



FOUNDATION TECHNICAL MEMORANDUM

For

JERSEYVILLE ROAD UNDERPASS

HIGHWAY 403

MTO WEST REGION 59 STRUCTURE REHABILITATIONS

SITE 1-190, CONTRACT 8

GWP 3094-12-00

GEOGRAPHIC TOWNSHIP OF BRANTFORD

BRANT COUNTY, ONTARIO

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:

- 1 cc: MMM Group Limited for distribution to MTO + 1 digital copy (pdf)
 - Electronic Copy to MTO (David.Staseff@ontario.ca)
- 1 Electronic Copy to MTO
 - (Paul.Santos@ontario.ca; maha.almassri@ontario.ca)
- 1 Electronic Copy to MMM Group Limited
 - (LeitchS@mmm.ca) +1 digital copy (pdf)
- 1 cc: PML Kitchener
- 1 cc: PML Toronto

PML Ref.: 13KF006H-JV
Index No.: 340FTM
Geocres No.: 40P1-77
June 12, 2017



TABLE OF CONTENTS

1. INTRODUCTION	1
2. PROJECT SITE BACKGROUND AND GEOLOGY	2
3. SOURCE OF INFORMATION	2
4. SITE RECONNAISSANCE	3
5. PREVIOUS FOUNDATION INVESTIGATION AND SUBSURFACE CONDITIONS	4
5.1 General	4
5.1.1 Silty Clay	4
5.1.2 Silt	5
5.1.3 Bedrock	5
5.1.4 Groundwater	6
6. FOUNDATION	6
6.1 Previous Foundation Recommendations	6
6.1.1 Structure Foundation	6
6.1.2 Approach Embankments	7
6.1.3 Other Considerations	7
6.1.4 Drawings	8
6.2 Assessment of Foundation Parameters	9
7. DISCUSSION	10
8. CLOSURE	12

Table 1 - List of Standard Specifications

Figure 1 – Key Plan

Appendix A – Previous Foundation Investigation Reports (GEOCRE 40P1-77)

- Reference 1. Foundation Investigation Report for Jerseyville Rd. Interchange Underpass, 13.7 km West of Hwy. #2, W.P. 66-67-03, Site 1-190, Contract No. 90-95, Hwy. #403, District #4.
- Reference 2. Foundation Investigation Report for Jerseyville Rd. Interchange Underpass, 13.7 km West of Hwy. #2, W.P. 66-67-03, STR Site 1-190, Hwy. #403, District #4, dated December 1979.



- Reference 3. General Arrangement Drawing, Jerseyville Road Underpass at Hwy 403, DWG 1, Sheet 237, Site No. 01-190, Dist. No. 4, Hwy. 403, Cont. No. 90-95, WP. 66-67-03, dated Sept, 1989.
- Reference 4. Foundation Layout and Reinforcement Drawing, Jerseyville Road Underpass at Hwy 403, DWG 3, Sheet 239, Site No. 01-190, Dist. No. 4, Hwy. 403, CONT. No. 90-95, WP. 66-67-03, dated Sept, 1989.

Appendix B – Site Photographs

FOUNDATION TECHNICAL MEMORANDUM

For

Jerseyville Road Underpass Highway 403
MTO West Region 59 Structure Rehabilitations
Site 1-190, Contract 8, GWP 3094-12-00
Geographic Township of Brantford
Brant County, Ontario

1. INTRODUCTION

The Foundation Engineering Services for the present project involve the detail foundation investigation and design for the rehabilitation of 59 structures in MTO West Region along Highways 4, 6, 401, 402 and 403. Ten (10) Group Work Projects (GWP's) are contemplated to be completed between 2014 and 2020.

This technical memorandum summarizes the factual results of geotechnical data based on the review and compilation of existing subsurface information from relevant reports in the MTO GEOCRES Library for the Jerseyville Road Underpass Highway 403. The Foundation Engineering recommendations from the initial foundation reports are summarized with reference to the "Canadian Highway Bridge Design Code" (CHBDC) and follow in general the "Guidelines for Professional Engineers providing Geotechnical Engineering Services".

From the Minutes of Meeting Report, dated June 3, 2016, it is understood that semi-integral conversion may not be the preferred rehabilitation strategy and that Preservation Management Strategy (PMS) consisting of expansion joint replacement, patch, waterproof and pave, patch repairs and concrete sealer applied to the concrete barrier walls to be undertaken at this underpass structure location.

The purpose of the technical memorandum is to summarize the subsurface and groundwater conditions and foundation recommendations based on available reports at the structure location for the design project team's reference.

The elevations in this report are expressed in meters, unless otherwise noted.



2. PROJECT SITE BACKGROUND AND GEOLOGY

The Jerseyville Road Underpass Highway 403 is located about 8.0 km north of the Town of Burford in the Geographic Township of Brantford, Brant County, Ontario. A key plan is shown in Figure 1.

The existing underpass is a two span post tensioned concrete voided slab structure that carries two through lanes over Highway 403. The immediate vicinity is relatively flat. Agricultural lands at the north and south sides were observed in the vicinity of the structure.

Physiographically, the site of the underpass is located in the region referred to as the Haldimand Clay Plain, which includes thick deposits of clay and silt deposited by Whittlesey and Warren glacial lakes overlying dolomite bedrock of the Salina Formation of the Upper Silurian Epoch. The bedrock surface at the site location is between elevation 181.0 and 183.0, typically 36.0 to 38.0 m below ground based on previous investigation.

3. SOURCE OF INFORMATION

The following reports and drawings, appended in Appendix A, were available for review and information for the underpass structure, subsoil information and original foundation recommendations.

1. Foundation Investigation Report for Jerseyville Rd. Interchange Underpass, 13.7 km West of Hwy. #2, W.P. 66-67-03, Site 1-190, Contract No. 90-95, Hwy. #403, District #4, Hamilton, Ministry of Transportation and Communications, GEOCRE No. 40P1-77. (Reference 1)
2. Foundation Investigation Report for Jerseyville Rd. Interchange Underpass, 13.7 km West of Hwy. #2, W.P. 66-67-03, STR Site 1-190, Hwy. #403, District #4, Hamilton, Engineering Materials Office, Pavement and Foundation Design Section, Ministry of Transportation and Communications, dated December 1979, GEOCRE No. 40P1-77. (Reference 2)



3. General Arrangement Drawing, Jerseyville Road Underpass at Hwy 403, DWG 1, Sheet 237, Site No. 01-190, Dist. No. 4, Hwy. 403, Cont. No. 90-95, WP. 66-67-03, Morrison Hershfield Limited Consulting Engineers, dated Sept, 1989. (Reference 3)
4. Foundation Layout and Reinforcement Drawing, Jerseyville Road Underpass at Hwy 403, DWG 3, Sheet 239, Site No. 01-190, Dist. No. 4, Hwy. 403, CONT. No. 90-95, WP. 66-67-03, Morrison Hershfield Limited Consulting Engineers, dated Sept, 1989. (Reference 4)

4. SITE RECONNAISSANCE

As part of the current foundation engineering assessment study, a site reconnaissance of the Jerseyville Road Underpass Highway 403 was carried out on August 28, 2015.

The site photographs present the conditions of the Jerseyville Road Underpass including visible portions of the abutments and pier, and abutment slope assessment based on visible areas, apparent areas of soil erosion and abutment slope cover.

The site inspection revealed that the vicinity of the underpass structure abutment locations was covered by vegetation. Both abutment front slopes were covered by concrete panels (Photographs 1 and 4). Weep holes were observed in the abutment walls, which appeared to be open and functioning satisfactorily to drain the water from behind the walls. Shrubs and grasses were observed growing between the concrete panels where the sealant between the panels was degraded (Photographs 1 and 2). Vertical cracks were observed on the abutment walls. Sloughing of ground beneath the concrete panels were observed. Cracks, breaks and dislodge of the concrete panels were also observed (Photographs 1 to 6). Both east and west slopes adjacent to the abutments were observed to be vegetated and no evidence of slope erosion was noted (Photographs 8 to 11). Minor cracks were observed on the east and west wingwalls of the north and south abutments. The centreline pier at the time of site reconnaissance was observed with minor surficial cracks with no spalling of concrete or exposure of rebar (Photograph 7).



5. PREVIOUS FOUNDATION INVESTIGATION AND SUBSURFACE CONDITIONS

A revised foundation investigation report (Reference 1) presenting only the factual subsurface and groundwater conditions was prepared by the Department of Transportation and Communications. The general subsurface conditions presented in this section are based on the foundation investigation report referred in Reference 1.

The field investigation was carried out on March 15 and 16, 1976 and July 26 to 30, 1979. A total of three sampled boreholes 1 to 3 accompanied by dynamic cone penetration test (DCPT) adjacent to boreholes 2 and 3 were carried out at the site location. The sampled boreholes were advanced to 36.6 to 37.1 m, elevation 181.5 to 182.3. In borehole 1, the borehole was extended 11.8 m into the bedrock to 38.4 m, elevation 180.3. The two DCPTs were advanced to approximate termination depths of 6.4 and 9.7 m, elevation 213.0 to 208.8.

The Foundation Investigation Report (Reference 1) includes the Borehole Locations & Soil Strata Drawing (DWG 2). The general layout of the structure is shown in Reference 3.

The boreholes were drilled by employing continuous flight auger machines, mounted either on a muskeg vehicle or on an all-terrain vehicle and equipped with 83 mm I.D. hollow stem augers and rotary core (BX) drilling.

5.1 General

Generally, in the sampled boreholes, uniform soil conditions were encountered at the site location which included a firm to very stiff 36.6 to 37.1 m thick stratified silty clay followed by dolomite bedrock.

5.1.1 Silty Clay

A 36.6 to 37.1 m thick firm to very stiff stratified silty clay stratum was encountered in all boreholes surficially, elevation 218.5 to 219.4, which extended to the bedrock/probable bedrock, elevation 181.5 to 182.3. The stratification was random and ranged approximately



between 5 and 150 mm. Silt seams were observed throughout this stratum. The plasticity of the individual layers varied from low to high. N values recorded ranged from 7 to 19. At depths where bedrock/probable bedrock was encountered in the boreholes, N values recorded were 38 and above 100. The field vane test results obtained unconfined shear strength values between 41 and over 100 kPa with sensitivity ranging from 2 to 5.

Grain size distribution results of selected silty clay samples determined the samples included 0 to 1% sand, 44 to 91% silt and 8 to 56% clay sizes particles. The Atterberg liquid limits ranged from 22 to 63 and the corresponding plastic limits ranged from 13 to 21. The plasticity index values ranged from 5 to 42. Moisture content determinations ranged from 13 to 46%.

The range of shear strength values obtained in the laboratory test results are summarized in the following table:

Test Method	Range (kPa)
Unconfined	39 to 96
Quick Triaxial	48 to 101

One consolidation test performed on a selected silty clay sample indicated that the soil was overconsolidated with a preconsolidation pressure of 435 kPa.

The report recommended an average undrained shear strength value of 60 kPa for design purposes in terms of total stresses.

5.1.2 Silt

A very loose to dense 2.0 to 2.4 m thick silt layer was encountered in all three boreholes within the silty clay deposit at 1.3 to 1.7 m, elevation 217.3 to 217.7, which extended to 3.4 to 4.1 m, elevation 214.8 to 215.3. N values recorded within this layer ranged between 4 and 27.

5.1.3 Bedrock

Probable bedrock was encountered in boreholes 2 and 3 at termination depths 37.1 and 37.0 m, elevation 182.3 and 181.5, respectively.



In borehole 1, a 1.8 m bedrock core length (98% recovery) was obtained from 36.6 to 38.4 m, elevation 182.1 to 180.3. The bedrock core consisted of moderately fractured, hard, light grey to white dolomite.

5.1.4 Groundwater

In boreholes 1 and 3, groundwater was observed at 3.9 m, elevation 214.8 and 214.6. In borehole 2, groundwater was not observed. It was noted that the subsoil encountered was relatively impermeable and that a considerable amount of time was required for the groundwater to stabilize.

The report assumed the groundwater level at this site at approximate elevation 215.0 for design and construction purposes. It was noted that the groundwater levels may be influenced by seasonal fluctuations.

6. FOUNDATION

6.1 Previous Foundation Recommendations

The previous foundation recommendations presented in the following sections are based on the original report (Reference 2). The recommendations were provided for a two span (25.5 m each span) structure at the junction of Highway 403 and Jerseyville Road. The profile of Highway 403 was designed to be at elevation 216.2 (H.O.T 14+196.308 CL median of Highway 403) with a 1.065% gradient increase from west to east. The profile of Jerseyville Road was designed to be raised approximately 3.4 m to elevation 222.4 (H.O.T. 10+000.000 CL Jerseyville Road). The average original ground level was at approximate elevation 219±. At the site location, firm to very stiff 36.6 to 37.1 m thick stratified silty clay was encountered overlaying dolomite bedrock.

6.1.1 Structure Foundation

The original foundation investigation report (Reference 2) recommended that spread foundations for the proposed structure were not feasible due to relatively low bearing capacity and settlement



considerations. It was recommended that the pier and abutments be supported on piles driven to bedrock, approximate elevation 182 ±. The report indicated that steel tubular piles 323.9 mm O.D. @ 49.73 kg/m or HP 310 x 110 steel 'H' piles with reinforced tips could be driven to bedrock. The maximum permissible load recommended in the report was 1100 kN. As for concrete piles, the report indicated that the maximum load would be dependent on the manufacturers' specification. In order to successfully drive the steel tube piles to bedrock without damage to the piles, the report recommended that the energy of the hammer used at the instant of contact with bedrock must not exceed 40,000 joules/blow and that the energy had to be restricted accordingly for the last 2.0 m of driving.

6.1.2 Approach Embankments

The report (Reference 2) indicated that up to 3.0 m deep cuts and up to 4.0 m high fills will be required, respectively, to establish the proposed profile grades of Highway 403 and Jerseyville Road. No stability problems were expected for the approach fills provided the approaches were constructed with 2 horizontal to 1 vertical forward and side slopes. It was recommended that the approach fills at the abutment locations be devoid of bouldery fill material with no larger grain sizes than 50 mm through which piles had to be driven. It was estimated that the silty clay subsoil would settle in the range of 75 to 100 mm due to the construction of approach fills and that the settlement would take place over a long term period. In order to minimize the settlement effects, it was recommended that the approach embankments be built in advance of the final grading and paving for as long a period as possible.

6.1.3 Other Considerations

The report recommended a minimum 1.3 m cover for the pile caps for frost protection.

No major dewatering problems were anticipated at the site location due to the impermeable nature of the subsoil encountered. It was recommended that the topsoil and/or soft surficial material be removed according to Ministry of Transportation and Communications (M.T.C.) practices.



It was anticipated that the abutments of the proposed underpass structure in part would be located over the then existing Jerseyville Road alignment. The report recommended that to avoid damages to the abutment piles during driving, the entire roadbed (pavement and base coarse) be excavated to its full vertical and horizontal extent.

The following values were recommended to estimate the earth pressures on the abutment walls.

Unit Weight of Granular Backfill: 21.2 kN/m³
Coefficient of Active Earth Pressure: $K_a = 0.35$
Coefficient of Earth Pressure at Rest: $K_o = 0.5$

The report recommended that a suitable drainage system be provided to relieve the build-up of excess hydrostatic pressure behind the abutment walls.

Further, the report recommended that the structure be designed with approach slabs to provide a smooth transition between the structure and the approaches, which would undergo settling for a long duration.

Erosion protection was to be provided for the exposed cut and fill slopes according to M.T.C. standards.

6.1.4 Drawings

Based on the General Arrangement Drawing (Reference 3), the proposed structure was going to carry the Jerseyville Road over Highway 403. The grades at the approaches were raised approximately 3.0 to 4.0 m from then existing ground levels. A post-tensioned structure type was to be constructed. The top of the 1000 mm thick pile caps was to be placed at elevation 217.500 and 217.000 at the north and south abutments, respectively and the top of the 1500 mm thick pile cap for the centre pier was to be placed at elevation 214.000. In the drawing, it was shown that HP 310x110 piles at the abutments and centre pier were to be driven to bedrock. A 150 mm diameter perforated subdrain was to be placed behind each abutment wall.



Based on the Foundation Layout and Reinforcement Drawing (Reference 4), the 310x110 H piles were to be driven to bedrock. The following pile design data was provided in the drawing:

Capacity at S.L.S., Type II = 1150 kN
Factored Capacity at U.L.S = 1600 kN

The following table summarizes the pile data based on the Footing Layout and Reinforcing Drawing (Reference 4). The pile lengths shown in the following table were the theoretical lengths below cut-off.

SUMMARY OF PILE DATA				
LOCATION (PILE TYPE)	BATTER	NUMBER REQUIRED	LENGTH, mm	PILE CUT-OFF ELEVATION, m
North Abutment (HP 310x110 Piles)	1:3	7	36 500	216.800
	1:5	2	35 000	
	1:12	2	34 500	
Centre Pier (HP 310x110 Piles)	1:5	12	31 500	212.800
	Vertical	2	31 000	
South Abutment (HP 310x110 Piles)	1:3	7	36 000	216.300
	1:5	3	34 500	
	1:12	2	34 000	

6.2 Assessment of Foundation Parameters

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



FOUNDATION DESIGN PARAMETERS					
FOUNDATION TYPE (LOCATION)	PROBABLE PILE TIP ELEVATION ¹	PREVIOUS LIMIT STATE DESIGN VALUES ²		LIMIT STATE DESIGN VALUES UPDATED TO CURRENT INDUSTRY PRACTICE ³	
		SLS TYPE II BEARING CAPACITY	ULS FACTORED CAPACITY	SLS BEARING REACTION/LOAD	ULS FACTORED GEOTECHNICAL RESISTANCE/LOAD
North abutment (HP 310x110 Piles)	181.5 to 182.3	1150 (kN)	1600 (kN)	1150 (kN)	1600 (kN)
Centre Pier (HP 310x110 Piles)					
South abutment (HP 310x110 Piles)					

Notes: 1. Founding elevations from Reference 1 (Pile founding elevations) and 4.
2. Limit State Design values from Reference 4.
3. Limit State Design values based on CHBDC (2014 Edition).

The Peak Ground Acceleration (PGA) for the site is 0.109 (National Building Code of Canada, 2015). The soil classification for seismic design should be in accordance with Clause 4.4.3.2 of the CHBDC (2014).

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.4 of the CHBDC 2014 Edition.

The foundation frost penetration depth at the site is 1.2 m according to OPSD 3090.101.

7. DISCUSSION

It is understood that semi-integral conversion may not be the preferred rehabilitation strategy and that Preservation Management Strategy (PMS) consisting of expansion joint replacement, patch, waterproof and pave, patch repairs and concrete sealer applied to the concrete barrier walls to be undertaken at this underpass structure location. From a geotechnical point of view, at the present time, foundation work for underpass structure is not expected.

However, if any major rehabilitation is undertaken for the proposed interchange at this location, it is recommended that the foundation capacity at the abutment locations should be verified prior to



any major construction work. Further, the Structural Engineer should verify the pile type and configuration used for the underpass structure.

A temporary support system may be required for the rehabilitation of the underpass structure and the construction for temporary support system should conform to OPSS 404 and 539. The contractor is responsible for the selection, detailed design and performance of the roadway protection scheme. The contractor should monitor the movement of the roadway protection system.

The concrete panels on the front slopes of north and south abutments require rehabilitation based on the current conditions observed. The slopes adjacent to both abutments are visually stable without signs of erosion.

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



8. CLOSURE

This Technical Memorandum was prepared by Mr. N. Rahman, P.Eng, Project Engineer and was reviewed by Mr. B. R. Gray, M.Eng, P.Eng., Principal Consultant. Mr. R. Ng, MBA, PhD, P.Eng., MTO Designated Principal Contact conducted an independent review of the report.

We trust this memo is sufficient for your immediate needs. Please, do not hesitate to contact us if you have any inquiries and/or comments.

Yours very truly,

Peto MacCallum Ltd.



Nazibur Rahman, P.Eng.
Project Engineer



Brian R. Gray, M.Eng, P.Eng.
Principal Consultant



Robert Ng, MBA, PhD, P.Eng.
MTO Designated Principal Contact



TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 404	Construction Specification for Support Systems
OPSS 539	Construction Specification for Temporary Protection Systems
OPSD 3090.101	Foundation Frost Depth for Southern Ontario

Figure 1 – Key Plan





APPENDIX A

Appendix A – Previous Foundation Investigation Reports (GEOCRE 40P1-77)

- Reference 1. Foundation Investigation Report for Jerseyville Rd. Interchange Underpass, 13.7 km West of Hwy. #2, W.P. 66-67-03, Site 1-190, Contract No. 90-95, Hwy. #403, District #4.
- Reference 2. Foundation Investigation Report for Jerseyville Rd. Interchange Underpass, 13.7 km West of Hwy. #2, W.P. 66-67-03, STR Site 1-190, Hwy. #403, District #4, dated December 1979.
- Reference 3. General Arrangement Drawing, Jerseyville Road Underpass at Hwy 403, DWG 1, Sheet 237, Site No. 01-190, Dist. No. 4, Hwy. 403, Cont. No. 90-95, WP. 66-67-03, dated Sept, 1989.
- Reference 4. Foundation Layout and Reinforcement Drawing, Jerseyville Road Underpass at Hwy 403, DWG 3, Sheet 239, Site No. 01-190, Dist. No. 4, Hwy. 403, CONT. No. 90-95, WP. 66-67-03, dated Sept, 1989.

FOUNDATION INVESTIGATION REPORT

CONTRACT NO 90-95



Ministry of
Transportation and
Communications

INDEX

<u>Page No:</u>	<u>DESCRIPTION</u>
1	Index
2	Abbreviations & Symbols
3 - 64	Foundation Investigation Report for: Jerseyville Rd. Interchange Underpass W.P. 66-67-03, Site 1-190 Hwy. 403, District 4 (Burlington) Fairchild Creek Twin Structures W.P. 66-67-05, Site 1-192 Hwy. 403, District 4 (Burlington) Johnson Rd. Underpass W.P. 66-67-06, Site 1-193 Hwy. 403, District 4 (Burlington) Garden Ave. Interchange Underpass W.P. 66-67-07, Site 1-194 Hwy. 403, District 4 (Burlington)

Note: For purposes of the contract, this report supercedes all other Foundation Reports prepared by, or for the Ministry in connection with the above mentioned project.

EXPLANATION OF TERMS USED IN REPORT

2

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 492 J IMPACT ENERGY ON 1" SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

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

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

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

	SPACING	30mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING		VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING		VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
t_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{VO}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	°	APPARENT ANGLE OF INTERNAL FRICTION
τ_r	kPa	RESIDUAL SHEAR STRENGTH
S_r	1	SENSITIVITY = $\frac{c_u}{\tau_f}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1	VOID RATIO	e_{min}	1	VOID RATIO IN DENSEST STATE
γ_s	kn/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	% PERCENT - DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kn/m^3	UNIT WEIGHT OF DRY SOIL	i_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1	VOID RATIO IN LOOSEST STATE	j	kn/m^2	SEEPAGE FORCE
γ'	kn/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

FOUNDATION INVESTIGATION REPORT

for

Jerseyville Rd. Interchange Underpass
13.7 km West of Hwy. #2
W.P. 66-67-03, Site 1-190
Hwy. #403, District 4, Hamilton

INTRODUCTION

This report contains the results of the foundation investigation carried out for the proposed structure at the junction of the existing Jerseyville Rd. and New Hwy. #403, during the periods of 1976 03 15 - 16 and 1979 07 26 - 30. The fieldwork consisted of three sampled boreholes and two dynamic cone penetration tests. The borings were advanced by employing continuous flight auger machines, mounted either on a muskeg vehicle or on an all-terrain vehicle and equipped with 82 mm I.D. hollow stem augers.

SITE DESCRIPTION

The site is located at the future junction of the existing Jerseyville Rd. and proposed New Hwy. #403, in the Township of Brantford.

The surrounding terrain is relatively flat, cultivated agricultural land.

The site lies in the physiographic region known as the Haldimand Clay Plain.

This extensive plain consists of stratified clay, silts and sands deposited by glacial lakes Whittlesey and Warren.

SUBSURFACE CONDITIONS

General

Generally uniform subsoil conditions were found to exist across the site. The subsoil (apart from the existing Jerseyville Rd. roadway material) consists of a deep deposit (36 - 38 m) of stratified silty clay with trace of sand followed by dolomite type bedrock.

The boundary between the overburden and bedrock together with the field and laboratory tests results are shown on the Record of Borehole Sheets contained in the Appendix. The stratigraphical profile shown on Drawing No. 2 of the Contract Documents is based on this information. The drawing also shows the locations and elevations of the borings. A detailed description of the encountered subsurface conditions is given below.

Silty Clay Trace of Sand

This stratum was intersected in all borings and extends from immediately below the ground surface to the bedrock for a depth of 36 - 38 m. The material in the deposit is stratified and classified as silty clay with trace of sand. The stratification is rather random and ranges in thickness from 5 mm to about 150 mm. The plasticity of the individual layers varies from low to high. Occasional silt seams were also observed throughout the stratum. The Atterberg Limit test results for the overall deposit are plotted on the plasticity chart (Figure 1). The consistency of the stratum varies randomly from stiff to very stiff. The assessment is based on a number of field vane and laboratory unconfined and quick triaxial tests. The test results are plotted on Figure 2 and summarized below, together with other physical properties.

	Range
Natural Moisture Content (W) %	13 - 46
Liquid Limit (W _L) %	22 - 63
Plastic Limit (W _P) %	13 - 21
Undrained Shear Strength (c _u) %	7 - 14
Unconfined	39 - 96 kPa
Quick Triaxial	48 - 101 kPa
Field Vane	41 - over 100 kPa
Unit Weight (γ)	19.3 - 20.5 kN/m ³
Sensitivity (Based on Field Vane Tests)	2 - 5

Grain size distribution curves are presented in an envelope form on Figure 3 of the Appendix.

One consolidation test was performed on a sample obtained from this stratum. The tests indicate that the soil is overconsolidated with a preconsolidation pressure of 435 kPa.

For design purposes in terms of total stresses, an average undrained shear strength value of 60 kPa is recommended.

Silt

A loose to dense, approx. 2 to 2.4 m thick silt with traces of sand and clay zone was found to be sandwiched between the brown and grey portion of the silty clay deposit.

Bedrock

Bedrock was found at depths about 36.6 below ground (elevation 182±) which consists of moderately fractured, hard, light grey to white dolomite.

Groundwater Conditions

The following groundwater levels were observed at the boring locations:

- B.H. #1 - Elevation 214.8
- B.H. #2 - not observed
- B.H. #3 - Elevation 214.6

It is pointed out that the subsoil is relatively impermeable, therefore a considerable time is required for the water levels to stabilize.

For design and construction purposes, it should be assumed that the groundwater level at this site is probably at elevation 215±. Seasonal changes may influence the groundwater levels.



P. Payer
P. Payer, P. Eng.
Sr. Foundation Engineer

M. Devata
M. Devata, P. Eng.
Chief Foundation Engineer

APPENDIX

RECORD OF BOREHOLE No 1 (0.0 - 30.2 m) METRIC

W P 56-67-03 LOCATION Co-ords. N 4 871 755.0; E 250 314.0 ORIGINATED BY PRK
DIST 4 HWY 403 BOREHOLE TYPE Hollow Stem Auger, BX-Corr COMPILED BY PRK
DATUM Geodetic DATE 79 07 26/27 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES			20 40 60 80 100	100	W _p	W	W _L		
218.7	Ground Level							SHEAR STRENGTH kPa		WATER CONTENT (%)				
								O UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL x LAB VANE						
								25 50 75 100 125		15	30	45		
0.0	Silty clay						218							
217.3	Stratified		1	SS	21									
	Very stiff - brown		2	SS	17									
1.4	Silt		3	SS	15									
	Trace of sand and		4	SS	27									
215.3	clay		5	SS	13									
	Trace of sand		6	SS	10									
3.4	Occasional layers and		7	SS	8									
	seams of silt		8	TV	PH									
	Firm to very stiff		9	SS	10									
	Grey coloured		10	TV	PH									
			11	SS	11									
			12	TV	PH									
			13	SS	10									
			14	TV	PH									
			15	TV	PH									
			16	TV	PH									
			17	SS	19									
186.53														

OFFICE REPORT ON SOIL EXPLORATION

Continued

*3, *5: Numbers refer to
Sensitivity

20
15
10
5
0
5
10
15
20
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 1 Cont. (30.2 - 38.4 m)

METRIC

W P 66-67-03 LOCATION _____ ORIGINATED BY PRR
DIST 4 HWY 403 BOREHOLE TYPE Hollow Stem Auger, BT-Core COMPILED BY PRR
DATUM Guelph, Ont DATE 79 07 25/27 CHECKED BY _____

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100				
184.53																
30.2	Silty clay	N														
			18	SS	15											
182.13			19	SS	40/	0.0 m	Bounding									
36.6	Bedrock moderately fractured Dolomite		20	RC	521											
180.33				EX	REC											
38.4	End of Borehole															

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 2 (0.0 - 30.2 m)

METRIC

W P 66-67-03 LOCATION Co-ords. N 4 871 308.0; E 350 326.0 ORIGINATED BY PRK
DIST 4 HWY 403 BOREHOLE TYPE Hollow Stem Auger COMPILED BY PRK
DATUM Gendetta DATE 19 07 30 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
219.4	Ground Level																
0.0	Silty clay Stratified		1	SS	20	Not Observed	218										0 0 88 12
217.7	Very stiff - brown		2	SS	17		216										0 0 60 40
1.7	Silt Trace of sand and clay Compact		3	SS	21		214										0 0 45 55
213.3			4	SS	17		212										0 0 90 10
4.1	Silty clay Stratified		5	SS	6		210										
	Traces of sand		6	SS	9		208										
	Occasional layers and seams of silt		7	TW	PH		206										
	Firm to very stiff		8	SS	8		204										
	Gray coloured		9	TW	PH		202										
			10	TW	PH		200										
			11	SS	14		198										
			12	TW	PH		196										
			13	SS	7		194										
			14	TW	PH		192										
198.84			15	TW	PH		190										

OFFICE REPORT ON SOIL EXPLORATION

30.2 Continued

+3, x5: Numbers refer to
Sensitivity

20
15
10
% STRAIN AT FAILURE



Ministry of
Transportation and
Communications

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

10

RECORD OF BOREHOLE No 2 Cont. (30.2 - 37.1 m)

METRIC

W P 66-67-03 LOCATION _____ ORIGINATED BY ZBK
 DIST 4 HWY 403 BOREHOLE TYPE Hollow Stem Auger COMPILED BY ZBK
 DATUM Geodetic DATE 79.07.30 CHECKED BY _____

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL MOISTURE LIMIT CONTENT LIMIT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
DEPTH M	DESCRIPTION	NUMBER	TYPE			20	40	60	80	100	W _p	W	W _L		
188.4															
30.20	Silty clay	16	SS	18	188										
					186										
					184										
37.1	Probable Bedrock End of borehole	17	SE	13-6	180										

OFFICE REPORT ON SOIL EXPLORATION

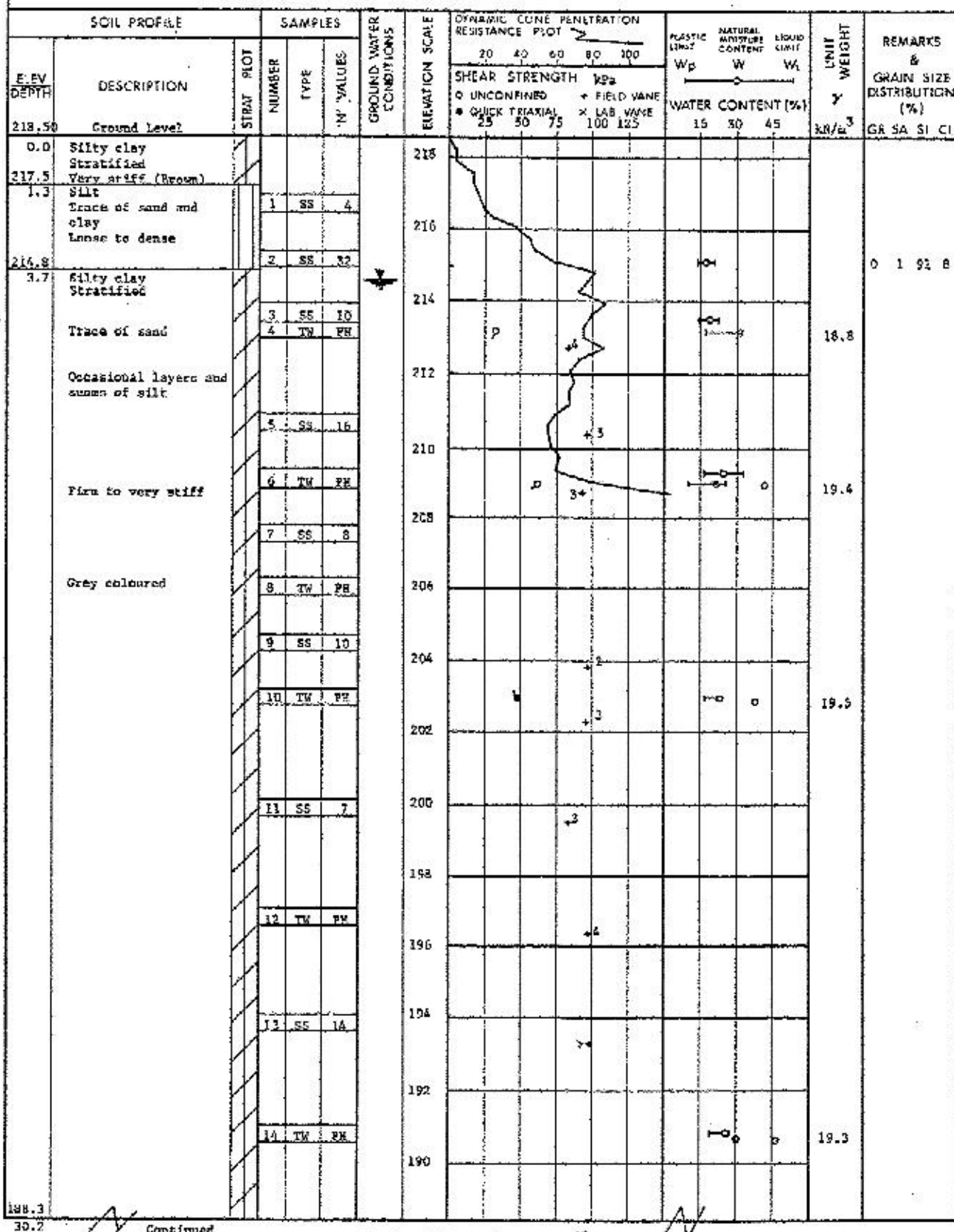
4, 5: Numbers refer to
Sensitivity

20
15
10
5
0
5
10
15
20
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3 (Formerly R.H. #5)
(R.P. 66-67-01)

METRIC

W.P. 66-67-03 LOCATION Co-ords. N 4 871 274.1; E 250 292.6 ORIGINATED BY MK
DIST 4 HWY 403 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MK
DATUM Geodetic DATE 76 03 15/16 CHECKED BY



+3, x5: Numbers refer to
Sensitivity

20
15
10
+5 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

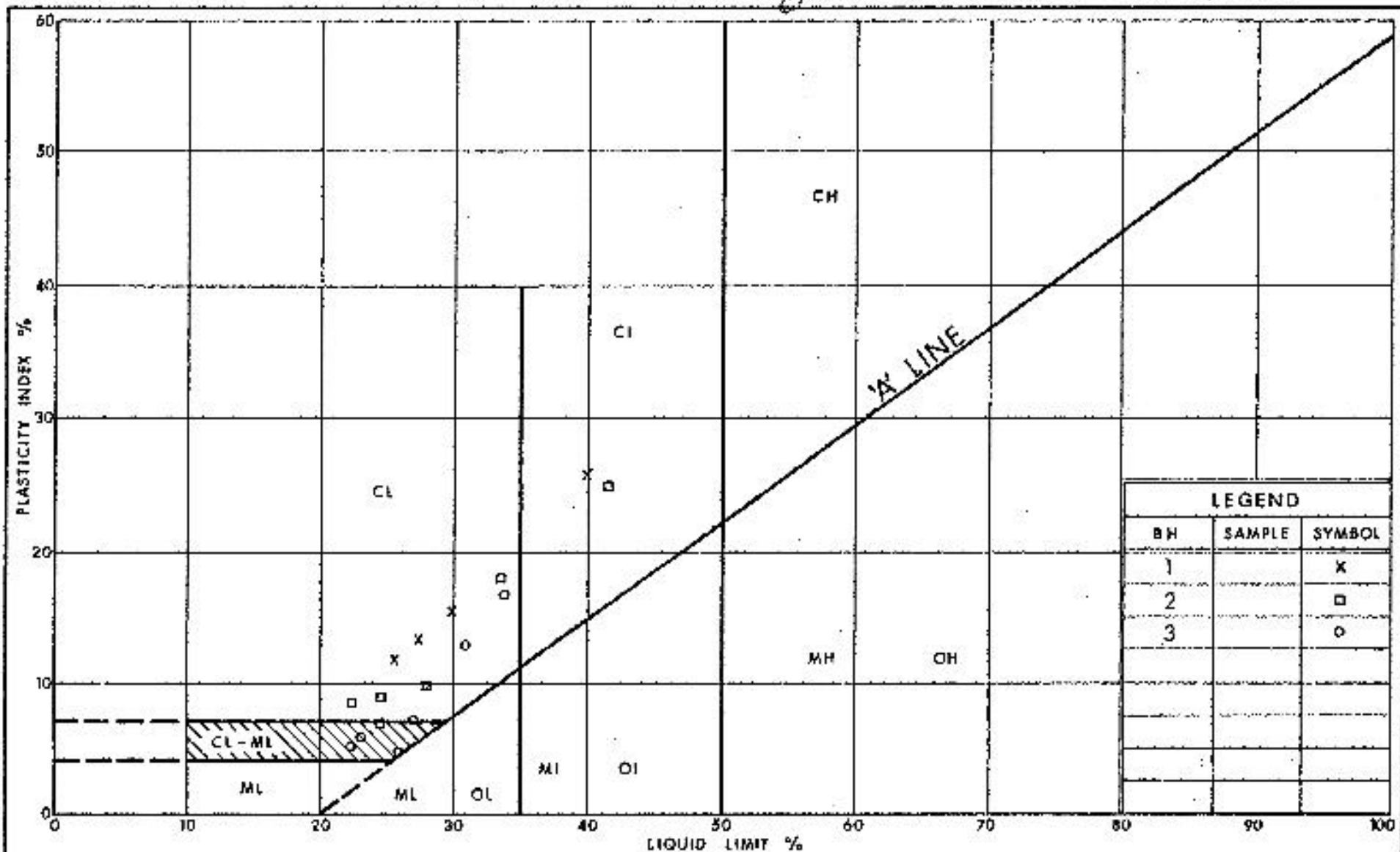
RECORD OF BOREHOLE No 3 Cont. (Formerly E.H. #5)

METRIC

W P 66-57-03 LOCATION _____ ORIGINATED BY MR
DIST 4 HWY 403 BOREHOLE TYPE Hollow Stem Auger COMPILED BY _____
DATUM Roadside DATE 75 03 15/16 CHECKED BY _____

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
188.30																	
180.20	Silty clay		15	SS	12		188										
							186										
			16	T2	PR		184								19.5		
							182										
181.5	Some sand and gravel		17	SS	36												
37.0	End of Borehole Probable Bedrock																

OFFICE REPORT ON SOIL EXPLORATION



Ministry of
Transportation and
Communications

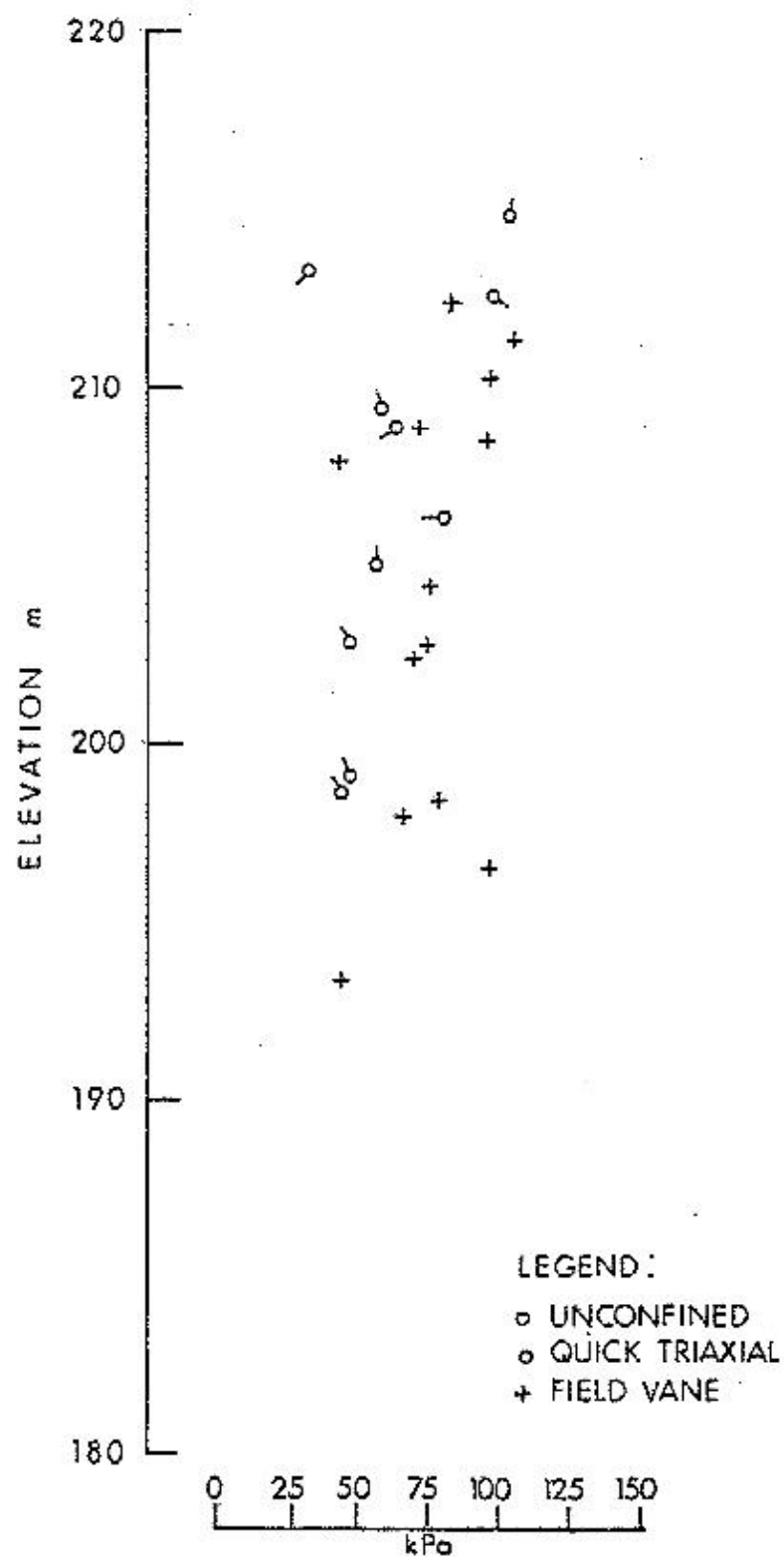
Ontario

ENGINEERING SERVICES BRANCH

PLASTICITY CHART SILTY CLAY STRATIFIED TRACE OF SAND

FIG No 1

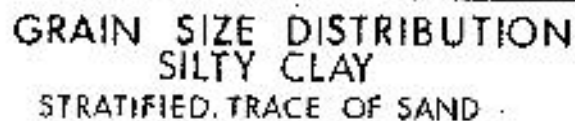
W P 66-67-03



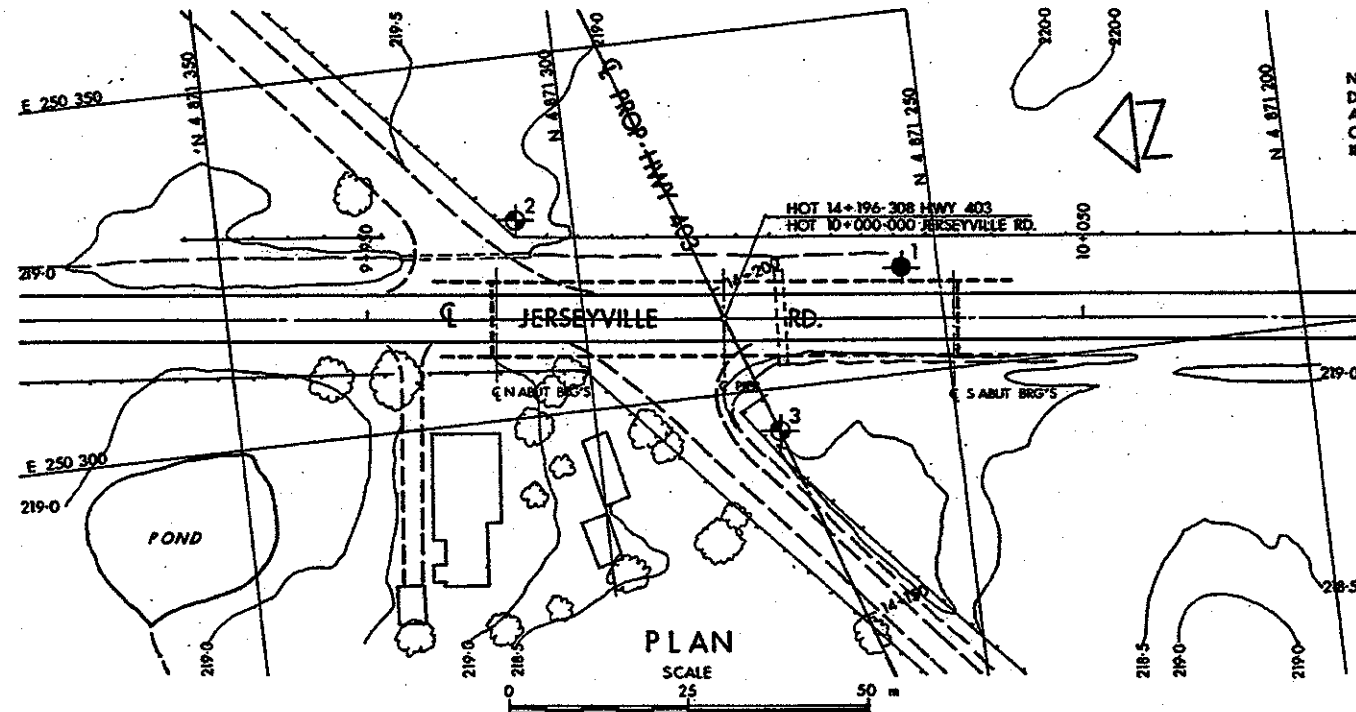
SHEAR STRENGTH VS ELEVATIONS

FIG 2

W.P. 66-67-03



W P 66-67-03



METRIC

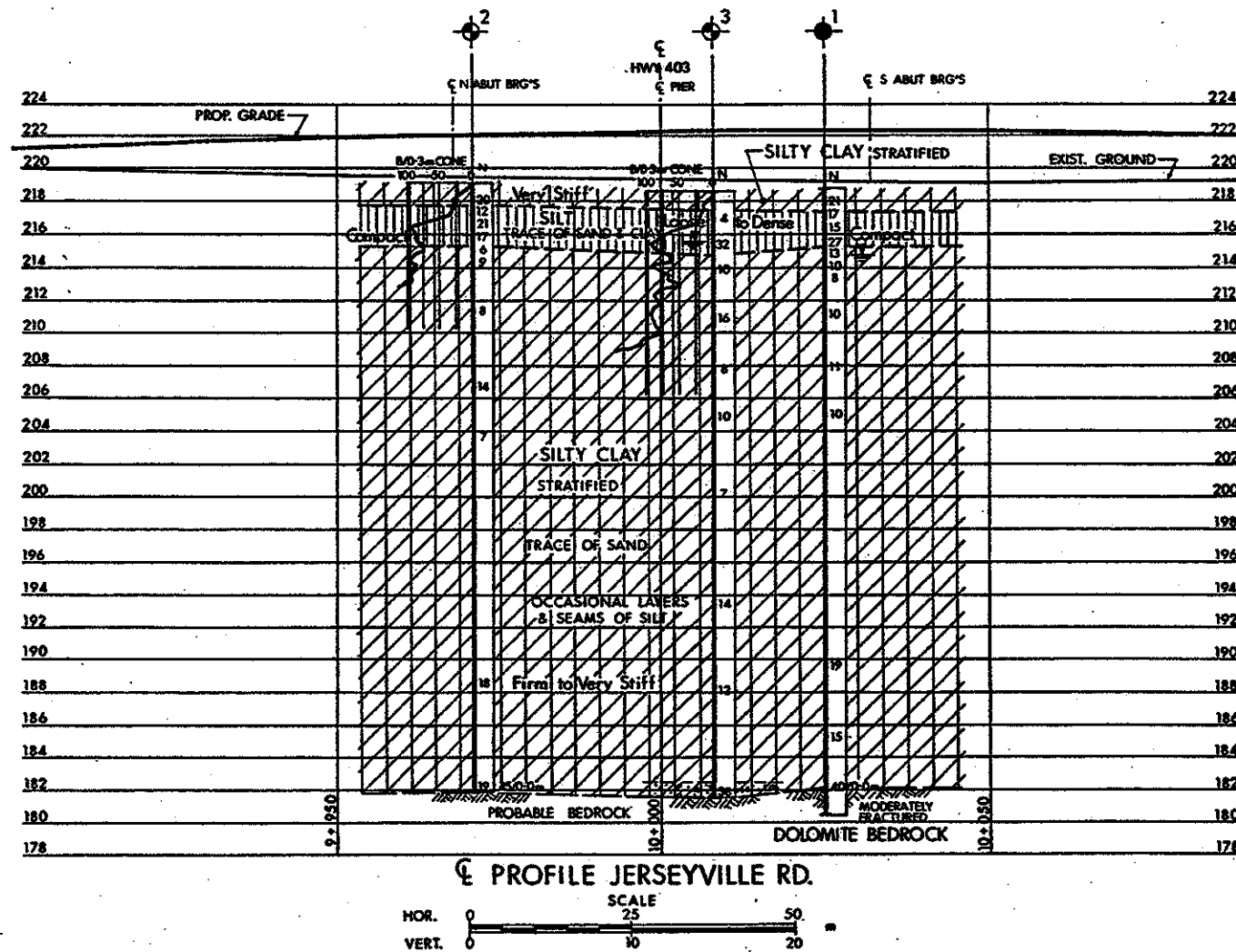
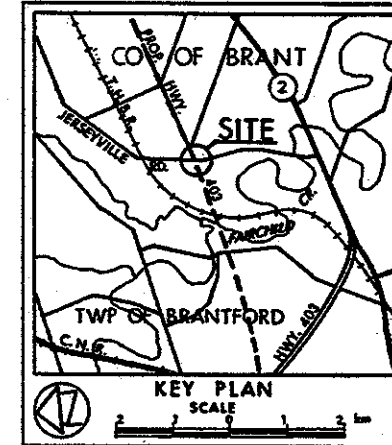
NOTE:
DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETERS + METERS

CONT No
WP No 66-67-03

HWY 403 & JERSEYVILLE RD.
UNDERPASS
BORE HOLE LOCATIONS & SOIL STRATA



SHEET
238



LEGEND

- ◆ Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W/L at time of investigation
BH No 1 79 07 26/77
BH No 3 76 03 15/76
NO W/L Established BH No 2

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	218.7	4 871 255-0	250 314-0
2	219.0	4 871 308-0	250 326-6
3	218.5	4 871 274-1	250 292-6

NOTE

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

REVISIONS		DESCRIPTION	
DATE	BY		
Geocres No 40P1-77			
HWY No 403	DATE 79 12 14	DIST 4	SITE 1-190
SUBMIT P.P. CHECKED	DATE 79 12 14	DIST 4	SITE 1-190
DRAWN/NO. 1 CHECKED	DATE 79 12 14	DIST 4	DWG 2



Ministry of
Transportation and
Communications

FILE COPY

Reference 2

FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

CONT 90-95
ENGINEERING MATERIALS OFFICE
PAVEMENT & FOUNDATION DESIGN SECTION

WP 66-67-03 DIST 4
HWY 403 STR SITE I-190

Jerseyville Rd. Interchange Underpass
13.7 km West of Hwy. 42

DISTRIBUTION

G. C. E. Burkhardt (3)
R. D. Gunter
I. V. Oliver
D. E. Thrasher (2)

C. Grebski
B. J. Giroux

R. Hore

R. Fitzgibbon)
J. Anderson) cover only
T. J. Kovich)

Files

FOUNDATION INVESTIGATION REPORT

For

Jerseyville Rd. Interchange Underpass
13.7 km West of Hwy. #2
W.P. 66-67-03, Site 1-190
Hwy. #403, District #4, Hamilton

INTRODUCTION

This report contains the results of a foundation investigation carried out for the proposed structure at the junction of the existing Jerseyville Rd. and New Hwy. #403, during the periods of 1976 03 15 - 16 and 1979 07 26 - 30. The fieldwork consisted of three sampled boreholes and two dynamic cone penetration tests. The borings were advanced by employing continuous flight auger machines, mounted either on a muskeg vehicle or on an all-terrain vehicle and equipped with 82 mm I.D. hollow stem augers.

SITE DESCRIPTION

The structure site is located at the future junction of the existing Jerseyville Rd. and proposed New Hwy. #403, in the Township of Brantford some 13.7 km west of Hwy. #2.

The surrounding terrain is relatively flat, cultivated agricultural land.

SUBSURFACE CONDITIONS

General

Generally uniform subsoil conditions were found to exist across the site. The subsoil (apart from the existing Jerseyville Rd. roadway material) consists of a deep deposit (36 - 38 m) of stratified silty clay with trace of sand followed by dolomite type bedrock.

The boundary between the overburden and bedrock, together with the obtained field and laboratory tests results are shown on

the Record of Borehole Sheets contained in the Appendix. The stratigraphical profile shown on Drawing No. 666703-A is based on this information. The drawing also shows the locations and elevations of the borings. A detailed description of the encountered subsurface conditions is given below.

Silty Clay Trace of Sand

This stratum was intersected in all borings and extends from immediately below the ground surface to the bedrock for a depth about 36 - 38 m. The material in the deposit is stratified and classified as silty clay with trace of sand. The stratification is rather random and ranges in thickness from 5 mm to about 150 mm. The plasticity of the individual layers varies from low to high. Occasional silt seams were also observed throughout the stratum. The Atterberg Limit test results for the overall deposit are plotted on the plasticity chart (Figure 1). The consistency of the stratum varies randomly from stiff to very stiff. The assessment is based on a number of field vane and laboratory unconfined and quick triaxial tests, the results of which are plotted on Figure 2 and summarized below, together with other physical properties determined from field and laboratory tests.

	<u>RANGE</u>
Natural Moisture Content (w)	13 - 46%
Liquid Limit (w_L)	22 - 63%
Plastic Limit (w_p)	13 - 21%
Undrained Shear Strength (c_u)	
Unconfined	39 - 96 kPa
Quick Triaxial	48 - 101 kPa
Field Vane	41 - over 100 kPa
Unit Weight (γ)	19.3 - 20.5 kN/m ³
Sensitivity (Based on Field Vane Tests)	2 - 5

Grain size distribution curves are presented in an envelope form on Figure 3 of the Appendix.

One consolidation test was performed on a sample obtained from this stratum. The tests indicate that the soil is overconsolidated with a preconsolidation pressure of 435 kPa.

For design purposes in terms of total stresses, an average undrained shear strength value of 60 kPa is recommended.

Silt

A loose to dense, approx. 2 to 2.4 m thick silt with traces of sand and clay zone was found to be sandwiched between the brown and grey portion of the silty clay deposit.

Bedrock

Bedrock was found at depth about 36.6 below ground (elevation 182 ±) which consists of moderately fractured, hard, light grey to white dolomite.

Groundwater Conditions

The following groundwater levels were observed the boring locations:

B.H. # 1 - Elevation 214.3

B.H. # 2 - not observed

B.H. # 3 - Elevation 214.6

It is pointed out that the subsoil is relatively impermeable, therefore a considerable time is required for the water levels to stabilize.

For design and construction purposes, it should be assumed that the groundwater level at this site is probably at elevation 215 ±. Seasonal changes may influence the groundwater levels.

DISCUSSION AND RECOMMENDATIONS

General

It is proposed to construct a two span (25.5 m - 25.5 m) structure at the junction of future New Hwy. #403 and the existing Jerseyville Rd. As per present proposals, the profile grade of Hwy. #403 will be located at elevation 216.2 (H.O.T. 14 + 196.308 @ median of Hwy. #403) with a gradient of 1.065% increase from west to east. The profile grade of Jerseyville Rd. will be raised by about 3.4 m to elevation 222.4 (H.O.T. 10+000.000 @ Jerseyville Rd.). The average original ground level is at approx. elevation 219 ±. The subsoil at this site was found to consist of a 26 - 38 m deep, firm to very stiff, stratified silty clay and followed by dolomite type bedrock.

Structure Foundation

The encountered subsurface conditions (relatively low bearing capacity value and settlement considerations) do not favour spread footing type foundations. Therefore, piled foundations are recommended. End-bearing piles driven to bedrock (elevation 182 ±) appear to be the most practical solution. The type of pile selected (steel tube, steel 'H' or reinforced concrete) should be based on economic considerations. For steel tubes, 323.9 mm O.D. @ 49.73 kg/m or HP 310 x 113 steel 'H' piles with reinforced tips driven to bedrock, the maximum permissible load is 1100 kN. For concrete piles, the maximum load will be dependent on the manufacturers specification. If steel tube piles are selected, they can be driven successfully to bedrock without damage to the piles provided that the energy of the hammer used at the instant of contact with bedrock does not exceed 40 000 joules/blow. For this reason, the energy should be restricted accordingly for the last 2 m of driving. A suitable note should be provided on the design drawings in this matter.

Approach Embankments

To accommodate the proposed profile grades of New Hwy. #403 and Jerseyville Rd. up to 3 m deep cuts and up to 4 m high fills will be required respectively. No stability problems are anticipated for the approaches (cuts and fill) of this magnitude, constructed with 2:1 forward and side slopes. The fill should consist of well compacted acceptable material. Care should be taken to ensure that no bouldery fill is placed within the approaches through which piles have to be driven, and it is recommended that this portion of the fill contain no larger grain sizes than 50 mm. Settlement of the silty clay subsoil induced by the construction of approach fills is estimated to be in the range of 75 - 100 mm and will take place over a long term period. In order to minimize the effect of these settlements on the performance of the pavement, it is recommended that the approach embankments be built in advance of the final grading and paving for as long a period as possible.

Other Considerations

The pile caps should be located not less than 1.3 m below finished ground level so as to provide for frost protection.

No major dewatering problems are anticipated due to the relatively impermeable nature of the subsoil. Topsoil and/or soft surficial material should be removed in accordance with current M.T.C. practices.

The future abutments in part, will be located over the existing Jerseyville Rd. roadway. To avoid damages to the piles during driving, it is recommended that the entire roadbed (pavement and base coarse) be excavated to its full vertical and horizontal extent.

In order to estimate the earth pressures on the abutment walls, the following values are recommended.

Unit Weight of Granular Backfill: 21.2 kN/m^3
Coefficient of Active Earth Pressure: $K_a = 0.35$
Coefficient of Earth Pressure at Rest: $K_o = 0.5$

A suitable drainage system should be provided to relieve the build-up of excess hydrostatic pressure behind the abutment walls.

To provide a smooth transition between the structure and the approaches which will undergo settling for a long period of time, it is recommended that the structure be designed with approach slabs.

The exposed cut and fill slopes should be protected against erosion according to M.T.C. standards.

MISCELLANEOUS

The fieldwork for this project was carried out by Mr. M. Kalapaca and Mr. P. R. Karpol. This report was written by Mr. P. Payer, and reviewed by Mr. K. G. Selby. The equipments used were owned by Atcost Soil Drilling Inc., and Dominion Soil Investigation Inc.

P. Payer

P. Payer, P. Eng.
Foundations Engineer.



K. G. Selby

K. G. Selby, P. Eng.
Senior Foundations Engineer.

December, 1979.

APPENDIX

MOBILE

SOIL PROFILE			SAMPLERS			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	SHEET PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100					
								SHEAR STRENGTH KPa O UNCONTAINED * FIELD VANE ● QUICK TRIAXIAL x LAB VANE 25 50 75 100 125					
216.7	Ground Level												
0.0	Silty clay Stratified												
217.3	Very stiff - brown		1	SS	23								
218.4	Silt Trace of sand and clay Compact		2	SS	17								
219.3			3	SS	15								
215.3			4	SS	27								
214.4	Silty clay Stratified Trace of sand Occasional layers and seams of silt Firm to very stiff Grey coloured		5	SS	13								
			6	SS	10								
			7	SS	8								
			8	TW	PH								
			9	SS	10								
			10	TW	PH								
			11	SS	11								
			12	TW	PH								
			13	SS	10								
			14	TW	PH								
			15	TW	PH								
			16	TW	PH								
			17	SS	19								

NETRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			f_c					
196.53													
30.2	Silty clay.	N					188						
			18	SS	LS		186						
							184						
182.13			19	SS	LO		flouring						
36.6	Bedrock moderately fractured Dolomite		20	BC	EX		182						
180.33													
38.4	End of Borehole						180						

0 0 18 22

RECORD OF BOREHOLE No 2 (0.0 - 33.2 m)



METRIC

W.P. 66-67-03 LOCATION Co-ords. N 4 871 308.0; E 250 326.0 ORIGINATED BY PER
 DIST 4 HWY 502 BOREHOLE TYPE Hollow Stem Auger COMPILED BY PER
 DATUM Geodetic DA'E 79.07.30 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE *LO*		PLASTIC LIMIT Wp	NATURAL MOISTURE CONTENT W	LIQUID LIMIT Wl	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	120					
219.4	Ground Level													
0.0	Silty clay													
217.7	Stratified		1	SS	26	Not Observed	218							0 0 88 12
	Very stiff - brown		2	SS	12									
1.7	Silt		3	SS	21		216							0 0 60 40
	Trace of sand and clay		4	SS	17									
	Coarse		5	SS	6		214							
215.3	Silty clay		6	SS	9									
	Stratified		7	TK	PH		212						19.5	0 0 45 55
	Traces of sand		8	SS	8									
	Occasional layers and seams of silt		9	TK	PH		210							
	Firm to very stiff		10	TK	PH		208							
	Grey coloured		11	SS	14		206							
			12	TK	PH		204						19.5	0 0 90 10
			13	SS	7									
			14	TK	PH		202							
							200							
							198						20.0	
							196							
			15	TK	PH		194							
							192							
							190							

RECORD OF BOREHOLE No 2 Cont. (30.2 - 37.1 m)

W P 66-67-03 LOCATION _____ ORIGINATED BY PRX
DIST 6 HWY 403 BOREHOLE TYPE Hollow Stem Auger COMPILED BY PRX
DATUM Geodetic DATE 79 07 30 CHECKED BY _____

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PARTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) OF SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	SHEAR STRENGTH	Wp	W			PL
188.4	Silty clay		16	SS	18			UNCONFINED + FIELD VANE QUICK TRIAXIAL * LAB VANE 25 75 125	WATER CONTENT (%) 15 30 45					
188														
186														
184														
30.20			17	SS	13-6	0.07	- 45	0.0	100	100				
37.1	Probable Bedrock End of Borehole						Bouncing							
							180							

RESULTS

W P 66-67-03 LOCATION Co-ords. N 4 871 274.3; E 250 292.6 ORIGINATED BY MR
DST 4 MMVY 403 BOREHOLE TYPE Hollow Stem Auger COMPILED BY MR
DATUM Geodetic DATE 76 03 15/16 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT		UNIT WEIGHT Y KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	W _p W _L W _U	WATER CONTENT (%) 15 30 45			
218.50	Ground Level												
0.0	Silty clay Stratified					218							
217.5	Very stiff (brown)					216							
1.5	Silt Trace of sand and clay Loose to dense		1	SS	4								
14.8			2	SS	32								
3.7	Silty clay Stratified					214							
	Trace of sand		3	SS	10								
			4	TX	PR								
	Occasional layers and seams of silt					212							
			5	SS	16								
	Firm to very stiff		6	TX	PR								
			7	SS	8								
	Gray coloured		8	TX	PR								
			9	SS	10								
			10	TX	PR								
						208							
						206							
						204							
						202							
						200							
						198							
						196							
						194							
						192							
						190							
188.3			14	TX	PR								

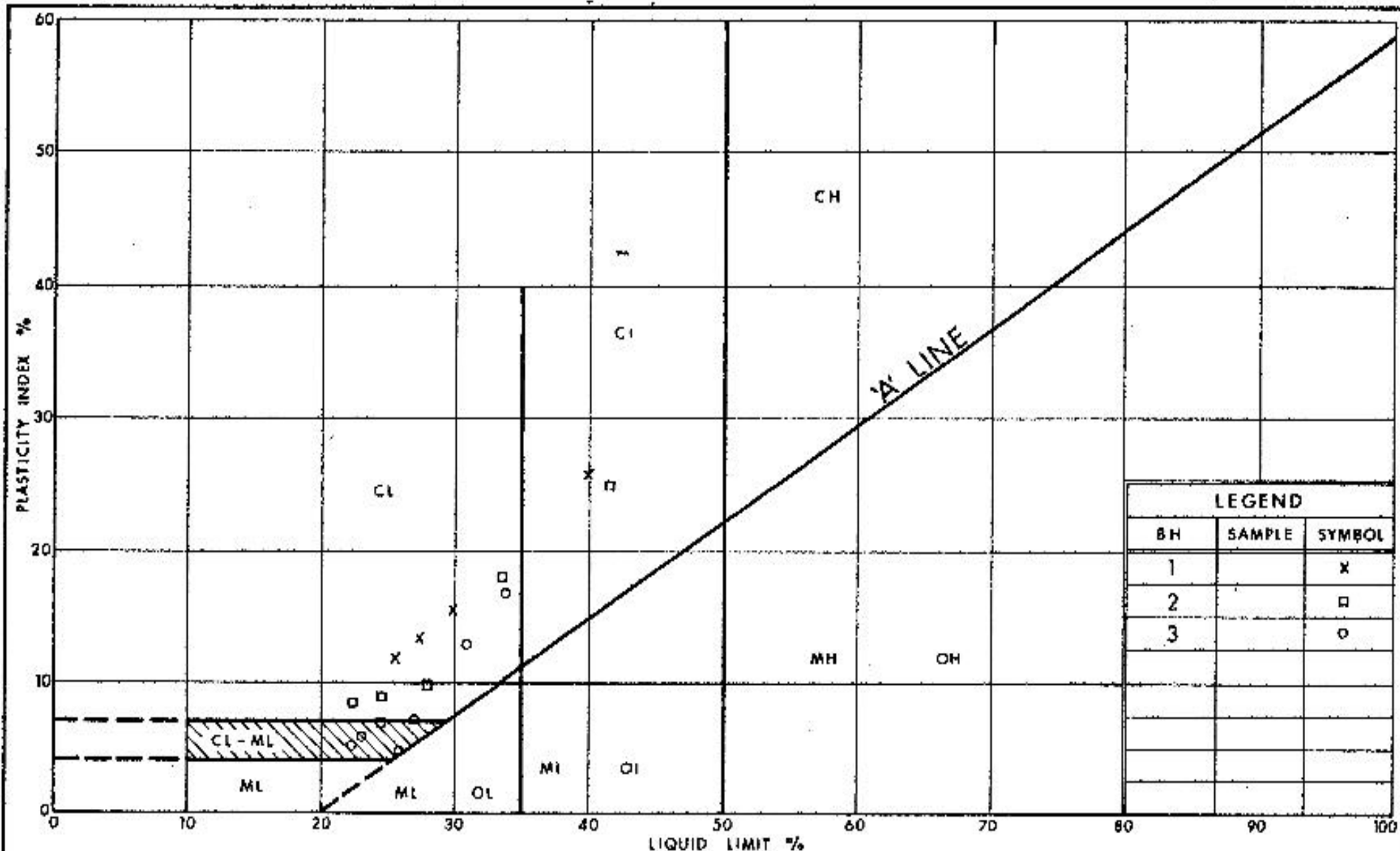
MEYRIC

W/ P 66-67-03 LOCATION _____ ORIGINATED BY NE

DIST 4 HWY 403 BOREHOLE TYPE Rollow Stem Auger COMPILED BY

DATUM Gedatyo DATE 26.03.15/16 CHECKED BY _____

[illegible]

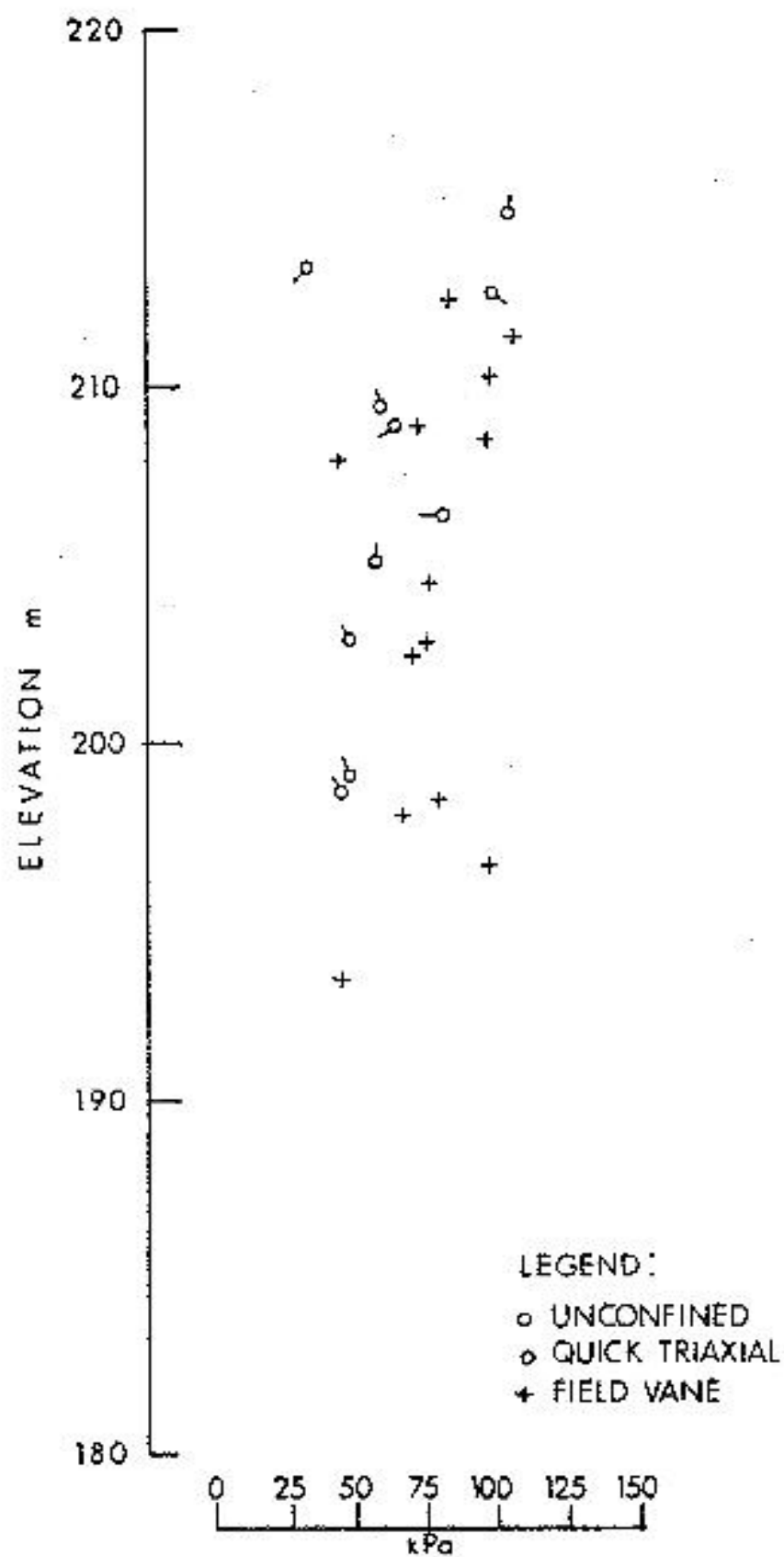
Ministry of
Transportation and
Communications

ENGINEERING SERVICES BRANCH

PLASTICITY CHART
SILTY CLAY STRATIFIED
TRACE OF SAND

FIG No 1

W P 66-67-03



SHEAR STRENGTH VS ELEVATIONS

FIG 2

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

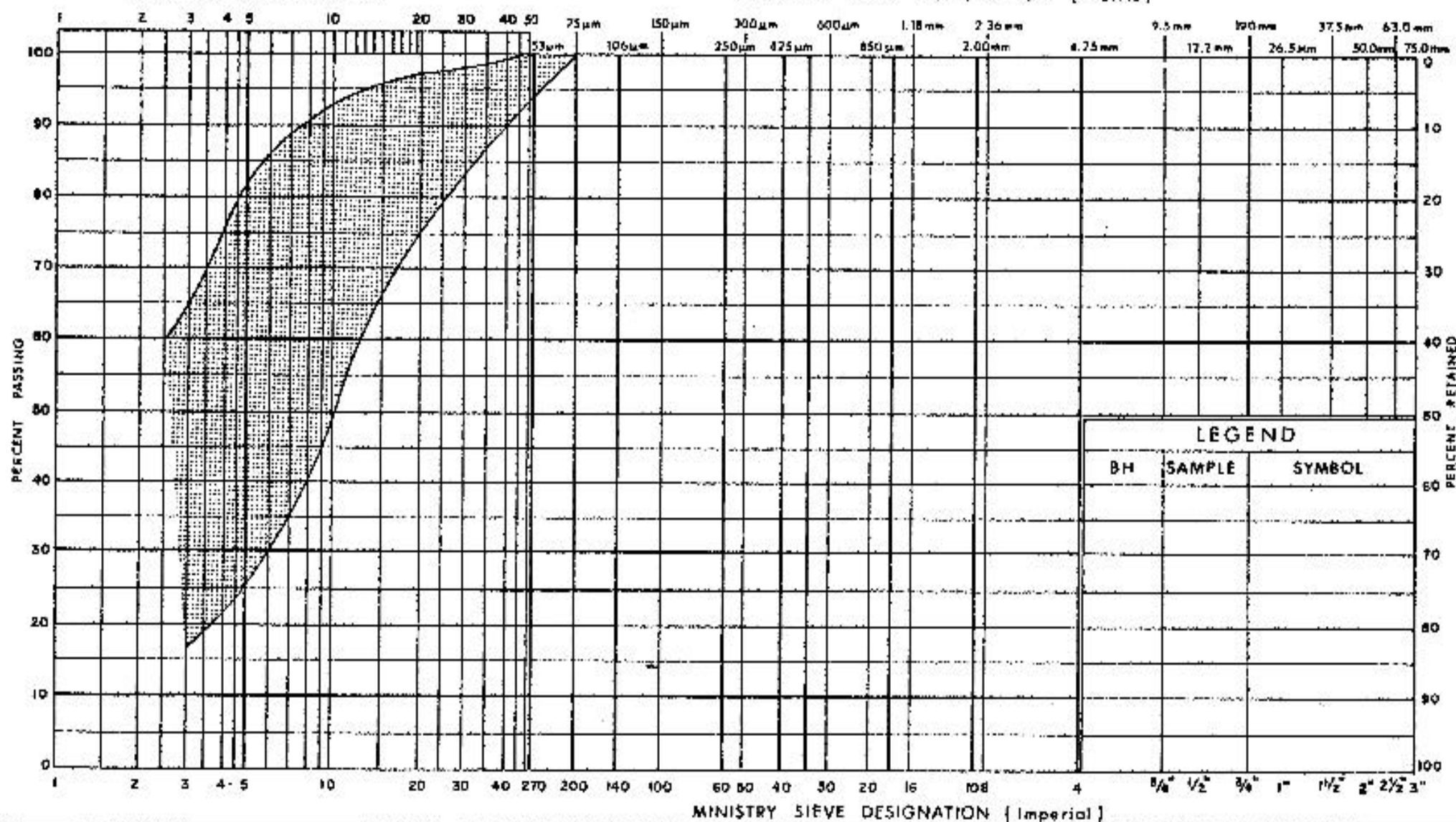
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



LEGEND

BH SAMPLE SYMBOL

GRAIN SIZE DISTRIBUTION
SILTY CLAY
STRATIFIED TRACE OF SAND

FIG No 3

W P 66-67-03



Ministry of
Transportation and
Communications

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 1" SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 20	20 - 30	> 30
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

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

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

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES 100mm* IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

SPACING	50cm	50 - 300cm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S SPLIT SPOON	T P THINWALL PISTON
WS WASH SAMPLE	OS OSTERBERG SAMPLE
ST SLOTTED TUBE SAMPLE	RC ROCK CORE
BS BLOCK SAMPLE	PH T.W. ADVANCED HYDRAULICALLY
CS CHISEL SAMPLE	PM T.W. ADVANCED MANUALLY
TW THINWALL OPEN	FS FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

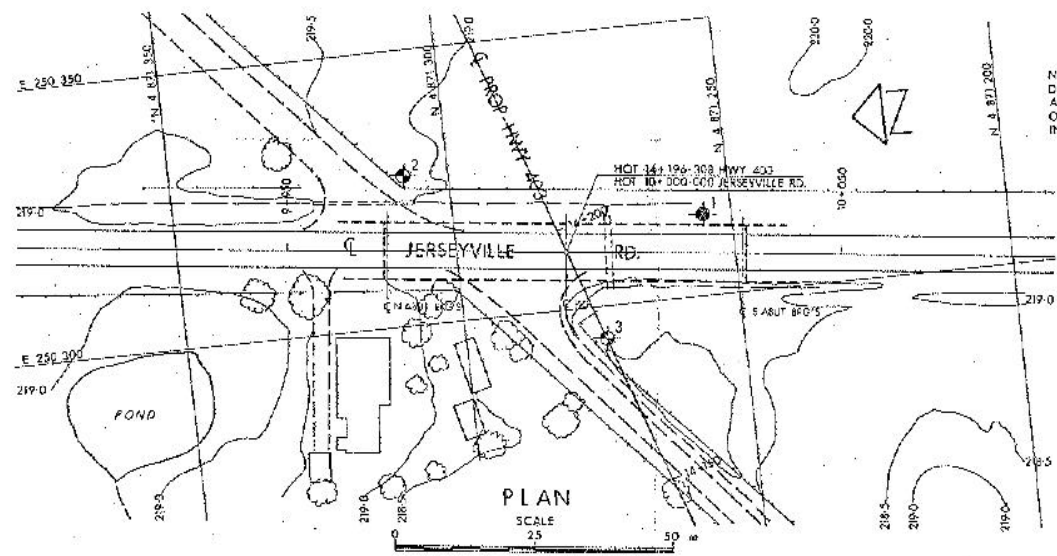
MECHANICAL PROPERTIES OF SOIL

m_v	kPa	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
N	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_r	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
T_R	kPa	REMOULDED SHEAR STRENGTH
S_r	1	SENSITIVITY = $\frac{c_u}{T_R}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - D. AMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ² /s	RATE OF DISCHARGE
γ_d	kn/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL				j	kn/m ²	SEEPAGE FORCE

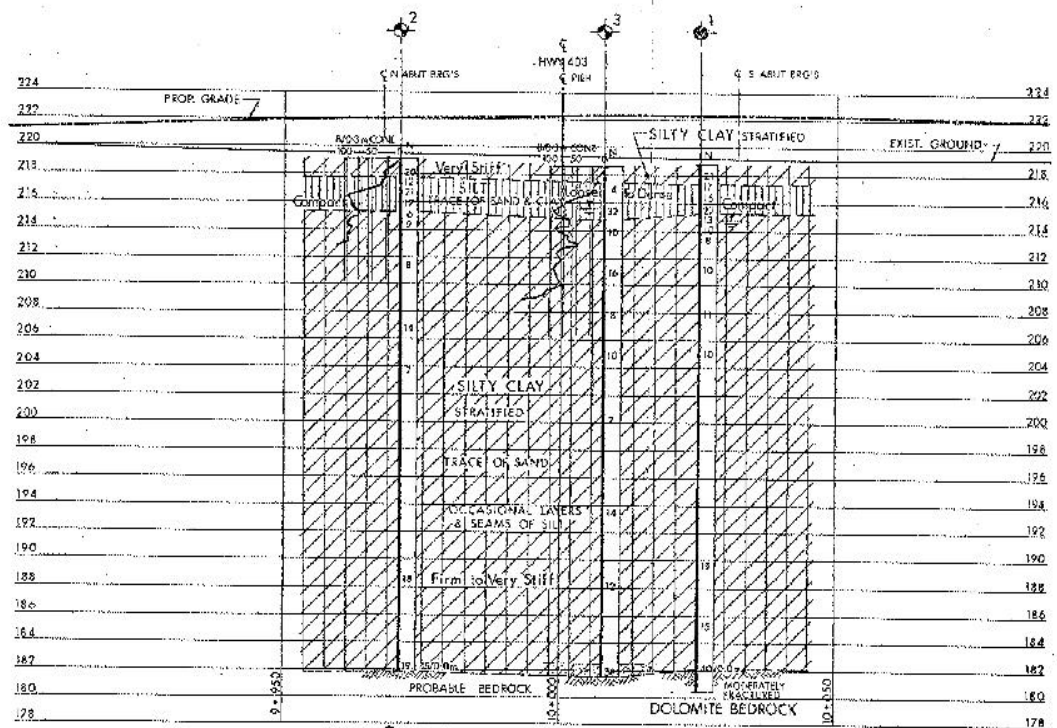
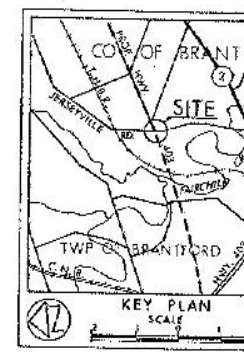
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, OTTAWA, ONTARIO, G9-1M7-165M 4-78



METRIC

NOTE:
DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETERS + METERS

CONT No
WP No 66-67-03
HWY 403 & JERSEYVILLE RD.
UNDERPASS
BORE HOLE LOCATIONS & SOIL STRATA



PROFILE JERSEYVILLE RD.



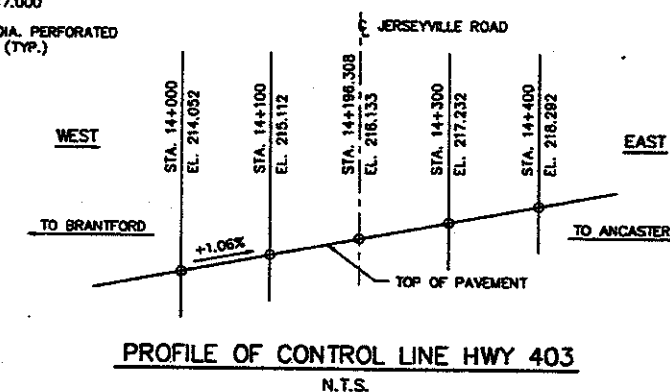
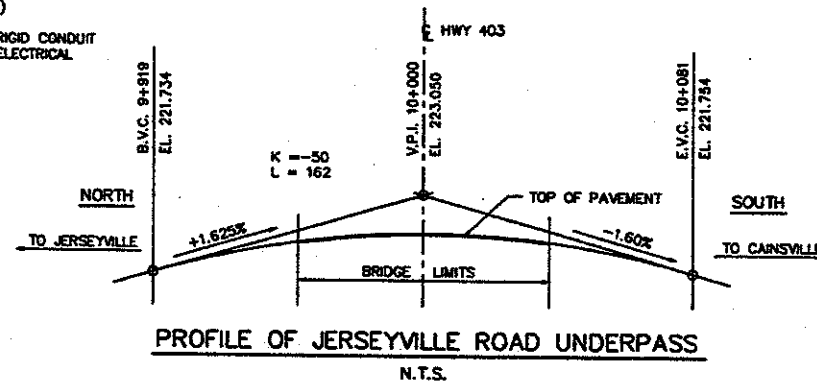
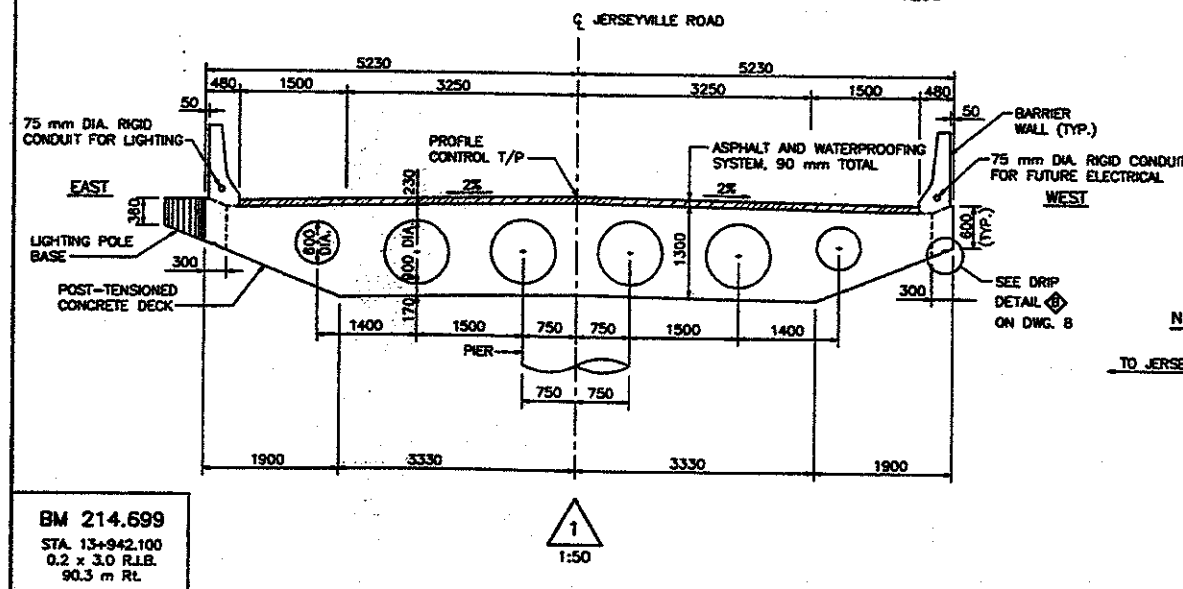
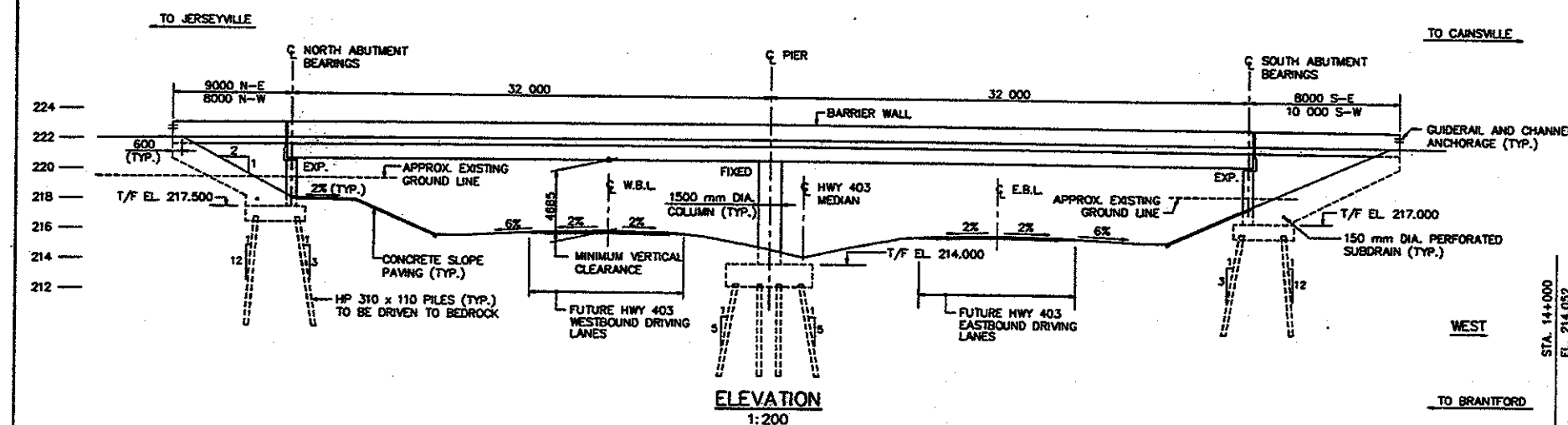
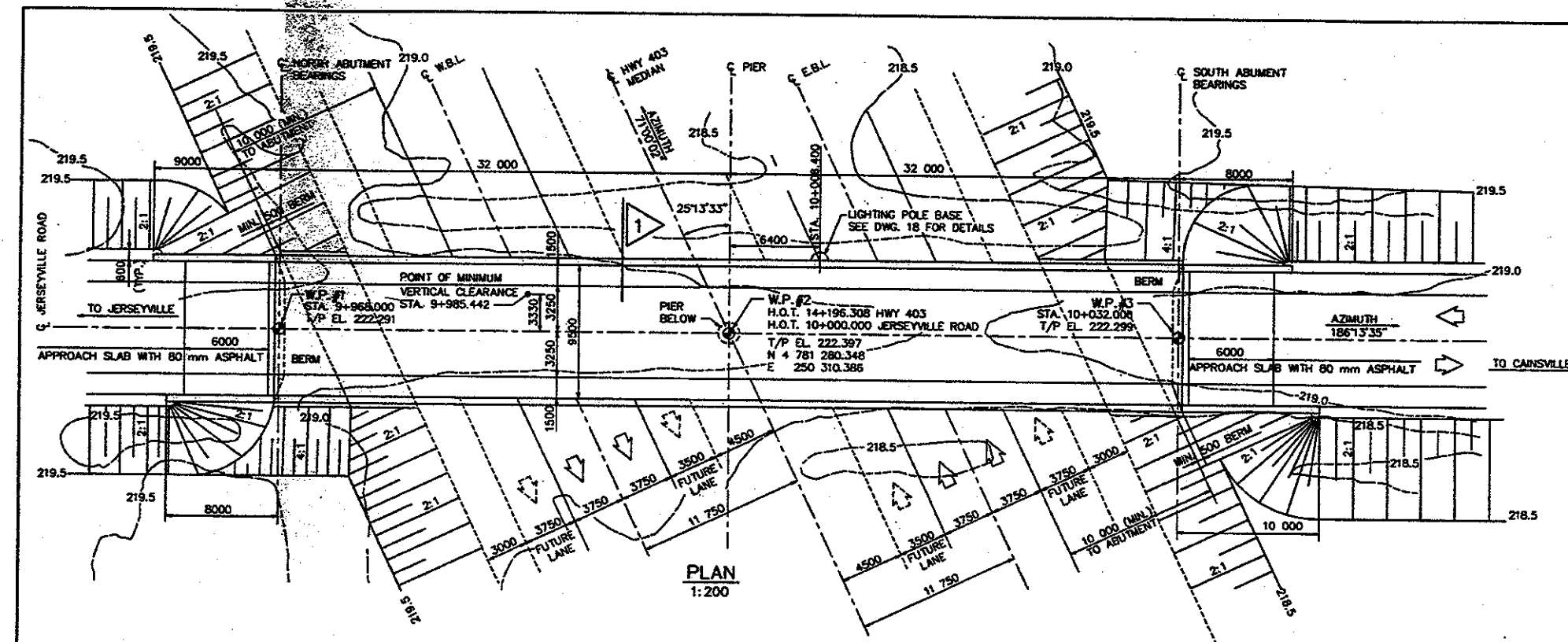
LEGEND			
	Bore Hole		
	Dynamic Cone Penetration Test		
	Bore Hole & Cone		
N	Blows/0.3m (Std Pen Test, 475)		
CONE	Blows/0.3m (60° Cone, 475)		
	Wt at time of investigation		
	BH No 1 79 07 26/27		
	BH No 3 76 03 15/26		
	NO Wt. Entered BH No 2		

No	ELEVATION	COORDINATE	
		NORTH	EAST
1	218.7	4 871 255.0	250
2	219.0	4 871 303.0	250
3	218.5	4 871 274.1	250

NOTE
The boundaries between soil strata have been
only at Bore Hole locations. Between Bore
boundaries are assumed from geological

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No 40P1-77	
HWY No 403	613
SUBMIT P P CHECKED	DATE 79 12 13
DRAWN L CHECKED	DATE 79 12 13



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST. No. 4
CONT. No. 90-95
WP. No. 66-67-03



JERSEYVILLE ROAD UNDERPASS AT HWY 403	SHEET
GENERAL ARRANGEMENT	237

Morrison Hershfield Limited
Consulting Engineers

GENERAL NOTES

- | | |
|--|--------|
| 1. CLASS OF CONCRETE | |
| DECK | 35 MPa |
| REMAINDER (UNLESS NOTED) | 30 MPa |
| 2. REINFORCING STEEL | |
| REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. | |
| BAR MARKS WITH SUFFIX 'C' DENOTE COATED BARS. | |
| 3. CLEAR COVER TO REINFORCING STEEL | |
| FOOTINGS | 100±25 |
| ABUTMENTS AND WINGWALLS | |
| FRONT FACE | 80±20 |
| BACK FACE | 70±20 |
| PIERS | 80±20 |
| DECK SLAB | |
| TOP | 70±20 |
| BOTTOM AND SIDES | 50±10 |
| REMAINDER (UNLESS NOTED) | 70±20 |
| 4. CONSTRUCTION NOTE | |
| IF THE ACTUAL BEARING HEIGHTS ARE DIFFERENT FROM THE ASSUMED HEIGHTS GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE BEARING SEAT ELEVATIONS AND THE REINFORCING STEEL TO SUIT THE ACTUAL HEIGHTS. | |

LIST OF DRAWINGS

- 01-190-1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATION AND SOIL STRATA
3. FOUNDATION LAYOUT AND REINFORCEMENT
4. NORTH ABUTMENT
5. SOUTH ABUTMENT
6. WINDSWALL LAYOUT AND REINFORCEMENT
7. PIER AND BEARING DETAILS
8. DECK LAYOUT AND SCREED ELEVATIONS
9. LONGITUDINAL CABLE LAYOUT
10. TRANSVERSE CABLE LAYOUT
11. DECK REINFORCEMENT
12. PRESTRESSING AND REINFORCING DETAILS - I
13. PRESTRESSING AND REINFORCING DETAILS - II
14. JOINT ANCHORAGE AND ARMOURING
15. BARRIER WALL
16. 6000 mm APPROACH SLAB
17. DETAILS OF CONCRETE SLOPE PAVING
18. STANDARD DETAILS
19. AS CONSTRUCTED ELEVATIONS AND DIMENSIONS
20. ELECTRICAL EMBEDDED WORK
21. QUANTITIES SHEET - STRUCTURAL I
22. QUANTITIES SHEET - STRUCTURAL II

LEGEND:

- | | |
|------|-----------------|
| T/F | TOP OF FOOTING |
| T/C | TOP OF CONCRETE |
| T/P | TOP OF PAVEMENT |
| W.P. | WORKING POINT |
| C | CENTRE LINE |

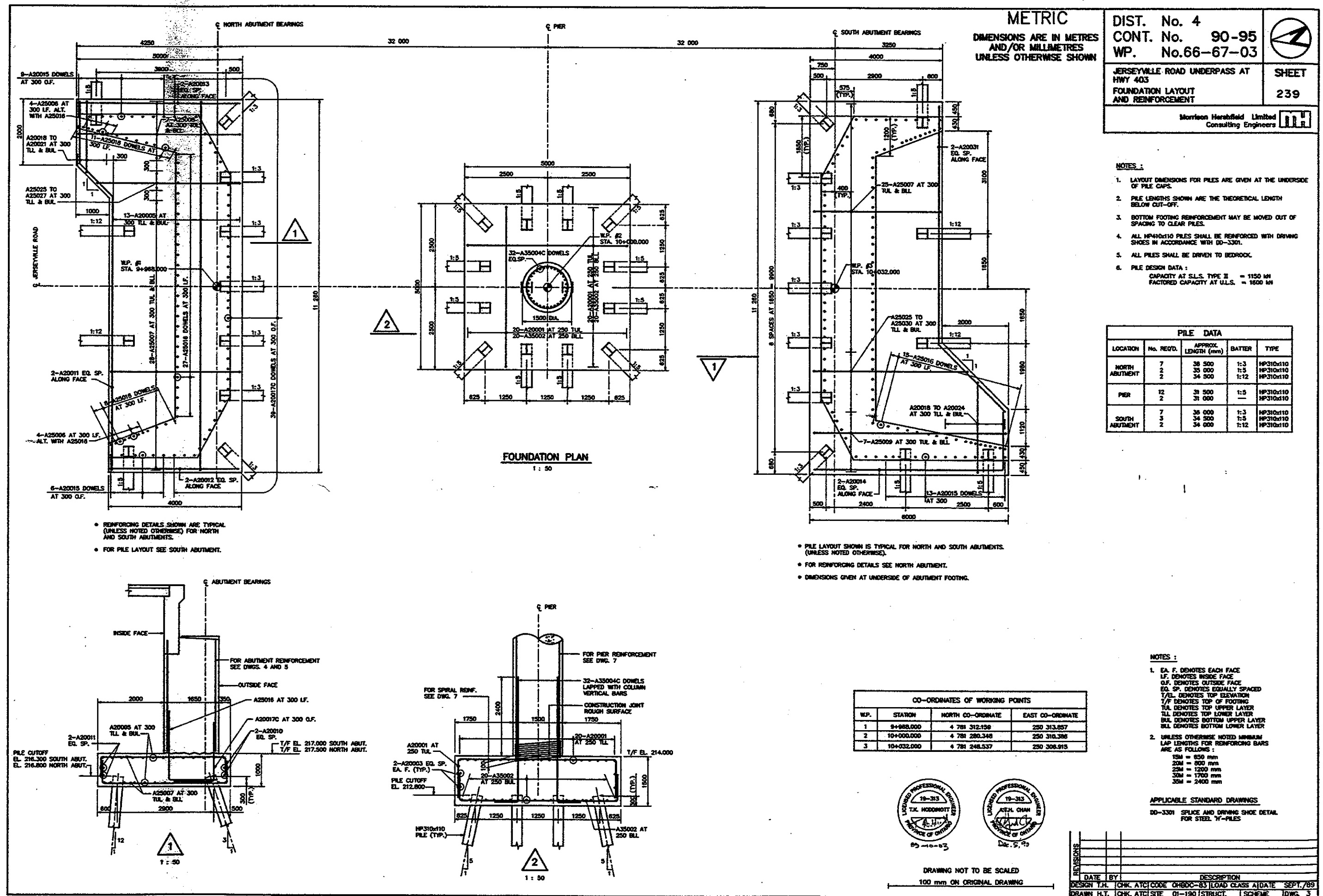
APPLICABLE STANDARD DRAWINGS

DD-3502 MINIMUM GRANULAR BACKFILL REQUIREMENTS

DRAWING NOT TO BE SCALED

100 mm ON ORIGINAL DRAWING

REVISIONS								
DATE		BY		DESCRIPTION				
DESIGN T.H.	CHK. ATC	CODE	0HSDC-83	LOAD	CLASS	A	DATE	SEPT./89
DRAWN T.H.	CHK. ATC	SITE	01-190	STRUCT.	SCHEME		DWG.	1





APPENDIX B

Site Photographs



Photograph 1: Looking at the south abutment of the Jerseyville Road Underpass Structure. Vertical cracks were observed on the abutment wall. Weep holes were observed in the abutment wall. Sealant between the concrete panels appeared degraded at locations where grasses and shrubs are growing. Sloughing of ground underneath the concrete panels was observed. Cracks, breaks and dislodge of the concrete panels were also observed. The toe of the front slope was covered with vegetation (August 28, 2015).



Photograph 2: Looking at the concrete panels on the front slope of the south abutment. Breaks and dislodge of concrete panels observed (August 28, 2015).



Photograph 3: Looking at the gap between the abutment wall and concrete panel (August 28, 2015).



Photograph 4: Looking at the north abutment of the Jerseyville Road Underpass Structure. Vertical cracks were observed on the abutment wall. Weep holes were observed in the abutment wall. Sealant between the concrete panels appeared degraded at locations where grasses and shrubs are growing. Sloughing of ground underneath the concrete panels was observed. Cracks, breaks and dislodge of the concrete panels were also observed. The toe of the front slope was covered with vegetation (August 28, 2015).



Photograph 5: Looking at the concrete panels on the front slope of the north abutment. Cracks and dislodge of concrete panels observed (August 28, 2015).



Photograph 6: Looking at the concrete panels on the front slope of the north abutment. Cracks on concrete panels observed (August 28, 2015).



Photograph 7: Looking north at the pier of Jerseyville Road Underpass Structure. Surficial cracks were observed on the pier (August 28, 2015).



Photograph 8: Looking at the east wingwall and the adjacent slope of the south abutment of the Jerseyville Road Underpass Structure. Surficial cracks were observed on the wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (August 28, 2015).



Photograph 9: Looking at the west wingwall and the adjacent slope of the south abutment of the Jerseyville Road Underpass Structure. Surficial cracks were observed on the wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (August 28, 2015).



Photograph 10: Looking at the west wingwall and the adjacent slope of the north abutment of the Jerseyville Road Underpass Structure. Surficial cracks were observed on the wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (August 28, 2015).



Photograph 11: Looking at the east wingwall and the adjacent slope of the north abutment of the Jerseyville Road Underpass Structure. Minor cracks were observed on the wingwall. The slope is vegetated and effect of erosion on the slope face was not observed (August 28, 2015).