Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION TECHNICAL MEMORANDUM

For

RALEIGH PLAINS DRAIN BRIDGE EBL ON HIGHWAY 401
MTO WEST REGION 59 STRUCTURE REHABILITATIONS
SITE 13-228-1, CONTRACT 7
GWP 3084-11-00
GEOGRAPHIC TOWNSHIP OF RALEIGH
KENT COUNTY, ONTARIO

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TABLE OF CONTENTS

FOUNDATION TECHNICAL MEMORANDUM
For
Raleigh Plains Drain Bridge EBL on Highway 401
MTO West Region 59 Structure Rehabilitations
Site 13-228-1, Contract 7, GWP 3084-11-00
Geographic Township of Raleigh, Ontario

1. INTRODUCTION

The Foundation Engineering Services for the present project involve the detail foundation investigation and design for the 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 the review and compilation of existing subsurface information from relevant reports in the MTO GEOCRES Library for the Raleigh Plains Drain Bridge EBL (Jeannette Creek Crossing). 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 July 24, 2014, it is understood that rehabilitation of the overpass structure is anticipated and that the rehabilitation will be completed in a single stage construction using median crossovers.

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 Raleigh Plains Drain Bridge EBL on Highway 401 is located about 10.0 km south of Chatham in the Township of Raleigh, Kent County, Ontario. A key plan is shown in Figure 1.

The existing structure is a single span reinforced concrete rigid frame structure that carries two lanes of Highway 401 Eastbound traffic. The general layout drawing titled Raleigh Township Bridge No. 7, Jeanette Creek Crossing - General Plan, Elevation & Sections, The King's Highway No. 401, District No. 1, Lots 15 &16. Con. VII, W. P. 13-59, Department of Highways Ontario, October 1959, GEOCRES No. 40J08-008, is enclosed as appendix A.

Physiographically, the site is located in the St. Clair Clay Plain, which consists of a flat and relatively deep deposit of typically very stiff clayey silt and silty clay till deposits. The bedrock underlying the Highway 401 alignment throughout the Geographical Township of Raleigh comprises mostly of the black bituminous shale containing locally grey shale of the Kettle Point Formation.

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 Hwy 401, Line "A" and Jeannette Creek Crossing, Lots 15 & 16, Con. VII, Township of Raleigh – Approx. 6.5 miles south of Chatham, W.P. 3074-11-09, W.J. F-59-14, Ontario, Materials and Research Section, Ministry of Transportation and Communications Ontario, May 11, 1959. GEOCRES NO. 40J08-008.

2. Raleigh Township Bridge No. 7, Jeanette Creek Crossing - General Plan, Elevation & Sections, The King's Highway No. 401, County Kent, Township of Raleigh, District No. 1, Lots 15 &16. Con. VII, W.P. 13-59, T.W.P 103-228-1-A, Department of Highways Ontario, October 1959.

4. SITE RECONNAISSANCE

As part of the current foundation engineering assessment study, a site reconnaissance of the Raleigh Plains Drain was carried out on October 20, 2013. A photographic record of the site visit is attached in Appendix B.

The adjacent slopes of the abutments were observed to be vegetated (Photographs 1 to 3). No erosion of the slope faces was observed. Further, scouring of the adjacent slope toes was also not observed at this bridge structure location. The front earth slopes of the east and west abutment walls (Photographs 4 and 5), were partially covered with rip-rap and the exposed earth was observed to be affected by scouring, most likely due to numerous fluctuating cycles of the creek water level over time. No obvious major cracks were observed on the abutment except some map cracks. Weep holes were observed on the abutment walls.

Concrete deteriorations and rebar exposure was observed on the wingwalls, deck and barriers, which will require rehabilitation.

At the time of the site reconnaissance, the water level of the creek was about 0.8 m deep and the direction of the water flow was towards the north.

5. PREVIOUS FOUNDATION INVESTIGATION AND SUBSURFACE CONDITIONS

A foundation investigation report (GEOCRES 40J08-008) was prepared by the Materials and Research Section, Department of Highways Ontario dated May 11, 1959, enclosed in Appendix A. The purpose of the previous investigation was to conduct a foundation investigation at the proposed bridge structure over the Raleigh Plains Drain (Jeanette Creek) at the site location.

The foundation investigation report includes the borehole location plan (Drawing No. F59-14A), Record of Borehole sheets (1 to 3) and summary of the Field and Laboratory tests.

The field investigation included three boreholes which were drilled between February 14 and 16, 1959. The boreholes were drilled to depths of 9.9 to 12.8 m below ground surface. One dynamic cone penetration test (DCPT) was conducted directly adjacent to the location of borehole 3. The DCPT met refusal at 5.1 m, elevation 173.7. The three boreholes were drilled using a trailer-mounted coredrill machine adapted for soil sampling. Conventional auger boring procedures were followed and samples were recovered at depths required. Thin wall Shelby tube samplers, 50 mm (2 in.) inner diameter (I.D.), were used in cohesive soils to obtain soil samples

Samples were visually examined and identified in the laboratory and routine index tests were performed on the samples.

Generally, topsoil over stiff to hard silty clay was encountered at the site location. Bedrock was not encountered in any of the three boreholes.

Frost and Topsoil

A layer of 0.3 to 0.6 m frost and topsoil was encountered surficially in the three boreholes and extended to elevation 178.2 to 178.5.

Sandy Clay

A 0.9 m local deposit of hard brown desiccated sandy clay was encountered below the topsoil in borehole 3 at 0.3 m, elevation 178.5, and extended to 1.2 m, elevation 177.6. The deposit was found to be oxidized. The laboratory shear strength obtained for the sandy clay sample was 358.6 kPa (7490 psf). The unit weight of a sample was 20.3 kN/m^3 (129.0 pcf). A moisture content determination obtained was 16.8%.

Silty Clay

Below the topsoil layer a stiff to hard brown to grey silty clay deposit was encountered in boreholes 1 and 2 at 0.6 m, elevation 178.5 and 178.2, respectively and below the sandy clay deposit in borehole 3 at 1.2 m, elevation 177.6. The silty clay extended to the borehole termination depths 9.9 to 12.8 m, elevation 166.0 to 168.9.

Generally, the clay samples contained 46% clay, 25% silt, 20% sand and 9% fine to medium gravel sized particle throughout the stratum. The average unit weight and moisture content were 20.7 kN/m^3 (132 pcf) and 18%, respectively. Laboratory average shear strength of 95.8 kPa (2000 psf) was determined to be representative for the silty clay stratum. Based on the moisture content and Atterberg limits, it appeared that the silty clay was saturated and preconsolidated.

The upper 3.0 m of the silty clay stratum had been oxidized giving its brownish color, below which the color was predominately grey and was generally hard to very stiff in consistency which appeared to be the result of desiccation. The laboratory shear strengths of the upper 3.0 m silty clay ranged between 159.0 and 360.1 kPa (3320 and 7920 psf) with a local shear strength of 66.3 kPa (1385 psf) in borehole 2. The unit weight of the samples ranged from 19.0 to 21.0 kN/m^3 (121 to 133.8 pcf). The Atterberg liquid and plastic limits of a sample were 31.0 and 18.3, with plasticity index of 12.7. Moisture content determinations ranged between 15.6 and 31.6%.

The silty clay encountered below 3.0 m exhibited very stiff to stiff consistency. Laboratory shear strengths obtained for the silty clay samples ranged from 112.6 to 73.3 kPa (2560 to 1530 psf). The Atterberg liquid limit ranged from 24.0 to 28.4 and the plastic limit ranged from 15.2 to 17.5 for the silty clay samples. The plasticity index ranged from 7.9 to 12.8. Further, unit weight of the silty clay samples varied from 19.5 to 21.5 kN/m^3 (124.0 to 136.8 pcf). Moisture content determinations ranged from 15.2 to 20.0%. Based on the moisture content determinations and Atterberg limits, the silty clay appeared to be saturated and preconsolidated.

Groundwater

Groundwater was not established in the three boreholes during the investigation because of low permeability of the silty clay deposit. The level of groundwater table was assumed at the seasonal water level of the Raleigh Plains Drain (Jeannette Creek) at approximate elevation 176.8 to 178.3 (580 to 585 ft.). No water-bearing sand seams or artesian water conditions were encountered during the investigation.

6. FOUNDATION

6.1 Previous Foundation Recommendations

The foundation report recommended that spread footings be placed at about elevation 175.3 (575 ft.) or lower; however, in order to avoid possible undermining of the footing through erosion and scouring actions and for future deepening of the drain channel, it was recommended that the footings be founded at approximate elevation 172.8 (567 ft.), about 2.5 m (8 ft.) below assumed stream bed elevation 175.3 (575 ft.). The report recommended that at elevation 175.3 (575 ft.) or below for a footing width of 2.1 to 3.0 m (7 to 10 ft.), an allowable bearing pressure of 240 kPa (2.5 tsf) incorporating a safety factor of 3, could be used for the design of the spread footing.

The long term settlements under the footings, due to an abutment pressure of 240 kPa (2.5 tsf) and embankment load due to the weight of the 2.1 to 3.0 m (7 to 10 ft.) fill, were estimated not to exceed 127 mm (5 in.). Further, it was contemplated that if a single span structure was preferred, then the differential settlement would be little in view of the relatively uniform subsoil conditions at the site and that each abutment would anticipate to settle the same amount. If a multi-span structure was preferred with centre piers, differential settlement would arise between the center piers and the abutments because the consolidation under the footings of the piers would be unaffected by the approach fill adjacent to the abutments. Hence, it was recommended that for a contemplated multi-span structure a rigid-frame design would only be preferred if a differential settlement of 50 to 63 mm (2 to 2.5 in.) could be tolerated.

Based on the investigation, no excessive seepage problems were anticipated during the excavation operation with respect to the footings.

The maximum fill height estimated was approximately 2.1 to 3.0 m (7 to 10 ft.) under the original proposed grade line and that it was indicated that the subsoil would have sufficient strength to safely support the embankment. Further, it was recommended to place rip-rap on the bank slopes of the upstream side of the bridge structure.

Based on the drawing titled Raleigh Township Bridge No. 7 - General Plan, Elevation & Sections, dated October 1959, the footings were to be founded at about elevation 174.7 (573 ft.). Further, it was indicated that steel sheet piles were to be driven to approximate elevation 171.0± (561± ft.). However, during the site reconnaissance, the presence of sheet piles could not been verified visually. At the abutment locations, the original ground elevation was to be increased by 2.1 m (7 ft.) to achieve an approximate elevation 181.0 (593.9 ft.). In addition, it was shown in the drawing that rip-rap was to be placed on the proposed 2H:1V slopes and continue 0.5 m below the lowest level of water. However, rip-rap was not observed on the adjacent slopes of the bridge structure during site reconnaissance.

6.2 Assessment of Foundation Parameters

Based on the previous investigation and subsurface conditions encountered, the following table summarizes the foundation design parameters that were recommended in the previous report and the updated geotechnical reaction at SLS and factored geotechnical resistance at ULS are provided.

FOUNDATION DESIGN PARAMETERS

Foundation Type	Elevation of Footings (m)	Previous Safe Bearing Resistance (tsf)[1]	Previous Equivalent Limit State Design Values		Limit State Design Values Updated to current industry practices[2]	
			SLS Geotechnical Reaction (kPa)	ULS Geotechnical Resistance Factored (kPa)	SLS Geotechnical Reaction (kPa)	ULS Geotechnical Resistance Factored (kPa)
East Abutment on Spread Footing	174.6 (573 ft.)	2.5	240	360	350	525
West Abutment on Spread Footing						

Notes:
1. Working stress design values. The Ultimate Limit State design values are based on the working stress. No field verifications were made.
2. Resistance Factor = 0.5 for shallow foundation (CFEM 4th edition)
Assumed Factor of Safety is 3 (CFEM 4th edition)

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

From a geotechnical point of view, at the present time, foundation work for the Raleigh Plains Drain Bridge EBL structure is not expected provided that the dead load on the overpass does not increase or decrease by more than 10%.

It is understood that rehabilitation of the bridge structure is anticipated and that rehabilitation will be completed in a single stage construction using median crossovers.

Further, it is suggested that the weep holes in the abutment walls should be maintained and cleaned at a regular basis to prevent any clogging of the holes. Regular maintenance of the weep holes will keep the water flowing from behind the abutment walls and will mitigate hydrostatic pressure to build-up behind the abutment walls.

8. CLOSURE

This Technical Memorandum was prepared by Mr. N. Rahman, P.Eng with the assistance of Mr. M. Khorsand, EIT and was reviewed by Mr. R. Ng, PhD, P.Eng. Mr. B. 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,

Peto MacCallum Ltd.

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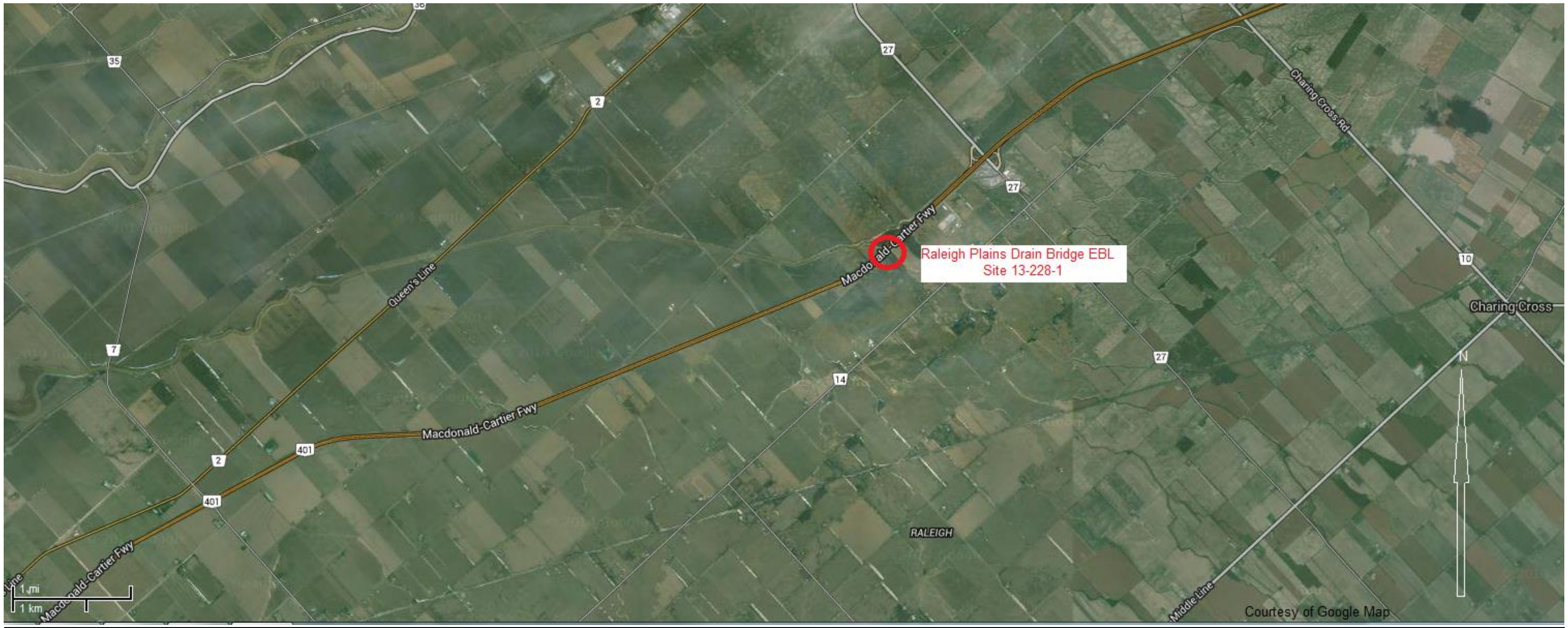
NR/RN/BRG:nr-mi-jk

TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSD 3090.101	Foundation Frost Depth for Southern Ontario

Figure 1 – Key Plan

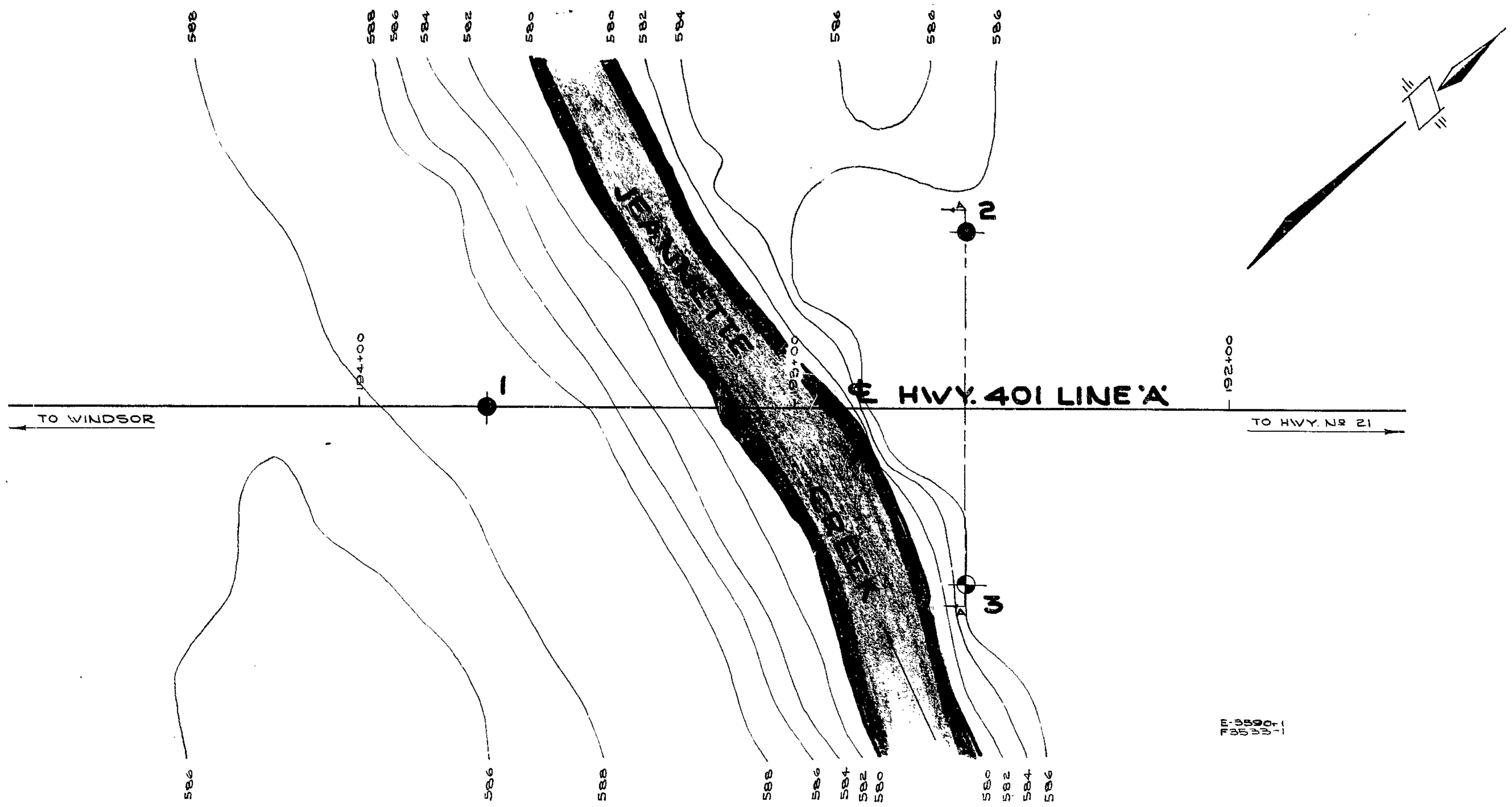


APPENDIX A

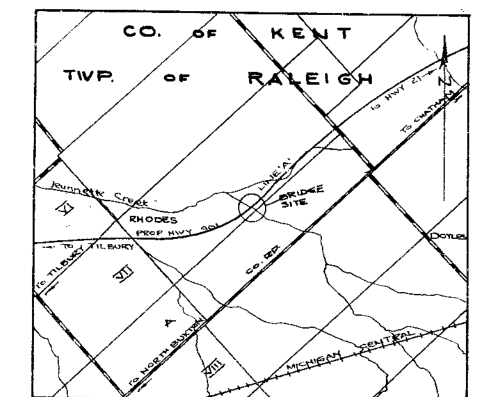
Foundation Report at Raleigh Plains Drain Bridge (GEOCRES 40J08-008)

Raleigh Township Bridge No. 7 - General Plan, Elevations & Sections, dated October 1959

59-F-14
W.P. # 13-59
Hwy. # 401
CROSSING
JEANNETTE CR.
6½ MILES S. OF
CHATHAM



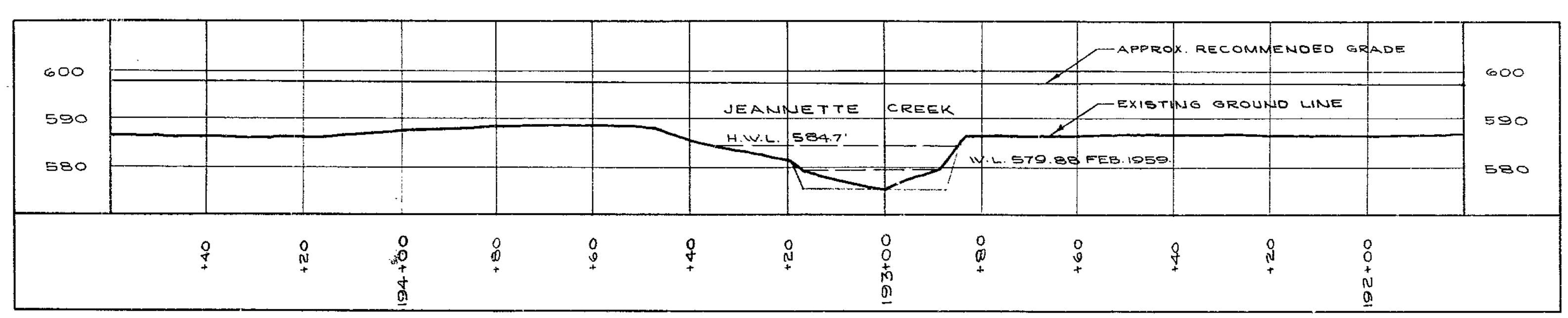
PLAN



KEY PLAN
SCALE 1 IN.= 1 MI.

LEDGEND	
BORE HOLE	
PENETRATION HOLE	
BORE & PENETRATION HOLE	

HOLE NO.	ELEVATION	STATION	DISTANCE
1	587.70	193+70	℄
2	586.70	192+60	40' RT.
3	586.70	192+60	40' LT.

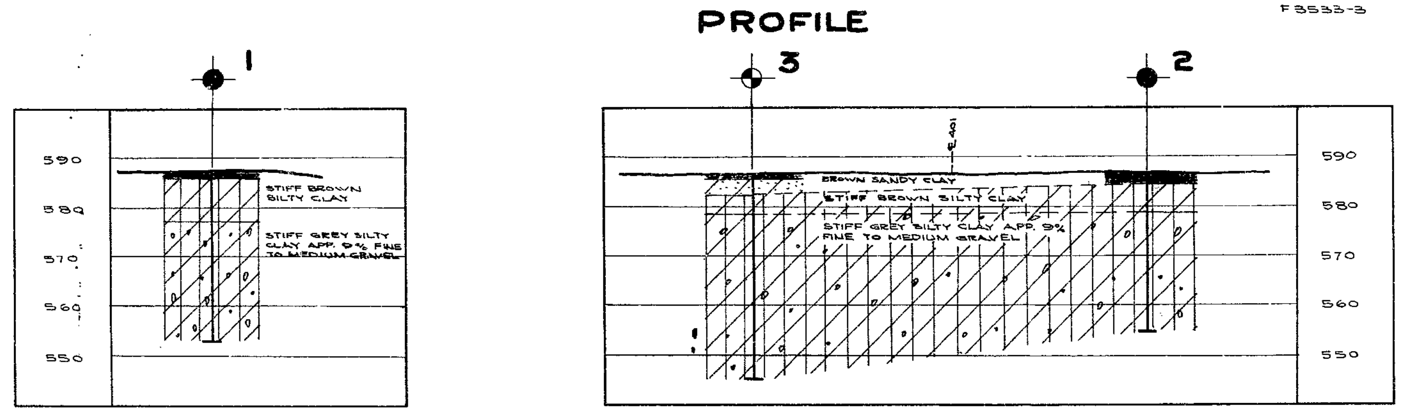


PROFILE

F3533-3

- 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.



A-A

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

DITCH (JEANNETTE CREEK) PROPOSED CROSSING

SHOWING POSITIONS & ELEVATIONS OF HOLES

HWY. 401 DISTRICT 1 COUNTY KENT
TOWNSHIP RALEIGH LOT 15 & 16 CON. VII
LOCATION APP. 6½ MI. S. of CHATHAM

DRAWN BY: T. MELLORS	CHECKED BY:	W.P. 13-59
DATE 6 MAY 1959.	APPROVED BY.	DRAWING NO. F59-14A
SCALE 1" = 20'		

cc: Mr. A. M. Toye

Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Section.

May 11, 1959.

FOUNDATION REPORT -

Attention: Mr. S. McCombie.

Re: Hwy. #401, Line 'A' and
Jeannette Creek Crossing
Lots 15 & 16, Con. VII,
Twp. of Raleigh - Approx.
6 1/2 Miles South of Chatham.

Enclosed herewith is our Foundation Report on the subsoil conditions existing at the above noted site. Reference to the contents of this report shows that the site is underlain by a layer of stiff silty clay which was proven for a depth of 42 feet.

Recommendations pertinent to the foundation design, are summarized as follows:-

(1) Subsoil conditions are such that at Elev. 575' or below in the stiff silty clay, for footings typically 7' to 10' wide, a safe footing pressure of 2 1/2 t.s.f. can be used for spread footing design. In order to protect footings from stream erosion and scour, and to allow for future deepening of the channel, it is recommended that footings be founded at Elev. 567' (approximately 8 ft. below stream bed elevation.)

(2) Long-term settlements resulting from abutment and embankment loadings have been estimated not to exceed 5 inches. For a single-span structure, differential settlements are considered tolerable.

(3) No excessive seepage problems with respect to footing excavations are anticipated.

cont'd. /2 ...

(4) No approach fill stability problems are anticipated. Bank slope on the upstream side of the structure should be protected by rip-rap.

L. G. Soderman

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATION ENGR.
per:

for

AL/MdeF
Encl.

(A. Loh,
FOUNDATION ENGR.)

cc: Messrs. A. Toye
H. A. Tregaskes
D. G. Ramsay
G. U. Howell
J. Roy
Dr. P. Karrow

Foundatinns Office.

File

FOUNDATION REPORT

on

Hwy. #401, Line 'A' and
Jeannette Creek Crossing
Lots 15 & 16, Con. VII,
Twp. of Raleigh - Approx.
6 1/2 Miles South of Chatham.

Site Plan No:	E-3590-1
Plan No:	F-3533-1
Profile No:	F-3533-3
Chainage:	Sta. 193+00.

Distribution:

Mr. A. M. Toye, Bridge Engineer.	(2)
Mr. H. A. Tregaskes, Cnnstruction Engineer.	(1)
Mr. D. G. Ramsay, Design Engineer.	(2)
Mr. G. U. Howell, District Engineer, Chatham, Ontario.	(1)
Mr. J. Roy, Regional Soils Engr. London Regional Office.	(1)
Dr. P. Karrow, Department of Mines.	(1)
Foundation Office.	(1)
File.	(1)

W.P. 13-59.

W.J. F-59-14.

INTRODUCTION:

Presented in this report are the results of a subsoil investigation carried out at a structure location approximately 6 1/2 miles south of Chatham where proposed Hwy. 401, Line 'A' crosses Jeannette Creek in Lots 15 & 16, Con. VII, Township of Raleigh (Sta. 193+00, Profile No. F-3533-3). This report contains the results of field and laboratory findings and recommendations for the foundation of the proposed structure.

The field work commenced on February 14, 1959 and was completed on February 16, 1959.

DESCRIPTION OF THE SITE AND GEOLOGY:

The site and its surrounding areas are generally flat farmlands; the areas on both sides of Jeannette Creek are presently under cultivation. At the time of the investigation the creek and its banks were covered with ice and snow.

Physiographically, the site is located on the St. Clair Plains, inundated by Glacial Lakes Whittlesey and Warren. According to available geological information, these extensive plains, covering a large area of South-Western Ontario, are covered by deep deposits of clay, underlain by limestone bedrock. At this site the upper 10 feet of the clay stratum has been subjected to oxidation, resulting in its present brownish colour.

DESCRIPTION OF FIELD & LABORATORY WORK:

Field work consisted of 3 sampled boreholes carried out by means of a trailer-mounted coredrill machine adapted for soil sampling. Conventional auger boring procedures were followed

cont'd. /2 ...

DESCRIPTION OF FIELD & LABORATORY WORK: (cont'd.) ...

and samples were recovered at depths required. 2" I.D. thin walled shelby tube samplers were used in the cohesive subsoil.

Upon receipt in the laboratory, samples were visually examined and identified. Routine index tests were performed on selected representative samples. Laboratory test results have been presented in the borehole logs and detailed in tabular form.

The location plan and subsoil profile are presented in Drawing No. F-59-14A.

SUBSOIL CONDITIONS:

Subsoil conditions are similar to all other sites previously investigated in this area. Reference to the borehole logs shows that the site is underlain by a stiff silty clay stratum, the upper zone of which has been subjected to oxidation. According to our boring data in this locality, this stiff clay stratum is underlain by a deep deposit of soft to medium clay extending over a considerable depth over bedrock.

In each of the sampled boreholes the frozen topsoil was found to be underlain by the stiff silty clay stratum. The upper 10 feet of the clay stratum has been oxidized to its present brownish colour. Below the oxidized zone the colour is predominantly grey. The stiff condition of the clay stratum is believed to be the result of desiccation. This stratum was explored to a depth of 42 feet below the existing ground surface (i.e. Elev.545') to confirm the stiff nature of the clay. In view of the similiarity

cont'd. /3 ...

SUBSOIL CONDITIONS: (cont'd.) ...

in geological formation, as well as subsoil conditions, between this site and the other sites previsouly investigated in this area, it is believed that the thick stratum of soft to medium clay would most likely be encountered at some depth below the stiff clay stratum.

In general, the stiff clay contains 25% silt, 20% sand and 9% fine to medium gravel throughout. The average unit weight and moisture content were found to be 132 p.c.f. and 18%. Liquid and plastic limits averaged 27% and 16%. Laboratory shear strength tests show an average of 2000 p.s.f. to be representative for the stiff clay stratum. A plot of shear strength versus depth has been presented and is included in this report under Appendix I. Judging from its moisture content and Atterberg limits, it appears that the stiff silty clay is saturated and preconsolidated.

Laboratory and field test results have been summarized in Table No. 1 and are included in this report under Appendix I.

WATER CONDITIONS:

Due to the low permeability of the clayey subsoil, it was not possible to accurately establish the ground water table of the site during the boring programme. Samples recovered from the boreholes were saturated and the ground water table has been assumed to be at the seasonal water level of Jeannette Creek at approximately Elev. 580' to 585'. In view of the fact that no water-bearing sand seams of any significance or artesian water conditions were encountered during the boring programme, seepage inflow during footing excavations will be local and of minor quantities, only.

cont'd. /4 ...

FOUNDATION CONSIDERATIONS:

The stiff silty clay stratum is competent to provide adequate foundation support for the proposed structure. Laboratory strength and compressibility characteristics are such that spread footing support can be obtained at Elev. 575' or below. At this elevation or below, for footings of 7' to 10' wide, an allowable bearing pressure of 2 1/2 t.s.f., incorporating a safety factor of 3 can be used for spread footing design. In order to avoid undermining of footings due to stream erosion and scour action, and to allow for future deepening of the channel, it is recommended that footings be founded at Elev. 567' (approximately 8 ft. below stream bed assuming stream bed elevation at 575').

Long-term settlements under the footings as a result of application of 2 1/2 t.s.f. abutment pressure and embankment load due to the weight of 7' to 10' fill have been estimated as not to exceed 5 inches. In view of the relatively uniform subsoil conditions at the site, little differential settlement need be anticipated of a single-span structure since each abutment will virtually settle the same amount. If centre piers are incorporated in the design, long-term differential movement between the piers and abutments would result, since consolidation under the footings of the piers would be unaffected by the approach fill adjacent to the abutments. It appears that if a multi-span structure is contemplated, a rigid-frame design is favourable only if it can tolerate differential settlement of the order of 2" to 2 1/2".

No excessive seepage problems during footing excavations are anticipated.

cont'd. /5 ...

FOUNDATION CONSIDERATIONS: (cont'd.) ...

Under the proposed grade line, the maximum height of fill is approximately 7' to 10'. The subsoil has sufficient strength to safely support this embankment loading. Bank slopes on the upstream side of the structure should be protected by rip-rap.

CONCLUSIONS & RECOMMENDATIONS:

(1) The site is underlain by stiff silty clay, followed by deep deposits of soft to medium silty clay.

(2) Subsoil conditions are such that spread footing support can be obtained in the stiff clay at Elev. 575' or below. At this elevation or below, for footings of 7' to 10' in width, an allowable bearing pressure of 2 1/2 t.s.f. can be used for spread footing design. In order to avoid undermining of footings due to stream erosion and scour action, and to allow for future deepening of the channel, it is recommended that footings be founded at Elev. 567' (approximately 8 ft. below stream bed elevation).

(3) Long-term settlements under the footings resulting from abutment and embankment loadings, have been estimated as not to exceed 5 inches. If a single-span structure is used, differential settlements are considered tolerable.

(4) No excessive seepage problems with respect to footing excavations are anticipated.

cont'd. /6 ...

CONCLUSIONS & RECOMMENDATIONS: (cont'd.) ...

(5) The proposed grade line does not present any approach fill stability problems. Bank slopes on the upstream side of the structure should be protected by rip-rap.

L G Soderman
for A. Loh,
FOUNDATION ENGR.

APPENDIX I.

TABLE NO. I .

SUMMARY OF FIELD & LABORATORY TESTS

JOB F 59-14.
W.P. 13-59.

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	T1	5'-7'	Stiff brown silty clay with some sand.	-	31.6	-	-	-	-	Approximately 9% fine to medium gravel throughout.
	T2	10'-12'	Stiff grey silty clay.	-	20.7	18.3	31.0	3720	130.0	
	T3	15'-17'	" " " "	-	18.5	-	-	2002	133.0	
	T4	20'-22'	" " " "	-	18.8	16.8	26.1	-	130.2	
	T5	25'-27'	" " " "	-	17.6	-	-	1928	131.0	
	T6	33'-35'	" " " "	-	17.2	15.2	28.0	-	129.4	
2	T1	5'-7'	Med. Stiff brown silty clay.	-	22.0	-	-	1385	121.0	Approximately 9% fine to medium gravel throughout.
	T2	10'-12'	Stiff grey silty clay.	-	19.5	-	-	3320	133.8	
	T3	15'-17'	" " " "	-	17.9	-	-	2430	132.6	
	T4	20'-22'	" " " "	-	18.9	-	-	1820	132.2	
	T5	25'-27'	" " " "	-	18.0	15.7	26.2	-	132.0	
	T6	30'-32'	" " " "	-	17.2	17.5	28.4	3100	128.7	
3	T1	3'-5'	Stiff brown sandy clay.	-	16.8	-	-	7490	129.0	Approximately 9% fine to medium gravel throughout.
	T2	6'-8'	Stiff brown silty clay.	-	15.6	-	-	7920	133.0	
	T3	10'-12'	Stiff grey silty clay.	-	15.6	16.1	24.0	-	136.8	
	T4	15'-17'	" " " "	-	17.8	16.1	27.8	2140	133.3	
	T5	20'-22'	" " " "	-	19.2	-	-	1640	124.0	
	T6	25'-27'	" " " "	-	19.0	16.7	27.1	1930	131.2	
	T7	30'6"-32'6"	" " " "	-	19.0	-	-	2560	132.0	
	T8	40'-42'	" " " "	-	20.0	16.0	25.5	1530	129.0	

T1 - denotes thin walled shelby sample.

FORM OB-ML-126 (REV. 1959.)—58-5691

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 13-59
JOB F 59-14
DATUM Geodetic
BORING DATE Feb.16/59

BORE HOLE NO. 1
STATION 193+70 ¢
COMPILED BY B.K.
CHECKED BY A.L.

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

LEGEND

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

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Ground Level.	587.7	
	Frost and Topsoil.	585.7	
	Stiff brown silty clay.	577.7	10
	Stiff grey silty clay - approximately 9% fine to medium gravel.	552.7	
	End of Borehole.		

STRENGTH AND PENETRATION RESISTANCE: P.S.F. 2000 4000 6000 8000; BLOWS FT. 50 100 150 200

CONSISTENCY: MOIST. CONTENT - % DRY WT. 10 20 30

DEPTH FEET: 10 20 30 40

SAMPLE	NATURAL UNIT WT. # C.F.
TW 1	-
TW 2	130.0
TW 3	133.0
TW 4	130.2
TW 5	131.0
TW 6	129.4

Borehole No. 1

FORM OB-ML-126 (REV. 1959.)—58-5691

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 13-59 BORE HOLE NO. 2.

JOB F 59-14. STATION 192+60(40'Rt.)

DATUM Geodetic COMPILED BY B.K.

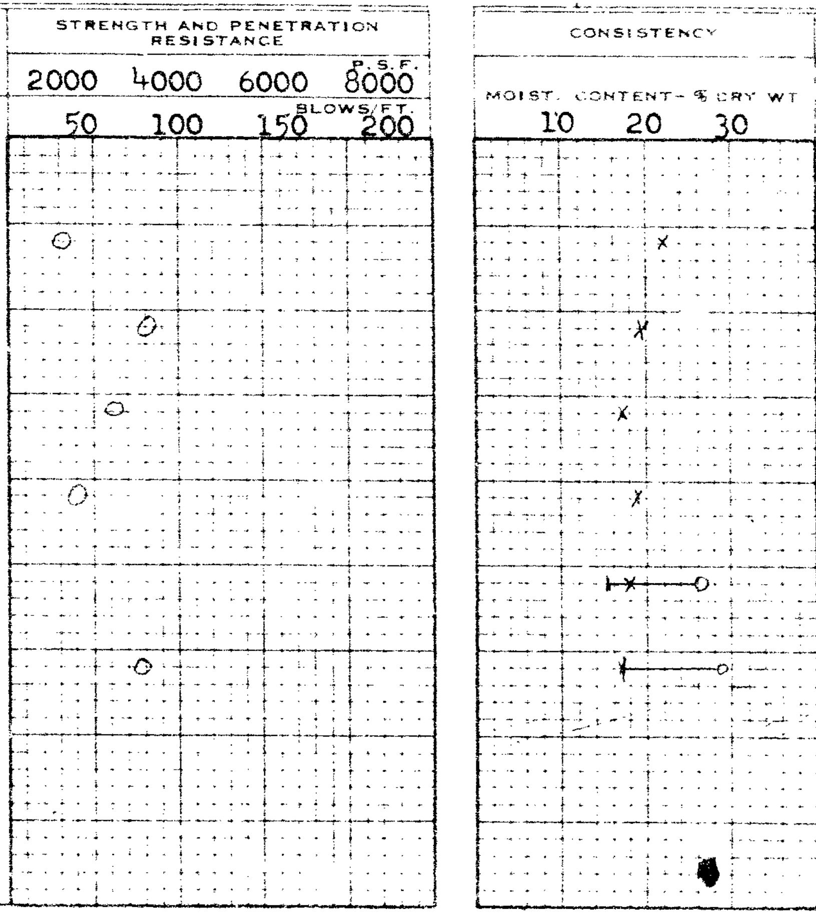
BORING DATE Feb.16/59. CHECKED BY A.L.

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)	$+^S$
NATURAL MOISTURE AND LIQUIDITY INDEX	X
LIQUID LIMIT	—o
PLASTIC LIMIT	⊢

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	Ground Level.	586.7			
	Frost and Topsoil.	584.7			
	Stiff brown silty clay.	578.7		TW 1	121.0
	Stiff grey silty clay - approximately 9% fine to medium gravel.		10	TW 2	133.8
				TW 3	132.6
			20	TW 4	132.2
				TW 5	132.0
		554.7 32.0'	30	TW 6	128.7
	End of Borehole.		40		



Borehole No. 2.

OFFICE REPORT ON SOIL EXPLORATION

Borehole No. 3.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 13-59.
JOB F 59-14.
DATUM Geodetic.
BORING DATE Feb. 14/59.

BORE HOLE NO. 3.
STATION 192+60 (40'Lt.)
COMPILED BY B.K.
CHECKED BY A.L.

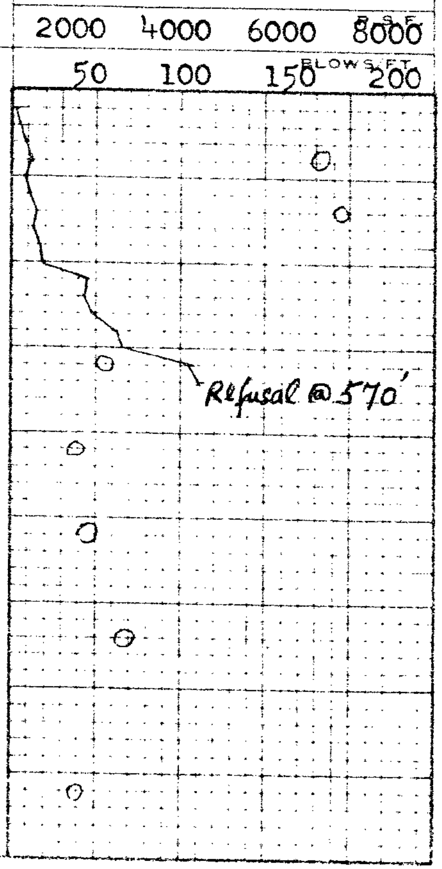
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

LEGEND

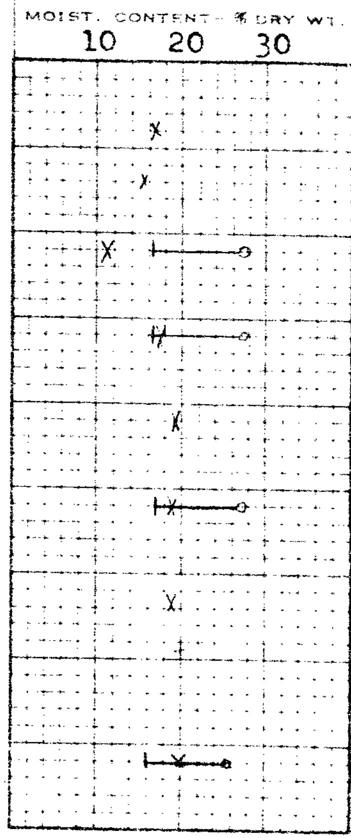
1/2 UNCONFINED COMPRESSION (Qu) ○
VANE TEST (C) AND SENSITIVITY (S) +S
NATURAL MOISTURE AND LIQUIDITY INDEX X
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Ground Level.	586.7	
	Topsoil.	585.7	
	Brown sandy clay.	582.7	
	Stiff brown silty clay.	578.7	
	Stiff grey silty clay - Approximately 9% fine to medium gravel.	544.7	
	End of Borehole.	42.0	

STRENGTH AND PENETRATION RESISTANCE



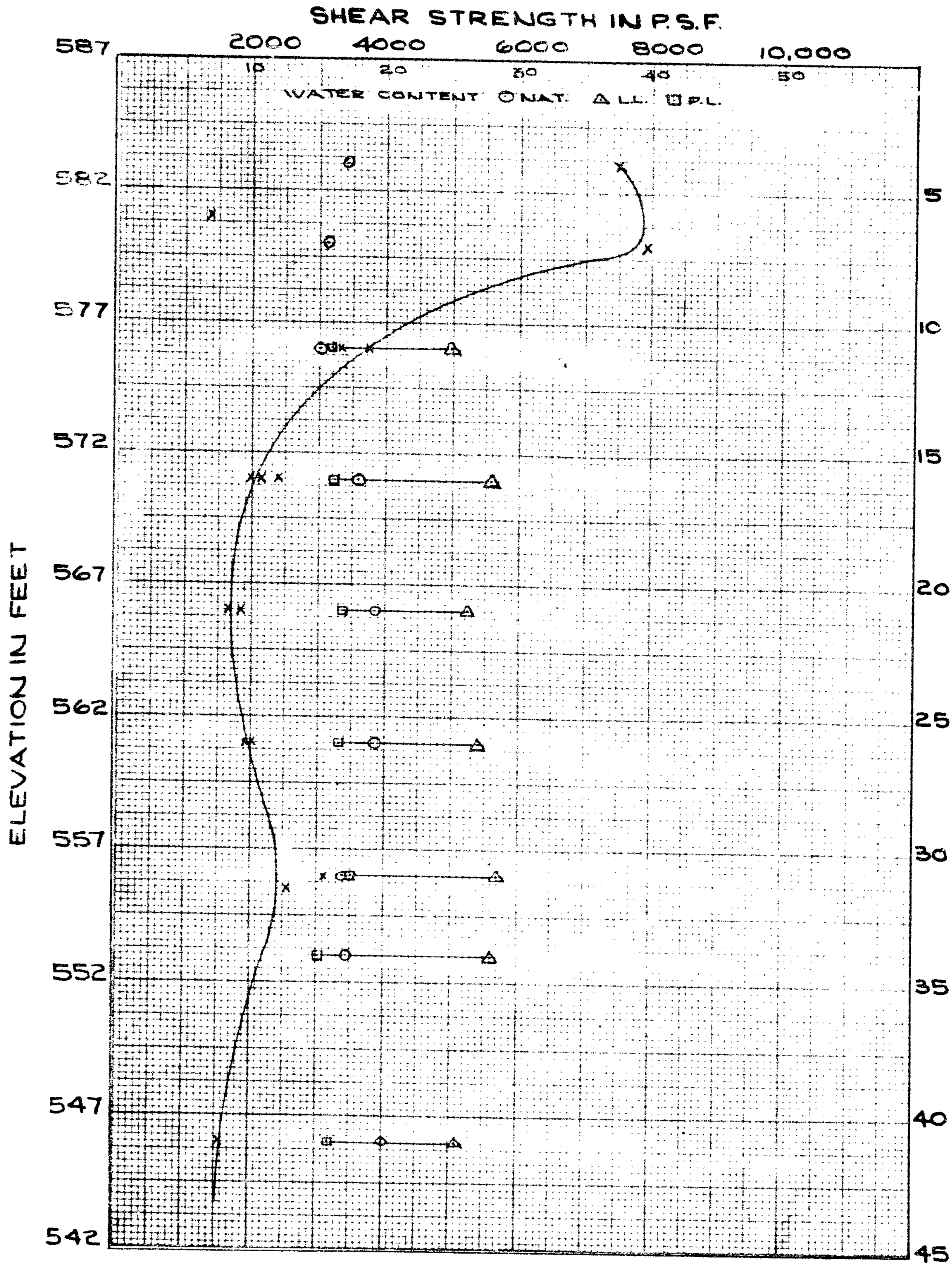
CONSISTENCY



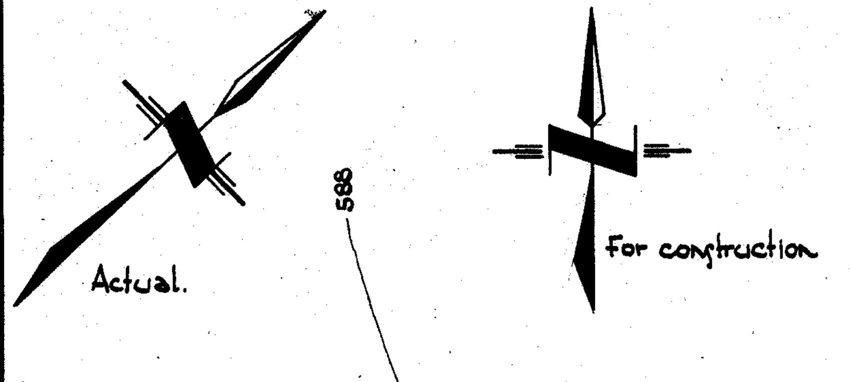
SAMPLE	NATURAL UNIT WT. P.C.F.
TW 1	129.0
TW 2	133.0
TW 3	136.8
TW 4	133.3
TW 5	124.0
TW 6	131.2
TW 7	132.0
TW 8	129.0

Borehole No. 3

W. P. 13-59 JOB F-59-14



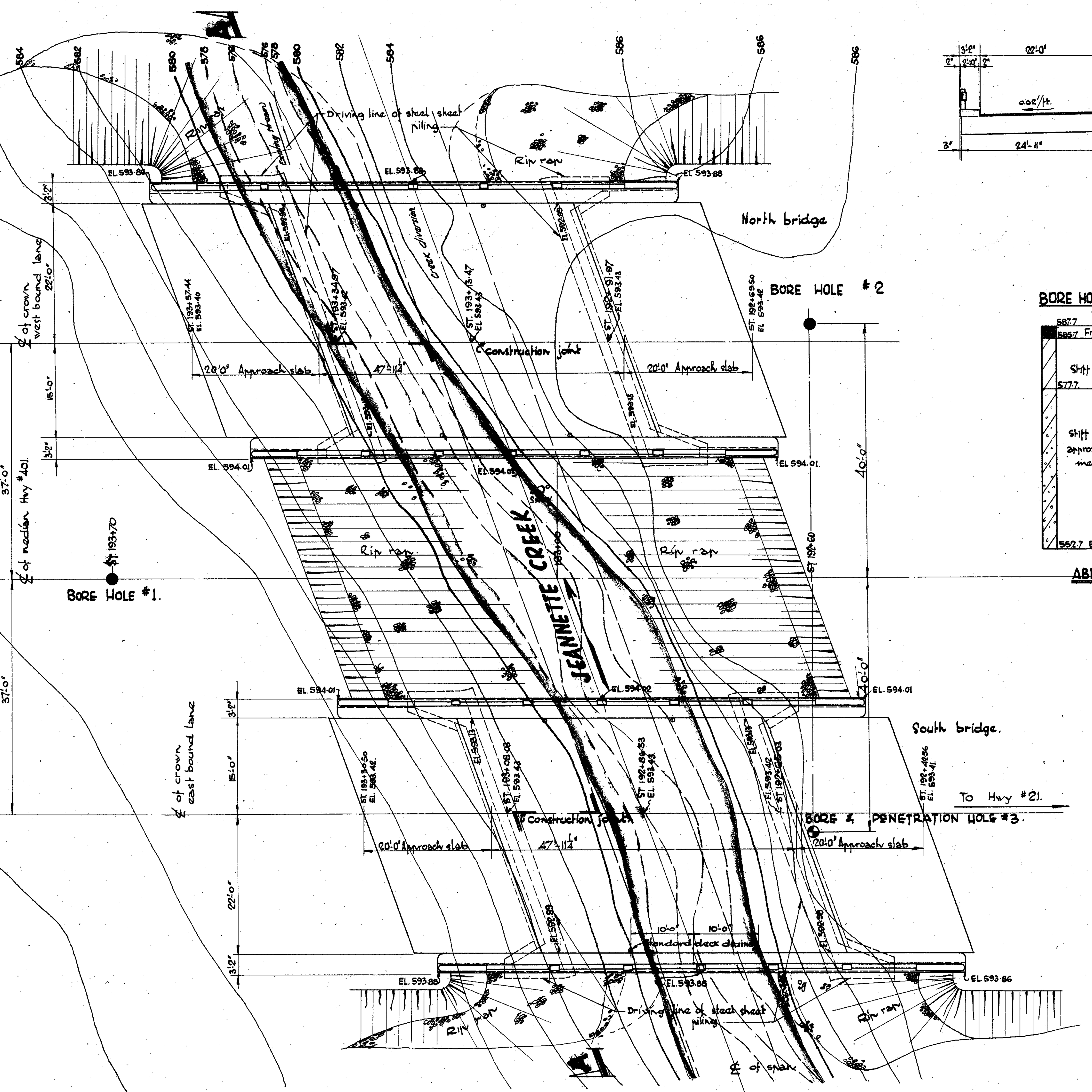
GRAPH FOR FOUNDATION REPORTS.



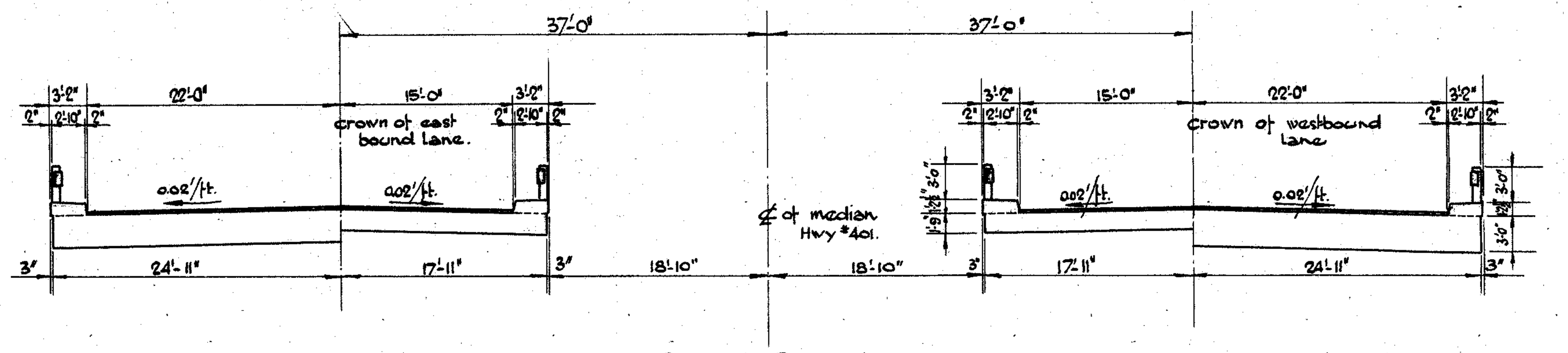
SKEW DETAILS:

Skew angle 20°

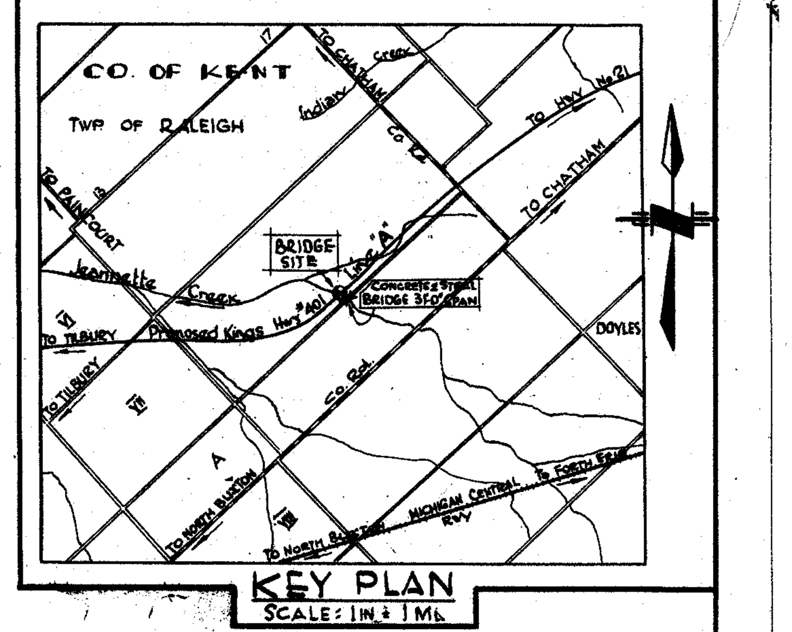
Sin.	0.34202
Cos	0.93969
Tan	0.36397
Cot	2.74748
Sec	1.06418
Cosec	2.92380



PLAN
Scale: 1" = 10'-0"



SECTION A-A
Scale: 1" = 10'-0"



KEY PLAN
SCALE: 1 IN = 1 MI.

BORE HOLE No 1

- 587.7 Ground level.
- 585.7 Frost and topsoil.
- Stiff brown silty clay.
- 577.7.
- Stiff grey silty clay approx. 9% fine to medium gravel.
- 552.7 End of bore hole.

BORE HOLE No 2

- 586.7 Ground level.
- 584.7 Frost & Topsoil.
- Stiff brown silty clay
- 578.7
- Stiff grey silty clay approx. 9% fine to medium gravel.
- 554.7 End of Bore hole

BORE & PENETRATION HOLE No 3.

- 586.7 Ground level.
- 585.7 Topsoil.
- 582.7 Brown sandy clay
- Stiff brown silty clay.
- 578.7
- Stiff grey silty clay approx. 9% fine to medium gravel
- 544.7 End of Bore hole

ABRIDGED VERSION OF SOIL INVESTIGATION REPORT BA 895.

GENERAL NOTES:

Note to district ENGINEER

Concrete work on this structure must not be commenced until monuments to fix control points have been erected and checked by the district ENGINEER.

Note to CONTRACTOR

Structure to be built in accordance with Form No 9 and the Special Provisions, extra copies of which may be obtained from the District ENGINEER.

All construction joints must be approved by the Bridge ENGINEER.

Concrete Mix

Minimum Strength @ 28 days { 3000 p.s.i. Structure; 2500 p.s.i. Footings

An approved admixture supplied by the Contractor will be added to all concrete as specified by the ENGINEER.
Max. size of aggregate ¾".

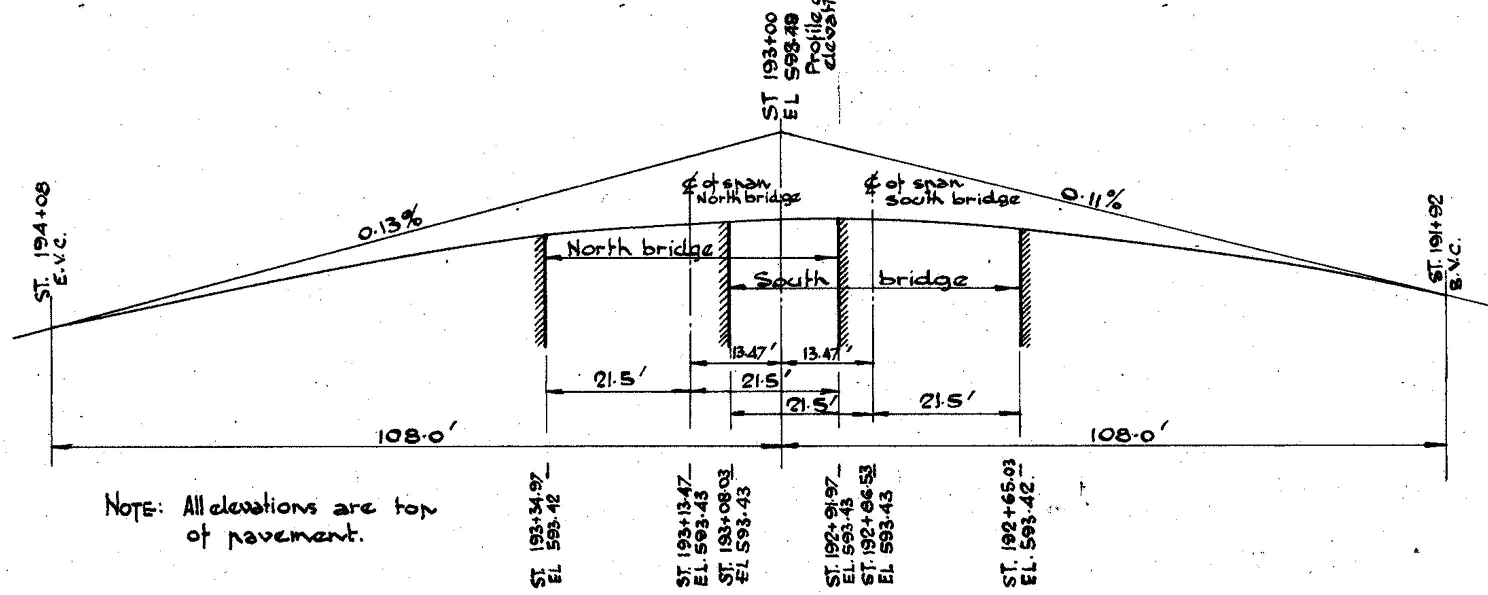
Boring Data

The complete soil investigation report BA 895 may be examined at the Bridge Office, Downsview, Toronto. The Department does not guarantee the accuracy of this report or the abridged version shown on this drawing.

Clear cover: 3" footings and on surfaces in contact with ground and water;
2" top of deck slab;
2" in remainder, unless otherwise stated.

Construction notes

1. All exposed edges to be chamfered 1" unless otherwise stated.
2. Formwork supporting wing walls to be left in position until after placing of curb concrete, and shall be removed only with the approval of the ENGINEER.
3. Backfill behind both abutments to be placed simultaneously.
4. Contractor to cut off deck drains 3" below bottom of deck



Note: All elevations are top of pavement.

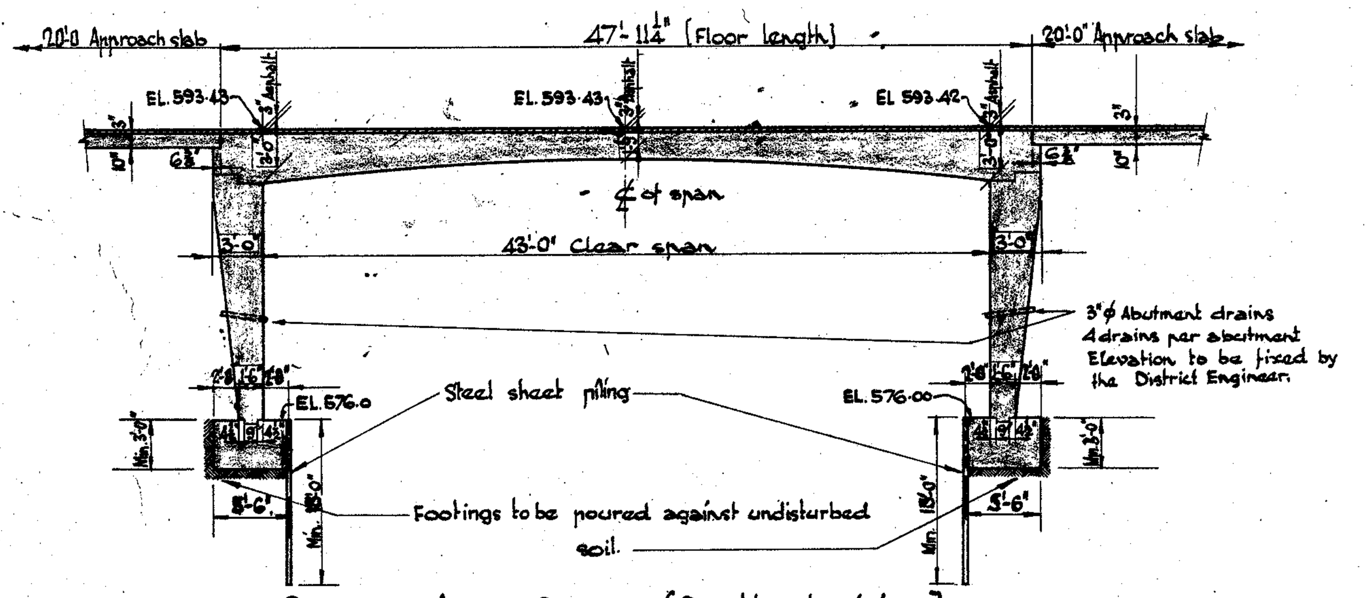
VERTICAL CURVE DETAILS
SCALES: Horiz. 1" = 20'-0"; Vert. 10" = 1'-0"

1800' V.C.
L.V.C. 216'

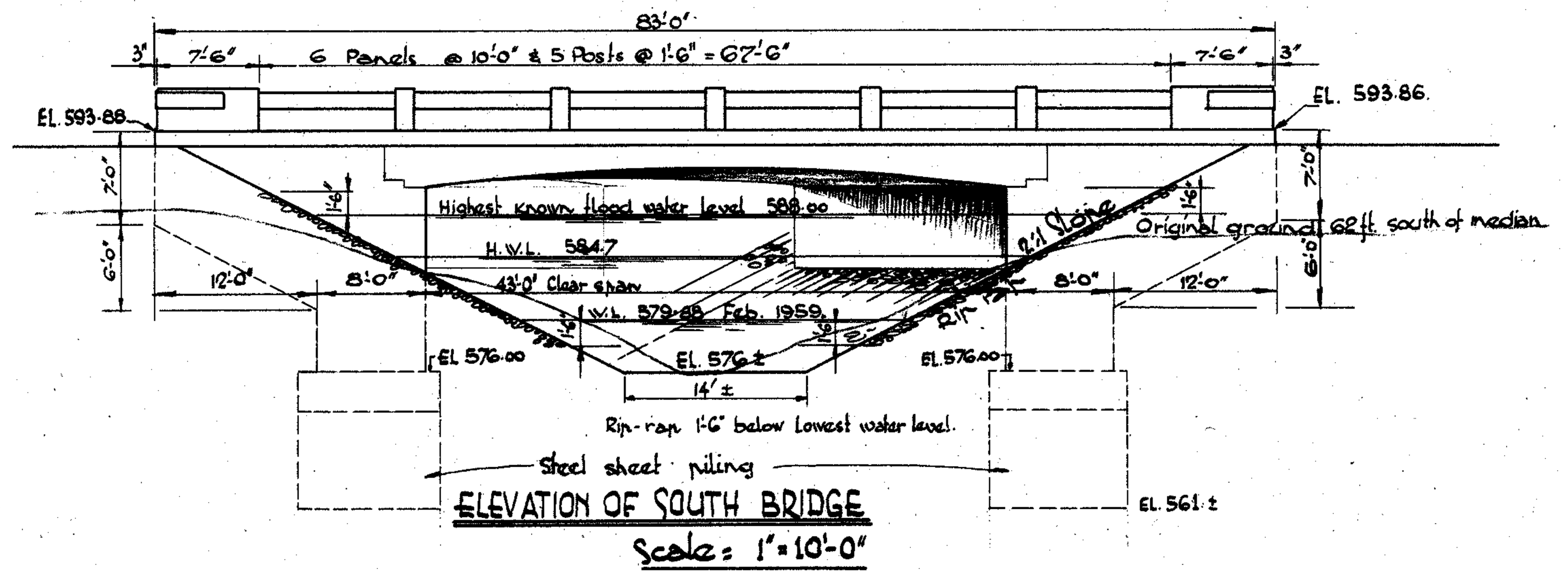
DRAWING LIST:

D-4374-1.	General plan, elevation an sections.
D-4374-2.	Footing plan and reinforcement.
D-4374-3	Frame details & reinforcement, handrails.
D-4374-4	Wing walls
D-4374-5.	Approach slabs.
D-4374-6.	Reinforcing steel schedule (hard grade)
D-4374-7	Do Do Do (structural grade).

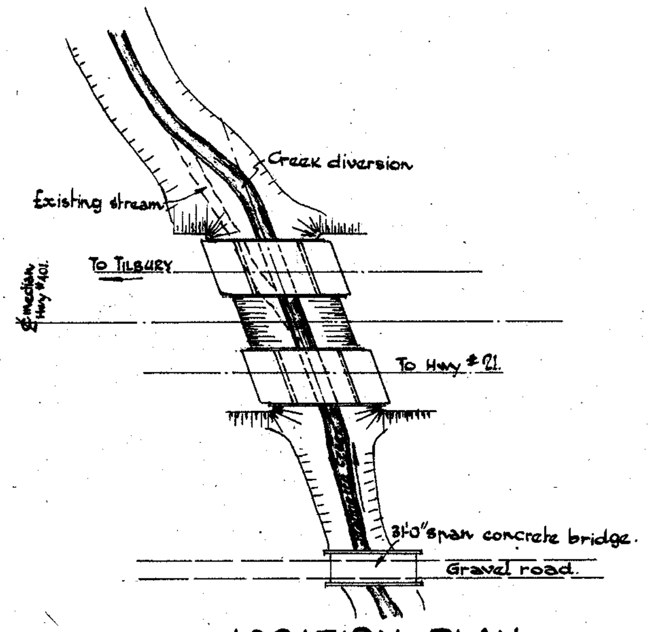
PRINT RECORD		
No.	FOR	DATE
30	Tender	14.2.61
10	D4C4F2	6.4.61
30	Tender	1.12.61
10	D4C4T2	25.1.62



SECTION ALONG CROWN (South bridge)
SCALE: ⅛" = 1'-0"



ELEVATION OF SOUTH BRIDGE
Scale: 1" = 10'-0"



LOCATION PLAN
Scale 1" = 100'-0"

W.P. 13-59.

DEPARTMENT OF HIGHWAYS:-ONTARIO-
BRIDGE OFFICE:-TORONTO

RALEIGH TOWNSHIP BRIDGE No 7.
Jeannette Creek crossing.

THE KING'S HIGHWAY No. 401 — DIST. No. 1.
CO. KENT
TWP. RALEIGH — LOT 15 & 16 — CON. VII

GENERAL PLAN, ELEVATION & SECTIONS.

APPROVED
BRIDGE ENGINEER — DESIGN ENGINEER

DESIGN	Adopted.	CHECK	P.R	CONTRACT NUMBERS	64-74 / 61-86
DRAWING	V.B.	CHECK	P.R		
TRACING		CHECK		LOADING H-20 S-16	DRAWING NUMBER ~~D-4374-1.~~
DATE October 1959.					

REFERENCE PLANS: E-3586-1. E-3533-1 E-3533-3 D-4374-P1.

REVISIONS — DATE — BY — DESCRIPTION

TWP. #103-228-1-A

APPENDIX B

Site Photographs

Photograph 1: Looking west at the adjacent south slope of the west abutment from the south slope of the east side of the bridge. The slope was heavily vegetated. No erosion or scouring was observed. (October 20, 2013).

Photograph 2: Looking east at the north slope adjacent to the east abutment. Slope face was densely vegetated. No erosion or scouring was observed. (October 20, 2013)

Photograph 3: Looking west at the south slope adjacent to the east abutment. A pile of debris was observed on the slope face. No erosion and scouring observed. (October 20, 2013)

Photograph 4: Looking west at the west abutment wall from the east abutment. Weep holes in the abutment wall were observed to be open and wet. Scouring of the exposed earth was observed due to probable numerous fluctuated cycles of the water level over time. In addition, adequate rip-rap protection of the slope face was not observed and may need to rehabilitated to protect the exposed ground from scouring effect. (October 20, 2013)

Photograph 5: Looking east from the north slope of the west side of the bridge towards the east abutment wall. Weep holes observed in the abutment wall were open and wet. Scouring of the exposed earth was observed most likely due to numerous fluctuated cycles of the water level over time. adequate rip-rap protection of the slope face was not observed and may need to rehabilitated to protect the exposed ground from scouring effect. (October 20, 2013)