MESSRS. C. G. RUSSELL ARMSTRONG
CONSULTING ENGINEERS
BARTLET BUILDING
WINDSOR ONTARIO

Report on

SOIL INVESTIGATION

for

ROAD BRIDGE

LOT 17, CONCESSIONS II & III

TOWNSHIP OF COLCHESTER SOUTH

64-F-150 M

by

DOMINION SOIL INVESTIGATION LIMITED
363 Queens Avenue
LONDON ONTARIO
Reference No. 4-1-L3
January 1964

CONTENTS

ENCLOSURES

SUMMARY

The strata consist of cohesive fill and natural till to El.82 feet (±). These strata are underlain by a compact to very dense deposit of fine silty sand which extends at least to El.52 feet.

Several types of foundation design are technically feasible, and the choice will be governed by cost. Timber piles are not suitable because they are unlikely to achieve the required penetration without damage. Steel pipe or steel H-sections are suitable, although the latter will penetrate considerably deeper before achieving the same bearing capacity.

Spread footing foundations are also feasible provided that suitable dewatering procedures are employed.

The construction problems associated with each type of design are discussed in the text.

I INTRODUCTION

Verbal authorization was received from Mr. E. O. LaFontaine to carry out a soil investigation at a site in the township of Colchester South where it is proposed to replace an existing road bridge with a new structure. The new bridge will be of freely-supported beam design with a span of 65 feet.

A survey plan was provided showing the outlines of the existing and proposed new bridges, and the approximate positions of two boreholes. It was indicated that preliminary borings had revealed the presence of wet sand strata, and that some form of piled foundation was being considered.

The purpose of this investigation has been to reveal the subsurface conditions and to determine the necessary soil properties for the design and construction of foundations.

II FIELD WORK

Field work was carried out during the period 6th to 8th of January 1964 and consisted of 2 boreholes at the locations shown on enclosure 2. The holes were advanced by wash-boring and lined with Bx (3-inch) casing.

Standard Penetration tests were performed at frequent intervals of depth to determine the relative density or consistency of the soil and to recover disturbed samples. Dynamic cone penetration tests were performed adjacent to each borehole. These tests provide a continuous record of penetration resistance and give a qualitative indication of the resistance which may be encountered during pile driving.

The results of the field tests are recorded on geotechnical data sheets comprising enclosures 3 and 4. Elevations have been referred to a local datum (top of North curb at ℄ of existing bridge, assumed El.94.05 feet).

III SUBSURFACE CONDITIONS

Details of the stratification at each borehole are shown on the data sheets and a general picture of the subsurface conditions is given by the profile on enclosure 2.

Both boreholes were advanced from or near the top of the existing road embankment, and penetrated through cohesive, mainly unsaturated soil to an average elevation of 82.2 feet. This material is a sandy clayey silt till, containing only a trace of gravel. It is of firm to stiff consistency. The upper layers have been remoulded and constitute the embankment fill, made up of local natural material.

Throughout the remaining depth explored by the boreholes, the soil is a fine, dark, grey, silty, cohesionless sand. The colour comes from the presence of finely divided organic matter which is distributed throughout the stratum. The quantity of organics is insufficient to affect the compressibility of the deposit; on the contrary the density increases rapidly with depth as demonstrated by the cone penetration tests and consequently the compressibility is estimated to be quite low.

IV FOUNDATIONS

The bed of the creek extends to El.79 feet which is several feet below the surface of the cohesionless sand stratum. Consideration of erosion in such conditions is important, and the highest safe elevation for a footing foundation would be about El.72 or 73 feet, subject to hydrological considerations. In these circumstrances several types of foundations have been considered and will be discussed in turn.

(a) Timber piles

To provide sufficient protection against erosion, it is estimated that the tips of the piles should penetrate at least to El.65 feet. In the very dense sand stratum it would probably be impossible to drive such piles much below El.75 without damaging them. It will be noted that the 2-inch diameter cones met refusal at elevations 73 and 77 in boreholes 1 and 2 respectively. The fibre stress during the driving of the piles would be excessive and it is concluded that this type of construction is unlikely to be satisfactory.

(b) Steel pipe piles

On the basis of the field penetration tests and the inferred relative density of the soil, it is estimated that 10-inch diameter steel pipe piles designed for a safe working load of 60 tons per pile will reach a satisfactory set at El.70. The required depth of penetration of the pile tips should be determined by a hydrological study, but in the absence of such a study it is recommended that the tips should penetrate to El.65. No difficulty is anticipated in driving closed-ended piles to this level, but should difficulty be encountered, the piles can be driven open-ended.

If it is advantageous to use 100-ton piles, a 12-inch diameter section should reach a satisfactory set at approximately the same level.

(c) Steel H-piles

The ultimate bearing capacity of steel H-sections has been calculated for piles driven to Els. 65 and 55 with the following results:

Section	Theoretical ultimate bearing capacity (tons) El.65	El.55
BP10 x 57 lb./ft.	60	93
BP12 x 74 lb./ft.	75	117

A factor of safety of at least 2 should be applied to these figures. While the theoretical estimate of pile depths is inherently crude and inaccurate, particularly with H-sections, the foregoing figures indicate that the length of H-piles required to achieve a safe unit load is considerably greater than with pipe piles. The choice between these types will be governed by economy, taking into account the availability of equipment.

Irrespective of any theoretical prediction made here as to the required length of driven piles, the piles should be driven to a satisfactory set in accordance with an established dynamic pile driving formula such as the Hiley formula, no matter at what elevation the set may be achieved.

It is further recommended that at least one pile loading test should be performed.

The settlement under load of the pile groups, provided they are driven to the required set, will not be of significant magnitude in a freely-supported structure.

Dewatering of the excavation for the pile caps may be a difficult process unless steel sheeting is driven to such an elevation that it prevents the sand from boiling. A less expensive but less reliable procedure would be to drive timber sheeting as far as possible into the dense sand, and to dewater with high-volume pumps. Speed of excavation of the work will help to reduce the problem. It may be necessary to re-drive the sheeting if boiling occurs.

(d) Footing foundations

The use of spread footing foundations is also feasible, providing that proper dewatering of the cohesionless sand stratum is carried out. The expense of the dewatering procedure should be weighed against the cost of a piled foundation.

Considering first the bearing capacity of the undisturbed soil, bearing pressures which will limit settlement to a tolerable value (1 to 1-1/2 inches) are:

El.75 to 70	12,000 p.s.f.
Below El.70	20,000 p.s.f.

Settlement would be immediate upon the application of load, and there would be no long term settlement. However, in view of the construction difficulties associated with dewatering the cohesionless sand stratum and the likelihood of some disturbance, it is recommended that the *gross* soil pressure should not exceed *5000 p.s.f.*, a value which should be adequate for the size of the structure proposed.

Careful dewatering of the excavation is of paramount importance and two methods are available. Well-points will operate satisfactorily in this type of material and are capable of lowering the water by the required 10 to 15 feet. The pumping yield will be high. Excavations into the dewatered soil should be braced or sloped at 1 to 1. It is important, to prevent disturbance, that the dewatering should *precede* the excavation, and that the water table should be maintained at the depressed level until after the footings have been poured and the concrete has set.

It is recommended that the abutments should be protected from erosion by light-gauge sheet piling driven to El.68 or deeper.

An alternative method of dewatering would be to construct each footing within a steel sheet pile enclosure, where the piles are driven to such a depth that the excavation can be pumped dry without heaving. This depth is determined by the level of the prevailing water table. The distance from the pile tip to the footing grade should be about 2 feet more than the distance from the grade to the water table. For a water level of El.84.6 feet and a grade level of El.73 feet, the required level of the pile tips would thus be El.59.4 feet.

If the sheet piling is left in position permanently for protection against erosion, the soil pressure may be increased to 8000 p.s.f. If the piling is withdrawn, then shallow light-gauge piling should be installed for erosion protection.

Both of the methods outlined above are technically feasible. The choice between them, or the choice of a piled foundation is entirely a matter of economy and the availability of suitable equipment.

V REFERENCES

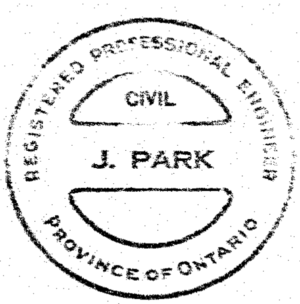
1. The Physiography of Southern Ontario by L. J. Chapman and D. F. Putnam of the Ontario Research Foundation - University of Toronto Press 1951.

2. Procedures for Testing Soils, ASTM, April 1958. pp.186 to 198. (Unified Soil Classification System - by A. A. Wagner).

3. Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering (Research on Determining the Density of Sands by Spoon Penetration Testing - by H. J. Gibbs and W. G. Holtz of the United States Bureau of Reclamation.) London, 1957.

4. Terzaghi and Peck: Soil Mechanics in Engineering Practice. John Wiley and Sons, New York 1948.

5. Standard Penetration Tests and Bearing Capacity of Cohesionless Soils, by G. G. Meyerhof, ASCE Paper 866, January 1956.



DOMINION SOIL INVESTIGATION LIMITED

James Park

James Park, M.Sc.,P.Eng.

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.

SYMBOL:

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 322

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

SOIL PROPERTIES.

W %	Water content	γ	Natural bulk density (unit weight)	k	Coeff. of permeability	
LL %	Liquid limit	e	Void ratio	C	Shear strength	in terms of total stress
PL %	Plastic limit	RD	Relative density	ϕ	Angle of int. friction	in terms of total stress
PI %	Plasticity index	C_v	Coeff. of consolidation	C'	Cohesion	in terms of effective stress
LI	Liquidity index	m_v	Coeff. of volume compressibility	ϕ'	Angle of int. friction	in terms of effective stress

UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

TRIAXIAL COMPRESSION TEST
UNCONFINED COMPRESSION TEST St
LABORATORY VANE TEST XSt
FIELD VANE TEST +St
POCKET PENETROMETER TEST St

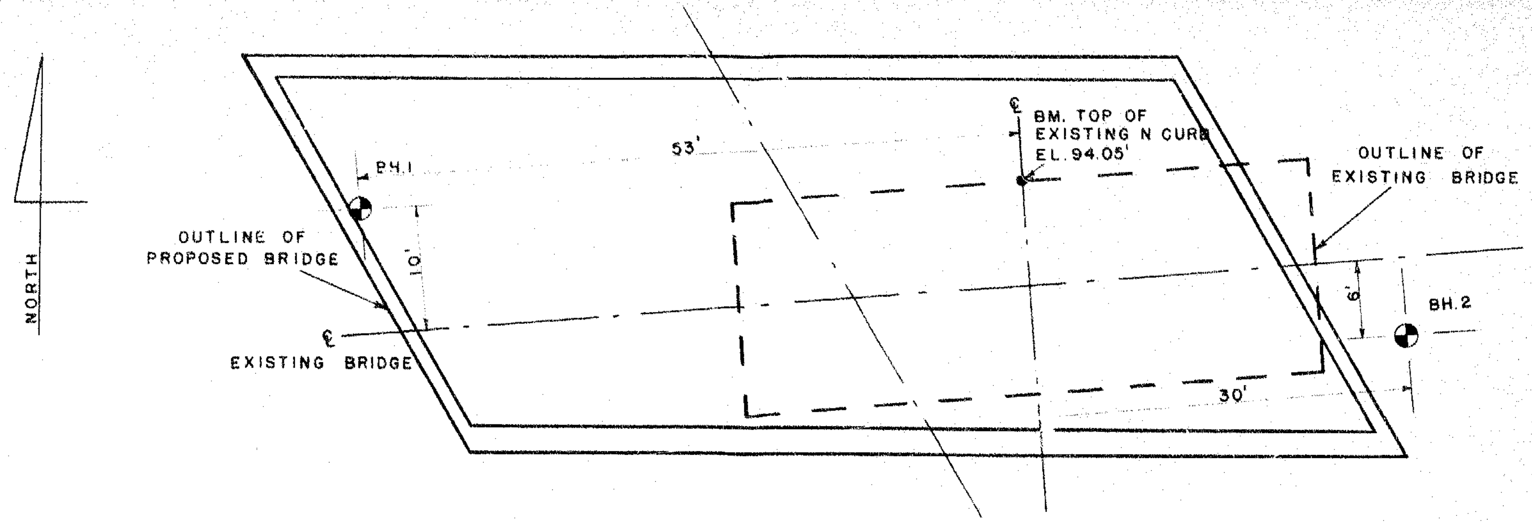
Strain at failure is represented by direction of stem: 20% (up), 5% (right), 10% (down), 15% (left)

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

SOIL DESCRIPTION.

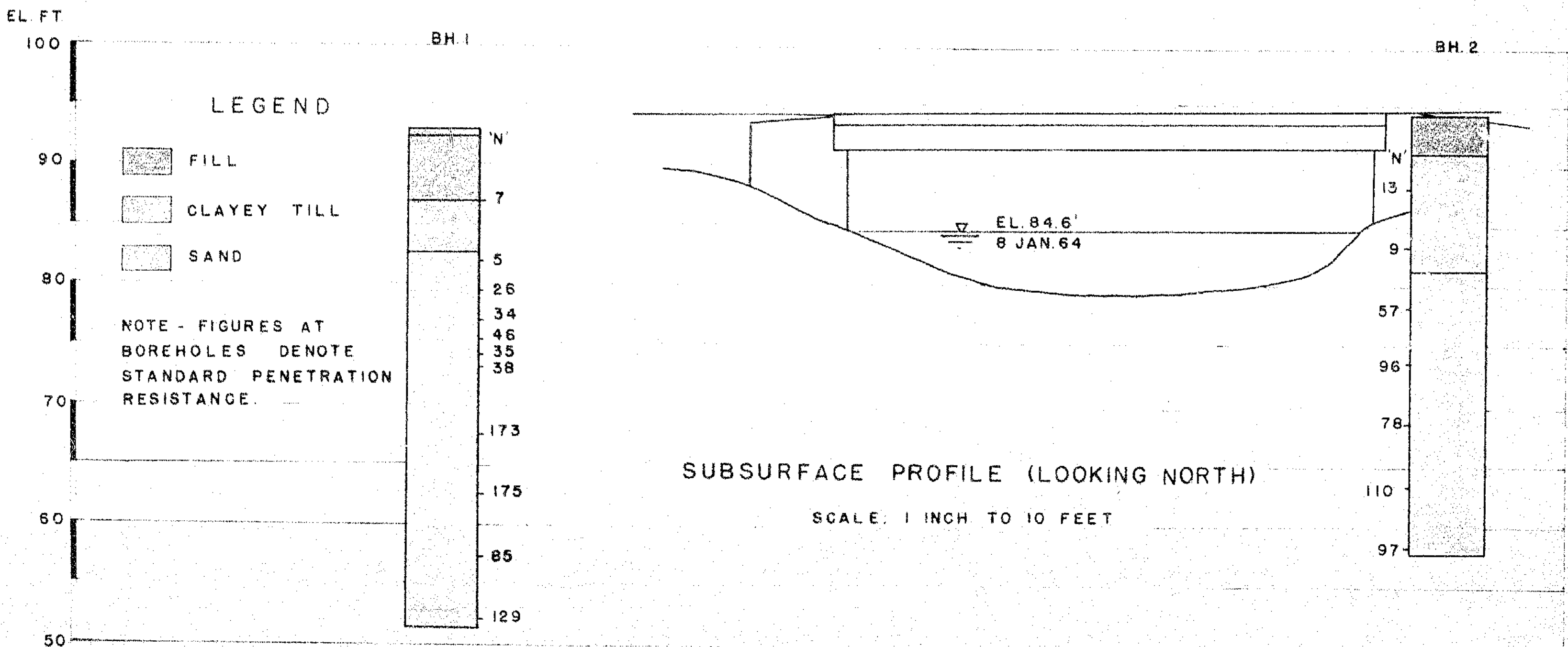
COHESIONLESS SOILS :	RD :
Very loose	0 – 15 %
Loose	15 – 35 %
Compact	35 – 65 %
Dense	65 – 85 %
Very dense	85 – 100 %

COHESIVE SOILS :	C lbs/sq.ft.
Very soft	less than 250
Soft	250 – 500
Firm	500 – 1000
Stiff	1000 – 2000
Very stiff	2000 – 4000
Hard	over 4000



LOCATION OF BOREHOLES

SCALE: 1 INCH TO 10 FEET



SUBSURFACE PROFILE (LOOKING NORTH)

SCALE: 1 INCH TO 10 FEET

GEOTECHNICAL DATA SHEET FOR BOREHOLE 1

OUR REFERENCE NO. 4-1-13

CLIENT: Messrs. C. G. Russell Armstrong
PROJECT: Road Bridge
LOCATION: Township of Colchester South
DATUM ELEVATION: 100.0' (see encl.2)

METHOD OF BORING: Washboring
DIAMETER OF BOREHOLE: NX (3-inch)
DATE: January 1964.

ENCLOSURE NO. 3

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	TYPE	N — or Advancement of Sampler	REMARKS
93.3	0	Ground Surface				
		Road fill.				
90		Brown sandy clayey fill, firm, damp.				
	5		1	SS	7	
85		Brown sandy clayey silt till, firm to stiff, damp.				WL in creek El.84.6' 8 Jan.1964.
	10	loose	2	SS	5	
80		cohesionless	3	SS	26	
	15	dense silt seam	4	SS	34	
75			5	SS	46	
			6	SS	33	
	20	very dense	7	SS	38	
70						
	25		8	SS	173	
65		Fine dark grey silty sand.				
	30		9	SS	175	Details of extrapolated 'N' value Sample 9-35/6" 60/3"
60						
	35		10	SS	85	
55						
	40		11	SS	129	
		End of borehole				

PENETRATION RESISTANCE blows per foot 20 40 60 80 100 / SHEAR STRENGTH lbs/sq ft

CONSISTENCY water content % PL W LL

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: SB CH'D: JB

GEOTECHNICAL DATA SHEET FOR BOREHOLE 2

OUR REFERENCE NO. 4-1-43

CLIENT: Messrs. C. C. Russell Armstrong
PROJECT: Road Bridge
LOCATION: Township of Colchester South
DATUM ELEVATION: 100.0' (see encl.2)

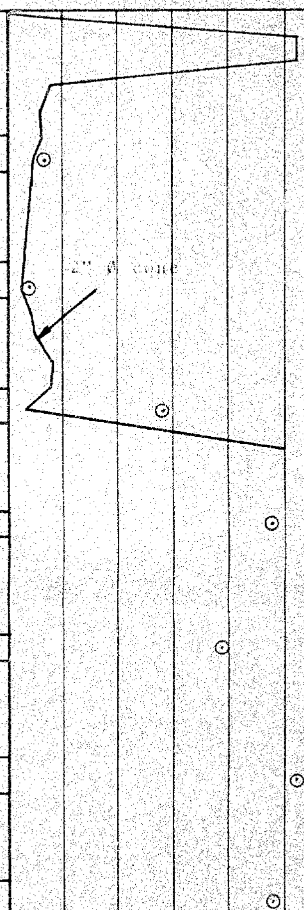
METHOD OF BORING: washboring
DIAMETER OF BOREHOLE: BX (3-inch)
DATE: January, 1964.

ENCLOSURE NO. 4

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLE NUMBER	TYPE	"N" Advancement of Sampler	REMARKS
94.4		Ground Surface				
	0	Brown sandy clayey fill, firm, damp. changing to				
90	5	Brown sandy clayey silt till, firm to stiff, damp.	1	SS	15	
85	10		2	SS	[illegible]	WL in creek El.84.0' 8 Jan.1964.
80	15	compact / very dense	3	SS	57	
75	20		4	SS	96	
70	25	Fine dark grey silty sand.	5	SS	78	
65	30		6	SS	110	
60	35		7	SS	97	
		End of borehole				

PENETRATION RESISTANCE blows per foot: 20 40 60 80 100
SHEAR STRENGTH lbs/sq ft

CONSISTENCY water content %: PL W LL



VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: SB CH'D: JP

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: A. Stermac, P. Eng.,
Principal Foundation Engineer,
Materials and Research Section,
Room 107, Lab. Bldg.

From: Bridge Division,
Downsview, Ontario.

Date: June 1, 1964.

Our File Ref.

In reply to

Subject: Twp. of Colchester South,
Bridge over the Richmond Drain,
Lot 17 Con. II/III
County of Essex
Structure Site No. 6-180
Our File No. BA 1843

Attached please find one copy of the Preliminary Plans and one copy of the Foundation Report, by Dominion Soil Investigation Limited, for your comments.

We would appreciate it very much if we could have your comments at your earliest convenience.

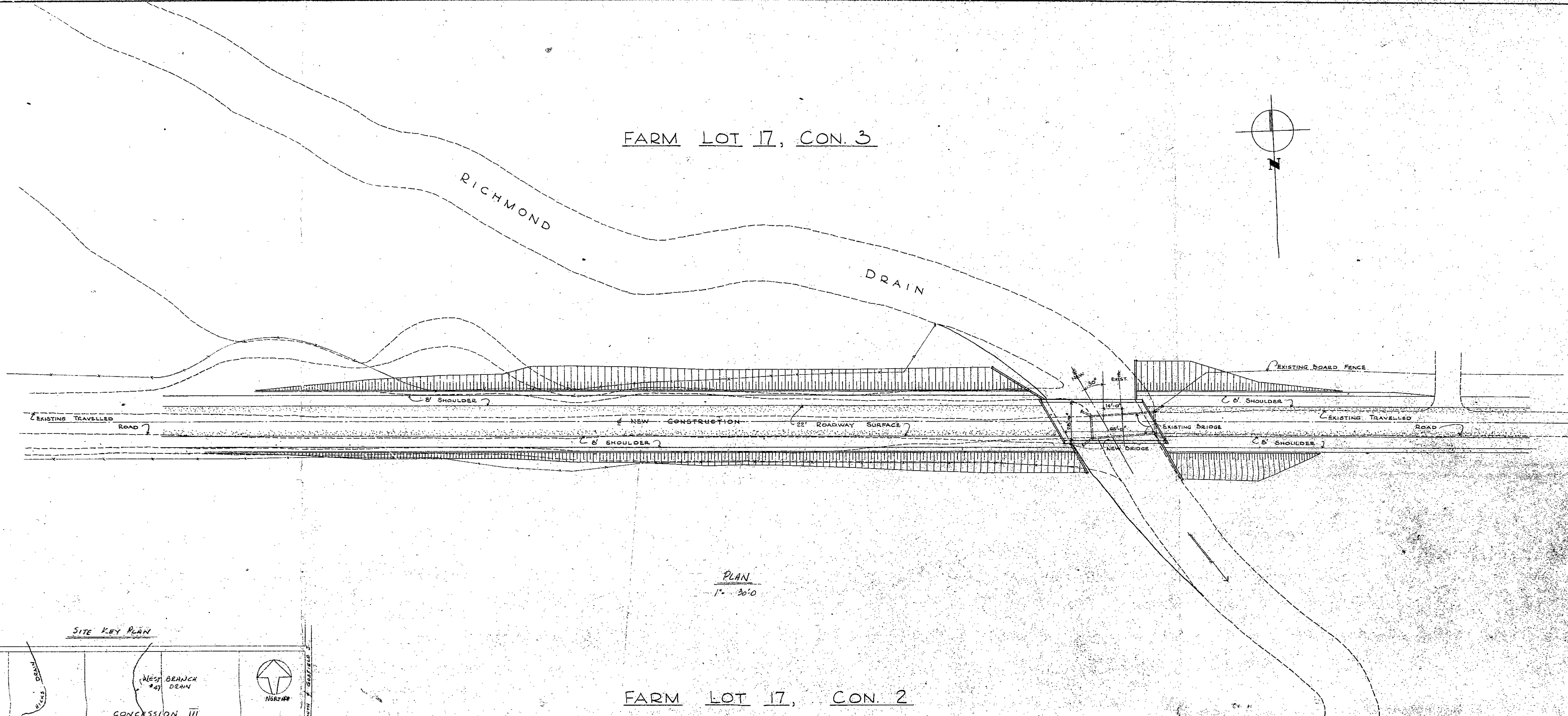
GCEB/sp

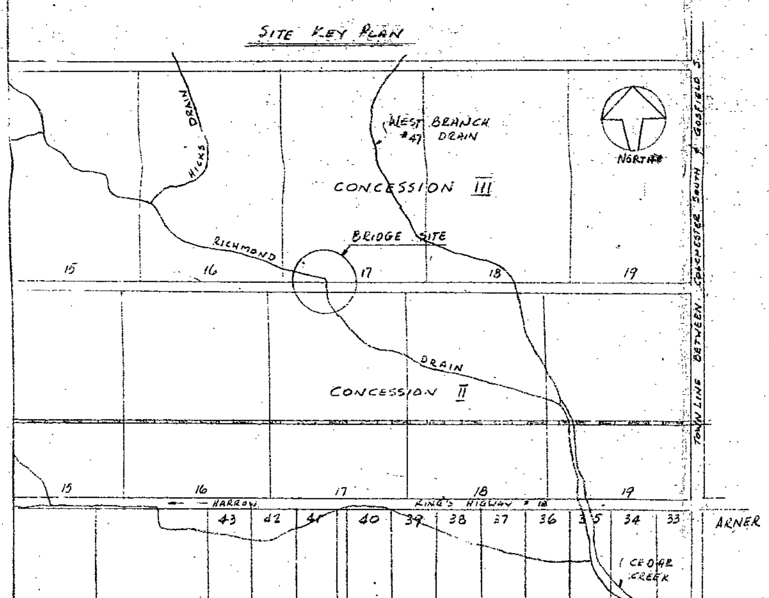
G.C.E. Bauld
for K.L. Kleinsteiber,
Municipal Bridge Liaison Engineer.

Comment:
Sheet piling only necessary to Elev. ~62.0
(2 feet + as mentioned in the report is too conservative)
(Water ~84, footing ~73, sheeting tip ~62)
By phone to G.B.
June 2. 1964

A. Stermac

64-F-250M
RICHMOND DRAIN
LOT 17, CON. 2+3
COLCHESTER S.





FOLLOW SEPARATE INSTRUCTIONS FOR PREPARATION OF BRIDGE SITE PLAN WHEN MAKING BRIDGE SURVEY

DATA

1. NONE

2. (1) 50'-0" SPAN. FIRST BRIDGE UPSTREAM

(3) NEW BRIDGE BUILT IN 1961

3. EXISTING BRIDGE VERY NARROW PRESENTING BAD TRAFFIC CONDITION.

DATA (CONT'D.)

4. ALLOWANCE HAS BEEN MADE FOR FUTURE CLEANING IF REQUIRED.
5. NIL
6. NIL
7. NIL

8. NIL

9. ROAD WILL BE CLOSED

10. HIGH WATER LEVEL 90.00' ±
LOW WATER LEVEL 84.00' ±

11.

STRUCTURE DATA

1. 65'-0" SPAN W/ PRE-STRESSED CONC. BEAMS
2. 28'-0"
3. NIL
4. 30°-00'
5.
6. 710 ± CU YDS
7. NIL
8.
9. 2800 CU YDS ±
10. 10,500 AC.

STRUCTURE SITE No. 6-180

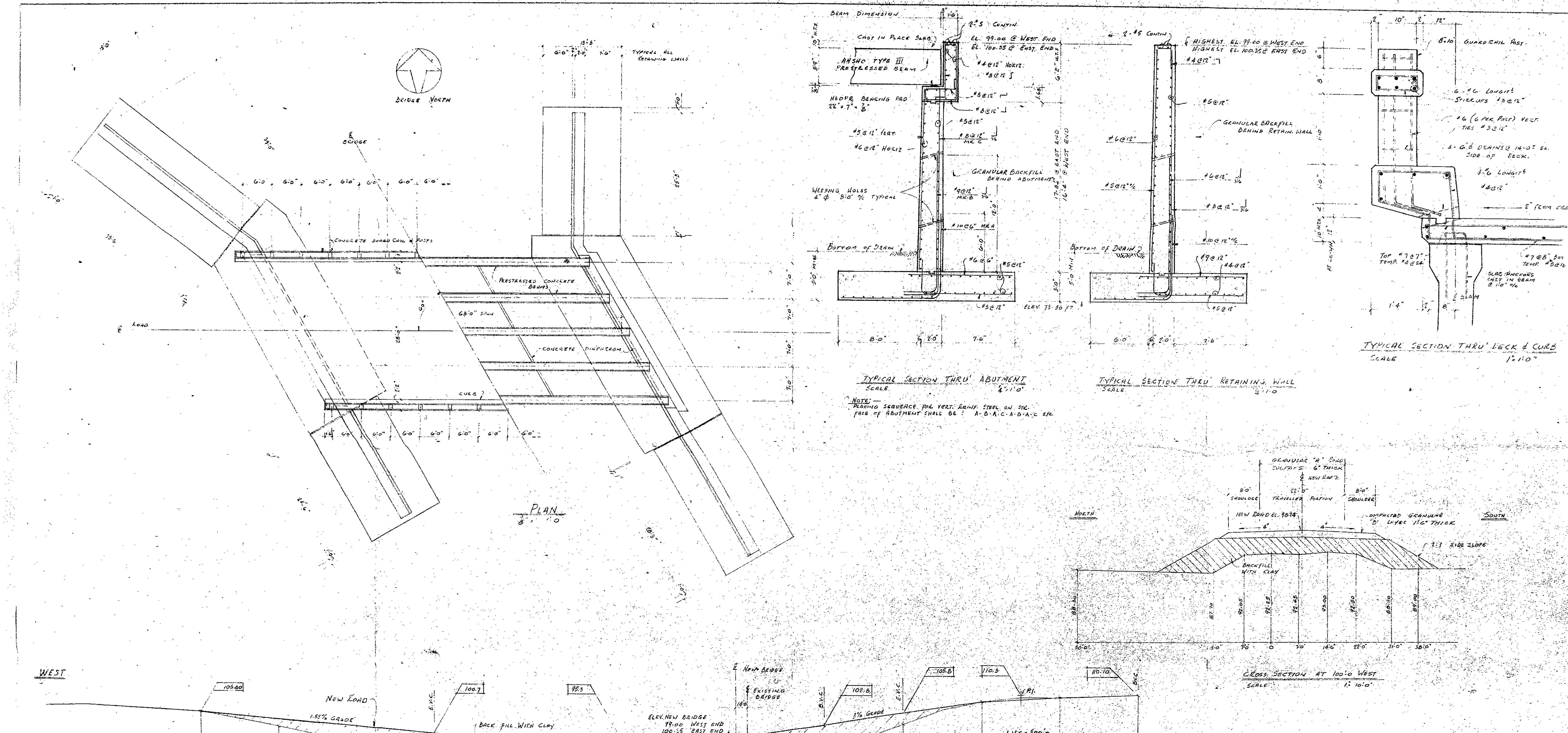
C. G. RUSSELL ARMSTRONG
CONSULTING ENGINEERS
WINDSOR - ONTARIO

PROJECT: BRIDGE OVER RICHMOND DRAIN ON ROAD BETWEEN 2 & 3 CONCESSIONS AT FARM LOT 17 - CONC. 2
TOWNSHIP OF COLCHESTER SOUTH

SHEET TITLE

REGISTERED PROFESSIONAL ENGINEER
CIVIL
C. G. R. ARMSTRONG
PROVINCE OF ONTARIO

DRAWN:- D.L.	APPROVED:-	DATE:-
CHECKED:-	SCALE:- AS NOTED	FIELD BOOK:-
OFFICE FILE No.:-	CONTRACT No.:-	
SHEET No.:- 1 OF 2	DRAWING No.:- PRELIMINARY	



C. G. RUSSELL ARMSTRONG
CONSULTING ENGINEERS
WINDSOR - ONTARIO

PROJECT
BRIDGE OVER RICHMOND DRAIN ON ROAD BETWEEN 2ND & 3RD CONCESSIONS AT FARM LOT 17 CONC. 12 TOWNSHIP OF COLCHESTER SOUTH

SHEET TITLE

DRAWN:- D.L.	APPROVED:-	DATE:-
CHECKED:-	SCALE:- AS NOTED	FIELD BOOK:-
OFFICE FILE No.:-	CONTRACT No.:-	
SHEET No.:- 2 of 2	DRAWING No.:- PRELIMINARY	

REGISTERED PROFESSIONAL ENGINEER
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