

Mr. A. M. Toye,

July 12, 1960.

Bridge Engineer.

FOUNDATION INVESTIGATION REPORT

Materials & Research Section.

by - William A. Trow & Associates, Ltd.

Attention: Mr. S. McCombie.

Re: Underpass of Ramp 'M' - W.P. 261-60,
Chedoke Expressway, Hamilton, Ontario.
District No. 4.

Attached, we are forwarding to you, the above mentioned report, submitted by the Consultant, W. A. Trow & Associates, Ltd.

We have reviewed the presented factual data and believe that the conclusions and recommendations contained in the report will be adequate for your future design work.

Should there be any other questions in connection with the above structure, please feel free to call on our Office.

L. G. Soderman,
PRINCIPAL FOUNDATIONS ENGR.
Per:

AS/Mdef
Attach.

cc: Messrs. A. M. Toye (2) ✓

H. A. Tregaskes

D. G. Ramsay

I. Campbell

R. E. Richardson

T. J. Kovich

C. C. Parker & Associates. (4)

A. Watt

Foundations Office

Gen. Files.

A. Stermac
(A. Stermac,
FOUNDATIONS OFFICE ENGR.)

BA 1092
60-F-302C

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

884 WILSON AVE.,
DOWNSVIEW, ONT.
ME. 5-5921

Project: J 516

July 8, 1960.

Mr. A. Rutka,
Acting Materials and Research Engineer,
Materials and Research Section,
Dept. of Highways of Ontario,
Parliament Buildings,
Toronto, Ont.

Attention: Mr. L. G. Soderman, P. Eng.,
Principal Soils & Foundations Engineer

Foundation Investigation
Underpass of Ramp 'M' - W. P. 261 - 60
Chedoke Expressway, Hamilton, Ontario.

Dear Sirs:

The enclosed report describes the soil conditions existing along part of the route of Ramp 'M' of the Chedoke Expressway. It deals specifically with the foundation requirements for the proposed underpass structure which carries ramp 'M' over the expressway and with the stability of the high earth embankments which lead uphill to this bridge.

No foundation problem of consequence appears to exist at either of these locations. Simple spread footings bearing at approximate Elev. 290 feet can be used to support the piers of the underpass. Cylindrical piles driven through the fill to refusal in the underlying dense sand can be used to carry the weight of the abutments. Some soft organic muck lies in the path of the embankment fill to the north-west of this underpass. It is probable that it will be displaced by the weight of fill.

We hope that the information contained in this report will assist you in the planning of this part of the expressway. Please contact us if you require additional information on the soil mechanics phases of this project.

Yours very truly,

W. Trow

William A. Trow (P. Eng.)

WAT/lt
Encl.

DEPARTMENT OF HIGHWAYS OF ONTARIO
MATERIALS AND RESEARCH SECTION
PARLIAMENT BUILDINGS, TORONTO, ONT.

FOUNDATION INVESTIGATION
UNDERPASS OF RAMP 'M' - W. P. 261 - 60
CHEDoke EXPRESSWAY, HAMILTON, ONTARIO.

Project: J 516

William A. Trow & Associates Ltd.

July 8, 1960

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FOUNDATION INVESTIGATION
UNDERPASS OF RAMP 'M' - W. P. 261 - 60
CHEDOKE EXPRESSWAY, HAMILTON, ONTARIO

This report describes the results of an investigation which was carried out to determine the soil conditions that exist at the above site. The types of foundations which are compatible with these conditions are outlined, and the safe bearing capacity of each type is indicated. Included in the report are comments on the stability of the approach embankments.

The Project

As proposed, the Chedoke Expressway will run at mid-height along the west side of the narrow sand and gravel bar at the west end of Hamilton Harbour. Ramp 'M' will rise from the westbound lane of the expressway up to the level of York Boulevard - a difference in elevation of about 50 feet. The ramp will first swing to the west of the expressway and then back over the proposed four lane divided highway on its way to York Boulevard. The swing away from the expressway will bring the ramp over low lying ground. The maximum height of embankment will occur on this section where about 40 feet of fill will be required.

The project can thus be considered in two parts. The first part deals with the foundation requirements of the underpass structure while the second involves the stability of that part of the ramp embankment over the low lying area.

The route of the expressway and the system of ramps that are proposed are illustrated on Dwg. 1.

Description of Site

The gravel bar, over which the expressway will pass, has a broad base 700 feet or more in width. The ridge of the bar, at elevation 345, is only 100 feet wide, however, and this width is taken up by York Boulevard. To the west, in the direction of the expressway, the ground drops sharply from the ridge to elevation 305 feet. A flat bench exists at this latter elevation. This bench varies in width along the bar, but at the underpass location, it is about 300 feet wide. The existing Longwood Road passes through the site at the western edge of this terrace. Beyond Longwood Road, the ground again drops rapidly to the level of Old Guelph Road at elevation 265. More gentle slopes bring the bar to the level of Cootes Paradise - a marshy expanse to the west. The plan and profile of Dwg. 2 clearly illustrate the large vertical relief of the bar at the underpass location.

Soil Encountered(1) Underpass Area:

Investigation of the soil at the proposed location of the underpass structure disclosed uniform conditions over this area. Brown sand with gravel was encountered in all five borings made in the locations shown on Dwg. 1 and 2. This soil is stratified and contains thin seams of very stiff silt or clay. This sandy soil is loose to medium dense at shallow depth, but becomes dense below 5 to 20 feet of the ground surface depending on hole location. The elevation at which the soil becomes dense can be seen on the plot of penetration resistance versus depth, prepared for each boring and presented as Dwg. 3 to 7. These levels are also indicated on the subsoil profile of Dwg. 2.

(2) Ramp Embankment Area:

Two boreholes were put down at the edge of Cootes Paradise during this investigation. Two other borings had been made in the area prior to this work. The locations of all four borings are shown in Dwg. 1, together with an estimated subsoil profile prepared from the logs of these holes. The borehole logs themselves are presented as Dwg. 8 to 11.

Referring to these drawings, it is seen that grey clay underlies about 10 to 15 feet of sand and gravel or organic muck. The sand and gravel exists at the surface at holes 15 and 16 on shore. At holes 8 and 8A locations in the marsh, 6 feet of organic muck or silt overlies the sand or a thin brown clay layer.

The surface sand deposit is loose to medium dense and contains organics at times. The clay has a minimum shear strength of about 1300 p.s.f., but is, in general, quite stiff. The compressibility of the clay was not determined during this investigation, but it is probably of a low order for the pressures anticipated from the proposed embankment.

Foundation Considerations - Underpass Structure

If maximum foundation elevation is considered to be a minimum of five feet below the road surface of the expressway for frost protection, the three piers of the four span structure will be founded at elevation 290 approximately. The soil in the area of the piers at and below this elevation is dense. Spread foundations can, therefore, be used under the piers.

The safe bearing capacity of the dense sand can be estimated from published charts based upon an empirical relationship between the standard penetration resistance of the soil and the allowable footing load for 1 inch limiting settlement. In addition, the standard penetration resistance of the

soil at footing level can be estimated from the results of dynamic cone and standard penetration tests in the sand. A conservative resistance value for this soil, as proven in borings 2, 4 and 5, is 30 blows per foot. The corresponding safe bearing capacity from the charts* is 7000 p.s.f. for spread footings of the size that probably will be constructed at this site. Settlement associated with this load should not be greater than one inch and it will occur simultaneously with application of load.

Displacement piles driven through the embankment fill will be the most practical type of foundation for the spill-through abutments. This is particularly true at the west abutment where about 20 feet of loose natural soil must be penetrated before dense conditions are encountered.

Virtual refusal to driving the large diameter displacement piles should be reached within a few feet of the depth at which the soil becomes dense. At hole 3, however, the soil does not become very dense until a depth of about 30 feet is reached so that the piles of the west abutment may penetrate to this level. Steel H piles should not be used at this site because of the uncertainty of their reaching refusal conditions at an economic depth.

The capacity of the displacement piles when driven to refusal will be determined by their structural properties when considered to act as a short column. Settlement will be of a very low order.

Stability of the Approach Embankment

The stability of the high approach fill can be estimated, conservatively, by considering the bearing capacity of the weakest soil zone which was noted at depths of about 25 to 30 feet in holes 15 and 16. Because the weak zone is apparently of only limited thickness and it exists below stiffer material, the actual factor of safety against total shear failure will be somewhat higher. In addition to this, the sides of the embankment will be sloped which will also increase the factor of safety.

The net ultimate capacity of a long spread footing on clay is given by the expression:

$$Q_u = cN_c$$

where: Q_u = ultimate bearing capacity in p.s.f.

c = lowest undrained shear strength about 1300 p.s.f.

N_c = bearing capacity factor = 5

solving the above equation, Q_u = 6,500 p.s.f.

* Page 225, - "Foundation Engineering" - Peck, Thornburn & Hanson

The bearing pressure that will be exerted by 45 feet of fill will be of the order of 5300 p.s.f. This is sufficiently below the "ultimate" indicated above to preclude the need for a more involved consideration of stability.

The above discussion is based upon the assumption that all organic muck or silt will be removed from the path of the embankment. This is necessary for stability. Failure of the embankment would probably occur in the area of holes 8 and 8A if this material were to be left in place. It is recommended, therefore, that the marsh area be excavated to elevation 238 to ensure that all of the organic material is removed. Alternatively, it could be displaced by the weight of the embankment provided that the first approximately 15 feet of fill is placed in one lift and deposited in such a manner that the organic material is pushed ahead and to the west of the earth structure.

Recommendations

1) Underpass Structure

(a) Loose to dense sand with gravel underlies the ground surface in this area. Footings for the central piers of the structure, if placed at El. 290 feet approximately, will rest upon dense sand.

(b) The safe bearing capacity of spread footings on the dense sand is estimated to be 7000 p.s.f. Settlement will be limited to 1 inch.

(c) Large diameter displacement piles are recommended for the support of the spill-through abutments. These piles can be driven through the fill to refusal in the dense to very dense sand.

(d) No problems with water entering excavations, either for the expressway or for the underpass footings, can be foreseen.

2) Ramp Embankment

(a) The maximum height of embankment occurs over the marsh area to the west of the gravel bar. This area was outlined by four borings. Ten to fifteen feet of sand overlies stiff grey clay on the shore while 6 feet of soft organic muck followed by stiff brown clay or fine sand overlies the grey clay in the water area.

(b) An embankment of the order of 40 feet high should be safe from failure provided that the organic muck is removed or displaced in the marsh area.


D.H. Shields, P.Eng.

APPENDIXFIELD INVESTIGATION METHODS

The five holes in the underpass area and hole 16 were put down using continuous flight auger equipment. The holes were 5 inches in diameter and were uncased to full depth.

Hole 15 and the two holes put down previously, numbered 8 and 8A, were advanced using standard diamond drilling equipment modified for soil sampling. Standard NX or BX casing was driven as required and then washed clean.

In cohesionless sand and gravel soils, a standard 2-inch O.D. split tube sampler was used to obtain specimens. This split sampler was driven into the undisturbed soil ahead of the boring by a 140 lb. hammer dropping 30 inches. The number of blows required to drive the sampler one foot after an initial penetration of 6 inches was recorded as the penetration resistance of the soil at the depth of sampling. On withdrawal, the sampler was dismantled and the soil identified.

In clay soils, relatively undisturbed specimens were recovered in thin-walled 2-inch I.D. shelly tubes. These tubes were either levered or driven into the ground using the same energy as for the split sampler. The soil was jacked from the tubes in the laboratory. The moisture content and Atterberg limits of each sample were determined. Samples from hole 15 were subjected to compression tests under conditions of lateral pressure equal to the total overburden pressure that existed at their respective positions in the ground. The results of this laboratory work are summarized on Dwg. 12. The actual test curves for the shear strength determinations are plotted on Dwg. 13.

A dynamic cone penetration test was performed adjacent to many of the holes. This test consists of driving a 2-inch diameter 60° cone into the ground from the surface. A driving energy of 350 ft.lbs. is used and the hammer blows for each foot of penetration are recorded. This information supplements the results of the standard penetration test performed at the sampling intervals.

The results of the field and laboratory work are plotted on the logs prepared for each boring - Dwgs. 3 to 11.

PROJECT NO. J 516

DRAWING NO. 3

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Underpass Ramp "M"

LOCATION Chedoke Expressway

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION AND DATUM 314.6 S. Arm of 1st.

Hydrant N. of High Level Bridge, W. Side York = 344.72

BOREHOLE NO. 1

FIELD SUPERVISOR

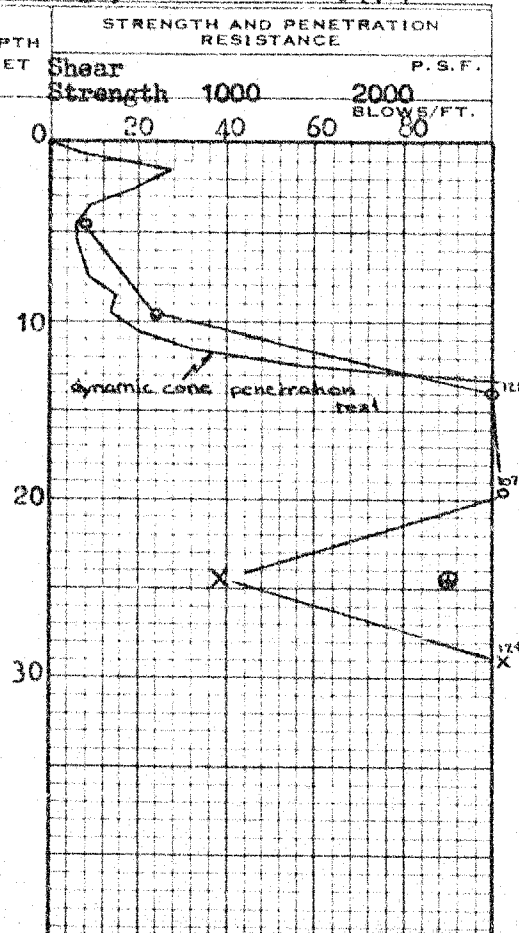
DRILLER

PREP.

LEGEND

- 2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (Qu)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				Shear Strength	P. S. F. BLOWS/FT.
	6" Topsoil	314.6	0	1000	2000
	Brown well graded sand with fine to coarse gravel-dense below 13 feet.		0 to 13		
	0.2" to 5" seams & layers of clean red clay & brown silty clay noted below 22 ft.		22 to 30		
	End of hole	285	30		
Notes: 1) Boring by continuous flight auger, hole uncased to full depth. 2) Sampler & cone driven under energy of 350 ft. lbs. per blow. 3) Hole becomes wet at 21 ft., caves to 20 1/2 ft. on completion.					



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.		
	1	
	2	
	3	
	4	
	5	128
	6	

PROJECT NO. J 516

DRAWING NO.

4

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SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Underpass Ramp "M"

LOCATION Chedoke Expressway

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION AND DATUM 304.6 BM as in hole 1.

BOREHOLE NO. 2

FIELD SUPERVISOR

DRILLER

PREP.

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (QU)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				P.S.F.	
	6" Topsoil	304.6	0	20 40 60 80 BLOWS/FT.	
	Reddish brown fine sand with fine to med. gravel sizes.				
	1 ft. layer of red silty fine sand from 8 1/2 to 9 1/2 ft.		10	dynamic cone penetration test	
	Alternate layers of brown & reddish brown sand with thin seams of silt and silty clay. Dense below 13 ft.				
			20		
	End of hole	274.6	30		

Notes: 1) & 2) as in hole 1.
3) Trace of water at 30'

CONSISTENCY		SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.			
		1	
		2	
		3	
		4	
		5	
		6	

PROJECT NO. J 516

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Underpass Ramp "M"

LOCATION Chedoke Expressway

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION AND DATUM 299.6 BM as in hole 1.

BOREHOLE NO. 3

FIELD SUPERVISOR

DRILLER

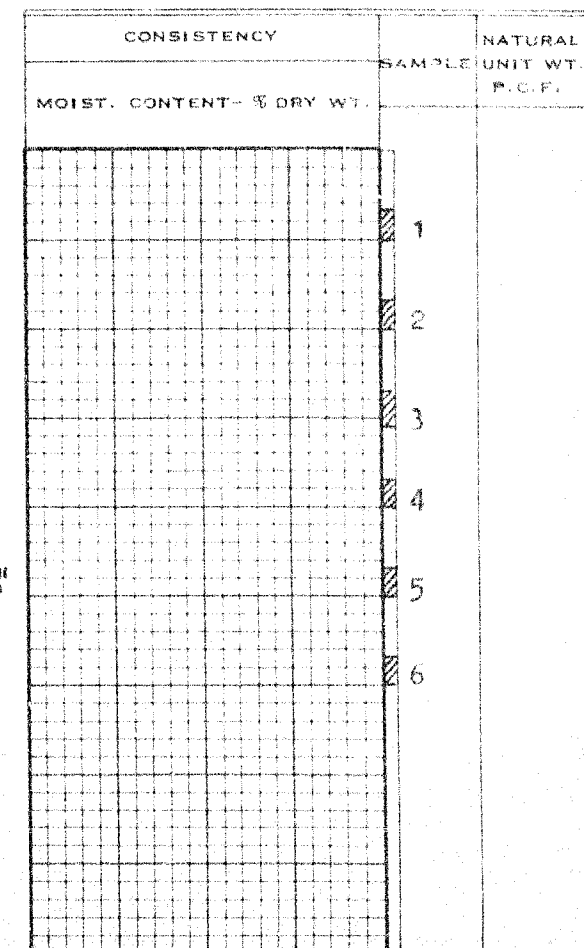
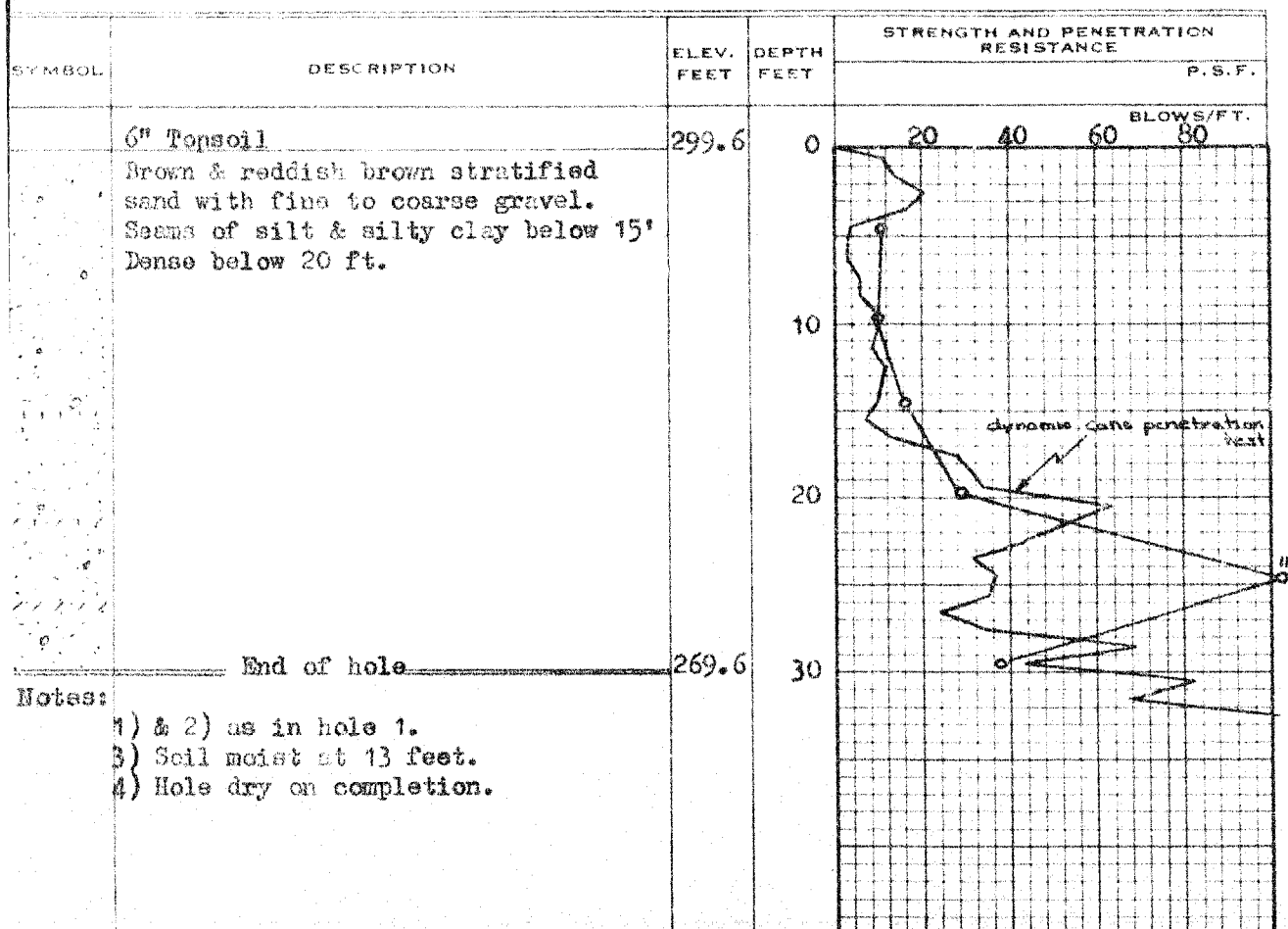
PREP.

DRAWING NO.

5

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (Qu)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



Notes:

- 1) & 2) as in hole 1.
- 3) Soil moist at 13 feet.
- 4) Hole dry on completion.

PROJECT NO. J 516

DRAWING NO. 6

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SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Underpays Ramp "H"

LOCATION Chedoke Expressway

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION AND DATUM 296.4 BW as in hole 1.

BOREHOLE NO. 4

FIELD SUPERVISOR

DRILLER

PREP.

LEGEND

- 2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				P. S. F.	
	4" of concrete at surface	296.4	0	BLOWS/FT.	
	Wet brown silty fine sand to 5 ft. changing to reddish brown & brown stratified sand with fine to coarse gravel sizes.		0		
	Dense below 5 ft.		10		
	Seams of silt & silty clay below 13'.		20		
	End of hole	266.4	30		
Notes:	1) & 2) as in hole 1.				
	3) Water enters hole from silty sand at surface.				

[illegible]

PROJECT NO. J 516

DRAWING NO.

7

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Underpass Ramp "M"

LOCATION Chadoke Expressway

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION AND DATUM 304.5 BM as in hole 1.

BOREHOLE NO. 5

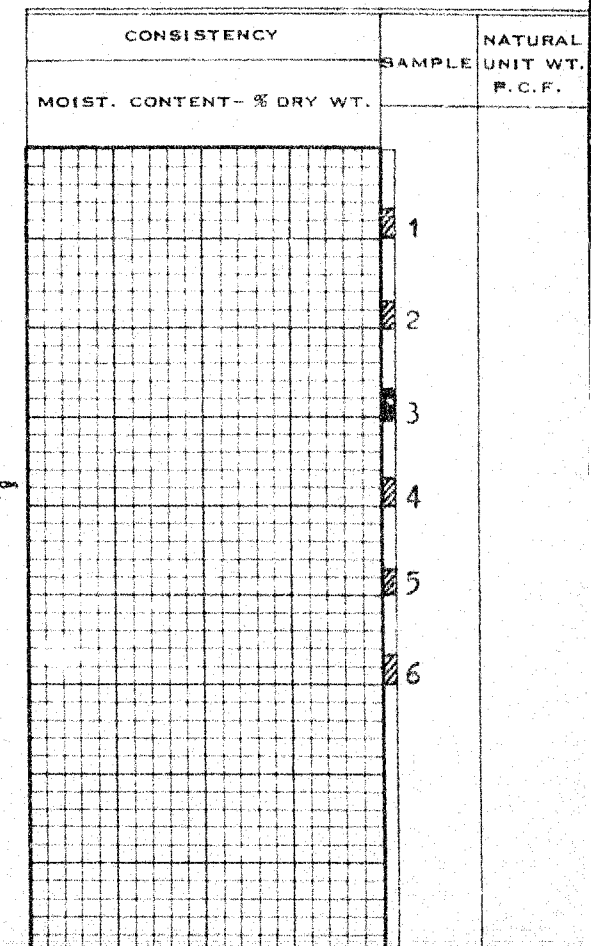
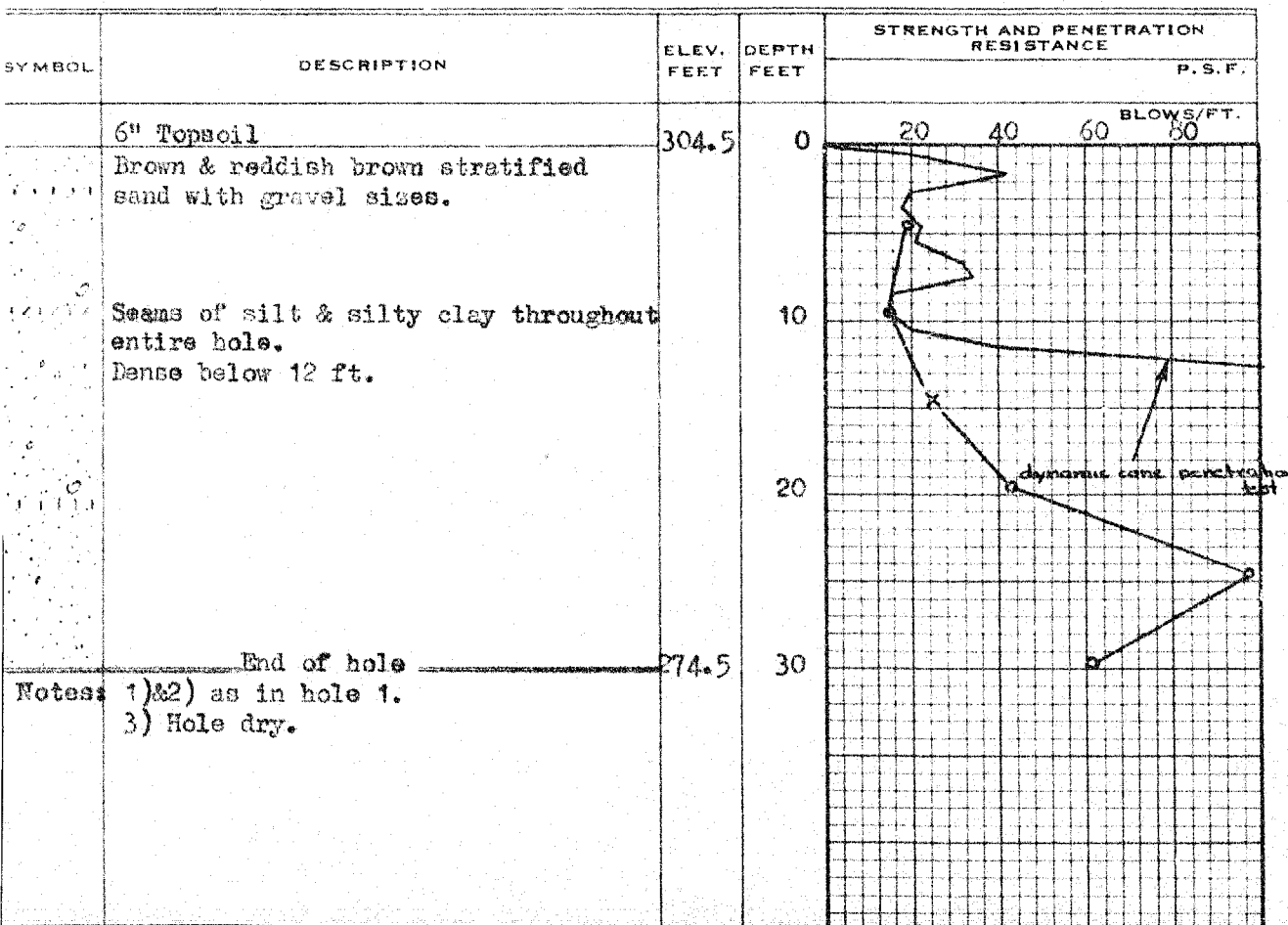
FIELD SUPERVISOR

DRILLER

PREP.

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (QU)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



Notes: 1)&2) as in hole 1.
 3) Hole dry.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING NO. 10
PROJECT NO. J319

LEGEND

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
2" I.D. SHELBY TUBE *—*—*—*—
2" DIA. CONE ————

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) +^s

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT —○—

PLASTIC LIMIT ———

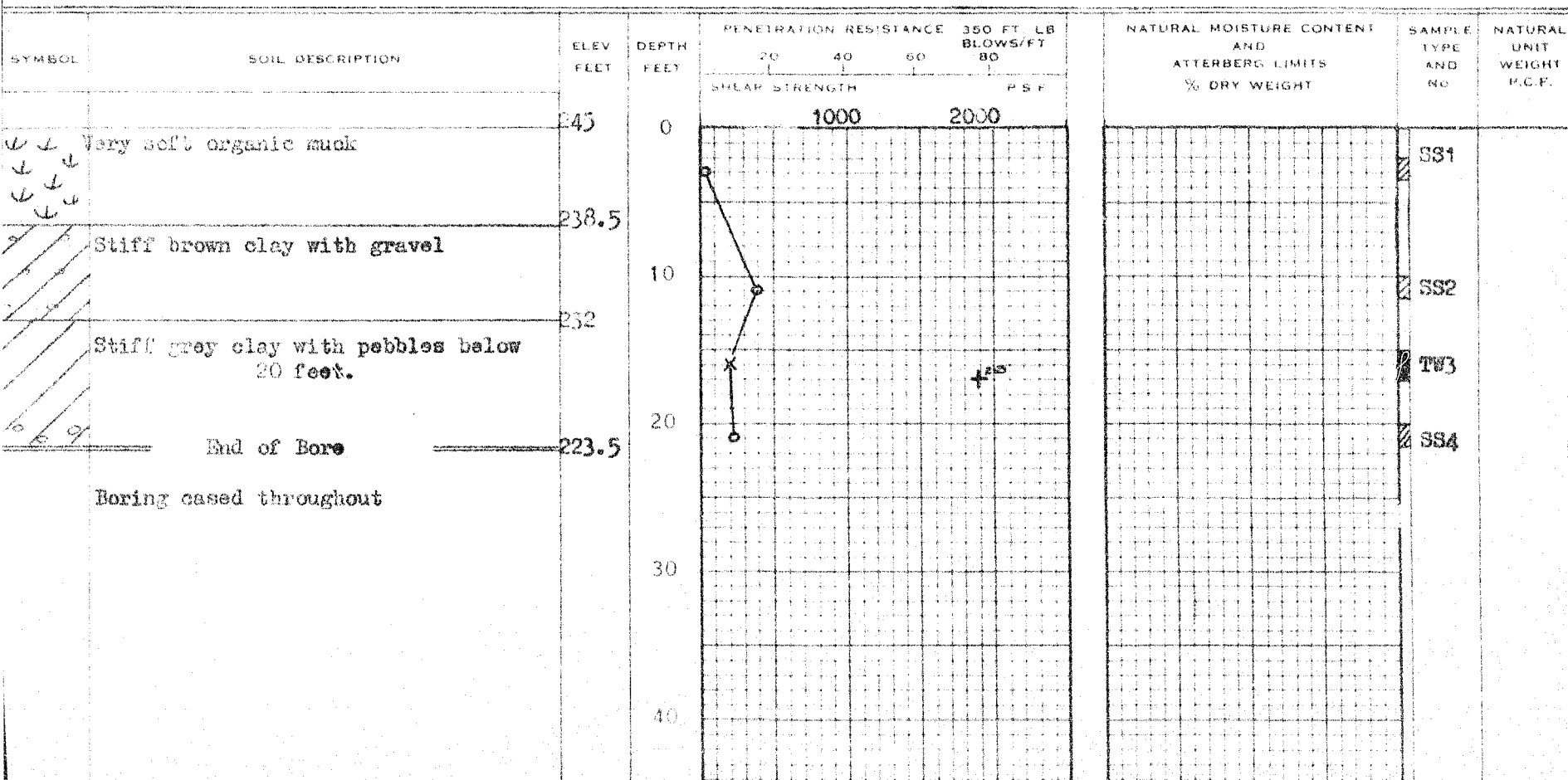
SAMPLE TYPE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

3" O.D. SHELBY TUBE

BOREHOLE NO. 8
PROJECT Chedoke Expressway
LOCATION South From Wolf Island Interchange
HOLE LOCATION See borehole location plan
HOLE ELEVATION
DATUM



PROJECT NO. J465

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Chelake Expressway
LOCATION Cootes Paradise

HOLE LOCATION

HOLE ELEVATION AND DATUM 245.0

BOREHOLE NO. 9A

FIELD SUPERVISOR

DRILLER

PREP.

DRAWING NO. 11

LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE		
				Sheet Str.	1000 P.S.	2000 P.S.
	Ice surface water	245.0	0			
		244.4				
	Soft dark brown organic silt					
		239.5				
	Brown silty fine sand with gravel sizes-some root fibres, slightly cohesive	237.0				
	Coarse brown sand and gravel up to 1/2 ins.	232.5	10			
	Stiff gray silty clay with fine gravel sizes.		20			
	End of hole	217.	30			
	NOTES: 1) Boring by wet sampling method. 2) Hole cased with 3-in. pipe to 20 ft.; wash ahead with AX pipe below this level to advance hole. 3) All Shelby tubes levered into ground except where noted.					

CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.		
	1	
	2	
	3	125.3
	4	128.9
	5	125.3

BOREHOLE No. 15
 PROJECT Ramp "A"
 LOCATION Chedoke Expressway, Hamilton
 HOLE LOCATION See Drawing 1A
 HOLE ELEVATION 248.5
 DATUM Elev. from 1" = 40' topography sheet

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
 2" I.D. SHELBY TUBE —x—x—x—x—
 2" DIA. CONE ————

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
 UNCONFINED COMPRESSION ⊗
 VANE TEST AND SENSITIVITY (S) †

NATURAL MOISTURE CONTENT
 AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT —○—

PLASTIC LIMIT ———

SAMPLE TYPE

2" O.D. SPLIT TUBE —○—

2" I.D. SHELBY TUBE —x—

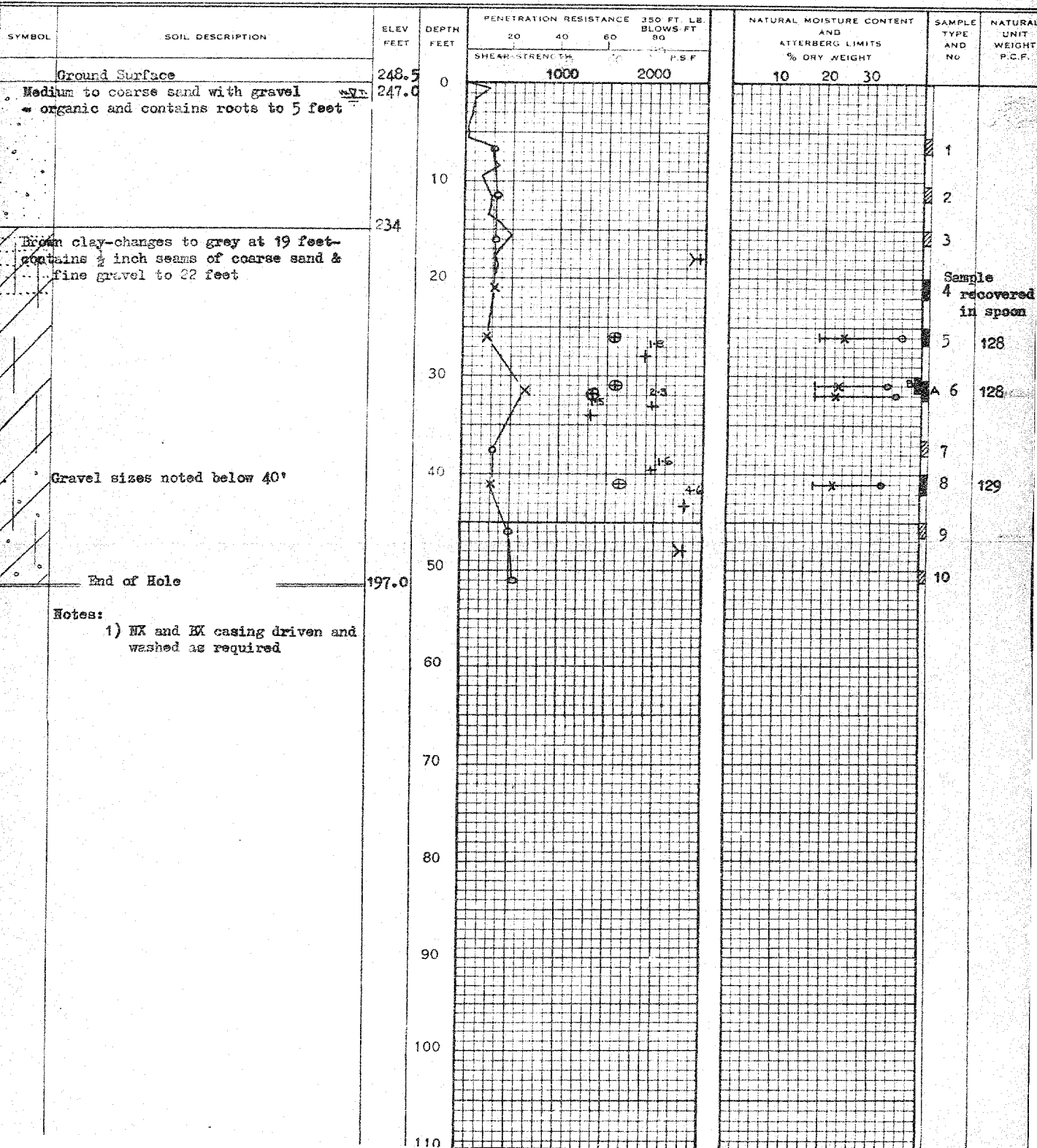
3" O.D. SHELBY TUBE —■—

X LI

—○—

—x—

—■—



BOREROLE NO. 16
 PROJECT Road "M"
 LOCATION Chedoke, Expressway, Hamilton
 HOLE LOCATION See Dwg. 1
 HOLE ELEVATION 253.0
 DATUM Elev. from 1"-40' Topography Sheet

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—
 2" I.D. SHELBY TUBE —+—+—+—
 3" DIA. CONE ————

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
 UNCONFINED COMPRESSION ⊗
 VANE TEST AND SENSITIVITY 15, 15

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

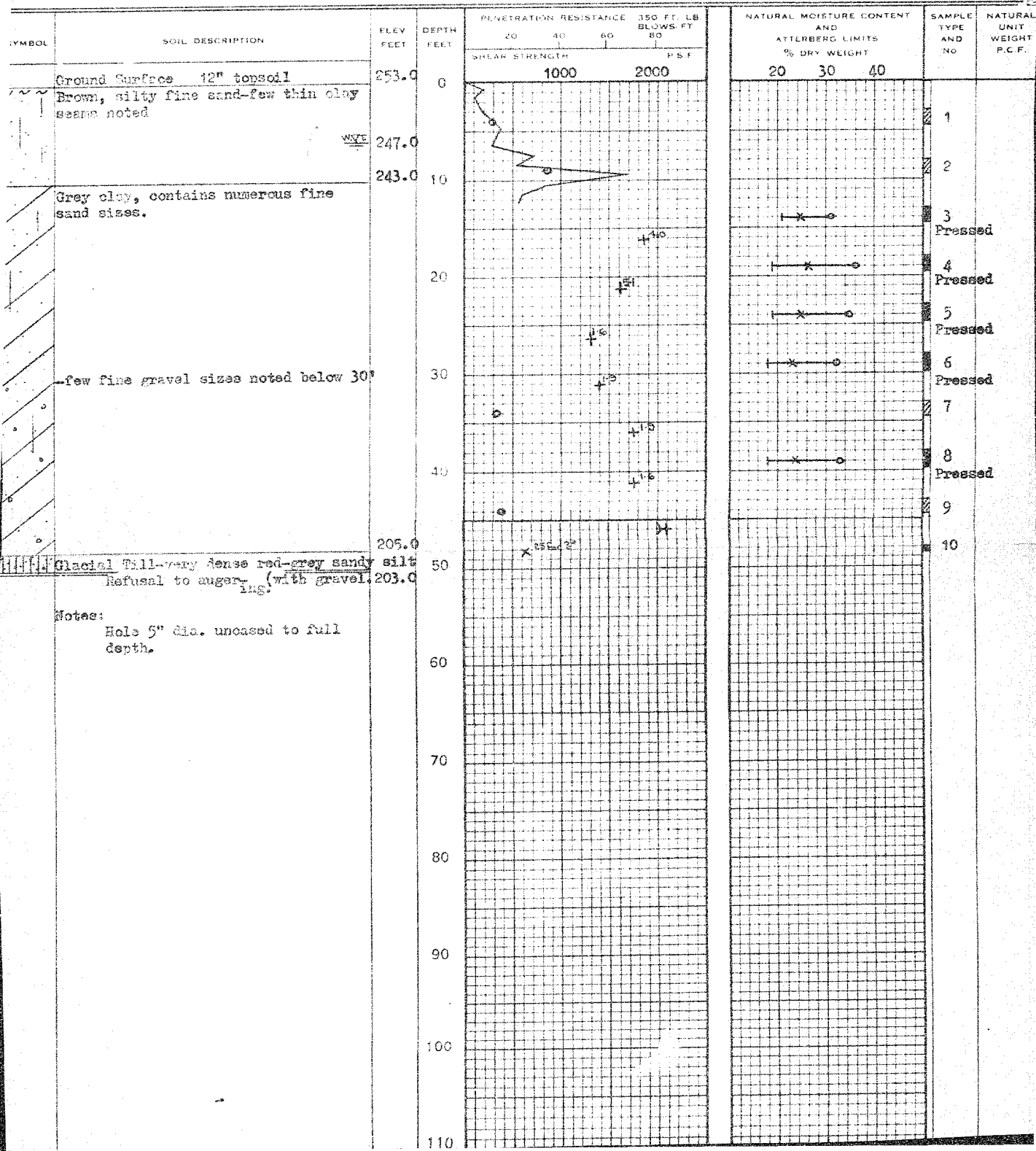
ATTERBERG LIMITS

LIQUID LIMIT —○—

PLASTIC LIMIT ———

SAMPLE TYPE

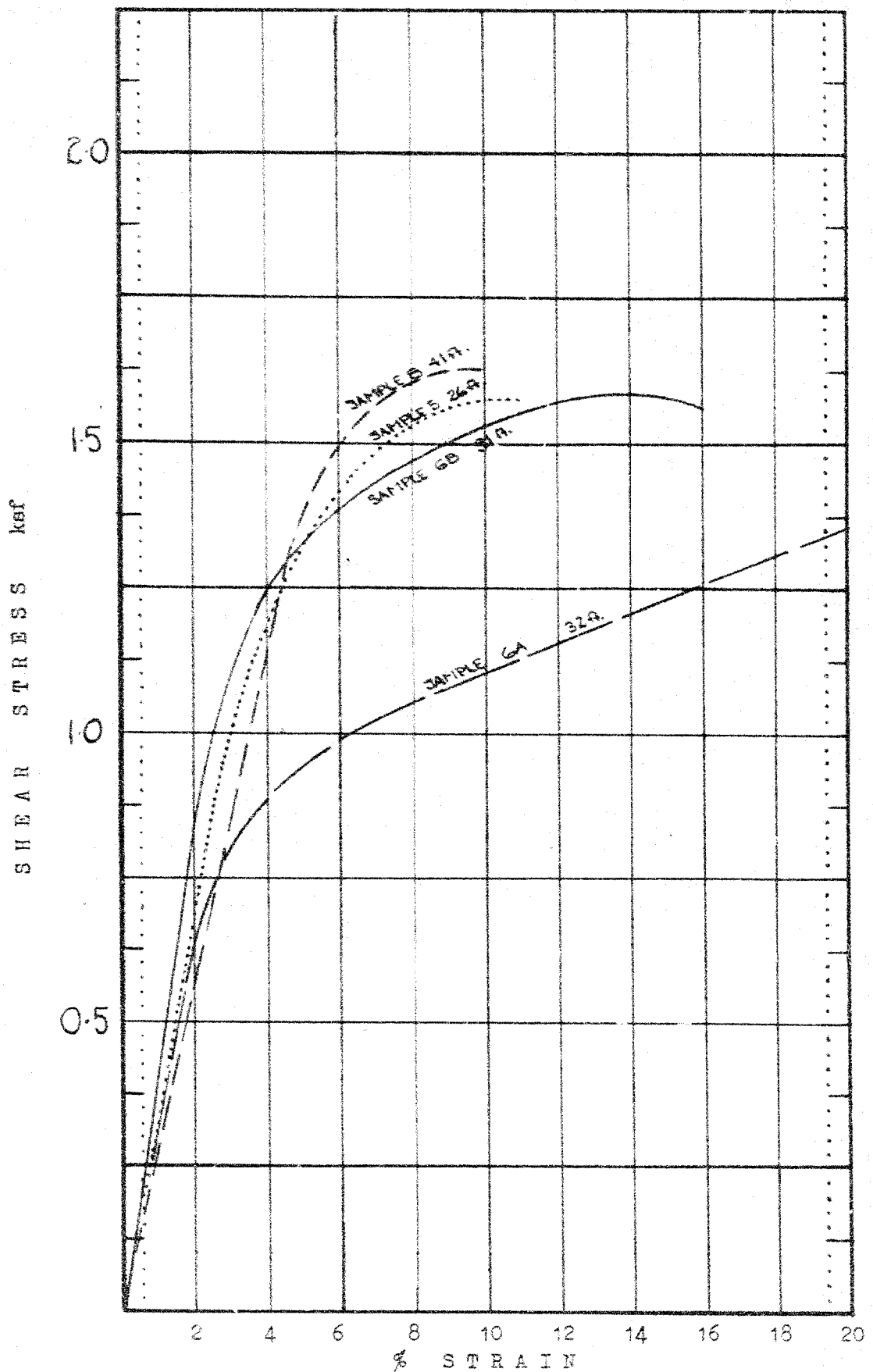
2" O.D. SPLIT TUBE —○—
 2" I.D. SHELBY TUBE —+—
 3" O.D. SHELBY TUBE —■—



LABORATORY REPORT ON UNDISTURBED SHELBY SAMPLES

Hole Number	Sample	Depth Feet	Description of Sample	Natural Moisture Content*	Liquid Limit*	Plastic Limit*	Natural Unit Weight	Shear Strength p.s.f. Undrained Triaxial
				Content*	Limit*	Limit*	Weight	Undrained Triaxial
15	5	25'0"-26'8"	Grey, slightly silty clay, stiff	24.0	36	19	128	1570
	6A	30'6"-32'6"	Stiff, grey, clay, few gravel sizes	22.3	35	18	127	1350
	6B	30'0"-31'3"	Stiff, grey, clay	23.0	33	18	128	1580
	6	40'0"-41'9"	Stiff grey clay	21.8	32	18	129	1620
16	3	13'0"-14'9"	Stiff grey clay	24.6	31	21		
	4	18'0"-19'9"	Grey clay	26.1	36	19		
	5	23'0"-24'4"	Grey clay	24.4	35	19		
	6	28'0"-29'9"	Grey clay	23.0	32	18		
	8	38'1"-39'9"	Grey clay with few gravel sizes	23.6	33	18		

* Percent Dry Weight



TRIAXIAL TEST CURVES - SAMPLES FROM HOLE 15

FEB. 1961

12^{3/4}" O.D. TUBE PILES

E & W ABUTMENTS

SOIL DATA - DENSE SAND

MEYERHOF $Q_f = 4N A_p + \bar{N} A_b / 50$

E.L.O.	E. ABUTMENT W. ABUTMENT	
	N	N
300	20	
295	15	12
290		10
285	22	18
280	30	24
275	60	111
270		40

W. ABUTMENT

$N = 40$
 $A_p = 0.886$

$\bar{N} = 35$
 $A_b = 100$

$Q_f = 142 + 70 = 212$

E. ABUTMENT

$N = 60$
 $A_p = 0.886$

$\bar{N} = 40$
 $A_b = 100$

$Q_f = 213 + 90 = 303$

A DESIGN LOAD OF 50 TONS / PILE IS IN ORDER

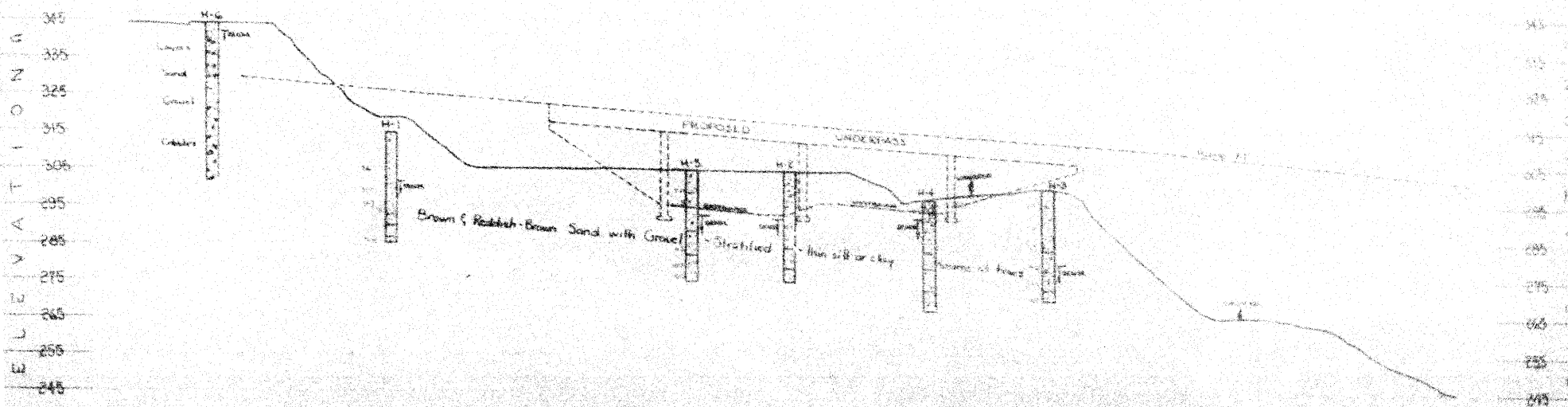
cont 61-174
 Block 23

PLAN

Scale 1"=40'

Spot markers from Checkers plan 1943 plotted on ground plan as shown

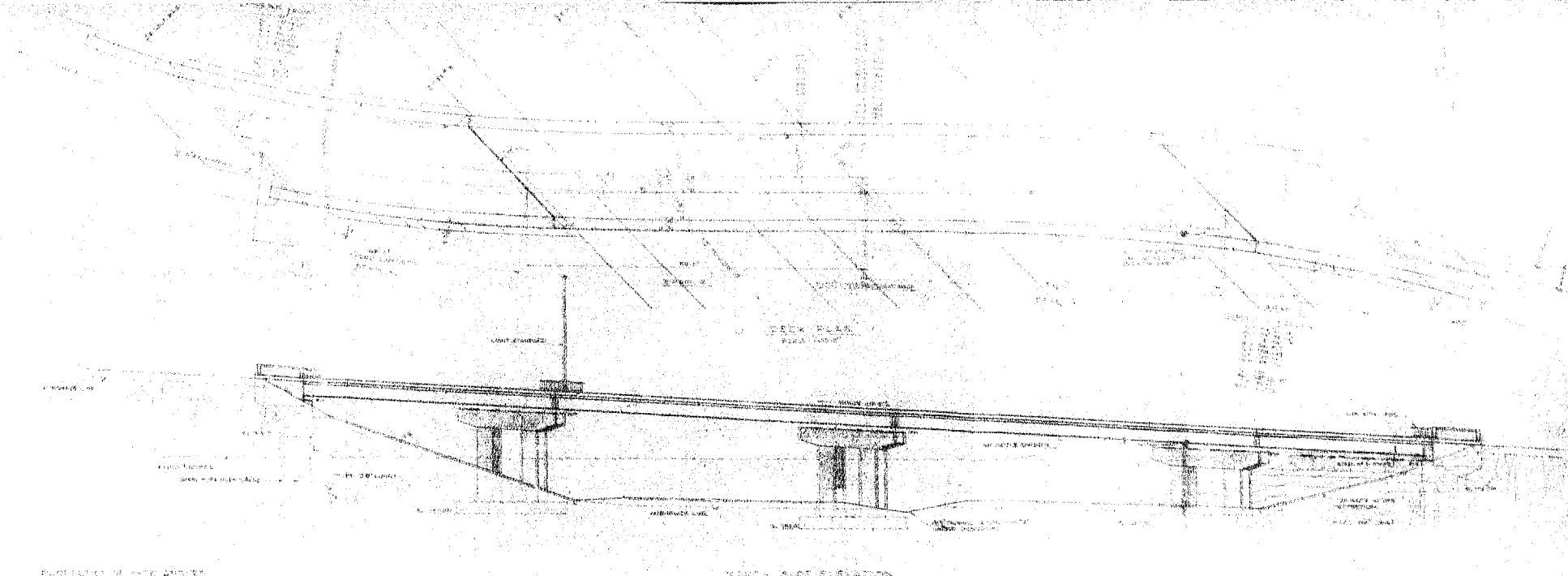
23-6-174-5



ESTIMATED SUBSOIL STRATIGRAPHY

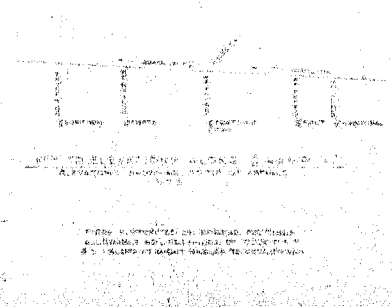
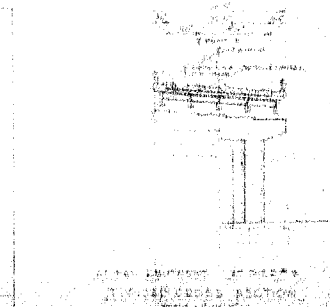
Scale: Hor. 1"=40' Vert. 1"=20'

23-61-174-G



Summary of Bridge Data

Span	Length	Area	Volume
1	100.00	100.00	100.00
2	100.00	100.00	100.00
3	100.00	100.00	100.00
4	100.00	100.00	100.00
5	100.00	100.00	100.00
6	100.00	100.00	100.00
7	100.00	100.00	100.00
8	100.00	100.00	100.00
9	100.00	100.00	100.00
10	100.00	100.00	100.00



DEPARTMENT OF HIGHWAYS, ONTARIO
BRIDGE OFFICE, TORONTO

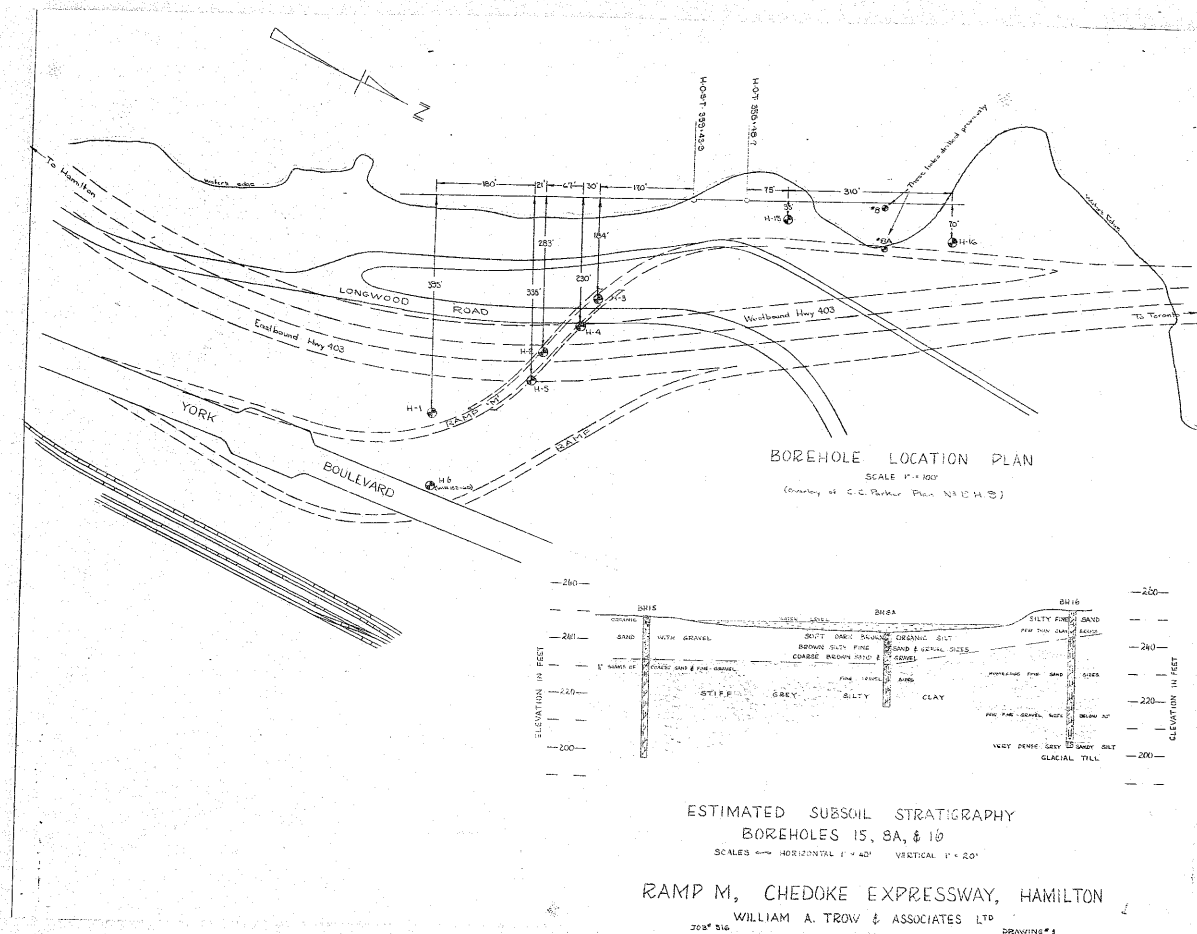
CHECKED BY: [Signature]
UNDERPASS AT RAMP

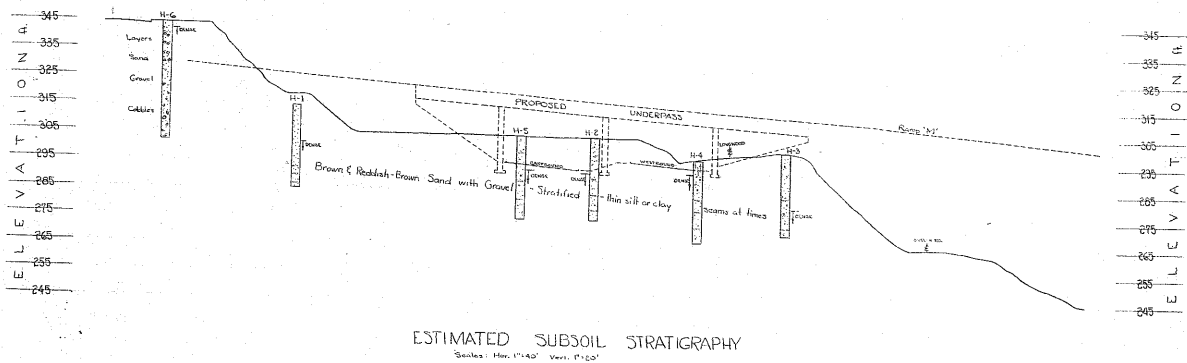
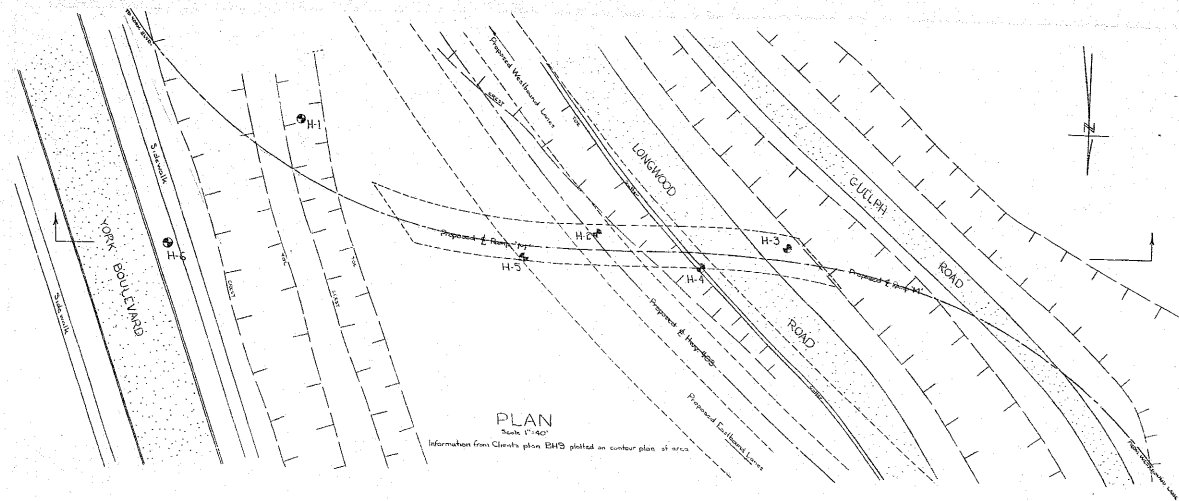
THE SPAN SHOWN IS AT: [Blank]
AND IS NOT TO SCALE

DATE: MAY - 9 1983

BY: [Signature]

DATE: [Blank]





W^m. A. Trow & Associates, Ltd.
DRAWING 2