



FOUNDATION TECHNICAL MEMORANDUM

For

**MEDWAY CREEK BRANCH BRIDGE (ST. JOHN'S)
MTO WEST REGION 59 STRUCTURE REHABILITATIONS
HIGHWAY 4, SITE 19-162, CONTRACT 5
GWP 3062-11-00
GEOGRAPHIC TOWNSHIP OF LONDON
MIDDLESEX COUNTY, ONTARIO**

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August, 1960

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FOUNDATION TECHNICAL MEMORANDUM

For

Medway Creek Branch Bridge (St. John's)
MTO West Region 59 Structure Rehabilitations
Highway 4, Site 19-162, Contract 5, GWP 3062-11-00
Geographic Township of London
Middlesex County, Ontario

1. INTRODUCTION

The Foundation Engineering Services for the present project involves the detail foundation investigation and design for rehabilitation of 59 structures in MTO West Region along Highways 4, 6, 401, 402 and 403. Ten (10) Group Work Projects (GWP's) are contemplated to be completed between 2014 and 2020.

This technical memorandum summarizes the factual results of geotechnical data based on review and compilation of existing subsurface information from relevant reports in the MTO GEOCREST Library for the Medway Creek Branch Bridge (St. John's) on Highway 4. The Foundation Engineering recommendations from the existing bridge foundation reports are summarized with reference to the "Canadian Highway Bridge Design Code" (CHBDC) and follow in general the "Guidelines for Professional Engineers providing Geotechnical Engineering Services".

From the Minutes of Meeting Report, dated January 23, 2015, it is understood that the rehabilitation of the structure will be full superstructure replacement. The superstructure will be replaced in two stages while maintaining a single lane of traffic in each direction.

The purpose of the Technical Memorandum is to summarize the subsurface and groundwater conditions and foundation recommendations based on available reports at the bridge location for the design project team's reference.

The elevations in this report are expressed in meters, unless otherwise noted.



2. PROJECT SITE BACKGROUND AND GEOLOGY

The Medway Creek Branch Bridge (St. John's) on Highway 4 is located about 8.0 km (5 miles) north of London in the Geographic Township of London, Middlesex County, Ontario. A site key plan is shown in the Figure 1.

The existing structure is a three span reinforced concrete slab on steel girders that carries four lanes of traffic. The Arva Flour Mill is located just southeast of the structure. In addition, a dam is located about 100 m east of the bridge structure. A residential neighbourhood is located southeast of the site; agricultural lands are located to the north and west of the site location and a vegetated area is located northeast of the site location. The creek flows from east to west under the bridge.

Physiographically, the site is located in the region known as the Stratford Till Plain. The till in this area is fairly uniform, being a brown calcareous silty clay whether on the ridges or the more level ground moraine. It is a product of the Huron ice lobe. The topography is flat to undulating. The geology of the subsoil is mainly moraine till composed of silty clay and gravel of glacial origin. In the Geographic Township of London the drift thickness exceeds 46 m and in some areas 61 m and is underlain by bedrock of the Dundee Formation, a light brown, medium-grained limestone with some chert (Aggregate Resources Inventory of London Township, Middlesex County, Southern Ontario by Ontario Geological Survey, dated 1983).

3. SOURCE OF INFORMATION

The following foundation report and drawing, appended in Appendix A, were available for review and provided information for the bridge structure, subsoil information and original foundation recommendations.

1. Foundation Report on New Bridge at Highway No. 4 and Medway River Crossing some 5 miles north of London, W.J. F-57-5, W.P 519-56, Department of Highways Ontario, dated July 4, 1957. GEOCREs No. 40P03 -002. (Reference 1)
2. General Plan and Elevation Drawing, St. John's Bridge at Arva over Medway River, The King's Highway No. 4, Lot 16 and 17 Con. VII, W.P 519-56, TWP 93-162-1-C by Department of Highways Ontario - Bridge Office Toronto, dated August 1960. (Reference 2)



4. SITE RECONNAISSANCE

As part of the current foundation engineering assessment study, a site reconnaissance of the Medway Creek Branch Bridge (St. John's) was carried out on July 24, 2014, 2013. A photographic record of the site visit is attached in Appendix B.

Photographs 1, 6 and 7 show the bridge structure from various corners at the site location. The west and east slopes adjacent to north and south abutments were partially vegetated with strands of trees and bushes. However, evidence of erosion activity on the slope faces and toes was also observed (Photographs 2, 6, 9 and 12), particularly the east slope adjacent to the south abutment (Photograph 12). No obvious major cracks, except for minor surface cracks were observed on the north and south piers (Photographs 3 and 10). Further, erosion activity was also observed on the front slope faces at the abutments (Photographs 4, 5 and 8). It was observed that the rip-rap stones were present at the bottom of the slopes adjacent to of the piers (Photographs 5 and 11).

At the time of the site reconnaissance, the depth of the water in the creek varied between 0.1 to 0.6 m.

5. PREVIOUS FOUNDATION INVESTIGATION AND SUBSURFACE CONDITIONS

The bridge site is located about 8.0 km (5 miles) north of London on Highway 4 in the Geographic Township of London, Middlesex County, Ontario. The general subsurface conditions presented in this section are based on the Foundation Report, GEOCRE 40P03-002, dated July 4, 1957 (Reference 1).

The foundation report included the borehole location plan (Drawing No. F59-5A) and Record of Borehole sheets (1 to 4).

At the time of the foundation investigation (Reference 1), it was proposed to replace the original single span arch bridge with a new structure at the same location. The bridge was to be widened some 0.9 m (3 ft.) by addition to the older arch and the structure was to be supported on piers resulting a three span reinforced concrete bridge.



The foundation investigation comprised four boreholes which were drilled between February 26 and March 21, 1957. The boreholes were drilled to depths of 8.1 to 14.9 m (26.5 to 49.0 ft.), elevation 244.5 to 253.1 (802.3 to 830.5 ft.). Boreholes 1 to 3 were accompanied with dynamic cone penetration tests (DCPTs) that were conducted directly adjacent to the location of boreholes. The depth of DCPTs ranged from 4.3 to 5.5 m (14.0 to 18.0 ft.), elevation 253.8 to 255.3 (832.8 to 837.5 ft.), where refusal was met. The boreholes were drilled using a skid mounted core drill machine.

Topsoil

A 300 mm (1.0 ft.) thick surficial topsoil layer was encountered in all four boreholes and extended to elevation 259.0 to 260.9 (849.8 to 856.0).

Fill

A 0.6 to 0.9 m (2.0 to 3.0 ft.) thick layer of gravelly fill material was encountered below the surficial topsoil in all four boreholes at 0.3 m (1 ft.), elevation 259.0 to 260.9 (849.8 to 856.0 ft.) and extended to 0.9 to 1.2 m (3.0 to 4.0 ft.), elevation 258.3 to 260.0 (847.5 to 853.0 ft.).

Clay Till

A deep deposit of very stiff to hard gravelly clay till material was encountered below the fill material at 0.9 to 1.2 m (3.0 to 4.0 ft.), elevation 258.3 to 260.0 (847.5 to 853.0 ft.) and extended to borehole termination depths between 8.1 and 14.9 m (26.5 to 49.0 ft.), elevation 244.5 and 253.1 (802.3 to 830.5 ft.).

Laboratory unconfined compression test results were inaccurate due to the presence of gravel and pebbles. The unconfined compression results on clay samples without boulders showed an average shear strength of over 191.5 kPa (2 tsf). The Atterberg liquid limits ranged from 14.0 to 24.0 and plastic limits ranged from 9.0 to 13.0 for the gravelly clay samples. The plasticity index ranged from 4.0 to 12.0. Further, unit weight of the gravelly clay samples varied from 18.1 to 23.0 kN/m³ (115.4 to 146.4 pcf). The average unit weight was about 22.0 kN/m³ (140 pcf). Moisture content determinations ranged from approximately 7.0 to 37.0%.



Groundwater

At the top of the clay till some seepage water was encountered during the investigation; however, no groundwater was encountered and the layer was considered to be impervious.

6. FOUNDATION

6.1 Previous Foundation Discussions and Recommendations

The foundation report recommended that spread footings could be placed at about elevation 256.0 (840 ft.). An allowable bearing pressure of 240 kPa (2.5 tsf) was recommended for the design of the spread footings. It was recommended to place the footings deep enough in the till layer to protect the structure against scouring hazard. No stability problem was anticipated for the approach fill. The report advised to improve the approach grades to the structure by raising the elevation of the new replacement bridge.

Based on the General Plan and Elevation drawing (Reference 2), the proposed bridge would be constructed as a three span steel girder structure with spans of 15.2, 18.9, 15.2 m (50, 62, 50 ft.). The abutment foundations were to be supported by steel H piles type HP 310 x 79 (12 BP 53). The piers (walls) would be constructed on continuous pile foundations. All H-piles for the abutments and piers were to be driven to refusal in hard gravelly gray clay stratum to elevation 253.0 (830 ft.). The embankments were to be raised about 6.0 m (20 ft.) from the original grade level of the river bank. The piles were designed for 444 kN (50 tons) per pile.



6.2 Assessment of Foundation Parameters

Piles were to be driven to the typically very stiff to hard gravelly clay till and the toes of the piles were to be placed at approximate elevation 253.0 (830.0 ft.); the HP 310 x 79 piles are considered to be capable of providing the resistance values summarized in the table below.

Foundation Type	Founding Elevation (ft.)	Founding Elevation (m)	Vertical Load per Pile (ton)	Geotechnical Axial Reaction at SLS (kN)	Factored Geotechnical Axial Resistance at ULS (kN)
H-Pile (12BP@53 / HP 310 x 79)	830.0	253.0	50	650	900

The seismic site coefficient for the conditions at this site is 1.0 (soil profile Type 1, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6).

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The foundation frost penetration depth at the site is 1.2 m according to OPSD 3090.101.

7. DISCUSSION

It is understood that the rehabilitation of the structure will be full superstructure replacement. Construction will be completed in two stages while maintaining a single lane of traffic in each direction.

From a geotechnical point of view, at the present time, foundation work for the Medway Creek Branch Bridge (St. John's) structure is not expected provided that the total dead load on the bridge does not increase or decrease by more than 10%.

The slope faces in front of the abutments and adjacent slopes should be rehabilitated with rock protection, rip-rap or equivalent materials to mitigate erosion and scouring effects. The aggregate materials should conform to OPSS.PROV 1004 and the construction of the rock protection, rip-rap or equivalent should conform to OPSS 511.



8. CLOSURE

This technical memorandum was prepared by Mr. Nazibur Rahman, P.Eng with the assistance of Mr. Mansoor Khorsand, EIT and was reviewed by Mr. Robert Ng, PhD, P.Eng., Senior Project Engineer. Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Principal Contact conducted an independent review of the report.

We trust this memo is sufficient for your immediate needs. Please, do not hesitate to contact us if you have any inquiries and/or comments. Yours very truly,

Yours truly,

Peto MacCallum Ltd.



Nazibur Rahman, P.Eng.
Project Engineer, Geotechnical Services



Robert Ng, MBA, PhD, P.Eng.
Senior Project Engineer



Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

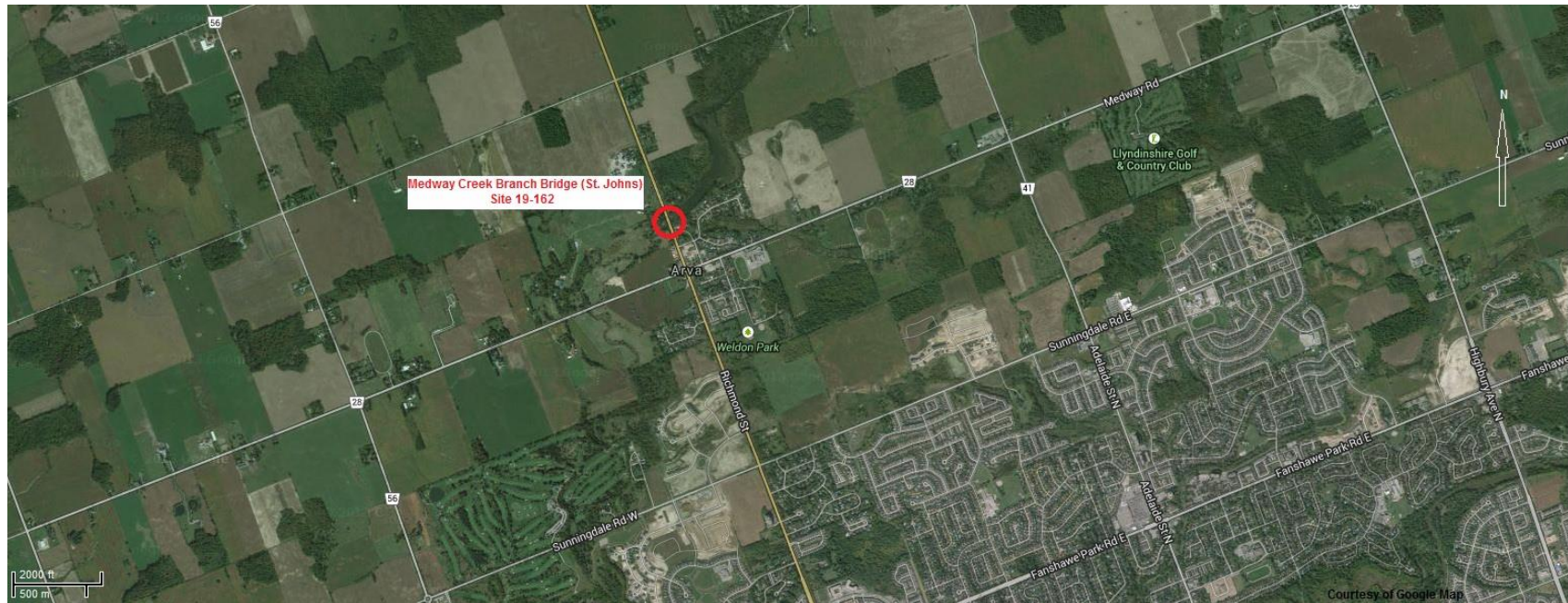


TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSD 3090.101	Foundation Frost Depth for Southern Ontario

Figure 1 – Key Plan





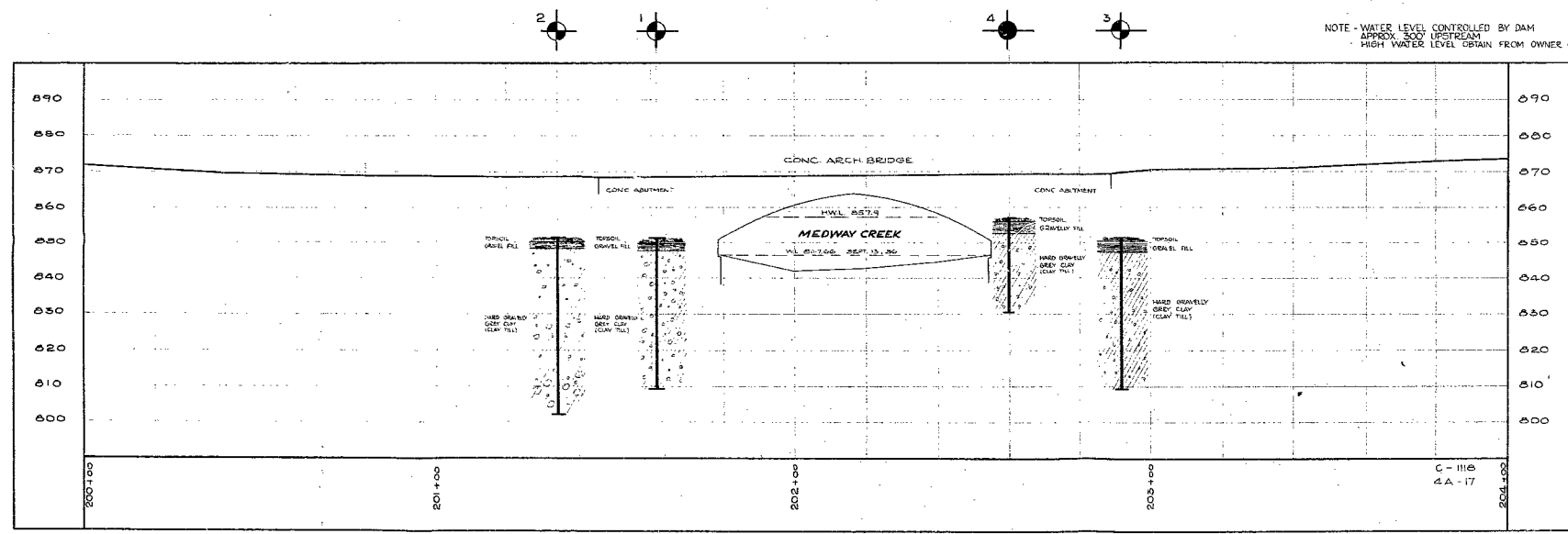
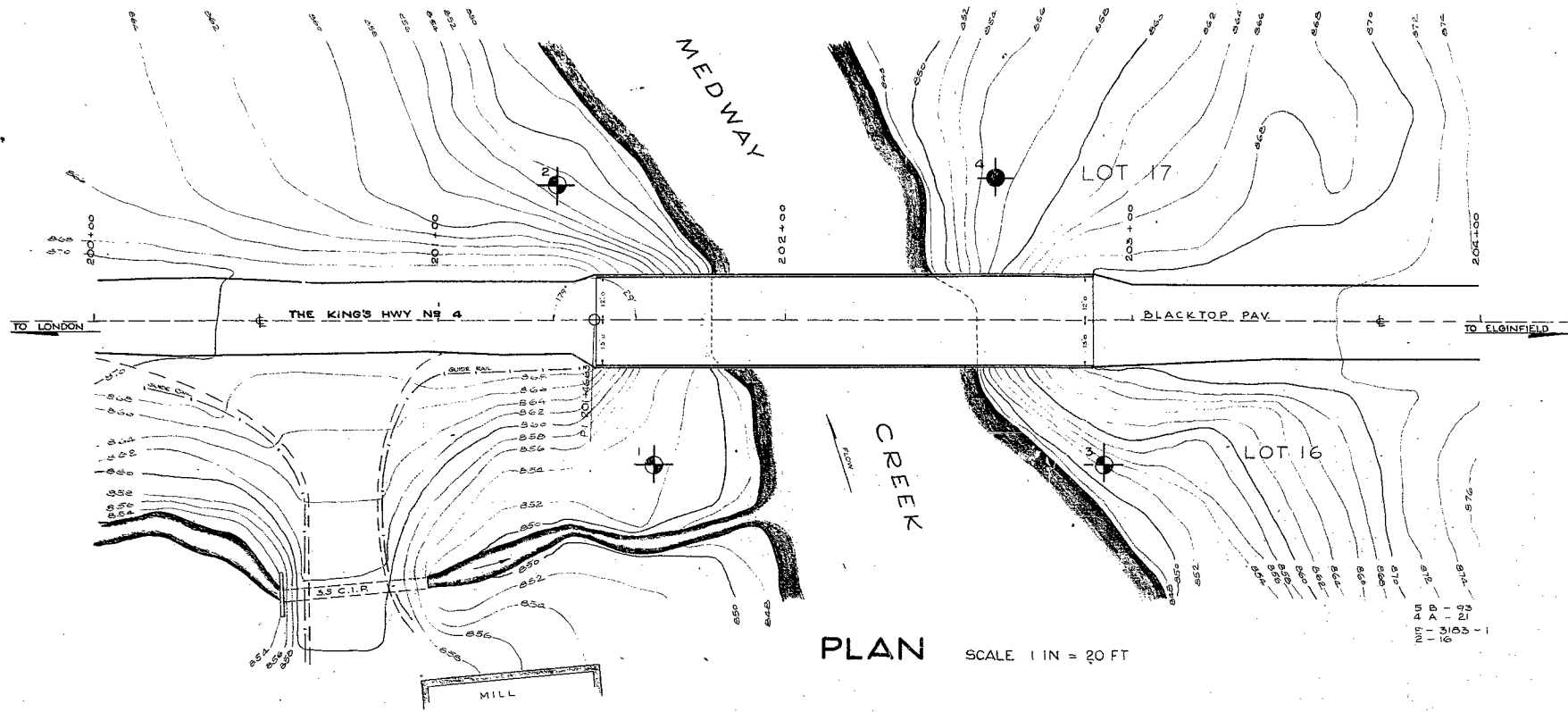
APPENDIX A

Foundation Investigation Report (GEOCRES 40P03-002)

General Plan and Elevation Drawing – St. John's Bridge at Arva, dated August, 1960

57-F-5
W.P.#519-56
Hwy.# 4
MEDWAY RIVER
CROSSING
LONDON





LEGEND			
BORE HOLES			●
PENETRATION HOLE			⊕
BORE & PENETRATION HOLE			⊗
HOLE NO.	ELEVATION	STATION	DISTANCE FROM E
1	650.8'	201+62'	41.5' RT
2	651.3'	201+34'	36.5' LT
3	651.58'	202+92'	41.3' RT
4	657.0'	202+60'	40.5' LT

NOTE
THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BORE HOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR.

DEPARTMENT OF HIGHWAYS, ONTARIO
MATERIALS & RESEARCH SECTION - DOWNSVIEW

**MEDWAY CREEK
PROPOSED CROSSING
3.5 MILES NORTH OF LONDON**

THE KING'S HIGHWAY No. 4 DIV. No. 2
CO. MIDDLESEX
TWP. LONDON LOT 16 & 17 CON. VII

POSITION & ELEVATION OF HOLES

APPROVED

ENGINEER CHIEF ENGINEER

DESIGN CHECK CONTRACT NO. W.P. 519-56
DRAWING D.F. CHECK DRAWING NO. F-57-5A
TRACING CHECK
DATE MAY 2, 1957

cc: Foundation Section

Mr. A. Toye,
Bridge Engineer.

F. C. Brownridge

per: A. Rutka

July 4, 1957.

Re: Foundation Report -
Medway River at Arva
Hwy. #4 - W.P. 519-
W.J. F-57-

Attached herewith are two copies of the above mentioned foundation report. Spread footing foundations appear to be satisfactory for this structure; however, it also appears that some protection against scouring will also be required.


This structure was indicated on the 1957-1958 construction program; however, it has now been deleted and does not appear on any recent list for re-construction.

AR/ndef
Attach.

cc: Messrs. A. Toye
H. Tregaskos
D.G. Ramsay
W. L. Fraser

F. C. Brownridge,
MATERIALS & RESEARCH ENGR.

Per:


A. Rutka,
PRINCIPAL CIVILS ENGINEER.

Foundation Section
File

FOUNDATION REPORT

on

New Bridge at Highway No. 4 and Medway River crossing
some 5 miles north of London.

Site Plan No. E-3183-1

Station: 202+20

Distribution:

Mr. A. Toye
Bridge Engineer (2)

Mr. H. Tregaskes
Construction Engineer (1)

Mr. D.C. Ramsay
Design Engineer (1)

Mr. W.L. Fraser
Dist. Eng. London, Ont. (1)

Foundation Section (1)

FILE (1)

W. J. F-57-5

W. P. 519-56

INTRODUCTION

A subsoil investigation was carried out to determine the bearing values of the layers to support the foundations of the proposed structure.

The location is some five miles north of London where Highway No. 4 crosses the Madway River (Station 202+20, profile No. C-1118). The proposed new structure is meant to replace the existing one at the same location. The existing bridge is a single span arch which has been widened some three feet by addition to the older arch. The structure is supported on piers. Some hundred yards to the east the waters of the creek have been dammed. The water head is being used by an operating flour mill located on the southeast side of the bridge. In between the mill and the highway there is a small stream flowing into the creek not very far from the south abutment of the bridge. It was observed that the eastern corner of the south pier has been exposed and the water depth is quite considerable at this point. The existing situation at this point is due to scouring.

The investigation work started on January 22, 1957 and was completed on February 23, 1957.

PROCEDURE

The subsoil investigation was carried out by means of a skid mounted core drill machine.

Four boreholes were made two on each side of the existing bridge. The location of the boreholes is shown on drawing F-57-5A and the elevations on log sheets under Appendix I.

SUBSOIL FINDINGS AND ANALYSIS

The terrain is till plain. The flow of the river is controlled by the existing dam some hundred yards to the east of the bridge. Although during the times of flood the high water level is recorded approx. 10' higher than the normal level.

The subsil investigations revealed the following stratigraphy:

Under the topsoil some fill material was encountered which reached down to elevation 847.5 ft. Below this the soil is hard gravelly clay till. The boreholes were carried down some 40 ft. below the surface ground and stopped. At the top of the layer some seepage water was encountered but no underground water exists and the layer is considered to be impervious.

Some undisturbed samples were extracted and the laboratory tests performed. From the test results the average liquid limit is about 20% and plastic limit about 10%. The average natural moisture content is about 9% and the average density of the layer about 140 p. c. f. The soil is identified as inorganic clay of low plasticity.

The unconfined compression results on clay samples without boulders, etc. showed an average shear strength of over 2 T.s.f. The average standard penetration within the top twenty foot depth is about 80 blows per foot. From these results the soil can provide a conservative bearing value of 2.5 T.s.f. with a safety factor of 3.

CONCLUSIONS AND RECOMMENDATIONS

From the above discussion it will follow that:

1. The soil at the site has all the properties of bouldery clay till.
2. From the field and laboratory test results the layer can be credited with a bearing value of 2.5 T.s.f. to support spread footing foundations.
3. It will be convenient to place spread footing foundations at elevation about 840 feet.
4. Unless the footings are placed deep enough in the till layer some scouring hazards can not be eliminated.
5. The approach fills to the structure do not present any problem.
6. It would appear advisable to improve the approach grades to the structure

CONCLUSIONS AND RECOMMENDATIONS (continued)

by raising the elevation of the new replacing bridge. This raise will not affect the above conclusions about the new foundations.

V. Korlu
Foundation Engineer

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-1 OPERATION BORE & PENET'N JOB F-57-5 WP 519-56 BORING 1 STA. 201+62 (415' RT.)
CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT APRIL 1957
SAMPLER HAMMER WT. 250 LBS. DROP 20 1/4 INCHES COMPILED BY H.J. CHECKED BY A.L. DATE BORING 26 FEB 1957

ABBREVIATIONS

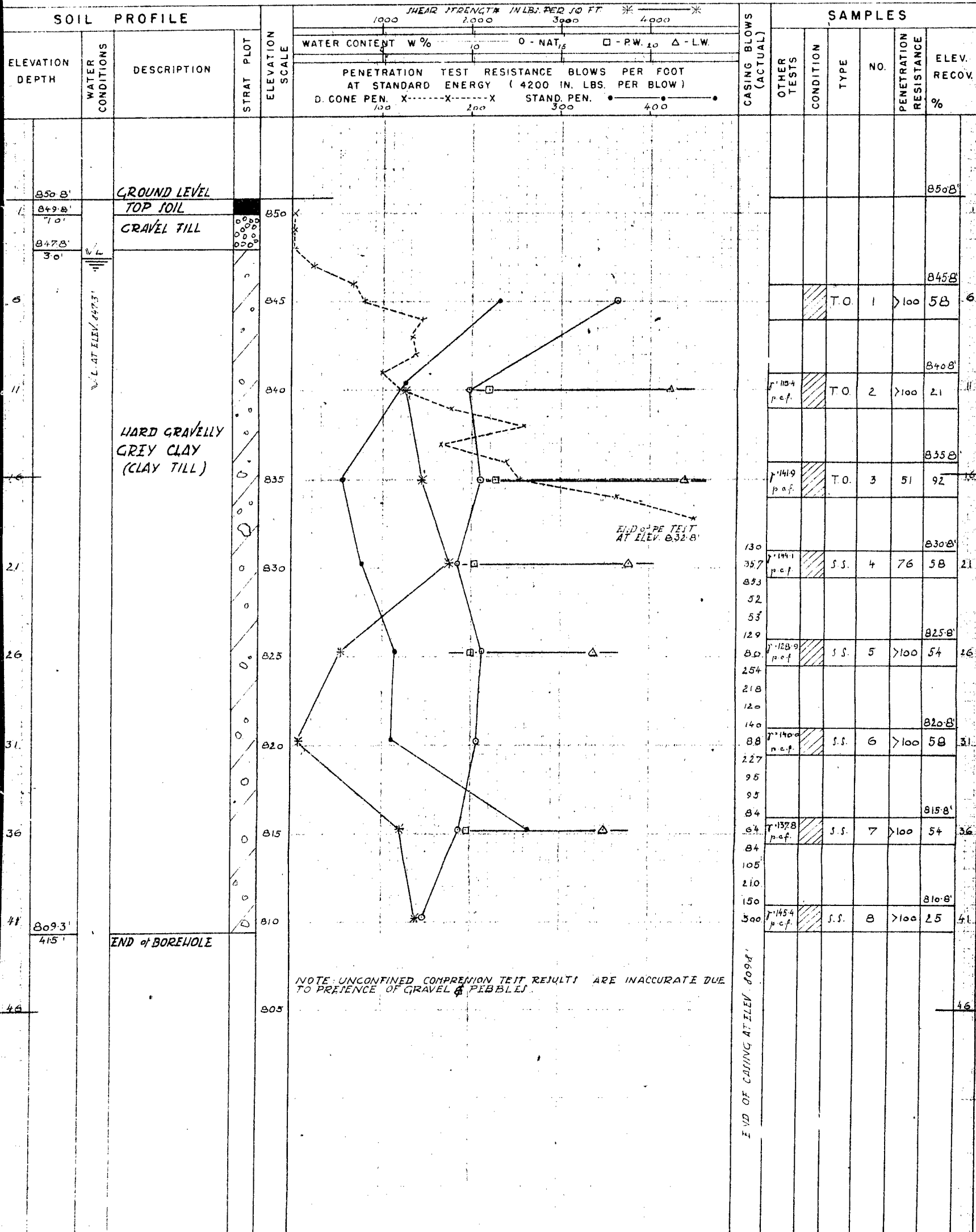
V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY
M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION
U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING
QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

SAMPLE TYPES

C.S. - CHUNK S.S. - SLEEVE SAMPLE
D.O. - DRIVE OPEN PS - PISTON SAMPLE
D.F. - DRIVE FOOT VALVE WS - WASHED SAMPLE
T.O. - THIN WALLED OPEN RC - ROCK CORE

SAMPLE CONDITION

 - DISTURBED
 - FAIR
 - GOOD
 - LOST



DRILL RIG 54-1 OPERATION BORI & PENIT'N JOB F-57-5 W.P. 519-56 BORING 2 STA. 201+34(38.5' LT)
CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT APRIL 1957
SAMPLER HAMMER WT. 250 LBS. DROP 20 1/2 INCHES COMPILED BY H.J. CHECKED BY A.L. DATE BORING 6 MARCH 1957

SAMPLE TYPES

SAMPLE CONDITION



- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLES

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-1 OPERATION BORE & PENET'N JOB F-57-5 WP. 519-56 BORING 3 STA. 202+92(413' 21")
CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT APRIL 1957
SAMPLER HAMMER WT. 250 LBS. DROP 20 1/4 INCHES COMPILED BY H.J. CHECKED BY AL DATE BORING 14 MARCH 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY
M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION
U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING
QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

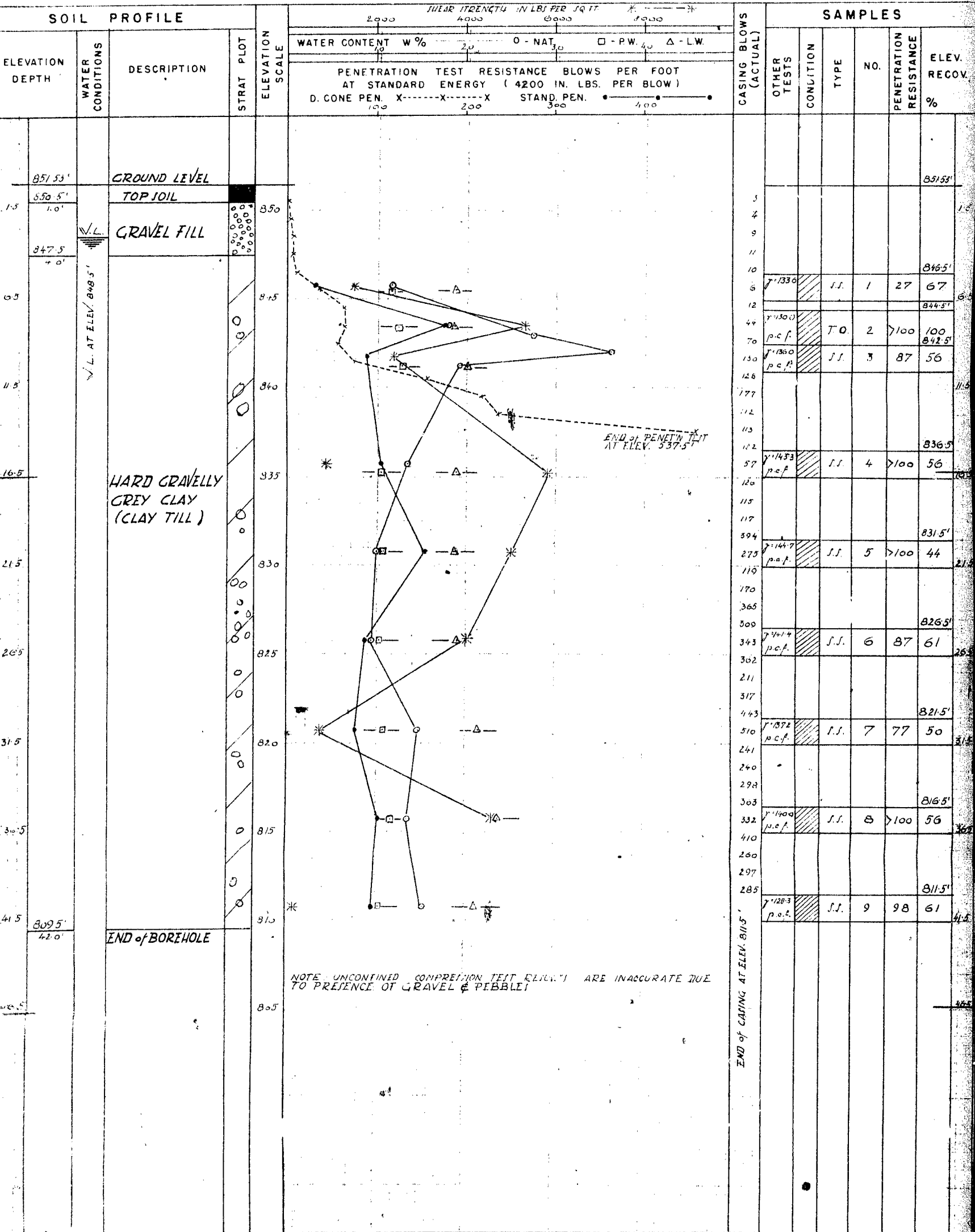
SAMPLE TYPES

SS - SLEEVE SAMPLE
PS - PISTON SAMPLE
WS - WASHED SAMPLE
RC - ROCK CORE
CS - CHUNK
DO - DRIVE OPEN
DF - DRIVE FOOT VALVE
TO - THIN WALLED OPEN

SAMPLE CONDITION



- DISTURBED
- FAIR
- GOOD
- LOST



DRILL RIG 54-1 OPERATION BORI JOB F-57-5 W.P. 5/9-56 BORING 4 STA. 202+60(40.5' L)
CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT APRIL 1957
SAMPLER HAMMER WT. 250 LBS. DROP 20 1/2 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 21 MARCH 1957

SAMPLE TYPES

- DISTURBED
- FAIR
- GOOD
- LOST

V - INSITU VANE SHEAR TEST	Q - TRIAXIAL QUICK	K - PERMIABILITY
M - MECHANICAL ANALYSIS	S - TRIAXIAL SLOW	C - CONSOLIDATION
U - UNCONFINED COMPRESSION	WL - WATER LEVEL IN CASING	CA - CASING
QC - TRIAXIAL CONSOLIDATED QUICK	WT - WATER TABLE IN SOIL	γ - UNIT WEIGHT

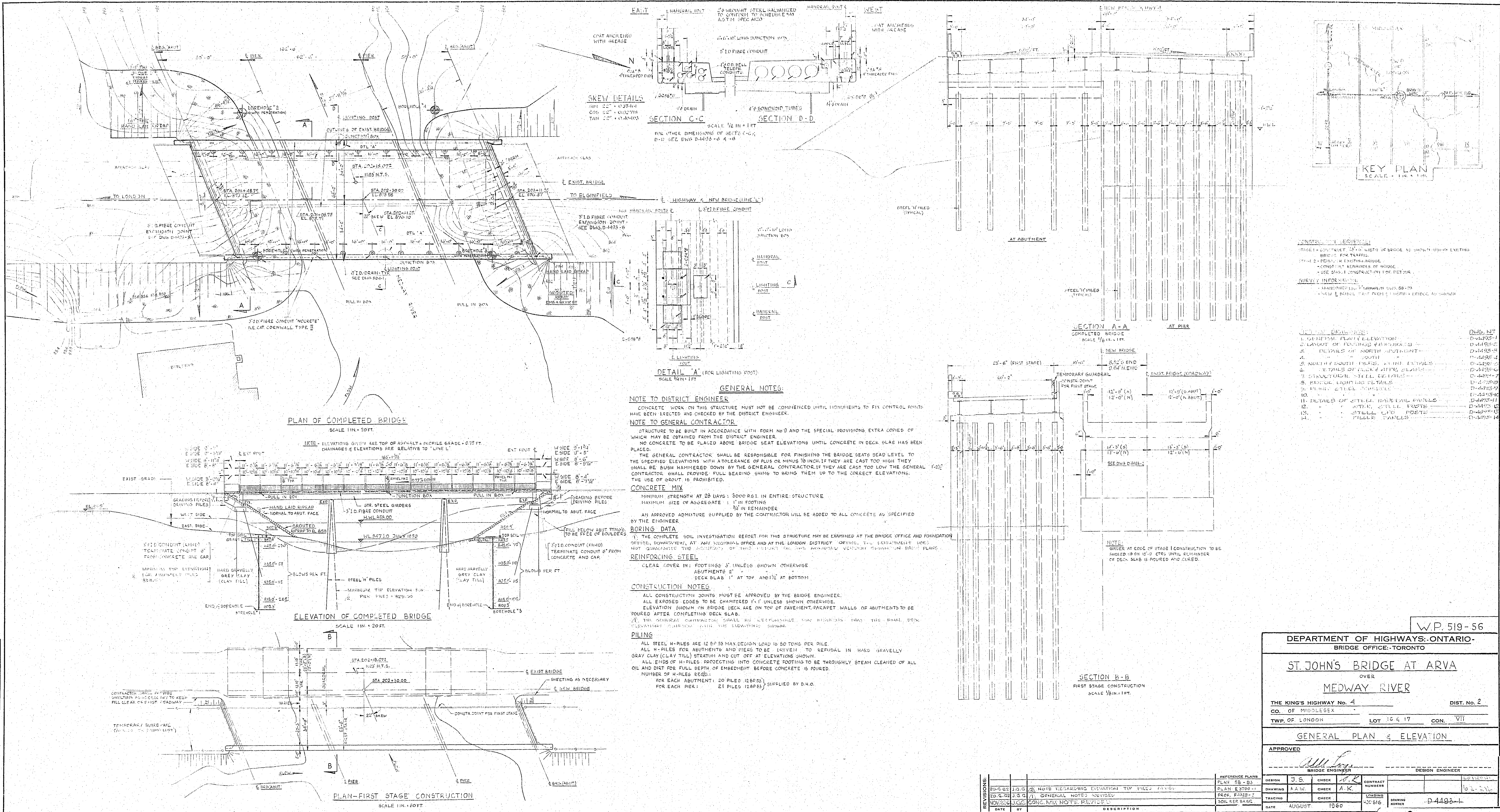
C.S. - CHUNK	S.S. - SLEEVE SAMPLE
D.O. - DRIVE OPEN	P.S. - PISTON SAMPLE
D.F. - DRIVE FOOT VALVE	W.S. - WASHED SAMPLE
T.O. - THIN WALL OPEN	R.C. - ROCK CORE

WEAR STRENGTH IN LBS. PER SQ. FT.

SAMPLES

SOIL PROFILE																							
ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W %			O - NAT			□ - P.W.			△ - L.W.			CASING BLOW (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV.
					PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW)			D. CONE PEN. X-----X-----X			STAND. PEN. ●-----●-----●												
857.0'		GROUND LEVEL																					857.0'
856.0'		TOP SOIL																					
853.0'		GRAVELLY FILL		855																			
850.0'				850																			852.0'
847.0'				845																			847.0'
840.0'		HARD GRAVELLY GREY CLAY (CLAY TILL)		840																			842.0'
837.0'				835																			837.0'
830.5'		END of BOREHOLE		830																			832.0'
26.5'																							

NOTE: UNCONFINED COMPRESSION TEST RESULTS ARE INACCURATE
DUE TO PRESENCE OF GRAVEL & PEBBLES.





APPENDIX B

Site Photographs



Photograph 1: Looking west at the bridge structure from the southeast corner. Slight erosion was observed on the slope face and the exposed earth adjacent to the south pier. (July 24, 2014)



Photograph 2: Looking north from the east side of the bridge. The northeast slope was covered with bushes. Slight erosion of the slope face and scouring of the slope toe adjacent to the abutment was observed. (July 24, 2014)



Photograph 3: Looking north at the north pier wall from the southeast corner. No obvious cracks were observed on the concrete pier. Scouring effect below the water could not be established during the site reconnaissance. (July 24, 2014)



Photograph 4: Looking south at the slope face in front of the south abutment. Effects of erosion were observed on the slope face. (July 24, 2014)



Photograph 5: Looking down at the slope face in front of the south abutment. Effects of erosion were observed on the slope face and the exposed earth in front of the south pier. There were no obvious cracks on the pier wall. (July 24, 2014)



Photograph 6: Looking east at the bridge structure from the southwest corner. Erosion of exposed earth was evident on the slope face and adjacent to the south pier. (July 24, 2014)



Photograph 7: Looking east at the bridge structure from the northwest corner. Erosion of exposed earth was evident on the slope face and adjacent to the north pier. (July 24, 2014)



Photograph 8: Looking down at the slope face in front of the north abutment. Effects of erosion were observed on the slope face and the exposed earth in front of the north pier. The rip-rap stones gathered at the bottom of the slope probably due to erosion activity of the slope face. There were no obvious cracks on the pier wall. (July 24, 2014)



Photograph 9: Looking south from the west side of the bridge. The west slope adjacent to the south abutment was covered with bushes. Erosion of the slope face and scouring of the slope toe adjacent to the south abutment was observed. (July 24, 2014)



Photograph 10: Looking south at the south pier wall from the northwest corner. No obvious cracks were observed on the concrete pier. Scouring effect below the water could not be established during the site reconnaissance. (July 24, 2014)



Photograph 11: Looking west at the north pier structure from the northeast corner. Rip-rap stones gathered at the bottom of the slope in front of the pier. No obvious cracks were observed on the pier. (July 24, 2014)



Photograph 12: Looking south from the east side of the bridge. The east slope adjacent to the south abutment was greatly affected by erosion activity. (July 24, 2014)