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DEPARTMENT OF HIGHWAYS OF ONTARIO

HIGHWAY 401 - C.P.R. CROSSING

W.P. 68-57 TOWNSHIP OF CORNWALL

57-F-242C

SOILS REPORT

by

E. M. PETO ASSOCIATES LTD.

TORONTO, ONTARIO

May, 1956

AN INDEPENDENT ORGANIZATION PROVIDING A COMPLETE SOIL ENGINEERING & INVESTIGATION SERVICE

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 57148

850 roselawn avenue,
TORONTO, ONTARIO.
RUssell 1 - 4955.

May 2nd, 1958.

Mr. A. M. Toye,
Chief Bridge Engineer,
Department of Highways of Ontario,
280 Davenport Road,
Toronto, Ontario.

Attention: Mr. J. C. McAllister

Re: Site Investigation,
Highway 401 - C.P.R. Crossing
Cornwall, Ontario.

Dear Sirs:

We are enclosing herewith four (4) copies of our report on this site for your attention. We were authorized by Mr. J. C. McAllister on December 9th, 1957, to carry out a soil investigation at this site. These instructions provided for the inclusion of an investigation of the stability of the approach embankments.

We have considered the site conditions in detail in the soils report attached hereto together with supporting Appendices. Here for your convenience we are summarizing our findings and recommendations as follows:

1. a) The soil conditions on this site consist of:
A layer of peat and organic matter for a depth of approximately 7 feet.

b) A stratum of saturated silty clay of low strength and of variable thickness.

c) A stratum of generally compact sandy till with a thickness varying inversely with the clay stratum above.

d) Limestone bedrock with a general inclination of less than 5°.

2. The water table is virtually at ground surface.

3. Consideration of the soil conditions on this site have led us to form the opinion that the bridge and embankment can be constructed subject to the exercise of certain precautions in regard to the method and order of construction as follows:

a) The highly organic peat should be stripped over the area to be occupied by the new embankments and replaced with clean granular material.

b) In order to control the consolidation of the silty clay stratum beneath the embankments, and with the view to accelerating the rate of construction the installation of vertical sand drains, below the granular material and into the silty clay stratum is considered necessary. These drains will need to be augered, to a minimum diameter of 16 inches and located approximately on a 20 foot grid.

This recommendation does not exclude the alternative employment of stage loading over a longer period, without the use of sand drains, which may be considered more appropriate on the grounds of economy.

c) The side slopes of the approach embankments adjacent to the railway should be constructed with an inclination of 3:1 rather than 2:1 as presently contemplated.

Alternatively 2:1 side slopes may be constructed subject to the provision of toe berms 12 feet high for a distance of 30 feet beyond the toe of the 2:1 side slopes.

Despite the recommendation of 3:1 side slopes or the alternative of a toe berm, we feel that the rate of accelerated construction should be controlled by the installation of piezometer tubes within the immediate area of the embankments.

d) The sandy till stratum overlying bedrock is not considered suitable as a load carrying medium for a heavy bridge structure. Accordingly the bridge should be founded on bedrock and to this end we recommend the adoption of steel "H" piles for this purpose.

d) (Cont'd)

These should be driven after the placing of the granular material recommended in (a) above, and before the construction of the embankment. This type of pile will reduce disturbance of the silty clay stratum to a minimum and will facilitate penetration of the sandy till layer where some resistance can be anticipated; and in view of the general site conditions hard driving of the steel "H" piles into bedrock is not desirable or necessary.

This site presents a number of problems which you may well like to discuss further and accordingly we shall be pleased to attend for any such discussion at your convenience.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,

E. M. Peto

CFF:sb

E. M. Peto, P. Eng.

HIGHWAY 401 - C.P.R. CROSSING

Contents

e. m. peto associates ltd., 850 roselawn avenue, Toronto 10, Ontario

Job No. 57148 Client's Ref. No. Date May 1st, 1958.

Report on

SOIL CONDITIONS

at

HIGHWAY 401 - C.P.R. CROSSING

W.P. 68-57 TOWNSHIP OF CORNWALL

for

DEPARTMENT OF HIGHWAYS OF ONTARIO

INTRODUCTION:

We were authorised to proceed with this investigation by Mr. J. C. McAllister under cover of his letter dated December 9th, 1957. Included were the following documents; a) an unnumbered plan of the site on which were marked six tentative borehole locations, b) portion of profile P-3165-14 and c) copy of letter from Mr. F. C. Brownridge, Materials and Research Engineer to Mr. McAllister, extending the foundation investigation to include the approach fills to the structure.

PROGRAMME OF WORK:

December 11th, 1957. Initial reconnaissance of site.

January 27th, 1958. Test holes staked out and water supply problems resolved, and field elevations taken.

January 28th, 1958. Field work commenced on site by unit moved from completed Raisin River Bridge investigation.

February 13th, 1958. Field work completed, Heavy snowfall during final week's work.

February 14th, 1958. Equipment removed and returned to Toronto.

GENERAL:

a) Standard soil sampling procedures were followed. These are described in Appendix "A".

b) Two test holes on either side of the railway tracks were diamond drilled (AX size) to prove reliability and continuity of bedrock. Most of the other holes were terminated at casing refusal depth.

c) In every test hole, after penetration of the frozen crust, ground water was observed to be within one or two feet of ground surface. Therefore only a limited number of further ground water level readings were taken.

d) Laboratory test results and detailed borehole logs are attached, together with a site plan showing the location of the test holes. The pier and abutment locations shown on the plan are preliminary and subject to revision. Elevations are to Geodetic datum referred to a D.H.O. bench mark in the form of a nail in the top of a 5" Maple stump 200 ft. right of station 541 + 30. The elevation of this bench mark was taken to be 184.37.

e) On completion of the field work we were advised that twin bridges were being considered for this location, to carry East and Westbound traffic respectively.

The two structures would be totally independent and separated by a boulevard. The transverse centre lines of the two bridges would not coincide with the centre line of the present C.P.R. tracks, since we understand that another set of tracks will be added in the future. It is unfortunate that these plans were not known earlier, before the field work was complete.

Our test holes were based on, or centred around line "G". It is now proposed that the longitudinal centre lines of the twin bridges will probably be located 42 feet North and South of Line "G". With an embankment height of 30 feet, the top and base width of the approach fill would be of the order of 130 and 250 feet respectively, and these figures have been used for our soil stress analyses.

SITE AND GEOLOGY:

Although the general area is located in the physiographic region known as the Glengarry till plain, specialized soil conditions exist at this site. The topography at the site is depressional, and the natural drainage is very poor. The surface soil type which has developed, as a consequence, is highly organic peat, classified agriculturally as muck. Underlying the peat is a thickness of marine clay, and beneath this is a sandy till material which is not generally dense, except immediately adjacent to the bedrock.

The sandy till directly overlies bedrock, which occurs at a depth of approximately 22 feet. The underlying bedrock is Black River limestone.

SOIL CONDITIONS:

The site under consideration is level, and soil conditions are uniform as shown on the attached plot of the soil stratigraphy. Only three main soil types were encountered above bedrock on this site, viz; a) highly organic peat, b) very soft silty clay and c) compact to dense sandy till.

a) Highly Organic Peat

Overlying the site at all points tested, but possibly not existing below the present railway grade due to excavation in the past, is a stratum of dark brown highly organic peat. This material was found to range in thickness from 4'4" up to 8'3", with 7 feet being a good average value. The upper three or four feet of peat, although frozen at the time of our investigation, may actually be in a semi-liquid condition in the spring. Below this, the peat is very loose. During early February, however, it was supersaturated throughout, with natural moisture contents in the order of 400%. It is stone free, and the organic material is well decomposed. Near the lower boundary of the peat at many points, minor pockets of silty clay have intruded, and were encountered during the site investigation.

b) Very Soft Silty Clay

Directly underlying the organic peat is a stratum of grey, very soft silty clay. The clay varies in thickness from 4'4" to 18'6", generally being much thicker on the West approach to the proposed bridge. The average thickness of the clay is 10-1/2 feet.

SOIL CONDITIONS:

b) Very Soft Silty Clay (cont'd)

At the immediate upper boundary of the silty clay stratum, the clay material contains fairly large amounts of sand, and tends to a bluish-grey colour. At the lower boundary of the clay stratum, the material is horizontally stratified, and contains very thin seams of fine light grey sand, spaced at roughly 1" intervals. The major intermediate portion of the silty clay stratum contains no foreign matter.

The clay fraction of this material is quite high, as shown by a hydrometer grain size test carried out in the laboratory. The grain size distribution determined is 50% clay, 43% silt, and 7% fine sand.

A large number of natural moisture content determinations were made throughout the clay stratum, and were correlated with the results of two consistency limit tests. The Liquid Limit, Plastic Limit, and Plasticity Index of the silty clay are in the order of 60, 23, and 37 respectively. The natural moisture contents are generally in excess of the Liquid Limits, a very undesirable condition.

The very low strengths, high natural moisture contents in excess of the Liquid Limit, and high compressive index as calculated from the Atterberg Limit tests, suggest that the silty clay material has not been preconsolidated to any extent in the past. However, a consolidation test on a clay specimen of similar characteristics at the adjacent Cornwall C.N.R. - Highway 401 crossing, which must be considered to have had the same geologic history, showed that the silty clay has, in point of fact, been subjected to a preconsolidation pressure of at least 3.0 kips per sq. foot.

The silty clay has a very pronounced nuggetty and blocky texture at some points, particularly beneath the sandy portion at the top boundary. This might be attributable in part to the leaching out of some of the soluble salts from the saline pore water of the clay, although it is more likely that the nuggetty, fissured texture of the silty clay material developed principally as a consequence of the removal of this preconsolidation load. We believe that the present nuggetty texture of this material is the reason for its behaviour as though it were only normally consolidated.

SOIL CONDITIONS:

b) Very Soft Silty Clay (Cont'd)

The clay is certainly fully saturated, and the consolidation test revealed that it may even be supersaturated at some points.

The silty clay is exceptionally weak. Thirteen separate unconfined compressive strength tests on undisturbed samples gave values ranging from only 90 lbs. per square foot to 748 lbs. per square foot. It is a reasonable assumption that this clay has very little, if any, effective internal friction. A quick undrained triaxial test performed with lateral pressures approximating the effective soil stress, plus probable preconsolidation load on the particular sample tested, bears out the results of the unconfined compression tests. The values determined from this triaxial test are: cohesion = 209 p.s.f., and angle of internal friction = 3-1/2°.

All samples of the clay material were much too wet and sticky for proper remoulding, and therefore no remoulded strength tests could be carried out. However, in our opinion the sensitivity can be closely approximated by using Bjerrum's correlation of sensitivity and liquidity index for marine clays. This has been done, and the senstivities were found to vary from 4.4 at the upper boundary of the silty clay stratum to 10.0 in the greater portion of the layer.

The wet density of a typical grey silty clay sample from borehole 11 was found to be 144.0 lbs. per cubic foot, and the dry density was computed to be 78.9 lbs. per cubic foot. Since the clay stratum is below the ground water table on this site, only the submerged effective unit weight of 82 lbs. per cubic foot has been used for our calculations.

c) Sandy Till

Beneath the grey silty clay, is a grey sandy material containing numerous angular limestone fragments, which we have designated as sandy till. The occurrence of the sandy till is exactly converse to the occurrence of the silty clay above. On the West approach where the clay stratum is thickest, there is very little sandy material between the clay and the bedrock; however, on the East approach to the bridge the sandy till attains its maximum thickness of roughly 11 feet.

SOIL CONDITIONS:

c) Sandy Till (Cont'd)

The sandy till is basically a fine to medium sand, with many grits and limestone fragments, and silt and clay binder. At some points there is a considerable increase in the binder content.

Standard penetration test results in this material ranged from a low of 13 blows to a high of over 100 blows per foot, the density tending to increase with depth.

The unit wet weight of the sandy till in situ is in the order of 132 lbs. per cubic foot.

d) Limestone Bedrock

Underlying this site at a depth of approximately 22 feet is the parent material, a Black River limestone. This material is dark grey in colour, fine-grained, and contains occasional fossils and thin seams of black shale. In all four of the test holes where rock cores were obtained the limestone was of excellent quality.

A rather important feature is the existence at approximately 18 to 24 inches below the upper surface of the limestone, of a thin water-bearing seam, from 1" to 6" in thickness.

The upper boundary of the limestone bedrock was determined to be between elevations 160 and 151; the centre line profile illustrates the relatively minor variations. The bedrock surface under the railway tracks appears to dip gently to the North-East.

WATER CONDITIONS:

The water table at the time of the investigation was within 2 feet of ground surface, which is attributable to the very poor surface drainage. It is unlikely that the water table will fluctuate to any extent, unless major remedial measures are undertaken, or a prolonged drought occurs.

ENGINEERING CONSIDERATIONS:

1. The highly organic peat is a totally undesirable material, because of its very high compressibility, and extremely low strength. There is no doubt that it must be removed over the area, which will be beneath the full base width of the embankment.

2. The grey silty clay, beneath the organic peat, is also an undesirable soil, because of its very low strength, and its behaviour as a highly compressible material. The excavation of this silty clay, however, will not be easy, due to the high water table.

   Based on the assumption that the 7 ft. of highly organic peat will be comletely stripped over the full base width of the proposed fill and replaced with clean granular material, we have analysed the shearing stresses imposed on the silty clay stratum by the 7 ft. of granular material and the full 30-foot height of embankment. The average laboratory shear strength of the clay stratum is 152 p.s.f. Using this figure, the ultimate shear strength under a strip load was calculated to be 780 p.s.f., according to the Prandtl theory.

   Under the West approach embankment, where there is an 18 foot thickness of silty clay (taking the worst case), the shear stress induced at the bottom of the clay stratum due to the combined surcharge and present overburden will be 2795 p.s.f., which is in excess of the laboratory ultimate strength, but which does not exceed the probable preconsolidation stress on this material.

   Under the East approach embankment, where there is only a 10 foot thickness of silty clay, the shear stress induced at the bottom of the clay stratum by the combined weight of the new embankment plus present overburden would be 1808 p.s.f., which is greater than the laboratory ultimate strength.

   Turning now to consideration of in situ shear strength tests results on similar clays at the nearby Highway 401 - C.N.R. crossing, we find the values obtained here are, in some cases, three to four times as high as the laboratory shear strengths. Furthermore, published reports of the results of very extensive tests on apparently virtually identical clay including deliberately conducted field failures by deep

ENGINEERING CONSIDERATIONS:

2. (Cont'd)

excavation, at the American Grass River locks on the St. Lawrence River Seaway, suggest that the in situ vane test results are too high. With this aspect of the problem in mind, it is as well to remember that all the laboratory tests give results in terms of total stresses rather than effective stresses, since these tests were all of the quick undrained, rather than, the slow type of test.

It would appear logical therefore from consideration of these varying results to assume that the true shear strength lies somewhere between these two sets of results, a reasonable compromise being a figure approaching twice the laboratory strength.

Accordingly, we have re-examined the embankment stability problem adopting a shear strength value double that given by the laboratory test results. Thus with a figure of C = 304 p.s.f. which is close to the actual field failure result obtained in the Grass lock tests, we have calculated that the embankment on the West approach would fail, and the East side embankment would be at the point of failure. Such failure would take the form of plastic flow in the silty clay under the embankment, with subsequent subsidence under the centre of the embankment. The existing ground surface beyond the toe of the slope would displace or heave.

3. Increasing the base width of the proposed embankment from 250 feet to 310 feet, thus reducing the side slopes from 2:1 to 3:1, would assist overall conditions by:

a) Strengthening the embankment, and minimize local bank failures which normally can be anticipated when settlement is occurring due to consolidation of the soft clay stratum.

b) Extend the surcharge effect of the shoulders of the full embankment, reducing the danger of local shear failures and relaxing the subsoil shear stress conditions under the railway tracks. By this means only the West embankment would approach failure, whilst the East embankment would be stable.

ENGINEERING CONSIDERATIONS: (Cont'd)

4. Consideration of the consolidation conditions at both the East and West approaches lead us to believe that the vertical pressures induced under the full embankment will be in the order of 6000 p.s.f. Based on our appreciation of the consolidation test results on the similar material from the nearby C.N.R. crossing site, this figure will far exceed the estimated preconsolidation load on this clay of 3500 p.s.f. and appreciable amounts of settlement will occur.

5. As a further means of assisting consolidation of the silty clay and enabling this stratum to accept the imposed load, consideration has been given to the incorporation of vertical sand drains or wells, below the embankment. These drains will provide additional means for the release of pore water pressure during consolidation; but it must be remembered that the topography of this site is depressional, and surface drainage is poor. Thus the operation of sand drains though advantageous to the purpose in mind, will not be so efficacious as they might appear to be at first sight, since both these drains and the granular backfill will become saturated with surface water.

Unfortunately any suggested spacing of the vertical sand drains is purely empirical. They cannot be spaced too closely or the disturbance of the fairly sensitive clay around them will more than counteract any beneficial effects. The actual spacing which should be used is most difficult to assess quantitatively, since it is dependent upon the permeability of the silty clay in a radial horizontal direction, the load superimposed on this clay, the three-dimensional consolidation characteristics of the clay, and the ratio of the diameters of the prismatic soil cylinder being drained to the diameter of the sand well. With due regard for these factors and taking into account the sensitivity of the clay, we suggest 16 inch diameter sand wells spaced on a 20 foot grid, giving an $\frac{R}{r}$ ratio of 15.

6. The sandy till stratum does not occur uniformly over the site, the densities are quite variable, and are generally only compact; accordingly this stratum has not been considered as a load carrying medium in view of the proximity of bedrock below.

E. M. PETO ASSOCIATES LTD.,

E. M. Peto, P. Eng.

EMP:sb

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401 - C.P.R. Crossing Job No. 57148 Borehole No. 1

Client Dept. of Highways of Ontario. Casing BX (2-1/2" diam.) Boring Date Feb. 5th-6th, 1958

Datum Geodetic. Compiled By M. Mindess Checked By E.M. Peto

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q_u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 182.1					GROUND FROZEN TO 3' DEPTH.
Highly organic peat.	Dark Brown	Semi-Liquid.			1	S.S.	PUSHED*	SATURATED.
			5' 0"					
As above					2	S.S.	PUSHED*	SUPERSATURATED.
			7' 0" 175.1					
Silty clay,	Grey	Very soft.			3 A B C	2" S.L.	PUSHED	NAT. M.C. = 52.0%
			10' 0"					
Silty clay	Grey	Very soft			4	S.S.	PUSHED	NAT. M.C. = 69.7%. Much wetter than Plastic Limit.
Silty clay, very nuggetty.	Grey	Very soft.			5 A B C	2" S.L.	PUSHED	Near Liquid Limit. Q_u 14'-14½' = 186 P.S.F. NAT. M.C. = 88.4%
			15' 0"					
Silty clay, some grits and very fine sand.	Grey	Almost in liquid condition.			6	S.S.	PUSHED	
			17' 0" 165.1					
Matrix of silty clay with many angular rock fragments.	Grey	Compact			7	S.S.	25	Saturated
			20' 0"					
Fine to medium sand, grits and pebbles.	Grey	Dense			8	S.S.	38	Wet
			22' 0" 160.1					
Fine grained limestone	Grey to Grey-Black	1" WATER-BEARING SEAM.			9	AXT R.C.		CORE RECOVERY = 96.7%
Fine grained limestone, occasional fossils and thin seems of black shale.	As above	Excellent quality throughout.	27' 0"					
					10	AXT R.C.		CORE RECOVERY = 100%
			32' 0" 150.1					

HOLE TERMINATED.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name: Hwy. 401 - C.P.R. Crossing Job No.: 57148 Borehole No.: 2

Client: Dept. of Highways of Ontario Casing: BX (2-1/2" diam.) Boring Date: Feb. 6 - 7th, 1958.

Datum: Geodetic Compiled By: M. Mindess Checked By: E.M. Peto

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
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- Q_u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVEL, SOIL MOISTURE & REMARKS
			0' 0" 181.3					GROUND FROZEN TO 3 FT DEPT
								* WITH TRAP VALVE IN BOTTO
Highly organic peat.	Dark Brown	Semi-Liquid			1	S.S.	PUSHED*	Supersaturated.
			5' 0"					
As above.	Dark Brown	Semi-liquid			2	S.S.	PUSHED*	SUPERSATURATED.
			7' 0" 174.3					
Silty clay	Grey	Very soft			3 A B C	2" S.L.	PUSHED	NAT. M.C.$_{9'}$ = 84.5% Wetter than Liquid Limit.
			10' 0"					
Silty clay, nuggetty texture.	Grey	Very soft.			4 A B C	2" S.L.	PUSHED	Q_u $_{10'-11\frac{1}{2}'}$ = 166 P.S.F. (2 TESTS). NAT. M.C.= 102.8%–97.0
As above					5	S.S.	PUSHED	NAT. M.C.$_{14'}$ = 70.0%
			15' 0"					
As above	Grey	Very soft			6 A B C	2" S.L.	PUSHED	NAT. M.C.$_{16\frac{1}{2}'}$ = 67.3%
			17' 0" 164.3					
Clay and silty sand, numerous grits and pebbles.	Grey	Compact to Dense			7	S.S.	30	Saturated.
			20' 0"					
Matrix of silty sand, with numerous angular limestone fragments.	Light Grey	Dense			8	S.S.	48	Saturated.
			22' 0" 159.3		CASING REFUSAL AT 22'.			
			24' 6" 156.8					

ABSOLUTE REFUSAL. BEDROCK.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE — TORONTO, ONTARIO

BOREHOLE LOG

Job Name: Hwy. 401 - C.P.R. Crossing Job No.: 57148 Borehole No.: 3

Client: Dept. of Highways of Ontario Casing: BX (2-1/2" diam.) Boring Date: Feb. 8-10th, 1958.

Datum: Geodetic Compiled By: M. Mindess Checked By: E. M. Peto

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
FAIR	S.L. SPLIT BARREL WITH LINERS	Q/u UNCONFINED COMPRESSIVE STRENGTH
DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 181.5					
Highly organic peat.	Dark Brown	Semi-Liquid	5' 0"		1	S.S.	PUSHED*	Supersaturated.
As above.	Dark Brown	Very loose	7' 9" 173.7		2	S.S.	PUSHED*	
Silty clay, nuggetty.	Grey	Very soft	10' 0"		3 A B C	2" S.L.	PUSHED	NAT. M.C. = 70.0 %.
Silty clay.	Grey	Very soft			4 A B C	2" S.L.	PUSHED	NAT. M.C. = 82.1 %.
Silty clay.	Grey	Very soft.	15' 0"		5	S.S.	PUSHED	NAT. M.C.$_{14'}$ = 95.4 % Wetter than Liquid Limit
As above.	Grey	Very soft.			6 A B C	2" S.L.	PUSHED	NAT. M.C.$_{15\frac{1}{2}'-16'}$ = 87.0 % $Q/u_{16'-16\frac{1}{2}'}$ = 207 P.S.F. NAT. M.C. = 88.3 %
As above. Fine to medium sand.	Light grey	Compact	18' 6" 163.0		7 A B	S.S.	24	Sand: Wet Clay: near Liquid Limit
Fine to medium sand, rock fragments up to 1-1/4"	Light grey	Compact			8	S.S.	13	Saturated.
			22' 10" 158.7 24' 1"					
Fine grained limestone, occasional thin seams of black shale. As above.	3" WATER-BEARING SEAM → Dark Grey	Excellent quality throughout.			9	AX R.C.	—	CORE RECOVERY VIRTUALLY 100 %.
Fine grained shale, many tiny fossils.	Black		31' 0" 32' 3"		10	AX R.C.	—	
Fine grained limestone.	Grey		33' 0" 148.5					

HOLE TERMINATED.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name: Hwy. 401 – C.P.R. Crossing
Job No.: 57148
Borehole No.: 4
Client: Dept. of Highways of Ontario
Casing: BX (2-1/2" diam.)
Boring Date: Feb. 4th-5th, 1958.
Datum: Geodetic
Compiled By: M. Mindess
Checked By: E. M. Peto

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
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SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 181.2					
Highly organic peat.	Dark Brown	Semi-Liquid			1	S.S.	PUSHED*	*WITH TRAP VALVE IN BOTTOM OF SAMPLER.
			5' 0"					SUPERSATURATED.
As above, mixed with some grey silty clay.	Dark Brown	Very loose.	6' 6" 174.7		2	S.S.	PUSHED*	NAT. M.C. OF CLAY AT 6 FT = 522
Silty clay, nuggetty	Grey	Very soft.	10' 0"		3 A B C	2" S.L.	PUSHED	$Q_{u\ 8'-8\frac{1}{2}'}$ = 468 p.s.f. NAT. M.C = 87.7% NAT. M.C. $_{9\frac{1}{2}'}$ = 92.6%
Silty clay	Grey	Very soft	11' 0" 170.2		4	S.S.	PUSHED	NAT. M.C. = 79.1% Wetter than Liquid Limit
Broken limestone fragments	Grey-Black	Dense	15' 0"		5	S.S.	33	WET
Silty medium to coarse sand, with broken rock fragments.	Grey to Grey Black	Compact			6	S.S.	29	"
Silty fine to medium sand, with many grits and rock fragments.	Grey	Very dense	20' 0"		7	S.S.	64	"
As above.	Grey	Very dense	21' 6" 157.7		8	S.S.	96	"
Very fine grained shale.	Grey-Black	Generally of excellent quality throughout. Shale is hard.						
Fine-grained limestone, occasional small marine fossils.	Grey				9	AXT R.C.	–	CORE RECOVERY = 100%
Fine-grained dolomitic limestone, occasional thin seams of shale.	Grey		30' 0"		10	AXT R.C.	–	
Very fine-grained shale, numerous tiny fossils and some pyrites.	Grey-Black		32' 0" 149.2					

HOLE TERMINATED.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE — TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401 - C.P.R. Crossing Job No. 57148 Borehole No. 5

Client Dept. of Highways of Ontario Casing PX (2-1/2" diam.) Boring Date Feb. 3rd, 1958.

Datum Geodetic. Compiled By M. Mindess Checked By E. M. Peto

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
FAIR	S.L. SPLIT BARREL WITH LINERS	Q_u UNCONFINED COMPRESSIVE STRENGTH
DISTURBED	S.T. THIN WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 160.9					GROUND FROZEN TO 3 FT. DEPTH. WT = 1'6", FEB. 3, 1958.
Highly organic peat.	Dark Brown	Semi-Liquid	5' 0"		1	S.S.	PUSHED*	Super saturated and spongy.
As above, mixed with some grey silty clay	Dark Brown	Very loose	7' 0" 173.9		2	S.S.	PUSHED*	SUPERSATURATED. M.C. OF CLAY AT 6 FT. = 37.8%
Silty clay, organic odour.	Bluish Grey	Very soft	10' 0"		3 A B C	2" S.L.	PUSHED	Very wet. $Q_{u\ 9'-9\frac{1}{2}'}$ = 380 P.S.F. NAT. M.C. = 79.4%
Silty clay, nuggetty	Grey	Very soft			4	S.S.	PUSHED	NAT. M.C. = 93.7%
As above	Grey	Very soft	15' 0"		5 A B C	2" S.L.	PUSHED	Wetter than Liquid Limit. $Q_{u\ 14'-14\frac{1}{2}'}$ = 231 P.S.F. NAT. M.C. = 84.6%
Silty clay, 1/8" seams of fine sand at 1" intervals	Grey	Very soft	17' 0" 163.9		6	S.S.	PUSHED	NAT. M.C. = 46.1% Horizontally stratified.
Sandy and silty clay with numerous rock fragments.	Grey	Compact			7	S.S.	27	SATURATED.
Sand and fine limestone fragments.	Grey-Black	Probably Dense	20' 10" 160.1		8	W.S.	—	

REFUSAL BEDROCK.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name: Hwy. 401 -C.P.R. Crossing Job No.: 57148 Borehole No.: 6

Client: Dept. of Highways of Ontario Casing: BX (2-1/2" diam.) Boring Date: Feb. 1st - 2nd, 1958

Datum: Geodetic Compiled By: M. Mindess Checked By: E.M. Peto.

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 181.6					▽ W.T. = 1'0", FEB. 1, 1958.
Highly organic peat.	Dark Brown	Semi-Liquid			1	S.S.	PUSHED*	Super saturated and spongy.
" " "			5' 0"					
Fine sand and silty clay, odd grits, considerable organic content.	Grey	Very soft	7' 2" 174.4		2	2" S.T.	PUSHED	NAT. M.C.$_{8'}$ = 44.0%.
Silty clay, some peat.	Grey	Very soft			3	S.S.	PUSHED	NAT. M.C.$_{9'}$ = 62.7%.
			10' 0"					Much wetter than Plastic Limit.
Silty clay	Grey	Very soft.			4	2" S.T.	PUSHED	
Silty clay, slightly nuggetty.	Grey	Very soft.			5	S.S.	PUSHED	Close to Liquid Limit.
			15' 0"					
As above.					6	2" S.T.	PUSHED	SAMPLE TOO SOFT FOR STRENGTH TESTS, TOO WET FOR REMOULDED TEST. NAT. M.C.$_{17'-17'8''}$ = 63.8%.
			17' 8" 163.9					
Silty medium to coarse sand, many angular limestone ~~fragments.~~	Grey	Compact			7	S.S.	13	NAT. M.C.$_{18'}$ = 15.9%. SATURATED.
~~Sand and fine rock fragments.~~	~~Grey to Black~~	Extremely Dense	20' 10" 160.8		8	S.S.	100/6"	WASH SAMPLE RETAINED.
Fine-grained dolomitic limestone.	Grey	4" WATER-BEARING SEAM.	23' 0"		9	AXT R.C.	–	82.5% CORE RECOVERY.
As above			25' 0"					
Fine-grained shale.	Black	Except for section from 23'0"-24'0" core was generally of excellent quality.	26' 4"		10	AXT R.C.	–	
Limestone as above.	Grey		28' 0" 153.6					

HOLE TERMINATED.

ATTERBERG LIMITS, SA. 6 FROM $16\frac{1}{2}'-17\frac{1}{2}'$:
LIQUID LIMIT = 58.6
PLASTIC LIMIT = 22.8

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name C.P.R. Crossing Hwy 401. Job No. 57148 Borehole No. 7

Client Dept. of Highways of Ontario Casing BX (2-1/2" diam.) Boring Date Jan. 31 Feb. 1,1958

Datum Geodetic. Compiled By M. Mindess Checked By E.M. Peto

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
FAIR	S.L. SPLIT BARREL WITH LINERS	Q/u UNCONFINED COMPRESSIVE STRENGTH
DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 181.2					
			▽					▽ W.T. = 1'6", JAN. 31, 1958.
Highly organic peat, occasional grey silty clay.	Dark Brown	Semi-Liquid						TOO SOFT AND FLUID TO PERMIT SAMPLING.
			5' 0"					
AS ABOVE.					1	S.S.	PUSHED*	SUPERSATURATED.
			6' 6" 174.7					
Sandy and silty clay	Grey	Very soft			2	2" S.L.	PUSHED	NAT. M.C. = 73.5%. VERY WET.
			10' 0"					
Silty clay	Grey	Very soft			3	2" S.L.	PUSHED	Wetter than Liquid Limit NAT. M.C.$_{11\frac{1}{2}'}$ = 89.0%.
			13' 0" 168.2					
Clayey and silty fine to coarse sand, many limestone fragments.	Grey	Compact			4	S.S.	20	Saturated.
			15' 0"					
As above.	Grey	Compact to Dense			5	S.S.	30	
As above.	Grey	Dense			6	S.S.	34	
			20' 0"					
Silty fine sand and broken limestone.	Grey	Very dense			7	S.S.	95	Wet.
			22' 6" 158.7					

REFUSAL PROBABLY BEDROCK.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401 - C.P.R. Crossing Job No. 7348 Borehole No. 8

Client Dept. of Highways of Ontario Casing BX (2-1/2" diam.) Boring Date Jan. 30-31st, 1958.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
FAIR	S.L. SPLIT BARREL WITH LINERS	Q_u UNCONFINED COMPRESSIVE STRENGTH
DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

Soil Description	Colour	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Water Levels, Soil Moisture & Remarks
			0' 0" 181.2					W.T. = 1'0", JAN. 30, 1958.
Highly organic peat.	Dark Brown	Semi-Liquid.	5'0"		1	S.S.	PUSHED	SUPERSATURATED. M.C.= 444.0%
As above.					2	S.S.	PUSHED	" M.C.=363.0%
Sandy and silty clay, some peat	Bluish Grey	Very soft	7' 3" 174.0 / 10' 0"		3A 3B	2" S.L.	PUSHED	Q_u 7½'-8' = 470 PSF NAT. M.C. = 62.8 NAT. M.C. 8½' = 74.7%
Silty clay, nuggetty texture.	Grey	Very soft			4	S.S.	1	Much wetter than Plastic Limit.
As above, some 1/16" fine sand seams at 1" intervals.	Grey	Very soft.	12' 8" 168.5 / 15'0"		5A 5B	2" S.L.	PUSHED	NAT. M.C. 12'-12½' = 78.0%
Clayey and silty fine to coarse sand, many rock fragments.	Grey	Compact			6	S.S.	20	Saturated.
Many angular limestone fragments in matrix of silty sand.	Grey	Dense	20'0"		7	S.S.	44*	Saturated.
Silty fine to coarse sand, many grits and rock fragments.	Grey	Very dense			8	S.S.	84	Wet
As above.	Grey	Extremely Dense	24' 6" 156.7		9	S.S.	120	Quite moist.

REFUSAL PROBABLY BEDROCK.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401 - C.P.R. Crossing Job No. 57148 Borehole No. 9

Client Dept. of Highways of Ontario Casing BX (2-1/2" diam.) Boring Date Jun. 28-30th, 1958.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto.

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
FAIR	S.L. SPLIT BARREL WITH LINERS	Q_u UNCONFINED COMPRESSIVE STRENGTH
DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

SOIL DESCRIPTION	COLOUR	Density Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 181.2					▽ W.T. = 1'5", JAN. 28, 1958
Highly organic peat, some light grey clayey sand.	Dark Brown	Very loose			1	S.S.	PUSHED	PEAT: SUPERSATURATED. *CLAYEY SAND: SATURATED. NAT.
5" STRATUM GREY SANDY CLAY			4' 4" 176.9		2 A B C	2" S.L.	PUSHED.	M.C. = 31.1%. Q_u * $5\frac{1}{2}'$ = 748 P.S.F.
Silty clay containing considerable coarse sand. Very nuggetty.	Grey	Very soft			3	S.S.	PUSHED	NAT. M.C.$_{5\frac{1}{2}'}$ = 51.0% - 49.0%. SATURATED. NAT. M.C.$_{7'}$ = 53.5%.
Silty clay, some fine sand.	Grey	Very soft	10' 0"		4 A B C	2" S.L.	PUSHED	NAT. M.C.$_{9\frac{1}{2}'}$ = 68.3%
Silty clay, slightly nuggetty.	Grey	Very soft.			5	S.S.	PUSHED	NAT. M.C.$_{11'}$ = 83.9%. Wetter than Liquid Limit.
As above.			15' 0"		6	2" S.T.	PUSHED	NAT. M.C.$_{15'}$ = 63.3%
Silty clay, nuggetty texture. Seams of light grey fine sand.	Grey	Very soft			7	S.S.	PUSHED	NAT. M.C.$_{17'}$ = 60.7%
			19' 0" 162.2		8	2" S.T.	PUSHED	
Fine to medium sand with black rock fragments. Minor silt content.	Light Grey	Compact			9	S.S.	30	WET. NAT. M.C. = 12.5%
As above.		Dense	25' 0"		10	S.S.	38	
As above.		Dense			11	S.S.	37	
Silty fine sand with many grits and rock fragments.	Grey	Compact			12	S.S.	27	WET
AS ABOVE.	DARK GREY	VERY DENSE	30'6" 150.7		13	S.S.	60/6"	QUITE MOIST

REFUSAL. PROBABLY BEDROCK.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name: Hwy. 401 - C.P.R. Crossing Job No.: 57146 Borehole No.: 10

Client: Dept. of Highways of Ontario Casing: BX (2-1/2" diam) Boring Date: February 11th, 1958.

Datum: Geodetic Compiled By: W. Mindess Checked By: E. M. Peto.

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 181.6					GROUND FROZEN TO 3 FT. DEPTH. W.T. = 1'0", FEB. 11, 1958.
Highly organic peat and decayed wood.	Dark Brown	Semi-Liquid	5' 0"		1	S.S.	PUSHED*	SUPERSATURATED. Spongy.
As above	Dark Brown	Very loose	7' 0"		2	S.S.	PUSHED*	"
6" CLAYEY SAND & FINE GRAVEL.			174.6					
Silty clay	Grey	Very soft	10' 0"		3	S.S.	12	NAT. M.C.$_{5'}$ = 75.0%. Close to Liquid Limit.
Silty clay, slightly nuggetty.	Grey	Very soft			4	2" S.L.	PUSHED	Q/u $_{10'-10\frac{1}{2}'}$ = 214 PSF NAT. M.C. = 72.3 – 76.7%
			15' 0"		5	2" S.T.	PUSHED	NAT. M.C.$_{15'}$ = 87.0%.
As above.	Grey	Very soft			6	2" S.T.	PUSHED	NAT. M.C.$_{17'}$ = 100.0%.
			20' 0"		7	S.S.	PUSHED.	NAT. M.C.$_{19'}$ = 62.0%.
As above, with 1/8" fine sand seams at 1-1/2" intervals.	Grey	Very soft	21' 6" 160.1		8	S.S.	PUSHED.	NAT. M.C.$_{21'}$ = 17.9%.
Clayey medium sand.	Grey	Compact			9	S.S.	16	Saturated
Medium sand and limestone fragments.	Grey	Probably Dense	24' 0" 157.6					

VIRTUAL REFUSAL. HOLE TERMINATED.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name: Hwy. 401 - C.P.R. Crossing
Job No.: 57148
Borehole No.: 11
Client: Dept. of Highways of Ontario
Casing: BX (2-1/2" diam.)
Boring Date: February 11th-12th/58
Datum: Geodetic
Compiled By: M. Mindess
Checked By: E. M. Peto.

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q_u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. & Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 181.5					
						W.T. = 1'0"	[illegible]	* WITH TRAP VALVE IN BOTTOM OF SAMPLER.
Highly organic peat.	Dark Brown	Semi-Liquid	5' 0"		1	S.S.	PUSHED*	Super saturated and spongy.
As above.	Dark Brown	Very loose			2	S.S.	PUSHED*	
Sandy and silty clay, minor organic content.	Bluish Grey	Very soft	8' 3" 173.3		3	2" S.T.	PUSHED	$Q_{u\ 8\frac{1}{2}'-9'}$ = 367. P.S.F. NAT. M.C.=508%
			10' 0"					
Silty clay	Grey	Very soft			4	S.S.	PUSHED	NAT. M.C. = 70.1% Near Liquid Limit.
			15' 0"		5	2" S.T.	PUSHED	NAT. $M.C._{15'}$ = 80.3.
Silty clay	Grey	Very soft			6 A B C	2" S.L.	PUSHED.	NAT. $M.C._{16\frac{1}{2}}$ = 100.0%.
			20' 0"		7	2" S.T.	PUSHED	NAT. $M.C._{19\frac{1}{2}'}$ = 98.0%
Silty clay, nuggetty texture.	Grey	Very soft			8	S.S.	PUSHED	NAT. $M.C._{21}$ = 73.3%
Silty clay, slightly nuggetty.	Grey	Very soft	25' 0" 156.5		9	2" S.T.	PUSHED	Close to Liquid Limit. $Q_{u\ 23\frac{1}{2}'-24'}$ = 144. P.S.F. NAT. M.C.=82.8% NAT. $M.C._{25'}$ = 87.0%
Clayey silt and angular rock fragments, some sand.	Grey	Compact			10	S.S.	24	Saturated.
			28' 0" 153.5					

VIRTUAL REFUSAL. HOLE TERMINATED

WET DENSITY$_{23\frac{1}{2}'-24\frac{1}{2}'}$ = 144.0 P.C.F.

DRY DENSITY = 78.9 P.C.F.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401 - C.P.R. Crossing   Job No. 57148   Borehole No. 12

Client Dept. of Highways of Ontario   Casing BX (2-1/2" diam.)   Boring Date Feb. 12th, 1958.

Datum Geodetic   Compiled By M. Mindess   Checked By E.M. Peto

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
FAIR	S.L. SPLIT BARREL WITH LINERS	Q/u UNCONFINED COMPRESSIVE STRENGTH
DISTURBED	S.T. THIN WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 181.6					W.T. = 1'11", FEB. 12, 1958.
Highly organic peat	Dark Brown	Semi-Liquid	5'0"		1	S.S.	PUSHED*	* WITH TRAP VALVE IN BOTTOM OF SAMPLER. Super-Saturated and spongy.
As above, some decayed wood.	Dark Brown	Very Loose			2	S.S.	PUSHED*	
Silty clay, with considerable sand and some organic peat.	Grey	Very soft	10' 0"		3	S.S.	PUSHED	NAT. M.C.8' = 58.0%
Silty clay.	Grey	Very soft			4	2" S.T.	PUSHED	Almost at Liquid Limit.
			15' 0"		5	2" S.T.	PUSHED	NAT. M.C.14½' = 83.0%
As above, some peat and some sand and grits.	Grey	Very soft			6 A B C	2" S.L.	PUSHED	M.C.15½'-16' = 91.7% Q/u 16'-16½' = 90. P.S.F. NAT. M.C. = 92.8% - 110.5%
Silty clay	Grey	Very soft	20' 0"		7	S.S.	PUSHED	NAT. M.C.19' = 89.0%
Silty clay.	Grey	Firm	25' 0"		8	2" S.T.	PUSHED	Clay is wetter than Liquid Limit. NAT. M.C.23' = 84.0%
Silty clay, with medium to coarse sand and rock fragments.	Grey	Firm	26' 6" 155.1		9	S.S.	13*	

REFUSAL, PROBABLY BEDROCK.

APPENDIX II

METHOD OF OPERATION

The field investigation work is carried out by means of a skid-mounted diamond drill rig.

Standard sampling procedures are followed. Casing is driven and cleaned, either by tubes or by wash water.

Samples are recovered ahead of the casing at frequent intervals with either a 2" or 3" O.D. split barrel sampling tube, Shelby tube, or split barrel sampling tube fitted with brass liners and special sharp cutting nose.

The standard penetration test results are recorded when sampling with the regular 2" O.D. split barrel sampler, these being the number of blows of a 140 pound hammer falling 30 inches required to drive the sampling tube a distance of one foot into undisturbed soil.

The Dutch cone probe test is made by driving the drill rods into the ground with a 2-1/4" - 90° cone tip. The number of 4200 inch pound blows per foot of penetration are recorded, as in the standard penetration test.

Where required, "in situ" shear strength tests are made ahead of the casing, using modified Acker vane test equipment.

Disturbed samples are visually classified in the field, sealed in sample jars, and are re-examined, and tested as necessary, in the soils laboratory. Undisturbed samples are returned to the laboratory for later examination and testing, as required.

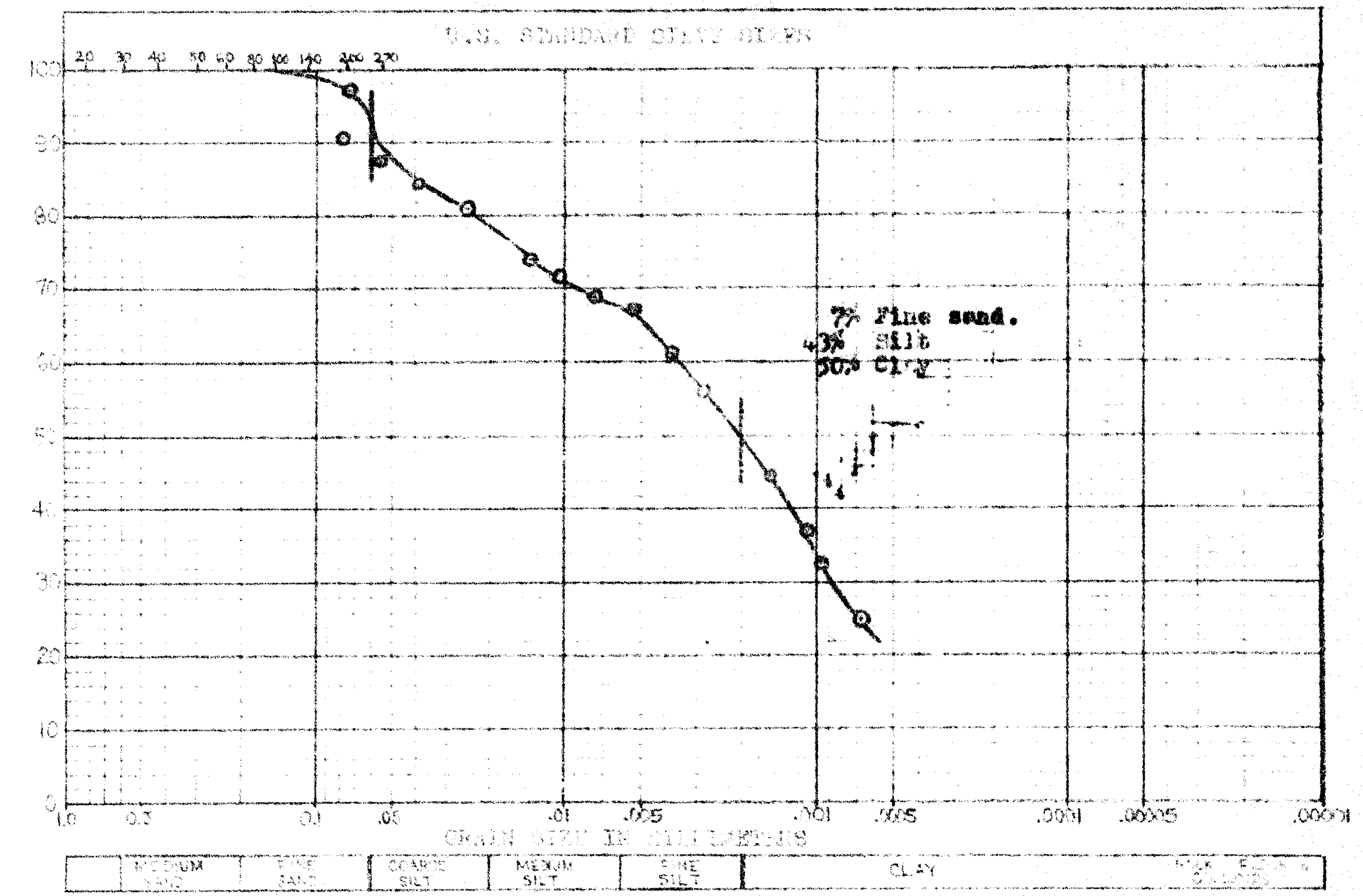
The test holes are bailed at the end of the day and on completion. Subsequent water level readings are taken for the duration of the field work. Water pressure readings are recorded when artesian water conditions are encountered. Moisture content samples are recovered at frequent intervals to assist in the soil classification and the interpretation of water table results.

APPENDIX III

Laboratory Test Results

R. M. PETO ASSOCIATES LTD.

HYDROMETER GRAIN SIZE DISTRIBUTION DIAGRAM



Job Name Hwy. 401 - C.P.R. Crossing Job No. 57148 Borehole No. 6 Sample No. 6

Depth 16-1/2' - 17-1/2' Elevation 164.6 Remarks S. G. of this sample = 2.72

e. m. peto associates ltd.

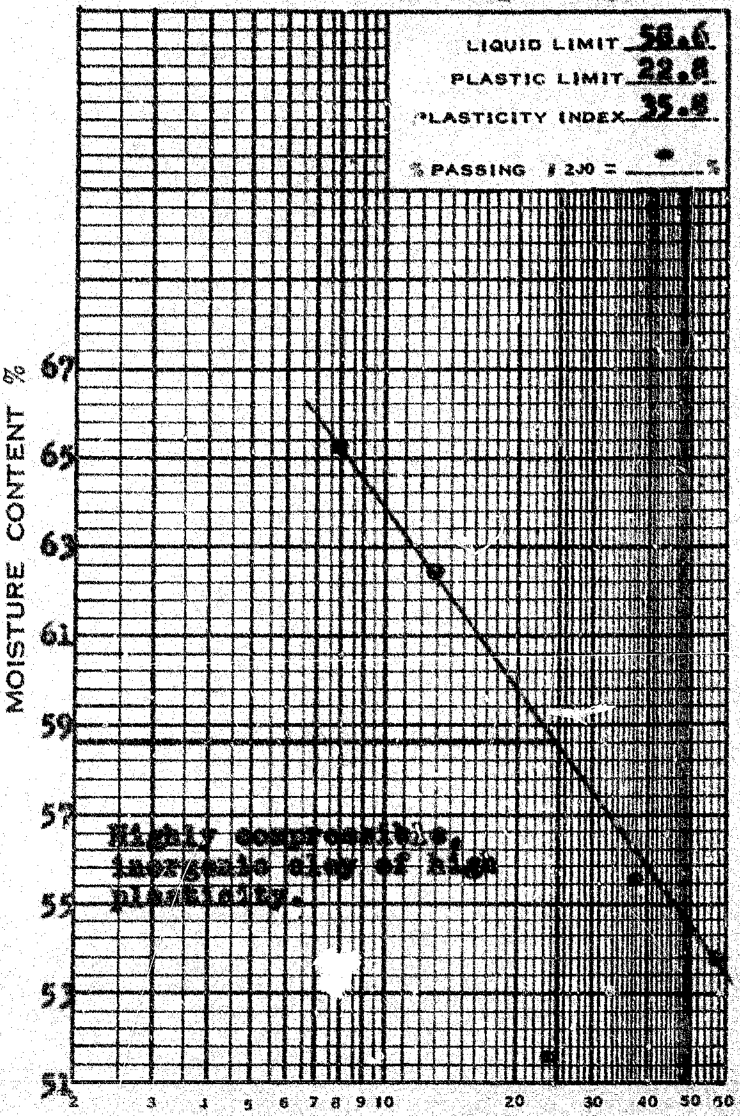
SOIL TESTING LABORATORY

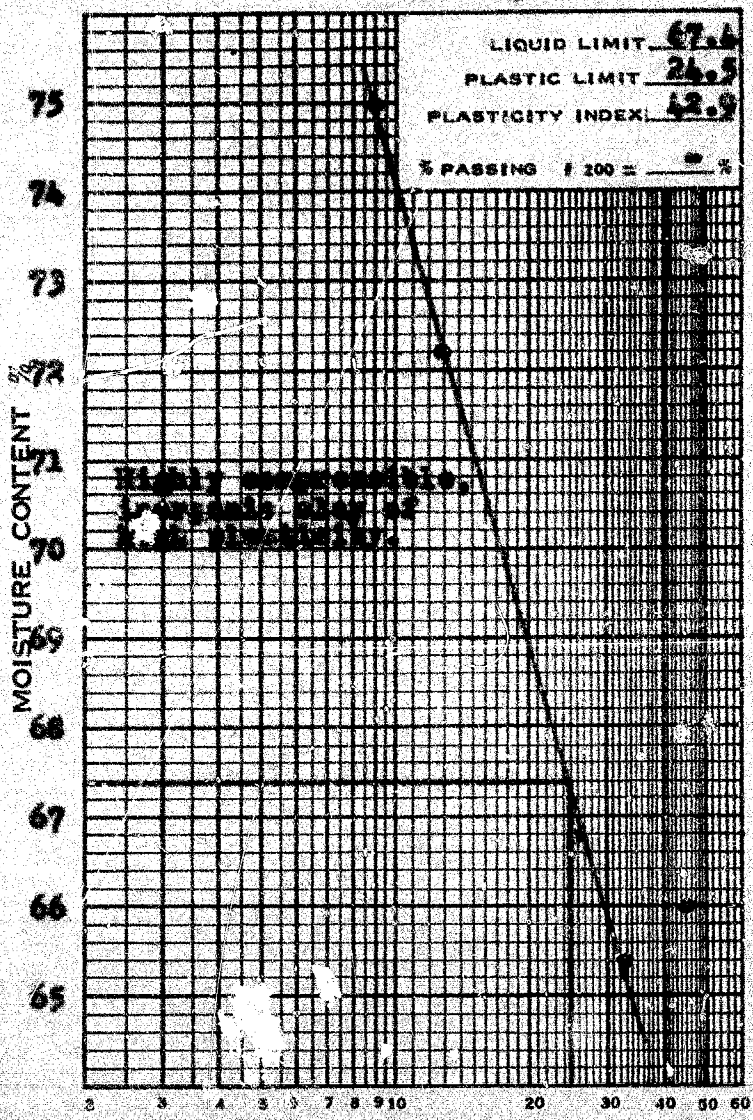
LIQUID LIMIT TEST **FLOW LINE CHARTS**

Job No. 57148 Project Hwy. 401 - C.P.R. Crossing

Sample From B.H. 6, Sa. 6
Depth 16½' - 17½'

Sample From B.H. 11, Sa. 3
Depth 8½' - 9'





NO. OF BLOWS (LOG SCALE)

Job No. 57148

SUMMARY OF UNCONFINED COMPRESSION STRENGTH TESTS

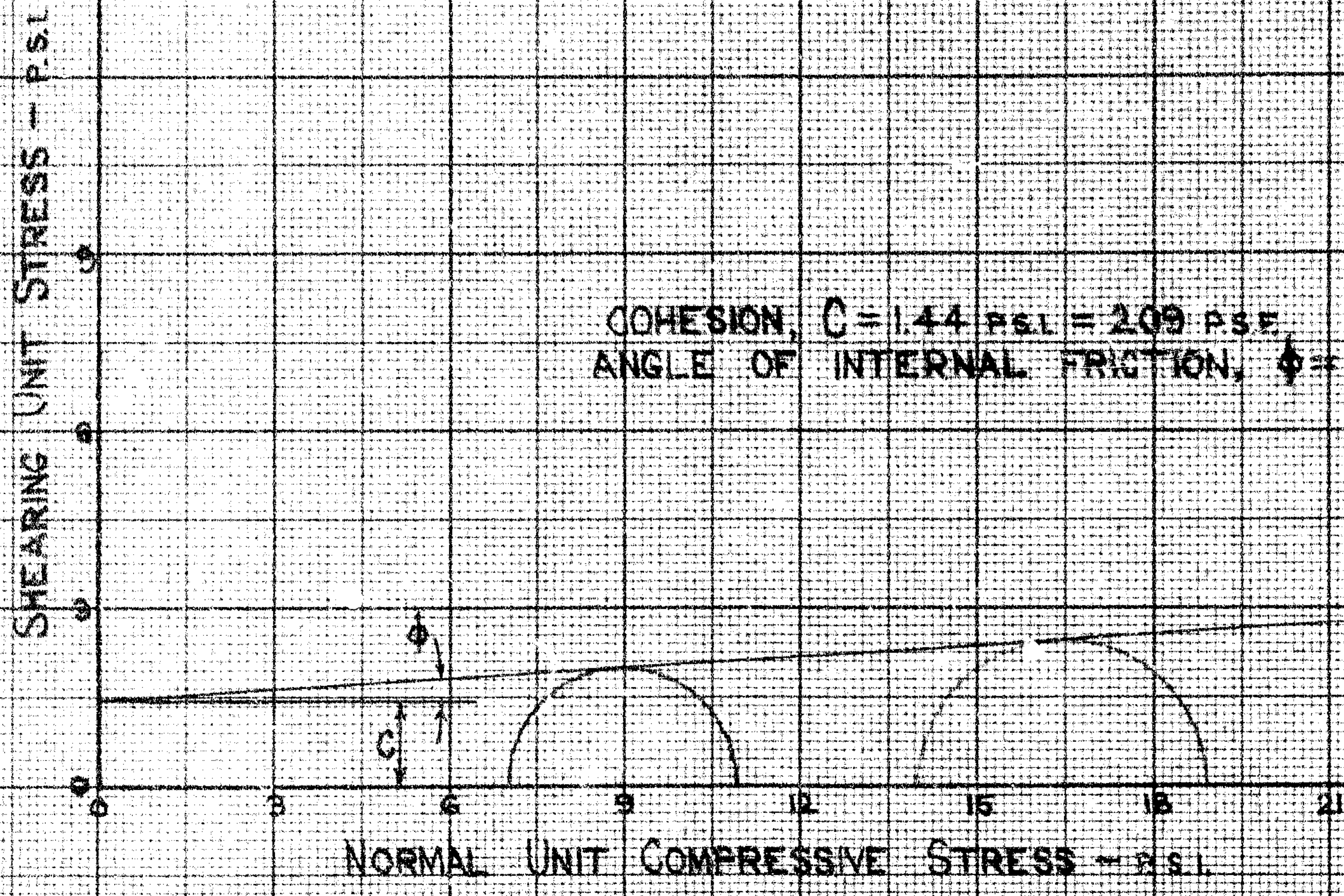
B.H.#	Sample No.	Depth	Type of Failure	Q/u (p.s.f.)	Natural M.C.	Remarks
1	5C	14'-14-1/2'	Shear along plane @ 54° to horizontal.	186	88.4%	Very nuggetty texture.
2	4A	10'-10-1/2'	Sudden shear along one well defined plane.	166	102.6%	Nuggetty texture.
2	4C	11'-11-1/2'	Sudden shear at one end along plane @ 56° to horizontal.	146	-	" "
3	6C	16'-16-1/2'	Shear	207	88.3%	
4	3A	8'-8-1/2'	Sudden shear at low unit strain.	468	87.7%	Nuggetty texture.
5	3C	9'-9-1/2'	Gradual shear.	380	79.4%	Very nuggetty.
5	5C	14'-14.1/2'	Shear at on end of specimen.	231	84.6%	Nuggetty.
8	3B	7-1/2'-8'	Brittle type failure.	476	62.8%	Very nuggetty. Some organic content.
9	2C	5'-5-1/2'	Shear at low unit strain.	748	49.0%	Very nuggetty.
10	4C	11'-11-1/2'	Well-defined shear failures, but at low unit strain.	214	72.3%	Slightly nuggetty.
11	3	8-1/2'-9'	Plastic, then shear.	367	58.8%	Minor organic content. Much wetter than Plastic limit.
11	9	23-1/2'-24'	Gradual, very plastic.	144	82.8%	Slighty nuggetty at one end, close to Liquid Limit.
12	6C	16'-16-1/2'	Shear along plane @ 62-1/2° to horizontal.	90	92.8%	Mild organic odour. Saturated.

Note: The samples were generally too wet and sticky for proper remoulding. Many of the samples could not be tested for the same reason.

QUICK, UNDRAINED TRIAXIAL TEST

ON SAMPLE OF VERY SOFT, NUGGETTY, GREY
SILTY CLAY

JOB NO. 57148
B.H. 3 SAMPLES: 4A, 4B
DEPTH: 8'–9'
NATURAL M.C.'s: 38.6%, 40.6%



cc: Mr. A. M. Toye

57-F-242C

Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Section.

September 22, 1959.

Re: Cornwall Twp. Bridge #16;
C.P.R. Overhead;
Hwy. 401 - District #9.

Attention: Mr. S. McCombie.

Attached hereto is a report prepared by Dr. H.Q. Golder, spelling out in detail, the remedial measures required in the design of the above noted overpass structure.

The initial investigation at this site was carried out by E.M. Peto & Associates, who reported that the marine clay at this site had a shear strength of the order of 150 lbs./sq.ft. With this value, the initial design of the approach embankments consisted of the installation of frictional keys, parallel to the track, and displacement of the soft subsoil by surcharging. If the reported shear strength had been, in fact, correct, this design would have worked satisfactorily.

During the period of August '59, the contractor commenced excavation at the location of the West frictional key. Failure of the side walls of this excavation took place before the required depth had been reached. At this time, the Foundation Section was called in and a detailed investigation of the engineering properties of the marine clay was carried out. In addition, the National Research Council, Soils Section, was interested in this failure and they also carried out a limited number of borings and vane tests at this site. The data collected, showed conclusively, that the information given the Department by E.M. Peto & Associates, was grossly in error. More specifically, the shear strength was found to be in excess of 400 lbs./sq.ft. Marine clay deposits with shear strengths of the order of 400 lbs. cannot be satisfactorily displaced by surcharging.

Because of this, it was necessary to alter the initial construction procedure and design, and the classical solution of counterbalancing berms for the embankment fill was adopted. Typical fill sections were designed by the Foundation Section, and these have been checked in detail, by Dr. Golder, who is an international expert in the field of soil mechanics.

cont'd. /2 ...

The principal comments and recommendations contained in Dr. Golder's report that affect the design of the overhead structure at this site, are summarized as follows:-

1. The initial structure at this location was designed as 7 spans. In order to accommodate the berms required for stability of the fill in a direction perpendicular to the track, it is necessary to increase the structure by one complete span on the West side of the rail centre-line. No change in pier locations or abutment location on the East side of the structure are required.

   The new structure will, then, have to consist of 8 spans, rather than the 7 spans initially designed.

2. In order to accommodate the berm sections required, additional property will have to be obtained. The District have been advised of the changes in fill sections and, also, of the changes required in the designing of the overhead structure, and they are presumably, arranging any additional property requirements.

3. Our memo to the District has pointed out to them, that the footings for piers and abutments will be supported on end-bearing piles. During the site visit, it was learned that the contractor had placed large diameter boulders in the fill material, and the precise location of these large-size boulders has not been determined. It is possible that these boulders may exist at the locations of the piers and abutments and, if so, problems would develop during attempts to place the required piles. We have advised the District of the difficulty that could result in this instance, and requested that they have us define the location and extent of any such boulders that they feel may be in the fill already placed.

The problem that has developed at this site, and the additional work required to modify the design of the structure and fill sections, is clearly the result of incorrect factual data provided us by the so-called "high-price" consultants. You may find some comfort in the fact that this particular organization has been virtually cut off.

L. G. Soderman

LGS/MdeF
Attach.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

Hugh Q. Golder, P.Eng.,
D.Eng., M.I.C.E.

1662, Avenue Road,
Toronto, 12,
ONTARIO.

16th.September, 1959

Mr. A. Kutka,
Materials and Research Section,
Department of Highways,
Parliament Buildings,
Toronto 2, Ont.

Dear Sir,

Following the investigation and analysis which I have made with the engineers of your department of the stability of the embankment at the crossing of the Canadian Pacific Railway at Cornwall, I now have pleasure in submitting my report and recommendations.

I trust that these will be satisfactory to you and I will be happy to discuss the problem with you further at any time.

Yours faithfully,

Hugh Q Golder

H.Q. Golder.

904 REPORT ON STABILITY OF EMBANKMENT AT CORNWALL, ONTARIO.

INTRODUCTION.

This report deals with the problems arising in the construction of a road embankment over the Canadian Pacific Railway tracks at Cornwall, Ontario.

At present there is only one railway track but provision has to be made for a second track in the future. In the first instance one two-lane pavement will be built with an appropriate bridge, but a second pavement and bridge will follow shortly and the embankment is to be designed to accommodate this second pavement. The ~~[illegible]~~ [illegible] height of the embankment above existing ground level will be 29 feet.

SITE CONDITIONS.

The ground conditions at the site are difficult but not complicated. There is a layer of muskeg up to 8 ft. thick overlying a soft clay 7 feet to 25 feet thick, which in turn overlies a compact glacial till overlying limestone bedrock.

The muskeg is essentially a peat, is saturated with water, has very little shear strength and a low weight. It must be removed and be replaced by a more suitable material.

The soft clay has a shear strength measured by vane tests of just over 400 lb/sq. ft. Its moisture content is about 80% and its liquid and plastic limits are about 60% and 23% respectively.

The glacial till contains material from clay sizes up to very large boulders several feet across. It is compact in its natural condition. The percentage of clay present is not high. The properties of the till are described in Appendix I. Borings and vane tests have been carried out at the site and the above information has been taken from records of this previous work. (See Appendix II)

VISIT TO SITE

On the 9th September I visited the site with Mr. A. Rutka to inspect the materials, the general site conditions, and a slip which had previously taken place, and also to talk to the men who had been eye witnesses of this event.

The glacial till which was inspected both in the bank and in the borrow pits is a good filling material. If properly compacted, which is essential, it should form a hard mass with an angle of internal friction of at least 35°. Its cohesion will not be high and is probably best ignored in calculations. This material can be compacted only if placed at a suitable moisture content. This must be carefully controlled in the field during construction. If placed too wet the soil will rapidly slurry and construction traffic will bog down. To some extent therefore the control on the wet side will be automatic. This is not so if the material is placed too dry. If voids are left in the dry material due to inadequate compaction, softening can occur later when water penetrates. Large boulders (over 12") should not be placed in the fill as it will be impossible to compact the material around them adequately.

The best method of site control is by many rapid density tests. In this connection it may be worth trying seismic measurements of the speed of wave transmission and correlating these with density measurements.

The gravel fill which has been placed in the frictional key on the East side is quite suitable for its purpose, and after discussing the formation of this key with the personnel on the Site I see no reason to doubt its effectiveness.

From inspection of samples of the soft clay which were available, the figure of 400 lb/sq.ft. for the shear strength seems quite reasonable. This clay is sensitive, but not unduly so.

The slip which had occurred while making the excavation for the frictional key on the West side was examined. This is a cylindrical type of slip which apparently took place in three stages. It is not necessary to postulate a virtual liquefaction of the clay to explain it, as had previously been thought. Apparently a considerable height of gravel fill was built up too near to the excavation, and the weight of this combined with the unsupported face left by the excavation caused failure of a small section. The remoulding of the clay in this area reduced the stability of the section behind this and two further failures occurred. The scars of these failures were clearly visible.

POSSIBLE SOLUTIONS TO THE PROBLEM.

The problem is to carry the roadway over the railway at a height of 27 ft. above the track level.

A simple embankment will overstress the soft clay to such an extent that failure is to be expected.

The following solutions have been proposed:-

1) Displace the clay sideways by tipping good fill on top of it.
2) Combined with the above, construct frictional keys of gravel fill to protect the railway and prevent displacement in this direction.
3) Construct a large box culvert to carry the railway through the embankment.

4) Construct berms at each side of the embankment, and combine berms and a bridge to carry the road over the railway.

Displacement.

Following the first site investigation it was thought that the shear strength of the clay was about 150 lbs/ sq. ft. It would have been easy to displace this material by tipping good fill on top of it. A bank so built would have been stable and this was a correct solution.

Frictional Keys.

The idea to use frictional keys of good material extending down to the till to prevent displacement of the soft clay towards the railway was also soundly conceived. The key on the East side is in position and will serve its purpose in the new design.

When the slip occurred on the West side further investigation of the shear strength of the clay was carried out and it was found that this strength was not 150 lb/ sq. ft. but 400 lbs/ sq. ft. and over. These results were obtained by vane measurements.

It would not be possible to displace this material with any certainty by piling fill on it, and the idea of displacement was then rightly abandoned.

Culvert.

From the point of view of the soil problems encountered this solution is a good one. However, it is understood that there are objections to this solution from the railway company's point of view and following a meeting with the Bridge Engineer it was rejected.

Berms and Bridge.

This is the traditional solution to the problem and in fact cannot be separated from the solutions discussed above since they would have included either berms and a bridge or berms alone.

The purpose of the berms is to reduce the shear stresses in the foundation material. Fortunately this solution can be applied in this case although the berms are large. The details are discussed below.

DESIGN OF BERM SECTIONS.

There are several different approaches to the analysis of the stability of a bank section. Although it is wise to consider all of these, most weight must be given to the method which simulates most closely the type of failure which is to be expected. The choice of method is best based on experience of the way in which other similar banks have failed.

In the present case, in which a relatively thin layer of soft clay is sandwiched between two layers of compact till the method known as the two-circle method is applicable. When the layer of clay is very thin this method agrees closely with the block method, but as the thickness of clay increases the block method gives pessimistic results which do not accord with the method of failure which is observed in nature. The results should be checked by calculating the shear stresses under the bank and comparing these with the shear strength of the clay. Typical calculations are given in Appendix III.

The factor of safety required is also a matter of judgment and experience. A value of 1.3 has been worked to in the present case. This value has been chosen after fair consideration of the consequences of a failure, the reliability of the strength data, and the economics of the project. It must be emphasised that good site supervision will be required during construction to ensure the strength assumed in the fill material.

The stability of the top and bottom sections of the berm must be considered independently. These have been analysed using simple circular arcs, a method which is appropriate to this case. The factor of safety is in no case lower than that for the main section.

The bridge piers will be founded on Steel H piles driven to rock. These piles will pass through the soft clay under the piers and when driven will remould this clay and reduce its strength at least temporarily. This has been allowed for by assuming zero shear strength in the area under the piers. This effect reduces the factor of safety by 5%. An appropriate increase in berm length has been made to counteract this reduction. (An increase in berm length of 10 ft. increases the factor of safety by 6%.) In fact the strength will not be reduced to zero and some regain in strength will occur after the piles are driven so that the above approach is conservative.

It must be borne in mind that large boulders must not be placed in the fill where piles have to be driven through it. This condition has been observed already in placing the fill in the areas of the piers, but it may be necessary to remove the fill under the new position of the abutment on the West side.

Table 1 shows the effect on the factor of safety of different variables.

RECOMMENDED SOLUTION.

1) Adopt the solution using berms and a bridge.

2) Remove the muskeg from below the bank and replace by till. This till must contain no boulders larger than 12" in diameter, and must be compacted to at least 95% of Proctor density. This excavation and filling is best carried out at a time when it can be done in the dry. The filling must follow immediately on the excavation.

3) The cross section of the main bank perpendicular to the centre line of the pavement is shown in figure 1. The top width of the bank is 105 ft. At each side of this there is a slope of 2:1 (2 horizontal : 1 vertical) for a height of 14 ft. At the toe there is a slope of 1½:1 for a height of 10 ft. Between these two slopes there is a berm with a drop of 5 ft. along its length for drainage purposes. The width of the berm varies as shown on the figure. Where there is no berm the side slope is 2:1. Where the berm tapers to zero width this can be done at such a rate that the appearance is pleasing.

4) In addition to the lateral berms it will be necessary to have a berm on each side of the railway and perpendicular to it. The section of the berm on the approach to the railway on the West side is shown in figure 2. This section is drawn perpendicular to the direction of the rail track. This section includes 4 spans. This is one more than in the original design and is necessary in order to get the required factor of safety. The muskeg is excavated up to a line 16 ft. from the centre line of the existing railway track. This line forms the toe of the berm.

5) The section of the berm on the approach to the railway on the East side is shown in figure 3. This section is also perpendicular to the track. On this side the conditions are better because of the frictional key which is already in place. Here the bridge can be built as designed with 3 spans.

The muskeg will be removed to a line 16 ft. from the centre line of the existing track and the area backfilled with till. The toe of the berm however, will be 16 ft. from the centre line of the future second track.

6) Some settlement will occur in the embankment due to the consolidation of the underlying clay. For this reason the pavement in this region should be flexible and not concrete, at least for the first few years.

7) It would be of advantage to instal measuring points in the berms so that warning can be obtained of any incipient failure. These points can consist of steel rods or tubes about 1½ to 2 inches diameter, driven say 8 to 10 feet into the fill on the berms at about 100 ft. intervals. Their positions should be related to fixed points well outside the bank. These positions should be carefully checked as the bank is raised to its full height. Possibly weekly intervals will suffice, but this depends on the rate of bank building.

8) If till can be placed over the muskeg alongside the rail track to a level of 183 a useful increase in factor of safety will be obtained.

9) The existing excavation on the West side should be filled with gravel starting from the North end.

Hugh Q. Golder

H.Q. Golder,
P.Eng.

17th. September, 1959.

APPENDIX I.

Properties of East Ontario Drumlin Till.

The following information has been collected on the engineering properties of the till which is to be used as fill on this job. It is given here for record purposes.

Field description by E.M.Peto Associates Ltd., in their report dated May 1958.

"The sandy till is basically a fine to medium sand, with many grits and limestone fragments, and silt and clay binder. At some points there is a considerable increase in the binder content.

Standard penetration test results in this material ranged from a low of 13 blows to a high of over 100 blows per foot, the density tending to increase with depth.

The unit wet weight of the sandy till in situ is in the order of 132 lbs. per cubic foot."

Information from D. Bazett of Ontario Hydro.

In many tests carried out by Mr. Bazett drumlin till in East Ontario gave $\phi' = 39^\circ$ consistently in drained triaxial and box shear tests, and in undrained tests with pore pressure measurements. The corresponding value of c' was 150 - 200 lb/sq.ft.

In these tests the material was compacted to ordinary Proctor density in the laboratory.

All the American design wo k on the Seaway in the East Ontario area was done using $c' = 0$ and ϕ' here have been no failures.

Extracts from "A Survey of Drumlin Formations in Southern Ontario"

by Ed. M. Gordon, Design Engineer, Trans-Canada Highway.

Under the heading "Classification" Mr. Gordon writes:-

"Drumlin soils exhibit a remarkable uniformity in grading, plasticity index and Proctor density. These relationships will serve to establish a definite category for such soils when an overall engineering classification is established.

For the present time it will be expedient to consider all drumlin soils as belonging to the same engineering soil group. Some drumlin soils show a slight variation from the normal, but they do not occur in sufficient numbers to warrant a separate group being established."

"The results of the routine tests conducted on representative drumlin parent materials are presented in the following tables and graphs."

Eastern Ontario	%Sa	%Si	%Cl	S.G.	Dry Den.	O.M.	L.L.	P.L.	P.I.
Cornwall	54	30	16	2.72	124	11.4	17.9	15.3	2.6
Kemptville	56	32	12	2.72	125	10.5	16.5	14.2	2.3
Winchester	65	25	10	2.72	124	12.2	20.4	15.8	4.6
Morrisburg	63	24	13	2.73	121	13.0			

%Sa. = % sand 1.0 mm to 0.050 mm
%Si = % silt 0.050 mm to 0.005 mm
%Cl = % clay 0.005 mm
S.G. = Specific Gravity
Dry Den = Proctor dry density lb/cu. ft.
O.M. = Optimum moisture content.
L.L. = Liquid limit
P.L. = Plastic limit
P.I. = Plasticity index

" - - - - Eastern Ontario drumlin soils are more sandy than their counterparts in other regions; - - "

" - - - - the lower portions of the grading curves - - - - - - - are almost identical to the corresponding section of the Weymouth ideal grading curve. This would indicate that drumlin till approaches an ideal grading and consequently is quite a dense soil in its natural occurrences."

Figure 4 Typical drumlin grading curves.

APPENDIX II.

Previous Tests and Investigations.

1) A site investigation was carried out by E.M. Peto Associates Limited and reported on in their report dated May, 1958.

This report showed the general succession of strata in the area. The values for the properties of the clay (except shear strength) which have been quoted in the main body of this report have been taken from Messrs. Peto's report, and their description of the till has been quoted above.

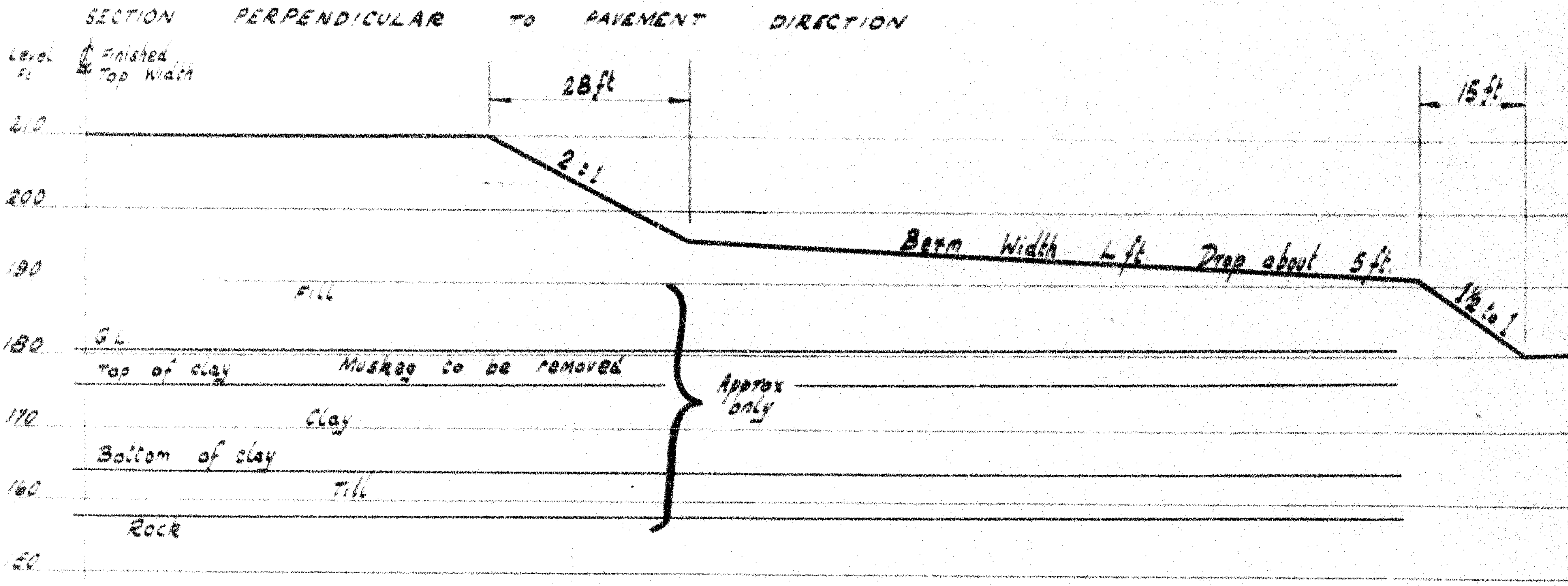
2) A geophysical investigation of the site was carried out by Messrs. Geocon in June, 1958. This showed the relative levels of the muskeg, the clay, the till and the bedrock over the whole site.

3) Vane tests were carried out in the clay by the Division of Building Research in August 1959. The results are given in the letter attached to this appendix.

4) Vane tests were also carried out by the Department of Highways in August 1959. The results showed that the clay in the area of the slip was only slightly less strong than the undisturbed material. These results are also attached to this appendix.

SIDE BERM TO BANK

SCALE :- 1" to 20 Ft.



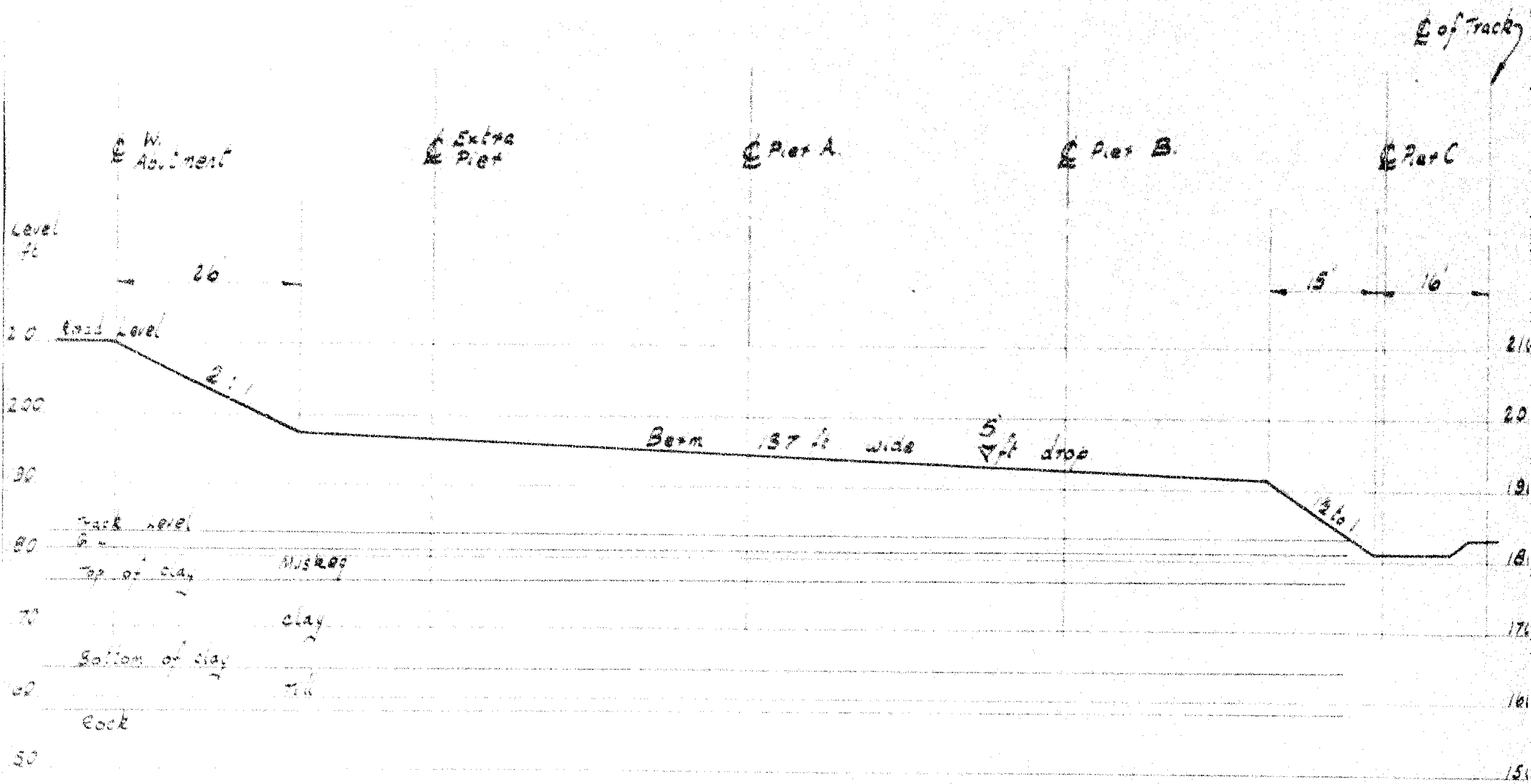
CHAINAGE	BERM WIDTH L Ft.
530 – 531	L = 0 Side slope 2:1
531 – 533	L increases gradually to 75 Ft.
533 – 536	L increases gradually to 110 Ft.
536 – 540	L = 110 Ft.
543 – 547	L = 110 Ft.
547 ⟶	L = reduced to zero gradually
548 ⟶	2 : 1 side slope No berm required.

FIG. 1.

BERM WEST SIDE OF RAILWAY

SCALE :- 1" to 20 ft.

SECTION PERPENDICULAR TO TRACK



BERM EAST SIDE OF RAILWAY   SCALE:- 1" to 20 ft.

SECTION PERPENDICULAR TO TRACK.

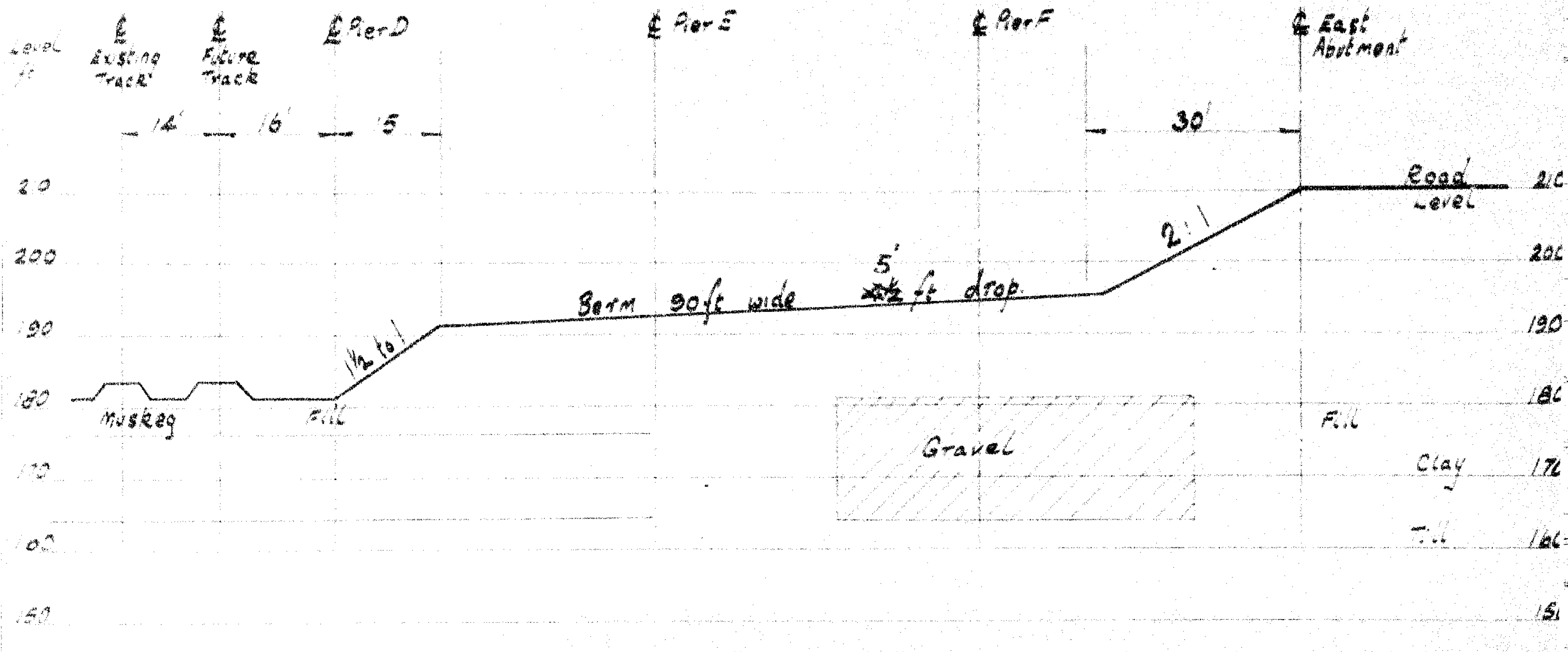


FIG. 3.

TABLE I

STABILITY CALCULATIONS

Conditions Assumed.

Levels.	Road	210
	Track	183
	Ground.	181
	Water	181
	Top of clay	176
	Bottom of clay	164
	Bedrock	158

Strength and Weight.

	Clay	110 lb/cu ft
		S = 400 lb/sq.ft.
	Till	130 lb/cu ft
		$\phi' = 35°$ $c' = 0$
	Muskeg	70 lb/cu ft
		strength = zero

Length of Berm = 100 ft

Muskeg replaced by Till up to top of bank

Factor of Safety F = 1·25 for above conditions.

10 ft. increase in length of berm increases F by 0·06

Muskeg replaced by Till to 30 ft. in front of toe to level 181
increases F by 0·05

Till placed over Muskeg up to level 183 to edge of
rail ties both sides increases F by 0·05

Gravel barrier on East side increases F by 0·80
(if fully effective)

F for top section alone - single circle 1·47

F for bottom section alone - single circle 1·30

$\phi' = 30°$ in Till - reduces F by 0·16

Top only 0·10 bottom only 0·04

S in clay reduced to zero in failed zone reduces F by 0·05

Top only 0·06 bottom only 0·04

FINAL FACTOR OF SAFETY FOR EACH SECTION

Figure 1 Beam 110 ft.

		F	F
With 100 ft beam		1·25	
Increase for 110 ft beam	+	0·06	1·31

Figure 2 West Side

With 100 ft beam		1·25	
Increase for 137 ft beam $\frac{0.06 \times 37}{10}$	+	0·22	
Reduction for piling	−	0·05	1·42
Top only – single circle		1·42	
Reduction for piling	−	0·06	1·36
Bottom only – single circle		1·30	
Reduction for piling	−	0·04	
Increase for fill over muskeg to level 183	+	0·05	1·31

Figure 3 East Side

With 100 ft beam		1·25	
Reduction for 90 ft beam	−	0·06	
Reduction for piling	−	0·05	
Increase for fill at toe	+	0·05	
Increase for Gravel barrier	+	0·60	1·80

TYPICAL CALCULATION — (See next page)

Circle 1. - Radius 48 ft.

Water pressure in crack $\frac{1}{2}\times 62.4\times 16$ = 496 16 ft

lever arm 8' Moment 496×8 = 4 000

Disturbing Moment

Section 1 $\frac{2}{3}\times 24\times 24\times 130\times 36$ = 2 180 000

2 $\frac{1}{2}\times 20\times 14\times 130\times 18.5$ 470 000

3 $(18\times 20\times 130 + 9 \ldots)14$ 900 000

3 558 000

Resisting Moment

Section A1 $\frac{64}{F}$ 4,800 normal force 6 000

" A2 5 500 " 14 000

" A3 " 22 400 " 18 000

38 000 x Tan Φ

Moment = $\frac{38000}{F}\times \ldots \times 48 \times 1/1$ = $\frac{1\,250\,000}{F}$

Moment in clay = $\frac{S.L.R.}{F}$ = $\frac{400\times 30\times 48}{F}$ = $\frac{575\,000}{F}$ lb-ft

$$P_1 = \frac{3\,558\,000 - \frac{1}{F}(1\,250\,000 + 575\,000)}{34}$$

$$= \frac{3\,558\,000 - \frac{1\,825\,000}{F}}{34}$$

Circle 2 $R = \infty$

Resisting force $= \frac{400}{F}\times 118 = \frac{47\,200}{F}$

$P_2 = P_1 + \frac{47\,200}{F}$

$$= \frac{3\,558\,000 - \frac{1\,825\,000}{F}}{34} - \frac{47\,200}{F}$$

Circle 3 R = 96'

Resisting M due to weight of quaywall = 208 000 lb-ft [or 2 000]

" " " shear strength = $\frac{400\times 4030}{F}$ = $\frac{1\,520\,000}{F}$

Moment from P_2 = $P_2\times 91$

$P_2\cdot 91$ = 208 000 + $\frac{1\,520\,000}{F}$

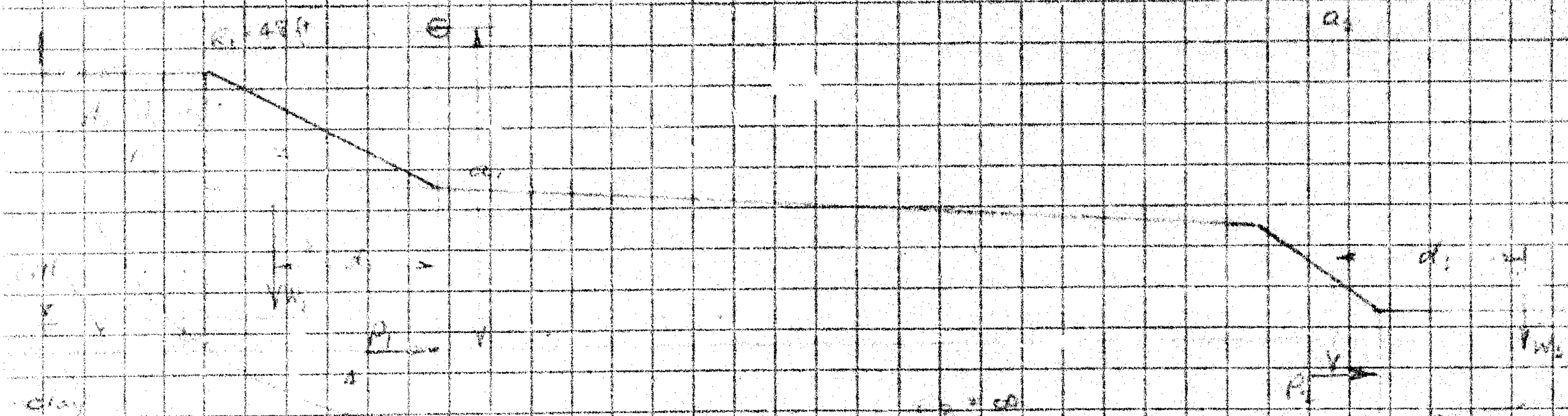
Hence F = 1.25

Circle 1 $\quad W_1 d_1 = \frac{S L_1 r}{F} \quad R_1 h_1$

Circle 2 $\quad R_2 a = R_2 + R_1 - \frac{S L_2}{F}$

Circle 3 $\quad R_2 a_2 + W_1 d_1 + \frac{S L_3 R_2}{F}$

Hence F



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

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FILE No. M43-4-4

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BUILDING RESEARCH

OTTAWA 2.

28 August 1959

Mr. L. Soderman,
Principal Soils & Foundation Engineer,
Materials & Research Section,
Dept. of Highways,
Parliament Buildings,
Toronto, Ontario.

Dear Larry:

With this letter, I am sending a transcript of the field vane tests conducted at the site of the Highway 401 crossing of the CPR, north-east of Cornwall.

Vane tests were conducted with a 55 x 110 mm. shear vane made by the Norwegian Geotechnical Institute. There are no corrections for rod friction with this apparatus. The two holes were 2 + 50 - 25 feet right and hole 2 + 00 baseline; hole 2 + 50 was believed to be outside the failure zone and hole 2 + 00 was inside. I believe Mr. Loh was conducting tests in adjacent holes with your apparatus.

We are most interested in this site and would like to be informed of future developments. We would also like to obtain some classification data on the clay at this site and if possible to have one or two Shelby tubes with which we could determine the pore water salt concentration, sensitivity etc.

I hope our vane tests have served to provide a good check for your apparatus and we would be pleased to assist you in this matter in the future.

Yours sincerely,

M. J. Eden
Soil Mechanics Section

MJE:vsl

59-6544

Field Vane Tests - CPR Crossing of Highway 401 North-east of Cornwall

All tests conducted with NGI 55 x 110 mm. vane apparatus

Depth to Vane Tip	Undisturbed Str. psf	Remoulded Str. psf
Hole 2 + 50 - 25° rt.		
12' 0"	635	Unable to remould
14' 0"	660	55
16' 0"	815	45
18' 0"	305	25
20' 0"	580	45
22' 0"	475	30
24' 0"	525	45
26' 0"	540	45
28' 0"	470	25
30' 0"	580	30
31' - refusal		
Hole 2 + 00 baseline		
13' 0"	810	45
15' 0"	400	25
17' 0"	275	25
19' 0"	510	45
21' 0"	275	25
23' 0"	515	45
25' 0"	570	Unable to remould
27' 0"	580	50
27' 4" - refusal		

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3355 WEST BROADWAY AVE.
VANCOUVER 8, B.C.
TEL. CH. 5810

Rexdale, Ontario,
July 2nd, 1958.

Department of Highways, Ontario,
Parliament Buildings,
Toronto 2, Ontario.

Attention: Mr. A. Rutka,
Materials and R.search Branch.

Dear Sirs:

Thank you for the use of your report No. BA 734 on the soil conditions, Highway 401 - C.P.R. Crossing, Cornwall, Ontario. Please find it enclosed.

Yours very truly,

GEOCON LTD

V. Milligan

V. Milligan, P. Eng.,
District Engineer.

VM/dw
Encl.
S-6687

BA734-A

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TEL. CH. 4-8841

3355 WEST BROADWAY AVE.
VANCOUVER 8, B.C.
TEL. CH. 6810

Rexdale, Ontario,
June 30th, 1958.

Department of Highways, Ontario,
Parliament Buildings,
Toronto 2, Ontario.

Attention: Mr. F. C. Brownridge, P. Eng.,
Materials and Research Engineer.

Re: Resistivity Survey,
Highway 401 - C.P.R. Crossing,
Cornwall, Ontario.

Dear Sirs: 57-F-242 C

This letter accompanies our factual report on the above resistivity survey carried out along three proposed lines for the crossing.

We find that the subsurface conditions in the local area covered by the three proposed lines are essentially similar. A variable thickness of peat deposits is underlain by about 10 to 20 feet of soft to firm clay, overlying dense glacial till, then limestone bedrock. The inferred soil stratigraphy for the area is shown on Drawing S6687-1 at the rear of this report.

An explanation of the electrical resistivity method, together with typical resistivity profiles compared to borehole records supplied to us, are given in the appendices.

We believe that this report gives you all the information which you require. If we can be of any further service, we would be pleased if you would call us.

Yours very truly,

GEOCON LTD

V. Milligan

V. Milligan, P. Eng.,
District Engineer.

VM/dw
S6687

S6657

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

RESISTIVITY SURVEY

HIGHWAY 401 - C.P.R. CROSSING

CORNWALL ONTARIO

Distribution:

4 copies - Department of Highways, Ontario,
Toronto, Ontario.

3 copies - Geocon Ltd,
Toronto, Ontario.

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INDEX

INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario, as outlined in our proposal dated May 26th, 1958, to carry out an electrical resistivity survey at the site of the proposed Highway 401 - C.P.R. crossing in the Township of Cornwall, Ontario. The purpose of this investigation was to determine and interpret, by the resistivity method, the general soil conditions at the site within limits approximately 500 feet north and south of the proposed centre line of the crossing. Information from boreholes previously put down at the site by others and supplied to us was used as control for the resistivity survey. The details of these boreholes are given in the Department of Highways report No. BA 734 dated May 2nd, 1958.

SITE AND GEOLOGY

The site is located in the Township of Cornwall, Ontario, about 4 miles north-east of the Town of Cornwall.

The local area at the site is generally flat and for the most part covered by swamp deposits. From a previous soil investigation carried out by others, it is known that the swamp deposits and an upper stratum of peat are underlain by about 10 to 20 feet of very soft silty clay overlying compact to dense sandy till, then limestone bedrock.

PROCEDURE

The field work was carried out between May 26th and May 31st, 1958. Fifty-five electrical resistivity depth determinations were made using a 115V, 60 cycle, A.C. generator and a sensitive vacuum tube voltmeter capable of measuring to 0.01 millivolts. An explanation of the electrical resistivity method is given in Appendix I.

Three possible lines for the proposed crossing were investigated:

(a) Line "C" - the initial proposed location for the crossing,
(b) Line "G" - a possible line located approximately 500 feet north of Line "C",

(c) "New Line" - a possible line located approximately 500 feet south of Line "O".

All the lines investigated extended both to the east and west of the existing Canadian Pacific Railway line.

The location of the area of the survey together with the inferred soil stratigraphy is shown on Drawing S6687-1, at the rear of this report.

Typical individual results showing the apparent and cumulative resistivity curves for individual locations are given in Appendix II.

SUMMARY OF RESULTS

Initially, two resistivity depth determinations were made adjacent to boreholes put down in the previous soil investigation for control purposes. The vertical resistivity profiles plotted against inferred soil stratigraphy for these boreholes are shown on Figures 1 and 2, Appendix II.

The depth determination made adjacent to borehole No. 12, shown on Figure 1, indicated the peat-clay interface to be at a depth of about 8 feet compared to 8 feet in the borehole, and the clay-bedrock interface to be at 27 feet which is the same as that recorded in the borehole. A further break in the cumulative resistivity curve may be observed at a depth of 21 feet. This may be due to a change in the consistency of the clay stratum. However no such change has been noted in the borehole records.

Similar agreement was found between the resistivity profile and the records for borehole No. 9. The comparative vertical profile is plotted on Figure 2.

It was, however, found that the results of an auger hole put down at station 543+40 on line "O" showed an apparent inconsistency in comparison with the records of the previous soil investigation for this location supplied to us. It was noted that the consistency of the clay stratum to a depth of about 10 feet below the peat stratum was "firm to stiff" and not "very soft" as previously recorded. Consequently, five further auger holes were then put down by the Department of Highways, Ontario, located on the "new line", to act as additional control to the resistivity survey. The maximum depth of these auger holes was 10 feet. The consistency of the clay stratum within the depth explored by the auger holes was found to be firm to stiff. Figures 3 to 7 inclusive show the vertical resistivity profiles obtained adjacent to the auger holes.

The results of these determinations were used for correlation and control purposes so that it was possible to infer the soil stratigraphy along the three possible locations studied. The longitudinal sections inferred are shown on Drawing 86687-1.

Generally the "new line" has a higher measured resistivity throughout in comparison with lines "G" and "C". The inferred till surface is at a comparatively shallow depth and increases in resistivity within this stratum indicate the till to be dense to very dense overlying bedrock.

From the control auger holes, it is further indicated that the consistency of the clay stratum is softer at line "G" than at either of the other lines investigated.

In conclusion, the resistivity survey showed that the subsurface conditions at each of the three proposed lines were essentially similar and that the upper surface of the till stratum rose gradually towards the south. The inferred soil stratigraphy for the general area is shown on Drawing 86687-1 at the rear of this report.

This report was written by Mr. E. B. Nicholls and checked by Mr. V. Milligan.

REGISTERED PROFESSIONAL ENGINEER
V. MILLIGAN
PROVINCE OF ONTARIO

V. Milligan, P. Eng.,
Principal Engineer

VM/dw

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APPENDIX I

EXPLANATION OF THE ELECTRICAL RESISTIVITY METHOD

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EXPLANATION OF THE ELECTRICAL RESISTIVITY METHOD

The electrical resistivity method of subsurface exploration consists of introducing a known current into the ground and measuring the current between the energizing electrodes placed at two selected points and also measuring the potential difference created between two auxiliary electrodes placed at other points. For convenience in analysis, the electrodes are usually kept in the same straight line.

The effective depth of measurement is governed to a considerable extent, by the spacings of the various electrodes involved in the measurements and by the relative resistivities of the various geological strata included in the measurement.

Interferring factors are always present. These include natural ground currents, polarization phenomena, stray ground currents, such as leakage from hydro lines and uneven topography. However, these can be minimized by proper technique in the field and in interpretation.

Numerous electrode configurations may be used. However, the one first suggested by Wenner is the most widely used. In this system two potential electrodes are placed in line with two current electrodes so that all four electrodes are situated at equal distances from each other with the potential electrodes inside the two current electrodes. The spacing between the electrodes is equivalent to the depth being measured. As the distance between the electrodes increases, the effective depth of penetration into the ground increases. However, as overburden or rock mantle is not homogeneous but contains material with different resistances, there will be changes observed in the resistivity values with change in electrode spacing. The resistivity value at each electrode spacing represents a composite resistance derived from several rock layers. Hence, only the relative changes in values are found and a plot of measured resistivity values as a function of electrode spacing is used to detect changes in stratification at depth.

The theory of the measurements involve around the elementary Ohms Law:

$$I = \frac{V}{R} \quad \text{or} \quad R = \frac{V}{I}$$

where I = current in amperes

V = voltage

R = resistance in ohms

Using this as a starting point we derive a simple formula to use with a Wenner configuration, that is

$$p = 2\pi \, a \frac{V}{I}$$

where p = apparent resistivity at a depth "a"

V = voltage between potential electrodes

I = current supplied by current electrodes

a = spacing between electrodes

Generally the calculated apparent resistivity is plotted graphically as a function of depth, that is, electrode spacing. An additional plot of the cumulative resistivity values against depth is also made. This cumulative curve is the summed total of the calculated resistivities at the electrode spacing. This method is known as Moore's or the cumulative method. The electrode spacings at which there are changes in slope of the straight line segments are interpreted as being depths at which there are changes in the subsurface strata. Points of inflection and points of marked change of line curvature, on the apparent resistivity curve can be interpretated as being related to subsurface changes in geology.

At the site in question, due to the swamp and thick bush conditions, it was decided to use a modified Wenner configuration called the single probe. For this purpose one current electrode is placed at the station under investigation, and the second current electrode is placed at an infinite distance away, or at least six times the depth of the investigation. The potential electrodes are kept at a fixed distance apart and moved along a straight line away from the initial current electrode. The distance between the current electrode and the potential electrode is equivalent to the depth of the investigation.

APPENDIX II

TYPICAL RESISTIVITY PROFILES

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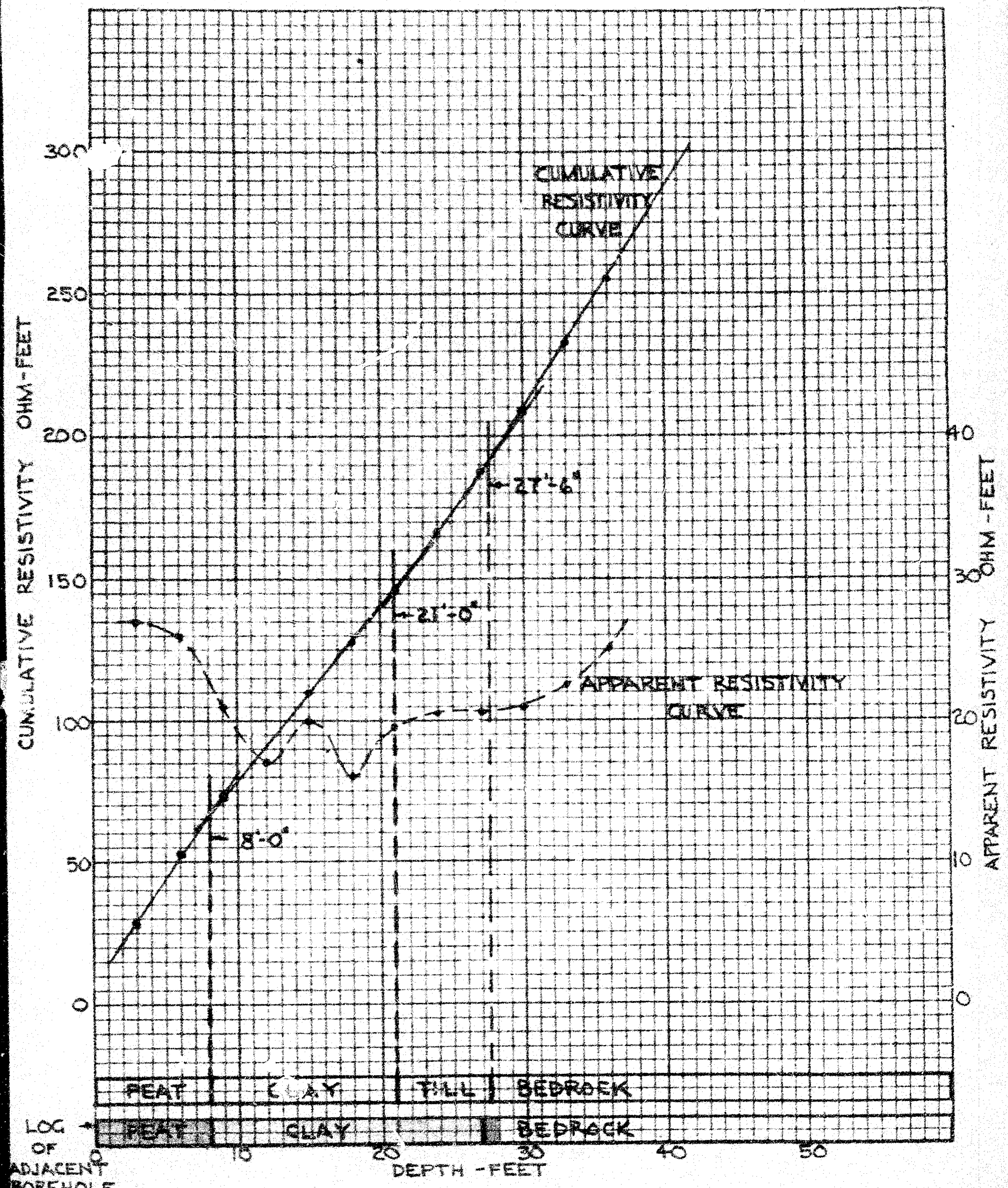
RESISTIVITY PROFILE

LINE "G" – STATION 538+63

APPENDIX II
FIGURE 1
PROJECT S6687



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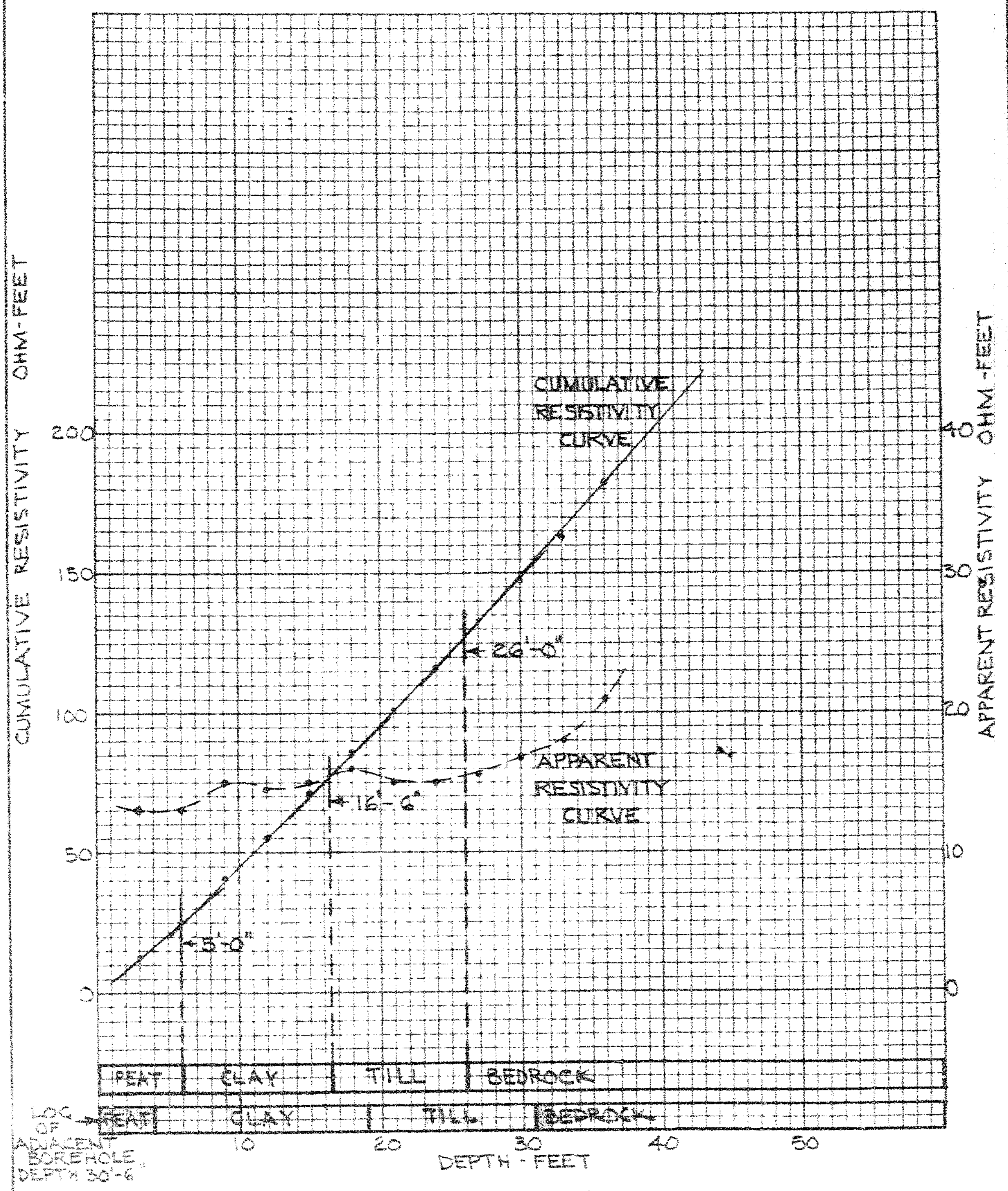
RESISTIVITY PROFILE

LINE "G" – STATION 544+60

APPENDIX	II
FIGURE	2
PROJECT	5668T



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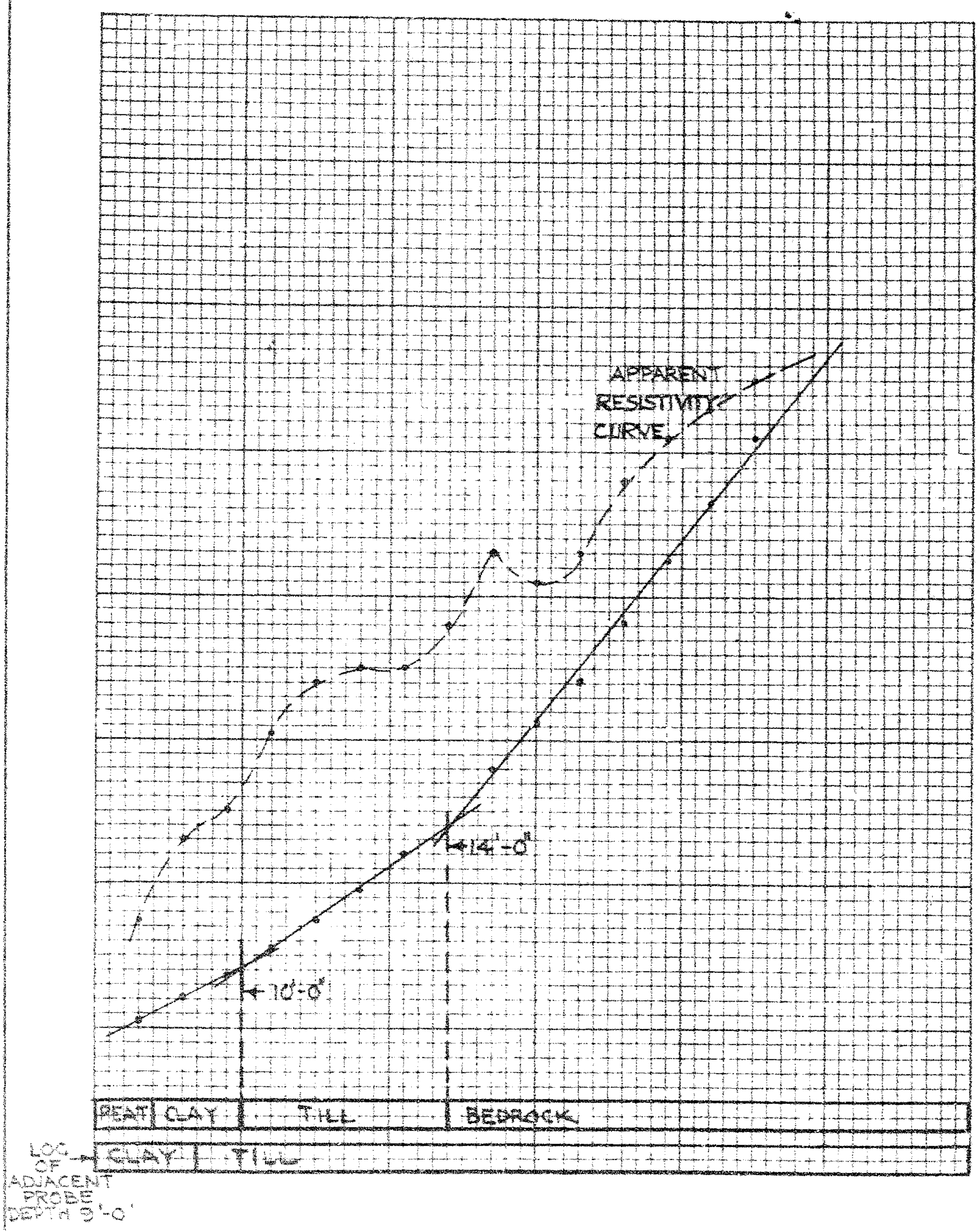
RESISTIVITY PROFILE

NEW LINE - STATION 3+00 EAST

APPENDIX	II
FIGURE	3
PROJECT	S6687



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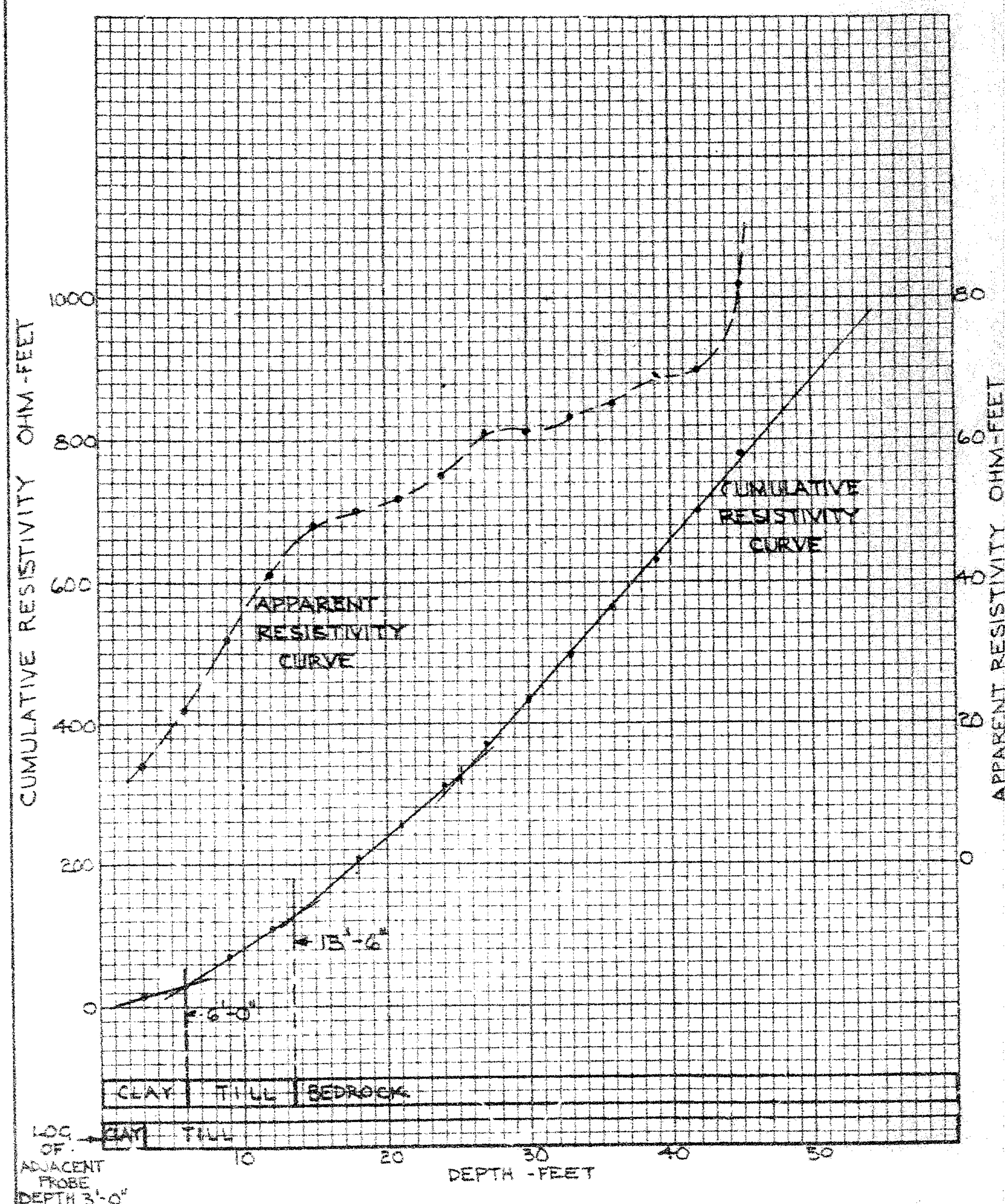
RESISTIVITY PROFILE

NEW LINE - STATION 6+00 EAST

APPENDIX II
FIGURE 4
PROJECT 5668T



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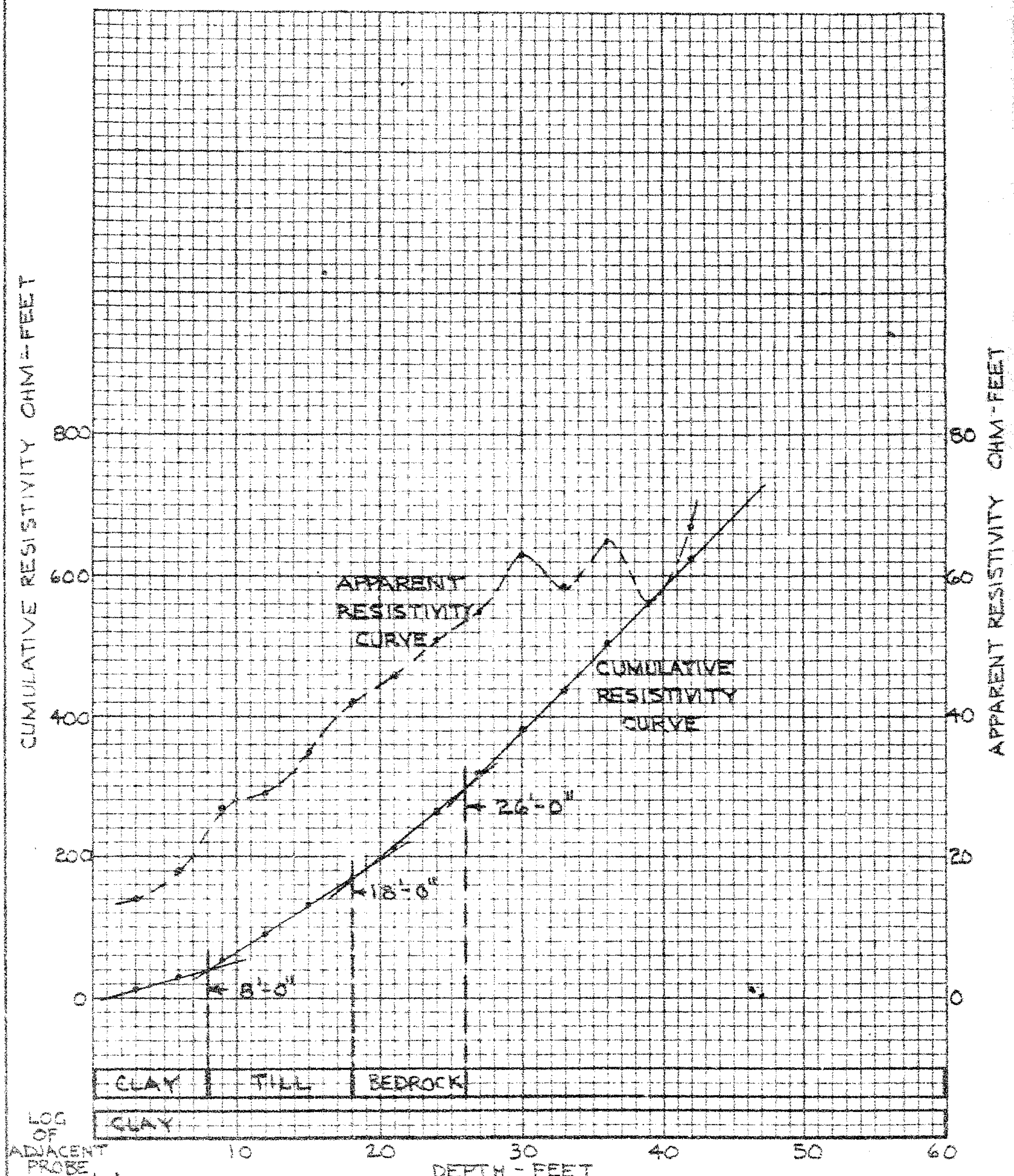
RESISTIVITY PROFILE

NEW LINE - STATION 7+00 EAST

APPENDIX II
FIGURE 5
PROJECT S6687



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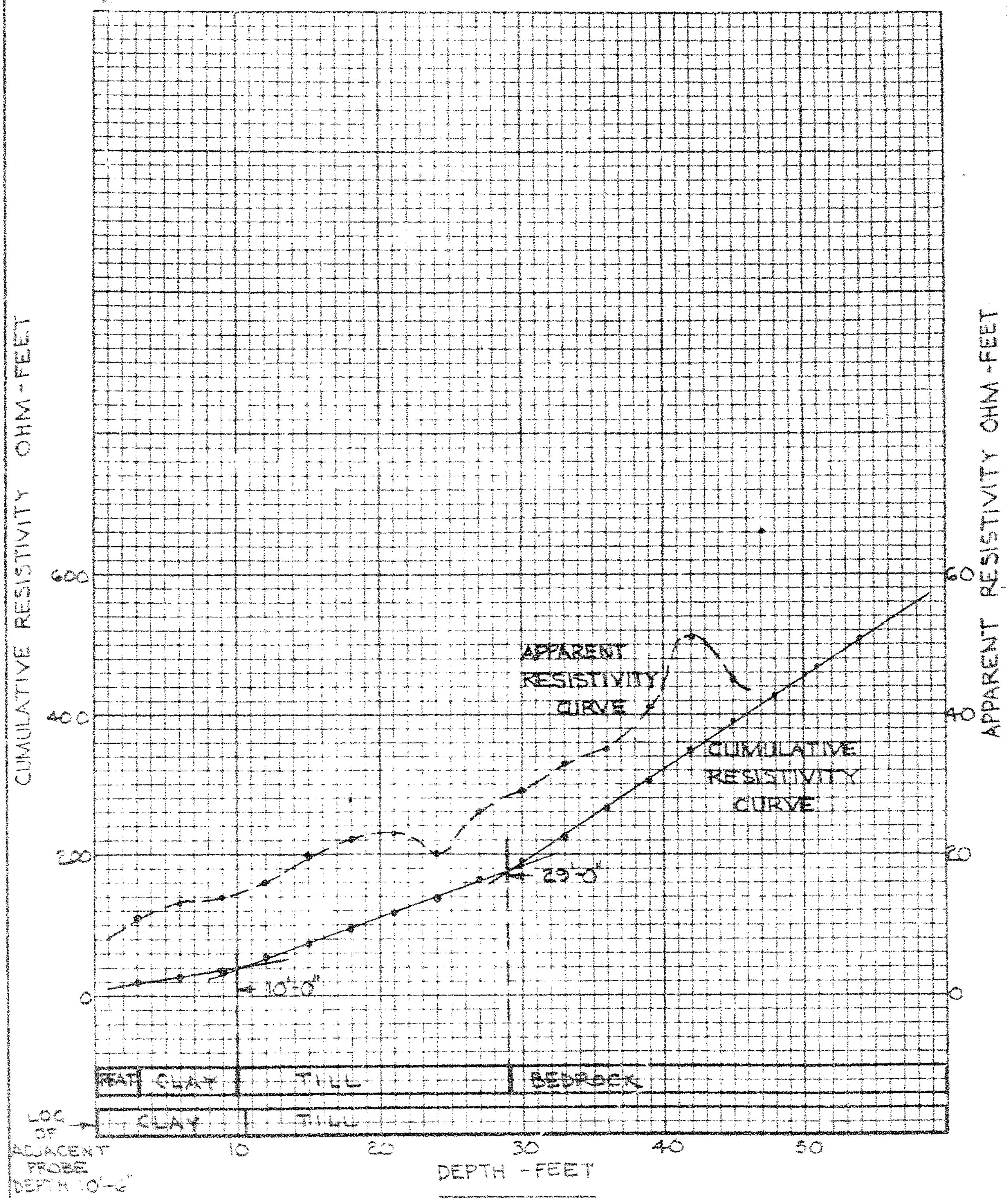
RESISTIVITY PROFILE

NEW LINE - STATION 8+00 EAST

APPENDIX	II
FIGURE	6
PROJECT	S6687



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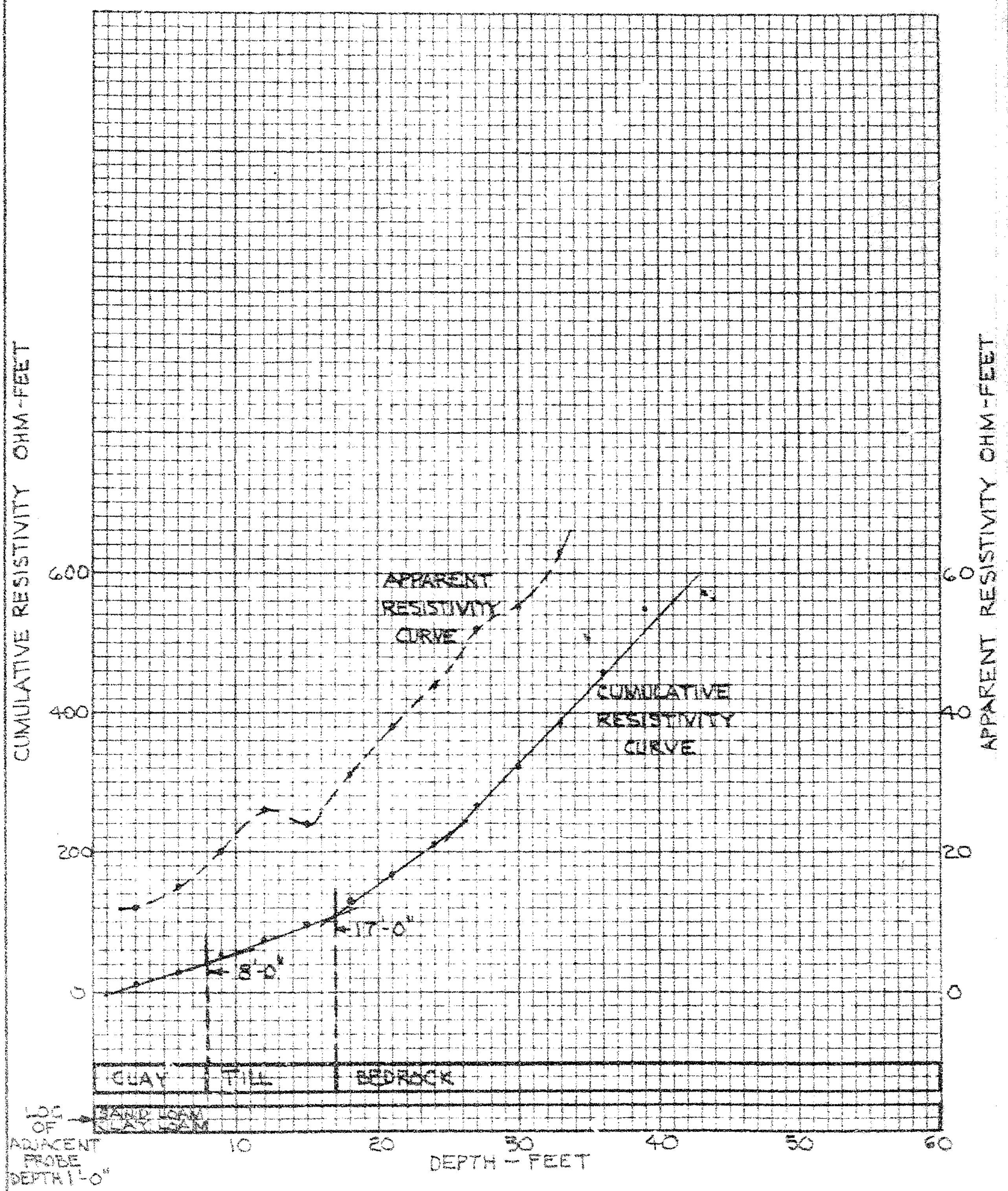
RESISTIVITY PROFILE

NEW LINE - STATION 10+00 EAST

APPENDIX II
FIGURE 7
PROJECT S6687



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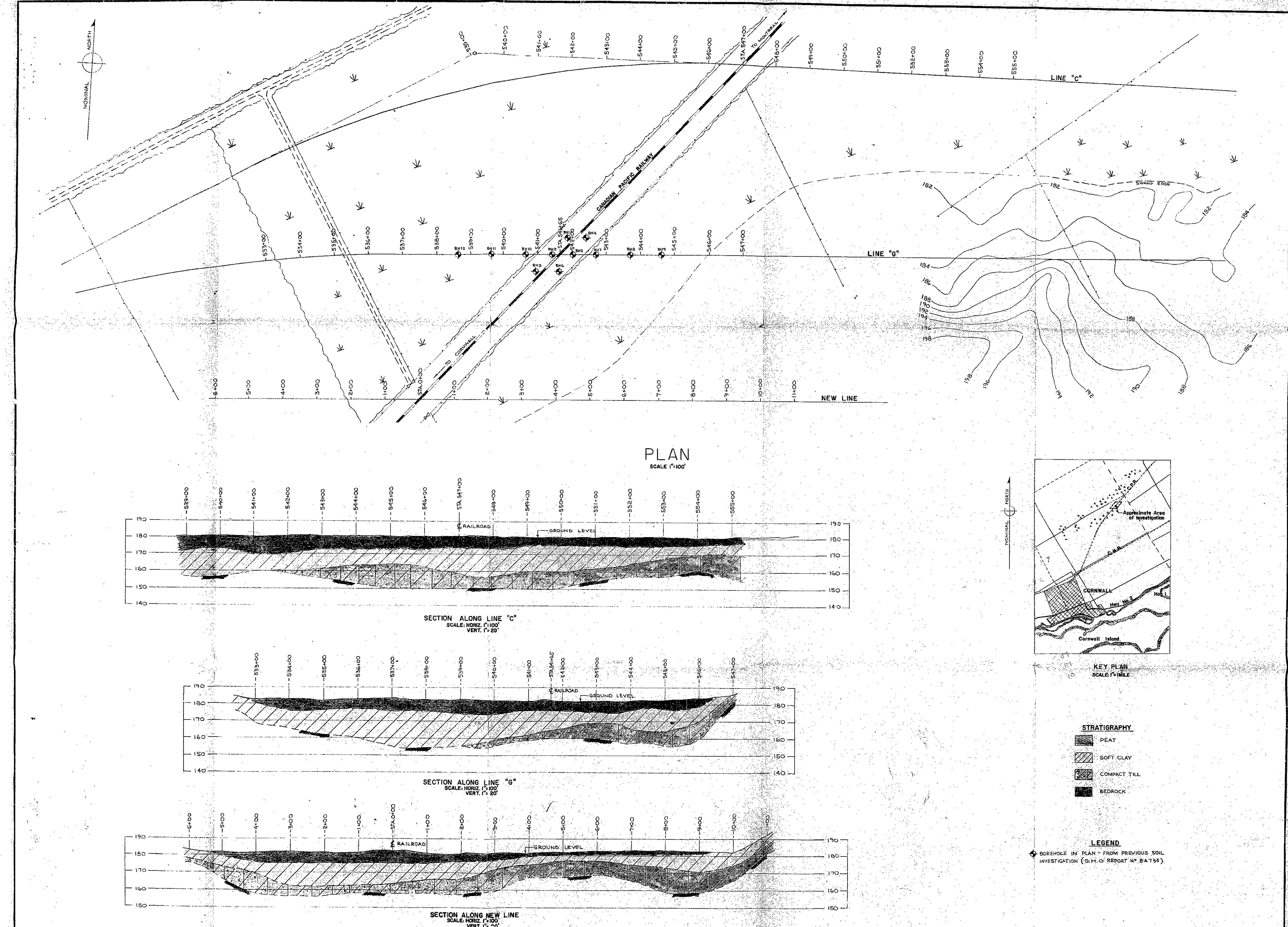
57-F-242C

W.P. 68-57

Hwy. # 401

C.P.R. Crossing



REVISIONS			REVISIONS			REFERENCE		REFERENCE	
MARK	DATE	DESCRIPTION	MARK	DATE	DESCRIPTION	DWG. NO.	DESCRIPTION	DWG. NO.	DESCRIPTION
								F3165-9	PLAN SUPPLIED BY THE DEPARTMENT OF HIGHWAYS, ONTARIO
								57-148	D.H.O. REPORT Nº BA-734 - SOIL INVESTIGATION AT HWY 401 - C.P.R. CROSSING FOR DEPARTMENT OF HIGHWAYS ONTARIO - DATED FEB. 19, 1958.

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO

PROPOSED HIGHWAY 401 & C.P.R. CROSSING
CORNWALL ONTARIO

SOIL STRATIGRAPHY DETERMINED BY RESISTIVITY

GEOCON LTD

DATE JUNE 12, 1958 SCALE AS SHOWN

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J.A.	E.G.N.	[illegible]	No. S 6687-1