GEOCRES No. 30M15-55

DIST. 7 REGION

W.P. No. 59-75-11

CONT. No.

W. O. No.

STR. SITE No.

HWY. No. 401

LOCATION CNR BOWMANVILLE

CREEK BRIDGE South of Hwy 401

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

30M15 - 55
GEOCRES No.

Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

REPORT

TO

MINISTRY OF TRANSPORATION
AND COMMUNICATIONS

SUBSURFACE INVESTIGATION
PROPOSED C.N.R. BOWMANVILLE CREEK BRIDGE
SOUTH OF HIGHWAY 401
W.P. 59 75-11 DISTRICT 7

NEWCASTLE ONTARIO

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March, 1980 801-1046

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ABSTRACT

A subsurface investigation was carried out by Golder Associates for the Ministry of Transportation and Communications at the site of a proposed C.N.R. bridge over Bowmanville Creek to the south of Highway 401 in Newcastle, Ontario.

In summary, the borings indicate that very loose to compact lacustrine deposits overlie a compact to very dense sandy silt to silty sand till stratum which overlies limestone bedrock at a relatively shallow depth. A sample from the creek bed indicates that the bed material of Bowmanville Creek consists of sand and gravel, containing some silt and cobbles. Groundwater levels measured during the investigation were at, or slightly above, the water level in the creek.

It is recommended that the proposed structure be supported on shallow spread footings founded on competent till or bedrock or supported on piles founded on bedrock. Recommendations related to the geotechnical aspects of bridge abutment design and channel lining are also presented.

1. INTRODUCTION

As part of the Highway 401 widening project from Waverly Road to Highways 35 and 115, Golder Associates have been retained by the Ministry of Transportation and Communication (letter dated February 29, 1980) to carry out a foundation investigation for a proposed C.N.R. bridge over Bowmanville Creek to the south of Highway 401 (W.P. 59-75-11).

The purpose of this investigation was to determine the subsurface conditions at the site and, based on our interpretation of these conditions, to provide engineering recommendations for the geotechnical design of the foundations for the proposed structure.

The investigation was carried out, and this report was prepared, in accordance with our proposal letter, dated February 21, 1980 and Consultant's Agreement No. 4242-9079-130.

2. DESCRIPTION OF PROJECT

Details of the project were provided during a meeting between Mr. Devata of the Ministry of Transportation and Communications and Messrs. Crooks and Hubble of Golder Associates on February 12, 1980. Additional details were provided on M.T.C. Plan E 5466-1, "Bridge Site Plan, Proposed Crossings at Canadian National Railways and Bowmanville Creek South of Highway 401", dated January 1980.

We understand that the proposed structure is to carry the C.N.R. main line over Bowmanville Creek, immediately upstream of the existing bridge and approximately 600 ft. to the south of Highway 401. We further understand that the bridge is proposed to be a single, 60 ft. span reinforced concrete structure.

3. SITE AND GEOLOGY

The site is located in the Town of Newcastle, Ontario. The ground surface in the area is generally flat lying. The Bowmanville Creek, which drains the surrounding area, has eroded a shallow valley that, at its maximum depth, extends about 15 ft. below the surrounding area at the proposed bridge location. Immediately to the south of the proposed bridge site, embankment fill has been placed as an approach to the abutments of the existing C.N.R. bridge over the creek.

The site is located in the physiographic region known as the Lake Iroquois Plain, which was formed by post-glacial Lake Iroquois. Surficial deposits of silts and sands overly deposits of glacial till, with Ordovician shale and limestone bedrock occurring at a relatively shallow depth. Recent sand and gravel deposits are found on the flood plain of Bowmanville Creek.

4. SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each of the boreholes together with the results of laboratory tests carried out on representative samples of the soil strata are given on the attached Record of Borehole sheets. A simplified stratigraphy for each borehole is shown on the longitudinal section of Sheet 1. It should be noted that the stratigraphic boundaries indicated on the borehole logs and longitudinal section are inferred from non-continuous samples. These boundaries typically represent a transition from one soil type to another and do not necessarily indicate an exact plane of geologic change. Further, the subsurface conditions may vary between boreholes.

In general, the ground surface at the site is underlain by about 0.5 ft. of topsoil, which overlies a stratum of very loose to compact lacustrine silty sand, typically containing a trace to some clay, gravel and organic matter. This stratum extends to a depth of 5.0 ft. and 7.5 ft. below ground surface in Boreholes 1 and 2, respectively. In Borehole 1, this deposit was found to have an 'N'[1] value of 10 blows/ft. and a natural water content of 18 per cent, while in Borehole 2, 'N' values were 1 and 3 blows/ft. and water contents were 28 and 23 per cent respectively.

Underlying the upper lacustrine deposit, a stratum of compact to very dense silty sand to sandy silt till containing some clay, gravel and cobbles was encountered. The 'N' values measured in this stratum were generally greater than 20 blows/ft. and the water content of the samples obtained were all less than 10 per cent. However, in Borehole 2 (Sample 3) an 'N' value of 12 blows/ft. and a water content of 13 per cent were recorded, suggesting that the upper portion of the deposit has been loosened.

At elevation 237.9 and 237.5 in Boreholes 1 and 2, respectively practical refusal to advance of the hollow stem augers were met, at which depths BXL rock core was extracted for a length of 10.0 ft. and 10.5 ft., respectively. The bedrock was found to be a highly weathered to fresh, irregularly banded light and dark grey, calcareous limestone, with clay seams at about elevation 235.0 ft. and elevation 232.0 ft. The per cent core recovery and Rock Quality Designation (RQD) of the core extracted from Borehole 1 were, on average, 93 per cent and 30 per cent respectively. In Borehole 2, the per cent recovery and RQD values were, on average, 100 per cent and 56 per cent, respectively.

[1]Standard Penetration Resistance - see Explanation of Terms, Appendix B.

Details of the creek bed material, sampled at the location shown on Sheet 1, are given on the Record of Test Pit 3. A deposit of brown sand and gravel, containing some silt was encountered to a depth of 0.5 ft. below the creek bed.

In Borehole 1, a filtered standpipe was sealed into the sandy silt till stratum. On February 26, 1980 the water level in the piezometer was 246.3 ft., or about 2.4 ft. below ground surface at that location. On February 26, 1980 the water level in the open hole of Borehole 2 was at elevation 247.2. These water levels indicate that at the time of the investigation groundwater level was at or slightly above the elevation of water in Bowmanville Creek.

5. DISCUSSION AND RECOMMENDATIONS

5.1 Bridge Foundations

5.1.1 Spread Footings

The bridge abutments may be founded on shallow spread footings placed in compact to dense sandy silt to silty sand till. An allowable bearing pressure of 6 ksf may be used for the design of footings placed on this stratum below approximately elevation 243 ft. at the location of Borehole 1 and elevation 240 ft. at Borehole 2. Based on the available data, it is anticipated that the total settlement of footings designed using this allowable bearing pressure and founded on undisturbed, compact to very dense till will be relatively small; generally less than about 1/2 in. However, if a rigid frame type structure is adopted, differential settlements may exceed the tolerable structural limits and spread footings may not be adequate.

As an alternative to founding the proposed structure in the till, consideration should be given to carrying the footings to rock which underlies the site at about elevation 237 to 238. In this case, an allowable bearing pressure of as much as 20 ksf may be used for design and total and differential settlments should be negligible.

In order to allow construction of footings under "dry" conditions, a suitable river diversion will be required to prevent entry of surface (river) water into the excavation. Some groundwater seepage may also be expected to enter excavations carried through the pervious alluvial materials. This will require suitable shoring of the excavation, and control of groundwater inflow by a method such as pumping from sumps. In this regard, it should be noted that the glacial till material is susceptible to disturbance due to construction activities, particularly in the presence of water.

To ensure that the footings are placed on competent, undisturbed material, it is recommended that the base of all footing excavations be inspected by a qualified geotechnical engineer immediately prior to placement of concrete.

5.1.2 Pile Foundation

As an alternative to spread footings, consideration could be given to founding the proposed bridge on short driven piles, end-bearing on the limestone bedrock which underlies the site at about elevation 237 to 238. In particular, pile foundations may be the best alternative if a perched or spill through type of abutment is to be constructed. It is understood that for practical reasons, a minimum pile length of 10 ft. will govern whether spread footings or pile foundations will be chosen.

If pile foundations are to be constructed, conventional steel H-piles, such as 12 BP53 pile section driven to a set of 20 blows/in. on or in the upper portion of the rock using a driving energy of at least 22,500 ft.-lb per blow may be designed using an allowable load of as much as 90 tons per pile. Settlement of such piles should be within tolerable limits for conventional structures.

5.2 Bridge Abutments

It is understood that the proposed abutments will act as earth retaining structures to support the approximately 15 ft. high railway approach embankments. The lateral earth loads on the retaining walls will depend on the type and method of placement of the fill materials.

The following recommendations are made in respect to the design of the abutment retaining walls:

(i) Selected granular fill, such as M.T.C. Granular 'B' should be used as backfill immediately behind the structures. The granular fill should be placed in the wedge-shaped zone defined by a 45 degree line extending up and back from the rear face of the structure;

(ii) All granular fill should be compacted in thin lifts to 95 per cent of the standard Proctor dry density of the material. However, heavy compaction equipment should not be used behind any structure within a lateral distance equal to the current height of the fill above the base of the structure;

(iii) Provided that the above criteria are satisfied, a coefficient of active earth pressure K_a of 0.3 may be used in computing lateral earth pressures, if an outward deflection of approximately 1/2 per cent of the wall height can be tolerated. If no outward wall deflection may be allowed, then an at-rest coefficient of earth pressure, K_o, equal to 0.5, should be used in calculating the lateral earth pressures. A coefficient of friction less than or equal to 0.55 may be assumed between the concrete footings and the sandy silt to silty sand till or the bedrock. A bulk unit weight of 130 lb/cu.ft. may be assumed for the Granular 'B' backfill.

(iv) An adequate drainage system should be provided behind the abutments to prevent build-up of hydrostatic forces. The drainage system should include a properly designed filter to prevent clogging of the pipes. Provision should be made to allow cleaning or rodding of the pipes, should they become clogged.

5.3 Channel Lining

If the bridge abutments are founded on bedrock, a channel lining will not be necessary. For abutments founded on till, it is considered that the glacial till and the more compact portion of the lacustrine sands will provide a suitable foundation material for a separate concrete slab-type channel lining beneath the bridge. However, the very loose lacustrine sands (e.g. Borehole 2, Sample 2) and any alluvium, fill or other deleterious materials should be subexcavated and replaced with select, compacted granular material. Adequate protection, in the form of a cut-off wall into the underlying soil should be provided to prevent scour and seepage beneath the concrete liner.

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J. B. DAVIS
PROVINCE OF ONTARIO

DWH:JBD:ee
801-1046

APPENDIX A

INVESTIGATION PROCEDURE AND LABORATORY TESTING PROGRAM

March, 1980

801-1046

INVESTIGATION PROCEDURE

The field work for this investigation was carried out on February 20 and 22, 1980 when two boreholes and one manual test pit were put down. Boreholes 1 and 2 were put down to depths of 20.8 ft. and 22.8 ft., respectively, at the locations shown on the attached Sheet 1. The boreholes were advanced using a track mounted CME 75 power auger supplied and operated by Atcost Drilling Inc., a specialist drilling contractor. In each boring, standard penetration tests were carried out at 2.5 ft. intervals of depth, using a standard 2 in. O.D. split spoon sampler, which was advanced by a 140 lb. weight falling freely for 30 in. Details of the drilling and sampling operations are summarized on the Record of Borehole sheets. A filtered standpipe was sealed into Borehole 1 to allow monitoring of the groundwater level at that location.

A sample was obtained from the creek bed at the location (designated Test Pit 3) shown on Sheet 1, using a split spoon sample pushed manually into the creek bed.

The field work was supervised throughout by a member of our engineering staff, who located the borings in the field, directed the drilling and sampling operations, and logged the borings. The borehole locations and elevation of ground surface at each boring were determined by survey personnel from the Ministry of Transportation and Communications and supplied to us on March 5, 1980. It is understood that all ground surface elevations are referred to Geodetic datum.

LABORATORY TESTING PROGRAM

Following field identification and logging, all samples obtained during the investigation were placed in air-tight containers and brought to our laboratory where they were examined in detail by the project engineer and classified by visual and tactile methods. Samples for laboratory testing were selected on the basis of the classification and the requirements of the project. The results of grain size distribution determinations performed on selected samples are presented on Figures 1 and 2. The natural water content of each soil sample was determined and is plotted on the Record of Borehole sheets.

APPENDIX B

EXPLANATION OF TERMS

ABBREVIATIONS AND SYMBOLS

March, 1980 801-1046

Golder Associates

EXPLANATION OF TERMS USED IN REPORT

'N' VALUE: AN INDICATOR OF SUBSOIL QUALITY. IT IS OBTAINED FROM THE STANDARD PENETRATION TEST (CSA STD. A119.1). SPT 'N' VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 2 INCH O.D. SPLIT-BARREL SAMPLER TO PENETRATE 12 INCHES INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WEIGHING 140 POUNDS, FALLING FREELY A DISTANCE OF 30 INCHES. FOR PENETRATIONS OF LESS THAN 12 INCHES 'N' VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. 'N' VALUES CORRECTED FOR OVERBURDEN PRESSURE ARE DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST (CSA STD. A119.3): CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (2" O.D. 60 CONE ANGLE) DRIVEN BY 350 FT-LB IMPACTS ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 12 INCH ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOIL QUALITY: SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSITY.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH AS FOLLOWS:

S_u (PSF)	0 - 250	250 - 500	500 - 1000	1000 - 2000	2000 - 4000	> 4000
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF SPT 'N' VALUES AS FOLLOWS:

'N' (BLOW/FT)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCK QUALITY: ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH DRILLED IN THAT CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE NATURALLY FRACTURED CORE PIECES, 4"+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	2"	2" - 12"	1' - 3'	3' - 10'	> 10'
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS & SYMBOLS

LABORATORY TESTING

TRIAXIAL TESTS ARE DESCRIBED IN TERMS OF WHETHER
THEY ARE CONSOLIDATED (C) OR NOT (U)
ISOTROPICALLY (I) OR NOT (A)
AND SHEARED DRAINED (D) OR UNDRAINED (U)
WITH PORE PRESSURE MEASUREMENTS (BAR OVER SYMBOLS)
EG. $\overline{CIU}$ = CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENT
UNLESS OTHERWISE SPECIFIED IN REPORT ALL TESTS ARE IN COMPRESSION

FIELD SAMPLING

- S S — SPLIT SPOON
- W S — WASH SAMPLE
- S T — SLOTTED TUBE SAMPLE
- B S — BLOCK SAMPLE
- C S — CHUNK SAMPLE
- T W — THINWALL OPEN
- T P — THINWALL PISTON
- O S — OSTERBERG SAMPLE
- F S — FOIL SAMPLE
- R C — ROCK CORE
- P H — T.W. ADVANCED HYDRAULICALLY
- P M — T.W. ADVANCED MANUALLY

EARTH PRESSURE TERMS

- μ — COEFFICIENT OF FRICTION
- δ — ANGLE OF WALL FRICTION
- k_o — COEFFICIENT OF EARTH PRESSURE AT REST
- k_A — COEFFICIENT OF ACTIVE EARTH PRESSURE
- k_p — COEFFICIENT OF PASSIVE EARTH PRESSURE
- i — ANGLE OF INCLINATION OF SURCHARGE
- ω — SLOPE ANGLE-BACKFACE OF WALL
- β — ANGLE OF SLOPE
- N_γ, N_q, N_c — BEARING CAPACITY FACTORS
- D_f — DEPTH OF FOOTING
- B,L — FOOTING DIMENSIONS

INDEX PROPERTIES

- γ — UNIT WEIGHT OF SOIL (BULK DENSITY)
- γ_w — UNIT WEIGHT OF WATER
- γ_d — UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
- γ' — UNIT WEIGHT OF SUBMERGED SOIL
- G_s — SPECIFIC GRAVITY OF SOLIDS
- e — VOIDS RATIO
- e_o — INITIAL VOIDS RATIO
- e_{max} — e IN LOOSEST STATE
- e_{min} — e IN DENSEST STATE
- D_r — RELATIVE DENSITY = $\frac{e_{max} - e}{e_{max} - e_{min}}$
- n — POROSITY
- w — WATER CONTENT
- w_L — LIQUID LIMIT
- w_p — PLASTIC LIMIT
- w_s — SHRINKAGE LIMIT
- I_p — PLASTICITY INDEX = $w_L - w_p$
- I_L — LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
- I_c — CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
- A_c — ACTIVITY = $\frac{I_p \text{ of soil}}{\% < 2\mu m \text{ Soil Fraction}}$
- Om — ORGANIC MATTER CONTENT
- S_r — DEGREE OF SATURATION
- S — SENSITIVITY = $\frac{S_u \text{ (undisturbed)}}{S_u \text{ (remoulded)}}$

STRENGTH PARAMETERS

- ϕ — ANGLE OF SHEARING RESISTANCE
- τ_f — PEAK SHEAR STRENGTH
- τ_R — RESIDUAL SHEAR STRENGTH
- c — COHESION INTERCEPT
- $\sigma_1, \sigma_2, \sigma_3$ — NORMAL PRINCIPAL STRESSES
- u — PORE WATER PRESSURE
- u_e — EXCESS u
- r_u — PORE PRESSURE RATIO
- q_u — UNCONFINED COMPRESSIVE STRENGTH
- s_u — UNDRAINED SHEAR STRENGTH
- ϵ — LINEAR STRAIN
- γ — SHEAR STRAIN
- ν — POISSON'S RATIO
- E — MODULUS OF ELASTICITY
- G — MODULUS OF SHEAR DEFORMATION
- k_s — MODULUS OF SUBGRADE REACTION
- m,n — STABILITY COEFFICIENTS
- A,B — PORE PRESSURE COEFFICIENTS

NOTE: EFFECTIVE STRESS PARAMETERS ARE DENOTED BY USE OF APOSTROPHE ABOVE THE SYMBOL, THUS:
ϕ' = EFFECTIVE ANGLE OF SHEARING RESISTANCE;
σ' = EFFECTIVE NORMAL STRESS

HYDRAULIC TERMS

- h — HYDRAULIC HEAD OR POTENTIAL
- q — RATE OF DISCHARGE
- v — VELOCITY OF FLOW
- i — HYDRAULIC GRADIENT
- j — SEEPAGE FORCE PER UNIT VOLUME
- η — COEFFICIENT OF VISCOSITY
- k — COEFFICIENT OF HYDRAULIC CONDUCTIVITY
- k_h — k IN HORIZONTAL DIRECTION
- k_v — k IN VERTICAL DIRECTION
- m_v — COEFFICIENT OF VOLUME CHANGE
- c_v — COEFFICIENT OF CONSOLIDATION
- C_c — COMPRESSION INDEX
- C_r — RECOMPRESSION INDEX
- d — DRAINAGE PATH DISTANCE
- T_v — TIME FACTOR
- U — DEGREE OF CONSOLIDATION
- O_r — OVERCONSOLIDATION RATIO (OCR)

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1

W P 59 - 75 - 11 LOCATION CO-ORDS 15,951,060 N ; 1,216,579 E. ORIGINATED BY K.E.S.

DIST 7 HWY 401 BOREHOLE TYPE HOLLOW STEM AUGERS, BXL ROCK CORE COMPILED BY D.M.

DATUM GEODETIC DATE FEBRUARY 22, 1980 CHECKED BY DWG

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
248.7	GROUND SURFACE							
0.5	Topsoil Silty Sand, some gravel, some organic matter Compact grey-brown	1	SS	10	CAVED SEAL	245	O	
243.7 5.0	Sandy Silt, some clay, gravel and cobbles. (TILL) Compact to very dense grey	2	SS	26	CAVED		O	
		3	SS	100		240	O	14 34 39 13
237.9		4	SS	50/3"			O	
10.8	Limestone Bedrock highly weathered to fresh irregularly banded light and dark grey Calcareous Limestone 0.5ft. clay seams at elev. 235.0 and elev. 232.0	5	RC BXL	92%		235		RQD 24%
		6	RC BXL	94%		230		RQD 36%
20.8	END OF BOREHOLE					225		W.L. elev. 246.3 on Feb. 26/80

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ

$+^3, \times^5$: Numbers refer to Sensitivity

15 -○- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 2

W P 59-75-11 LOCATION CO-ORDS: 15,951,082 N; 1,216,636 E. ORIGINATED BY K.E.S.
DIST 7 HWY 401 BOREHOLE TYPE HOLLOW STEM AUGER, BXL ROCK CORE COMPILED BY D.M.
DATUM GEODETIC DATE FEBRUARY 20, 1980 CHECKED BY DWH

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
249.8	GROUND SURFACE							
	TOPSOIL							
0.5	Silty sand, trace clay, some organic matter	1	SS	3	FEB. 26/80		≈ 28	
	Dark Brown					245		
	Very loose mottled grey-brown and reddish-brown	2	SS	1			≈ 23	0 59 40 1
242.3 / 7.5	Silty Sand to Sandy Silt some clay, gravel and cobbles (Till) Brown	3	SS	12			≈ 14	
	Grey					240		
237.5	Compact	4	SS	23			≈ 8	
12.3	Limestone Bedrock highly weathered to fresh irregularly banded light and dark grey calcareous Limestone	5	RC BXL	100%		235		RQD 58%
	0.5 ft. clay seam at elev. 233.0 ft.	6	RC BXL	100%		230		RQD 55%
								W/L. at elev. 247.3 on Feb. 26/80
22.8	END OF BOREHOLE					225		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ

$+^3$, $\times^5$: Numbers refer to Sensitivity   15 ◇ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF TEST PIT 3

LOCATION: See Figure 59711-A

EXCAVATION DATE: February 20, 1980

TEST PIT TYPE: Manual

GROUND SURFACE ELEVATION: 244 (Approx.)

TEST PIT SIZE: 2 in. O.D. Split Spoon

DATUM: Geodetic

DEPTH (ft.)	DESCRIPTION	SAMPLE DEPTH (ft.)	WATER CONTENT (per cent)	REMARKS
0.0 - 0.5	Brown SAND and GRAVEL, some silt	0.0 - 0.3		Water level in creek at about 0.5 ft. above creek bed with ice cover to about 2.0 ft. above creek bed. Split Spoon (1 5/8 in. I.D.) unable to sample coarse gravel or cobbles.

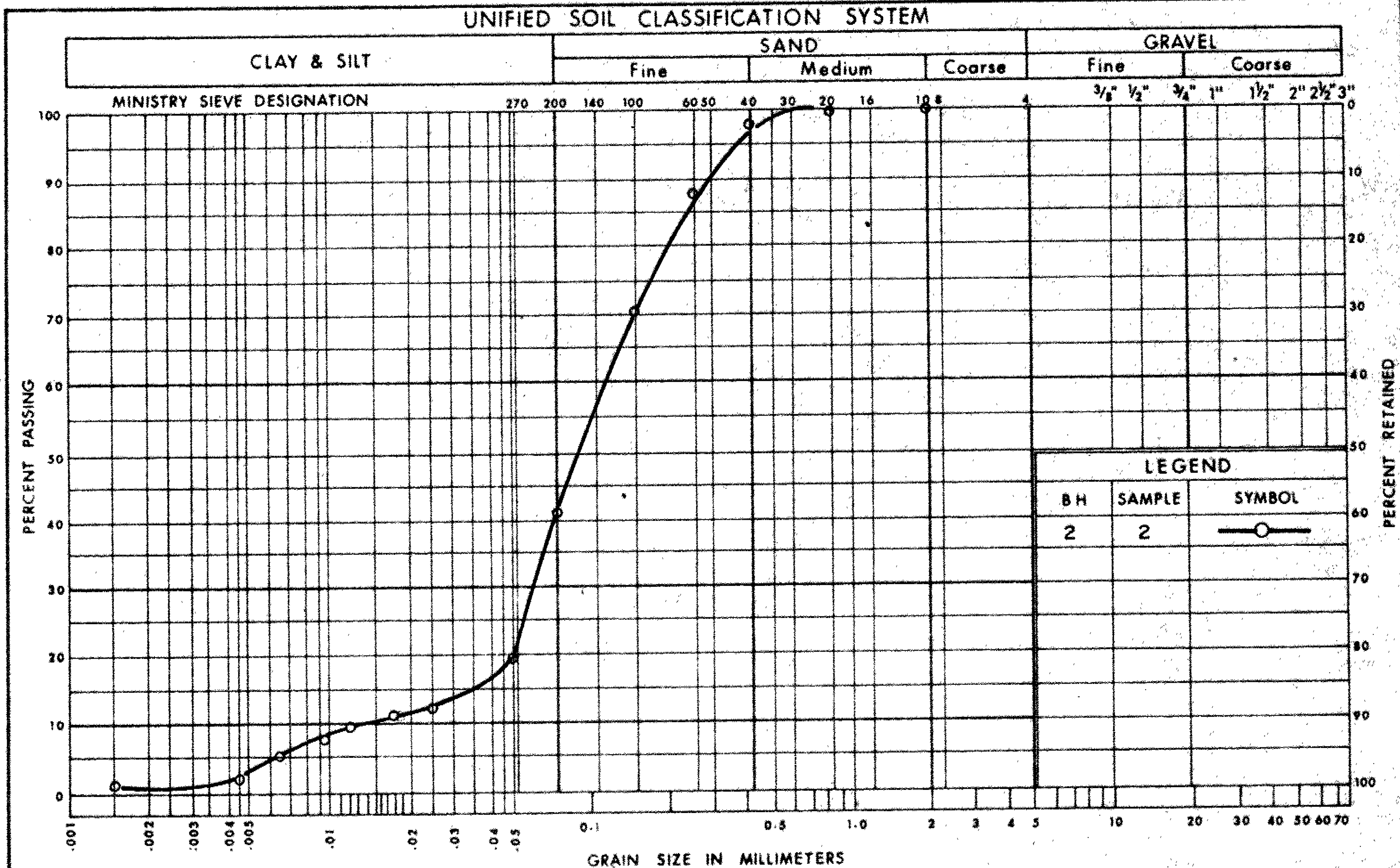
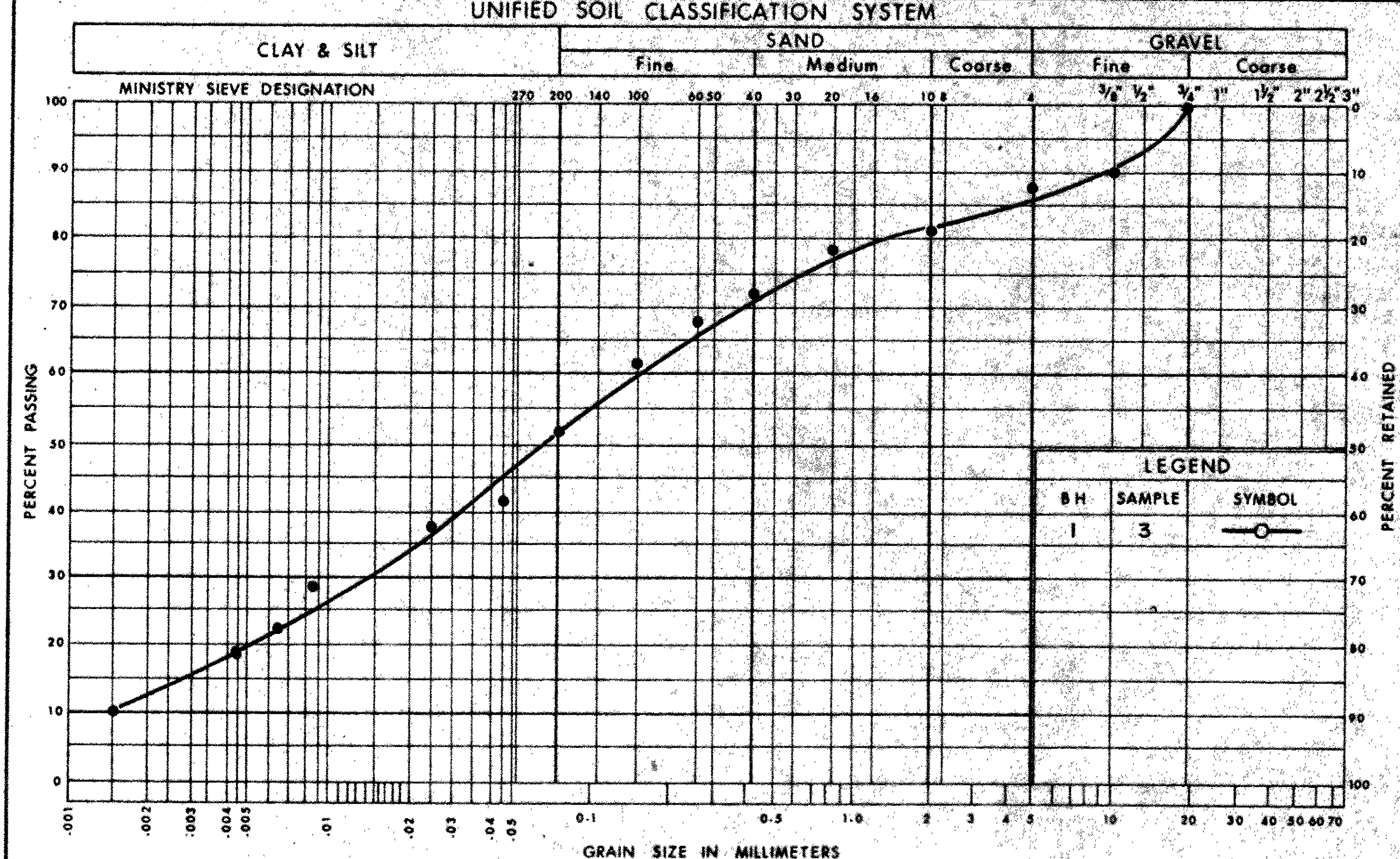


FIG No	1
W P	59-75-11

Ministry of Transportation and Communications
Ontario

GRAIN SIZE DISTRIBUTION
SILTY FINE SAND

Oct 75, FF-S-22



Ministry of Transportation and Communications
Ontario

GRAIN SIZE DISTRIBUTION
SANDY SILT (TILL)

FIG No 2
W P 59-75-11

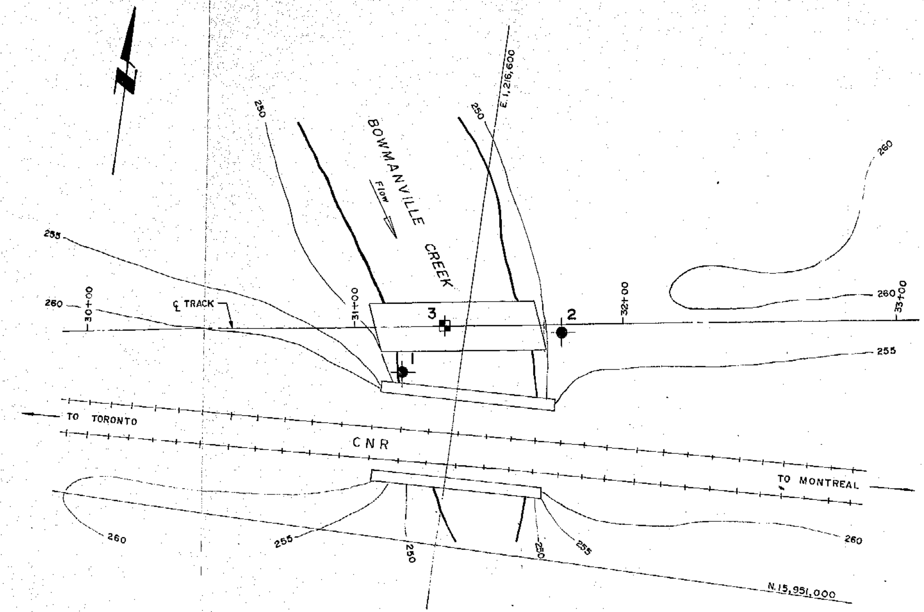
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

CONT No
WP No 59-75-11

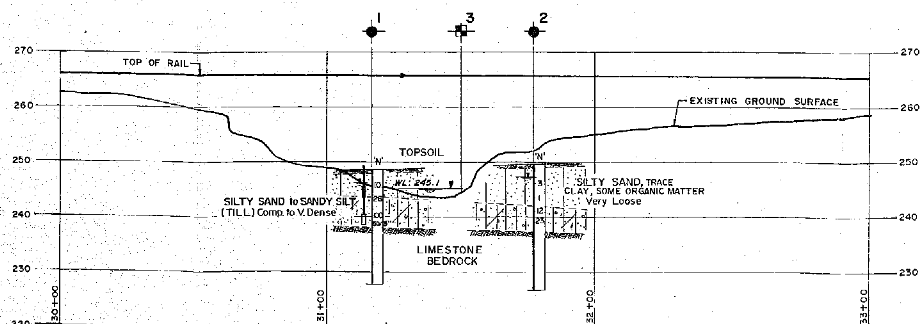
BOWMANVILLE CREEK
(600 FEET SOUTH OF HIGHWAY 401)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET 1 of 1

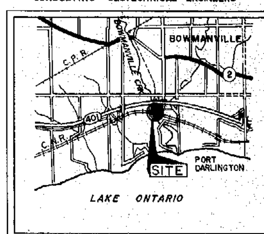
GOLDER ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS



PLAN
SCALE
20 10 0 20 40 FT.



PROFILE
SCALE
HOR 20 10 0 20 40 FT
VERT 10 5 0 10 20 FT



KEY PLAN
0.5 0 1 2 3 Miles

LEGEND

Bore Hole
Dynamic Cone Penetration Test (Cone)
Bore Hole & Cone
'N' Blows/ft (Std Pen Test 350 ft lbs energy)
CONE Blows/ft (60° Cone, 350 ft lbs energy)
WL at time of investigation FEB. 1980
TEST PIT
PIEZOMETER

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	248.7	15,951,060	1,216,579
2	249.8	15,951,082	1,216,637

30M15-55
GEOCRES No.

-NOTE-
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS		
DATE	BY	DESCRIPTION

30M15-55
GEOCRES No.

REF. No. E 5466-1; JAN. 1980

HWY No. 401			DIST 7
SUBM'D	CHECKED	DATE MAR. 6, 1980	SITE
DRAWN DM	CHECKED DMK	APPROVED	DWG 59-75-11