G.I.-30 SEPT. 1976

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 40P9-31

DIST. 3 REGION

W.P. No. 533-91-04

CONT. No. 96-37

W. O. No.

STR. SITE No. 35-579

HWY. No. 6

LOCATION Hwy 6 & Speed River Ramp S-E/W

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

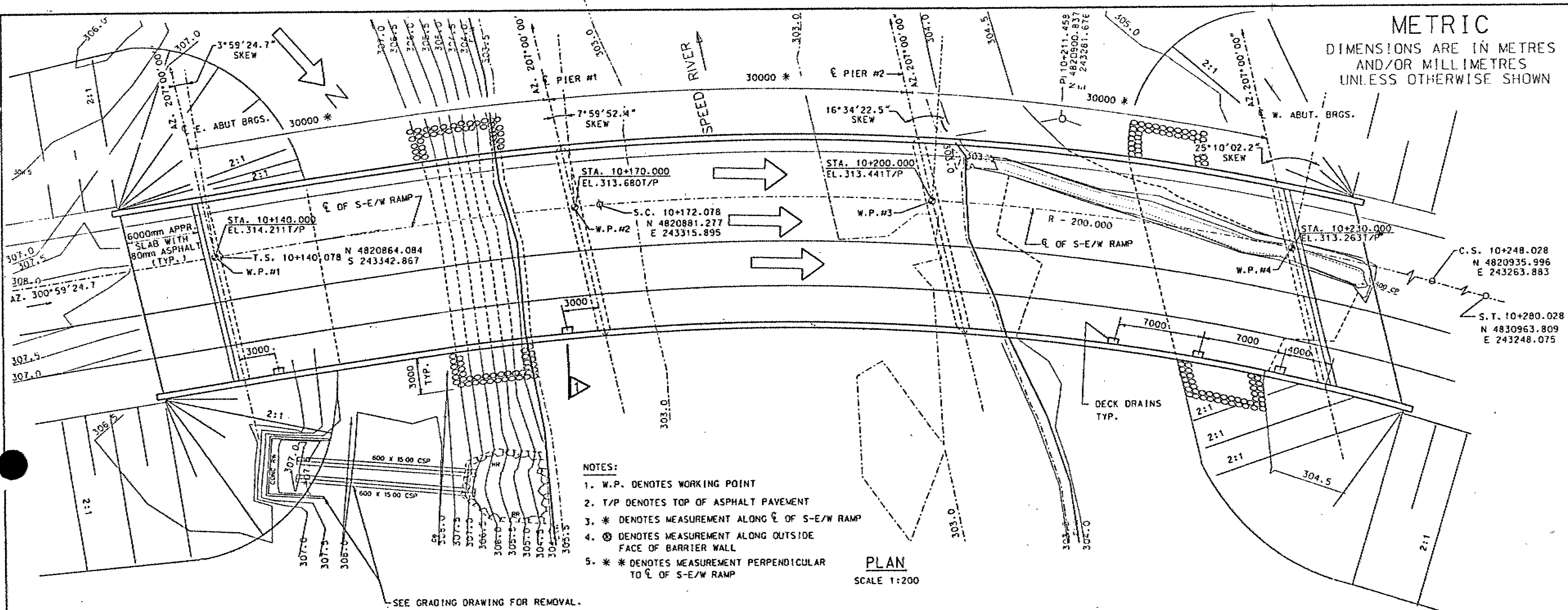
METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST No 3
CONT No 96-37
WP No 533-91-04

SPEED RIVER BRIDGE
S-E/W RAMP
HWY.6(HANLON EXPRESSWAY)
GENERAL ARRANGEMENT

SHEET 201



NOTES:
1. W.P. DENOTES WORKING POINT
2. T/P DENOTES TOP OF ASPHALT PAVEMENT
3. * DENOTES MEASUREMENT ALONG ℄ OF S-E/W RAMP
4. ⊗ DENOTES MEASUREMENT ALONG OUTSIDE FACE OF BARRIER WALL
5. ** DENOTES MEASUREMENT PERPENDICULAR TO ℄ OF S-E/W RAMP

GENERAL NOTES

CLASS OF CONCRETE

PRECAST GIRDERS	40 MPa
REMAINDER (UNLESS OTHERWISE NOTED)	30 MPa

CLEAR COVER TO REINFORCING STEEL

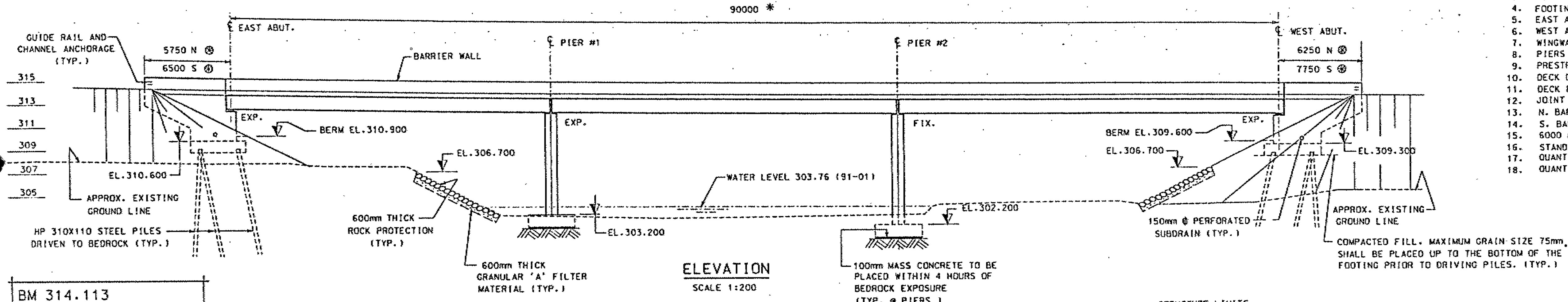
FOOTINGS	100 ± 25
ABUTMENTS & WINGWALLS	
FRONT FACE	70 ± 20
BACK FACE	70 ± 20
PIERS	70 ± 20
DECK	
TOP	70 ± 20
BOTTOM	40 ± 10
REMAINDER (UNLESS OTHERWISE NOTED)	70 ± 20

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH THE SUFFIX "C" DENOTE COATED BARS.

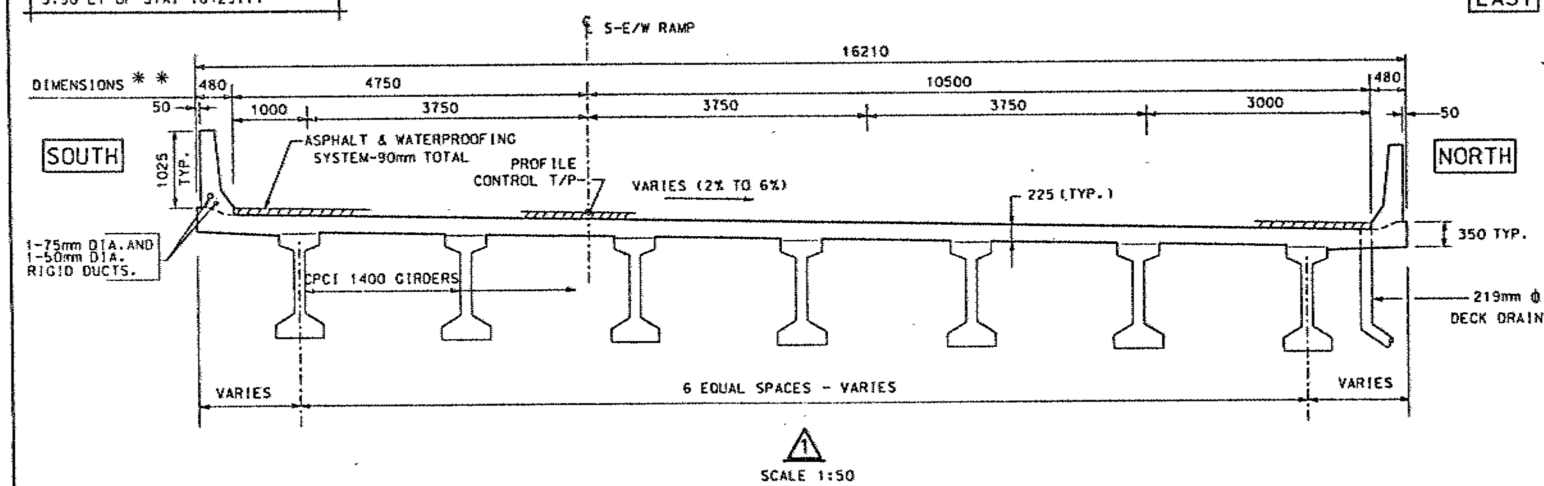
CONSTRUCTION NOTES

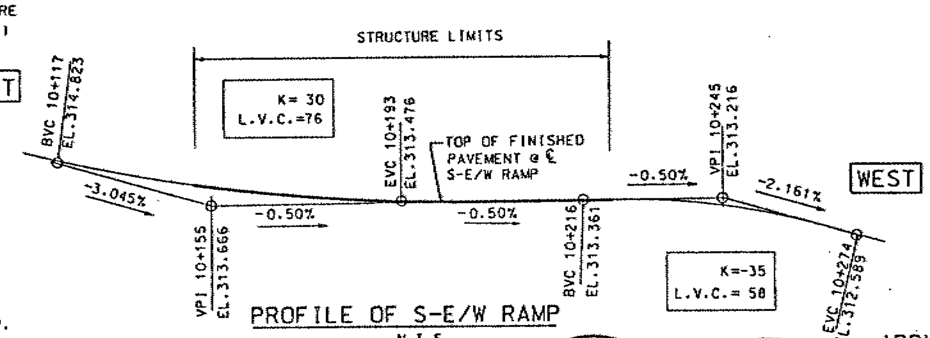
THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

LIST OF DRAWINGS



BM 314.113
CC ON CONCRETE CURB OF S'LY STRUCTURE OVER SPEED RIVER. 5.90 LT OF STA. 16+297.1





LICENSED PROFESSIONAL ENGINEER A.J.MERLO 16/07/30 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.S.Y.WONG PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

APPLICABLE STANDARD DRAWINGS

OPSD - 3501.00 GRANULAR BACKFILL REQUIREMENTS
OPSD - 4010.00 GUIDE RAIL AND CHANNEL ANCHORAGE

REVISIONS

		DESCRIPTION			
DESIGN AJM	CHK S.W.	CODE	OHBDC-91	DATE APRIL 1994	
DRAWN J.F.	CHK AJM	SITE 35-579	STRUCT	SCHEME	DWG 1

FILE COPY

Ministry of Transportation

Ontario

FOUNDATION DESIGN SECTION

foundation investigation and design report

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

CONT 96-37

WP 533-91-04 DIST 3

HWY 6 & 24 STR SITE 35-579

Proposed S-E/W Ramp Structure Crossing
Over Speed River and
Adjacent Retaining Wall

DISTRIBUTION

A. Ho (2)
C.M. Bond
A.E. Irving
E.J. Zavitski (2)
B. Farago
G.E. Greene
E.A. Joseph
G. Laithwaite (Cover Only)
F. Bacchus (Cover Only)
File

GEOCRES 40P9-31 DATE AUG 04 1993

FOUNDATION INVESTIGATION REPORT

For

Proposed S-E/W Ramp Structure Crossing

Over Speed River

and

Adjacent Retaining Wall

W.P. 533-91-04, Site 35-579

Hwy. 6 & Hwy. 24 District 3, Stratford

INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the proposed Hwy. 6 South - Hwy. 24 E/W Ramp structure and adjacent retaining wall in the City of Guelph, District 3 (Stratford) within the Southwestern Region. The ramp structure is proposed to cross the Speed River and the retaining wall is proposed immediately south of the structure extending beyond the ramp structure abutment and the existing Hydro transmission tower which is located approximately 30 metres south of the abutment. The structures are planned in conjunction with the new Hanlon Expressway (Hwy. 6) and Wellington Street (Hwy. 24) interchange. The new interchange will replace the existing level intersection that is now present. The scope of the report is limited to the structures and immediate approaches.

SITE DESCRIPTION AND GEOLOGY

General

The site is located approximately 75 to 100 metres east of the existing Hwy. 6 Northbound/Speed River Crossing structure in the City of Guelph, Wellington County. Wellington Street (Hwy. 24) is located approximately 0.5 km north of the proposed structure location. The existing northbound structure is located adjacent to a twin southbound structure. The structures are reinforced concrete four (4) span structures constructed in 1971.

The Speed River flowing in a westerly direction is naturally the dominant feature at the site. At the site location, the river is approximately thirty-five metres in width. Embankment slopes, approximately 1H:1V and five (5) metres in height are present on the south river bank whereas on the north river bank the land is flat adjacent to the river. The south embankment appears to be the result of fill placed for the construction of an access roadway in the area. The river level at the time of the investigation was approximately 1 metre in depth.

Beyond the river on the south side, the slope of the ground rises gradually. A hydro corridor exists within the southern limits of the site. On the north side, the ground surface rises gradually as well but approximately 100 metres north of the site, the ground level is flat. An abandoned landfill site is situated at this location. It appears that this area was the location of a former gravel pit.

Land Use and Terrain

The site is mainly grassland, which was snow covered for the duration of the investigation. Some wooded areas are also located along the Speed River.

There are several storm water outfalls in the general site area. The storm sewers discharge storm waters into the Speed River.

Land use surrounding the site is primarily residential and industrial. A new residential subdivision was partially completed in the area south of the site at the time of the investigation. The Guelph Dolome Quarry is situated just west of the site and the Bryden Apartments and Manor Motel are located north of the site.

Geology

Physiographically, the site is located within the region known as the "Guelph Drumlin Field". Within this area, there are approximately 300 drumlins of all sizes. The drumlins in this area are not closely spaced and there is intervening low lying grounds between the drumlins. This is for the reason that during the most recent Wisconsinan Glaciation period (approximately 12,000 years ago), the ice which moulded this drumlin field advanced from the southeast whilst the receding glacier moved perpendicular to this direction. As a result, the drainage of the ice front was directed to lower and lower outlets and hence the drumlin field is furrowed by parallel valleys running perpendicular to the trend of the drumlins. Along the sides of these valleys there are broad sand and gravel terraces.

As a result of the glacial activity, the general landform pattern consists of drumlins or groups of drumlins fringed by gravel terraces. The dominant soil materials are the unstratified, unsorted drumlin tills consisting of a heterogeneous mixture of gravels, sand and silts and the deep gravel terraces of the old meltwater spillways. Overburden in the site area is underlain by dolostones of the Amabel and Guelph Formations. Overburden thicknesses are shallow in the site and usually less than five (5) metres.

INVESTIGATION PROCEDURE

General

Soil and rock data and inherent properties were obtained by conducting both an in situ field investigation and laboratory analysis. Details of the field investigation and laboratory testing program are discussed below.

Field Investigation

The fieldwork for this project was conducted between 93 02 01 and 93 03 11 and consisted of a total of thirteen(13) boreholes. The boreholes were advanced to depths ranging from 2.1 m to 11.9 m. Two dynamic cone penetration tests advanced to depths of 4.9 m and 6.7 m were also conducted. Six (6) of the boreholes were advanced in conjunction with the proposed S-E/W Ramp structure foundation locations and two (2) of the boreholes were advanced at the proposed embankment fill locations in advance of this structure. A total of five(5) boreholes were advanced in conjunction with the proposed retaining wall.

Eleven of the boreholes were advanced on land using a conventional track mounted Central Mining Equipment (CME) 55 drilling unit. Hollow stem augering techniques were used to penetrate the overburden at these borehole locations.

Two boreholes were advanced offshore and within the Speed River utilizing a raft and a more portable, diamond drill unit. The diamond drill used was a skid mounted Boyles Bros. No. 1 unit that had a weight of approximately 700 kg. Conventional diamond drilling techniques that included washboring within rotary driven casing was used to advance the boreholes.

Disturbed subsoil samples were retrieved in the overburden using a 50 mm diameter split spoon sampler driven in accordance with the Standard Penetration Test (SPT-ASTM D1586). The samples were generally retrieved at 0.76 m intervals for the surficial 4.5 metres and at 1.5 m intervals thereafter. The track mounted unit employed an automatic hydraulic tripping hammer mechanism to impart the standard driving energy. Bedrock underlying the overburden was cored up to 3.1 metres in depth using conventional rock coring techniques. A NX core barrel within NW Casing was used in the coring process.

All subsoil samples were identified in the field and then properly sealed in plastic containers to preserve natural moisture contents in the soil. The samples were then transported to the laboratory where additional visual classifications were carried out and pertinent laboratory tests were conducted as described in the next section below.

Rock core samples were also identified in the field and physical properties were determined by visual examination and also by measurement of rock quality designations (RQD's) and rock core recovery. All rock core were placed in standard rock core boxes and carefully transported to the laboratory.

Groundwater levels were determined by monitoring the water levels in the open boreholes and the lake level was also monitored throughout the duration of the field investigation. All boreholes were backfilled upon completion of the fieldwork.

The survey related to the location and elevation of the individual boreholes was provided by Southwestern Region Surveys and Plans. A boat was required to determined the lake bottom and lake level elevations at the borehole locations off-shore.

Laboratory Analyses

All subsoil samples were carefully visually examined in the laboratory in accordance with the procedures outlined in the Visual Method described in Chapter 2 of the MTO Soil Classification Manual. The behaviour, gradation and natural moisture contents of the soil were determined by conducting the appropriate laboratory tests on representative samples.

Sample preparation and testing were conducted in accordance with the MTO Laboratory Testing Manual.

Detailed rock core logging was conducted in the laboratory by an in-house resident geologist. The rock core logging included descriptions of colour, grain size, bedding, jointing and strength.

Laboratory test results have been summarized below in the subsequent section of this report entitled "Subsurface Conditions" and are illustrated on the corresponding boreholes and figures included in the Appendix to this report.

SUBSURFACE CONDITIONS

General

The subsurface conditions across the site consist of a shallow native deposit comprised of a heterogeneous mixture of gravel, sand and silt underlain by dolostone bedrock. The heterogeneous mixture of gravel, sand and silt has a thickness ranging from 0.7 m to 4.4 m and is considered a glacial till deposit. Boulders and cobbles, which are characteristic of these types of deposits, were also encountered. The dolostone bedrock encountered were from two formations, the Guelph and the Amabel. The bedrock surface elevation varies from 299.3 m to 308.2 m.

As mentioned earlier, an embankment exists on the south river bank attributable to the placement of fill material placed and supposedly compacted over the native heterogeneous mixture of gravel, sand and silt. The fill material was placed up to a thickness of 5.0 metres and consists primarily of an irregular mixture of silt, sand and gravel. Traces of black organic inclusions are also present randomly within the fill material. The fill material overlies the native soil at the ramp structure location but the native soil thickness diminishes in a southerly direction and hence the fill material is directly underlain by bedrock at the southern limits of the proposed retaining wall.

Fill material also exists at the proposed north approach embankment location. The fill material, comprised of an irregular mixture of silt, sand and gravel with traces of organics and incinerated refuse is suspected to be part of the abandoned landfill. The thickness of this fill material is approximately 1.8 metres.

A plan of the site illustrating the locations and elevations of the boreholes is shown on Dwg. No. 5339104-A in the Appendix. A subsoil stratigraphical profile is also provided. The boundaries between the various soil types, in situ and laboratory test results as well as groundwater levels established at the time of investigation are shown on the stratigraphical profile and also on the individual Record of Borehole sheets in the Appendix.

SOIL/ROCK DESCRIPTIONS

Water

Approximately 0.8 to 0.9 m of standing water was present in the Speed River at the time of the investigation. The water had frozen, although not completely, at the time of the investigation. Some flow was evident within the middle of the river.

Irregular Mixture of Silt, Sand and Gravel (Fill Material)

Fill material comprised of an irregular mixture of silt, sand and gravel exists at the proposed south abutment, south pier, retaining wall and also at the proposed north approach embankment location. The thickness of the fill material ranges from 1.5 m to 5.0 m south of the river and has a thickness of approximately 1.8 metres at the north approach location. Traces of black organic material was evident within the predominantly brown material. Incinerated refuse material was also encountered at the north approach location.

A grain size distribution envelope as determined by hydrometer and mechanical sieve analysis that illustrates the gradation of the fill material is shown on Figure 1 in the Appendix. The envelope depicts a broad range of particle sizes ranging primarily from silt to gravel.

Atterberg Limit tests conducted on the fine grained portion (less than 425 micrometres) of some representative samples indicate that the fine grained portion ranges from a non-plastic silt to a plastic silt (ML). In general, however, it can be concluded that the material is cohesionless.

The 'N' values as determined by the Standard Penetration Test ranged from 3 blows/0.3 m to 60 blows/0.08 m. In general however, 'N'values were less than 30 blows/0.3 m indicating a state of denseness ranging from very loose to compact. The higher 'N' values are perhaps attributable to the larger gravel particle sizes within the fill material.

<u>Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till)</u>

Underlying the fill material on land and the water offshore, a native heterogeneous mixture of gravel, sand and silt is present across the site. This deposit of glacial till origin is unsorted and unstratified and as observed by samples retrieved in the water and by observation of the lake bottom and auger grinding inference, the deposit also contains boulders and cobbles. The deposit which varies in colour from brown to grey has a thickness ranging from 0.7 to 4.4 m. At some locations across the site, the surficial 1 to 1.5 metres of the deposit also contains traces of organics and hence is black in colour.

A grain size distribution envelope produced by mechanical sieve and hydrometer analysis is given on Figure 2 in the Appendix. The results reveal a broad range of particle sizes ranging primarily from silt to gravel. The envelope does not include particle sizes larger than gravel. The broad range of particle sizes is typical of deposits of glacial till origin.

The 'N'values as determined by the Standard Penetration Test range from as low as 2 blows/0.3 m to as high as 60 blows/0.03 m indicating a very loose to very dense state of denseness. The low 'N' values are the result of the organics in most cases and the larger 'N' values are the result of perhaps the natural denseness of the deposit and the larger gravel, cobble and boulder particle sizes.

Bedrock

The bedrock at the site underlies the heterogeneous mixture of gravel, sand and silt and consists of a "vuggy" fossiliferous dolostone of the Guelph and the underlying Eramosa Member of the Amabel Formation. The Guelph dolostone, the younger formation, was encountered at the boreholes within the water at an elevation ranging from 300.7 m to 302 m. Offshore and south of the river the Amabel dolostone was encountered at elevations ranging from 299.3 m to 299.8 m at the S-E/W ramp structure and along the proposed retaining wall, the bedrock surface elevation rises from Elevation 302.7 m to Elevation 308.2 m in a southerly direction. The bedrock surface varies from 301.4 m to 301.8 m north of the river. Up to 3 metres of NX rock core was retrieved at each of the individual boreholes advanced at the proposed structure foundation locations.

The dolostone bedrock is a chemical sedimentary rock that is medium grained. The rock is unweathered to slightly weathered and is featured by a porous "vug" texture and stylolites. The rock is very pale orange to yellowish brown (Guelph Formation) to dark yellowish brown (Amabel Formation) in colour and contains thin horizontal beds and very close to moderately close spaced vertical fractures. Detailed descriptions of the bedrock are attached in the Appendix in a report entitled "Description of Rock Core".

An assessment of the quality of strength of the rock was carried out by measuring core recoveries and Rock Quality Designations (RQD's) in the field and conducting physical index property tests. Recoveries were in the order of 95 to 100% and RQD's ranged from 0% to 66% indicating that the rock is of very poor to fair quality. Rock strengths can be described as medium strong for the Amabel Formation and weak for the Guelph Formation. Rock Core penetration rates were generally rapid which is indicative of the weaker nature of this sedimentary rock.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water levels in the open boreholes and monitoring the lake level throughout the duration of the field investigation. The river level was approximately 303.7 m at the time of the investigation and remained constant throughout the investigation. On shore and adjacent to the Speed River the water level appeared to reflect the river level and ranged from elevation 303.2 m to 303.9 m. Most of the boreholes advanced in conjunction with the proposed retaining wall further beyond the Speed River were dry upon completion.

Groundwater levels in general, are subject to seasonal fluctuations and hence can vary from the values given in this report.

DISCUSSION AND RECOMMENDATIONS

In conjunction with the proposed interchange at the Hanlon Expressway (Hwy. 6) and Wellington Street (Hwy. 24), it has been proposed to construct a three equal span beam-type structure that will carry the Hwy. 6 S to Hwy. 24 E/W ramp over the Speed River. The structure will be upstream of the existing Hwy. 6 NB/SB twin structures and is located immediately north of a former landfill site. In fact, the north approach for the structure will be situated within the abandoned rehabillitated landfill site. Two of the structure foundations are planned within the Speed River.

In addition, in view of the presence of a Hydro transmission tower located approximately 30 metres south of the ramp structure, a fifty(50) metre long retaining wall has been proposed to retain the approach embankment fill material. Twenty(20) metres of the retaining wall is proposed north of the northwestern leg of the existing tower and thirty(30) metres of the retaining wall is proposed south of this leg. The existing Hydro transmission tower is founded on concrete caissons.

The ramp structure width varies to a maximum 14.75 m increasing in the number of lanes from two to three in the northwardly direction. The structure also slopes downward at an approximate 5% gradient in a northwardly direction. The proposed profile grade therefore varies from approximately 315 m at the south approach to approximately 314 m at the north approach. The original ground is approximately 308 m and 304 m at the south and north approach respectively. Therefore, approximately seven (7) metres of approach embankment fill material will be required at the south approach and up to ten (10) metres of approach embankment fill material will be placed at the north approach. The proposed retaining wall will retain approximately seven(7) metres of fill material.

Dwg. 5339104-A in the Appendix illustrates a plan of the proposed structures and related structure foundations. The drawing also includes the proposed profile grade superimposed on a subsoil stratigraphical section.

Contained in the scope of this report are recommendations to facilitate the design and construction of the proposed structure foundations and related earth/rock works. Foundation and geotechnical recommendations are provided for:

1) Structure Foundations
   - S-E/W Ramp
   - Retaining Wall
2) Backfill to Structures
3) Approach Embankments
4) Construction Considerations

1) STRUCTURE FOUNDATIONS

S-E/W RAMP STRUCTURE

In view of the varying conditions at the site due to the existing fill material south of the river and at the north approach, the shallow overburden thickness at the proposed south pier, north pier and north abutment, and in consideration of the magnitude of the proposed approach embankment fills, it is recommended that the structure foundations be founded as summarized in Table 1 below.

Table 1 - Structure Foundation Scheme	
Structure	**Foundation Type**
South Abutment	Deep Foundations (Steel H-Piles or Concrete Caissons)
South Pier	Spread Footings
North Pier	Spread Footings
North Abutment	Deep Foundations (Steel H-Piles or Concrete Caissons)

Detailed recommendations for each structure foundation is given below.

<u>Deep Foundations</u>

<u>General</u>

In view of the existing fill material at these proposed structure foundation locations and the significant proposed approach embankment fill heights, it is recommended that both abutments be founded on end bearing deep foundation units consisting of either steel H-piles driven to the bedrock surface or alternatively concrete caissons augered and installed to bedrock. The most cost effective and technically feasible option shall be selected. Major consideration shall be given to employing an open abutment-type design because of its cost effectiveness. Abutments can be perched within the embankment fill material supported by the deep foundation units.

Driven Steel H-Piles

Both abutments can be founded on steel H-piles driven to the bedrock surface. For purposes of the O.H.B.D.C., the steel H-piles can be designed employing the axial capacities tabulated in Table 2 below.

Table 2 - Driven Steel H-Piles

Structure	Pile Type	Factored Capacity at U.L.S. (kN)	Axial Capacity at S.L.S. (kN)	Estimated Pile Tip Elevation (m)
South Abutment	HP310x110 HP310x79	1600 1150	1100 890	299.5 ±
North Abutment	HP310x110 HP310x79	1600 1150	1100 890	301.5 ±

To prevent pile installation impedment and difficulty, it is recommended that the maximum particle size of any newly placed fill be restricted to 75 mm. This specification shall be included in the contract documents as a Non-standard special provision (NSSP).

Axial capacities provided in Table 2 are for vertical piles only. Reductions of axial capacities for inclined loadings shall conform to factors provided in Section 6-8.3.4.3 of the O.H.B.D.C.

It is recommended that to facilitate the pile driving process, all piles be equipped with reinforced tips. Driving shoe details are given on OPSD 3301.00.

Pile spacing shall conform with Section 6-8.3.10 of the O.H.B.D.C. For centrally loaded piles equal load sharing on the deep foundation units can be assumed. The design of eccentric loaded deep foundation units shall comply with Section 6-8.3.4.2 of the O.H.B.D.C. All piles shall have a minimum three (3) metres edge distance from the crest of a slope such as that located at the south river bank.

The lateral resistance for both vertical and battered piles shall be computed in accordance with Section 6-8.3.8 of the O.H.B.D.C. Pertinent unfactored soil parameters to facilitate the design of vertical piles is given in Table 3 below. The corresponding soil depths can be obtained from the relevant borehole logs and having knowledge of the proposed heights of fill. In the computation of the lateral resistance within fill material, it is recommended that the upper 60% of the embedment length within the fill (taken from the frost penetration depth) shall be disgarded and to account for uncertainties in method of placement and material selection, it is recommended that the shear strength parameters be reduced by 10%.

Pile caps shall be protected against frost penetration by providing a minimum 1.2 m earth cover or equivalent frost protection.

Table 3 - Horizontal Resistance Design Parameters		
Soil	Angle of Internal Friction (ϕ)	Bulk Unit Weight (kN/m^3) (γ)
Irregular Mixture of Silt, Sand and Gravel (Fill Material)	30°	20
Het. Mixt. of Gravel, Sand and Silt	30°	20

Concrete Caissons

Alternatively, the abutments can be founded on concrete caissons augered and placed on bedrock at or below the elevations provided in Table 2. To facilitate the design of the concrete caissons, a vertical factored bearing capacity equivalent to 3500 kPa can be employed for the dolostone bedrock at the U.L.S. In view of the unyielding nature of the bedrock, the Serviceability Limit State (SLS) will not govern the design because the stresses required to induce detrimental settlements at the SLS will exceed the factored capacity at U.L.S.

The designer can use the bearing capacity provided to select the size of the caisson and the respective ultimate capacity. For instance, a 0.9 m diameter caisson will yield a capacity equivalent to approximately 2300 kN at U.L.S.

The lateral resistance for vertical or battered concrete caissons can be computed in accordance with Section 6-8.3.8 of the O.H.B.D.C. and using the data given in Table 3. Lateral resistance can be further augmented by socketing the caissons into the bedrock. The socket shall be a minimum 0.5 m and an unconfined compressive strength (qu) of 10 MPa can be used to compute the horizontal capacity of the caisson in the rock. Again, only the Ultimate Limit State (U.L.S) will govern the lateral capacity design.

Pile caps shall be protected against frost penetration by providing a minimum 1.2 m earth cover or equivalent frost protection.

Construction of the concrete caissons will require augering holes through the native heterogeneous mixture of gravel, sand and silt in most cases submerged beneath the groundwater table. As a result, special measures will be required to prevent the collapse of the shaft of the hole under the created condition of the unbalanced hydrostatic head. The construction of the caissons will be discussed under the title "Construction Considerations" later in this report.

Shallow Foundations

In view of the shallow overburden thickness at the proposed south pier and north pier locations, it is recommended that these piers be founded on conventional spread footings bearing on the competent dolostone bedrock. For purposes of the O.H.B.D.C., all footings founded on the dolostone bedrock can be designed as summarized in Table 4 below.

Table 4 - Spread Footings on Bedrock - Vertical Bearing Capacity			
Structure	Factored Capacity at U.L.S. (kPa)	Bearing Capacity at S.L.S. Type II (kPa)	Founding Elevation (m)
South Pier	1500	N/ A	302
North Pier	1500	N/ A	301

As indicated in Table 4 above, only the factored capacity at U.L.S. governs the design because of the unyielding nature of the bedrock. Stresses required to induce detrimental settlements at the Serviceability Limit State would exceed the factored capacity at U.L.S.

The capacities tabulated in Table 4 pertain to vertical normal loads only. Reductions of bearing capacities to account for inclined loadings shall conform to factors provided in Section 6-7.3.3.5 of the O.H.B.D.C.

The sliding resistance of the spread footings founded on the bedrock surface can be computed by employing an unfactored angle of friction of 30° between the concrete footing and the bedrock surface. Should additional horizontal resistance to sliding be required, shear keys or dowels can be incorporated. An unconfined compressive strength of 10 MPa and a bond stress of 500 kPa (between cement grout and bedrock) at U.L.S. are relevant shear key/ dowel design parameters within the dolostone bedrock. The lateral resistance of shallow foundations shall be computed in accordance with Section 6-7.3.3.2 of the O.H.B.D.C.

It is prudent that the footing base be protected against weathering during construction. To preserve the integrity of the bedrock surface during construction, it is recommended that a 100 mm thick lean mix concrete coating be placed on the footing bedrock surface within four(4) hours of exposure. Any previously weathered or loosened rock shall be removed prior to the placement of the concrete coating.

The construction procedure of the footings within the Speed River including the dewatering method will be discussed under the subheading "Construction Considerations" later in this report.

RETAINING WALL

General

Subsurface conditions at the proposed retaining wall are irregular and non-uniform and therefore special consideration must be given to the varying conditions. Both the ground surface and bedrock surface slopes upwardly in a southerly direction along the proposed retaining wall alignment. The bedrock is overlain by fill material consisting of an irregular mixture of silt, sand and gravel but its thickness decreases from approximately 5 metres to 1.5 metres. As a result, in the selection of a foundation scheme, it is imperative that these varying depths of fill material be considered. All foundations will be founded on bedrock but the selection of a shallow and/or deep foundation must be done such that the definitions of shallow and deep foundation are satisfied. A deep foundation is defined by a minimum D/B ratio (usually about 2 to 2.5) where D is the embedment depth and B is the foundation width or diameter. In addition, a minimum three(3) metre embedment is required to satisfy lateral rigidity requirements for a deep foundation. For D/B ratios less than 2 to 2.5, the foundation can be defined as a shallow foundation.

In view of the subsurface conditions and in consideration of the shallow/deep foundation definitions, it is recommended that the retaining wall be founded on:

1) deep foundation concrete caissons on bedrock

or

2) shallow foundations on bedrock (from Station 10+070 to 10+085), and
   deep foundation concrete caissons on bedrock (from Station 10+085 to 10+120)

The retaining wall design must satisfy a number of geotechnical related criteria and hence the following must be prevented:

1) Foundation Bearing Capacity Failure
2) Foundation Sliding Failure
3) Overturning Failure
4) Overall Instability

Foundation and geotechnical parameters to facilitate a safe retaining wall design is given below. The retaining wall must be designed to satisfy ultimate and serviceability limit states as described in Section 6-9 of the O.H.B.D.C.

The type of retaining wall can be a conventional reinforced concrete type of retaining wall or alternatively a reinforced earth or mechanically stabilized earth retaining wall. Should a reinforced earth wall be chosen, it is recommended that a non-standard special provision (NSSP) be included in the contract documents that outlines specifications for submission and design requirements, materials, construction and performance of the wall. A copy of the NSSP can be obtained from this office. The internal stability of the reinforced earth module will address the sliding and overturning design criteria mentioned above.

The type of retaining wall that proves to be the most encountered and technically feasible shall be selected.

1) **Foundation Bearing Capacity**

Concrete Caisson Option

The retaining wall can be supported by concrete caissons founded on bedrock. In view of the shallow overburden thickness between Station 10+070 and 10+085, caissons installed within this area must be augered and socketted into the bedrock such that the minimum embedment is achieved.

Recommendations for the founding elevations are summarized in Table 5 below.

Table 5 - Retaining Wall Caisson Founding Elevations	
Station	Foundation Elevation (m)*
10+070 to 10+085	305±
10+085 to 10+120	302.7±

*Minimum Embedment criteria must be satisfied.

Concrete caissons can be designed for vertical bearing capacity as previously described for the caisson option supporting some of the structure foundations for the S-E/W ramp structure. Frost protection criteria of 1.2 m earth cover or equivalent is also applicable for the design of the caissons caps.

It is anticipated that no major dewatering problems will arise during the construction of the caissons because of the fact that the groundwater table was below or at the bedrock surface. However, should the groundwater table rise within the fill, sloughing of the shaft of the augered hole due to unbalanced hydrostatic head conditions can occur. Caisson construction will be discussed under the title "Construction Considerations" later in this report.

Shallow Foundations/ Deep Foundation Combination Option

Alternatively, the retaining wall ca be supported on conventional spread footings founded on the bedrock surface between Stations 10+070 and 10+085 inclusive and on concrete caissons as described above beyond Station 10+085. The design of spread footings on the bedrock can be carried out using the parameters previously recommended for spread footings for the south and north piers (see table 4). The founding elevation can be "stepped" between 305.1 m and 308.2 m.

No dewatering problems are anticipated during the excavation and construction of the spread footings. The footing base shall be protected as previously described.

2) **Sliding**

Horizontal Capacities

The horizontal capacities of shallow foundations and concrete caissons can be determined as previously described for the foundations of the S-E/W ramp structure. An unfactored angle of friction of 30° between the concrete footing and the bedrock can be used for the shallow foundations and the lateral resistance for vertical or battered concrete caissons can be computed in accordance with Section 6-8.3.8 of the O.H.B.D.C.

Additional horizontal capacity can be achieved using shear keys or dowels for spread footings and socketting caissons in the rock. An unconfined rock compressive strength (qu) of 10 MPa can be used for the shear key/socket design and a bond stress of 500 kPa (between cement grout and bedrock) at U.L.S. can be used for any dowel design as previously discussed. A minimum 0.5 m socket is recommended. The lateral resistance of the caissons attributable to the overlying fill material shall be as previously described (see page 19).

Backfill, Drainage and Lateral Earth Pressure

Backfill, drainage and lateral earth pressures to the retaining wall shall be designed and constructed as described in the "Backfill to Structure" section below.

3) **Overturning**

Overturning failure involves rotation of the wall about its toe. Earth pressures to be used in the analysis can be computed as described in the "Backfill to Structure" section below. The retaining wall design must safeguard against overturning.

4) **Overall Stability**

In view of the cohesionless nature of the fill material present at the location of the proposed retaining wall, and the close proximity of the bedrock surface at some locations, there are no overall stability problems anticipated.

2) BACKFILL TO STRUCTURE

Material

It is recommended that Granular 'A' or Granular 'B' material be placed within a wedge behind the abutments and retaining wall bounded by a plane rising at 60° to the horizontal as shown in Figure 6-9.6.1 of the O.H.B.D.C. The application of granular material combined with weep holes in the abutment/retaining walls or pipe subdrains to drain any accumulation of water in the backfill will prevent hydrostatic pressure build-up.

Design parameters of the soil are given in Table 6 below. Computations of lateral earth pressure shall be in accordance with Section 6-6.1.2 of the O.H.B.D.C.

Table 6 - Backfill Properties

	Granular 'A'	Granular 'B'
Angle of Internal Friction (ϕ) (unfactored)	35°	30°
Unit Weight (kN/m^3), γ	22.8	21.2
*Coefficient of Active Earth Pressure (K_a)		
- S.L.S.	0.27	0.33
- U.L.S.	0.33	0.40
*Coefficient of Earth Pressure at Rest (K_o)		
- S.L.S.	0.43	0.50
- U.L.S.	0.50	0.58

*These earth pressure coefficients apply to horizontal backfill surfaces only. The appropriate consideration shall be given to account for sloping backfill. The coefficient of earth pressure at rest shall be applied for rigid and unyielding walls.

Backfill and Compaction

The backfill shall be placed in 300 mm lifts in accordance with OPSS 902 series and compacted to achieve the target maximum dry density as outlined in OPSS 501.07.08.

Heavy vibratory equipment should be avoided in the backfill construction adjacent to the structure. It is therefore recommended that hand compaction equipment be employed in backfilling behind the abutment and retaining wall within a lateral distance equal to the current height of fill above the wall footing, in order to minimize deflection or possible damage of the wall.

3) APPROACH EMBANKMENTS

General

As mentioned earlier, up to seven(7) metres and up to ten(10) metres of approach embankment fill material will be required at the south and north approach respectively. At the north approach location, an abandoned landfill site is present. A previous report produced by Gartner Lee Ltd. (see W.P. 186-89-00) who was retained by MTO to conduct both environmental and geotechnical investigations contain recommendations regarding the design and construction of the embankment fills placed at this location. This area was not explored during our more recent foundation investigation and hence recommendations for the design and construction of the north approach embankments in this area are not included in the scope of this report.

The design of embankments such as those proposed at the site must satisfy two major criteria:

(1) Stability

(2) Settlement

These criteria are discussed below. Embankment material and construction is also discussed.

(1) Stability

Global

In view of the cohesionless nature of the fill material and underlying heterogeneous mixture of gravel, sand and silt, there are no deep seated global stability problems anticipated for embankment slopes constructed at 2H:1V both in the transverse and longitudinal directions. It is essential, however, that any deleterious material be subexcavated prior to fill placement.

Internal

To preserve the internal stability of the proposed embankments and to avoid surficial slope failures, the following guidelines shall be followed:

1. Earth fills up to eight(8) metres in height shall be constructed at 2H:1V slopes or flatter.

2. Earth fills exceeding eight(8) metres as proposed at the north approach shall be constructed at 2H:1V slopes with a nominal two(2) metre midheight berm constructed with a 2% gradient towards the toe of the embankment to promote surface turnoff or alternatively constructed at 2.5H:1V slopes.

3. Embankment slopes adjacent to the Speed River shall be protected against the scouring and erosive water forces. A revetment system consisting of a 0.3 metre thick rip rap or gabion stone material as outlined in OPSS 1004.05.06 is recommended. A filter material between the rock protection and the base embankment material is also recommended. Specifications for the gradation and thickness of the filter material are dependent on the gradation of the base embankment material and can be provided by this office once the composition of the base material is known.

(2) Settlement

Settlements induced as a result of the applied embankment loading will be the result of the elastic compression of the native subsoil and as a result of settlements within the fill material itself including the fill material that already exists at the site. It is anticipated that approximately 50 mm of settlement attributable to the elastic compression of the existing fill and native soil at the south approach and approximately 25 mm of settlement within the native soil at the north abutment location will be realized. This settlement is expected to be elastic in nature and hence should be realized during or shortly after the construction period.

Settlements within the embankment fill material are also anticipated as the result of internal stress induced by the self weight of the material. It is anticipated that approximately 75 mm to 100 mm of settlement will occur within a ten(10) metre earth fill and 25 mm to 50 mm within the seven(7) metre earth fill embankment.

Settlements within the earth fill should occur almost instantaneously and hence should occur during or immediately following construction for a granual material. Settlements of cohesive fill embankments will be more time dependent and anticipated to be realized within a three(3) month time period following placement.

Embankment Construction

All organic material shall be excavated at the base of the proposed embankment fills prior to the construction of the embankment proper. Any subexcavation within the cohesionless organic surficial materials submerged below the prevailing water table as present at the north abutment location will necessitate a dewatering scheme. One method of achieving this subexcavation is to excavate an initial pilot trench within the central area of the planned area and sump pumping the drained water seepage from this trench. The excavation can then proceed laterally in sequence until the entire area is effectively drained. Excavation can then be advanced vertically if required. This dewatering scheme controls and contains any soil sloughing within the confines of the excavation. The contractor shall submit his proposal of subexcavation and dewatering for review by this office prior to construction. A NSSP should also be included in the contract documents that states that the excavation of any cohesionless material submerged below the groundwater table is susceptible to conditions of unbalanced hydrostatic head and hence can "boil" under these conditions.

Embankment fills shall be placed and compacted as specified in OPSS 206.07.07 and OPSS 501 series.

4) **CONSTRUCTION CONSIDERATIONS**

Pier Construction in the Speed River

A dewatering scheme will obviously be required to facilitate the construction of the south pier and north pier located within the Speed River. In view of the larger boulder and cobble sizes that comprise the native soil situated at lake bottom and overlying the bedrock, the driving of an interlocking steel sheet pile wall to form a water tight enclosure is not considered a practical alternative because of the impedment that would result. It is therefore recommended that temporary barriers or dams be constructed to divert the water flow from the structure foundations and then excavation of the overburden be advanced using a "box-type" scheme. Excavation can take place within the box. As the overburden is being excavated, the box or prefabricated enclosure can then be displaced until the bedrock surface is encountered. The bedrock surface-box interface can then be sealed effectively to prevent water inflow and additional pumping can the be carried out if required.

It is recommended that the Contractor submit dewatering scheme plans prior to construction. Our office can then assist in the review of these drawings to determine acceptability of the proposal.

Caisson Construction

As mentioned previously, caisson construction within augered holes penetrating the cohesionless irregular mixture of silt, sand and gravel or the heterogeneous mixture of gravel, sand and silt submerged below the groundwater table will require a dewatering system. This is for the reason that soil sloughing and cave-in will result due to the unbalanced hydrostatic head condition produced during construction. One method of controlling this condition is to use mud drilling and tremie techniques. In employing this technique the quality of the bentonite slurry (density, viscosity) should be kept under constant control to ensure that it performs satisfactorily.

Alternatively, the caisson can be constructed within a temporary steel liner installed at the appropriate depth to prevent cave-in of the cohesionless submerged soils. After, the liner has been cleaned out and the required reinforcing installed, the concrete should be placed in the dry. An overzealous rapid withdrawal of the temporary casing should be avoided to prevent the intrusion of soil in the concrete (necking). Conversely, the temporary liner should not be allowed to get stuck in partial set concrete.

The proposed method of caisson installation shall be in accordance with OPSS 903.07.03 and subject to review by this office. It is prudent that the contractor submit a caisson construction for approval as outlined in OPSS 902.04.01.

As mentioned earlier, it is recommended that a NSSP be included in the contract documents that states that the cohesionless irregular mixture of silt, sand and gravel and heterogeneous mixture of gravel, sand and silt submerged below the prevailing groundwater is subjected to conditions of unbalanced hydrostatic head and hence can boil.

Environmental Considerations

Instream and shoreline construction can increase sediment depositions to a waterbody. As stated in Gartner Lee Environmental Report and hereby reiterated special environmental construction precautions must be placed and implemented. A number of construction mitigative techniques have been given in their report and these guidelines should be carefully followed to control and reduce impacts.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer, utilizing equipment owned and operated by Malones Soil Samples and Master Soils Investigation Ltd. Logging of the rock core in the laboratory was carried out by D. Williams, Petrographer.

The project was carried out by T. Sangiuliano under the general supervision of P. Payer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed by P. Payer and approved by M. Devata, Chief Foundation Engineer.

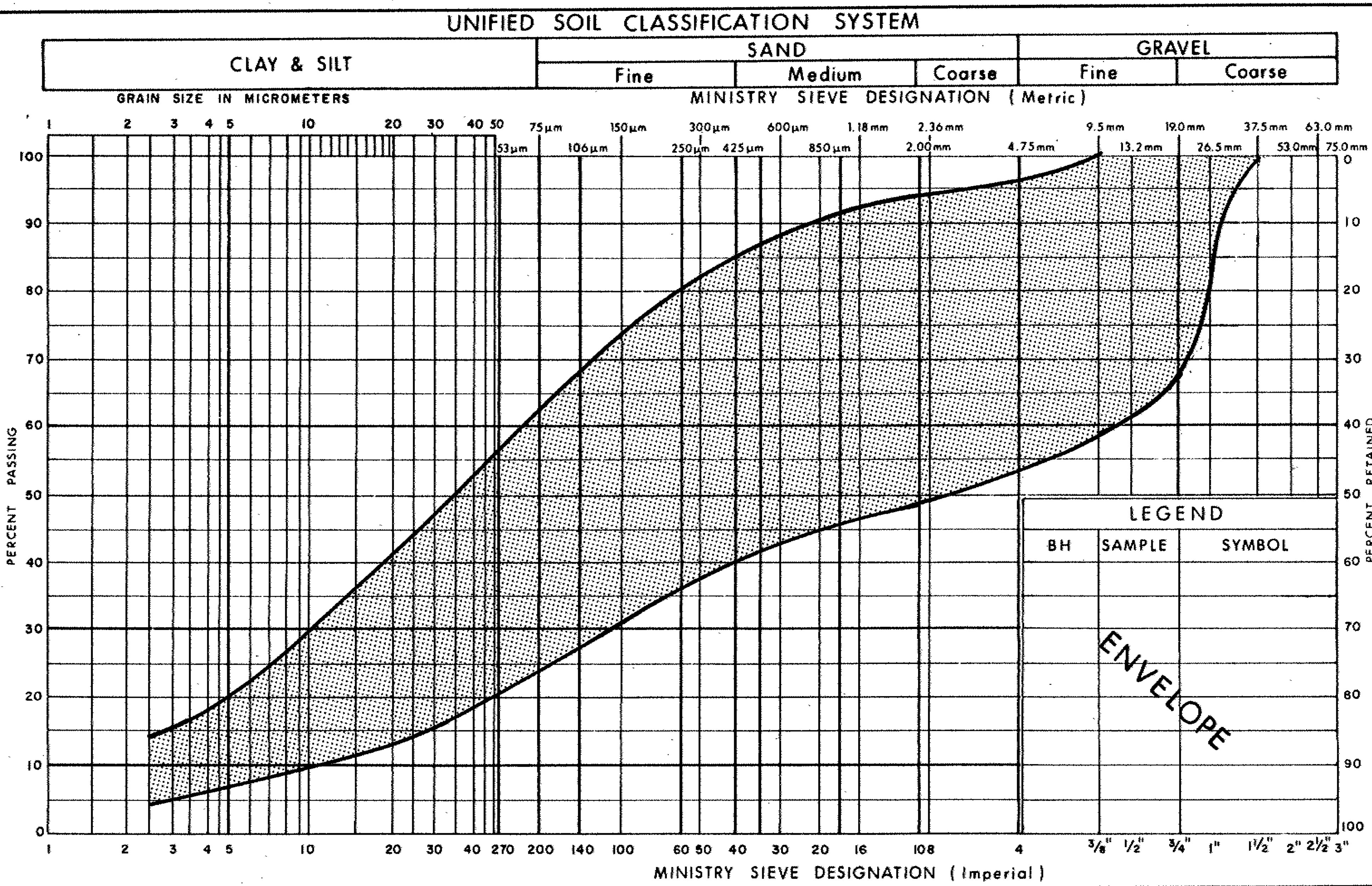
LICENSED PROFESSIONAL ENGINEER
A. J. Sangiuliano
PROVINCE OF ONTARIO

T. Sangiuliano, P.Eng.
Foundation Engineer

M. Devata, P.Eng.
Chief Foundation Engineer

APPENDIX

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

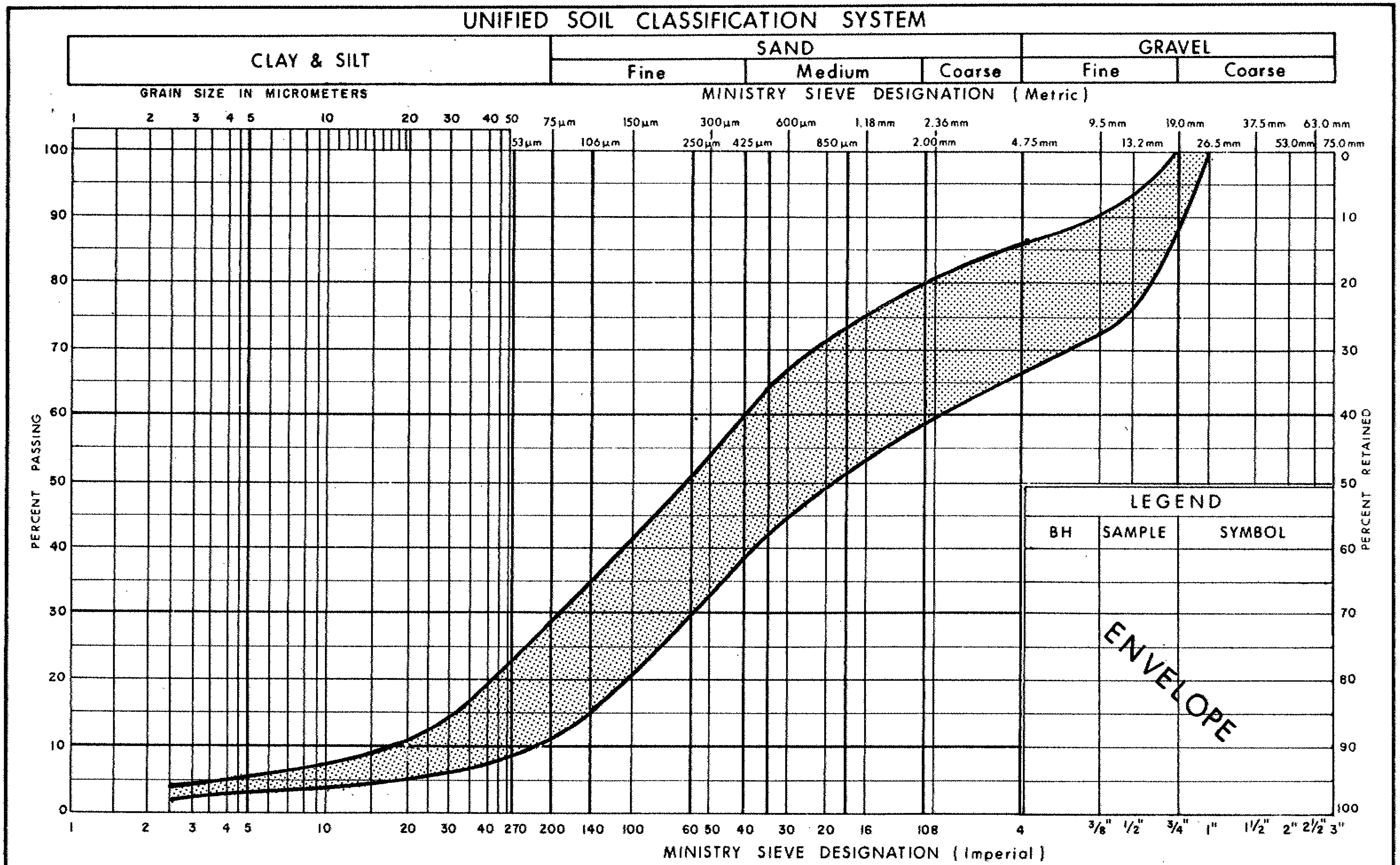
IRREGULAR MIXTURE OF

SILT, SAND & GRAVEL (FILL MATERIAL)

FIG No 1

W P 533-91-04

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

HETEROGENEOUS MIXTURE OF

GRAVEL, SAND & SILT (Glacial Till)

FIG No 2

W P 533-91-04

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. 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 475 J IMPACT ENERGY ON 'A' 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 (R Q D), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

SPACING	50mm	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	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_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^{-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
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 1

METRIC

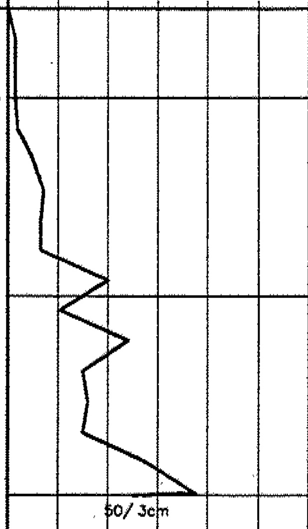
W.P. 533-91-04 LOCATION Co-ords: N 4 820 867.0; E 243 352.0 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core and Cone Test COMPILED BY TS

DATUM Geodetic DATE 93 02 01-02 CHECKED BY PP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
306.9	Ground Surface							
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material)	1	SS	4		306		
	Brown with traces of Black Organics	2	SS	4			o	
	Very Loose to Compact	3	SS	16				
303.7						304		
3.2	Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till) Brown	4	SS	78	▼*			
	Compact / V. Dense	5	SS	17				
		6	SS	74		302	o	22 50 25 3
		7	SS	60	/8cm			
						300		
299.3								
7.6	Dolostone Bedrock Weak to Medium Strong, Slightly Weathered to Unweathered	8	RC	REC 100%		298		RQD = 33%
		9	RC	REC 100%				RQD = 66%
296.2								
10.7	End of Borehole * 93 02 03							

DYNAMIC CONE PENETRATION RESISTANCE PLOT



SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)

WATER CONTENT (%): PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L (10 20 30)

UNIT WEIGHT γ kN/m^3

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

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 2

METRIC

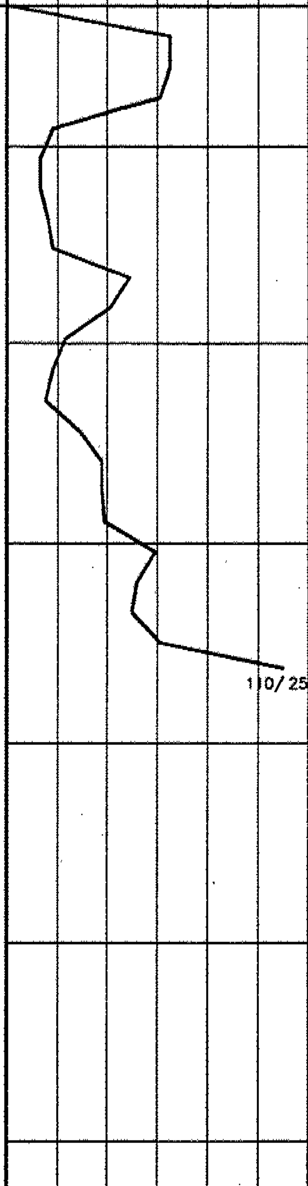
W.P. 533-91-04 LOCATION Co-ords: N 4 820 866.5; E 243 330.0 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core and Cone Test COMPILED BY TS

DATUM Geodetic DATE 93 02 02-03 CHECKED BY PP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) (w_P — w — w_L)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
307.4	Ground Surface								
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material) Compact to Very Dense	1	SS	35					
		2	SS	15		306			
		3	SS	35			o ├┤		22 31 35 12
	Brown / Brown with Black, trace Organics	4	SS	10		304			
		5	SS	60/8cm	* ▼				
302.8 / 4.6	Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till) Grey, Compact	6	SS	15		302			
		7	SS	23			o		14 66 18 2
299.8 / 7.6	Dolostone Bedrock Medium Strong, Slightly Weathered to Unweathered	8	SS	60/3cm		300			
		9	RC	REC 95%		298			RQD = 20%
		10	RC	REC 100%		296			RQD = 40%
295.5 / 11.9	End of Borehole								
	* 93 02 04								

DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)



+³, ×⁵ : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 3

 METRIC

W.P. 533-91-04 LOCATION Co-ords: N 4 820 887.5; E 243 322.5 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE NW Casing, NX Core COMPILED BY TS

DATUM Geodetic DATE 93 02 10 CHECKED BY PP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
303.7	Water Surface										
0.0	Water										
302.8 / 0.9	Heterogeneous Mixture of Cobbles, Gravel, Sand and Silt (Glacial Till) Brown, Very Dense										
302.0 / 1.7	Dolostone Bedrock Weak to Medium Strong, Slightly Weathered to Unweathered		1	SS	75 /13cm		302				
			2	RC	REC 100%						RQD = 19%
			3	RC	REC 100%		300				RQD = 27%
299.0 / 4.7	End of Borehole										

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

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 4

METRIC

W.P. 533-91-04 LOCATION Co-ords: N 4 820 907.0; E 243 301.5 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE NW Casing, NX Core COMPILED BY TS

DATUM Geodetic DATE 93 02 11 CHECKED BY PP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	'N' VALUES	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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
303.7	Water Surface										
0.0	Water										
302.9 / 0.8	trace Organics		1	SS	21						
	Compact										
	Very Dense		2	SS	85	/15cm	302		o		98 0 (2)
	Heterogeneous Mixture of Cobbles, Gravel, Sand and Silt (Glacial Till) Brown		3	SS	100						
300.7 / 3.0			4	SS	75	/10cm					
	Dolostone Bedrock Weak to Medium Strong, Slightly Weathered to Unweathered		5	RC	REC 100%		300				RQD = 30%
299.0 / 4.7	End of Borehole										

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

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 4A

1 OF 1 METRIC

W.P. 533-91-04 LOCATION Co-ords: N 4 820 898.0; E 243 289.0 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core COMPILED BY TS

DATUM Geodetic DATE 93 02 04 CHECKED BY PP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
304.1	Ground Surface							
0.0	Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till) Black, trace Organics, Very Loose	1	SS	2	*			
	Grey, Very Dense	2	SS	60	/15cm		o	28 60 10 2
301.8 / 2.3		3	SS	**		302		
	Dolostone Bedrock Weak to Medium Strong, Slightly Weathered to Unweathered	4	RC	REC 100%				RQD = 0%
		5	RC	REC 100%		300		RQD = 0%
298.8 / 5.3	End of Borehole							

* 93 02 05

** Sampler Bouncing

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 (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 5

METRIC

W.P. 533-91-04 LOCATION Co-ords: N 4 820 923.5; E 243 286.0 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core COMPILED BY TS

DATUM Geodetic DATE 93 02 05 CHECKED BY PP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
304.4	Ground Surface								
0.0	Black, trace Organics, Very Loose				▼*	304			
		1	SS	2					
	Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till) Brown, Compact to Very Dense	2	SS	21					
		3	SS	60 /15cm		302	~9		33 53 12 2
301.4 / 3.0	Dolostone Bedrock Weak to Medium Strong, Slightly Weathered to Unweathered	4	RC	REC 100%					RQD = 0%
						300			
		5	RC	REC 100%					RQD = 16%
298.3 / 6.1	End of Borehole								
	* 93 02 08								

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 (%) 10 20 30

+3, ×5: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 6

METRIC

W.P. 533-91-04 LOCATION Co-ords: N 4 820 939.5; E 243 273.0 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE HS Auger COMPILED BY TS

DATUM Geodetic DATE 93 02 05 CHECKED BY PP

ELEV DEPTH	SOIL PROFILE 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 (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
304.8	Ground Surface										
0.0	Irregular Mixture of Silt, Sand and Gravel with traces of Organics and Incinerated Refuse (Fill Material)		1	SS	3	▼*	304				
303.0	Black, Very Loose to Compact		2	SS	12						
1.8	Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till)										
302.3	Grey, Very Dense										
2.5	End of Borehole Auger Refusal (Probable Bedrock) * 93 02 08										

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

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

RECORD OF BOREHOLE No 7

METRIC

W.P. 533-91-04 LOCATION Co-ords: N 4 820 849.0; E 243 370.0 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE HS Auger COMPILED BY TS

DATUM Geodetic DATE 93 02 03 CHECKED BY PP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
308.0	Ground Surface										
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material)										
	Brown to Black, trace Organics		1	SS	11		306				
	Loose to Compact		2	SS	25				o		47 30 19 4
						▼*	304				
303.0			3	SS	8						
5.0	Heterogenous Mixture of Gravel, Sand and Silt (Glacial Till)		4	SS	60	/4cm	302				
300.4	Brown, Very Dense										
7.6	End of Borehole Auger Refusal (Probable Bedrock)										
	* 93 02 03										

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

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

RECORD OF BOREHOLE No 8

 METRIC

W.P. 533-91-04 LOCATION Co-ords: N 4 820 865.9; E 243 369.6 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core COMPILED BY TS

DATUM Geodetic DATE 93 03 04 CHECKED BY PP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) w_P – w – w_L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
306.9	Ground Surface								
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material) Brown and Black, trace Organics Loose to Compact					306			
		1	SS	5			plotted		4 34 49 13
		2	SS	6		304			
		3	SS	14					
302.7		4	SS	36					
4.2	Slightly Weathered / Unweathered Dolostone Bedrock Weak	5	SS	60/10cm	* ▼	302			
		6	SS	60/3cm					
300.0		7	RC	REC 100%					RQD = 8%
6.9	End of Borehole								

* 93 03 05

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

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

$15 \overset{20}{\underset{10}{\diamond}} 5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 8A

METRIC

W.P. 533-91-04 LOCATION Co-ords: N 4 820 851.5; E 243 384.3 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE HS Auger COMPILED BY TS

DATUM Geodetic DATE 93 03 04 CHECKED BY PP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	SAMPLE TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
308.2	Ground Surface										
0.0	Irregular Mixture of Silt, Sand and Gravel with random Zones of Clayey Silt (Fill Material)					DRY *	308				
			1	SS	4						
			2	SS	5						
	Dolostone fragments		3	SS	61		306				
	Brown with Black, trace Organics		4	SS	12						
	Very Loose to Compact		5	SS	21		304				
303.2			6	SS	66 **						
5.0	End of Borehole (Auger Refusal – Probable Bedrock)										

* 93 03 05

** Sampler Bouncing (Probable Bedrock)

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

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 9

METRIC

W.P. 533-91-04
LOCATION Co-ords: N 4 820 834.2; E 243 415.9
ORIGINATED BY TS

DIST 3 HWY 6
BOREHOLE TYPE HS Auger, NW Casing, NX Core
COMPILED BY TS

DATUM Geodetic
DATE 93 03 04
CHECKED BY PP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
309.7	Ground Surface						
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material) Brown with Black, trace Organics Loose				DRY *		
308.2 1.5	Dolostone Bedrock Weak, Unweathered to Slightly Weathered	1	SS	65	/15cm	308	
		2	SS	60	/5cm		
		3	RC	REC 100%		306	RQD = 53%
305.6 4.1	End of Borehole						
	* 93 03 05						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; 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 (%) 10 20 30; UNIT WEIGHT γ kN/m³; GR SA SI CL

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

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

RECORD OF BOREHOLE No 9A

METRIC

W.P. 533-91-04 LOCATION Co-ords: N 4 820 840.3; E 243 397.3 ORIGINATED BY TS

DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core COMPILED BY TS

DATUM Geodetic DATE 93 03 04 CHECKED BY PP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
308.5	Ground Surface						
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material) Brown with Black trace Organics Loose to Compact				DRY *	308	
		1	SS	10			
		2	SS	10			
		3	SS	18		306	37 29 25 9
305.1		4	SS	65			
305.0	**						
3.5	End of Borehole (Auger Refusal)						

* 93 03 05

** Dolostone Bedrock

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; 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 (%) 10 20 30; UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No 10

METRIC

W.P. 533-91-04
LOCATION Co-ords: N 4 820 843.3;E 243 405.2
ORIGINATED BY TS
DIST 3 HWY 6
BOREHOLE TYPE SS Auger
COMPILED BY TS
DATUM Geodetic
DATE 93 03 04
CHECKED BY PP

SOIL PROFILE			SAMPLES			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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
308.7	Ground Surface										
0.0 / 306.6	Irregular Mixture of Silt, Sand and Gravel (Fill Material) Brown with Block, Trace Organics					DRY *	308				
2.1	End of Borehole (Auger Refusal – Probable Bedrock) * 93 03 04										

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

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

ROCK CORE DESCRIPTION
WP 533-91-04

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1	8 9	7.62-9.14 9.14-10.67	100 100	33 66	7.62-10.67	**DOLOSTONE** (with stylolites, abundant small vugs, and larger vugs up to 5 cm in diameter, commonly containing calcite crystals), very pale orange to pale yellowish brown (7.62-8.59 m) and dark yellowish brown to pale yellowish brown (8.59-10.67 m); medium grained; weak (7.62-8.59 m) to medium strong (8.59-10.67 m); unweathered to slightly weathered; fractures moderate to very close spaced, flat to dipping, undulating to planar, smooth to rough.
2	9 10	8.84-10.36 10.36-11.89	95 100	20 40	8.84-11.89	**DOLOSTONE** (with abundant small vugs and larger vugs up to 5 cm in diameter, commonly containing calcite crystals), dark yellowish brown to pale yellowish brown; medium grained; medium strong; unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.
3	2 3	1.96-3.18 3.18-4.70	100 100	19 27	1.96-4.70	**DOLOSTONE** (with stylolites, abundant small vugs, and larger vugs up to 3 cm in diameter), very pale orange to pale yellowish brown (1.96-4.44 m) and dark yellowish brown to pale yellowish brown (4.44-4.70 m); medium grained; weak (1.96-4.44 m) to medium strong (4.44-4.70 m); unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section

ROCK CORE DESCRIPTION
WP 533-91-04

	CORE RECOVERY				CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
4	5	2.44-3.96	100	30	2.44-3.96	**DOLOSTONE** (with stylolites, abundant small vugs, and larger vugs up to at least 4 cm in diameter, commonly containing calcite crystals), very pale orange to pale yellowish brown (2.44-3.51 m) and dark yellowish brown to pale yellowish brown (3.51-3.96 m); medium grained; weak (2.44-3.51 m) to medium strong (3.51-3.96 m); unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.
4A	4 5	2.29-3.81 3.81-5.33	100 100	0 0	2.29-5.33	**DOLOSTONE** (with stylolites, abundant small vugs, and larger vugs up to at least 7 cm in diameter, commonly containing calcite and sphalerite crystals), very pale orange to pale yellowish brown (2.29-3.10 m) and dark yellowish brown to pale yellowish brown (3.10-5.33 m); medium grained; weak (2.29-3.10 m) to medium strong (3.10-5.33 m); unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.
5	4 5	3.05-4.57 4.57-6.10	100 100	0 16	3.05-6.10	**DOLOSTONE** (with stylolites, abundant small vugs, and larger vugs up to at least 7 cm in diameter, commonly containing calcite crystals), very pale orange to pale yellowish brown (3.05-4.27 m) and dark yellowish brown to pale yellowish brown (4.27-6.10 m); medium grained; weak (3.05-4.27 m) to medium strong (4.27-6.10 m); unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section

ROCK CORE DESCRIPTION
WP 533-91-04

	CORE RECOVERY				CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
8	7	5.36-6.88	100	8	5.36-6.88	**DOLOSTONE** (with stylolites and abundant small vugs), very pale orange to pale yellowish brown; medium grained; weak; unweathered to slightly weathered (moderately weathered, 5.36-5.56 m); fractures close to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.
9	3	2.59-4.11	100	53	2.59-4.11	**DOLOSTONE** (with stylolites, abundant small vugs, and some larger vugs up to 4 cm in diameter, commonly containing calcite crystals), very pale orange to pale yellowish brown; medium grained; weak; unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

Note: Depths are approximated where core recovery is less than 100%
Logged by: DAW, Soils and Aggregates Section

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

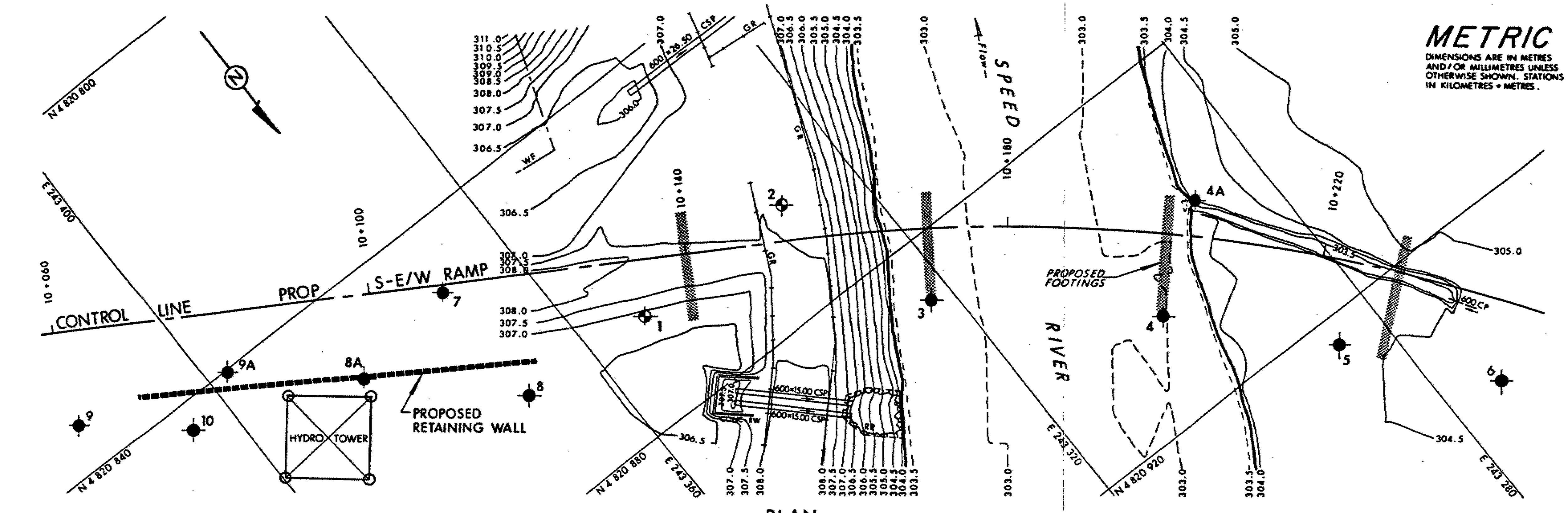
CONT No

WP No 533-91-04

SPEED RIV & PROP S-E/W RAMP (INTERCHANGE AT HWY 6 & WELLINGTON ST)

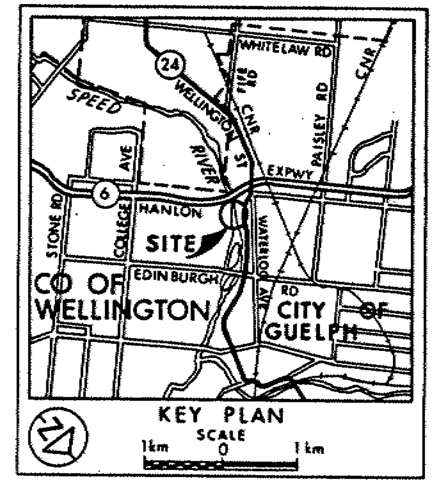
BORE HOLE LOCATIONS & SOIL STRATA

SHEET



PLAN

SCALE 6m 0 6m



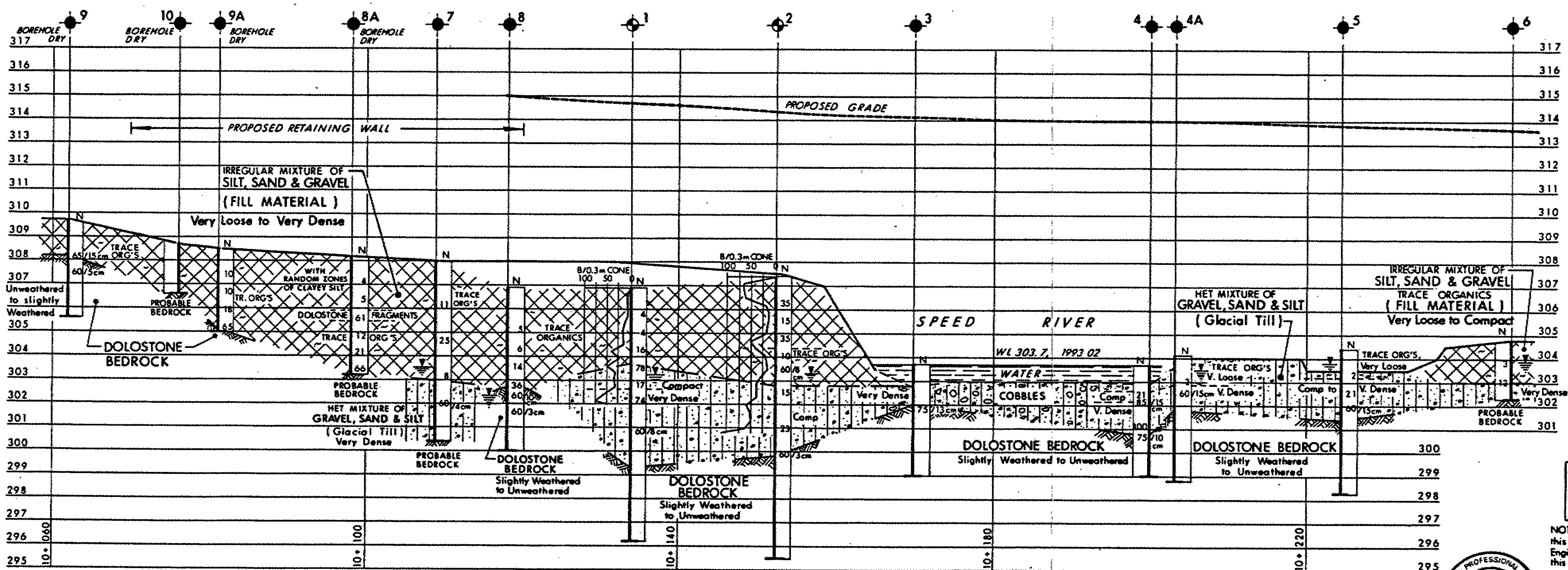
KEY PLAN

SCALE 1km 0 1km

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)
- WL at time of investigation 1993 02 and 03

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	306.9	4 820 867.0	243 352.0
2	307.4	4 820 866.5	243 330.0
3	303.7	4 820 887.5	243 322.5
4	303.7	4 820 907.0	243 301.5
4 A	304.1	4 820 898.0	243 289.0
5	304.4	4 820 923.5	243 286.0
6	304.8	4 820 939.5	243 273.0
7	308.0	4 820 849.0	243 370.0
8	306.9	4 820 865.9	243 369.6
8 A	308.2	4 820 851.5	243 384.3
9	309.7	4 820 834.2	243 415.9
9 A	308.5	4 820 840.3	243 397.3
10	308.7	4 820 843.3	243 405.2



PROFILE CONTROL LINE PROP S-E/W RAMP

SCALES 6m 3 0 6m HOR; 2m 1 0 2m VERT

NOTE

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen.Cond.

REV.	DATE	BY	DESCRIPTION

Geocres No 40P9-31

REF No E-153-6-3; 1992 06

HWY No 6 (HANLON EXPWY)			DIST 3
SUBM'D T S	CHECKED TS	DATE 1993 07 23	SITE 35-579
DRAWN R S	CHECKED	APPROVED	DWG 5339104-A