

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

Attention: Mr. S. McCombie

Mr. A. G. Stermac,  
Principal Foundation Engr.,  
Foundation Section,  
Materials & Research Division.  
October 9, 1963.

FOUNDATION INVESTIGATION REPORT BY -  
William A. Trow & Associates Ltd. -  
Proposed Crossing over Young Creek,  
Dev. Rd. #684, Port Ryerse, Ontario.  
W.P. 701-62

Attached we are forwarding to you the above mentioned report submitted by the Consultant W. A. Trow and Associates Ltd. of Toronto.

We have reviewed the report and herewith submit our comments for your consideration.

The investigation consisted of four boreholes of which three were located along the site of the proposed new structure. These three boreholes have indicated that the general subsoil stratification is relatively uniform but they have also shown that variations exist in the upper 10 to 20 ft. This is especially evident in B.H. #3.

The proposed footing elevation for the barrel arch culvert is at 594.0 which would allow for somewhat variable soil below the footings. The variation would be in density as well as nature of the subsoil. It would be therefore justified to assume that when the full load is applied some differential settlements will result.

Since in the case of the barrel arch the structure is completed before the main load (resulting from the embankment) is applied any differential movement is detrimental irrespective whether it results from the immediate or consolidation settlement. It is therefore our recommendation that consideration be also given to other types of structures.

The Consultant has pointed out that some construction difficulties can be expected mainly connected with excavations and related dewatering. Some of the subsoil materials are very prone to boiling if subject to excessive hydraulic heads.

The Consultant's proposal to build the structure in an open excavation of which the sides should always be kept deeper and from which pumping is to be carried out seems to be practical and feasible. However, because of the nature of the subsoil difficulties could be expected and this should be clearly stated in the Contract documents. The importance of always pumping from trenches that are lower than the rest of the excavation cannot be overemphasized.

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

AGS/tt  
Attach.

*agsternmac*  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. H. A. Tregaskes  
H. D. McMillan  
A. Gater  
H. C. Dernier  
J. R. Roy  
A. Watt

Foundations Office  
Gen. Files

63 F 215C

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS  
LABORATORY TESTING  
SOIL MECHANICS CONSULTATION

W.P. 701-62

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

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

Project: J1214

September 26, 1963

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

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

Re: Foundation Investigation  
Proposed Crossing over Young Creek  
Development Road No. 684  
Port Ryerse, Ontario  
W.P. 701-62

Dear Sirs:

In conformance with your authorization of August 28, 1963, we have investigated the subsoil conditions at this culvert site crossing of Development Road No. 684, over Young Creek.

We have found that the subsoil consists almost entirely of dense silt down to limestone bedrock which lies about 60 feet below river bed level. Since the soil is granular in nature, there will be no stability problem associated with the installation of fill to a height of about 50 feet above the level of the flood plain.

We have suggested that the footings of the culvert be taken 7 feet below stream level to El 594 feet or to the base of the river alluvium. The recommended pressure to apply at this depth is 6500 psf, which is the approximate surcharge pressure exerted by the central sections of

the fill. This pressure is much higher than is normally permitted for a building footing but the problem here is not similar. Flexible culvert construction is recommended.

Some ground water problems should be expected during excavation, but the alluvial sand is sufficiently free-draining to permit this operation to proceed in the form of a large open excavation. Comments on this subject are given in the report.

We shall be pleased to discuss any queries you may have after you have reviewed this information.

Yours very truly,

*W. A. Trow*

William A. Trow, P.Eng.

WAT/gc  
Encls.

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

FOUNDATION INVESTIGATION  
PROPOSED CROSSING OVER YOUNG CREEK  
DEVELOPMENT ROAD NO. 684  
PORT RYERSE, ONTARIO  
W.P. 701-62

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FOUNDATION INVESTIGATION  
PROPOSED CROSSING OVER YOUNG CREEK  
DEVELOPMENT ROAD NO. 684  
PORT RYERSE, ONTARIO  
W.P. 701-62

PROJECT AND SITE

This proposed crossing of Young Creek comprises one portion of Development Road No. 684, which will extend west from County Road No. 12 toward Walsh Station. It is located about 1 mile northwest of Port Ryerse, Ontario.

It is understood that a 30 foot arch culvert about 270 feet long will be installed at this site and that the final road grade will be at El 653, or approximately 54 feet above the bottom of the creek.

Young Creek, at this crossing location, flows near the south end of a broad flood plain about 500 feet wide. The course of the creek is in a general easterly direction toward its outlet into Lake Erie at Port Ryerse. The valley supports a heavy growth of trees, up to  $2\frac{1}{2}$  feet in diameter and accompanying undergrowth. The valley floor is covered with a layer of peat up to 3 feet thick. The table farm land on top of the valley lies at a level of about 100 feet above the creek bed on the north, and 40 feet on the south.

The creek is approximately 20 feet wide and 5 feet deep and the water level in it during the period of the field work was at El 601.5 feet. Local information indicates that the water level, during the Spring flooding, rises to the top of the creek bank at El 604 feet. The flow in the creek is erratic, due to the obstruction of numerous trees lying on the creek bed.

SUBSOIL CONDITIONS

The results of the borings are described in detail on the borehole logs, Dwgs. 1 to 4 of this report, and in more general form on the site plan. It can be seen that the predominant subsoil stratum is the medium dense to very dense silt which extends to bedrock and is approximately 55 feet thick. From visual inspection, reddish grey clay seams about 1/32 inch thick were occasionally found in the upper portion of the silt stratum, and they were more frequently encountered, with a slight increase in thickness, in the lower 20 feet. On the basis of penetration resistance measurements obtained during sampling, the silt is indicated to be in a medium dense condition for the upper 2 to 10 feet and it becomes very dense with depth. According to measurements of moisture content and relative density, however, it is concluded that the silt at upper levels is dense, even though the resistance to penetration was not very great. The results of these computations are presented in Table 1.

The silt stratum is covered by a layer of alluvial sand, extending from the ground surface to approximately 5 feet below the creek bed level at the locations of holes 1, 2 and 3, where the proposed arch culvert is to be placed. At the location of hole 4, the alluvium only extends to the level of the creek bed. The upper portion of the alluvium that lies above the creek bed level was found to be in a very loose to loose condition, and the remaining portion exists generally in a medium dense state. Some timbers were encountered in holes 1, 2 and 3 just below the creek bed elevation. As indicated in the opening paragraphs, the upper 3 feet of soil in the vicinity of hole 4 consists of compressible peat.

Limestone bedrock was encountered underlying the silt at El 540 feet in holes 2 and 4. Coring in AX core size to a depth of 10 feet was made in hole 4, and the recovery was 100 percent. Although bedrock was not positively proved in hole 2, the record of positive bouncing refusal to the



wash rods and the emergence of gas from this level leaves little doubt that the bedrock surface is horizontal under the site. A water well was drilled approximately 1000 feet north of the site, and the surface of bedrock was revealed to be at a similar elevation.

### FOUNDATIONS

As indicated in the opening paragraphs of this report, the crossing of Young Creek will take the form of a 30 foot arch culvert and embankment fill reaching a height of 50 feet above the creek valley. Since the subsoil underlying the crossing location is granular in nature, it will adjust immediately to the application of culvert and fill load. Therefore, bearing capacity, embankment stability and long term settlement will not be a problem in this construction. The embankment fill will remain stable after it reaches full height and its settlement will be virtually complete at that time. The 5 foot layer of peat should be removed from the embankment route before fill is placed, however, in order to assure that this satisfactory construction situation is achieved.

The decision concerning the appropriate bearing value to use for the design of the culvert footings will be determined by the type of construction proposed. If a flexible pipe culvert of the Armco type is used, differential movement is not of any particular concern. The culvert will deform and settle sufficiently to take advantage of the internal resistance, arching forces and lateral support that will be developed in the surrounding fill. Careful and adequate compaction of the surrounding fill adjacent to the sides of the pipe becomes a major requirement in this type of construction.

If the culvert is to be supported on strip footings, the selection of a design bearing value requires more careful study. The pressure exerted by 50 feet of embankment fill will reach a maximum value in the

order of 6500 psf. If a lower bearing stress than this were to be used for the culvert footings, somewhat less settlement could be expected at the bridge location, and therefore some additional weight of the adjacent fill would be passed on to the structure. In addition, considering that the culvert will be carrying about 35 feet or more of fill above it, footings designed to a pressure much lower than 6500 psf will probably use up all of the 30 foot available span below river bed. Therefore there is a definite lower bearing pressure limit that is practical for this construction. Any projection of the footing under the adjacent fill must carry the surcharge pressures of this fill and therefore its efficiency as a bearing medium is reduced.

According to conventional empirical rules suggested by Terzaghi and Peck\* for building footings, the safe net bearing pressure to apply to this soil for a limiting settlement of 1 inch is about 2000 psf. Since the movement of the culvert will be determined almost entirely by the settlement of the surrounding fill, however, the reference to a limiting settlement of 1 inch has no practical significance in this project. In addition, the empirical rules of Terzaghi and Peck have been shown to be extremely conservative for bearing situations close to the ground surface or below the water table.\*\* The relative density computation shown in Table 1 supports this view.

It is proposed therefore that the footing be designed on the basis of ultimate bearing capacity considerations only. The ultimate bearing capacity of a strip footing of width B on the surface of sand - the extreme, but applicable situation applying here if river scour were to extend to footing level, - is given by the expression:

$$q = \frac{1}{2} \gamma B N_{\gamma} \quad ***$$

\* "Soil Mechanics in Engineering Practice" Pg. 423. Terzaghi & Peck

\*\* "Research on Determining the Density of Sands By Spoon Penetration Testing" Gibbs, H.J. & Holtz, W.G. (1957) Proc. of the 4th Int. Conf. on Soil Mechanics & Foundation Engineering.

\*\*\* "Soil Mechanics in Engineering Practice" Terzaghi & Peck Pg. 170

where:  $\gamma$  the submerged weight of the soil = 65 pcf approximately

$N_3$  is a bearing capacity factor estimated to be at least equal to 100 for this sand.

Solving  $q$  ultimate = 3250 B psf

For a footing 10 feet wide, - the order of width probably required here, -  $q = 16$  tsf approximately. It is concluded therefore, that the footings of the culvert can be designed to a net bearing stress of 6500 psf without danger of failure.

In view of the variable loading condition applying across the length of the culvert, a flexible type of construction is recommended. In order to provide for differential movement between the centre and edge of the fill, the centre section of the culvert should be founded about 3 inches higher than the end sections. This recommendation is based upon the following very approximate elastic settlement computation.

$$S_c = 0.5 \frac{h \Delta p}{E}$$

where:  $h$  is the thickness of soil below the fill and above bedrock  
 = 660 inches approximately  
 $\Delta p$  is the average pressure transmitted by the fill and culvert into the soil. A value of 5 ksf is conservatively assumed  
 $E$  the modulus of elasticity of the lower sand is estimated to be at least equal to 500 ksf

Solving in this expression,  $S_c = 3\frac{1}{2}$  inches approximately. Beyond the edges of the footing there should not be any settlement, and therefore this computation provides an approximate indication of the maximum differential movement to be expected.

Since the alluvial sands above El 594 feet are loose and organic materials are present in them, it is recommended that the footings be installed at this elevation on the top of the silt stratum. This bearing level will be about 7 feet below the river level, and consequently, excavation difficulties must be expected when digging to it. These ground water difficulties can be minimized by diverting the river through a temporary channel made farther to the north and by making an excavation much wider than is normally required for a simple footing. The digging operations should be conducted in such a manner that the proposed footing location will always be higher than the outside limits of the excavation. The alluvial sand, being relatively permeable, will drain to the perimeter ditches and a gradual reduction in the water table will be achieved.

It may be necessary to support the steep south bank of the river at the west end of the culvert during this footing installation work. Alternatively, the slope could be cut back, and free-draining pit run gravel could be placed against it to permit drainage of the seepage emerging at this location.

When the footing bearing surface is exposed it should be covered with a layer of weak concrete in order to provide a clean firm working surface.

It is understood that the channel of the creek will be paved in order to protect the footings from scour. It is assumed that the exposed slopes of the embankment will receive rip rap protection for the same reason.

In order to provide lateral support to the arch culvert, a high degree of compaction of the adjacent fill should be specified. A density in the order of 100 percent standard Proctor (A.S.T.M. 498 42T) is recommended.

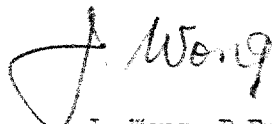
The resistance to sliding developed along the base of the footing under the rough working slab will equal the normal load times the tangent of the angle of internal friction of the silt. A value of  $\tan \phi = 0.7$  is believed to be a conservative value to use for this material when full load is applied.

#### CONCLUSIONS AND RECOMMENDATIONS

The foregoing observations and comments can be summarized briefly as follows:

- 1) The site of this development road crossing of the valley of Young Creek, is underlain by dense silt down to hard limestone bedrock which lies about 60 feet below the water surface.
- 2) Since the subsoil is granular, no embankment stability problem exists in this construction. Settlements will occur immediately with load application. An elastic movement in the order of 3 inches has been estimated.
- 3) The footings for the culvert should be designed to the same bearing stress as is exerted by the embankment fill. This surcharge value is estimated to be in the order of 6500 psf.
- 4) The culvert footings should bear on the dense silt at El 594 feet, or about 7 feet below creek level. Excavation difficulties should be expected, but gravity drainage to sumps of the sand alluvium above the silt appears feasible.

JW/gc  
J1214  
Sept./63

  
J. Wong, P.Eng.


  
William A. Trow, P.Eng.

TABLE 1  
RELATIVE DENSITY DETERMINATIONS

In place dry density  $\gamma$  determined indirectly on sample from hole 2, 15 feet, by measurement of moisture and assumption of specific gravity.

$$\gamma = \frac{62.4S}{1+W.S}$$

where:  $W$  = moisture content = 16.3 % dry weight

$S$  = 2.70 assumed

Solving  $\gamma = 115.6$  p.c.f.

Maximum density determined in two separate tests in a consolidometer  $2\frac{1}{2}$  inches diameter, 1.4 inches deep. Sample was placed in saturated condition with drainage at top and bottom and no water in the oedometer. Maximum load of 27 t.s.f. applied and sides of oedometer vibrated.

Maximum dry density = 124.2 p.c.f. and 123.7 p.c.f.;  
 take  $\gamma_{\max.} = 124$  p.c.f. (Probable that density too high because of crushing of grains).

Minimum dry density determined by pouring dry sand through a funnel into a water-filled 1000 ml. flask.

Minimum dry density = 76.6 p.c.f.

$$\text{Relative density} = \frac{\gamma_{\max.} (\gamma - \gamma_{\min.})}{\gamma (\gamma_{\max.} - \gamma_{\min.})} \% = \frac{124.0 (115.6 - 76.6)}{115.6 (124 - 76.6)} = 88.3\%$$

Note: 65% - 85%: Dense\*  
 >85% Very Dense

\* "Research on Determining the Density of Sand by Spoon Penetration Testing"  
 Gibbs, H.J. & Holtz, W.G. (1957) Proc. of the 4th Int. Conf. Vol. 1.

## 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 — 1/2°

## NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

## ATTERBERG LIMITS

LIQUID LIMIT —○—

PLASTIC LIMIT ———

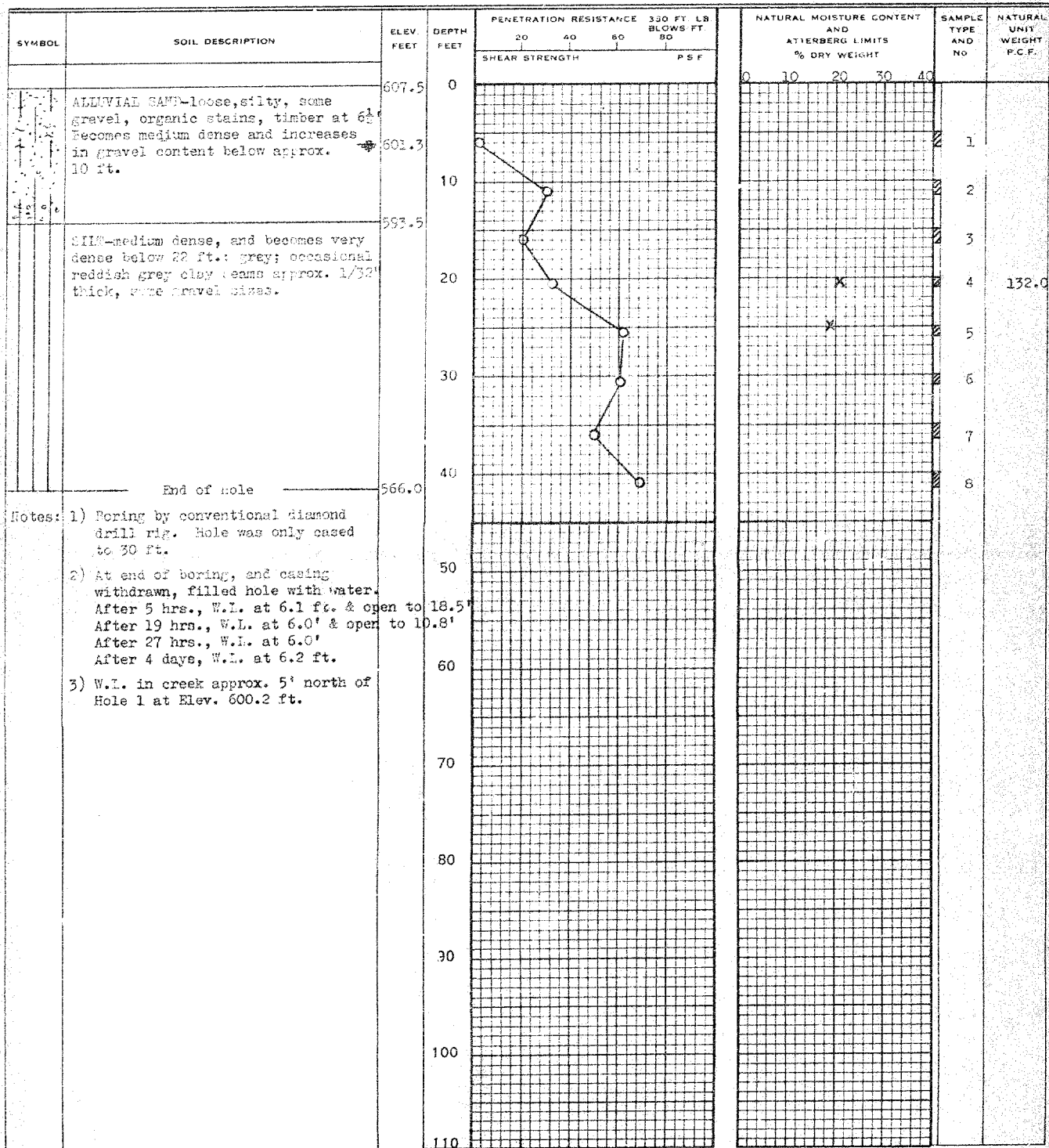
## SAMPLE TYPE

2" O.D. SPLIT TUBE —■—

2" I.D. SHELBY TUBE —■—




3" O.D. SHELBY TUBE —■—

BOREHOLE No. 1  
 PROJECT Proposed Crossing over Young Creek, W.P. 701-62  
 LOCATION Port Rverge, Ontario  
 HOLE LOCATION See Site Plan Drawing  
 HOLE ELEVATION 607.5 ft.  
 DATUM Geodetic






BORSHOLE NO. 2PROJECT Proposed Crossing over Young Creek, W.P. 701-62LOCATION Port Ryerse, OntarioHOLE LOCATION See Site Plan DrawingHOLE ELEVATION 604.4 ft.DATUM Geodetic

## PENETRATION RESISTANCE



2" O.D. SPLIT TUBE 2" I.D. SHELBY TUBE 2" DIA. CONE 

## SHEAR STRENGTH



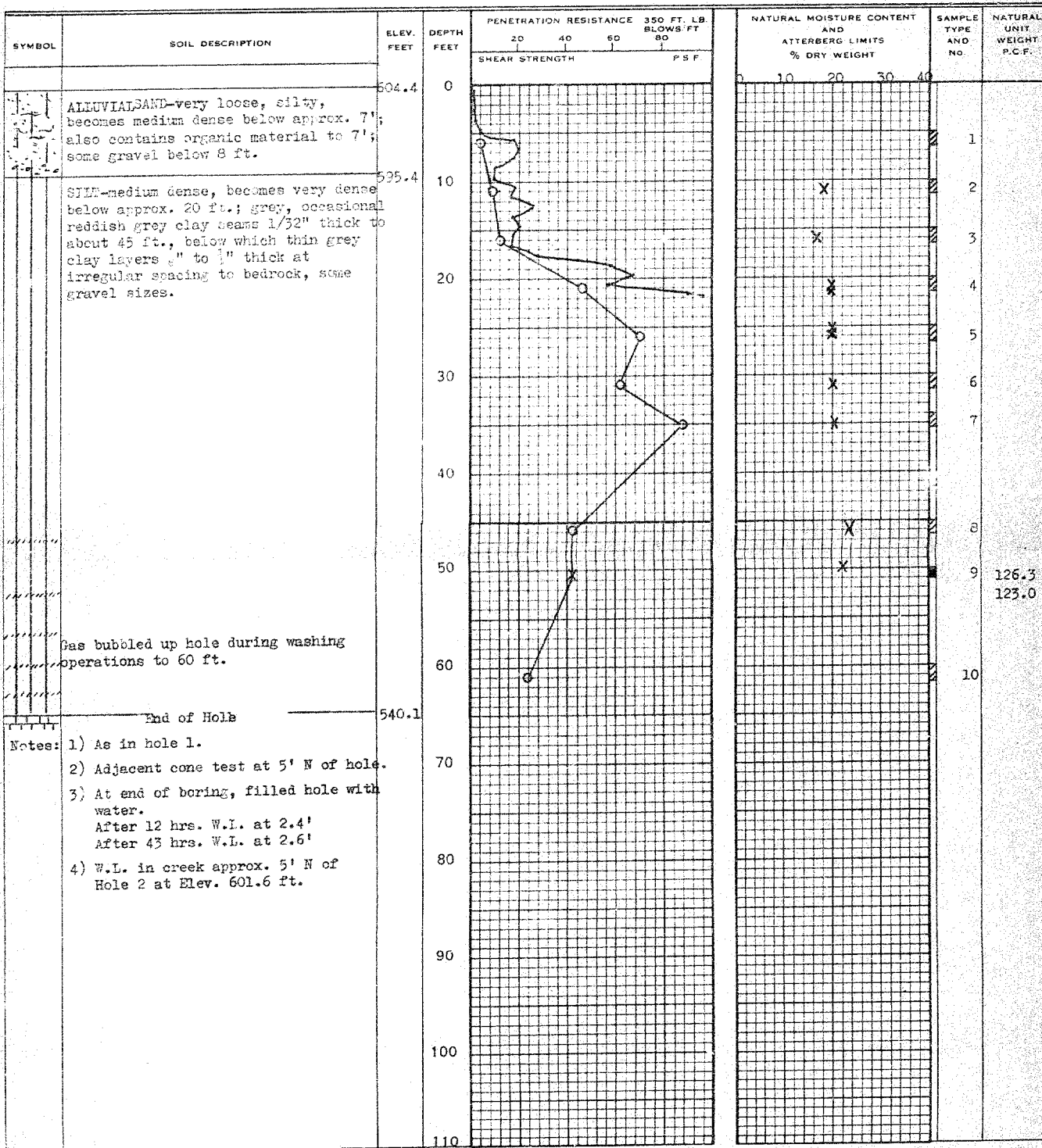

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE UNCONFINED COMPRESSION VANE TEST AND SENSITIVITY (S) 

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 









# WILLIAM A. TROW & ASSOCIATES LTD.







SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

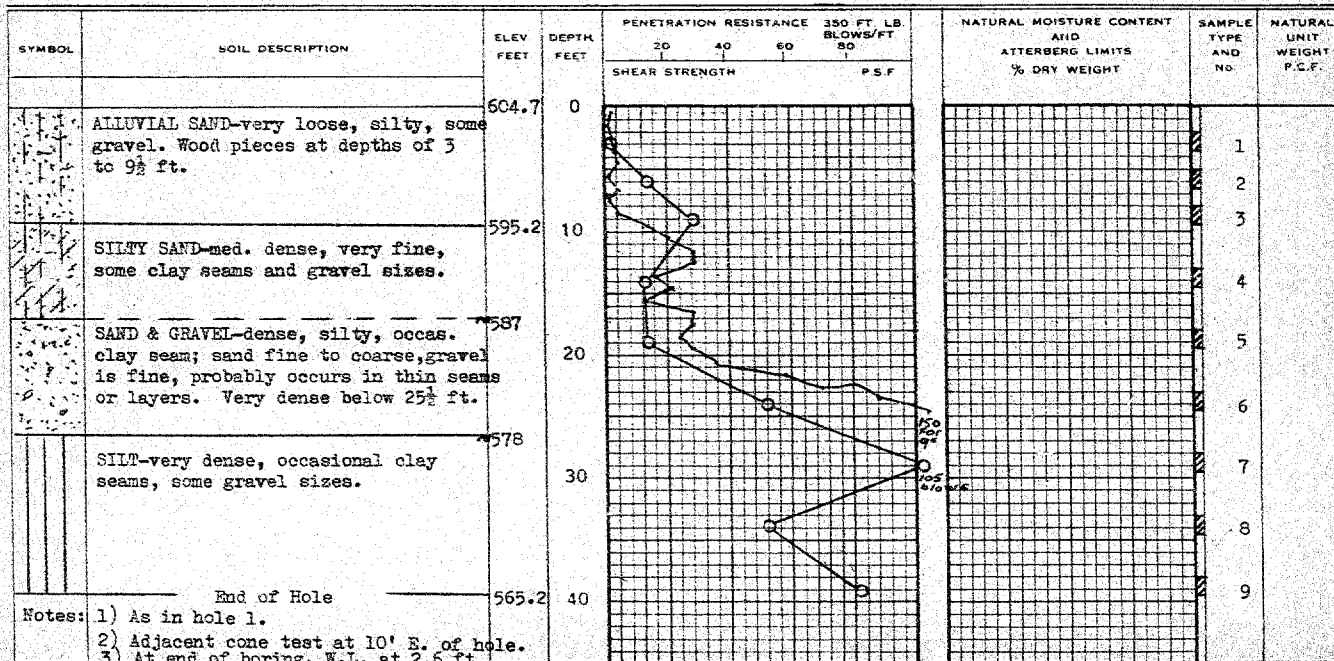
## LEGEND

DRAWING No. 3  
PROJECT No. J1214

BOREHOLE No. 3  
PROJECT Proposed Crossing over Young Creek, W.P. 701-62  
LOCATION Port Ryerse, Ontario  
HOLE LOCATION See Site Plan Drawing  
HOLE ELEVATION 604.7 ft.  
DATUM Geodetic

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 



# WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

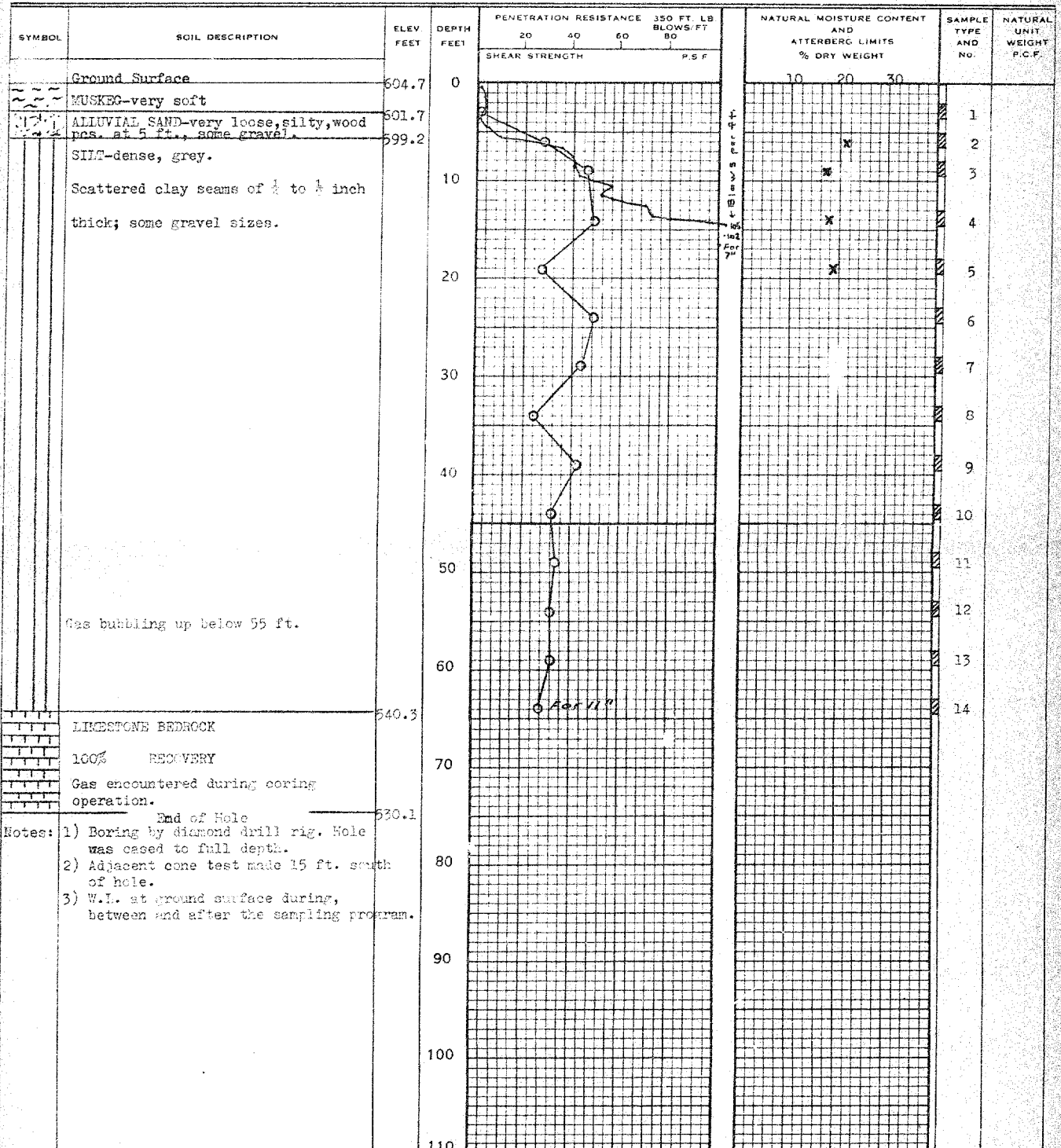
## LEGEND

DRAWING No. 4  
PROJECT No. 01214

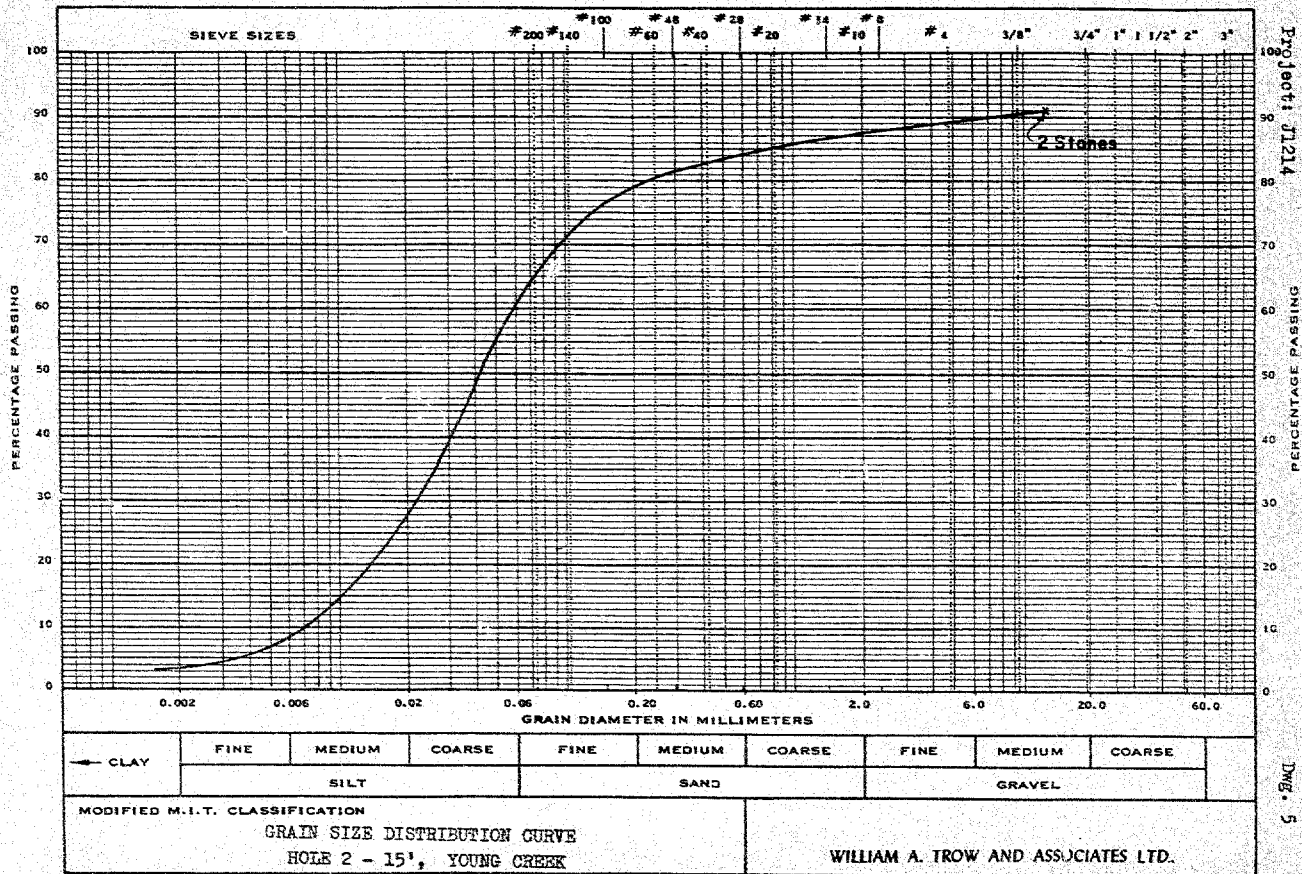
BOREHOLE NO. 4  
PROJECT Proposed Crossing over Young Creek, W.P. 701-62  
LOCATION Fort Ryerse, Ontario  
HOLE LOCATION See Site Plan Drawing  
HOLE ELEVATION 604.7 ft.  
DATUM Geodetic

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 15.4

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



# MECHANICAL ANALYSIS



Materials and Research Division

August 28, 1963

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

Attention: Mr. W. A. Trow

Re: W.P. 701-62, Dev. 76, 684, Young's Creek,  
District #2, London, Ontario.

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 August 26, 1963.

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

Ten copies of the completed foundation report with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to October 2, 1963. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawing accompanying the foundation report, showing the location of borings, the inferred subsoil conditions, etc., is to become one of the contract drawings, you are requested to prepare it in accordance with the D.R.C. standards. To enable you to do this, we are enclosing a sample drawing with all the necessary explanations, together with a linen sheet for your drawing. You are also requested to provide the D.R.C. with a Cronaflex copy of the drawing.

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

WAT/MiaF  
Encls.(2)

Yours very truly,

cc: A. McCombie  
A. Gater  
H. C. Gernier  
J. Roy  
H. D. Smith (2)  
Mrs. T. Tate

  
A. Butka,  
MATERIALS & RESEARCH ENGINEER

Foundations Office  
Gen. Files

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

Mr. A. G. Stermac,  
Principal Foundation Engr.,  
Foundation Section,  
Materials & Research Division.  
October 9, 1963.

Attention: Mr. S. McCombie

FOUNDATION INVESTIGATION REPORT BY -  
William A. Trow & Associates Ltd. -  
Proposed Crossing over Young Creek,  
Dev. Rd. #684, Port Ryerse, Ontario.  
W.P. 701-62

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We have reviewed the report and herewith submit our comments for your consideration.

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The proposed footing elevation for the barrel arch culvert is at 594.0 which would allow for somewhat variable soil below the footings. The variation would be in density as well as nature of the subsoil. It would be therefore justified to assume that when the full load is applied some differential settlements will result.

Since in the case of the barrel arch the structure is completed before the main load (resulting from the embankment) is applied any differential movement is detrimental irrespective whether it results from the immediate or consolidation settlement. It is therefore our recommendation that consideration be also given to other types of structures.

The Consultant has pointed out that some construction difficulties can be expected mainly connected with excavations and related dewatering. Some of the subsoil materials are very prone to boiling if subject to excessive hydraulic heads.

The Consultant's proposal to build the structure in an open excavation of which the sides should always be kept deeper and from which pumping is to be carried out seem to be practical and feasible. However, because of the nature of the subsoil difficulties could be expected and this should be clearly stated in the Contract documents. The importance of always pumping from trenches that are lower than the rest of the excavation cannot be overemphasized.

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

AGS/tt  
Attach.

*A. G. Stermac,*  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. H. A. Tregaskes  
H. D. McMillan  
A. Gater  
H. C. Dernier  
J. R. Roy  
A. Watt

Foundations Office  
Gen. Files

## MEMORANDUM

To: Mr. A. Stermac,  
Principal Foundation Eng.,  
Room 107,  
Lab. building.

FROM: N. Zoltay

Attention: Mr. N.D. Smith

DATE: January 23, 1964.

OUR FILE REF.

IN REPLY TO

SUBJECT: Bridge Site # 21-144  
W.P. 701-62  
Young's Creek Bridge  
Dev. Rd. # 684 Dist. #2

We are sending to you two copies of our preliminary proposal D-5443-P1 for the above structure.

Our previous proposal for this structure was an arch type culvert. However it is now believed that by using steeper approach slopes a bridge type structure will be more suitable.

We will be pleased to have your views regarding the foundations for this structure.

*N. Zoltay*

NZ/kd

c.c. J. Walter  
S. McCombie  
G. Scott  
N.D. Smith  
M. Gvildys  
R. Fitzgibbon

N. Zoltay  
for G. Scott  
Bridge Location Engineer.

Mr. G. Scott,  
Bridge Location Engineer,  
Bridge Division.

Attention: Mr. N. Zoltay

Mr. A. G. Stermac,  
Principal Foundation Engr.,  
Foundation Section,  
Materials & Research Division.

January 28, 1964

Your Memo - Jan. 23/64.

Bridge Site #21-144  
W.P. 701-62  
Young's Creek Bridge  
Dev. Rd. #684, Dist. #2.

We have received your memo of January 23, 1964, with the enclosed two copies of your preliminary proposal drawing D-5443-P1 for the above structure.

The proposed solution seems much better suited for the particular subsoil conditions than the previously suggested arch culvert.

There are basically two types of piles that could be used to support the footings: end-bearing piles driven down to limestone bedrock, or friction piles driven down to approx. elevation 570.0.

Steel H-piles (12-BP-53) could be loaded up to 70 tons/pile if driven to refusal on bedrock.

Friction piles, either steel tube piles or timber piles, could be loaded up to 40 and 30 tons respectively, if driven to elevation 570.0. However, for friction piles, a pile loading test is suggested in order to establish the allowable load more reliably. It is felt that the same bearing load in the case of a friction pile could be achieved also by using an H-pile, but this would have to be substantiated by a field load test.

AGS/HdeP

cc: Foundations Office  
Gen. Files

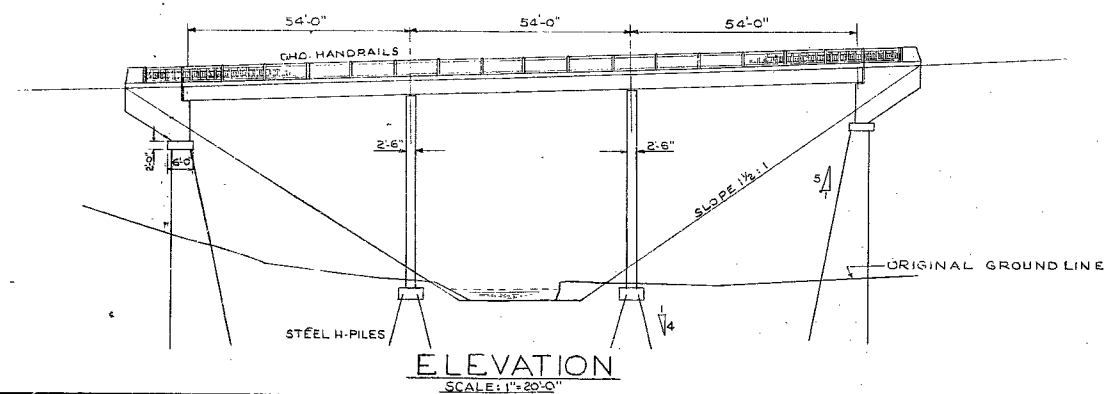
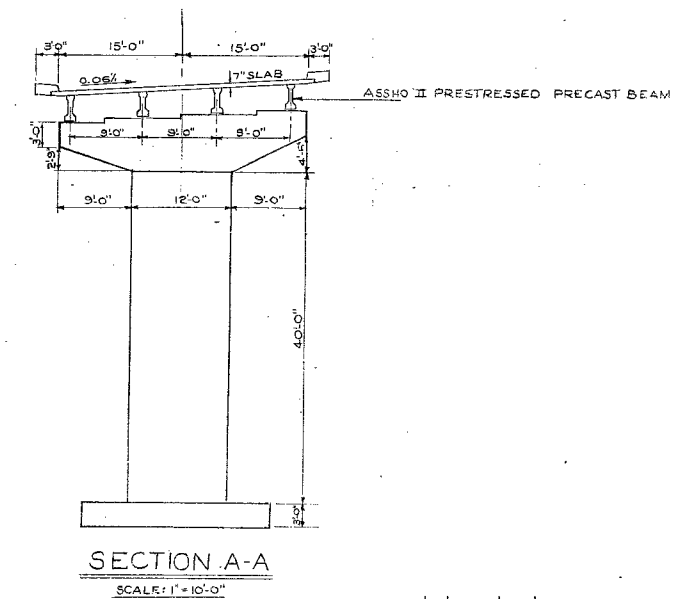
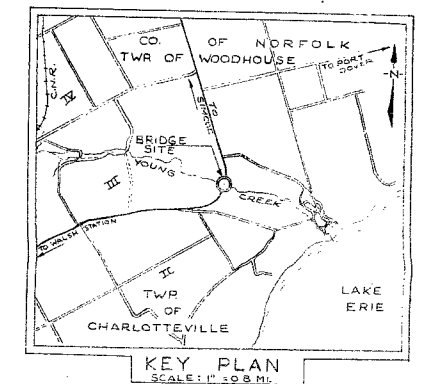
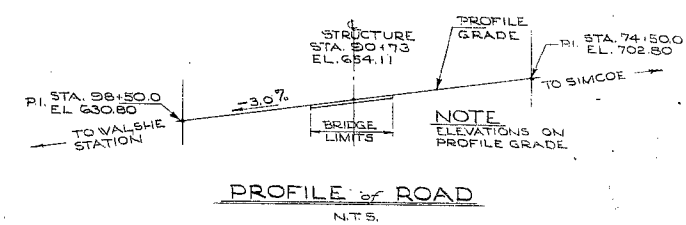
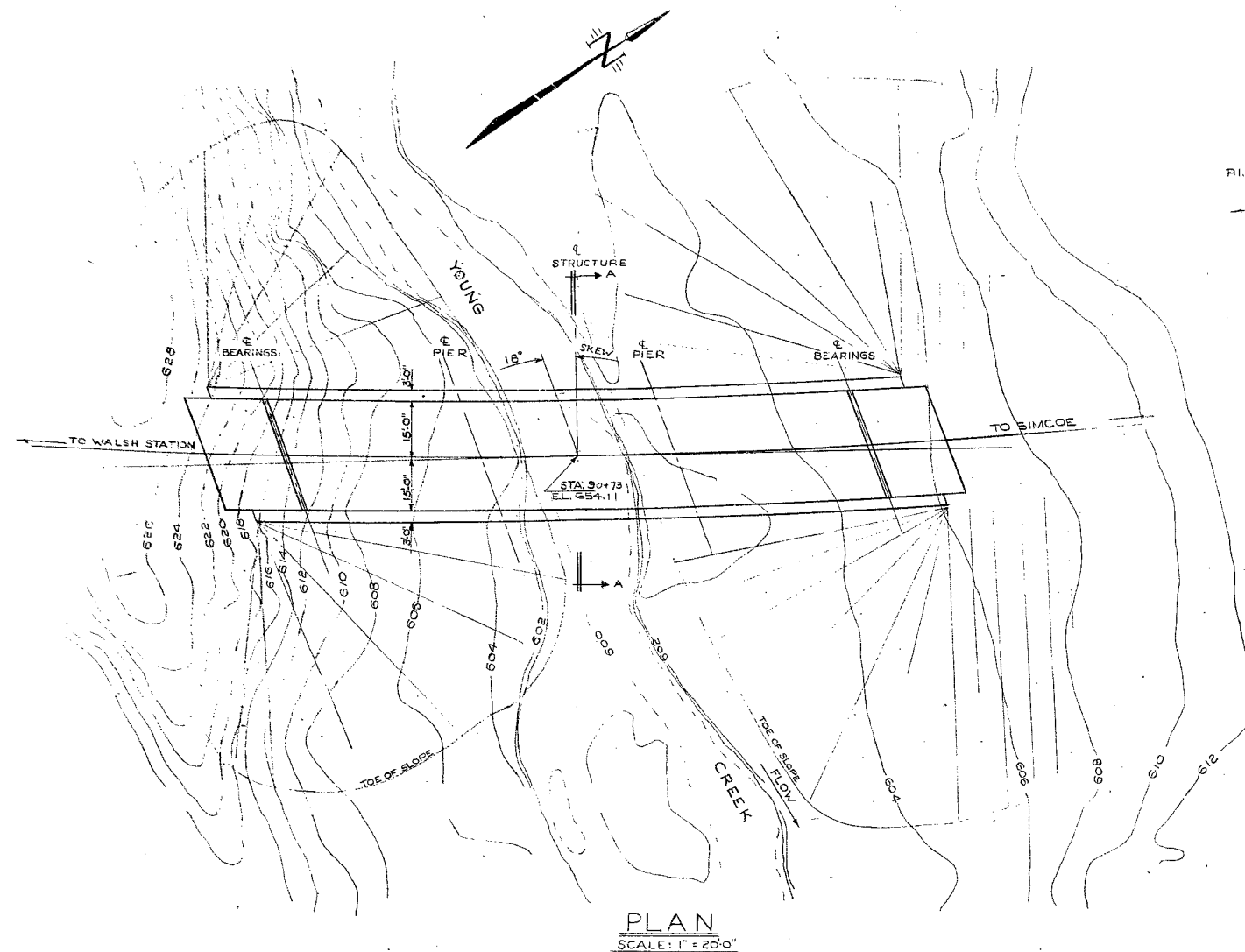
*A. G. Stermac*  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGINEER



# 63-F-215  
W.P.# 701-62  
YOUNG CREEK  
& DEV. RD. 684  
X-ING  
PORT RYERSE

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REVISED									
DATE	BY								
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Soul

**DEPARTMENT OF HIGHWAYS ONTARIO**  
BRIDGE DIVISION

YOUNG CREEK BRIDGE

KING'S HIGHWAY No. PROPOSED DEVELOPMENT RD. 684 DIST. No. 2

CO.

TWP. WOODHOUSE

LOT 1

CON.

PRELIMINARY PLAN

APPROVED \_\_\_\_\_

L.D.S. ENGINEER

SITE No.

21-144

W.P. No.

701-62

CONTRACT  
Nos.

DRAWING

R.H.

CHECK

DATE

JAN-64

LOADING

DRAWING  
No.

D-5443-P1