

MEMORANDUM

asp

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

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

ATTENTION: Mr. S. McCombie

DATE: April 8, 1969

OUR FILE REF:

IN REPLY TO

APR 14 1969

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For

-- Structure #7 --
Of the

Proposed Hwy. #20, Q.E.W. and Q.E.
Extension Complex, Niagara Falls, Ont.
District No. 4 (Hamilton)
W.J. 69-F-2-7 -- W.P. 168-64-24

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

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

H. G. Selby

KGS/MdeF
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
G. K. Hunter (2)
H. Greenland
W. S. Melinyshyn
T. J. Kovich
B. A. Singh

K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Sternac,
PRINCIPAL FOUNDATION ENGR.

Foundations Files
Gen. Files

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FOUNDATION INVESTIGATION REPORT
For
-- Structure #7 --
Proposed Hwy. #20, Q.E.W. and Q.E.
Extension Complex, Niagara Falls, Ont.
District No. 4 (Hamilton)
W.J. 69-F-2-7 -- W.P. 168-64-24

1. INTRODUCTION:

In a memo, dated January 14, 1969, Mr. W. S. Melinyshyn, Regional Bridge Location Engineer, Central Region, requested foundation investigations at the locations of the proposed structures of the Hwy. #20, Q.E.W. and Q.E. Extension interchange. The request called for investigations at the piers and abutment locations of the proposed 9 new structures within this interchange complex.

The subsequent field work, as well as the laboratory testing program, was undertaken by the Foundation Section. The boreholes were located and surveyed by personnel of the Engineering Surveys Central Region. This report contains the results of the portion of the investigations carried out at the location of Bridge #7.

2. DESCRIPTION OF THE SITE AND FIELD INVESTIGATION:

The site of the proposed bridge is situated on relatively flat grassland. An existing combined sewer of 60-inch diameter runs under the proposed bridge site roughly parallel to and immediately north of the future Hwy. #20 Westbound lanes. A 12-inch diameter pipe of the sewer branches out southward at this location as shown on Drawing #69-F-2-7A.

Geologically the area is part of the physiographic region known as the "Haldimand Clay Plain". This region consists of a series of parallel belts. The site lies within the first such belt

2. DESCRIPTION OF THE SITE AND FIELD INVESTIGATION: (cont'd.) ...

adjoining the Niagara Escarpment. The height of the land is above 600 ft. comprising of recession moraines, built by the ice lobe that occupied the basin of Lake Ontario.

A total of seven sampled boreholes and adjacent to the holes, seven dynamic cone penetration tests were carried out at the approximate locations of the abutments and piers. The boreholes were sunk by means of diamond core drills adapted for soil sampling purposes. Standard penetration 'N' values were obtained by conventional methods described at the end of this report.

The locations and elevations of the boreholes, also the stratigraphical profiles, are shown on Drawing #69-F-2-7A.

3. SOIL CONDITIONS:

3.1) General:

The entire site of the proposed interchange complex is covered by a predominantly silt deposit of 25 - 40 ft. thickness, underlain by a layer of heterogeneous glacial drifts, which in turn, is followed by dolomite bedrock. The bedrock surface was found to vary between El. 583 ft. and El. 602 ft., the lowest level being observed near the existing Dorchester traffic circle.

The results of the field and visual classification tests were confirmed by some laboratory tests. The natural moisture contents of each sample was measured; further tests of Atterberg limits and grain-size analyses were carried out on representative specimens. Laboratory and field test results are plotted on the borelogs accompanying this report.

A brief description of the subsoils at this particular bridge site is given below:

3.2) Silts:

Between ground level and El. 603 - 607 ft., a silt deposit was encountered in each borehole. In Holes #33, 34 and 35 the

3. SOIL CONDITIONS: (cont'd.) ...

3.2) Silts: (cont'd.) ...

material was found to be somewhat disturbed, which was likely caused by the excavations for the existing combined sewer some years ago. The remoulded nature of the soil is well documented by the greatly reduced values of consistencies and relative densities experienced in B.H.'s #33 and 35 near the sewer. Values of cone penetration tests in these holes vary between 10 and 20 blows per ft. within the upper 20 - 35 ft., as opposed to the values within the undisturbed material, where below El. 620 ft., the cone penetration resistance exceeded 100 blows per ft. The average 'N' values of the standard penetration tests were estimated to be 20 blows per ft. near the sewer, and approx. 40 blows per ft. in the rest of the holes. In the disturbed material, 'N' values of 9 - 11 are not uncommon. Grain-size analyses revealed that the samples tested contained approx. 1 - 12% sand, 74 - 99% silt and 3 - 20% clay-size particles. The stratum, in its natural state, is finer near the surface, becoming somewhat sandier with depth. The cohesive portion of the layer has very slight plasticity, having an average of 18% plastic limit, whereas the liquid limit moisture content ranges from 23 to 32%. On account of the variations in grain-size distribution, the material was identified to be silt, sandy silt or clayey silt. The natural moisture contents were measured to range from 16% to 28%, averaging about 20% by dry weight.

3.3) Clayey Silts and Sandy Silts with Gravel (Glacial Till):

Underlying the silt deposit a heterogeneous mixture of sandy silt with clay and gravel or clayey silt with sand and gravel was observed. The overall thickness of the layer is 5 - 8 ft., the consistency of the cohesive variety being stiff to hard and the relative density of the granular portion generally dense to very dense. Plasticity tests were performed on a few samples of the glacial deposit, resulting in an average value of plastic limits of 12.5% and liquid limits of 18%. The natural moisture contents

3. SOIL CONDITIONS: (cont'd.) ...

3.3) Clayey Silts and Sandy Silts with Gravel (Glacial Till):
(cont'd.)

falling usually below the plastic limits, further indicating the preconsolidated history of the deposit. Samples subjected to grain-size analyses, exhibited a wide scatter of the amount of constituent particles, as marked on the borelog sheets. Due to the unsorted deposition and the well-packed nature of the stratum, it is less permeable than the overlying silts, consequently it is believed to stay fairly stable under the conditions of water flow.

3.4) Bedrock:

Bedrock was proved at two locations by diamond drilling in B.H.'s #31 and 34. The elevation of the rock surface shows some slight variations between 597 ft. and 600 ft. The bedrock is classified to be a medium grey dolomite with thin argillaceous seams and some seams and nodules of gypsum. Five ft. thickness of the rock was drilled with AX core barrel and diamond bit at each location, yielding 90 - 99% rock core recovery.

4. GROUNDWATER CONDITIONS:

Groundwater observations were carried out in the boreholes for several days. Since the water-bearing silt stratum lacks adequate clay binder, the boreholes usually closed in very soon after the withdrawal of the casings. The elevation, where the borehole "caved in" under the influence of water, was assumed to be the groundwater level. The observed water levels were found to range between El. 611 ft. and 614 ft. - i.e., some 20 - 22 ft. below existing ground elevation.

5. DISCUSSION AND RECOMMENDATIONS:

5.1) General:

Bridge #7 is proposed to carry the Q.E.W. extension over Hwy. #20 Westbound and the ramp Dorchester - West. The bridge will

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

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

likely be designed as a three-span structure with either closed or spill-through type of abutments. The height of the approach fill of the Q.E.W. extension will be around 9 ft. at the east and approx. 15 - 16 ft. at the west. The proposed grade of the Dorchester Westbound ramp and Hwy. #20 Westbound at the crossing will be between El. 621 and 623.5 ft., some 10 - 12 ft. deeper than the existing ground level.

The silt subsoil, in its original state, is well packed and hard or very dense above the groundwater level. As was mentioned earlier, however, the excavations for the existing sewers disturbed the soils quite considerably, thereby reducing the relative density or consistency of the deposit. It is felt that such soils do not possess sufficient strength to support the structure economically on spread footings. Based on the findings at the borehole locations only, the extent of the earlier excavations cannot be clearly established. The width of the excavations at ground level might have been anywhere between 30 and, say 120 ft., depending on whether the construction was carried out with or without sheetpiling or bracing.

5.2) Structure Foundations:

Due to above uncertainties, it is suggested that the abutments as well as the piers, be supported on end-bearing piles driven to refusal. Adopting piled foundations for the abutments, the pile caps could be formed either within the approach fills or below the existing ground, with sufficient cover for frost protection (4 ft.). The pile caps for the piers could also be constructed with the base of pile caps at four ft. below finished grades. The use of steel H-piles appears to be the most economical choice, piles being driven to refusal on bedrock. In this case, design loads equal to the full structural strength of the H-section used, may be employed on the footings. The estimated bedrock elevations at the locations of the proposed abutments and piers, are tabulated on the following page:

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

5.2) Structure Foundations: (cont'd.) ...

Structure	Locations	Assumed Elevations of Refusal of Piles (ft.)
North Structure	East Abutment	600
	East Pier	598 - 600
	West Pier	598 - 600
	West Abutment	597 - 598
South Structure	East Abutment	595 - 597
	East Pier	598 - 600
	West Pier	598 - 600
	West Abutment	598 - 600

The groundwater level was found to be a few ft. lower than the estimated bottom of the pier footing excavations at the time of the field work. A rise of the water level may, however, occur at any time, on which occasion the excavations will be under water. The silt subsoil is susceptible to conditions of unbalanced hydrostatic head, consequently some dewatering scheme for the excavations below the groundwater level has to be provided for. The prevailing water level should be established right before commencing the construction in order to determine whether sheet piling of the excavations or installation of well-points will be necessary in order to ensure stability.

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

5.3) Foundations in the Vicinity of Utilities:

It is not as yet known by this Section whether or not the existing sewer will remain operational after the construction of the new interchange. Regardless of this, suggestions for the construction of piled foundations in the ^{vicinity} ~~proximity~~ of underground utilities are given in the following paragraphs:

(1) Where piles will be 12 feet or more from the edge of a utility, no special precautions need be taken.

(2) All piles closer than 12 feet from a utility should be prebored to a depth of 6 ft. below the pipe bottom. The size of the augered hole need only be slightly larger than the pile section.

(3) Where holes are augered in non-cohesive subsoil, casing may be required to prevent the holes from caving in.

The above procedure was followed in Contracts 63-182 and 68-24 with satisfactory results.

5.4) Structure Approaches:

No stability problems are foreseen for the proposed approach embankments, provided they are built with 2 horizontal to 1 vertical slopes.

It is assumed that the bottom of the proposed cut of Hwy. #20 Westbound will be well above the prevailing water level, and the slopes will be stable with 2 horizontal to 1 vertical slopes. Nevertheless, in the rather unlikely event of seepage through the slopes, the following protective steps should be taken:

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

5.4) Structure Approaches:

(a) A drainage blanket, consisting of suitable granular material which will perform as a filter, having a minimum thickness of 12", should be constructed over the affected areas.

(b) Sub-drains, consisting of suitably sized perforated pipes (6" - 8"), should be constructed at the toes of slopes with the pipe inverts at a sufficient depth for frost protection (4 ft.). The pipes should be backfilled with granular material, so as to permit free drainage of the blanket and the roadbed.

6. MISCELLANEOUS:

The field work was carried out during the period February 19 - 28, 1969 under the supervision of Messrs. A. K. Barsvary, Senior Foundation Engineer, P. Payer and G. Allen, Project Foundation Engineers.

Equipment used was owned and operated by Dominion Soil Investigation Ltd., Toronto, and Peninsula Soils Investigation Company, Welland, Ontario.

This report was written by Mr. A. K. Barsvary and reviewed by Mr. K. G. Selby, Supervising Foundation Engineer.

April 1969.

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 33

FOUNDATION SECTION

JOB 69-F-2-7

LOCATION Co-ord. 659,118 N; 102,613 E.

ORIGINATED BY PP

W.P. 168-611-24

BORING DATE Feb. 25-26, 1969

COMPILED BY AKB

DATUM Geodetic

BOREHOLE TYPE Washboring BX Casing

CHECKED BY

SOIL PROFILE

SAMPLES

DYNAMIC PENETRATION RESISTANCE

BLOWS / FOOT

20 40 60 80 100

SHEAR STRENGTH P.S.F.

LIQUID LIMIT — FL

PLASTIC LIMIT — PL

WATER CONTENT — W

WATER CONTENT %

10 20 30

BULK
DENSITY

REMARKS

ELEV.
DEPTH

DESCRIPTION

STRAT. PLOT

NUMBER

TYPE

BLOWS / FOOT

EV. SCALE

633.6

Ground Level

0.0

Clayey silt & silt
with traces of sand

(Backfill)

Stiff to hard

1 SS 16

2 SS 13

3 SS 40

4 SS 11

5 SS 23

6 SS 36

7 SS 31

8 SS 71/6"

9 SS bounces

630

620

610

600

Gr. Sa. Si. Cl

0 4 76 20

611.6

0 1 94 5

34 31 29 6

603.1

30.5 Sandy silt with clay
& gravel (Till)
598.7 Very dense34.9 End of Borehole
Probably Bedrock

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 36

FOUNDATION SECTION

JOB 69-F-2-7

LOCATION Co-ord. 659,088 N; 102,523 E.

ORIGINATED BY PP

W.P. 168-64-24

BORING DATE Feb. 27 & 28, 1969

COMPILED BY AKB

DATUM Geodetic

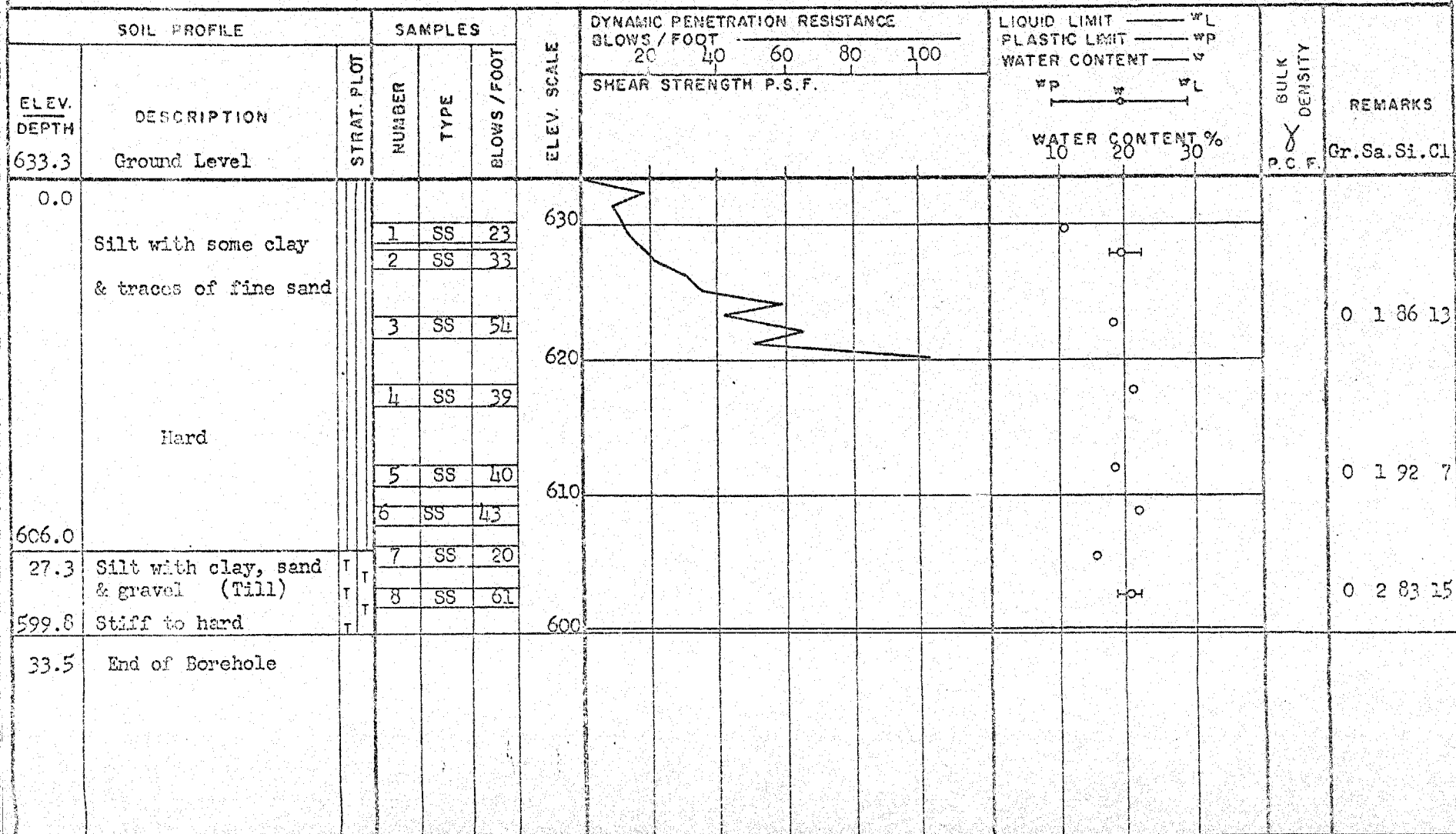
BOREHOLE TYPE Washboring, BX Casing

CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W		BULK DENSITY Y P.C.F.	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80			100	Wp
632.6	Ground Level													
0.0	Silt with some clay, becoming silt with traces of fine sand		1	SS	37									
			2	SS	45									
			3	SS	54									
	Hard and dense to loose		4	SS	40									
			5	SS	43									
			6	SS	38									
			7	SS	20									
			8	SS	9									
604.6														
28.0	Sandy silt with clay & gravel (Till)	T	9	SS	32									
600.0	Dense	T												
32.6	End of Borehole													

▽ 611.8

CHECKED BY



ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

TYPE OF SAMPLE

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

SOIL TESTS

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

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

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

STRESS AND STRAIN

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

EARTH PRESSURE

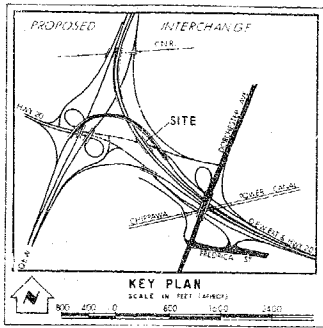
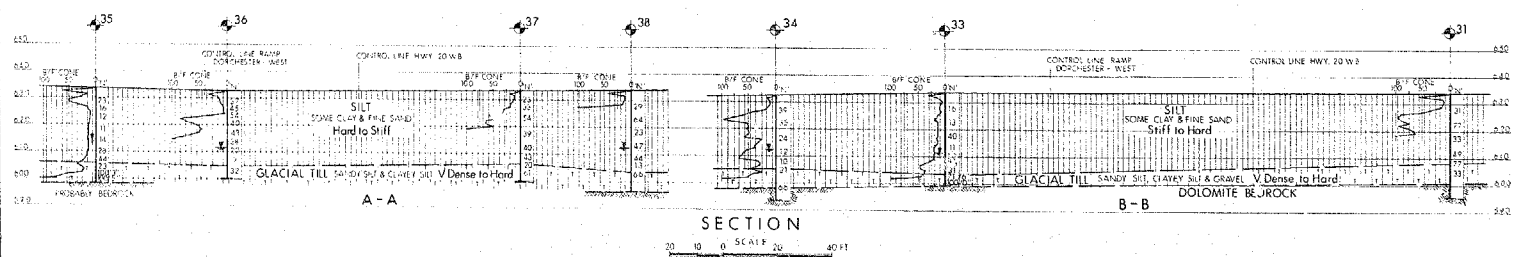
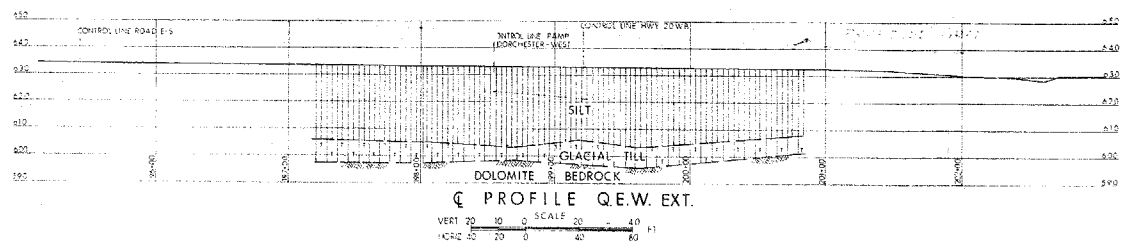
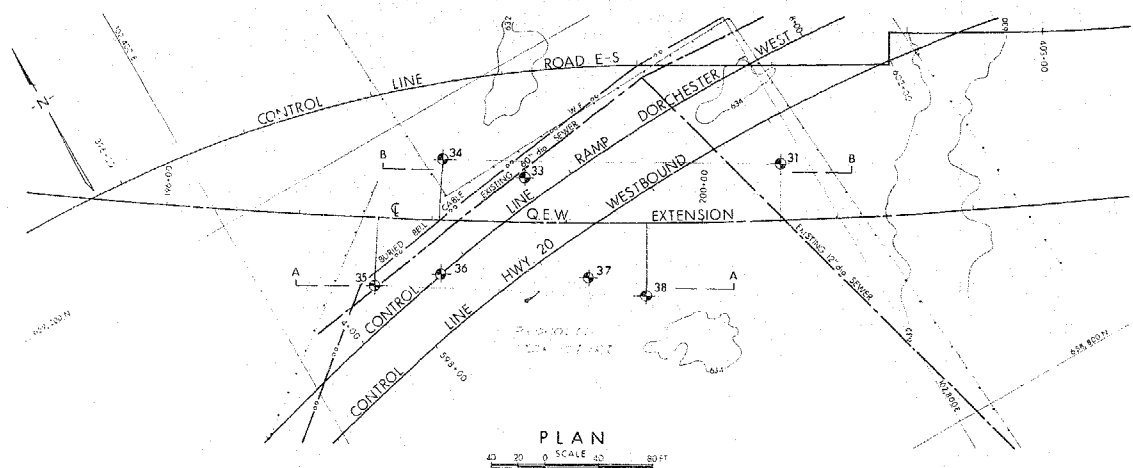
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

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

SLOPES

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



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation. Feb. 1969

NO.	ELEVATION	CO-ORDINATES	FEET
31	633.1	659,033	102,747
33	633.0	659,118	102,812
34	637.0	659,160	102,567
35	633.0	659,105	102,475
36	647.0	659,088	102,523
37	633.3	659,030	102,617
38	633.4	658,997	102,618

- 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.

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE - FOUNDATION SECTION

BRIDGE No 7
Q.E.W. EXTENSION OVER HWY 20 WB

KING'S HIGHWAY NO 20 & Q.E.W. INTERCHANGE DIST. NO. 4
CO. WELLAND NIAGARA FALLS
TWP. LOT CON.

BORE HOLE LOCATIONS & SOIL STRATA

DRAWN: A.B. CHECKED: W.P. NO. 158-64-24
DATE: 10 APRIL 1969 SITE NO. 69-F-2-7A
APPROVED: [Signature] DATE: 10 APRIL 1969

PRINT RECORD

NO.	FOR	DATE

Department of Highways Ontario

Copy for the information of
Mr. K. Selby,
Foundation Section, Lab. Bldg.

Mr. A.E. McKim,
Bridge Control Engineer,
Bridge Office

B.S. Richardson

February 10, 1970

Mr. M. Stoyanoff

Q.E.W./Hwy. 20 Interchange
Bridge #7, Bridge Site No. 34-245
District No. 4

69-F-2-7

We have just received from the Foundation Section the results of their probing for the existing 60" I.D. storm and sanitary sewer. The results do not confirm the survey carried out within the sewer but agree reasonably well with the centre-line location obtained by joining the manholes on the photogrammetric survey.

There is no way to establish positively which, if either, set of data is correct in the few days available before the project goes to Contract Control. I propose to assume that the soil probings are correct. The augering for the piles will show whether or not this assumption is correct. If revisions to the footing prove to be necessary, a delay of a few days could arise. For this reason the Contractor should be advised by Special Provision of the possibility of a delay so that he can schedule the driving of augered piles first and thus minimize possible idle time.

The revisions to the drawings will be completed in about two days. There will be small changes to piling, footing concrete, footing steel, footing excavation and cement. There should be time to incorporate these changes before the Contract documents go to Contract Control.

The Foundation Section believe that the sewer could be damaged by a carelessly used auger. The District staff should supervise the operation very carefully.

It is to be hoped that in future projects, the locating of underground services in areas where piles are to be used will be carried out before structural design commences.

BSR:rd

B.S. Richardson,
Regional Bridge Design Engineer

c.c. C.S. Grebski
H. Greenland
W. Friedmann
K. Selby
W. Melnyshyn



LIST OF PILES

LOCATION	NO.	LENGTH	TYPE
WEST ABUT.	32	39'-0"	12 BP53
EAST ABUT.	38	34'-0"	Do
PIER #1	40	22'-0"	Do
PIER #2	40	24'-0"	Do

LAYOUT OF W.P.'S

NOTES:

* AZIMUTH OF TANGENT AT W.P. 1

FOOTING LAYOUT

PIER #2

NOTES:

LOCATION OF W.P.'S

CO-ORDINATES

W.P. #	STATION ON Q.E.W.	N	E
0-1	197+81.69	659 130.53	102 521.83
1	198+41.87	659 147.45	102 539.56
2	197+24.13	659 123.81	102 448.59
1-0	198+34.69	659 107.50	102 568.94
1	198+79.35	659 115.08	102 623.33
2	198+54.76	659 111.32	102 592.62
3	198+15.49	659 103.50	102 542.08
4	197+95.00	659 098.35	102 514.36
2-0	199+54.08	659 047.02	102 669.78
1	200+13.95	659 049.41	102 736.36
2	199+50.25	659 048.66	102 700.68
3	199+23.67	659 044.84	102 640.37
4	199+03.53	659 041.64	102 608.07
3-0	200+18.06	659 015.87	102 725.69
1	201+05.09	659 021.69	102 829.35
2	199+38.99	659 010.25	102 631.33

PLAN SCALE: 1"=1'-0"

PIER #2 FOOTINGS

SCALE: 3/4"=1'-0"

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

67-1-2-7

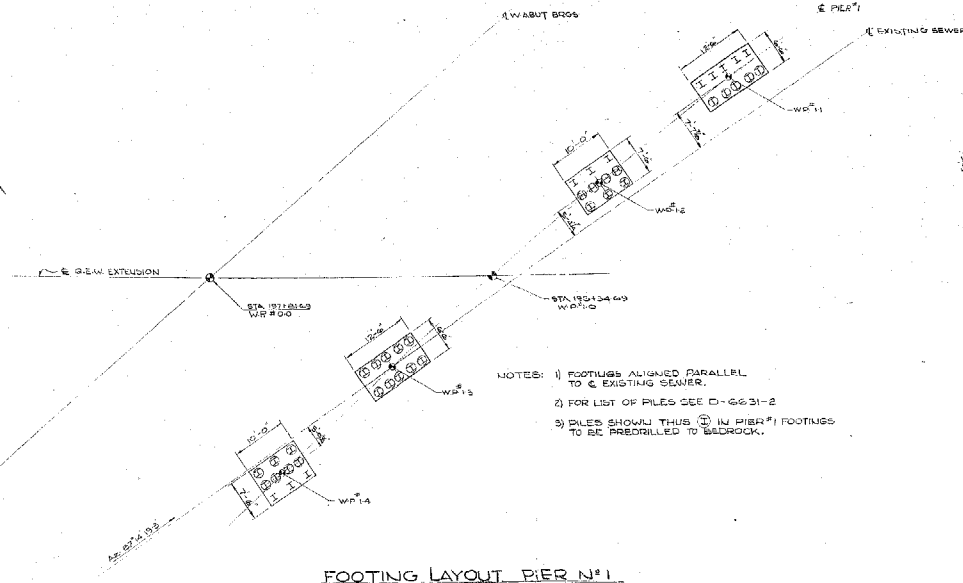
BRIDGE #7

HWY #20 O'PASS @ Q.E.W. EXTENSION
KING'S HIGHWAY No. Q.E.W.
Q.E. WELLAND
CITY OF NIAGARA FALLS DIST. CON.

FOOTING LAYOUT I

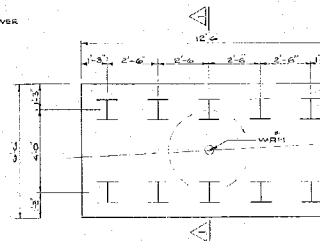
APPROVED:	DESIGNER:	CONTRACT NO.	77-1-2-2
CUBSON	CHECK: 77C	BRIDGE NO.	77-1-2-2
DRAWING: 1/4"=1'-0"	CHECK: 47.77	DRAWING NO.	D 6631-2
DATE: 1 SEP 69	LOADING: 100,000 LB		



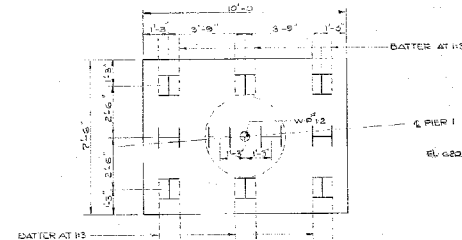


- NOTES: 1) FOOTINGS ALIGNED PARALLEL TO EXISTING SEWER.
2) FOR LIST OF PILES SEE D-6631-2
3) PILES SHOULD BE PREDRILLED TO BEDROCK.

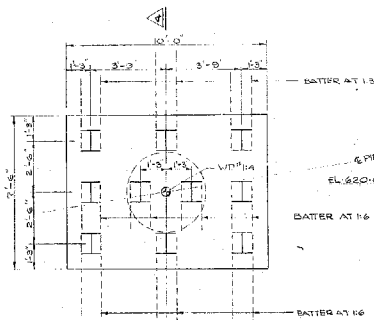
FOOTING LAYOUT PIER N°1
SCALE 1/100



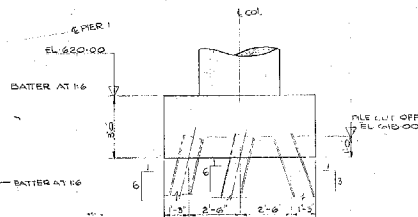
PLAN AT WP 11
SCALE 3/10



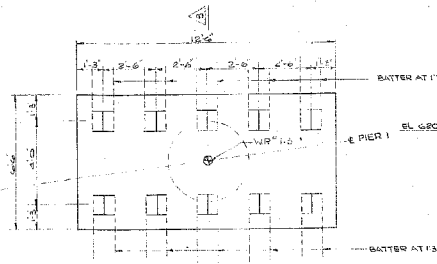
PLAN AT WP 12
SCALE 3/10



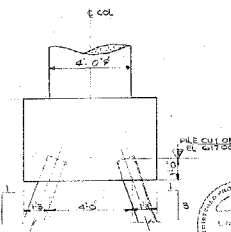
PLAN AT WP 14
SCALE 3/10



SCALE 3/10



PLAN AT WP 13
SCALE 3/10



SCALE 3/10

REVISIONS
DATE BY DISCUSSION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

19-P-1-7

BRIDGE N°7

HWY. N°40.0000 & C.R.W. EXTENSION
CITY OF KILGORE, N.B. 4
CITY OF KILGORE, N.B. 4
CITY OF KILGORE, N.B. 4

FOOTING LAYOUT II

APPROVED	DESIGN	CHECK	DATE	CONTRACT	BRIDGE	NO.	DATE

FOR REDUCED PLAN
USE SCALE BELOW
1" = 3' INCHES ON ORIGINAL PLAN

PRINT RECORD
No. FOR DATE