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GEOCRES No. 31C-156DIST. 8 REGION W.P. No. CONT. No. W. O. No. 87-11001STR. SITE No. HWY. No. locLOCATION Sir John A. Mac Donald
Blod & CNR, Grade SeparationNo of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

Peto Smith

6837

Geocres 31C-156

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED OVERPASS SEPARATION
SIR JOHN A. MACDONALD BOULEVARD EXTENSION
KINGSTON, ONTARIO

JAMES NEILSON & ASSOCIATES, INC.

Reference 85-3-S1

March 1985

Prepared for
CORPORATION OF THE CITY OF KINGSTON
C/O TOTTEN SIMS HUBICKI ASSOCIATES (1981) LIMITED
675 BATH ROAD, LASALLE PARK PLAZA
KINGSTON, ONTARIO K7M 4X2

1.0 INTRODUCTION

James Neilson & Associates, Inc. a firm of consulting geotechnical engineers, was retained by Totten Sims Hubicki Associates (1981) Limited, acting in behalf of the Corporation of the City of Kingston, to conduct a preliminary geotechnical investigation for a proposed overpass-type grade separation. The overpass will be located where the proposed extension of Sir John A. MacDonald Blvd. will cross the existing CNR right-of-way in Kingston, Ontario.

The scope of the assignment was defined by Mr. Peter A. Smith, P.Eng., of Totten Sims Hubicki, in a letter request for Proposals dated February 22, 1985, and in subsequent meetings and telephone conversations. The purpose of the preliminary investigation was to provide the subsurface data needed to enable the design consultant to select the appropriate type of overpass structure to be built, together with possible foundation design alternatives. The investigation also would provide information re the extent of soft, compressible soils within a swamp area located north of the existing tracks. This latter information will provide the basis for a tentative culvert design on a small creek located within the proposed development area. The investigation would also reveal the type of subsurface materials which can be expected to support the north approach of the overpass structure.

Our Proposal was submitted on February 26, 1985. Formal acceptance and authorization to proceed with the work was received from the Corporation of the City of Kingston through Purchase Order No. 153158, dated March 12, 1985.

This report contains a factual account of our work; a brief description of the site and its geology; and it presents a profile of the subsurface conditions encountered. Field and laboratory data are interpreted and preliminary recommendations are presented regarding foundation design alternatives for the proposed overpass structure. Geotechnically-related comments are also provided to assist in the design of proposed structures in the swamp area. In addition, suggestions are advanced for future geotechnical investigations, if and where warranted.

2.0 SITE AND GEOLOGY

The project involves an approximately 800 m extension of Sir. John A. MacDonald Blvd. from Terry Fox Drive in the south to Dalton Ave. in the north. A small creek flows through the development area in an east to west direction approximately 200 m south of Dalton Ave. The creek has established a floodplain 100 m wide on both sides of its banks. The CNR right-of-way is situated at the approximate centre of the project site just beyond the south floodplain boundary (i.e., about 400 m south of Dalton Ave.).

The natural topography of the site drops off in elevation from south and north towards the creek. However, the relief has been modified somewhat by the deposition of miscellaneous fill recently placed along the alignment of the proposed extension.

Bedrock is not exposed because the site is located in an eroded stream valley filled with postglacial sediments. It is known, however, that Ordovician limestone common to the Kingston area underlies the site and the limestone, in turn, overlies Precambrian granite and granite gneiss at depth.

The postglacial sediments referred to above were laid down at intervals during the past 10,000 years in the valley eroded in the bedrock. In this area they consist predominantly of silts and clays with more recent organic materials. The area at one time formed an embayment in Glacial Lake Iroquois whose shorelines were several tens of feet higher than the present Lake Ontario and this accounts for the in-filling of the creek valley with clay, silt, and other fine-grained materials.

3.0 METHODOLOGY

The preliminary geotechnical investigation consisted of a subsurface exploratory drilling program followed by limited laboratory testing and analysis of the soil and rock.

3.1 FIELD WORK

Exploratory drilling was conducted during the period March 18-20, 1985, using a track-mounted mobile power auger. Three boreholes were put down at pre-selected locations chosen by the Client as shown on the Borehole Location Plan, Enclosure 1, attached. Boreholes 1 and 2 were put down at the proposed overpass location while

Borehole 3 was situated where it is tentatively planned to locate the new culvert.

Borehole depths ranged from 6.9 m to 19.8 m below the existing ground surface. Collar elevations of the boreholes were established geodetically using the benchmark indicated on the Location Plan. Borehole locations were tied in to the survey stations located on the site by City of Kingston surveyors.

The boreholes were advanced with hollow-stem augers and conventional soil sampling equipment. Both disturbed and undisturbed soil samples were recovered. Disturbed samples were taken at regular depth intervals with a 50 mm O.D. split-spoon sampler. The sampler was driven into the undisturbed soil at the bottom of the borehole with a constant driving energy (a 6.3 kg hammer dropping 750 mm). The number of blows required to advance the sampler 0.3 m yields the Standard Penetration Resistance ('N'-values) of the soil.

Undisturbed samples of cohesive soils were obtained with a 75 mm O.D. thin-walled (Shelby-tube) sampler. For undisturbed samples, the sampler is advanced its full length (700 mm) under a constant downward pressure from the hydraulic head of the power auger.

In addition, undrained shear strength tests were performed, in situ, in cohesive soil by means of a four-bladed torque vane. Both the undisturbed and remoulded strengths (sensitivity) were measured.

The bedrock encountered in the boreholes was explored by coring. The rock was diamond cored for 1.3 to 1.6 m using BX equipment (40 mm diameter core). Core recovery and pressure during coring as well as the approximate quantity of return water were noted. The "rate of recovery" (REC) and the "rock quality designation" (RQD) were calculated. These indices provide a useful measure of bedrock quality and the degree of weathering and fracturing.

The field program was supervised by one of our staff engineers who logged, labelled and stored the samples which were then transferred to our laboratory for further examination and testing. Groundwater levels were monitored throughout the field investigation and for several days following completion of the drilling. Standpipes were installed in the boreholes to permit future monitoring of the groundwater level, as needed.

3.2 LABORATORY WORK

The soil samples taken to our laboratory were re-examined and classified by visual and tactile methods in accordance with the Unified Soil Classification System. A geologist from our staff re-logged the recovered bedrock cores.

Natural water contents and unit weights of representative samples were determined by standardized laboratory methods. In addition, several penetration and compression tests were conducted to determine the undrained shear strength of the cohesive strata. The compressibility

and consolidation characteristics of the subsoil were determined by laboratory consolidation tests. The test results are plotted on the borehole logs and/or are presented as separate Enclosures.

4.0 SUBSURFACE CONDITIONS

4.1 STRATIGRAPHY

The stratigraphy encountered in the boreholes is illustrated on the Borehole Logs comprising Enclosures 2 to 4b. Available information indicates that although the subsurface materials are quite similar in each of the boreholes, the strata depths and thicknesses vary over the site. The geotechnical details of each of the deposits encountered are discussed below.

i) Overpass Structure (Boreholes 1 and 2)

Boreholes 1 and 2 were drilled at the proposed location of the overpass structure. The stratigraphy in both boreholes is quite similar except for the existence of a thin surficial cover of fill encountered in BH2.

The City has reported that selected fill material from external sources was dumped and levelled along the proposed Boulevard extension at various times in the past. This fill material encountered as the surficial deposit in BH 2 extends 0.6 m below the present grade. The fill is comprised of a mixture of sand and gravel, clay, organics, and occasional cobbles and boulders. The presence of wood, limestone rubble, and miscellaneous

construction debris attests to the man-made origin of the deposit. The fill is in a loose, uncompacted condition.

The uppermost natural soil deposit in both boreholes is a 200 mm-thick layer of dark, brown, clayey topsoil. This deposit was encountered immediately below the fill in BH 2 and as the surficial stratum in BH 1.

The predominant soil type below the topsoil in both boreholes is a varved silty clay interlayered with clayey silt (at depth). This stratum is characterized by a banded brown-grey and grey colour; localized seams of silt and fine sand; a silt content increasing with depth; embedded sand and gravel at the base of the deposit; and a slightly fissured and moist to wet condition (W.C. = 25% to 35%). The "varved" structure is a seasonal phenomenon indicative of deposition in a glacial lake.

In BH 1, the varved stratum extends from 0.2 to 6.2 m below the existing ground surface and, in BH 2, the deposit occurs between 0.8 and 5.5 m depths.

With the exception of a thin stiffening zone at the topsoil/clay interface, the upper 1.5 to 1.8 m of the deposit has a very stiff crust. The consistency of this zone is based on in-situ Standard Penetration Resistance ('N'-values) of 16 to 22 blows/0.3 m and field shear strengths and laboratory penetration tests of 215 to 265 kPa. Below the crustal section, the clay exhibits a stiff consistency as indicated by 'N'-values of 6 to 9 blows/0.3 m, and field shear strengths and laboratory compression and penetration tests of 50 to 180 kPa.

Some inconsistencies were experienced in the various test results and these are attributed to the presence of cohesionless material within the stratum.

Sensitivity values (the ratio of undisturbed to remoulded shear strength) of 3.3 to 5.6 indicate that the clay is "medium sensitive" to "sensitive"; that is, the shear strength of the soil may be reduced to a low of 15% - 30% of its original strength if it is disturbed.

The unit weight of the soil deposit was also determined by laboratory tests and found to be relatively consistent with depth (i.e., 19.4 to 20.1 kN/m³).

The compressibility of this cohesive deposit was measured in the laboratory by consolidation tests which were conducted on samples retrieved from 2.5 m and 5.0 m depths in BH 1. The results are presented on Enclosures 5 and 6. These tests indicate that the deposit is overconsolidated, that is, it has carried loads in excess of the present overburden pressure in its past geological history.

Limestone bedrock was encountered below the clay stratum. The bedrock surface was penetrated at approximately Elevation 75.0 m (i.e., 6.2 and 5.5 m below the existing ground surface in BH 1 and BH 2, respectively). The rock was cored in both boreholes for a depth of approximately 1.5 m with between 95% to 98% of the core being recovered. An examination of the core indicates the bedrock at this location is a grey, fine-grained limestone with occasional shale partings, stylolites and small vugs. The upper section of the rock is weathered and fractured as indicated by RQD values of 21% to 31%.

ii) Culvert Structure (Borehole 3)

Borehole 3 was located immediately adjacent to an existing creek in the approximate centre of a 200 m-wide floodplain. The borehole was sited at the proposed location of a new culvert structure designed to allow the creek to pass beneath the north approach of the overpass. Although the subsurface materials at this location appear quite similar to those encountered at the proposed overpass location, the exploratory drilling reveals that the overburden thickness in the vicinity of BH 3 is significantly greater than that at BH 1 and BH 2 and, in fact, is 18.5 m.

As was the case with BH 2, the upper, surficial stratum of BH 3 is comprised of a man-made fill deposit. Composed predominantly of material similar to that found in BH 2, the deposit in BH 3 extends to 0.7 m below the present grade. The fill is loose and uncompacted.

The natural subsoil below the fill in BH 3 consists of silty clay interlayered at depth with clayey silt. No topsoil underlies the fill in BH 3 but the upper 0.8 m of the clay is highly organic with numerous roots and humus pockets. The stratum extends from the base of the fill to a depth of 18.5 m below the existing ground surface.

No colour-banding or layering was observed within the upper 3.3 m of the stratum and the clay has a grey colour with brown mottling and contains a trace of fine sand. Below this level, the soil changes to a uniform grey with some brown-grey colour-banding and exhibits

a layered structure. It contains localized seams of silt and fine sand and has an increasing silt content with depth and possesses a trace of embedded sand and gravel at the base of the deposit.

The clay has a firm consistency ('N' = 7 blows/0.3 m) within the upper organic portion. Below this portion and extending to 7.0 m below present grade, the consistency of the clay increases to stiff as inferred from penetration indices of 5 to 8 blows/0.3 m, field vane shear strength tests of 107 to 180 kPa, laboratory compression tests of $C_u = 46$ to 81 kPa and penetration tests of 60 to 120 kPa. Extending from 7.0 m to the base of the deposit at 18.5 m, the clay exhibits a soft to firm consistency as based on 'N'-values of 2 to 3 blows/0.3 m, field vane shear tests of 60 to 85 kPa, and laboratory compression and penetration tests of 12 to 35 kPa. , Again, the presence of cohesionless materials in the soil has created some non-uniformity in the test results.

A variable sensitivity condition ("medium" to "extra-sensitive") is indicated throughout the deposit by ratios in the order of 2.4 to 9.3. In addition, natural moisture content measurements of 25% to 35% suggest an increase in moisture with depth while unit weight measurements decreased slightly from 19.9 kN/m³ in the upper section of the soil deposit to 18.9 kN/m³ near the base of the stratum.

The bedrock surface at the BH 3 location was penetrated at Elevation 59.4 m (i.e., 18.5 m below ground surface). The rock was cored for a depth of 1.3 m and was determined to be a granite gneiss. Although 86% of the core was

recovered, the upper sections of the rock are severely weathered and fractured as indicated by a 0% RQD value.

4.2 GROUNDWATER

The water levels measured in the standpipes installed in the boreholes are shown on the Borehole Logs. The groundwater movement is towards the creek adjacent to BH 3 where the water level was recorded at Elevation 77.6 m on March 12, 1985. On April 9, between 20 to 22 days after completion of the field investigation, the GW elevation recorded at BH 1 was Elevation 77.9 (i.e., 3.2 m below the existing ground surface) while in BH 3 the GWT was recorded at Elevation 77.4 m (i.e., 0.5 m below the existing ground surface). In BH 2, the GW elevation was measured at Elevation 79.8 m (i.e., 0.7 m below the existing ground surface). This latter reading may not be representative of a 'static' groundwater level and, given sufficient time to stabilize, the water level in BH 2 should establish itself more or less at the elevations being recorded in BH 1.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

Since this is a preliminary geotechnical investigation, the emphasis of this report is on fact-finding coupled with limited interpretation of the subsurface conditions that may influence foundation design. The type or structure planned for this site is presently unknown but we can make several assumptions that may assist in the consideration of alternatives for preliminary foundation design. In any case, we suggest that after the structural selection process has been completed, we be retained to examine the design drawings to ensure continuity with the assumptions made herein and to advise on the need for any further geotechnical work.

The present investigation indicates that the subsurface conditions at the test locations are non-uniform due to the deposition of materials of varying thicknesses in an eroded stream valley.

5.2 FOUNDATIONS - OVERPASS STRUCTURE (Boreholes 1 and 2)

The surficial fill, topsoil and any other highly organic materials underlying the site at this location are not competent to carry planned foundation loadings due to the risk of large total and differential settlements which would result from the compressibility, variable thicknesses, and potential for decomposition of these deposits. For these reasons, all structural loadings should, in our opinion, be extended to the natural, silty

clay/clayey silt stratum or to bedrock. To accomplish this, we have considered the following preliminary alternative foundation methods.

5.2.1 Spread Footing Foundations

Tentatively, both lightly and/or heavily loaded structures could be supported on conventional shallow spread footings.

The recommended founding stratum for shallow spread footings will be the very stiff, varved, silty clay. For structures founded on the silty clay, the foundation design will be governed by the shear strength of the clay below the foundation within a depth of 1.5 to 3.0 times the width of the footings. The presence of a softer clay zone at depth suggests that all footings founded on the silty clay should be placed at the highest elevation possible in the upper, very stiff crustal zone to avoid overstressing the weaker underlying material.

With this in mind, and provided that the proposed shallow spread footings are established on a carefully prepared, "undisturbed" surface of the very stiff silty clay, a tentative bearing pressure of 200 kPa can be used for preliminary design purposes. This figure assumes a minimum 1.2 m soil surcharge factor for frost protection. For overpass (bridge) structural design, the aforementioned value can be considered as a factored bearing capacity at Ultimate Limit States, but the recommended figure may be subject to review when the footing shape and depth/breadth ratio become known.

Without definite information concerning the type of overpass structure to be built and hence the amount of settlement it can tolerate, the bearing capacity at Serviceability Limit States for the structure cannot be properly determined. However, should the recommended bearing pressure of 200 kPa be fully mobilized as the resultant structural load and the proposed footing widths are infinitely large, it is estimated from the consolidation tests that total settlements for footings placed on an undisturbed surface of the clay will be in the order of approximately 50 mm (2 inches).

Based on the available information in the present investigation, no unusual geotechnically-related problems are anticipated in excavating for shallow foundations and dewatering in the project site.

5.2.2 Deep Foundations

If higher bearing pressures are required, heavy structures could be supported on deep foundations which establish their load-carrying capacity by end-bearing on the bedrock. Since the bedrock surface is generally within 6 m of the existing ground surface in this section of the site, suitable deep foundation types could be:

- i) Concrete caissons, cast-in-place in pre-bored holes,
- ii) Piles driven with minimum displacement

For sections established on a stable bedrock surface, a factored bearing capacity at Ultimate Limit States of 2,000 kPa can be used for preliminary foundation design purposes. No detrimental settlements are anticipated

for structures supported by deep foundations utilizing this bearing pressure.

5.3 FOUNDATIONS - Culvert Structure (Borehole 3)

The surficial fill and organic soils revealed in BH 3 are unsuitable as a bearing medium for structural foundation loads. All loadings should extend either to the natural, stiff, silty clay-clayey silt stratum or to bedrock. The foundation design parameters of these materials will be discussed in the following paragraphs.

Conventional shallow spread footings in this area should be placed as high as possible in the stiff silty clay stratum after giving consideration to frost protection and/or scour. This is provided that the footing width permits the vertical stress beneath the base of the footing (imposed by the structural load) to fall within the stiff zone and within 7.0 m of the ground surface.

Tentatively, a factored bearing capacity at Ultimate Limit States of 150 kPa can be used for the preliminary design phase. This value would be reduced significantly if the stresses penetrate the softer clay zone underlying the upper stiff section. This eventuality might be caused by greater footing widths or by an overlying deposit of imported fill (e.g., the north approach of the overpass).

We suggest, therefore, that we be afforded an opportunity to reassess the recommended pressure following the initial design work and also to provide an estimate of

settlement-related movement caused by the proposed structure(s).

Construction of shallow foundations in the vicinity of BH 3 will require the temporary diversion of the existing creek and the construction of a cofferdam or levee to protect the culvert site.

The thick (18.5 m) overburden at this site suggests driven end-bearing piles may be a practical construction alternative. For preliminary pile design, a factored bedrock bearing capacity at Ultimate Limit States of 2,000 kPa can be used for units founded on a stable bedrock surface.

5.4 FUTURE GEOTECHNICAL ENGINEERING WORK

As previously mentioned, the discussion and recommendations in this preliminary report are based on information obtained from limited field work and laboratory testing. The magnitude of the project suggests that the subsurface and groundwater conditions between and beyond the test sites may differ from those observed at the actual test locations. These possibilities point to the need for further geotechnical work to provide additional parameters for the final design phase of the project.

It is recommended that at least two additional boreholes should be put down along the alignment of the proposed extension north of the existing CNR right of way. One borehole is required midway between BH's 2 and 3 and another near STA 1+100. The information obtained

will enable a more accurate determination of the extent of the floodplain and will provide the parameters needed to predict the behaviour of the subsurface materials with respect to the north overpass approach.

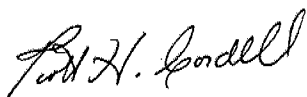
The subsurface conditions south of the existing CNR right-of-way are represented only by BH 1. One or two boreholes should be put down in this area to determine if the conditions encountered in BH 1 are consistent to the south along the Boulevard extension. In addition, they will provide information regarding the quality and mechanical properties of the existing fill in this area.

Additional laboratory work is also recommended to obtain further information re plasticity characteristics, shear strength parameters, compressibility, etc. This test work will be performed on samples presently stored in our laboratory as well as on samples retrieved from future borings.

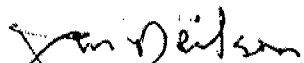
The additional data will be interpreted and used to verify the recommended bearing capacities already outlined; to establish additional foundation design alternatives; and to enable the determination of bearing capacities at Serviceability Limit States, Type II. They will also serve to anticipate excavation difficulties, and to predict settlement, dewatering problems and backfill requirements. In addition, the data may provide information re earth and hydrostatic pressures and uplift.

The scope of the additional geotechnical investigations should be reviewed by one of our engineers upon completion of the preliminary design studies. In this way, the needs of the Client and the structural requirements of the project will be best met.

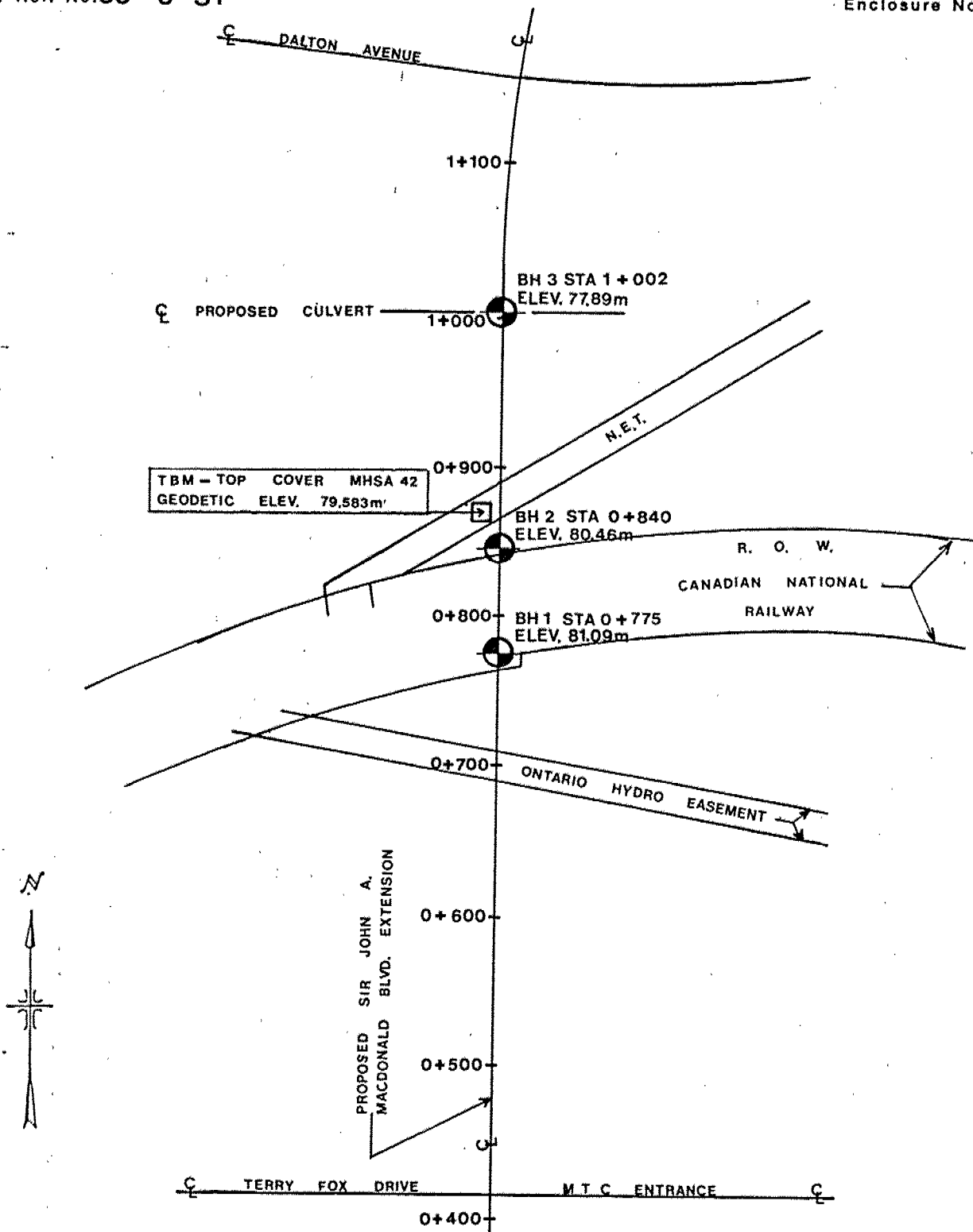
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President



PRELIMINARY GEOTECHNICAL INVESTIGATION
 PROPOSED OVERPASS GRADE SEPARATION
 SIR JOHN A. MACDONALD BLVD. EXTENSION
 KINGSTON, ONTARIO

BOREHOLE LOCATION PLAN

JAMES NEILSON & ASSOCIATES, INC.

SCALE AS SHOWN

85-3-S1

Our Reference No.

LOG OF BOREHOLE .1. (STA. 0. + 775)

ENCLOSURE No. 2.

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension
 DATUM ELEVATION: Geodetic

DRILLING DATA
 Method: Auger (Hollow Stem); Diamond Drilling
 Diameter: 150 mm O.D.-85 mm I.D.; 60mm O.D.-40 mm I.D.
 Date: March 18, 1985

SUBSURFACE PROFILE		SAMPLES		PENETRATION RESISTANCE					WATER CONTENT %			UNIT WEIGHT kN/m ³	REMARKS			
DEPTH m	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows/30 cm	20 40 60 80 100							PLASTIC NATURAL LIQUID LIMIT		
							UNDRAINED SHEAR STRENGTH kPa							LIMIT		
							PENETROMETER - COMPR. TEST							LIMIT		
+3% FIELD VANE TEST & SENSITIVITY							Wp			WL						
50 100 150 200 250							20 40 60									

81.1	0	GROUND SURFACE	Seal																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													</
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CITY: Corporation of the City of Kingston
PROJECT: Proposed Overpass Grade Separation
LOCATION: Sir John A. MacDonald Blvd. Extension
DATUM/ELEVATION: Geodetic

DRILLING DATA

Method: Auger (Hollow Stem); Diamond Drilling
Diameter: 150 mm O.D.-85 mm I.D.; 60 mm O.D.-40 mm I.D.
Date: March 19, 1985

[illegible]

Our Reference No. 85-3-S1.....

LOG OF BOREHOLE .3 (STA. 1 + 002)

ENCLOSURE No. 4.a.....

CLIENT: Corporation of the City of Kingston
PROJECT: Proposed Overpass Grade Separation
LOCATION: Sir John A. MacDonald Blvd. Extension
DATUM ELEVATION: Geodetic

DRILLING DATA
Method: Auger (Hollow Stem); Diamond Drilling
Diameter: 150 mm O.D.-85 mm I.D.; 60 mm O.D.-40 mm I.D.
Date: March 19, 20, 1985

ELEVATION m	DEPTH m	SUBSURFACE PROFILE	DESCRIPTION	SYMBOL	GROUND WATER	SAMPLES		PENETRATION RESISTANCE Blows/30cm					WATER CONTENT %			UNIT WEIGHT 1N/m ³	REMARKS	
						NUMBER	TYPE	N	UNDRAINED SHEAR STRENGTH kPa					PLASTIC LIMIT	NATURAL LIQUID LIMIT			
									Blows/30cm	20	40	60	80		100			Wp
									PENETROMETER COMPR. TEST					FIELD VANE TEST & SENSITIVITY				
									50	100	150	200	250	20	40	60		

77.9	0	GROUND SURFACE																
		clay, sand, gravel															Groundwater in standpipe	
		rubble, constr. debris																
77.2	0.7	(FILL)															Date	Elev.
	1	firm numerous organics				1	SS	7									Mar. 26	77.3 m
		stiff															Apr. 9	77.4 m
	2	grey brown mottling				2	TW	-									19.9	
	3	SILTY CLAY				3	SS	8										
		trace fine sand																
						4	SS	5										
		moist to wet																
	4	grey wet				5	SS	6										
		some brown-grey banding																
						6	TW	-										
	5																19.0	
		interlayered with																
	6	CLAYEY SILT																
						7	SS	6										
		stiff																
	7	soft to firm																
	8	occ. silt and fine sand seams				8	SS	2										
	9																	
6	9																	
	10	Borehole continues on Enclosure 4b																

Our Reference No. 85-3-S1

LOG OF BOREHOLE . 3 (STA 1 + 002) cont'd

ENCLOSURE No. 4 b

CLIENT: Corporation of the City of Kingston
PROJECT: Proposed Overpass Grade Separation
LOCATION: Sir John A. MacDonald Blvd. Extension
DATUM ELEVATION: Geodetic

DRILLING DATA

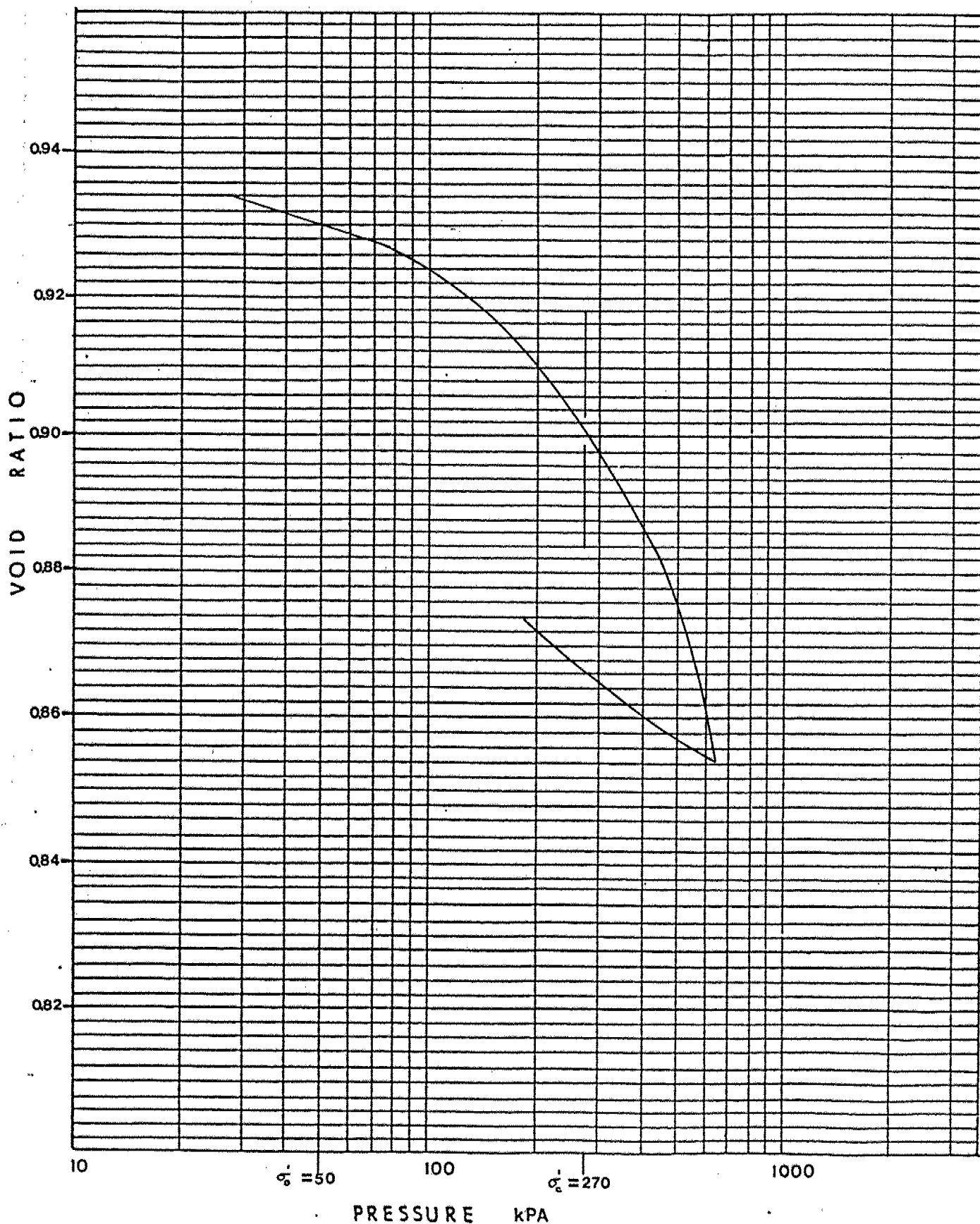
Method: Auger (Hollow Stem); Diamond Drilling
Diameter: 150 mm O.D. - 85 mm I.D.; 60 mm O.D. - 40 mm I.D.
Date: March 19, 20, 1985

SUBSURFACE PROFILE		SAMPLES			PENETRATION RESISTANCE					WATER CONTENT %			UNIT WEIGHT t/m ³	REMARKS
DEPTH	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N	Blows/0.3 m	UNDRAINED SHEAR STRENGTH kPa	PENETROMETER	COMPR. TEST	PLASTIC LIMIT	NATURAL LIMIT		
							20	40	60	80	100			
							FIELD VANE TEST & SENSITIVITY							
							50	100	150	200	250			
												Wp	Wl	
												20	40	60

67.9	10	(For soil conditions above see Enclosure 4a)														
11	soft to firm	wet		9	SS	2	▼	3.7								Mar.19
	grey						○									Mar.20 (11.43 m)
12																
13	SILTY CLAY															
	interlayered with															
14	CLAYEY SILT			10	SS	3	▼	2.4								
	occ. silt and fine sand seams						○									
15																
16																
17				11	TW	-	▼								18.9	
18	embedded sand and gravel															
18.5	weathered GRANITE GNEISS			12	BX	86% REC										RC - Rock Core
19	BEDROCK horizontal to diagonal fracturing			RUN 1, 2, 3	RC	0% RQD										REC - Rock Core Recovery
19.8	END OF BOREHOLE															RQD - Rock Core Designation
20																
21																

RC - Rock Core
REC - Rock Core Recovery
RQD - Rock Core Designation

CONSOLIDATION TEST



BH No. 1 (STA 0+775)

SAMPLE: TW-4

DEPTH: 2.5m

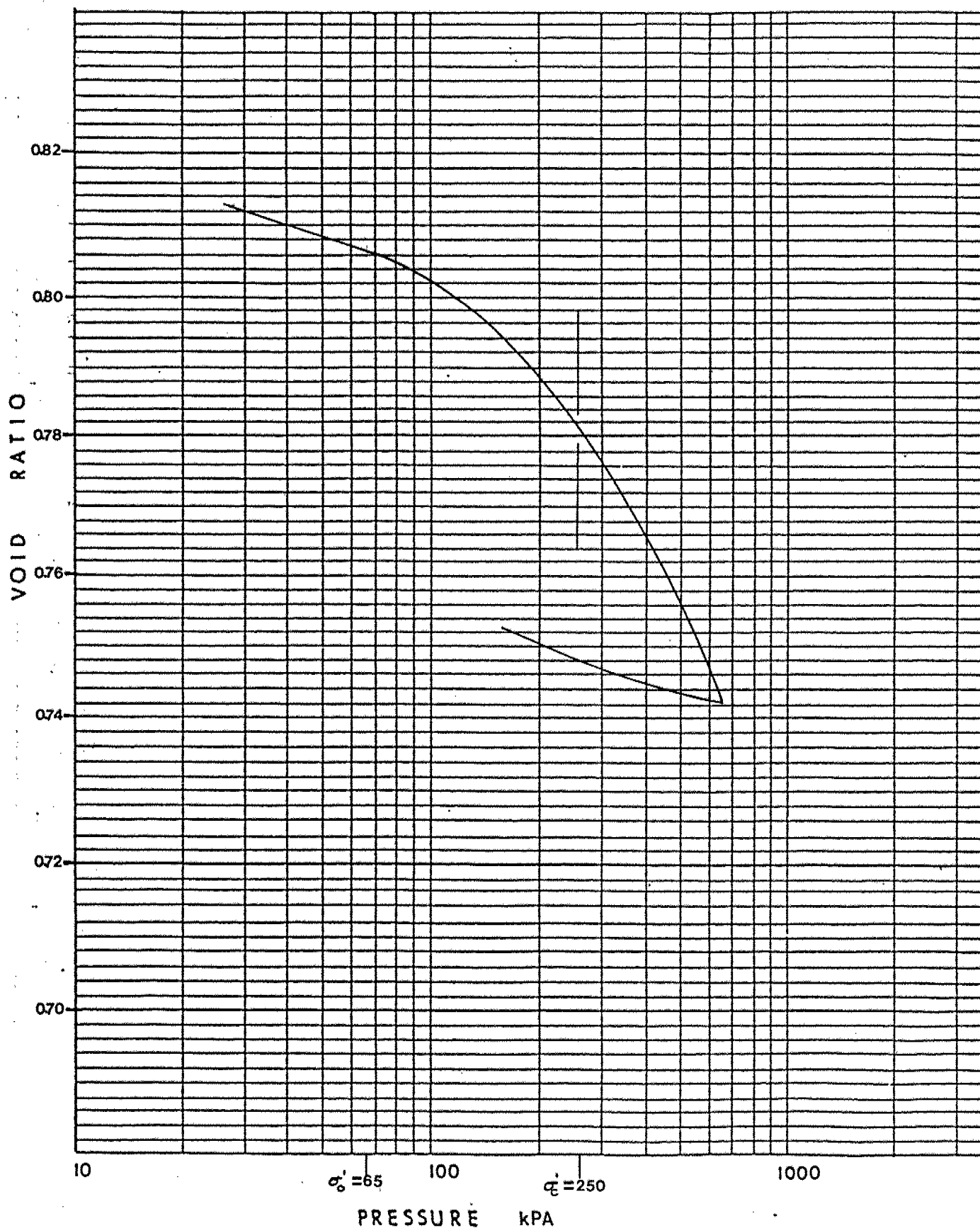
L.L. = - %

P.L. = - %

P.I. = -

w = 32 %

CONSOLIDATION TEST



BH No. 1 (STA 0+775)

SAMPLE: TW-7

DEPTH: 5.0m

L.L. = - %

P.L. = - %

P.I. = -

W = 25 %

GEOCOR ENGINEERING INC.

Geotechnical, Materials Engineers & Geologists



11 Parkwood Place
Kingston, Ontario, Canada, K7M 2C9
Tel. (613) 544-6474

Geocres # 31C-156
SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED OVERPASS SEPARATION
SIR JOHN A. MACDONALD BOULEVARD EXTENSION
AT THE C.N.R.
KINGSTON, ONTARIO
No 1 July 86

Reference No. 86-5-S3
July 1986

Prepared For:

The Corporation of the City of Kingston
c/o Totten Sims Hubicki Associates (1981) Limited
675 Bath Road, LaSalle Park Plaza
Kingston, Ontario
K7M 4X2

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- 14 Consolidation Test
- 15 Triaxial Tests

1.0 INTRODUCTION

GeoCor Engineering Inc., a firm of consulting geotechnical engineers, was retained by Totten Sims Hubicki Associates (1981) Limited, acting on behalf of the Corporation of the City of Kingston, to undertake a supplemental geotechnical investigation where it is proposed to develop a overpass-type grade separation. The development will take place where the proposed extension of Sir John A. MacDonald will cross the existing C.N.R. right-of-way and a tributary of the Little Cataraqui Creek in Kingston, Ontario.

In particular, the project involves an approximately 750 m extension of Sir John A. MacDonald Blvd. from Terry Fox Drive in the south to Dalton Ave. in the north. The tributary or creek flows through the study area in roughly an east to west direction approximately 150 m south of Dalton Ave. near project co-ordinates STA 1 + 000. The creek has established a floodplain from roughly STA 0 + 900 to 1 + 300. To permit the uninterrupted flow of water in the creek, the north approach of the overpass will incorporate twin S.P.C.S.P.A. culverts at about STA 1 + 000. The C.N.R. right-of-way is situated near the center of the project site just outside of the south floodplain boundary ie., about STA 0 + 800. The overpass structure will be constructed at this location and will span several railway tracks.

Although the natural topography of the site descends in elevations from the south and north towards the creek, the relief has been modified over the years by the deposition of miscellaneous fill materials placed within the limits of the proposed extension for the purpose of embankment construction.

As directed, the information gained from the present investigative exercise is intended to determine the physical and mechanical properties of both the prevailing subsurface man-made and natural conditions, which would form the basis for recommendations relative to the design and construction of the proposed overpass separation. The data obtained from this program is considered supplementary in nature, in that, it is used in conjunction with information supplied

from previously conducted subsurface work at the site.

This report provides a factual account of our work and includes the equipment and methodologies used; profiles the subsurface conditions encountered in the boreholes; and presents the results of the field and laboratory tests. The data gathered are interpreted and comments are presented on the geotechnically-related engineering parameters applicable to foundation design alternatives of the overpass structure; condition and treatment of the existing fills within the limits of the approaches; the development of structures (ie. north approach, culvert) within the confines of the floodplain of the tributary or creek; surface, base and sub-base recommendations for pavement profiles of the approaches; recommended borrow sources; the effects of the proposed development on the existing N.E. trunk sewer which passes under the study site at approximately STA 0 + 870 on a roughly 33° skew; and, the bedrock surface profile between Dalton Ave. and the south limit of the MacDonald Cartier Freeway. In addition, suggestions are advanced for future geotechnical work where warranted.

2.0 FIELD WORK

2.1 General

The field work for the investigative phase of the project was performed during the interim of May 30 to July 14, 1986, and consisted of:

- layout and survey of the test areas
- underground utilities clearances
- subsurface exploratory borehole program
- site visits (groundwater level monitoring, trunk sewer inspections)

2.2 Layout and Survey

The test locations for the investigation were laid out with the aid of a site plan entitled 'Plan & Profile', plate no. 2, project 14-6348, Sept. 1985,

prepared by Totten Sims Hubicki Associates. The project was oriented to the bearings shown and horizontal measurements to the test areas were referenced to Station points and existing landmarks. The borehole locations of the present investigation with that of the previously conducted work are shown on the accompanying drawing, Borehole Location Plan & Stratigraphic Section, Enclosure 1.

The elevations of the test locations were geodetically referenced to the following three City of Kingston benchmarks:

BM #42 - R.R. spike in hydro pole, STA 1 + 145, Elev. 79.623 m

BM #111 - R.R. spike in hydro pole, STA 0 + 422, Elev. 86.535 m

BM #285 - Top of S.I.B., STA 0 + 770, Elev. 81.428 m

The recorded readings are indicated on Enclosure 1 and on the Log of Borehole sheets, Enclosures 2 to 13.

2.3 Underground Utilities

Within the C.N.R. right-of-way there exists underground services for signalling and switching purposes plus a fiber-optics cable. Prior to initiating the drilling program, the borehole locations were cleared for these services by C.N. personnel. For future references, the services have been surficially delineated.

2.4 Borehole Program

The exploratory drilling program consisted of ten boreholes (B.H. 4 through 13) which were put down at pre-selected locations chosen by this firm in conjunction with the requirements of the project. The holes were advanced using a track-mounted mobile power drill, utilizing hollow-stem augers and conventional soil and bedrock sampling equipment to penetrate the substratum between 4.5 to 21.0 m

below grade. Both disturbed and undisturbed soil samples were recovered. Disturbed samples were taken at regular depth intervals with a 50 mm O.D. split-spoon sampler. The sampler was driven into the undisturbed soil at the bottom of the borehole with a constant driving energy (a 6.3 kg hammer dropping 750 mm). The number of blows required to advance the sampler 0.3 m yields the Standard Penetration Resistance ('N'-values) of the soil.

Undisturbed samples of cohesive soils were obtained with a 75 mm O.D. thin-walled (Shelby-tube) sampler. For undisturbed samples, the sampler is advanced its full length (700 mm) under a constant downward pressure from the hydraulic head of the power auger.

In addition, undrained shear strength tests were performed, in situ, in cohesive soil by means of a four-bladed torque vane. Both the undisturbed and remoulded strengths (sensitivity) were measured.

The basal stratum (ie. the bedrock) in B.H. 8 was explored by coring. The rock was diamond cored for 1.8 m using BX equipment (40 mm diameter core). Core recovery and pressure during coring as well as the approximate quantity of return water were noted. The "rate of recovery" (REC) and the "rock quality designation" (RQD) were calculated. These indices provide a useful measure of bedrock quality and the degree of weathering and fracturing.

The field program was supervised by members of our technical staff who logged, labelled and stored the samples which were then transferred to our laboratory for further examination and testing.

2.5 Site Visits

Groundwater levels were monitored throughout the field investigation and for a subsequent period following the drilling operation. Standpipes were installed in selected boreholes to permit future monitoring of the groundwater level, as needed.

Since the records regarding information of the N.E. trunk sewer were found to be somewhat ambiguous, one of our field engineers descended down MH42 to conduct an internal inspection of the sewer. Where possible, measurements were taken and observations made regarding the type and condition of the pipe.

3.0 LABORATORY WORK

All soil samples taken to our laboratory were re-examined by a soils engineer and classified by visual and tactile methods in accordance with the Unified Soil Classification System. See also:

ASTM C 2487 Standard Test Method for Classification of Soils for
Engineering Purposes

and Canadian Foundation Engineering Manual, Second Edition, 1985

A geologist from our staff re-logged the recovered bedrock core.

Natural water contents, unit weights and plasticity limits of representative samples were determined by standardized laboratory methods. In addition, numerous penetration and compression tests were conducted to determine the undrained shear strength of the cohesive units. The compressibility and consolidation characteristics of the subsoil were determined by laboratory consolidation tests. The test results are plotted on the borehole logs or are presented on separate enclosures.

4.0 SUMMARIZED SUBSURFACE CONDITIONS

4.1 Stratigraphy

The stratigraphy, as encountered in the boreholes, is illustrated on the Log of Borehole sheets comprising Enclosures 2 through 13. Enclosure 1 duplicates

some of the borehole information in conjunction with the results of a previous boring investigation conducted at the site by others (ie. James Neilson & Associates, Inc., Ref. No. 85-3-S1, March 1985).

Bedrock does not outcrop at the site because the study area is located in an eroded stream valley which has been in-filled with postglacial sediments. These sediments were laid down at intervals during the past 10,000 years and based on the borehole data consist predominately of clays and silts with more recent organic materials.

Although the now buried bedrock shoreline contains steps, it does drop quite abruptly in elevation (some 15 m) approximately between STA 0 + 875 \pm and STA 1 + 300 \pm as shown on Enclosure 1. Where encountered at shallow depths, the bedrock is an Ordovician limestone common to the Kingston area, whereas, a Precambrian granite was found to lie at the greater depths.

The predominate soil type encountered at the site is a varved silty clay interlayered with clayey silt (at depth) which generally extends from the bedrock surface to the ground surface (except where either a thin veneer of topsoil or a man-made surficial layer of fill material exists. The varved stratum has been desiccated in the upper sections and exhibits a stiff to very stiff consistency, however, below this crustal layer, the shear strength of the soil decreases to a firm then soft consistency.

As outlined above, besides the surficial organic deposits, the other surface material encountered above the silty clay/clayey silt stratum is a miscellaneous fill. Generally restricted to those areas of the site where the south and north approaches are to exist, the fill contains of mixture of large concrete, asphalt and limestone slabs with various quantities of clay, silt, sand, gravel, organic matter and construction debris. The sporadic field penetration tests and the results of visual observations indicate this material was placed without the use of controlled compaction procedures.

Rather than dealing at this stage, with a lengthy discussion summarizing the borehole data, further details concerning the stratigraphy will be outlined in the subsequent Section 5.0 Discussion and Recommendations as it pertains to the development of the individual facets of the project.

4.2 Groundwater

The groundwater levels measured in the open boreholes and standpipes are shown recorded on the Borehole Logs, Enclosures 2 to 13 and Enclosure 1. Generally, the groundwater movement is towards the open creek adjacent to B.H. 3 (Ref. No. 85-3-S1) where the water level was recorded at Elev. 77.6 m on March 12, 1985. Recorded variations in the continuity of the GWT is attributed in part to a 'perched' groundwater effect and the fact that the groundwater table had not fully stabilized during the investigative interim.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 General

The geotechnical study of the proposed overpass-type grade separation development area, indicates that the subsurface conditions at the test locations are non-uniform due to the deposition of natural materials of varying thicknesses in an eroded stream valley-floodplain environment and the subsequent in-filling of a majority of the site with man-made deposits. Since the physical and mechanical properties of the various strata differ significantly, their relationship to the individual facets of the project will be dealt with as they pertain to each of the proposed structures. It is realized, however, that the full design details of the project are presently unknown and, as such, we have made several assumptions that will assist in the consideration and selection of alternatives for design purposes. Because of this, we are available throughout the designing stage for additional comments and recommendations regarding geotechnical input

and to ensure that there is continuity with the assumptions made herein and to advise on the need for further geotechnical work if required.

Both heavily and lightly loaded structures are being considered for the project. The presence of fill, organic deposits, deep lying compressible soils, buried side-slopes, high groundwater levels, etc., suggest that the following recommendations be taken into consideration in the design work.

5.2 Foundations - Overpass Structure (Boreholes 5,6,7 & 8; Boreholes 1 & 2, Ref. No. 85-3-S1

It is our present understanding that the proposed overpass structure which will span the existing C.N.R. right-of-way, will be supported by two rows of piers located at approximately STA 0 + 795 and 0 + 827 and abutments located at roughly STA 0 + 763 and 0 + 850. The piers are tentatively planned to be founded below the present grade while the abutments are to be placed at a higher elevation within an engineered fill.

As indicated on Enclosure 1, the overpass structure will be positioned just beyond the southern limit of a now buried steep bedrock side-slope, in the area of a relatively flat lying bedrock step or ledge. The surface of the rock in this area was found to lie at approximately Elev. 75.0 \pm m. The bedrock is overlain by 4.5 to 6.0 m of varved silty clay/clayey silt. The clay possesses a very stiff, upper crustal layer which gradually decreases in consistency with depth. Surficial deposits of organic clay, topsoil and/or fill overlie the varved stratum with the thickest fill sections being recorded in the areas of the proposed abutment locations.

The surficial fill, topsoil and any other highly organic material found in this area of the site are not competent to carry planned foundation loadings distributed by the proposed structure due to the risk of potential settlement and shear failure problems. For these reasons, all structural loadings should, in our opinion, be extended to the natural very stiff, upper crustal section of the

silty clay/clayey silt or to bedrock where the former alternative is considered not feasible. To accomplish this, we have examined the following foundation methods.

5.2.1 Spread Footings On Natural Stratigraphy

Potentially, the overpass structure could be supported on conventional shallow spread footings. In this regard, the recommended founding stratum for the footings would be the upper crustal zone of the very stiff varved silty clay/clayey silt, which lies at approximately Elev. 79.5 ± 0.5 m in this particular area of the development. Based on both the field and laboratory test results, footings founded on an "undisturbed" surface of the very stiff, varved subsoil can be designed using a maximum allowable bearing pressure of 250 kPa (5,200 p.s.f.). The presence of a softer cohesive zone at depth, however, tends to restrict the dimensions of footings utilizing this bearing pressure. For example, to minimize total settlement of the overpass structure in the order of approximately 50 mm, the maximum allowable strip footing width that could be used would be 1.5 m. Likewise, to maintain total settlements in the area of about 65 mm, strip footing widths would be restricted to a maximum of 3.0 m.

The feasibility of using foundation loads and dimensions other than those listed above, should be reviewed by a geotechnical consultant to determine the stresses imposed on the bearing materials and the settlements that may result. It should also be noted that the recommended bearing pressure and associated settlements have assumed a minimum 1.2 m soil surcharge cover factor for frost protection. However, the depth of engineered fill at the abutment locations will be significantly larger than this and will have to be compensated for in the foundation design when the applied loads from the structure and depth of fill are known.

Another area of concern regarding shallow foundations, will be the proximity of the north abutment footing with respects to the underlying steep bedrock side-slope. Although the investigation indicates the drop-off occurs between B.H. 8 STA 0 + 855 and B.H. 9 STA 0 + 925, there is warrant to more accurately defining

Why not
done when
drill there.

the location of the slope with respects to the vertical stresses applied from the footing.

The clayey bearing materials are susceptible to loosening by disturbance, water-ponding, etc. during construction. For these reasons, it is imperative that construction be carried out with the utmost care and water not be allowed to collect on the founding surfaces. We recommend that prior to actually constructing the foundations, the proposed bearing surfaces be inspected by one of our geotechnical engineers to confirm the natural, undisturbed condition of the subgrade and the validity and extent of the recommended bearing pressure, indicated above.

The available information from both the present and previous investigations indicate that no unusual geotechnically-related problems are anticipated in excavating for shallow foundations and dewatering the footing areas.

5.2.2 Deep Foundations

Should higher bearing pressure and minimal settlements be required, the proposed overpass structure could be supported on deep foundations which would establish their load-carrying capacity by end-bearing on the bedrock. This foundation alternative also eliminates the need to compensate for the softer clay zones at depth in the overburden stratigraphy; it minimizes the strict compaction control requirement for shallow footings bearing on engineered fill; and, perhaps eliminates a potential slope stability problem associated with the steep bedrock side-slope immediately adjacent the development area.

Available data indicates that pile founding levels will occur approximately at Elev. 75.0 m in this section of the site and coupled with other subsurface information, suggests that the most suitable deep foundation types would be i) concrete caissons, cast-in-place in pre-bored holes or ii) piles driven with minimum displacement.

Concrete caissons are preferred because of their high-capacity, ease of variable lengths, ability to inspect potential bearing surfaces and minimal heaving or displacing effects of adjacent piles or structures. For sections established on a stable bedrock surface, a bearing capacity of 2,000 kPa (41,750 p.s.f.) can be used for design purposes. It is anticipated that the loads required to produce detrimental settlements to the proposed structure will far exceed the aforementioned capacity.

Likewise, minimum displacement, driven piles are considered acceptable under the prevailing site conditions. Preference is given to steel H-pile sections which are equipped with a rock point to effect some penetration of upper weathered layers in the bedrock and grip against potential sloping surfaces. For H-sections that are vertically driven to practical refusal in the bedrock, the capacity of pile can be estimated from the cross-sectional area of the unit by multiplying it with the allowable unit stress of this pile material. *use*

The following pile sections and allowable load-carrying capacities are given as examples for H-piles with a steel stress of 83 MPa (12 k.s.i.):

PILE SECTIONS		PILE CAPACITY	
IMPERIAL	METRIC	TONS	KN
HP 8 x 36	HP 200 x 54	65	570
HP 10 x 57	HP 250 x 85	100	990
HP 12 x 53	HP 310 x 79	95	830
HP 12 x 63	HP 310 x 94	110	990
HP 12 x 74	HP 310 x 110	130	1170

No detrimental settlements are anticipated for the overpass structure wholly supported by deep foundations provided they are designed and installed in accordance with the above guidelines and good engineering practice.

It should be pointed out, that on previously monitored construction projects, data has revealed that serious heaving/settlement of existing, or for that matter, proposed structures, can result from adjacent driving of displacement piles (ie, closed-ended pipe piles) in deposits of clay. This effective zone of influence has been recorded to be laterally as much as 30 m from the point of piling. Should displacement piles be considered for this project, the potential for heave/settlement should be monitored during the driving operation of these units.

5.3 Approaches

5.3.1 South Approach (Boreholes 4 & 5)

The natural stratigraphy along the alignment of the proposed south approach appears from the borehole information to be similar to that encountered at the location of the proposed overpass structure. The bedrock, however, was observed to be rising in elevation towards the south which consequently results in a thinner soil overburden. Generally, the natural soil overburden is a varved silty clay interlayered with clayey silt and was found to range from approximately 2.9 m in thickness at B.H. 4 to 4.9 at B.H. 5. As previously found, besides a thin stiffening zone near the surface of the deposit, the varved stratum exhibits a very stiff, crustal section decreasing in consistency with depth.

At the time of the investigation, the cohesive soils were overlain by a wedge of miscellaneous fill material with recorded thicknesses of 2.8 m at B.H. 4 to 6.4 m at B.H. 5. The fill is considered miscellaneous both in its composition and in-situ strength parameters. As evidenced by field penetration tests and from visual observations, proper controlled compaction procedures were not adhered to during the placement of the existing approach material. The test data indicates sporadic values with interlayers of compact with soft to firm material. Likewise, the material being dumped for the subsequent use as approach fill material is inconsistent. Of particular note is the presence of large concrete, asphalt and limestone slabs which provide very little interlocking

(frictional) strength. Such concentrations of this material may constitute preferred failure planes. Also, significant void spaces have been observed between the over-lapping sections of these large fill pieces. Over a prolonged period of time, this condition may result in localized settlement problems as fine grained material wash into the void space. Elsewhere, the fill is extremely non-homogeneous consisting of a mixture of clay, silt, sand, gravel and varying degrees of construction debris. The presence of organic matter in the form of topsoil, trees and miscellaneous wood fragments were also observed.

Assigning engineering parameters to the on-site fill material in order to facilitate stability assessments, is extremely difficult due to the unpredictable behaviour of some of the materials noted above. Short of re-excavating and compacting the existing fill properly, it will be difficult to upgrade the quality of this in-situ material. Where the fill is only a few meters deep, large vibratory compaction equipment may prove successful. Ideally large slabs should be broken into irregular 'fist' size pieces to provide greater frictional strength, eliminate the potential nucleation of failure planes and decrease void spaces and associated localized settlements.

Future filling programs should specify improved fill quality to prevent the dumping of organic matter and other deleterious materials. The embankment fills should be deposited and spread in uniform layers for the full width of the embankment. The methodology whereby the embankment is built up by constructing the core and thereafter side dumping, should be avoided. Each successive layer should be shaped to the line and cross-section and thoroughly compacted before the succeeding layer is placed. Depending on the compaction equipment available, as a guideline, the individual lifts or layers should be within 200 to 300 mm in thickness and compacted to a minimum of 95 % of its Standard Proctor Maximum Dry Density (SPMDD). Granular base and sub-base courses require different installation specifications which have been outlined along with other details concerning the construction of the approaches, in the subsequent sections of this report.

In the area of the highest embankment height (10m) along the south approach alignment, total settlements in the order of 100 mm can be expected. *4 inches*

5.3.2 North Approach (Boreholes 9 & 10; Borehole 3, Ref. No. 85-3-S1)

As can be readily seen on Enclosure 1, the north approach will span almost entirely along its alignment, the now buried gorge created by the former and existing tributary of the Little Cataraqui Creek found in this area of the study site. From an elevation of approximately 75.0 m in the vicinity of the proposed overpass structure location, the bedrock abruptly drops to Elev. 59.5 ± m along the north approach alignment until roughly Dalton Ave. where it commences to rise again. Averaging about 18 m in thickness, the eroded valley has been in-filled with a varved silty clay interlayered with clayey silt. Besides its most noticeable overall increase in thickness and associated thickness increase in the deeper lying softer zones, the varved stratum here shows similar characteristics to the cohesive deposits found elsewhere on the site.

Fill of similar quality to that encountered along the south approach was also found to overlie the natural clayey soils in this area. However, the maximum recorded thickness of this man-made deposit was calculated to be 3.7 m at B.H. 9 with lesser depths being recorded at B.H. 3 and 10. The thinner profile will allow large vibratory equipment to be more effective throughout the fill profile. As to the additional treatment and future backfilling procedures of imported material along the north approach, reference is made to the comments and recommendations given in Section 5.3.1, herein.

Besides the need to construct a solid embankment under controlled conditions, the north approach will also be largely governed by the behaviour of the deep natural underlying deposit of the silty clay/clayey silt. Of particular concern will be the stability of the embankment with respects to slip failure and settlement of the varved stratum. However, with adequate monitoring and design work, potential problems in this regard can be anticipated, accounted for and implemented into the final design.

Based on the recovered data from the field investigation and subsequent laboratory testing program, a detailed stability analyses has been completed for the proposed north approach where the prevailing site conditions are the most adverse. Our analyses indicates that should the embankment (at its highest location) be constructed in a one stage effort, the factor of safety against basal slip failure is less than one. Basal failure, that is, failure at the bedrock surface, is almost a certainty due to the crustal nature of the varved clay deposit.

Therefore, in order that the design grade of the north approach can be achieved, special design considerations must be examined. One alternative would be the extensive use of stabilizing berms. The berms would be expensive to build and would depend on the availability of additional quantities of select fill material. Depending on the engineering parameters of the soil underlying the berms, it may be possible to build the embankment without using stage construction which generally necessitates a longer construction period. However, additional geotechnical information would be required to further comment of this possible design alternative.

Another and possible more feasible alternative, would be to construct the north approach in stages. This procedure of pre-loading the site causes consolidation of the cohesive subsoils which is essential in order that the soils will gain sufficient strength for subsequent staging to achieve the design grade. Detailed calculations show that a maximum fill height of 6 m or a grade elevation of 83.5 m should be permitted for the initial pre-consolidation stage. This embankment height provides a factor of safety against basal failure of 1.2 . Subsequent consolidation of the soft underlying varved soils will increase the shear strength sufficiently that the second stage may be constructed to design grade with a safety factor of 1.4 against basal shear failure. Until monitoring of underlying strata, however, has indicated that the required consolidation has taken place and the shear strength has sufficiently increased, the placement of fill for the second stage should be avoided. A suggested monitoring program is outlined in Section 5.9 entitled Further Geotechnical Work.

A preliminary estimate of settlement for the north approach indicates that final total movement may be in the order of 400 mm. Since this analyses is based on consolidation tests in the crustal and basal layers only, it should be regarded only as an estimate. Further testing is recommended in this regard.

5.4 Culvert (Borehole 3, Ref. No. 85-3-S1)

As documented, the new proposed twin structural plate corrugated steel pipe arch (S.P.C.S.P.A.) culverts are to be centered on or about STA 1 + 002 which is where B.H. 3, Ref. No. 85-3-S1, was put down. The dimensions of the proposed structures will individually be 3.89 x 2.69 (span x rise) with an approximate overall length of 52 m. Positioned roughly in a east-west direction, the culverts will provide a westward drainage path for the existing tributary or creek.

The finished embankment grade at the culvert locations will provide approximately 4 m of soil cover over the pipes. To accommodate the water flow, the proposed invert has been tentatively set at Elev. 76.8 m. ← ok.

In terms of bedding requirements, the presence of yielding and frost susceptible subsoils at the site, suggests that the new pipes bear directly on a 300 mm thick shaping bed which, in turn, is underlain by a 600 to 900 mm layer of compacted granular bedding. The latter 900 mm thickness is required if it is anticipated that low water levels in the creek could be at or near the design culvert invert level in times of - 0° C temperatures. Maintaining continuity with the aforementioned elevations, the above-noted bedding thickness will result in underside elevations of 76.5 m for the shaping bed and 75.9 to 75.6 for the lower bedding profile.

The geotechnical study indicates that the excavation at the tentative design sub-cut elevation (75.6 to 75.9 m) will be into the stiff, varved silty clay just below an upper clayey organic soil. Our analyses indicates that the stiff, inorganic varved deposit possesses adequate bearing capacity to sustain the culvert structures, bedding materials and 4 m of proposed soil cover. However,

memorandum



To: Mr. W.E. Blum
District Municipal Engineer
District 08, Kingston

Date: 86 09 02

FROM: Geotechnical Section
Eastern Region, Kingston

RE: C.N.R. Overpass, Sir John A. MacDonald Blvd.
Pavement Designs

This is in reply to your request for comments relative to pavement design and in answer to P.A. Smith's query in his letter to you of August 14, 1986.

We feel that an asphalt depth of 130 mm is adequate in this instance and I believe that such depth is what was designed for the additional lanes on the same road between Princess and Counter Streets. The Ministry generally uses a 40 mm surface course depth for two reasons: 1) Surface course mixes are usually more expensive and increased depth increases costs, and 2) 40 mm is considered the minimum depth to obtain proper compaction and yet thick enough to provide durability and good rideability.

We would suggest a durable skid resistant surface course be used in this instance (H.L. 1, H.L. 3 Mod. or D.F.C.). This could be placed on lower and upper binders (50 mm and 40 mm) of conventional asphalt [H.L. 4 or Kingston City's H.L. 6 (Mod.)]. If additional strength is required, the binder depths could be increased on the basis of Granular Base Equivalency (G.B.E.). As you are aware, G.B.E. requirements are dependent on fill material type and granular base and sub-base type and depth. This information was not available in the package sent to this office.

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1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																				

the loads distributed by the culvert profile will result in an estimated 175 to 250 mm of total settlement at this location.

The excavation to accommodate the culverts, and hence the bedding width, should extend along their entire length to a width of approximately 1.2 m ← 1.3. on both sides of the culverts' greatest width. The bedding backfill material should be free-draining, non-frost susceptible (less than 8 % passing the 75 micron size) granular material and compacted to approximately 95 % SPMDD. Fill material within 300 mm of the pipe should not exceed 75 mm in size. The shaping bed should be stable but relatively yielding.

Culverts of pipe-arch shape exert greater pressures against the soil at the corner plates than elsewhere around the structure, hence, the need to insure haunches are well compacted (100 % SPMDD). In addition, corner bearing pressures also specify the minimum pipe wall thickness required. Our estimation of the corner pressure is 200 kPa.

5.5 Pavement Profile

The pavement profile for the approaches will be governed by the traffic usage of the structure and the type of subgrade upon which the profile (surface, base and sub-base courses) will bear. It is our understanding the proposed overpass-type grade separation will be multi-lane with a future ten year daily traffic usage of approximately 12,000 vehicles. We have assumed for the purpose of this assignment, that the type of subgrade at the finished approach grades will be of similar composition to the existing fill material found at the project site. However, should the quality of approach fill material improve as the backfilling program continues to the final grades, adjustment in the proposed sub-base thicknesses can be made.

Based on the present available data, the following pavement profile design is recommended for the proposed overpass approaches.

130 145 mm - Hot Mix Asphalt

150 mm - Granular 'A' base course

450 to 800 mm * - Granular 'B' sub-base course

* Thickness subject to assessment of future subgrade quality

The subgrade material below the paved areas should be adequately prepared before placing the pavement structure. After grading, the subgrade should be compacted to not less than 95 % of its SPMD.

The granular base and sub-base materials should be placed in loose lifts not exceeding 200 mm in thickness and each layer compacted to 100 % of its SPMD. The asphalt pavement should consist of a 75 mm - thick HL 8 binder course and a 70 mm - thick HL 3 surface course. 90

40

5.6 Borrow Sources

Preferred borrow sources for the type of structures being considered for this project are granular pits containing material which can be re-deposited and compacted to achieve a high friction angle (ie. well-graded, non-plastic sands and gravels). Organic materials, frozen lumps of soils, large slabs or boulders and other extraneous materials should be avoided.

Since the proposed exit from the MacDonald Cartier Freeway at Sir John A. MacDonald Blvd. is tentatively designed to be an underpass structure, the inorganic soil overburden as encountered in B.H. 11, 12 and 13 could be considered as a potential borrow source. The soil is a varved silty clay/clayey silt with measured moisture contents of 32 to 40 %. The material would be restricted to areas where frost penetration is not considered a problem and would require air drying prior to being re-deposited and compacted. However, further analyses would have to be conducted on this material to determine its optimum moisture content and compactability characteristics.

The end result is to achieve a solid embankment containing little or no voids and with predictable engineering parameters so that the future behaviour of the structure can more accurately be determined.

5.7 Trunk Sewer

As shown on Enclosure 1, the existing NE trunk sewer will pass under the proposed development just north of the overpass structure location where the north

approach will be at its highest elevation. An internal inspection of the sewer revealed the pipe invert to be at approximately Elev. 73.0 m at MH 42 or about 6.5 m below the present grade at the site. The pipe is a 1067 mm internal diameter concrete culvert with 115 mm wall thickness. The concrete is circular reinforced with approximately 8 mm wire mesh at 75mm centers longitudinally and 150 mm centers around the pipe perimeter. For the most part, both the concrete and pipe joints appear to be in good structural condition in the vicinity of MH 42. At the time of the inspection approximately 20 cm of water lay in the sewer.

Extrapolating the borehole data indicates that the pipe is probably bearing on bedrock at the west end of the proposed north approach, however, moving eastward through the embankment, the bedrock appears to drop in elevation and the sewer is bearing on an undetermined depth of varved silty clay/clayey silt. To more accurately define the subsurface conditions along the pipe alignment within the development area, additional field work would be necessary which should concentrate both on revealing the external 'as constructed' conditions of the sewer, as well as, the extent of the stratigraphy below the pipe.

Some initial design analyses, however, can be determined from the available information. If we assume the worst condition whereby the west end of sewer is bearing directly on bedrock (B.H. 8 data) and the east end is bearing on a varved silty clay/clayey silt of the extent found in B.H. 9, then differential settlements along the sewer, with the introduction of the north approach, could potentially reach 250 to 300 mm. However, when the subsurface conditions are more accurately plotted out, a more realistic estimate of anticipated differential settlements at this location would be 100 to 150 mm.

It is estimated that the existing concrete sewer pipe can sustain the loads introduced by the embankment without failure from crushing but, the magnitude of expected differential settlement could conceivably cause the pipe joints to open. It has been reported that the sewer was supposedly encased in concrete along certain parts of its length which would help prevent some of the damaging

effects created by differential settlement, however, the assessment of this condition requires further investigative work.

5.8 Dalton Avenue to MacDonald Cartier Freeway (Boreholes 11, 12 & 13)

Three boreholes were put down along the proposed alignment of the Sir John A. MacDonald Blvd. extension from Dalton Ave. to the south limits of the MacDonald Cartier Freeway (Hwy. 401) for the purpose of defining the bedrock surface in this area.

As can be readily seen from the Log of Borehole sheets, Enclosures 11, 12 & 13 and Enclosure 1, this area defines the now buried north shore of the tributary. From a depth of 8.4 m below ground surface (Elev. 72.3 m) in B.H. 11 to a depth of 7.0 m (Elev. 79.0) in B.H. 13 the bedrock rises in elevation towards the north at a more gradual increase than was observed along the southern shore.

The soil stratigraphy encountered above the bedrock surface in all three boreholes consisted of a varved silty clay/clayey silt similar to that encountered elsewhere within the study area.

5.9 Further Geotechnical Work

The following is a list of recommendations for future geotechnical engineering involvement for this project and does not exclude the previously outlined suggestions for additional design details made elsewhere in this report.

1. It is essential that deformation of the north embankment be monitored as backfilling procedures advance so that a potential slope failure can be anticipated. In this regard, the following is recommended:
 - i) Slope indicator at the toe.
 - * ii) Settlement plugs on the surface (preferably on the side-slopes and at the toe where they should be left undisturbed).
 - iii) A settlement platform on the original ground surface.
2. The second stage of construction, should not proceed until the underlying strata has sufficiently consolidated (ie. pore pressures have dissipated).

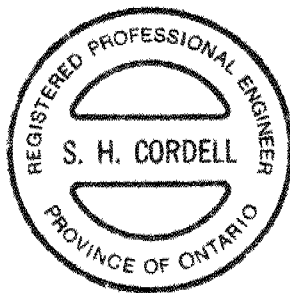
This can be indicated by piezometers, preferably installed at the quarter points in the underlying strata. Due to the heavy flow of construction traffic, electronic piezometers are recommended, as standpipes will suffer damage resulting in the loss of valuable information. Several standpipes on the site have already been destroyed, despite efforts to mark them well.

3. Anticipating the time for consolidation and hence the construction period for stage one is necessary. If it is paramount to expediate the construction schedule, sand drains may be necessary. However, further consolidation tests are recommended, although these tests should not be considered redundant to on-site field instrumentation.
- * 4. Additional subsurface work in the form of test pits and possibly boreholes to more accurately define the prevailing conditions along the existing NE trunk sewer/north approach location.
5. Proctor analyses and other quality control tests on potential embankment fill materials.
6. A review of the final design details to ensure continuity with the assumptions and recommendations made in this report.

GEOCOR ENGINEERING INC.

Scott H. Cordell

Scott H. Cordell, P.Eng.
President



SHC:jsl
Encls.

ENCLOSURES

BOREHOLE LOCATION PLAN & STRATIGRAPHIC SECTION

PROPOSED
OVERPASS
AT
SIR J.A. MACDONALD BLVD.
&
C.N.R.

PLAN SCALE

1 : 4000

SECTION SCALE

HORIZONTAL - 1 : 4000

VERTICAL - 1 : 250

NOTE: ALL DIMENSIONS ARE IN METRES
ALL ELEVATIONS ARE GEODETIC
REFERENCED

LEGEND

- 10 BOREHOLE NO. & LOCATION
(REF. NO. 86-5-S3)
- 3 BOREHOLE NO. & LOCATION
(REF. NO. 85-3-SI, James Neilson & Assoc.)
- MH MANHOLE

▽ GROUNDWATER LEVEL
86-5-S3 JULY/86, 85-3-SI APR/85

UNIT 1 FILL

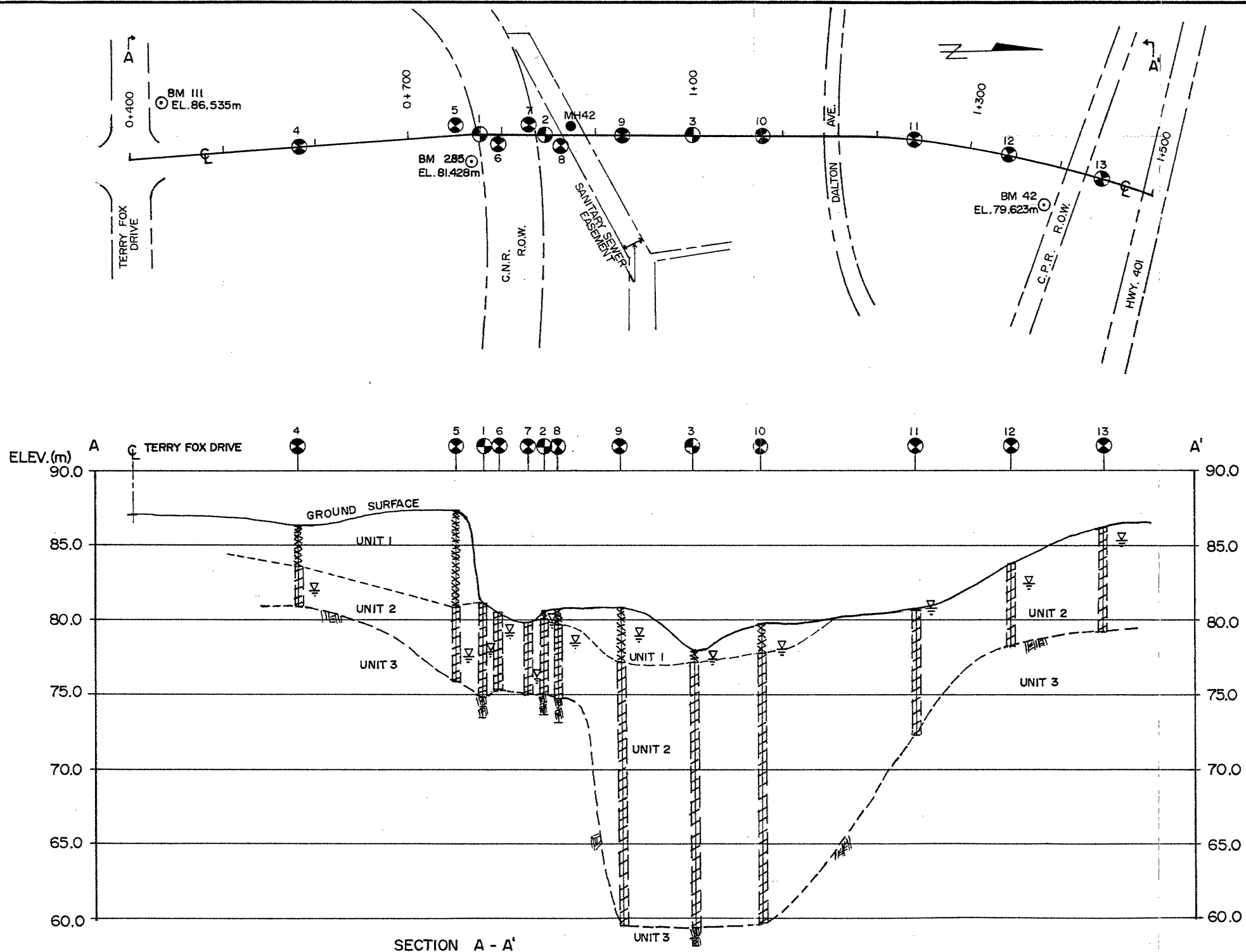
UNIT 2 varved SILTY CLAY /
CLAYEY SILT

UNIT 3 BEDROCK

NOTE: The boundaries between strata have
been established only at Boreholes.
Between Boreholes the boundaries
have been interpolated.

REF. NO.	DATE	DWN. BY	CH'D BY
86-5-S3	JULY 1986	G J G	<i>[Signature]</i>

GEOCOR ENGINEERING INC.



Our Reference No. 86-5-S3....

LOG OF BOREHOLE .4 (STA 9+580) .

ENCLOSURE No. 2

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA
 Method: Augering (Hollow-Stem)
 Diameter: 150 mm O.D. - 85 mm I.D.
 Date: July 3, 1986

ELEVATION E	DEPTH E	SUBSURFACE PROFILE		SAMPLES		PENETRATION RESISTANCE $\frac{Blows}{30.5\text{ cm}}$					WATER CONTENT %		UNIT WEIGHT $\frac{t}{m^3}$	REMARKS
		DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N	Blows/30.5 cm	UNDRAINED SHEAR STRENGTH kPa	Y PENETROMETER	COMPR. TEST	PLASTIC LIMIT	NATURAL LIQUID LIMIT	
									50	100	150	200	250	
												20	40	60

86.30	0	GROUND SURFACE												
		compact damp	F											Groundwater in open borehole
	1	mixture of clayey silt, sand gravel, rubble, misc. constr. debris (FILL)	F		1	SS	28							Date July 14
	2		F		2	SS	50+							Elev. 81.95
	2.75		F											SS2 - High 'N' caused by rubble
83.55	3	firm moist banded brown-grey & grey			3	SS	13							
	4	stiff varved to SILTY CLAY very stiff			4	SS	23							
	5	interlayered CLAYEY SILT embedded			5	SS	20							
80.66	5.64	soft gravel wet			6	SS	50+							
	6	END OF BOREHOLE Auger Refusal (inferred bedrock)												SS6 - High 'N' caused by bedrock
	7													+ 'N' - penetration less than 0.3 m.
	8													
	9													
	10													
	11													

GEOCOR ENGINEERING INC.

DRAWN BY: GJG CHECKED BY: Hle

Our Reference No. 86-5-S3....

LOG OF BOREHOLE 5 (STA 0+746.8) ..

ENCLOSURE No. 3

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA
 Method: Augering (Hollow-Stem)
 Diameter: 150 mm O.D. - 85 mm I.D.
 Date: July 3, 1986

ELEVATION E	DEPTH E	SUBSURFACE PROFILE	SYMBOL	GROUND WATER	SAMPLES		PENETRATION RESISTANCE Blows/30 cm					WATER CONTENT %		UNIT WEIGHT t/m ³	REMARKS	
					NUMBER	TYPE	N	UNDRAINED SHEAR STRENGTH kPa					PLASTIC LIMIT			LIQUID LIMIT
								FIELD VANE TEST & SENSITIVITY								
								20	40	60	80	100				
								50	100	150	200	250				
87.22	0	GROUND SURFACE														
		firm moist to compact	F		1	SS	12								Groundwater in open borehole	
	1	mixture of silty clay, sand gravel, trace organics			2	SS	32								Date Elev. July 3 77.82 m July 14 77.42 m	
	2	soft to firm	F		3	SS	4									
	3	silty clay with embedded sand & gravel			4	SS	9									
	4	compact sand, silty clay, gravel	F		5	SS	23									
	5	large limestone & concrete rubble	F		6	SS	50+								SS6 - High 'N' caused by rubble	
	6	(FILL)														
80.82	640	banded brown-grey & grey														
	7	moist			7	SS	39									
	8	very stiff stiff			8	SS	28									
	9	varved SILTY CLAY			9	SS	15									
	10	stiff firm			10	SS	9									
	11	interlayered CLAYEY SILT			11	SS	7								▼ → shear strength exceeds capacity of penetrometer. + 'N' - penetration less than 0.3 m	
75.91	11.31	soft embedded wet gravel														
		END OF BOREHOLE														
		Auger Refusal (inferred bedrock)														

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DRAWN BY: GJG CHECKED BY:

Our Reference No. B6-5-S3....

LOG OF BOREHOLE .6 (STA 0+793.6) .

ENCLOSURE No.4.....

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA
 Method: Augering (Hollow-Stem)
 Diameter: 150 mm O.D. - 85 mm I.D.
 Date: July 2, 1986

ELEVATION E	DEPTH E	SUBSURFACE PROFILE DESCRIPTION	SYMBOL LOG	GROUND WATER	SAMPLES		PENETRATION RESISTANCE $\frac{\text{Blows}}{30 \text{ cm}}$					WATER CONTENT %		UNIT WEIGHT LN/m ³	REMARKS
					NUMBER	TYPE	UNDRAINED SHEAR STRENGTH kPa					PLASTIC LIMIT	LIQUID LIMIT		
							20 40 60 80 100								
							UNDRAINED SHEAR STRENGTH kPa								
							PENETROMETER - COMPR. TEST								
							FIELD VANE TEST & SENSITIVITY								
							50 100 150 200 250								
							20 40 60								
80.31	0	GROUND SURFACE													
		highly organic													Groundwater in open borehole.
		banded													Date Elev.
	1	brown-grey & grey trace organics			1	SS	22								July 2 78.48
	2	very stiff varved SILTY CLAY			2	SS	21								July 14 79.01
	3	stiff moist			3	SS	12								
	4	firm moist to wet			4	SS	6								
	4	interlayered CLAYEY SILT													▼ → shear strength exceeds capacity of penetrometer
75.36	4.95	END OF BOREHOLE Auger Refusal (inferred bedrock)													
	6														
	7														
	8														
	9														
	10														
	11														

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DRAWN BY: GJG CHECKED BY: *AG*

Our Reference No. 86-5-53.....

LOG OF BOREHOLE .7 (STA. Q+823) ..

ENCLOSURE No. 5

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA
 Method: Augering (Hollow-Stem)
 Diameter: 150 mm O.D. - 85 mm I.D.
 Date: June 25, 1986

ELEVATION E	DEPTH E	SUBSURFACE PROFILE	SYMBOL	GROUND WATER	SAMPLES		PENETRATION RESISTANCE $\frac{Blows}{30\text{ cm}}$					WATER CONTENT %		UNIT WEIGHT $\frac{1}{m^3}$	REMARKS
					NUMBER	TYPE	20	40	60	80	100	PLASTIC LIMIT	NATURAL LIMIT		
79.44	0	GROUND SURFACE													
		highly organic													Groundwater in open borehole
	1	very moist stiff			1	SS	28								Date July 14
	2	banded brown-grey & grey			2	SS	15								Elev. 75.74
	3	varved SILTY CLAY			3	SS	13								
	4	stiff			4	SS	9								
	4.57	wet interlayered CLAYEY SILT embedded gravel			5	TW	-							20.9	shear strength exceeds capacity of penetrometer
74.87	5	END OF BOREHOLE Auger Refusal (inferred bedrock)													
	6														
	7														
	8														
	9														
	10														
	11														

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Our Reference No. 86-5-S3.....

LOG OF BOREHOLE .8 (STA 0+855) . .

ENCLOSURE No.6.....

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA
 Method: Augering (Hollow-Stem); Diamond Drilling
 Diameter: 150 mm O.D. - 85 mm I.D.; 60 mm O.D. - 47 mm I.D.
 Date: July 2, 1986

ELEVATION E	DEPTH E	SUBSURFACE PROFILE	SYMBOL	GROUND WATER	SAMPLES			PENETRATION RESISTANCE Blows/30cm					WATER CONTENT %		UNIT WEIGHT t/m ³	REMARKS
					NUMBER	TYPE	Blows/30cm	20	40	60	80	100	UNDRAINED SHEAR STRENGTH kPa PENETROMETER COMPR. TEST FIELD VANE TEST & SENSITIVITY	PLASTIC NATURAL LIQUID LIMIT Wp WL		
80.64	0	GROUND SURFACE														
		loose moist to silty clay, firm sand, gravel, trace organics (FILL)			1	SS	12									Groundwater in open borehole
79.59	1.05	firm														Date Elev. July 14 78.29
	2	trace organics			2	SS	23									
	3	very moist stiff			3	SS	24									
	4	varved SILTY CLAY			4	SS	22									
	5	stiff			5	SS	15									
	6	soft wet interlayered CLAYEY SILT embedded gravel			6	SS	10									
74.85	5.79	grey			7	SS	4									
	6	fine grained LIMESTONE BEDROCK occ. shale partings			8	BQ	100% REC									▼ → shear strength exceeds capacity of penetrometer
73.08	7.56				RUN I	RC	99% RQD									RC - Rock Core REC - Rock Core Recovery RQD - Rock Quality Designation
	8	END OF BOREHOLE														
	9															
	10															
	11															

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DRAWN BY: GJG CHECKED BY: *THC*

Our Reference No. 86-5-S3.....

LOG OF BOREHOLE 9 (STA 9+925)...

ENCLOSURE No.7.....

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA
 Method: Augering (Hollow-Stem)
 Diameter: 150 mm O.D. - 85 mm I.D.
 Date: July 1, 1986

ELEVATION E	DEPTH E	SUBSURFACE PROFILE		SYMBOL	GROUND WATER	SAMPLES			PENETRATION RESISTANCE $\frac{\text{Blows}}{30 \text{ cm}}$					WATER CONTENT %		UNIT WEIGHT LN/m ³	REMARKS
		DESCRIPTION	NUMBER			TYPE	N	UNDRAINED SHEAR STRENGTH kPa					PLASTIC LIMIT	LIQUID LIMIT			
								PENETROMETER - COMPR. TEST									
									50	100	150	200	250	Wp	Wl		

80.71	0	GROUND SURFACE																
		firm moist																Groundwater in open standpipe
	1	to compact mixture of large 1st. rubble, concrete, asphalt, with silty clay, sand, gravel, wood frags., misc. constr. debris, some organics	F	1	SS	50+												Date Elev. July 14 78.79 m
	2		F	2	SS	50+												SS 1 & 2 - High 'N' caused by rubble
	3	(FILL)	F	3	SS	16												
	4	banded brown-grey & grey		4	SS	29												
77.05	3.66	firm moist		5	SS	13												
	5	to stiff		6	TW	-												20.9
	6	soft wet highly organic clayey silt		7	SS	5												
	7	grey		8	TW	-												18.6
	8	soft moist to firm wet		9	SS	4												
	9	varved SILTY CLAY		10	SS	15												
	10	interlayered CLAYEY SILT		11	TW	-												20.3
	11	stiff to very stiff																

Borehole continues
 on Enclosure 8

GEOCOR ENGINEERING INC.

DRAWN BY: GJG CHECKED BY: TAC

Our Reference No. 86-5-S3....

LOG OF BOREHOLE .9 (STA 0+925) cmt'd.

ENCLOSURE No. 8.....

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA
 Method: Augering (Hollow-Stem)
 Diameter: 150 mm O.D. - 85 mm I.D.
 Date: July 1, 1986

SUBSURFACE PROFILE										PENETRATION RESISTANCE					WATER CONTENT %			UNIT WEIGHT LN/m ³	REMARKS
ELEVATION E	DEPTH E	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows/0.3 m	20 40 60 80 100					PLASTIC NATURAL LIQUID LIMIT LIMIT						
								UNDRAINED SHEAR STRENGTH kPa PENETROMETER • COMPR. TEST FIELD VANE TEST & SENSITIVITY					Wp ——— Wl						
								50 100 150 200 250					20 40 60						

68.71	12	(For soil conditions above, see Encl. 7)			12	SS	10	⊙																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																</
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GEOCOR ENGINEERING INC.

DRAWN BY: GJG. CHECKED BY: *AKP*

Our Reference No. 86-5-S3....

LOG OF BOREHOLE 10 (STA 1+075)...

ENCLOSURE No. 9.....

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA
 Method: Augering (Hollow-Stem)
 Diameter: 150 mm O.D. - 85 mm I.D.
 Date: June 24, 1986

ELEVATION E	DEPTH E	SUBSURFACE PROFILE		SAMPLES		PENETRATION RESISTANCE Blows/0.3 m					WATER CONTENT %		UNIT WEIGHT LN/m ³	REMARKS
		DESCRIPTION	SYMBOL	NUMBER	TYPE	UNDRAINED SHEAR STRENGTH kPa PENETROMETER COMPR. TEST FIELD VANE TEST & SENSITIVITY	20	40	60	80	100	PLASTIC LIMIT Wp	LIQUID LIMIT Wl	
79.82	0	GROUND SURFACE												
	0	firm moist to compact mixture of large 1st. rubble, concrete, asphalt with silty clay, sand (FILL)	SEAL											Groundwater in open standpipe
	1													Date Elev.
	2	banded brown-grey & grey very moist stiff		1	SS	26								July 14 78.08 m
77.82	3			2	SS	29								
	4	varved SILTY CLAY	STANDPIPE	3	SS	28								
	5	occ. silt and fine sand seam		4	SS	20								
	6	firm to stiff		5	SS	11								
	7	grey soft wet		6	TW	-								19.3
	8	interlayered		7	SS	3								
	9	CLAYEY SILT		8	TW	-								18.1
	10			9	SS	1								
	11													

Borehole continues
 on Enclosure 10

GEOCOR ENGINEERING INC.

DRAWN BY: G J G CHECKED BY: JAG

Our Reference No. 86-5-93.....

LOG OF BOREHOLE 10 (STA 1+0.75) cont'd.

ENCLOSURE No.10.....

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA

Method: Augering (Hollow-Stem)
 Diameter: 150 mm O.D. - 85 mm I.D.
 Date: June 24, 1986

ELEVATION E	DEPTH E	SUBSURFACE PROFILE DESCRIPTION	SYMBOL	GROUND WATER	SAMPLES		PENETRATION RESISTANCE Blows/30"					WATER CONTENT %			UNIT WEIGHT LN/m ³	REMARKS
					NUMBER	TYPE	UNDRAINED SHEAR STRENGTH kPa	UNDRAINED SHEAR STRENGTH kPa	UNDRAINED SHEAR STRENGTH kPa	UNDRAINED SHEAR STRENGTH kPa	UNDRAINED SHEAR STRENGTH kPa	PLASTIC LIMIT	NATURAL LIMIT	LIQUID LIMIT		
67.82	12	(For soil conditions above, see Encl. 9)			10	SS	1									
	13	grey														
	14	soft wet														
	15	varved SILTY CLAY														
	16	interlayered CLAYEY SILT			11	TW	-								19.9	TW11 - Consolidation test, see Encl. 14. Triaxial test, see Encl. 15
	17															
	18															
	19	embedded sand and gravel			12	SS	3									▼ → shear strength exceeds capacity of penetrometer
59.70	20	END OF BOREHOLE														
	20.12	Auger Refusal (inferred bedrock)														
	21															
	22															
	23															

GEOCOR ENGINEERING INC.

DRAWN BY: GJG CHECKED BY: *PHG*

Our Reference No. 86-5-S3.....

LOG OF BOREHOLE. II (STA 1+240) ..

ENCLOSURE No. 11

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA
 Method: Augering (Hollow-Stem)
 Diameter: 150 mm O.D. - 85 mm I.D.
 Date: June 24, 1986

SUBSURFACE PROFILE		SAMPLES			PENETRATION RESISTANCE $\frac{\text{Blows}}{30 \text{ cm}}$					WATER CONTENT %		UNIT WEIGHT $\frac{\text{kg}}{\text{m}^3}$	REMARKS
ELEVATION E	DEPTH E	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows/30 cm	UNDRAINED SHEAR STRENGTH $\frac{\text{kPa}}{\text{cm}}$	UNDRAINED SHEAR STRENGTH $\frac{\text{kPa}}{\text{cm}}$	PLASTIC LIMIT	NATURAL LIMIT		
								20	40	60	80	100	
								UNDRAINED SHEAR STRENGTH $\frac{\text{kPa}}{\text{cm}}$	UNDRAINED SHEAR STRENGTH $\frac{\text{kPa}}{\text{cm}}$	PLASTIC LIMIT	NATURAL LIMIT		
								50	100	150	200	250	
								20	40	60			

80.65	0	GROUND SURFACE											
	1	banded brown-grey & grey											
	2	very stiff moist			1	SS	27						
	3	varved SILTY CLAY			2	SS	30						
	4	occ. silt and fine sand seams											
	5	firm to stiff			3	SS	10						
	6	interlayered CLAYEY SILT			4	SS	13						
	7												
	8	soft wet embedded gravel			5	SS	18						
72.27	8.38	END OF BOREHOLE Auger Refusal (inferred bedrock)											
	9												
	10												
	11												

Groundwater in open borehole

Date	Elev.
June 25	80.65 m
July 14	80.65 m

▼ → shear strength exceeds capacity of penetrometer

Our Reference No. 86-5-S3.....

LOG OF BOREHOLE . 12 (STA 1+340) ..

ENCLOSURE No. 12

CLIENT: Corporation of the City of Kingston

PROJECT: Proposed Overpass Grade Separation

LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.

DATUM ELEVATION: Geodetic

DRILLING DATA

Method: Augering (Hollow-Stem)

Diameter: 150 mm O.D. - 85 mm I.D.

Date: June 24, 1986

SUBSURFACE PROFILE		SAMPLES		PENETRATION RESISTANCE $\frac{\text{Blows}}{0.3 \text{ m}}$					WATER CONTENT %			REMARKS
ELEVATION E	DEPTH E	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N	Blows/0.3 m	UNDRAINED SHEAR STRENGTH kPa	PLASTIC LIMIT	NATURAL LIQUID LIMIT	
									20 40 60 80 100	Wp	Wl	
									50 100 150 200 250			
83.82	0	GROUND SURFACE										
		banded brown-grey & grey										Groundwater in open borehole
	1	stiff to very stiff										Date Elev.
		moist										June 25 83.01 m
	2	varved SILTY CLAY			1	SS	24					July 14 82.42 m
	3	interlayered CLAYEY SILT			2	SS	19					
	4	occ. silt and fine sand seams										
	5	firm wet embedded gravel sand seams			3	SS	18					▼ → shear strength exceeds capacity of penetrometer
78.18	5.64	END OF BOREHOLE Auger Refusal (inferred bedrock)										
	6											
	7											
	8											
	9											
	10											
	11											

Our Reference No. 86-5-S3.....

LOG OF BOREHOLE . 13 (STA (+440)).

ENCLOSURE No. 13.....

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

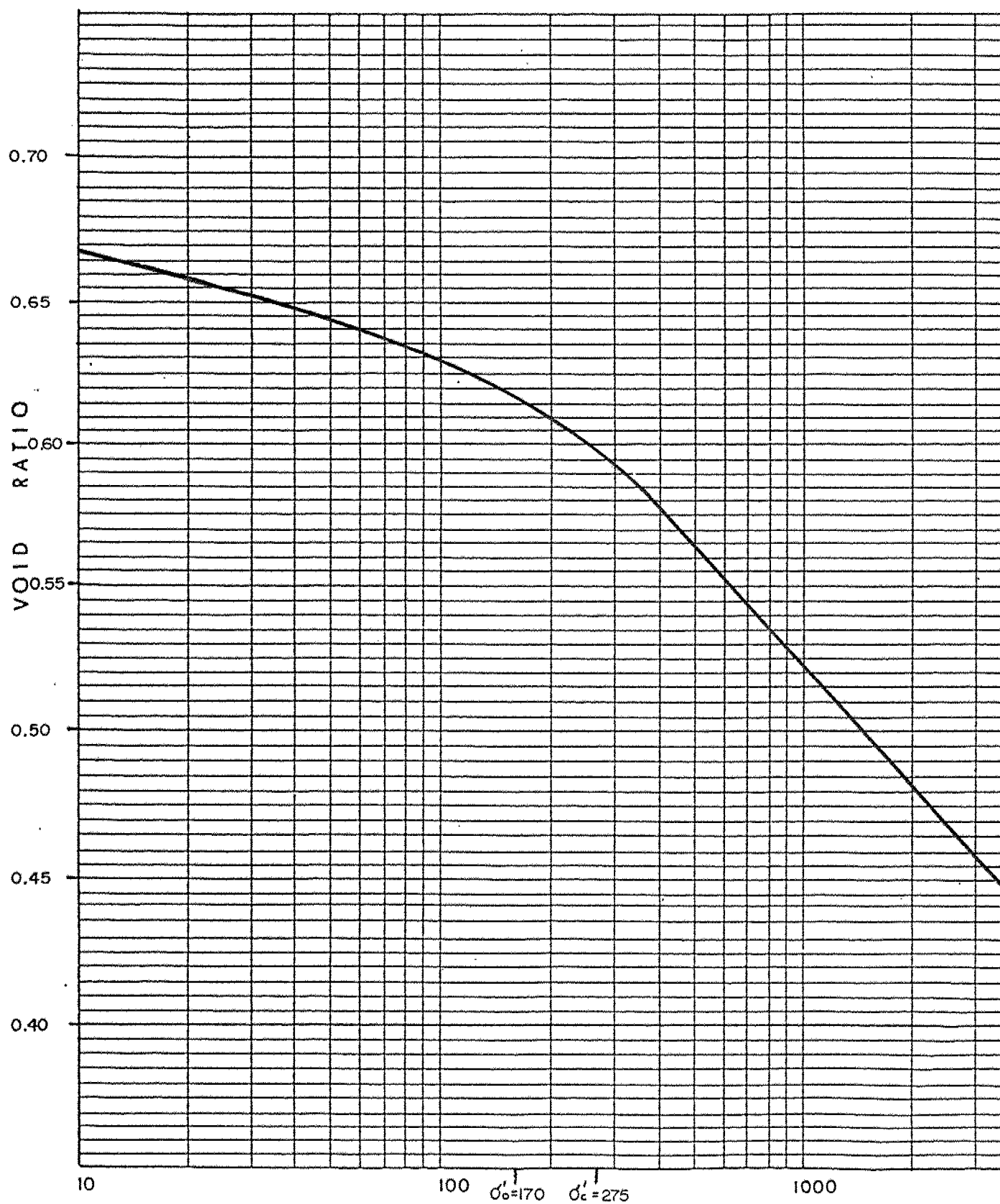
DRILLING DATA
 Method: Augering (Hollow-Stem)
 Diameter: 150 mm O.D. - 85 mm I.D.
 Date: June 23, 24, 1986

SUBSURFACE PROFILE		SAMPLES		PENETRATION RESISTANCE $\frac{Blow}{303E}$					WATER CONTENT %		UNIT WEIGHT $\frac{LN}{m^3}$	REMARKS						
ELEVATION E	DEPTH E	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows/303m	20	40	60			80	100	UNDRAINED SHEAR STRENGTH $\frac{1}{2}Po$ PENETROMETER * COMPR. TEST FIELD VANE TEST & SENSITIVITY	PLASTIC NATURAL LIQUID LIMIT	Wp	Wl
								50	100	150	200	250			20	40	60	
86.04	0	GROUND SURFACE																
	1	banded brown-grey & grey																Groundwater in open borehole
	2	stiff to very stiff			1	SS	17											
	3	varved SILTY CLAY			2	SS	7											June 25 85.13 m
	4	firm																July 14 85.04 m
	5	interlayered CLAYEY SILT			3	SS	8											<div>↑</div> <div>↓</div> <div>June 23</div> <div>June 24</div>
	6	firm to stiff			4	SS	25											
79.03	7.01	embedded sand and gravel																
	8	END OF BOREHOLE																
	9	Auger Refusal (inferred bedrock)																
	10																	
	11																	

GEOCOR ENGINEERING INC.

DRAWN BY: GJG CHECKED BY:

CONSOLIDATION TEST



PRESSURE kPA

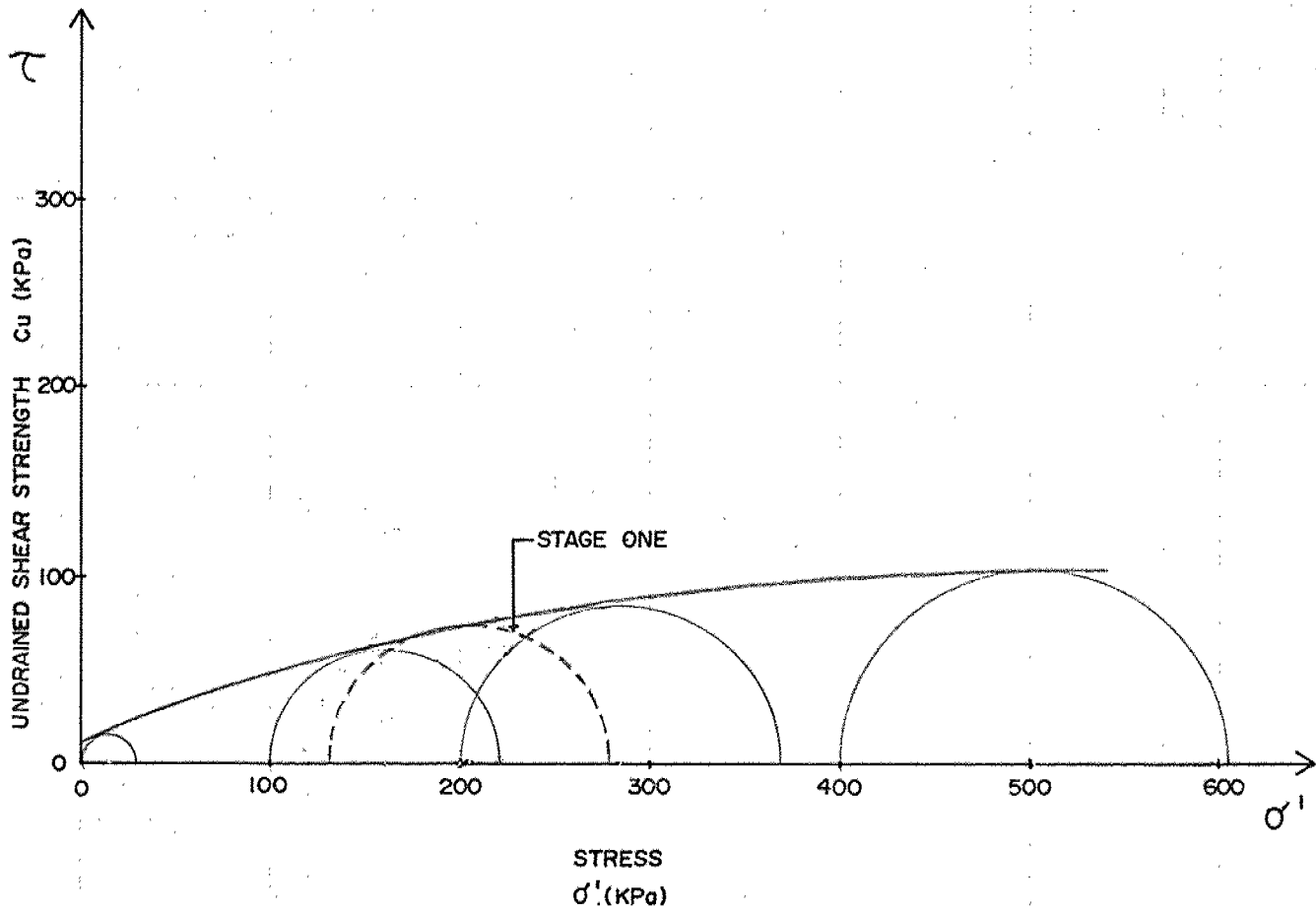
BHNo. 10
SAMPLE: TWII
DEPTH : 15.5m

L.L. = 36 %
P.L. = 17 %
P.I. 18
W = 38 %



Prep. By GJG

PLOT of C_u vs. σ'_1



Geocres 31C-156

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED OVERPASS SEPARATION
SIR JOHN A. MACDONALD BOULEVARD EXTENSION
AT THE C.N.R.
KINGSTON, ONTARIO

No 2 Dec. 86

GeoCor Engineering Inc.
Reference No. 86-5-S3
December 1986

Prepared For:

The Corporation of the City of Kingston
c/o Totten Sims Hubicki Associates (1981) Limited
675 Bath Road, LaSalle Park Plaza
Kingston, Ontario
K7M 4X2

GEOCOR ENGINEERING INC.

Geotechnical, Materials Engineers & Geologists



11 Parkwood Place
Kingston, Ontario, Canada, K7M 2C9
Tel. (613) 544-6474

Reference No. 86-5-S3

December 8, 1986

The Corporation of the City of Kingston
c/o Totten Sims Hubicki Associates (1981) Limited
675 Bath Road
LaSalle Park Plaza
Kingston, Ontario
K7M 4X2

Attention: Mr. Peter A. Smith, P.Eng.
Branch Manager

Re: Supplemental Geotechnical Investigation
Proposed Overpass and Approaches
Sir John A. MacDonald Boulevard Extension at C.N.R.
Kingston, Ontario

Dear Sir:

This submission details the results of the most recent geotechnical investigation for the above-noted project based on our letter proposal October 14, 1986 and authorization to proceed with the same, received from your office dated November 4, 1986.

The present study concentrated on determining the subsurface conditions along the alignment of the existing northeast trunk sewer where the north approach of the proposed grade separation is to be constructed. The data obtained from

this exercise was used to estimate the amount of potential settlement that can be expected along the alignment of the sewer as a result of the proposed approach construction. As well, the borehole information provided additional parameters to assist in securing a more detailed deep foundation design in the vicinity of the proposed overpass north abutment location.

In conjunction with our letter of November 5, 1986 detailing the present status and future backfilling programme of the north and south earth approaches, further comments are directed herein in this regard.

Laboratory testing conducted on selected samples retrieved from the former and present studies, were undertaken to better define consolidation time interims for the stage construction programme of the north approach. The information gained from the analyses has resulted in the recommendation that a subsurface drainage system be implemented into the north approach construction sequence. Preliminary design criteria are outlined in this report with regards to suggested methodologies for the system.

FIELD AND LABORATORY WORK

The exploratory drilling program was conducted during the interim of November 5 to 7, 1986 and consisted of three originally scheduled boreholes (BH 14, 15 and 16), which were put down at pre-selected locations along the alignment of the NE trunk sewer and, an additional hole (BH 17), which was drilled adjacent the NW corner of the proposed overpass north abutment location. The extra boring was necessitated to better define the bedrock surface profile in the latter mentioned area. The borehole locations are shown on the accompanying drawing, Borehole Location Plan & Stratigraphic Section, Enclosure 1.

Cuts in the existing embankment fills near the boring locations were also made by a bulldozer which was used to provide site access for the drill rig. These cuts were visually logged as to composition and compaction.

The boreholes were advanced using a truck-mounted mobile power drill, utilizing hollow-stem augers and conventional soil and bedrock sampling equipment to penetrate the substratum between 7.5 to 18.5 m below grade. Both disturbed and undisturbed samples were recovered through the overburden which was accompanied by penetration and shear strength tests. Bedrock was cored in each hole using BQ equipment.

The field programme was supervised by members of our technical staff who assigned geodetic collar elevations to the boreholes; logged, labelled and stored the retrieved samples for subsequent transfer to our laboratory; monitored ground-water levels; and, photographed various features of the existing north approach backfill materials.

All soil samples taken to our laboratory were re-examined by a soils engineer and classified by visual and tactile methods in accordance with the Unified Soil Classification System. A geologist from our staff re-logged the recovered bed-rock core.

Natural moisture contents and unit weights of representative samples were determined by standardized laboratory methods. The compressibility and consolidation characteristics of the subsoil were determined by laboratory consolidation tests. The tests results are plotted on the borehole logs or are presented on separate enclosures.

SUMMARIZED SUBSURFACE CONDITIONS

The stratigraphy, as encountered in the boreholes, is illustrated on the Log of Borehole sheets comprising Enclosures 2a through 5. Enclosure 1 duplicates some of the borehole information.

Borehole 14 and 16 were located beyond the existing limits of the most recent backfilling operation for the north approach, while BH 15 and 17 penetrated this

latest fill deposit in the upper section of the holes. As encountered elsewhere on the development site, this recent fill material is comprised of a miscellaneous mixture of large concrete, limestone and asphalt slabs intermixed with clay, sand, gravel, organics and construction debris. Described as in a loose to firm condition, the fill was observed to be moist to wet.

Below this upper fill layer in BH 15 and encountered as the surficial deposit in BH 14 and 16, another man-made deposit was uncovered in the borings. Associated with the backfilling operation during the construction of the NE trunk sewer, this fill material is comprised predominantly of silt and clay with some minor amounts of sand, gravel and organic matter. The penetration indices ('N'-values) indicates this material is in a firm to stiff consistency.

The predominate natural soil type encountered in all of the borings is a varved silty clay interlayered with clayey silt which extends in a wedged-shape pattern from the fill interface to the underlying bedrock surface. The stratum generally exhibits a desiccated upper stiff to very stiff section, however, below this crustal layer, the shear strength decreases to a firm then soft consistency where the deposit is thickest.

Unexpectedly, the bedrock surface drops in elevation in a east to west direction rather than vice versa as was anticipated in this most recently investigated area of the project site. Encountered at its greatest depth in BH 14 (ie. 17.1 m below grade, Elev. 61.1 m), the bedrock (granite in this location) steps up abruptly towards the proposed centerline of the embankment where in BH 15 the rock (limestone in this location) was intersected at 9.9 m below ground surface or Elev. 70.1 m. Beyond BH 15, towards the east, the bedrock surface profile rises at a more gradual rate and was penetrated at its its shallowest investigated depth of 6.3 m below grade (Elev. 71.7 m) in BH 16 (limestone in this location).

The groundwater level in this location of the site is somewhat lower than that measured elsewhere on the project and is probably depressed by the presence of

the trunk sewer. At the time of the field work programme, the level in this investigated area was recorded at approximately Elev. 76.5 ± 0.2 m.

Further details of the subsurface conditions as they pertain to the development of the various facets of this latest design phase of the project are discussed in the subsequent section of this report.

DISCUSSION AND RECOMMENDATIONS

In conjunction with the previously conducted investigative work and the data obtained from the latest geotechnical study, the following additional comments and recommendations are offered for the proposed overpass grade separation development:

1. The cuts made in the latest placed embankment fills and the results of the former and recent borehole information, indicate that soil materials (especially clay) make up a good percentage of the existing north approach backfill. The clay is presently in a moist to wet (saturated in some areas) condition and in our opinion generally beyond its optimum moisture content for compaction purposes. Therefore, prior to undertaking a quality control backfill compaction programme, this material will have to be air dried in an environment with $+0^{\circ}$ C temperatures, in order to achieve the desired moisture/density combination. For additional details regarding approach backfilling procedures, reference is made to our previously submitted letter report of November 5, 1986.
2. The inclination of the bedrock surface in the vicinity of the proposed overpass north abutment location suggest that the H-piles which will be used to support the north abutment structure, be provided with rock injector points to enable chipping into the bedrock. In this regard, Titus Rock Injector Pile Points, H.P.P. series or equivalent are recommended. Driving of piles equipped with injector points are to be conducted in

a predetermined set sequential manner until proper seating has been achieved.

3. As can readily be seen on Enclosure 1, the stratigraphy along the alignment of the NE trunk sewer where the north approach of the proposed grade separation is to be constructed, is non-uniform due to the deposition of natural (and later, man-made) materials of varying thicknesses along the banks of a now buried, eroded stream valley. The nature of the overburden materials suggest that differential settlement can be expected in this area due to the proposed development and, especially so, where these materials are shown to be thickest. Based on the available field data and subsequent laboratory testing of selected samples retrieved from the borings, our analyses of the estimated settlement that can be anticipated along the sewer as a result of the approach construction, is diagrammatically shown on Enclosure 9 attached. In summary, very little settlement is expected with the east half of the embankment where the overburden is of limited thickness and the bedrock surface is at shallow depths, however, this is not the case for the embankment's west half. In this latter location, it is estimated that approximately 200 mm of differential movement can be expected between the centerline and west crest of the approach when final grade has been achieved, or about 13 mm per meter of sewer length in this area. Beyond the crest, towards the west, the magnitude of movement diminishes as would be expected along the embankment side-slopes.

It is difficult to predict the extent of damage to the sewer that would result from differential settlement of the order outlined above, however, unless some form of protection is provided to compensate for this movement, the potential for open pipe joints area realistic. Generally in this situation, the solution is to create a more rigid sewer in the effected area or underpin the structure. Other techniques such as soil improvement, deflection structures, light weight embankment materials, etc. have also been used. A review of the City of Kingston's documentation concerning the construction of NE trunk sewer within the subject

development location, indicates that sections of the sewer were supposedly encased in concrete, however, this was to have occurred along areas of the proposed east half of the embankment; not the west half where the majority of movement is expected to take place.

Besides having to protect the sewer from movement caused by the consolidation process of the founding soils, the new protection system will have to take into account the added applied loads the sewer will have to sustain as a result of the proposed construction. It is estimated that the stress at the pipe surface will be increased to approximately 265 kPa due to the introduction of the embankment.

4. As outlined in our initial report, it is recommended that the construction of the north approach be conducted in stages, two in this case. Based on our most recent settlement calculations, Stage I, which will involve the placement of approach fill materials to approximately grade elevation 83.5 m, will result in a total settlement of about 500 mm before the consolidation process is completed. As well, an additional approximate 350 mm of settlement can be expected for the Stage II phase of the embankment construction.

Although the magnitude of estimated settlement is quite substantial for the two stages, it is the time interim over which the consolidation process will take place that is of greater concern. As an example, in order to achieve about 80% of the anticipated consolidation for the Stage I phase of the north approach development, it has been determined that in excess of 150 years will be required for the process to be completed. Hardly realistic. However, the slow rate of consolidation in wet, saturated clays of low permeability may be accelerated by means of vertical sand drains which shorten the drainage path within the clay. Consolidation is then due mainly to horizontal radial drainage, resulting in the faster dissipation of excess pore water pressure; vertical drainage becomes of minor importance. In theory the final

magnitude of consolidation settlement is the same, only the rate of settlement is affected.

Sand drains installed in the clay would enable the embankment to be brought into service much sooner and there would be a quicker increase in the shear strength of the clay. A degree of consolidation of the order of 80% would be desirable at the end of Stage I.

Sand drains are installed by drilling vertical boreholes through the clay layer and backfilling with a suitably-graded sand. The sand should be capable of allowing the efficient flow of water without permitting fine clay particles to be washed in. As the object is to reduce the length of drainage path, the spacing of the drains is the most important design consideration. The drains may be spaced in square, rectangular or triangular patterns.

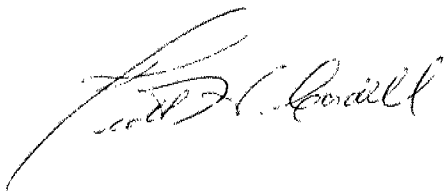
The effectiveness of sand drains can be seen in the following example. If 200 mm diameter sand drains were installed in a 1.8 m square pattern along the alignment of the effective area of the proposed north approach, then the time rate to complete 80% of the consolidation for the Stage I phase of construction would be reduced to approximately 1.4 years. Depending on the desired construction period, the pattern configurations and spacings can be adapted. Encl. 10 presents various alternatives.

The information presented above is based on limited available data and does not preclude the importance of a field instrumentation programme during the stage construction period of the north approach development. This programme is really the best method for monitoring the performance of the structure under actual site conditions (since unpredictable situations often present themselves) and for observing first hand, potential hazards. As previously mentioned, Queen's University, Civil Engineering Department has shown interest in participating in the

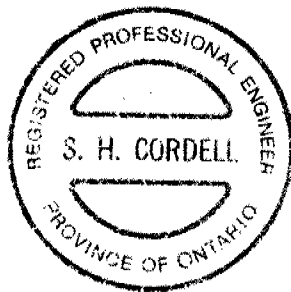
monitoring program with us to obtain case studies for research purposes.

Since this latest report introduces new facets into the design work, we are available to discuss any matter further with you, once your review is completed.

Yours very truly,
GEOCOR ENGINEERING INC.



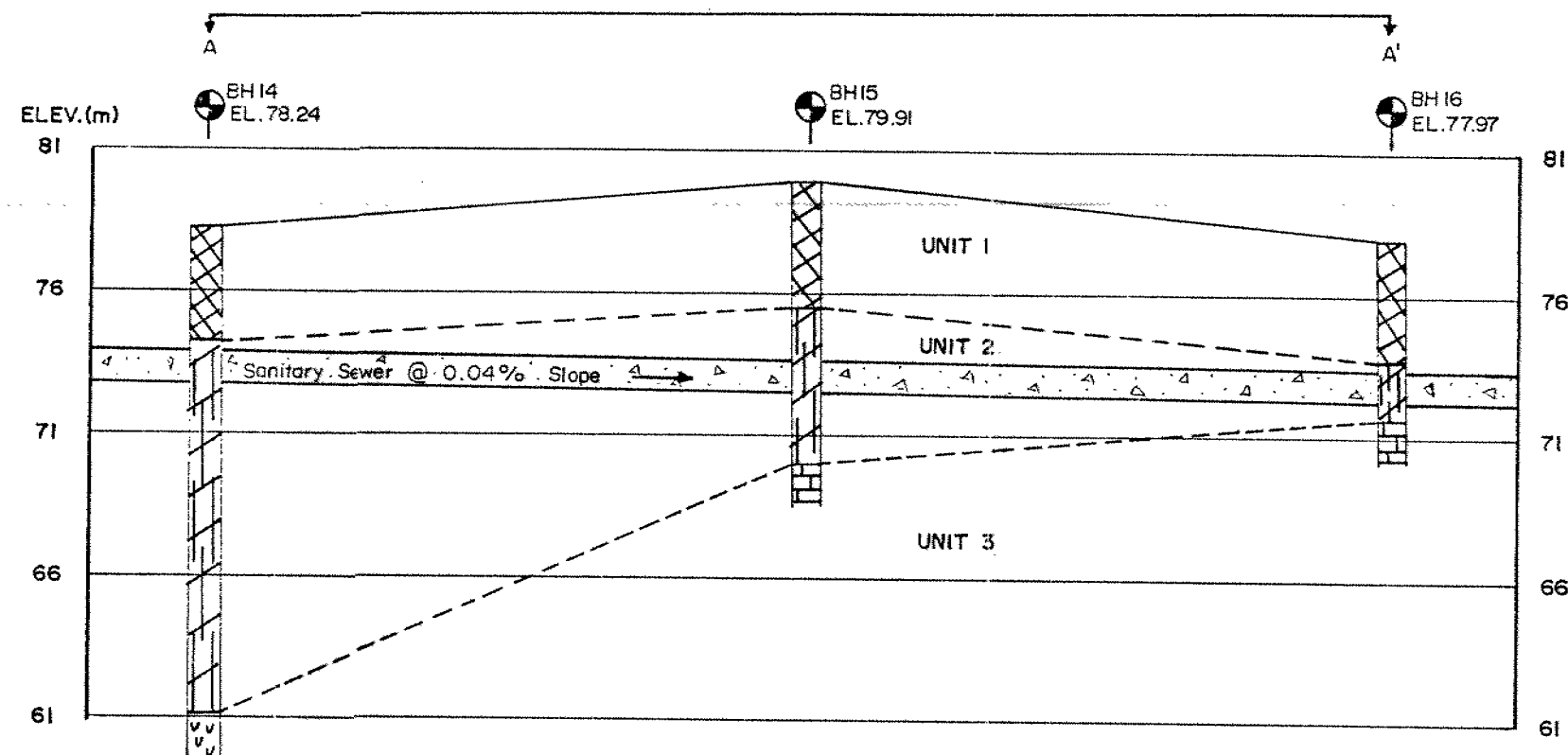
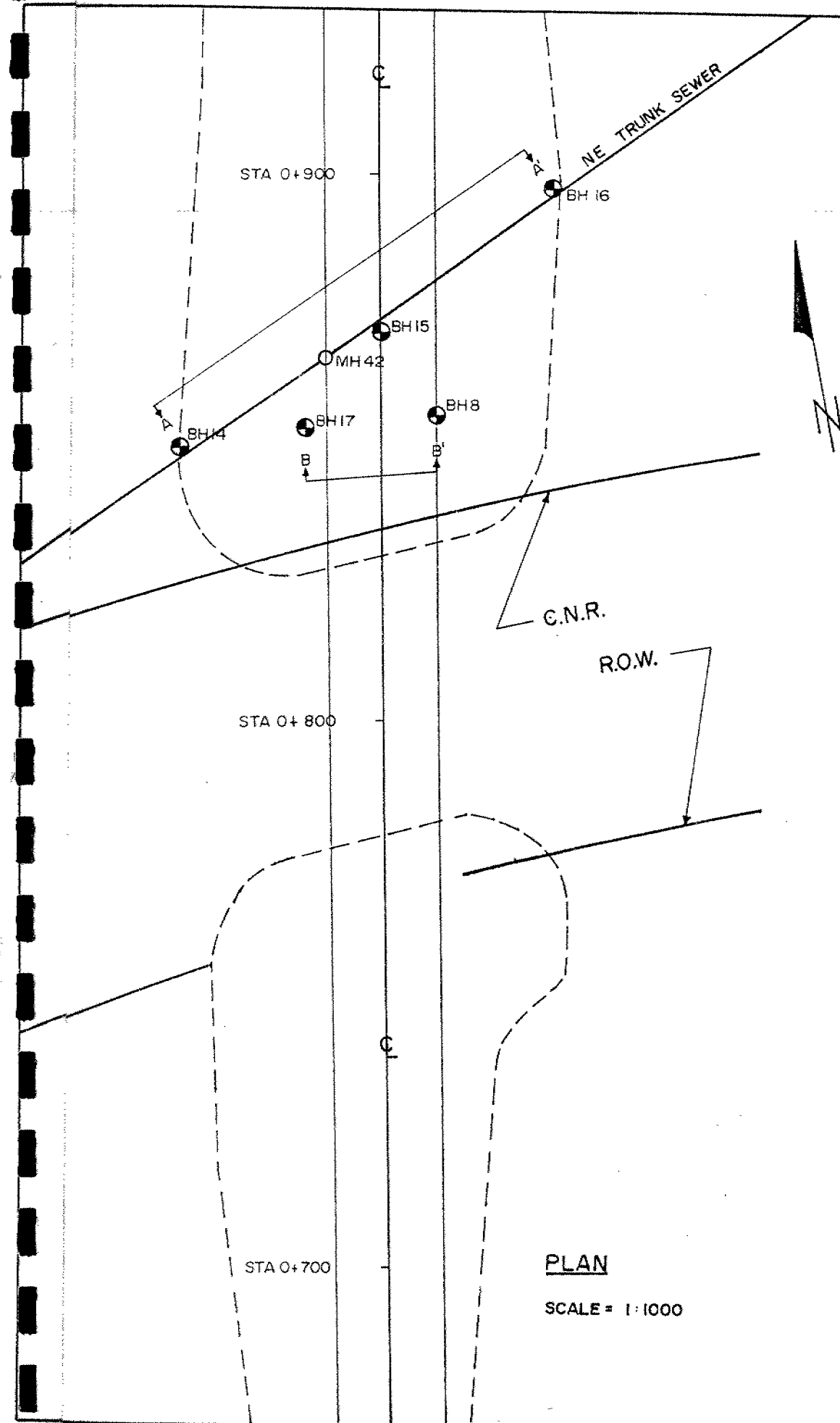
Scott H. Cordell, P.Eng.
President



SHC:jsl

Encls.

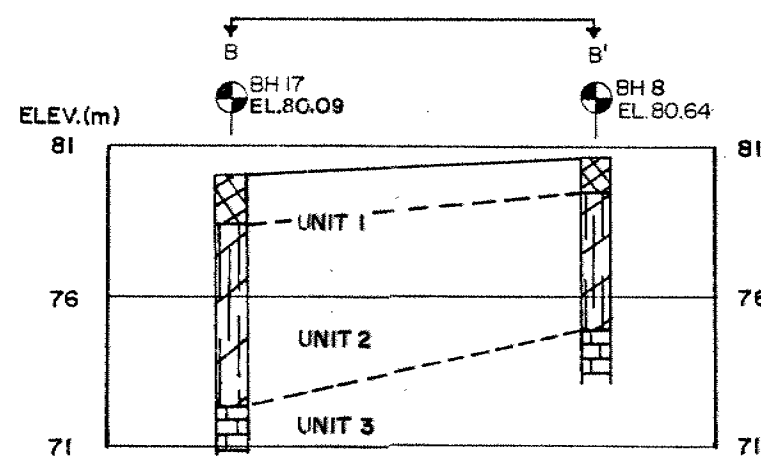
ENCLOSURES



LEGEND

- BH 8 BOREHOLE NO. & LOCATION
- MH MANHOLE
- ▨ UNIT 1 FILL
- ▧ UNIT 2 VARVED SILTY CLAY/CLAYEY SILT
- ▩ UNIT 3 BEDROCK (LIMESTONE)
- ▩ UNIT 3 BEDROCK (GRANITE)

NOTE: The boundaries between strata have been established only at Boreholes. Between Boreholes the boundaries have been interpolated.



BOREHOLE LOCATION PLAN

A

STRATIGRAPHIC SECTION

PROPOSED OVERPASS AT

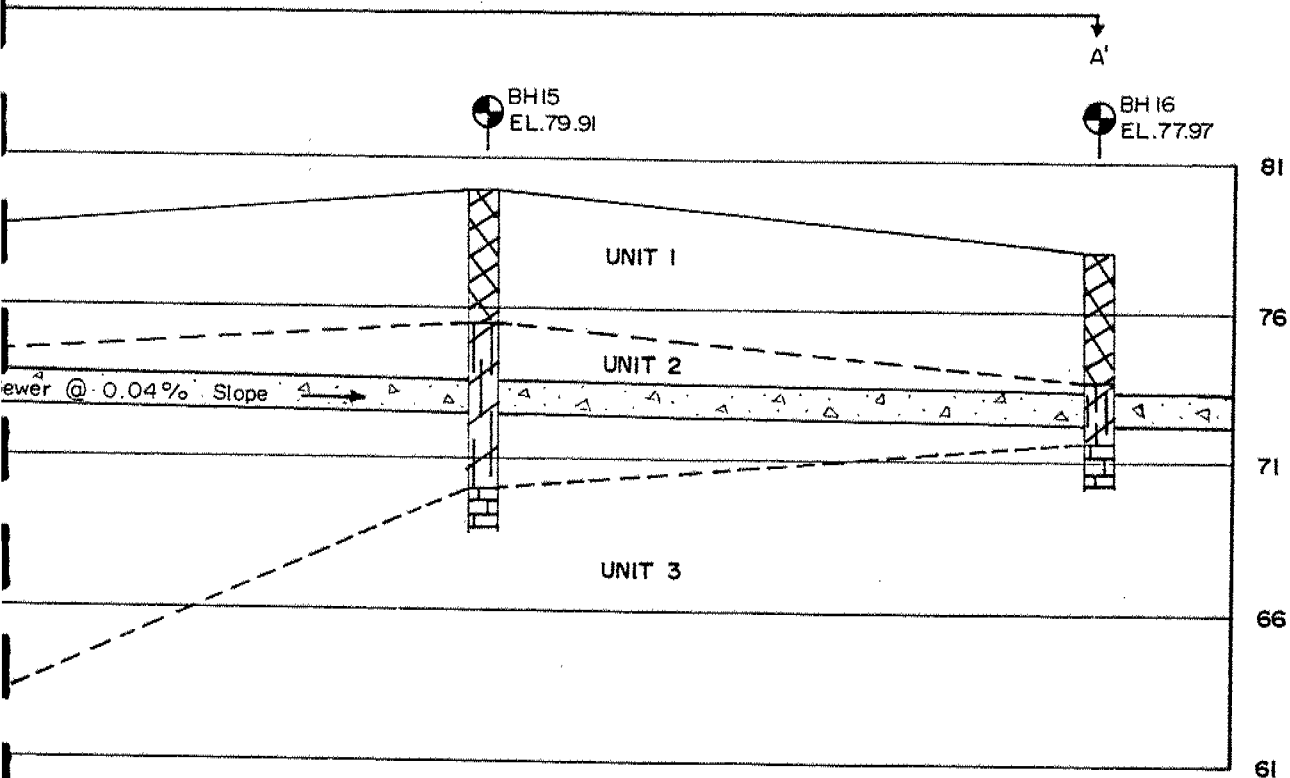
SIR J. A. MacDONALD BLVD. & C.N.R.

KINGSTON, ONTARIO

REF. NO.
96-5-S3DATE
NOV. 1986DWN BY
G.J.G.CH'D BY
G.J.G.

GEOCOR ENGINEERING INC.

NOTE: All dimensions are in metres



SECTION A - A'

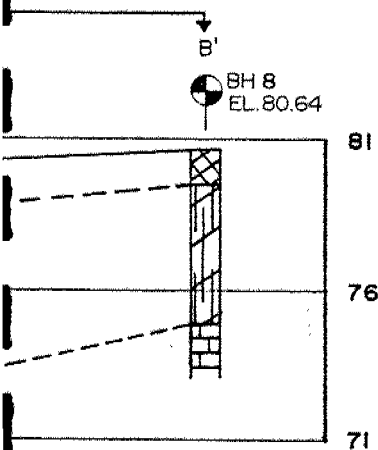
SCALE - horiz 1:500
vert 1:250

(NE TRUNK SEWER)

LEGEND

- BH 8 BOREHOLE NO. & LOCATION
- MH MANHOLE
- UNIT 1 FILL
- UNIT 2 VARVED SILTY CLAY/CLAYEY SILT
- UNIT 3 BEDROCK (LIMESTONE)
- UNIT 3 BEDROCK (GRANITE)

NOTE: The boundaries between strata have been established only at Boreholes. Between Boreholes the boundaries have been interpolated.



SECTION B - B'

horiz 1:500
vert 1:250

(ABUTMENT)

BOREHOLE LOCATION PLAN

&

STRATIGRAPHIC SECTION

PROPOSED OVERPASS AT

SIR J. A. MacDONALD BLVD. & C.N.R.

KINGSTON, ONTARIO

REF. NO.
86-S-S3

DATE
NOV. 1986

DWN BY
G J G

CH'D BY
[Signature]

GEOCOR ENGINEERING INC.

Dimensions are in metres

CLIENT: Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA

Method: Augering (Hollow-Stem); Diamond Drilling
 Diameter: 150 mm O.D. - 85 mm I.D.; 60 mm O.D. - 47 mm I.D.
 Date: November 5 & 6, 1986

SUBSURFACE PROFILE		SAMPLES		PENETRATION RESISTANCE $\frac{\text{Blows}}{100 \text{ mm}}$					WATER CONTENT %		UNIT WEIGHT LN/m ³	REMARKS
ELEVATION E	DEPTH E	DESCRIPTION	SYMBOL GROUND WATER	NUMBER	TYPE	N Blows/100 mm	UNDRAINED SHEAR STRENGTH $\frac{1}{2} P_u$ PENETROMETER - COMPR. TEST	FIELD VANE TEST & SENSITIVITY	PLASTIC NATURAL LIQUID LIMIT	Wp		
							20 40 60 80 100					
							50 100 150 200 250					

78.24	0	GROUND SURFACE																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
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Borehole 14 continues
on Enclosure 2b

GEOCOR ENGINEERING INC.

DRAWN BY: G J G CHECKED BY: *PHG*

LOG OF BOREHOLE ... 14 CONT'D ...

CLIENT: The Corporation of the City of Kingston

DRILLING DATA

PROJECT: Proposed Overpass Grade Separation

Method: Augering (Hollow-Stem); Diamond Drilling

LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.

Diameter: 150 mm O.D. - 85 mm I.D.; 60 mm O.D. - 47 mm I.D.

DATUM ELEVATION: Geodetic

Date: November 5 & 6, 1986

ELEVATION E	DEPTH E	SUBSURFACE PROFILE	DESCRIPTION	SYMBOL	GROUND WATER	SAMPLES				PENETRATION RESISTANCE <small>Blows/30 cm</small>					WATER CONTENT %			UNIT WEIGHT <small>t/m³</small>	REMARKS
						NUMBER	TYPE	N	Blows/30 cm	UNDRAINED SHEAR STRENGTH <small>kPa</small>					PLASTIC NATURAL LIQUID LIMIT				
										PENETROMETER - COMPR. TEST					LIMIT				
										FIELD VANE TEST & SENSITIVITY					LIMIT				
					50	100	150	200	250	Wp ———— Wl			20	40	60				

66.24	12		(For soil conditions above, see Encl. 2a)			9	SS	2	0																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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+ SS12 - High 'N' caused by bedrock

Nov. 5
Nov. 6

RC - Rock Core
REC - Rock Core Recovery
RQD - Rock Quality Designation

GEOCOR ENGINEERING INC.

DRAWN BY: GJG CHECKED BY: HLG

LOG OF BOREHOLE 15 STA 0+871

CLIENT: The Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA

Method: Augering (Hollow-Stem); Diamond Drilling
 Diameter: 150 mm O.D. - 85 mm I.D.; 60 mm O.D. - 47 mm I.D.
 Date: November 6, 1986

ELEVATION E	DEPTH E	SUBSURFACE PROFILE		SAMPLES		PENETRATION RESISTANCE					WATER CONTENT %		UNIT WEIGHT LN/m ³	REMARKS
		DESCRIPTION	SYMBOL	NUMBER	TYPE	Blows/0.3 m	UNDRAINED SHEAR STRENGTH kPa PENETROMETER	COMPR. TEST	FIELD VANE TEST & SENSITIVITY	PLASTIC NATURAL LIQUID LIMIT	Wp	Wl		
							20 40 60 80 100							
							50 100 150 200 250							
79.91	0	GROUND SURFACE												
	1	loose to clay, sand, firm gravel, limestone rubble concrete, asphalt	F											Groundwater level in open borehole.
		grey-brown												Date Elev.
	2	firm moist to wet stiff	F	1	SS	12								Nov. 7 75.81 m
	3	silty clay intermixed with sand, gravel, organics		2	SS	7								Nov. 11 76.86 m
	4	(FILL)	F											
75.51	4.40	banded brown-grey & grey		3	SS	16								
	5													
	6	stiff moist to very stiff		4	SS	14								
	7	varved SILTY CLAY												
	8	slightly fissured		5	TW	-							18.4	Tw5 - Consolidation test. See Encl. 8
	9	interlayered CLAYEY SILT												
	10	embedded wet sand & gravel		6	SS	16								
70.05	9.86													
	11	fine grained LIMESTONE BEDROCK		7	RC	REC 95%								RC - Rock Core REC - Rock Core Recovery RQD - Rock Quality Designation
	11.38	shale partings												
68.53	11.38	END OF BOREHOLE												

GEOCOR ENGINEERING INC.

DRAWN BY: GJG CHECKED BY: HLG

LOG OF BOREHOLE ... 16. STA Q+896

CLIENT: The Corporation of the City of Kingston
 PROJECT: Proposed Overpass Grade Separation
 LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.
 DATUM ELEVATION: Geodetic

DRILLING DATA

Method: Augering (Hollow-Stem); Diamond Drilling
 Diameter: 150 mm O.D. - 85 mm I.D.; 60 mm O.D. - 47 mm I.D.
 Date: November 7, 1986

SUBSURFACE PROFILE		PROFILE		SAMPLES		PENETRATION RESISTANCE					WATER CONTENT %		UNIT WEIGHT 1N/m ³	REMARKS	
ELEVATION E	DEPTH E	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows/0.3 m	20	40	60	80	100			PLASTIC LIMIT
UNDRAINED SHEAR STRENGTH kPa															
PENETROMETER COMPR. TEST															
FIELD VANE TEST & SENSITIVITY															
50 100 150 200 250															
20 40 60															

77.97	0	GROUND SURFACE													
		grey-brown													
		firm moist	F												
	1	to to													
		stiff wet													
	2	silty clay intermixed with sand, gravel, organics			1	SS	8								
	3	(FILL)			2	SS	6								
	4														
73.77	4.20	banded brown-grey & grey			3	TW	-								
	5	firm moist													
		to varved													
		stiff SILTY CLAY													
	6	interlayered CLAYEY SILT													
		embedded sat.			4	SS	50+								
71.67	6.30	sand & gravel													
	7	fine grained LIMESTONE BEDROCK			5	RC	REC 96%								
		shale partings				BQ	RQD 62%								
70.45	7.52	END OF BOREHOLE													
	8														
	9														
	10														
	11														

Groundwater level
in open borehole.
Date Elev.
Nov. 11 76.32 m

+ SS4 - High 'N'
caused by bedrock.
RC - Rock Core
REC - Rock Core
Recovery
RQD - Rock Quality
Designation

GEOCOR ENGINEERING INC.

DRAWN BY: GJG CHECKED BY: JHC

LOG OF BOREHOLE ... 17 STA 0+853.

CLIENT: The Corporation of the City of Kingston

DRILLING DATA

PROJECT: Proposed Overpass Grade Separation

Method: Augering (Hollow-Stem); Diamond Drilling

LOCATION: Sir John A. MacDonald Blvd. Extension & C.N.R.

Diameter: 150 mm O.D. - 85 mm I.D.; 60 mm O.D. - 47 mm I.D.

DATUM ELEVATION: Geodetic

Date: November 7, 1986

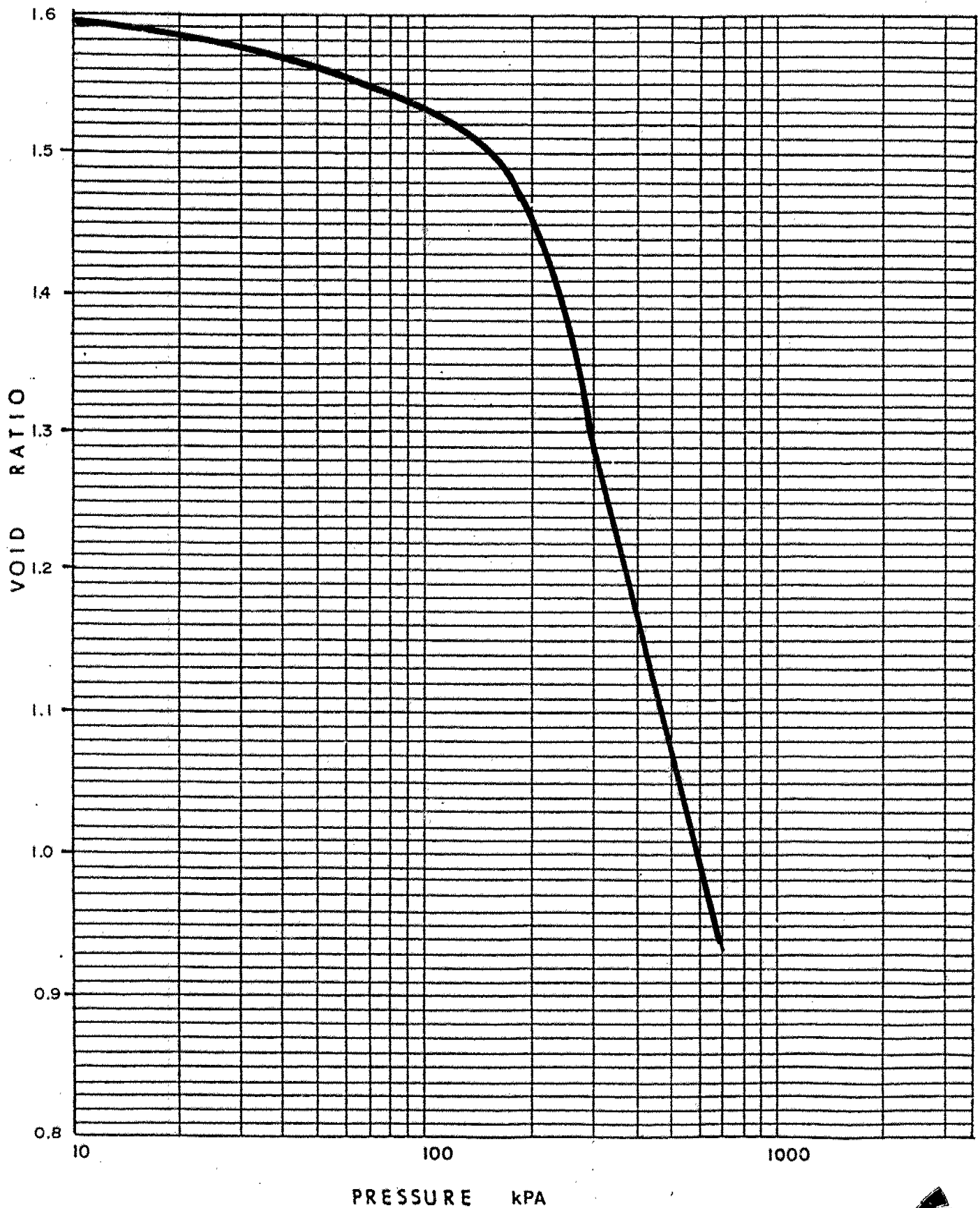
ELEVATION E	DEPTH E	SUBSURFACE PROFILE	DESCRIPTION	SYMBOL	GROUND WATER	SAMPLES		PENETRATION RESISTANCE $\frac{\text{Blows}}{103 \text{ mm}}$					WATER CONTENT %		UNIT WEIGHT LN/m ³	REMARKS		
						NUMBER	TYPE	N	Blows/103 mm	UNDRAINED SHEAR STRENGTH kPa							PLASTIC LIMIT	LIQUID LIMIT
										PENETROMETER	COMPR. TEST	FIELD VANE TEST & SENSITIVITY						
								20	40	60	80	100						
								50	100	150	200	250			20	40	60	

80.09	0	GROUND SURFACE															
		loose to firm	moist to wet														Groundwater level in open borehole.
	1	silty clay, sand, gravel, organics, rubble, asphalt (FILL)															Date Elev.
78.49	1.60	brown				1	SS	32									Nov. 11 76.34 m
	2	very stiff	moist														
		SILTY CLAY															
	3	with some fine sand				2	SS	29									
	4	banded brown-grey & grey															
	5	firm to stiff	moist			3	SS	10									
		varved SILTY CLAY															
	6	interlayered CLAYEY SILT				4	SS	8									
	7	embedded wet sand and gravel occ. cobbles & boulders				5	SS	50+									+SS5 - High 'N' caused by bedrock.
72.39	7.70	fine grained					RC	REC 95%									RC - Rock Core
		LIMESTONE BEDROCK				6		RQD 78%									REC - Rock Core Recovery
	9	shale partings															RQD - Rock Quality Designation
70.87		END OF BOREHOLE															
	10																
	11																

GEOCOR ENGINEERING INC.

DRAWN BY: GJB CHECKED BY: JH

CONSOLIDATION TEST

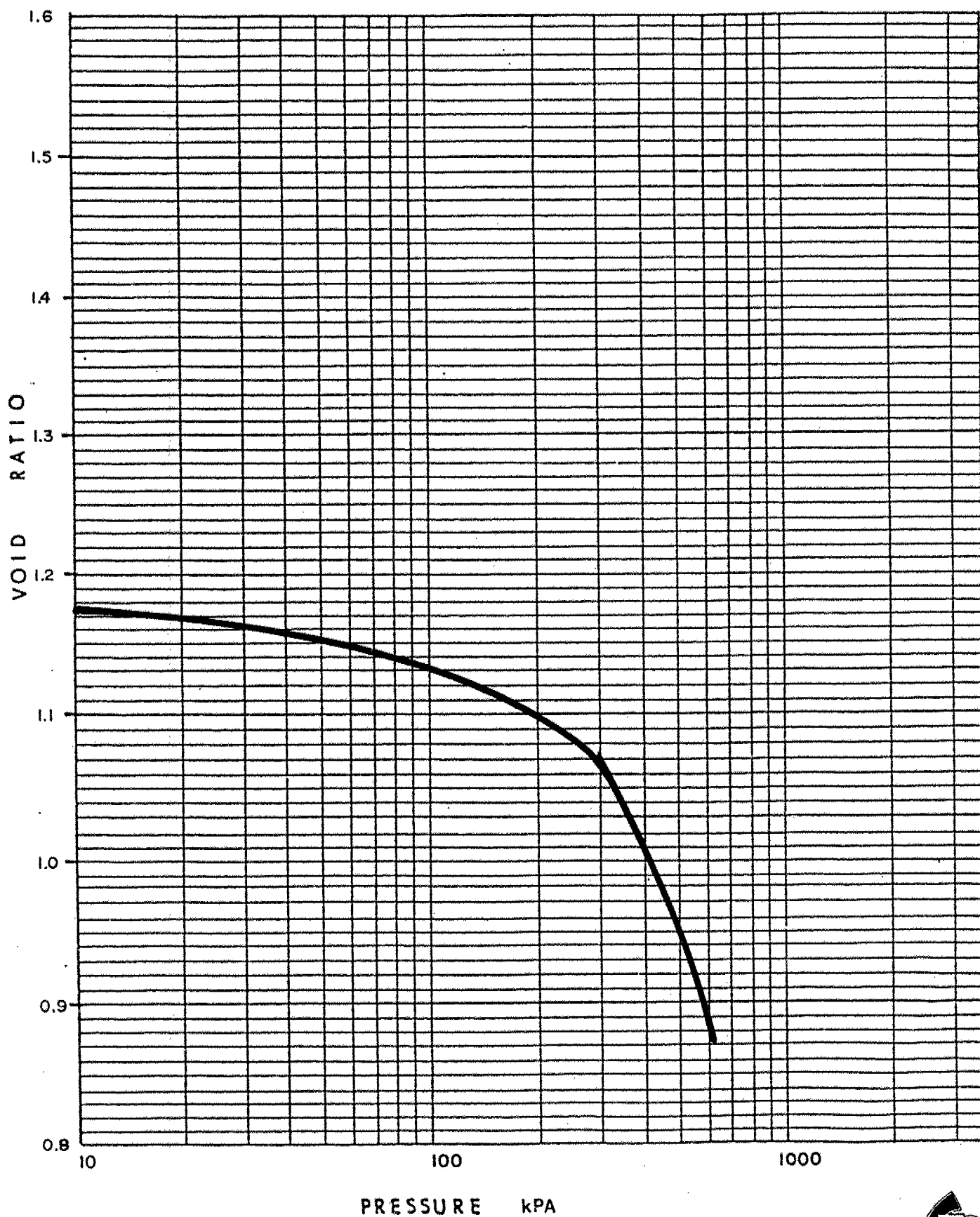


BH No. 10
SAMPLE: TW8
DEPTH : 9.5m

L.L. : - %
P.L. : - %
P.I. : -
W = 60%



CONSOLIDATION TEST

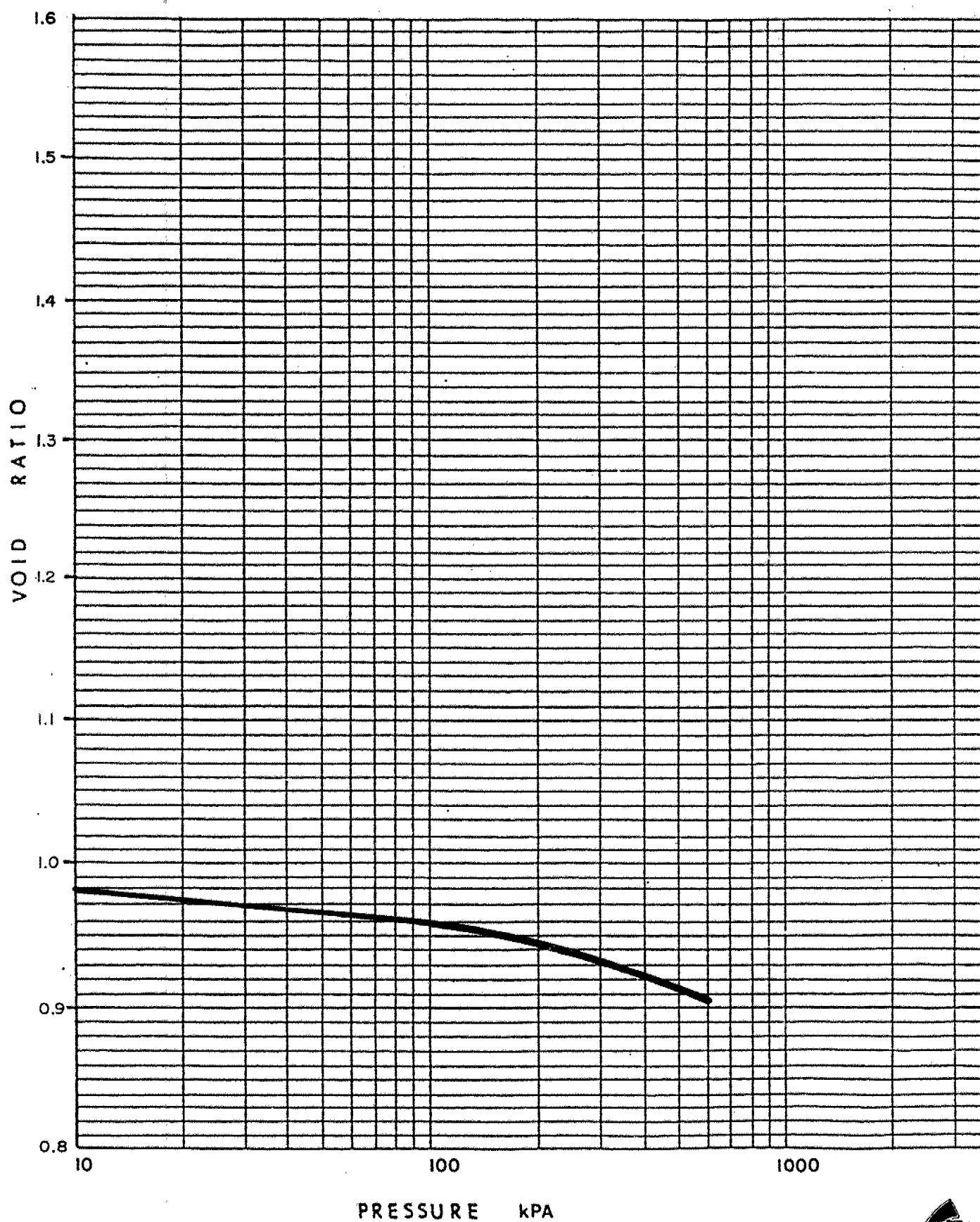


BHNo. 14
SAMPLE: TW7
DEPTH: 9.5m

L.L. = — %
P.L. = — %
P.I. = —
W = 48 %



CONSOLIDATION TEST

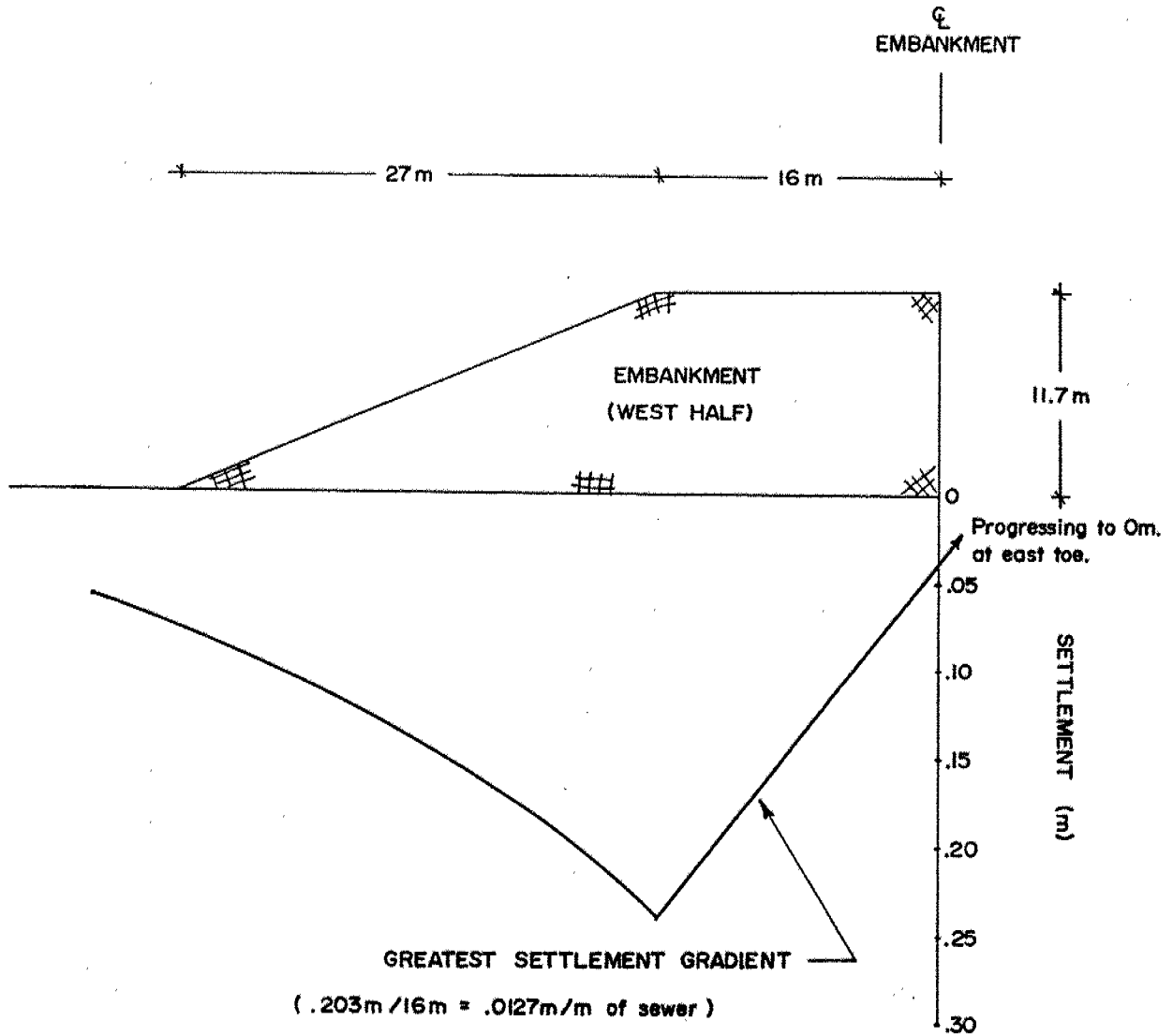


BH No. 15
SAMPLE: TW5
DEPTH: 7.9m

L.L. = — %
P.L. = — %
P.I. = —
W = 37 %



Prep. By G J G



ESTIMATED SETTLEMENT OF EMBANKMENT AT NE TRUNK SEWER

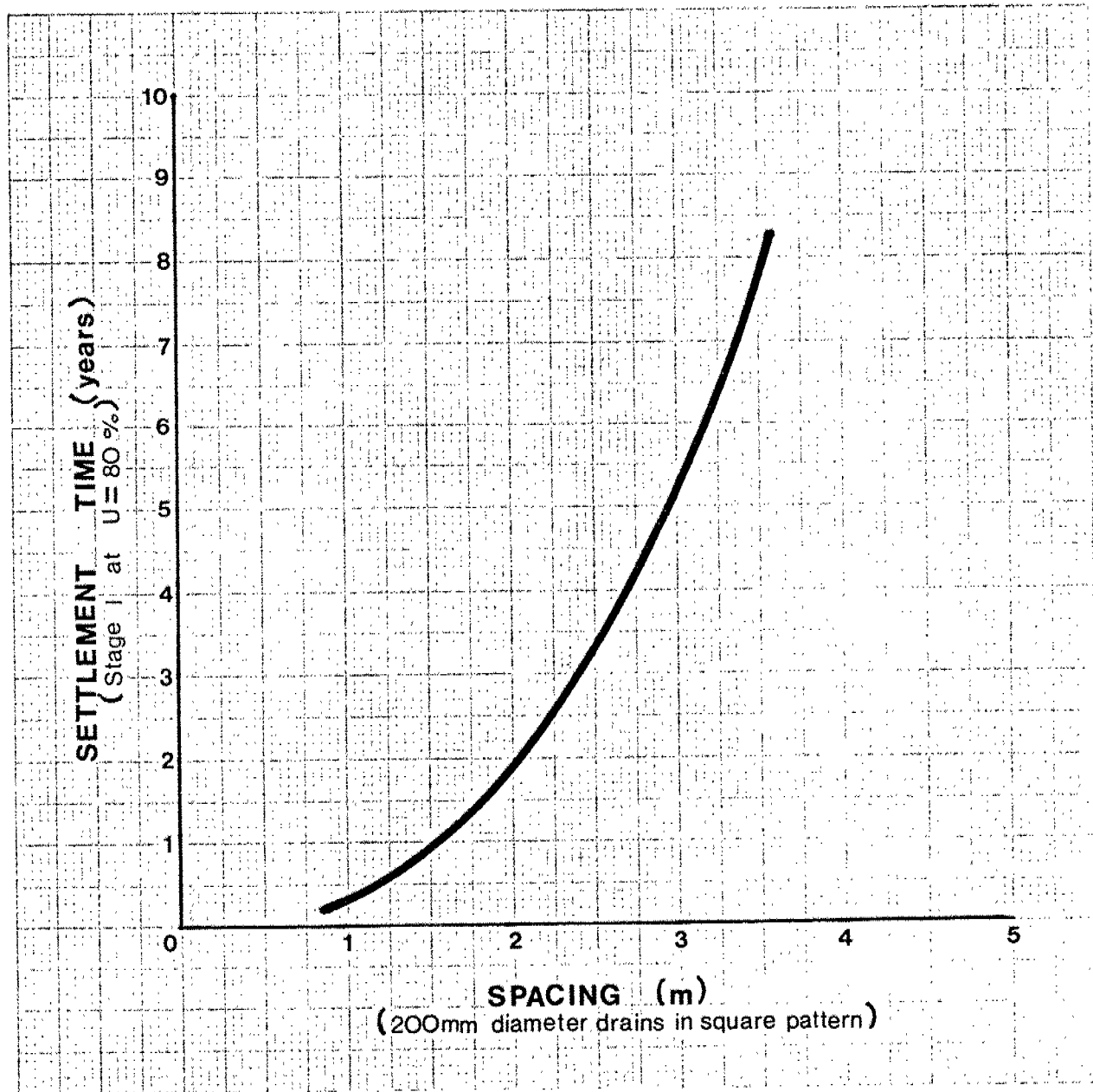
SCALE AS SHOWN

DECEMBER 1986

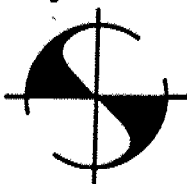
GEOCOR ENGINEERING INC.

Prep. By SHC

SETTLEMENT TIME
VRS
SPACING OF VERTICAL SAND DRAINS
STAGE I (U=80%)



GEOCOR ENGINEERING INC.



STRATA ENGINEERING CORP.

RESEARCH . ENGINEERING . SCIENCE

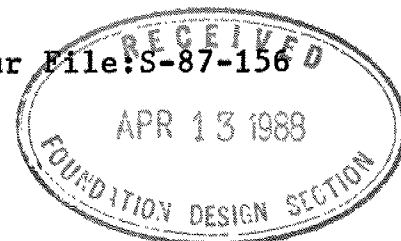
Tel: (416) 441-2560

Suite 410, 170 The Donway West,
Don Mills, Ontario, Canada M3C 2G3
Telex: 06-966637, Fax: 416-441-2862
~~441-446~~

1988 04 11

Our File: S-87-156

Totten Sims Hubicki Associates
Consulting Engineers
675 Bath Road
Lasalle Park Plaza
Kingston, Ontario
K7M 4X2



Subject: Sir John A. MacDonald/CNR Grade Separation
Kingston, Ontario
Tests by University of Western Ontario

Dear Sirs:

Attached is a copy of the most recent report submitted to us by Dr. Bob Quigley of the University of Western Ontario, on the consolidation testing of the "hard" bands of the varved clay deposit from the above site.

Our settlement calculations are now being revised in light of this new information, and will be detailed in the final report.

Preliminary calculations show that the total amount of settlement will be somewhat less than earlier predicted; however, the magnitudes will still be of concern with respect to the long term integrity of the sewer pipe below the north fill approach embankment.

The final report will be issued within ten working days.

Yours very truly,

C. Mirza, P.Eng.

cc: M.S. Devata, Chief Foundation Engineer, MTO
W.E. Blum, P.Eng., MTO Kingston



The University of Western Ontario
GEOTECHNICAL RESEARCH CENTRE

Faculty of Engineering Science
London, Ontario, Canada N6A 5B9

Dr. K.Y. Lo (519) 661-2125
Dr. M. Novak 661-2003
Dr. R.K. Rowe 661-2126
Dr. R.M. Quigley 661-3344

Telex No. 064-7134

1988 03 28

Strata Engineering Corp.
170 The Donway West, Suite 410
Don Mills, Ontario
M3C 2G3

Attention: Mr. C. Mirza, P.Eng.

Dear Sirs:

Re: Special Consolidation Test - "Hard" Clay Bands
Sir John A. MacDonald/CNR Grade Separation
Kingston, Ontario

In accordance with your letter of December 17, 1987, we have carried out a single consolidation test on one of the "hard" clay layers from the set of waxed tube samples sent to us.

The 15 mm thick sample tested consisted of a winter clay layer from the complex proximal varves described in our earlier report dated December 10, 1987. Trimming and installing the sample in the consolidation ring was difficult because the sample appears to have consisted of both the brittle, high water content clay and the underlying plastic clay of lower water content. Determination of the water content also caused some problems since the brittle clay had a w_o of 83.5 compared to much lower values in the plastic clay below it.

... 2/

MAR 30 1988

A summary of the initial moisture variations in the sample is as follows:

Sample	BH #3A, Sa #13C
w_o (brittle clay)	83.5%(1)
w_o (test sample average)	62.1%(2)
D_R (specific gravity of clay layer)	2.86

-
- Notes: (1) Measured on cuttings of clay.
(2) Back calculated from the final moisture content (48.2%) at the end of consolidation testing.

Two consolidation curves are presented on Figures 1 and 2 using e_o values calculated for the w_o values of 83.5 and 62.1%. Figure 2 is more representative of the bulk soil tested than Figure 1. Load increment ratios of 0.5 were used to more clearly identify the preconsolidation pressure of the sample. Using the Casagrande construction procedure, σ'_p of this layer of brittle clay is estimated to be approximately 420 kPa. Both curves give the same σ'_p value indicating that this particular item is fairly independent of e_o used to plot the curves.

The only major difference from your data is that the winter layer appears to contain abundant clay minerals yielding a high D_R value of 2.86.

We trust that these results on this most interesting clay are adequate for your current needs on this project.

Yours truly,

GEOTECHNICAL RESEARCH CENTRE



R.M. Quigley, Professor

RMQ:em

Attachs: Figures 1 and 2.

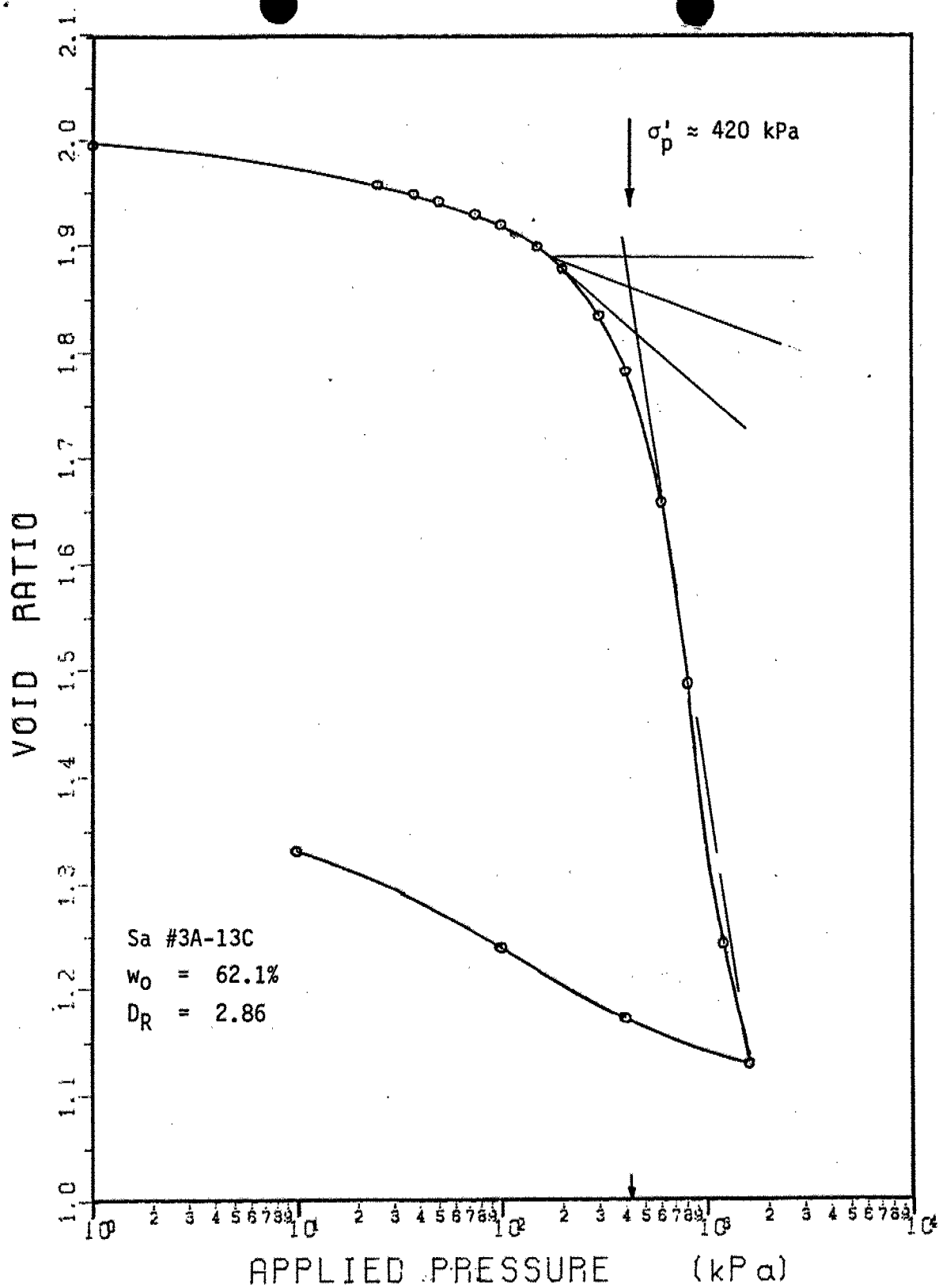


FIGURE 1. CONSOLIDATION CURVE FOR WINTER CLAY LAYER USING $e_0 = 2.0$

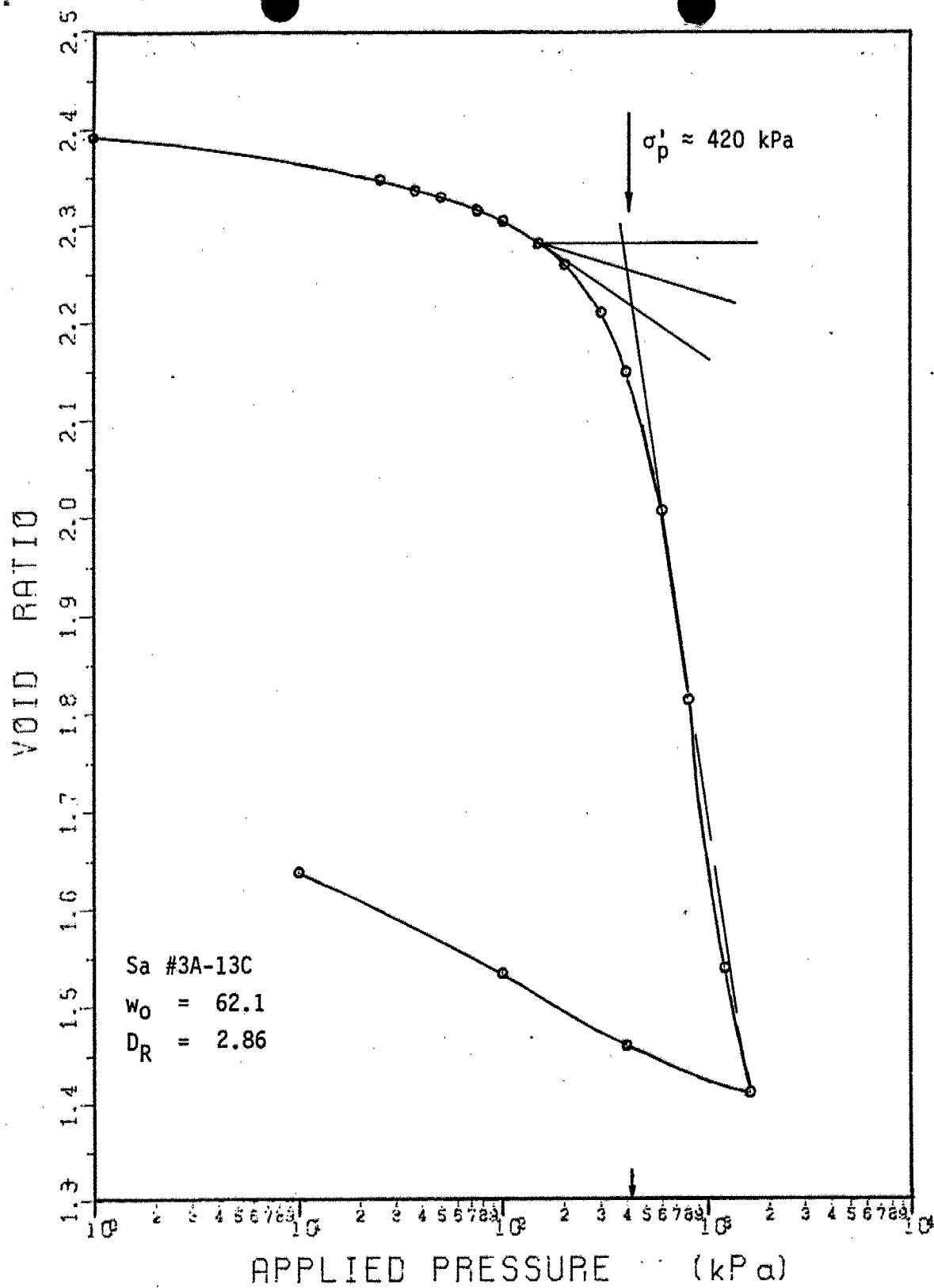
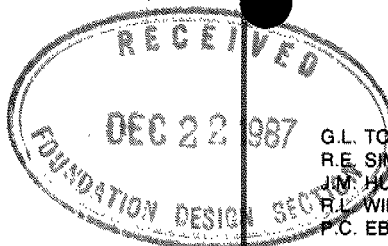
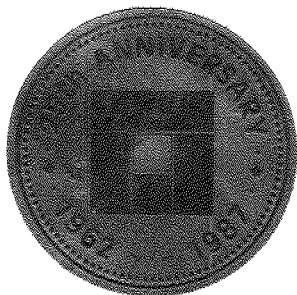


FIGURE 2. CONSOLIDATION CURVE FOR WINTER CLAY LAYER USING $e_0 = 2.39$



G.L. TOTTEN B.Sc., P.Eng.
R.E. SIMS B.A.Sc., P.Eng.
J.M. HUBICKI B.A.Sc., OAA, P.Eng.
R.L. WINDOVER M.Sc., P.Eng.
P.C. EBERLEE B.A.Sc., P.Eng.

TOTTEN SIMS HUBICKI ASSOCIATES (1981) LIMITED
675 BATH ROAD, LASALLE PARK PLAZA, KINGSTON
ONTARIO, CANADA, K7M 4X2 (613) 389-3703

totten sims hubicki associates

MINUTES OF MEETING HELD DECEMBER 16, 1987 AT 11:00 A.M., AT THE MINISTRY OF TRANSPORTATION OFFICES ON COUNTER STREET IN KINGSTON, REGARDING THE SIR. JOHN A. MACDONALD BOULEVARD, C.N.R. OVERPASS AND APPROACHES, CITY OF KINGSTON.

Present:

Mr. K. Linseman, P.Eng. - Commissioner of Works, City of Kingston
Mr. G. de Lugt, P.Eng. - Deputy City Engineer, City of Kingston
Mr. J. Campbell, P.Eng. - City of Kingston
Mr. M. Devata, P.Eng. - Ministry of Transportation, Foundation Section
Mr. B. McKinnon, P.Eng. - District Engineer, Ministry of Transportation, Kingston
Mr. W. Blum, P.Eng. - District Municipal Engineer, Ministry of Transportation, Kingston
Mr. C. Mirza, P.Eng. - President, Strata Engineering
Mr. P. Smith, P.Eng. - Totten Sims Hubicki Associates Ltd

Purpose of the Meeting:

The purpose of the meeting was to discuss the various alternatives for solving the settlement problems which will occur with the 42" diameter sanitary trunk sewer on the north side of the C.N.R. should a conventional approach fill be constructed across it.

Background:

Mr. Blum opened the meeting by reviewing the background of the project to date. Originally, Geocor Engineering Consultants were engaged to do the foundation work for this project but their recommendations were not accepted by the Design Consultant or the City Engineering staff. Subsequently, Strata Engineering was engaged to carry out further investigative work and M. Devata of the M.T.C. Foundation Section agreed to review the findings and recommendations.

Possible Solutions:

The Soils Consultant outlined four (4) possible solutions to the problem which were as follows:

- a) Construct an additional span in the proposed structure to bridge the sewer area. Approximate cost of this alternate is \$1,200,000.00.
- b) Install flexible joints in the sewer. This would entail complete excavation of the existing sanitary sewer, bypassing of the flow in the sewer, cutting out the existing joints and installing flexible neoprene type joints and rebackfilling. An estimated cost for this work is not available but it would be significant due to the amount of excavation and backfill required.
- c) Use lightweight fill for the north approach fill in the area of the sanitary sewer. The Consultant noted that either lightweight slag or styrofoam is suitable for this installation but since slag would have to be hauled from the Hamilton area, only styrofoam is economically feasible for this project. The estimated cost of the styrofoam option is \$150,000.00.

MINUTES OF MEETING HELD DECEMBER 16, 1987 REGARDING THE SIR. JOHN A. MACDONALD BOULEVARD, C.N.R. OVERPASS AND APPROACHES, CITY OF KINGSTON

Possible Solutions(cont'd.)

- d) Removal of the existing sanitary sewer in the area of the fill and replace it with a plastic type pipe. No cost estimate is available for this alternate.

After some discussion, it was felt that because of the very flat grade of the existing sanitary sewer (0.08%) and due to the fact that the flexible joint pipe or the plastic pipe would still reflect under the load of the approach fill and therefore affect the flow characteristics of the sanitary sewer, that the only viable alternate was the lightweight fill alternate using styrofoam.

Recommendations:

- 1) The Soils Consultant will check on the availability and cost of styrofoam and write to TSH with a cost estimate and recommendation for review and approval by the City of Kingston. M. Devata will comment on this submission. Should the styrofoam solution appear to be economically feasible, the Soils Consultant will recommend that solution and will make recommendations in his report on how to install the styrofoam etc.
- 2) Due to the observed high velocities of flow in the sanitary sewer, it was recommended that a television survey be carried out on the sewer to check its condition and if there are any problems, remedial action would be carried out prior to constructing the fill. TSH will make the necessary arrangements to have the sewer televised between manhole No. 41 and manhole No. 43.
- 3) Manhole No. 42 will be buried by the new fill. TSH was instructed by the City Engineering staff to install two new manholes, one at each side of the proposed new fill and to cap the existing manhole No. 42. This will ensure that if there are ever problems with the sewer under the fill, the flows can easily be by-passed from new manhole to new manhole while remedial work is being carried out.

Miscellaneous:

C. Mirza noted that on his last visit to the site, considerable deleterious material was observed in the north approach fill area. Specifically, fenceposts brush etc., had been piled in the area and this should be discouraged and should be removed prior to constructing the new fill within that area.

Adjournment:

The meeting adjourned at 12:20 p.m.

End of Minutes



P.A. Smith, P.Eng.

PAS/fb

- c.c: Mr. K. Linseman, P.Eng.
✓ : Mr. M. Devata, P.Eng.
: Mr. B. McKinnon, P.Eng.
: Mr. W. Blum, P.Eng.
: Mr. C. Mirza, P.Eng.



The University of Western Ontario
GEOTECHNICAL RESEARCH CENTRE

Faculty of Engineering Science
London, Ontario, Canada N6A 5B9

Dr. K.Y. Lo (519) 661-2125
Dr. M. Novak 661-2003
Dr. R.K. Rowe 661-2126
Dr. R.M. Quigley 661-3344

Telex No. 064-7134

1987 12 10

Strata Engineering Corp.
170 The Donway West, Suite 410
Don Mills, Ontario
M3C 2G3

Attention: Mr. C. Mirza, P.Eng.

Dear Sirs:

Re: Mineralogical Assessment of Soil Samples
Sir John A. MacDonald/CNR Grade Separation
Kingston, Ontario

In accordance with your letter of instruction dated November 6, 1987, we have carried out detailed visual observations and mineralogical analyses on the waxed soil specimen provided to us from the above project. The primary purpose of our studies has been to determine the geological or mineralogical explanation for the presence of unusual, brittle, very high moisture content clay layers within the stratigraphic section from which the sample came.

Visual Description of Soil Sample

A notch was cut into the waxed specimen and both macroscopic and microscopic observations were made of the sample. The sketch presented on Figure 1 is a true-scale representation of the sample along with a short description of each layer.

On the basis of these observations, it is suggested that the soil specimen comes from a complex stratigraphic sequence of proximal varved clays in

DEC 14 1987

which the summer layers consist of a considerable thickness of silt containing thin laminae of more clayey material. The winter clay layer, while not clearly demonstrating graded bedding, does consist of plastic silty clay in the lower half, becoming very brittle, blocky and more clayey in the upper half (see sketch).

Moisture contents run on the various parts of the sample yielded values of 18 and 21% for the lower and upper silt layers, and values of 42 and 90% for the lower and upper halves of the clay layers, respectively. The very high water content of the brittle zone indicates a highly flocculated fabric as well as abundant clay minerals.

Mineralogical Analyses

X-ray powder patterns run on the four main soil layers are presented on Figures 2 and 3. The traces for the two silt laminae indicate the presence of abundant quartz, feldspar, dolomite and calcite. Although clay minerals are present in both silts, they seem more abundant in the lower silt which contains distinct 1.5 mm thick laminae of clayey material.

The x-ray powder patterns for the two halves of the clay layer (Figure 3) yield very strong clay peaks and correspondingly weaker peaks for the primary minerals. The two traces are distinctly different, with very little quartz, feldspar and carbonate present in the upper brittle, very clayey zone.

X-ray diffraction traces obtained on $< 2 \mu\text{m}$ fines from the two halves of the clay layer are presented on Figures 4 to 6. The clay minerals present consist of abundant illite (1.0 nm peak) and smectite (1.45 nm to 2.8 nm peaks). The two specimens heated to 550°C (Figure 6) yielded small peaks at 1.42 nm suggesting that a small amount of iron chlorite is present in the samples, yielding part of the 0.7 nm peaks on Figures 4 and 5.

Potassium saturation (Figures 4b and 5b) caused very little collapse of the swelling clays indicating that they are low charge smectites rather than vermiculites.

The results of carbonate analyses on four separate layers are presented in Table 1 with the water contents. The silt layers contain 24 to 26% carbonate consisting of both dolomite and calcite. The two halves of the clay layer contain less carbonate, especially the brittle upper half which contains only 7% carbonate.

Discussion and Recommendations

An attempt was made to assess the fabric of the two halves of the clayey layer by x-ray diffraction on air dried fragments oriented in solidified plastic. The peaks were too weak for the method to yield any results. If it is deemed important to confirm a highly flocculated fabric in the brittle part of the clay layer which has a water content of 90%, we can do it later by scanning electron microscopy on fracture surfaces of freeze-dried samples.

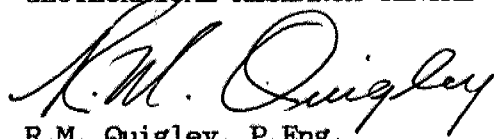
It is understood that the flocculated clay layers represent an important portion of the stratigraphic section, especially as they influence potential consolidation. We suggest, therefore, that an attempt be made to run consolidation tests on a couple of these brittle layers using a $\Delta p/p$ ratio of 0.5 up to the preconsolidation pressure. This would be a very tricky assignment in view of the very brittle blocky nature of the clay. Please send us a few more samples if you would like us to try.

- - - - -

We trust that these preliminary analyses meet with your present needs on this very unusual clay.

Yours truly,

GEOTECHNICAL RESEARCH CENTRE


R.M. Quigley, P.Eng.
Professor

RMQ:em

Attach: Table 1, Figures 1 to 6.

TABLE 1. TEST RESULTS (Sa #3A/13-A)

Sample	Water Content (%)	Carbonate (%)
Upper silt	21	26
Upper half clay layer	90	7
Lower half clay layer	42	18
Lower silt	18	24

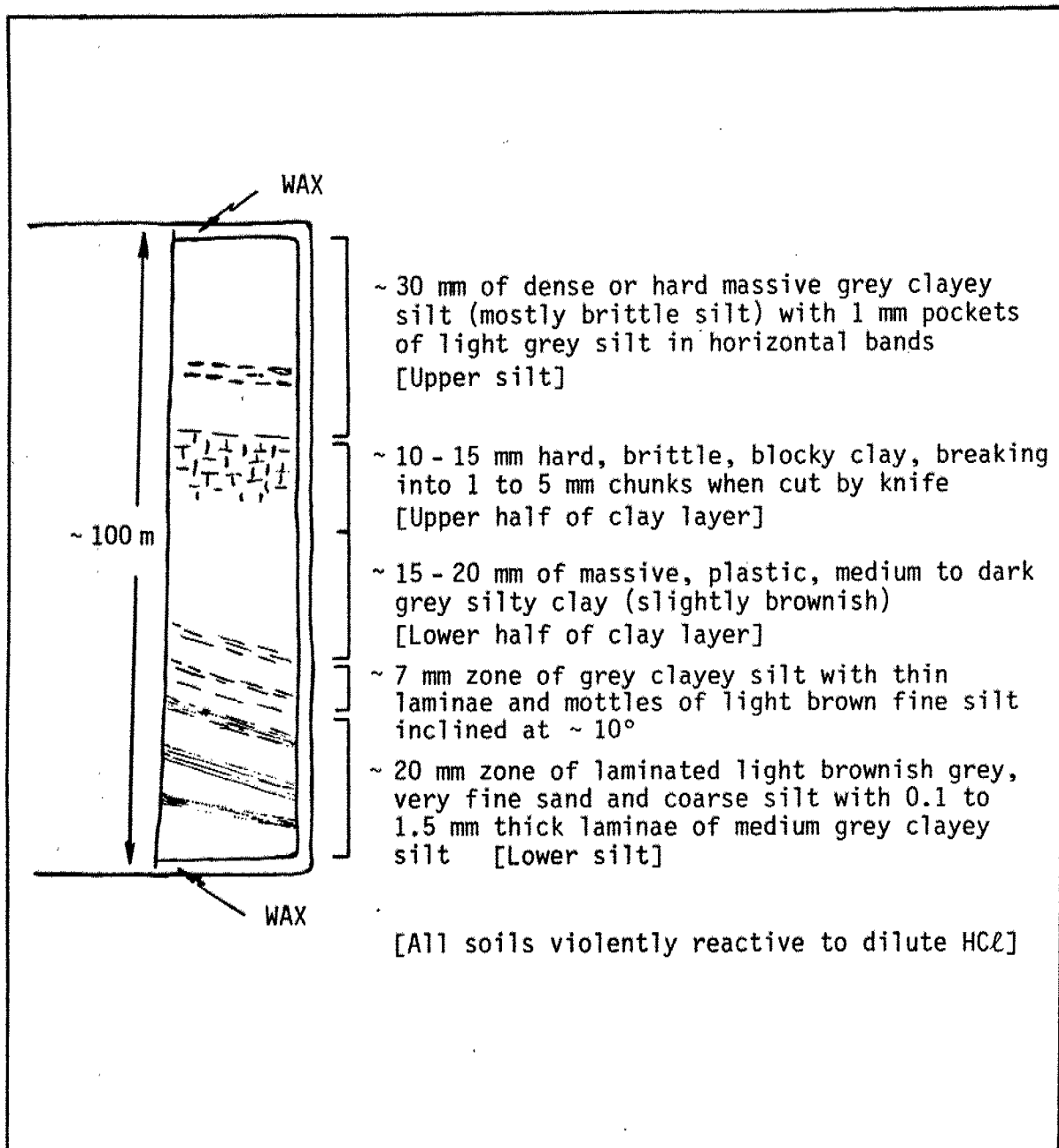


FIGURE 1. SKETCH AND DESCRIPTION OF TUBE SAMPLE #3A/13-A,
STRATA ENGINEERING CORP. PROJECT #S-87-156,
KINGSTON, ONTARIO

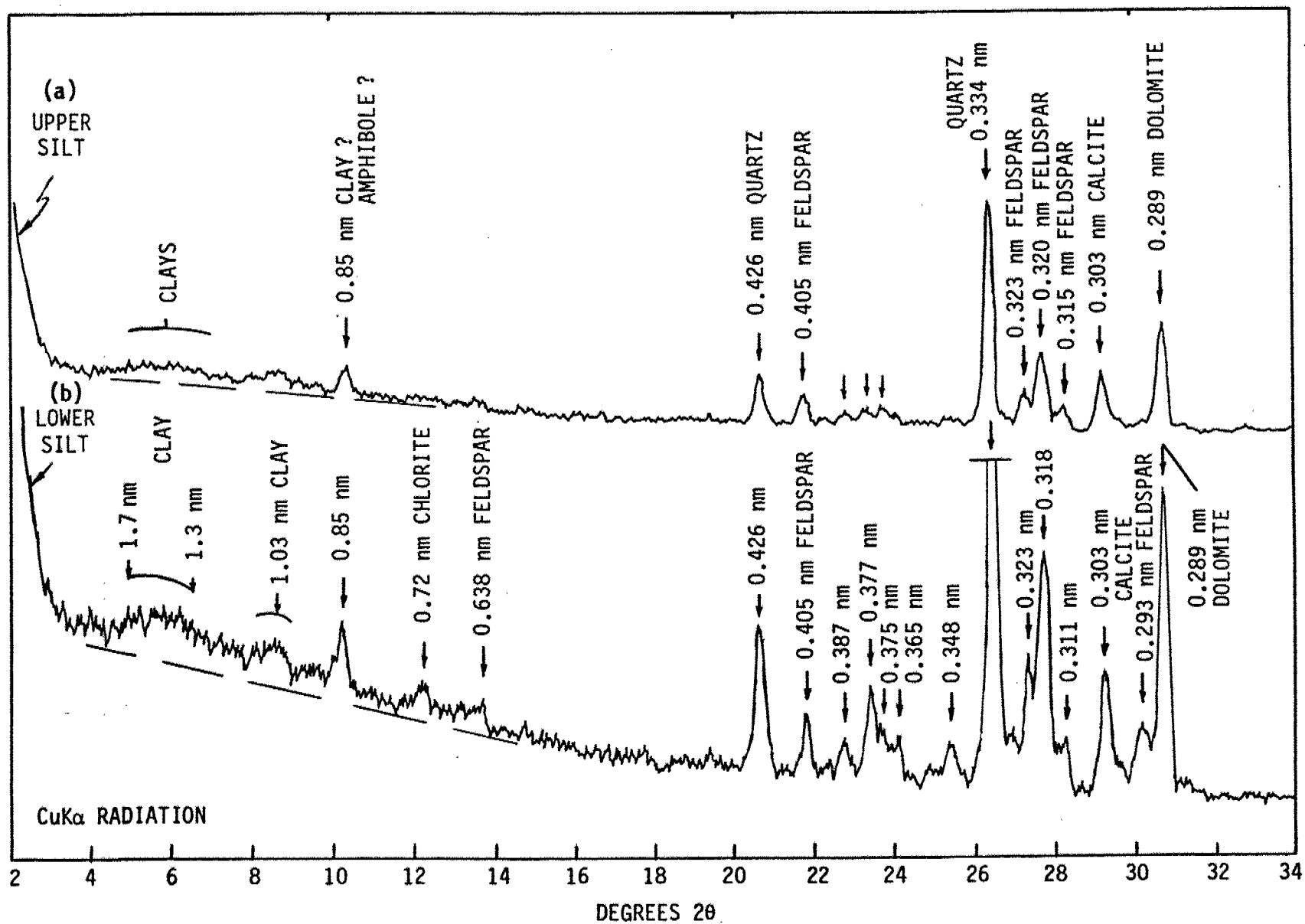


FIGURE 2. X-RAY POWDER PATTERNS OF WHOLE SOIL FROM (a) UPPER AND (b) LOWER SILT LAYERS IN SA #3A/13-A (See Figure 1)

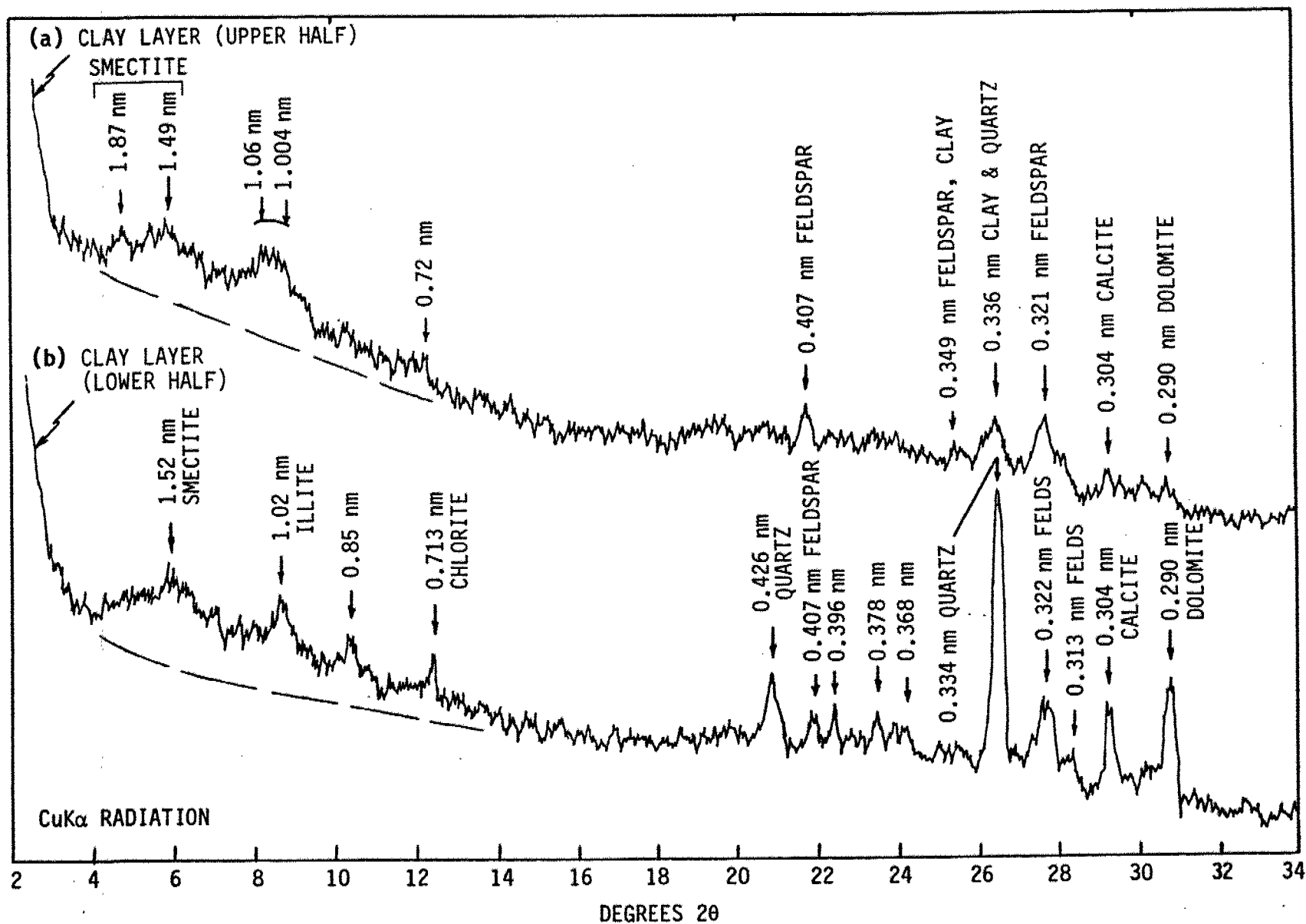


FIGURE 3. X-RAY POWDER PATTERNS OF WHOLE SOIL SAMPLES FROM (a) UPPER, BRITTLE, BLOCKY HALF OF CLAY LAYER AND (b) LOWER PLASTIC HALF OF CLAY LAYER FROM SA #3A/13-A (See Figure 1)

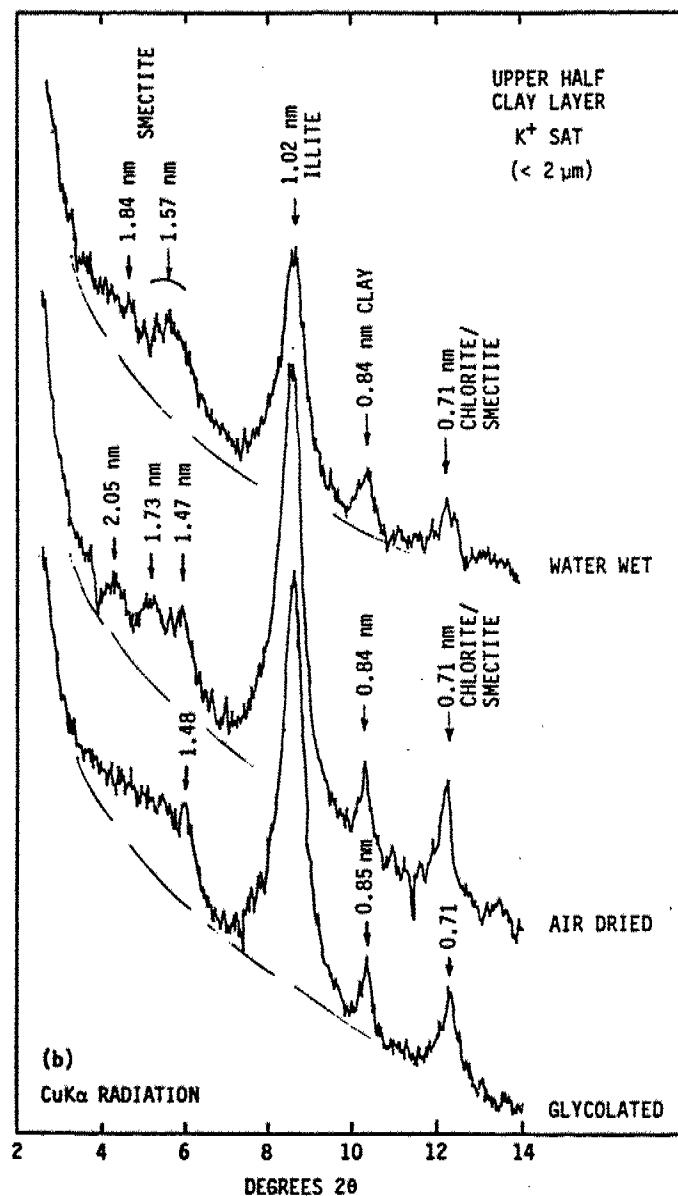
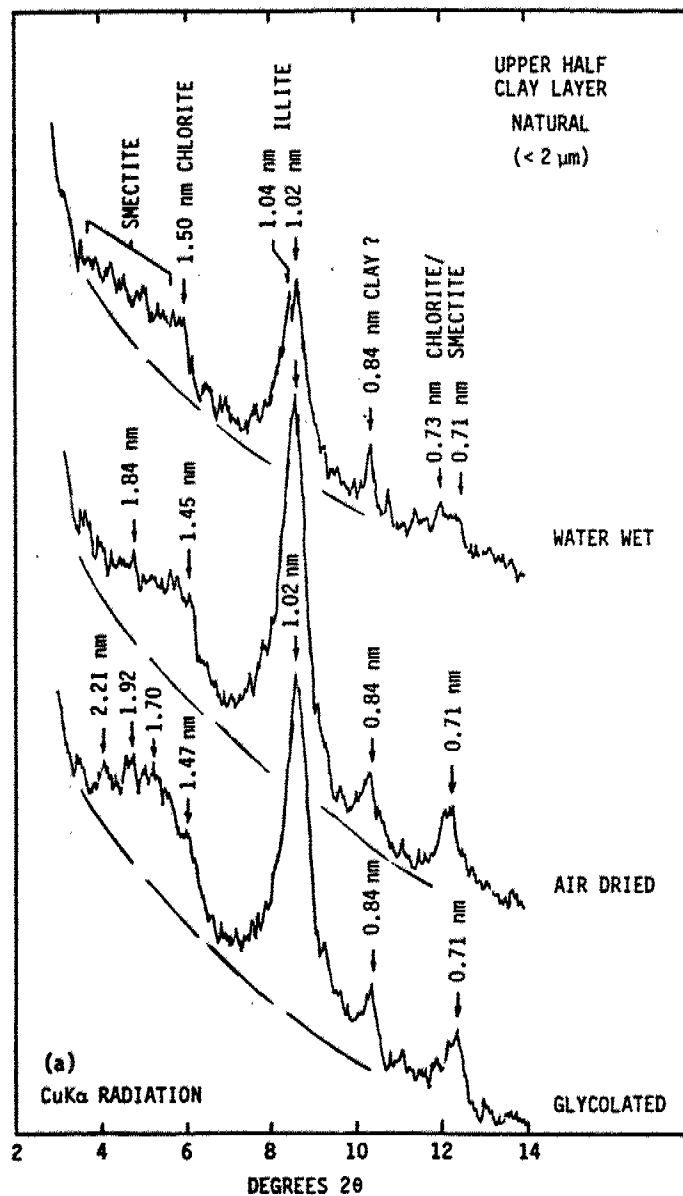


FIGURE 4. X-RAY TRACES OF ORIENTED, $< 2 \mu\text{m}$ FINES OF UPPER HALF OF CLAY LAYER (a) NATURAL AND (b) K^+ SATURATED

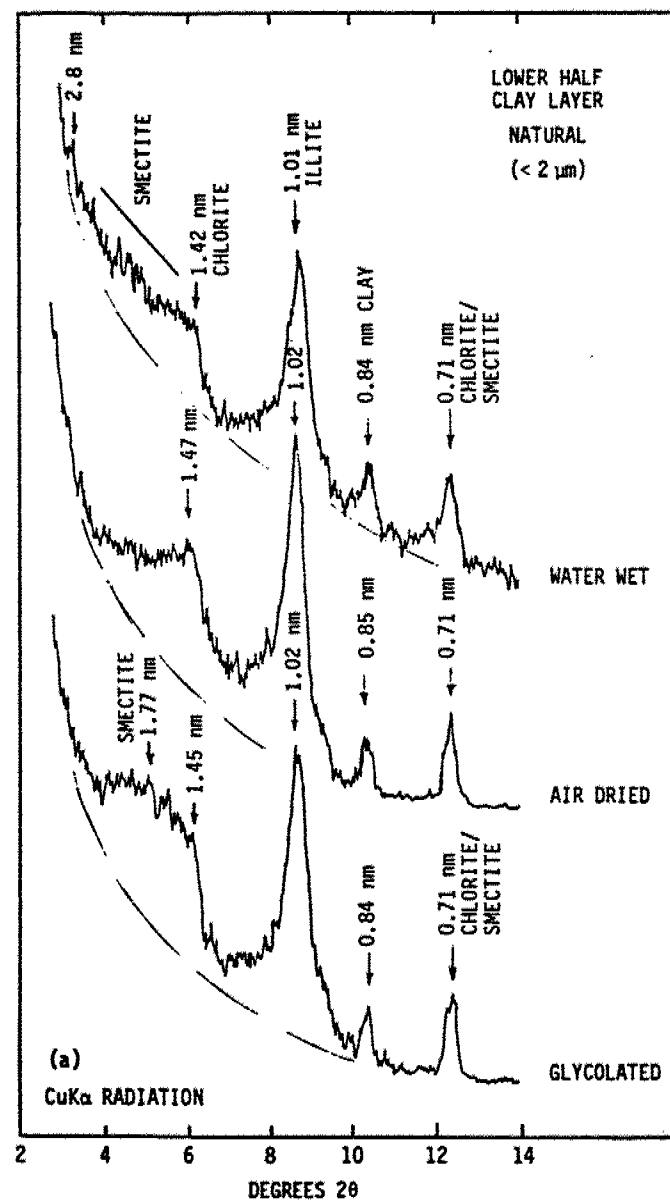
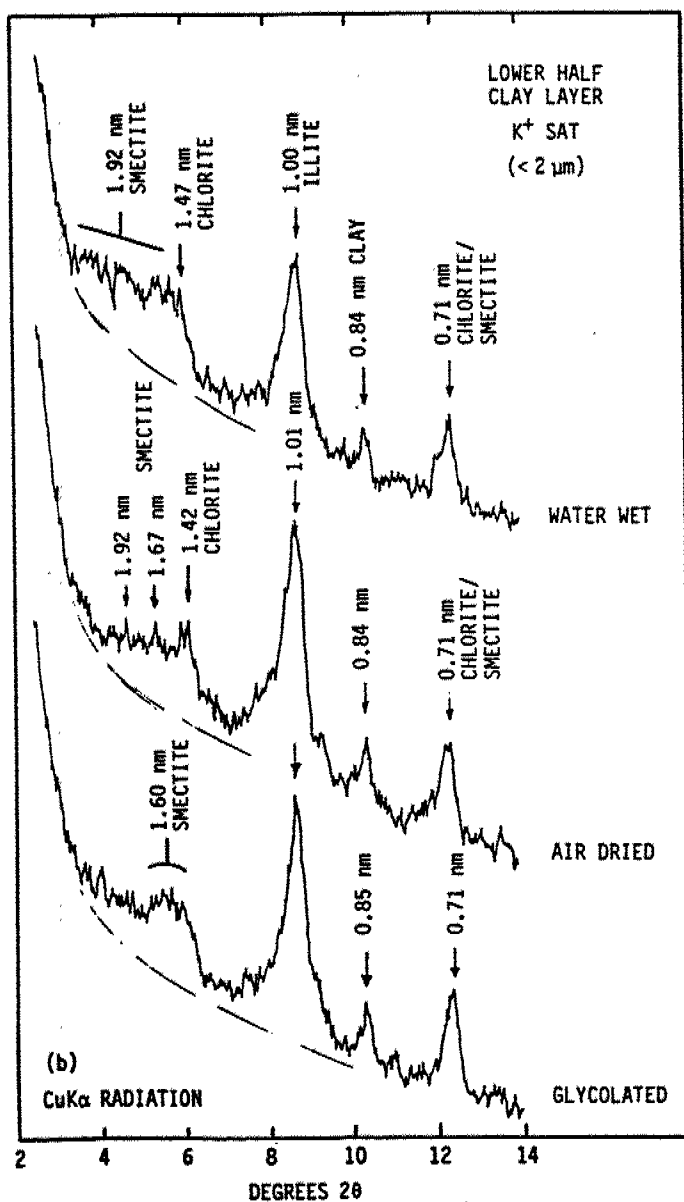


FIGURE 5. X-RAY TRACES OF ORIENTED, $< 2 \mu\text{m}$ FINES OF LOWER HALF OF CLAY LAYER (a) NATURAL AND (b) K⁺ SATURATED

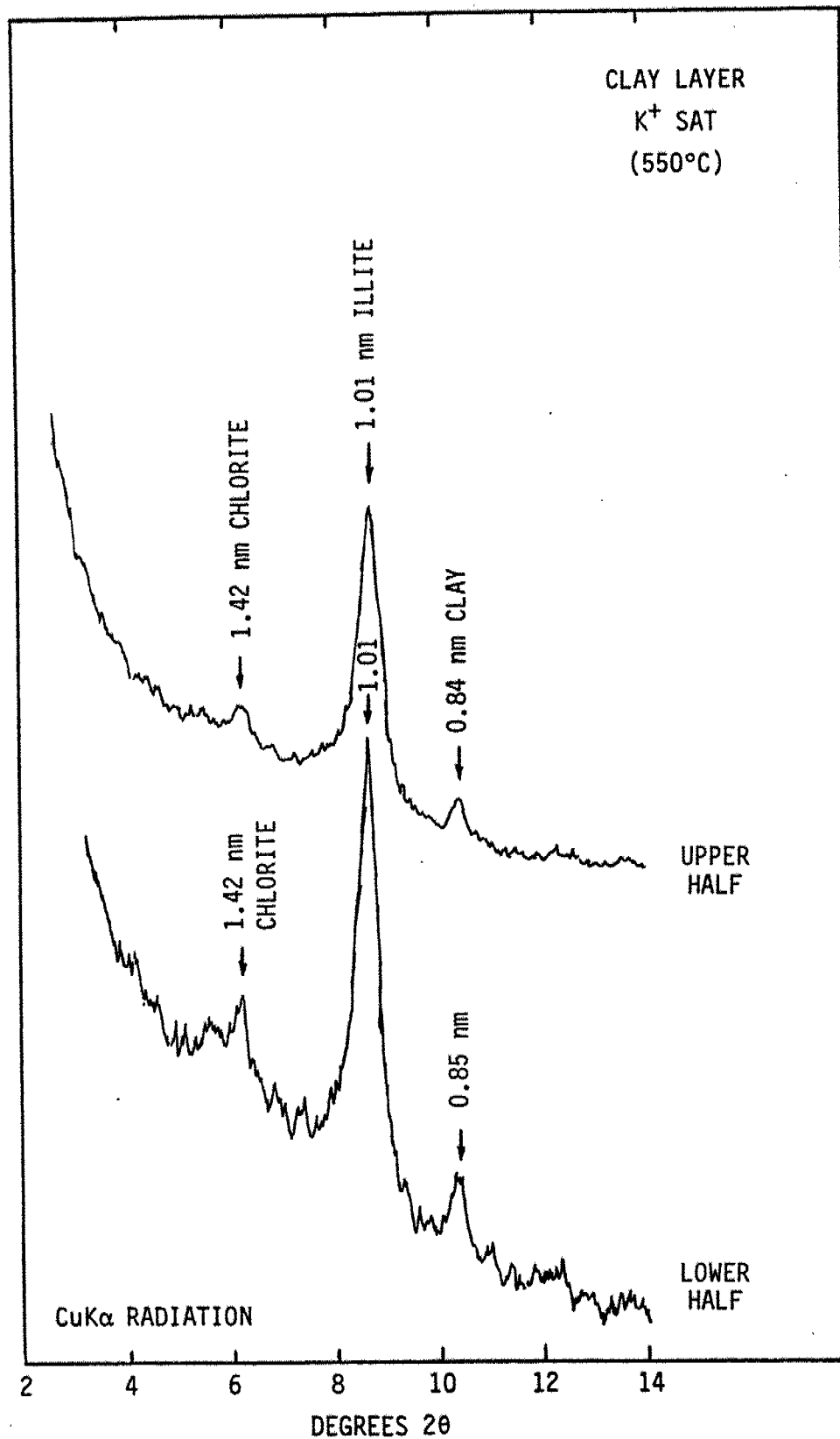


FIGURE 6. X-RAY TRACES OF ORIENTED, < 2 μ m FINES, K⁺ SATURATED AND HEATED TO 550°C

STRATA ENGINEERING CORPORATION - PROGRESS REPORT No: 3

To: City of Kingston, c/o Totten Sims Hubicki Associates, Kingston, Ontario

Date: November 26, 1987

Our Project No: S-87-156 Totten Sims Hubicki Project No: 6987

Location/Title: Sir John A. MacDonald / CNR Overpass - Terry Fox Dr. to Dalton Ave.

The following constitutes our third progress report on the above captioned project. It covers the following items:

1. Settlement Analyses
2. Decisions with respect to sewer pipe/north fill
3. Schedule
4. Budget

1. Settlement Analyses

The results of detailed settlement analyses are summarized on the enclosed three sketches. The first sketch shows the depth to bedrock below the sewer pipe. The clay thickness varies from 700 mm to over 10 m at a distance of 10 m from the west toe of the proposed 10 m high embankment fill on the north approach.

From our consolidation tests on the clay below the sewer pipe, a preconsolidation pressure of about 100 kPa has been established. The clay above the sewer pipe is desiccated and not likely to suffer from any long term consolidation settlements of significant magnitude.

The soil parameters used in the theoretical calculations of expected ultimate settlements are shown on the second sketch. The numbers in brackets after the numbers representing ultimate settlement values refer to the expected settlements at the end of first year after construction. The maximum settlement below the sewer pipe is expected to be in the order of 108 mm at the west shoulder of the approach fill (Point 4) on the sketch.

The time rate of consolidation settlement is shown on the third sketch. Essentially, it is expected that all consolidation will be completed within a period of five years after completion of loading. The maximum ultimate differential settlement is expected to occur between Points 4 and 3, and amounts to about 60 mm over a five year period.

These calculations show that settlements due to the consolidation of the clay underlying the sewer pipe will be much less than was predicted in the earlier investigation and reports by others. Therefore, the question arises as to whether or not to proceed with construction as planned, assuming the sewer pipe will be able to withstand the differential settlements predicted by these calculations.

The case for a shift in the alignment to the east by 10 m was also analysed, and no significant improvement was noted in the calculated settlement values. It is therefore our conclusion that the alignment as selected is the most optimum and preferred.

In order to determine the capability of the pipe to withstand the embankment loadings, and to determine whether or not a negative trench condition needs to be established to protect the pipe against overloading (for structural, and not settlement reasons), we have asked the Firm of SIMPSON, GUMPERT & HAGER of Boston (USA) to carry out an analysis using computer programmes they have developed for use by the American and Ontario Concrete Pipe Associations. Their results are expected shortly, and will be included in our final report, which will be withheld until some decisions can be reached as to the ultimate disposition of the sewer pipe.

We will shortly be receiving a report from Dr. R. M. Quigley of the University of Western Ontario. Dr. Quigley has reported orally to us that the clay below the pipe is smectitic, and shows a large quantity of carbonates. He also indicates the clay which we thought was not varved, may in fact be varved, but of considerable age (which has considerable significance with respect to interpretation of preconsolidation pressures), and of near-ice lacustrine deposition environment. He has asked for additional samples of the clay so that the University may attempt a consolidation test on the dark grey hard bands found in the lower regions of the clay deposit. He suggests the University has the equipment and skills capability to attempt such a test. It should be recognized that all of our consolidation tests were carried out on the intermediate relatively softer soils found between the hard dark grey bands of clay. We feel it would be very beneficial to allow the University to attempt such a consolidation test, as it will improve our confidence in our own assessment of the

pre-consolidation characteristics of the clay below the sewer pipe. The ultimate settlement values we have derived based on our consolidation tests are on the conservative side, by choice. Therefore, actual settlements may be less than 100 mm in the long term. The UWO tests will help confirm this assumption of conservatism.

These relatively low estimates of settlement below the sewer pipe now raise the question of what to do - build over it as proposed or avoid settlements altogether?

2. Decisions with respect to sewer pipe / north fill

Assuming the sewer pipe has sufficient strength to withstand the imposed loading from the north approach fill embankment (with or without a negative trench condition), a decision needs to be made with respect to whether or not the pipe can safely accommodate the expected differential movements over the next five years after completion of the embankment. The Ontario Concrete Pipe Association has not been able to help in that regards, although Mr. Jim Bartley of that organization feels the concrete can deflect sufficiently to accommodate such movements, and also that should cracks develop they could easily be grouted from within, given the large diameter of the pipe.

If zero or only very minor (less than 25 mm) settlement is to be allowed below the sewer pipe, the north abutment position has to be moved northwards a distance of 60 m from its present intended location, resulting in the addition of two extra spans to the bridge. One extra pier would be located just south of the pipe alignment, and the abutment would need to be moved back northwards to allow the toe of fill to be at a sufficient distance from the pipe centreline to reduce imposed stresses on the clay below the pipe, as well as to result in a proportionate span ratio across the length of the deck, assuming standard CPCI beams. Assuming a deck width of 25 m, the extra two spans represent an additional deck area of 1500 m² which at an estimated cost of \$800 per square metre could increase the cost of the structure by \$1,200,000!

We have examined other alternatives for reduction of settlement, and short of using styrofoam as a backfill, nothing else comes even close to reducing the anticipated settlements due to consolidation of the underlying clay to less than 25 mm. The cost of styrofoam, such as HI 40, is expected to be \$100,000 (material, taxes, supply, installation) for a core 20 m in width, 6 m thick, and built at 1:1 side slopes for a full thickness distance of approximately 25 m back of the abutment, and then suitably graded down northwards (for a transition) into the full earth fill zone. This central core would then be covered with normal fill material, sloped at 2:1.

The cost of treating in some appropriate manner the existing fill will be common to any solution adopted, since the existing north fill consists of randomly placed, uncompacted construction debris mixed with soil and rock. Two solutions for its treatment are: (1) complete removal and disposal elsewhere, or (2) covering with a geogrid and geotextile after imparting sufficient compactive effort. If a negative trench condition must be adopted to safeguard the sewer pipe against imposed loadings, then alternative (1) would be the most sensible solution. However, if styrofoam is used, there would be no need for a negative trench condition to be instituted to safeguard the sewer pipe, and alternative (2) could be considered for adoption.

There are case records of approach fills built with a central styrofoam core to reduce anticipated settlements. We feel that the use of lightweight high strength styrofoam fill would in the ultimate analysis be the most cost-effective and reassuring solution, especially when compared with the alternates of (1) either increasing the bridge spans from three to five, or (2) building a pumping station to handle the sewage through an alternate alignment from the existing manhole on the CNR ROW to connect with a new manhole on the east side of the toe of north fill.

3. Scheduling

In order to discuss these alternatives and review the work we have completed to date, we request a meeting be set up, perhaps in Kingston, so that our results can be presented to all parties concerned, and a decision made as to the most effective course of action. We can then finalize our report in terms of the design requirements which may arise from this meeting.

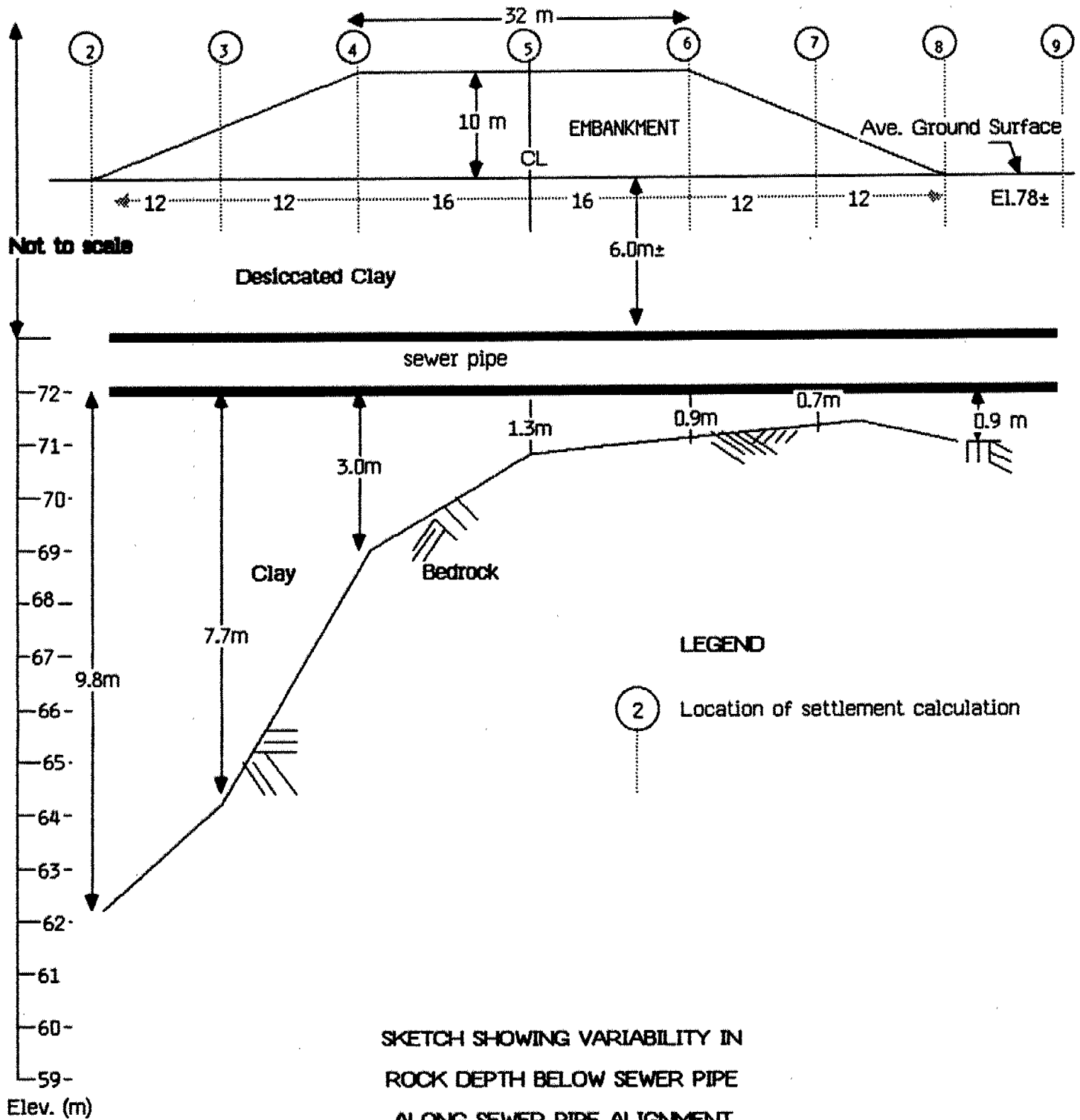
4. Budget

To date, the project budget is on target. However, an over-run of approximately \$2,500 is anticipated because of the work being carried out by the University of Western Ontario and the consulting firm in the USA. This additional work was not allowed for in our original estimates, and it would be appreciated if the original budget could be adjusted to reflect these anticipated extras which represent less than a 10 per cent increase in the original budget.

submitted by: _____

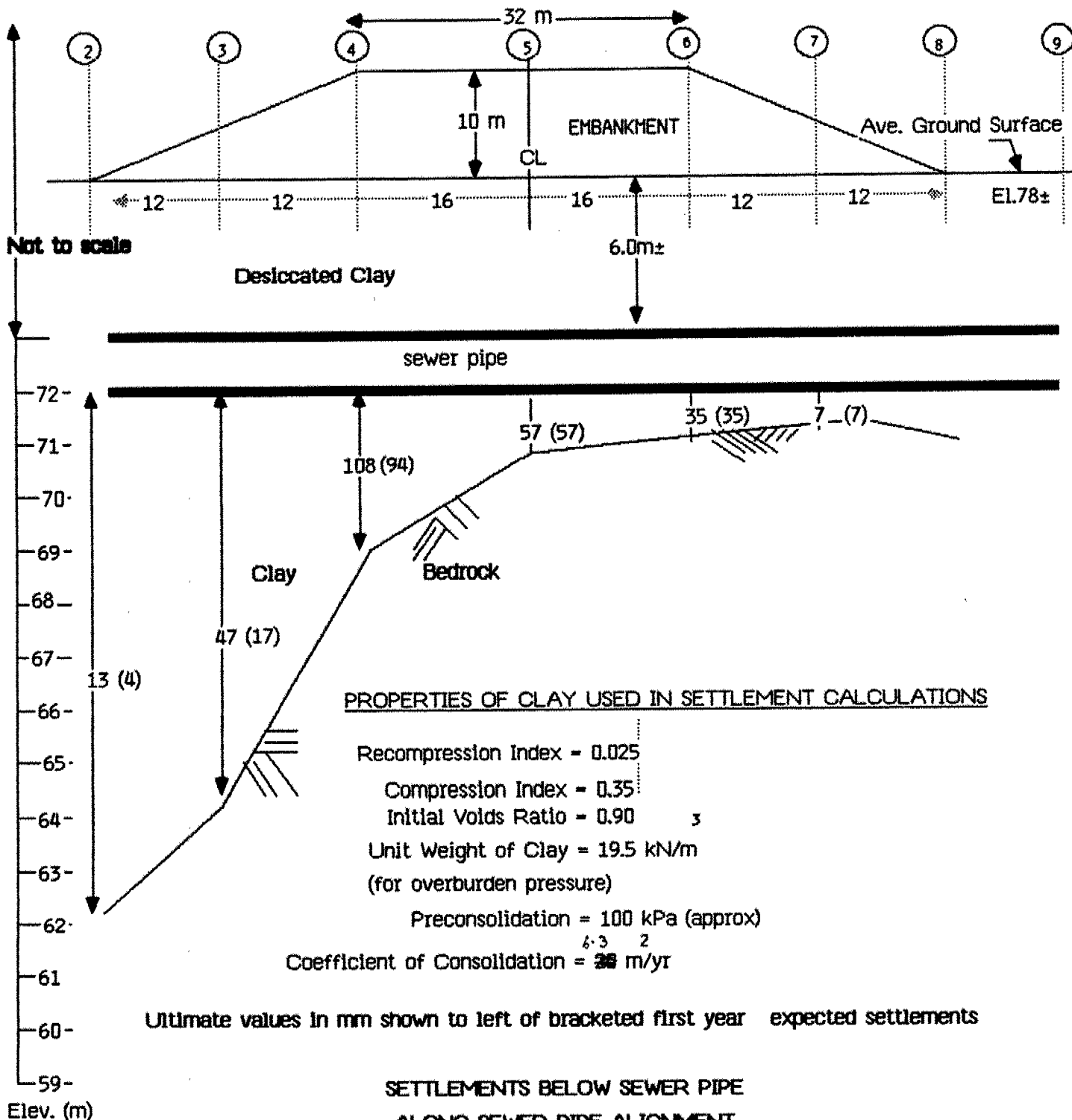
Cam Mirza, P.Eng.

cc: Peter A. Smith, P.Eng., Totten Sims Hubicki Associates (3 copies)
 M.S. Devata, P.Eng., MTO, Downsview (1 copy)
 W.E.M. Blum, P.Eng., MTO, Kingston (1 copy)
 Strata File S-87-156 (1 copy)



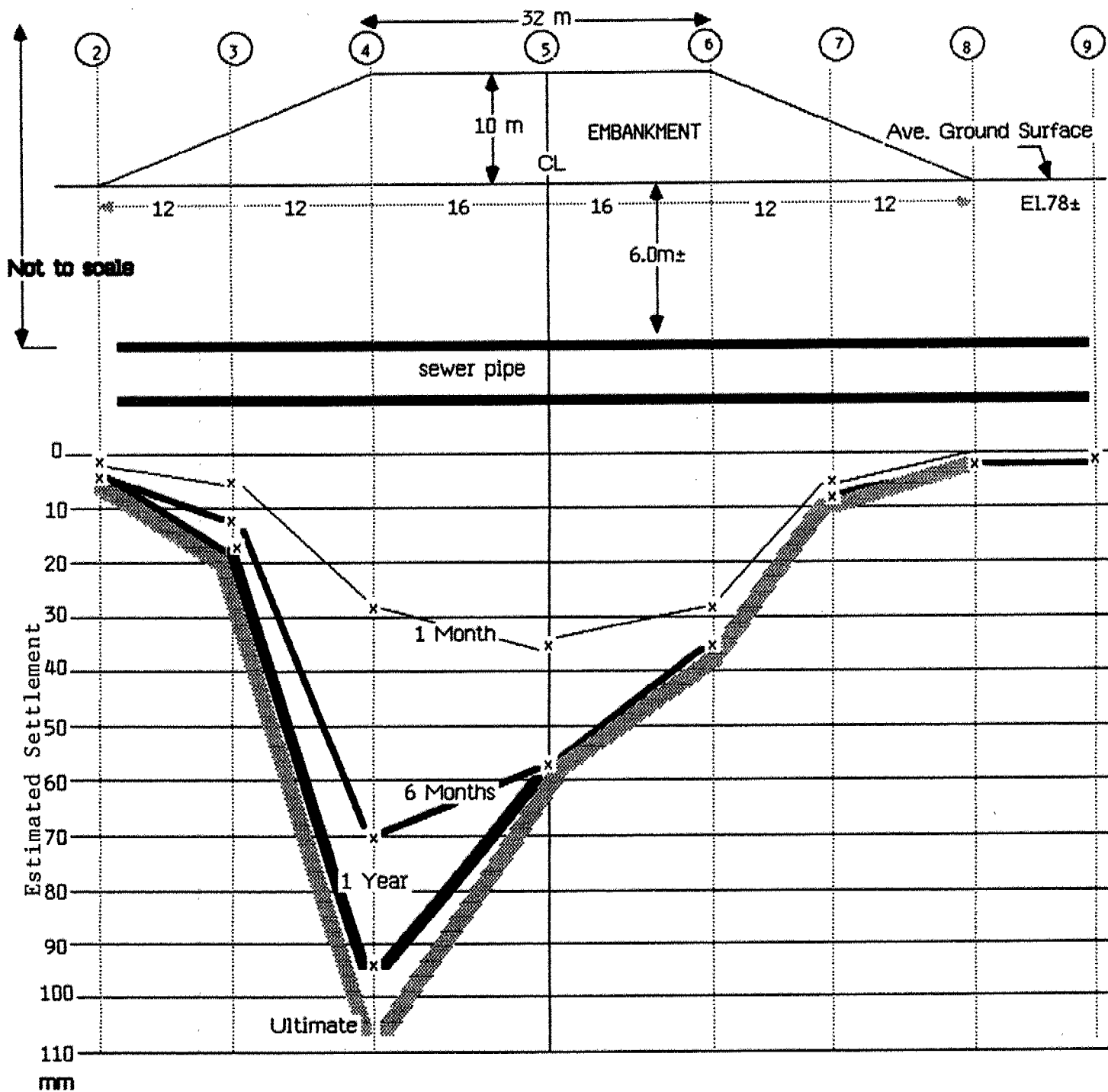
Sir John A. MacDonald/CNR Bridge Location

Kingston, Ontario



Sir John A. MacDonald/CNR Bridge Location

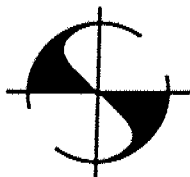
Kingston, Ontario



TIMES RATES OF SETTLEMENTS BELOW NORTH APPROACH FILL
ALONG SEWER PIPE ALIGNMENT

Sir John A. MacDonald/CNR Bridge Location

Kingston, Ontario

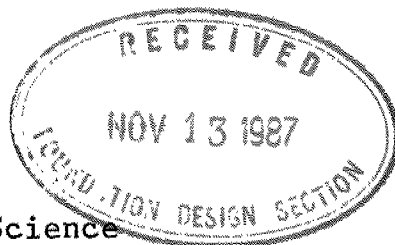


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Dr. R.M. Quigley
Faculty of Engineering Science
University of Western Ontario
London, Ontario
N6A 5B9

1987 11 06

Re: Our Project: S-87-156
Sir John A. MacDonald/CNR Grade Separation
Kingston, Ontario

Dear Dr. *Quigley* Quigley:

Further to our discussion at the Regina CGS Conference, I'm enclosing a sample of clay from one of the boreholes we drilled at the above site.

The sample number is 3A/13-A. It was taken from Borehole 3A in a 75 mm thin wall tube at a depth of 20 m below ground surface. At present the ground surface consists of previously placed fill on original ground, of about 2 m height. Thus, this sample has been subjected to about 40kPa+overburden pressure. Groundwater is at original ground surface (elevations are as follows: original ground = 78 m; fill at BH 3A = 80±m; groundwater = 78±m).

When you open this sample, you will find it to be banded. The dark grey relatively hard bands are about 5 mm thick. Our tests show moisture contents in these hard bands as high as 100 per cent! The balance of the clay matrix shows a moisture content of about 30 per cent.

The stratigraphy at this site consists of the following:

1. A desiccated hard layer of brown-grey silty clay, with in situ undrained shear strengths (field vane tests) of in excess of 100 kPa, followed by
2. A thick deposit of homogeneous to varved sediments with undrained shear strengths of in the order of 40-60 kPa, followed at a depth of about 14 m by
3. The banded silty clay, with hard bands 5 mm thick exhibiting very high moisture contents, and undrained shear strengths in the order of 60 to 80 kPa.

A rough draft of the field borehole log sheet for BH 3A is enclosed.

...../2

Dr. R.M. Quigley
S-87-156
1987 11 06
page 2

The arrow on the sample covering indicates the sample orientation. The specimen we are sending you is from the top end of the shelby tube.

In light of the hard nature (at least by tactile examination) of the dark grey bands, we are at a loss to explain the reason for the very high moisture content. These bands are thin, so that we have been unable to gather sufficient material to carry out Atterberg Limits, which might have helped us with a mineralogical assessment of the clay type constituent.

We are therefore requesting your laboratories to conduct whatever type of x-ray diffraction (random, oriented, etc.) tests you feel may be necessary to identify the dominant clay minerals; it might also be helpful to run a test on a glycogenated specimen to check for smectitic minerals.

The results of two consolidation tests on samples 11 and 13 are also enclosed. The letters A,B,C, etc. after the sample number denote the section of the shelby tube from which the sample was taken. Essentially each shelby tube sample was cut with a wire saw at 125 mm intervals into 4 or 5 sections, the topmost section being labelled A, the last section being D or E depending on the recovered length within the tube.

Our Client on this project is the City of Kingston, and we have made some allowance in our testing budget for the work we are now asking you to do. Please call me if you feel your charges for the requested analysis will exceed \$500 to \$750.

All shelby tube samples were extruded hydraulically and wrapped as per the specimen being sent to you. I'm sure we have other samples as well should you need more; in any case, please treat this specimen as the only one available for the moment.

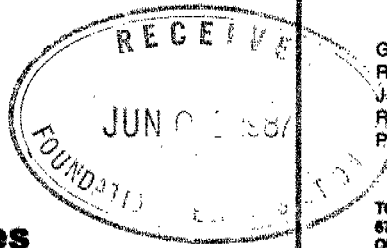
Your analysis is essential to our appraisal of the consolidation history of this clay. At issue is an old sewer pipe over which the approach fills have to be constructed to avoid adding an extra span to the bridge. A major part of the total estimated settlement below the sewer pipe results from this banded clay which underlies the pipe.

Yours sincerely,

Cam Mirza, P.Eng.

bcc: M. Devata, P.Eng., MTO
W. Blum, P.Eng., MTO Eastern
P. Smith, P.Eng., TSH Kingston

STRATA



G.L. TOTTEN B.Sc., P.Eng.
R.E. SIMS B.A.Sc., P.Eng.
J.M. HUBICKI B.A.Sc., OAA, P.Eng.
R.L. WINDOVER M.Sc., P.Eng.
P.C. EBERLEE B.A.Sc., P.Eng.

TOTTEN SIMS HUBICKI ASSOCIATES (1981) LIMITED
675 BATH ROAD, LASALLE PARK PLAZA, KINGSTON
ONTARIO, CANADA, K7M 4X2 (613) 389-3703

totten sims hubicki associates

Minutes of Meeting held at 2:00 p.m. May 27, 1987 at the Ministry of Transportation and Communications Board Room on Counter Street in Kingston, Ontario

Purpose of the Meeting

The purpose of the meeting was to discuss the proposed extension of Sir John A. MacDonald Boulevard in the City of Kingston including the CNR Overpass and Approaches. In particular, the purpose of the meeting was to discuss the Foundation Report and the comments by independent Consultants on the field data and lab results.

Present:

Mr. W.E. Blum, P.Eng. District Municipal Engineer	- M.T.C. Kingston
Mr. M. Devata, P.Eng. Chief Foundations Engineer	- M.T.C. Kingston
Mr. K. Linseman, P.Eng. Commissioner of Works	- City of Kingston
Mr. G. deLugt, P.Eng. Deputy Commissioner of Works	- City of Kingston
Mr. J. Campbell, P.Eng. Sewer Engineer	- City of Kingston
Mr. P. Smith, P.Eng.	- Totten Sims Hubicki Assoc

General:

Mr. Devata reviewed the problems with the original Soils Report and the comments by the two independent Soils Consultants and outlined the general problems with the site which are as follows:

1. There is a large trunk sewer (42" dia.) which crosses the area of the north approach fill and will be overlain by approximately 10 metres of new fill. Because this sewer is underlain by soft deposits which are deeper at the west side than the east side, the new fill will cause differential settlement in the soft underlying material and this will cause the joints of the sewer to open.
2. Based on the available field data and lab results, there is a potential problem with the stability of the deep soft deposit which will be under the north approach fills. The placing of the new approach fill may require special treatment to prevent sheer failure in this deposit.
3. There will be severe settlement when the north approach fill is placed over the deep soft deposit. The settlement can be accelerated if certain construction techniques are utilized.

Possible Solutions

1. According to available soils information, the depth of the soft deposit appears to reduce towards the east. More field information might indicate that a change in the alignment to the east would reduce the problems in the area.
2. It may be possible to bridge the sewer area with an additional span to the bridge. Cost estimates should be prepared to compare the cost of relocating the sewer versus adding the additional span. Extremely deep piling or caissons would be required to support the more northerly end of the additional span.

Minutes of Meeting held at the M.T.C. Board Room on May 27, 1987.

Possible Solutions (cont'd.)

- 3 The fill height could be reduced by a grade change or some form of light fill material could be utilized. However, due to the haul distance for light material, this option is not very feasible.
4. The deep soft deposit could be surcharged for a period of six months to a year and this would result in the majority of the settlement taking place during that period. The surcharge could then be removed when the final construction of the north approach is commenced. Surplus fill material from the City of Kingston Works projects could be used to construct the surcharge.

Recommendations

It was agreed that a more experienced Soils Consultant should be engaged and additional field investigations carried out and a new Report prepared. If requested by the City in writing, the M.T.C. would become involved and Mr. Devata would set up terms of reference for the additional soils work and would monitor the work on behalf of the City of Kingston. After some discussion regarding suitable Soils Consultants, it was agreed that Strata Engineering Incorporated would be contacted for a proposal and budget figure for the work. The principal of this Company is Mr. C. Mirza who before going into private practice was the Chief Foundations Engineer for the Ministry of Transportation and Communications.

Adjournment

The meeting adjourned at 3:30 p.m.

End of Minutes



P.A. Smith, P.Eng.

PAS/fb

c.c: Mr. W.E. Blum, P.Eng. MTC Kingston

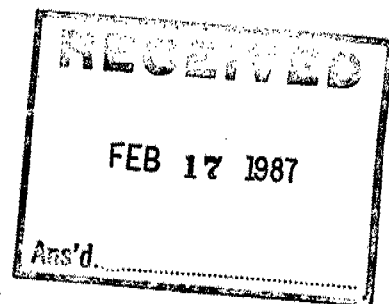
✓: Mr. M. Devata, P.Eng. MTC Kingston *Downsview*

: Mr. K. Linseman, P.Eng. City of Kingston



Golder Associates

CONSULTING GEOTECHNICAL AND MINING ENGINEERS



February 16, 1987

Our ref: 871-8006

TOTTEN SIMS HUBICKI ASSOCIATES
675 Bath Road
Lasalle Park Plaza
Kingston, Ontario
K7M 4X2

ATTENTION: Mr. P. A. Smith, P. Eng.

RE: PROPOSED OVERPASS NORTH APPROACH FILL
CNR, KINGSTON, ONTARIO

Dear Sirs:

Further to your letter of January 21, 1987 and in keeping with the terms of reference set out in your letter of January 13, 1987, we have reviewed the data provided and examined potential problems associated with construction of the north approach fill to the overpass structure. This letter gives our preliminary opinion and recommendations.

General Site Conditions

The ground surface in the area of the proposed overpasses and fills is relatively flat but ranges from about 78 m to 80 m, or more, in elevation because of variable thicknesses of random fill recently placed at the site. This fill overlies stratified (or layered) silty clays or clayey silts which range in total thickness from several metres, or less, to almost twenty metres; the deposits overly bedrock. It is the range in thickness of these deposits which typifies the problem - where they are thick

they tend to be weak and compressible. As the proposed north approach fill will be about 12 metres in total thickness at its highest point, there are potential problems of stability and settlement of the fill. These problems are compounded by the presence of an existing sanitary sewer (1070 mm diameter), some 7 m to 8 m below present ground surface, which crosses the line of the proposed fill close to the location where it is thickest.

A general outline of the fill, the line of the sewer and the location of adjacent boreholes are plotted on Sketch 1. It may be seen that, to the south of the existing sewer, bedrock is at relatively shallow depth, some 3 m to 5 m below ground surface; to the north, bedrock is much deeper, about 20 m, more or less, below ground surface. It is particularly significant that the existing sewer, at the proposed western edge of the fill, is underlain by a considerable thickness of silty clay or clayey silt whereas, at the eastern edge of the fill, these deposits are very thin, the sewer invert being close to bedrock. Any movement of the sewer caused by deformation of the clays/silts will therefore be concentrated where the deposits are thickest below the western side of the fill. To assess the degree of such differential movement, it is essential to define the detailed topography of the soil/bedrock surface in the area and the soil properties.

Properties of the Clays/Silts Underlying the North Approach Fill

It is difficult, at this stage, to assess the properties of the silty clays or clayey silts, variously described as Unit 2, in a reasonably exact manner because of the limited data and moreover because of discrepancies in the data. For example, consolidation test data are given for three (3) samples, all of which conflict. In the first plot (BH.20/Sa.11), the initial void ratio, or what may be reasonably inferred from the data to be the initial void ratio, is grossly at variance with the reported natural water content. Either the sample has dried out or was disturbed, or both. Further, the existing overburden pressure is apparently indicated as 170 kPa and the maximum past pressure, or preconsolidation pressure, is apparently represented by $\sigma'_c = 275$ kPa; how this can be derived so precisely from the data as plotted is difficult to understand; however, if true, it suggests that the compression index (C_c), a measure of the material's compressibility for loads in excess of the maximum past pressure, is about 0.15 which disagrees with common experience for glacial clays of plasticity and water content corresponding to that of the sample.

In the second plot (BH.14/Sa.7), the relationship between the inferred initial void ratio and that corresponding to the reported water content is better, however no Atterberg limits are reported and despite a more credible data pattern from which to estimate the maximum past pressure, none is indicated. One could however postulate a possible C_c value of about 0.6 which is possible for glacial clays of this water content.

The third plot (BH.15/Sa.5) is of little value except to suggest that for a clay of the water content reported the sample is markedly disturbed.

Further conflict is provided by your letter of January 21, 1987 which states that you were informed that - "the cv value used was 0.45 m² per yr or 1.43×10^{-4} cm² per sec". We do know to which consolidation test samples this refers but, in our experience, the coefficient of consolidation (a measure of the time rate at which a material tends to dissipate excess pore pressures) of most glacial clays differs by about an order of magnitude from the figure(s) quoted.

Thus, with such discrepancies and lacking exact knowledge of the soil properties, our opinion is based on a review of the general pattern of the data and on approximations or estimates which fit with experience.

Estimated Undrained Shear Strength: Data from BH's significant to the stability of the north approach fill (BH's 9 and 14) and from those BH's containing a significant thickness of similar clay/silt deposits (BH's 3 and 10) are synthesized on Sketch sheet 2. From the pattern of the standard penetration resistances ('N' Values) and of the vane shear test data, it may be inferred that the "softest" zone in three of the BH's (3, 10, 14) is about 10 m thick lying between about El. 71 (± 1 m) and El. 62, whereas in BH 9 an apparent "soft zone" of about 4 m thick exists at a higher elevation, between El. 72 and El. 76.

It is further inferred that the minimum undrained shear strength in these "soft zones" is about 50 kPa (± 10 kPa). While it is possible that the lowest undrained strengths may only exist in layers or zones of limited local extent, nevertheless, where present, they create problems of foundation stability for fills.

Estimated Compressibility: The inferred pattern of undrained strength, as discussed above, suggests that the deposits have been subjected to a maximum past pressure of at least about 200 kPa to 250 kPa, or possibly more. (This also fits with the most credible of the data as represented by BH 14/Sa.7.) To this level of total applied loading, fill settlements should be modest; above this level, settlements could be significant and approximating to that computed on the basis of a compression index of about 0.5 (± 0.1).

Stability of the Fill - End of Construction

Assuming that the approach fill were built to final height relatively quickly, the stability of the fill is largely controlled by the minimum undrained shear strength of the underlying clay/silt deposits. This minimum strength could be as low as 40 kPa at El. 72 m (± 1 m); in which case, for a fill elevation of 90 m, the computed factor of safety is approximately unity. [Even if the minimum strength were 50 kPa, the factor of safety is still less than 1.3.] We could expect considerable shear deformations within the foundation soils even if outright failure did not result. The amount of such deformation cannot be readily assessed; we can however reasonably assume that it would tend to be erratic and that it would cause erratic movement of the sewer. Consequently, to maintain stability even if the sewer were not present, consideration should be given to building the approach fill in two stages - the initial stage to a maximum thickness of about 8 m, the second stage only being added as pore pressures are observed to diminish; as a guess, this may mean fill construction over two seasons.

Possible Settlement of the Fill and Underlying Sewer

As discussed, the clay/silt deposits are probably preconsolidated to some degree, the probable maximum past pressure being estimated at about 250 kPa. Assuming the fill at its maximum height and that the conditions represented by BH's 9, 14, 15 are typical, a total maximum settlement of about 0.2 m to 0.3 m is readily possible. Any such settlement, whether elastic or long term, tends to be differential with respect to the sewer because of possible erratic variations in thickness of the clay/silt deposits. I doubt if moving the alignment of the fill by 10 m \pm would reduce significantly the amount of possible differential movement of the sewer. Similarly, I doubt the possible benefits of any ground improvement processes or underpinning the sewer in this local area and suggest that we accept the fact of differential deformation of the sewer as inevitable to some degree. As a guess, we should consider the possibility of deformation of the sewer amounting to some 0.2 m to 0.3 m over a length of say 15 m to 20 m.

The details and condition of the existing sewer are not known to me; however, two possible means of dealing with such differential movement could be examined viz. inserting a steel liner within the existing sewer accepting modified flow conditions, or, relocating the sewer to a slightly more southerly transverse route where bedrock is relatively shallow and the clay/silt deposits are thin.

Summary and Recommendations

On the basis of the available data, it is considered that:

- o The stability of the proposed north approach fill is marginal; it should be built in two stages, the first stage not exceeding 8 m; the time needed to add the second stage may be about 1 year, more or less.
- o Settlement of the fill and sewer is, to some extent, inevitable; ground improvement or underpinning the sewer are not recommended; the sewer should be examined in detail and if it cannot accept a possible differential movement of about 1 in 50, even if reinforced with a steel liner, it should be relocated. [As the depth to effect relocation is relatively shallow, it is suggested that it be considered as the better option.]

It should be recognized that the opinions above are preliminary and intended only "to examine viable options". It is probable that better definition of soil properties and of the local bedrock topography would improve and modify the opinions to some degree. It is recommended that:

- o A boring (1) should be put down to continuously sample the clay/silt deposits in the general area between existing BH's 9 and 14. The purpose of this boring is to define the minimum shear strength, assess strength/deformation properties of the "weakest" layer(s), and determine possible settlement and pore pressure response of such

layer(s). The intent of this data is to review stability, the need and timing of fill stages and the most likely deformation below the fill.

- o A series of dynamic cone tests or probings should be put down in the same general area and also along the possible route(s) of sewer relocation. The purpose is to define bedrock topography and thus improve our estimates of possible differential movement and also permit evaluation of possible sewer routes.

I trust this letter adequately covers the terms of reference. If I can be of further service, please contact Mr. D. K. J. Noonan at our Whitby office or me directly at this office.

Yours truly,

GOLDER ASSOCIATES



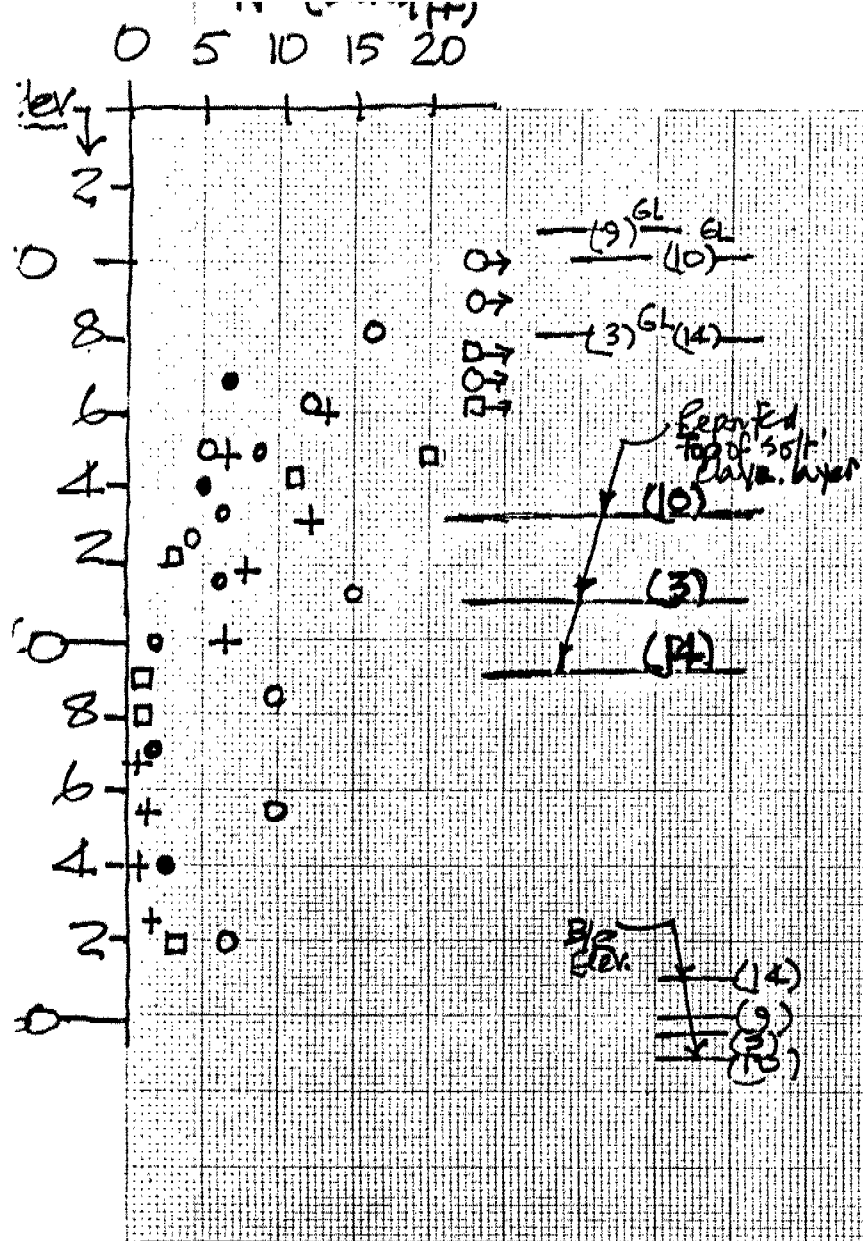
V. Milligan, P. Eng.

VM/cg

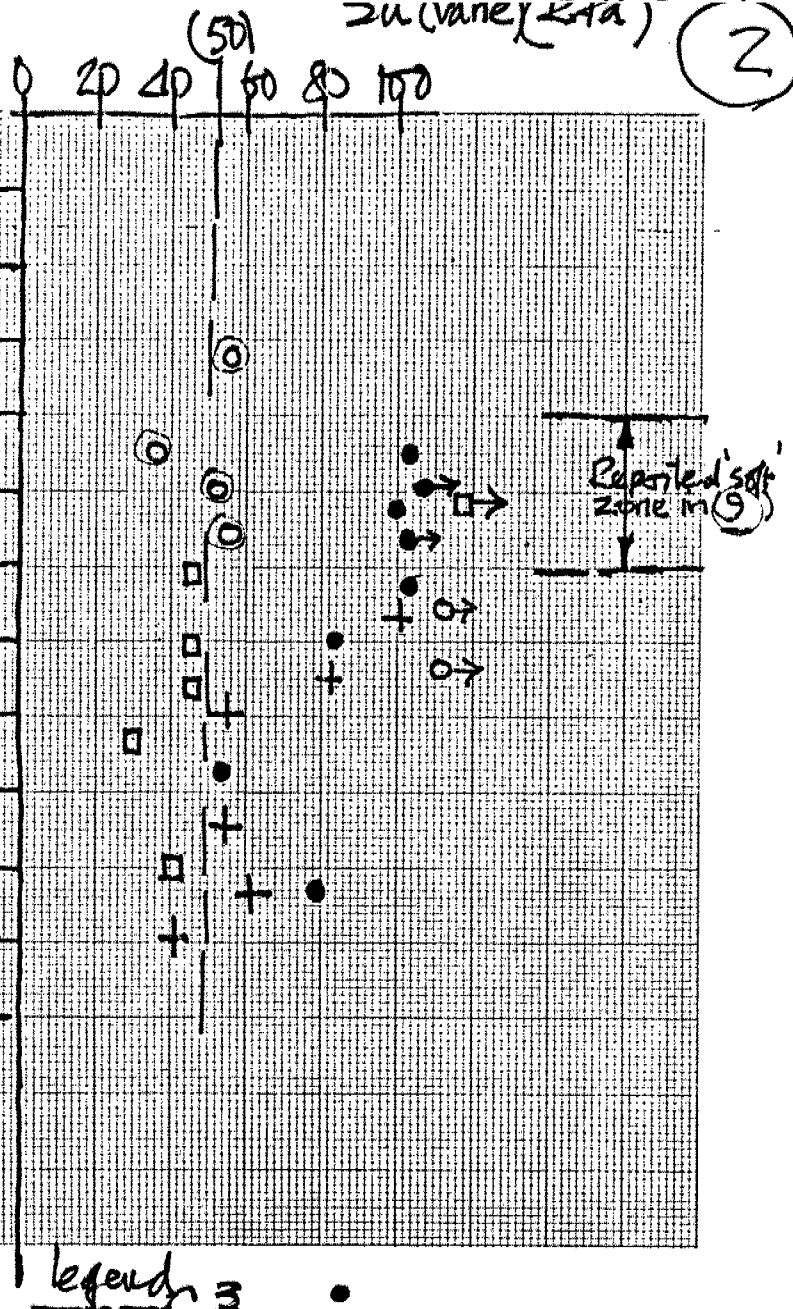
Att: as noted above

c.c. Mr. D. K. J. Noonan, P. Eng.
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Golder Associates



SYNTHESIS OF DATA
FROM BHS 3, 9, 10, 14



Legend

3

9

10

14

Adjacent to highest fill.