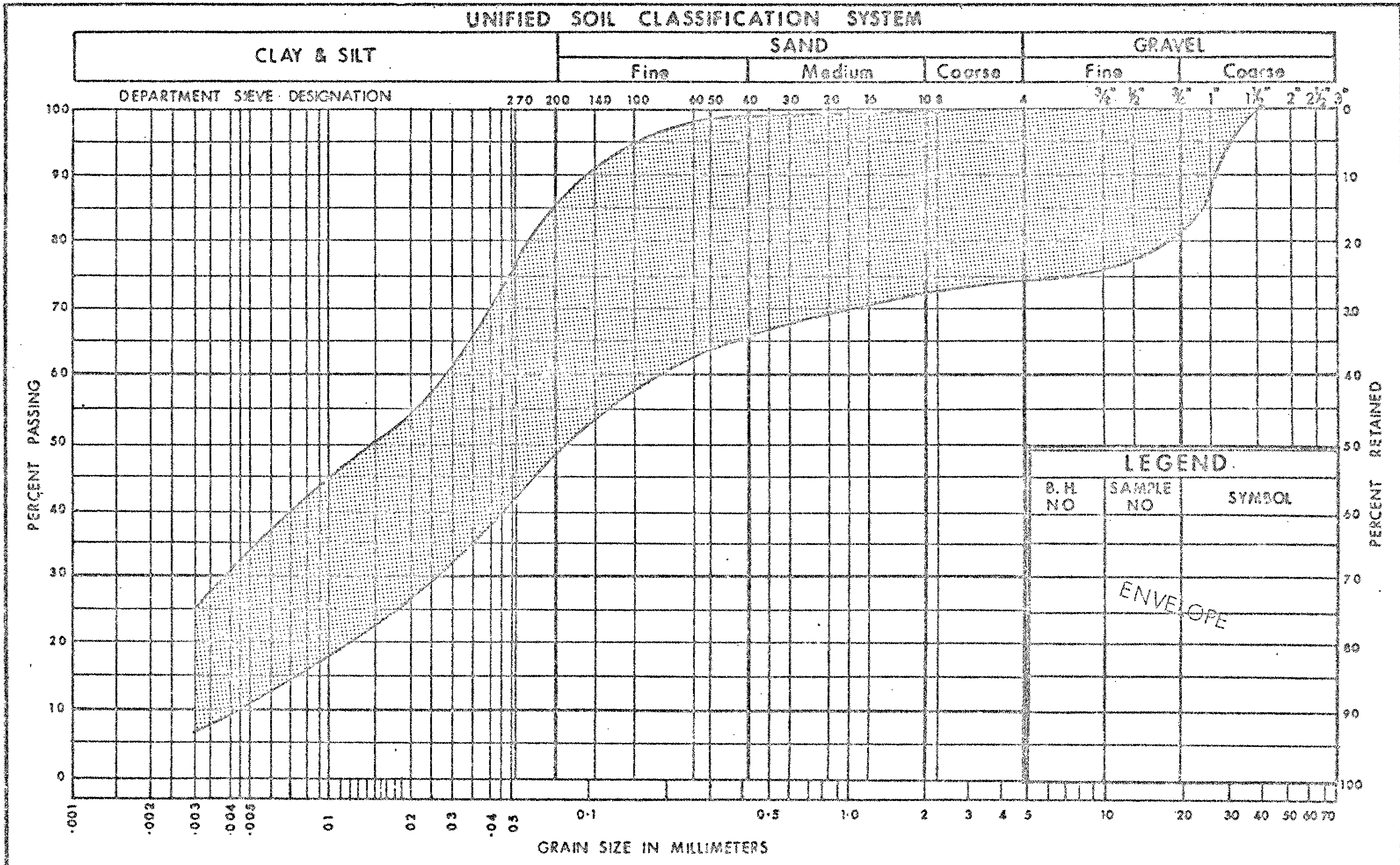
20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

REVISED JULY, 1966

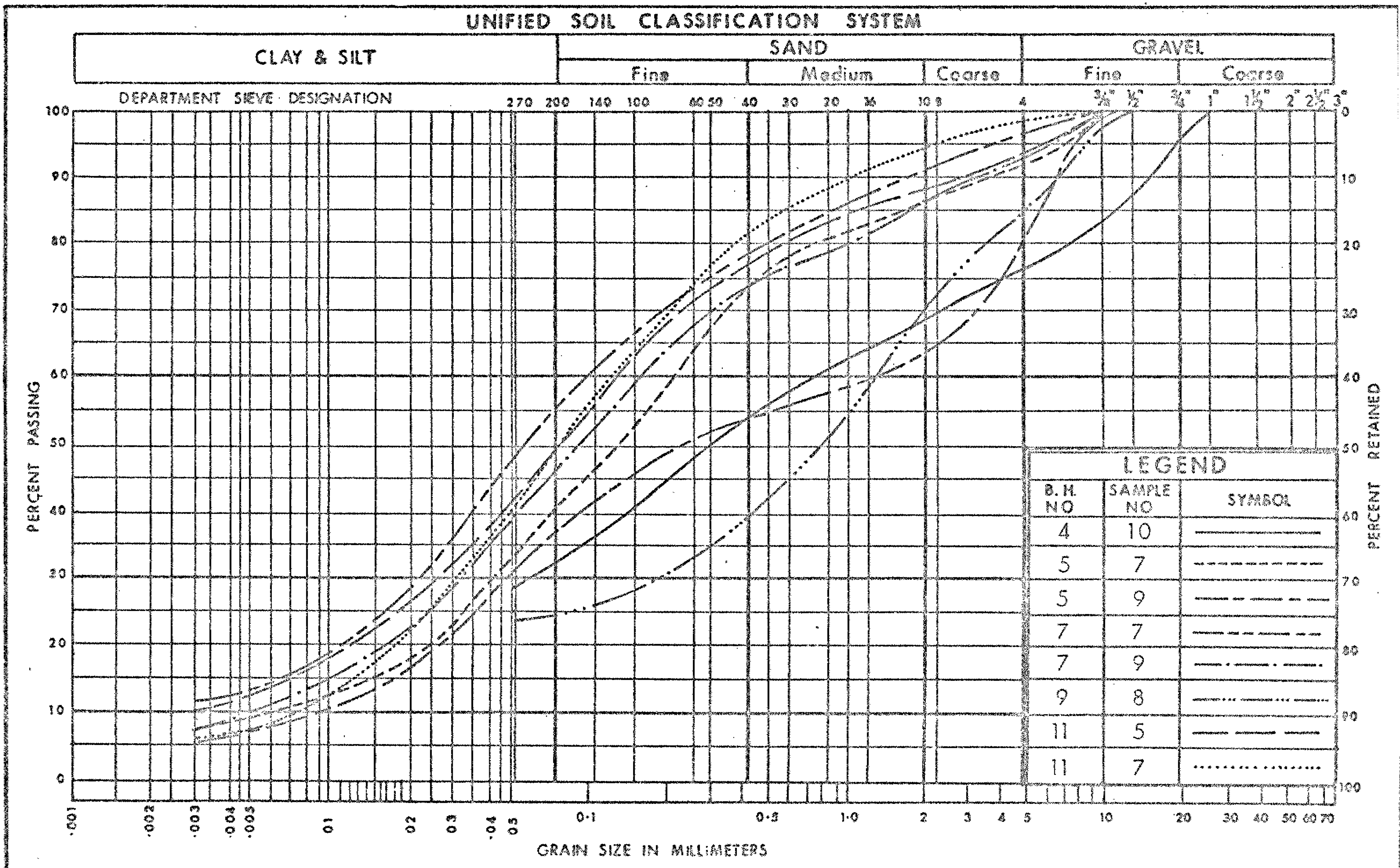


DEPARTMENT OF HIGHWAYS MATERIALS and TESTING DIVISION	PLASTICITY CHART GLACIAL TILL HET. MIX. OF CLAYEY SILT, SAND & GRAVEL	W.P. No. 127 - 66 - 12 JOB No. 73 - 11032 FIG. 1

FORM OD-MT
REV. AUG. 19



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS DESIGN SERVICES BRANCH	GRAIN SIZE DISTRIBUTION GLACIAL TILL HET. MIX. OF CLAYEY SILT, SAND & GRAVEL	W.P. No. 127 - 66 - 12 JOB No. 73 - 11032 FIG. 2



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS DESIGN SERVICES BRANCH	GRAIN SIZE DISTRIBUTION SILT, SILTY SAND OR SANDY SILT	W.P. No. 127-66-12 JOB No. 73-11032 FIG. 3

GROUND WATER REGIME

EGLINTON AVE (FORMERLY BASE LINE RD.) & CAWTHRA RD.

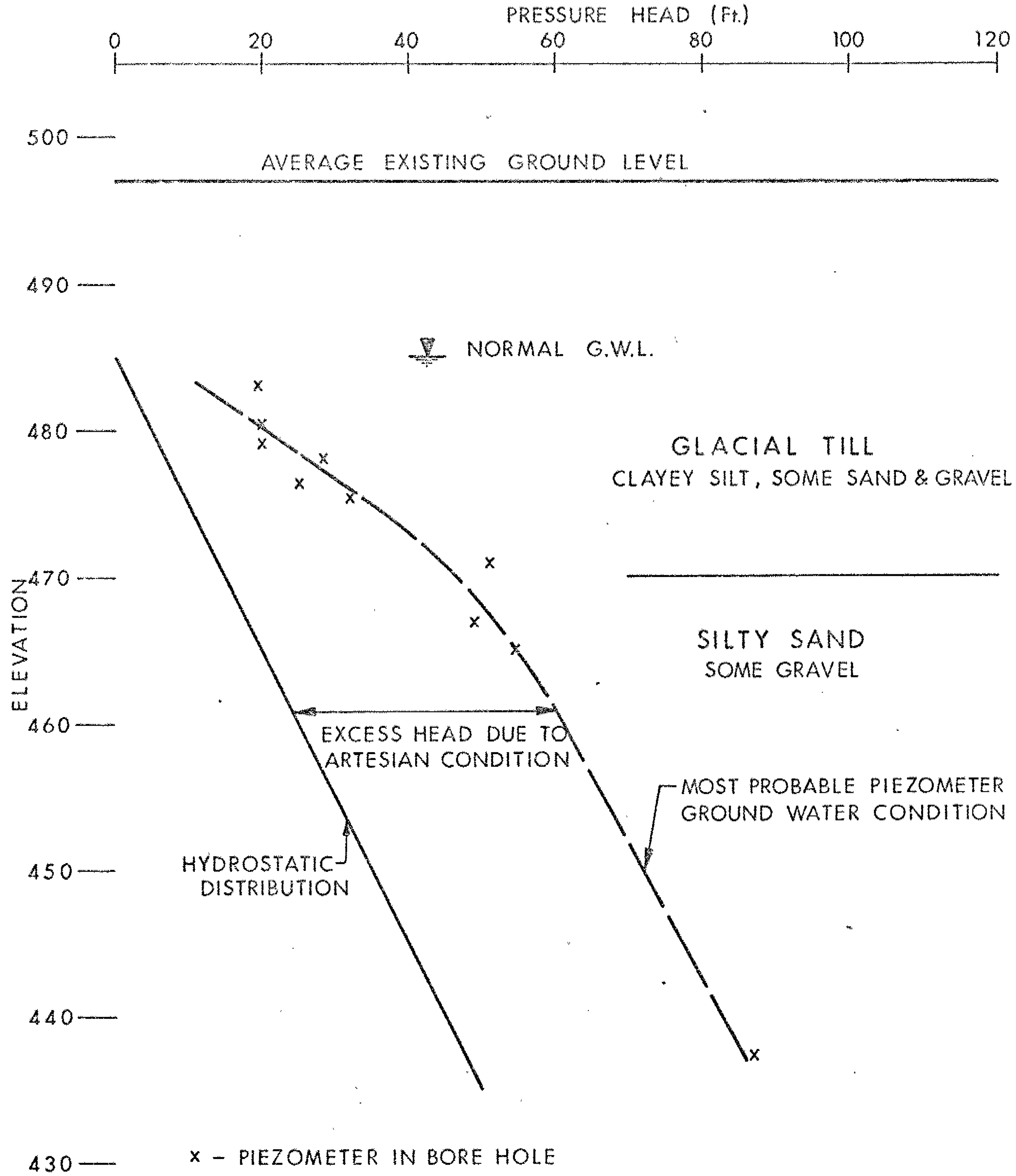


FIG. 4

JOB No: 73-11032

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

'N'=STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

CONSISTENCY	c LB./SQ.FT.
VERY SOFT	0 - 250
SOFT	250 - 500
FIRM	500 - 1000
STIFF	1000 - 2000
VERY STIFF	2000 - 4000
HARD	> 4000

DENSENESS	'N' BLOWS / FT.
VERY LOOSE	0 - 4
LOOSE	4 - 10
COMPACT	10 - 30
DENSE	30 - 50
VERY DENSE	> 50

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10 % , SOME 10-25 % , WITH 25-40 % , > 40 % SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CIU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

FD-9b (Rev. Jan. 73)

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

SOIL PROPERTIES

γ UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s UNIT WEIGHT OF SOLID PARTICLES
γ_w UNIT WEIGHT OF WATER
γ_d UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ' UNIT WEIGHT OF SUBMERGED SOIL
G SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e VOID RATIO
n POROSITY
w WATER CONTENT
S_r DEGREE OF SATURATION
w_L LIQUID LIMIT
w_P PLASTIC LIMIT
I_P PLASTICITY INDEX
w_S SHRINKAGE LIMIT
I_L LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max} VOID RATIO IN LOOSEST STATE
e_{min} VOID RATIO IN DENSEST STATE
I_D DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
RELATIVE DENSITY D_r IS ALSO USED
h HYDRAULIC HEAD OR POTENTIAL
q RATE OF DISCHARGE
v VELOCITY OF FLOW
i HYDRAULIC GRADIENT
k COEFFICIENT OF PERMEABILITY
j SEEPAGE FORCE PER UNIT VOLUME
m_v COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma'}$
c_v COEFFICIENT OF CONSOLIDATION
C_c COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma'}$
T_v TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U DEGREE OF CONSOLIDATION
τ_f SHEAR STRENGTH
c' EFFECTIVE COHESION INTERCEPT
ϕ' EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
(for c' and ϕ':) IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u APPARENT COHESION
ϕ_u APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
(for c_u and ϕ_u:) IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ COEFFICIENT OF FRICTION
S_t SENSITIVITY

GENERAL

π $= 3{\cdot}1416$
e BASE OF NATURAL LOGARITHMS $2{\cdot}7183$
$\log_e a$ OR $\ln a$ NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$ LOGARITHM OF a TO BASE 10
t TIME
g ACCELERATION DUE TO GRAVITY
V VOLUME
W WEIGHT
M MOMENT
F FACTOR OF SAFETY

STRESS AND STRAIN

u PORE PRESSURE
σ NORMAL STRESS
σ' NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ SHEAR STRESS
ε LINEAR STRAIN
γ SHEAR STRAIN
ν POISSON'S RATIO (μ IS ALSO USED)
E MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G MODULUS OF SHEAR DEFORMATION
K MODULUS OF COMPRESSIBILITY
η COEFFICIENT OF VISCOSITY

EARTH PRESSURE

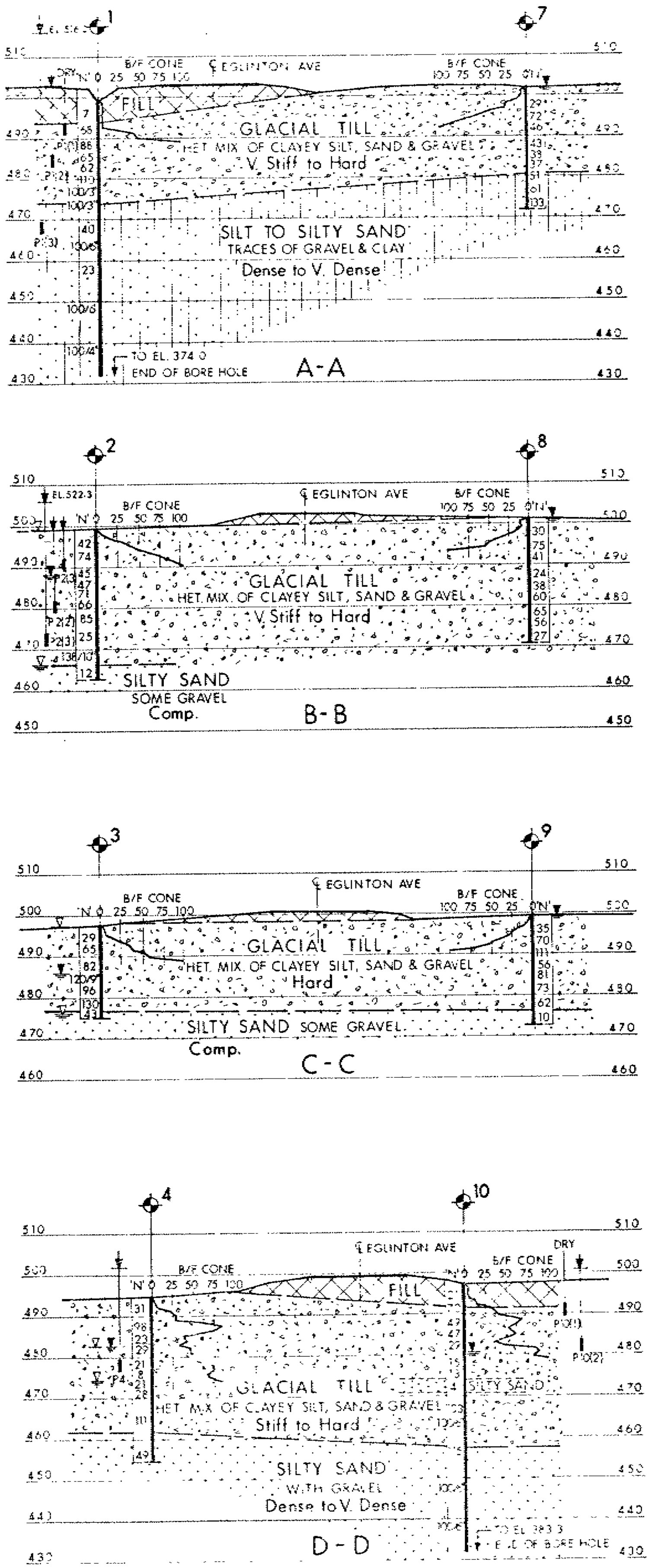
d DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ ANGLE OF WALL FRICTION
K DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o COEFFICIENT OF EARTH PRESSURE AT REST

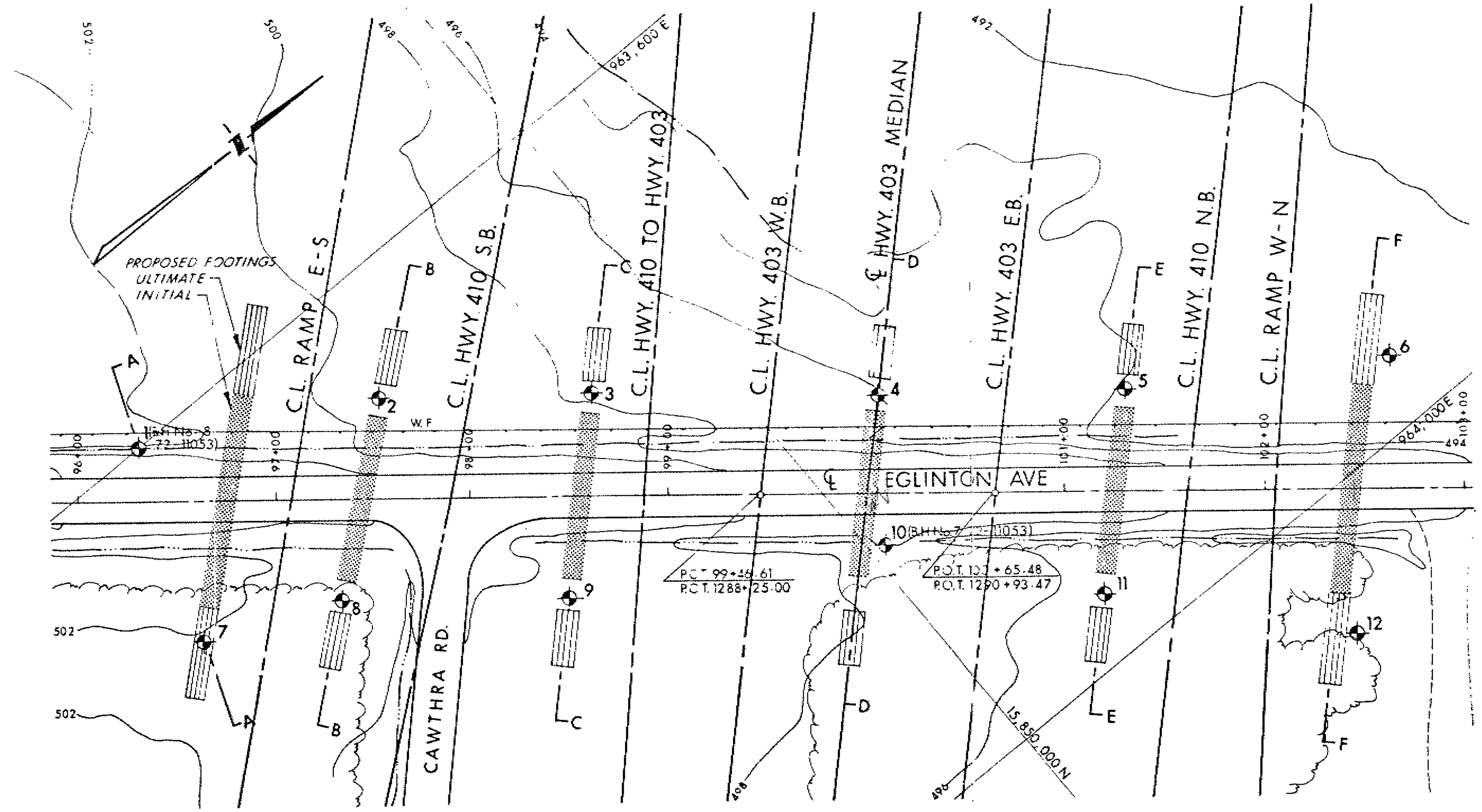
FOUNDATIONS

B BREADTH OF FOUNDATION
L LENGTH OF FOUNDATION
D DEPTH OF FOUNDATION BENEATH GROUND
N DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s MODULUS OF SUBGRADE REACTION

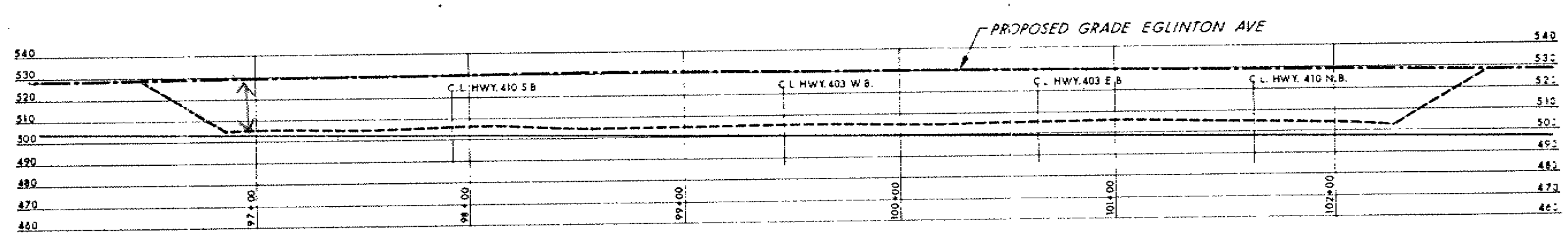
SLOPES

H VERTICAL HEIGHT OF SLOPE
D DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β ANGLE OF SLOPE TO HORIZONTAL

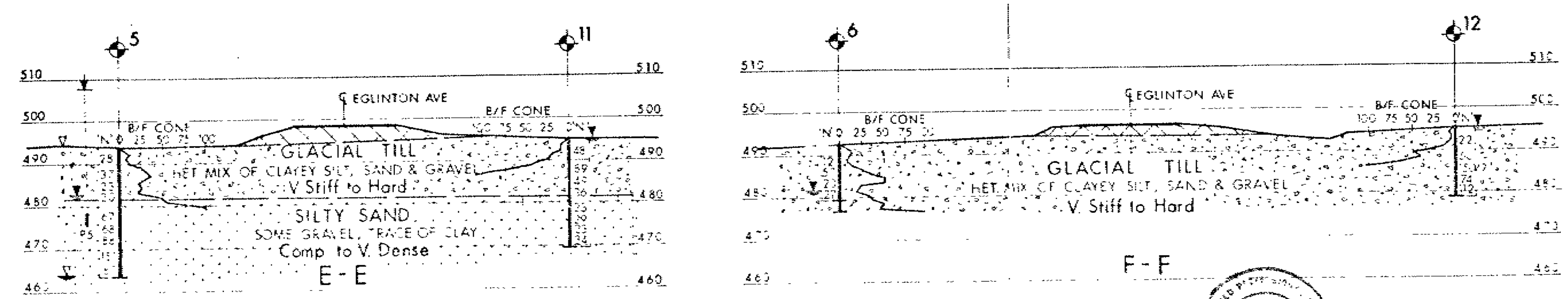




PLAN

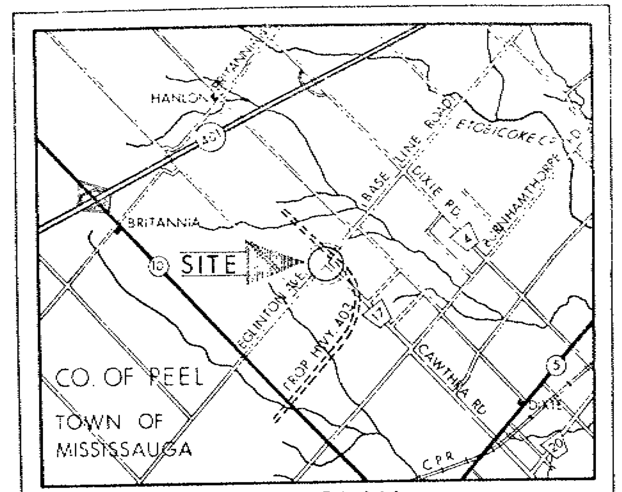


PROFILE



SECTIONS

REF No 3953-4K-1



KEY PLAN

SCALE IN MILES

LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation. April 72 & June 73.
- Piezometers
- Head / Encountered — Artesian Water Levels

NO.	ELEVATION	CO-ORDINATES NORTH	EAST
1	499·0	15,849,750	963,600
2	499·1	15,849,859	963,658
3	497·6	15,849,943	963,725
4	494·9	15,850,054	963,818
5	493·8	15,850,152	963,895
6	492·8	15,850,265	963,966
7	501·7	15,849,711	963,697
8	501·1	15,849,779	963,725
9	499·2	15,849,867	963,798
10	497·3	15,850,008	963,879
11	495·2	15,850,078	963,969
12	495·4	15,850,162	964,065

— NOTE —

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

REVISIONS — DATE — BY — DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

BRIDGE No. 37
EGLINTON AVE

HIGHWAY NO 403 DIST NO 6
CO PEEL
TOWN OF MISSISSAUGA LOT CON

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD V.K. CHECKED — W.P. NO 127-66-72 — DRAWING NO 73-11032A
DRAWN S.R. CHECKED — W.O. NO 73-11032
DATE JULY 23 1973 — SITE NO — BRIDGE DRAWING NO
APPROVED — PRINCIPAL FOUNDATION ENGINEER — CONT NO

79-77

Ministry of Transportation and Communications
Ontario

Memorandum

To: Mr. R. Northwood,
Area Construction Engineer,
Central Region.

Attention: Mr. I. Tremain,
Construction Supervisor.

Our File Ref.

From: Pav't. & Foundation Design Section,
Engineering Materials Office,
Room 315, Central Building,
Downsview, Ontario.

Date: 79 10 05

In Reply to

Subject: Re: Seepage Problems During Construction
Around Pier #2 and #3, Bridge #37,
Hwy. 403 and Eglinton Avenue,
W.P. 127-66-12, Cont. 79-77,
District 6, Toronto.

In response to a request from Mr. I. Tremain, the above mentioned site was visited by Messrs. T. Kazmierowski and M. Devata of this office on 79 09 21. This memo summarizes our observations and verbal recommendations provided to you at the site meeting.

Excessive seepage was observed in the area of the west side of the south pier #2 footing and also on the north side of the north pier #3 footing. In addition, a pool of water was present between the footings at the pier locations. The seepage was quite active visibly carrying out some fine grained material. This could be attributed to the presence of localized springs or due to excess hydrostatic head generated by artesian water conditions. The site investigation revealed the presence of high artesian water conditions with a head up to elevation 500 for piezometers located at elevation 480 or below. This aspect is well documented in the Contract Foundation Investigation Report on Figure #4. In view of this, the founding elevation of the footing was restricted to elevation 491. It is understood that under a separate grading contract, the old sewer backfill material was removed to elevation 486 and backfilled with mass concrete to the base of the new footing construction. It is possible that this operation might have intercepted some of the pervious sand layers. According to local residents, the general area is quite active with small springs and it is also possible that these springs might have been intercepted during the excavation operation of the pier foundations.

Recommended Remedial Measures:

The primary concern is to prevent loss of fine grained subsoil from and around the footing area. In order to prevent such problems, an effective drainage relief system should be adopted immediately. This should consist of a continuous perimeter drainage system incorporating a 6" diameter perforated sub-drain enclosed in a filter fabric and further encased in a

cont'd. 2

free draining material wrapped with appropriate filter fabric. The invert elevation of the sub-drain should be 12 inches below the footing base and the drain should be located 12 inches away from the edge of the constructed footing. These drains should be extended into the main drainage system so that the continuous flow can be maintained at all times. The low areas between the north and south footing for each pier should be filled with mass concrete upon the approval of the Structural Office.

It was agreed at the site meeting that the aforementioned remedial measure will be adopted immediately without any undue delay so that there will not be any danger of undermining the foundation material due to loss of fine grained materials in the subsoil.

We believe that this information will be adequate for your immediate requirements and should you require any clarification with regard to this project, please contact us.

M. Devata

M. Devata,
Senior Foundation Engineer.

MD/cy

c.c. C. S. Grebski
R. D. Gunter
Files

Mr. C.S. Grebski
Structural Design Engineer
Structural Office
West Building, Downsview

Soil Mechanics Section
Engineering Materials Office
West Building, Downsview

77 10 11

Re: Eglinton Ave., Underpass, Bridge No. 37
W.P. 127-66-12, Site 24-319
Hwy. 403, District 6, Toronto

We have reviewed the final design drawings No. 1 and 3 dated September 13, 1977 for the above mentioned structure.

The indicated pier and abutment footing base elevations comply with our latest comments presented in our memorandum dated February 18, 1977. We want to remind you again that the treatment of the proposed footing with regard to the existing 24" Ø watermain and 18" Ø storm sewer should be as per our memorandum to the Structural Planning Office of January 7, 1977.

We have no further comments.

V. Korlu
Project Engineer

For: M. Devata
Supervising Engineer

VK/MD/bh

cc: Files

Mr. C.S. Grebski
Structural Design Engineer
Structural Design Section
West Building, Downsview

Mr. W. Lin

Soil Mechanics Section
Engineering Materials Office
West Building, Downsview

February 18, 1977

Re: Eglinton Avenue Underpass, Bridge 37
W.P. 127-66-12, Site 24-317
Hwy. 403, District 6, Toronto

We have reviewed the preliminary design drawing (#P1) dated January, 1977 for the above mentioned structure and our comments are as follows.

The pier and abutment footing elevations as shown on the drawing comply with our previous recommendations.

The treatment of the proposed footing with regard to the existing 24" ø watermain and 18" ø storm sewer should be as per our memorandum to the Structural Planning Office of January 7, 1977.

C. Johnson

C. Johnson
Project Engineer

For: M. Devata
Supervising Engineer

CJ/gs

cc: G.C. Burkhardt
N. Sen
Files
Record Services

Ministry of Transportation and Communications

Ontario

Memorandum

To: Mr. C. F. Farrell,
Act. Reg. Structural Planning Engineer,
Structural Planning Office,
Central Region, 3501 Dufferin Street,

Attention:
Mr. D. H. Bye

Our File Ref.

From: Soil Mechanics Section,
Geotechnical Office,
West Building, Downsview.

Date: January 7th, 1977

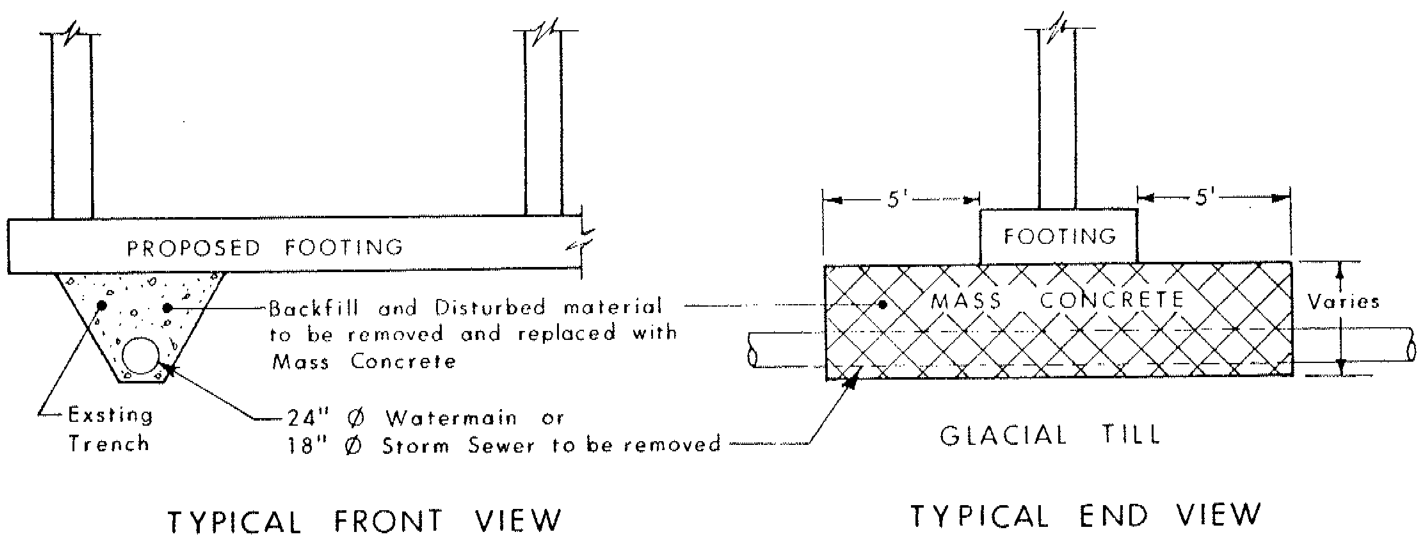
In Reply to

Subject: EGLINTON AVENUE UNDERPASS
(BRIDGE #37)
SITE 24-319 W. P. 127-66-12
HWY. 403, DISTRICT 6 TORONTO

In response to your memorandum of December 22nd, 1976, with regard to the treatment of the existing 24"ø watermain and 18"ø storm sewer necessitated by the construction of the structure foundations in the above mentioned project, we have reviewed the pertinent subsurface data and submit the following recommendations:

1. The existing utilities under the proposed footings should be relocated to prevent the footings and the utilities from being damaged by differential settlements.

2. All previous backfill material to the pipes and all disturbed soil at the locations of the proposed footings should be removed and replaced with mass concrete to the dimensions shown in the figure below. The mass concrete should be placed simultaneously with the 3" working slab.



N.T.S.

C. Johnson

for M. Devata
Supervising Engineer

c.c. N. Sen
W. Lin
Files
Record Services

Mr. G.C. Burkhardt
Regional Bridge Planning Engineer
Central Region
3501 Dufferin Street, Downsview

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

November 19, 1976

Eglinton Avenue Underpass (Bridge #37)
W.P. 127-66-12, Site 24-319
Hwy. 403, District #6, Toronto

Further to our memorandum of November 1, 1976, Mr. Bye would like to obtain footing elevations for various elements of the structure. Taking into account the latest available information, our recommendations are as follows:

1. Pier footing elevations should be as per our memorandum of November 1, 1973.

2. The base of the west abutment should be located at elevation 493 (formerly known as pier #1), whereas the east abutment should be located at elevation 492,with an allowable load of 3 tons per square foot.

3. In view of the presence of the artesian condition, the use of piles is not advisable.

B. Ly

B. Ly
Senior Engineer

For: M. Devata
Supervising Engineer

MD/BL/gs

cc: C.S. Grebski
W. Lin
D. McDonald
Files ✓
Record Services

Ministry of Transportation and Communications

Ontario

Memorandum

To: Mr. G.C.E. Burkhardt
Regional Bridge Planning Engineer
Bridge Planning Office
Central Region, 3501 Dufferin St.

Attention: Mr. D.H. Bye

Our File Ref.

From: Soil Mechanics Section
Geotechnical Office
West Building, Downsview

Date: November 1, 1976

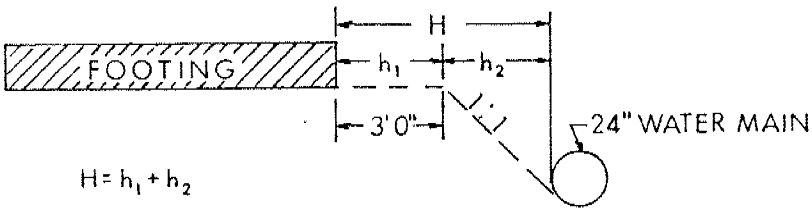
In Reply to

Subject: Eglinton Avenue Underpass (Bridge 37)
W.P. 127-66-12, Site 24-319
Hwy. 403, District 6, Toronto

In response to your two memoranda with regard to the above mentioned project, we have reviewed the pertinent subsurface data and submit the following comments:

1. The revised footing locations necessitated by the redesign due to median width and reduction in the lane requirements were carefully reviewed. We conclude that our recommendations and subsurface data contained in the foundation report (W.O. 73-11032 dated August 2, 1973), are still applicable.

2. With regard to the treatment of the existing 24" ø watermain on the north side of the Eglinton Avenue which will be left in place, we suggest the following measures to be adopted during the time of design and construction.



Minimum horizontal distance between the face of the footing and the inner face of the watermain should be as follows:

a) If the sewer is at the same base elevation of the footing then $h_2 = 0$ and $H = 3$ feet.

b) If the sewer is lower than the base elevation of the footing then $H = h_1 + h_2$

$= 3$ ft. $+ h_2$ (minimum 1:1 slope as shown on the figure)

cont'd.......

The excavation of the footing in the vicinity of the existing 24" ø watermain should be carried out under the close supervision of the Ministry's District personnel. Care should be exercised so that material in this critical area is not disturbed during excavation operations. The backfill in the vicinity of the pipe should be carried out with well compacted granular material to the finished grade.

V. Korlu
Project Engineer

For: M. Devata
Supervising Engineer

MD/VK/gs

cc: C.S. Grebski
W. Lin
D. McDonald
Files
Record Services

Ministry of Transportation and Communications

Ontario

Memorandum

To: Mr. C. Mirza
Head
Soil Mechanics Section
West Building

Attention: Mr. M. Devata

Our File Ref.

From: G.C.E. Burkhardt
Structural Planning Office
3501 Dufferin Street

Date: September 27th, 1976

In Reply to

Subject: EGLINTON AVENUE UNDERPASS (BRIDGE 37)
SITE 24-319 W.P. 127-66-12
HWY. 403 DISTRICT 6 :

This is further to my letter of September 10th, 1976.

Please find attached for your information and use more detailed plans and cross-section for the above structure as follows:

Plan showing approx. footing locations and Hwy. 403 pavement elevations

Plan showing Eglinton Avenue pavement elevations

Cross-section through Hwy. 403 at structure.

I have indicated in blue on the plan and cross-section the approx. location of the 24" dia. watermain on Eglinton Avenue. In plan the location of the watermain has been confirmed in the field. The profile though is based on the "as designed" drawings and the location has not been confirmed in the field.

I trust these plans will assist you in the review of the previous Foundation Report.

D.H. Bye

D.H. Bye
STRUCTURAL PLANNING SUPERVISOR
for:
G.C.E. Burkhardt
REG. STRUCTURAL PLANNING ENG.

Ans. Letter of Nov. 1/76.

RECEIVED
GEOTECHNICAL OFFICE
SEP 28 1976
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

Ministry of Transportation and Communications
Ontario

Memorandum

To: Mr. C. Mirza
Head, Soil Mechanics Section
West Building

From: Structural Planning Office
Central Region

Attention: Mr. M. Devata

Date: 10th September, 1976

Our File Ref.

In Reply to

Subject: EGLINTON AVE. UNDERPASS (BRIDGE 37)
SITE 24-319 WP 127-66-12
HWY. Q.E.W. DISTRICT 6

As you are aware the concept of Hwy. 403 between Hwy. 401 and Hwy. 10 has changed considerably. The most notable of the changes being a reduction in the median width and a reduction in the lane requirements.

These changes will necessitate a redesign of the above mentioned structure. Due to these changes and in light of the very sensitive nature of the subsoil in the area of the structure, it would be appreciated if you could review the original Foundation Report (WO 73-11032 dated Aug. 2, 1973).

Could you also make a recommendation regarding the treatment of the 24"Ø watermain on the north side of Eglinton Ave. which will be left in place. The watermain is indicated in plan and profile on the enclosed plans.

I have enclosed for your use the following material. The approx. location of the footings is indicated in red on the plan.

Site Plan
Profile Hwy. 403 E.B.
Profile Hwy. 403 W.B.
Profile of Eglinton Ave. is unchanged.

The scheduled completion date for the review of the report is October 20th, 1976.

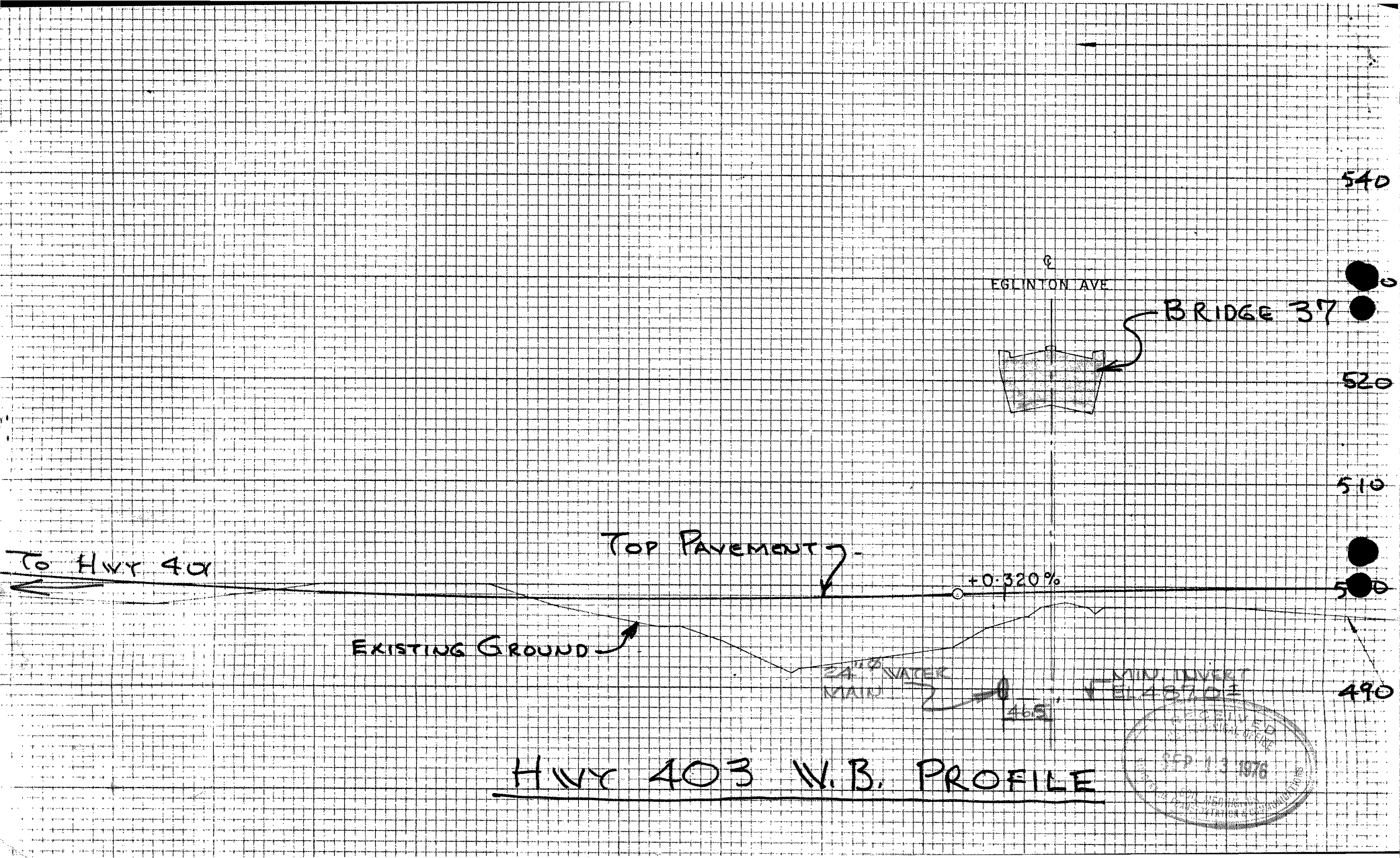
Our letter 8 Nov. 1/76

D.H. Bye
D.H. Bye
STRUCTURAL PLANNING SUPERVISOR
for:
G.C.E. Burkhardt
REG. STRUCTURAL PLANNING ENG.

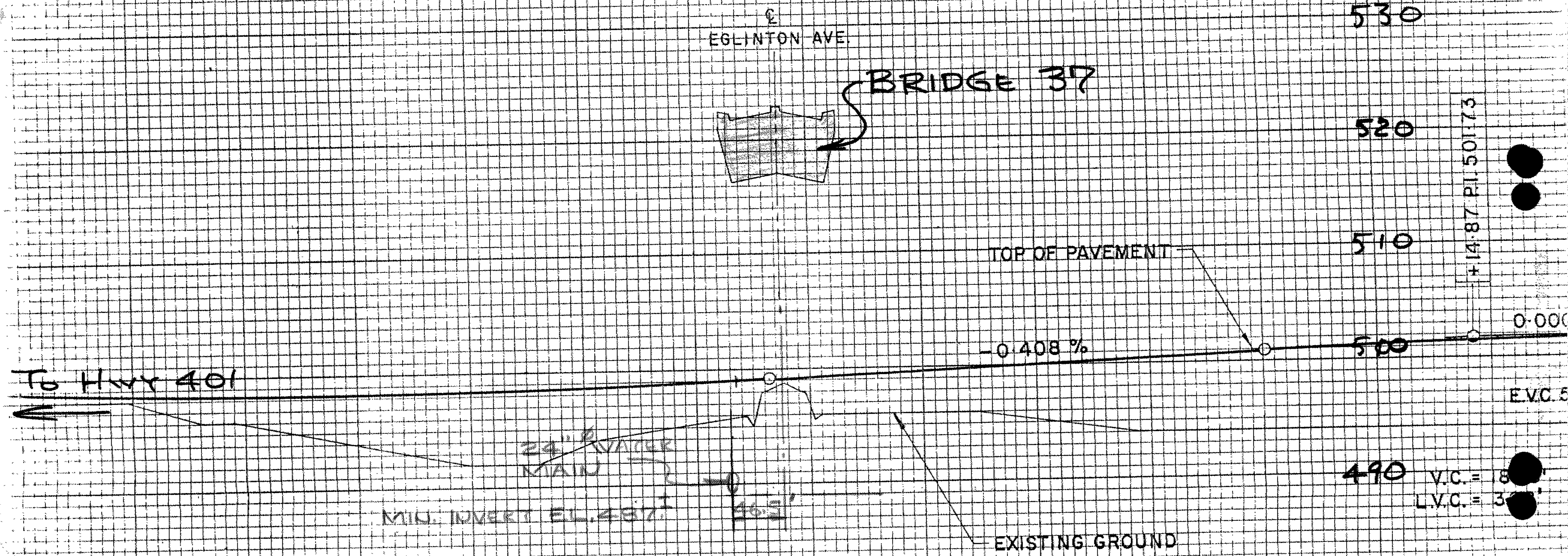
c.c. R. Fitzgibbon
J. Anderson
W. Roters
E. Shedler

RECEIVED
GEOTECHNICAL OFFICE
SEP 13 1976
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

M.D.



HWY 403 W.B. PROFILE



HWY 403 E.B. PROFILE

Mr. W. Roters,
Area Manager,
Regional Planning and Design,
Central Region.
N. Sen

G. C. E. Burkhardt,
Structural Planning Office,
3501 Dufferin Street.

June 7, 1976.

Eglinton Avenue Underpass (Bridge #37)
Site 24-319, W.P. 127-66-12,
Cawthra Road S.B. Underpass (Bridge #38)
Site 24-327, W.P. 127-66-25,
Highway 403, District 6.

This is in reference to the Minutes of Progress Meeting 76-1 Items 1-(d) and (m), for Hwy. 403 - W.P. 127-66-01 and -38 and further to Mr. B. Ly's memo of May 27, 1976.

As indicated in Mr. Ly's memo a two month preloading of approach fills will be required on Structures #37 and #38 to accommodate any settlement within the fill.

Mr. Ly's second comment is in reference to the recommended founding elevation for the centre pier footing for Bridge #38. Elevation 490 as indicated is the elevation of the bottom of the footing. Since the pier footings will be approximately 5' deep with 1' of cover, the road profile at the shoulder in the area of the structure would have to be approx. El. 496.0 or higher, in order to avoid any conflict.

If any further information is required on this matter please contact this office.

D H Bye

DHB:lm

D. H. Bye,
STRUCTURAL PLANNING SUPERVISOR,
for:
G. C. E. Burkhardt,
REG. STRUCTURAL PLANNING ENG.

c.c. W. Lin
M. Devata ✓

Mr. C.S. Grebski,
Structural Design Engineer,
West Building, Downsview.

Soil Mechanics Section,
Geotechnical Office,
West Building, Downsview.

July 18th, 1974.

RE: Eglinton Avenue Underpass (Bridge #37),
W.P. 127-66-12, Site #24-319,
Highway 410, District #6 (Toronto).

We have reviewed the preliminary drawings for the abovementioned structure and our comments are as follows:

As shown on the drawings the piers for this structure are to be supported on spread footings, founded within the parent hard glacial till stratum ranging from elevations 490 to 491, which is acceptable. However, the founding elevations may be raised to those as recommended in our memo to Mr. G.C.E. Burkhardt (November 1st, 1973), in order to minimize the depth of footing excavation, provided that frost protection requirements are fulfilled.

Should you have any queries, please contact this Office.

C.S. Poon
Project Engineer
For:
M. Devata
Supervising Engineer

CSP/mk

c.c. C.G.E. Burkhardt.
Files
Documents

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO:	Mr. G.C.E. Burkhardt, Regional Structural Planning Eng., Central Region, 3501 Dufferin St., Downsview.	FROM:	Foundations Office, Design Services Branch, West Bldg., Downsview.
ATTENTION:	Mr. D. H. Bye.	DATE:	November 1, 1973.
OUR FILE REF.		IN REPLY TO	

SUBJECT: Eglinton Avenue Underpass (Bridge #37)
W.P. 127-66-12, Site #24-319, Hwy. #403
W.O. 73-11032, District #6 (Toronto)

Further to your memo of October 17, 1973, with regard to lowering the grades of Hwy. 403, Hwy. 410, and Eglinton Avenue in this area, we have reviewed all the available data and submit the following comments:

1) Recommendations regarding the abutments as given in our Foundation Report W.O. 73-11032 will still be applicable.

2) The revised founding elevations for the piers, taking into consideration of the new grades, are as follows:

Pier No.	Station	Refer to B.H.'s	Recommended Minimum Founding Elevation
1	97+45	#2 & #8	493
2	98+56	#3 & #9	491
3	100+00	#4 & #10	491
4	101+25	#5 & #11	492

An allowable bearing pressure of up to 3 t.s.f. may be used in designing the footings founded as recommended. Other comments contained in our Foundation Report W.O. 73-11032 are still applicable.

Due to the prevailing artesian conditions in this area, some special measures should be adapted for the construction of the pier foundations.

1) Excavation for the construction of the pier foundation should be carried out in a short period of time in order to minimize dewatering problems and possible softening of the foundation material. If foundation excavations have to be exposed for a considerable period of time, a working slab should be poured as soon as the excavation reaches the founding level.

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2) Backfilling to the finish grade in the excavated area should be completed without any undue delay.

Should you have any queries regarding this project, please do not hesitate to contact this office.

M. Devata

M. Devata,
SUPERVISING FOUNDATIONS ENGINEER.

MD/ao

c.c. E. J. Orr
B. R. Davis
A. Rutka
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H. Greenland
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G. A. Wrong
B. A. Singh
C.C. Parker Ltd. (Attn: R. Lewis)

Foundations Files
Documents