

REPORT ON FOUNDATION INVESTIGATION
for the
PROPOSED CROSSING OF HIGHWAY NO. 17
OVERPASSING
C.P.R. RIGHT OF WAY, NEAR COBDEN, ONTARIO

for the
DEPARTMENT OF HIGHWAYS - ONTARIO

by the
Engineering Division
HUNTING TECHNICAL AND EXPLORATION SERVICES LIMITED
Toronto, Ontario

October, 1958.

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Section 1.1

PURPOSE OF REPORT

1.11 General

The purpose of this report is to present the results of a sub-surface soil investigation on the site of the proposed crossing of Highway No. 17 overpassing the C.P.R. Right of Way in the Township of Westmeath.

The original programme of four boreholes was carried out at locations of the proposed crossing. A fifth borehole together with a number of cone penetration holes were also made along and near the centre line of the approaches.

As bedrock was found to be somewhat shallow by preceded borings, coupled with evidence of outcrops of rock stratum in surrounding areas, the original programme was later extended to include more cone penetration tests and seismic depth-to-rock determinations in the neighbourhood of the proposed crossing and along both sides of the railway line with the object of locating another site where bedrock is shallow and footing foundation could be economically reached by excavation.

DISCUSSION OF PROCEDURES1.21 Location of Test Borings and Seismic Depth-to-Rock Determinations

The field location of the four boreholes intended for the original programme of investigation was established by Department of Highways engineers. Hunting Technical and Exploration Services Limited engineers established the actual borehole locations by chaining to all boreholes from the centre line of proposed revision line "G" Highway No. 17. Location for the boreholes differ somewhat from that originally proposed because of head-room limitations imposed by overhead power and telegraph lines.

Location for the other test borings and the seismic depth-to-rock determination points was chosen at the discretion of the field engineer in charge of the project.

Elevations for all test borings and seismic determination points were established by a level relative to El. 443.06 on top of rail located at chainage 115+16.78 on the centre line of the proposed revision line "G", Highway No. 17.

At the completion of the work, the boreholes were marked each with a large stake denoting the hole number for future reference.

The locations and elevations of top of all test borings and seismic depth-to-rock determination points are shown in Appendix 1.81.

1.22 Subsurface Drilling and Sampling

The original programme, specified by the client, of 4 soil borings was carried out in the vicinity of the proposed site of the new overpassing bridge. A fifth hole was decided later and made on the north approach along the centre line of the proposed revision line "G".

One trailer-mounted and one skid-mounted junior Longyear hydraulic head drilling rigs were used on this project. All borings and sampling operations were completed by experienced soil sampling crews under the supervision of engineering personnel experienced in soil sampling procedure.

All soil borings were performed by the standard wash boring procedure. By this method, drill casing was driven into the soil by a 350 lb. hammer to a depth determined by the boring supervisor. All the soil contained inside the casing during this operation was thoroughly washed out to the bottom of the casing. Sampling tools were then lowered to the bottom of the hole. The sample was then taken and the sampling tools removed from the hole. Additional lengths of casing were added as required and the procedure repeated.

"Undisturbed" samples were taken in 2-inch Shelby Tubes which were pushed into the soil and extracted. All tube samples were classified, tagged and sealed immediately upon recovery from the hole.

Undisturbed and remolded in situ vane shear tests were conducted in the cohesive soils using 2-inch and 3-inch diameter vanes. Pocket penetrometer tests were performed on all cohesive soils.

Attempts were made to obtain samples in the less cohesive soils by means of a 2-inch O.D. standard split spoon sampler. The standard penetration test using a 140 lb. hammer falling 30 inches was recorded for each foot of sampler penetration. When necessary, recovery of samples for identification and correlation was obtained with a side slit sampler. All samples were visually examined and classified on the site, then placed in jars and forwarded to the engineering office. Where samples obtained were representative and relatively undisturbed, apparent density tests were made on site to obtain the approximate specific weight of the material.

The cone penetration tests were used where a quick means of probing onto shallow rock was desirable. These were done, also using the 140 lb. hammer falling 30 inches, and the penetration for each foot driven was recorded.

1.23 Soil Testing

Selective disturbed and "undisturbed" samples from each strata were forwarded to the laboratory as a check on the visual field classification, and for unconfined compression and other standard soil tests as required.

The results of all tests are given in the Office Logs and in the Appendices.

The laboratory tests were performed by:

Donald Inspection Limited,

340 Richmond Street West,

Toronto 1, Ontario.

1.24 Seismic Depth-to-Rock Investigation

From August 28th to September 2nd inclusive, our Geophysics Division carried out a seismic depth-to-rock investigation on the site of the proposed crossing.

The survey was performed using a portable MD-1 instrument operated by one of our geophysicists and a helper.

The survey is divided into two parts. The first part is a detailed profile of the bedrock along the Revision Line "G". Depth determinations were obtained at 100 foot intervals. Additional depths to bedrock were estimated at the 50 foot chainages wherever possible. The second part of the survey consisted of seismic probes at specified points chosen by the field engineer along both sides of the railway line.

Depths to bedrock were calculated assuming either of the two cases where applicable, namely:

- (a) two-layer case, that is, a fairly homogeneous overburden with an average velocity of 1,300 ft./second, and the bedrock with an average velocity of 15,000 ft./second.
- (b) the overburden is composed of two layers; the top layer with the low velocity of 1,300 ft./second, and the bottom layer with an immediate velocity of 5,400 to 5,600 ft./second.

Depths to bedrock so obtained are average values. The range of uncertainty in all cases however, may lie within a maximum of 2.5 feet plus or minus.

The results of the seismic depth-to-rock determinations are shown in tabulated form in Appendix 1.81.

Section 1.3DISCUSSION OF SITE1.31 Geographic Location

The proposed bridge site is located on the King's Highway No. 17 at the proposed crossing of the Canadian Pacific Railway right of way. The site lies in the County of Renfrew, Township of Westmeath, Lots 6 and 7, Concessions I and II.

1.32 Site Geology

This area consists primarily of silty-clay glacio-lacustrine or glacio-marine deposits overlying more or less flat-lying or gently dipping granite-gneiss bedrock. In places, as shown on the annotated aerial photograph, bedrock is at or very near the surface. Elsewhere, channels cut in the bedrock are filled with drift. No deposits of organic soil were noted within the area of interest, nor were any areas of impeded drainage. Outside the area, in the southwest corner of the photograph, a well defined glacio-fluvial gravel ridge was noted and may form a source of construction material, if required.

The surface geological features of the site are shown in the air photo in Appendix 1.83.

1.33 Water Condition

At the time of exploration, no apparent water table was observed except for the water which remained inside the hole after boring operations.

However, it was observed in Borehole No. 4 that water rose in the hole after diamond drilling through the bedrock was completed. This issuance of water was suspected to have come from fissures within the granite bedrock punctured by drilling and water for boring operation in other boreholes was tapped from this hole.

1.34 Soil Condition

The borings indicate that the soil stratification is fairly uniform throughout the site.

Bedrock was found to occur between El. 428 and El. 420.

The overburden soil, varying from about 11 feet to a maximum of about 19 feet from bedrock to top of boreholes, consisted of three structural types in the following order:

1. Sand and organic top soil
2. Medium to stiff grey clay and silt
3. Sand and gravel with clay.

The physical properties of the soil types are summarized below in the order of their occurrence below ground surface.

1. Sand and organic top soil:

This layer of top soil exists about 2 feet to 3 feet in depth at the site. This material is highly compressible and is considered to have little or no structural value, and should be removed before the construction of the abutments and the approaches to the bridge.

2. Medium to stiff grey clay and silt:

This is a layer of grey glacial clay whose consistency varies with depth from stiff to medium.

The top portion, generally extending to about El. 427, is stiff clay. Vane shear tests performed in boreholes and laboratory unconfined compression tests indicate that the material has a shearing strength in the range of 2,700 to 4,000 lbs./sq. foot. The underlying portion, which is believed to be medium clay of the same type, has shearing strength varying from 800 to 1,400 lbs./sq. foot.

The moisture content in the clay averages about 38% and its liquid limit 53% more or less. It is difficult to ascertain the compres-

sibility of clay unless a laboratory consolidation test has been performed on undisturbed samples, but in our opinion, we believe this layer of clay has been precompressed by desiccation.

The void ratio of the clay averages about 0.9, and in volumetric measure there is about 47% of voids in the material. In the analysis of settlement the effect of preconsolidation will, however, not be taken into account.

3. Loose to medium sand and gravel with clay:

This layer of soil is located immediately above bedrock.

The soil consists mainly of coarse sand and gravel (some large stones) intermixed with silt and clay. The material appears to be saturated and for this reason it is believed to be a layer of glacial till which has probably lost its stiff consistency by the action of free water occurring in the region between the layer and the bedrock surface.

Standard penetration tests performed in this stratum gave an average value of 7 blows per foot.

1.35 Bedrock Conditions

Granite-gneiss bedrock was encountered at the site. This is believed to be medium hard rock with slight defects and weathering along fractures and seams, especially predominant along the surface.

Under the east abutment bedrock occurs fairly even at El. 420 approximately. But under the west abutment depths to rock average about El. 425 and the rock surface appears to dip gently in a southerly direction. Within about 700 feet to the north and south, along both sides of the railway line, from location of the proposed crossing, bedrock profile may generally be described as irregular, bedrock being shallower to the north.

The recovery of rock core from drilling in boreholes is within the range 90% to 100% indicating the bedrock is quite solid in

composition. We estimated that such rock should have a bearing capacity as high as 40 tons/sq. foot. But for design purposes we would recommend the use of 20 tons/sq. foot in spread footing foundation. Depending on design requirements, where horizontal forces are large, such footings on rock should be made into at least 12 inches cut beyond the surface of bedrock.

Section 1.4COMMENTS ON FOUNDATIONS OF STRUCTURE1.41 General

Our understanding of the proposed bridge structure is that abutments are contemplated in the vicinity of chainages 114+80 and 115+65 with a skew angle of $17^{\circ}06'$ with respect to the C.P.R. right of way.

The maximum height of fill along the approaches to the structure is assumed to be in the order of 32 feet and that the fill will be selected granular material contained and protected by wing-walls and retaining-walls where necessary.

Crossing of the bridge structure at a location approximately 400 feet further north along the railway has also been taken into consideration as an alternative site to that originally proposed.

1.42 Spread Footing Foundation

Assume the use of a spread footing foundation placed in the medium to stiff clay at El. 433 which is 5 feet to 6 feet more or less below the present ground level.

At El. 433 the maximum bearing capacity of the soil is about 1.8 tons/sq. foot, factor of safety being 3.

However, the allowable bearing capacity of soil at El. 433 is governed by the presence of weaker clay in the underlying regions at about El. 427. At El. 427 the clay was estimated to have a maximum bearing capacity of about 1.0 tons/sq. foot. By using Westengaard's Influence Chart for vertical pressure and assuming width of continuous footings to be 14 feet, it was calculated that the design bearing capacity of soil at El. 433 would be in the order of 1.4 tons/sq. foot.

Under a bearing load of 1.4 tons/sq. foot, however, the estimated ultimate settlement due to the clay below the west abutment is about 5.5 inches and that under the east abutment is about 8.0 inches. An ultimate differential settlement in the order of 2.5 inches is expected between the two abutments and therefore a simply-supported type of structure is advisable if the use of a spread footing foundation is contemplated. This settlement analysis is based on test results and assumptions as mentioned in Section 1.34, considering the same type of clay exists under both abutments.

For a rigid-framed structure on spread footings, the differential settlement should be brought within the magnitude tolerable by the rigidity of the structure. This will mean that the design bearing capacity of 1.4 tons/sq. foot will have to be reduced so that the ultimate differential settlement can be made within a value allowable for the design of such a structure. However, in our opinion, we reckon the reduced bearing value will be inadequate to meet any economical design of such footings.

1.43 Pile Foundations

There is a possibility of acquiring a sound foundation by using short piles bearing on bedrock. If the cost of excavation is not justified, we would like to suggest the use of cast-in-place concrete piles. Such piles are expected to carry 100 tons or more per pile and they may be driven either with a mandrel or with an auger. Companies such as Raymond Pile Co., Franki Pile Co., or Spencer, White and Prentice, and others, could be asked to perform the foundation work. Using this kind of foundation it should be safe and economical to design a single-span rigid-framed type of bridge structure on this location.

Section 1.5STABILITY OF APPROACH FILLS

The stability of the fills on the approaches has been investigated with respect to failure by sliding both in the longitudinal and transverse directions. Different cases of slide failures have been tried out.

When abutments are built on spread footing foundation, sliding in a longitudinal direction into the railway embankment and having the slip surface tangential to the bedrock surface seems to be the most dangerous case. Assuming a cohesion of 600 lbs./sq. foot for the backfill material and an average cohesion of 1100 lbs./sq. foot for the entire overburden soil from ground surface to bedrock surface, the factor of safety is in the order of 1.30 for a 32-foot height of fill. Embankments of the fills at the approaches and any berms that may be required in front of the abutments in the case of a three-span bridge are advised to be made on a 1 to 2.5 slope.

However, when the abutments are located close to the railway line, (say 15 feet from centre line of track), there is no apparent guarantee as to the absolute stability of the approach fills since the nature of the clay beneath the railway embankment is not known. Vertical pressure from the 32 feet of fill exerting onto the ground may cause certain degree of movement within the underlying clay from highly stressed zone to a lower stressed zone. Consequently this could result in the heaving up of the ground under the railway track. With due consideration given to the safety of the railway line, it is advisable to locate such abutments further away from the railway. This, in our opinion, will require a three-span bridge or some kind of structure that can be trestled across the railway line with intermediate piers founded on each side of the track. If properly located it may not be necessary to use retaining walls around

the approaches.

However, when the foundations of the abutments are made on short piles or spread footings on rock, we do not anticipate this danger of sliding in the longitudinal direction.

The settlement under the approach fills depends on the depths of the underlying clay. With a 15-foot layer of clay, the ultimate settlement would be roughly about 20 inches. If required, the rate of settlement may also be roughly estimated from knowledge of the physical properties of the clay.

Section 1.6

RECOMMENDATIONS

(1) With due consideration given to the safety of the railway line we do not advise spread footing foundation on the clay unless abutments are placed at least 70 feet away from centre line of the railway track.

Footings for abutments or for any intermediate piers may be designed with a soil bearing value of 1.4 tons/sq. foot.

The bottom of footings may be located at El.433 more or less.

(2) Foundation on short piles or spread footings brought down to bedrock may be used if the abutments, carrying a single-span bridge structure, are located at about 14 to 20 feet from the centre line of the railway track.

Our comments under Section 1.43 outlined the use of several types of heavy short piles.

Spread footing on rock should be made into at least 12 inches cut beyond the surface of bedrock. A design load of 20 tons/sq. foot may be used.

(3) The maximum height of fill that may be placed over the approaches is 32 feet on a slope of 1 to 2.5 (see Section 1.5).

(4) If an alternative site for the new crossing is desirable, we would like to suggest that the revision line be relocated more or less parallel and about 120 feet north of the present proposed revision line "G".

According to our preliminary investigation by both cone penetration and seismic depth-to-rock tests, rock was observed to occur at more or less 7 feet below ground surface at this particular crossing area. With bedrock at such shallow depths, it will be, undoubtedly, most economical to design the new bridge on spread footing foundation.

In the event that this alternative site is chosen for the

new structure, we would advise at least two more boreholes be made down to bedrock at that site to ascertain the nature of the overburden soil and bedrock.

Section 1.7

PERSONNEL

The field work for this project was performed under the supervision of Mr. A. B. McArthur, B.A.Sc., Mr. W. Naumko, P.Eng., and Mr. J. Kilgour, P.Eng.

The seismic survey and depth-to-rock determinations were performed by Mr. C. W. Faessler, Geophysicist.

The airphoto interpretation of the site was provided by Mr. D. K. Erb, P. Eng.

Mr. W. W. F. Wong, P.Eng., was responsible for the writing and completion of this report.

Section 1.8

APPENDICES

**1.81 Tabulated Results of Cone Penetration Tests
and Seismic Survey**

and

General Plan of Site and Subsurface Sections

RESULTS OF CONE PENETRATION TESTS AND SEISMIC SURVEY

(Chainages and offsets referred to C. P. R. Right End of East Rail, Station 26+00 taken as Chainage 0+00).

<u>Test Point No.</u>	<u>Chainage</u>	<u>Offset</u>	<u>Ground Elevation</u>	<u>Depth to Bedrock</u>	<u>Bedrock Elevation</u>
P1	2+04 N	27'W	438.2	7.8'	430.4
P2	1+09 S	19'W	437.6	17.7'	419.9
P3	2+10 S	19'W	436.9	21.6'	415.3
P4	3+11 S	18'W	438.5	15.5'	423.0
P5	4+11 S	16'W	439.9	7.4'	432.5
P6	5+11 S	14'W	439.4	7.0'	432.4
P7	6+12 S	14'W	438.4	11.1'	427.3
P8	7+12 S	14'W	438.1	20.9'	417.2
P9	8+12 S	15'W	436.1	27.3'	408.8
P10	9+12 S	15'W	436.0	10.0'	426.0
P11	10+12 S	16'W	435.0	1.0'	434.0
P12	11+12 S	19'W	436.1	6.8'	429.3
P13	11+17 S	30'W	436.3	10.0'	426.3
P14	7+11 S	21'E	436.8	24.0'	412.8
P15	6+11 S	13'E	438.2	13.0'	425.2
P16	4+91 S	14'E	439.6	9.0'	430.6
P17	3+97 S	14'E	439.7	6.8'	432.9
P18	3+11 S	19'E	440.1	9.3'	430.8
P19	2+10 S	21'E	439.1	17.4'	421.7
P20	1+20 N	31'E	437.8	16.5'	421.3
P21	2+20 N	32'E	438.2	12.2'	426.0
P22	3+20 N	32'E	439.2	4.5'	434.7
P23	4+20 N	33'E	438.8	7.0'	431.8
P24	5+20 N	33'E	438.7	7.1'	431.6
P25	6+20 N	33'E	439.1	10.0'	429.1

<u>Test Point No.</u>	<u>Chainage</u>	<u>Offset</u>	<u>Ground Elevation</u>	<u>Depth to Bedrock</u>	<u>Bedrock Elevation</u>
P26	1+20 N	100'E	437.4	17.8'	419.6
P27	0+20 N	100'E	436.8	8.7'	428.1
GP1	3+19 N	26'W	439.4	4.8'	434.6
G1	4+19 N	24'W	438.6	6.8'	431.8
G2	5+19 N	22'W	439.1	8.3'	430.8
G3	6+19 N	20'W	438.7	9.0'	429.7
G4	4+19 N	100'W	440.4	14.5'	425.9
G5	5+19 N	100'W	440.7	16.1'	424.6
G6	6+19 N	100'W	440.3	8.7'	431.6
G7	7+19 N	100'W	440.2	7.9'	432.3
G8	0+80 S	100'E	440.9	6.0'	434.9
G9	1+80 S	100'E	441.8	7.2'	434.6
G10	2+80 S	100'E	440.7	9.7'	431.0

(Chainages for Points G11 to G20 inclusive referred to Revision Line "G" Highway No. 17)

G11	116+28	-	438.0	12.4'	419.6
G12	117+28	-	439.0	9.6'	429.4
G13	117+78	-	439.0	9.2'	429.8
G14	118+28	-	440.0	10.1'	429.9
G15	118+78	-	440.4	9.6'	430.8
G16	119+28	-	440.4	8.9'	431.5
G17	119+78	-	440.0	10.9'	429.1
G18	120+28	-	440.0	12.2'	427.8
G19	120+78	-	440.0	12.5'	427.5
G20	121+28	-	440.0	12.0'	428.0

1.82 Office Logs of Boreholes

BORING			LOG
SCALE	DEPTH	ELEV	
FT	FT	FT	
0	0.0	438.2	Ground Surface
	2.5	435.7	Organic Top Soil
5			Medium to Stiff Grey Clay and Silt Some Sand
10	10.0	428.2	
	11.2	427.0	Sand and Gravel, Clay
15			Granite - Gneiss Bedrock
20			(End of Boring)
	21.0	417.2	
25			

	silt		gravel
	clay		peat
	sand		fill

SAMPLE CONDITION

 — undisturbed
 — disturbed but represent.
 — fair
 — lost

SS --- split spoon
ST --- Shelby tube
T.W.R --- thin walled piston
D.B. --- diamond bit - Ro

C — consolidation test
M — mechanical analysis
T — triaxial shear
R — permeability
U — unconfined compression

FIELD TESTS

LABORATORY TESTS

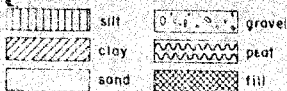
[illegible]

JOB # H583/58 LOCATION near Cobden, Ontario
 CLIENT Department of Highways - Ontario
 COORDINATES Sta. 115+15.0; offset 23.0' left of
 ELEV. (surface) 437.2 (color) Datum D.H.O.
 BOREHOLE NUMBER 2
 DATE (started) Aug. 27/58 (finished) Aug. 27/58
 R/S No. TYPE Longyear

HUNTING TECHNICAL

AND EXPLORATION SERVICES

BOREHOLE No. 2



x — standard penetr. 2 s.s.
 Δ — vane shear
 O — pocket penetrometer

SAMPLE CONDITION



W.S. - Washed sample

SS — spirit spoon
 ST — Shelby tube
 TWP — thin walled piston
 D.B. — diamond bit - rock core

C — consolidation test
 M — mechanical analysis
 T — triaxial shear
 K — permeability
 U — unconfined compression

BORING LOG

FIELD TESTS

LABORATORY TESTS

SCALE	DEPTH	ELEV.	WATER OBSERVATION	LOG	DESCRIPTION	SHEAR STRENGTH (TONS PER SQUARE FOOT) 1/2 1 1/2	STANDARD PENETRATION TEST (BLOWS PER FOOT) 20 40 60	N.	COND.	DEPTH FROM TO	TYPE	RECOVERY LENGTH REC. DIST. DRIV.	PENETRATION RESISTANCE (BLANKS PER FOOT)	ATTERBERG LIMITS W.P. — O.W.I.	WATER CONTENT	REMARKS
FT.	FT.	FT.								FT.	FT.	%		20% 40%	ksf pcf pcf	
0	0.0	437.2			(Ground Surface)											
	2.0	435.2			Organic Top Soil											
5					Stiff Grey											
10	10.0	427.2			Clay and Silt Some Sand			1		5.0	6.5	S.T.	100		58 120 87	
15					Sand and Gravel with Clay			2		10.0	11.5	S.T.	100		118 121 110	
	16.0	421.2						3		15.0	16.0	W.S.				
20	20.0	417.2			Granite Bedrock (End of Boring)			4	X	18.0	20.0	D.B.				

25

BORING LOG

SAMPLE CONDITION		W
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consolidation test

undisturbed

SS split spoon

S.T. — Shelby tube
T.W.P. — thin walled piston

D.B. diamond bit-Rock

[illegible]

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[illegible]

C — consolidation test
M — mechanical analysis
T — triaxial shear
K — permeability
U — unconfined compression

LABORATORY TESTS

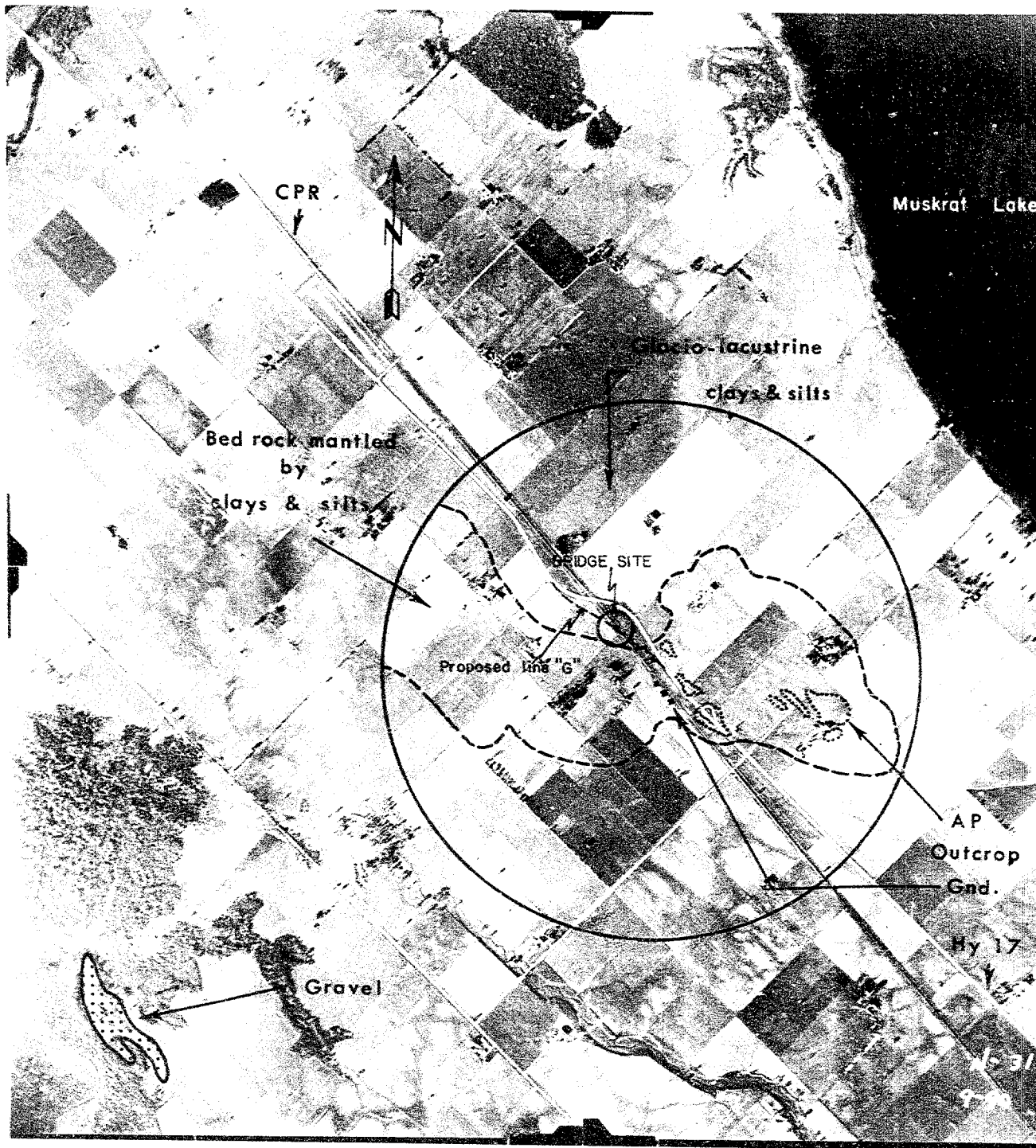
SLALF	DEPTH	ELEV	WATER OBSERVATION	LOG	DESCRIPTION
FT	FT	FT			
	0.0	440.4			Ground Surface
	0.0	438.4			Greenish Top Soil
					Medium to Stiff
	7.0	431.4			Gray Clay and Silt
					W/lt. Sand
	8.5	429.9			Sand and Gravel, Clay
					End of Boring
					Assumed Top of Bedrock

1.83 Air Photo of Site



SITE OF THE PROPOSED CROSSING OF HIGHWAY No. 17
 OVERPASSING C.P.R. RIGHT OF WAY
 TOWNSHIP OF WESTMEATH
 ONTARIO

SCALE - 1 in. = 1320 ft.

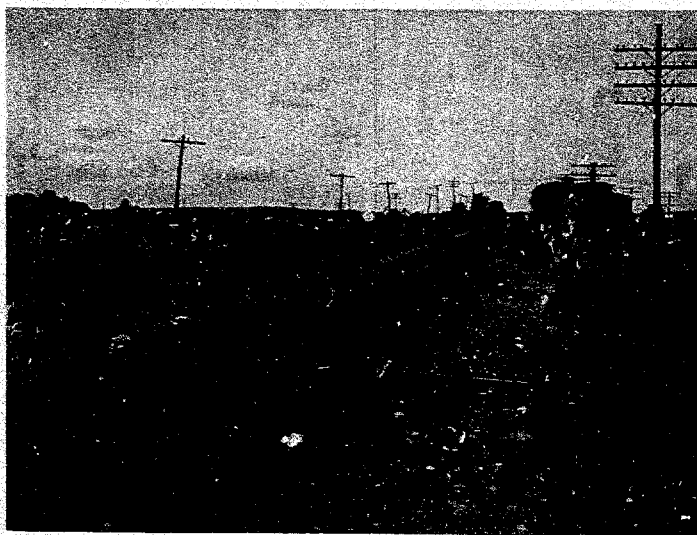


SITE OF THE PROPOSED CROSSING OF HIGHWAY NO. 17
OVERPASSING C.P.R. RIGHT OF WAY

TOWNSHIP OF WESTMEATH
ONTARIO

SCALE 1" = 100 FEET

1.84 Photos of Site



General view of site looking north-west from
location of B. H. 4 (approx.)



General view of site looking west from
location of G8 (approx.)

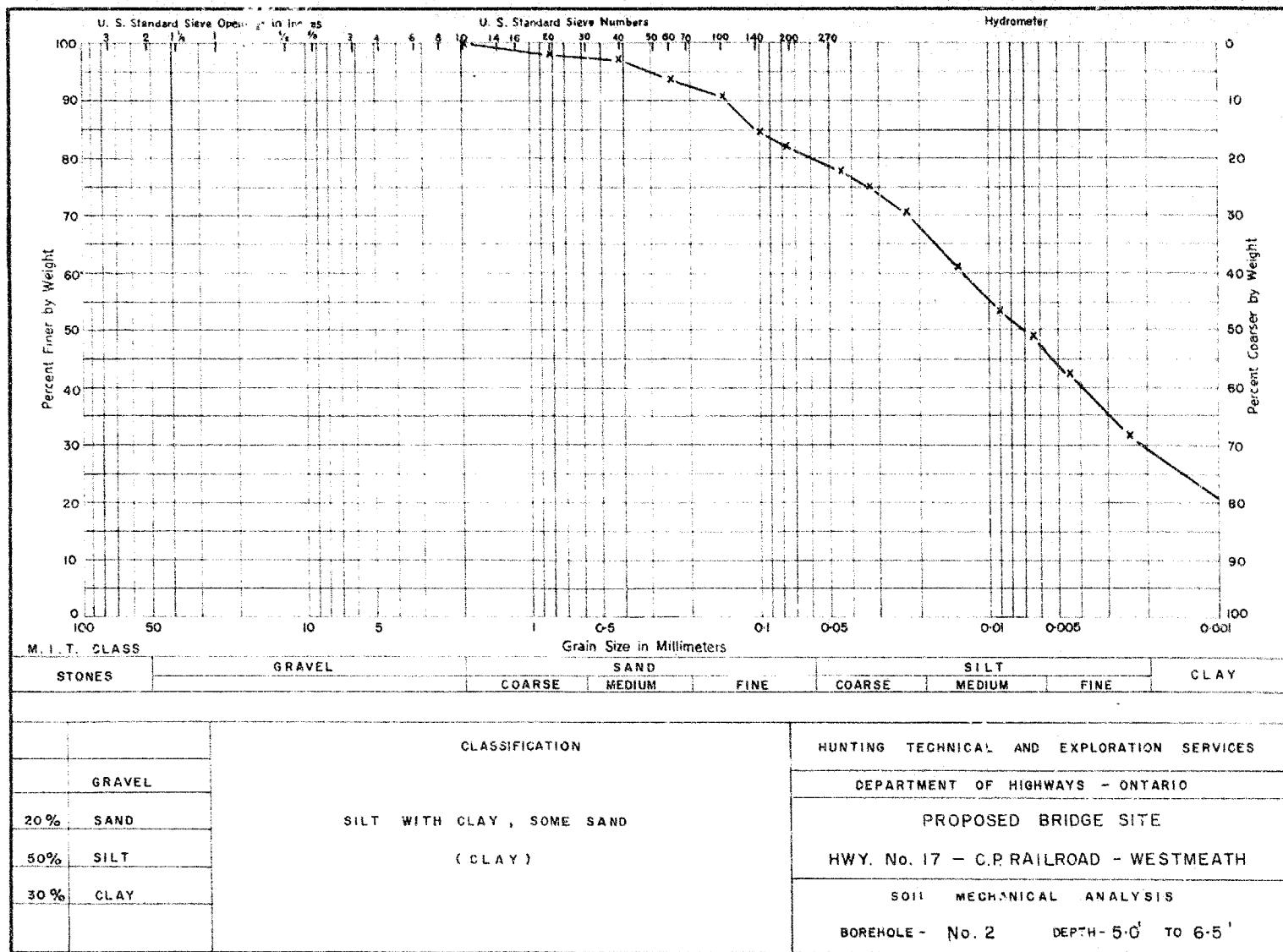


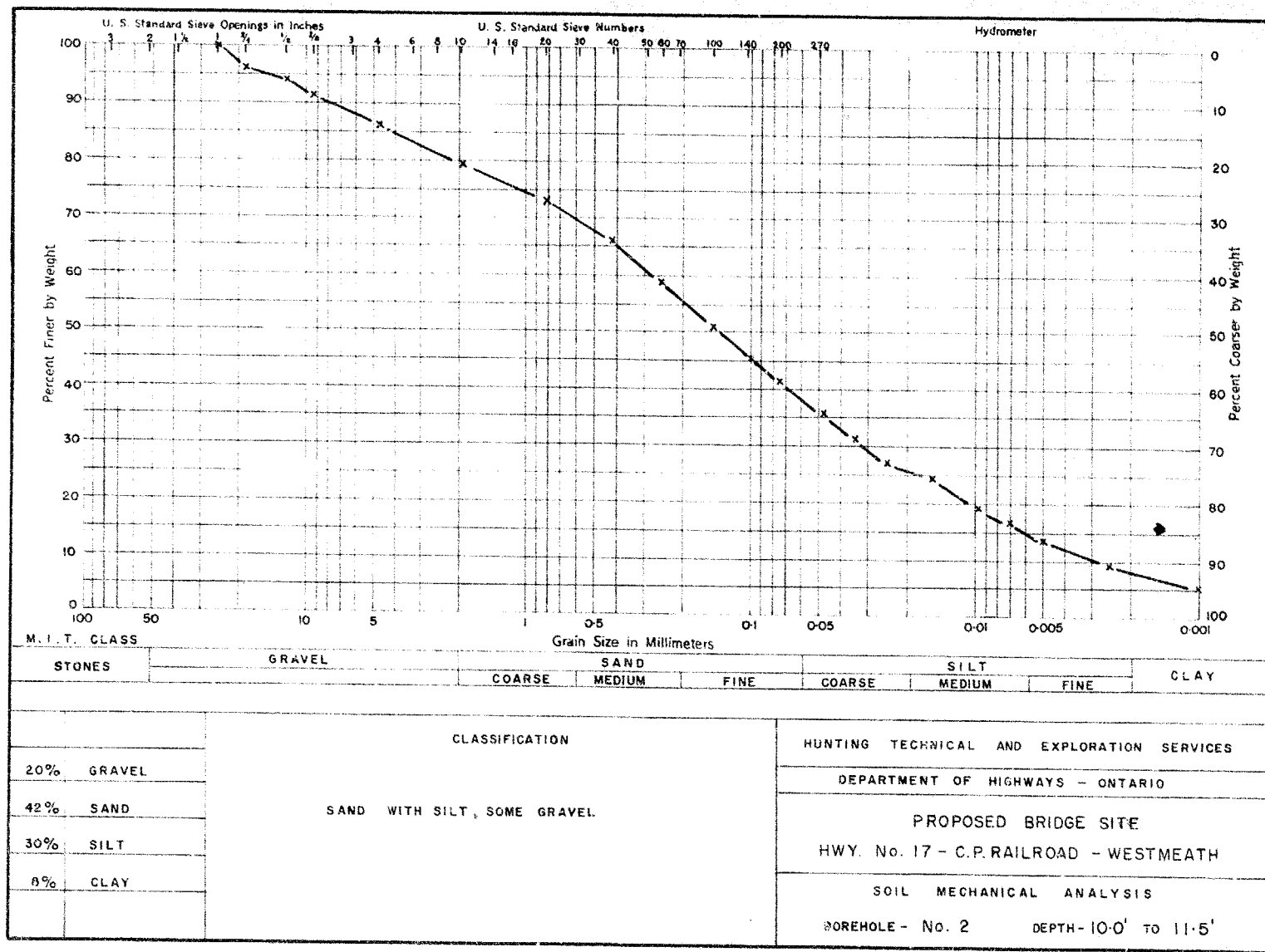
General view of site looking north-west from
location of B. H. 4 (approx.)

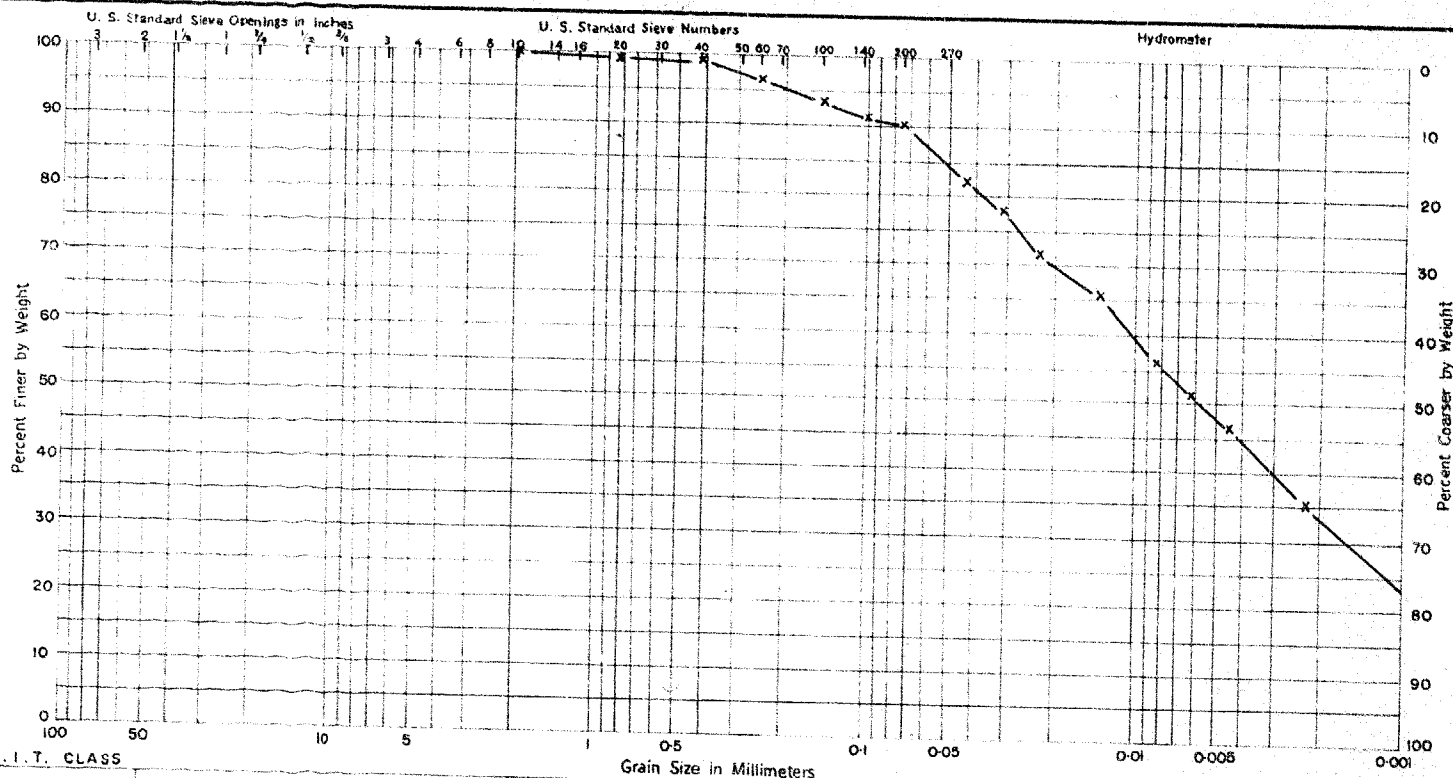


General view of site looking west from
location of G8 (approx.)

1.85 Soil Classification Charts







CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

Soils encountered in sub surface exploration for engineering purposes are composed of organic or inorganic materials, water, air and dissolved salts. The water and air are generally considered to be uniform so that identification is primarily in the nature of organic or inorganic (mineral grains) and dissolved salts.

In the field a soil is generally identified in terms of grain size characteristics, color and mineral content -- properties of the mineral grains. Occasionally, the origin of a soil is included in the identification.

The systems used to describe soils in terms of engineering properties are called classification systems. In the system described below, the soils are first identified and then classified in terms of strength characteristics which are of prime importance in utilizing the soil boring data in designing a safe and economical foundation.

Penetration measured by dropping 140 lb. hammer 30" on 2" O.D. split spoon sampler.

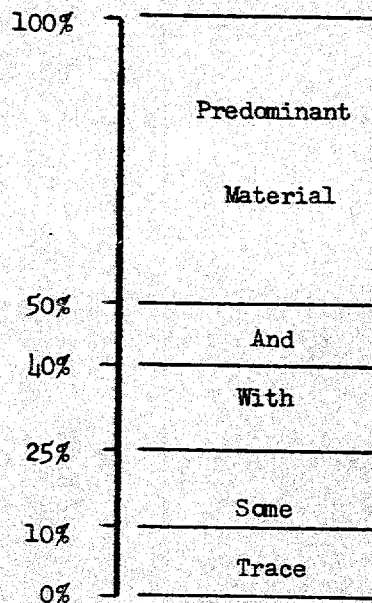
Identification (Soil Type)	Classification	<u>Classification Criteria</u> Unconfined Compressive Strength	
Clay	Soft	Less than 0.50 Tons/Sq. Ft.	
	Medium	0.50 to 1.00 Tons/Sq. Ft.	
	Stiff	1.00 to 2.00 Tons/Sq. Ft.	
	Very Stiff	2.00 to 4.00 Tons/Sq. Ft.	
	Hard	Greater than 4.00 Tons/Sq. Ft.	
Silt	Loose	<u>Density</u> Less than 80 lbs./Cu. Ft.	
	Medium Dense	80 to 95 lbs./Cu. Ft.	
	Dense	Greater than 95 lbs./Cu. Ft.	
Sand	Loose	<u>Relative Density</u> 0 - 30%	<u>Penetration Resist.</u> 0 - 10 Blows/Ft.
	Medium Dense	30 - 60%	10 - 30 Blows/Ft.
	Dense	60 - 90%	30 - 50 Blows/Ft.
	Very Dense	90 -100%	Over 50 Blows/Ft.
			<u>Penetration Resist.</u> Less than 30 Blows
Gravel	Loose	Over 30 Blows/Ft.	
Hardpan	Dense	Cemented or partially cemented sandy gravels, sands, gravels with or without some clay and silt and having unconfined compression strength greater than 5 tons/sq. ft.	
Fill	Organic	Very Loose	0 - 4 Blows/Ft.
		Loose	4 - 10 Blows/Ft.
		Medium	10 - 30 Blows/Ft.
	Inorganic	Dense	30 - 50 Blows/Ft.
		Very Dense	Over 50 Blows/Ft.
Peat	Very Soft	<u>Unconfined Compressive Strength</u> Less than 0.30 Tons/Sq. Ft.	
	Soft	0.30 to 0.60 Tons/Sq. Ft.	
	Stiff	Greater than 0.60 Tons/Sq. Ft.	
Organic Silt (Muck)	Loose	<u>Density</u> Less than 30 lbs./Cu. Ft.	
	Medium Dense	Greater than 80 lbs./Cu. Ft.	

HUNTING TECHNICAL & EXPLORATION SERVICES

1450 O'Connor Drive Toronto, Ontario

SOIL TYPES

The following system was used in classifying the various soils by name:



Example:

Medium dense grey silt with fine sand
(Penet. resist.) (colour) (pred. type) (25%-40%) (other type)
or relative density

Unless believed to have a significant effect on the soil characteristics the minor soil types (i.e. traces) present are disregarded in the name used on the boring log and cross-sections. The complete classification is given with the gradation analysis.

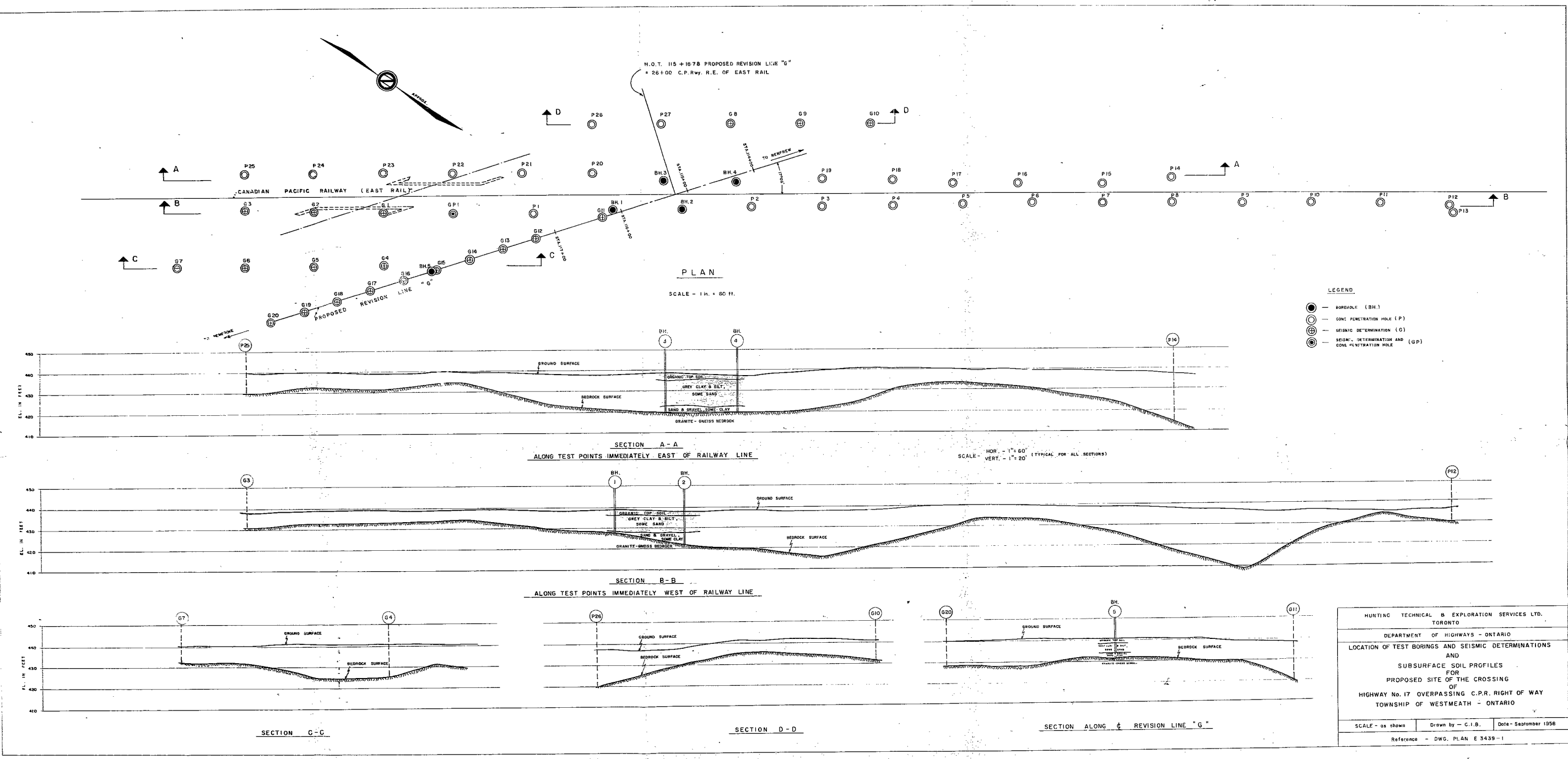
In all cases the strength characteristics (e.g. penetration resistance) is quoted first, followed by the colour and finally the descriptive name based on the mechanical analysis.

#58-F-218-C

HWY. #17

OVERPASS C.P.R.,

NEAR COBDEN



HUNTING TECHNICAL & EXPLORATION SERVICES LTD.
TORONTO

DEPARTMENT OF HIGHWAYS - ONTARIO

LOCATION OF TEST BORINGS AND SEISMIC DETERMINATIONS
AND
SUBSURFACE SOIL PROFILES
FOR
PROPOSED SITE OF THE CROSSING
OF
HIGHWAY No. 17 OVERPASSING C.P.R. RIGHT OF WAY
TOWNSHIP OF WESTMEATH - ONTARIO

SCALE - as shown Drawn by - C.I.B. Date - September 1958

Reference - DWG. PLAN E 5439-1