

A. M. Toye,  
Bridge Engineer.  
Materials & Research Section.

June 13, 1960.

D.H.O. FOUNDATION INVESTIGATION  
(W.P. 244-60) -- W.J. 60-7-32.

Attention: Mr. S. McCombie.

Re: Hwy. 500 at Pusey and Grace Lake Junction,  
Township of Cardiff, District No. 11.

Attached to this memo, we are forwarding to you, the Foundation Investigation Report for the above mentioned site. The report has been prepared in our Section. The conclusions and recommendations are self-explanatory and we believe, adequate and sufficient for your future design work.

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

L. G. Soderman,  
PRINCIPAL FOUNDATIONS ENGR.  
Per:

AS/MdeF  
Attach.

(A. Stermac,  
FOUNDATIONS OFFICE ENGR.)

cc: Messrs. A. M. Toye (2)  
E. A. Tregaskes  
D. G. Ramsay  
G. K. Hunter  
H. C. Dernier  
T. J. Kovich  
A. Watt

Foundations Office  
Gen. Files.

# FOUNDATION REPORT

For

Hwy. 500 at Pusey and Grace Lake Junction,  
Township of Cardiff, District 11.

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Station: 8

Plan No: F-340

## Distribution:

Mr. A. M. Teye,  
Bridge Engineer. (2)

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Construction Engineer. (1)

Mr. D. G. Ramsay,  
Road Design Engineer. (1)

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W.J. 60-P-32.

W.P. 244-60.

## I. INTRODUCTION:

This report covers the soil investigation carried out to determine the bearing capacity of the subsoil for supporting the foundations of the proposed culvert.

The original plan was to replace the existing culvert. Later, it was decided to move the centre line about 100 feet to the South.

The site is on Hwy. 500 about one mile North-East of Wilberforce, where Furry Lake and Grace Lake are separated by Hwy. 500 fill embankment and a culvert. Twp. of Cardiff (Station 8 + 25, Profile No. C 1649).

The field work started on April 4, 1960 and was completed on April 26, 1960.

## II. DESCRIPTION OF SITE AND GEOLOGY:

The topography of the country is hilly, covered with forests, and spotted with lakes.

The geology of the area is referred to as the Precambrian Shield, which consists of igneous bedrock. The overburden is reshaped by glacial drifts and spillways, etc.

## III. FIELD AND LABORATORY WORK:

The field work was carried out by means of a core-drill machine adapted for soil sampling.

The original plan was to replace the existing culvert and accordingly, three boreholes were made. Later, it was decided to move the centre line about 100 feet South. This necessitated the extension of the explorations and, in addition, two boreholes and two dynamic cone tests were completed.

The coredrill was mounted on a raft, and the explorations were carried out from this floating raft on the lake.

The sampling was done with a 2" O.D. split-barrelled spoon sampler. The dimensions of the spoon sampler and the energy used in driving it, conform to the requirements of the Standard

### III. FIELD AND LABORATORY WORK: (cont'd.) ...

Penetration Test. In addition, a 2" Diameter cone was dynamically driven from existing ground surface (lake bottom) to refusal depth in order to define a cone resistance profile.

The split spoon samples were visually examined and identified in the field. Some representative samples were brought to the laboratory for further tests.

Laboratory and field test results have been summarized in Table No. 1 and are included under Appendix I. The location of the boreholes is shown on Drawing No. 60-F-32 A.

### IV. SOIL TYPES ENCOUNTERED:

The investigations at the site revealed the following subsoil stratification:-

Under the water, the lake bottom is filled with muck, made up of black sandy clay, clayey sand with decayed organic matter. Underlying this muck layer, is gray, very fine silty sand and sandy silt material. In order to determine the depth of this layer, boreholes (No's. 5 & 6), were driven to a depth of 100 and 120 ft. However, no bedrock was encountered and the depth of the layer was left undefined.

#### 1. Black, Sandy Clay (Muck).

This material is made up of clay and sand and decayed wood which has been deposited at the lake bottom, recently. Dynamic cone penetration resistance registered, indicates that the material is in a loose state. It is more sandy clay at the top, changing to clayey sand by depth. The intersected thickness of this layer was found to range from 3 ft. to 10 ft. (see attached Drawing No. 60-F-32 A).

#### 2. Grey, Fine Silty Sand, Sandy Silt:

This layer was encountered immediately underneath the muck. The explorations were carried about 120 ft. down in search of hard bottom, but having failed, the boring was stopped without defining the true depth of the layer.

IV. SOIL TYPES ENCOUNTERED: (cont'd.) ...

2. Grey, Fine Silty Sand, Sandy Silt: (cont'd.) ...

The material is made up of clayey silt and fine sand. The textural analysis indicated an average of 70% silt, and 30% of fine sand. The material is non-plastic and in a loose state. The Standard Penetration Test results in the upper 40 ft. of the stratum, indicated an average of 7 blows per foot. Below that, the average was 18 blows per foot, indicating higher relative density.

V. FOUNDATION CONSIDERATIONS:

The relatively loose state of the upper horizon of the grey fine silty sand layer makes spread footing foundations impractical. The relatively denser state of the lower horizon of this layer indicated by the Standard Penetration Tests and the dynamic cone penetration refusals, lead us to recommend the use of piles as a means of support.

For this purpose, either wooden piles or steel tube displacement piles, will be suitable. Steel tube piles of 12" diameter, 28 lbs./foot and 60 ft. long, can provide 35 tons safe bearing value ( $P_s = 3.0$ ) at 50 blows per foot penetration with a Delmag 12 Hammer (2200 ft./lbs. energy). Actually, the bearing capacity of 35 tons of these piles is theoretically obtained already at 36 blows per foot penetration, but, because of the silty character of the soil, the criterion has been raised to 50 blows. After the dissipation of excess pore water pressures which occur during pile driving, the number of blows per foot penetration will decrease and eventually become equal to the required theoretical value.

This load value could be checked during the pile driving operation. In case, the required set of .24 inches per blow could not be attained at the indicated elevation, further driving of the piles will be necessary.

cont'd. /4 ...

VI. SUMMARY AND RECOMMENDATIONS:

From the above discussion, it will follow that:-

1. The site is located at the junction of two small lakes, which are separated by Hwy. 500 fill embankment and a culvert.
2. The lake bottom is covered with black sandy clay and decayed organic matter varying from 2 ft. to 10 ft. in thickness. It is underlain by a deposit of grey, non-plastic, fine silty sand and sandy silt material. The boreholes were terminated in this layer, at a depth of 120 ft.
3. The loose state of the subsoil makes the use of spread footing support impractical. It is recommended to support the footings on displacement piles. These piles could be either wooden piles or steel tube piles. It is estimated that 60 ft. of 12" diameter steel tube piles of 28 lbs. per foot weight, could provide 35 tons bearing capacity at a set of .24 inches per blow, or 50 blows per foot penetration with a Delmag 12 Hammer exerting 2200 ft. lbs. energy per blow. It is assumed that this set value will be reached at about 55 - 60 ft. depth. If not, further driving of the piles will be necessary until the indicated set value is attained.
4. No change in the alignment or elevation of the existing grade line is indicated. Accordingly, no fill embankment stability problem is anticipated.

June 1960.

REPORT PREPARED BY: V. Korlu,  
PROJECT PDN. ENGR.

REPORT APPROVED BY:   
PDNS. OFFICE ENGR.

APPENDIX I.

## SUMMARY OF FIELD &amp; LABORATORY TESTS

JOB 60-F-32W.P. 244-60

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
1										Omitted
2A	S1	10.5'-12.5'	Black sandy clay (decayed matter)	3	-	-	-	-	-	
	S2	15.5'-17.5'	Grey very fine silty sand, sandy silt.	2	-	-	-	-	-	
	S3	21'-23'	" " " "	5	-	-	-	-	-	
	S4	26'-28'	" " " "	5	-	-	-	-	-	
	S5	31'-33'	" " " "	4	-	-	-	-	-	
	S6	36'-38'	" " " "	6	-	-	-	-	-	
	S7	43'-45'	" " " "	5	-	-	-	-	-	
	S8	48'-49.9'	" " " "	4	-	-	-	-	-	
	S9	54'-56'	" " " "	16	-	-	-	-	-	
	S10	58.5'-60.5'	" " " "	3	-	-	-	-	-	
3	S1	12'-14'	Grey very fine silty sand, sandy silt.	3	-	-	-	-	-	
	S2	15'-17'	" " " "	4	-	-	-	-	-	
	S3	20'-22'	" " " "	6	-	-	-	-	-	



## SUMMARY OF FIELD &amp; LABORATORY TESTS

JOB 60-F-32W.P. 244-60

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
3	S4	25'-27'	Grey very fine silty sand, sandy silt.	6	-	-	-	-	-	
	S5	35'-37'	" " " "	6	-	-	-	-	-	
	S6	46'-48'	" " " "	10	-	-	-	-	-	
	S7	52.5'-54.5'	" " " "	32	-	-	-	-	-	
4	S1	7'-9'	Black brown clayey sand (decayed matter)	1	-	-	-	-	-	
	S2	11'-13'	" " " "	5	-	-	-	-	-	
	S3	16'-18'	Grey very fine silty sand, sandy silt.	3	-	-	-	-	-	
	S4	21'-23'	" " " "	4	-	-	-	-	-	
	S5	31'-33'	" " " "	4	-	-	-	-	-	
	S6	41.5'-43.5'	" " " "	7	-	-	-	-	-	
	S7	53'-55'	" " " "	14	-	-	-	-	-	
5	S1	10.7'-12.7'	Black brown clayey sand (decayed matter)	4	-	-	-	-	-	
	S2	14.9'-16.9'	Grey very fine silty sand, sandy silt.	5	-	-	-	-	-	

## SUMMARY OF FIELD &amp; LABORATORY TESTS

JOB 60-F-32

W.P. 244-60

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
5	S3	19.2'-21.2'	Grey very fine silty sand, sandy silt.	8	-	-	-	-	-	
	S4	24'-26'	" " " "	3	-	-	-	-	-	
	S5	34'-36'	" " " "	4	-	-	-	-	-	
	S6	44'-46'	" " " "	4	-	-	-	-	-	
	S7	54'-56'	" " " "	29	-	-	-	-	-	
	S8	65.7'-67.7'	" " " "	11	-	-	-	-	-	
	S9	75'-77'	" " " "	22	-	-	-	-	-	
	S10	85'-87'	" " " "	9	-	-	-	-	-	
	S11	95.3'-97.3'	" " " "	11	-	-	-	-	-	
	T12	104.5'-106.5'	" " " "	24	-	-	-	-	-	

## SUMMARY OF FIELD &amp; LABORATORY TESTS

JOB 60-F-32W.P. 244-60

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT	MOIST. CONT. %	ELASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
6 7										Cones only
8	S1	14'-15.5'	Grey very fine silty sand, sandy silt.	7	22.6	-	-	-	-	
	S2	24'-25.5'	" " " " "	5	22.8	-	-	-	-	
	S3	34'-35.5'	" " " " "	7	25.4	-	-	-	-	
	S4	45.5'-47'	" " " " "	7	22.8	-	-	-	-	
	S5	54.5'-56'	" " " " "	2	22.4	-	-	-	-	
	S6	70'-71.5'	" " " " "	16	22.2	-	-	-	-	
			S denotes split spoon sample T " shelby tube "							

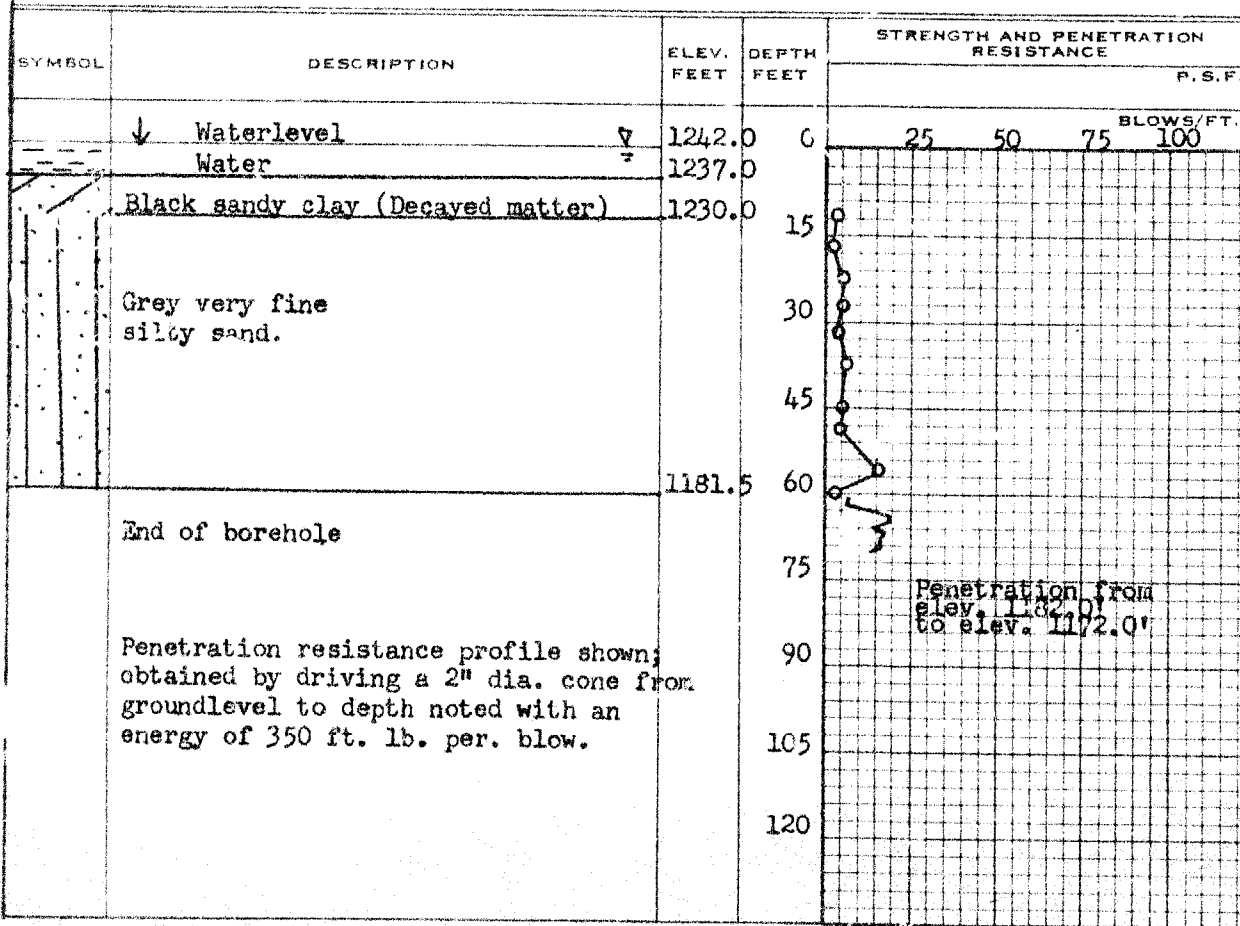
# DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 244-60 BORE HOLE NO. 2  
 JOB 60-F-32 STATION 9+45 (21' Lt.)  
 DATUM 1242.0' COMPILED BY B. K.  
 BORING DATE April 6/60 CHECKED BY V. K.

2" DIA. SPLIT TUBE  
 2" SHELBY TUBE  
 2" SPLIT TUBE  
 2" DIA. CONE  
 2" SHELBY  
 CASING

## LEGEND

1/2 UNCONFINED COMPRESSION ( $Q_u$ )  
 VANE TEST ( $C$ ) AND SENSITIVITY ( $S$ )  
 NATURAL MOISTURE AND LIQUIDITY INDEX  
 LIQUID LIMIT  
 PLASTIC LIMIT



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.		
	S1	-
	S2	-
	S3	-
	S4	-
	S5	-
	S6	-
	S7	-
	S8	-
	S9	-
	S10	-

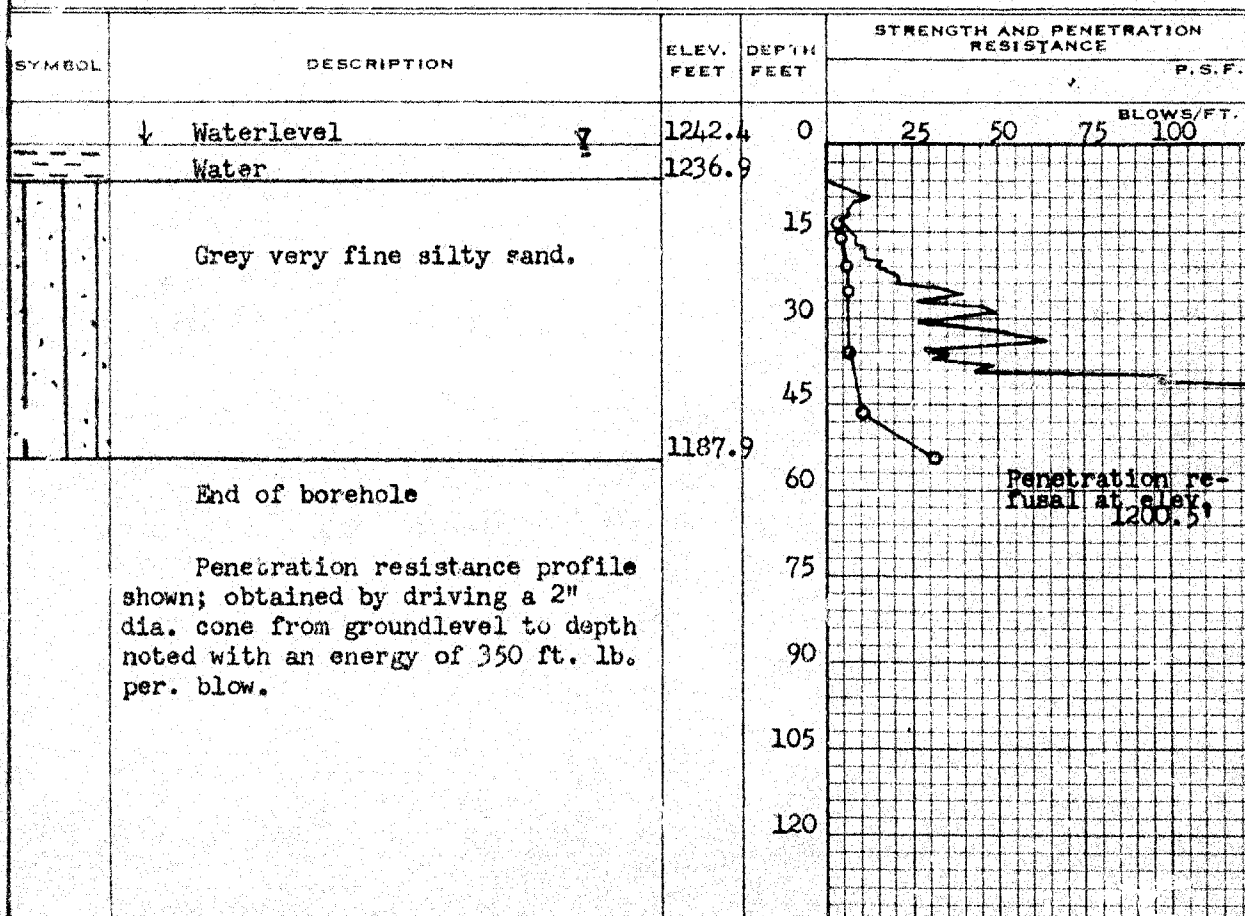
# DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 244-60 BORE HOLE NO. 3JOB 60-F-22 STATION 9+00 (21' Lt.)DATUM 1242.4' COMPILED BY B. K.BORING DATE April 11/60 CHECKED BY V. K.

2" DIA. SPLIT TUBE \_\_\_\_\_ ☒  
 2" SHELBY TUBE \_\_\_\_\_ ☒  
 2" SPLIT TUBE \_\_\_\_\_ ☐  
 2" DIA. CONE \_\_\_\_\_ ☐  
 2" SHELBY \_\_\_\_\_ ☐  
 CASING \_\_\_\_\_ ☒ ☒

## LEGEND

1/2 UNCONFINED COMPRESSION (Qu) \_\_\_\_\_ O  
 VANE TEST (C) AND SENSITIVITY (S) \_\_\_\_\_ +  
 NATURAL MOISTURE AND LIQUIDITY INDEX \_\_\_\_\_ LI  
 LIQUID LIMIT \_\_\_\_\_ X  
 PLASTIC LIMIT \_\_\_\_\_



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	S1	-
	S2	-
	S3	-
	S4	-
	S5	-
	S6	-
	S7	-

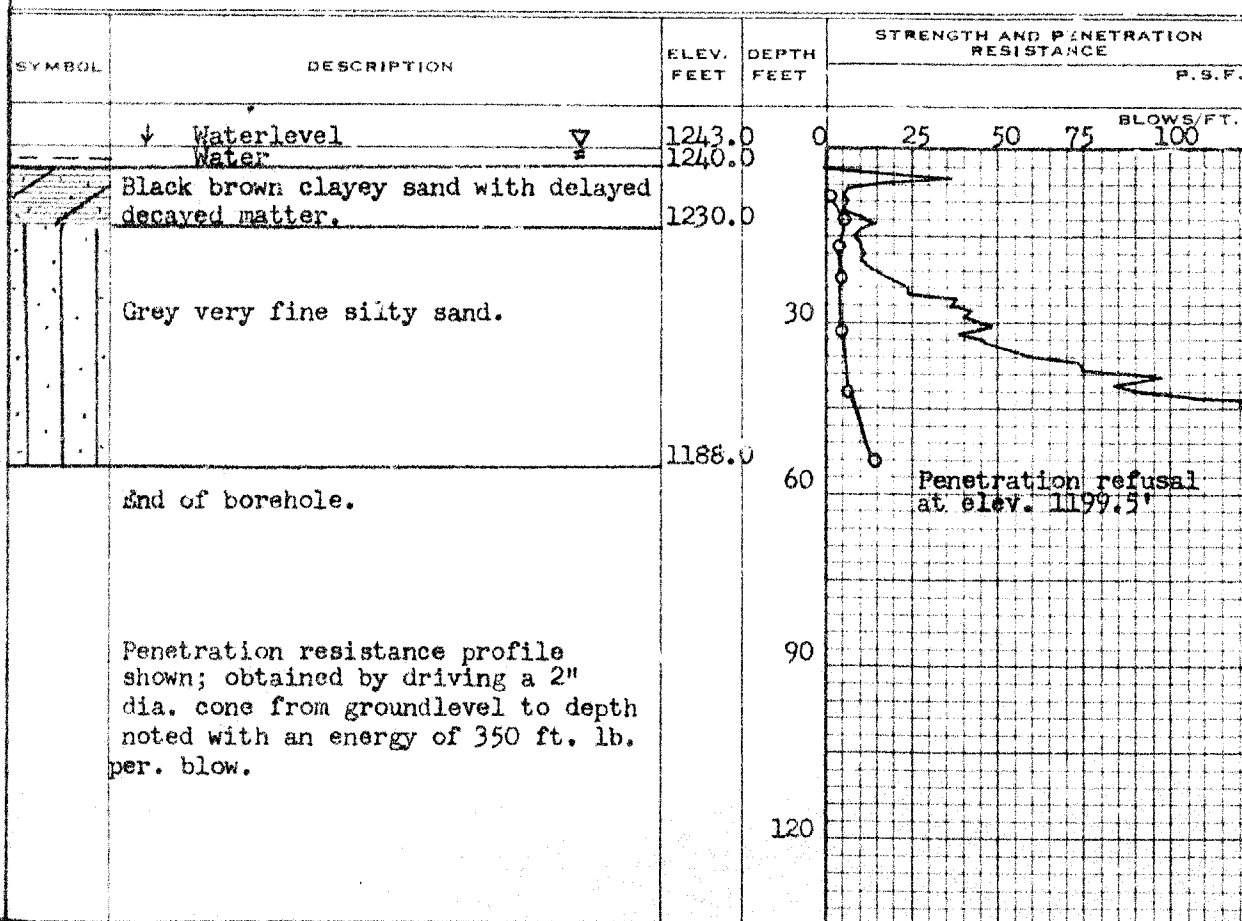
# DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 244-60 BORE HOLE NO. 4  
JOB 60-F-32 STATION 9+15 (26' Rt.)  
DATUM 1243.0' COMPILED BY B. K.  
BORING DATE April 12/60 CHECKED BY V. K.

2" DIA. SPLIT TUBE  
2" SHELBY TUBE  
2" SPLIT TUBE  
2" DIA. CONE  
2" SHELBY  
CASING

## LEGEND

1/2 UNCONFINED COMPRESSION ( $Q_u$ )  
VANE TEST (C) AND SENSITIVITY (S)  
NATURAL MOISTURE AND LIQUIDITY INDEX  
LIQUID LIMIT  
PLASTIC LIMIT



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	S1	-
	S2	-
	S3	-
	S4	-
	S5	-
	S6	-
	S7	-

# DEPARTMENT OF HIGHWAYS - ONTARIO

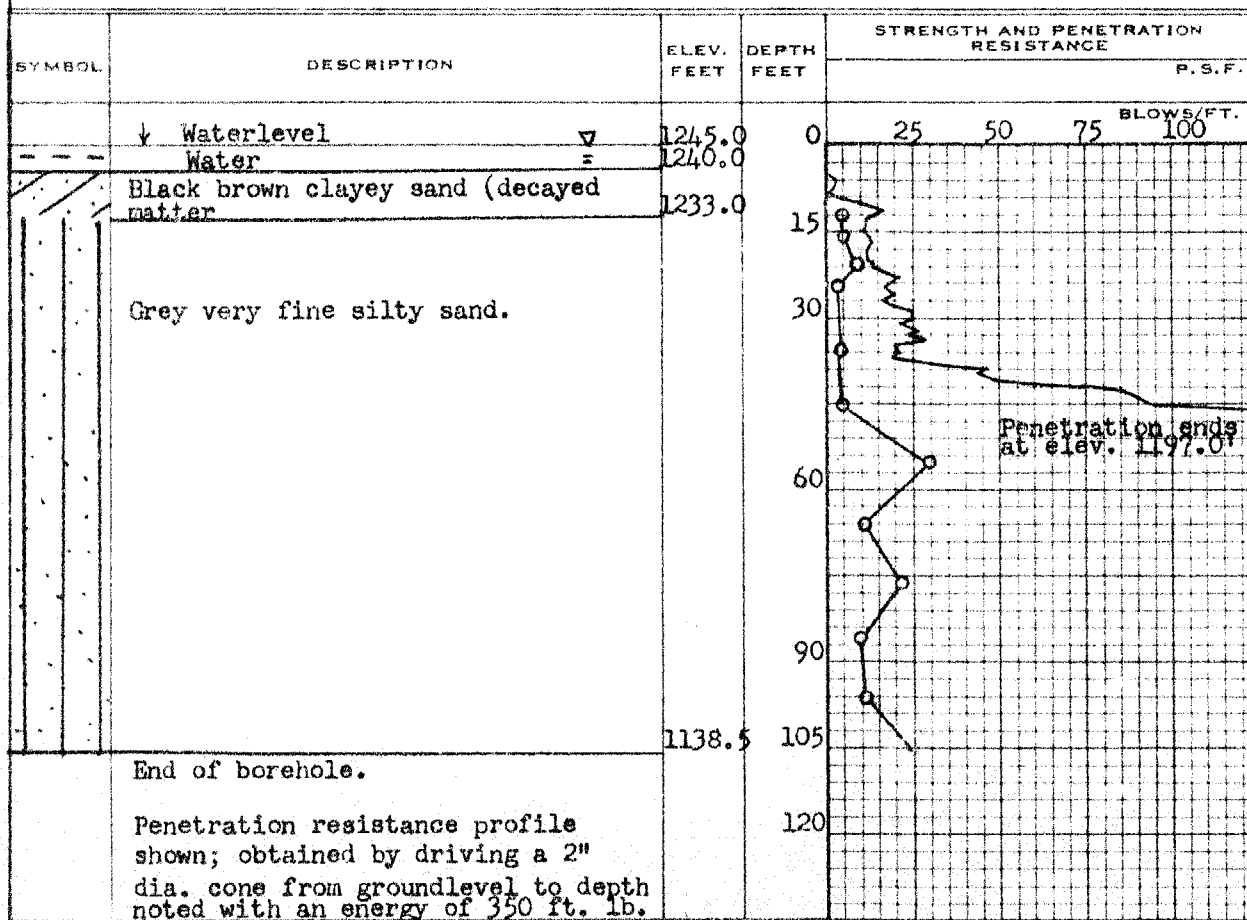
## MATERIALS AND RESEARCH SECTION

W.P. 244-60 BORE HOLE NO. 5  
 JOB 60-F-32 STATION 8+10 (42' Rt.)  
 DATUM 1245.0' COMPILED BY B. K.  
 BORING DATE April 13/60 CHECKED BY V. K.

2" DIA. SPLIT TUBE \_\_\_\_\_  
 2" SHELBY TUBE \_\_\_\_\_  
 2" SPLIT TUBE \_\_\_\_\_  
 2" DIA. CONE \_\_\_\_\_  
 2" SHELBY \_\_\_\_\_  
 CASING \_\_\_\_\_

### LEGEND

1/2 UNCONFINED COMPRESSION ( $Q_u$ ) \_\_\_\_\_  
 VANE TEST (C) AND SENSITIVITY (S) \_\_\_\_\_  
 NATURAL MOISTURE AND LIQUIDITY INDEX \_\_\_\_\_  
 LIQUID LIMIT \_\_\_\_\_  
 PLASTIC LIMIT \_\_\_\_\_



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	S1	-
	S2	-
	S3	-
	S4	-
	S5	-
	S6	-
	S7	-
	S8	-
	S9	-
	S10	-
	S11	-
	T12	-

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS AND RESEARCH SECTION

W.P. 244-60	BORE HOLE NO. 6
JOB 60-F-32	STATION 840 (50' Rt.)
DATUM 1245.0'	COMPILED BY B. K.
BORING DATE Apr. 21/60	CHECKED BY V. K.

2" DIA. SPLIT TUBE  
2" SHELBY TUBE  
2" SPLIT TUBE  
2" DIA. CONE  
2" SHELBY  
CASING

### LEGEND

1/2 UNCONFINED COMPRESSION (Qu) \_\_\_\_\_ O  
VANE TEST (C) AND SENSITIVITY (S) \_\_\_\_\_ +  
NATURAL MOISTURE AND LIQUIDITY INDEX \_\_\_\_\_ LI  
LIQUID LIMIT \_\_\_\_\_ X  
PLASTIC LIMIT \_\_\_\_\_

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE			
				P. S. F.			
↓	Waterlevel	1245.0	0	25	50	75	100
---	Water	1238.0		BLOWS/FT.			
			30	Penetration ends at elev. 1204.0'			
			60				
			90				
			120				

Penetration resistance profile shown; obtained by driving a 2" dia. cone from groundlevel to depth noted with an energy of 350 ft. lb. per. blow.

[illegible]



DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS AND RESEARCH SECTION

W. P. 244-60 \_\_\_\_\_ BORE HOLE NO. 7 \_\_\_\_\_

JOB 60-F-32 STATION 8-40 (50' Lt.)

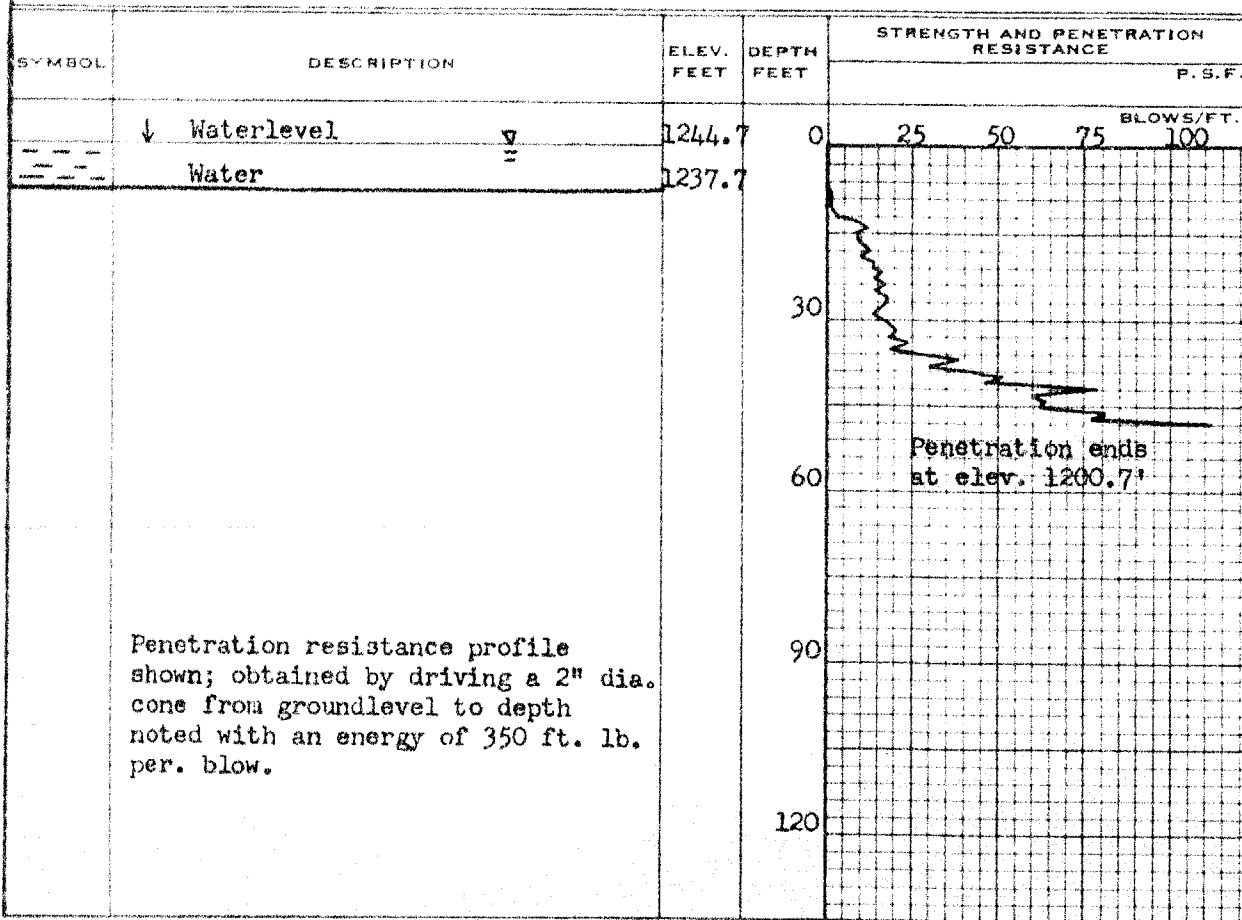
DATUM 1244.7' \_\_\_\_\_ COMPILED BY B. K. \_\_\_\_\_

BORING DATE Apr. 29/60 CHECKED BY V. K.

2" DIA. SPLIT TUBE  
2" SHELBY TUBE  
2" SPLIT TUBE  
2" DIA. CONE  
2" SHELBY  
CASING

### LEGEND

1/2 UNCONFINED COMPRESSION (Qu) ———— O  
VANE TEST (C) AND SENSITIVITY (S) ———— +<sup>s</sup>  
NATURAL MOISTURE AND  
LIQUIDITY INDEX ———— LI  
LIQUID LIMIT ———— X  
PLASTIC LIMIT ————

[illegible]

W.P. 244-60 \_\_\_\_\_ BORE HOLE NO. 8 \_\_\_\_\_  
JOB 60-F-32 \_\_\_\_\_ STATION 8/10 (50' Lt.) \_\_\_\_\_  
DATUM 1245.0' \_\_\_\_\_ COMPILED BY B. K. \_\_\_\_\_  
BORING DATE Apr. 26/60 \_\_\_\_\_ CHECKED BY V. K. \_\_\_\_\_

2" DIA. SPLIT TUBE  
2" SHELBY TUBE  
2" SPLIT TUBE  
2" DIA. CONE  
2" SHELBY  
CASING

1/2 UNCONFINED COMPRESSION (QU)	---	O
VANE TEST (C) AND SENSITIVITY (S)	---	+ <sup>s</sup>
NATURAL MOISTURE AND		
LIQUIDITY INDEX	---	LI
LIQUID LIMIT	---	X
PLASTIC LIMIT	---	

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE			
				P. S. F.	BLOWS/FT.		
	↓ Waterlevel	1245.0	0	25	50	75	100
	Water	1237.6					
	Black clayey sand (decayed matter)	1234.0					
	Grey very fine silty sand.						
		1123.0	120				
	End of borehole,						

CONSISTENCY		SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.			
X		S1	-
X		S2	-
X		S3	-
X		S4	-
X		S5	-
X		S6	-

G.I-30 SEPT 1976

GEOCRES No. 31E-01DIST. 10 REGION EasternW.P. No. 76-73-01CONT. No. 78-08

W. O. No. \_\_\_\_\_

STR. SITE No. 40-52HWY. No. 648LOCATION Wilberforce Bridge

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 4REMARKS ① Documents to be unfolded  
before microfilming② TO BE ADDED TO EXISTING MICROFILM



## Memorandum

To: Mr. K.G. Bassi  
Head, Eastern Section  
Structural Office  
2nd Floor, West Building

Attention:

Mr. C. Farrell  
Our File Ref.

From: Soil Mechanics Section  
Engineering Materials Office  
Room 315, Central Building

Date: 78 08 22 ,

In Reply to

Subject:

Re: Wilberforce Bridge Site, Contract 78-08

On 78 08 16 this Section was informed of possible pile movement at the Wilberforce Bridge site, Contract 78-08 by Mr. C. Farrell and Mr. B. Hashizume of the Structural Office. A meeting between the two Offices was held the following day to further discuss the situation. In order to gain firsthand information and better assess the situation, the author visited the site on 78 08 18 and observed the following:

- i) The pile butts of the south pier bent had moved laterally towards the south abutment and settled vertically. Likewise, the north pier piles had moved towards the north abutment and settled. The magnitude of movements are shown on the attached diagram. Tension cracks in the soil surrounding the pier piles indicated the piles had moved laterally towards the abutments.
- ii) The author could not inspect the abutment piles since the pile caps had already been poured and the south abutment wall was in the process of being poured.

Personal communication with the on site construction field staff Alvin Armstrong, Project Supervisor and Pat O'Connor, Bridge Inspector, verified the following:

- i) The abutment piles were driven according to contract drawings with no apparent problem. The field staff considered the abutment piles to be sound and, therefore, had approved the placement of the abutment pile caps.
- ii) The construction sequence for driving and pouring of concrete for the tube piles is shown on the attached bar chart. All piles were cut off to the required elevations.
- iii) Although no complete driving records were taken, the driving resistance for the length of the piles averaged from 8 to 10 blows/foot. Resistance at the required tip elevation averaged 10 blows/foot.
- iv) Initial movement of the pier piles was observed in pile No. 19 during driving of abutment batter pile No. 10. Driving resistance increased slightly for pile No. 10

cont'd.....

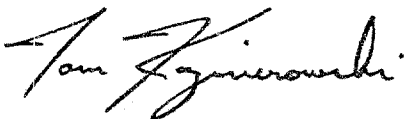
between 40 to 45 feet; this corresponds to a greater depth than when pile No. 10 commenced movement. No apparent increase in driving resistance was noted for any of the other abutment batter piles

- v) All pier piles commenced movement only during the driving of the abutment batter piles. No relative movement occurred after driving the abutment piles.
- vi) No movement of the abutment piles had occurred during driving the adjacent piles except for pile No. 6 which settled 7 inches during the driving of pile No. 7.
- vii) Slight movement of the south pier piles was observed during the driving of the north abutment batter piles.

Considering the magnitude of lateral and vertical movement of the pier piles and the sensitivity of the structure to differential settlement, concern was expressed whether the piles would settle excessively under the design loading.

Since no unusual movement or problems of the abutment piles were reported by the construction field staff, we feel these piles will behave as designed. In order to verify the load-movement behaviour of the displaced pier piles, it is recommended to carry out a full scale load testing program on selected pier piles.

Details of the testing program were discussed at a meeting with Bill Hashizume and Augustine Liu on 78 08 21. It was agreed that reaction system design, contractor arrangement, and region/district liaison would be carried out by the Structural Office and that the Soil Mechanics Section would supervise the load testing operations.



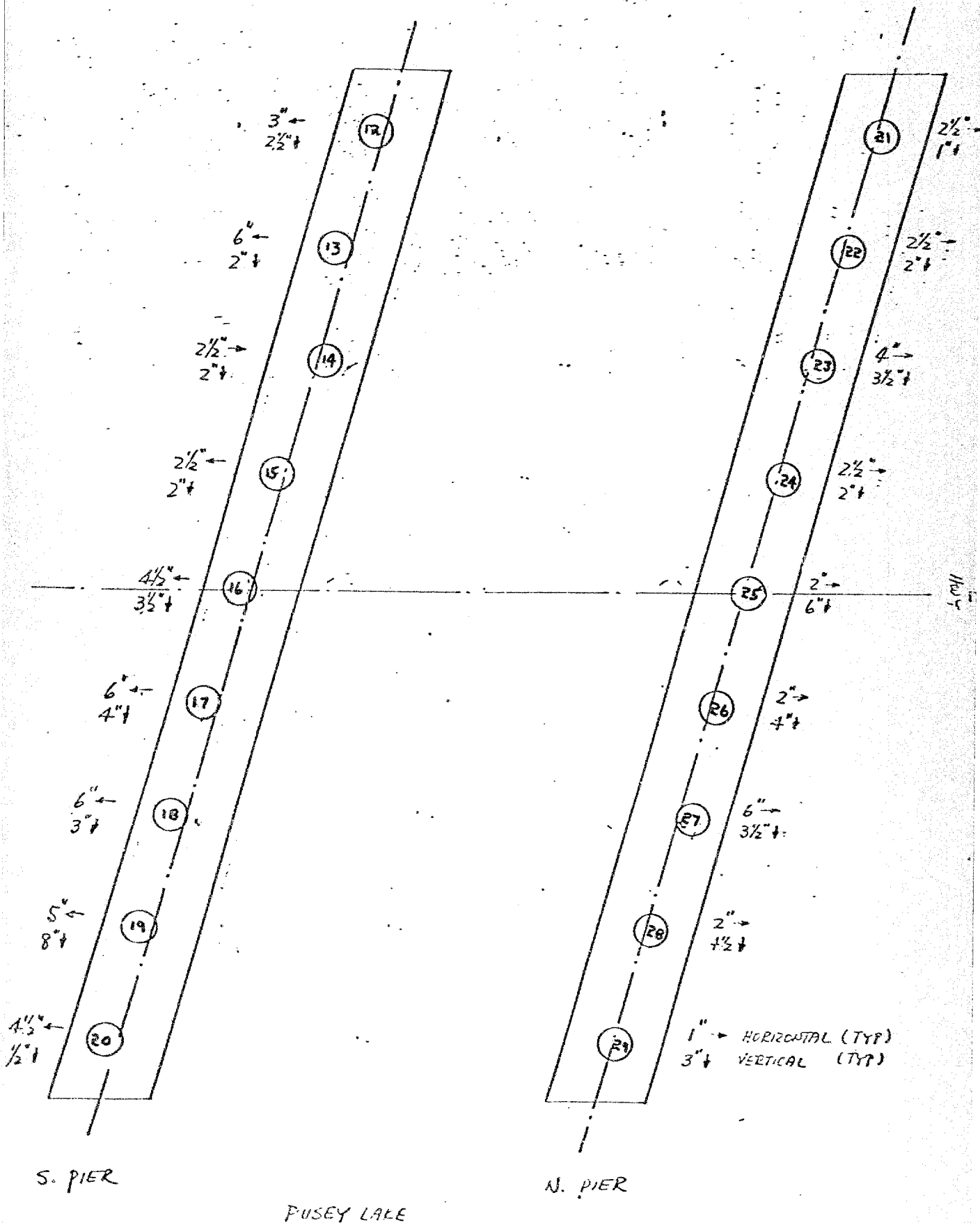
T. Kazmierowski  
Project Engineer

TK/gs

Attach.

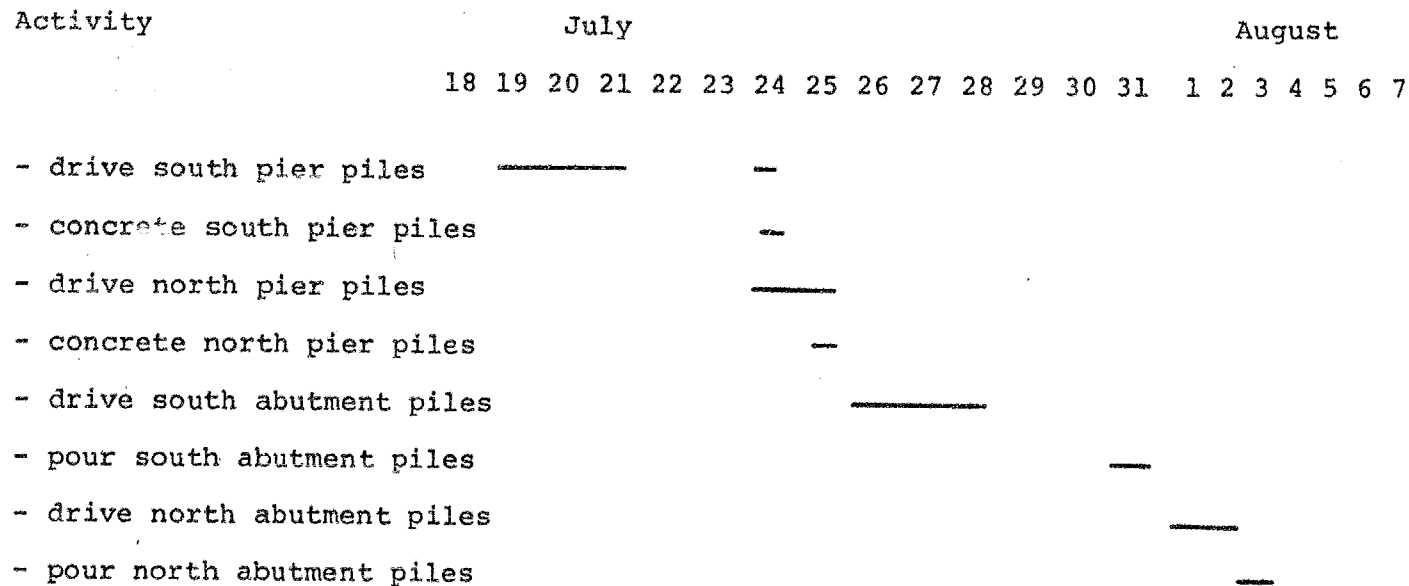
cc: W. Hashizume  
A. C-Y. Liu  
T.C. Kingsland  
R.W. Franks  
Files ✓

# Measured Pile Movements Contract 78-08



# BAR CHART OF CONSTRUCTION SEQUENCE

Contract 78-08



Pile Details  
Contract 78-08

Contract 78-CF



08-02 PILE DRIVEN AUG 2  
7' PILE LENGTH LEFT IN PLACE  
3/FT NO. OF BLOWS PER FOOT

DB-01  
57' 10/ET

08-01  
57' 13/FT



Mr. K.G. Bassi  
Head, Eastern Section  
Structural Office  
2nd Floor, West Building

Soil Mechanics Section  
Engineering Materials Office  
Room 315, Central Building

78 09 22

Mr. W. Hashizume

Re: Wilberforce Bridge Site, Contract 78-08

The construction of the 3 span structure at the above mentioned site is proceeding with work performed by Gaffney Construction under the supervision of M.T.C. Regional Construction personnel. During driving of the abutment batter piles into the loose to compact sandy silt underlying the site, movement of the previously driven north and south pier piles was observed. The magnitude of these movements ranged to a maximum of 6 inches laterally and 3 inches vertically. No further movement of these pier piles were observed upon completion of pile driving operations. Considering the magnitude of lateral and vertical movement and the sensitivity of the structure to differential settlement, concern was expressed whether the piles would settle excessively under the design load. The Soil Mechanics Section was made aware of this problem on 78 08 16 and on 78 08 21 it was recommended to carry out a full scale load testing program on selected pier piles in order to verify the pier pile performance under actual load.

We have now completed a full scale load testing program of six selected pier piles at the above mentioned site. Testing was carried out under the supervision of the Soil Mechanics Section during the period of 78 09 13 to 78 09 19. The selected test piles, numbers 15, 18, 19, 23, 25, 27, were loaded to maximum loads ranging between 50 and 60 tons using a 200 ton jack according to standard A.S.T.M. procedures. Measurements of pile butt movements were taken with four dial gauges, accurate to 0.01 millimeter, mounted parallel to the longitudinal axis of the pile. Results of these measurements are plotted on Load/Movement curves appended to this memorandum. As well, measurements of the batter of each pier pile can be found in chart form in the appendix.

Based on the results of the pile load test, we feel that the ultimate bearing capacity of the pier piles can be conservatively estimated at approximately 45 to 50 tons with a maximum pile movement of 6.0 mm (0.25 inches) and net settlement of 4.0 mm. The safe allowable design load per pile should be limited to 22.5 tons which would result in negligible elastic pile movements in the range of 1 mm.

cont'd.....

Due to the shift in the centerline of the pier caps and eccentricity in loading of the piles as a result of the batter of the various piles, we feel that the original pier cap drawings should be reviewed by your office and redesigned as required. We trust that the above information will be sufficient for design purposes and, hence, have no intention of submitting a formal report.

If any additional information and/or clarification are required, please do not hesitate to call this Section.

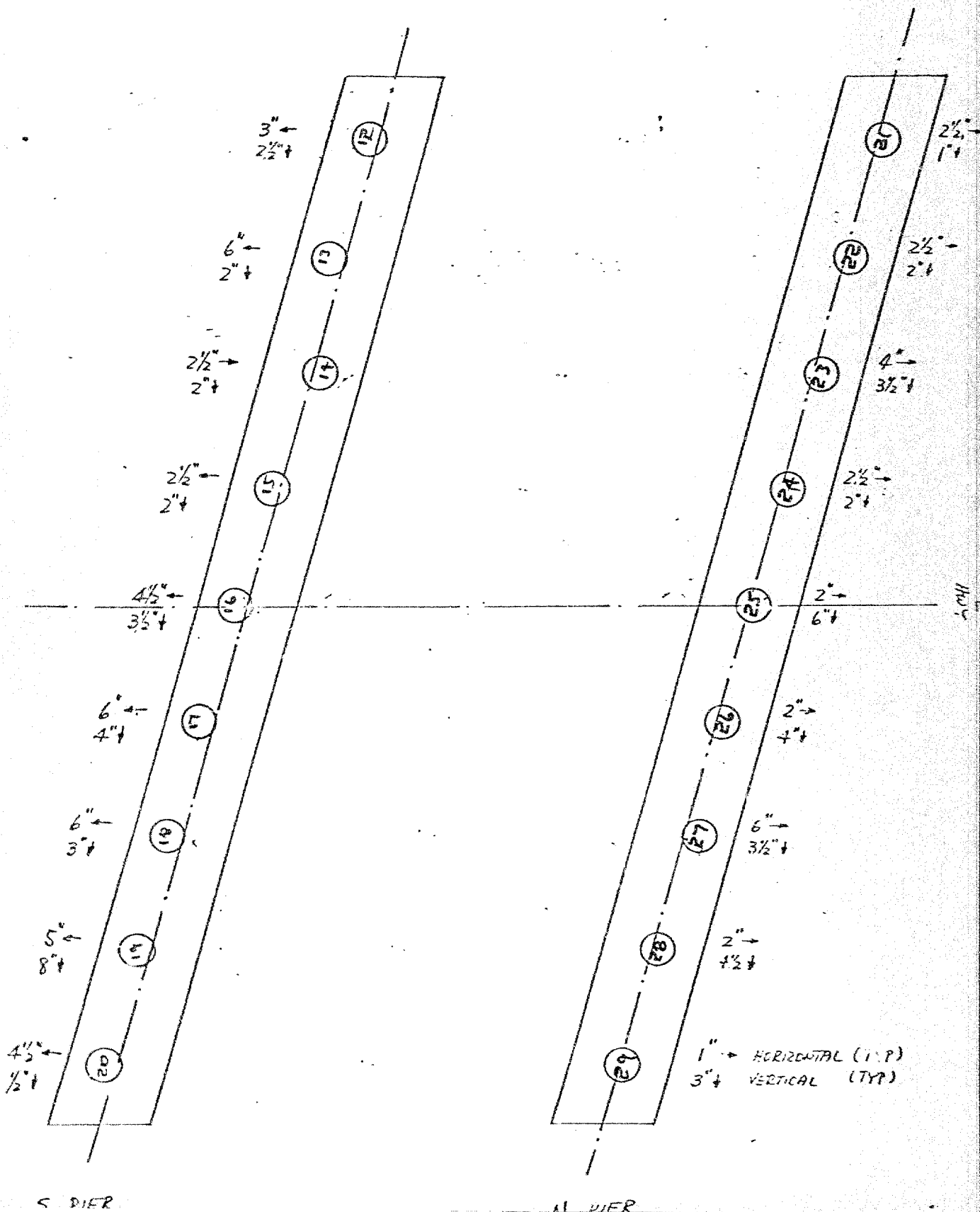
T. Kazmierowski  
Project Engineer

TK/gs

Attach.

cc: A.C-Y. Liu  
R.W. Franks  
Files ✓

# Measured File Movements Contract 78-C8



Contract 78-08  
Measured Batter of Pier Piles

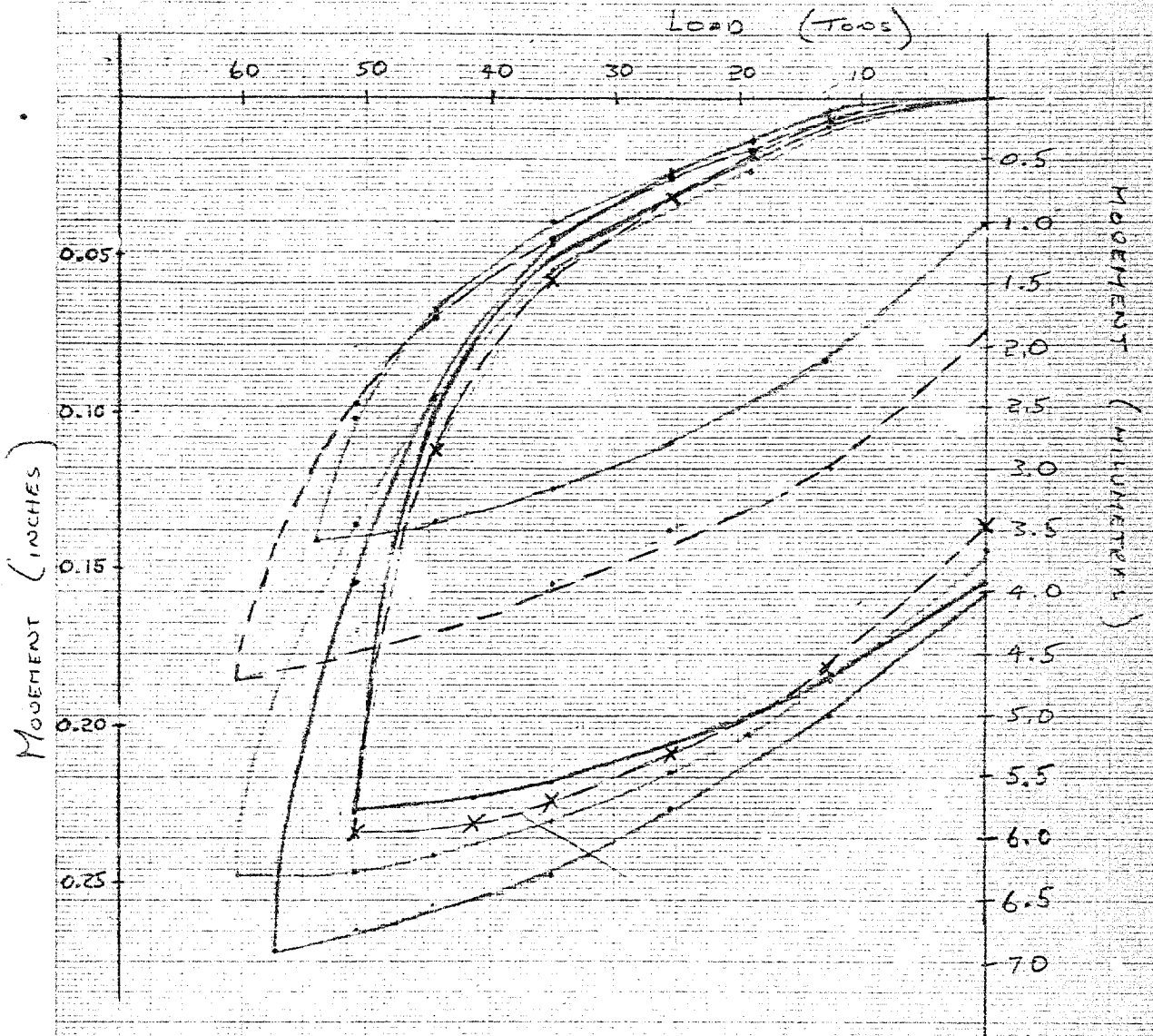
South Pier Bent (No.)	Pile But Leaning at a Batter (Horizontal:Vertical) Towards the			
	North	South	East	West
12 (Batter Pile)	1:29		1:2.8	
13		1:112		
14	1:21		1:29	
15				1:36
16		1:48	1:21	
17		1:41		1:36
18		1:29		1:24
19		1:72		1:29
20 (Batter Pile)				1:3.5
North Pier Bent (No.)				
	North	South	East	West
21 (Batter Pile)			1:3.3	
22	1:48		1:36	
23				
24	1:58			1:26
25			1:48	
26				1:72
27	1:36		1:144	
28	1:288		1:288	
29 (Batter Pile)		1:72		1:3.5

## Ultimate Bearing Capacity of Test Piles

<u>Test Pile</u>	<u>Load (tons) at Which Pile Behaviour Deviates From Straight-Line Elastic Deformation</u>
<b>South Pier Bent</b>	
15	50
18	50
19	45
<b>North Pier Bent</b>	
23	45
25	50
27	50

Test Pile # 18  
 Test Pile # 15  
 Test Pile # 19

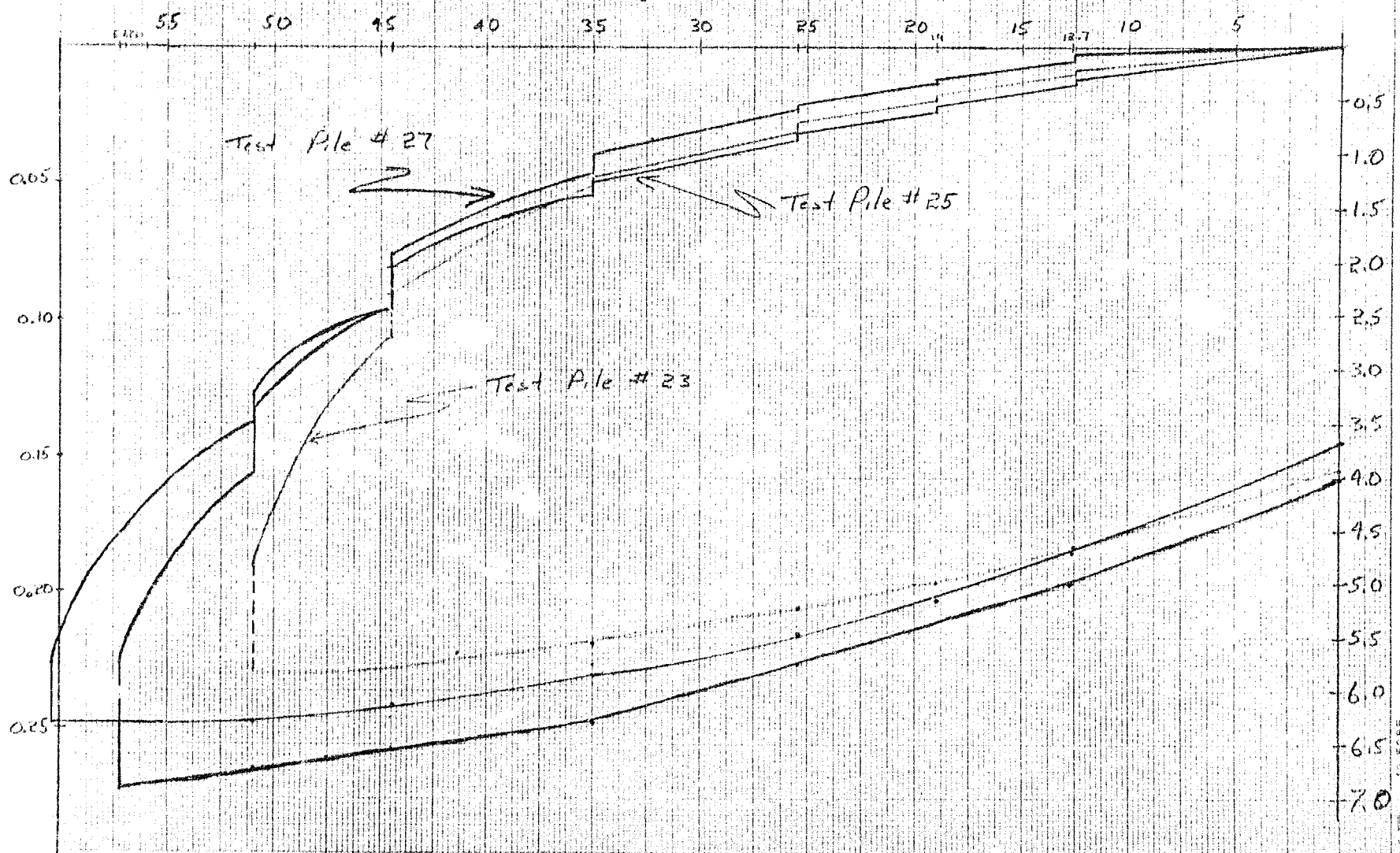
Test Pile # 23  
 Test Pile # 27



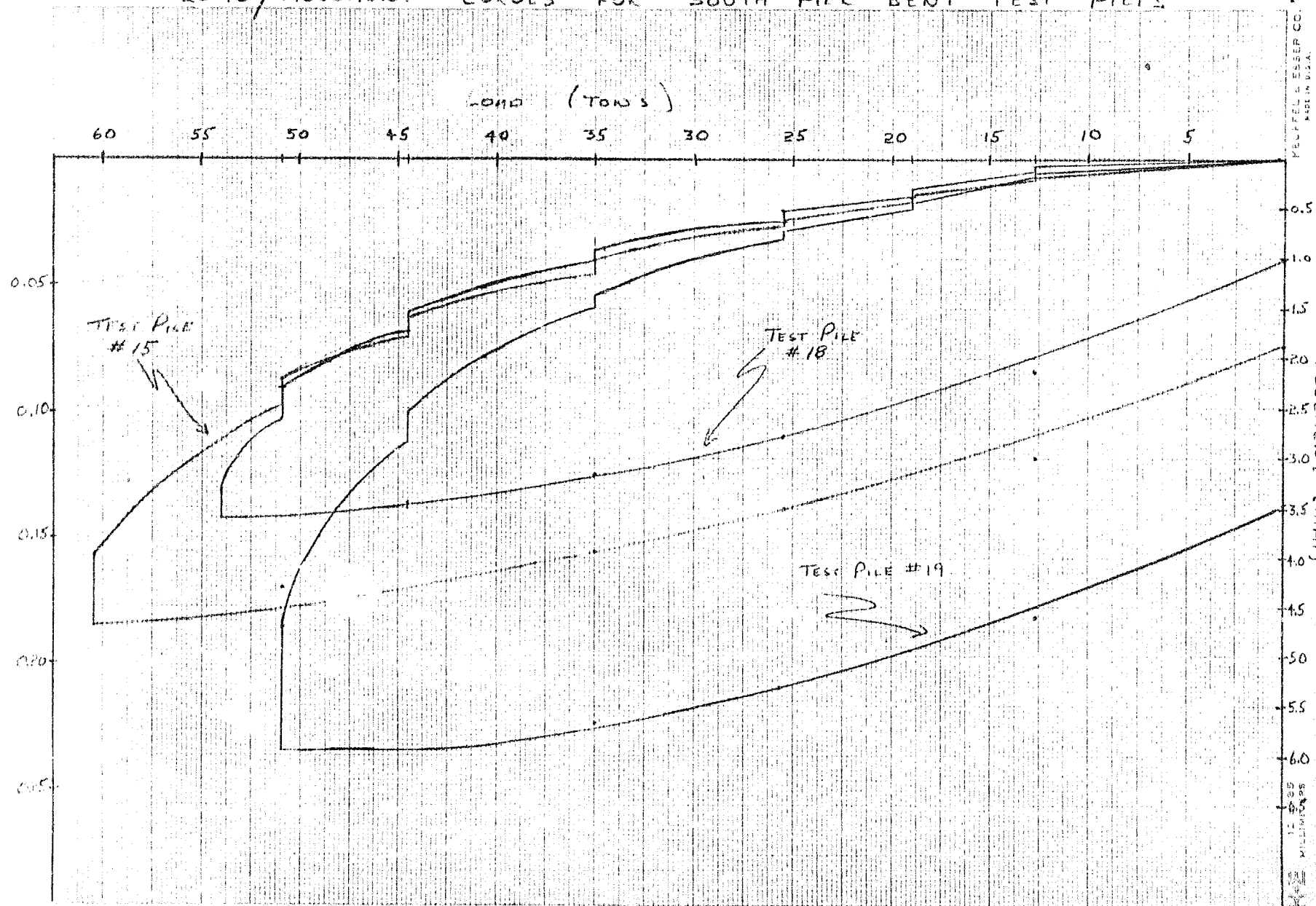
# LOAD/MOVEMENT CURVES FOR NORTH PIER BENT TEST PILES

Movement (inches)

Load (Tons)



# LOAD / MOVEMENT CURVES FOR SOUTH PIPE BENT TEST PILES







## Memorandum

To: Mr. K.G. Bassi  
Head, Eastern Section  
Structural Office  
2nd Floor, West Building

From: Soil Mechanics Section  
Engineering Materials Office  
Room 315, Central Building

Attention:  
Mr. C. Farrell  
Our File Ref.

Date: 78 08 22 .

In Reply to

Subject:

Re: Wilberforce Bridge Site, Contract 78-08

On 78 08 16 this Section was informed of possible pile movement at the Wilberforce Bridge site, Contract 78-08 by Mr. C. Farrell and Mr. B. Hashizume of the Structural Office. A meeting between the two Offices was held the following day to further discuss the situation. In order to gain firsthand information and better assess the situation, the author visited the site on 78 08 18 and observed the following:

- i) The pile butts of the south pier bent had moved laterally towards the south abutment and settled vertically. Likewise, the north pier piles had moved towards the north abutment and settled. The magnitude of movements are shown on the attached diagram. Tension cracks in the soil surrounding the pier piles indicated the piles had moved laterally towards the abutments.
- ii) The author could not inspect the abutment piles since the pile caps had already been poured and the south abutment wall was in the process of being poured.

Personal communication with the on site construction field staff Alvin Armstrong, Project Supervisor and Pat O'Connor, Bridge Inspector, verified the following:

- i) The abutment piles were driven according to contract drawings with no apparent problem. The field staff considered the abutment piles to be sound and, therefore, had approved the placement of the abutment pile caps.
- ii) The construction sequence for driving and pouring of concrete for the tube piles is shown on the attached bar chart. All piles were cut off to the required elevations.
- iii) Although no complete driving records were taken, the driving resistance for the length of the piles averaged from 8 to 10 blows/foot. Resistance at the required tip elevation averaged 10 blows/foot.
- iv) Initial movement of the pier piles was observed in pile No. 19 during driving of abutment batter pile No. 10. Driving resistance increased slightly for pile No. 10

cont'd.....

between 40 to 45 feet; this corresponds to a greater depth than when pile No. 10 commenced movement. No apparent increase in driving resistance was noted for any of the other abutment batter piles

- v) All pier piles commenced movement only during the driving of the abutment batter piles. No relative movement occurred after driving the abutment piles.
- vi) No movement of the abutment piles had occurred during driving the adjacent piles except for pile No. 6 which settled 7 inches during the driving of pile No. 7.
- vii) Slight movement of the south pier piles was observed during the driving of the north abutment batter piles.

Considering the magnitude of lateral and vertical movement of the pier piles and the sensitivity of the structure to differential settlement, concern was expressed whether the piles would settle excessively under the design loading.

Since no unusual movement or problems of the abutment piles were reported by the construction field staff, we feel these piles will behave as designed. In order to verify the load-movement behaviour of the displaced pier piles, it is recommended to carry out a full scale load testing program on selected pier piles.

Details of the testing program were discussed at a meeting with Bill Hashizume and Augustine Liu on 78 08 21. It was agreed that reaction system design, contractor arrangement, and region/district liaison would be carried out by the Structural Office and that the Soil Mechanics Section would supervise the load testing operations.



T. Kazmierowski  
Project Engineer

TK/gs

Attach.

cc: W. Hashizume  
A. C-Y. Liu  
T.C. Kingsland  
R.W. Franks  
Files ✓

# BAR CHART OF CONSTRUCTION SEQUENCE

Contract 78-08

Activity

July

August

18 19 20 21 22 23 24 25 26 27 28 29 30 31 1 2 3 4 5 6 7

- drive south pier piles

—

- concrete south pier piles

—

- drive north pier piles

—

- concrete north pier piles

—

- drive south abutment piles

—

- pour south abutment piles

—

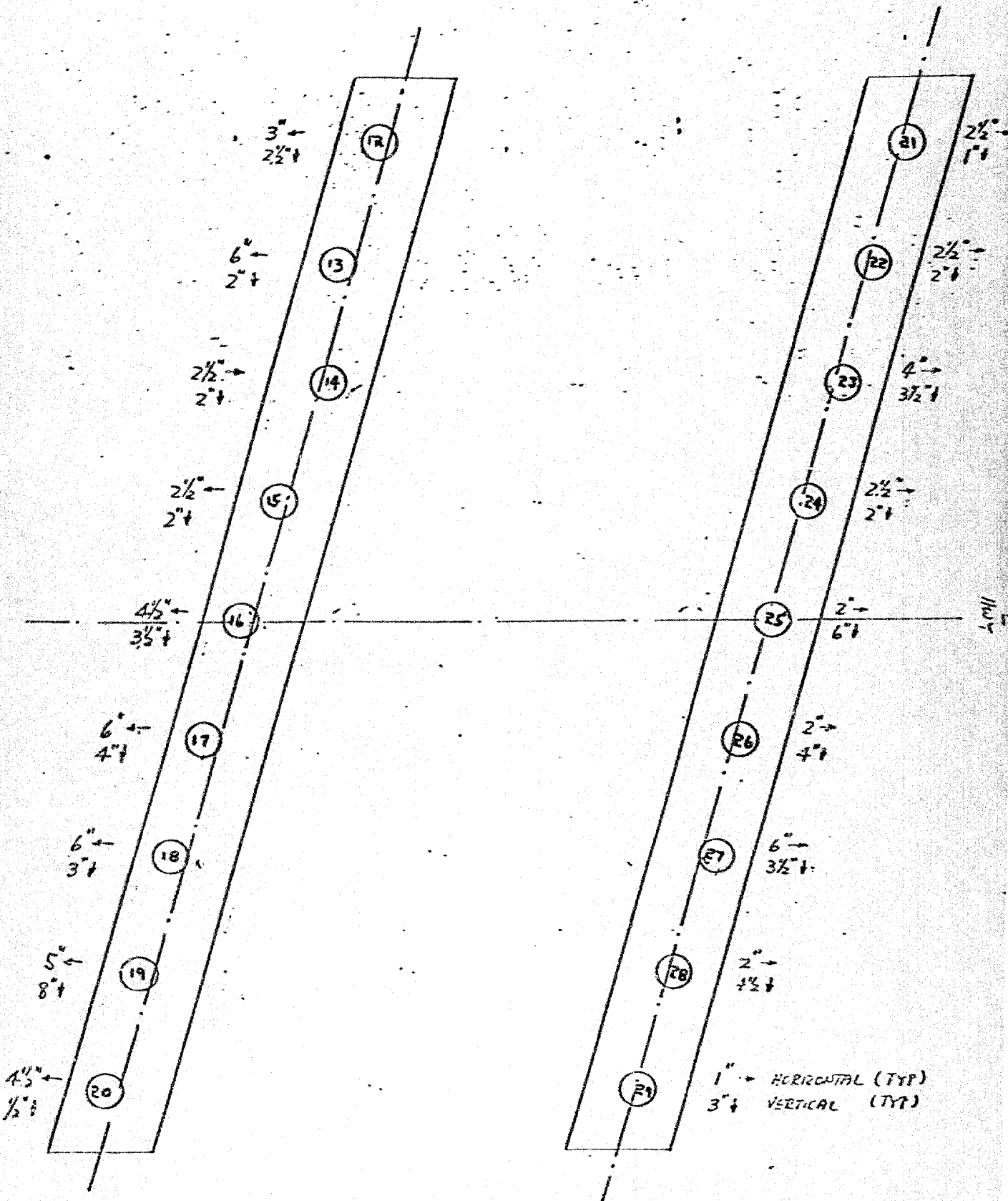
- drive north abutment piles

—

- pour north abutment piles

—

# Measured Dike Movements Contract 78-C8



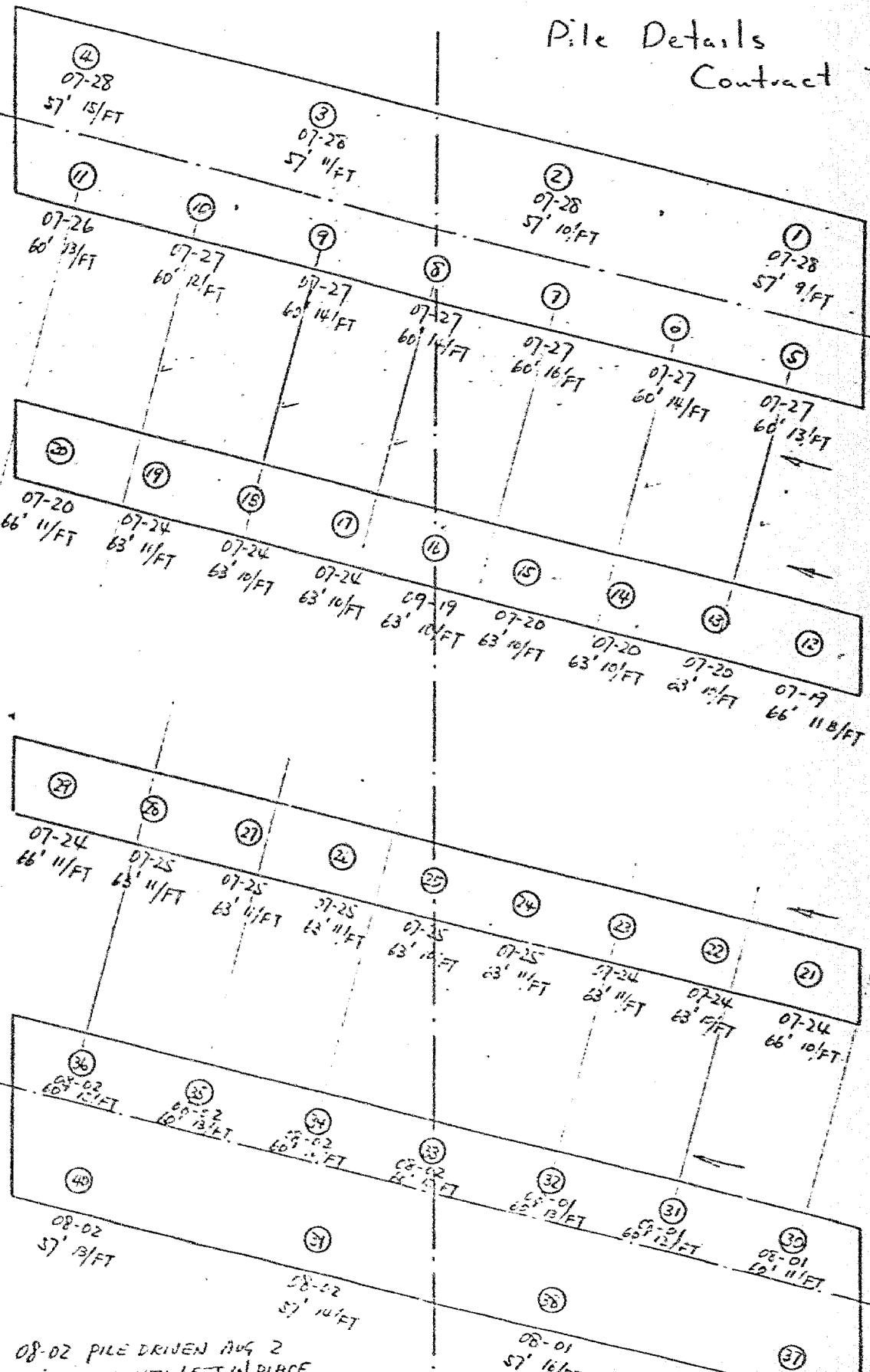
# Pile Details Contract 78-CE

S. ABUT.

S. PIER

N. PIER

N. ABUT.



08-02 PILE DRIVEN AUG 2  
LEFT IN PLACE

# PROJECT CONSTRUCTION REPORT

RADBONE

ENGINEERING & POW OFFICE		
	SECTION	Initial
✓	PLANNING & DESIGN	
	PROPERTY	
	STRUCTURAL PLANS	
	GEOTECHNICAL	
✓	RECORDING	
	CONSTRUCTION	
	FILE	

CONTRACT No.: 78-08

CONTRACTOR: O.J. Gaffney

LOCATION: Gr., Dr., Gr. Base, H.M.P. and  
Structure.  
Wilberforce Bridge, 3.8 Miles  
West of Harcourt, Hwy. 648 -  
0.3 Miles.

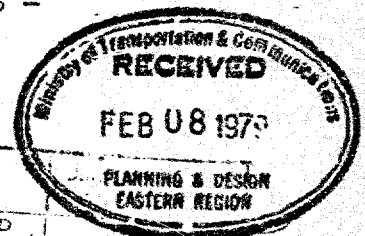
REGION: Eastern

DESIGNED BY: M.T.C.

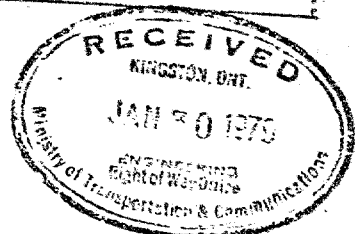
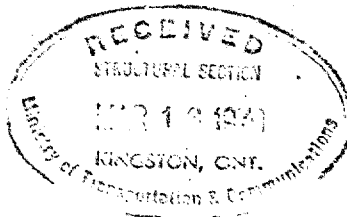
PREPARED BY: E.J. Armstrong

RECOMMENDED BY: W.G. Howe

DATE: January 4, 1979



HD. P&D	
SR. PM	
PM	
DGL	
D.S.S.	✓
C.C.O.	✓
B/P.C.	
C.L.C.	
C.R.O.	✓
ENV. PL.	
SECY	
COMMENTS:	



In general, the Contract package was put together in good order.

Adequate information was supplied in the contract drawings, special provisions, and specifications to construct the job.

Several construction problems were encountered during construction.

These are explained briefly below and detailed elsewhere in this report.

- (a) No provision for traffic so that piles in the north abutment could be drove.
- (b) Movement in pier cap piles.
- (c) Prestressed slabs too narrow.
- (d) Prestressed slab reinforcing steel too short.
- (e) Incorrect granular backfill standard.

No problems were encountered in the quality assurance portion of this contract.

1:2

RECOMMENDATIONS

x Yes it could have been  
but there were good  
reasons why it wasn't.  
(or weren't.)

The structure on this contract consisted of a three span bridge with box type prestressed beams. The centre span was 33 feet long and the two end spans from the abutment to the pier caps were 15 feet each.

It was on account of these short spans that trouble with the piles was encountered.

I feel that this structure could have been designed differently and the two piers eliminated.

Closer inspection to materials which are manufactured and supplied by others off the contract - (ex.-See Prestressed Slabs).

More extensive boring should be done at the exact locations of footings, pier caps, etc.

Why? The  
sub-soil was  
completely uniform

Detours should be scrutinized closer at the design stage to ensure proper traffic control while at the same time they will not interfere with the contractor's operations.

I would like to recommend that all scales be equipped with electronic print-out devices. This would eliminate the M.T.C. Weighman, most human errors, and save the Ministry a considerable amount of money.

2-

CONTRACT PACKAGE

The contract package for this contract was good. No problems were encountered in the specifications or the special provisions. They appeared to adequately cover their appropriate items.

2:1:1

DESIGN PROBLEMS (ROADWAY)

Provision for traffic - One detour was required at the location of the new structure.

Trucks with trailers over 45 feet had problems negotiating this detour.



Property problems - Due to the short length of the contract, no problems were encountered in this area. No complaints were received from the property owners.

Layout information - Sufficient information was received to properly lay out the contract. This information appeared to be fairly accurate.

Drainage problems - Two extra entrance culverts were required to provide proper drainage.

Environmental concerns - Granular "C" was used in the detour and fill areas in the causeway between Grace and Pussey Lakes. This material was clean and there was very little contamination noticed in the water.

Rip rap was placed on these slopes to prevent erosion.

No future maintenance problems from the design features can be foreseen on this contract.

2:1:2

#### COMPUTER OUTPUT

The computer outputs was received from the Project Manager by mail. These outputs were not reviewed with the Design Staff possibly because of the contract length. (0.3 Miles).

Cross Section Plots - These plots were received with the theoretical final templates, and sub-grade templates plotted on them.

The breakpoints, grade indicators, etc. was also enclosed.

The above templates helped considerably during the final calculations and should be applied on future contracts.

Reports - The following reports were received on this contract.

- (a) Road elevations and breakpoints.
- (b) Sub-grade elevations
- (c) Horizontal and vertical curve data.  
(Included in the contract drawings)
- (d) Bridge drawings.
- (e) Soils reports.
- (f) Environmental reports.

The above reports were used extensively and proved <sup>provided</sup> useful information.

2:2

DESIGN PROBLEMS (STRUCTURES)

The contract drawings for the structure was quite adequate. They provided enough elevations, deminsions, and layout information that no problems were encountered in building the structure.

Traffic was protected on the detour by temporary steel beam guide rail, and the proper construction signing.

The reinforcing steel and concrete schedules were checked and found to be quite accurate.

2:3

FOUNDATION AND SOILS INFORMATION

The information contained in the above reports was fairly accurate, however, the following problem was encountered at the pile driving stage. The original crossing between the two lakes appeared to be corduroy. Old logs 12" to 14" in diameter was dug up during the excavation. At some unknown time 8 to 10 feet of rock fill was <sup>placed</sup> over this corduroy.

The contract drawings called for pre-augering 12" diameter holes in this fill in order to drive the piles.

Due to the size and amount of rock involved in this fill, the contractor decided to excavate the rock and backfill with suitable earth, rather than try to auger.

The contractor did not claim for any extra compensation for his method of operation. I feel that augering for the piles would not have worked in this type of material.

I would suggest more extensive boring, test holes, etc. be done by the M.T.C. at the actual locations of footings, pier caps during the design stage.

2:4

SPECIFICATIONS AND STANDARDS

The specifications and standards were adequate enough to cover the items on this contract. The special provisions included in contract appropriately covered the items where the standard specifications did not apply.

One error was found in the structure standards which had to be changed.

Granular backfill to structure standard #S.S.- 5-2 was the incorrect one for this contract. Structural office was contacted and this standard was changed to standard #S.S.-5-3. The granular backfill was paid for under item #47 which was a lump sum item. The contractor was informed of the standard change, and he stated that due to the minor change in quantities, no extra compensation would be requested.

There was no duplication of standards or specifications on this contract.

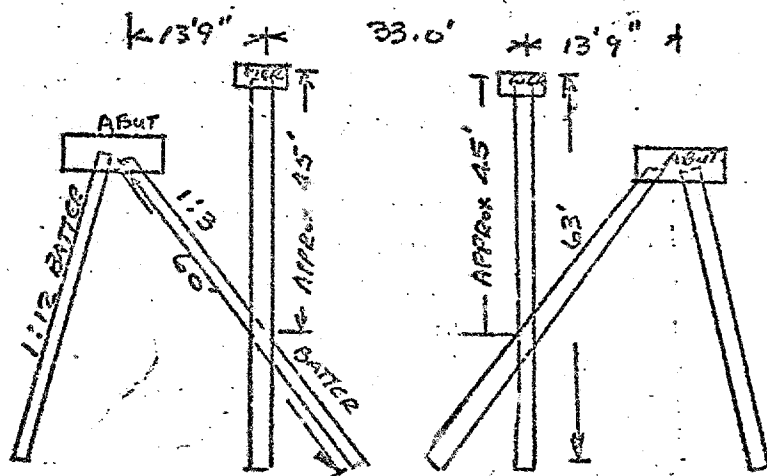
3-1

### CONSTRUCTION PROBLEMS

#### (a) PILE DRIVING

Donn Const'n. (Pile Driving Sub-Contractor) arrived on the contract on July 17/78. He decided to drive the piles in the pier caps first. After the rock fill was excavated, these piles were drove without difficulty.

Donn Const'n. then commenced driving the piles in the south abutment. While driving the 1:3 batter piles in this abutment movement was noticed in the south pier cap piles. Movement was also recorded in the north pier cap piles when the 1:3 batter piles were being driven in the north abutment footing. The reason for this movement was that these batter piles was crossing between the pier piles which was 3' 9" apart. This crossing occurred at approximately 45' (feet) down on the pier piles.  
- SEE SKETCH BELOW -



PIER PILES ARE 3'9" APART.

The piles in these pier caps moved approximately 1" to 6" horizontal and  $\frac{1}{2}$ " to 8" vertical.

Structural office, Toronto was notified. Mr. W. Hashizume of the above office, and Mr. T. Kazmierowski, of the Soils Mechanics office investigated the bridge site. It was decided by them that these piles could have lost their load capacities and that certain piles should be load tested.

These piles were load tested by Gaffney Const'n. and carried out under force account #40-78-55.

The width of the pier caps were then changed from 3' 0" to 3' 8" wide at this time.

The extra concrete required by this change was paid for under Quotation Request #Q-40-78-58.

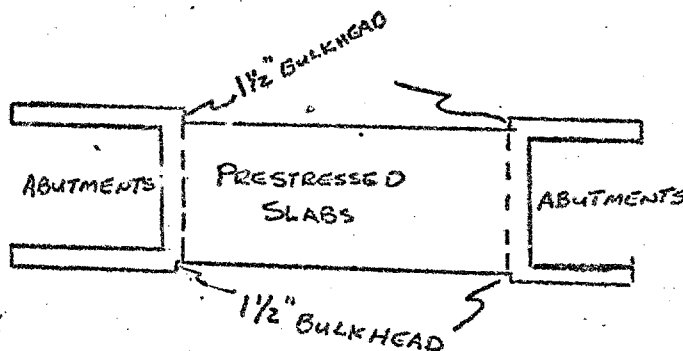
Letters covering the above changes are included in the correspondence.

(b) PRESTRESSED SLABS

The prestressed slabs were manufactured and erected by Stanley Structures of Belleville, Ontario.

When these beams arrived on the contract, every beam was approximately  $\frac{1}{4}$ " to  $\frac{3}{8}$ " narrower than the 4.0' as shown on the contract drawings. A bulkhead of approximately  $1\frac{1}{2}$ " had to be built on the outside of the barrier walls at the expansion joints on each side of the structure to compensate for this.

SEE SKETCH BELOW



During the placing of the reinforcing steel in the barrier walls by the prime contractor, it was found that the reinforcing steel, Re-Bars #M-500 placed in the prestressed slabs by Stanley Structures was 7" too short. Extra ties were used to compensate for this.

### MATERIAL SUPPLY

The contractor obtained his granular materials from two pits which were listed on the strip map. A small problem in one pit occurred, which is detailed under section 3:3 - Quality Assurance.

There were no problems with the supply of any materials listed on the Designated Sources List, with the exception of the steel piles. The contract stated that the steel piles were to be 12 3/4" O.D., .250 wall thickness.

Piles with this size of wall thickness could not be supplied at this time. The contractor requested in the wall thickness to .228 in lieu of .250.

This change was approved by Construction Office.      ??!

### WORKING DAYS

The contract was delayed for twenty working days while testing of the pier cap piles was carried out. Working days were not charged against the contract during this operation. There were three working days remaining when the contract was accepted. In view of this I feel that the number of working days allotted for the contract was appropriate.

The contract was awarded on May 31/78. This date would have been good except that the contractor was held up for 20 working days while the piles were being tested. This carried the work into the middle of November. However, we had very nice weather and was able to complete the contract. I feel that a contract of this type should be awarded in the spring as soon as weather permits. (Example - May 1st instead of May 31st.)

The staff on this contract was adequate and well qualified to do the work. *However their English were near enough to drive me to distraction.*

### 3:2 MISCELLANEOUS

Work Orders - Due to the change in the size of the pier caps, Quotation Request #Q-40-78-58 was issued to pay for the extra 4.49 cu. yds. of concrete involved.

Force Accounts - There were three force accounts initiated on this contract which are listed below:

F-40-78-45 - Placing and removing the 48" C.S.P. required

in the detour extension when the existing channel was closed so that the piles could be driven in the north abutment footing.

F-40-78-55 - This force account covers the load testing of the piles in the pier bents.

F-40-78-85 - To reclean ditches, fill washouts and rake topsoil back onto the slopes to repair the damage caused by heavy rains.

No claims or intent of claim was filed by the contractor on this contract.

#### Summation of Staff

1 - Project Supervisor	- 8 Months
1 - Inspector	- 5 Months
1 - Tech. 1 Const.	- 7 Months
1 - H.I.A.3, Inspector	- 7 Months

NOTE: There were various other personnel on th is contract for short periods of time.

#### ENGINEERING EXPENDITURES

A total of \$45,000. was allotted on this contract for Engineering.

There was \$39,400. spent as of December 15/78. Approximately \$2,100. will be spent yet doing the final estimate. This will bring the total expenditure to approximately \$41,500. An underrun of \$3,500. (approx.) will occur.

The money allotted (\$45,000.) against what was spent (\$41,500.) is quite close with what was estimated.

#### 3:3 QUALITY ASSURANCE

The amount of testing carried out on th is contract compared favourably with what was specified.

The contractor elected to crush his Gran. 'A' aggregate in Pit #7, as listed on the strip map. The strip map showed that there was no Gran. 'A' in this pit.

When the crusher moved in and started to attempt to produce this material, it was unacceptable.

The crusher then moved to Pit #6 and this material was acceptable.

No further aggregate problems were encountered.

The H.L.4 material was supplied and placed by Royel Paving, Lindsay. No problems encountered in this item.

The contractor used single drum vibratory rollers and hand plate vibratory compactors. These units achieved the desired amount of compaction required by the specifications.

#### 4 OVERRUNS AND UNDERRUNS

All tender items on this contract were either close to the estimated tender or in the case of the lump sum items they were exact.

No item varied over \$5,000.00, however, a few of the items changed and the reasons for this are listed below.

	<u>Tender</u>	<u>Actual</u>
Item #2 - Earth Excavation	5,100 c.y.	5,514 c.y.
" #5 - Granular 'A'	3,600 Tons	3,806 Tons
" #23 - Temp. Steel Beam G.R.	600 L.F.	829 L.F.

The above items overran due to the detour extension Rt. Sta. 110+00 to 111+50.

Item #10 - 18" C.S.P. Culverts - Tender 200 L.F., Actual 280 L.F.

Two additional entrance culverts were required in the M.T.C. park entrances to provide proper drainage. See Bk.4 - Pages 67 & 95.

Item #28 - Water for Sod - Tender 10,000 Gals., Actual - 0

Due to the late time in the season that the sod was placed (Nov. 16 & 17), it was impracticable to place any water on this sod.

#### 5 TENDER COST AND ACTUAL COST

The total tender cost submitted by Gaffney Construction was \$281,430.96.

The actual cost to construct the contract was \$297,135.99. The reason for the overrun was due to extra work. 3 Force Accounts had a total value of \$14,529.06 and one work order for \$613.10. The total cost of this extra work was \$15,142.16.

c.c.: J. E. Callaghan  
D. M. Hopper  
W. G. Wagle  
S. C. J. Radbone ✓  
R. W. Franks  
W. G. Howe  
E. J. Armstrong  
Final Estimate  
Final Estimate (District copy)

W. V. Sibley  
Regional Director  
Kingston, Ontario

Engineering Audit Office

1978 09 12

	DOUGLAS & PETER	
	GEORGE HARRIS	
	SCHWARTZ & BROWN	
cc	<input checked="" type="checkbox"/> STRUCTURAL	<input checked="" type="checkbox"/>
	FILE	

I.T.C. Contract 78-06

T.C.K. please draw this  
up into Structural Design

*HL*

Attached is an Office Audit Report by H. V. Sudds  
dated 1978 09 08.

Although the records reviewed were found to be  
generally satisfactory in their support of progress  
payment, additional payments are warranted for  
granular backfill to culverts and reinforcing  
steel. Detail of this is found in the body of the  
report.

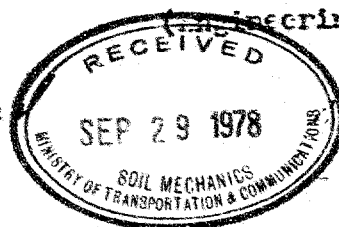
Our Auditor notes that the detour had to be extended  
in length to enable pile driving to be done without  
closing the road to traffic. Also, the abutment piles  
driven to the designed batter made contact with the  
vertical pier piles causing these to move. It was  
decided to load test the piers, resulting in additional  
cost being incurred.

In the view of the foregoing two problems being associated  
with the design of this structure and detour a copy of  
this report is forwarded to Mr. S.C.J. Radbone, Manager,  
Engineering and Right-of-Way Office.

C. V. McClelland  
Asst. Chief Internal Auditor  
(Engineering)  
(For) J. H. Crannie  
Chief Internal Auditor  
(Engineering)

CL/JHW

c.c. S.C.J. Radbone  
G. Luyt





Note

This project consists basically of replacing an existing structure with a new three span bridge with a precast, pre-stressed hollow slab deck. The grading at the adjacent approaches and detour was designed with a surplus of material, with the excess material to be disposed of outside the right-of-way.

The grade work has generally been completed beyond the limits of the detour. The Contractor has constructed the abutments and is currently placing forms and steel in the ballast walls. Topsoil was presently being placed and it was noted that final cross-sections had not yet been obtained. Mr. E. Armstrong, Project Supervisor stated an allowance would be made where required.

Item #2, Earth Excavation (Grading)

Tender 5,100 cu.yds.

P.O.R. 3,187 cu.yds.

Progress payment consists of underfill stripping, subgrade cut, ditch excavation to place granular backfill and detour removal. 62% of the tender quantity has been calculated for payment to date. The cross section template with required elevations and distances was issued to the Contractor representative. It was pointed out to the field staff that plotting of final subgrade cross sections was not required and reference to the field book was all that was required on the final cross section rolls.

Detour Removal

The detour was originally laid out and constructed according to the contract drawings; however, due to limited working space the Contractor would have been unable to drive the north abutment piles without closing Highway #648. Consequently it was decided to extend the detour past the existing structure to enable the pile driver to set on the existing bridge. This also necessitated placing of a 48 inch diameter culvert in the existing channel. The extended detour was required for four days. A quantity of 485 cubic yards for removal of this portion of the detour has been calculated for payment under this item.

Items #10 - #13. Placing Culverts - Various Types and Sizes

Payment has been summarized in final form and referenced to the Inspector's diary for the lengths placed. Standard specification 421 is applied against the above tender items and consequently the earth excavation is included in the price bid per lineal foot. The placing of the culverts to the standard required was confirmed by the Inspector. However no measurements were obtained to confirm that excavation had been carried-out to theoretical requirements. There is only one centre-line and two sideroad culverts requiring excavation on this project. It was suggested on future projects that supportive measurements be obtained.

After a general review of the diary records and discussion with the field staff it was noted that no payment had been made for the granular backfill to culverts. Mr. Armstrong stated he was under the impression that the backfill was included in the price bid per lineal foot. The field staff had summarized the loads used for the Contractor's payment to the pit owner and consequently payment is to be made under Item #6, Granular "C". The design quantity was noted to be 32 tons.

Item #37. Reinforcing Steel (Bridge and Approach Slabs)

Tender 15 ton

P.O.R. 5 ton

Payment for the reinforcing steel placed to date has been made according to the reinforcing steel schedule. The Inspector notes daily on the steel schedule the amount of reinforcement placed. The steel schedule received did not include the reinforcing steel for the approach slabs, however, the total was included in the tender quantity. The approach slab steel schedule was requested and received from the Structural Office.

It was noted that all of the reinforcing steel had been received on the project. It was pointed out to Mr. Armstrong that additional payment was warranted as supplemental specification 905 S allows 75% payment for the supply of the reinforcing steel.

Item #47. Granular "C" Backfill to Bridge

Tender Lump Sum

P.O.R. 100%

This item has been summarized in final form referenced to the diaries for the details of the backfill operation. Final excavation sections were obtained to substantiate that excavation had been carried out as per standard. Summaries were also compiled for the backfill material placed.

A letter is on file from the Structural Office indicating that the backfill standard on sheet #29 should have been S.S.5-3 rather than S.S.5-2. Mr. Armstrong stated the difference in the two standards resulted in an additional 25 - 30 tons of backfill being required. The Contractor has indicated that the change is of a minor nature and no re-negotiation of the lump sum item would be necessary.

### Testing of Pier Piles

The Contractor is currently being delayed on the bridge work due to the Soils Mechanics Section of the M.T.C. wanting to load test certain pier piles. The charging of working days has been suspended until the load testing on the south and north piers is completed and the steel and forms is replaced on the pier caps. The test is expected to be conducted within the next week.

According to the contract drawings the batter on the abutment piles was to be 1:3 and the pier piles were to be vertical. When driving the abutment piles on the required batter contact was made with the pier piles and eventual displacement was noted. The north pier piles moved from 1-6 inches horizontally and 1-6 inches vertically, whereas the south pier piles moved from 2 $\frac{1}{2}$  - 6 inches horizontally and  $\frac{1}{2}$  - 8 inches vertically. Consequently after an on-site inspection by Mr. T. Kazmierowski of the Soils Mechanics Section it was decided to carry out a full scale load testing program. Three piles in each pier bent is to be test loaded and the test carried out to failure of the pile or to a maximum of 50 ton test load. The Contractor is to supply all the material, equipment and instruments to conduct the test. Payment is to be by force account.

### Force Accounts

Two force accounts have been required to date necessitated by the extra work noted previously in the report. The initial force account was

to place and eventually remove the 48 inch diameter culvert placed in the existing channel when the detour was extended. The other force account was to remove the steel and formwork from the south pier and the steel that had been placed in the north pier prior to the decision to load test the piles.

Daily work records were on file and appear in order.

#### Contract Management and Design Package

The various proposals of the contract management package have been instituted on this project with no particular problems noted. A master list of equipment was not submitted by the Contractor. Mr. Armstrong stated the Contractor's own equipment consisted of pumps only and other equipment was rented locally as required. The equipment inventory section in the diaries have been maintained.

Mr. Armstrong stated no particular problems were encountered with the design package received at the field office. Original ground and theoretical subgrade cross sections were computer plotted on the final cross section rolls. It was noted that no meeting was held with the design personnel.

#### Summary of Report

The records reviewed were found to be in generally satisfactory order to substantiate the payments made to date. The following points were discussed with the Project Supervisor:

- suspension of working days due to N.T.C. decision to load test certain pier piles.
- extension of detour to enable pile driving to be completed without closing Hwy. #648.
- no payment made for granular backfill to culverts contrary to specification 421.
- additional payment warranted for Item #37, Reinforcing Steel.
- taking of final cross sections after the placement of topsoil.

*B.V. Sudds*  
B.V. Sudds  
Project Auditor

EVS/jhw

ENGINEERING AUDIT OFFICE  
OFFICE AUDIT REPORT BY B. V. SUDDS  
1978 09 08

This report is based on an Audit of the records on file in support of quantities shown on Progress Quantity Report No. 3, dated August 31, 1978. The Audit was carried out during the period of September 6 - 7, 1978.

M.T.C.	Contract #78-08
DISTRICT	#10, Bancroft
CONSTRUCTION MANAGER	R. W. Franks
TYPE	Grading, Drainage, Granular Base, Hot Mix Paving and structure
LOCATION	Sec. Hwy. #648 - Wilberforce Bridge and Approaches between Grace and Puscy Lakes 3.8 miles west of Harcourt
LENGTH	0.3 miles
CONTRACTOR	O. J. Gaffney Ltd.
TENDER CLOSING DATE	May 31, 1978
TIME FOR COMPLETION	80 Working Days from June 26, 1978
FREE TIME	Nil
ESTIMATED COMPLETION DATE	On Schedule
WORKING DAYS LEFT ON CONTRACT	40 as of September 1, 1978
CONTRACT SUPERVISED BY	M.T.C. Staff
CONSTRUCTION SUPERVISOR	W. Howe
PROJECT SUPERVISOR	E. Armstrong
INSTRUMENTMAN	C. Lacombe
JR. INSTRUMENTMAN	J. Hawkins
INSPECTORS	P. O'Connor



# INDUSTRIAL INSTRUMENTS SERVICE LTD.

80 HALE ROAD, UNIT 10, BRAMPTON, ONTARIO L6W 3N9

(416) 459-2032 - 457-3413

June 22/78

Birmingham Construction Ltd.  
Wellington Street Marine Terminal  
Hamilton, Ontario  
L8L 4Z9

Dear Sir

With reference to your purchase order 31228 covering the repairs of three  
(3) Pressure Gauges 0/10,000 PSI.

These gauges were calibrated and certification was done using our dead weight tester, which in turn has standards traceable to the National Research Council, Ottawa, as per their report # PMM-84 dated 7 August, 1973.

The results were as follows:-

L-K 5558 Birmingham Construction

0-10000 PSI #BA5491-B  
Actual Pressure

1000 PSI  
5000  
9000

Gauge Reading

1000 PSI  
5000  
8950

\* 0-10000 PSI #A9816 F  
Actual Pressure

1000 PSI  
5000  
9000

Gauge Reading

1000 PSI  
4975  
9000

0-10000 #LK4211  
Actual Pressure

1000 PSI  
5000  
9000

Gauge Reading

980 PSI  
5000  
9020

Yours very truly

T. W. Rimmer

TWR/lb

SERVICE OFFICE OF INDUSTRIAL PROCESS CONTROLS FOR TEMPERATURE - PRESSURE - FLOW - LIQUID LEVEL - ALARM SYSTEMS

Sept. 6 1978

\* This gauge never used since calibration

Lawrence Arthur

Birmingham Const. Ltd.

PILE LOAD TEST JACK  
 CALIBRATION CHART

USING ONE OR TWO 9"Ø HYDRAULIC JACKS  
 (RAM AREA - 63.6174 SQ. IN EACH)

PRESSURE (psi)	ONE JACK (tons)	TWO JACKS (tons)	PRESSURE (psi)	ONE JACK (tons)	TWO JACKS (tons)
100	3.181	6.36	3400	108.15	216.30
200	6.36	12.72	3500	111.33	222.66
300	9.54	19.09	3600	114.51	229.02
400	12.72	25.45	3700	117.69	235.38
500	15.90	31.81	3800	120.87	241.75
600	19.08	38.17	3900	124.05	248.11
700	22.27	44.53	4000	127.23	254.47
800	25.45	50.89	4100	130.42	260.83
900	28.63	57.26	4200	133.60	267.19
1000	31.81	63.62	4300	136.78	273.55
1100	34.99	69.98	4400	139.96	279.92
1200	38.17	76.34	4500	143.14	286.28
1300	41.35	82.70	4600	146.32	292.64
1400	44.53	89.06	4700	149.50	299.00
1500	47.71	95.43	4800	152.68	305.36
1600	50.89	101.79	4900	155.86	311.73
1700	54.08	108.15	5000	159.04	318.09
1800	57.26	114.51	5100	162.22	324.45
1900	60.44	120.87	5200	165.41	330.81
2000	63.62	127.24	5300	168.59	337.17
2100	66.80	133.60	5400	171.77	343.53
2200	69.98	139.96	5500	174.95	349.90
2300	73.16	146.32	5600	178.13	356.26
2400	76.34	152.68	5700	181.31	362.62
2500	79.52	159.04	5800	184.49	368.98
2600	82.70	165.41	5900	187.67	375.34
2700	85.88	171.77	6000	190.85	381.70
2800	89.06	178.13	6100	194.03	388.07
2900	92.25	184.49	6200	197.21	394.43
3000	95.43	190.85	6288	200.01	400.03
3100	98.61	197.21			
3200	101.79	203.58			
3300	104.97	209.94			

Mr. R. W. Franks  
Manager, Construction Office  
Eastern Region, Kingston

Structural Office  
West Building, Downsview

78 09 14

Contact 78-08, W.P. 76-73-01,  
Wilberforce Bridge, Site 40-52,  
Highway 648, District 10

---

This will confirm telephone conversation of 78 09 14 with Mr. E. Armstrong of your Wilberforce field office regarding the above bridge.

Because of shifting of the pier piles during driving, the centre line of the south pier cap has been moved 4" towards the south abutment (see sketch "A") and the centre line of the north pier cap has been moved 3" towards the north abutment (see sketch "B").

The width of the pier cap has been increased to 3'-8" from 3'-0" (see sketch "C").

Rebar C6004 is now redundant, and in its place, C6006 should be used (see sketches "C" and "D").

At Mr. Armstrong's request, three (3) Xerox copies of this memorandum including the 4 sketches will be hand delivered to your field office by Mr. T. Kazmierowski of the Soil Mechanics Section on 1978 09 18.

WTH/wh  
Attch.

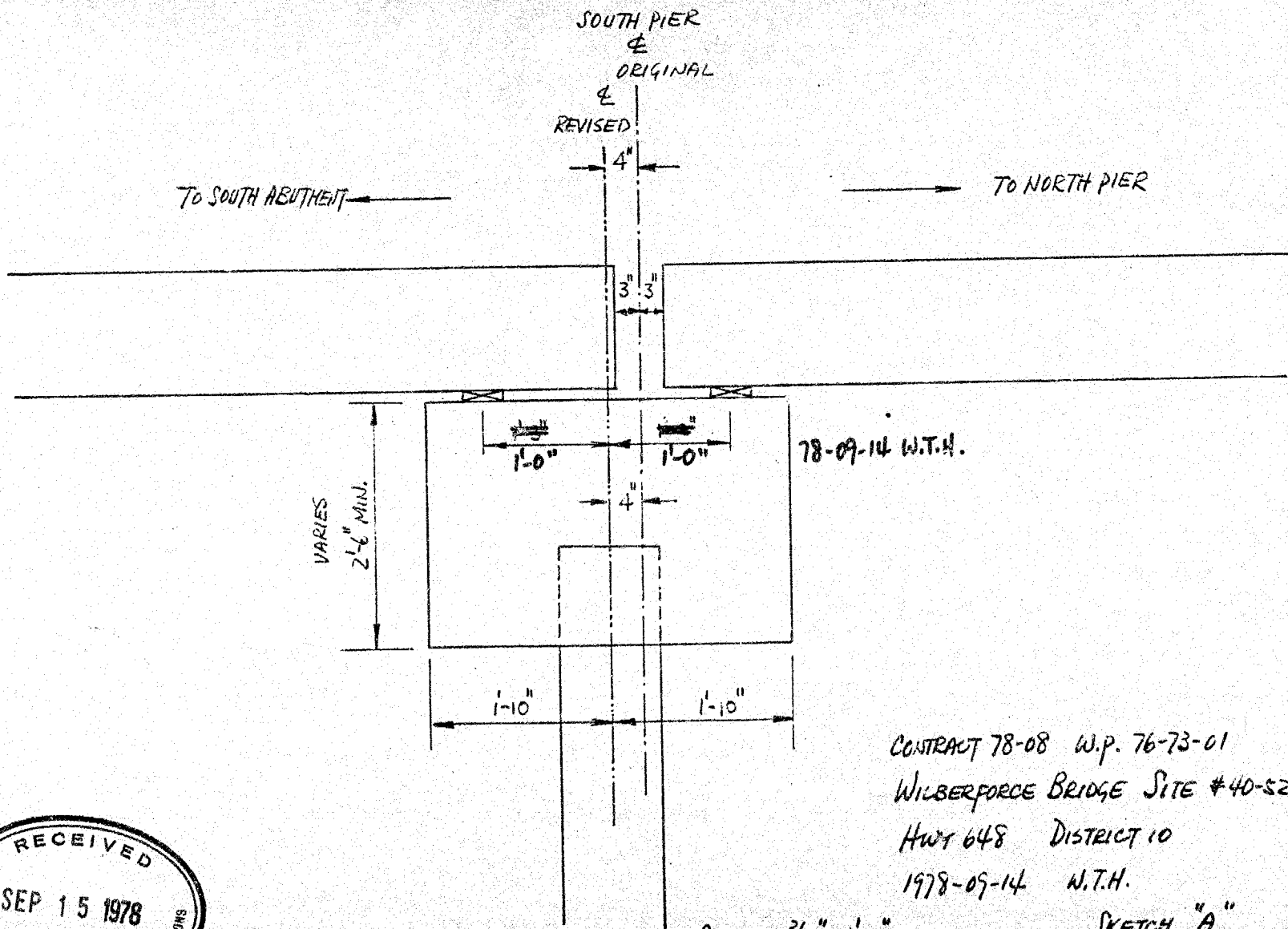
*W. T. Hashizume*  
W. T. Hashizume  
Inspection Engineer  
Eastern Section

c.c. C. Mirza ✓  
K. Bassi

*ln → m-devala → Fth.*





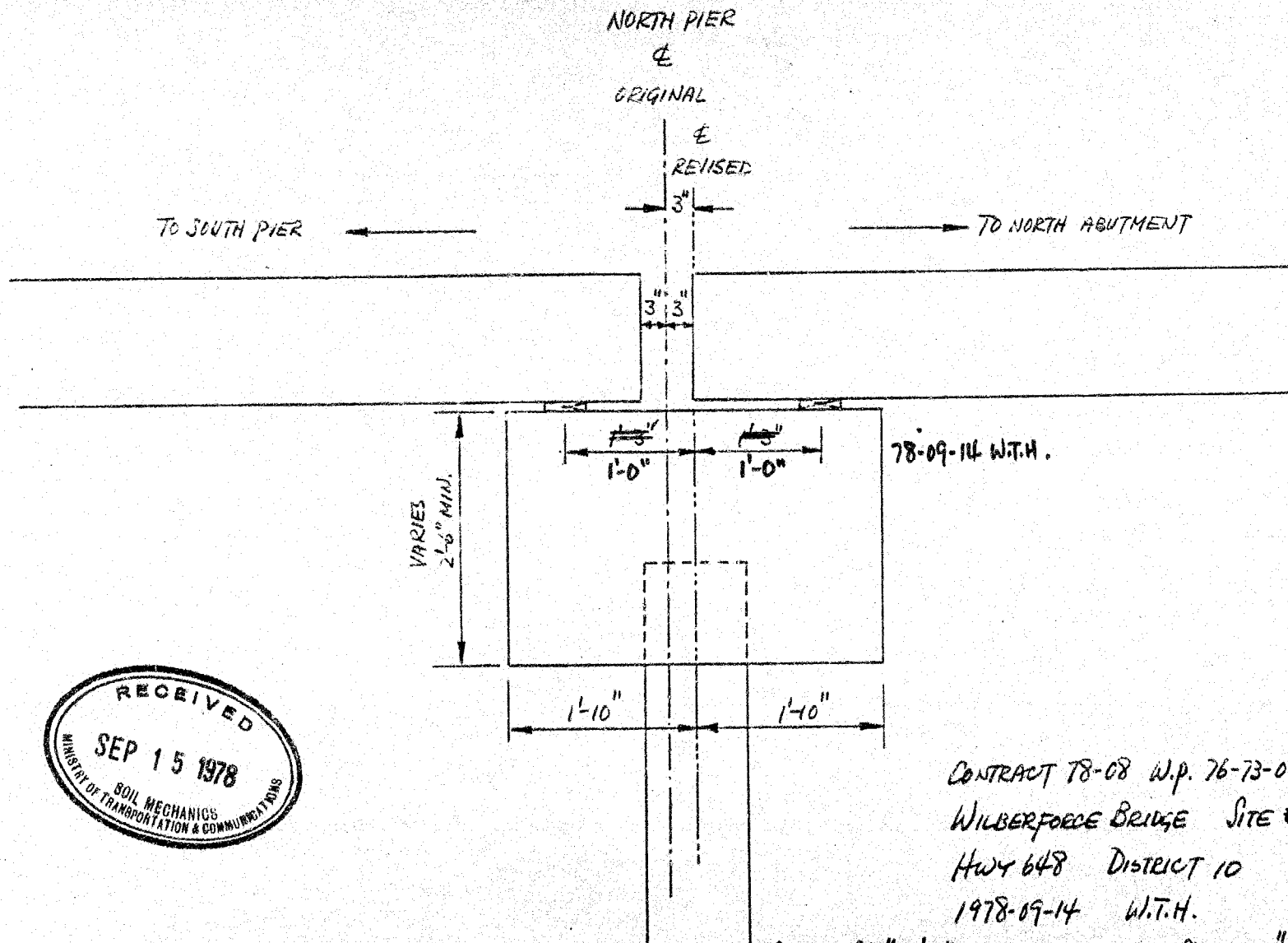


CONTRACT 78-08 W.P. 76-73-01  
 WILBERFORCE BRIDGE SITE #40-52  
 HWY 648 DISTRICT 10  
 1978-09-14 W.T.H.

SCALE:  $\frac{3}{4}" = 1'-0"$

SKETCH "A"

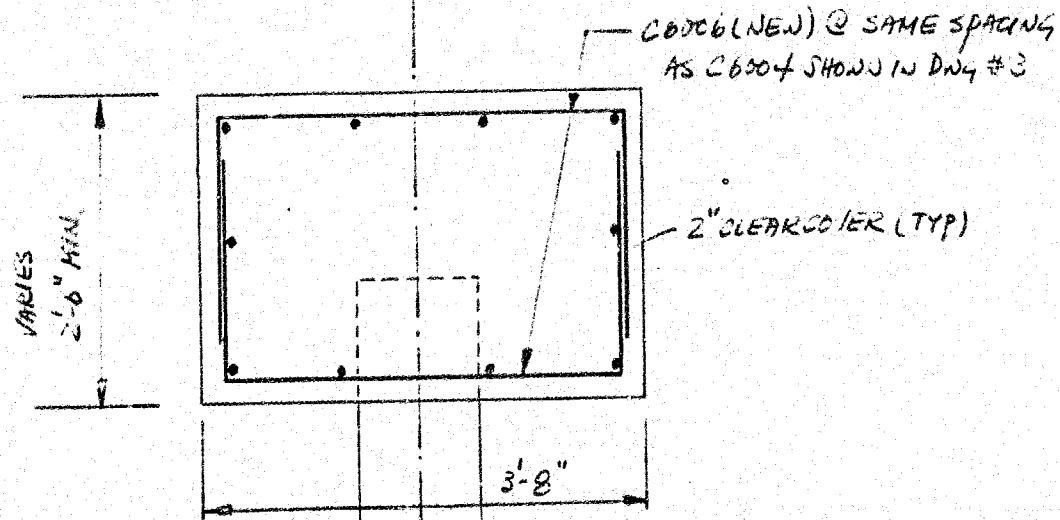




CONTRACT T8-08 W.P. 76-73-01  
 WILBERFORCE BRIDGE SITE #40-52  
 HWY 648 DISTRICT 10  
 1978-09-14 W.T.H.

SCALE:  $\frac{3}{4}" = 1'-0"$  SKETCH "B"

REVISED  
&  
PIER



CONTRACT T8-08 WP 76-73-01  
WILBERFORCE BR. SITE # 40-52  
HWY 648 DISTRICT 10  
1978-07-14 W.T.H.

SCALE  $\frac{3}{4}" = 1'-0"$

SKETCH "C"

[illegible]

CONTRACT 78-08 W.P. 76-73-01  
WILBERFORCE BRIDGE, SITE # 40-52  
HWY 648 DISTRICT 10  
1978-09-14 W.T.H.

SKETCH "D"

Sept. 5/78  
Wilberforce, Ont.

Cont. 78-08 - WILBERFORCE BRIDGE

A meeting was held on the above date at the M.T.C. Field Office in Wilberforce commencing at 11:00 A.M.

**THOSE PRESENT WERE:**

W. Howe -	Const'n. Sup'r. - M.T.C.
E. Armstrong -	Proj. Sup'r. - M.T.C.
T. Kasmierowski -	Soils Section - M.T.C. ✓
G.G. Gaffney -	Gaffney Const'n.
Tom Sloan -	" "

The letter from W. Hashizume, Structural Office, dated 78 08 25 -  
RE: Method of Load Testing Piles - was discussed with the Contractor,  
and the following points were mentioned.

1. A 100 Ton Jack is to be used. This Jack should have guages which have been calibrated recently.
2. Since the design of the Piles is 25 Tons and we want to load Test them at 50 Tons it was felt by some that these piles would fail, however T. Kasmierowski explained the safety factors involved in the design and did not think the Piles would fail.
3. The Contractor was advised that extra plates would be required to cover the tops of the Piles which are to be tested. The Contractor should also have extra shims on hand in case these are required.
4. G. Gaffney stated he was having difficulty obtaining the Reaction Beam. He was advised by T. Kasmierowski that "Birmingham Construction" had one, and perhaps he could obtain it. The Contractor is going to check this out.
5. The Contractor was also instructed that a Censor Beam would be required to record the pressure. This is to be separate from the Reaction Beam.

Meeting Adjourned at 11:30 A.M.

EJA/dh

E. J. Armstrong  
Project Supervisor

c.c. J.W. Reid  
Those in Attend.  
Files.





## Memorandum

To: Mr. R. W. Franks,  
Manager, Construction Office,  
Eastern Region, Kingston.

From: Structural Office,  
West Building, Downsview.

Attention:

Date: 78 08 25

Our File Ref.

In Reply to

Subject:

Contract 78-08, W.P. 76-73-01,  
Wilberforce Bridge, Site 40-52,  
Highway 648, District 10

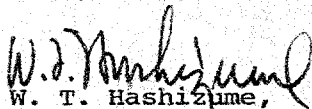
Based on field inspection of 78 08 16, and subsequent consultation with the Soil Mechanics Section, it has been decided that 3 piles in each pier bent (a total of 6 piles) will be test loaded. The decision to test load was conveyed via telephone to Mr. J. Reid of your office on 78 08 17.

The selection of the piles to be tested, the design of the necessary reaction beam, method of installation, construction notes etc. have been completed and we are forwarding herewith the following 6 sketches to you for implementation.

- Sketch "A" Layout of Piles  
"B" Location of Reaction Beams for Testing Piles in South Pier.  
"C" Location of Reaction Beams for Testing Piles in North Pier.  
"D" Typical Details of Reaction Beam  
"E" General Construction Notes  
"F" Scope of Work

Please note that force account has been recommended as basis of payment in Sketch "F", Scope of Work. This was recommended by the Soil Mechanics Section to get the load testing done as expeditiously as possible, and is based on their experience with similar testing done while a bridge was under construction.

WTH/cf  
Attch.

  
W. T. Hashizume,  
Inspection Engineer,  
Eastern Section.

c.c. C. Mirza ✓  
K. Bassi

Mr. K. Bassi  
Structural Design Section  
West Building, Downsview

Soil Mechanics Section  
Engineering Materials Office  
West Building, Downsview

February 11, 1977

Wilberforce Bridge  
W.P. 76-73-01, Site 40-52  
Hwy. 648, District 10, Bancroft

---

We would like to bring to your attention the following with regard to abutment foundations for the above mentioned structure.

The driving of 12  $3/4$ " O.D. x  $1/2$ " thick wall tubular piles should be controlled by Hiley Formula during construction as per current M.T.C. methods to attain the required safe design load. However, for pile length estimation purposes piles driven to approximate elevation 1187 will develop a safe load of 25 tons/pile.

We believe that you will make the necessary changes on the contract drawings.

M. Devata  
Supervising Engineer

MD/gs

cc: A.E. McKim  
M. Stoyanoff  
Files ✓  
Record Services

*Mr. M. Devata*  
December 22, 1976

MEETING OF  
STRUCTURAL REVIEW COMMITTEE

Time: 9:30 a.m., December 15th, 1976

Place: Boardroom B, West Building

Attending: Messrs. A. E. McKim - Construction Branch  
W. Hashizume - Construction Branch  
M. Stoyanoff - Structural Office  
K. Bassi - Structural Office  
R. Kan - Structural Office  
M. Devata - Soil Mechanics Section  
V. Boehnke - Hydrology Section  
F. Gormek - Structural Maintenance Section

Projects Reviewed: (a) Delisle River Bridge,  
Site 31-66, W.P. 28-73-01,  
Highway 34, District 9.  
(b) Wilberforce Bridge,  
Site 40-52, W.P. 76-73-01,  
Highway 648, District 10.  
(c) Madawaska River Bridge,  
Site 29-111, W.P. 127-73-02,  
Highway 41, District 10.  
(d) Rosedale Creek Bridge Widening,  
Site 15-120, W.P. 827-73-02,  
Highway 43, District 8.

Mr. Bassi presented each project to the Committee summarizing the design features, maintenance of traffic, and construction sequence where applicable.

The following comments were made, with recommendations as noted.

Rosedale Creek Bridge Widening (W.P. 827-73-02)

Foundations

The piling requirement was reviewed and the Committee concurred with the Soils Mechanics Section's recommendation that the pile lengths not be shortened but remain as proposed.

The location of the new wall drains will be at the same elevation as the existing wall drains.

Hydrology

The design has incorporated all requirements recommended by the Hydrology Section.

..... 2





### Structure

- (a) The specifications for the lightweight concrete are to be forwarded to Mr. P. Wilson for his comments and/or approval.
- (b) Structural Maintenance felt the depth of asphalt on the deck might be excessive in certain locations and that dishing of the riding surface could be a problem. Mr. Bassi checked this matter with Mr. Corkill of Quality Assurance who advised that no problem is anticipated or expected.
- (c) Machine finish of the deck is not required.

### Wilberforce Bridge (W.P. 76-73-01)

#### Foundations

The matter of unwatering was discussed and it was agreed that a tender item be provided in the contract for unwatering foundations.

#### Hydrology

The recommendations of the Hydrology Section have been incorporated in the design and no problems are expected.

### Structure

- (a) The need for a roadway protection scheme was questioned and it was pointed out that the detour is located far enough away from the new construction to eliminate the problem.
- (b) The design of the deck slab was questioned for adequacy and found satisfactory.
- (c) Polystyrene for forming is to be removed after construction is completed. A note is to be added to the drawings calling for this.
- (d) Machine finish of the deck is not required.

### Delisle River Bridge (W.P. 28-73-01)

#### Foundations

The depth of footings was discussed and it was concluded that as the footings are well into and cast against rock no foundation problems are expected. This also satisfies the hydrological requirements for protection against scour.

Structure

- (a) The extent of rip-rap was questioned by Hydrology and was found satisfactory.
- (b) The drawings have been amended to incorporate the requirements of the Regional Review meeting.

Madawaska River Bridge (W.P. 127-73-02)

The sequence of construction was reviewed and discussed to ascertain possible problems in carrying out the work.

The Committee recommends that the work be done in two stages and that stage I be fully completed including waterproofing and asphalt before any work commences on stage II.

Attached is a copy of the sequence of work as may be shown on the structure drawings. The Committee points out that no traffic is to be allowed on the exposed concrete deck.

No other points were brought up and the meeting adjourned at 11:45 a.m.

MS/im  
Attach.

*M. Stoyanoff*  
M. Stoyanoff,  
Structural Contract Engineer.

c.c. J. B. Wilkes  
W. G. Wigle  
E. J. Orr  
R. A. Dorton  
C. S. Grebski  
T. C. Kingsland  
W. Lin  
J. Keen  
A. Radkowski  
W. McFarlane  
All attending meeting

## Madawaska at Griffith

### Sequence of Work

Stage I : Restrict traffic to one northerly lane and carry out all the remedial work on the south half of the bridge including waterproofing and placing new asphalt on the deck.

Stage II: Restrict traffic to one southerly lane and carry out all the remedial work on the north half of the bridge.

The remedial work in the deck soffit may be carried out either during Stage I or Stage II.

No traffic shall be allowed on the exposed concrete deck at anytime.

Mr. C.S. Grebski  
Structural Design Engineer  
Structural Design Section  
West Building, Downsview

Mr. K. Bassi

Soil Mechanics Section  
Geotechnical Office  
West Building, Downsview

July 13, 1976

Wilberforce Bridge  
W.P. 76-73-01, Site 40-52  
District 10, Bancroft

Attached please find the enclosed subsoil data for the above mentioned site which was carried out by the Regional Materials and Testing Office as per our request. The field work was performed to determine, a) the type and extent of the organic material in the lake bottom within the plan limits of the proposed embankment widening to the west, and b) the type of fill material used in the construction of that portion of the existing causeway where tubular piles are proposed to be driven. In view of this information, we are providing the following comments.

1. Organic material of 3 to 5 feet in thickness was encountered within the lake bottom and was determined by means of hand augers. This organic material should be excavated completely within the plan limits of the proposed widening and replaced with suitable granular material.
2. Borings through the fill material were put down by means of a truck mounted power auger. Few cobbles and/or boulders were encountered at 4 to 6 ft. depths. This aspect should be brought up to the attention of the contractor so that he has the necessary equipment to remove them and thus be able to drive the proposed 12 3/4" O.D. tubular piles.

H. Shah  
Project Engineer

For: M. Devata  
Supervising Engineer

cc: T.C. Kingsland  
Files  
Record Services

Due to possible presence of cobbles and/or boulders within the top 8' from the existing ground level the piles shall be driven through 12"  $\phi$  holes Preaugered down to elev. 1238.00 The piles shall be driven to EL. 1187.00

H. Shah  
July 19, 1976.



## Memorandum

To: Mr. M. Devata  
Supervising Engineer  
Soils Mechanics Section  
Geotechnical Office  
West Bldg., Downsview

Attention: H. Shah, Project Engineer

Our File Ref.

From: Materials & Testing Office  
Kingston, Ontario

Date: June 18, 1976

In Reply to

Subject:

Wilberforce Bridge, W.P. 76-73-01, Site 40-52,  
Hwy 648, District 10 - Bancroft

The field work which you requested our office to undertake for the above bridge has now been completed, and the borehole data is appended.

The existing roadway mainly consists of mulch over crushed gravel, over fine sand with a few stones. The subgrade on the embankment is comprised of fine gravel, and a few large boulders (24" to 26" maximum diameter) were encountered at 4'-6' depths. Large boulders up to 36" diameter were placed on the west bank for slope stabilization and protection from wave erosion.

Our field crew could not locate any large organic deposits in the lake bed. Boreholes showed organic material for depths of 3', 4', and 5', in the deeper areas of the lake, underlain by fine sandy loam or clean fine sand. The lake bed in the shallower spots consists of fine sandy loam over boulders.

Two hand auger shots were also taken at the present bridge location in the existing channel in order to help predict the stability of the future backfill to the channel. Here, gravelly fine sand was encountered.

We hope that the above information will be of some use in your bridge investigation.

D. G. Guibord  
Project Soils Engineer

DGG/rao



(Note: 108+00 60' Lt (-3.5') )  
( Edge of Water)

Note: All offsets taken from exist t

(1) 108+10 50' Lt (-4')  
0-36" Water 1241.3  
36-66" Br F Sa Lo 5.5  
66" NFP Blds 1233.8

(12) 111+35 45' Lt (-7')  
0-36" Water 1241.3  
36-60" Br F Sa Lo 5.0  
60" NFP Blds 1236.3

(Note: 111+50 45' Lt (-7') )  
( Edge of Water)

Shots taken from drains in Exist Struct

(9) 110+30 7' Lt  
0-66" Water 1241.3  
66-96" Br F Sa (Gravelly) 8.0  
96" NFP (Can't Twist) 1233.3

(10) 110+45 7' Rt  
0-66" Water 1241.3  
66-96" Br F Sa (Gravelly) 8.0  
96" NFP (Can't Twist) 1233.3

(2) 108+60 45' Lt (-4') 1241.3  
0-72" Water 11.0  
72-10' Blk Amor Gran Org 1230.3  
10-11' Br F Sa Lo  
Note: Logs & Garbage on bottom

(7) 109+50 45' Lt (-5')  
0-84" Water 1241.3  
84-10' Blk Amor Gran Org 12.0  
10-12' Br F Sa (Clean) 1229.3  
12' NFP (Can't Twist)

(11) 110+50 45' Lt (-7')  
0-96" Water 1241.3  
96-13' Blk Amor Gran Org 15.0  
13-15' Br F Sa (Clean) 1226.3

Note: After 109+50 Lt  
holes were drilled  
continuous because of  
"sloughing"

D.M.

109+50 ③ 8' Lt (Exist  $\phi$ )  
0-3" Mulch  $\frac{1248.0}{15.5}$   
3-8" Br Cr Gr  $\frac{1233.5}{15.5}$   
8-96" Br F Sa Few Stns  
96-13' Gry F Gr (Sat 96") 76-LL-85 ASC  
13-14' Org 1.7% P #270  
14-15½' Gry Co Sa 2.1% P #200  
4.2% P #100  
12.7% P # 50

N.B. offsets taken from existing  $\phi$   
D.G.G.

⑤ 109+20 8' Lt (Exist  $\phi$ )  
0-3" Mulch  $\frac{1248.0}{15.5}$   
3-8" Br Cr Gr  $\frac{1232.5}{15.5}$   
8-72" Br F Sa Few Stns  
72-15½' Gry F Gr (Few Lrg Blds @ 72")

109+00 to 110+00 — 20' Lt (-48") Lrg Blds up to 36"  
Dia for bank stabl.

③ 108+90 8' Lt (Exist  $\phi$ )  
0-3" Mulch  $\frac{1248.0}{15.5}$   
3-8" Br Cr Gr  $\frac{1232.5}{15.5}$   
8-72" Br F Sa Few Stns  
72-15½' Gry F Gr (Lrg Blds 48" to 72")

④ 109+10 8' Rt (Exist  $\phi$ )  
0-3" Mulch  $\frac{1248.0}{15.5}$   
3-7" Br Cr Gr  $\frac{1232.5}{15.5}$   
7-72" Br F Sa Few Stns  
72-15½' Gry F Gr (Few Blds @ 72")

⑥ 109+75 8' Rt (Exist  $\phi$ )  
0-3" Mulch  $\frac{1248.0}{15.5}$   
3-8" Br F Gr  $\frac{1233.5}{15.5}$   
8-72" Br F Sa Few Stns  
72-15½' Gry F Gr (Few Blds @ 72")

Mr. C.S. Brebski  
Structural Design Engineer  
Structural Office  
West Bldg.

Soil Mechanics Section  
Geotechnical Office  
West Bldg.

Mr. K. Bassi

June 4, 1976

**Wilberforce Bridge**  
W.P. 76-73-01, Site 40-52  
District 10, Bancroft

---

Our comments pertaining to the review of the Preliminary Bridge Plan Dwg. 40-52-P1 for the above mentioned structure are as follows:

1. At this location 12 3/4" O.D. tubular piles (friction piles) will be driven through the existing fill material. It is possible that the fill may contain cobbles and boulders which may obstruct the driving of piles through the fill material. In order to ascertain this aspect, we have requested the Regional Materials and Testing Office, Kingston, to carry out the necessary field work.
2. The roadway embankment will be widened some 12 ft. on the west side of the existing roadway in the immediate vicinity of the structure. Our subsurface investigation carried out in 1960 revealed the presence of organic material in the upper portion of the lake bed. In view of this, we have requested the Regional Materials and Testing Office, Kingston, to determine the extent and depth of the organic material in the area of the widened portion of the embankment. The subexcavation requirements of this organic material will be provided after the receipt of the requested data from the Regional M & T office.
3. The portion of the new embankment should be 'keyed' into the existing embankment in accordance with current M.T.C. practices.

H. Shah  
Project Engineer

For: M. Devata  
Supervising Engineer

HS/bp

cc: R. Forrest  
T.C. Kingsland  
S. Radbone  
E. Saint - Att: Mr. A.M. Batten

Files  
Record Services



Mr. K.C. Bassi  
Regional Structural Design Engineer  
Structural Design Section  
West Building, Downsview

Soil Mechanics Section  
Geotechnical Office  
West Building, Downsview

April 9, 1976

Wilberforce Bridge (Pusey Lake)  
Hwy. 648, Line 'J'  
District 10, Bancroft  
W.P. 76-73-01, Site 40-52

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Further to your verbal request, we have reviewed the subsurface conditions at the above mentioned site and submit the following:

It is understood that the new structure will be located at Sta. 109 + 36 Hwy. 648 (Line 'J'), some 102 feet west of the present crossing. The new bridge will be a three span structure with perched abutments. There will be a slight raise in the grade approximately three feet above the existing roadway surface. The roadway embankment will be widened by some 12 feet on the west side. The creek bottom at the proposed crossing will be about elevation 1237.

The proposed slopes with 2 horizontal to 1 vertical will be stable, both in the longitudinal as well as in the transverse directions. The portion of the new embankment should be keyed into the existing one as per current MTC methods.

The perched abutments and the pier bents can be supported on steel tubular (12 3/4" O.D.) friction piles driven into the granular subsoil with the following design loads.

<u>Tip elev.</u>	<u>Allowable load/pile</u>
1207	15 tons
1197	20 tons
1187	25 tons

Alternatively, end-bearing expanded base franki type concrete caissons may be considered at this location. The capacity and pertinent detailed information on this type of caisson could be investigated further if needed.

*H*  
H. Shah  
Project Engineer

For: M. Devata  
Supervising Engineer

cc: R. Forrest	E. Saint
T.J. Kingsland	Files
H.K. Kirchner	Record Services
S. Radbone	

MEMORANDUM

TO: Mr. T.C. Kingsland  
Regional Structural Planning Engineer  
Structural Planning Office  
Kingston, Ontario

FROM: Soil Mechanics Section  
Geotechnical Office  
West Building

ATTENTION:

DATE: January 15, 1976

OUR FILE REF.

IN REPLY TO

SUBJECT: WILBERFORCE BRIDGE (PUSEY LAKE)  
Hwy. 648, Line "J"  
District 10, Bancroft  
W.P. 76-73-01 Site 40-52

Further to our telephone conversation, we are forwarding to you a copy of the report for the foundation investigation carried in the immediate vicinity of the above mentioned site. This investigation was undertaken during 1960 under W.P. 244-60 (W.J. 60-F-32), formerly Dist. 11 and Hwy. 500, with job description of Pusey Lake 1.0 mile north of Wilberforce.

We believe that the data contained in this report will be adequate for Preliminary Design requirements. When the project is finalized, if necessary, additional investigation with pertinent recommendations will be provided by this Section.

*H. Shah*

H. Shah  
Project Engineer

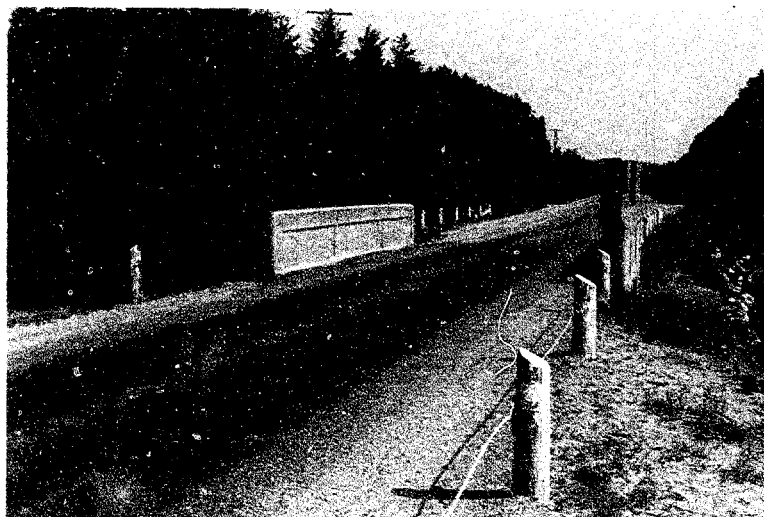
For: M. Devata  
Supervising Engineer

HS/bp

cc: J. Anderson )  
K. Bassi )  
R. Forrest ) Memo only  
C. Grebski )  
S. Radbone )



GRACE LAKE - PUSEY LAKE BRIDGE  
WILBERFORCE - HWY. 648 SITE 40-52  
LOOKING WEST



GRACE LAKE - PUSEY LAKE BRIDGE  
WILBERFORCE - HWY. 648 SITE 40-52  
LOOKING EAST

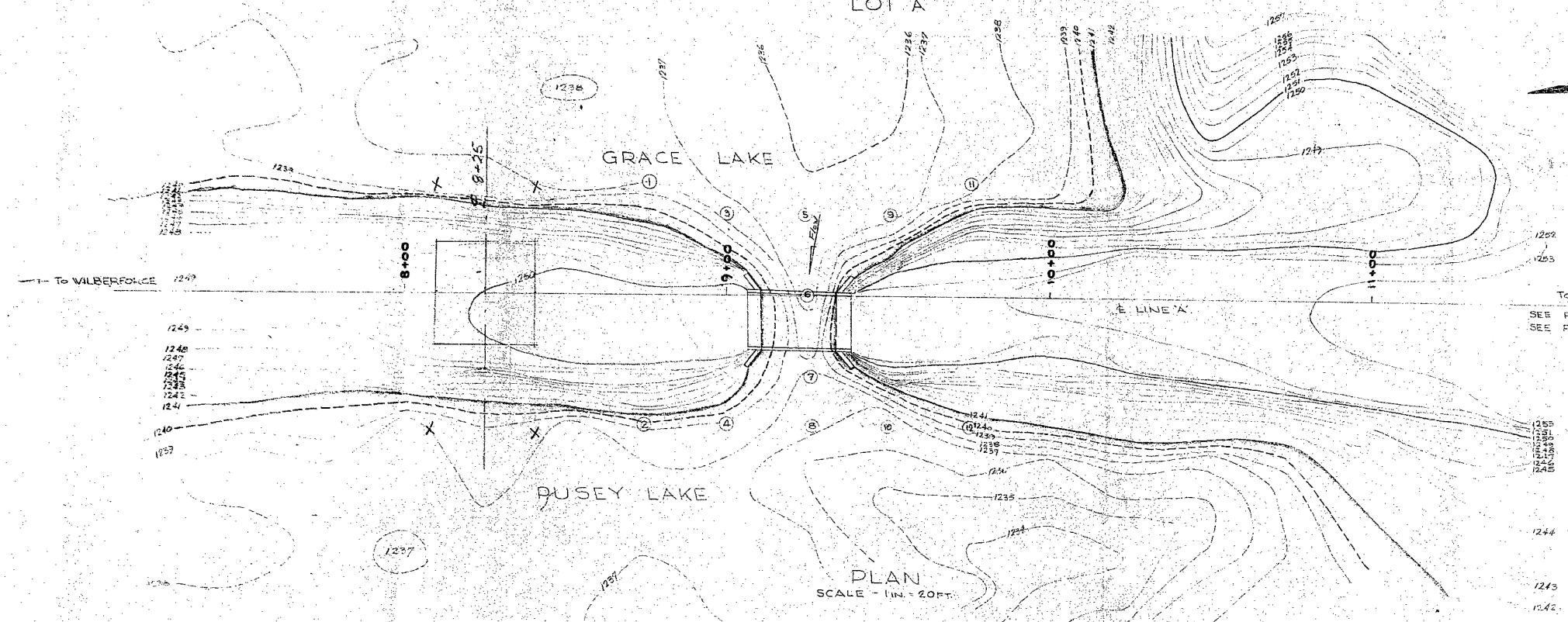
# 60-F-32

W.P. # 244-60

Hwy # 500

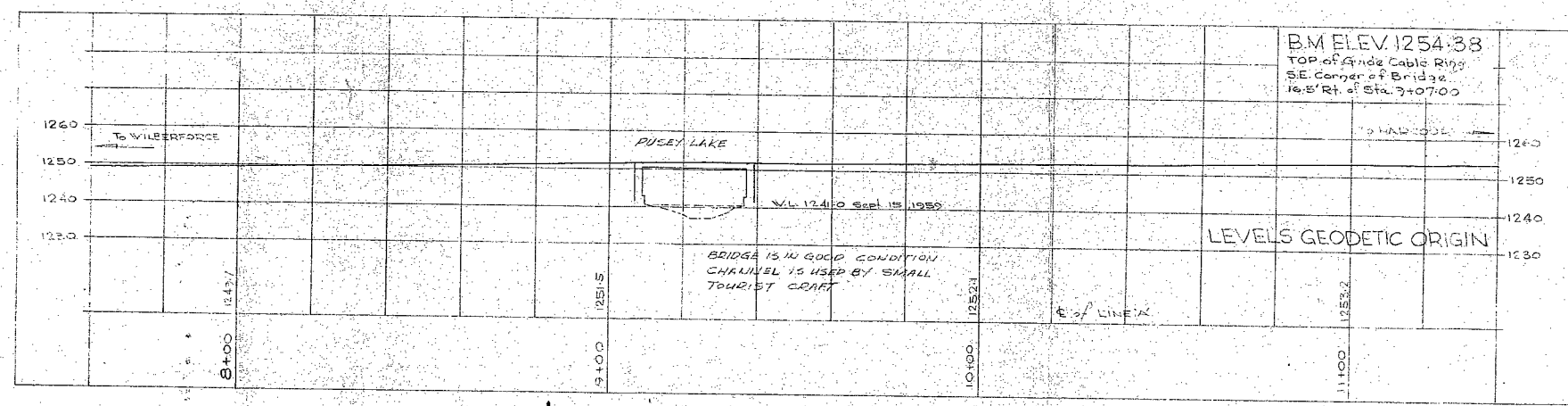
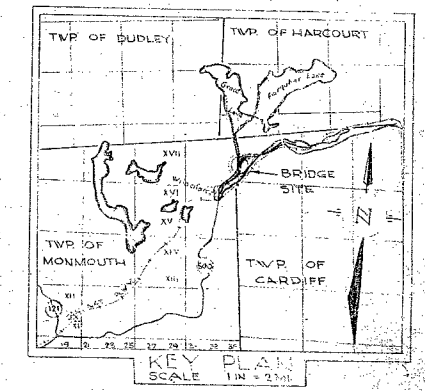
& PUSEY LAKE

COUNTY OF HALIBURTON  
TOWNSHIP OF CARDIFF  
CON 22  
LOT A



TO VALBERFORCE  
SEE PLAN N° E 3469-6  
SEE PROFILE N° C 1649

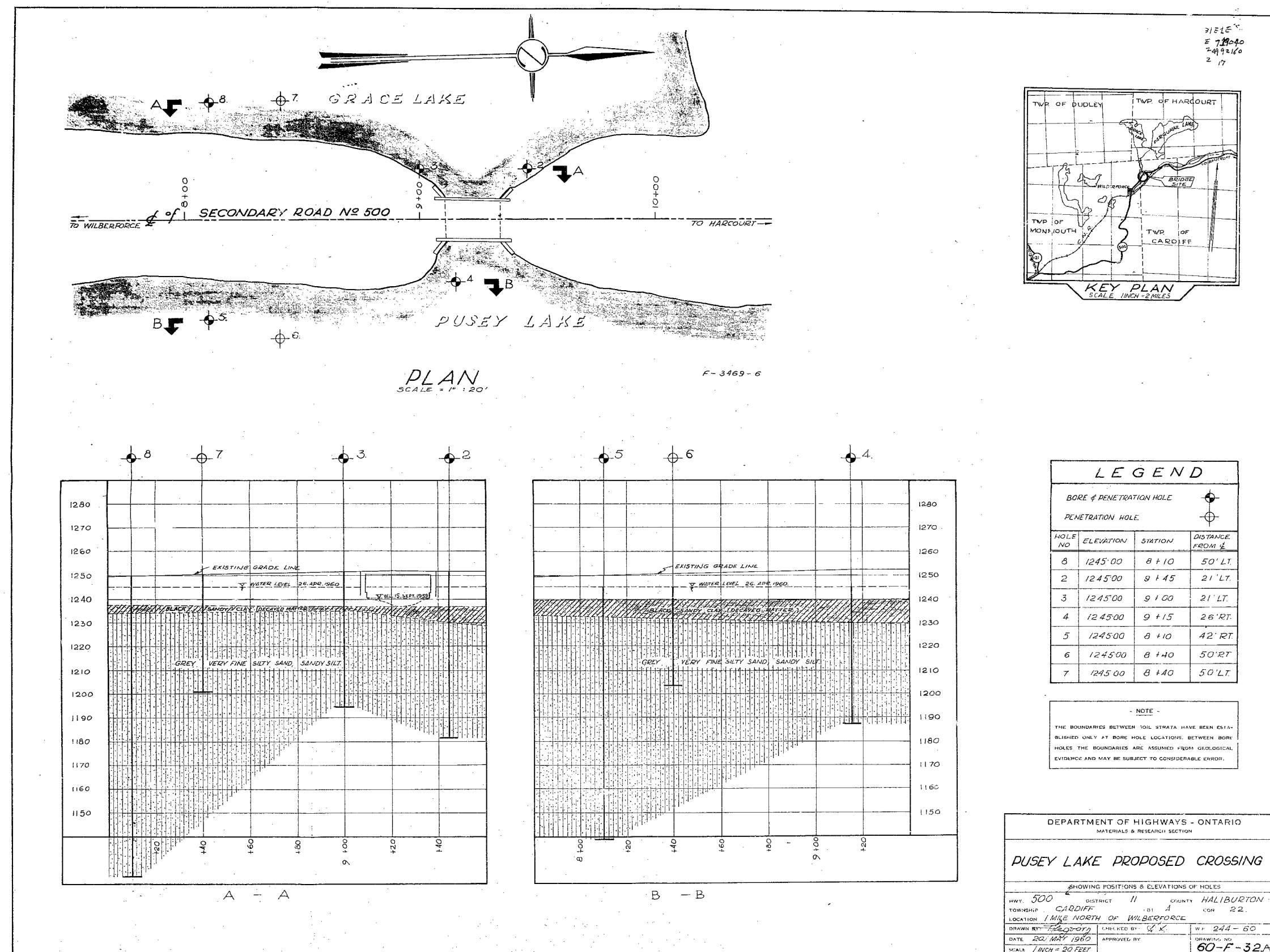
SOUNDING DATA			
N°	STATION	DEPTH	BOTTOM
1	8+75 35' Lt.	2 1/2' Water	2' 0" Sand to Rock
2	40' Rt.	3' 0" Water	2' 0" Sand to Rock
3	9+00 25' Lt.	3' 5" Water	3' 5" Sand to Rock
4	40' Rt.	2' 0" Water	5' 0" Sand to Rock
5	9+25 4'	3' 0" Water	4' 5" Sand to Rock
6	25' Lt.	5' 0" Water	4' 0" Sand to Rock
7	25' Rt.	3' 0" Water	1' 0" Sand to Rock
8	40' Lt.	5' 0" Water	3' 0" Sand to Rock
9	9+50 25' Lt.	4' 0" Water	4' 0" Sand to Rock
10	40' Rt.	3' 0" Water	5' 0" Sand to Rock
11	9+75 35' Lt.	3' 0" Water	5' 0" Sand to Rock
12	40' Rt.	1' 5" Water	4' 5" Sand to Rock



BM ELEV. 1254.38  
TOP of Grade Cable Ring  
SE. Corner of Bridge  
16.5' Rt. of Sta. 9+07.00

PROFILE  
SCALE - HOR - 1 in = 20 ft.  
VER - 1 in = 20 ft.

DEPARTMENT OF HIGHWAYS - ONTARIO  
SURVEYS BRANCH  
DISTRICT N° 11  
PROPOSED CROSSING  
AT  
PUSEY LAKE  
AND  
SECONDARY ROAD 118.000  
APPROX 1 MI. NORTH OF VALBERFORCE  
LOT A CON 22  
TOWNSHIP OF CARDIFF COUNTY OF HALIBURTON  
BRIDGE SITE  
SURVEYED BY N. N. LEVYMAN  
SUPERVISOR H. S. PATE  
DRAWN BY S. PETERSON  
SUPERVISOR J. E. KNUTT  
CHECKED BY  
DRAFTSMAN SUPERVISOR  
APPROVED  
SCALE - AS SHOWN  
DATE OF SURVEY - SEPT. 1959  
DATE OF PLAN - OCT. 1959  
PLAN - 10 F 1  
PLAN - E 3693-1



DOCUMENT AND NUMBER IDENTIFICATION

GEOCRES No. 31E-01

DIST. 10 REGION EASTERN

W.P. No. 76-73-01

CONT. No. 78-08

W. O. No. \_\_\_\_\_

STR. SITE No. 40-52

HWY. No. 648

LOCATION WILBERFORCE BRIDGE

OVERHEAD DRAWINGS TO BE INCLUDED WITH THIS REPORT 4

REMARKS: \_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

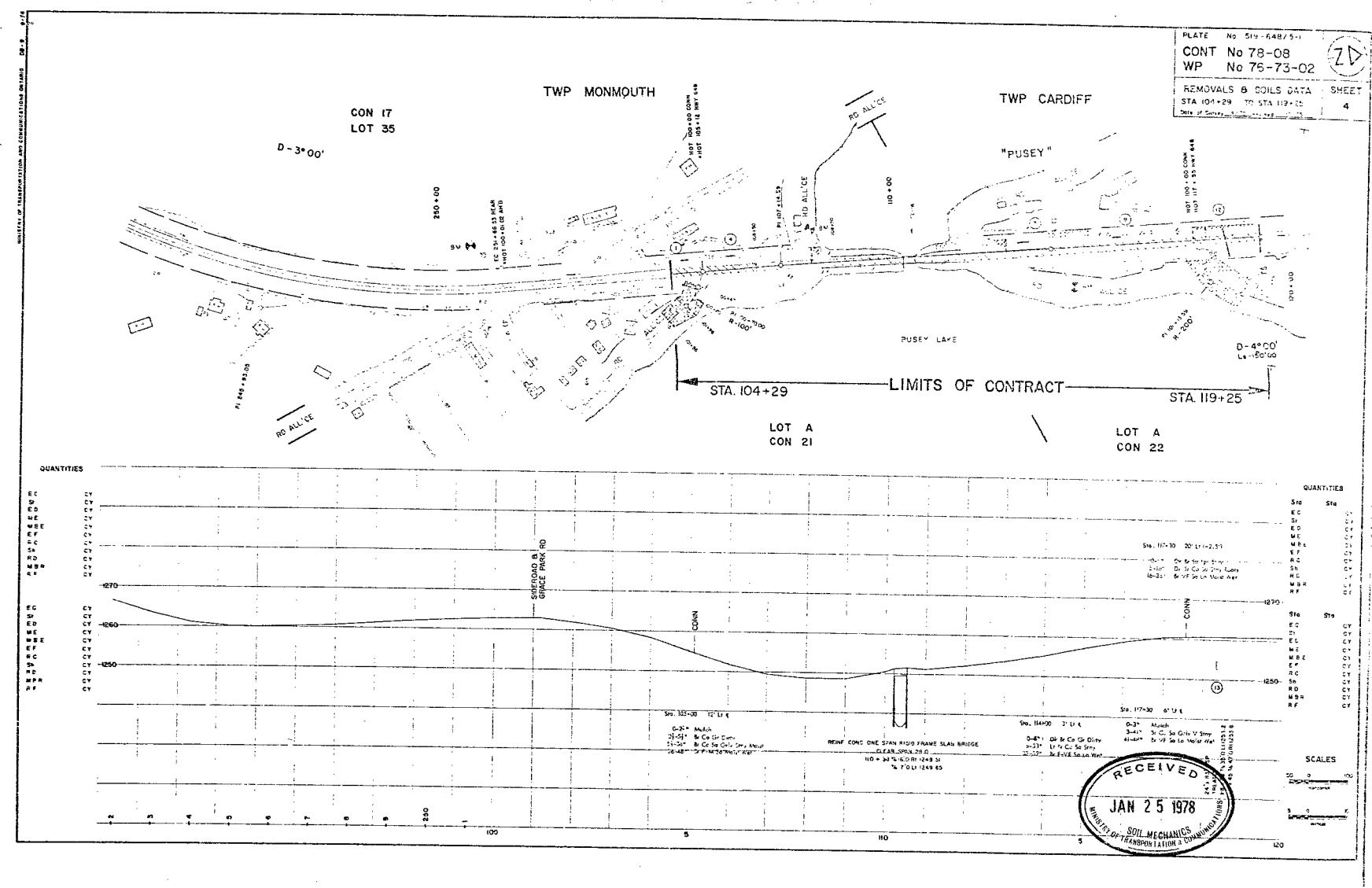


PLATE No 514-648/5-1  
CONT No 78-08  
WP No 76-73-02  
REMOVALS & SOILS DATA  
STA 104+29 TO STA 119+25  
SHEET 4

QUANTITIES

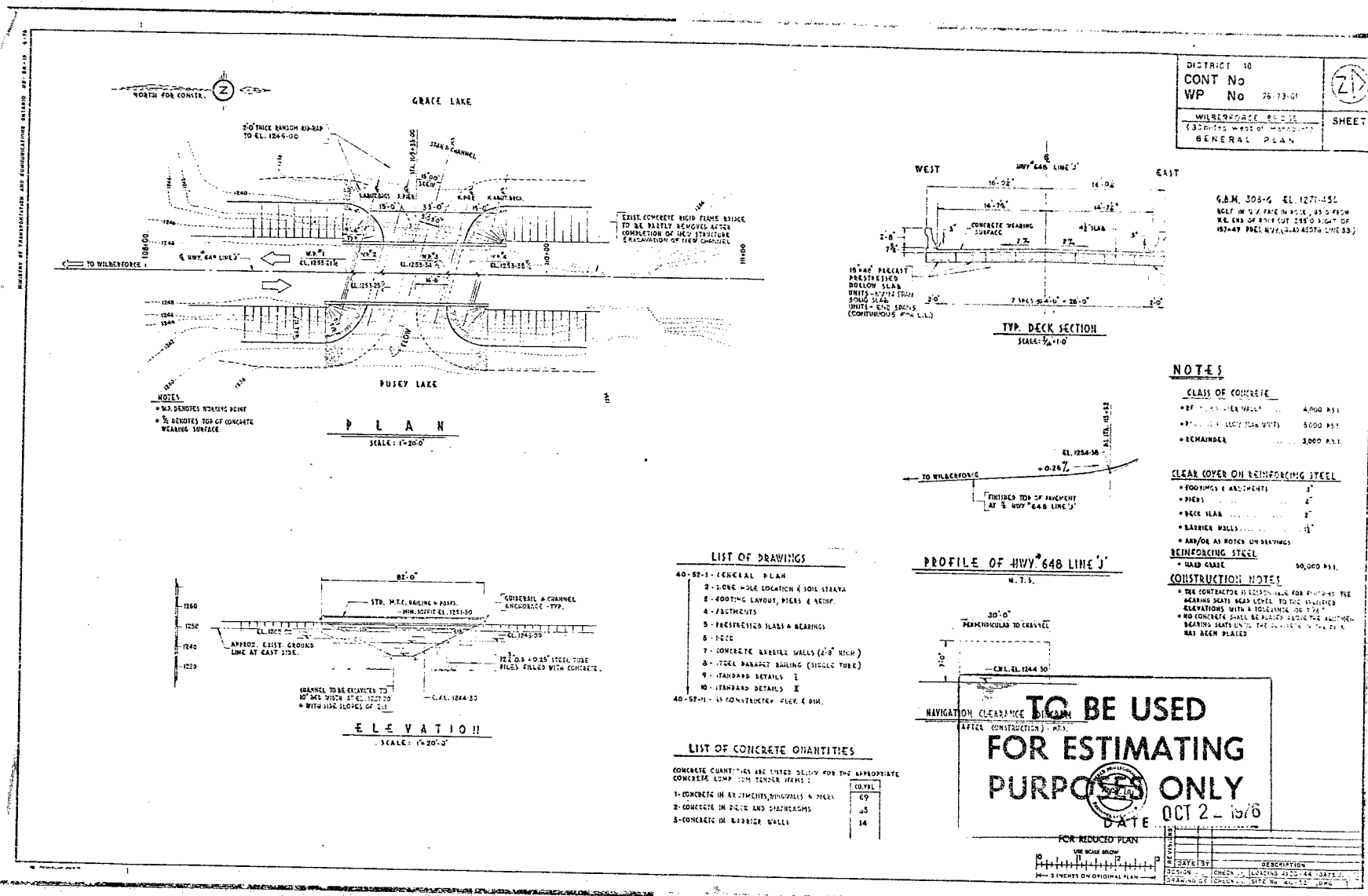
EC	CY
ED	CY
EE	CY
EF	CY
EG	CY
EH	CY
FI	CY
FJ	CY
FK	CY
FL	CY
FM	CY
FN	CY
FO	CY
FP	CY
FQ	CY
FR	CY
FS	CY
FT	CY
FU	CY
FV	CY
FW	CY
FX	CY
FY	CY
FZ	CY

QUANTITIES

Stn	Sta
EC	CY
ED	CY
EE	CY
EF	CY
EG	CY
EH	CY
FI	CY
FJ	CY
FK	CY
FL	CY
FM	CY
FN	CY
FO	CY
FP	CY
FQ	CY
FR	CY
FS	CY
FT	CY
FU	CY
FV	CY
FW	CY
FX	CY
FY	CY
FZ	CY

SCALES  
1" = 40'  
1" = 10'





DISTRICT 10  
CONT No  
WP No 78-73-01  
WILKESBORO, N.C.  
(3.3 MILES WEST OF WILKESBORO)  
GENERAL PLAN



SHEET

G.M. 304-6 EL. 1271-152  
N.E. END OF BRIDGE CUT 245.0' SIGHT OF  
157-49' PREL. WYE (2.43 ADJUSTED LINE 33)

NOTES

- CLASS OF CONCRETE
- 4000 PSI
  - 5000 PSI
  - 3000 PSI

- CLEAR COVER ON REINFORCING STEEL
- FOOTINGS & ABUTMENTS 4"
  - PIERS 4"
  - DECK SLAB 4"
  - BARRIER WALLS 4"
  - AND/OR AS NOTED ON DRAWINGS

- REINFORCING STEEL
- 50,000 PSI

- CONSTRUCTION NOTES
- THE CONTRACTOR IS TO BE RESPONSIBLE FOR THE PROTECTION OF THE EXISTING UTILITIES AND STRUCTURES AND FOR THE PROTECTION OF THE ADJACENT PROPERTIES.
  - NO CONSTRUCTION SHALL BE PERMITTED UNTIL THE NECESSARY PERMITS HAVE BEEN OBTAINED.
  - THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE PROTECTION OF THE EXISTING UTILITIES AND STRUCTURES AND FOR THE PROTECTION OF THE ADJACENT PROPERTIES.

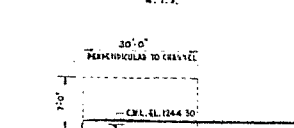
LIST OF DRAWINGS

- 40-82-1 - GENERAL PLAN
- 2 - SOIL LOG LOCATION & SOIL STRATA
- 3 - FOOTING LAYOUT, PIERS & BEAMS
- 4 - FOOTINGS
- 5 - PRESTRESSED SLAB & BEAMS
- 6 - PIER
- 7 - CONCRETE BARRIER WALLS (2'-8" HIGH)
- 8 - STEEL MANHOLE BAILING (SINGLE TUBE)
- 9 - STANDARD DETAILS I
- 10 - STANDARD DETAILS II
- 40-82-11 - CONSTRUCTION PLAN & DIM.

LIST OF CONCRETE QUANTITIES

CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE CONCRETE CLASS (SEE TENDER SHEET)	CUBIC YARDS
1 - CONCRETE IN ABUTMENTS, PIERS & BEAMS	69
2 - CONCRETE IN PIER AND SUBSTRUCTURES	25
3 - CONCRETE IN BARRIER WALLS	14

PROFILE OF HWY 648 LINE 'J'



TO BE USED  
FOR ESTIMATING  
PURPOSES ONLY  
DATE OCT 2 - 1966

FOR REDUCED PLAN

USE SCALE 1/2"=1'-0"

1"=1' ON ORIGINAL PLAN

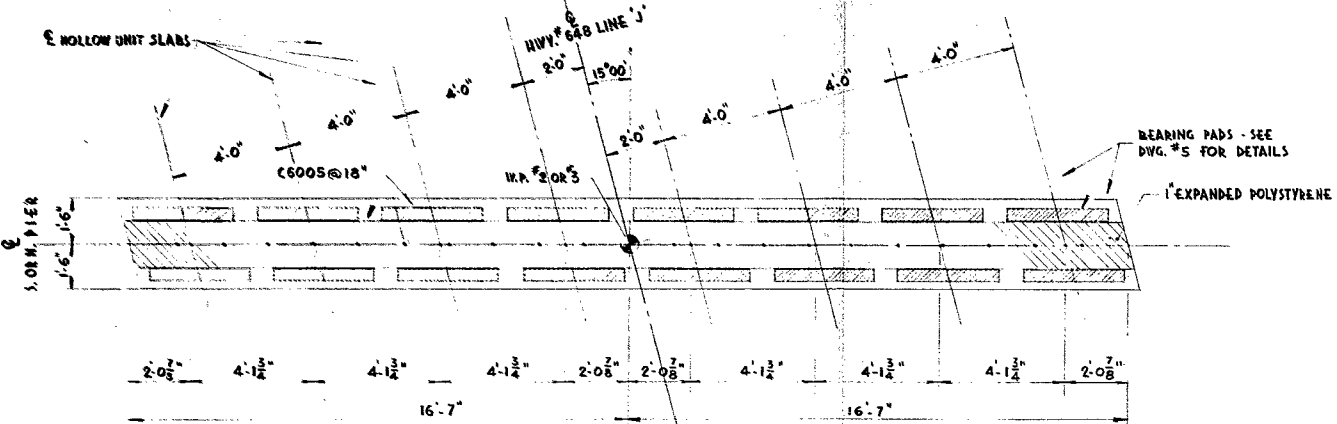
DATE	DESCRIPTION
10/2/66	DESIGN
10/2/66	CONSTRUCTION
10/2/66	AS BUILT

31E-01

CONT No  
WP No 76-73-01  
WILBERFORCE BRIDGE  
(38 miles west of Harcourt)  
FOOTING LAYOUT, PIERS & REIN.



SHEET



PIER CAP PLAN

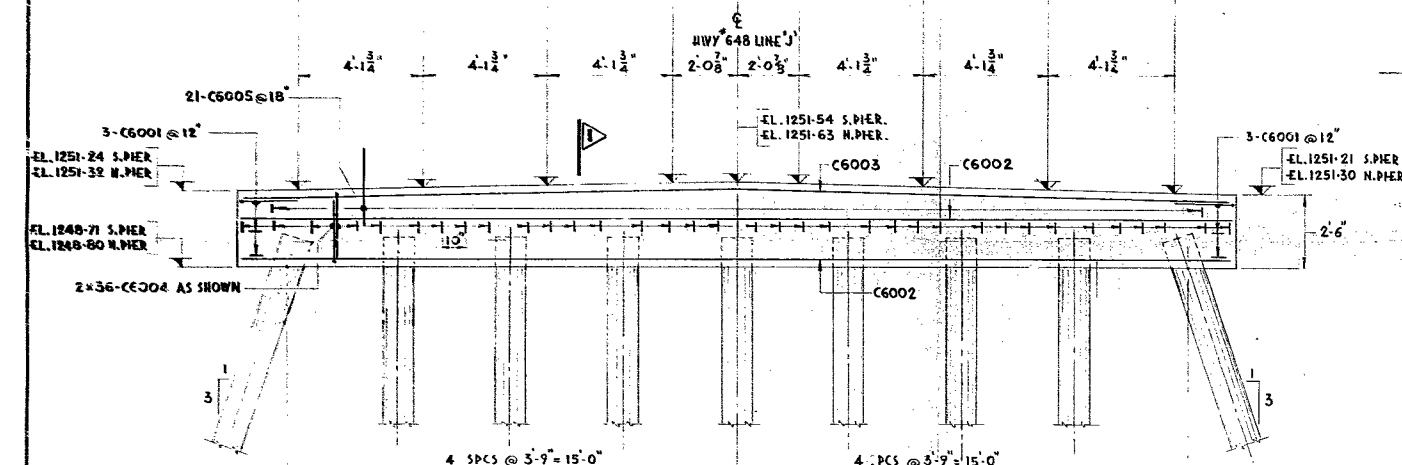
SCALE: 3/8" = 1'-0"

WEST

PIER CAP ELEVATIONS

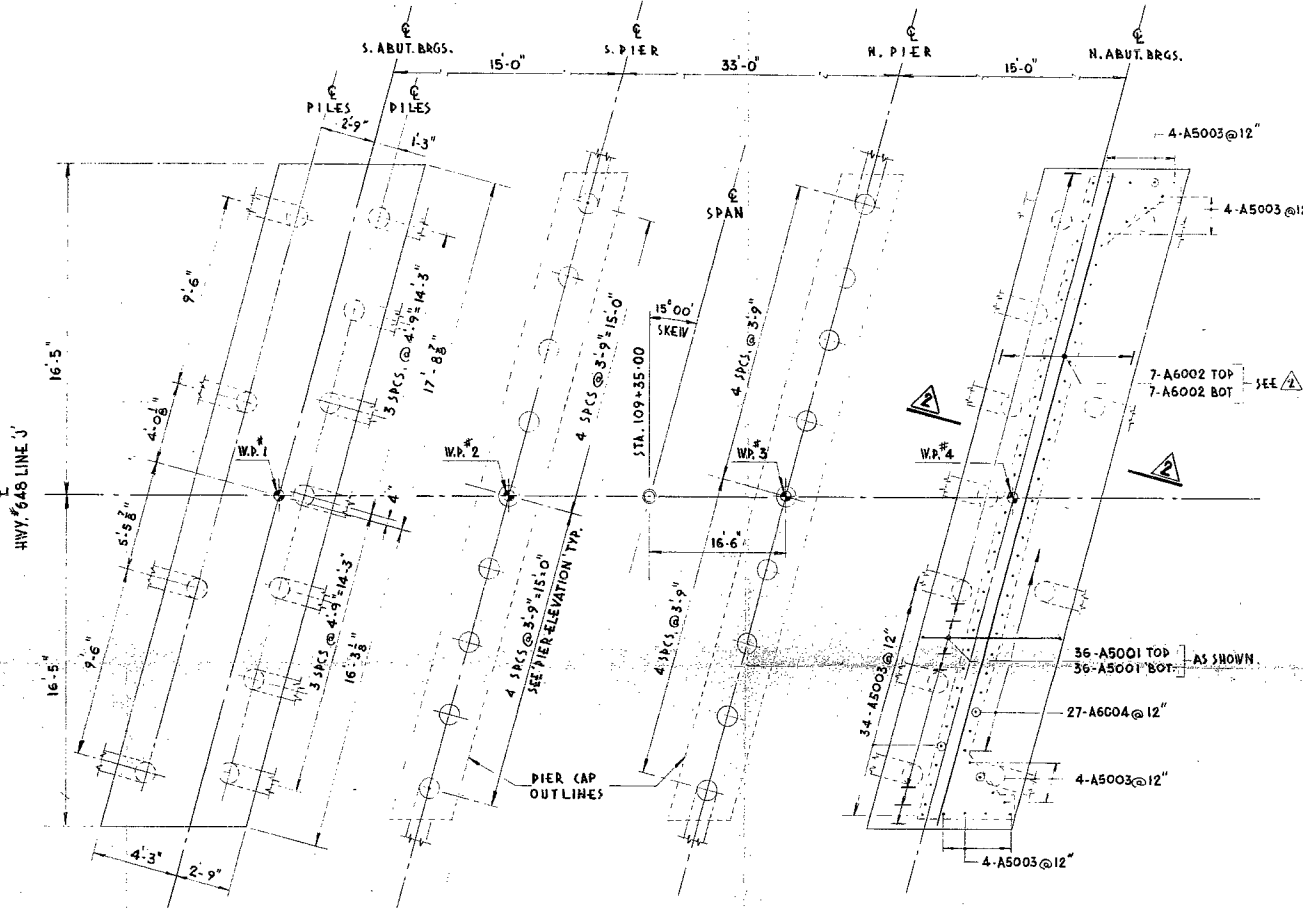
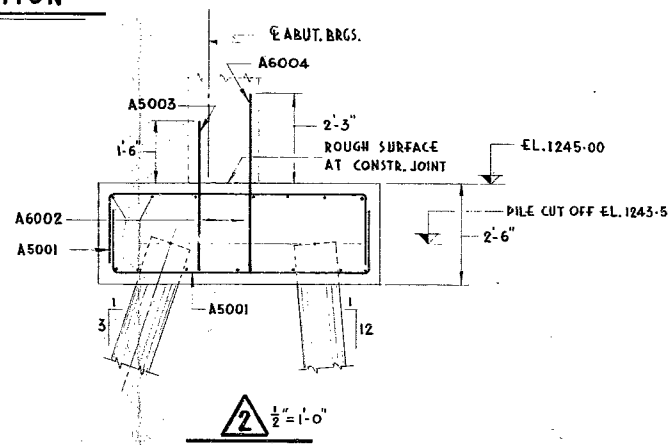
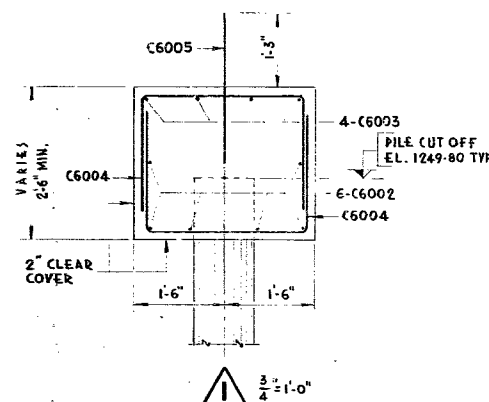
EAST

AT S. PIER	1251-27	1251-35	1251-43	1251-51	1251-50	1251-42	1251-34	1251-25
AT N. PIER	1251-36	1251-44	1251-51	1251-59	1251-59	1251-51	1251-42	1251-34



PIER ELEVATION

SCALE: 3/8" = 1'-0"



FOUNDATION LAYOUT

SCALE: 1/4" = 1'-0"

PILE DATA

LOCATION	BATTER	N <sup>o</sup> REQD.	LENGTH	TYPE	DESIGN LOAD
S. ABUT.	1:3	7	60'-0"	STEEL TUBE PILES	25 TONS / PILE
	1:12	4	57'-0"	12 1/2" O.D. 1/2" WALL THICKNESS FILLED WITH 3000 PSI. CONCRETE.	
S. PIER	1:3	2	66'-0"	STEEL FOR TUBE PILES TO HAVE 0-3% COPPER CONTENT.	
	VERT.	7	63'-0"		
N. PIER	1:3	2	66'-0"		
	VERT.	7	63'-0"		
N. ABUT.	1:3	7	60'-0"		
	1:12	4	57'-0"		

CONCRETE FILL FOR TUBE PILES 74CU.YDS.

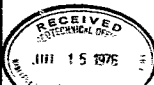
NOTES

- PILE LAYOUT, ABUT. FOOTING DIMENSIONS AND REIN. SIMILAR FOR BOTH ABUTMENTS EXCEPT WHERE NOTED.
- PILE LAYOUT, PIER CAP DIMENSIONS AND REINFORCING SIMILAR FOR BOTH PIER CAPS EXCEPT WHERE NOTED.
- PILE SPACING TO BE MEASURED AT UNDERSIDE OF ABUT. FOOTINGS OR PIER CAPS.
- PILES TO BE DRIVEN TO EL. 1187.00
- STEEL TUBES SHALL CONFORM TO A.S.M.T. SPECIFICATION A252 GRADE 2.
- FOR TUBE PILE SPLICE (IF REQUIRED) AND SHOE DETAILS SEE STD SS 3-2 ON DWG #9.

FOR REDUCED PLAN

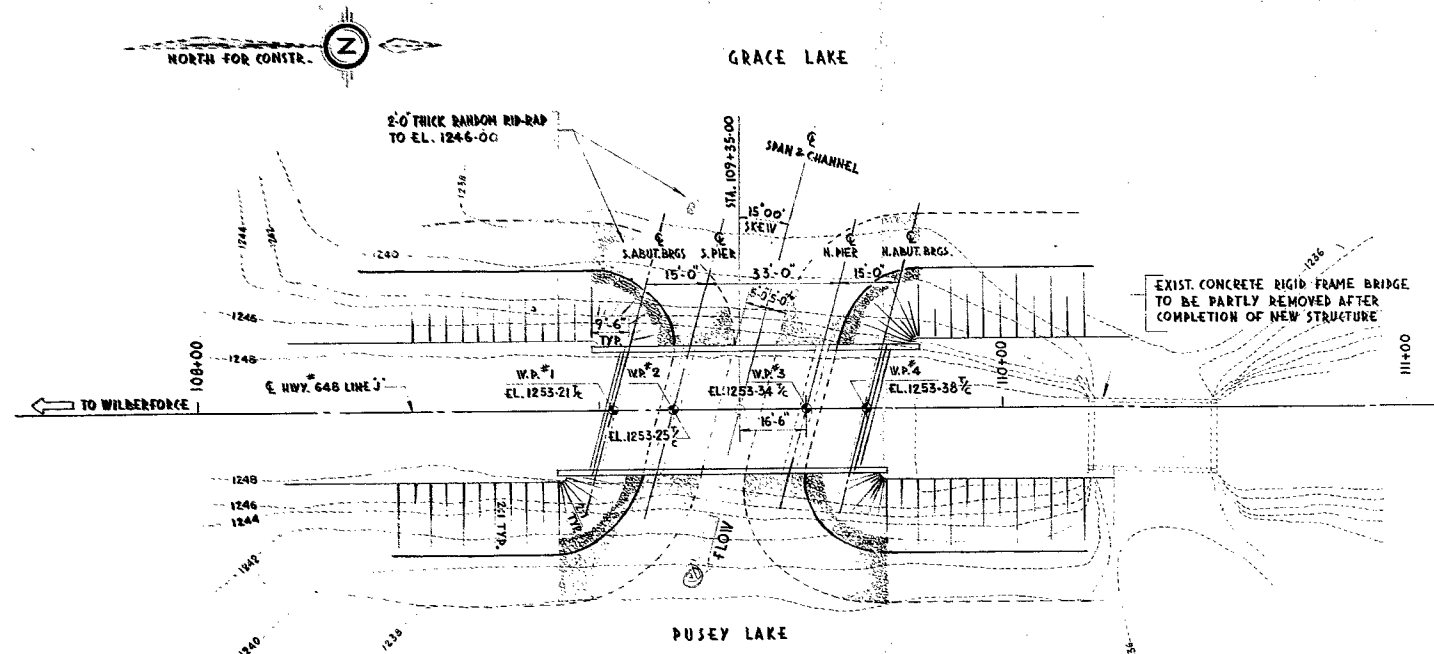
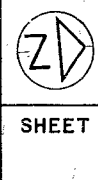


REVISIONS	DATE	BY	DESCRIPTION
DESIGN	7/15/76	AL	LOADING H520-44
DRAWING	7/15/76	G.C.	SITE No 40-52 DWG 3



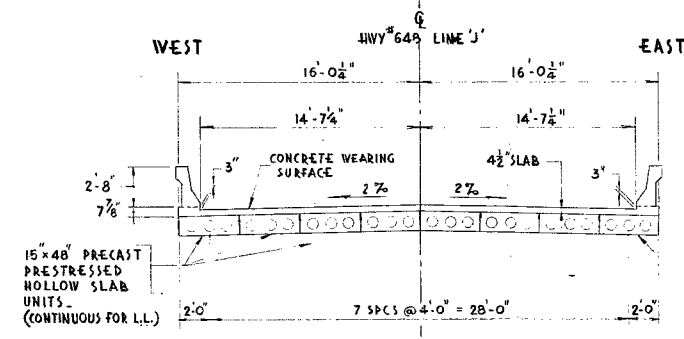
31E-01

DISTRICT 10  
CONT No  
WP No 76-73-01  
WILBERFORCE BRIDGE  
(3.8 miles west of Harcourt)  
GENERAL PLAN



NOTES  
• W.P. DENOTES WORKING POINT  
• 1/2" DENOTES TOP OF CONCRETE WEARING SURFACE

PLAN  
SCALE: 1"=20'-0"



TYP. DECK SECTION  
SCALE: 3/16"=1'-0"

G.B.M. 308-G EL. 1271-455  
BOLT IN N.W. FACE IN ROCK, 85'-0" FROM  
N.E. END OF ROCK CUT 255'-0" RIGHT OF  
157+49 PRES. HWY. (QUAD 45078 LINE 88)

NOTES

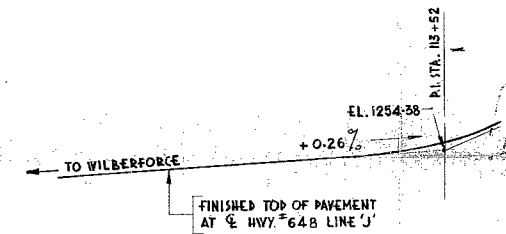
- CLASS OF CONCRETE
- DECK, BARRIER WALLS ..... 4,000 P.S.I.
  - PRECAST HOLLOW SLAB UNITS ..... 5,000 P.S.I.
  - REMAINDER ..... 3,000 P.S.I.

CLEAR COVER ON REINFORCING STEEL

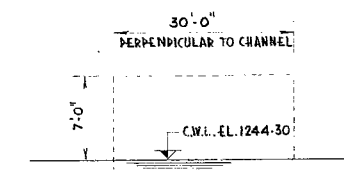
- FOOTINGS & ABUTMENTS ..... 3"
- PIERS ..... 2"
- DECK SLAB ..... 2"
- BARRIER WALLS ..... 1 1/2"
- AND/OR AS NOTED ON DRAWINGS

CONSTRUCTION NOTES

- THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF  $\pm 1/8"$ .
- NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.



PROFILE OF HWY #648 LINE 'J'  
N.T.S.



NAVIGATION CLEARANCE DIAGRAM  
(AFTER CONSTRUCTION) - N.T.S.

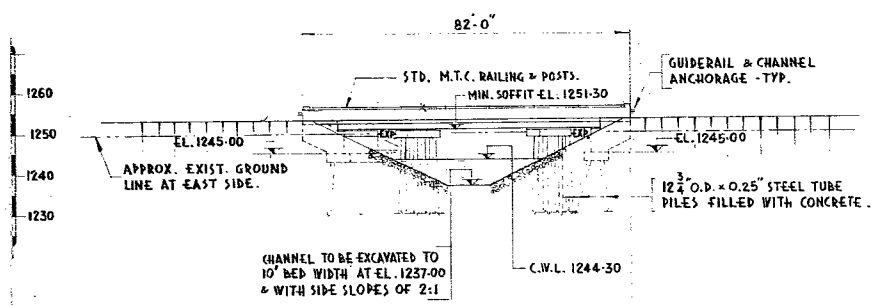
LIST OF DRAWINGS

- 40-52-1 - GENERAL PLAN  
2 - BORE HOLE LOCATION & SOIL STRATA.  
3 - FOOTING LAYOUT, PIERS & REINF.  
4 - ABUTMENTS  
5 - PRESTRESSED SLABS & BEARINGS  
6 - DECK  
7 - CONCRETE BARRIER WALLS (2'-8" HIGH)  
8 - STEEL PARAPET RAILING (SINGLE TUBE)  
9 - STANDARD DETAILS I  
10 - STANDARD DETAILS II  
40-52-11 - AS CONSTRUCTED ELEV. & DIM.

LIST OF CONCRETE QUANTITIES

CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE CONCRETE LUMP SUM TENDER ITEMS:

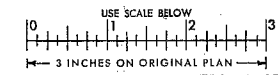
	CU. YDS.	P.S.I.
1- CONCRETE IN ABUTMENTS, WINGWALLS & PIERS		3000
2- CONCRETE IN DECK		
3- CONCRETE IN BARRIER WALLS		4000



ELEVATION  
SCALE: 1"=20'-0"



FOR REDUCED PLAN



REVISIONS	DATE	BY	DESCRIPTION
1			
2			
3			

DESIGN: AC CHECK: AC LOADING: H320-44 DATE: JULY 76  
DRAWING: G.C. CHECK: R.K. SITE No: 40-52 DWG: 1

