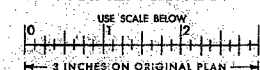
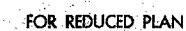
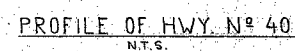
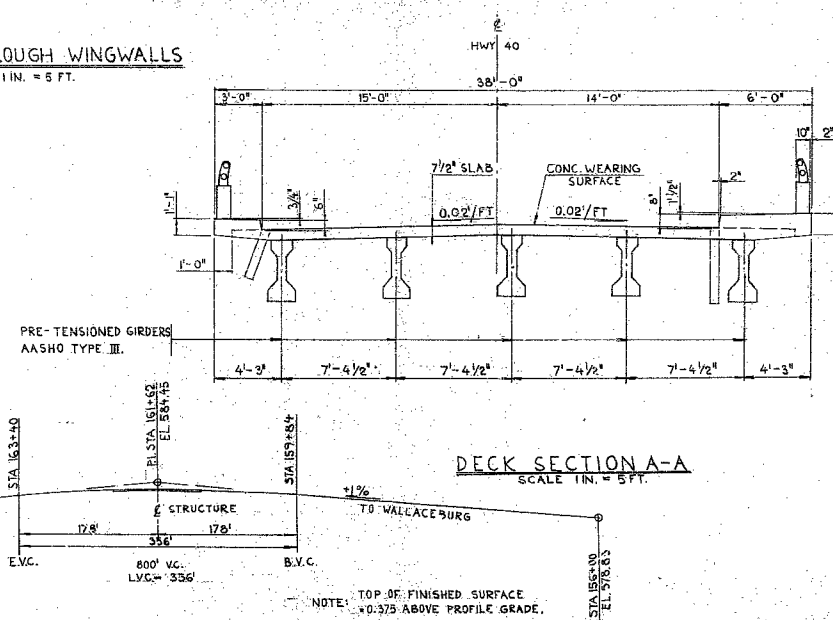
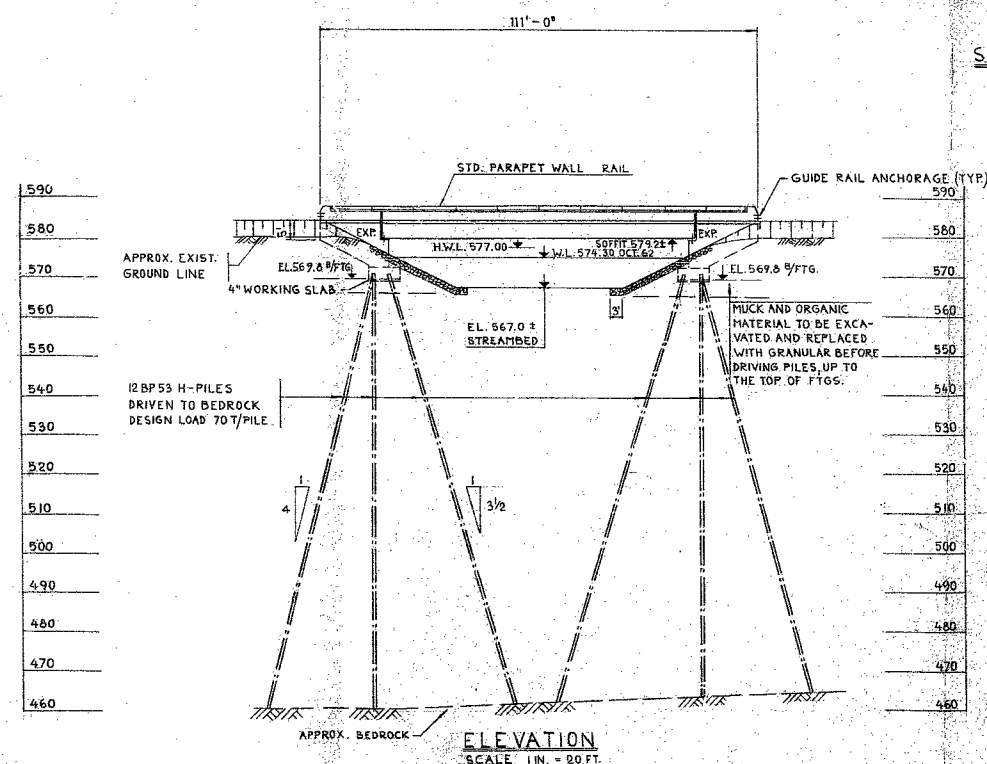
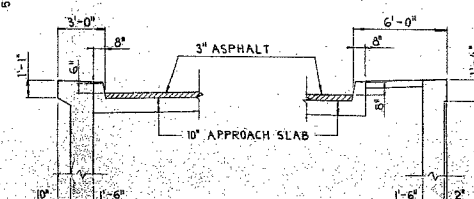
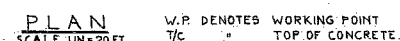
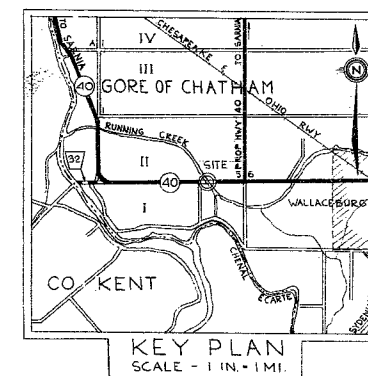
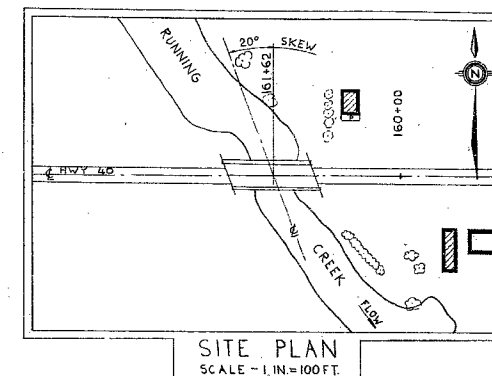


63-F-1
W.P. 323-61
Hwy. #40
RUNNING CREEK
BRIDGE #2



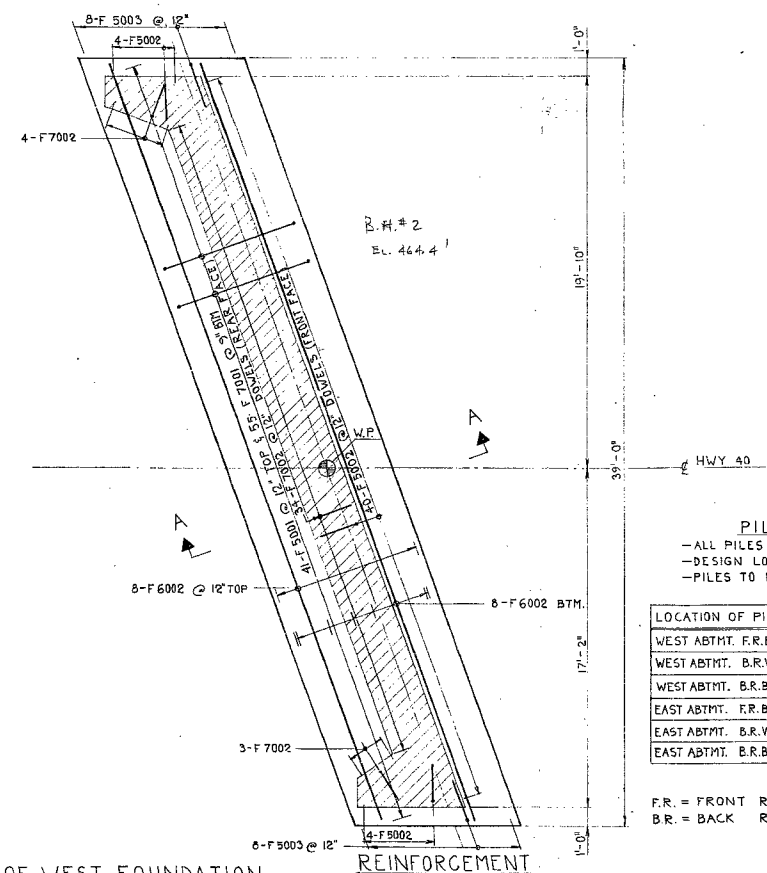
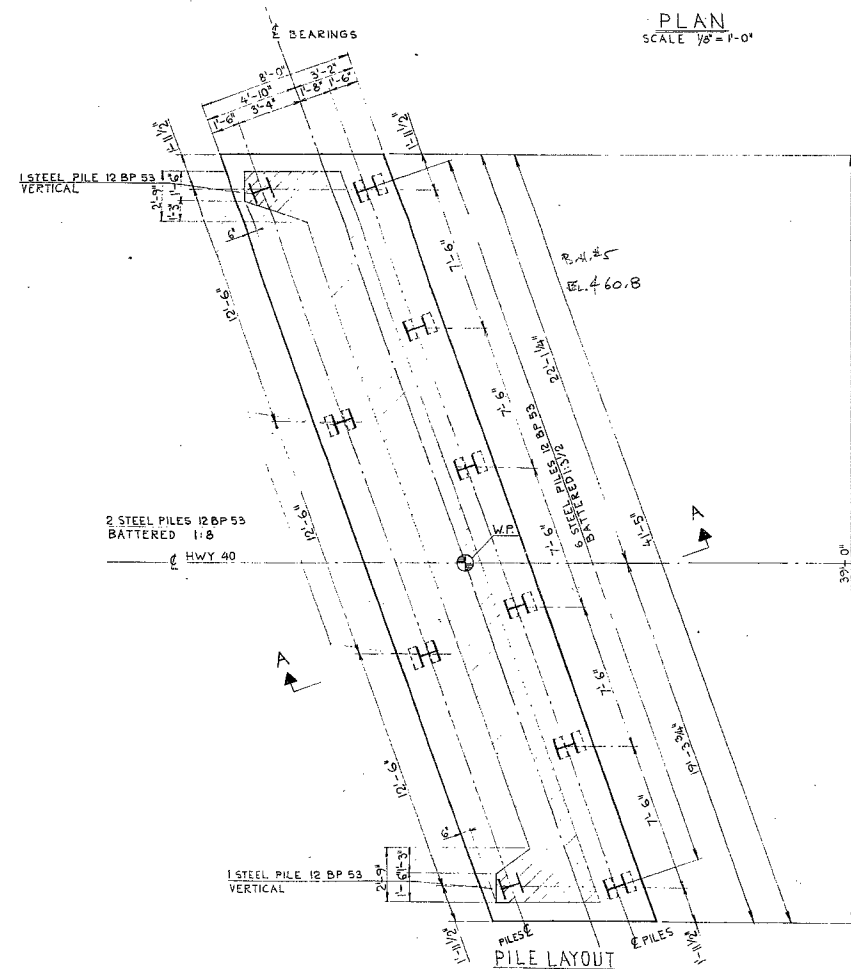
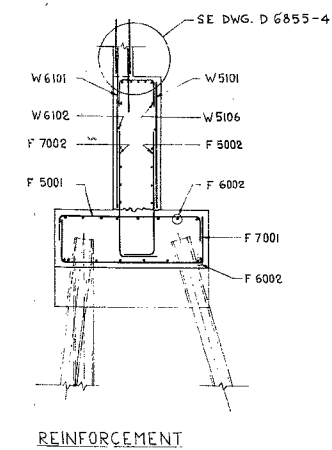
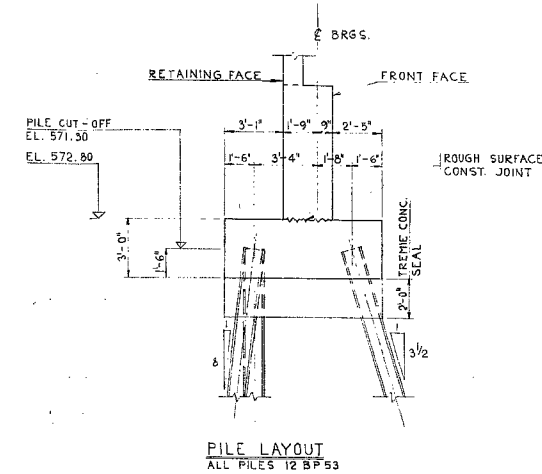
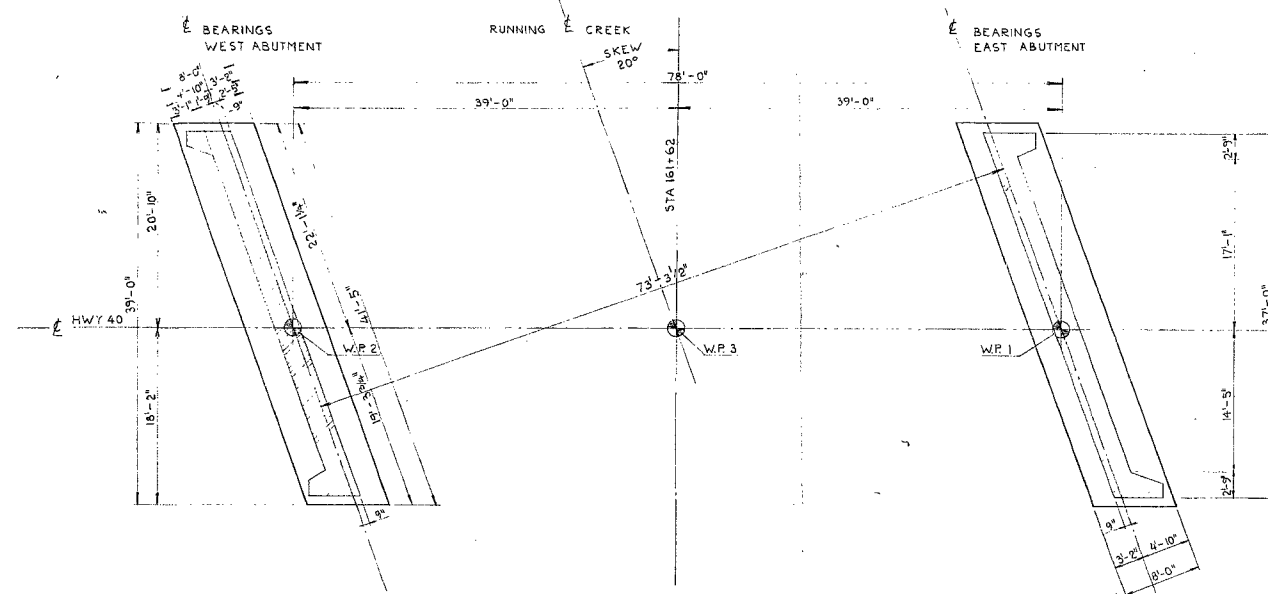
PRELIMINARY
DRAWING

ISSUED
DATE: OCT. 15 1970
Sam
MAKSYMIEC & ASSOCIATES LTD
CONSULTING PROFESSIONAL ENGINEERS

[illegible]

63-5.

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
MAKSYMEC & ASSOCIATES LTD. CONSULTING PROFESSIONAL ENGINEERS			
TOWN OF KENT		BRIDGE NO. 13-4	
RUNNING CREEK BRIDGE # 2 1.6 MILES WEST OF WALLACEBURG WEST LIMITS			
KING'S HIGHWAY NO. 40		DIST. NO. 1	
CO. OF KENT		LOT 4	
TWP. OF GORE OF CHATHAM		CON. 1 & 2	
GENERAL		ARRANGEMENT	
APPROVED _____ DESIGNER		SITE No. 13-4 W.P. No. 323-61-0	
CHECK _____ DATE		CONTRACT No. _____	
KS _____ DATE		DRAWING No. _____	
SEPT 70		D 6855-P1	



PILE DATA
-ALL PILES 12BP53
-DESIGN LOAD = 70 TON/PILE
-PILES TO BE DRIVEN TO BEDROCK

LOCATION OF PILES	N° OF PILES	LENGTH
WEST ABTMT. F.R.BATTERED	6	116'-0"
WEST ABTMT. B.R.VERTICAL	2	113'-0"
WEST ABTMT. B.R.BATTERED	2	113'-0"
EAST ABTMT. F.R.BATTERED	6	116'-0"
EAST ABTMT. B.R.VERTICAL	2	113'-0"
EAST ABTMT. B.R.BATTERED	2	113'-0"

F.R. = FRONT ROW
B.R. = BACK ROW



FOR REDUCED PLAN



REVISIONS			
	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
MAKSYMEC & ASSOCIATES LTD.			
Toronto	CONSULTING	PROFESSIONAL	ENGINEERS
Stouffville			
<h2 style="margin: 0;">RUNNING CREEK BRIDGE # 2</h2> <p style="margin: 0;">1.6 MILES WEST OF WALLACEBURG WEST LIMITS</p>			
KING'S HIGHWAY No. 40 _____		DIST. No. 1 _____	
CO. OF KENT _____		CON. 1&2 _____	
TWP. OF GORE OF CHATHAM _____		LOT 4 _____	

#

63-F-1

W.P. [#] 323-61

[#]
Hwy 40

RUNNING
CREEK

MEMORANDUM

20-70-182
RE: Foundation

Investigation Report

Running Creek Br. 2

HWY 40 West of

Wallaceburg Dist.

W.C. 63-11001

TO: Mr. B. R. Davis,
Bridge Engineer,
Bridge Office,
Admin. Bldg

ATTENTION: Mr. S. McCombie.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

DATE: June 4, 1970.

OUR FILE REF.

IN REPLY TO JUN 8 1970

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Running Creek Bridge #2
HWY. #40, West of Wallaceburg
District #1 (Chatham)
W.C. 63-11001 -- W.P. 323-61
REVISED JUNE 1970

Attached, we are forwarding to you a copy of our revised foundation report for the above-mentioned proposed structure. Revisions of recommendations relating to foundations are contained in a memorandum dated June 4th 1970 which is included in the Appendix of the report.

We believe you will find the information in the report sufficient for your future design purposes. Should there be any questions you would like to discuss please contact this Office.

AGS/hrd
Attach.

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

cc: Messrs. B. R. Davis,
H. A. Tregaskes,
D. W. Farren,
W. Zonnenberg,
F. C. Brown,
A. P. Watt,
J. Roy,
B. A. Singh

Foundation Files
General Files

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

Attention: Mr. S. McCombie

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

March 12, 1963

D.H.C. FOUNDATION INVESTIGATION REPORT FOR -
Proposed Crossing at Running Creek No. 2 and
Highway #40, 3.5 miles West of Wallaceburg,
Lot 4, Concession I & II, Township of Chatham,
District of Chatham (District #1)
W.J. 63-F-1 -- W.P. 323-61.

Attached, we are forwarding to you, our
detailed foundation report on the subsoil conditions
existing at the above-noted structure site.

We believe you will find the factual data
and recommendations therein, adequate for your future design
work. Should there be any questions you would like to
discuss, please feel free to call on our Office.

KYL/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
A. Gater
G. U. Howell
J. Poy
A. Watt

Foundations Office
Gen. Files./

KYL/Lo
K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

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 2. DESCRIPTION OF SITE.
 3. FIELD AND LABORATORY WORK.
 4. SOIL TYPES AND CONDITIONS:
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 - 4.2) Muck.
 - 4.3) Silt, Sandy Silt and Clayey Silt.
 - 4.4) Silty Clay.
 - 4.5) Sand.
 - 4.6) Bedrock.
 5. GROUND WATER CONDITIONS.
 6. DISCUSSION AND RECOMMENDATIONS:
 - 6.1) Structure Foundations.
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-

FOUNDATION INVESTIGATION

For

Proposed Crossing at Running Creek No. 2 and
Highway #40, 3.5 miles West of Wallaceburg,
Lot 4, Concession I & II, Township of Chatham,
District of Chatham (District #1)
W.J. 63-F-1 -- W.P. 323-61.

1. INTRODUCTION:

A request for a foundation investigation at the above-noted site was received from the Bridge Office in a memo dated November 6, 1962. A field investigation was subsequently carried out by this Section to determine the subsoil conditions existing at the site. The results of this investigation, together with laboratory test results, and the recommendations pertaining to the design of the proposed structure foundations, are contained in this report.

2. DESCRIPTION OF SITE:

The general ground surface on either side of Running Creek No. 2 is flat. The banks of the creek are raised slightly in the form of levees. These levees are the only topographical features in an otherwise very smooth clay plain.

3. FIELD AND LABORATORY WORK:

The field work consisted of four boreholes and one dynamic cone penetration test adjacent to one of the boreholes. The locations and elevations of these boreholes are shown on Drawing No. 63-F-1A, attached to this report.

3. FIELD AND LABORATORY WORK: (cont'd.) ...

Two of the boreholes were advanced by the repetitious procedure of driving, washing, sampling and in situ strength testing. Relatively undisturbed samples were recovered in thin-walled 2-inch diameter tubes by means of a fixed piston sampler. In situ vane tests were carried out immediately after sampling by means of the standard D.H.O. large vane.

In the other two boreholes, NX casing was driven to a depth of 20 ft. and then washed out. The Geonor vane apparatus was then used to carry out continuous in situ strength tests. The tests were performed at 8-inch intervals and extended to a depth of 81 ft. below ground elevation. The lower part of the vane apparatus was advanced by jacking against the weight of the drill rig. Representative test results are shown in the records of boreholes 3 and 4.

Bedrock was proved in borehole 2 by recovering 5 ft. of BXT size core.

Selected representative samples were tested in the laboratory to determine water content, Atterberg limits, shearing strength and consolidation characteristics of the soils. Some of the test results are given in the record of borehole 1.

4. SOIL TYPES AND SOIL CONDITIONS:

4.1) General:

Subsoil at the site was found to consist of relatively recent fluvial deposits of muck, silt, sandy silt and clayey silt

cont'd. /3 ...

4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.1) General: (cont'd.) ...

over a deep deposit of silty clay. The silty clay is underlain by a layer of sand over shale bedrock. A detailed description of the various soil types encountered during the investigation is shown in Appendix I of this report, and described in the paragraphs which follow. The estimated stratigraphical profile shown on Drawing No. 63-F-1A is based upon this information.

4.2) Muck:

A deposit of dark brown muck, ~7 ft. in thickness, covers the site within the banks of the creek. The material is very soft and has negligible shearing strength.

4.3) Silt, Sandy Silt and Clayey Silt:

Grey silt, sandy silt and clayey silt occur in irregular layers below the muck for a distance of approximately 13 ft. These soils are in a very loose state and appear to be relatively unconsolidated fluvial deposits.

4.4) Silty Clay:

Grey silty clay occurs from 20 ft. to approximately 100 ft. below ground elevation. The clay appears to be of fairly uniform composition, as evidenced by the fact that the Atterberg limits are relatively constant with depth. The clay is generally of intermediate plasticity.

The undrained shearing strength varies with depth in a manner which is typical of that at other sites in the area.

cont'd. /4 ...

4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.4) Siltv Clay: (cont'd.) ...

From ground surface, the shearing strength decreases until a minimum value is reached and beyond this level, it generally increases. At this site, however, it appears that the upper zone of high shearing strength has been eroded by the creek to a depth of about 20 ft., as indicated by the pattern of shearing strength shown in the record of boreholes. At a depth of approximately 35 ft. below ground level, a minimum shearing strength in the order of 400 p.s.f. occurs. Below this depth, the shearing strength increases; the c/p ratio is approximately 0.23. Sensitivity values range from 3 to 6.

4.5) Sand:

A layer of sand about 14 ft. in thickness was encountered in borehole 2 below the silty clay and above the bedrock. This sand contains boulders and is in a very dense state. Within the voids in this sand layer, there are pockets of gas. Gas was encountered in borehole 1 in sufficient quantities to burn for 4 hours at the surface before the fire was extinguished.

4.6) Bedrock:

Bedrock was proved by coring 5 ft. with BXT core barrel in borehole 2. The bedrock consists of dark grey shale and occurs at a depth of 109 ft. below ground elevation.

cont'd. /5 ...

5. GROUND WATER CONDITIONS:

Water level at the time of the investigation in January, 1963, was at elevation 572.8, which was the top of the ice on the creek. The water marks on the nearby boat docks indicate that the water level fluctuates significantly in one year.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) Structure Foundations:

The investigation has shown that the subsoil at this location is unsuitable for the use of spread footings. It will therefore be necessary to use piled foundations to support the new structure. Timber piles and steel H-bearing piles should be considered.

With timber piles about 40 ft. in length, an allowable load of 10 tons per pile could be utilized. Any increase in the allowable load per pile would have to be based on the results of a pile loading test at the site. Because of the likelihood of a fluctuating water table, these piles should be pressure creosoted or the pile tops protected by some other method. Settlements are likely to be significant, but the magnitude cannot be predicted with reasonable accuracy.

The structure can be supported on steel H-bearing piles driven to a sufficient depth in the sand or to bedrock. The allowable loads on such piles would be limited only by the structural capacity of the piles. - e.g., 14 BP 73 H-piles will provide a load carrying capacity of 70 tons. Settlements would be very small or non-existent.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Structure Approaches:

In view of the facts that the height of the approach fills are to be increased by only 2 ft., and that the present fills are in good condition, no stability problems are anticipated.

7. SUMMARY:

A foundation investigation carried out at the site of Highway 40 and Running Creek No. 2 is reported.

Subsoil at the site consists of fluvial deposits of muck, silt, sandy silt and clayey silt extending to a depth of 20 ft. over 80 ft. of silty clay. The silty clay is underlain by a 14-ft. layer of very dense sand and boulders over shale bedrock.

Piled foundations are recommended. Timber friction type piles, at least 40 ft. in length, with an allowable load of 10 tons per pile, can be used. The alternative is to use steel H-bearing piles driven into the sand layer or to bedrock.

No stability problems are anticipated with regard to the bridge approaches.

8. MISCELLANEOUS:

The field work was carried out during the period of 3 January to 24 January, 1963.

Department of Highways' equipment was used and operated by Departmental personnel under the supervision of Mr. R. J. Salvas. This report was prepared by Mr. R. J. Salvas under the supervision of Mr. K. Y. Lo, Foundation Section, Department of Highways, Ontario.

March 1963

APPENDIX I.

MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Office,
Admin. Bldg.

From: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

Attention: Mr. S. McCombie.

Date: June 4, 1970.

Our File Ref.

In Reply To

SUBJECT:

Running Creek Bridge #2 & Hwy. #40
Foundation Investigation Report 63-11001
W.P. 323-61 Bridge Site #13-4.
District #1 (Chatham)

We have reviewed the above-mentioned foundation report with regard to the present proposals for constructing a new three span structure at the site. As a result of this review we decided that additional work in the field was necessary to more accurately define the bedrock profile. Thus, BH #5 has been extended down to bedrock. We have also revised the report by preparing a new Drawing No. 63-11001A, and by submitting new recommendations. These recommendations are as follows:-

- (1) The proposed structure should be supported on piles end-bearing on the bedrock. The most practical type of pile would be a steel 'H' pile in which case the maximum allowable load for the particular steel section adopted may be assumed for design purposes.
- (2) All organic soil should be removed and replaced with suitable granular fill for a width equal to the width of the approach embankments (toe to toe) at the following locations:
 - (a) From 5 ft. west of the east pier footing to 20 ft. east of the east abutment footing.
 - (b) From 5 ft. east of the west pier footing to 20 ft. west of the west abutment footing.This applies also to organic soil which may exist under the existing fill. Outside of the above-mentioned limits excavation of organic soil should be according to recommendations of the Regional Materials Engineer.
- (3) If excavations are carried out for the pier or abutment footings below the ground or river water level a de-watering scheme will be required to prevent 'boiling' of the upper subsoil layers which consist of silt or sandy silt. This objective can be achieved by constructing an enclosed coffer dam which extends for a distance below the footing base equal to the height of the prevailing ground or river water level above the footing base.

June 4, 1970.

- (4) Settlements will occur under the proposed approaches due to the weight of new fill added. These settlements are not anticipated to exceed about 6 inches over a long term period but differential settlements are expected to occur between the existing fill and the widened portions. For this reason it is recommended that a slight surcharge be added to the widened portions (i.e. $1\frac{1}{2}$ - 2 ft.) and that final paving of the bridge approaches be delayed for as long a period as is possible.
- (5) No stability problems are anticipated provided 2:1 slopes are constructed and organic material is removed as specified above.

This memo is to be considered a part of the foundation investigation report and should be attached to your copy of the report. Recommendations given in this memo supercede recommendations given on pages 5 and 6 of the original report.

K. G. Selby

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

KGS/hrd

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
W. Zonnenberg
F. C. Brown
A. P. Watt
J. Roy
B. A. Singh

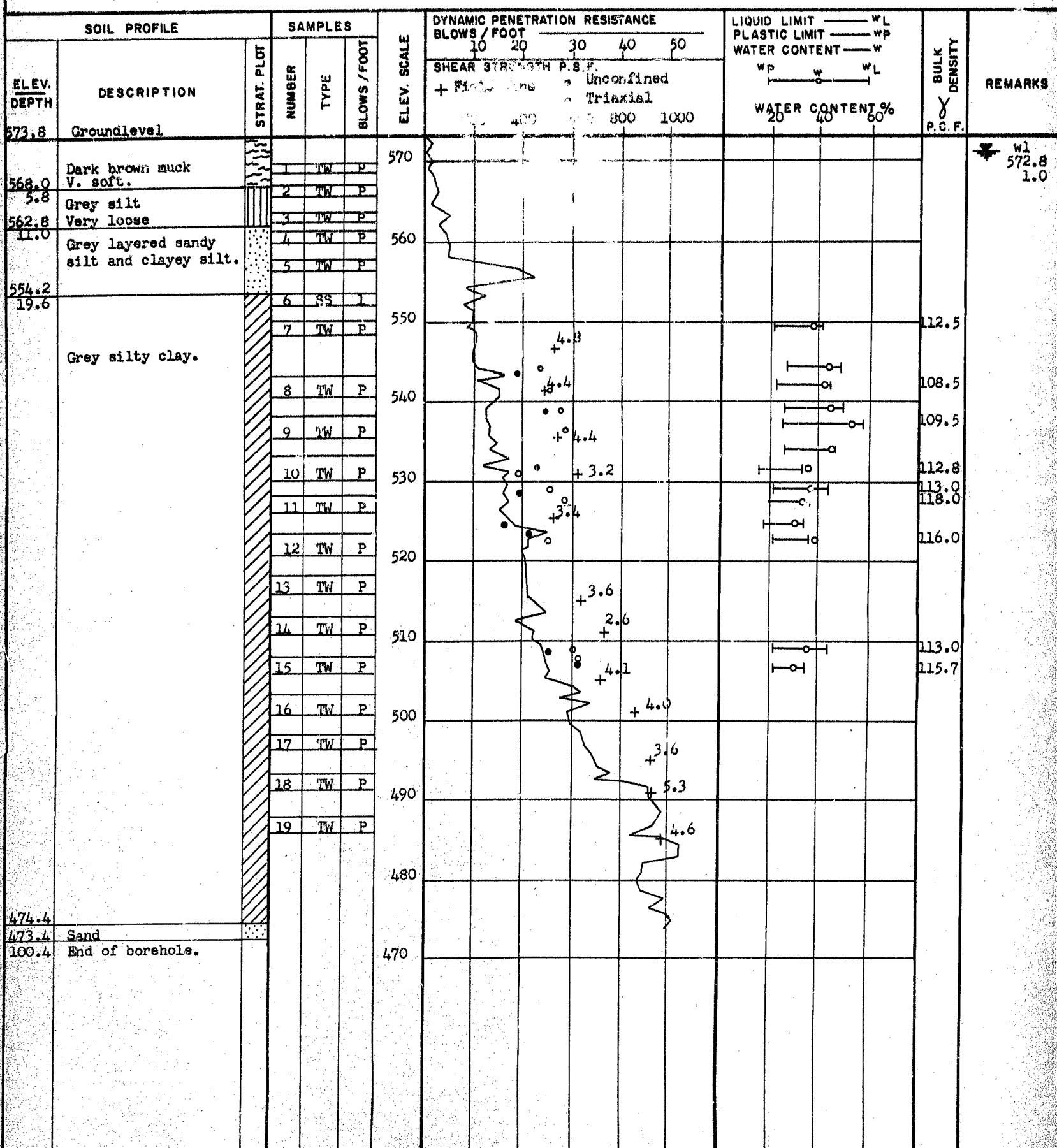
Foundation Files
General Files

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 63-11001 LOCATION Sta. 162+20 (37' Rt. of C.) ORIGINATED BY R.S.
W.P. 323-61 BORING DATE Jan. 3, 1963. COMPILED BY B.K.
DATUM 573.8' BOREHOLE TYPE Washboring CHECKED BY K.Y.L.



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 63-11001
W. P. 323-61
DATUM 573.3

LOCATION Sta. 161/14 (41' It.)

ORIGINATED BY R.S.

W. P. 323-61

BORING DATE Jan. 10, 1963.

COMPILED BY B.K.

DATUM 573.3

BOREHOLE TYPE Washboring.

CHECKED BY K.Y.L.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.		WATER CONTENT %			
							+ Field Vane		wp ——— w ——— WL			
						200	400	600	800	1000		
573.3	Groundlevel											
0.0	Dark brown muck. V. soft					570						
566.5												
6.8	Grey silt, sandy silt and clayey silt-very loose.		1	SS	P	560						
554.0												
19.0	Grey silty clay.		2	TW	P	550			+4.3			
			3	TW	P				+3.7			
			4	TW	P	540			+3.1			
			5	TW	P				+2.4			
			6	TW	P	530			+3.2			
			7	TW	P				+3.3			
			8	TW	P	520			+3.1			
			9	TW	P				+4.7			
			10	TW	P	510			+2.8			
			11	TW	P				+3.6			
			12	TW	P	500			+2.9			
			13	TW	P				+2.7			
			14	TW	P	490			+3.9			
			15	TW	P				+5.1			
478.6						480						
94.7	Grey sand with boulders v. dense					470						
464.4												
108.9	Dark grey shale					460						
459.4												
113.9	End of borehole.					450						

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 63-11001 LOCATION Sta. 161+19 36' Lt. E ORIGINATED BY R.S.
 W.P. 323-61 BORING DATE Jan. 21, 1963. COMPILED BY B.K.
 DAT 573.3 BOREHOLE TYPE Continuous vane tests only. CHECKED BY K.Y.L.

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. + Geonor Field Vane					WATER CONTENT %				
							200	400	600	800	1000	WP	W	WL		
573.3																
0	dark brown muck very soft					570										572.8 0.5
566.5																
6.8	Grey silt, sandy silt and clayey silt- very loose.					560										
554.0																
19.0	Grey silty clay.					550										
						540										
						530										
						520										
						510										
						500										
						490										
						480										
						470										

Note
The stratigraphy of borehole 3 is assumed to be similar to that of borehole 2.

FOUNDATION SECTION

ORIGINATED BY R.S.

COMPILED BY B.K.

CHECKED BY K.Y.L.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— w_L PLASTIC LIMIT ——— w_p WATER CONTENT ——— w		BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F. + Geonor Field Vane		w_p ——— w ——— w_L WATER CONTENT %				
	Groundlevel						200	400	600	800	1000		
	Same information as borehole 3.					570							
						560							
						550	4.9	6.2					
							7.7	6.3					
						540							
						530		5.9	6.2				
								5.1					
						520							
						510		3.4					
						500		3.7					
						490							
						480							
					470								
					460								

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

JOB 63-11001 LOCATION Sta. 162 + 23 16' Rt. ORIGINATED BY PP
W.P. 323-61 BORING DATE May 5 & 6, 1970 COMPILED BY PP
DATUM Geodetic BOREHOLE TYPE Washbore-BX Casing CHECKED BY PP

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w				BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.				WATER CONTENT %					
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE				w_p — w — w_L						
579.5	Ground Level															
0.0	Fill															
	Sandy silt with some clay					570										
						560										
	Silty clay					550										
						540										
						530										
						520										
						510										
						500										
						490										
						480										
477.5																
102.0	Sandy till					470										
	Very dense															
460.8																
118.7	Probable Bedrock End of Borehole					460										

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

2 PENETRATION RESISTANCE - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED
END OF DRILL ROD. 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS:-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	F.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR = $\frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

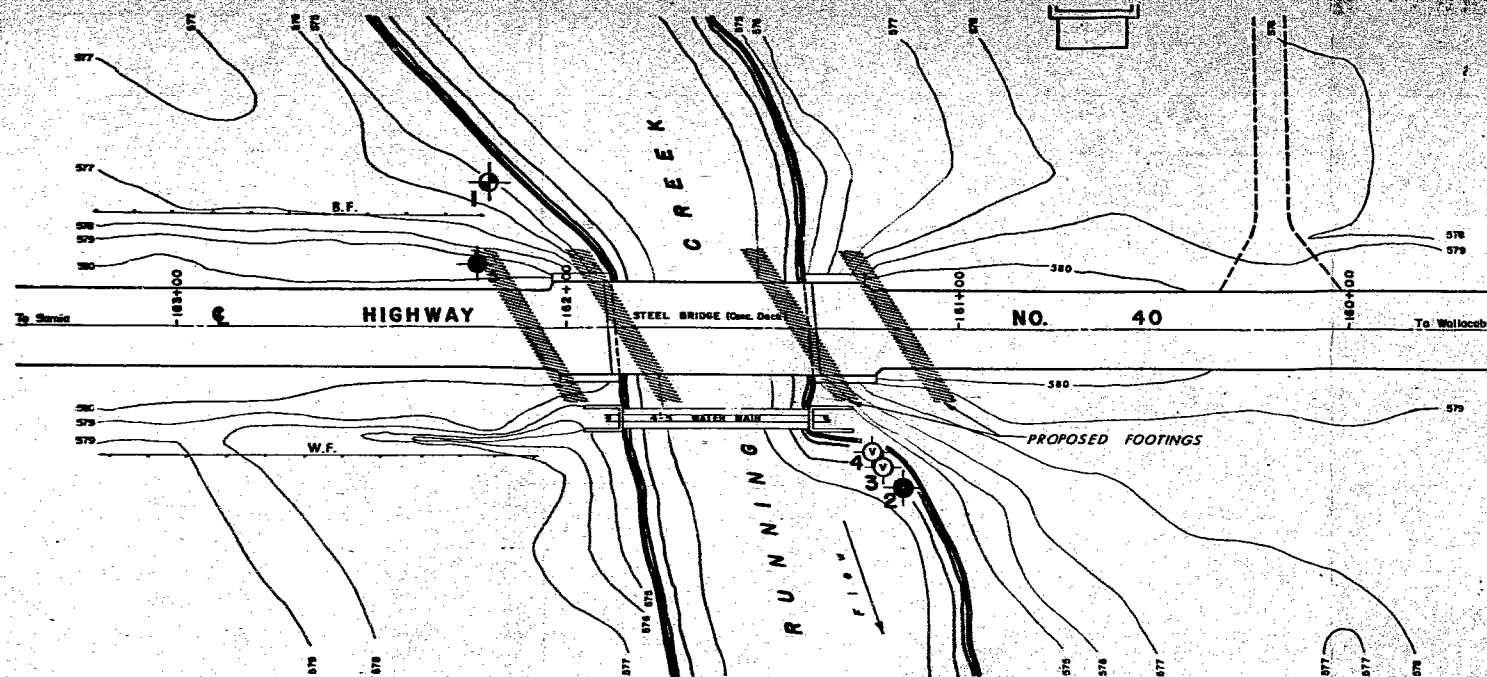
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

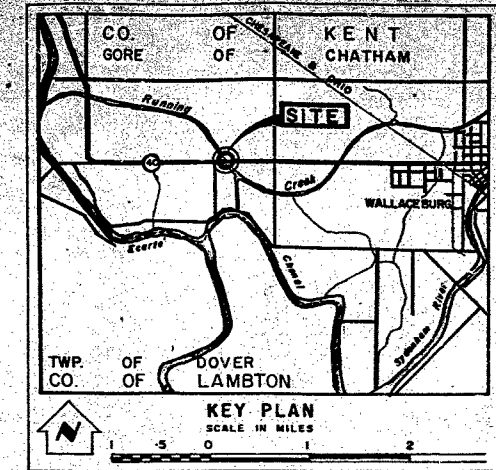
SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



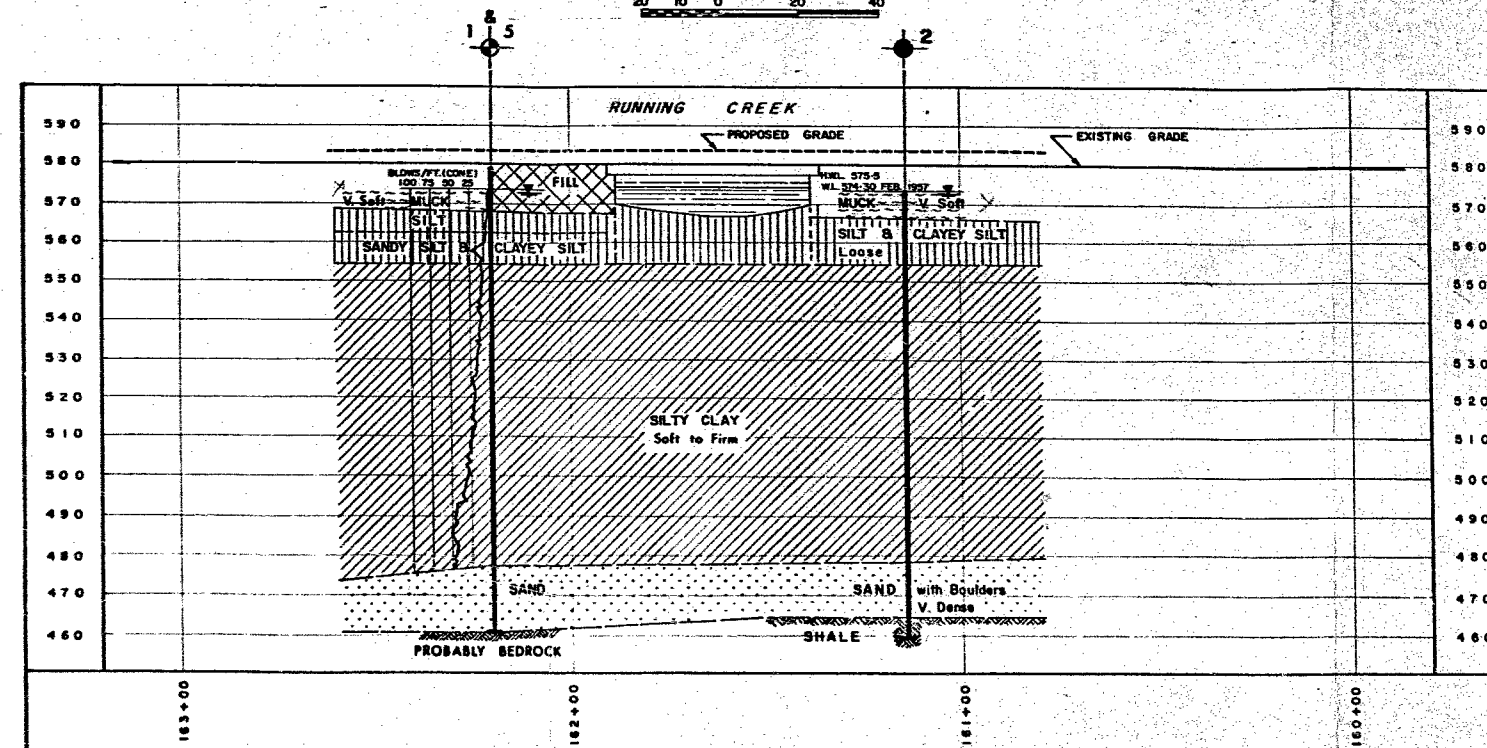
PLAN
SCALE IN FEET
20 10 0 20 40

- NOTE -
The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the CHATHAM District Office. The Department does not guarantee the accuracy of this report or the abridged version shown on these plans.



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation (Jan. 1963)		
	Vane Test		

NO.	ELEVATION	STATION	OFFSET
1	573.8	162+20	37' RT.
2	573.3	161+14	41' LT.
3	573.3	161+19	36' LT.
4	573.3	161+22	32' LT.
5	579.5	162+23	16' RT.



PROFILE
SCALE IN FEET
20 10 0 20 40

- 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
1	MAY 70	S.R.	BORE HOLE No. 5 ADDED ON PLAN & PROFILE

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

RUNNING CREEK
NO. 2

KING'S HIGHWAY NO. 40 DIST. NO. 1
CO. KENT
TWP. GORE OF CHATHAM LOT 4 CON. I & II

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. B. K.	CHECKED <input checked="" type="checkbox"/>	W.P. NO. 323-61	M.R. DRAWING NO.
DRAWN D. M.	CHECKED <input checked="" type="checkbox"/>	JOB NO. 63-F-1	63-11001A
DATE 4 MARCH 1963	SITE NO.		BRIDGE DRAWING NO.
APPROVED <i>[Signature]</i>	CONT. NO.		

REF. NO. E-4877-1
REF. NO. E-3311-1

MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Office,
Admin. Bldg

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

ATTENTION: Mr. S. McCombie.

DATE: June 4, 1970.

OUR FILE REF.

IN REPLY TO JUN 8 1970

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Running Creek Bridge #2
HWY. #40, West of Wallaceburg
District #1 (Chatham)
W.O. 62-11C01 -- W.P. 323-61
REVISED JUNE 1970

Attached, we are forwarding to you a copy of our revised foundation report for the above-mentioned proposed structure. Revisions of recommendations relating to foundations are contained in a memorandum dated June 4th 1970 which is included in the Appendix of the report.

We believe you will find the information in the report sufficient for your future design purposes. Should there be any questions you would like to discuss please contact this Office.

AGS/hrd
Attach.

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

cc: Messrs. B. R. Davis,
H. A. Tregaskes,
D. W. Parren,
W. Zonnenberg,
F. C. Brown,
A. P. Watt,
J. Roy,
B. A. Singh

Foundation Files ✓
General Files

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

Attention: Mr. S. McCombie

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

March 12, 1963

D.H.O. FOUNDATION INVESTIGATION REPORT FOR -
Proposed Crossing at Running Creek No. 2 and
Highway #40, 3.5 miles West of Wallaceburg,
Lot 4, Concession I & II, Township of Chatham,
District of Chatham (District #1)
W.J. 63-F-1 -- W.P. 323-61.

Attached, we are forwarding to you, our
detailed foundation report on the subsoil conditions
existing at the above-noted structure site.

We believe you will find the factual data
and recommendations therein, adequate for your future design
work. Should there be any questions you would like to
discuss, please feel free to call on our Office.

ZYL/mdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
E. D. McMillan
A. Cater
G. U. Howell
J. Poy
A. Watt

Foundations Office
Gen. Files./

K.Y.L.
K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

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1. INTRODUCTION.
 2. DESCRIPTION OF SITE.
 3. FIELD AND LABORATORY WORK.
 4. SOIL TYPES AND CONDITIONS:
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 - 4.2) Muck.
 - 4.3) Silt, Sandy Silt and Clayey Silt.
 - 4.4) Silty Clay.
 - 4.5) Sand.
 - 4.6) Bedrock.
 5. GROUND WATER CONDITIONS.
 6. DISCUSSION AND RECOMMENDATIONS:
 - 6.1) Structure Foundations.
 - 6.2) Structure Approaches.
 7. SUMMARY.
 8. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION

For

Proposed Crossing at Running Creek No. 2 and
Highway #40, 3.5 miles West of Wallaceburg,
Lot 4, Concession I & II, Township of Chatham,
District of Chatham (District #1)
W.J. 63-F-1 W.P. 323-61.

1. INTRODUCTION:

A request for a foundation investigation at the above-noted site was received from the Bridge Office in a memo dated November 6, 1962. A field investigation was subsequently carried out by this Section to determine the subsoil conditions existing at the site. The results of this investigation, together with laboratory test results, and the recommendations pertaining to the design of the proposed structure foundations, are contained in this report.

2. DESCRIPTION OF SITE:

The general ground surface on either side of Running Creek No. 2 is flat. The banks of the creek are raised slightly in the form of levees. These levees are the only topographical features in an otherwise very smooth clay plain.

3. FIELD AND LABORATORY WORK:

The field work consisted of four boreholes and one dynamic cone penetration test adjacent to one of the boreholes. The locations and elevations of these boreholes are shown on Drawing No. 63-F-1A, attached to this report.

cont'd. /2 ...

3. FIELD AND LABORATORY WORK: (cont'd.) ...

Two of the boreholes were advanced by the repetitious procedure of driving, washing, sampling and in situ strength testing. Relatively undisturbed samples were recovered in thin-walled 2-inch diameter tubes by means of a fixed piston sampler. In situ vane tests were carried out immediately after sampling by means of the standard D.H.O. large vane.

In the other two boreholes, NX casing was driven to a depth of 20 ft. and then washed out. The Geonor vane apparatus was then used to carry out continuous in situ strength tests. The tests were performed at 8-inch intervals and extended to a depth of 81 ft. below ground elevation. The lower part of the vane apparatus was advanced by jacking against the weight of the drill rig. Representative test results are shown in the records of boreholes 3 and 4.

Bedrock was proved in borehole 2 by recovering 5 ft. of BXT size core.

Selected representative samples were tested in the laboratory to determine water content, Atterberg limits, shearing strength and consolidation characteristics of the soils. Some of the test results are given in the record of borehole 1.

4. SOIL TYPES AND SOIL CONDITIONS:

4.1) General:

Subsoil at the site was found to consist of relatively recent fluvial deposits of muck, silt, sandy silt and clayey silt

cont'd. /3 ...

4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.1) General: (cont'd.) ...

over a deep deposit of silty clay. The silty clay is underlain by a layer of sand over shale bedrock. A detailed description of the various soil types encountered during the investigation is shown in Appendix I of this report, and described in the paragraphs which follow. The estimated stratigraphical profile shown on Drawing No. 63-F-1A is based upon this information.

4.2) Muck:

A deposit of dark brown muck, 7 ft. in thickness, covers the site within the banks of the creek. The material is very soft and has negligible shearing strength.

4.3) Silt, Sandy Silt and Clayey Silt:

Grey silt, sandy silt and clayey silt occur in irregular layers below the muck for a distance of approximately 13 ft. These soils are in a very loose state and appear to be relatively unconsolidated fluvial deposits.

4.4) Silty Clay:

Grey silty clay occurs from 20 ft. to approximately 100 ft. below ground elevation. The clay appears to be of fairly uniform composition, as evidenced by the fact that the Atterberg limits are relatively constant with depth. The clay is generally of intermediate plasticity.

The undrained shearing strength varies with depth in a manner which is typical of that at other sites in the area.

cont'd. /4 ...

4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.4) Silty Clay: (cont'd.) ...

From ground surface, the shearing strength decreases until a minimum value is reached and beyond this level, it generally increases. At this site, however, it appears that the upper zone of high shearing strength has been eroded by the creek to a depth of about 20 ft., as indicated by the pattern of shearing strength shown in the record of boreholes. At a depth of approximately 35 ft. below ground level, a minimum shearing strength in the order of 400 p.s.f. occurs. Below this depth, the shearing strength increases; the c/p ratio is approximately 0.23. Sensitivity values range from 3 to 6.

4.5) Sand:

A layer of sand about 14 ft. in thickness was encountered in borehole 2 below the silty clay and above the bedrock. This sand contains boulders and is in a very dense state. Within the voids in this sand layer, there are pockets of gas. Gas was encountered in borehole 1 in sufficient quantities to burn for 4 hours at the surface before the fire was extinguished.

4.6) Bedrock:

Bedrock was proved by coring 5 ft. with BXT core barrel in borehole 2. The bedrock consists of dark grey shale and occurs at a depth of 109 ft. below ground elevation.

cont'd. /5 ...

5. GROUND WATER CONDITIONS:

Water level at the time of the investigation in January, 1963, was at elevation 572.8, which was the top of the ice on the creek. The water marks on the nearby boat docks indicate that the water level fluctuates significantly in one year.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) Structure Foundations:

The investigation has shown that the subsoil at this location is unsuitable for the use of spread footings. It will therefore be necessary to use piled foundations to support the new structure. Timber piles and steel H-bearing piles should be considered.

With timber piles about 40 ft. in length, an allowable load of 10 tons per pile could be utilized. Any increase in the allowable load per pile would have to be based on the results of a pile loading test at the site. Because of the likelihood of a fluctuating water table, these piles should be pressure creosoted or the pile tops protected by some other method. Settlements are likely to be significant, but the magnitude cannot be predicted with reasonable accuracy.

The structure can be supported on steel H-bearing piles driven to a sufficient depth in the sand or to bedrock. The allowable loads on such piles would be limited only by the structural capacity of the piles. - e.g., 14 BP 73 H-piles will provide a load carrying capacity of 70 tons. Settlements would be very small or non-existent.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Structure Approaches:

In view of the facts that the height of the approach fills are to be increased by only 2 ft., and that the present fills are in good condition, no stability problems are anticipated.

7. SUMMARY:

A foundation investigation carried out at the site of Highway 40 and Running Creek No. 2 is reported.

Subsoil at the site consists of fluvial deposits of muck, silt, sandy silt and clayey silt extending to a depth of 20 ft. over 80 ft. of silty clay. The silty clay is underlain by a 14-ft. layer of very dense sand and boulders over shale bedrock.

Piled foundations are recommended. Timber friction type piles, at least 40 ft. in length, with an allowable load of 10 tons per pile, can be used. The alternative is to use steel H-bearing piles driven into the sand layer or to bedrock.

No stability problems are anticipated with regard to the bridge approaches.

8. MISCELLANEOUS:

The field work was carried out during the period of 3 January to 24 January, 1963.

Department of Highways' equipment was used and operated by Departmental personnel under the supervision of Mr. R. J. Salvas. This report was prepared by Mr. R. J. Salvas under the supervision of Mr. K. Y. Lo, Foundation Section, Department of Highways, Ontario.

March 1963

APPENDIX I.

MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Office,
Admin. Bldg.
ATTENTION: Mr. S. McCombie.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

DATE: June 4, 1970.

OUR FILE REF.

IN REPLY TO

SUBJECT:

Running Creek Bridge #2 & Hwy. #40
Foundation Investigation Report 63-11001
W.P. 323-61 Bridge Site #13-4.
District #1 (Chatham)

We have reviewed the above-mentioned foundation report with regard to the present proposals for constructing a new three span structure at the site. As a result of this review we decided that additional work in the field was necessary to more accurately define the bedrock profile. Thus, BH #5 has been extended down to bedrock. We have also revised the report by preparing a new Drawing No. 63-11C01A, and by submitting new recommendations. These recommendations are as follows:-

- (1) The proposed structure should be supported on piles end-bearing on the bedrock. The most practical type of pile would be a steel 'H' pile in which case the maximum allowable load for the particular steel section adopted may be assumed for design purposes.
- (2) All organic soil should be removed and replaced with suitable granular fill for a width equal to the width of the approach embankments (toe to toe) at the following locations:
 - (a) From 5 ft. west of the east pier footing to 20 ft. east of the east abutment footing.
 - (b) From 5 ft. east of the west pier footing to 20 ft. west of the west abutment footing.
 This applies also to organic soil which may exist under the existing fill. Outside of the above-mentioned limits excavation of organic soil should be according to recommendations of the Regional Materials Engineer.
- (3) If excavations are carried out for the pier or abutment footings below the ground or river water level a dewatering scheme will be required to prevent 'boiling' of the upper subsoil layers which consist of silt or sandy silt. This objective can be achieved by constructing an enclosed coffer dam which extends for a distance below the footing base equal to the height of the prevailing ground or river water level above the footing base.

June 4, 1970.

- (4) Settlements will occur under the proposed approaches due to the weight of new fill added. These settlements are not anticipated to exceed about 6 inches over a long term period but differential settlements are expected to occur between the existing fill and the widened portions. For this reason it is recommended that a slight surcharge be added to the widened portions (i.e. $1\frac{1}{2}$ - 2 ft.) and that final paving of the bridge approaches be delayed for as long a period as is possible.
- (5) No stability problems are anticipated provided 2:1 slopes are constructed and organic material is removed as specified above.

This memo is to be considered a part of the foundation investigation report and should be attached to your copy of the report. Recommendations given in this memo supercede recommendations given on pages 5 and 6 of the original report.

K. G. Selby

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

KGS/hrd

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
W. Zonnenberg
F. C. Brown
A. P. Watt
J. Roy
B. A. Singh

Foundation Files
General Files

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 63-11001

LOCATION Sta. 162+20 (37' Rt. of C)

ORIGINATED BY R.S.

W.P. 323-61

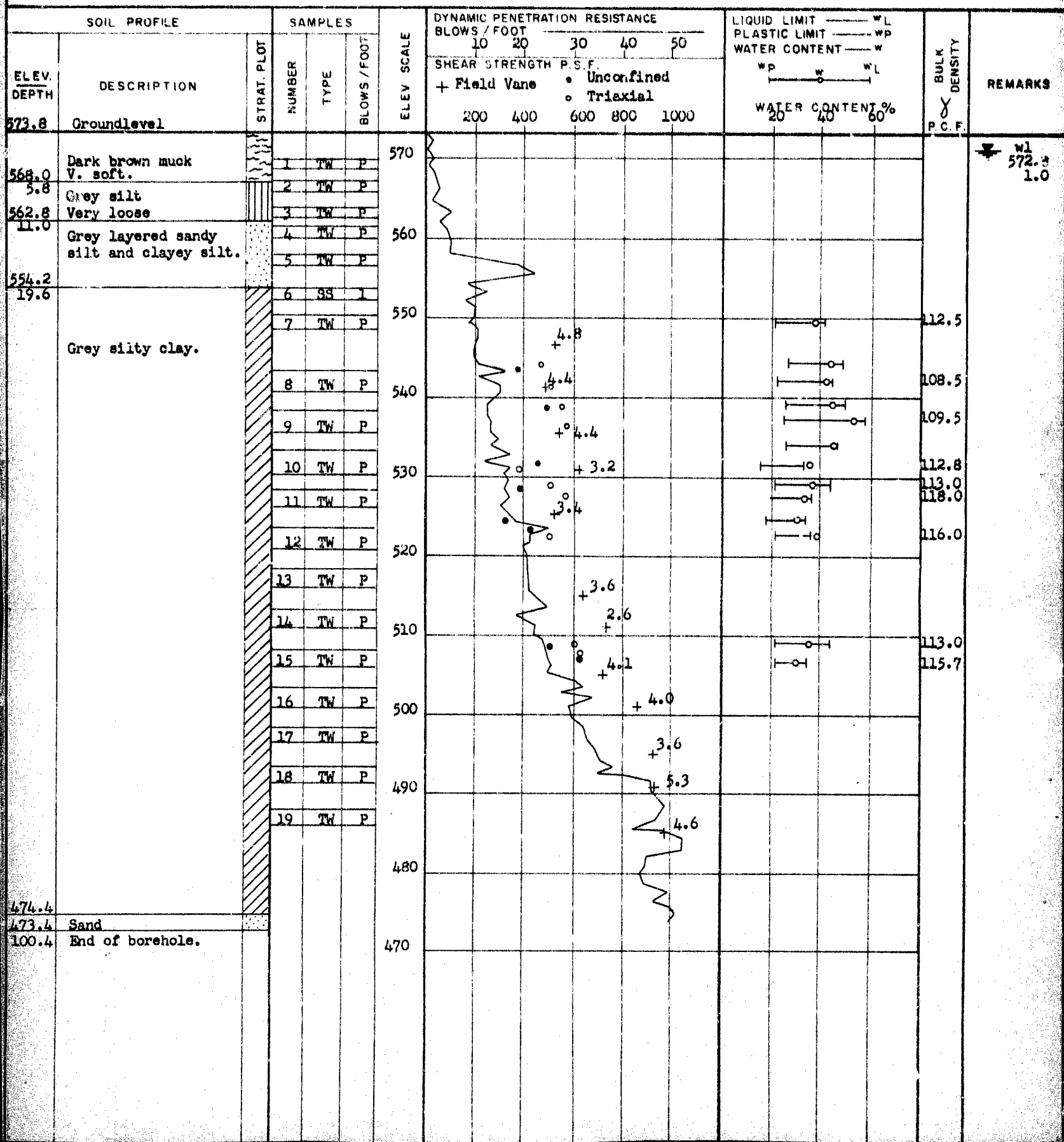
BORING DATE Jan. 3, 1963.

COMPILED BY B.K.

DATUM 573.8'

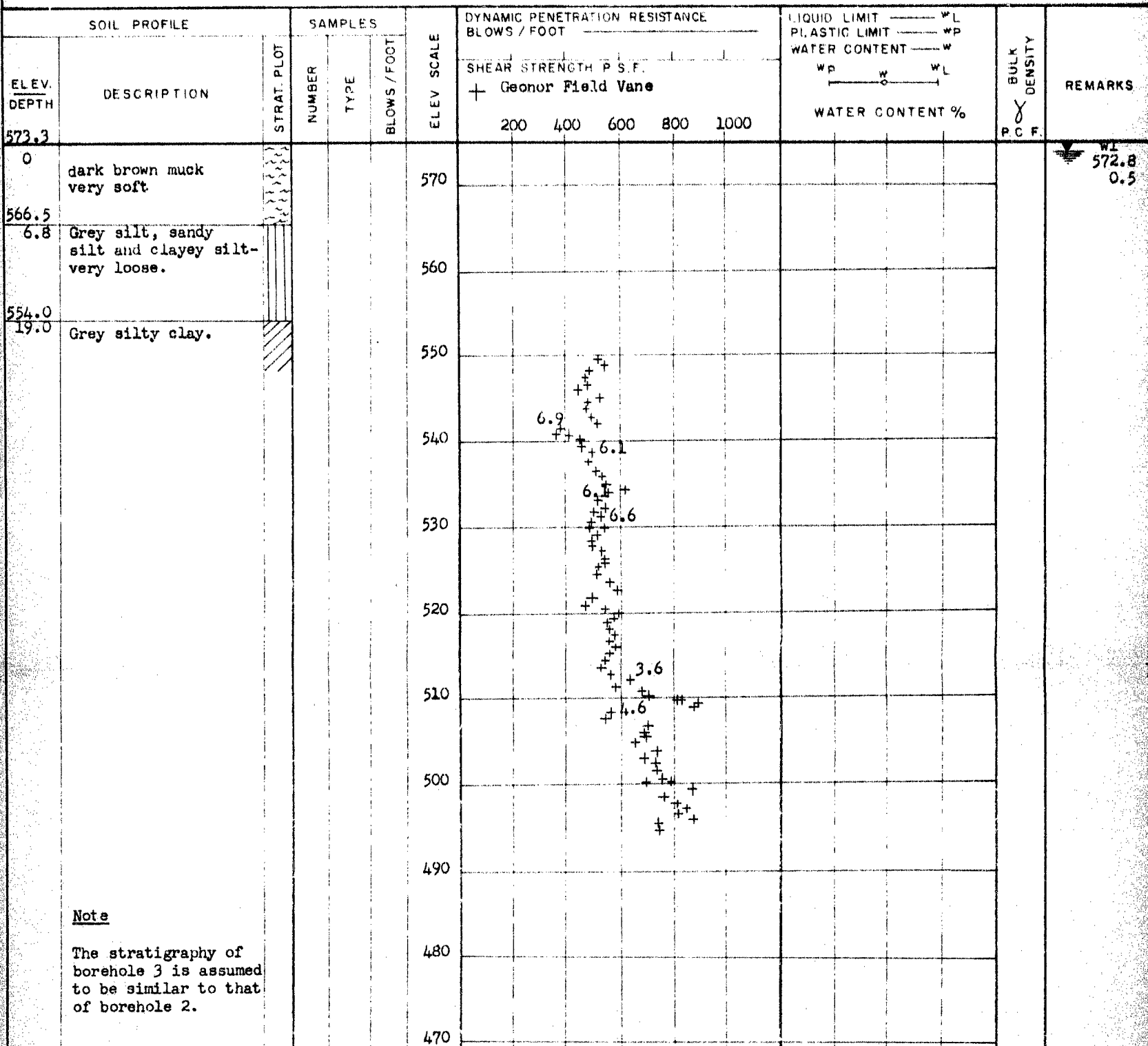
BOREHOLE TYPE Washboring

CHECKED BY K.Y.L.



SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LQUID LIMIT ——— WL	PLASTIC LIMIT ——— WP	WATER CONTENT ——— W	BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.	w p — w — w L			P.C.T.	
							+ Field Vane					
							200 400 600 800 1000	WATER CONTENT %				
573.3	Groundlevel											
0.0	Dark brown muck. V. soft					570						wl 572.8 0.5
566.5	Grey silt, sandy silt and clayey silt-very loose.		1	SS	P	560						
554.0	Grey silty clay.		2	TW	P	550		+ 4.3				
19.0			3	TW	P			+ 3.7				
			4	TW	P	540		+ 3.1				
			5	TW	P			+ 2.4				
			6	TW	P	530		+ 5.2				
			7	TW	P			+ 3.3				
			8	TW	P	520		+ 3.1				
			9	TW	P				+ 4.7			
			10	TW	P	510			+ 2.8			
			11	TW	P			+ 3.6				
			12	TW	P	500			+ 2.9			
			13	TW	P			+ 2.7				
			14	TW	P	490		+ 3.9				
			15	TW	P					+ 5.1		
478.6						480						
94.7	Grey sand with boulders v. dense					470						
464.4												
108.9	Dark grey shale					460						100% Recovery
459.4												
113.9	End of borehole.					450						

ORIGINATED BY R.S.
COMPILED BY B.K.
CHECKED BY K.Y.L.



Same information
as borehole 3.

JOB63-11001

LOCATIONSta. 162 + 23 16' Rt.

ORIGINATED BYFP

W.P.323-61

BORING DATEMay 5 & 6, 1970

COMPILED BYFP

DATUMGeodetic

BOREHOLE TYPEWashbore-BX Casing

CHECKED BY

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT ——— w _L PLASTIC LIMIT ——— w _p WATER CONTENT ——— w		BULK DENSITY Y	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.		w _p ——— w _L WATER CONTENT %			
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE					
579.5	Ground Level										P.C.F. GR. SA. SI. CL.	
0.0	Fill											
	Sandy silt with some clay					570						
						560						
	Silty clay					550						
						540						
						530						
						520						
						510						
						500						
						490						
						480						
477.5						470						
472.0	Sandy fill					460						
	Very dense					450						
460.8	Probable Bedrock					440						
118.7	End of Borehole											

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' -- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE -- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL. THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS:--

<i>COHESIVE</i>			<i>GRANULAR</i>	
CONSISTENCY	'N' BLOWS / FT.	c LB. / SQ. FT.	DENSENESS	'N' BLOWS / FT.
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H.	SAMPLE ADVANCED HYDRAULICALLY	
	P.M.	SAMPLE ADVANCED MANUALLY	

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c	EFFECTIVE COHESION
	INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	$= 3.1416$
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e \sigma$ OR $\ln \sigma$	NATURAL LOGARITHM OF σ
$\log_{10} \sigma$ OR $\log \sigma$	LOGARITHM OF σ TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

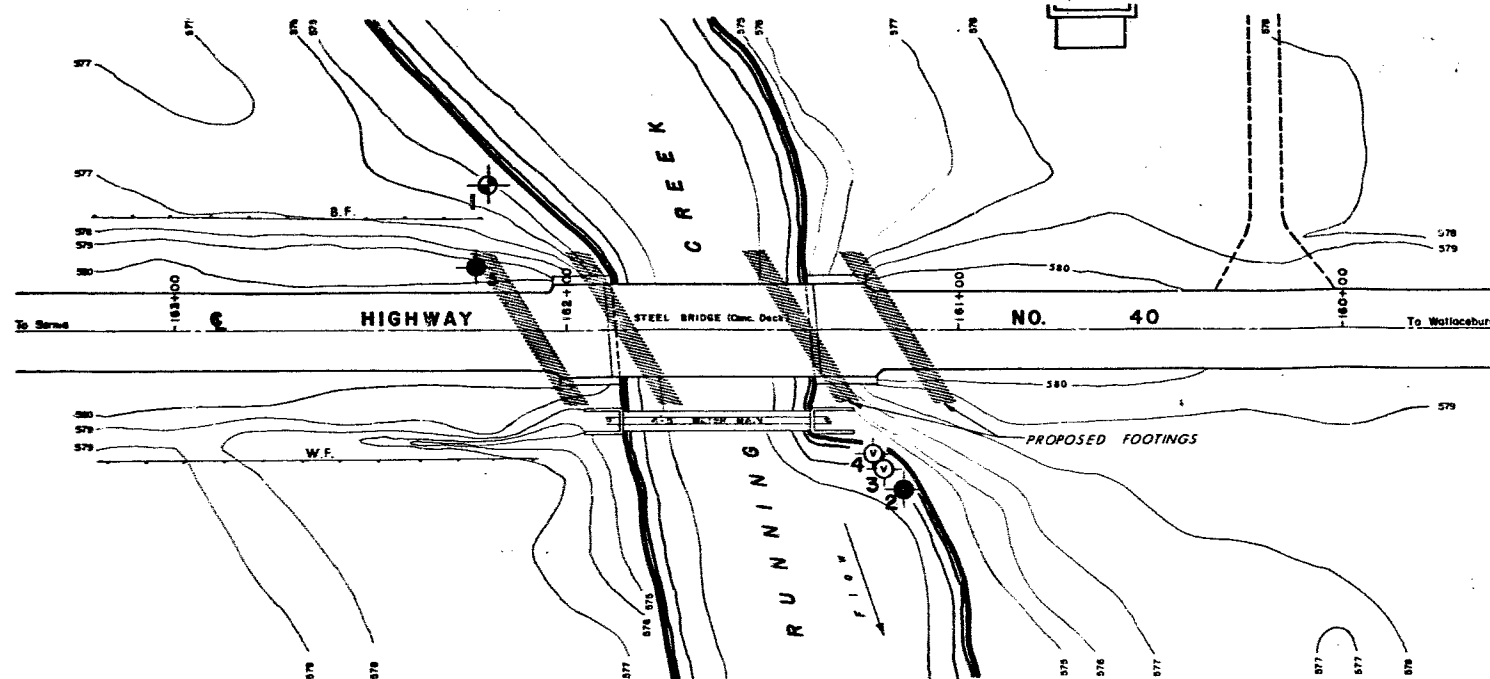
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGL. OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

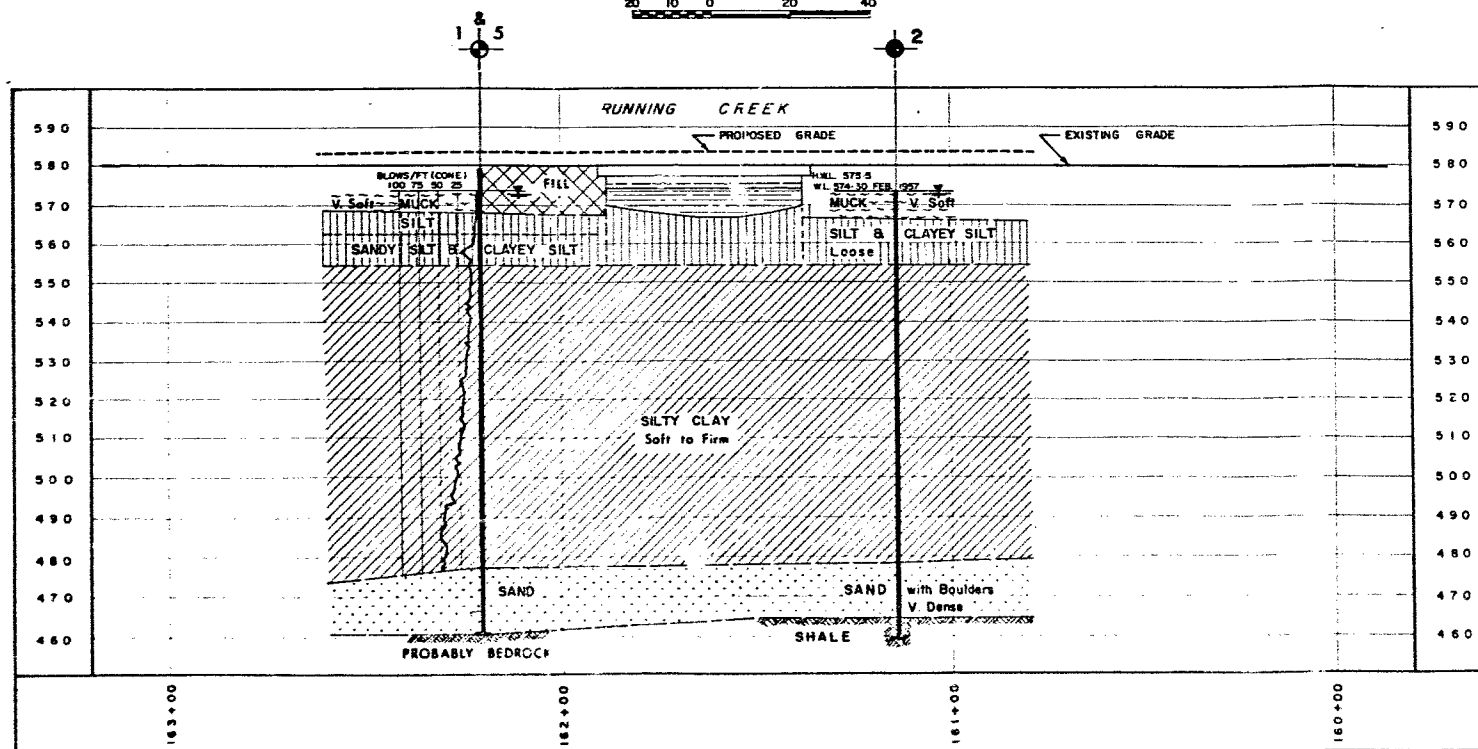
B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

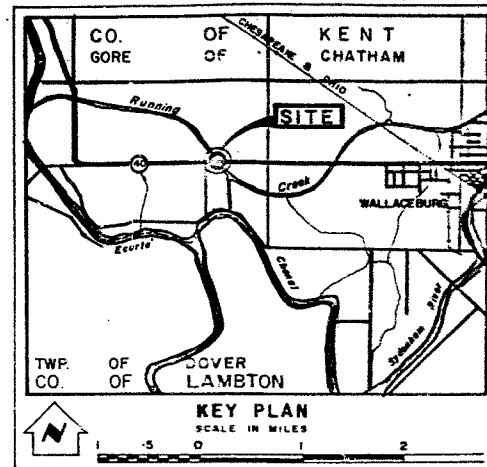


PLAN
SCALE IN FEET
20 10 0 20 40



PROFILE
SCALE IN FEET
20 10 0 20 40

- NOTE -
The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the CHATHAM District Office. The Department does not guarantee the accuracy of this report or the abridged version shown on these plans.



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation (Jan. 1963)		
	Vane Test		

NO.	ELEVATION	STATION	OFFSET
1	573.8	162+20	37' RT.
2	573.3	161+14	41' LT.
3	573.3	161+19	36' LT.
4	573.3	161+22	32' LT.
5	579.5	162+23	18' RT.

- 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
1	MAY 20 63	SR	BORE HOLE NO. 1 ADDED ON PLAN & PROFILE

DEPARTMENT OF HIGHWAYS - ONTARIO	
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION	
RUNNING CREEK	
NO. 2	
KING'S HIGHWAY NO. 40	DIST. NO. 1
CO. KENT	
TWP. GORE OF CHATHAM	LOT 4 CON. I & II
BORE HOLE LOCATIONS & SOIL STRATA	
SUB'D B. K. CHECKED 323-61	MR. D. M. 63-11001A
DRAWN O. M. CHECKED 63-63-F-1	BRIDGE DRAWING NO.
DATE 4 MARCH 1963	
APPROVED <i>[Signature]</i>	

REF. NO. E-4877-1
REF. NO. E-3311-1

Department of Highways Ontario

Copy for the information of

Foundation Office

Mr. A. Sternac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

C.S. Grebski,
Bridge Office

February 9, 1971

Running Creek Bridge #2
1.6 Mi. W. of Wallaceburg W. Limits
W.P. 323-61-00, Site 13-4
Highway 40, District No. 1

63-11-001

Attached herewith we are submitting the final
bridge drawings which show the foundation design for
this structure.

Kindly give us your comments at your earliest
convenience.

C.S. Grebski,
Bridge Design Engineer

CSG:rd

Attach.

c.c. Foundation Office

Pile lengths should be as below

12th Jan
Feb. 19th 1971

LOCATION OF PILES	NO. OF PILES	LENGTH
W. ABUTMENT F.R. EXT.	6	118'
W. ABUTMENT B.R. VENT.	2	114'
W. ABUTMENT E. EXT.	2	115'
E. ABUTMENT F.R. EXT.	6	115'
E. ABUTMENT B.R. VENT.	2	110'
E. ABUTMENT E. EXT.	2	111'

dkl
9 Mar. 71

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac
Mr. A.P. Watt,

Reg. Bridge Planning Engineer,
London Regional Office,
London, Ontario

C.S. Grebaki,
Bridge Office

October 19, 1970

Manning Creek Bridge #2
1.6 Mi. W. of Wallaceburg W. Limits
W.P. 323-61-00, Site 13-4
Highway 40, District No. 1

63-F-1
63-11001

Attached herewith are prints of the Preliminary Bridge Plan Drawing D-6853-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is \$75,000. This cost includes tender, materials, engineering and sundry construction but does not include cost of removal of the existing structure.

Any comments or revisions you may have should be submitted within three weeks.

CSG:rd

C.S. Grebaki,
Bridge Design Engineer

Attach.

c.c. S. McCubie
A. Starnes (2)
J. Anderson
A. Crowley

NO COMMENTS

OCT. 21/70

PP
K. L. S. 