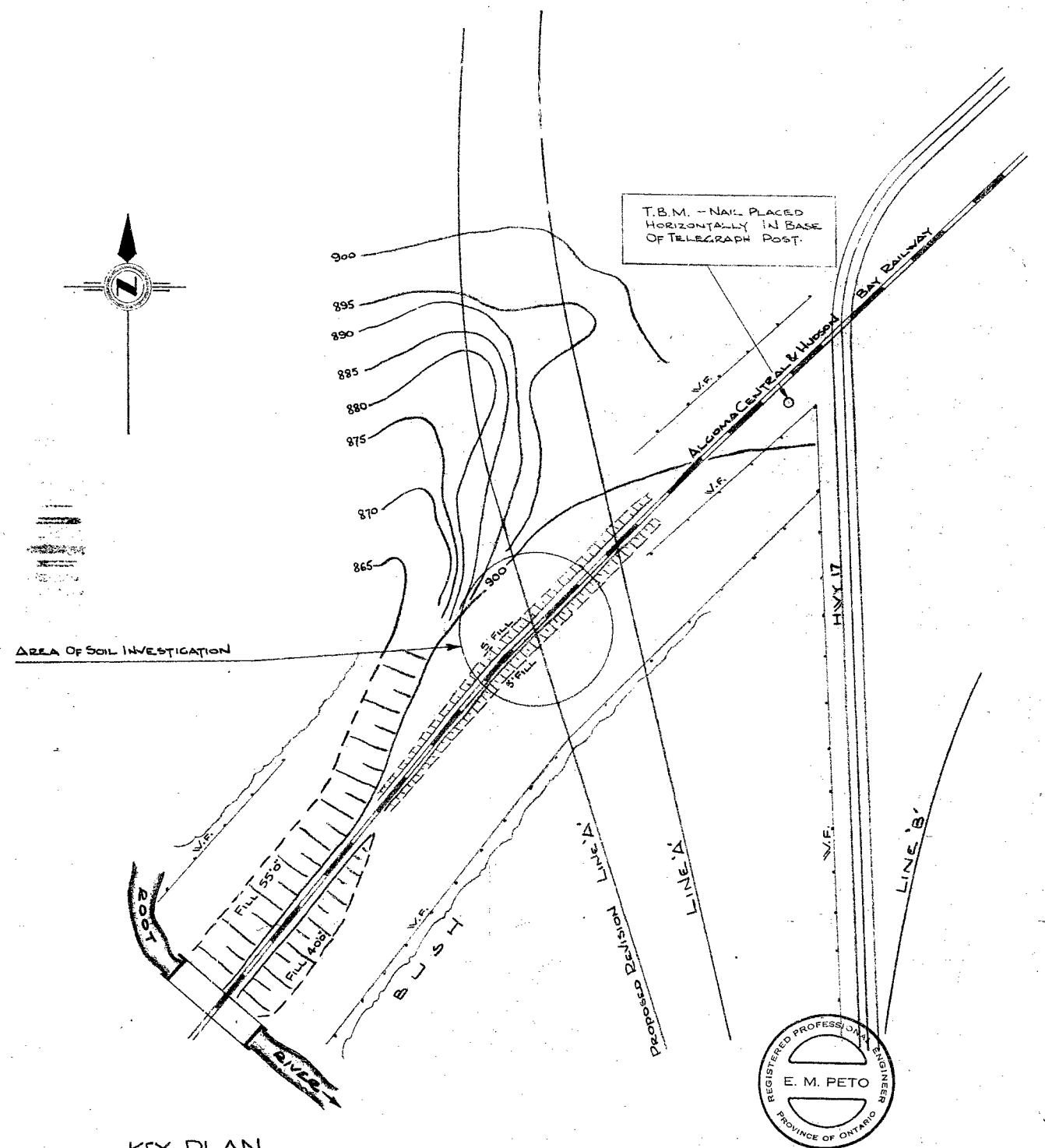


58-F-290C
W.P. 909-58
Hwy. # 17
A.C.+H.B. RAILWAY
CROSSING



KEY PLAN
SCALE 100' TO 1"

e.m. peto & associates ltd.

SOIL SITE INVESTIGATION
AT

HWY 17-A.C. & H.B. RAILWAY CROSSING

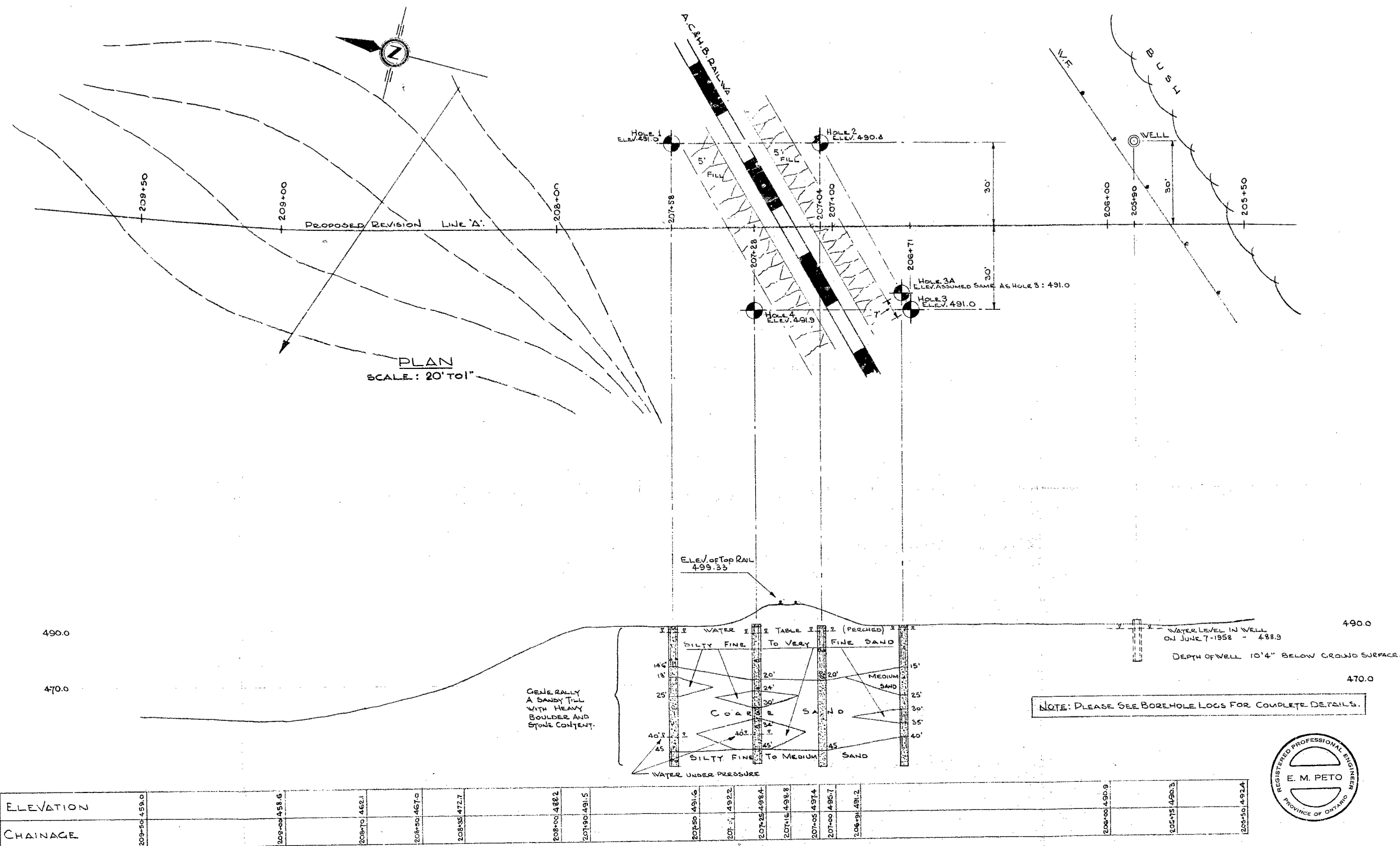
FOR
DEPARTMENT OF HIGHWAYS OF ONTARIO

OUR JOB No. 5869 I

DATE JUNE 25-58

CLIENTS PLAN No. 1-B-327

PER C.T.



PROFILE
HOR. 20' TO 1"
SCALE VERT. 20' TO 1"

NOTE: 1) THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS.
2) ELEVATIONS ARE TO AN ASSUMED T.B.M. AND ACCORDINGLY DO NOT AGREE WITH THOSE SHOWN ON KEY PLAN.
3) ASSUMED T.B.M. - 500.0 (SEE KEY PLAN FOR LOCATION)

e.m. peto & associates ltd.

SOIL SITE INVESTIGATION
AT
HWY 17-A.C. & H.B. RAILWAY CROSSING
FOR
DEPARTMENT OF HIGHWAYS OF ONTARIO

OUR JOB No. 5869-III DATE JUNE 30-58
CLIENTS PLAN No. 1-B-327 PER: G.T.



58-F 290c

DEPARTMENT OF HIGHWAYS OF ONTARIO

HIGHWAY 17 - A. C. & H. B. RAILWAY CROSSING
W. P. 909 - 58 ODENA, ONTARIO

SOILS REPORT

by

J. H. FETC ASSOCIATES LTD.

TORONTO, ONTARIO

July, 1958.

e. m. peto associates ltd.

YOUR REFERENCE:- **W. P. 909-53**
OUR REFERENCE:- **5869**

**850 roselawn avenue,
TORONTO, ONTARIO.
RUssell 1 - 4951.**

4th July, 1958.

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

For the attention of Mr. J. C. McAllister, P. Eng.

Dear Sir,

**Soil Site Investigation
Highway 17 - A. C. & E. B. Railway Crossing**

We refer to your letter dated 27th May, 1958, and have pleasure in forwarding herewith four (4) copies of our report on this investigation for your attention.

Generally we consider this site presents a problem related to drainage rather than to soil structural strength. We have considered both these aspects in some detail in the report, together with supporting appendices and drawings. Here for your convenience we are summarizing our findings and recommendations for your consideration.

1. The soil is basically a sandy till with high boulder and stone content, varying with depth; for simplicity of presentation we have described it under four strata entitled

- (a) Topsoil, which is a black organic sandy loam approximately 12 inches thick.**
- (b) A grey to brown silty very fine to fine sand, which is compact to very dense, and arises below the topsoil down to a depth varying from 20 to 25 feet below ground surface.**

- (c) A very dense brownish grey coarse sand, arising below the silty very fine to fine sand, down to a depth of approximately 45 feet.
- (d) A very dense fine to medium sand arising below the coarse sand stratum referred to in (c), down to the termination of the test holes at a depth of 50 feet below ground surface.

2. Free ground water was encountered at two distinct levels in considerable quantity. Firstly there is a perched water table arising in the upper 3 to 10 feet, and this appears to be surface drainage water collected and retained in the area. Secondly there appears to be a water bearing stratum occurring below a depth of 40 feet; this water appears to be under pressure and may well be subterranean water feeding the river. There were clear indications of this particular condition in test holes # 1 and 4, but this was not apparent at test holes # 2 and 3A. The implication at these two latter test holes at these depths was rather the presence of a permeable stratum, which at the time of the investigation was not water bearing.

3. It is apparent that below a depth of 5 feet the soil is in a dense to very dense state, and this condition is to a large extent due to the heavy boulder and stone content in the various strata. Accordingly, from this aspect there is a wide range of elevation suitable for a heavy structure foundation. Furthermore in view of the difficulty encountered in driving the casing we are of the opinion that consideration of a piled foundation can be ruled out on this score, apart from being unnecessary in view of the type and generally dense condition of the soils arising at this site.

4. We are not in possession of a profile showing the proposed road surface level. However, on the basis of a number of reasonable assumptions regarding construction depth of the rail girders and overhead clearance, etc. required, we have concluded that the road surface will be approximately 15 feet below existing ground level or at elevation 476.00 approximately to our datum. Having in mind the presence of a water table, we have been guided by the susceptibility of the soil at this elevation to frost damage, when considering the elevation at which the bridge foundation should be placed. In our opinion and with due regard to the location and fairly exposed conditions applicable to this site, we believe the foundations should be placed not less than 5 feet below finished grade, or at elevation 471.00 approximately.

5. Accordingly, and irrespective of the type of footing finally adopted, we recommend a permissible bearing load of 5 tons per square foot subject to a minimum dimension in any direction of 4 feet.

Settlement under such a loading intensity should not exceed 1 inch and differential settlement will be some fraction of this.

6. We have not carried out stability analyses of the material with respect to the safe slope to be adopted in the cut, as we consider this material will be stable at your standard 2:1 side slope provided suitable drainage provisions are incorporated in the work. To this end we suggest prior to opening up the ground two deep drains be excavated parallel to the centre line and outside the final area of the excavation in order to partially drain at least the looser material near the surface. We are of the opinion these drains should be cut to a depth of 5 feet below existing ground surface and should fall toward the blind valley at chainage 209 + 00 approximately.

During the course of the excavation below this depth, temporary ditches should be maintained at the foot of each slope in order to collect surface water draining through to the cutting.

Subsequent to the completion of the bulk excavations for the cutting, considerations of drainage requirements for both the cutting side slopes and the pavement subgrade lead us to recommend the provision of

- (a) Counterfort drains to the cutting side slopes
- (b) A deep blind drain below the inverts, and on the line of the pavement side drains.

These latter drains should be approximately 2 feet below the bridge foundation, and in addition to protecting the subgrade would also collect surface water from the cutting counterfort drains. Where these drains pass along and below the bridge footings they should be back filled up to the footing elevation with a weak mass concrete.

covering letter for **Mr. A. M. Toye,**
Chief Bridge Engineer.

Sheet No. **4.**

Similar but smaller (8 inches diameter) sub soil drains should also be provided behind and below the main abutment and wing wall footings. These drains would be connected to the main blind drain and the design regarding filter material surround and weak concrete back fill would be similar to the blind drain. The invert for these drains need not exceed a depth of 12 inches below the footing elevation.

We have discussed the use of deep well drains in the soils report, however we do not feel at this stage that the information on the water at depth is sufficiently conclusive to indicate the necessity of vertical drains. However, subsequent to the excavation of the cutting a condition may arise indicating the need for such drains, in which case they could be incorporated without great difficulty, and for this reason we suggest this decision should be made on the site, although provision for this work could be included in the contract documents.

7. In view of the high boulder and stone content as well as the dense condition of these soils excavation will be difficult, and in the bulk excavation for the cutting rooting may have to be resorted to in order to loosen the material for normal mechanical excavation. The soil removed will make good fill material for the embankment, which it is presumed will be provided to cross the blind valley immediately to the North of the bridge site; subject to similar soil conditions and proper compaction with normal bank protection work, bank side slopes of 1-1/2 : 1 will be quite in order.

Generally this material is frost susceptible and for this reason we suggest that it should not be used within 2 feet of final grade in the cutting, or the embankment where this is near a free water table.

We believe we have covered all matters arising from this investigation regarding which you wish to be informed. However, in the event of there being some point on which you require further information we shall be pleased to be of further service.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

GYS:pf


E. M. Peto, P. Eng.

Job No. 5869

Client's Ref. No. W. P. 909-58

Date 4th July, 1958.

Report on

SOIL SITE INVESTIGATION

at

HIGHWAY 17 - A.C. AND H.B. RAILWAY CROSSING

W.P. 909 - 58 ODENA, ONTARIO

for

DEPARTMENT OF HIGHWAYS OF ONTARIO.

INTRODUCTION

In accordance with written instructions from Mr. J. C. McAllister, dated May 27th, 1958, a soil investigation was carried out at the site of the proposed crossing of Highway 17 (T. C. H.) and A. C. and H. B. Railway.

The object of the investigation was

- (a) to determine the feasibility of constructing an underpass
- (b) to make recommendations for the type of foundations best suited to the soil condition at the site.
- (c) to determine the allowable bearing capacity of the soil at the elevation of the proposed foundation structure
- (d) to give advice on safe slopes and safe construction practice for the deep cut of the underpass.

SUPER IMPOSED DOCUMENT MAY
APPEAR AS MULTI-FEED ON FILM

PROGRAMME OF WORK

June 3rd, 1958	Equipment trucked to the site from Toronto.
June 4th, 1958	Reconnaissance of site by field engineer, test holes staked out.
June 4th - 6th, and 14th - 16th, 1958	Driving borehole # 1.
June 6th - 13th, 1958	Driving borehole # 4.
June 16th - 18th, 1958	Driving borehole # 3.
June 19th - 23rd, 1958	Driving borehole # 2.
June 23rd - 26th, 1958	Driving borehole # 3A.
June 27th, 1958	Crew and equipment moved from site to nearby bridge site.

GENERAL INFORMATION

1. Our standard soil sampling procedures were followed. These are described in Appendix II.
2. Boreholes # 1, 2, 3A and 4 were driven to a depth of 50 feet below ground surface; borehole # 3 was terminated at a depth of 27 feet, because of boulder interference and damaged casing.
3. A site plan showing the soil test hole locations together with a profile along the centre line of revised line "A" alignment, and detailed individual borehole logs are included.

The ground surface elevations are referred to an assumed benchmark, which is a 6 inch nail driven horizontally into the base of a telegraph pole 50 feet West of present Highway 17, and 20 feet South-West of the railway tracks at the existing level crossing some 500 feet North-East of the present site. The elevation of the nail was taken to be 500.00. The top of the South rail at the proposed new Highway 17 centre line is at elevation 499.2 to this assumed datum.

SITE AND GEOLOGY

The approximate general elevation of this site is \pm 900 (geodetic datum). The topography is gently rolling to hilly. There is a steep slope from the site down to the Root River and in addition there is a ravine draining directly to the river and within 100 feet of the site in a Westerly direction. The shortest distance between the site and the Root River is about 300 feet. The HWL of the river at the Highway 17 - Root River # 1 crossing is approximately at elevation 806.5 (Geodetic datum).

None of the boreholes at this site reached the underlying bedrock. However, from other investigations and from available geological information it is known that the area is underlain by hard, Precambrian bedrock, consisting mainly of Gneiss, Granite and other acidic rocks.

The soil profile under investigation was probably deposited in the Wisconsin era.

SOIL CONDITIONS

Below the topsoil there is a till with a heavy boulder and large stone content. This till is variable and for clarity has been described in four strata.

(a) Topsoil

The topsoil is a black organic sandy loam of about one foot thickness.

(b) Upper Silty Fine to Very Fine Sand

Below the topsoil and to a depth varying from 20 to 25 feet there is a light grey to brown silty fine to very fine sand with grits and pebbles. The layer is not uniform, since it contains seams of silt and coarse sand and this characteristic is reflected by natural moisture contents, which lie between 24.8% and 8.3%. In all probability the higher moisture contents correspond with silt pockets and the lower moisture contents with coarse sand seams.

(c) Coarse Sand

Below the silty fine to very fine sand and at a depth of 45 feet, there is a layer of brownish-grey to grey coarse sand.

In borehole # 1 a fairly thick seam of coarse sand was found in the silty fine to very fine sand between the depths of 14-1/2 to 18 feet.

The sand in borehole # 4 is stratified and alternating layers of the coarse sand and the silty fine to very fine sand were found.

(d) Lower Silty Fine to Medium Sand

Below the coarse sand the boreholes were terminated in a grey-brown silty fine to medium sand. This layer is similar to the upper fine material but coarser and more uniformly graded.

TEST RESULTS

1. Standard Penetration Test

The soil is compact to a depth of about 5 feet with an average standard penetration test result of 35 blows per foot.

Below this depth the relative density of the soil is dense to very dense with standard penetration values ranging from 49 to 720 blows expressed in terms of a drive of 12 inches. The average penetration is 295 blows per foot.

The wide scattering of the penetration values reflects the heterogeneity of the soil profile. Penetration values in the lower range probably indicate relatively loose pockets of fine-grained material. The occasional high penetration values were probably obtained where boulders or pockets of coarse sand and gravel were encountered.

There is a marked improvement in the penetration test results from the 30 feet depth down. At this depth an average penetration of 450 blows per foot was found.

2. Moisture Content

Moisture samples were taken and moisture content determination was made on samples taken at convenient intervals. The varying moisture contents reflect the heterogeneity of the profile and confirm the identification of coarse sand and silt pockets and seams.

3. Mechanical Analysis

The mechanical analyses run on representative samples show that the silty fine to very fine sand is frost susceptible.

WATER CONDITIONS

The general surface and subdrainage is toward the Root River.

The whole area under investigation presently appears to contain considerable ground water at two distinct levels.

In the upper 3 to 10 feet of the profile seams of medium to coarse sand were found frequently and free flow of water was noted from these seams, which suggests that these are interconnected.

At a depth of about 40 to 50 feet in holes # 1 and 4, a rich water bearing strata was found with water behaving like Artesian water. The water rose in the holes and the flow was sufficient to render bailing below a depth of 3 feet ineffective. This latter condition was not repeated in holes # 2 and 3A, where there appears to be a permeable stratum which may be water bearing, but was not so at the time of the investigation.

ENGINEERING CONSIDERATIONS

1. The depth of foundation at the site will be dictated by the structural design of the railway bridge, by the vertical alignment of the proposed highway, the standard clearance for the underpass and the soil condition at the site. Placing the foundation structure on the upper silty fine sand, the minimum depth of foundation below the free surface should be the depth of frost penetration which is estimated at the site to be 5 - 6 feet.

ENGINEERING CONSIDERATIONS (contd)

In considering the silty fine sand as the supporting media for the foundation structure it should be kept in mind that the soil is frost susceptible, and the ground water level will be near or may be near to any free surface if proper steps are not taken to lower the ground water.

The coarse sand is the superior material on which to place the foundation structure, but in this case too, the minimum depth of foundation below the free surface of 5 - 6 feet should be observed. If the lower silty fine to medium sand is chosen as the support for the foundation structure, the same precautions described above for the upper silty fine sand will be necessary.

Based on standard penetration test results available, we feel that a pile foundation at the site is not a practical or economical proposition. The penetration values indicate very high bearing capacity values, and either column or strip footings could be used.

2. Assuming that the bottom elevation of the footings for the railway bridge will be between 15 to 25 feet below the existing ground surface, the safe allowable bearing capacity is 5 tons per square foot for footings with minimum width of 4 feet.

It is not desirable to use footings less than 4 feet in width in this material because local weakness of the stratum may affect the foundation. It is essential that, with the relatively high bearing values recommended, the soil on which the foundation will be placed, remain undisturbed. This can be achieved using proper drainage facilities and careful excavation practice.

3. Due to the existing ground water condition it is of extreme importance to provide adequate drainage facilities to keep the excavation dry during construction, and the water table below the road and bridge structures during the life time of the road.

The Root River is a natural drainage outlet. The seepage water from the seams of the upper fine silty sand in the cutting can be collected via counterfort drains into side ditches along the excavation and directed toward the ravine to the North. It will be necessary to keep the excavation open to the ravine at all stages of the work.

ENGINEERING CONSIDERATIONS (contd.)

Opening up the excavation will relieve the overburden pressure and consequently the ground water which has been found in holes # 1 and 4 may break through. Accordingly, it may be advisable to relieve the water pressure from this layer by vertical drains connected to an underdrain system placed on either side of the excavation with outlets to the ravine. However, a final decision regarding the necessity of such relieving wells could be left to be made on the site before the final pavement is laid, in which case such wells will have to be excavated below the deep blind drains and connected to them. These blind drains should consist of pipes perforated on the lower third of the circumference and laid on a compacted 6 inch layer of fine filter material, followed by back filling the trench with similar filter material compacted to maximum density to a point 12 inches below the open side ditch invert. This latter 12 inches should consist of an impermeable plug of clay or similar material. The invert of the blind drains will be determined by the footing elevation of the bridge and in this connection we suggest the invert need not exceed a depth of 2 feet below the footing. Where this drain passes below and along the footings we suggest (a) the pipe is not perforated and (b) the drain trench is back filled with weak concrete to the footing elevation. A separate local subsoil drain should also be provided behind and below the footings of the wing walls and main abutment of the bridge. The invert of this drain should be approximately 12 inches below the footings and similar in design to the blind drains though they need not exceed 6 inches in diameter. The back fill to the underside of the footings should again consist of weak concrete. These local drains should be connected to the main blind drain to the immediate North of the structure.

4. Generally for granular material similar to that found at the site we would recommend side slopes of 1-1/2 horizontal to 1 vertical for deep excavation. However, due to the existing ground water condition at the site it is advisable to reduce these to slopes of 2 horizontal to 1 vertical.

Subject to similar subsoil conditions for fill, we consider a slope of 1-1/2 horizontal to 1 vertical adequate for the material found below the 2 feet depth on the slope.

5. The silty fine to very fine sand is frost susceptible, for this reason it is advisable to avoid the use of this material in the upper 2 feet in fill and remove this soil in the upper 2 feet below subgrade in cut where the free ground water level is near to the surface.

ENGINEERING CONSIDERATIONS (contd.)

6. Properly compacted, the soil to be excavated at the site is potentially good fill material subject to the restrictions mentioned in paragraph 5.

In the event of the existing moisture conditions remaining reasonably constant, it is likely that the natural moisture content of the soil when excavated will be higher than optimum, however since this material is granular this will not interfere unduly with the compaction operation.

E. M. PETO ASSOCIATES LTD.

GYS:pf


E.M. Peto, P. Eng.

BOREHOLE LOG

Checked By P. A.

ABBREVIATIONS

R. C. ROCK CORE

HOLE TERMINATED

BOREHOLE LOG

Borehole No. 2.

Boring Date JUNE 19TH. - 21ST. 1958.

Checked By F. A.

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

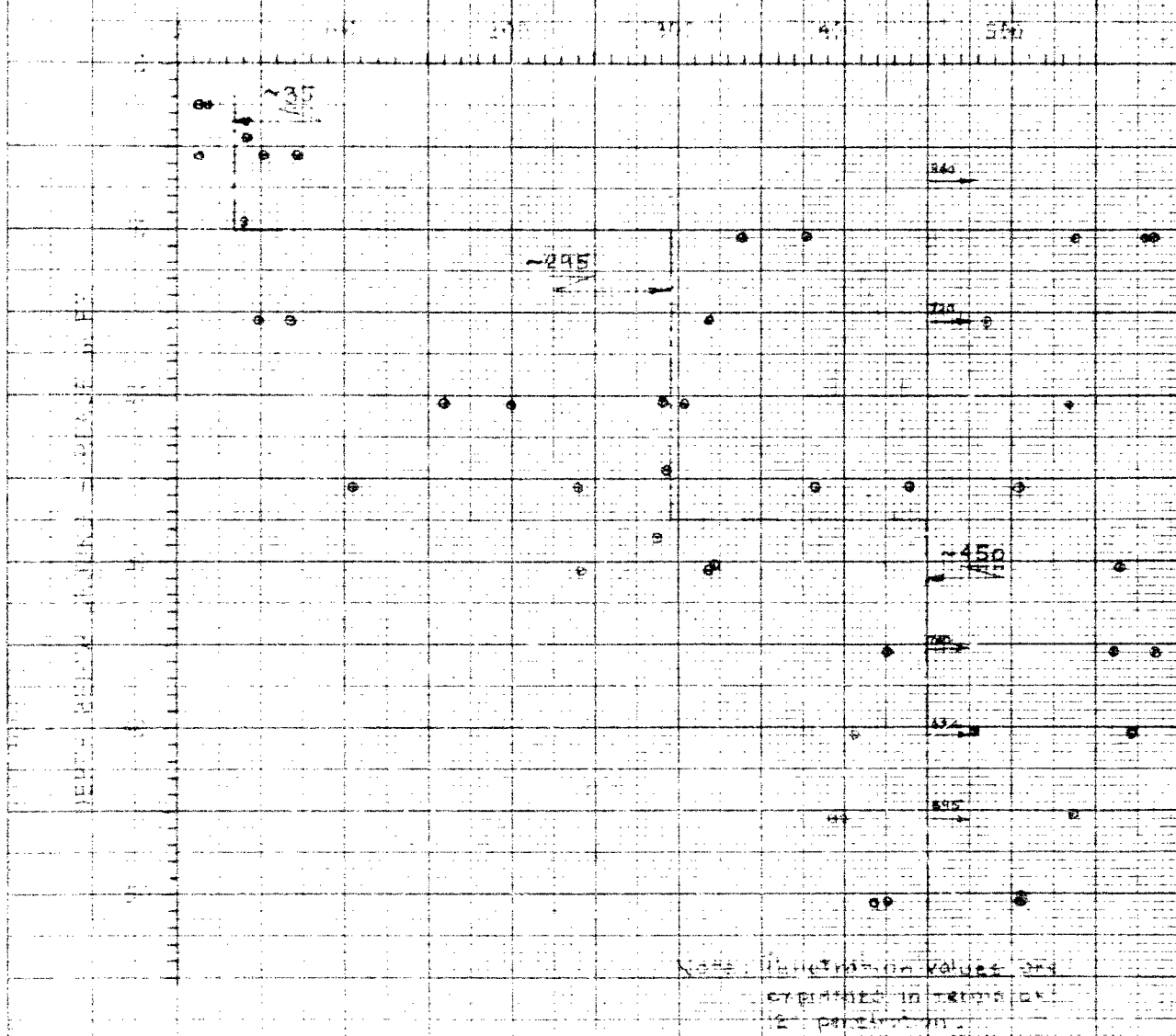
Q/4. UNCONFINED COMPRESSIVE STRENGTH

W. L. WATER LEVEL IN CASING

W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0' 0"					
SANDY ORGANIC LOAM	BLACK		4' 9.8"					(TOP SOIL)
			5' 0"					
SILTY FINE SAND, GRITS, ODD PEBBLE	LT. GREYISH-BROWN	VERY DENSE			1	S.S.	51	M.C.D. 8% MOIST BOULDERS FROM 7'-9'
			10' 0"					
AS ABOVE WITH ROCK FRAGMENTS	AS ABOVE	VERY DENSE			2	S.S.	130/6"	MOIST
			15' 0"					
MEDIUM FINE SAND	LT. GREYISH BROWN	VERY DENSE			3	S.S. W.S.	248/6" -	NET BOULDER FROM 16'-17' BOULDERS FROM 18'-19'
			17' 6"					
			20' 0"					
COARSE SAND	BROWNISH - GREY	VERY DENSE			4	S.S. W.S.	100/3"	NET
			25' 0"					
FINE TO MEDIUM SAND, SOME GRIT & PEBBLE	REDDISH BROWN	VERY DENSE			5	S.S.	150/6"	" 8.4% MOIST
			30' 0"					
AS ABOVE	-"-	-"-			6	S.S.	111/6"	" 8.4% MOIST
WIT MORE GRIT & PEBBLE			35' 0"					
			40' 0"					
AS ABOVE	-"-	-"-			7	S.S.	200/5"	" 8.4%
WITH ROCK FRAGMENTS			45' 0"					
			50' 0"					
AS ABOVE	-"-	-"-			8	S.S.	165/6"	MOIST
AS ABOVE	-"-	-"-			9	S.S.	185/6"	M.C.D. 8% MOIST
WITH OCCASIONAL GRAVEL								
AS ABOVE						W.S.		
NO GRAVEL					10	S.S.	175/6"	
				HOLE TERMINATED				NOTE: LADING 18.5" HOLE 50' 6", AFTER 30 MIN. WL FALL TO 33'

STANDARD PENETRATION TEST

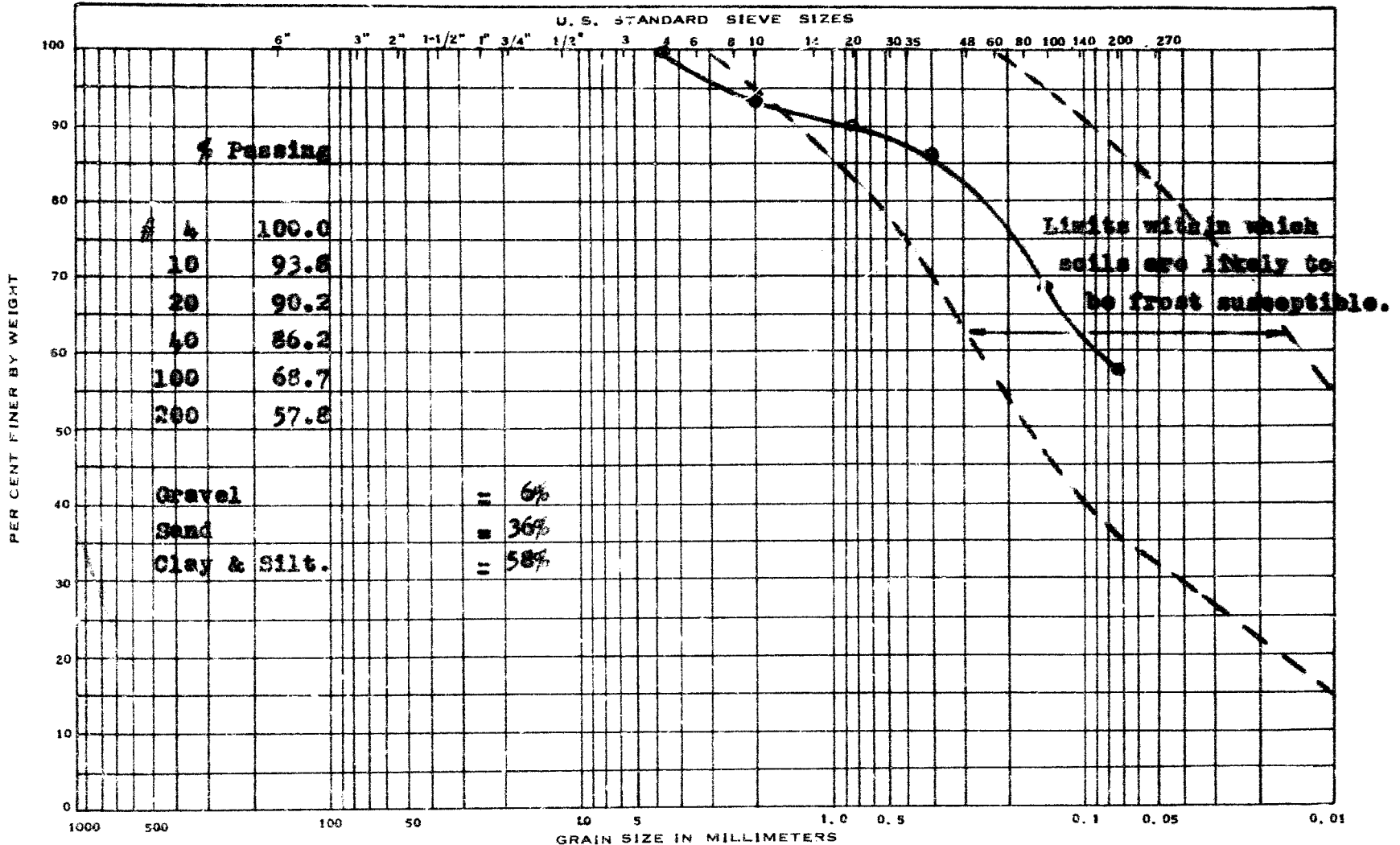


STANDARD PENETRATION TEST

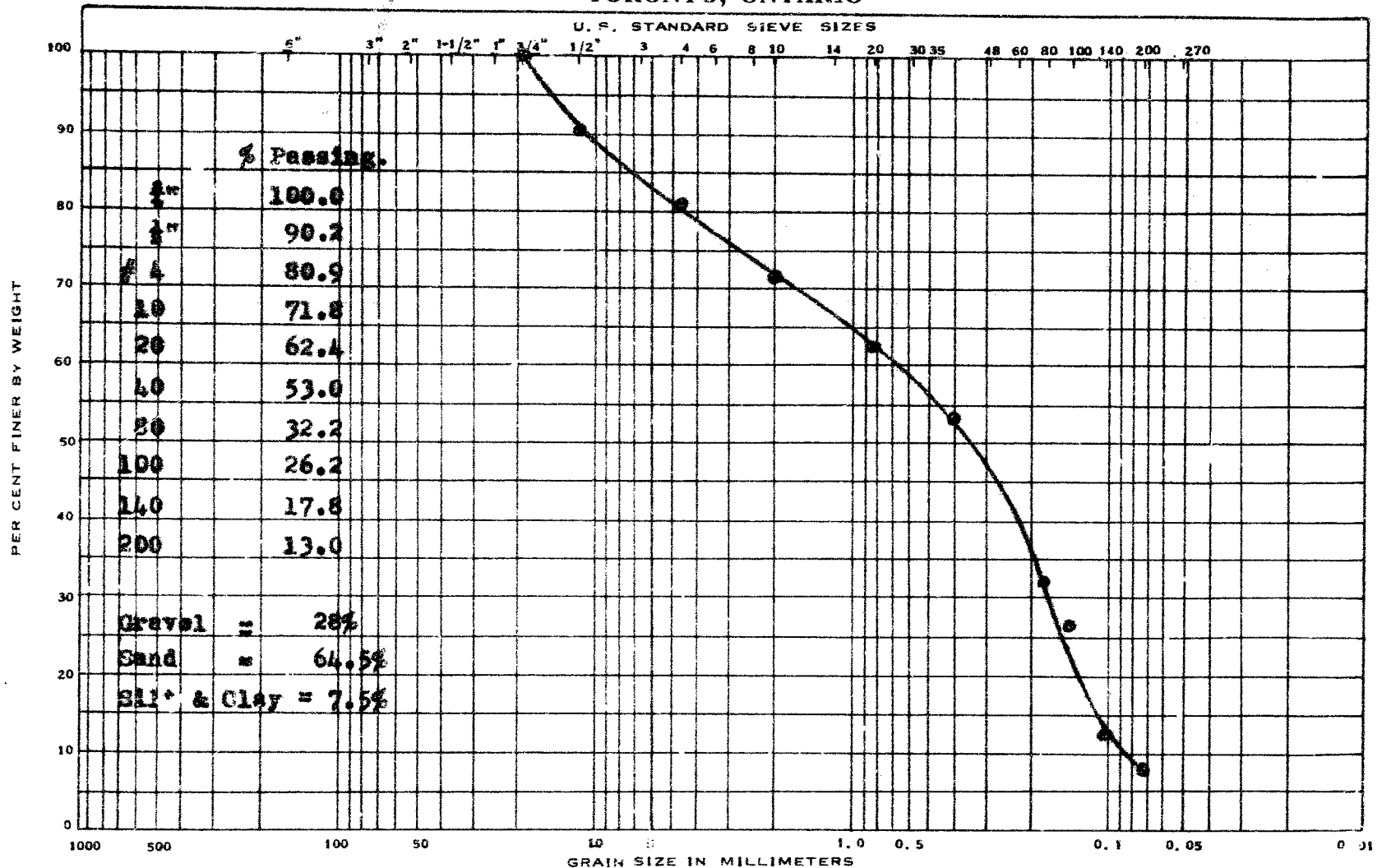
APPENDIX I

LABORATORY TEST RESULTS

e. m. peto associates ltd.
TORONTO, ONTARIO



e. m. peto associates ltd.
TORONTO, ONTARIO



BOULDERS	STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT
----------	--------	--------	-------------	-----------	-----------	-------------	-----------

Rwy. 17 - A.C. & H.B.

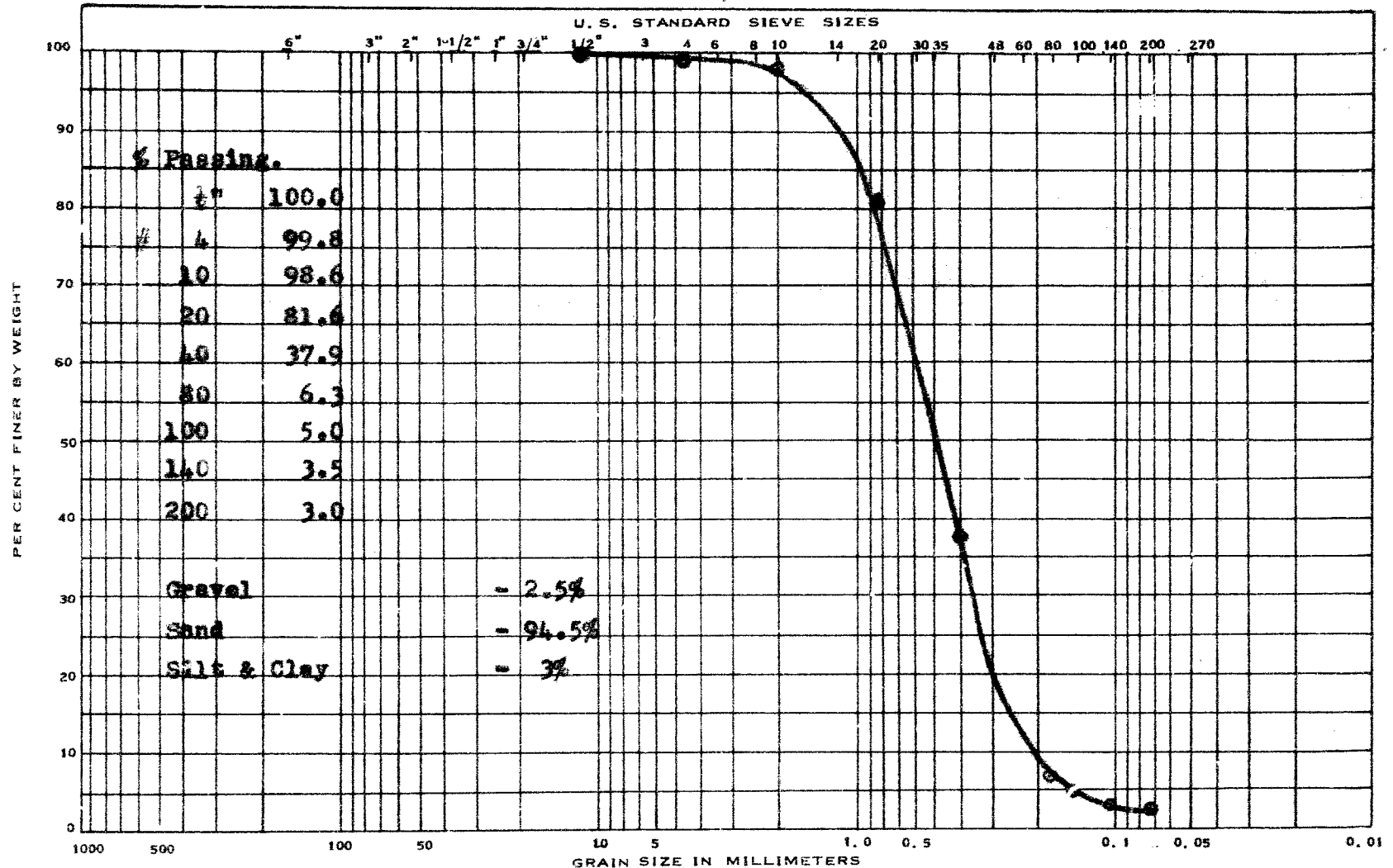
MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Railway Crossing. JOB NO. 5869 HOLE NO. 4 SAMPLE NO. 11

DEPTH 35'-35'6" ELEVATION _____ REMARKS Typical sample from the coarse sand layer.

GRAIN SIZE DISTRIBUTION DIAGRAM
COARSE MATERIALS

e. m. peto associates ltd.
TORONTO, ONTARIO



BOULDERS	STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT
----------	--------	--------	-------------	-----------	-----------	-------------	-----------

MASS. INST. OF TECH. CLASSIFICATION

Hwy. 17 - A.C. & H.B.

JOB NAME Railway Crossing

JOB NO. 5869

HOLE NO. 1

SAMPLE NO. 10

DEPTH 40-41'

ELEVATION

REMARKS Typical sample from the coarse sand layer.

GRAIN SIZE DISTRIBUTION DIAGRAM
COARSE MATERIALS

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 inch or 3 inch 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 inch 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 cased 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.