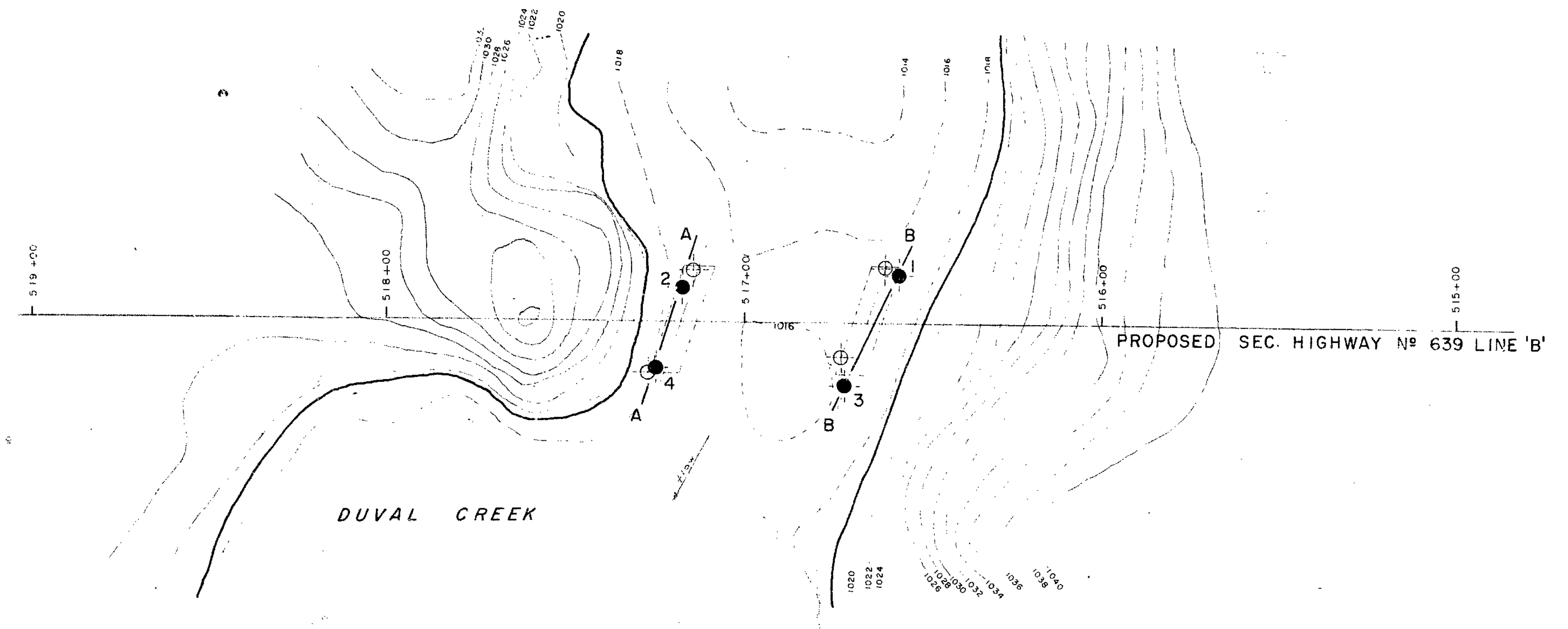
#65-F-258

W.P. # 17-65 &

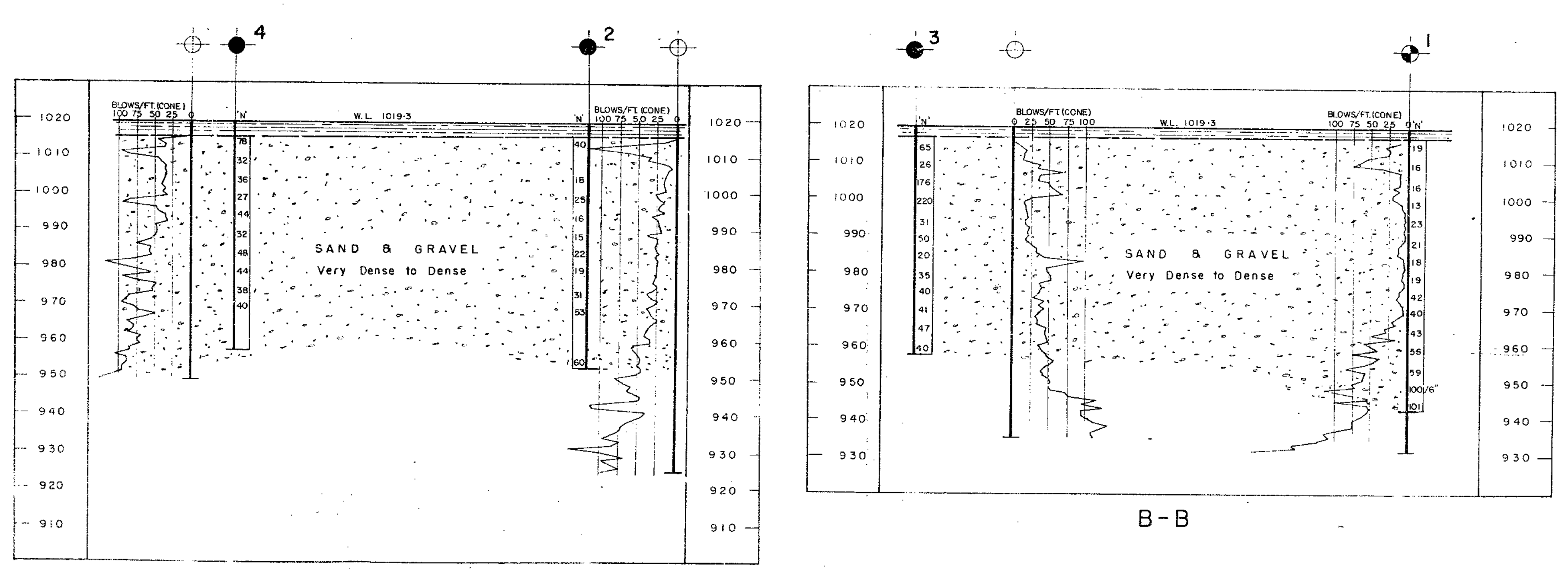
W.P. # 221-64-1

Hwy. # 639

DUVAL CREEK



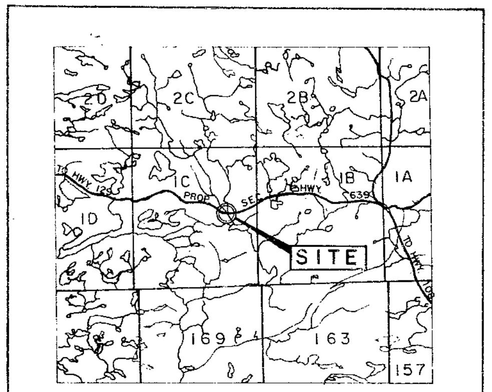
PLAN
20 10 0 SCALE 20 40 FT.



A-A

B-B

SECTIONS
SCALE
5 0 5 10 FT. HORIZ.
20 10 0 20 40 FT. VERT.



DISTRICT OF ALGOMA

KEY PLAN
4 2 0 4 8 12

LEGEND

Bore Hole
Cone Penetration Hole
Bore & Cone Penetration Hole
Water Levels established at time of field investigation.

NO.	ELEVATION	STATION	OFFSET
1	1019·3	516+56	14 R.
2	1019·3	517+17	9·5 R
3	1019·3	516+72	17·5 L
4	1019·3	517+25	13 L

– NOTE –

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS		
DATE	BY	DESCRIPTION

PRINT RECORD

NO.	FOR	DATE

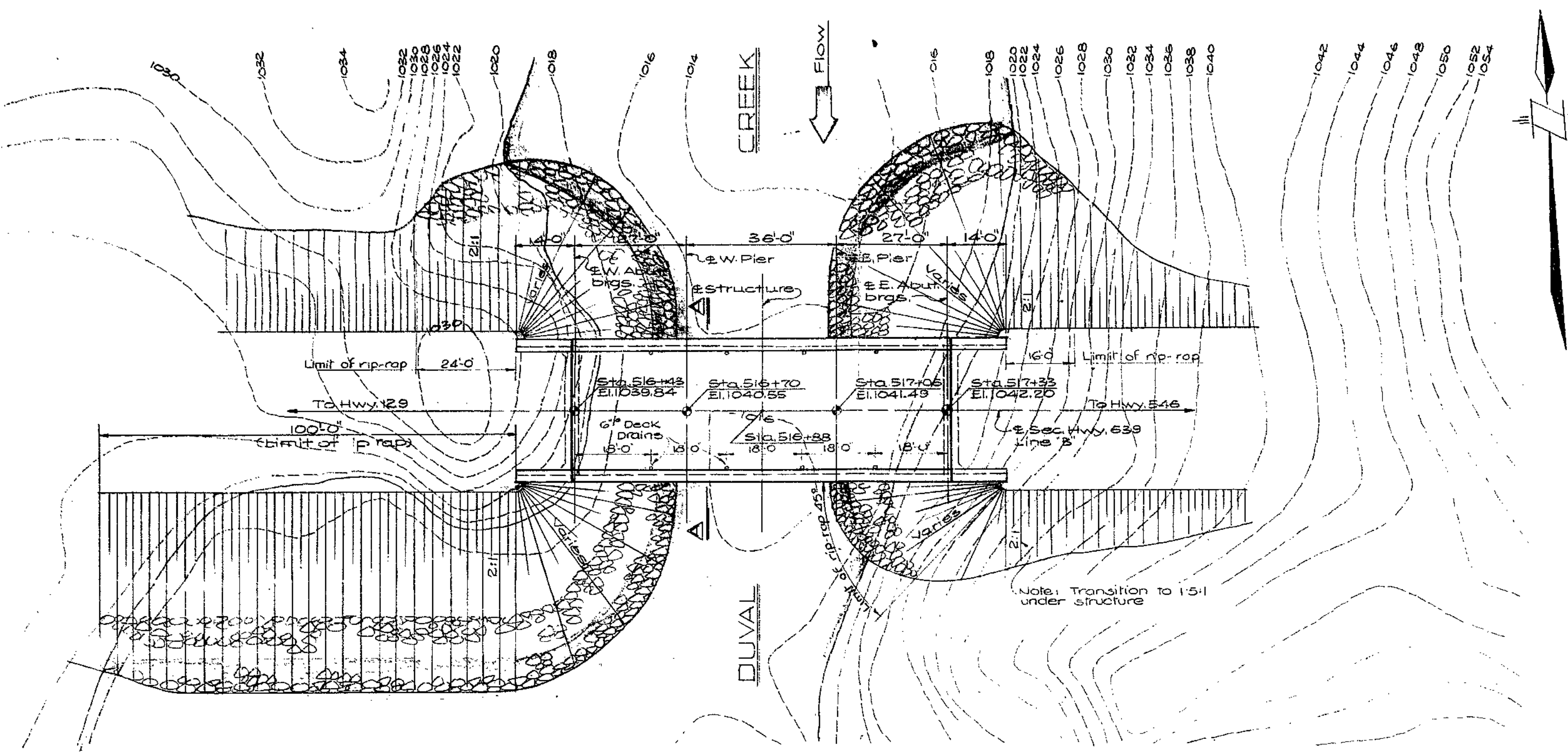
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & RESEARCH DIVISION – FOUNDATION SECTION

DUVAL CREEK

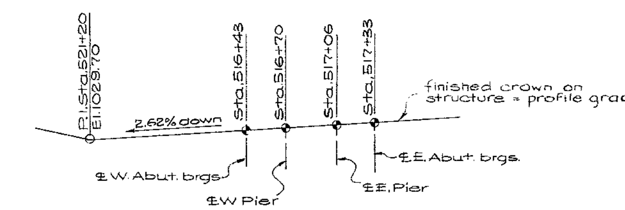
KING'S HIGHWAY NO. 639 LINE B REVISION DIST NO. 18
DIST. ALGOMA
TWP. Nº C LOT CON.

WILLIAM TROW ASSOCIATES LIMITED J-1754

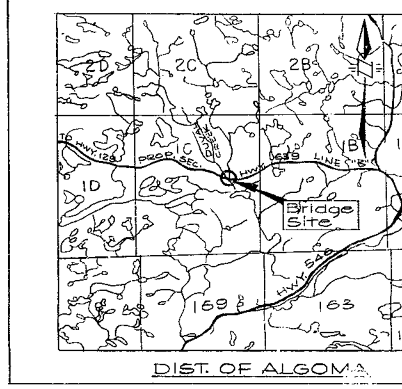
SUBM'D	CHECKED	W.P. NO. 17 - 65	M.&R. DRAWING NO.
DRAWN	CHECKED	JOB NO.	
DATE		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	



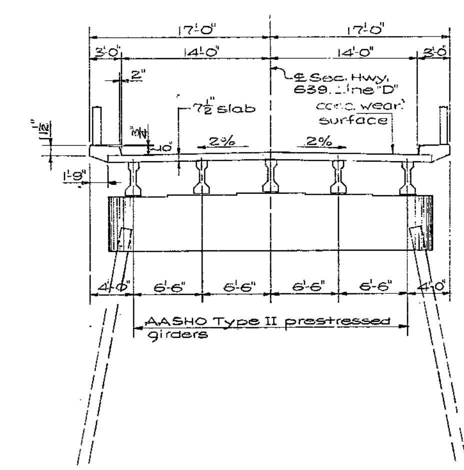
PLAN
1"=20'-0"



PROFILE AT CROWN OF HWY. 639
N.T.S.

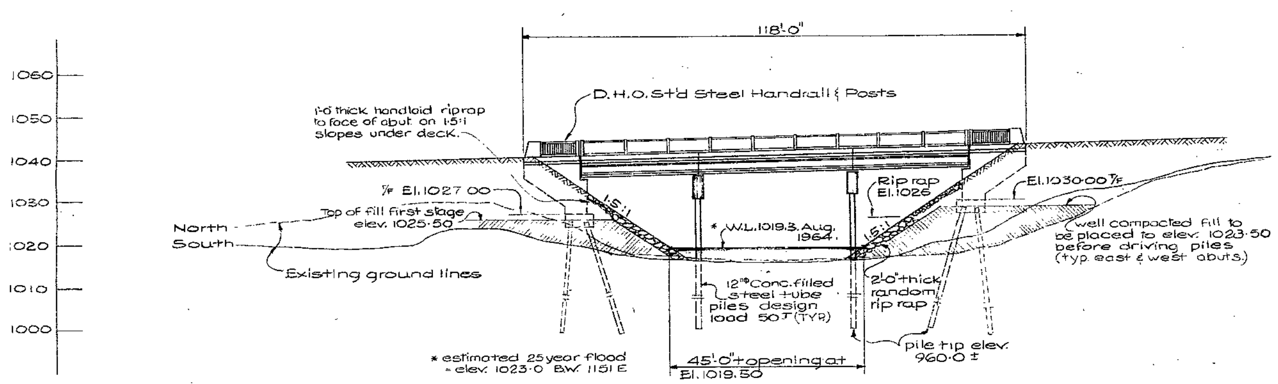


KEY PLAN
1"=4 Mi.



A 1/8"=1'-0"

B.M. El. 1030.71
Geodetic Origin
Nail in Top of 1.0' Cedar Stp 75.0' Lt. of Sta. 520+20



ELEVATION
1"=20'-0"

Reference Plans	
Bridge Site Plan	E-4524-1
Location Plan	B-1140-1
Profile	C-1140-1
Soils	SA-2047

PRINT RECORD		
No.	FOR	DATE

REVISIONS		
DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

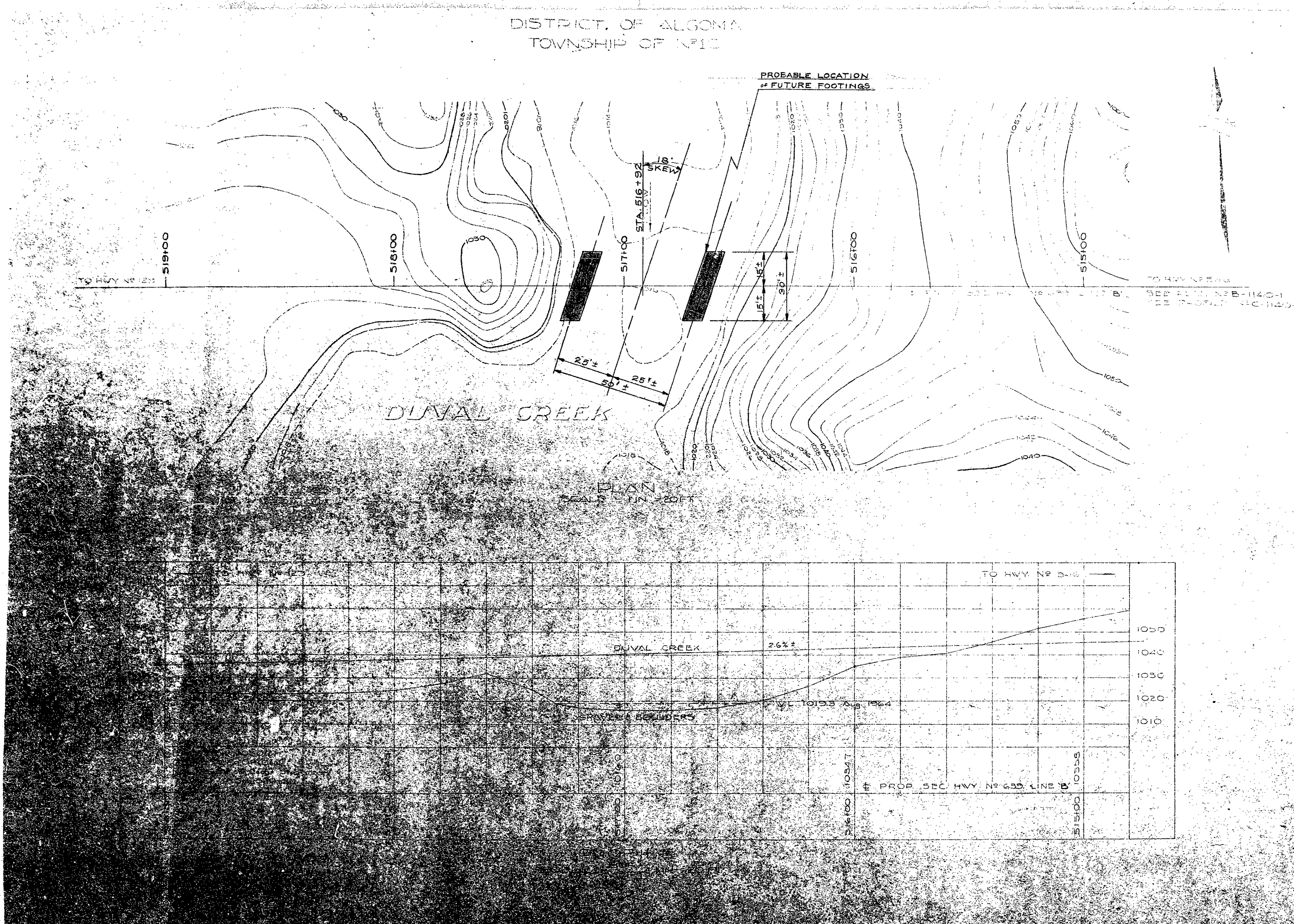
DUVAL CREEK BRIDGE
7.5 Miles West of Sec. Hwy. 546

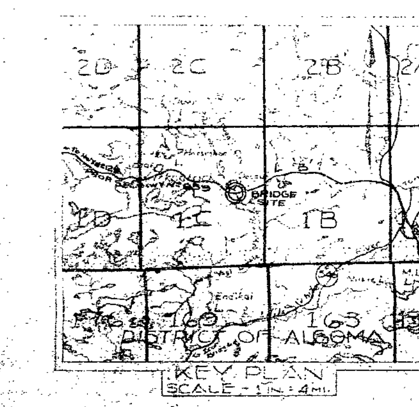
SEC. ~~KING'S~~ HIGHWAY No. 639 DIST. No. 18
[illegible]. Dist. of Algoma
TWP. No. 1C LOT CON.

PRELIMINARY

APPROVED	BRIDGE ENGINEER			SITE No. 285-276	W.P. No. 17-65
				CONTRACT Nos.	
DESIGN	J.M.	CHECK			
DRAWING	D.C.	CHECK	J.L.K.	DRAWING No.	D-5717-P1
DATE	OCT. 1965	LOADING	H20,S16		

SOME DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENTS





W.P. Nº 221-64-1

DATE	REVISIONS & ADDITIONS	BY	CH'KD

DEPARTMENT OF HIGHWAYS ONTARIO
DESIGN BRANCH
ENGINEERING SURVEYS DIVISION

BRIDGE SITE

PROPOSED CROSSING AT DUVAL CREEK AND PROP. SEC. HWY. Nº 639 REV. LINE 'B'

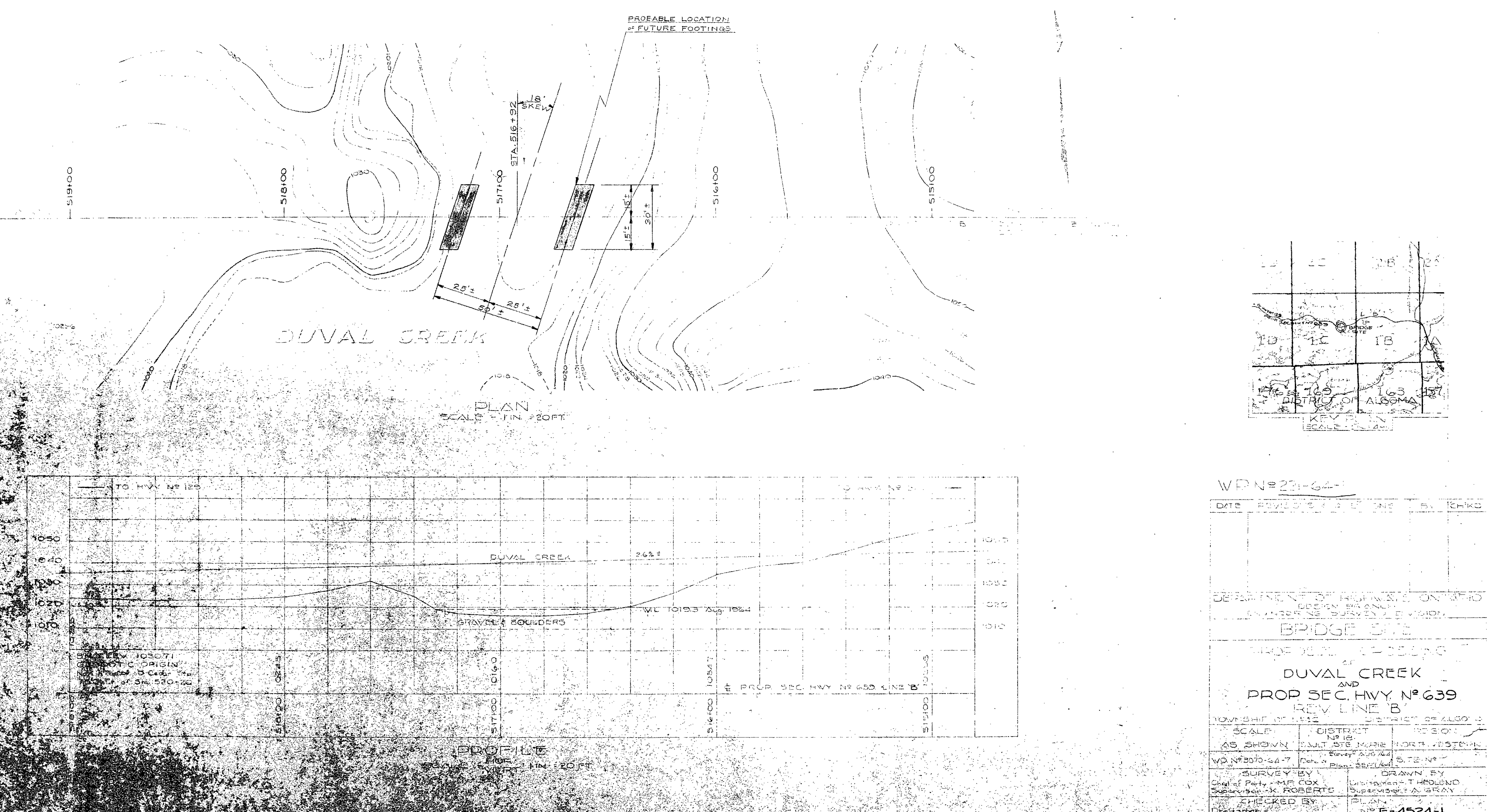
TOWNSHIP OF Nº 10 DISTRICT OF ALGOMA

SCALE	DISTRICT Nº 18	REGION
AS SHOWN	SAULT STE. MARIE	NORTHWESTERN
W.P. Nº 3070-64-7	Date: Survey AUG/64 Plans SEPT/64	SITE Nº

SURVEY BY	DRAWN BY
Chief of Party - M.J. COX Supervisor - K. ROBERTS	Draftsman - T. HEDLUND Supervisor - A. GRAY
CHECKED BY Draftsman - Supervisor -	PLAN Nº E-4524-1

SOME DEFECTS IN NEGATIVE DUE

TO CONDITION OF ORIGINAL DOCUMENTS



90 Milvan Drive
Weston, Ontario
749-1290

William Trow

WP-17-65

Project: J1754

Soil Mechanics
Consultants
W. A. Trow
MSc. MEIC. P. Eng.
K. Peaker
PhD. MEIC. P. Eng.
D. H. Shields
PhD. MEIC. P. Eng.

Associates Ltd.

March 31, 1965

Mr. A. Rutka, P.Eng.,
Chief Materials & Research Engineer,
Materials and Research Section,
Department of Highways of Ontario,
Parliament Buildings,
Toronto, Ontario.

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

Foundation Investigation
Proposed Crossing
Duval Creek
Highway No. 639
W.P. 17-65

Dear Sirs:

In conformance with your authorization of December 17th, 1964, a foundation investigation has been completed at the above mentioned site. The field work was carried out during the period of March 5th, to March 11th, inclusive. Our findings and recommendations on this subject are outlined in the following paragraphs.

1) The subsoil at the proposed bridge site consists of sand and gravel which varies in density from very dense to medium dense. The sand and gravel was found to be present to a depth of at least 94 feet.

No artesian conditions were evident at this site.

2) At the time of the investigation the depth of the river was about 4 feet and the complete ice cover indicated a very low rate of flow.

3) It is recommended that the bridge be founded on spread footings placed about 8 feet below the present river bed level or approximate El 1007 feet. On the basis of penetration resistance measurements it is considered that river scour, before the water was dammed downstream, extended almost to this level. The dense granular soil which exists at this site has a safe net bearing value of 2 tsf.

If it is proposed to perch the abutments on the approach embankment fill, piles should be driven into the underlying natural soil. Theoretical computations indicate that adequate capacity for cylindrical piles can be obtained if terminated at approximate El 1000 feet. For timber piles with 8 inch diameter tips, a safe net bearing capacity of 20 tons should be available if driven to this depth.

4) As the sand and gravel is free-draining and well-graded, excavation to footing depth below the water table should not be difficult provided that the working area is drained and that drainage ditches are installed.

5) All organic soil, which will be encountered in the heavily wooded areas east and west of the bridge site, should be removed to expose the underlying granular soil. No embankment stability problem exists in the approaches to the bridge.

The above recommendations and conclusions have resulted from the consideration of the following information.

PROJECT

The proposed bridge over Duval Creek in Northern Ontario is on the proposed section of Highway No. 639, Line 'B'. The subject of this report is the foundation investigation at the abutment locations which have been proposed by the Department of Highways for Ontario.

SITE DESCRIPTION

The proposed bridge will provide a crossing over Duval Creek. The creek flows in a southerly direction from Dougal Lake to Douglas Marsh. The rate of flow could not be obtained because of complete ice cover. This indicates, however, that the rate is very low. At the bridge site the creek was found to be about 60 feet wide and 4 feet deep. The creek bottom is generally sand and gravel.

The creek is contained, at the bridge site, by banks which rise sharply on the east and west. Of greater geological significance, however, are the massive rock outcrops which occur to the west and east of the bridge site,(see photographs). These indicate the presence of a flood plain which is 2 or 3 miles wide.

An ancient logging dam was observed 100 feet south of the bridge site, (see photograph).

FIELD WORK AND SUBSOIL STRATIGRAPHY

The field work consisted of 4 sampled boreholes and 4 cone penetration tests. The locations of the boreholes and accompanying cone penetration holes are shown on the site plan.

The boreholes were advanced using a standard diamond drill and conventional sampling equipment. Samples of the subsoil were obtained with split spoon type samplers which were driven into the soil with an energy conforming with the requirements of the standard penetration test.

The subsoil encountered at the site is shown in detail on the borehole logs (Dwgs.1 to 4 inclusive). The subsoil stratigraphy as interpreted from the logs is shown on the site plan.

All borehole elevations are referenced to a bench mark having geodetic origin. The bench mark is described on the Department of Highways Plan No. E-4524-1, as a nail in the top of a cedar stump located at Station 520 + 20, 75 L, having an elevation of 1030.71.

The subsoil at the proposed bridge site consists of sand and gravel. Borehole data indicates that sand and gravel is present to a depth of at least 94 feet. The soil is generally very dense near the surface. A sudden decrease in density at about El 1009 feet in Holes 1 and 2 suggests that scour has extended down to this level. Scour is further evidenced by the fact that the gravel sizes were much larger and more frequent above this elevation. This would account for the high penetration resistance above El 1009 feet.

Below about 20 feet the sand and gravel becomes dense to medium dense. At about 45 feet a distinct increase in density was noticed.

All boreholes were made through the ice on the creek, therefore, the water levels shown on the borehole logs correspond to the creek level, namely, 1019.3.

No artesian conditions were observed at this site.

FOUNDATIONS

a) Spread Footings - It is recommended that the proposed structure be founded below scour level or at approximate El 1007 feet. From the blows on the split spoon sampler and from the cone penetration test a safe net bearing pressure* of 2 tsf is indicated at this level.

Under this loading the settlement will be well within the tolerable limit of the structure (less than 1 inch). As the soil is granular and free-draining, almost all the settlement will take place as the structure is being built.

b) Pile Foundations - An alternative to spread footings is to perch the abutments on piles above the creek bed but below the ~~maximum~~ MINIMUM expected water level. It is expected that cylindrical piles will encounter adequate bearing in the dense sand and gravel at approximate El 1000 feet. This opinion is based upon the following assumptions and calculations.

The ultimate bearing capacity of a pile can be estimated from the expression:

$$Q = A\gamma' DN + \frac{pk\gamma' D^2}{2}$$

WOODEN PILES

* Gibbs & Holtz - 'Research on Determining the Density of Sand by Spoon Penetration Testing' - 4th Int. Conf. on Soil Mech. and Found. Eng., Vol. 1, 1957.

By assuming an ultimate bearing capacity, Q, and solving the expression for D, the required embedded pile depth can be obtained. The assumptions and definition of terms are given as follows:

Q = the required ultimate pile capacity, assumed to be 60 tons

A = the area of the pile tip, assumed to be 0.35 square feet for an 8 inch diameter pile tip

γ' = the submerged unit weight of the sand and gravel, assumed to be 65 pcf

N = the bearing capacity factor, estimated to be at least 300* for the soil density applicable

k = the resultant friction coefficient on the shaft, estimated to be 1.0

p = the perimeter of the pile, assumed to be 2.6 feet for a timber pile having an average diameter of 10 inches.

D = the required embedded depth of pile below stream bed level which will give an ultimate bearing capacity of 60 tons.

Solving the expression for D gives an embedded depth of about 15 feet. By applying the normal factor of safety of 3, a safe net bearing capacity of 20 tons will be available when the pile is driven through the natural soil to an elevation of about 1000 feet.

If it is decided to perch the abutments on the embankment approaches by driving piles through the fill and into the natural soil, the effect of the weight of unsaturated fill should be considered. D, in this case can be measured from the base of the perched abutment in the fill.

* Meyerhof G.G. - 'Some Research on the Bearing Capacity of Foundations', Canadian Geotechnical Journal, 1963.

If H piles are used, they may well penetrate to a depth greater than 80 feet before refusal is encountered.

FOUNDATION EXCAVATIONS

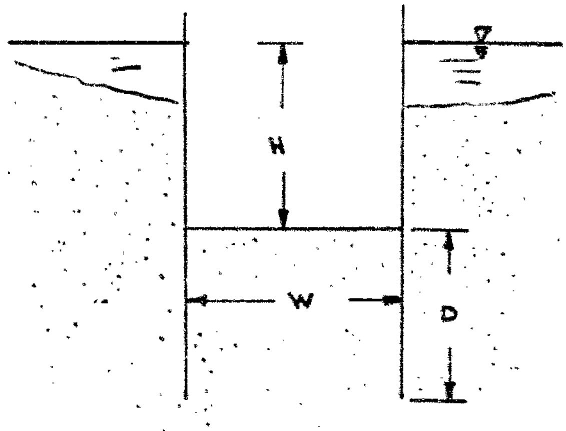
If the footing proposal is used, foundation excavations below the water table in porous sand and gravel will be necessary. Before any attempt is made to dewater the area, the footing excavation including perimeter drainage ditches should be made. The base of the excavation should be crowned, with the footing area being the highest point of the cut and with the ground sloped down beyond the edge of the footing at 3 horizontal to 1 vertical. The material which is removed should be placed on the river side of the excavation to form a dyke. In this manner the creek can be diverted around the excavation. The excavation can then be pumped out gradually. Continuous pumping will be required to keep the water table depressed in the working area.

The foregoing proposal will require an excavation which is much larger than that which would normally be needed for a bridge footing. In addition, because the soil is quite permeable, a considerable inflow of water can be expected. Provided the pumping equipment is adequate, the volume of seepage should gradually reduce with continued pumping. The flow can be minimized by directing the river well clear of the work. Since the sand and gravel is quite well graded, it is expected to remain stable as the dewatering program progresses, provided that the perimeter walls of the excavation are sloped at approximately 2 horizontal to 1 vertical. A sketch of the suggested shape of the excavation and an estimate of seepage flow is given in Dwg. 5.

An alternative to this dewatering method is one incorporating the use of well points and a sheeted excavation. If well points are impractical in this isolated area, however, another alternative would be to drive water tight sheeting well below footing elevation. The depth to which sheeting should be driven to obtain a factor of safety of 2 against piping can be obtained from the following table.**

W/H	D
0.5	1.75 H
1.0	1.4 H
2.0	1.0 H
3.0	0.8 H
4.0	0.75 H
5.0	0.7 H
10.0	0.6 H
20.0	0.52 H

F.S. = 2.0



The sheeting, if left in place, will provide excellent protection against scour.

** McNamee, J., - 'Seepage Into a Sheeted Excavation,' - Geotechnique, Volume 111, 1952 - 53.

SCOUR PROTECTION

Positive measures against possible scour and errosion must be provided. Once footings are placed, the excavation should be backfilled with the natural sand and covered with coarser rock at creek bed level. In addition, rip rap should be placed on the soil in front of the abutment and wing walls and on the adjacent sections of road fill, up to the highest anticipated flood level.

APPROACH EMBANKMENTS

Very little organic material was encountered at the site of the proposed bridge. Approach embankments, however, will extend into heavily wooded areas. In these areas it will be necessary to remove the organic soil to expose the alluvial deposit of sand and gravel.

Once the organic soil is removed, the embankments will rest on the underlying granular soil and stability as well as long term settlement will not be a problem. Typical grading curves are included in this report.

The natural soil at this site is quite satisfactory for use as embankment fill.

EARTH PRESSURES

If abutments and wing walls are used on this project, i.e. the approach fill does not spill through the abutments, they must be designed to withstand the lateral earth pressure exerted by the retained soils. The earth pressure that will act on the walls can be estimated using a value of earth pressure coefficient equal to 0.35. The earth pressure, p, on the walls at any depth, h, can be found from the expression:

$$p = K \{ \gamma (h - h_1) + \gamma_s h_1 + q \}$$

where:

K = 0.35, the recommended earth pressure coefficient assuming the walls to be rigid

γ = 125 pcf, the estimated unit weight of the retained soil

γ_s = 60 pcf, the estimated submerged weight of the retained soil

h_1 = height of water table above the point being considered

q = surcharge, if any, acting at the top of the wall.

The stability of the abutment and wing walls should be checked for horizontal sliding along the footing base. The resistance against the sliding is the frictional force acting along the footing base. The frictional force developed along the footing base can be calculated using a friction coefficient of 0.7 (concrete sliding on granular soils).

If the resisting force is less than 1½ times the estimated sliding force, the footing base can be extended under the fill to increase the weight of backfill carried by it. In this manner, the resistance to sliding can be increased.

Should any queries arise concerning the contents of this report we will be pleased to discuss them with you.

Yours very truly,

H.R. Krzywicki

HRK/ss
Encls.

H.R. Krzywicki, M.Eng.

K. Peaker

K. Peaker, P.Eng.

View of bridge site looking North

View from bridge site looking East

J1754
(b)

View from bridge site looking West.
Large rock outcrop can be seen in the
background.

J1754
(5)

Aerial photograph of bridge site. Large rock outcrop to the Northeast of bridge site can be seen in the background. A portion of Douglas Marsh can be seen on the right.

View from bridge site looking South. The remains of a wooden dam can be seen in the centre of the photograph.

Aerial photograph of bridge site. Large rock outcrop to the Northeast of bridge site can be seen in the background. A portion of Douglas Marsh can be seen at the north.

View from bridge site looking south. The remains of a wooden dam can be seen in the centre of the photograph.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No 1

PROJECT No J1754

LEGEND

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

BOREHOLE No. 1

PROJECT Proposed Bridge Site

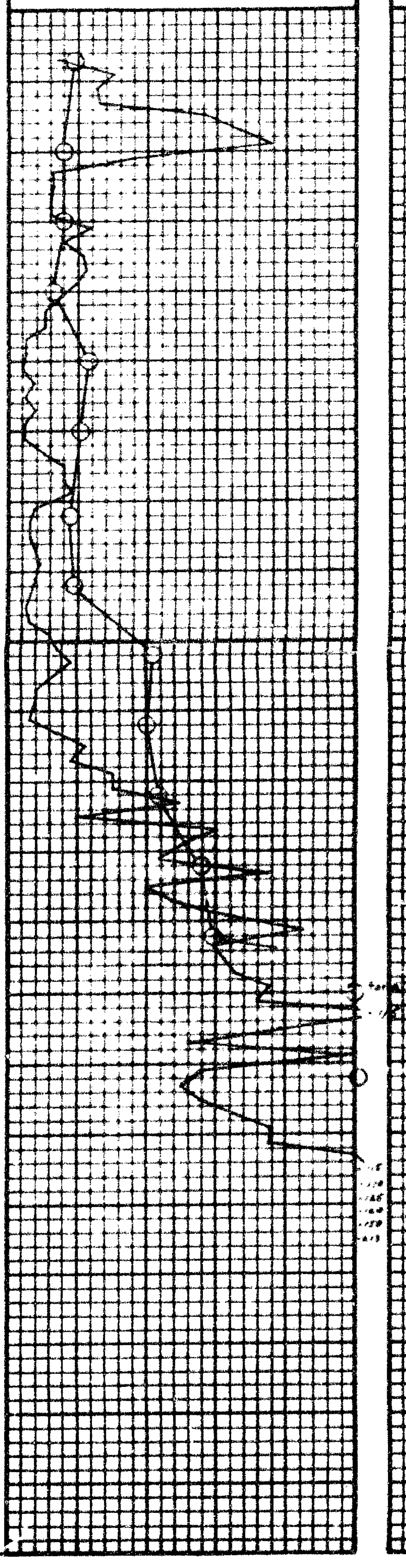
LOCATION Duval Creek [illegible]

HOLE LOCATION 316 + 58; [illegible]

HOLE ELEVATION 1019.[illegible] ft.

DATUM See [illegible]

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.
		[illegible]	0				
	WATER	[illegible].3					
	SAND AND GRAVEL-very dense to dense to about 1[illegible] feet, then medium dense to 45 feet.						
	- Sand is brown, well graded from fine to coarse.		10				
	- Gravel graded to [illegible] in. above 1[illegible] feet, becoming generally smaller ([illegible]") below 1[illegible] feet.		20				
	- Soil is much sandier below about 3[illegible] feet.		30				
	- Becoming dense at about 4[illegible] feet.		40				
			50				
			60				
			70				
	End of Bore	[illegible]					
Notes:	1) Borehole advanced using conventional wash boring techniques.		80				
	2) Cased with BX to 75 ft, Unable to drive casing beyond 75 ft.						
	3) Cone driven inside borehole from 7[illegible].[illegible] ft. to [illegible].[illegible] ft.						
	4) Cone driven beside borehole to 88 ft, at Sta. 316 +61; 1[illegible]. Refusal (100 blows/ft.) at [illegible] ft.	[illegible]	90				
			100				
			110				



WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

DRAWING No 2

PROJECT No [illegible]

LEGEND

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

BOREHOLE No ____

PROJECT [illegible]

LOCATION [illegible]

HOLE LOCATION [illegible]

HOLE ELEVATION [illegible]

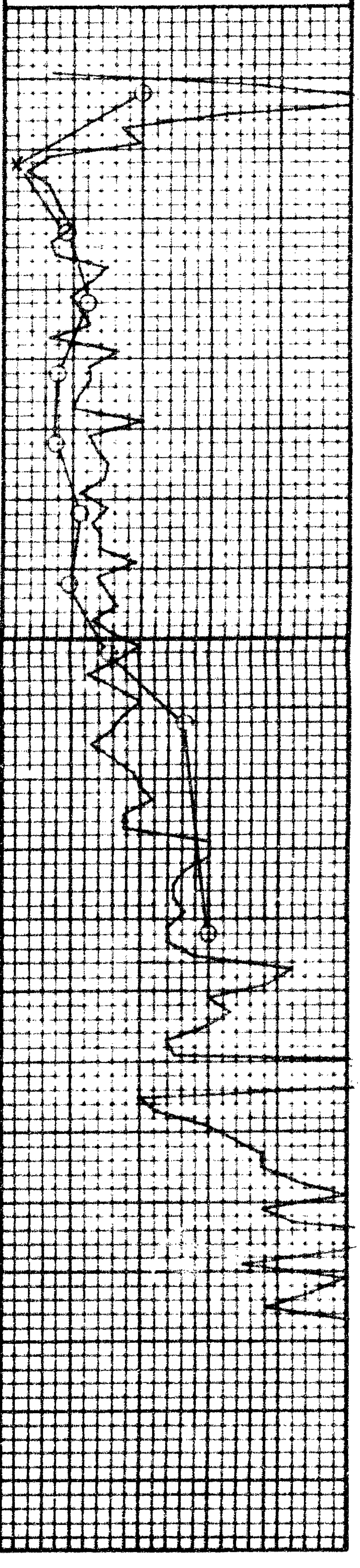
DATUM [illegible]

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET
		[illegible]	0
	[illegible]	[illegible]	
	[illegible] very dense to [illegible], becoming dense to [illegible] ft, then [illegible] dense to [illegible].		10
	– [illegible] brown, well graded fine, very fine to coarse.		
	– Gravel [illegible] above [illegible], becoming generally smaller [illegible] below [illegible].		20
	– [illegible] with sampler below about [illegible].		30
			40
	– Becoming dense at about [illegible].		50
	– Water drop of [illegible] in casing noted at [illegible].		60
	End of Bore	[illegible]	
Notes:	1) Borehole advanced using conventional wash boring techniques. 2) Cased with [illegible] to [illegible]. 3) Cone driven beside borehole to 9[illegible] ft, at [illegible]. Refusal [illegible] blows/ft. [illegible]		70
			80
		[illegible]	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

SAMPLE TYPE AND No

NATURAL UNIT WEIGHT P.C.F.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

DRAWING No. 3
PROJECT No. 1754

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]

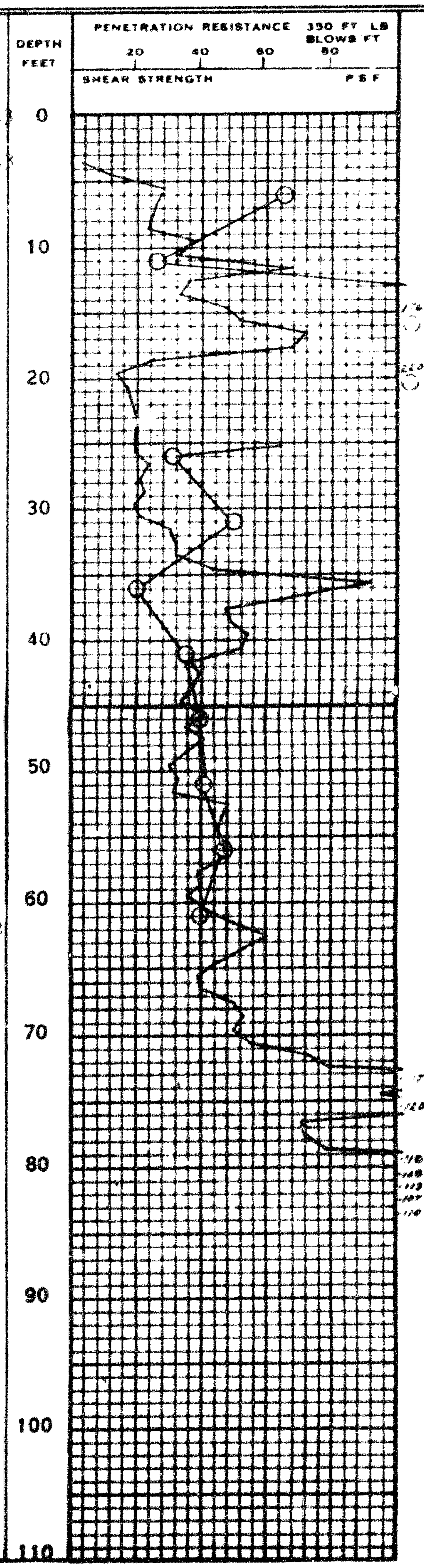
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT

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

BOREHOLE NO. 3
PROJECT Proposed Bridge Site
LOCATION Duval Creek W.P. 17-A
HOLE LOCATION 516 + 72; 17.5L
HOLE ELEVATION 1019.3 ft.
DATUM See Site Plan

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET
		1019.3	0
	WATER	1016.3	
	SAND AND GRAVEL-very dense above about 20 ft, and dense below 20 ft. to end of bore. - Sand is brown, well graded from very fine to coarse. - Gravel grades to 1 in. - Soil is much sandier below about 30 ft. - Water drop in casing noted at 40 ft. Pump capacity not sufficient to keep casing filled.		
	End of Bore	957.3	
		935.3	

Notes: 1) Borehole advanced using conventional wash boring techniques.
2) Cased with BX to 60 ft.
3) Cone driven beside borehole to 84 ft, at Sta. 516 + 73; 10'. Refusal (100 blows/ft.) at about 80 ft.



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

SAMPLE TYPE AND No.

NATURAL UNIT WEIGHT P.C.F.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

DRAWING No. ______
PROJECT No. ______

LEGEND

BOREHOLE No. ______
PROJECT ______
LOCATION ______
HOLE LOCATION ______
HOLE ELEVATION ______
DATUM ______

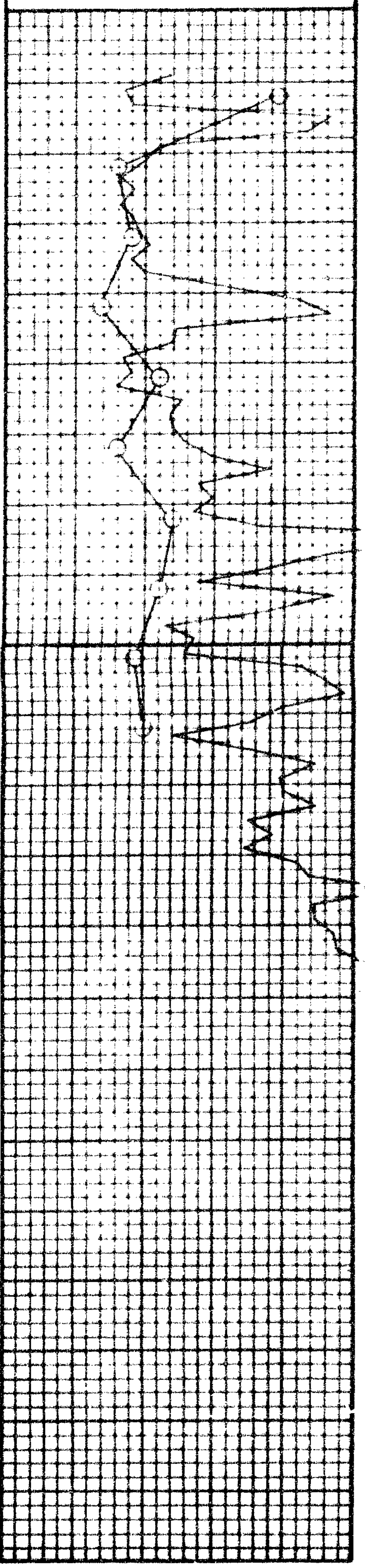
PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" O.D. SHELBY TUBE
2" D.A. CONE

SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY

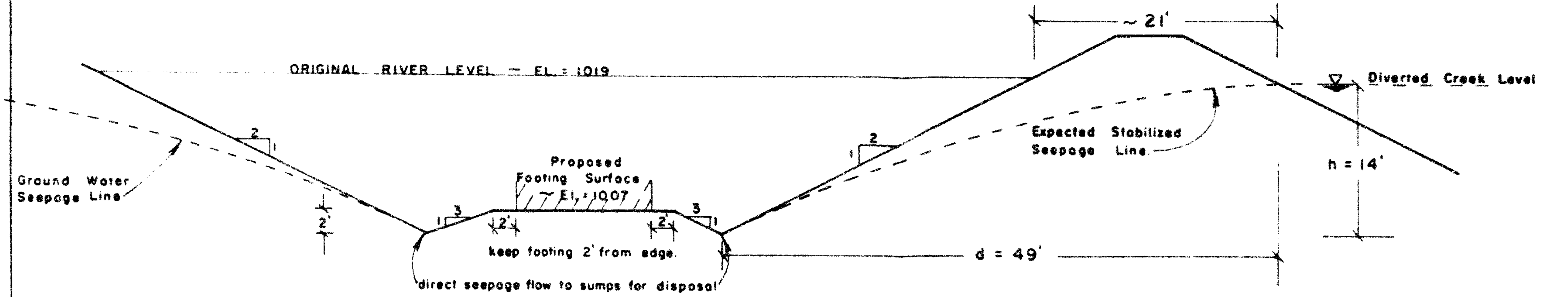
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT

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

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.
	[illegible]		0				
	[illegible]		10				
	[illegible]		20				
			30				
	[illegible]		40				
			50				
			60				
	Notes: 1) [illegible] with [illegible] 2) [illegible] 3) [illegible]		70				
			80				
			90				
			100				
			110				



FOOTING EXCAVATION AND SEEPAGE CALCULATION



PROCEDURE — 1) Divert creek.
2) Excavate below water level to approximate dimensions above.
3) Pump out water
4) Prepare footing bed and install footing.
5) Backfill to el. 1019 with adjacent sand and gravel with a rip rap cover of coarser stone.

ESTIMATE OF SEEPAGE FLOW

$Q = Pk(\sqrt{h^2+d^2} - d)$*

take coefficient of permeability $k = 2$ f.p.m. for sand and gravel** (conservative estimate)

P = perimeter of excavation (for base excavation 34' wide and 60' long — $P = 188'$)

Solving $Q = 715$ cu ft/min

It should be appreciated that the seepage line will become progressively flatter and therefore the value $\sqrt{h^2+d^2}$ and seepage will reduce. In any case, seepage should not exceed creek flow

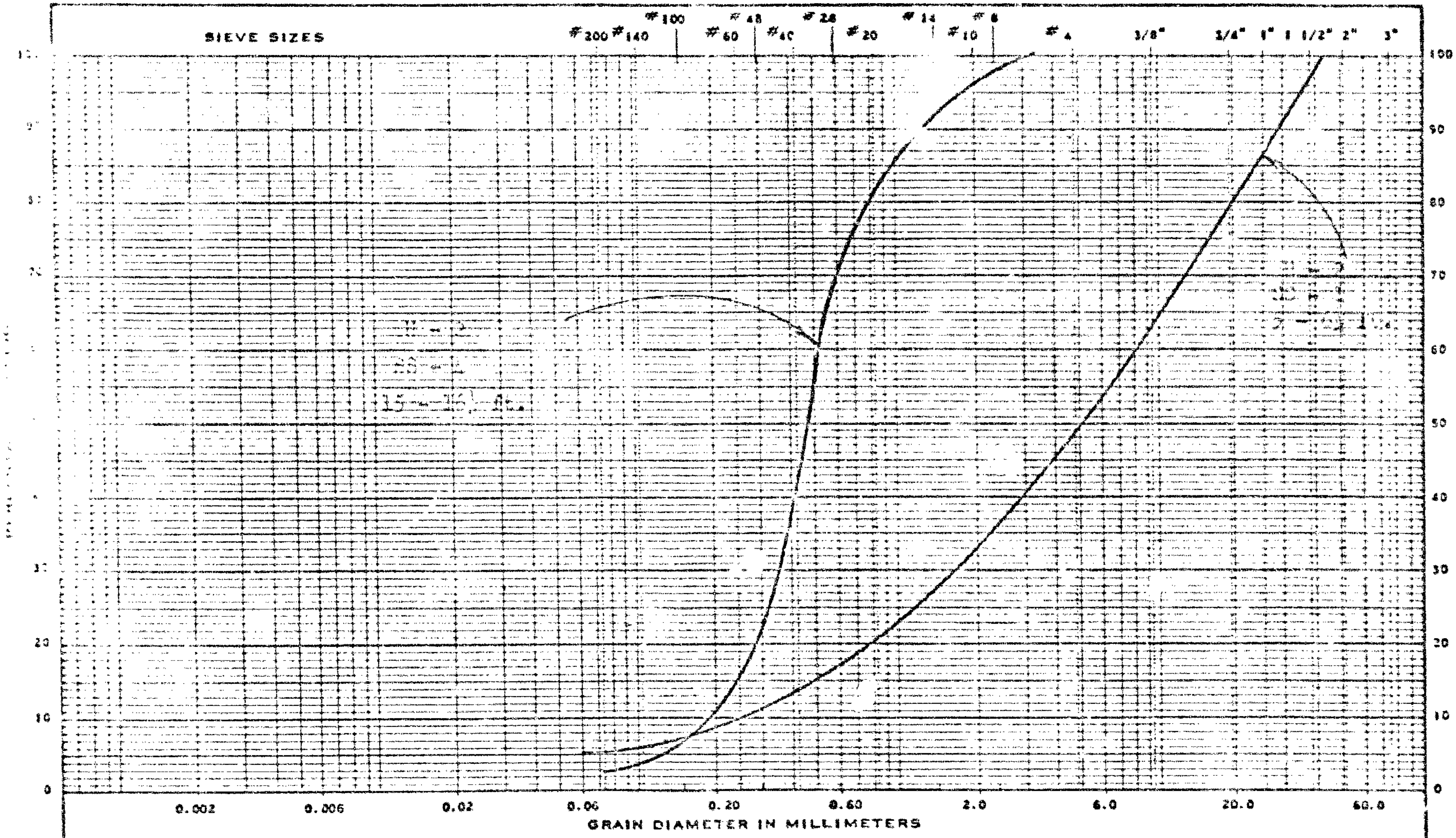
* Seepage Through Dams, 1933 Casagrande

** Soil Mechanics in Engineering Practice, Table 6, pg 48, Terzaghi and Peck

WILLIAM TROW ASSOCIATES LTD.

PROJ. J-1754 DRAWING 5 MARCH, 1965

MECHANICAL ANALYSIS



CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT			SAND			GRAVEL		

MODIFIED M.I.T. CLASSIFICATION

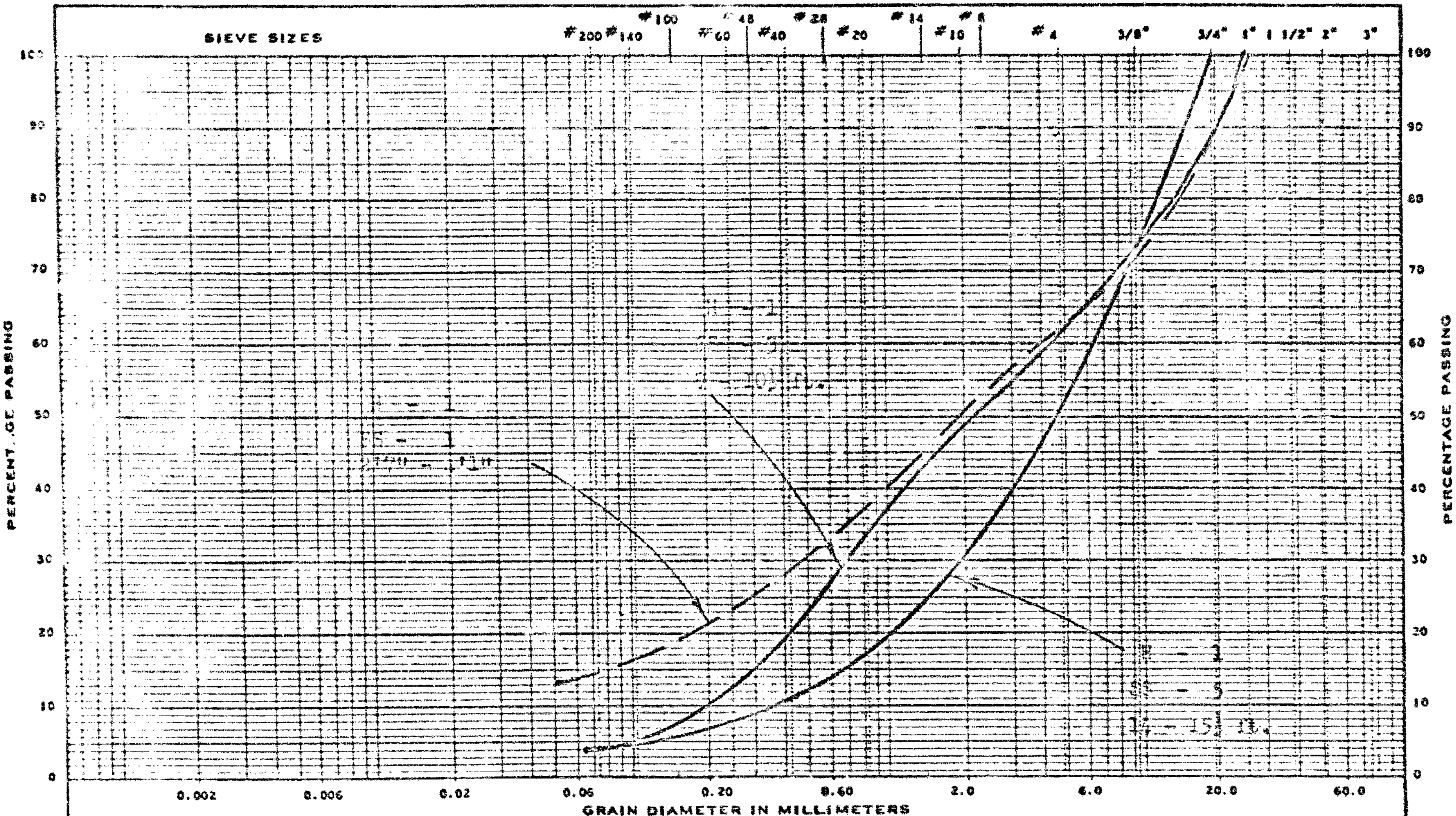
GRAIN SIZE ANALYSIS OF SAND NEAR REQUIRED FOUNDATION LEVEL

WILLIAM A. TROW AND ASSOCIATES LTD.

Project: J1754

Drawing No. 6

MECHANICAL ANALYSIS



← CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT			SAND			GRAVEL		

MODIFIED M.I.T. CLASSIFICATION

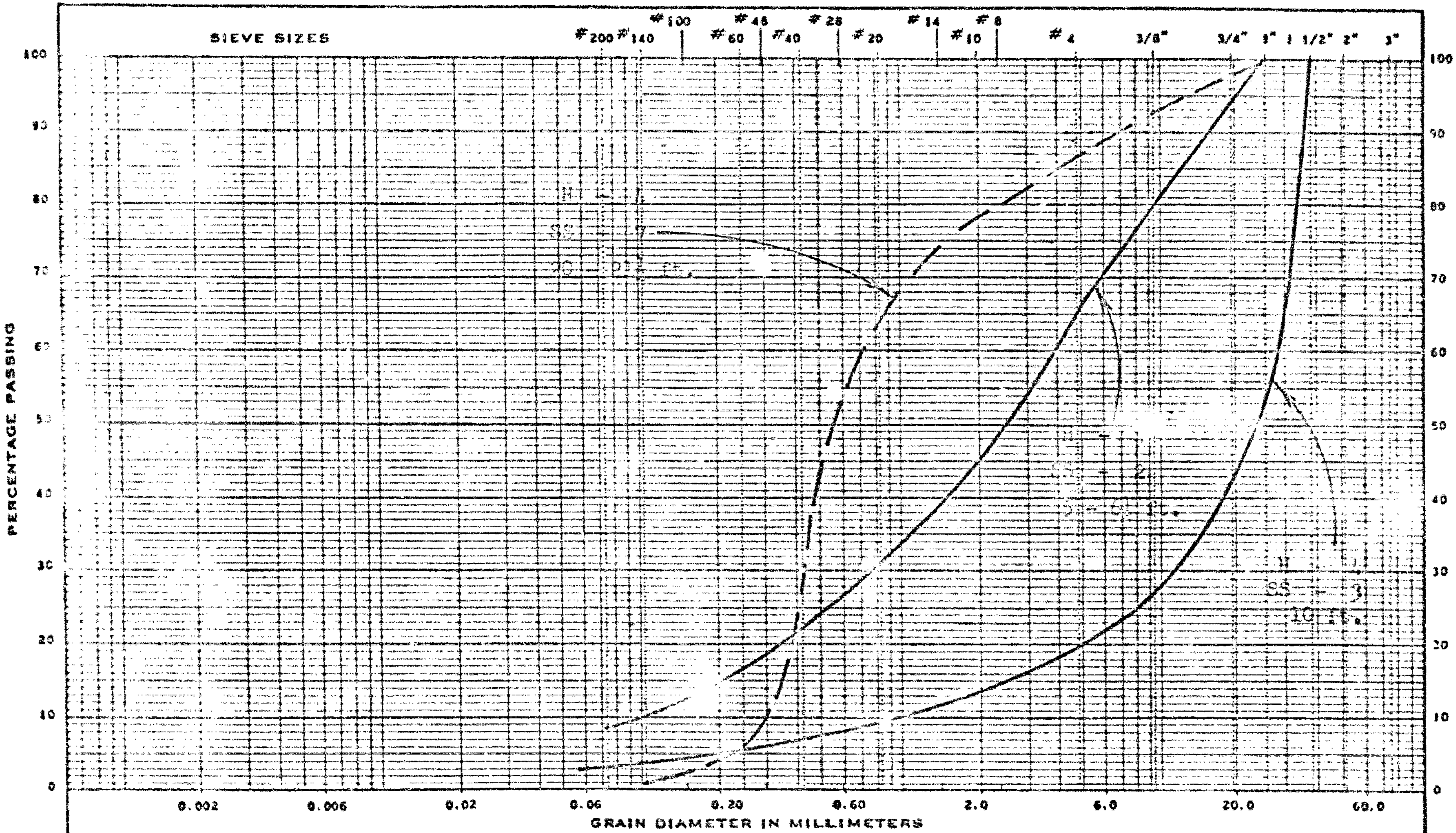
GRAIN SIZE ANALYSIS OF SAND NEAR REQUIRED FOUNDATION LEVEL

WILLIAM A. TROW AND ASSOCIATES LTD.

Project: J1754

Drawing No. 7

MECHANICAL ANALYSIS



CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT			SAND			GRAVEL		

MODIFIED M.I.T. CLASSIFICATION

GRAIN SIZE ANALYSIS OF SAND NEAR REQUIRED FOUNDATION LEVEL

WILLIAM A. TROW AND ASSOCIATES LTD.

Project: J1754

Drawing No. 8

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. G. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

From: Bridge Division,
Downsview, Ontario.

Date: December 11, 1964.

Our File Ref.

In reply to

Subject: Foundation Investigation
District [illegible]

Please make the necessary arrangements to have foundation investigations carried out at the following sites:

1)	W.P.	Kindiogami River (East Crossing)
2)	W.P. 16-65	Kindiogami River (West Crossing)
✓3)	W.P. 17-65	Duval Ck.
4)	W.P. [illegible]-65	West Little White River
5)	W.P. [illegible]-65	Rapid River
6)	W.P.	[illegible] River

Plans for the above crossings will be available to [illegible] the end of this month.

[signature]

[illegible]
[illegible] DeVisser

[illegible],
Bridge Planning Engineer.

cc: Foundations Office (Rm. 110)

Hwy. 401 & Keele St.,
Downsview, Ontario.

Materials and Testing Division

December 17, 1964

William A. Trow & Associates, Ltd.,
1850 Jane Street,
Weston, Ontario.

Attention: Mr. Wm. A. Trow

Re: W.P. - , Hwy. 639, Kindiogami River (East Crossing).
W.P. 16-65, Hwy. 639, Kindiogami River (West Crossing).
W.P. 17-65, Hwy. 639, Duval Creek.
W.P. 18-65, Hwy. 639, West Little White River.
W.P. 19-65, Hwy. 639, Rapid River.
W.P. - , Hwy. 639, Sharpsand River.
--- District 18, Sault Ste. Marie ---

Dear Sir:

Please consider this your authority to carry out foundation investigations at the above sites. Plans and profiles were provided to your representative on December 11, 1964.

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

Ten copies of each completed foundation report, with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to March 19, 1965. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawings accompanying the foundation reports, showing the location of borings, the inferred subsoil conditions, etc., are to become contract drawings, you are requested to prepare them in accordance with the D.H.O. standards. To enable you to do this, we are supplying you with sample drawings with all the necessary explanations, together with linen sheets for your drawings. You are also requested to provide the D.H.O. with Cronaflex copies of the drawings.

cont'd. /2 ...

Charges for the work performed will be in accordance with your Schedule of Rates, dated November 19, 1962, and invoices to be addressed to the attention of the undersigned.

Yours very truly,

A.R.

NDS/MdeF

A. Rutka,
MATERIALS & TESTING ENGINEER

cc: Messrs. C. McCombie
H. McArthur
A. A. Ward
E. R. Saint
Mrs. T. Tate
N. D. Smith (2)
Foundations Office
Gen. Files (2)

cc: Foundations Office (Rm. 110)

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attention: Mr. A. McCombie

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

April 13, 1965

FOUNDATION INVESTIGATION REPORT BY:
William A. Trow & Associates, Limited,
Proposed Crossing - Duval Creek, Hwy. 639,
W.P. 17-65 --- District 18

Attached, please find the above-mentioned report submitted by the Consultant, W. A. Trow & Associates, Ltd. We have reviewed the report and found the factual data both adequate and well presented. The conclusions and recommendations are straightforward and do not require any comments. However, should you have some additional questions to discuss, please feel free to contact our office.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/McP
Attach.
cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
R. McArthur
A. A. Ward
E. F. Saint
A. Watt
F. De Visser

Foundations Office
Gen. Files

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Laboratory Bldg.,
DOWNSVIEW, Ontario.

From: Bridge Division,
208 Simpson Street,
FORT WILLIAM, Ontario.

Date: October 27, 1965.

Our File Ref.

In reply to

Subject: Preliminary Plans

When preliminary plans are sent to you for your comments, it would probably be more satisfactory for you to make your comments directly to Mr. C. Grebski, Bridge Design Engineer. However, to keep me informed of any changes or additions would it be possible for you to send me a note if any changes or additions are recommended to the Bridge Design Office.

F. DeVisser

FDV/mcr

F. DeVISSER,
Regional Bridge Location Engineer.

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Laboratory Bldg.,
DOWNSVIEW, Ontario.

From: Bridge Division,
208 Simpson Street,
Fort William, Ontario.

Date: October 27, 1965.

Our File Ref.

In reply to

Subject: Site 38S-276, W. P. 17-65,
Duval Creek Bridge,
7.5 Miles West of Sec. Hwy. 546,
Hwy. 639, District 18.

Enclosed please find two prints of Preliminary Plan D-5717-P1 for the subject structure.

If you have any comments, please let me know.

F. DeVisser

FDV/mcr
Enc. (2)

F. DeVISSER,
Regional Bridge Location Engineer.

No comment
[illegible]
[illegible] 65

cc: Foundations Office (Rr. 110)

Wm. A. Trow

Mr. C. Grebski,
Bridge Design Engineer,
Bridge Division.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

November 10, 1965

W.P. 17-65 - Duval Creek Bridge - Plan #D-5717-P1
W.P. 18-65 - West Little White River Bridge - Plan #D-5679-P1
W.P. 16-65 - Kindiogami River Bridge - Plan #D-5754-P1
- District No. 18 -
(Sault Ste. Marie)

We have reviewed the Preliminary Plans for the above-mentioned proposed structures with regard to the subsoil conditions, as outlined in the Foundation Reports by Wm. A. Trow & Associates. We have no further comments.

K. G. Selby

KGS/MdeF

K. G. Selby,
SENIOR FOUNDATION ENGINEER
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office
Gen. Files

Re. Proposed Crossing Duval Creek
Hwy 639 W.P 17-65.

Mr Jim Keen called regarding the above mentioned Job and requested recommendations pertaining to the structure foundations. The new structure will be a three span structure, with perched abutments and piers on pile bents with caps. In view of the hydrological requirements and also because of the type of structure, pile foundation is ~~only~~ the most suitable type of foundation.

This section recommended 12¾" O.D tubular piles driven to approximate tip elev 960.0 should provide 50 tons/pile. However pile driving during construction should be controlled by the use of Hiley Formula.

M Devata
Sept 29th 1965.

F. De Visser Reg. Bridge Loc. Engr. Fort William
Re: WP 17-65 Duval Creek, Hwy 639, District 18

With reference to your telex of July 30. 1965 sentance on page 5 of report should read as follows: An alternative to spread footings is to perch the abutments on piles above the creek bed but below the minimum expected water level.

A.G. Stermac

JUL 30 AM 9:29

DOWN FTWR 2 JULY 30/65 823A

A STERMAC PRIN FOUNDATION ENGR ADMIN BLDG

RE: WP 17-65 DUVAL CREEK HIGHWAY 639, DISTRICT 18

ON PAGE 5 OF THE FOUNDATION REPORT UNDER ITEM B, UNDER PILED FOUNDATIONS

WHICH STATES THAT THE ABUTMENTS MAY BE PERCHED ON PILES ABOVE THE CREEK

BED, BUT BELOW THE MAXIMUM WATER LEVEL

WHAT DOES THIS MEAN?

F DEVISSER REG BRIDGE LOC ENGR

JO

TELETYPE

D

FTWR DOWN 9 AUG 5/65 1043A VR

FRANK DEVISSER REG BRIDGE LOC ENGR

RE WP17-65 DUVAL CREEK HWY 63) DIST 12

WITH REFERENCE TO YOUR TT OF JULY 30TH/65 SENTANCE ON PAGE 5

OF REPORT SHOULD READ AS FOLLOWS

"AN ALTERNATIVE TO SPREAD FOOTINGS IS TO PERCH THE ABUTMENTS

ON PILES ABOVE THE CREEK BED BELOW THE MINIMUM EXPECTED WATER

LEVEL

A G STERMAC PRINC FOUND ENGR MATS & TESTG

SW

Y
P
E