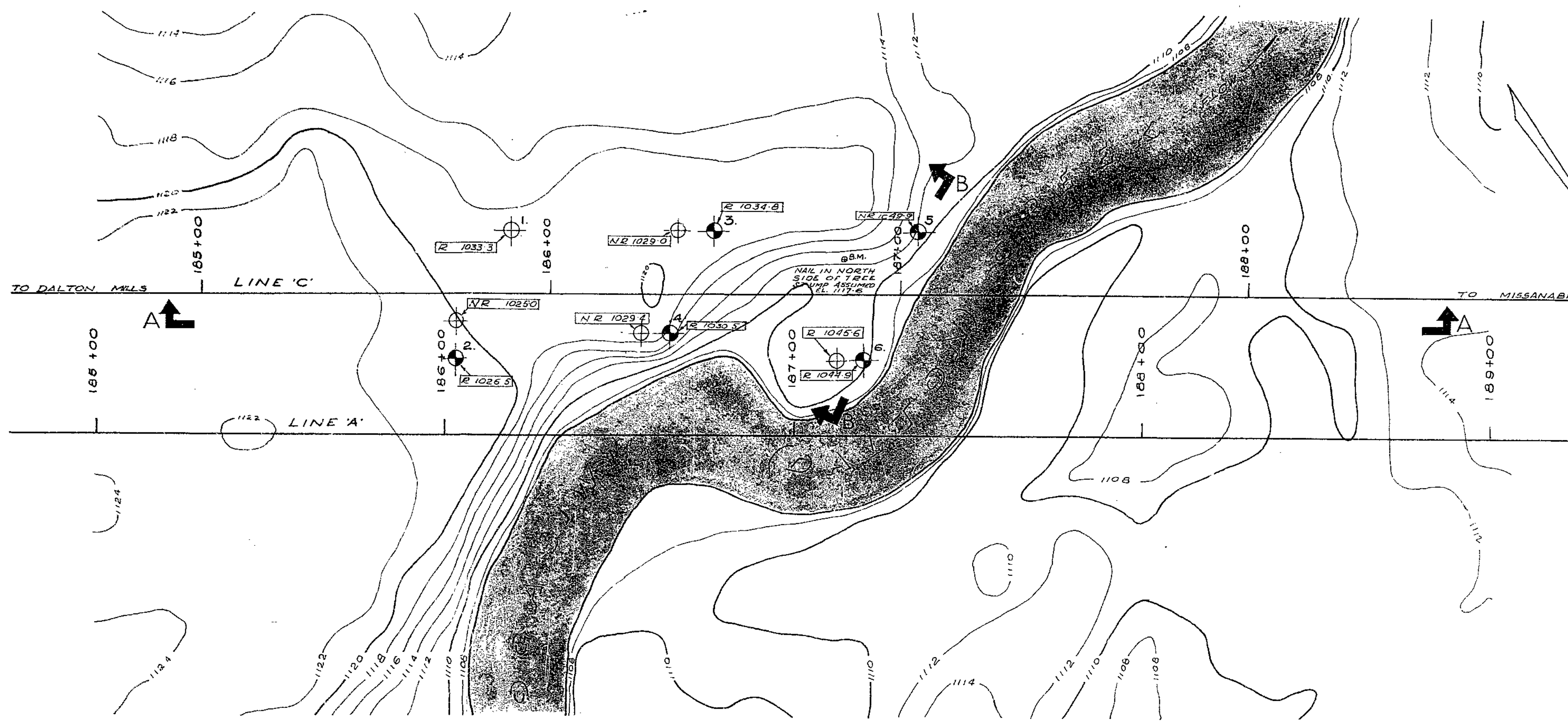
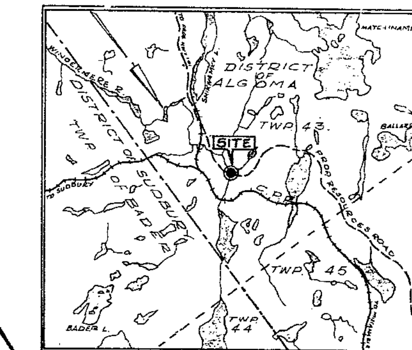
#62-F-244C
W.P. #231-61
Hwy #651
OGASIWI
CREEK



PLAN
SCALE 1"=20'



KEY PLAN
SCALE 1" = 2 MILES

LEGEND

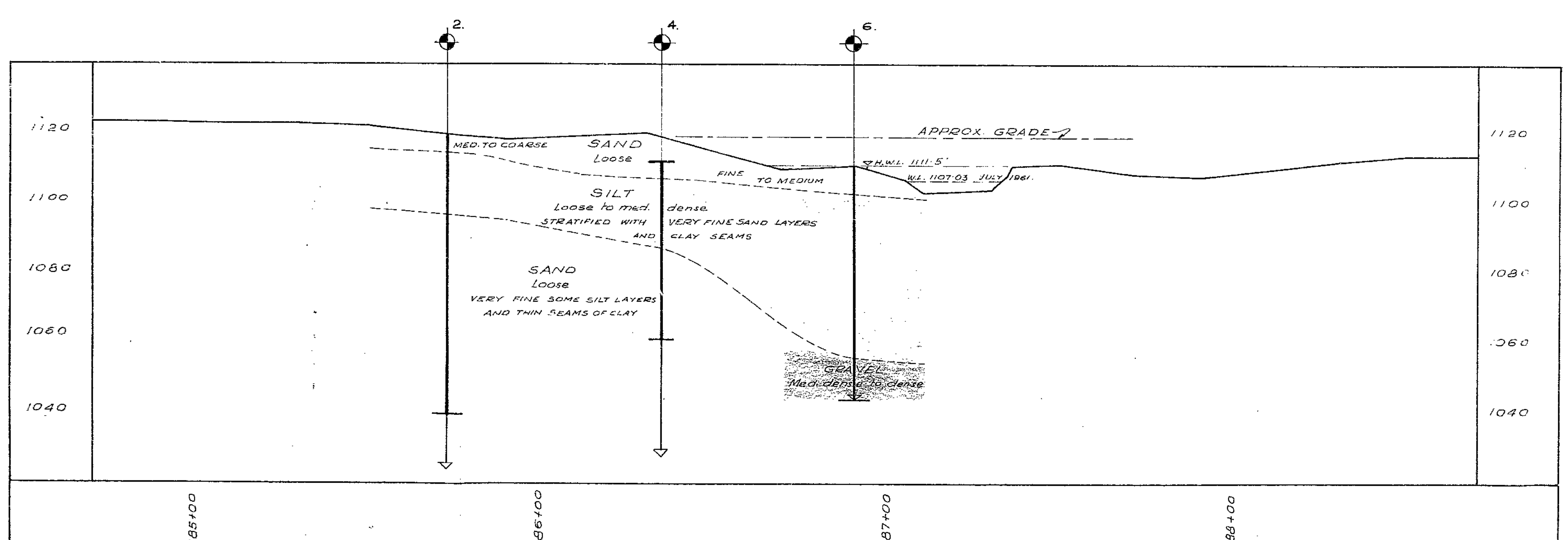
BORE HOLE & PENETRATION

PENETRATION TEST
R = REFUSAL NR = NO REFUSAL

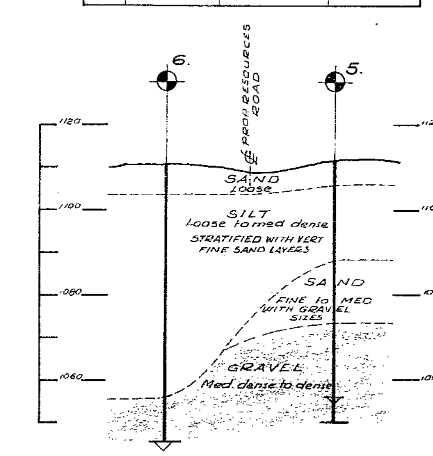
END OF BORE HOLE

END OF PENETRATION TEST

HOLE Nº	ELEVATION	STATION	DISTANCE FROM ℄
1	1118·5	185 + 89	18' LT.
2	1119·5	185 + 73	18' RT.
3	1118·9	186 + 46	18' LT
4	1112 4	186 + 34	11' RT
5	1110·7	187 + 05	18' LT
6	1110 4	186 + 89	18' RT



PROFILE LINE 'C' AND SECTION A-A
SCALE 1" = 20'



SECTION B-B
SCALE 1"=20'

W. A. TROW & ASSOC. LTD.

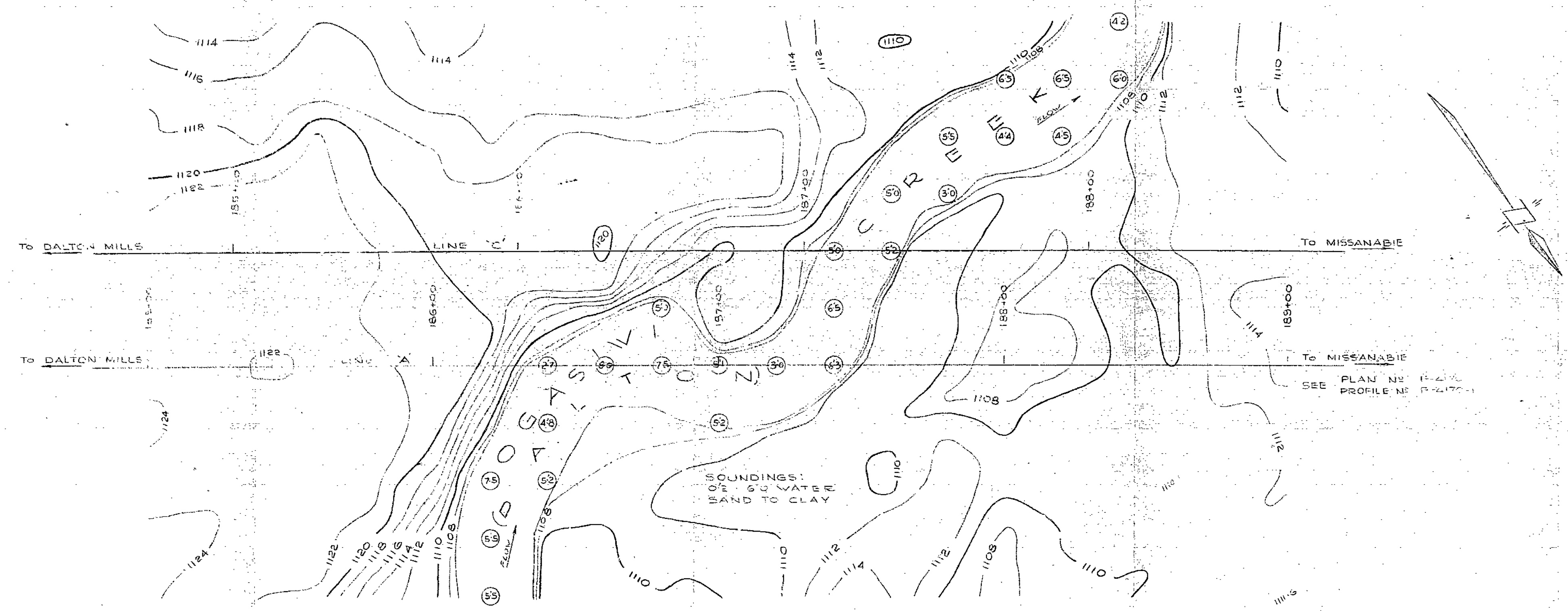
FOUNDATION INVESTIGATION

PROPOSED RESOURCES ROAD AND OGASIWI (DALTON) CREEK CROSSING

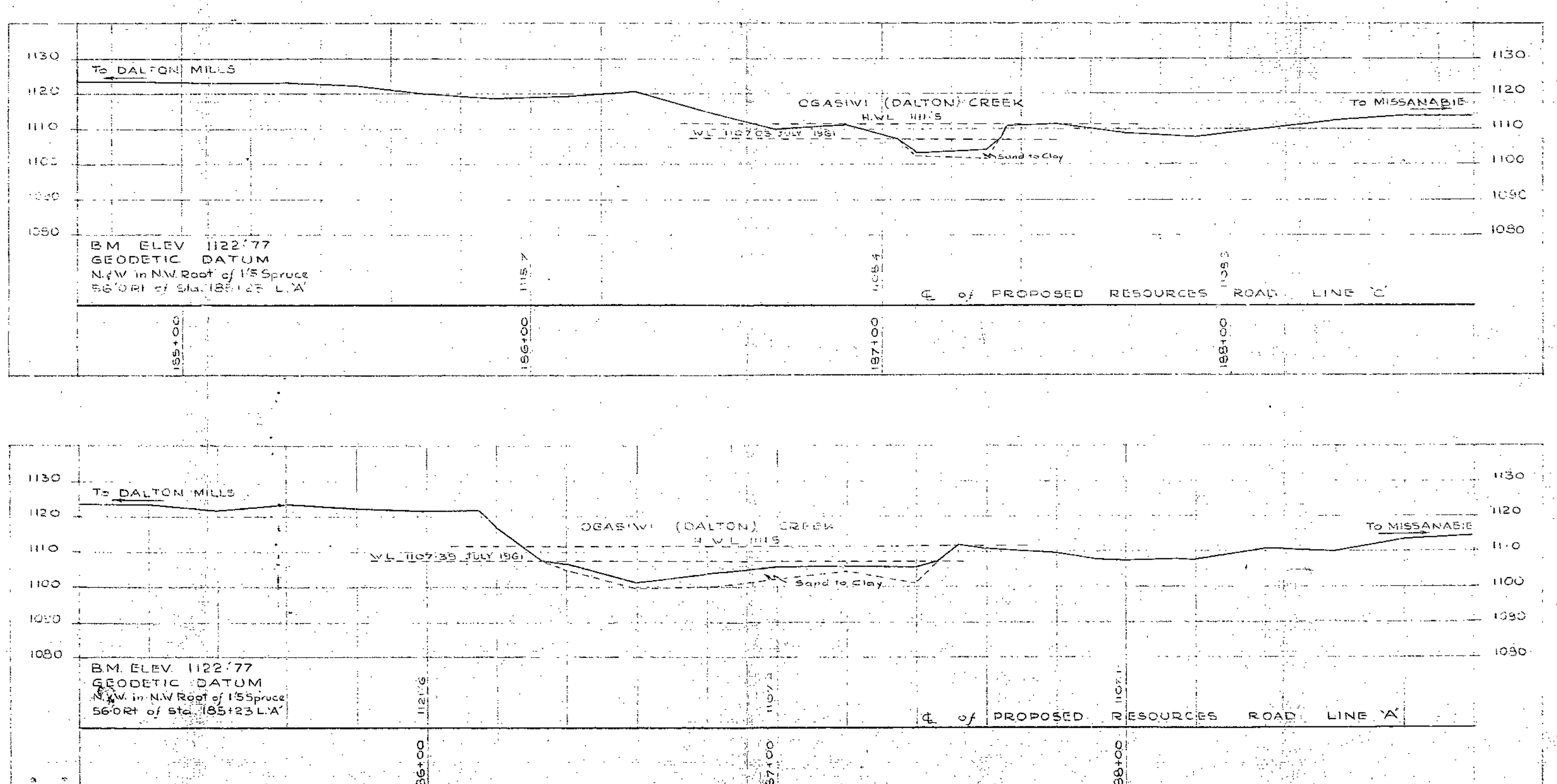
PROJECT Nº J 862.	W.P. Nº 231-61	DATE JUNE 1962.	DWG. 1.

E-4032-1

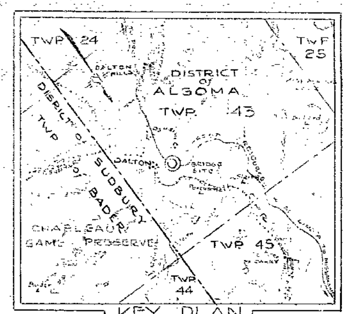
DISTRICT of ALGOMA
TOWNSHIP 43



PLAN
SCALE - 1 IN = 20 FT.



PROFILES
SCALES - HOR. / VER. - 1 IN = 20 FT.



KEY PLAN
SCALE - 1 IN = 2 MI

DALTON
G.B.M. No 2462 ELEV. 1123.929
Plate girder bridge on C.P.Ry., 1 mile southeast of station and at mileage 42.95 from Chapleau. Tablet in top of first concrete pier from southeast end of bridge, 7 feet from northeast or upstream end of pier.

WP No 231-61

DEPARTMENT OF HIGHWAYS - ONTARIO
PLANNING & DESIGN BRANCH
DISTRICT No 18

PROPOSED CROSSING
AT
OGASIWI (DALTON) CREEK
AND
PROP. RESOURCES ROAD LINES 'A' & 'C'
APPROX. 0.8 MI. N. WEST OF DALTON
TOWNSHIP 43 DISTRICT OF ALGOMA

BRIDGE SITE

SURVEY BY: CHIEF OF PARTY: G. REID SUPERVISOR: R. MADILL	APPROVED: John Walter, Director of Planning & Design
DRAWN BY DRAFTSMAN: R. TABASHNIUK & B. STUKALO SUPERVISOR: A. E. KAY	SCALE - AS SHOWN DATE OF SURVEY: JULY 1961 DATE OF PLAN: SEPT. 1961
CHECKED BY. DRAFTSMAN: A. AUGULIS SUPERVISOR: H. PLEASANCE	PLAN PLAN - E-4035-1

E-4032-1

Materials and Research Division

April 13, 1962

William A. Trow,
1350 Jane Street,
Weston, Ontario.

Attention: Mr. Wm. A. Trow.

Re: W.P. 231-61, Resources Road,
Ogasiwi (Dalton) W., 0.9 Miles
N.W. of Dalton, Sault Ste. Marie,
District #18.

Dear Sir:-

Please consider this your authority to carry out a foundation investigation at the above site. Plans and profiles were provided to your representative on April 6, 1962.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Ten copies, only, of the completed foundation report should be submitted to the Foundation Section as soon as possible. Previous requirements as to preliminary borehole information, and laboratory testing program, should be followed.

Charges for the work performed will be in accordance with your Schedule of Rates, dated May 24, 1959, and invoice to be addressed to the attention of the undersigned.

You will arrange directly with Mr. Hugh McArthur of our Regional Design Office in North Bay, for any surveying required at this site. Mr. McArthur may be contacted at: [illegible], North Bay.

Note:- As North Bay is the nearest recognized mobilization point, payment for mobilization will be from there, as discussed with your representative.

[illegible]/Mc27

Yours very truly,

A. Rutka,
MATERIALS & RESEARCH ENGINEER

[illegible] Messrs. [illegible] McCombie
G. E. Hunter
D. F. Collins
[illegible] R. Saint
N. D. Smith (2)
Foundations Office
Mrs.T. Tate
Gen. Files (2)

cc: Foundations Office (Rm. 110)

Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Division,

(Foundation Section)

Attention: Mr. [illegible]

July 11, 1962.

FOUNDATION INVESTIGATION REPORT

By: Wm. A. Trow & Assoc., Ltd.

Re: [illegible]gasiwi Creek Crossing,
Resources Road, Dalton, Ontario,
District No. 18, W.P. 231-61.

Attached, we are forwarding to you, the above-mentioned report submitted by the Consultant, W. A. Trow & Associates. We have reviewed the report and found the factual information well presented. We also agree with recommendations contained in the report and believe that they will be adequate for your future work. The Consultant has pointed out the discrepancies he found when determining the density of the sandy subsoil and is therefore suggesting that a pile load test be carried out to verify his assumptions and findings. We would, on our part, also strongly recommend that this be done because such valuable information would be acquired for future work.

Should there be any additional questions or problems that you would like to discuss, please feel free to call on our Office.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

[illegible]
Attach.

cc: Messrs. A. M. Toye (2)
E. A. Tregaskes
H. D. McMillan
K. McArthur
F. P. Collins
[illegible] Saint
T. J. Kovich
J. Roy
J. W. Gruspier
F. Norman
A. Watt
Foundations Office
Cen. Files.

62-F-244C

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

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

1850 JANE ST.,
WESTON, ONT.
CH. 1-4644

Project: J862

July 3, 1962

Mr. A. Rutka,
Materials and Research Engineer,
Department of Highways of Ontario,
Parliament Buildings,
Toronto, Ontario

Attention: Mr. A.G. Stermac, P.Eng.

Foundation Investigation
Ogasiwi Creek Crossing
Dalton, Ontario, WP 231-61

Hwy 651

Dear Sirs:

Enclosed herewith is our report on the foundation conditions existing under this development road crossing.

The subsoil here is characteristic of the fine grained deposits comprising the height of land separating the James Bay and the Lake Superior Watersheds. The soil consists almost entirely of silt and very fine sand with only very occasional thin clay partings or seams.

Although not proven, refusal and assumed bedrock lies at depths ranging from 58 to 78 feet below the creek bed. End-bearing piles should encounter refusal at these levels. Alternatively, the bridge can be supported on friction piles floating in the silt and very fine sand deposits. The estimated safe load for a Class B timber pile driven to 30 feet below the creek bed is 20 tons; similarly, a 12 inch O.D. cylindrical steel pile should develop a working load of 50 tons if stopped at 40 feet. Since the soil is essentially granular, the amount of settlement under these loads will be very small and it will occur as load is applied. A pile load test is recommended in order to confirm these estimates.

We hope that the information contained in this report assists you in the design of this creek crossing.

Yours very truly,

W. Trow

William A. Trow, P.Eng.

WAT/gc
Encls.

DEPARTMENT OF HIGHWAYS OF ONTARIO
MATERIALS AND RESEARCH BRANCH
PARLIAMENT BUILDINGS
TORONTO, ONTARIO

FOUNDATION INVESTIGATION
OGASIWI CREEK CROSSING
DALTON, ONTARIO, WP 231-61

Project: J862 July, 1962

William A. Trow and Associates Ltd.

TABLE OF CONTENTS

ENCLOSURES

FOUNDATION INVESTIGATION
OGASIWI CREEK CROSSING
DEVELOPMENT ROAD, DALTON, ONTARIO
WP 231-61

Project and Site

This crossing of the Ogasiwi Creek comprises one portion of the development road between Hwy. 101 and Missanabie. It lies about ½ mile to the west of Dalton, Ontario, a small railway stop some 40 miles northwest of Chapleau.

It is proposed to straighten the course of the creek at this crossing location. The bridge will be located immediately to the south of the existing stream. Foundation conditions permitting, a trestle-type structure is proposed for this site.

The terrain in this area is comparatively flat, although a large outcrop of rock rises, in the form of a steep hill, about 200 feet east of the railway and 600 to 800 feet north of Dalton Station. The bedrock surface must fall sharply below the ground surface, since no rock was encountered in a well dug to 55 feet in Dalton. Except for the highway clearing, the area is covered with dense bush.

The creek flowed quite swiftly at the time of the investigation and it probably was close to its highest level. The depth of the water was in the order of 6 feet. Some of the higher river banks were scoured.

Investigation Procedure

The borings of this investigation were performed using conventional wet sampling methods. The holes were cased with BX pipe to the maximum sampled depth of 80 feet. Because of the extremely high friction resistance on the casing, this was the maximum sampling depth obtainable with the light drill equipment used on this project. The decision to use a light drill was dictated by the necessity to fly to the various bridge sites in this area and the need to make borings from a raft.

A total of five sampled borings and six cone penetration tests were made at this bridge site. Each cone was driven at distances ranging from 4 to 10 feet from the adjacent hole and this test was performed first before the boring was made. Because the subsoil was found to be quite uniform, a boring was not made at the cone 1 test location.

Samples were recovered at 5 foot intervals of depth, or at greater intervals when it became obvious that no significant variation existed. They were obtained in the partially disturbed state using the conventional 2 inch O.D. split spoon which was driven into the ground under an energy of 350 ft.lbs. per blow. Attempts were made to recover shelby tube samples of the soil, but only one undisturbed specimen was recovered. In some instances the split spoon samples were lost. When this occurred, the soil was retrieved using a side sampler.

In an effort to establish refusal and probable bedrock levels, the conditions below the sampling limit were determined either by driving a cone or by jetting. These refusal levels were compared with the maximum penetration depths in the adjacent cone penetration tests.

The locations of the borings and the reference bench mark are indicated on Dwg. 1.

Subsoil

Descriptions of the soil underlying this site are given on the borehole logs, Dwgs. 3 to 7 of this report and in the estimated stratigraphical profile of Dwg. 1. It is seen that the predominant soil types are stratified very fine sand or silt. Typical gradings of these soils are presented in Dwg. 8; the finer grading of some of the samples probably reflects the presences of a thin layer of clay in the sample. These materials contain some very thin layers or partings of clay.

In the vicinity of the river bed the silt is extremely fine and the clay partings are slightly organic. At greater depths, however, the individual grains of the silt can be seen. Upon inspection under a microscope, they are found to have an oval or cube-like shape with rounded corners.

At greater depths in the area of holes 5 and 6, the soil becomes much coarser and is essentially gravel in hole 5 below approximate elevation 1073 feet. Refusal and assumed bedrock was encountered at depths ranging from El 1045 to El 1025 feet approximately. As indicated in the previous section, the drilling machine did not have sufficient power to penetrate to bedrock or to withdraw the casing after it had been driven to rock. Consequently, the refusal levels were established by cone penetration tests.

According to penetration resistance measurements recorded during sampling, the soil under this site exists in a loose to medium dense state only. In view of the high friction resistance exerted on the casing and the cone drill rods, however, it is felt that these empirical indications of relative density are misleading and that the sand and silt actually are reasonably dense. In order to obtain an approximate verification of this view, certain laboratory tests were performed on representative samples of these deposits. The results of these tests are presented in Table 1 and Dwg. 9.

An indication of the in-place density was obtained by making measurements of moisture content and specific gravity. The wet and dry density can be computed if it is assumed that the soil is completely saturated. Because the silt and sand had a relatively low permeability, this assumption is considered to be valid.

Selecting conservative values of moisture content and specific gravity the in-place density was computed to be 103 pcf. Maximum and minimum density tests also were made and the relative densities of three samples were determined. These computations show that the relative density of the sand and silt is in the order of 70 percent, which is equivalent to a dense sand having a penetration resistance or N value approaching 40 blows per foot.*

A multi-stage triaxial test was performed on a remoulded specimen of the silt. Unfortunately the placement density was greater than was estimated for field conditions, even though attempts were made to place it in a loose state. The results of this test are shown on Dwg. 9. The angle of internal friction of the soil at this density is shown to be 44 degrees. This value is in agreement with results obtained for the Giles Creek investigation.

Foundations

In view of the apparently loose condition of the subsoil, its low resistance to scour and the high level of the water table, the support of this creek crossing on piles would seem to be the only reasonable foundation scheme.

Two alternatives for a pile foundation are available. One is to use H piles end-bearing on bedrock. The other is to utilize displacement friction piles.

Unfortunately, because of the limited capacity of the drill equipment, it was not possible to confirm the presence of bedrock with certainty. However, the refusal pattern of cone tests, recorded on the borehole plan of Dwg. 1, can reasonably be interpreted as an indication of the bedrock surface. Assuming this to be the case, the permissible loading per pile, driven to this terminal surface, should be equal to its safe structural capacity when considered as a short column. The pile lengths below the creek bed at the west and east end of the bridge will range between about 58 and 78 feet respectively.

The alternative foundation scheme is to use friction piles for the support of the bridge. On the basis of the low penetration resistance measurements, recorded for each boring, this method of support would seem to be a poor alternative to end-bearing piles. However, it

* "Research on Determining the Density of Sands by Spoon Penetration Testing" Gibbs & Holtz - 4th Int. Conference on Soil Mechanics & Foundation Engineering, 1957.

is the opinion of this report that the silt and fine sand underlying this site are in a much denser and more competent condition than is suggested by these empirical dynamic tests.

It is for this reason that the laboratory testing, described in the previous section, was carried out. The conclusions resulting from this testing was that the silt and fine sand exist in a dense state with a relative density in the order of 70 percent. The approximate angle of internal friction of the silt was 44 degrees.

The confirmation of a high relative density for this foundation material is considered to be important, since a soil in this dense state will offer a greater resistance to displacement as a cylindrical displacement pile is driven into it. A greater horizontal force, for the mobilization of friction resistance, therefore, should be generated. The high friction resistance experienced in the withdrawal of the $2\frac{3}{4}$ inch O.D. flush-joint casing and even in the withdrawal of the 1 - 5/8 inch O.D. drill rods, after a cone penetration test, is submitted as an empirical on-site confirmation of this view. The opinion that the horizontal force against the shafts of displacement piles, driven in dense sand, approaches the passive pressure state was expressed in a recent report for another bridge site at Giles Creek, located a few miles to the south of this crossing.* As support of this view, reference was made principally to the results of pull-out tests on step-taper Raymond piles driven into coral sand.+ Some simple static penetration measurements on A drill rods, performed at the Giles crossing, also indicated this high friction resistance.

In the estimation of the capacity of cylindrical piles, driven into the very fine sand of that site, a resultant friction factor equal to unity was assumed. This friction value is much lower than was determined in the pull-out tests on the step-tapered piles; it is also much lower than the value obtained for a soil having an angle of internal friction equal approximately to 44 degrees, exerting an earth pressure approaching the passive state.

In the Giles Creek report, estimates were given of safe bearing capacities of cylindrical piles driven to various depths below creek bed level. These estimates were made using the Terzaghi expression for ultimate bearing capacity.**

$$Q = A(0.3\gamma DN_\gamma + \gamma ZN_q) + PZ(\tfrac{1}{2} K\gamma Z)$$

An explanation of the various terms, together with estimates of the capacities of Class B timber piles and cylindrical steel piles are presented in Table 2. The recommended pile loads were 20 tons for timber piles and 50 tons for 12 inch diameter steel piles; these loads would be developed at approximately 30 and 40 feet below stream bed level. Stream

* Foundation Investigation - Giles Creek Crossing - WP 70-62 Hwy. 101, June 14/62

\+ "Pulling Tests on Piles in Sand" - H.O. Ireland - 4th Int. Conf., Soil Mechanics - 1957

**"Soil Mechanics in Engineering Practice" - Terzaghi & Peck, Pg. 176

bed level was chosen as the reference in these computations in order to provide some compromise value for penetration depth Z. Under the river banks a higher value of Z will apply. However, this could be temporarily reduced if scouring of the river bed took place close to the abutments.

A factor of safety slightly greater than two is incorporated in the recommended pile loads noted above. These estimates have been submitted as a guide in the preliminary design of foundations for this bridge. At least one pile load test should be carried out before the final design decisions are made. It is not anticipated that the piles will encounter refusal when driven to the depths noted.

Since the soil in the river banks has a low resistance to scour, some protection should be provided for the tops of the piles below abutment level. Coarse granular materials seem to be in short supply in the area and therefore perimeter protection in the form of steel sheeting or a timber pile wall probably will be utilized.

There is no embankment stability problem at this site. The granular subsoil will adjust immediately to the fill loadings applied to it.

Recommendations

1) Because of the need for scour protection and the difficulties associated with the installation of footings below the water table in fine sand, it is recommended that the structure be founded on piles.

2) Refusal to end-bearing H piles should be encountered at depths ranging from 58 to 78 feet below creek bed level.

3) Timber friction piles, if stopped at a depth of 30 feet, should develop a safe capacity of 20 tons. Similarily, cylindrical steel piles 12 inches in diameter should develop a safe capacity of 50 tons when driven to 40 feet. There will be no refusal in either case. A pile load test should be performed to verify these estimates.

4) No embankment stability problem exists at this site.

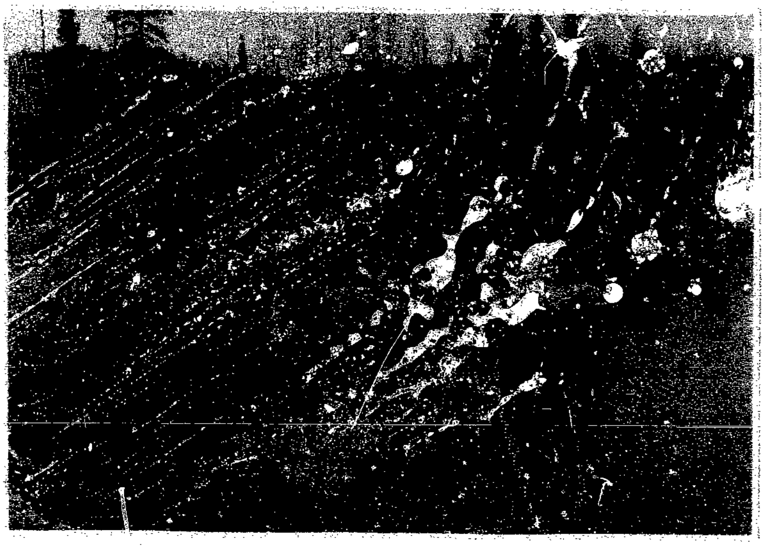
REGISTERED PROFESSIONAL ENGINEER
CIVIL
W. A. TROW
PROVINCE OF ONTARIO

W. Trow

William A. Trow, P.Eng.

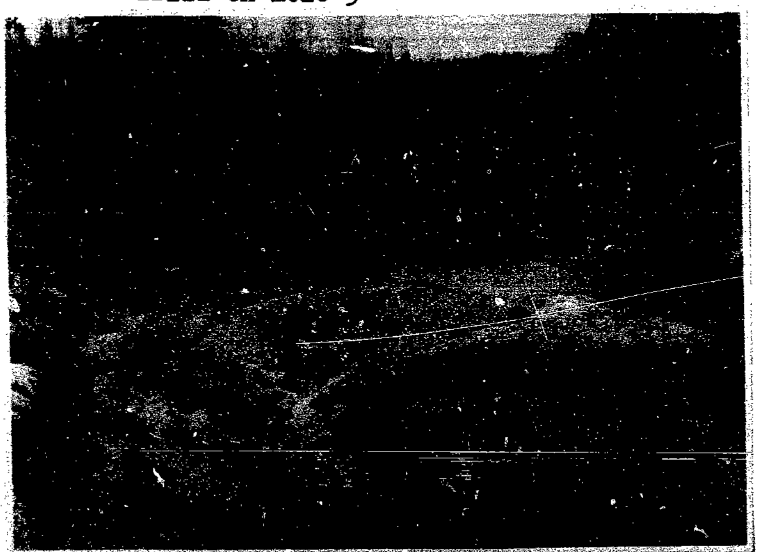
WAT/gc
J862
July, 1962

Looking Downstream
Drill on hole 5



View Looking West
Along the highway
right of way

View from the vicinity
of hole 4
Drill on hole 5



Looking Downstream
Drill on hole 5

View Looking West
Along the highway
right of way

View from the vicinity
of hole 4
Drill on hole 5

WILLIAM A. TROW AND ASSOCIATES LTD.

TABLE 1

SUMMARY OF LABORATORY TEST MEASUREMENTS

(a) Moisture Content Determinations

Depth (ft.)	10	15	20	25	30	35	40	45	50	60	70
Hole 2	17.3		17.2	19.5	18.0	20.6 20.7*	18.1		18.1	18.0 (16.5)	21.6
Hole 4		19.7	16.1	16.4	22.6 (21.8)	19.8	17.5+ 17.7+	18.3	20.5 (19.9)		

Note: Hole 2 - 10 feet sample recovered from centre of sealed Shelby tube. All other tests performed on split spoon samples. Each of these latter samples was placed in a container consisting of two plastic bags. Each bag was tied tightly separately. Moisture contents were performed 1 day after removal from shipping box.

As a check on the rate of loss of moisture, three checks were made on samples that were left in these sealed bags on the inspection table for 1 week. The loss in moisture content, (with ambient temperature about 75 degrees), ranged from 0.6 to 1.5 percent. The moisture contents of these samples are shown in brackets.

* Test on entire bag sample including moisture adhering to walls of bag.

+ Test on sample wrapped in tinfoil and placed with rest of soil in plastic bag.

(b) Specific Gravity Determinations

Hole 2 - 20 feet	S = 2.60 and 2.62	Average = 2.61
Hole 2 - 35 feet	S = 2.64 and 2.64	" = 2.64
Hole 2 - 40 feet	S = 2.59	

(c) Estimation of Unit Weight

Assume S = 2.60

Moisture Content W = 22%

For 100% saturation

$$\gamma D = \frac{62.4S}{W.S + 1} = 103 \text{ pcf} = \text{Dry unit weight}$$

$$\gamma = 103(1+W) = 126 \text{ pcf} = \text{Natural unit weight}$$

Hole 2 - 10 feet (Shelby tube). γ = 129 pcf W = 17.3 γ_D = 110 pcf

TABLE 1

SUMMARY OF LABORATORY TEST MEASUREMENTS

(d) Relative Density Determinations

Depth (ft.)	20	30	50
Hole 2			
Maximum Density γ max. pcf^{+}	119.2(111)	117.2	117.8
Minimum " γ min. pcf^{x}	78.6(83)	78.5(78.5)	78.6
Relative " R* %	69.8	72.2	71.2

$$* \; R = \frac{\gamma max.(\gamma - \gamma min.)}{\gamma(max. - \gamma min.)}$$

Assume γ = 103 pcf

$^{+}$ Vibrated in saturated condition under load for at least 1 hour with drainage permitted.

^{x} Allowed to settle in water in accordance with procedure for hydrometer test. No dispersing agent used.

Values in brackets indicate results of check tests.

TABLE 2

SUMMARY OF ESTIMATED PILE CAPACITIES IN TONS+ - OGASIWI CREEK CROSSING

Depth ** Below Stream Bed	Timber Pile *	Cylindrical Steel Pile ++
30	23.5	33.5
40	39.5	54.5
50	60	80.5

** Assume extra capacity due to weight of fill offset by possibility of scour to greater depths.

* Tip diameter = 8 ins.; average diameter = 10 ins.

++ O.D. = 12 ins.

\+ Estimated from expression:

$$Q = \frac{1}{F}\left\{A(0.3\gamma D N_\gamma + \gamma Z N_q) + \tfrac{1}{2}\gamma Z^2 P K\right\}$$

where: A = area of pile tip in sq. ft.

γ = 63 pcf is the submerged weight of the soil

D = tip diameter of pile in feet

Z = depth of pile below stream bed level

N_γ & N_q are bearing capacity factors - assume = 30

P = average perimeter of pile in feet

K is the friction coefficient - assume = 1*

F=2 is the factor of safety

Note: Recommended loadings - subject to load test

Timber pile = 20 tons at Z = 30 feet

Steel pile = 50 tons at Z = 40 feet

The factor of safety in these recommendations is somewhat greater than 2.

* Values well in excess of 1 determined by Ireland and in investigations for Giles Creek and Little Pine Lake, Hwy. 101.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

DRAWING No. 2

PROJECT No. J862

LEGEND

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE —o—o—o—
- 2" I.D. SHELBY TUBE —x—x—x—
- 2" DIA. CONE ————

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) +S

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

ATTERBERG LIMITS
- LIQUID LIMIT —o
- PLASTIC LIMIT |—

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

BOREHOLE No. Cone 1

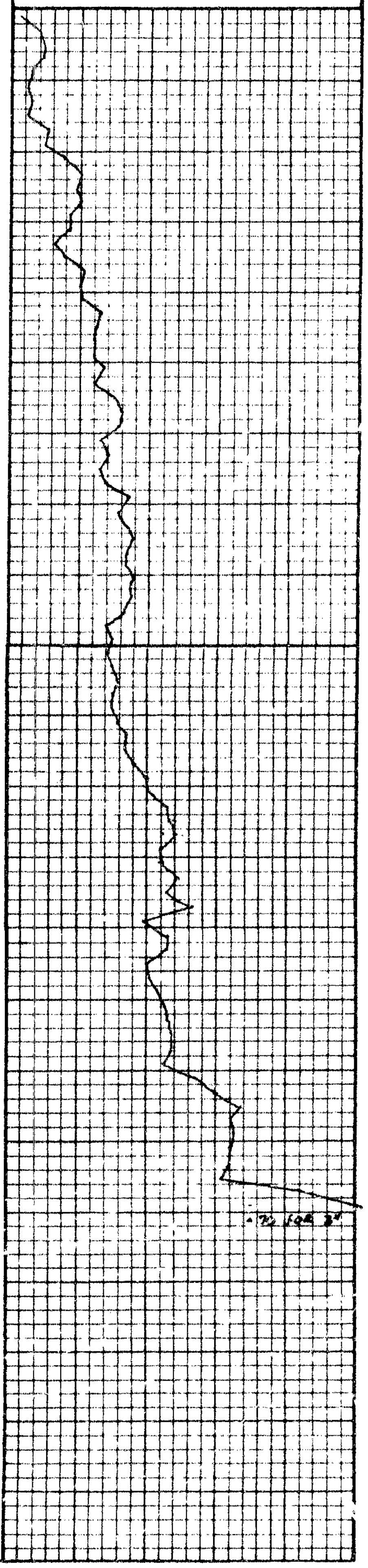
PROJECT Ogasiwi Creek,

LOCATION Dalton, Ontario

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 1118.5 ft.

DATUM See Dwg. 1.

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT 20 40 60 80 / SHEAR STRENGTH P.S.F.	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
	CONE PENETRATION TEST ONLY Refusal to cone = 85.25 feet or El 1033.3 Hammer bounce	1118.5	0 10 20 30 40 50 60 70 80 90 100 110				

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No. 3

PROJECT No. J862

LEGEND

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 2" DIA. CONE

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
- UNCONFINED COMPRESSION
- VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS
- LIQUID LIMIT
- PLASTIC LIMIT

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

BOREHOLE No. 2

PROJECT Ogasiwi Creek Crossing, WP 231-61

LOCATION Dalton, Ontario

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 1119.7 ft.

DATUM See Dwg. 1.

SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
Ground Surface	1119.7	0		
About 6 ins. topsoil, roots, etc. SAND-loose, yellow brown, medium to coarse			1	
	1114.2		2	
SILT-medium dense to loose, brown, becomes grey below about 9 ft., stratified with sandy silt, silty sand and some thin organic clay seams above about 9 ft.; layers of very fine sand increase in thickness below 15 feet.		10	3	
			4, 5, 6	Shelbys Here
	1097	20		
SAND-loose, very fine, grey, silty; some thin dark brown clay seams 1/16 inch to 1/4 inch below 40 feet; also some silt seams; silt predominates below 60 feet; more sand again below 79 feet		30	7, 8, 9	
		40	10	Static Cone Test
		50	11	
		60	12	
-virtual refusal to casing at 79 ft. terminated boring. Drove cone below casing to refusal and bouncing at 93.2 ft. or El 1026.5 ft.		70	13	
End of Hole	1039.2	80	14	Side Sampler

PENETRATION RESISTANCE 350 FT. LB BLOWS/FT — SHEAR STRENGTH P.S.F.

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

Notes:
1) Hole cased with BX pipe to full depth.
2) Pipe kept full of water during withdrawal of wash rods; rods withdrawn slowly to avoid disturbance to soil.
3) Cone driven 11 feet south of hole to virtual refusal at 94 feet. El 1025.0; no bounce. (Cone El 1119.0 ft.)

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING NO. 4

PROJECT NO. J862

LEGEND

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 2" DIA. CONE

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

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

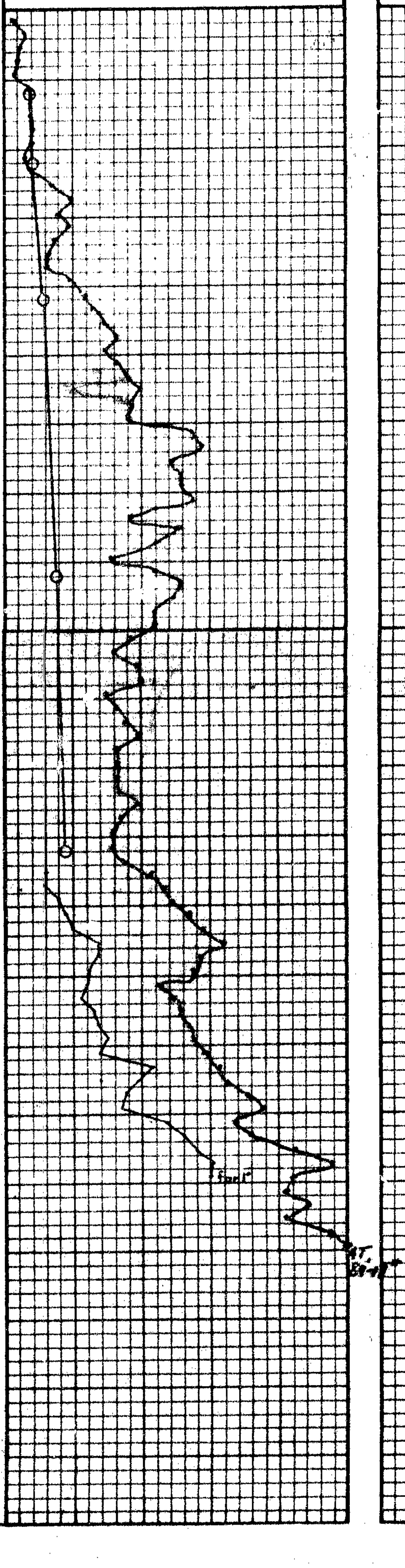
ATTERBERG LIMITS
- LIQUID LIMIT
- PLASTIC LIMIT

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

BOREHOLE No. 3
PROJECT Ogasiwi Creek
LOCATION Dalton, Ontario
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 1118.9 ft.
DATUM See Dwg. 1.

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.
		1118.9	0	
	½ inch topsoil SAND-loose, brown, silty, fine to medium.			1
			10	2
	SILT-loose, grey.	~1104.5		
	SAND-grey, medium, sand fine silty sand and sandy silt layers, and some thin clay seams; hard washing below about 25 feet -pieces of wood and layers coarse sand below 25 feet, some gravel sizes about 27 feet -sand becomes fine below about 30ft. -drove cone below casing to bouncing refusal at 84.1 ft. or El 1034.8 ft.	~1099.1	20	3
			30	
			40	4
			50	
			60	5
	End of Bore	1057.4		
Notes:	1) & 2) As in hole 1. 3) Cone driven 10 feet east of hole; refusal at 89.9 ft. or El 1029.0 ft.		70	
			80	
			90	
			100	
			110	

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. 20 40 60 80
SHEAR STRENGTH P.S.F



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

NATURAL UNIT WEIGHT P.C.F.

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 5
PROJECT No. J862

LEGEND

BOREHOLE No. 4
PROJECT Ogasiwi Creek.
LOCATION Dalton, Ontario
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 1112.4 ft.
DATUM See Dwg. 1.

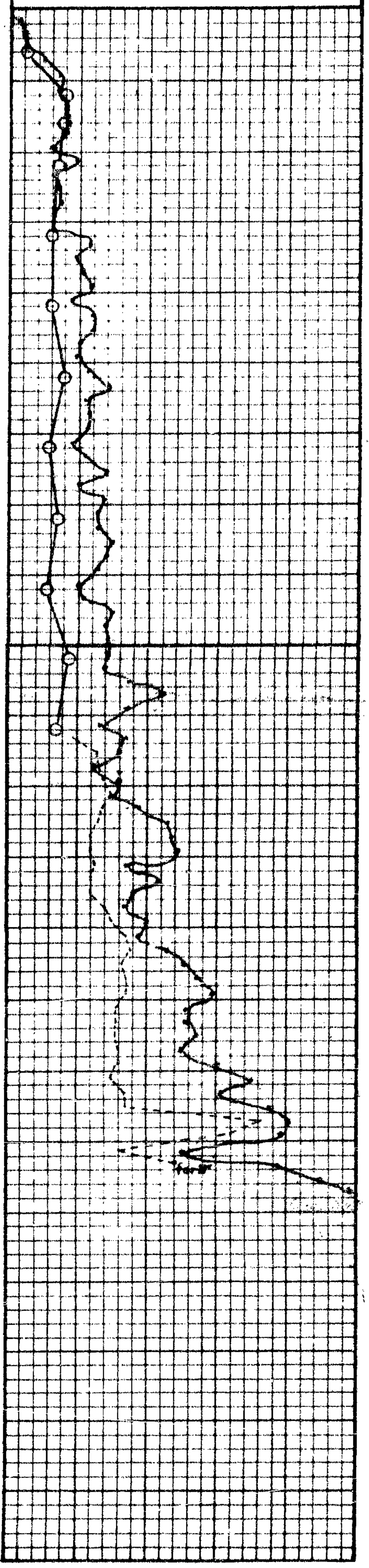
PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA. CONE
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
UNCONFINED COMPRESSION
VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X[LI]
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT
SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
	Bottom of bulldozer excavation	1112.4	0		
	SAND-fine to medium, loose	1110		1	
	SILT-med. dense, brown, stratified, with fine sand seams and some organic clay seams, above about 10 ft.; becomes greyish brown at 6 ft.		10, 20	2, 3, 4, 5, 6	
	SAND-very fine, grey, some silt layers 3 to 6 ins. thick; harder driving casing below 35 ft.; layers of sandy silt with some thin seams of clay more pronounced below 35 ft. -very hard to drive BX casing below 50 ft. - drove cone below 51 feet to refusal & bouncing at 81.9 ft. or El 1030.5 ft. End of Hole	1089.0 1061	30, 40, 50	7, 8, 9, 10, 11, 12	

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT 20 40 60 80 — SHEAR STRENGTH P.S.F.

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT



Notes: 1) & 2) As in hole 2.
3) Cone driven 8 feet east of hole test stopped at 84 ft. or El 1029.4 ft.; not refusal (Cone El 1113.4 ft.)

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

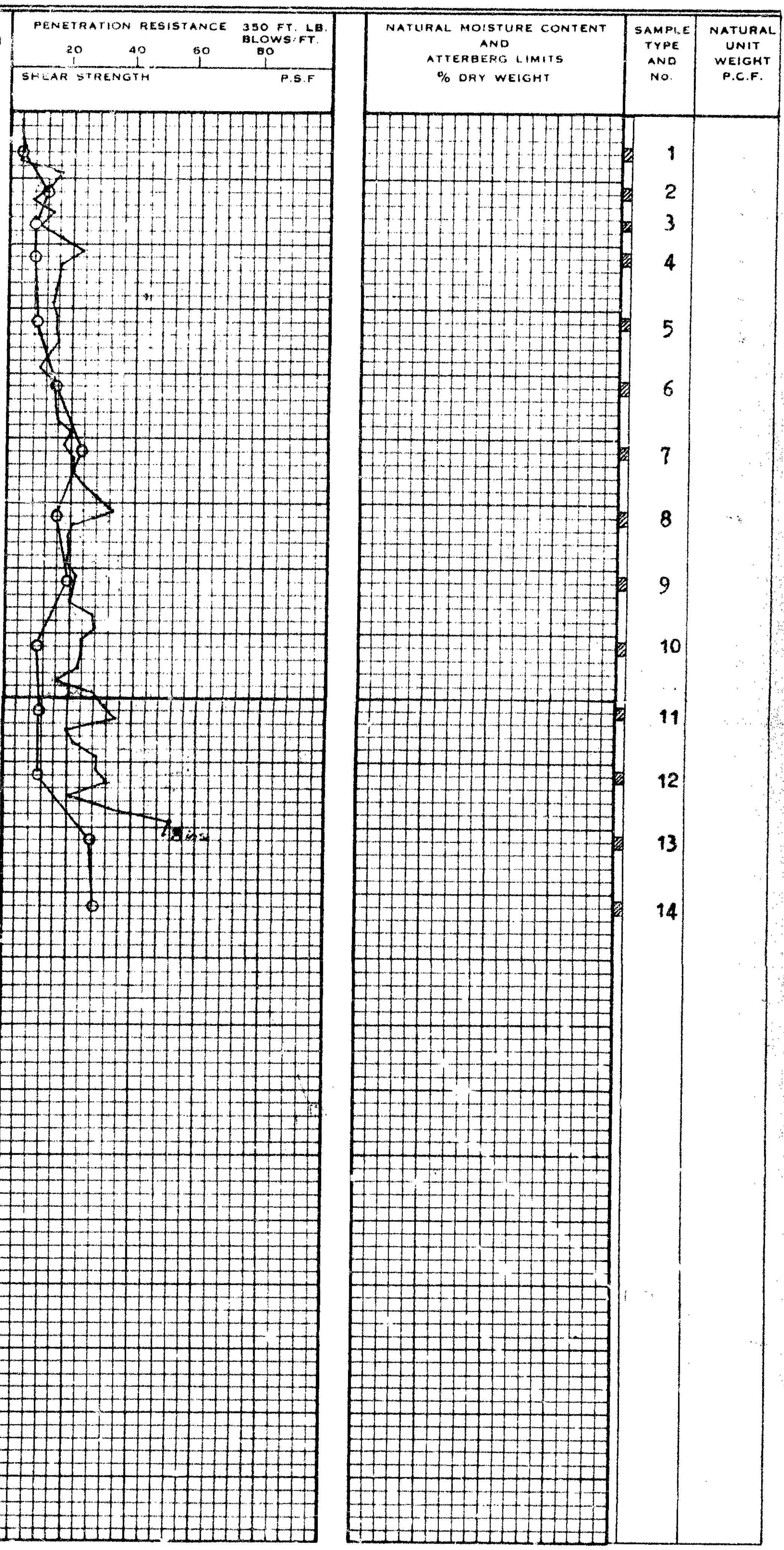
DRAWING No. 6
PROJECT No. J862

LEGEND

PENETRATION RESISTANCE	
2" O.D. SPLIT TUBE	—o—o—
2" I.D. SHELBY TUBE	—x—x—
2" DIA. CONE	———
SHEAR STRENGTH	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	
UNCONFINED COMPRESSION	⊗
VANE TEST AND SENSITIVITY (S)	$+^{S}$
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	X^{LI}
ATTERBERG LIMITS	
LIQUID LIMIT	——o
PLASTIC LIMIT	⊢——
SAMPLE TYPE	
2" O.D. SPLIT TUBE	
2" I.D. SHELBY TUBE	
3" O.D. SHELBY TUBE	

BOREHOLE No. 5
PROJECT Ogasiwi Creek,
LOCATION Dalton, Ontario
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 1110.7 ft.
DATUM See Dwg. 1.

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
		1110.7	0		
	SAND-fine to med. brown, wet below approx. 1 ft.; topsoil, tree roots in first 2 ft.			1	
	SILT-loose saturated, cohesive, few very fine clayey silt seams, greyish brown, some sand although less sand with depth, stratified; hard to drive casing.	1105.7	10	2, 3, 4, 5	
	SAND-fine to medium, wet, grey with some fine gravel sizes -gravel sizes increase with depth.	1087.7	20, 30	6, 7, 8, 9	
	GRAVEL-fine, clean, med. dense, some med. to coarse sand; lost wash water; essential refusal to casing at 60 ft. because of side friction. Attempted to wash ahead below 50 ft. but hole continuously caved.	1072.7	40, 50, 60	10, 11, 12, 13, 14	
	End of Sampling	1049.2			



Depth scale: 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110

Notes:

1) & 2) As in hole 2.

3) Cone test stopped at 55.7 ft. at final resistance of 50 blows for 8 ins. penetration. (Cone El 1111.3 ft.)

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 7

PROJECT No. J862

LEGEND

PENETRATION RESISTANCE	
2" O.D. SPLIT TUBE	—o—o—o—
2" I.D. SHELBY TUBE	—*—*—*—
2" DIA. CONE	—— ——
SHEAR STRENGTH	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	◡
UNCONFINED COMPRESSION	⊗
VANE TEST AND SENSITIVITY (S)	+S
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	X LI
ATTERBERG LIMITS	
LIQUID LIMIT	—o
PLASTIC LIMIT	⊢—
SAMPLE TYPE	
2" O.D. SPLIT TUBE	
2" I.D. SHELBY TUBE	
3" O.D. SHELBY TUBE	

BOREHOLE No. 6

PROJECT Ogasiwi Creek,

LOCATION Dalton, Ontario

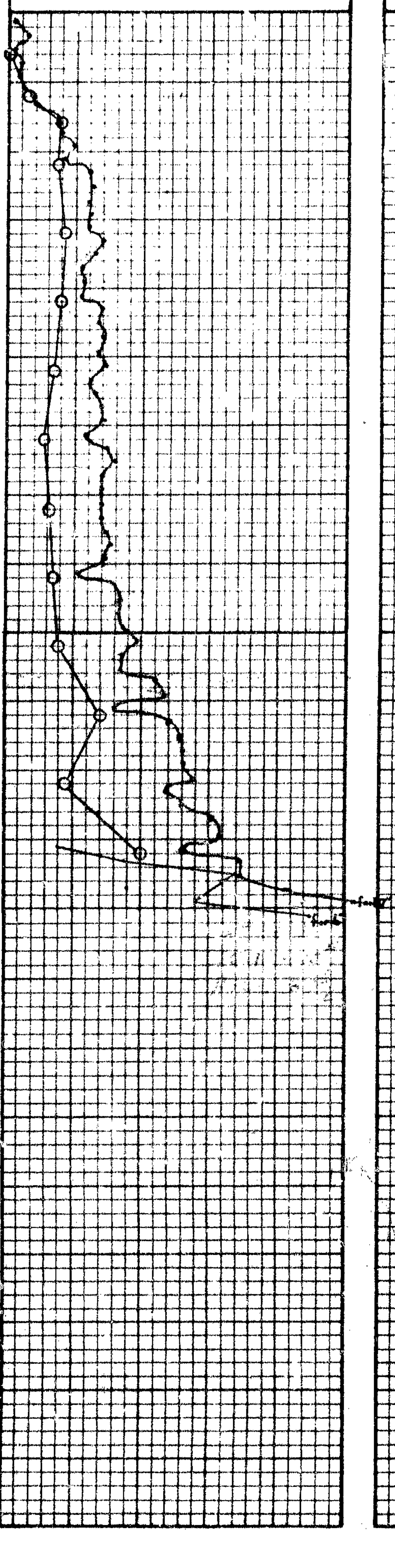
HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 1110.4 ft.

DATUM See Dwg. 1.

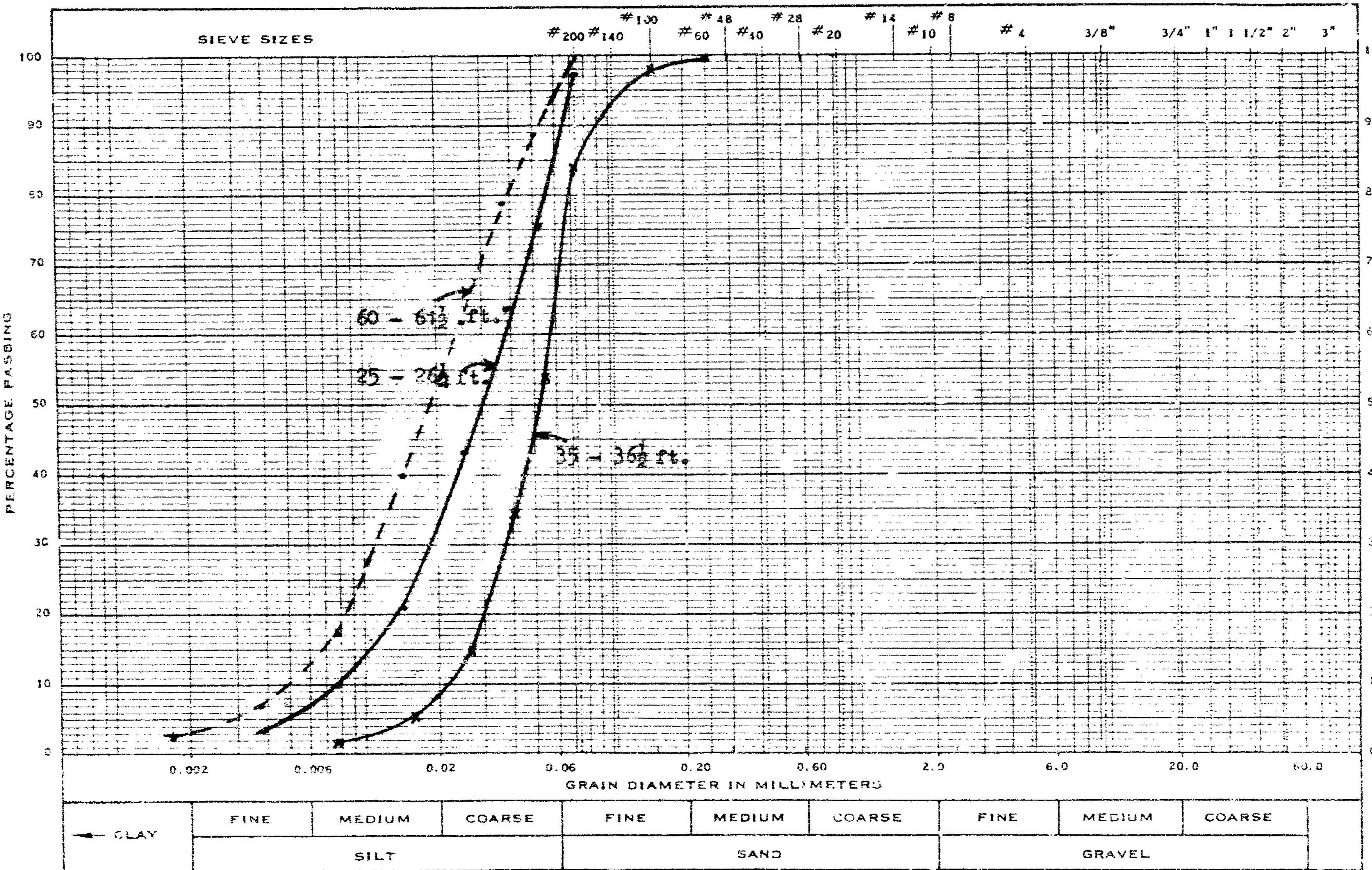
SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
		1110.4	0		
	SAND-loose, brown, fine to medium with some peat and pieces of wood.			1, 2	
	SILT-loose to medium dense greyish brown, stratified with very fine sand layers, some pieces of wood near 7 ft.; very occasional thin clayey silt seams; sand more prominent with depth.	1103	10, 20, 30, 40	3, 4, 5, 6, 7, 8, 9, 10	
	-casing hard to drive below 45 ft.		50	11, 12	
	GRAVEL-med. dense to dense, some sand could not drive casing below 60 ft. Drove cone to bouncing refusal at 65½ ft. or El 1044.9 feet.	1055.4	60	13, 14	
	End of Hole	1045			
Notes:	1) & 2) As in hole 2. 3) Cone driven 7 feet east of hole encountered refusal at 64.8 ft. or El 1045.6 ft.		70, 80, 90, 100, 110		

PENETRATION RESISTANCE 350 FT. LB BLOWS/FT · 20 40 60 80 · SHEAR STRENGTH P.S.F.



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

MECHANICAL ANALYSIS



MODIFIED M.I.T. CLASSIFICATION

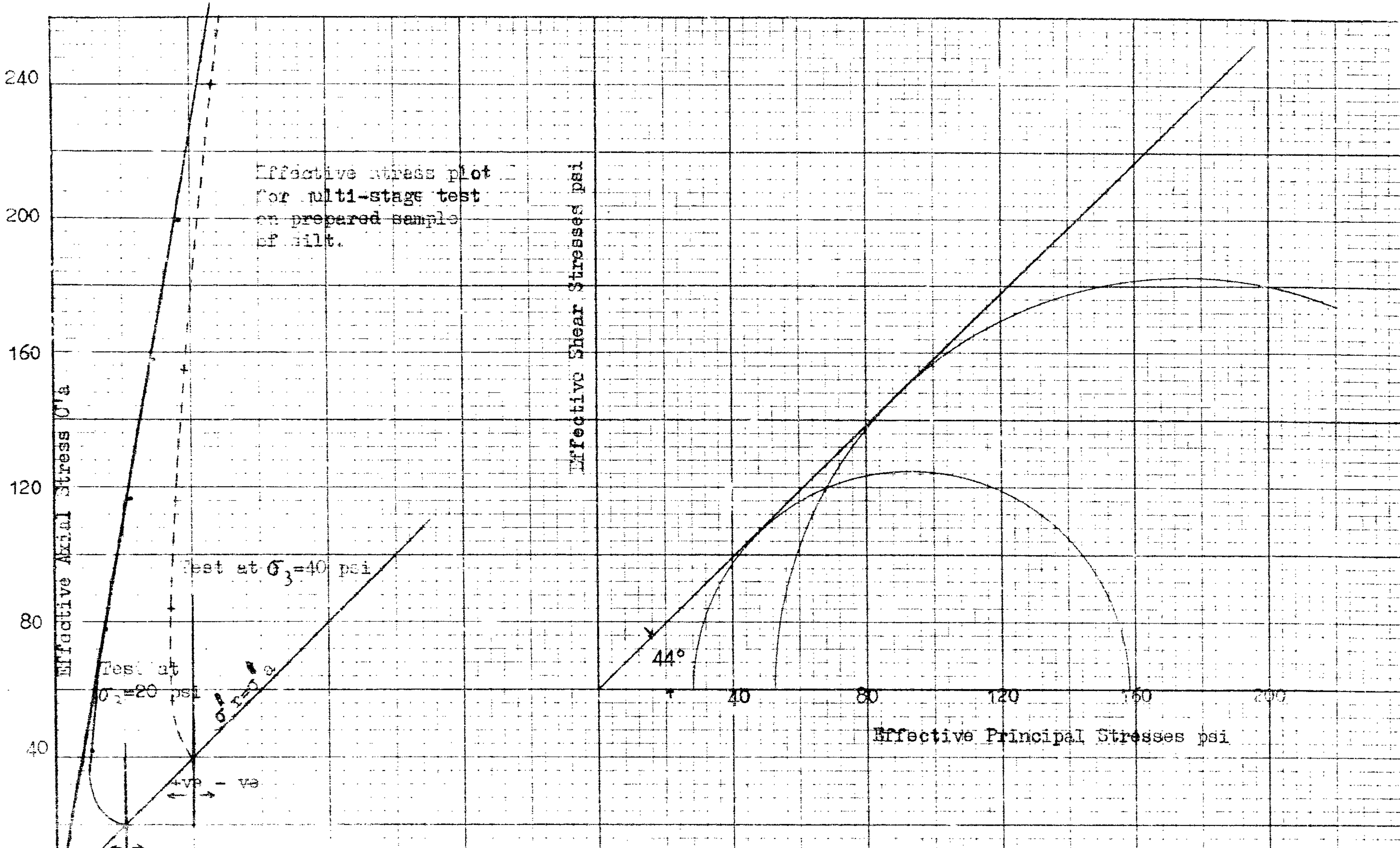
GRAIN SIZE DISTRIBUTION CURVES
HOLE 2 - OGASIWI CREEK

WILLIAM A. TROW AND ASSOCIATES LTD.

J862

Dwg. 8

Project: J862



RESULTS OF MULTI-STAGE TRIAXIAL TEST
TO DETERMINE ANGLE OF INTERNAL FRICTION OF SILT
HOLE 2 – 30 – 31½ FT.
(Prepared Density at start of test = 110 pcf)

Dwg. 9