FOUNDATION INVESTIGATION AND DESIGN REPORT
MUNICIPAL SERVICE ROAD OVER HIGHWAY 11
HIGHWAY 11, HIGHWAY 518 WEST TO HIGHWAY 520
G.W.P. 480-93-00, W.P. 5405-04-01, SITE 44-427

Geocres Number: 31E-246

Report to

Marshall Macklin Monaghan

Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

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TABLE OF CONTENTS

PART 1: FACTUAL INFORMATION

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

Appendices

FOUNDATION INVESTIGATION AND DESIGN REPORT
MUNICIPAL SERVICE ROAD OVER HIGHWAY 11
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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed underpass structure to carry the Municipal Service Road over the realigned Highway 11 at the village of Katrine, Ontario. A previous, preliminary investigation had been carried out at the location of the proposed structure by Shaheen & Peaker Limited (S&P) and the factual data from that investigation has been used in the current assignment.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, borehole logs, stratigraphic profile and cross-sections and a written description of the subsurface conditions. A model of the subsurface conditions was developed using the data obtained during the previous S&P investigation, which was considered sufficient for the current study. This model describes the geotechnical conditions influencing design and construction of the foundations and approach embankments for the structure.

Thurber prepared the investigation report as a sub-consultant to Marshall Macklin Monaghan, under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-A-000285.

2 SITE DESCRIPTION

The site lies immediately west of the existing Highway 11 alignment approximately 800 m north of Three Mile Lake Road at the Village of Katrine, Armour Township. The structure will carry a new section of the Municipal Service Road (current Highway 11) over the new alignment of Highway 11.

The general site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges where exposed. The immediate bridge location, however, lies between bedrock ridges within an area underlain by relatively deep deposits of glacio-fluvial and glacio-lacustrine soils. The site is situated about 150 m west of the Magnetawan River at an elevation some 10 to 15 m above that of the river flood plain.

The site lies within an open meadow area bordered by heavily forested lands. The ground surface generally falls to the east. Several residential dwellings are present east of the site, on the east side of existing Highway 11.

3 SITE INVESTIGATION AND FIELD TESTING

The borehole data obtained during the preliminary foundation investigation was considered adequate for the current study and therefore no additional site investigation or field testing was carried out. Shaheen & Peaker Limited carried out the preliminary investigation between April 6 and 17, 2001.

The preliminary site investigation consisted of drilling and sampling three boreholes to refusal on bedrock at depths of 17.3 to 22.4 m at the abutment and pier locations, and two boreholes to depths of 6.6 (refusal) and 9.6 m at the approaches. Bedrock at the abutment and pier locations was proved by coring 2.9 to 3.9 m below refusal, to total borehole depths of 21.2 to 25.3 m.

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix D.

The positions of the principal boreholes considered in the preparation of this report, relative to the structure site are as shown in Table 3.1.

Table 3.1 – Borehole Locations Relative to Structure

Location on Structure	Boreholes Considered in Design
West Approach	BPR1
West Abutment	BPR2
Pier	BPR3
East Abutment	BPR4
East Approach	BPR5

The coordinates and elevations of the boreholes are given on the Borehole Locations and Soil Strata Drawing and on the individual Record of Borehole Sheets in Appendix A.

A standpipe piezometer was installed in borehole BPR3.

4 LABORATORY TESTING

The results of natural moisture content determinations, gradation analyses (sieve and hydrometer), and Atterberg Limits tests conducted during the preliminary investigation are shown on the Record of Borehole sheets in Appendix A. The results of the gradation analyses and Atterberg Limits tests are also plotted on the charts included after the borehole logs in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil stratigraphy are presented in this appendix and on the attached Borehole Locations and Soil Strata Drawing. An overall description of the stratigraphy is given in the following paragraphs however the factual data presented in the borehole logs governs any interpretation of the site conditions.

The soil stratigraphy encountered at this site is consistent with that encountered in much of the Highway 11 corridor between Huntsville and North Bay. The bedrock is mantled by sand and gravel containing cobbles and boulders which is overlain by glacial outwash soils deposited in glacio-fluvial and glacio-lacustrine environments.

In general terms, the site was found to be underlain by a thin veneer of topsoil; silt; silty sand; gravelly sand with cobbles and boulders; and bedrock. A clay layer was encountered below the topsoil at one location.

More detailed descriptions of the individual strata are presented below.

5.2 Topsoil

Topsoil was identified surficially in all boreholes drilled at the site. The topsoil thicknesses were established only at the borehole locations and ranged from 75 to 250 mm. The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.3 Silty Clay

A layered silty clay unit was encountered in the area of the east approach only. This unit was interbedded with silt layers, eventually grading to silt at 4.3 m depth (elevation 305.1). Standard penetration test N values obtained in the clay ranged from 5 to 15 blows per 300 m penetration, indicating a firm to stiff consistency. An N-value of 2 obtained near 4 m depth is believed to be low due to disturbance of the saturated silt layers.

The results of grain size analyses and Atterberg Limits testing on two samples of the silty clay are presented in Appendix A. The plasticity chart plot indicates that the clay varies from medium plastic (CI) to low plastic (CL-ML) with depth.

5.4 Silt

A silt deposit was encountered below the clay at the east approach and below the topsoil in the remaining boreholes. The silt is non-plastic and exhibits a layered structure, with occasional thin clay layers and laminations. In the boreholes at the east abutment and approach, cobbles were encountered in the silt near elevation 300. The lower boundary of the silt deposit generally falls towards the east, ranging from elevation 307.2 (5.0 m depth) at the west abutment to elevation 293.9 (13.2 m depth) at the east abutment. Borehole BPR5 at the east approach was terminated in the silt at elevation 299.8 (9.6 m depth).

STP N-values obtained in the silt varied widely from 2 to 29 blows per 300 mm penetration. Typically, the N-values range from about 6 to 16 blows/300 mm (loose to compact), with the values below and above this range possibly attributed to the presence of groundwater and cobbles, respectively. Moisture contents range from 18 to 29%.

Grain size distribution curves for the silt, prepared during the preliminary investigation, are included in Appendix A.

5.5 Silty Sand

The silt is underlain by a stratum of silty sand with some gravel. The thickness of this layer ranged from 2.7 to 3.0 m at the pier and east abutment, to 8.2 m at the west abutment. The lower boundary of the silty sand falls towards the east from elevation 299.0 (13.2 m depth) at the west abutment to elevation 290.9 at the east abutment.

Borehole BPR1 drilled at the west approach was terminated on a probable boulder in the silty sand at 6.6 m depth (elevation 305.9). Boulders were also encountered in this deposit in borehole BPR2, requiring rock coring procedures (diamond drilling) to penetrate.

SPT values measured in the silty sand at the pier and east abutment ranged from 22 to 38 blows per 0.3 m of penetration, indicating a compact to dense condition. At the west abutment, SPT N-values ranged from 44 to 95 where the sampler was driven the full 0.3 m, corresponding to a dense to very dense condition. On two occasions in borehole BPR2, the sampler contacted boulders and could not be driven the full 0.3 m.

Grain size distributions for the silty sand are reported on the Record of Borehole sheets and are plotted in Figure 4 in Appendix A. Moisture contents of about 4 and 11% were determined for two samples.

5.6 Gravelly Sand with Silt, Cobbles and Boulders

Very dense gravelly sand with numerous cobbles and boulders was encountered below the silty sand layer in the three deep boreholes. The upper boundary of this deposit drops towards the east, from elevation 299.0 (13.2 m depth) at the west abutment to elevation 290.9 (16.2 m depth) at the east abutment. The lower boundary rests on bedrock, at 17.3 to 22.4 m depth (elevation 294.9 to 285.7), falling from the west abutment to the pier location.

With only one exception, SPT testing obtained 50 blows for penetrations of less than 150 mm due to the very dense condition of the soil and the numerous cobbles and boulders. One SPT value of 73 blows per 300 mm of penetration was recorded. Rock coring methods were necessary to penetrate occasional bouldery zones.

The results of one grain size distribution analysis conducted on the gravelly sand matrix are presented in Appendix A. Moisture contents ranged from 8 to 18%.

5.7 Bedrock

The soils described above were found to be underlain by bedrock of the Pre-Cambrian Canadian Shield. The bedrock was proved by coring 2.9 to 3.9 m below the bedrock surface.

The rock is described as grey gneiss that is unweathered to moderately weathered, typically slightly weathered. The reported core recovery was 73 to 100%. RQD values ranged from 60 to 100%, indicating a fair to excellent quality rock.

5.8 Depths to Refusal

Effective refusal, defined as an SPT value exceeding 100 blows for 0.3 m of penetration (or 50 blows for less than 150 mm penetration), was encountered in the gravelly sand, cobbles and boulders above the bedrock surface in the three deep boreholes. The depths at which effective refusal was encountered, and the depth to the bedrock surface, are shown in Table 5.1.

Table 5.1 – Refusal and Bedrock Depths

Location	Borehole	Refusal		Bedrock	
		Depth (m)	Elevation	Depth (m)	Elevation
West Abutment	BPR2	13.7	298.5	17.3	294.9
Pier	BPR3	12.1	296.5	22.4	286.2
East Abutment	BPR4	16.6	290.5	21.4	285.7

5.9 Water Levels

The initial and final groundwater depths and elevations are shown in Table 5.2.

Table 5.2 – Groundwater Depths and Elevations

Location	Borehole	Date	Water Level (m)		Comment
			Depth	Elevation	
West Approach	BPR1	April 6, 2001	5.9	306.6	Not stabilized
West Abutment	BPR2	April 12, 2001	11.2	301.0	Not stabilized
Pier	BPR3	April 10, 2001	8.3	300.3	Not stabilized
		April 11, 2001	6.5	302.1	In piezometer, not stabilized
East Abutment	BPR4	April 17, 2001	1.5	305.6	Based on sample moisture condition
East Approach	BPR5	April 6, 2001	3.0	306.4	During drilling

The above values are short-term readings, not stabilized, and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

The geotechnical data used in the preparation of this report was obtained from the Preliminary Foundation Investigation Report, Proposed Highway 11, Municipal Service Road Underpass, Katrine, Ontario, W.P. 314-99-00, prepared by Shaheen & Peaker Limited, dated November 7, 2001.

Interpretation of the field data obtained during the preliminary investigation and preparation of the current investigation report were completed by Mr. Murray Anderson, P.Eng. Review of the report was performed by Mr. Alastair E. Gorman, P.Eng. The report was also reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.
Murray R. Anderson, P.Eng., M.Eng.
Senior Geotechnical Engineer

Alastair E. Gorman, P.Eng., M.Sc.
Senior Foundations Engineer

P.K. Chatterji, P.Eng., Ph.D.
Review Principal.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach fills for the proposed structure.

A two-span, 105.8 m long, post-tensioned trapezoidal voided deck structure is proposed at this site and integral abutments are under consideration. The new Highway 11 at the underpass location will be constructed in a cut extending up to 7.5 m below the existing ground surface. The undersides of the abutment stems will lie approximately 4 to 5 m below existing grade, and the base of the centre pier column will lie about 8 m below existing grade.

At the east abutment, the finished grade will be about Elevation 309.1. The original ground slopes down to the southeast from about Elevation 310 to Elevation 308, resulting in minor cut and fill of approximately 1 m.

At the west abutment, the finished grade will be about Elevation 313.1. The original ground slopes down to the east from about Elevation 310.5 to Elevation 309.0, resulting in an approach fill approximately 2.5 to 4.0 m high.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the preliminary investigation.

8 STRUCTURE FOUNDATIONS

Foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation scheme is recommended.

Based on the results of the exploratory boreholes drilled at the proposed abutment and pier locations, the stratigraphy consists of approximately 5 to 13 m of generally loose to compact silt overlying a compact to very dense silty sand layer, underlain by dense to very dense gravelly sand with cobbles and boulders. The gravelly sand was contacted at about 12 to 16 m depth and lies on bedrock at depths of approximately 17 to 22 m.

Initial consideration was given to the following foundation types:

- Spread footings on native soil
- Spread footings on engineered fill
- Driven steel H-piles
- Caissons (drilled shaft piles)

Appendix B contains a table presenting a comparison of the technical advantages and disadvantages of the different foundation schemes at this site.

8.1 Spread Footings

8.1.1 Footings on Native Soil

The upper deposit of silt is considered too loose to provide adequate support for spread footings due to the low bearing resistance available and the potential for comparatively large settlements. It is noted, however, that Highway 11 will be constructed in a cut at this location, and as a result, founding levels for the west abutment and pier are expected to extend below the silt unit and into the compact to very dense underlying sands.

The competent silty sand and gravelly sand deposits contain frequent cobbles and boulders. In view of the potential for disturbance of the founding surface when removing bouldery material, the difficulty in providing a clean level founding surface in these conditions, as well as the potential subgrade variability due to boulders below the founding level, it is recommended that a pad of engineered fill be placed over the sand deposits prior to construction of footings. Further discussion is provided in the next section.

At the east abutment, the silt extends to greater depth. Construction of spread footings would require substantial subexcavation of the silt and replacement with engineered fill. Considering the depth to the sand deposit, use of spread footings is not considered feasible at the east abutment.

8.1.2 Footings on Engineered Fill

Construction of spread footings on engineered fill placed over the dense to very dense silty sand and gravelly sand (west abutment and pier) and the compact silt (east abutment) may be considered. It is recommended that the pad of engineered fill be at least 2.0 m thick to provide a good quality founding surface.

The underside of the engineered fill pad should extend down to or below the elevations given in Table 8.1.

Table 8.1 – Maximum Elevation of Underside of Engineered Fill

Location	Borehole Number	Maximum Elevation of Underside of Engineered Fill Pad
West Abutment	BPR2	306.0
Pier	BPR3	298.0
East Abutment	BPR4	303.0

It must be noted that the sub-excavation depth at the east abutment extends approximately 2.6 m below the groundwater level observed in the borehole. Groundwater control measures will be required to enable excavation and compaction of the fill. Considering the unwatering requirements, supporting the east abutment on spread footings is not a favoured option.

Supporting the pier on a spread footing/engineered fill is considered feasible with the abutments supported on a different type of foundation, possibly driven H-piles. Excavation for fill pad construction would extend about 4 m below finished grade. A groundwater control scheme will be needed to prevent disturbance of the subgrade and to allow engineered fill to be placed in dry conditions.

The engineered fill must consist of OPSS Granular "A" placed in 150 mm lifts and compacted to 100% of its SPMDD at ±2% of optimum moisture content and generally conforming to the geometry illustrated in Figure 1.

Provided a minimum footing width of 2 m is maintained, a footing bearing on the engineered fill may be designed for a concentric, vertical geotechnical resistance of 900 kPa at factored ULS and a resistance of 350 kPa at SLS.

These resistance values are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is not expected to exceed 25 mm. Differential settlements are not expected to exceed 20 mm across the width of the structure.

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.7. This is an "ultimate" value and requires a degree of sliding movement to occur to fully mobilize the resistance.

8.2 Driven Steel Piles

The geotechnical conditions encountered at this site are considered suitable for driven steel H-pile foundations.

The piles are expected to encounter effective refusal in the layer of sand containing cobbles and boulders lying immediate above the bedrock. In some cases, a pile may penetrate this layer without being obstructed by boulders and will meet refusal on the bedrock.

The piles should be designed on the basis of the axial geotechnical resistances given in Table 8.2.

Table 8.2 – Pile Geotechnical Resistance

Pile Section	Piles Driven Into Sand with Cobbles and Boulders				
	ULS (Factored)	SLS (25 mm Settlement)	Estimated Pile Tip Elevation		
			West Abutment	Pier	East Abutment
HP 310 X 110	1,800 kN	1,600 kN	298.5	296.5	290.5
HP 360 X 132	2,100 kN	1,800 kN	298.5	296.5	290.5

The pile tip elevations shown in Table 8.2 should be used for cost estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.2.3 Pile Driving.

8.2.1 Pile Tips

Due to the presence of cobbles and boulders in the expected founding layer, the tips of all piles should be fitted with H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point), Pruyn Points, or approved equivalent.

The use of rock points is recommended for the following reasons:

- The piles will be driven into soil containing cobbles and boulders, which requires a higher level of protection than driving into soils containing only smaller particle sizes.
- Some piles may achieve refusal on large boulders, which will require the same pile tip protection and reinforcement as founding on bedrock.
- Some piles may fully penetrate the zone of cobbles and boulders and achieve refusal on the bedrock.
- In the case of partial bearing on bedrock, the cast steel point will provide better stress distribution without failure than would be achieved in a pile tip reinforced with a driving shoe.

8.2.2 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

The Contract Documents should contain a NSSP alerting the bidders to:

- The presence of cobbles and boulders in the sands just above bedrock.
- The possibility of piles within a group achieving the specified resistance at different elevations.
- The possibility of some piles meeting refusal on a large boulder.

- The possibility that some piles may fully penetrate the zone of cobbles and boulders and achieve refusal on the bedrock.

The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving. Suggested wording for the NSSP is included in Appendix C.

To facilitate pile installation, embankment fill through which piles will be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

8.2.3 Pile Driving

Pile driving within 2 m of the estimated pile tip elevation (Table 8.2) should be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile". "R" must have the minimum values shown in Table 8.3.

Table 8.3 – Ultimate Geotechnical Resistance of Piles

Pile	Ultimate Resistance, R (kN)
HP 310X110	3,600 kN
HP360X132	4,200 kN

The Contractor should be alerted to the fact that the piles may penetrate through the cobble and boulder layer and may contact the bedrock. If this happens, the Hiley formula is not applicable and a site decision must be made that bedrock has been encountered and that further pile driving must be controlled to adequately seat the pile in the bedrock without overdriving and damaging the pile. A suitable criterion for deciding that bedrock has been contacted is: 10 blows at full energy for 12 mm penetration, for two consecutive sets of 10 blows. The geotechnical resistances given in Table 8.2 remain valid in this situation.

8.2.4 Downdrag

The soils at the abutments are non-cohesive and settlements induced in the native soils around the piles by construction of the approach embankments will be substantially complete as construction of the embankment is completed. Post-construction downdrag on the piles is therefore not considered to be an issue at this site. However, it is recommended that the approach embankments be constructed three months in advance of pile driving. The embankment should be constructed up to the level of the abutment from the forward slope to a distance back sufficiently far to allow access and operation of construction equipment. Beyond that distance, the embankment should be constructed to full height.

8.2.5 Lateral Resistance of Piles

The lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h \cdot z / D \quad (kN/m^3)$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (kPa)$$

where z = depth of embedment of pile in metres

D = pile width in metres

n_h = coefficient of horizontal subgrade reaction (Table 8.4)

γ = unit weight (Table 8.4)

K_p = passive earth pressure coefficient (Table 8.4)

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \times L \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult}*L*D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in one pile be limited to no more than 150 kN at ULS and 50 kN at SLS.

Table 8.4 – Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m^3)	K_p	Unit Weight (kN/m^3)
West Abutment	OGL to 307	1,600	2.7	19
	307 to 299	6,000	3.1	20
	299 to 295	9,000	3.3	20
Pier	OGL to 300	1,200	2.7	19
	300 to 297	8,000	3.0	20
	297 to 286	9,000	3.3	20
East Abutment	OGL to 294	1,600	2.7	19
	294 to 291	6,000	3.0	20
	291 to 286	9,000	3.3	20

The modulus of subgrade reaction may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 8.5. Intermediate values may be obtained by linear interpolation.

Table 8.5 – Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Reduction Factor
Pile group oriented ***perpendicular*** to direction of loading	4D	1.0
	1D	0.5
Pile group oriented ***parallel*** to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

* where D is the breadth of pile

In the case of conventional abutments, i.e. not integral, horizontal loads may be resisted by means of battered piles.

8.3 Caissons

The soil conditions, and more particularly the groundwater conditions at this site are not considered to be suitable for caisson foundations. To achieve the high resistance necessary to justify the construction costs, the caissons would have to be founded on bedrock, or possibly in very dense gravelly sand.

When attempting to found on bedrock, there could be difficulties sealing the liner to allow unwatering of the caisson and placement of concrete in the dry. In the case of caissons founded in the very dense gravelly sand, it would be impossible to achieve a seal and slurry excavation and tremie concreting would be necessary.

Caissons are also not considered to be suitable for construction on a batter to resist horizontal loads.

On the basis of the installation difficulties and risks assessed for this site, caissons are not recommended.

8.4 Recommended Foundation

The recommended foundation system for both abutments at this site is steel H-piles driven to refusal as controlled by application of the Hiley formula. The pier could be founded on driven steel H-piles or on spread footings constructed on engineered fill.

8.5 Abutment Type

From a geotechnical perspective, the subsurface conditions at this site are considered to be suitable for the construction of conventional, semi-integral or integral abutments. However, the recommended abutment foundation system of H-piles makes integral abutments a feasible option.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. At this site, the upper 3 m of the pile length will lie in very loose sandy silt which, in its original state, would provide sufficient flexibility. However, if the upper 3 m of the piles lies in compacted fill or if the native soil becomes compacted by the construction processes, the required flexibility may be compromised. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by one of the following systems:

- For a "true abutment" supported on top of the piles - a 600 mm diameter CSP filled with sand, or
- For "false abutment" - concentric CSPs in accordance with standard integral abutment design procedures.

The sand must be placed in the CSP after the pile has been driven to avoid the danger of the sand being densified by pile driving.

Backfill sand should meet the gradation shown in Table 8.6.

Table 8.6 – Integral Abutment Sand Grading

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 μm	#30	80%-100%
425 μm	#40	40%-80%
250 μm	#60	5%-25%
150 μm	#100	0%-6%

8.6 Frost Protection

The depth of earth cover required to provide frost protection for footings and pile caps at this site is 1.8 m.

It is possible to reduce the thickness of earth cover by the substitution of synthetic insulation. Typically, 25 mm of rigid, extruded polystyrene insulation is equivalent to 600 mm of earth cover. Synthetic insulation must be covered to provide protection where it is used.

Rock fill is not equivalent to earth fill in terms of thermal resistance. Frost may penetrate deeper through rock fill than earth fill and the possibility exists for freezing conditions to develop below the pile cap. Therefore, non-frost susceptible free-draining granular fill with less than 5% particles by mass finer than 75 μm should be specified for construction of the pile driving pad within the rock fill.

9 EXCAVATION (LOCAL)

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site may be classed as Type 3 soils above the water table. This classification is based on the lack of cohesion in the soils and the resulting possibility that excavation slopes will slough if excavated vertically for the lower 1.2 m. Excavation slopes should not exceed 1V:1H above the groundwater level.

Excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater may cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work.

Prior to excavation below the natural groundwater level, the groundwater must be depressed to a level below the deepest excavation level sufficient to maintain a stable base and prevent soil disturbance by construction traffic.

10 UNWATERING

Stabilized groundwater levels were not recorded during the preliminary investigation but based on the available data are expected to be near elevation 306. Based on the preliminary GA for the bridge structure, it is expected that work at the abutments will require excavation extending some 3 m below the stabilized groundwater level at the time of drilling. Excavation at the pier is expected to extend some 7 m below the observed water levels.

Highway 11 will be constructed in cut at the structure location and therefore the actual water levels encountered at the time of construction will depend upon the construction sequencing. It is recommended that the mainline cut be completed in advance of bridge construction so that the highway drainage system can lower the groundwater prior to the start of localized excavation and dewatering.

The design of the dewatering system that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility and the need to engage a dewatering specialist. While the responsibility for dewatering should remain with the Contractor, suitable systems that might be employed include pumping from filtered sumps for penetration of no more than 0.5 m below the groundwater level and the use of vacuum wellpoints for deeper penetration below the groundwater level.

11 APPROACH EMBANKMENTS

Approach embankments approximately 2.5 to 4.0 m high are required at the west approach and will be founded on loose to compact silt. A fill approximately 1.0 m high will be required within 5 m of the east abutment and will be founded on the loose to compact silt. To the east of this area, the Municipal Service Road profile will require a cut in the order of 1.0 m deep that will result in an unloading effect on the layer of soft to stiff silty clay encountered in this section.

Stability analysis indicates that earth fill approach embankments with 2H:1V side slopes or rock fill embankments with 1.25H:1V side slopes constructed on the native soils will have a factor of safety greater than 1.5. Foundation settlements under the embankment loading will be less than 25 mm and are expected to be immediate in nature.

The embankments are considered to be stable under earthquake loading on the assumption of a stable foundation. This topic is dealt with more completely in Section 16: Seismic Considerations.

All topsoil, organics and disturbed material should be stripped from the embankment footprint prior to fill placement. Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002 and referenced in Appendix C.

In accordance with Northeastern Region policy, embankments higher than 6 m must be provided with mid-height berms. The berms should:

- extend for the length through which the embankment height exceeds 6 m
- be 2 m wide
- have 2% positive drainage in earth fill to shed run-off water.

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 572.

12 MAINLINE CUT

In the vicinity of the structure, Highway 11 will lie in a cut approximately 5 m deep formed in the loose to compact silt and penetrating into the dense to very dense sand at the west ditch line. The base of the cut will lie 1 to 2 m below the groundwater level at the site. The following recommendations relate to the cut within 20 m of the structure centreline.

It is recommended that the cut at the structure be constructed in conjunction with the remainder of the mainline cut in this area. The cuts should be completed at least 3 months in advance of bridge construction to allow drawdown of the groundwater to occur, thus reducing the dewatering effort required at the structure.

The backslopes in the cut must be no steeper than 2H:1V and must be provided with rock protection, including geotextile, from the ditch invert up to elevation 306. Earth cut slopes in areas not covered with rock protection must be provided with erosion protection in accordance with OPSS 572.

13 RETAINED SOIL SYSTEMS

RSS walls used in conjunction with bridge abutments must be "High Performance" and "High Appearance". Therefore it is important to limit the settlement that the RSS walls experience due to compression of the foundation soils and embankment fill.

Provided the approach fill is placed at least 3 months prior to RSS wall construction and proper ground preparation is carried out prior to construction of the walls, RSS systems are considered suitable for the subsurface conditions at this site and are expected to meet the aesthetic and structural requirements.

The following minimum preparation of the base below the RSS is recommended:

- The topsoil and zones of loose silt should be stripped from the footprint of the RSS.

- The RSS wall at the west abutment should be founded on the dense to very dense silty sand at or below elevation 307.2 m.
- The RSS mass at the east abutment must be founded on an engineered fill pad at least 2 m thick and extending at least 500 mm beyond the limits of the RSS mass and levelling strip. The engineered fill must consist of OPSS Granular A or Granular B in conformance with SP110F13 and compacted according to OPSS 501 and SP105S10.
- The highest permitted founding level for the underside of the engineered fill at the east abutment is Elevation 304.0 m. A lower founding elevation may be required to accommodate the required thickness of engineered fill.
- Approach fill should be in place for at least 3 months prior to construction of the RSS to reduce post-construction consolidation settlement of the foundation soils.

For a wall constructed on a subgrade prepared as outlined above, the geotechnical parameters to be used for the design of the RSS walls are presented in Table 13.1.

Table 13.1 – RSS Design Parameters

Parameter	East Abutment	West Abutment
Factored ULS Bearing Resistance at Contact of RSS Wall and Prepared Subgrade	300 kPa	900 kPa
SLS Bearing Resistance at Contact of RSS Wall and Prepared Subgrade	200 kPa	350 kPa
Coefficient of Sliding Resistance at Contact between RSS Wall and Prepared Subgrade	0.55	0.55

The design, supply and construction of RSS must be in accordance with SP 599S22. The supplier of the proprietary RSS system must demonstrate that it will meet the Ministry's specifications for performance and appearance. The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design.

14 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material.

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill to the abutment should be limited to fragments no greater than 75 mm and including adequate spalls to fill voids in the rock fill.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular "B" Type II.

The backfill to the abutment walls must be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill must be placed to the extents shown in OPSD 3501.000, and rock backfill must be placed to the extents shown in OPSD 3505.000.

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with SSP 105S10.

The design of the abutment must incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

15 EARTH PRESSURE COEFFICIENTS (ABUTMENTS)

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K(\gamma h + q)$$

where: p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table below)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 15.1.

Table 15.1 – Earth Pressure Coefficients (K)

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$, $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$, $\gamma = 21.2$ kN/m^3		Rock Fill (Limited to 300 mm size) $\phi = 42°$, $\gamma = 19$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.43*	0.2	.30*
At rest (Restrained Wall)	0.43	-	0.47	-	.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in Table 15.1 above are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

16 SEISMIC CONSIDERATIONS

For design purposes, the site is treated as lying in Seismic Zone 1.

16.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 1
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.08

The Soil Profile Type at this site has been classified as Type I. Thus, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" of 1.0 should be used in seismic design.

16.2 Liquefaction Potential

The potential for liquefaction of the foundation soils has been assessed using the Seed and Idriss (1971) method[1].

Using this method, it was determined that the foundation soils at the abutments are not in danger of liquefaction under earthquake loading.

16.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The seismic earth pressure coefficients to be used in design at this site are shown in Table 16.1.

[1] Seed, H.B. and Idriss, I.M. 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential" *Journal of Soil Mechanics and Foundations Division,* ASCE, Vol. 101, No. SM9, pp. 1249 – 1273.

Table 16.1 – Earth Pressure Coefficients (K) for Seismic Design

Condition	Earth Pressure Coefficient (K) for Earthquake Loading					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°, \delta = 17°$		OPSS Granular B Type I $\phi = 32°, \delta = 16°$		Rock Fill (Limited to 300 mm size) $\phi = 42°, \delta = 21°$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active*, K_{AE} (Unrestrained Wall)	0.30	0.45	0.33	0.54	0.23	0.31
At rest**, K_{OE} (Restrained Wall)	0.59	-	0.63	-	0.43	-
Passive*, K_{PE} (Movement Towards Soil Mass)	6.3	-	5.4	-	12.0	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods

In Table 16.1, the angle of friction between the wall and the backfill, δ, is taken as 50% of the angle of internal friction of the backfill, ϕ.

16.4 Slope Stability Considerations

Seismic effects were taken into account in the slope stability analyses conducted for this site using pseudo-static methods and assuming that the foundation soils would not be subject to liquefaction. Under these conditions, satisfactory factors of safety were obtained from the analysis, i.e. all values exceeded 1.0.

17 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- The possibility of piles reaching refusal on large boulders or bedrock. In this case, the Hiley formula is not appropriate and site staff must make a decision regarding pile resistance and the appropriateness of continued driving.
- The potential variability of pile lengths at refusal.
- Excavation for spread footings and engineered fill construction, if employed, is expected to encounter cobbles and boulders. Provision of a level base in this material may be difficult if large boulders are encountered at the excavation base level. Excavation should be carried out in a manner which minimizes disturbance to the subgrade.

- Excavation below the groundwater level at the time of construction will require an unwatering system suitable to maintain a sound, dry excavation for pile cap, footing or engineered fill construction.

18 CLOSURE

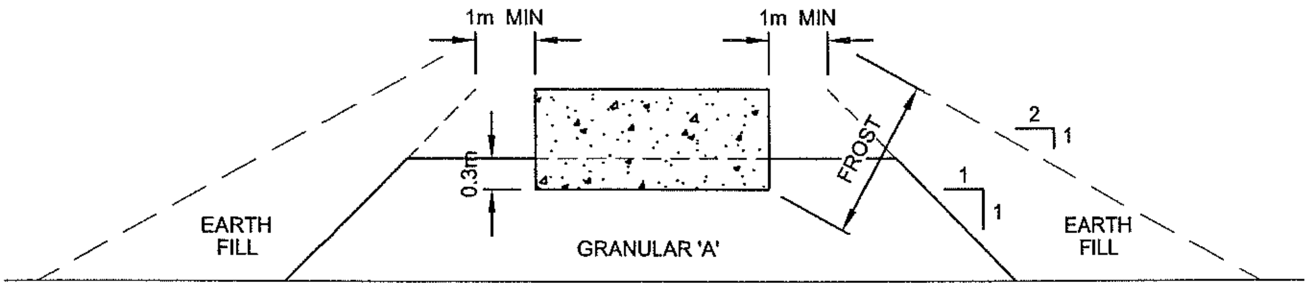
Engineering analysis and preparation of the foundation design report were carried out by Mr. Murray Anderson, P.Eng. The report was reviewed by Mr. Alastair E. Gorman, P.Eng. The report was also reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.
Murray R. Anderson, P.Eng., M.Eng.
Senior Geotechnical Engineer

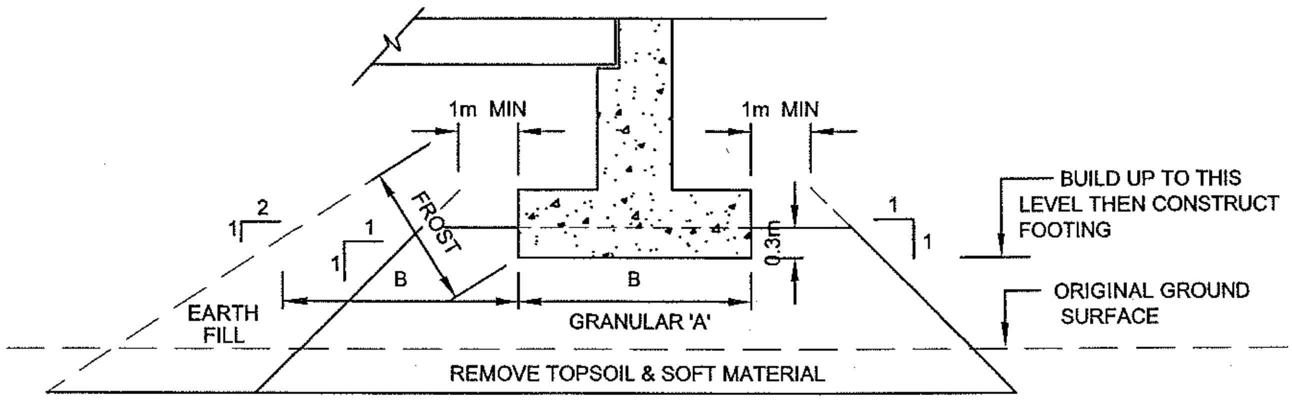
Alastair E. Gorman, P.Eng., M.Sc.
Senior Foundations Engineer

P.K. Chatterji, P.Eng., Ph.D.
Review Principal

March , 2004



CROSS-SECTION



LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ENGINEER	AEG
DRAWN	SS
DATE	April , 2004
APPROVED	PKC
SCALE	NTS

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE

THURBER

DWG. NO. FIGURE 1

TED35146 .DWG

Appendix A

Data From Preliminary Investigation Report

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT[1] 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

SS Split Spoon Sample
WS Wash Sample
AS Auger (Grab) Sample
TW Thin Wall Shelby Tube Sample
TP Thin Wall Piston Sample
PH Sampler Advanced by Hydraulic Pressure
PM Sampler Advanced by Manual Pressure
WH Sampler Advanced by Self Static Weight
RC Rock Core
SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

▼ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength** (MPa)	(psi)	**Field Estimation of Hardness***
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BPR1

METRIC

W.P. 314-99-00 LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 048 931.8; E 315 920.1 ORIGINATED BY G.I

DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY G.T

DATUM Geodetic DATE 06.04.01 CHECKED BY LSR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
312.5	Ground Surface									
0.0	75 mm Topsoil; SILT (laminated) moist to wet; loose ----- compact	1	SS	4		312				
		2	SS	14			0	2	98	0
		3	SS	13		311				
	-------- increasing fine sand content	4	SS	15		310				
	brownish	5	SS	10		309				
	------ grey	6	SS	13			0	12	88	0
		7	SS	16		308				
		8	SS	16		307				
306.6 / 5.9	SILTY SAND some gravel, very dense, grey, wet				▼*					
305.9		9	SS	60		306				
6.6	End of borehole. Refusal to augering at 6.6 m. *Water level at 5.9 m (not stabilized) and hole open to 6.0 m on completion									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ ³, × ³: Numbers refer to Sensitivity; 20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BPR2

METRIC

W.P. 314-99-00 LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 048 946.8; E 315 915.0 ORIGINATED BY G.I

DIST 52 HWY 11 BOREHOLE TYPE Solid and Hollow Stem Augering, Washboring & NQ Rock Coring COMPILED BY G.T

DATUM Geodetic DATE 11.04.01 & 12.04.01 CHECKED BY LSR

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

312.2 Ground Surface

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS / GRAIN SIZE (GR SA SI CL)
0.0	150 mm **Topsoil**; very loose	1	SS	2	312	
	SILT (laminated) wet; rootlets to 1.2 m; compact to loose	2	SS	12	311	0 4 92 4
		3	SS	10		
	increasing fine sand content; brown	4	SS	11	310	
	grey	5	SS	9	309	
		6	SS	8	308	
307.2		7	SS	8		0 14 82 4; HST augering
5.0	**SILTY SAND** some gravel, dense to very dense, grey	8	SS	60/13	307	washboring
	BOULDER	9	NQ			
		10	SS	48	306	
	moist to wet	11	SS	59	305	
	wet	12	SS	95	304	4 65 29 2
		13	SS	62	303	
	occasional sandy silt till lenses	14	SS	60/0	302	
	BOULDERS	15	NQ		301	April 11 (water level)
		16	NQ			April 12
		17	SS	44	300	38 48 (14)
299.0 / 13.2	**GRAVELLY SAND** with silt, cobbles and boulders very dense, grey, wet	18	SS	60/10	299 / 298	37 47 (16)
297.2 / 15.0						

Continued Next Page

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

15 –20/10– 5 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BPR2

METRIC

W.P. 314-99-00 LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 048 946.8; E 315 915.0 ORIGINATED BY G.I

DIST 52 HWY 11 BOREHOLE TYPE Solid and Hollow Stem Augering, Washboring & NQ Rock Coring COMPILED BY G.T

DATUM Geodetic DATE 11.04.01 & 12.04.01 CHECKED BY LSR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
297.2						
15.0	**GRAVELLY SAND** with silt, cobbles and boulders very dnese, grey, wet	19	SS	60/5	297	
					296	
		20	SS	60/14		
294.9	BOULDER	21	NQ		295	
17.3	BEDROCK (Gneiss) grey, moderately weathered	22	NQ RC	Rec. 73%		RQD=60%
					294	
		23	NQ RC	Rec. 85%	293	RQD=65%
	slightly weathered				292	
		24	NQ RC	Rec. 90%		RQD=77%
291.0						
21.2	End of borehole *Water level at 11.2 m (not stabilized in hollow stem augers). Hole open to 1.7 m on completion					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GR SA SI CL

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

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BPR3

METRIC

W.P.	314-99-00	LOCATION	Municipal Service Road Underpass - Katrine, ON - Coords: N 5 048 996.8; E 315 917.0	ORIGINATED BY	G.I
DIST	52 HWY 11	BOREHOLE TYPE	Hollow Stem Augering, Washboring & NQ Rock Coring	COMPILED BY	G.T
DATUM	Geodetic	DATE	06.04.01 to 11.04.01	CHECKED BY	LSR

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) w	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
308.6	Ground Surface							
0.0	125 mm Topsoil	1	SS	5	308	~21	18.8	
	SILT (laminated) loose to compact, wet; brown ----- grey; ----- clayey laminations -----	2	SS	6		~25		0 0 86 14
		3	SS	11	307	~23	18.9	
		4	SS	8	306	~21	18.5	0 4 96 0
		5	SS	10	305	~20	18.6	
		6	SS	9		~21	19.8	
		7	SS	8	304	~18	18.7	
	------ increasing fine sand content	8	SS	11	303			
		9	SS	9	302	~18		
		10	SS	6	301	~21		
300.1 / 8.5	SILTY SAND some gravel, grey, wet; compact ------ dense	11	SS	22	300			14 60 26 0
		12	SS	38	299 / 298			
297.4 / 11.2	GRAVELLY SAND with silt, cobbles and boulders, very dense, grey wet	13	SS	50/14	297 / 296	~6		
		14	SS	50/3	295	~5		
	BOULDERS	14A	NQ	Rec. 26%	294			
293.6 / 15.0								

Ground water conditions remarks: April 06 ----- April 09

Remarks: HST augering ------ washboring

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BPR3

METRIC

W.P. 314-99-00 LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 048 996.8; E 315 917.0 ORIGINATED BY G.I

DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering, Washboring & NQ Rock Coring COMPILED BY G.T

DATUM Geodetic DATE 06.04.01 to 11.04.01 CHECKED BY LSR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
293.6						
15.0	**GRAVELLY SAND** with silt, cobbles and boulders, very dense, grey wet	14A	NQ		293	
		15	SS	73	292	April 09 ----- April 10
		16	SS	50/8	291, 290	
		17	SS	50/13	289, 288	
		18	SS	50/14	287	
286.2	BOULDER					
22.4	**BEDROCK** (Gneiss) grey, slightly weathered	19	NQ		286	
		20	NQ RC	Rec. 100%	285	RQD=85%
		21	NQ RC	Rec. 95%	284	RQD=87%
283.3 25.3	End of borehole Hole caved at 3.2 m on completion *Water level in hollow stem auger at 8.3 m Piezometer installed on April 11/2001 to 15.2 m Water level in piezometer at 6.5 m depth on April 11/2001. Probably not stabilized					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

GR SA SI CL

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

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

RECORD OF BOREHOLE No BPR4

METRIC

W.P. 314-99-00 LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 049 045.7; E 315 924 ORIGINATED BY G.I

DIST 52 HWY 11 BOREHOLE TYPE Solid and Hollow Stem Augering, Washboring & NQ Rock Coring COMPILED BY G.T

DATUM Geodetic DATE 11.04.01 to 17.04.01 CHECKED BY LSR

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
307.1	Ground Surface						
0.0	250 mm Topsoil	1	SS	28		307	
	SILT (laminated) occasional thin clay layers; loose to compact, ------- compact,; moist brown ----- grey wet	2	SS	26	▼	306	0 2 76 22
		3	SS	7		305	
		4	SS	13		304	
		5	SS	19			
		6	SS	19		303	
		7	SS	29		302	0 9 91 0
		8	SS	27		301	April 11 ----- April 16
		9	SS	19			
	cobbles	10	SS	18		300	
		11	SS	13		299	
	fine sand content increasing with depth	12	SS	11		298	
		13	SS	2	**	297 / 296	** SS13 Low N-value probably due to hydrostatic uplift
							Hollow Stem Augering ------- Washboring
		14	SS	26		295	0 37 (63)
293.9 / 13.2	SILTY SAND some gravel, compact, grey, wet	15	SS	23		294 / 293	
292.1 / 15.0	Continued Next Page						

Dynamic cone penetration resistance plot: 20 40 60 80 100. Shear strength kPa: ○ Unconfined + Field Vane ● Quick Triaxial × Lab Vane; 20 40 60 80 100.

Plastic limit w_P — Natural moisture content w — Liquid limit w_L. Water content (%): 20 40 60.

Unit weight γ kN/m³

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

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No BPR4

METRIC

W.P. 314-99-00 | LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 049 045.7; E 315 924 | ORIGINATED BY G.I

DIST 52 HWY 11 | BOREHOLE TYPE Solid and Hollow Stem Augering, Washboring & NQ Rock Coring | COMPILED BY G.T

DATUM Geodetic | DATE 11.04.01 to 18.04.01 | CHECKED BY LSR

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
292.1 / 15.0	**SILTY SAND** some gravel, grey, wet, dense		16	SS	37		292	April 16 ----- April 17; 28 58 14 0
290.9 / 16.2	**GRAVELLY SAND** with silt, cobbles and boulders, very dense, grey, wet		17	SS	50/14		291, 290	
			18	SS	50/13		289, 288	
			19	SS	50/13		287, 286	
			20	SS	50/8			
285.7 / 21.4	BEDROCK (Gneiss) grey; slightly weathered		21	NQ RC	Rec. 100%		285	RQD=93%
	unweathered		22	NQ RC	Rec. 100%		284	RQD=100%
	slightly weathered		23	NQ RC	Rec. 100%		283	RQD=72%
282.6 / 24.5	End of borehole *Water level not stabilized on completion Borehole open to 7.9 m depth *Groundwater probably at 1.5 m depth from sample moisture condition							Casing withdrawn on April 18

+ ³, × ³: Numbers refer to Sensitivity   20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No BPR5

METRIC

W.P. 314-99-00 LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 049 066.8; E 315 920.0 ORIGINATED BY G.I

DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY G.T

DATUM Geodetic DATE 06.04.01 CHECKED BY LSR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.4	Ground Surface						
0.0	200 mm Topsoil	1	SS	5		309	
	SILTY CLAY with silt layers, brown ---- grey, damp to moist ----- moist	2	SS	15		308	0 4 46 50
		3	SS	11			
	silt content increasing with depth, stiff to soft	4	SS	6		307	
	---- wet	5	AS		▼*		High vane test results probably caused by silt layers
						306	
305.1		6	SS	2			0 0 77 23
4.3	SILT grey, wet	7	SS	3		305	
		8	SS	6		304	0 0 86 14
	very loose to loose	9	SS	6		303	
	-------------- compact					302	
	------ increasing fine sand content	10	SS	18			0 4 92 4
						301	
299.8	COBBLES	11	SS	21		300	
9.6	End of borehole *Water level not stabilized; encountered at 3.0 m during drilling Borehole caved at 4.9 m on completion						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

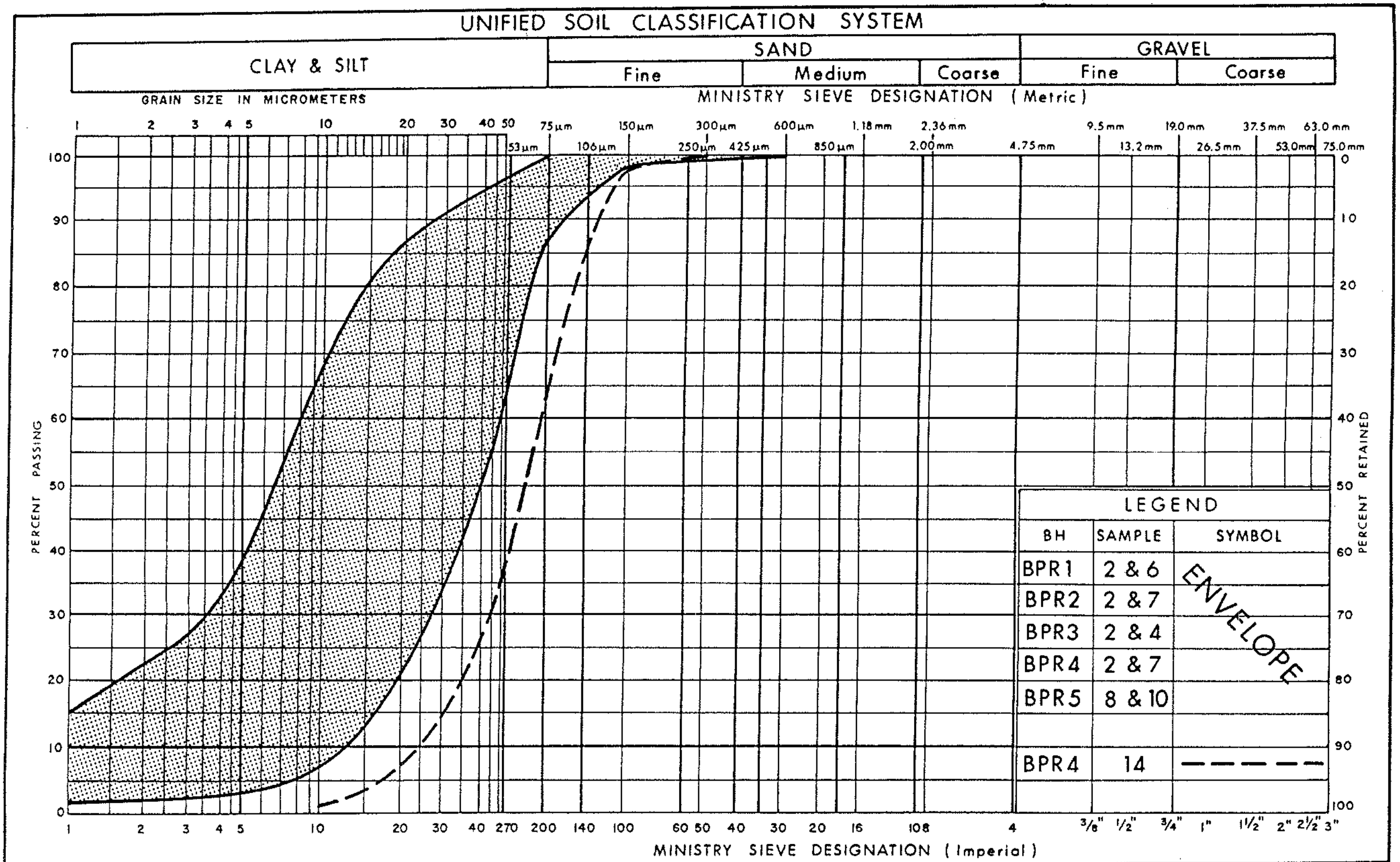
UNIT WEIGHT γ kN/m^3

Continued Next Page

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

78 12 M



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Ontario

GRAIN SIZE DISTRIBUTION

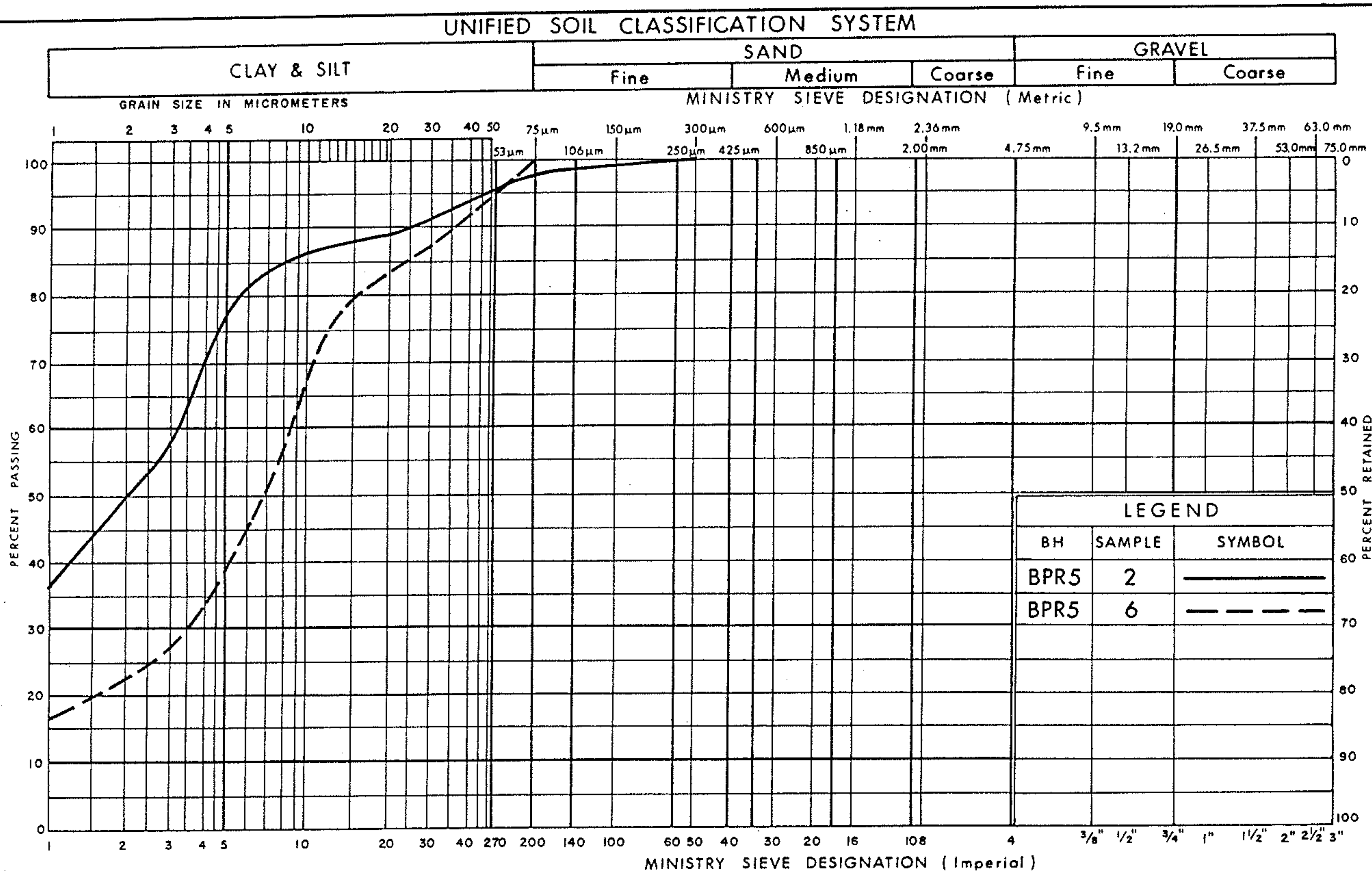
SILT

FIG No 1

W P 314-99-00

SPT 1010E

78 12 M



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GRAIN SIZE DISTRIBUTION

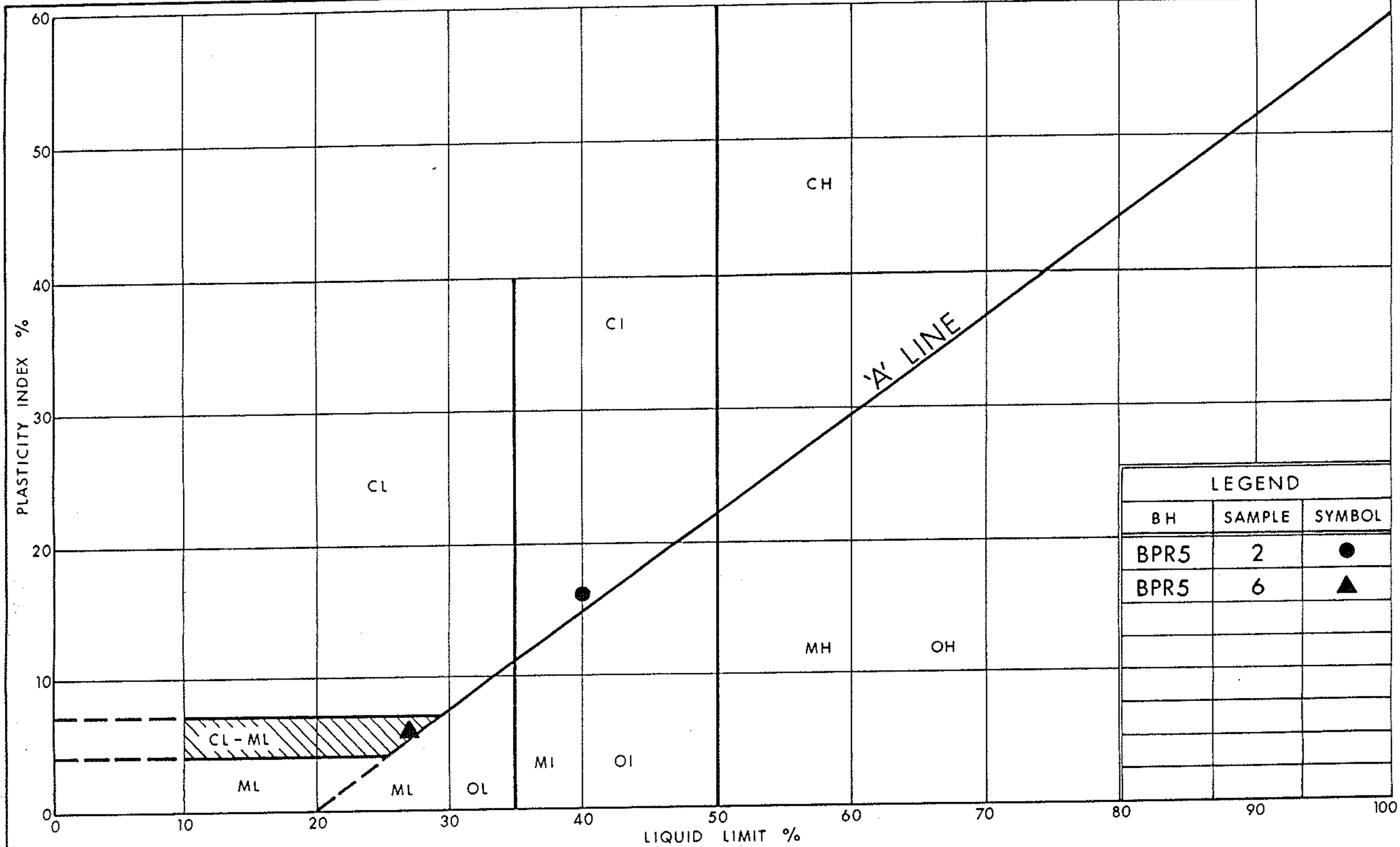
SILTY CLAY

FIG No 2

W P 314-99-00

SPT 1010E

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
BPR5	2	●
BPR5	6	▲

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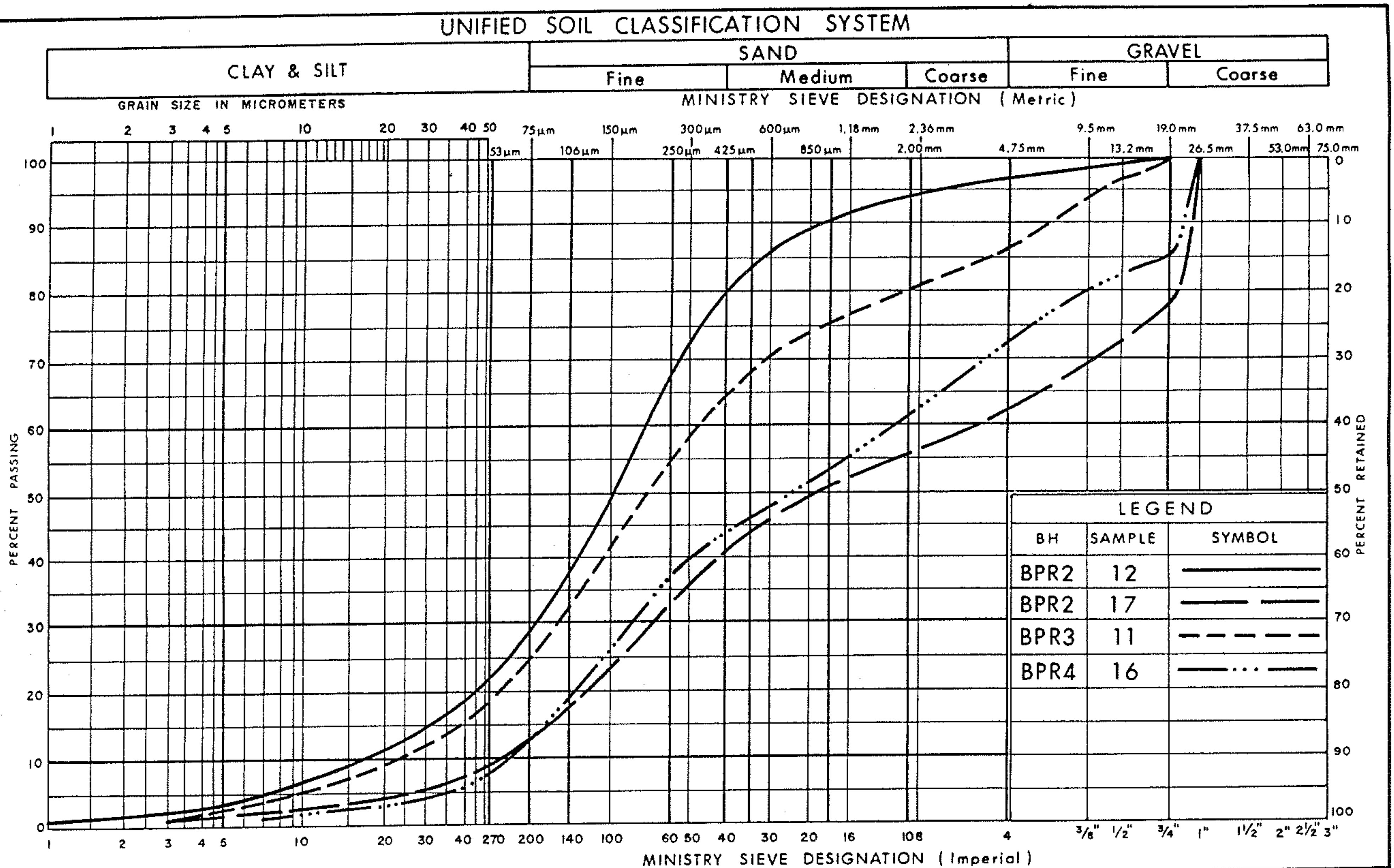
Ontario

PLASTICITY CHART
SILTY CLAY

FIG No 3
W P 314-99-00
SPT 1010E

78 12 M



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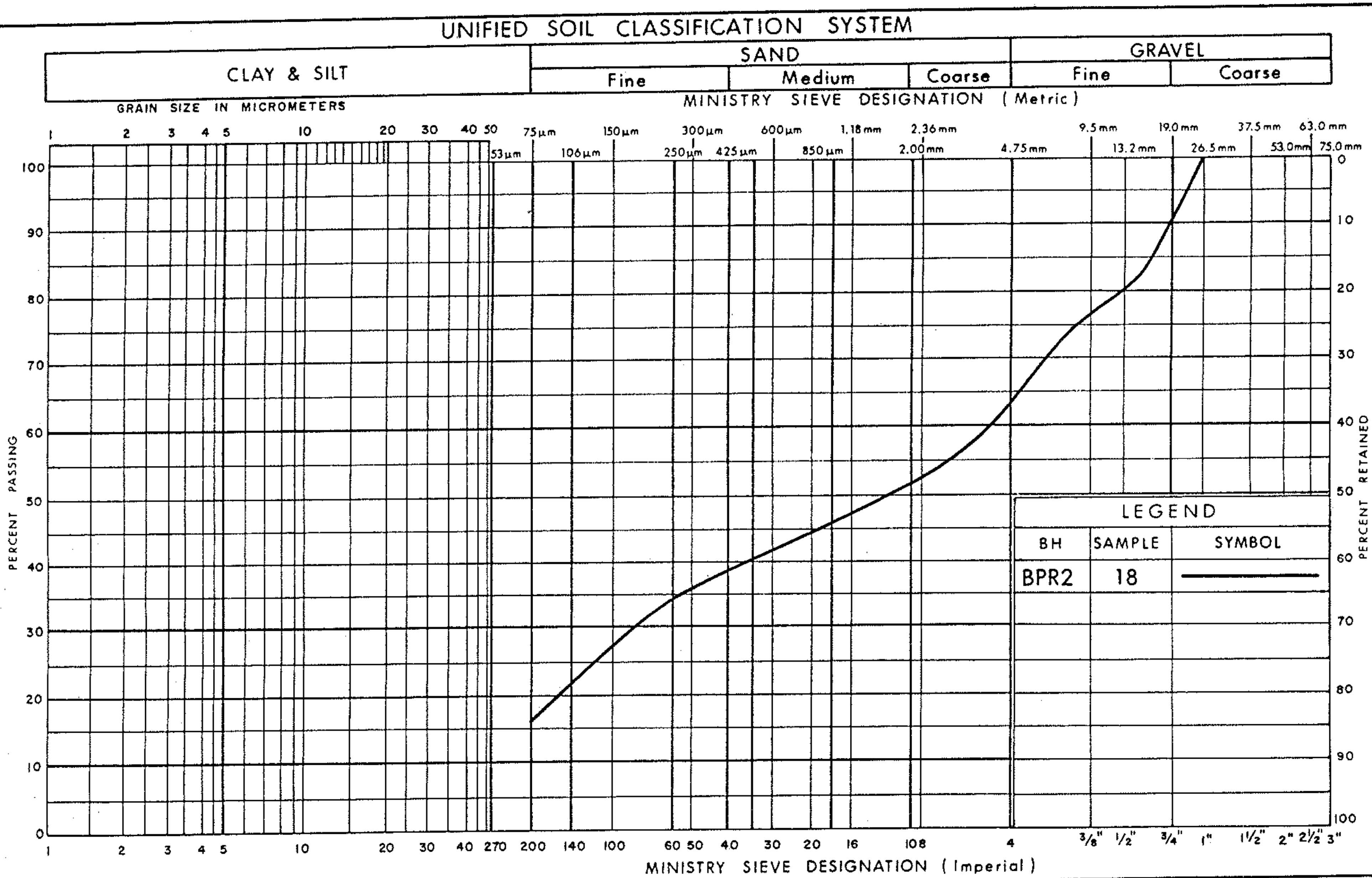
GRAIN SIZE DISTRIBUTION
SILTY SAND, SOME GRAVEL

FIG No 4
W P 314-99-00
SPT 1010E

78 12 M



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GRAIN SIZE DISTRIBUTION

GRAVELLY SAND WITH SILT, COBBLES & BOULDERS

FIG No 5

W P 314-99-00

SPT 1010E

Appendix B

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Driven Piles	Footing on Native Soil	Footing on Engineered Fill	Caisson
Advantages: i. Piles will develop high geotechnical resistance if driven to refusal in the very dense soil. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed. ***Disadvantages:*** i. Higher unit costs than footings. ii. Construction concerns related to the possibility of pile being obstructed by a boulder during driving. **RECOMMENDED**	***Advantages:*** i. Ease of construction. ii. Allows choice of conventional or semi-integral abutment. iii. Lower cost than deep foundations. ***Disadvantages:*** i. Low geotechnical resistance available at this site. ii. Potential for unacceptable magnitude of settlement. iii. Potential for subgrade disturbance during excavation of bouldery material. iv. Potential subgrade variability due to boulders below the founding level. v. Requires subexcavation and dewatering to construct. **NOT RECOMMENDED**	***Advantages:*** i. Would permit use of higher geotechnical resistance than is available on the native soil. ii. Allows choice of conventional or semi-integral abutment. iii. Allows use of perched abutments. iv. Lower cost than deep foundations. ***Disadvantages:*** i. Low geotechnical resistance available at this site. ii. Cost of constructing engineered fill. iii. Potential for unacceptable magnitude of settlement. iv. Requires subexcavation and dewatering to construct **NOT RECOMMENDED FOR ABUTMENTS**	***Advantages:*** i. High resistance is available for caissons founded on very dense soil. ii. Construction of caissons could continue in freezing weather. iii. Choice of conventional or semi-integral abutment design. ***Disadvantages:*** i. Soil conditions encountered at this site are considered to be unsuitable. **NOT RECOMMENDED**

Appendix C

Special Provisions

The following Special Provisions are referenced in this report:

- Amendment to OPSS 206, December 1993
- Special Provision No. 902S01
- Special Provision No. 903S01
- Standard Special Provision No. 105S10

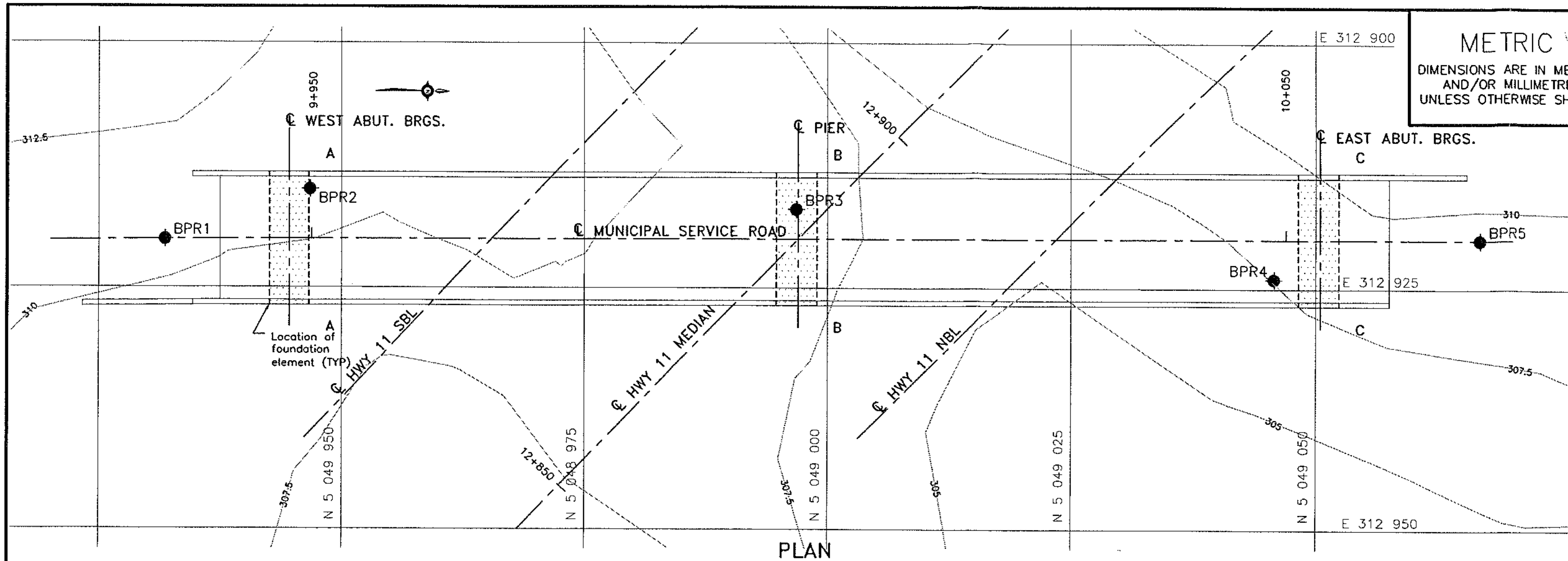
Suggested text for a NSSP on Pile Installation should contain the following:

" The soil at this site contains cobbles and boulders. The presence of cobbles and boulders will potentially have an impact on the installation of piles. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- *The need to provide protection to the pile tips in the form of rock points*
- *The cobbles and boulders may impede the driving of the piles resulting in more arduous driving*
- *Some piles may meet refusal on boulders that are large enough not to be dislodged or broken by the pile driving*
- *As a result of the presence of boulders, piles may meet refusal at varying depths*
- *Pile driving must be controlled according to the criteria specified for the site."*

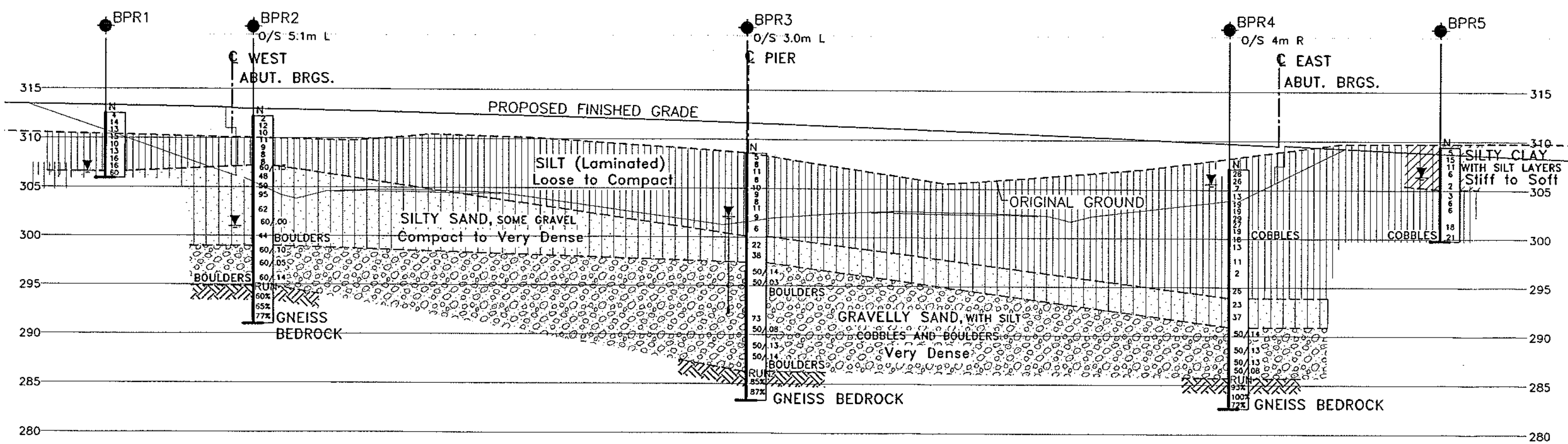
Appendix D

Drawings



PLAN

SCALE: 1:250



℄ PROFILE MUNICIPAL SERVICE ROAD

SCALE: 1:250

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

HWY 11
CONT No
WP No 5405-04-01

MUNICIPAL SERVICE ROAD UNDERPASS

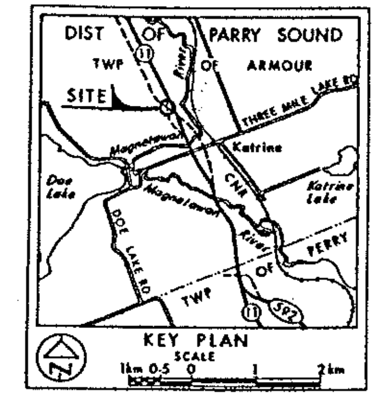
BOREHOLE LOCATION & SOIL STRATA

SHEET

Marshall Macklin Monaghan

PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.



LEGEND

●	BoreHole
⊕	Dynamic Cone Penetration Test (cone)
	BoreHole and Cone
N	Blows /0.3m (std pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	WL
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
BPR1	312.5	5 048 931.8	315 920.1
BPR2	312.2	5 048 946.8	315 915.0
BPR3	308.6	5 048 996.8	315 917.0
BPR4	307.1	5 049 045.7	315 924.0
BPR5	309.4	5 049 066.8	315 920.0

NOTE

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

LICENSED PROFESSIONAL ENGINEER M. R. ANDERSON MAR 16/06 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER A. E. GORMAN MAR 16/06 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER P. K. CHATTERJI Mar 16/06 PROVINCE OF ONTARIO

BENCH MARK

DRAWING NOT TO BE SCALED 100 mm ON ORIGINAL DRAWING

REVISIONS

DATE	BY	DESCRIPTION

DESIGN AEG	CHK AEG	CODE CHBDC 2000	LOAD CL-625-ONT	DATE Nov. 25
DRAWN SS	CHK PKC	SITE 44-427	STRUCT.	SCHEME. DWG 2

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

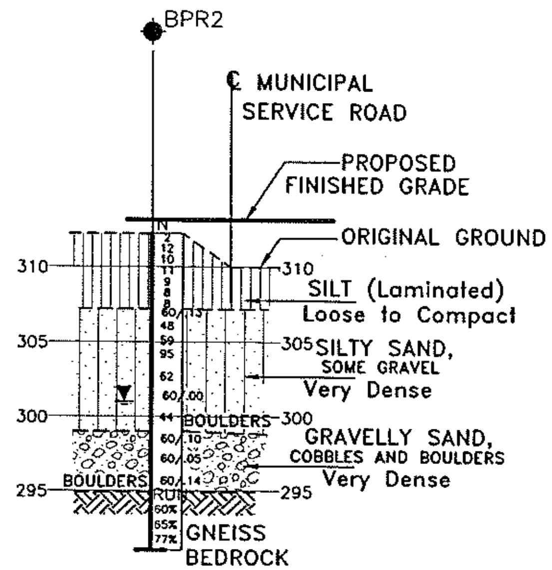
HWY 11
CONT No
WP No 5405-04-01

MUNICIPAL SERVICE ROAD UNDERPASS
SOIL STRATA

SHEET

Marshall Macklin Monaghan
PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.



SECTION A–A

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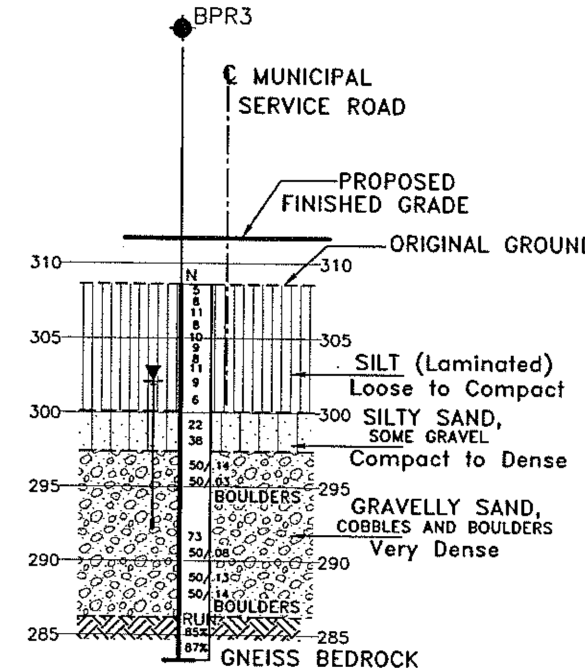

SECTION B–B

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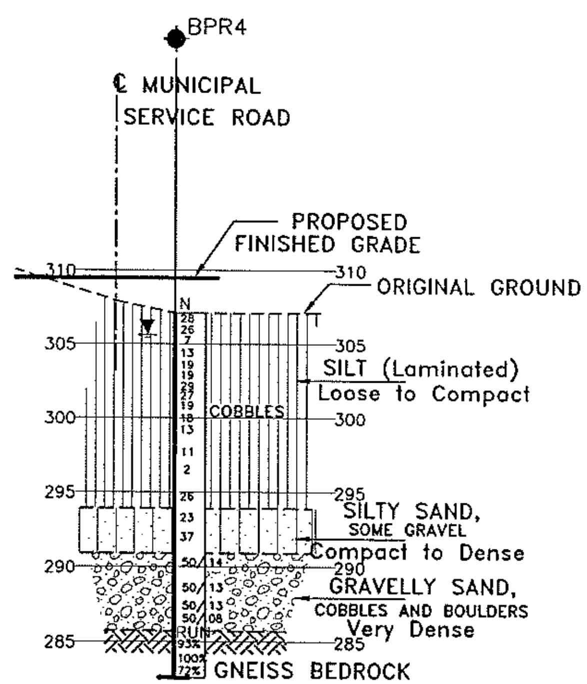
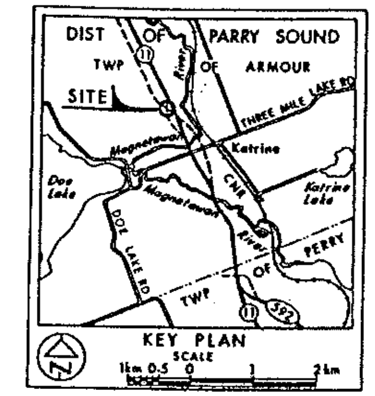
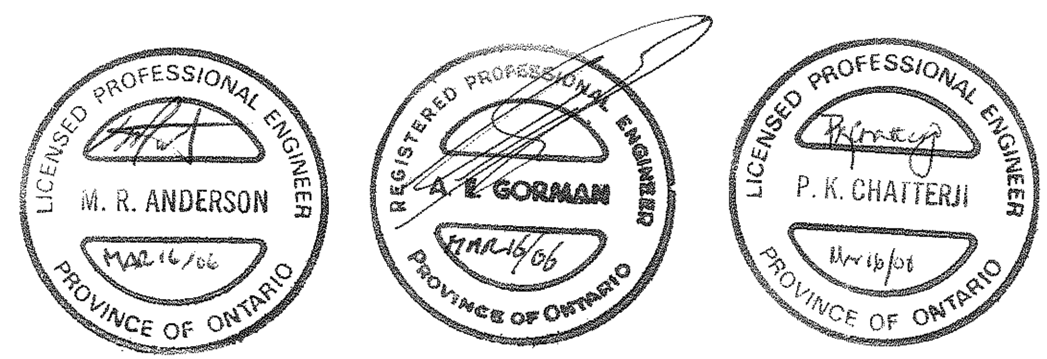


SECTION C–C

SCALE: 1:250



KEY PLAN



LEGEND

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BENCH MARK

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION

DESIGN AEG	CHK AEG	CODE CHBDC 2000	LOAD CL-625-ONT	DATE Nov. 25	
DRAWN SS	CHK PKC	SITE 44-427	STRUCT .	SCHEME .	DWG 2A