

#62-F-326 M

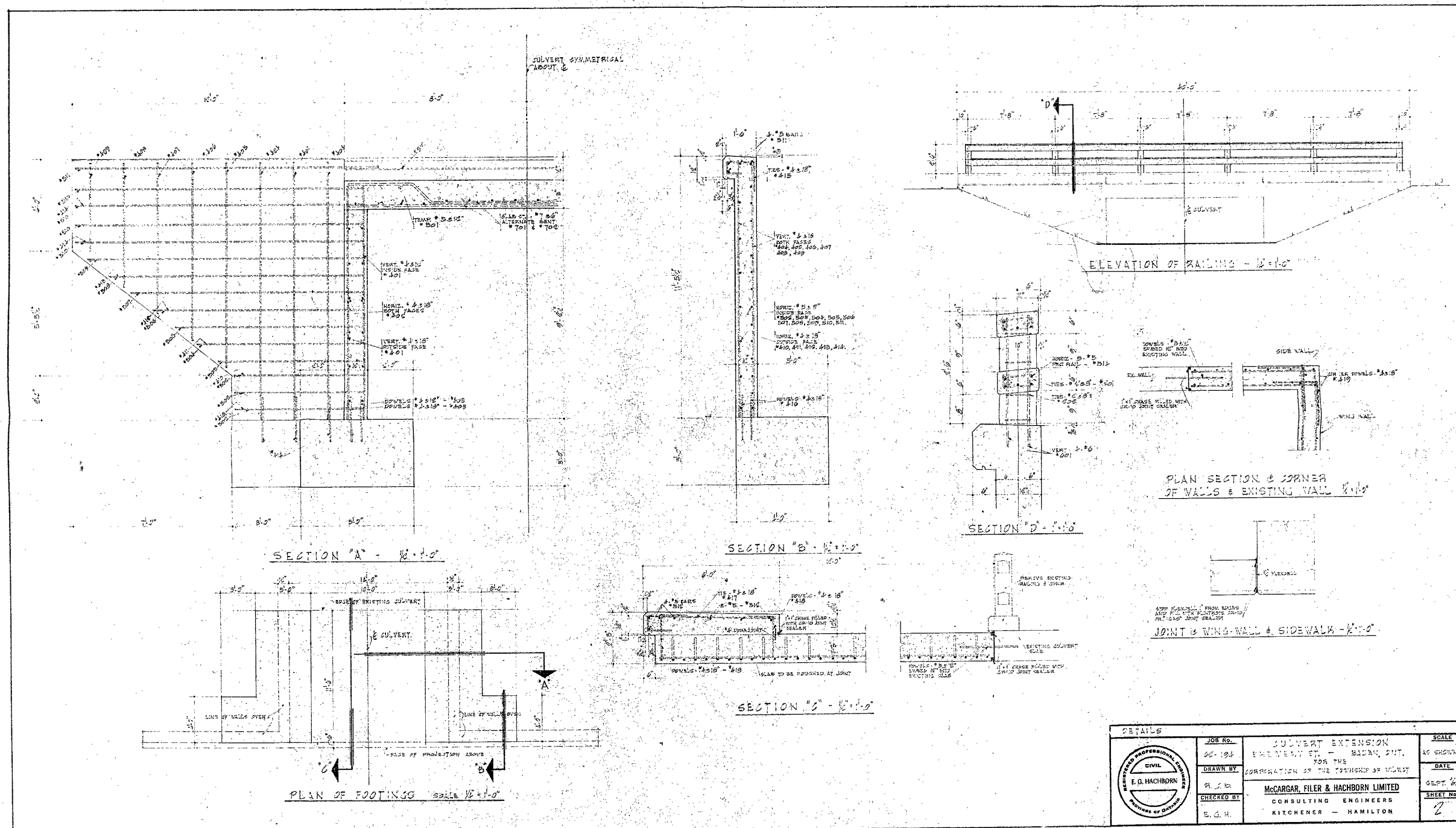
BRIDGE

BREWERY STREET

BADEN VILLAGE

WILMOT TWP.





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

Attn: Mr. K. L. Kleinstelber,  
Mun. Bridge Liaison Engr.

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

June 4, 1963

Bridge Widening on Brewery Street -  
Village of Baden, Twp. of Wilmot, Ont.  
(Bridge Office Ref. BA 1504)

In response to your verbal request of May 31st, the undersigned visited the above-mentioned site on June 3rd. The purpose of the visit was to establish, as reliably as possible, the causes that led to the interruption of work by the contractor, and to make recommendations for further action.

At the time of the visit, the excavation for the new footings was completely filled with water and, therefore, not much could be observed. The water level in the excavation corresponded to the creek level. The creek is presently being diverted through a corrugated pipe placed to the south of the existing bridge.

Difficulties were experienced during the excavation for the north abutment, and the bridge designer, Mr. Hachborn of McCargar, Filer and Hachborn, requested on May 27th, Dominion Soil Investigation, Ltd. to send their representative to review the situation. Mr. L. S. Rolke of Dominion Soil, who did the original investigation, has summarized his findings and recommendations in a letter to McCargar and Assoc., dated May 30, 1963.

According to this letter and the verbal information obtained by the undersigned from Mr. Hachborn, the contractor has, under the protection of the temporary earth dyke, completed the excavation for the south footing without encountering any difficulty. No boils were observed anywhere within the excavation.

After completing the excavation, which was done very quickly, the forms were lowered into the excavation and the concrete poured immediately.

However, in the north excavation, 'quick' conditions were encountered. According to eyewitnesses, many boils have developed, resulting in a complete failure of the bottom of the excavation. That made it impossible for the contractor to proceed, and after some futile attempts, the contractor finally gave up and temporarily pulled out from the site.

Mr. A. M. Toye,  
Bridge Division.  
Attn: Mr. K. L. Kleinsteinber

- 2 -

June 4, 1963

Everything that happened was to be expected because the recommendations contained in the foundation report were completely ignored. When the foundation report was reviewed in this Section, a special paragraph concerning the dewatering, was put in the covering memo of October 2, 1962. It read as follows: "The Contractor's attention should be drawn to the need of dewatering, and also the need to apply correct dewatering techniques because, otherwise, a great deal of trouble and expense could result."

Because no attention was paid to either the dewatering recommendations or the above warning, a great deal of trouble and expense has, and will result.

The fact that the contractor was able to complete the south excavation without trouble, was due to the fact that a layer of clayey silt existed at this location. This is evidenced by borehole No. 1 of the foundation investigation. However, even at this location, we are of the opinion that the contractor was taking too great a risk and was really, "pushing his luck". On the north side, the material is pure silt and, therefore, the boiling occurred immediately.

It appears that due to the created boiling and quickening conditions, the subsoil under the existing structure has been somewhat disturbed and the structure has moved. According to Mr. Diamond, Reeve of the Township of Wilmot, a large void was created under the structure's north footing. It is not known whether the structure is founded on piles or on spread footings.

A number of ways exist to remedy the present situation and complete the construction. However, before certain alternatives can be considered, the extent of the damage to the subsoil would have to be known.

It appears to the undersigned that, in view of the facts mentioned earlier and, also, in view of the condition of the present structure, which is far from good, serious consideration should be given to the alternative of complete change of design. This would involve the demolition of the existing structure, and the placement of a flexible culvert at this location. According to Mr. Haehborn, a 6-ft. diameter culvert would be adequate from a hydrological point of view. If this alternative is adopted, we would recommend that three feet of material below the culvert be replaced with coarse gravel, thus forming a good bedding for the pipe.

The use of piles for the north abutment would be another alternative. However, sheet piling would have to be used for the dewatering prior to pouring the concrete for the foundation, therefore, making it quite costly. Besides, this alternative would

Mr. A. M. Foye,  
Bridge Division.  
Attn: Mr. K.L. Kleinsteinber.

- 3-

June 4, 1963

result in different types of footings being used for a rigid frame structure, which is certainly not to be desired.

If the present design has to be preserved, we would suggest that a field investigation be carried out with the earlier mentioned intention of determining the extent of damage to the subsoil.

It is believed that this memo covers the problem adequately. However, should you wish to discuss the matter further, please feel free to call on the Foundation Section.

AGG/KdeF

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

cc: Messrs. K. L. Kleinsteinber (4)  
H. A. Tregaskes  
L. F. Radie

Foundations Office ✓  
Gen. Files

COPY

DOMINION SOIL INVESTIGATION LIMITED  
77 CROCKFORD BOULEVARD SCARBOROUGH, ONTARIO TELEPHONE 421-2567

BRANCH  
362 QUEENS AVENUE  
LONDON, ONTARIO  
TELEPHONE GE. 3-3851



FOUNDATION ENGINEERS

P.O. BOX 933  
SAULT STE. MARIE  
ONTARIO  
TELEPHONE AL. 4-2615

Scarborough, Ontario,  
May 30th, 1963.

McCargar, Filer and Hachborn,  
Consulting Engineers,  
30 Francis Street South,  
KITCHENER, Ont.

Att'n: Mr. E.G.Hachborn, P.Eng.

Re: Inspection of Foundation Construction  
for proposed Bridge Extension in  
Baden, Ontario - Our Ref: 3-5-22

R E P O R T

Gentlemen:

The writer visited the above site on May 28th, 1963 as requested by Mr. Hachborn the previous day. This report presents the findings and concluded recommendations:

OBSERVATIONS

The field visit took place on a rainy day; consequently, no work was being done at the construction site. The footing grade proper was not visible because the excavation for the foundations was inundated with water. According to information obtained from Mr. Hachborn and Mr. Haufschild (Road Superintendent for the Township of Wilmot), the contractor excavated the south foundations in a pit protected by a temporary dike and owing to the relatively low amount of water seepage, he was able to pour the concrete of the footing. The uppermost 12" was not completed because, at this stage, the temporary dike caved in and the incoming water pushed the earth onto the top of the partially completed foundation.

The construction of the foundation on the north side was not possible without adequate measures for dewatering because of the high rate of water seepage into the excavation. Therefore, it was decided that another method of construction would be employed.

#### RECOMMENDATIONS

A soil testing was performed for the proposed bridge extension and in our report dated July 30th, 1962 (Ref: 2-7-3) we stated that dewatering difficulties would be encountered and therefore we advised the use of sheet piles. These would have served a twofold purpose: (i) confining the dewatering to a limited area; and (ii) prevent the loosening of subgrade by the upward seeping water.

In the case of the southern footing, the dewatering difficulty was overcome by pumping from the unprotected subgrade but we do not know how much the latter was loosened up by the seeping water. Therefore, we recommend that the loads on the foundation should be kept as low as possible and the last foot of concrete should be bound to the existing footing by dowels. The increasing of the height of the footing (and thereby the foundation pressure) should be avoided.

It is understood that for the northern foundation, the contractor decided to employ the construction method recommended in our report - that is - enclosing by sheet piles. We can only approve of this and we would suggest that:

- (i) the sheet piles should be driven to a sufficient depth below the footing grade (see report), and
- (ii) the subgrade should be rigorously checked before the footings are poured because it may already have been loosened by the upward seeping water.



Last - but not least - the construction should be done without any further delay because further difficulties may be encountered.

We trust that the foregoing will meet with your requirements but should you have any further questions, please do not hesitate to contact us.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED



L. S. Rolko, P.Eng., Ass. Member A.S.C.E.  
CHIEF SOILS ENGINEER.

LSR/oed

Mr. K. L. Kleinsteinber,  
Municipal Bridge Liaison Engr.,  
Bridge Division.

Attn: Mr. G.C.E. Burkhardt.

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

October 3, 1962.

Your Memo - October 2, 1962.

Township of Wilmot,  
Culvert Extension,  
Brewer Street, Baden, Ontario,  
Bridge Office Ref. # BA 1504.

We have reviewed the foundation report for the above-mentioned structure, and would like to submit the following comments for your consideration:

It is evident from the borehole logs that although the basic and predominant material seems to be the same (grey silt), its properties are quite different. The penetration resistances recorded in B.H. No. 1 are very low, down to approx. 20 feet below the ground surface, while in B.H. No. 2, the resistances are higher. Since the recorded resistances are the only reliable source of information on the properties for such type of soil, it has to be concluded that there is quite a difference in the material's densities.

We cannot agree with the Consultants' reasoning on the "liquefaction" of the dense silt under the influence of dynamic loading. Liquefaction could, indeed, occur but only if the silt would be very loose. In such a soil, collapse of the soil structure occurs under loading, dynamic or static, resulting in "liquefaction".

We believe, on the basis of the presented information, that the soil where Borehole No. 1 was put down, is, in fact, loose and we would therefore recommend that the allowable pressure be reduced to 1,500 p.s.f.

The Contractor's attention should be drawn to the need of dewatering, and also the need to apply correct dewatering techniques because, otherwise, a great deal of trouble and expense could result.

AGS/MdeF

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

cc: Foundations Office  
Gen. Files.

BA 1504

DOMINION SOIL INVESTIGATION LIMITED  
77 CROCKFORD BOULEVARD SCARBOROUGH, ONTARIO TELEPHONE 421-2567

BRANCH  
153 QUEENS AVENUE  
LONDON, ONTARIO  
TELEPHONE GE. 3-3851



FOUNDATION ENGINEERS

P.O. BOX 953  
SAULT STE. MARIE  
ONTARIO  
TELEPHONE AL. 4-2615

Township of Wilmot,  
BADEN,  
Ontario.

C.C. OF WATERLOO

Scarsborough, Ontario,  
July 30th, 1962.

Att'n: Mr. N. A. Haufschild,  
Road Superintendent.

62-F-326M

Re: Subsoil Conditions  
Culvert on Brewery St.,  
Baden, Ontario.  
Our Reference: 2-7-3

REPORT

Gentlemen:

The soil investigation has been completed for the above project in accordance with your letter of authorization (dated June 26, 1962). The results of the subsurface exploration together with our concluded recommendations are presented in this report.

Field Work

Field work was carried out during the period July 5th to July 7th and comprised two boreholes and two dynamic cone penetration tests at the locations shown on Enclosure #2. The positions of the test holes were set out on the site together with Mr. N.A. Haufschild. Elevations were measured relative to a nail driven into a hydro pole immediately south of the bridge (=el 100.0) - also indicated on Enclosure #2.

The boreholes were of 2 7/8 in. diameter. They were lined with Bx casing advanced to the required sampling depths by the repetitive procedure of alternately driving and washing.

Standard penetration tests were made at frequent intervals using a 2 in. outside diameter split spoon driven into the bottom of the clean borehole by a constant driving energy (140 pound hammer dropping 30 ins.). The dynamic cone penetration test is one type of deep sounding in which the Bx rods with a 2 in. diameter 60 degree apex cone driving point are driven into the subsoil without casing and applying the same driving energy as above. The former test provided disturbed samples of the substrata indicating their relative density and consistency and the latter a continuous record of soil density.

Samples were also taken with 2 in. diameter thin walled tubes forced into the subsoil in one rapid continuous movement. However, owing to the nature of the material, the specimens were disturbed to a high degree.

In addition to the split spoon and thin walled tube samples, some subsurface material removed from the withdrawn casing and brought up by the washwater was also preserved.

The stratification of the subsoil, sampling depths and the results of the penetration tests are recorded on geotechnical data sheets comprising Enclosures #3 and #4 inclusive.

### Laboratory Work

The samples were shipped to our modern laboratory where they were thoroughly examined and classified. The most representative specimens and of primary importance considering the requirements of the present project, were subjected to a more detailed analysis in the laboratory. The common purpose of all experiments was to determine the geotechnical properties of the subsoil by the most accurate and up-to-date testing methods.

The laboratory test results are listed below and they are also recorded graphically on the corresponding geotechnical data sheets.

- (i) A sieve analysis was performed on B.H.#2, Sa. #5 (approx. elevation 88 ft.) and it was found that 96.9% of the material passed the No. 200 sieve and 100% passed the No. 100 sieve. Owing to the fact that the soil possesses no cohesion, it can be called a pure silt. (The grain size analysis was not continued below the No. 200 sieve size because it was not believed to be of practical importance.
- (ii) A relatively undisturbed sample was taken from the same silt. Its unit weight is 138 pcf with a natural moisture content of 18%.
- (iii) A sample from the clayey silt (B.H. #1, Sa. #4 (elevation 90 ft.) was also tested:

Liquid Limit	36.3
Plastic Limit	15.6
Water Content	24.3
Plasticity Index	20.7
Liquidity Index	0.25

The above values are characteristic to a firm, clayey silt material.

#### Subsurface Conditions

Marshy conditions prevail at the site, hence peat was encountered first. This material is not capable of supporting any load, has a high moisture content and it is very compressible.

In Borehole #1, firm, brown, clayey silt was encountered. This stratum is missing from Borehole #2 where the boggy conditions are more advanced.

A greyish, compact silt with layers of firm, grey, silty clay was found below Elevation 88 ft.(average). The material has no plasticity but the grains are finer than those of the finest sand. It is very sensitive to vibration, in fact, it turns into a "heavy liquid" when dynamic loading is applied on it. This is the reason why both dynamic tests (cone and split spoon) met little resistance. The soil, however, as proven by the high unit weight and low moisture content, has a low void ratio - i.e. it is compact.

The soil turns into a fine, dense sand around Elevation 75 ft.

The ground water level corresponds to that in the creek (Elevation 93.9 ft.).

#### Discussion and recommendations

It is understood that the existing bridge will be extended to the east and that it will not carry heavy traffic regularly. Our recommendations are based on this information. (No information was obtainable regarding how the existing structure is founded).

The grey, compact silt should serve as foundation material and an allowable bearing pressure of 2000 psf is recommended. Differential settlements may be experienced; therefore, either a statically

determinate structure should be designed or a rigid, box-type culvert which is insensitive to slight movements.

The construction of the foundations will present some difficulties. These, however, may be overcome if the procedure recommended below is followed:

The creek should be diverted and as much of the surface waters drained elsewhere as possible. Then, before starting the excavations, sheet piles should enclose the working area (timber sheet piles should be satisfactory) driven to a depth below the proposed footing grade to a distance at least equal to the distance between the outside water level and the footing grade. (e.g. the water table is at Elevation 94 ft., the proposed footing grade - Elevation 86 ft. - then sheet piles should be driven to Elevation 78 ft.). This measure would prevent the hydraulic failure of the footing grade. The seepage water can then be removed from the enclosed excavation pit and the footing grade constructed. As soon as the proposed base level has been arrived at, a lean concrete blanket should be spread over the bottom of the hole, which would make the work in the pit easier.

We wish to call attention to the following:

- (i) Pumping from an unprotected excavation (i.e. without enclosing by sheet piles) should not be allowed. These types of soils are extremely sensitive to the slightest disturbance, and the moving water (mobilized by pumping) would soon erode the bottom of the excavation, reduce its bearing capacity and increase the expectable settlements.

- (ii) The stability of the existing structure should be protected. No excavation should be done below the level of its foundations because the subsoil may be loosened and detrimental settlements may occur.

Consideration was given to dewatering by vacuum wellpoints. The success of this method may be dubious owing to the presence of clayey layers. Therefore, the following procedure is recommended:

The costs of dewatering in advance by wellpoints should be compared to the costs of the above described method (enclosing by sheet piles). If they are roughly the same, or the sheet piling does not cost much more than the wellpoints, the sheet piling should be adopted as a certainly successful dewatering method. Should, however, the sheet piling be significantly more expensive, the wellpoint dewatering method should be tried but with the thought in mind that the possibility exists that they will not be effective and that sheet piling will still be necessary to dewater the pits.

We also mention here that pile foundations do not seem to be feasible under the present circumstances because of the following two reasons:

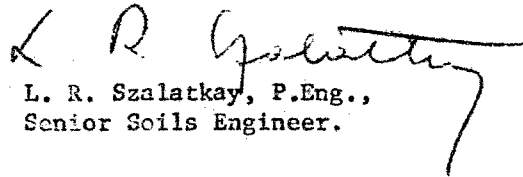
- (a) The pile driving operations may cause liquification of the subsoil under the existing bridge and if the existing bridge rests on spread footings, large settlements might be experienced.
- (b) The required length of piles would be in the order of 40 ft., which would make this foundation method very expensive.



We are glad to have had this opportunity of being of service to you and should any questions arise in connection with the report, please do not hesitate to contact us.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

  
L. R. Szalatnay, P.Eng.,  
Senior Soils Engineer.

LRS/oed

Enclosures

# LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

## SOIL COMPONENTS AND GROUND WATER CONDITIONS.

BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY	ORGANICS	BEDROCK	GROUND WATER LEVEL	DEPTH OF CAVE-IN
		COARSE	FINE	COARSE	MEDIUM	FINE						
Ø	> 8"	3"	3/4"	4.76mm	2.0	0.42	0.074	0.002	>	NO SIZE LIMIT		
U.S. Standard Sieve Size :				No. 4	No. 10	No. 40	No. 200					

## SAMPLE TYPES.

AS Auger sample

CS Sample from casing

ChS Chunk sample

RC Rock core

% Recovery

SS Split spoon sample

TP Piston, thin walled tube sample

TW Open, thin walled tube sample

WS Wash sample

SAMPLER ADVANCED BY static weight : w  
 " pressure : p  
 " tapping : t

OBSERVATIONS  
 MADE WHILE  
 CORING

Steady pressure  
 No pressure  
 Intermittent pressure

Washwater returns  
 Washwater lost

## PENETRATION RESISTANCES.

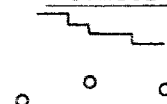
DYNAMIC PENETRATION RESISTANCE : to drive a 2"Ø, 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot.

STANDARD PENETRATION RESISTANCE, -N- : to drive a 2" outside dia, split spoon sampler 1 foot into the ground, expressed in blows per foot.

### EXTRAPOLATED -N- VALUE

The energy for the penetration resistances is supplied by a 140 lb. hammer falling 30 inches

SYMBOL :



322

## SOIL PROPERTIES.

W % Water content

LL % Liquid limit

PL % Plastic limit

PI % Plasticity index

LI Liquidity index

 $\delta^*$ 

Natural bulk density (unit weight)

e

Void ratio

RD

Relative density

C<sub>v</sub>

Coeff. of consolidation

m<sub>v</sub>

Coeff. of volume compressibility

k Coeff. of permeability

C Shear strength — in terms of total stress

 $\phi$ 

Angle of int. friction

C'

Cohesion — in terms of effective stress

 $\phi'$ 

Angle of int. friction

## UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

TRIAXIAL

UNCONFINED

LABORATORY

FIELD

COMPRESSION TEST

St

VANE TEST

St

POCKET PENETROMETER TEST

St

Strain at failure is represented  
 by direction of stem

20%  
 15% — 5%  
 10%

St : sensitivity =  $\frac{\text{shear strength in undisturbed state}}{\text{shear strength in remoulded state}}$

## SOIL DESCRIPTION.

COHESIONLESS SOILS :

RD :

COHESIVE SOILS :

c lbs/sq.ft.

Very loose

0 - 15 %

Very soft

less than 250

Loose

15 - 35 %

Soft

250 - 500

Compact

35 - 65 %

Firm

500 - 1000

Dense

65 - 85 %

Stiff

1000 - 2000

Very dense

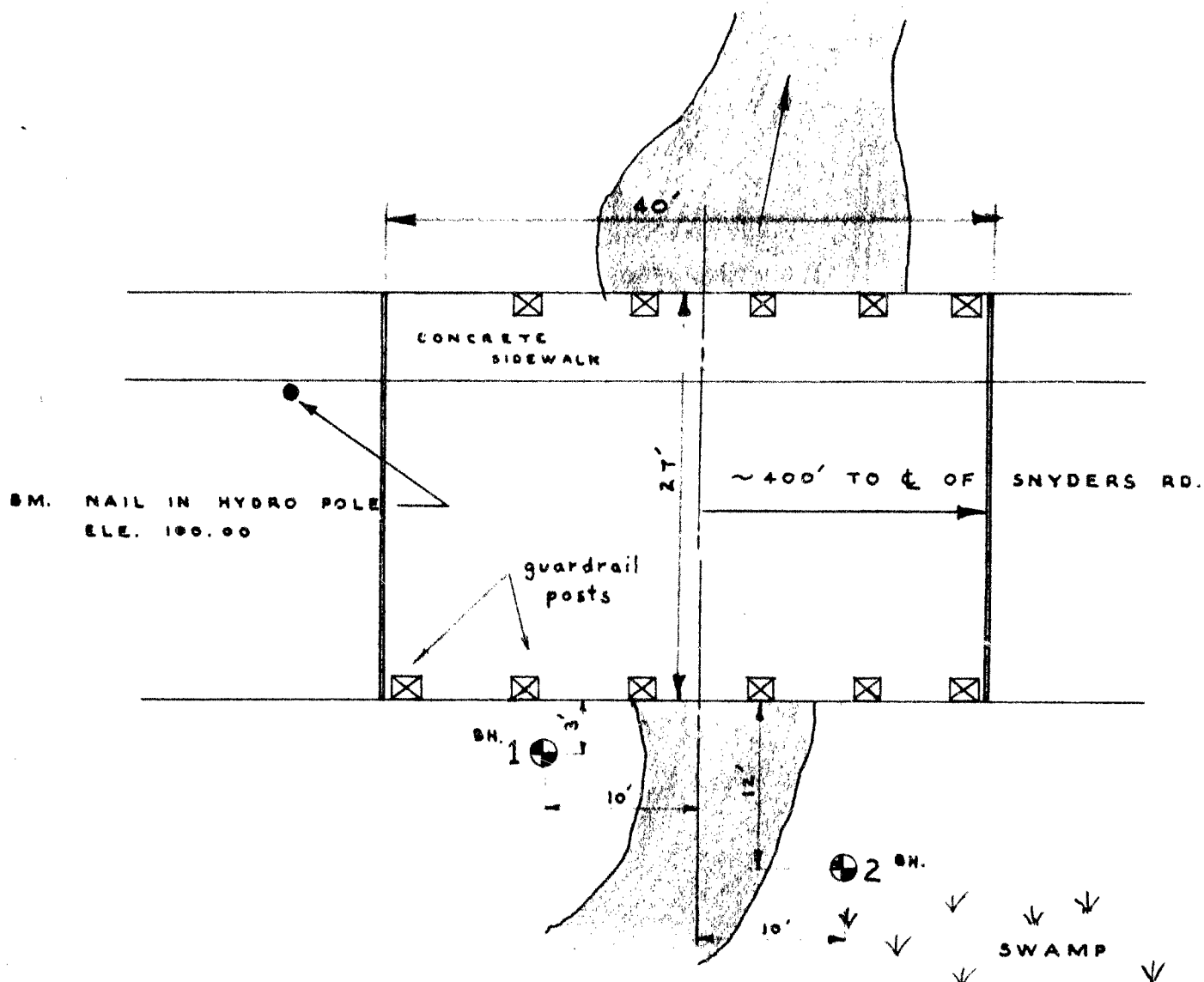
85 - 100 %

Very stiff

2000 - 4000

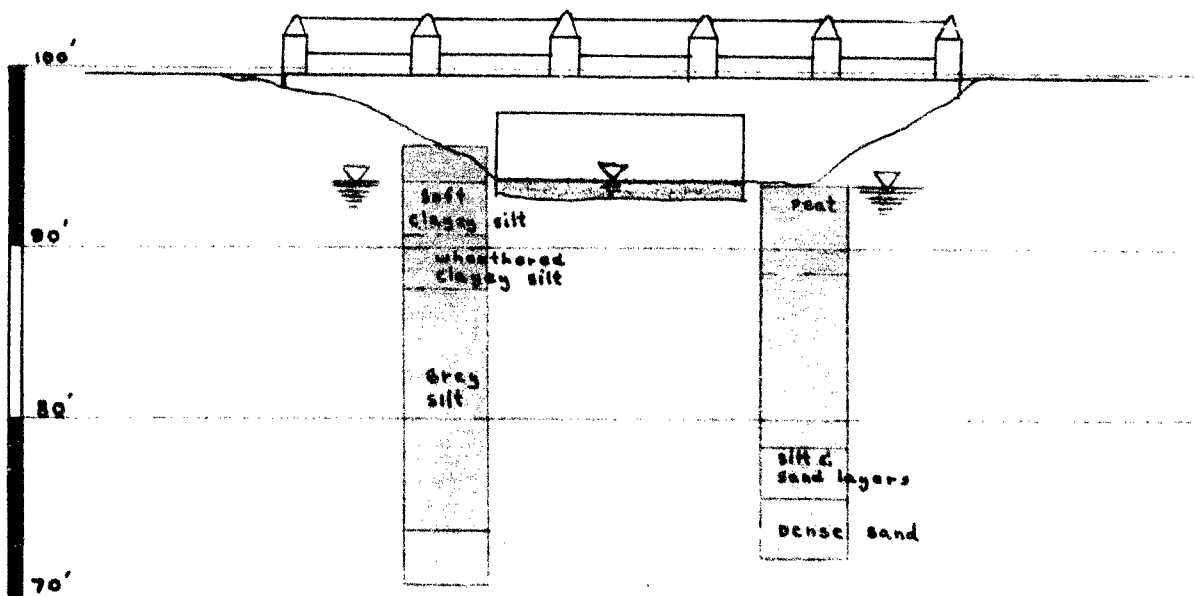
Hard

over 4000



### LOCATION OF BOREHOLES

SCALE: 1" TO 10'



### SUBSURFACE PROFILE

SCALE: 1" TO 10'

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 1 . . . .

OUR REFERENCE NO. 2-7-3

CLIENT: TOWNSHIP OF WILMOT  
PROJECT: CULVERT IN BADEN  
LOCATION: BADEN, ONTARIO  
DATUM ELEVATION: 95.30

METHOD OF BORING: WASHBORING  
DIAMETER OF BOREHOLE: 2 3/8"  
DATE: JULY 6, 1962

ENCLOSURE NO 3

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE		CONSISTENCY		REMARKS
				NUMBER	TYPE	N or Advancement of Sampler	blows per foot	SHEAR STRENGTH lbs/sq ft	water content % PL W LI		
95.5	0	PEAT MIXED WITH GRAVEL FILL		1	C.S.	—					A T.W. SAMPLE WAS OBTAINED FROM 5' TO 7' DEPTH, ABOUT 3' AWAY FROM THE BOREHOLE.
		GREY SOFT CLAYEY SILT WITH ORGANIC REMAINS.		2	SS	2					
				3	SS	4					
90	5	BROWN SOFT TO FIRM, WEATHERED CLAYEY SILT		4	SS	3					
				5	SS	5					
85	10	COMPACT GREY SILT WITH LAYERS OF FIRM GREY SILTY CLAY		6	SS	2					
				7	SS	2					
80	15			8	SS	0					
75	20	increasing sand content		9	SS	10					
		GREY DENSE SAND		10	WS	—					
70	25										FOR NOTATIONS SEE ENC. 1
65	30										
60	35										
55	40										

OUR REFERENCE NO. 2-7-3

METHODE OF BORING WASH BORING  
DIAMETER OF BOREHOLE: 2 7/8"  
DATE: JULY 7, 1962

ENCLOSURE NO. 4.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES	PENETRATION RESISTANCE blows per foot	CONSISTENCY water content % PL W LI	REMARKS
				NUMBER TYPE N or Advance ment of Sampler	SHEAR STRENGTH lbs./sq ft		
93.8	0	PEAT WITH SOFT CLAYEY SILT	[Symbol]	1 SS 0			A TW SAMPLE WAS OBTAINED FROM 5' TO 7' DEPTH, S EAST OF THE BOREHOLE
				2 SS 2			
90-				3 SS 2			
5		GREYISH COMPACT SILT	[Symbol]	4 SS 8			ANOTHER TW SAMPLE WAS OBTAINED FROM 5' TO 7' DEPTH 3' NORTH OF THE BOREHOLE
85-	10			5 TW W			
				6 SS 16			
80-	15	LAYERS OF COMPACT SILT & SAND	[Symbol]	7 TW W			
				8 SS 20			
75-	20	FINE DENSE SAND	[Symbol]	9 SS 17			
	25						
70-							
65-	30						