

GEOCRES No. 30M15-87DIST. 7 REGION W.P. No. 59-75-10CONT. No. W. O. No. STR. SITE No. HWY. No. 401LOCATION CNR BRIDGE, North of Hwy 401CROSSING BOWMANVILLE CREEKNo of PAGES - —=====
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:



Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

REPORT

TO

MINISTRY OF TRANSPORTATION
AND COMMUNICATIONS

SUBSURFACE INVESTIGATION
PROPOSED C.N.R. BOWMANVILLE CREEK BRIDGE
NORTH OF HIGHWAY 401
W.P. 59 75-10 DISTRICT 7

NEWCASTLE

ONTARIO

GEOCRES No 30M15-87

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March, 1980

801-1047

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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 north of Highway 401 in Newcastle, Ontario.

In summary, the borings indicate that loose lacustrine deposits overlies a compact to very dense sandy silt till stratum which overlies limestone bedrock at a relatively shallow depth. Samples from the creek bed indicate that the bed material of Bowmanville Creek at the proposed bridge location and at the existing Highway 401 bridge, consists of sand and gravel, containing some silt and cobbles overlying a silty sand till stratum. 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 spread footings founded on the 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

Golder Associates have been retained by the Ministry of Transportation and Communication of Ontario (letter dated February 29, 1980) to carry out a foundation investigation for a proposed C.N.R. bridge over Bowmanville Creek to the north of Highway 401 (W.P. 59-75-10) near Bowmanville, Ontario. This project is part of the Highway 401 widening project from Waverly Road to Highways 35 and 115.

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. Plane E 5465-1, "Bridge Site Plan, Proposed Crossing at Canadian National Railways and Bowmanville Creek north of Highway 401", dated January 1980.

We understand that the proposed structure will carry a C.N.R. spur line over Bowmanville Creek, approximately 220 ft. to the north of Highway 401. We further understand that the bridge is proposed to be a single, 90 ft. span reinforced structure.

3. SITE AND GEOLOGY

The site is located in the Town of Newcastle, Ontario. The ground surface is generally flat-lying, except that Bowmanville Creek flows in a shallow valley that is about 5 ft. below the surrounding area at the proposed bridge location.

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 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 and test pits together with the results of laboratory tests carried out on representative samples of the soil strata are given on the attached Record of Borehole and Record of Test Pit 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, test pit sheets 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.

*The boreholes and test pits pertaining to this investigation are numbered sequentially from 4 through 8. Boreholes and Test Pits 1 to 3 were put down during a concurrent investigation at a similar bridge crossing to the south of Highway 401 (Golder Associates report no. 801-1046, dated March 1980).

In general, the ground surface at the site is underlain by about 0.5 ft. of topsoil, which overlies a stratum of very loose lacustrine silty sand, containing some organic matter (Figure 1). This stratum was encountered to a depth of 7.5 ft. and 5.5 ft. below ground surface in Boreholes 5 and 6, respectively. In Borehole 5, the lacustrine deposit was found to have an 'N' value of 3 to 4 blows/ft. and an average natural water content of 32 per cent, while in Borehole 6, an 'N' value of 2 blows/ft. and water contents of 40 to 45 per cent were recorded.

Underlying the upper lacustrine deposit in each of Boreholes 5 and 6 is a relatively thin (1.5 ft.) stratum of loose to compact grey-brown sand and gravel. The natural water content of a sample of this material was 16 per cent.

Underlying the sand and gravel deposit a stratum of compact to very dense grey sandy silt till, containing some clay and gravel was encountered. Although no cobbles or boulders were encountered in the two borings, it is considered that they may occur in the till stratum, due to the glacial origin of the material. The water content of samples of the till was less than 10 per cent.

At elevation 234.9 in Borehole 5 and 239.8 in Borehole 6, practical refusal to advance of the hollow stem augers was encountered, at which time BXL rock core was obtained for lengths of 11.3 and 9.9 ft., respectively. The bedrock was found to be highly weathered to fresh, irregularly banded light and dark grey, calcareous limestone. A highly fractured zone was noted at between about elevation 233.5 and 233.0 ft. in Borehole 6. The per cent core recovery and Rock Quality Designation (RQD) of the core extracted from Borehole 5 were 100 per cent and from 21 to 50 per cent, respectively. In

*Standard Penetration Resistance - see Explanation of Terms, Appendix B.

Borehole 6, the per cent recovery and RQD values were, on average, 90 per cent and 47 per cent, respectively.

The creek bed material at the location of the proposed bridge was sampled manually (refer to Sheet 1 and Record of Test Pit 7). A layer of brown sand and gravel containing some silt and cobbles (Figure 2) was encountered to a depth of 1.0 ft. below the creek bed.

Two backhoe dug test pits (nos. 4 and 8) were put down in the creek bed beneath the existing Highway 401 bridge over Bowmanville Creek (Sheet 1). In both test pits a deposit of brown sand and gravel, containing some silt and cobbles (Figure 2) and occasional layers of silty fine sand was encountered to a depth of 4.0 ft. (Test Pit 4) and 3.5 ft. (Test Pit 8) below the creek bed. The gravel and cobble size particles of this fluvial deposit are rounded and subrounded. Between a depth of 4.0 and 5.0 ft. in Test Pit 4 and between 3.5 and 5.0 ft. in Test Pit 8, a grey silty sand till containing some clay and gravel (Figure 3) was encountered.

In Borehole 6, a filtered standpipe was sealed into the sandy silt till stratum. On February 26, 1980 the water level in the standpipe was at elevation 248.0 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 5 was at elevation 249.6 ft. 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 till. An

allowable bearing pressure of 6 ksf may be used for the design of footings placed on this stratum below approximately elevation 241 ft. at the location of each borehole. At the location of Borehole 6, bedrock was encountered at elevation 239.0. Therefore, the spread footing elevation given above (241 ft.) is 1.2 ft. above the surface of bedrock. Provided that the sandy silt till is not loosened during construction, it is not necessary to subexcavate this material and found on the bedrock. 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 235 to 240. In this case, an allowable bearing pressure of as much as 20 ksf may be used for design and total and differential settlements 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 existing 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 Foundations

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 235 to 240. 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 12BP53 pile section, driven to a set of at least 20 blows/in. on or in the upper portion of the rock using a rated 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 20 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 equal to 0.55 may be assumed between the concrete footings and the sandy silt 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 sand or fluvial sand and gravel, will provide a suitable foundation material for a separate concrete slab-type channel lining beneath the bridge. However, any loose lacustrine sand, 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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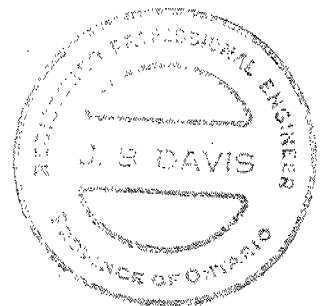
D. W. Hubble

D. W. Hubble

J. B. Davis

J. B. Davis, P. Eng.

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APPENDIX A

INVESTIGATION PROCEDURE
AND LABORATORY TESTING PROGRAM

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INVESTIGATION PROCEDURE

The field work for this investigation was carried out on February 20, 21, 22 and 26, 1980 when two boreholes, one manually dug and two backhoe dug test pits were put down (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. Boreholes 5 and 6 were put down to depths of 27.1 ft. and 20.5 ft., respectively. 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 6 to allow monitoring the groundwater level at that location. Since it was not possible to put down a borehole in the creek bed (due to the depth of water and thin ice cover) a sample was obtained from the creek bed at the location (designated Test Pit 7) shown on Sheet 1, using a shovel.

Similarly, a proposed borehole in the creek bed beneath the existing Highway 401 bridge over Bowmanville Creek was not possible. Therefore, on February 26, 1980, two test pits were put down beneath the Highway 401 bridge (Sheet 1) using a rubber tired backhoe supplied and operated by Robinson Quinney Ltd., a local contractor.

On February 26, 1980 stabilized water level readings were taken in the filtered standpipe installed in Borehole 6 and in the open hole of Borehole 5.

The field work was supervised throughout by a member of our engineering staff, who located the borings and test pits

in the field, directed the drilling, test pitting and sampling operations, and logged the borings and test pits. 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. The test pit locations were measured in the field by our staff. The approximate elevation of the ground surface at each test pit location was obtained from the contour information on Plan 5465-1.

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. Selected samples were then submitted for laboratory testing, based on the requirements of the project.

Grain size distribution determinations were performed on selected samples and the results are presented on Figures 1 to 3. The natural water content of selected soil samples is given on the Record of Borehole and Record of Test Pit sheets.

APPENDIX B

EXPLANATION OF TERMS
ABBREVIATIONS AND SYMBOLS

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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 N_c .

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. $\bar{C}IU$ = 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
 w SLOPE ANGLE-BACKFACE OF WALL
 β ANGLE OF SLOPE
 N_c, N_q, N_γ 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 = $\frac{I_p \text{ of soil}}{2 \mu m \text{ Soil Fraction}}$
 O_m 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_n MODULUS OF SUBGRADE REACTION
 m, n STABILITY COEFFICIENTS
A, B PORE PRESSURE COEFFICIENTS

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_c OVERCONSOLIDATION RATIO (OCR)

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



RECORD OF BOREHOLE No 5

W P 59-75-10 LOCATION CO-ORDS. 15,951,991 N; 1,216,073 E. ORIGINATED BY K.E.S.
DIST 7 HWY 401 BOREHOLE TYPE HOLLOWSTEM AUGERS, BXL ROCK CORE COMPILED BY D.M.
DATUM GEODETIC DATE FEBRUARY 21, 1980 CHECKED BY D.W.H.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100										WATER CONTENT (%)	10 20 30
								SHEAR STRENGTH											
251.7	GROUND SURFACE															GR SA SI CL			
0.5	TOPSOIL						250												
	Silty Sand, some organic matter, roots, pieces of wood and fine amorphous organics		1	SS	4	FEB 26/80													
	Very loose grey - brown		2	SS	3		245												
244.2																			
7.5	Sand and Gravel, compact grey - brown		3	SS	27														
242.7																			
9.0	Sandy Silt, some clay and gravel (Till)		4	SS	27		240												
	Compact to dense grey																		
234.9			5	RC BXL	100%		235												
16.8	Limestone Bedrock irregularly banded light and dark grey calcareous Limestone		6	RC BXL	100%		230									RQD 21%			
	Highly to moderately weathered																		
	Slightly weathered to fresh		7	RC BXL	100%											RQD 50%			
224.6							225									W.L. elev. 249.6 on Feb. 26/80			
27.1	END OF BOREHOLE						220												

+³, x⁵: Numbers refer to
Sensitivity

20
15 ◇ 5 (%) STRAIN AT FAILURE
10

RECORD OF TEST PIT 4

LOCATION: See Figure 597510-A

EXCAVATION DATE: February 26, 1980

TEST PIT TYPE: Backhoe Dug

GROUND SURFACE ELEVATION: 246 (Approx.)

TEST PIT SIZE: 2 ft. x 4 ft.

DATUM: Geodetic

DEPTH (ft.)	DESCRIPTION	SAMPLE DEPTH (ft.)	WATER CONTENT (per cent)	REMARKS
0.0 - 4.0	Brown SAND and GRAVEL, some silt and cobbles. Gravel and cobble particles are rounded and sub-rounded. Cobbles are up to 4 in. size	0.0 - 1.0		Water level in creek at about 1.0 ft. above creek bed with ice cover to about 2.0 ft. above creek bed.
		3.0 - 4.0		Some fines (i.e. sand and silt) washed from sample by current. See Figure 2
4.0 - 5.0	Grey SILTY SAND, some clay and gravel (TILL)	4.0 - 5.0	14	See Figure 3

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RECORD OF TEST PIT 7

LOCATION: See Figure 597510-A

EXCAVATION DATE: February 22, 1980

TEST PIT TYPE: Manual (Shovel)

GROUND SURFACE ELEVATION: 247 (Approx.)

TEST PIT SIZE: 1 ft. x 1 ft.

DATUM: Geodetic

DEPTH (ft.)	DESCRIPTION	SAMPLE DEPTH (ft.)	WATER CONTENT (per cent)	REMARKS
0.0 - 1.0	Brown SAND and GRAVEL, some silt and cobbles	0.0 - 1.0		Water level in creek at about 1.5 ft. above creek bed with ice cover to about 2.0 ft. above creek bed. Some fines (i.e sand and silt) washed from shovel sample by current.

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RECORD OF TEST PIT 8

LOCATION: See Figure 597510-A

EXCAVATION DATE: February 26, 1980

TEST PIT TYPE: Backhoe Dug

GROUND SURFACE ELEVATION: 248 (Approx.)

TEST PIT SIZE: 2 ft. x 4 ft.

DATUM: Geodetic

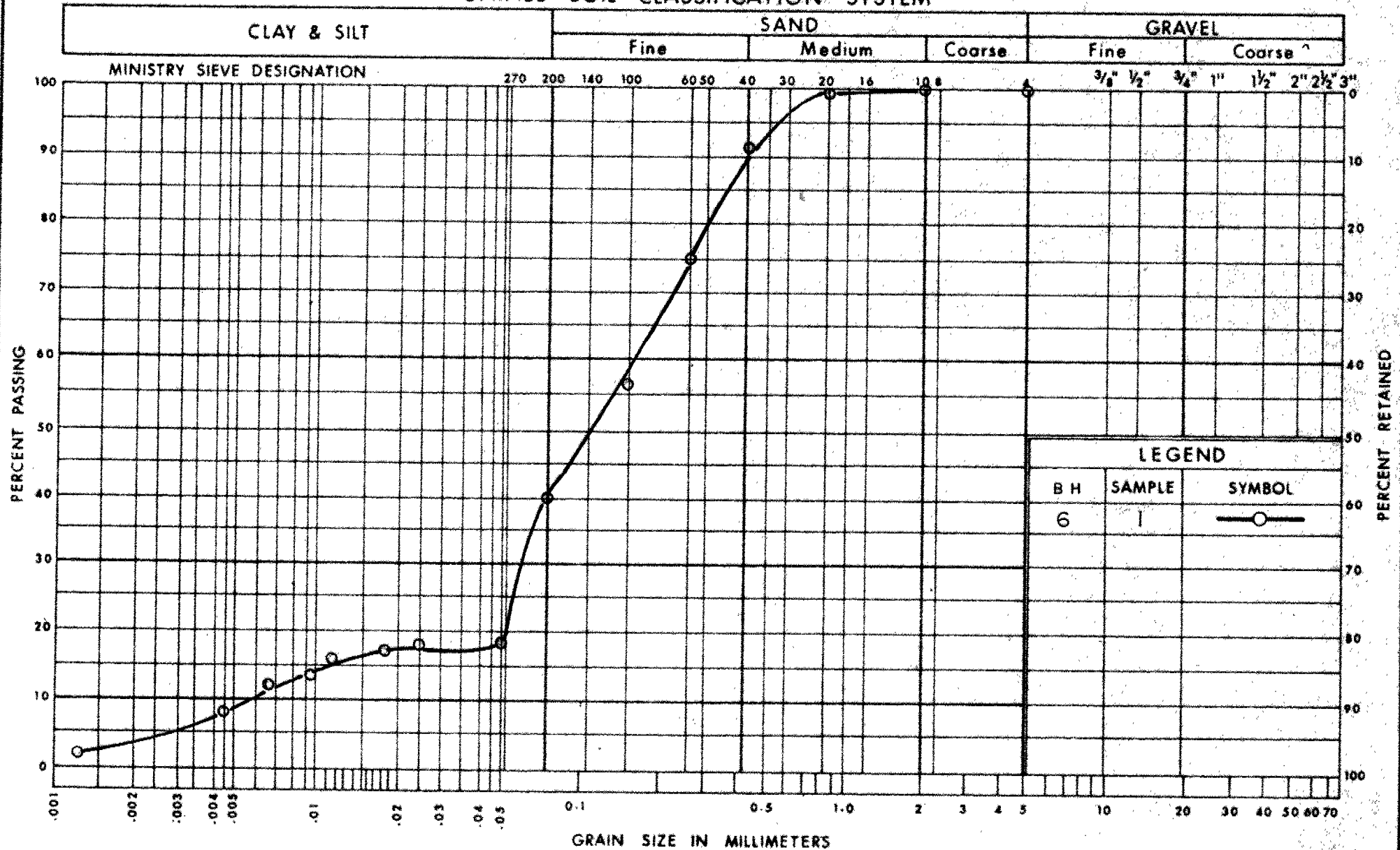
DEPTH (ft.)	DESCRIPTION	SAMPLE DEPTH (ft.)	WATER CONTENT (per cent)	REMARKS
0.0 - 3.5	Brown SAND and GRAVEL, some silt and cobbles, some layers of SILTY FINE SAND	1.5 - 2.0		Water level in creek at about 0.5 ft. above creek bed with ice cover to about 1.5 ft. above creek bed.
		2.1 - 2.5		See Figure 2
3.5 - 5.0	Grey SILTY SAND, some clay and gravel (TILL)	3.8 - 4.5	7	See Figure 3

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UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

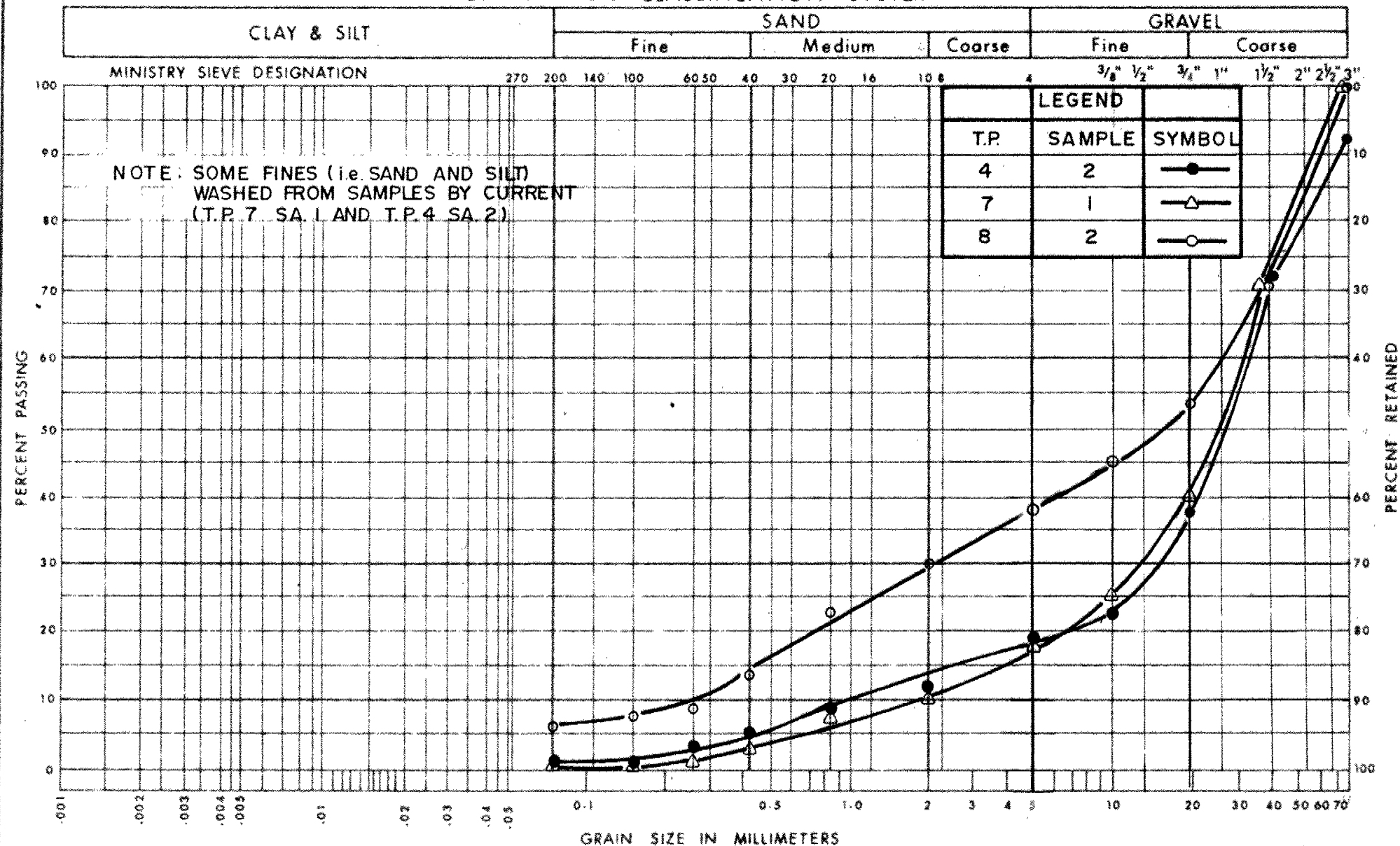
 Ministry of
Transportation and
Communications

 GRAIN SIZE DISTRIBUTION
SILTY SAND

FIG No. I

W P 59-75-10

UNIFIED SOIL CLASSIFICATION SYSTEM



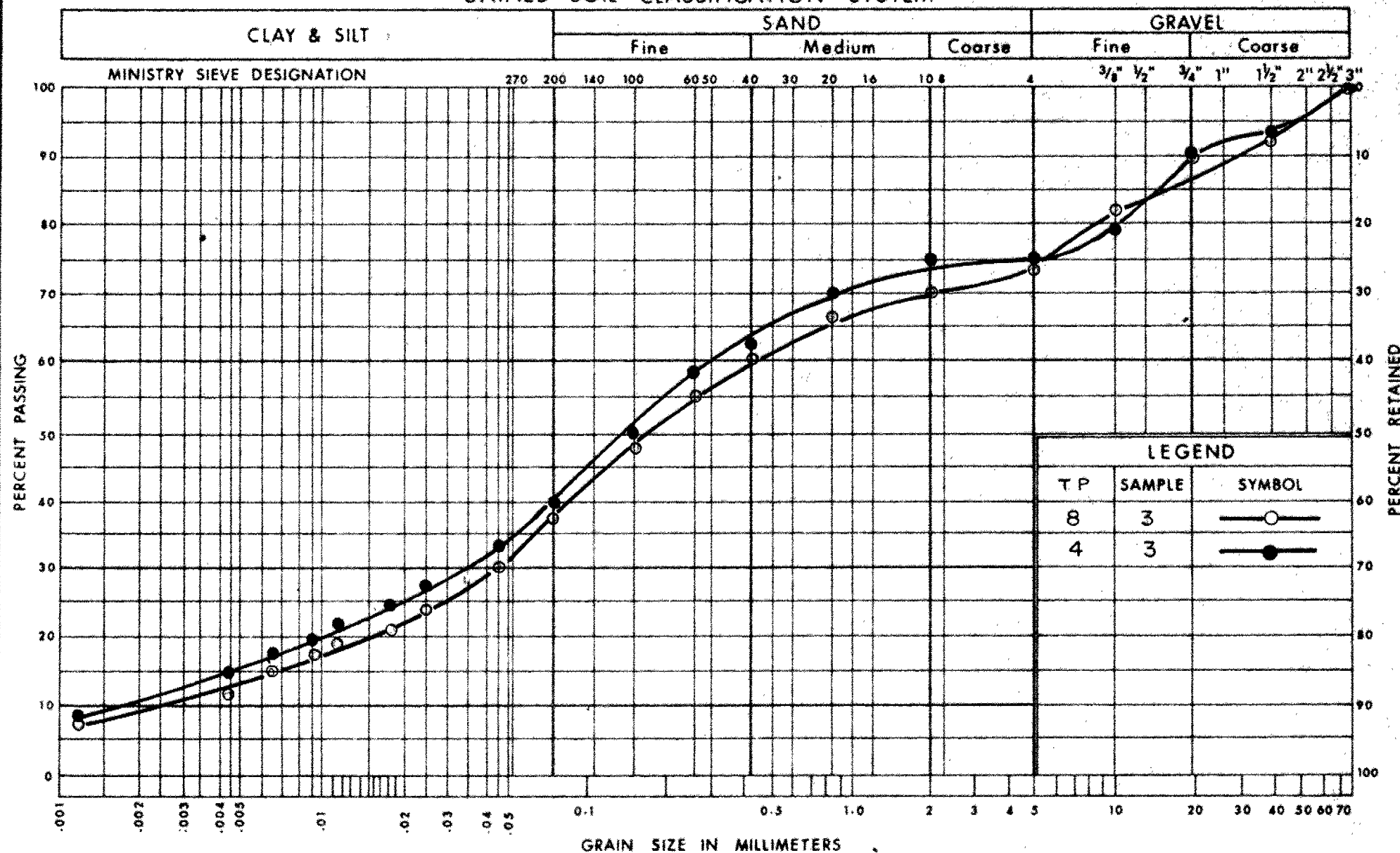
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL

FIG No 2

W P 59 - 75 - 10

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION SILTY SAND TILL

FIG No 3

W P 59 - 75 - 10

memorandum



To: Mr. G.C.E. Burkhardt
Head, Structural Office
Central Region

Date: 1980-04-08

From: Pavement & Foundation Design Section
Room 313, Central Building
Downsview

Subject: Foundation Investigation and Design Report
Proposed C.N.R. Bowmanville Creek Bridge
North of Hwy. 401
W.P. 59-75-10, Dist. #7 (Port Hope)

A subsurface investigation was carried out by Golder Associates, Geotechnical Consultants for the Ministry at the site of a proposed C.N.R. bridge at the above mentioned location in Newcastle, Ontario. The enclosed report contains the factual data together with the engineering recommendations for the design of the foundations and associated earth works for the proposed structure.

Our comments pertaining to structure foundations are as follows:

In our opinion the best alternative is a single span structure with perched or spill through type of abutments supported on piles. For pile foundations, conventional Steel 'H' piles such as 12 BP 74 driven to bedrock may be designed for an allowable load of up to 130 tons/pile.

To ensure that the pile foundations are placed in a 'dry' condition, the base should be located well above the prevailing high water level of the river.

The proposed 20 ft. high approach embankments will be stable with standard 2:1 slopes and constructed with well compacted acceptable earth material.

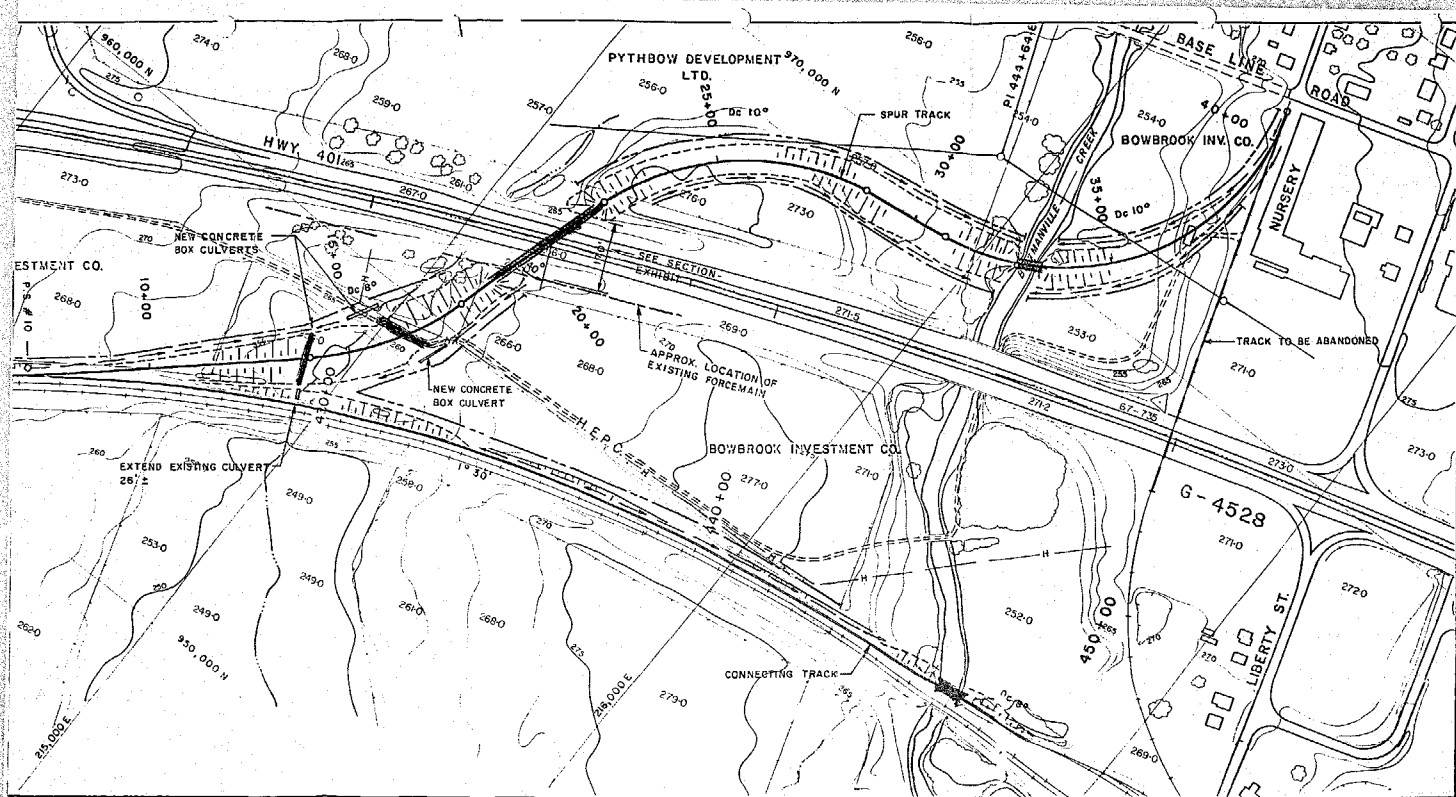
Should you require any further details related to foundation requirements, please contact us.

A handwritten signature in dark ink, appearing to read "M. Devata".

M. Devata
Senior Foundations Engineer

MD:ea

cc: R.D. Gunter
I.V. Oliver
D.E. Thrasher (2)
C. Grebski
B.J. Giroux
R. Hore
R. Fitzgibbon)
J. Anderson) Memo only
✓ T.J. Kovich)

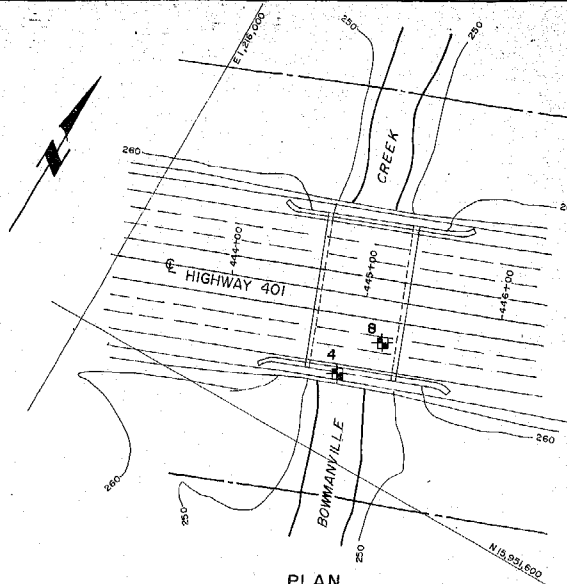
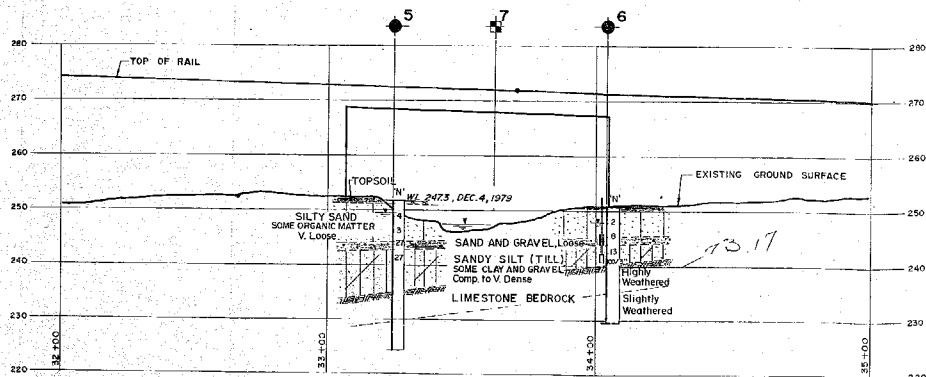
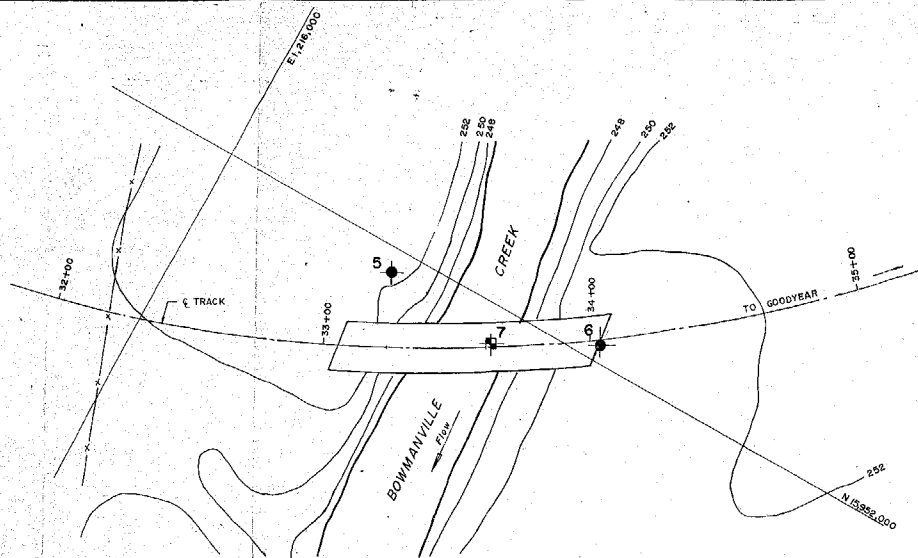


HIGHWAY 401 - OSHAWA
PROPOSED RELOCATION OF C.N.R. SPURLINE



LEGEND:
— TRACK TO BE RETIRED
- - - PROPOSED TRACKAGE
--- R.O.W. FOR CONSTRUCTION

EXHIBIT
2



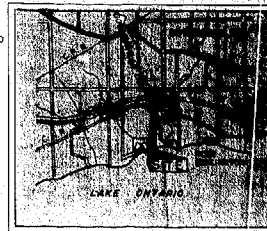
CONT No
WP No 59-75-10

BOWMANVILLE CREEK
(220 FEET NORTH OF HIGHWAY 401)
BORE HOLE LOCATIONS & SOIL STRATA



SHEET
1 of 1

GOLDER ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS



LEGEND

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

No	ELEVATION	CO-ORDINATES NORTH	EAST
5	251.7	15,851,918	1,216,073
6	250.4	15,952,006	1,216,155

-NOTE-

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

DATE	BY	DESCRIPTION

NO. 401	DATE	1980	1
BY	CHECKED	DATE	1980
DATE	1980	1	

REF No. E 5465-1, JAN., 1980