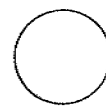


REMARKS: _____

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST. 3
CONT No 96-37
WP No 533-91-03

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



GENERAL NOTES

CLASS OF CONCRETE (UNLESS OTHERWISE NOTED)	30 MPa
CLEAR COVER TO REINFORCING STEEL	
FOOTINGS	100 ± 25
ABUTMENTS & WINGWALLS	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

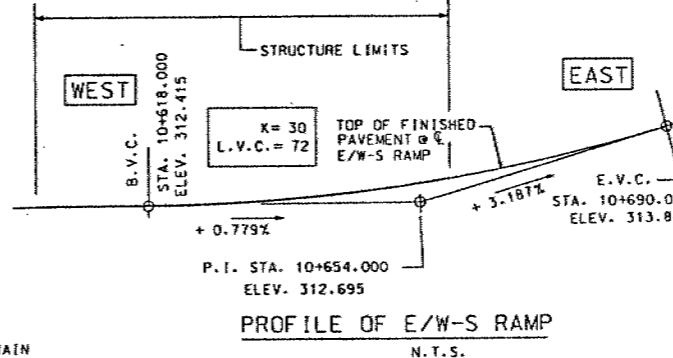
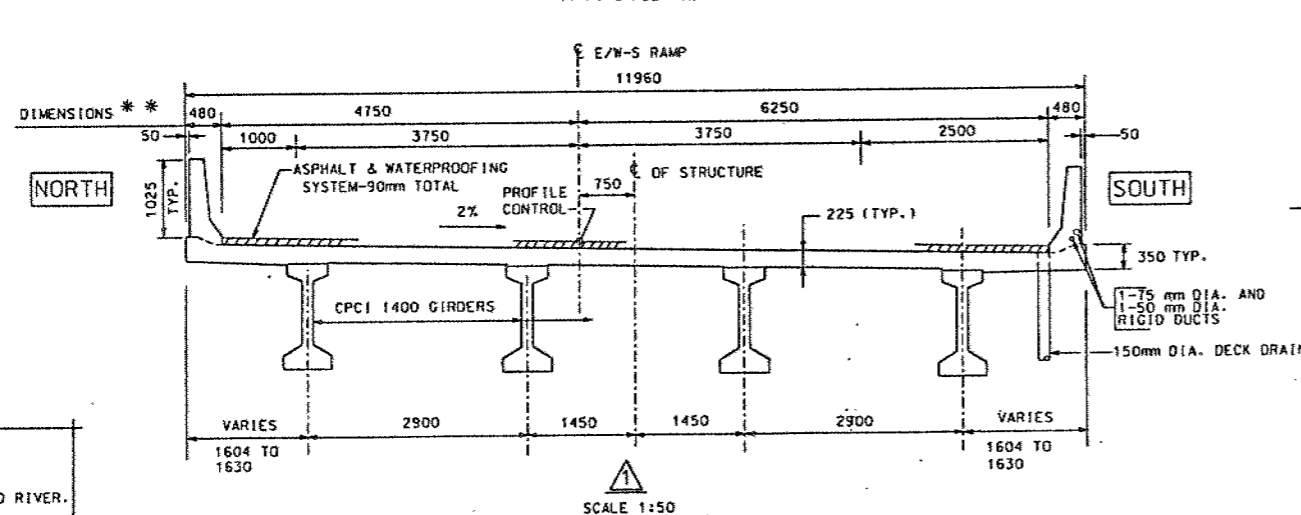
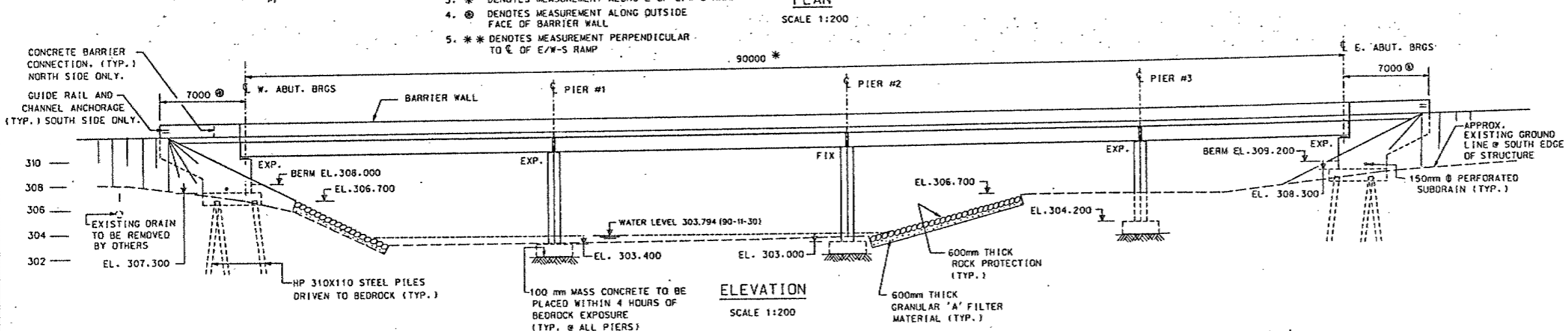
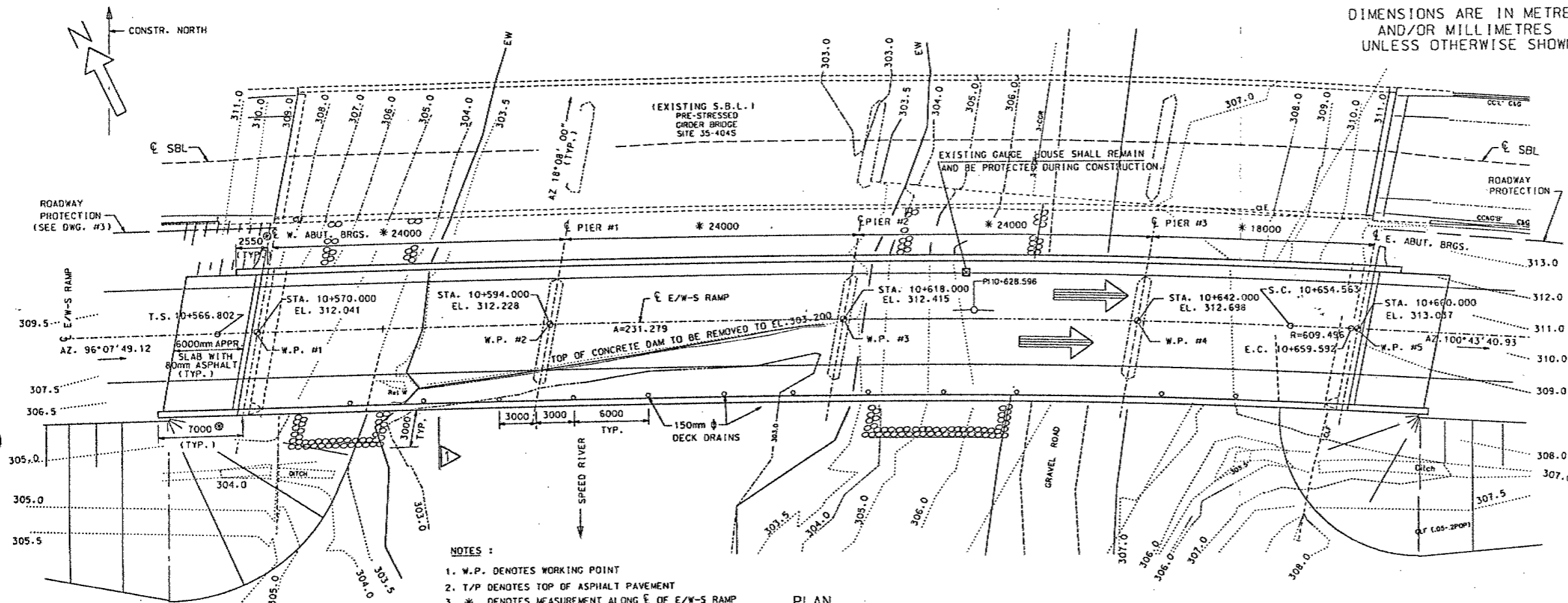
1. THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
2. FOR SUBSTRUCTURE CONSTRUCTION SEQUENCE, REFER ALSO TO THE CONTRACT DRAWINGS OF W.P. 604-90-01/02, "SPEED RIVER BRIDGE NBL & SBL REMEDIATION", SITE 35-404. DRAWING #2 OF THE CONTRACT GIVES "CONSTRUCTION SEQUENCE AND SCOPE OF WORK" THAT OUTLINES THE SEQUENCE THAT SHALL BE FOLLOWED.

LIST OF DRAWINGS

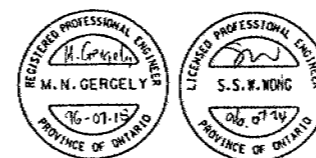
1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS & SOIL STRATA
3. ROADWAY PROTECTION
4. FOUNDATION LAYOUT
5. FOOTING REINFORCEMENT
6. WEST ABUTMENT
7. EAST ABUTMENT
8. WINGWALLS
9. PIERS
10. PRESTRESSED GIRDERS & BEARINGS
11. DECK DETAILS
12. DECK REINFORCEMENT
13. JOINT ANCHORAGE & ARMOURING
14. BARRIER WALL W/O RAILING
15. 6000 mm APPROACH SLAB
16. STANDARD DETAILS
17. QUANTITIES - STRUCTURE I
18. QUANTITIES - STRUCTURE II

APPLICABLE STANDARD DRAWINGS

OP50 - 3501.00 GRANULAR BACKFILL REQUIREMENTS
OP50 - 918.01 CONCRETE BARRIER TRANSITION TO STRUCTURE
OP50 - 4010.00 GUIDE RAIL & CHANNEL ANCHORAGE



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



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DESCRIPTION	DATE
1	DESIGN M.G. CHK S.W. CODE 08BDC-91	MAY 1994
2	DRAWN J.E. CHK M.G. SITE 35-578 STRUCT	
3	SCHEME	
4	DWG 1	

FILE COPY



Ministry
of
Transportation

FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

CONT 96-37

WP 533-91-03 DIST 3

HWY 6 & 24 STR SITE 35-578

Proposed Crossing at
E/W-S Ramp and Speed River

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FOUNDATION INVESTIGATION REPORT
FOR
PROPOSED CROSSING AT
E/W-S RAMP AND SPEED RIVER
WP 533-91-03, SITE 35-578
HWY 6 & HWY 24 - DISTRICT 3, STRATFORD

INTRODUCTION

This report summarizes the results of a Foundation Investigation conducted at the proposed Hwy 24 E/W - Hwy 6 south ramp structure in the City of Guelph, District 3 (Stratford), within the Southwestern Region. The structure is proposed to cross the Speed River and is one of many structures planned in conjunction with the revamped Hanlon Expressway (Hwy 6) and Wellington Street (Hwy 24) interchange. The scope of this report is limited to the structure itself and immediate approaches.

SITE DESCRIPTION AND GEOLOGY

The site is located at the Speed River immediately adjacent and downstream of the existing Hwy 6 Southbound River Crossing. The site location is situated approximately 0.5 km south of the existing Hwy 24 in the City of Guelph, County of Wellington. The forward and transverse embankment fill slopes of the existing Hwy 6 Southbound structure bound the northern limits of the site. The

structure and the adjacent existing northbound structure are reinforced concrete four (4) span structures constructed in 1971. The Guelph Dolime rock quarry is located southeast of the site. Significant earth and rock excavation is evident at the quarry and stockpiles of processed dolostone is indicative of the operation. A woodlot containing tall deciduous trees is located immediately northwest of the site.

The Speed River flowing in a southerly direction is naturally the dominant feature at the site. At the site location, the river is approximately thirty-five metres in width. Embankment slopes exist on either side of the Speed River downstream of the proposed structure. The embankment slopes which are approximately three (3) metres in height are approximately 1.5H:1V to 2H:1V and covered with trees and low lying shrubs. At the crest of the eastern embankment, a gravel roadway exists. This roadway and a bridge located downstream permit access to the site south of the River. A concrete weir is presently located in the water at the proposed structure location.

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 Wisconsinian Glaciation period (approximately 12,000 years ago), the ice which moulded this 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.

Therefore, the general landform pattern consists of drumlins or groups of drumlins fringed by gravel terraces. As a result, 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. Native overburden thickness are shallow at the site and usually less than four (4) metres.

INVESTIGATION PROCEDURE

General

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

Field Investigation

The fieldwork for this project was conducted between 93 02 05 and 93 02 16 and consisted of a total of twelve (12) boreholes. The boreholes were advanced to depths ranging from 3.8 to 12.3m. Nine (9) of the twelve (12) boreholes were advanced in conjunction with structure foundation locations whereas three (3) of the boreholes were advanced at the proposed embankment fill locations.

Nine of the boreholes were advanced on land using conventional truck and track mounted Central Mining Equipment (CME) 55 drilling units. Hollow stem augering techniques were used to penetrate the overburden at the site. Traffic protection provided by the nearest Patrol (#13) was required on the Hwy 6 Southbound lane to advance four of these boreholes.

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 drilled casing was used to advance the boreholes.

Disturbed subsoil samples were retrieved in the overburden using a 50mm diameter split spoon sampler driven in accordance with the Standard Penetration Test (SPT-ASTM D1586). The samples were generally retrieved at 0.76 intervals for the surficial 4.5 metres and at 1.5m intervals thereafter at the structure foundation locations. At the approach embankment locations, samples were retrieved at 1.5m intervals. The track/truck mounted units 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 NXL 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 index 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 determine the lake bottom and lake level elevation at the boreholes located offshore.

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 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 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 at the site reflect the previous placement of fill material and hence consists of an irregular mixture of silt to clayey silt, sand and gravel at the proposed abutment locations and also at the east pier location. The thickness of this fill material varies up to 10.7 metres. At the locations beyond the fill placement, the native surficial deposit consists of a heterogeneous mixture of gravel, sand and silt. This deposit underlies the fill material where the latter exists and underlies the waters of the Speed River at the proposed west and centre piers. The deposit also contains boulders and cobbles which were explored and also visible at the river bottom. The thickness of this deposit varies from 0.6m to 4.6m.

The native heterogeneous mixture of gravel, sand and silt is underlain by dolostone bedrock. The bedrock was encountered at Elevations ranging from 299.8m

to 303.9 indicating an irregular bedrock surface.

A plan of the site illustrating the locations and elevations of the boreholes and proposed structure foundation locations is shown on Dwg. No. 5339103-A in the Appendix. A profile of the E/W-S ramp and a number of stratigraphical sections at the proposed structure foundation locations are 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 sections and also on the individual Record of Boreholes sheets in the Appendix. A detailed description of the subsurface conditions are given below.

SOIL/ROCK DESCRIPTIONS

Water

Approximately 0.4m to 0.9m of standing water was present in the Speed River at the time of the investigation. The greater depths of water existed upstream of the concrete weir and water velocities produced turbulent flow immediately downstream of the concrete weir.

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

The composition of the fill material present at the site is predominately an irregular mixture of brown silt, sand and gravel. However, at various locations, random zones of an irregular mixture of clayey silt, sand and gravel were also encountered. The thickness of the fill material varies up to 10.7 metres with the greatest thickness encountered at the crest of the approach embankment slope. Boulders and cobbles as inferred by auger grinding are also randomly present

within the fill material. A grain size distribution envelope as determined by hydrometer and mechanical sieve analysis and shown on Figure 1 in the Appendix illustrates the broad gradation of the fill material.

Figure 2 in the Appendix shows the results of Atterberg limit tests conducted on the fine grained portion of the fill material (less than 425 micrometres). The results reveal that liquid limits (w_L %) range from 17% to 24% and the plasticity index (I_p %) ranges from 4% to 8%. Based on these results, it can be concluded that at some random locations, the fine grained portion of the fill material contains sufficient clay fractions to exhibit a low plasticity and hence can be categorized as a plastic silt (ML) to a clayey silt (CL). However, as Figure 1 illustrates, silt, sand and gravel percentages predominant and hence the fill material exhibits generally a cohesionless non-plastic behaviour.

The "N" values as determined by the Standard Penetration Test ranged from 2 blows/0.3m to 70 blows/0.28m indicating a denseness ranging from very loose to very dense. In general, however, "N" values ranged between 5 blows/0.3m to 30 blows/0.3m representing a loose to compact state of denseness. The cohesive fill material can be described as soft to firm as indicated by the "N" values.

Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till)

Underlying the fill material and water, and also occurring surficially at some locations across the site, a heterogeneous mixture of gravel, sand and silt exists. This deposit, which is predominantly brown in colour also contains a surficial black crust up to 1.5 metres in thickness at some locations. Boulders and cobbles are also present in this deposit as determined by actual sampling,

visual observation in the Speed River and frequent auger grinding that was encountered. The thickness of this deposit ranges from 0.6 metres to 4.6 metres.

A grain size distribution envelope produced by mechanical sieve and hydrometer analysis is given in Figure 3 in the Appendix. The results reveal a broad range of particle sizes ranging primarily from silt to gravel. (Note: the envelope does not include particle sizes larger than gravel). The broad range of particle sizes is typical of deposits of glacial till origin.

An Atterberg Limit test was conducted on a representative sample of the black soil which occurs at random locations (See BH 8, SS5) across the site. The test results reveals a liquid limit (w_L) of 53% and plasticity index of approximately 10%. Based on this result, the material can be categorized as an organic silt (MH).

The "N" values as determined by the Standard Penetration Test range from 8 blows/0.3m to 60 blows/0.3m. The larger "N" values may be the result of the larger boulder and cobble sizes and hence may not necessarily accurately represent the denseness of the deposit. The "N" values tend to generally range between 10 blows/0.3 and 50 blows/0.3m indicating a compact to dense state of denseness.

Bedrock

The bedrock that underlies the heterogeneous mixture of gravel, sand and silt at the site is primarily a light coloured dolostone of the Guelph formation. However, some darker coloured dolostone of the Amabel Formation was also

encountered underlying the Guelph Formation at the proposed east pier location. Bedrock surface elevations varied across the site and are summarized in Table 1 below.

TABLE 1 - BEDROCK SURFACE ELEVATIONS	
AREA	ELEVATION (m)
West Abutment/Approach	299.8 - 301.5
Particle Size Analysis <i>W.P.R.</i>	302.6 - 302.9
Natural Moisture Contents <i>F.P.R.</i>	303.1 - 303.9
Bulk Unit Weights <i>F.P.R.</i>	301.8 - 302.2

The dolostone bedrock is a chemical sedimentary rock that is medium grained. The rock is unweathered to slightly weathered and featured by a porous "vug" texture and stylolites. The rock is very pale orange to yellowish brown (Guelph Formation) and is light-grey 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 and 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 ranged from 67% to 100% and RQD's ranged from 0% to 88%. In general, however, recoveries exceeded 90% and RQD's ranged from approximately 25% to 60% indicating that the rock is of poor

to fair quality.

Rock strengths can be described as weak for the Guelph formation and medium strong for the Amabel Formation. Rock core penetration were generally rapid which is indicative of the weaker nature of the 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 at elevation 303.7m at the time of the investigation and remained constant throughout the duration of this investigation.

On shore, the water level ranged from elevation 301.9m to 304.2m which is approximately equal to the elevation of the water level.

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

DISCUSSION AND RECOMMENDATIONS

In conjunction with the proposed construction of the interchange at the Hanlon Expressway (Hwy. 6) and Wellington Street (Hwy 24), it has been proposed to construct a 4-span beam-type structure (approximately 24, 24, 24, 18 metre west to east spans) that will carry Hwy 6 SBL traffic from Hwy 24 over the Speed River. The structure will be positioned immediately adjacent and downstream of the existing Hwy SBL structure and in fact all proposed structure foundations will align with that of the existing structure. A plan illustrating the location of the proposed structure and related structure foundations is shown on Dwg 5339103-A in the Appendix.

The proposed structure width is eleven (11) metres and includes two lanes of 3.75m width and a shoulder. The structure also slopes downward at a 2% gradient in a southwardly direction.

The proposed profile grade varies from approximately 311.5m to 313.5m increasing in the easterly direction. The existing ground surface varies from 311.6m to 304.0m at the west approach and from 307.6m to 313.3m at the east approach. Therefore, a maximum height of approach fill required at the west and east approach embankment is approximately seven (7) metres.

Contained in the purview of the report are recommendations to facilitate the design and construction of the proposed structure foundations and related earthworks. Foundation and geotechnical recommendations provided include:

- 1) Structure Foundations

- 2) Backfill to Structure
- 3) Approach Embankments
- 4) Construction Considerations

1) STRUCTURE FOUNDATIONS

The abutments and east pier of both the existing Hwy 6 NB and SB structures are supported on deep foundation steel H-piles driven to the bedrock. The west pier and centre pier located in the water are supported on spread footings founded on the bedrock surface. Having completed the foundation investigation for the proposed E/W-5 ramp structure, in general the native subsurface conditions appear to be relatively uniform between the existing and proposed structure sites. It is therefore recommended that the structure foundation scheme consist of foundations as summarized in Table 2 below.

TABLE 2 - STRUCTURE FOUNDATION SCHEME	
Area	Foundation Type
West Abutment	Deep Foundations - (Steel H-piles or Concrete Caissons)
West Pier	Shallow Foundation
Centre Pier	Shallow Foundation
East Pier	Deep Foundation (Steel H-piles or Concrete Caissons) or Shallow Foundations
East Abutment	Deep Foundations (Steel H-piles or Concrete Caissons)

The most economical and technically feasible footing scheme shall be adopted for design.

The only noticeable departure in comparing the present foundation recommendations with the foundations of the existing structure is at the east pier where shallow foundations is also given as an alternative. This option has been provided because subsurface conditions are such that only a minimum of three (3) metres of overburden overlies the bedrock. This minimum thickness may be considered as insufficient to define a deep foundation steel H-pile design (See definitions in commentary - C6-2 - "Deep and Shallow Foundations") and also the piles may not provide the necessary lateral rigidity. The minimum criteria must be evaluated before finalizing the design option.

DEEP FOUNDATIONS

General

The following recommendations and design guidelines apply to the west abutment, east abutment and east pier. Deep foundation units are particularly suitable at the abutment locations because it allows the abutments to be "perched" within the fill material enabling a more economical "closed-type" abutment design.

Driven Steel H-Piles

The above mentioned structure foundations 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 as tabulated in Table 3 below.

TABLE 3 - DRIVEN STEEL H-PILES				
Structure	Pile Type	Factored Capacity at U.L.S. (kN)	Axial Capacity at S.L.S. (kN)	Estimated Pile Tip El _n (m)
West Abutment	HP 310 x 110	1600	1100	300.5±
	HP 310 x 79	1150	890	
East Pier	HP 310 x 110	1600	1100	303.5±
	HP 310 x 79	1150	890	303.0 ± TS 93 07 16
East Abutment	HP 310 x 110	1600	1100	302±
	HP 310 x 79	1150	890	

To prevent pile installation impediment at the abutment locations, it is recommended that the maximum fill material particle size be restricted to 75 mm. This specification shall be included in the contract documents as a non-standard special provision (NSSP). Furthermore, to facilitate the pile driving process, it is recommended that all piles be equipped with reinforced tips. Driving details are given in OPSD 3301.00.

Axial capacities provided in Table 3 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.

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.

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 4 below. The corresponding soil depths can be obtained from the relevant borehole logs and having knowledge of the proposed heights of fill.

TABLE 4 - LATERAL RESISTANCE DESIGN		
Soils	Angle of Internal Internal Friction (ϕ)	Bulk Unit Weight (kN/m^3), γ
Irregular Mixture of Silt, Sand and Gravel (Fill Material)	30°	20
heterogeneous Mixture of Gravel, Sand and Silt	30°	204

In view of the variable nature of the fill material, it is recommended that the upper 60% of the embedment length be disregarded for lateral resistance of deep foundations within the fill material. Furthermore, to account for uncertainties in material selection and method of placement, it is recommended that the shear strength parameters in Table 4 be reduced by 10%.

Pile caps shall be protected against frost penetration by providing a minimum

1.2m earth cover or equivalent frost protection.

Concrete Caissons

Alternatively, the west abutment, east abutment and east pier can be founded on concrete caissons augered and placed on bedrock at or below the elevations provided in Table 3. 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 (S.L.S.) will not govern the design because the stresses required to induce detrimental settlements at the S.L.S. 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 0.9m 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 4. Lateral resistance can be further augmented by socketing the caissons into the bedrock. The socket shall be a minimum 0.5m and an unconfined compressive strength (q_u) of 10 MPa can be used to compute the horizontal capacity of the caisson in the rock. Again, only the ultimate limit state will govern the lateral capacity design.

Pile caps shall be protected against frost penetration by providing a minimum 1.2 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 which in most cases is 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 pier locations, it is recommended that the west and centre pier be definitely founded on conventional spread footings bearing on the competent unweathered, dolostone bedrock and consideration also be given to supporting the east pier on shallow foundations. For the purposes of the O.H.B.D.C., all footings founded on the dolostone bedrock can be designed as summarized in Table 5 below.

TABLE 5 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 El (M)
Centre Pier	1500	N/A	301.8
West Pier	1500	N/A	302.2
East Pier	1500	N/A	303.0

As indicated in Table 5 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 of U.L.S.

The capacities tabulated in Table 5 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 employed 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 100 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 100mm 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 later in this report under the subheading "Construction Considerations".

2) BACKFILL TO STRUCTURE

Material

It is recommended that Granular "A" or Granular "B" material be placed within a wedge behind the abutments 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 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°) (factored)	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	0.41
* 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.

Backfilling and Compaction

The backfill shall be placed in 300mm lifts in accordance with OPSS 902 series and compacted to achieve the target maximum dry density as outlined in OPSS 501 series.

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 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 six (6) metres of approach embankment fill material will be required at the approaches to the structure. 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. In addition, embankment material and construction, which are prudent to the safe and reliable performance of embankments are also discussed.

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.

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 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 runoff or alternatively 2.5H:1V slopes.
3. Embankment slopes adjacent to the Speed River shall be protected against scouring and erosive water forces. A revetment system consisting of a 1 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.
4. In the construction of new fills that must be integrated into existing fills, it is essential that the construction be done in accordance with OPSD 208.01. This drawing describes a procedure of "Keying" the new fills into the existing embankment to prevent plane translational instabilities.

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 50mm of settlement attributable to the elastic compression of the existing fill and native soil 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 25mm of settlement will occur within the embankment fill material. Settlements within a granular fill embankment should occur almost instantaneously and hence should occur during or immediately following construction material. Settlement 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 existing organic material shall be excavated within the plan limits prior to the construction of the embankment proper. Any subexcavation within the cohesionless surficial materials submerged below the prevailing water table 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 excavation 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. Vertical excavation can then proceed if necessary. This dewatering scheme controls and contains any soil sloughing within the confines of the excavation. The contractor shall submit a subexcavation and dewatering proposal for review by this office prior to construction.

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

As mentioned earlier, embankment construction of the new fills shall be integrated into the existing fills in accordance with OPSD 208.01

4) CONSTRUCTION CONSIDERATIONS

Roadway Protection Scheme

To facilitate the construction of the abutment foundations, a roadway protection scheme will be required to maintain traffic on the Highway 6 southbound during construction. A timber lagging-soldier pile wall can be used as a shoring system to achieve the roadway protection. Cantilevered walls can be considered provided that reasonable depths of penetration are required to satisfy the necessary earth pressure equilibrium. Alternatively, rock anchors or rakers can be used to support the shoring wall. The shoring system that proves to be the most economical and technically feasible shall be selected.

The design of the shoring system shall include the appropriate earth pressures computed in accordance with Section 6.6.1.2 of the O.H.B.D.C. Adjustment of any sloping surfaces must be incorporated in the computation of the active earth

pressures. The design parameters to compute earth pressures can be obtained from Table 4.

Rock anchors can be installed in the dolostone bedrock if required. A bond stress of 100 kPa can be used for the design of rock anchors in the dolostone bedrock.

Alternatively, the shoring wall can be supported by rakers installed in front of the wall. Rakers must be installed while an earth berm remains in front of the soldier piles. Slots should be cut into this berm to install rakers before the supporting berm is removed. Raker footings can be founded on the bedrock surface at either approach location (see Table 1 for Bedrock surface elevations). Alternatively, at the west approach, raker footings can be found at Elevation 302m within the heterogeneous mixture of gravel, sand and silt submerged below the groundwater table. An allowable bearing capacity of 250 kPa at S.L.S.T. type II and 900 kPa at U.L.S. can be used in the raker footing design.

Solider piles can be installed employing conventional augering or driving techniques. A dewatering scheme will be required for soldier piles penetrating the heterogeneous mixture of gravel, sand and silt submerged below the groundwater table. The dewatering scheme described below in conjunction with caisson construction is also applicable for the soldier pile installation.

The Contractor shall provide working drawings for the traffic protection scheme as outlined in OPSS 902.04. Our office can assist in the review and approval of these drawings.

Pier Construction in the Speed River

The procedure of constructing the west pier and centre pier within the Speed River will necessitate:

- 1) removal of existing concrete weir, and
- 2) a dewatering scheme

The removal of the existing concrete weir may require a temporary dam upstream and/or temporary diversion. A construction staging sequence may also be necessary.

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 watertight enclosure is not considered a practical alternative because of the impediment 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 a "box excavation scheme" with sump pumps be employed to excavate the overburden. The scheme involves the excavation and then subsequent displacement of a prefabricated enclosure until the bedrock surface is encountered while pumping water to facilitate the excavation. Once the overburden has been excavated, and the prefabricated enclosure is positioned on the bedrock surface and sealed effectively at the bedrock surface to prevent water inflow, conventional sump pumping techniques can be used to discharge any additional water.

It is recommended that the contractor submit dewatering scheme plans prior to construction as referenced in OPSS 902.04. Our office can then assist in the review of these drawings to determine acceptability of the proposal.

East Pier Construction

Although the groundwater table at the time of the investigation was at the bedrock surface and hence below the previous heterogeneous mixture of gravel, sand and silt, groundwater conditions can fluctuate. Hence, the contractor should be prepared to employ mud drilling techniques in the case of caisson construction (see below) or employ an oversized excavation with perimeter ditches and sump pumps should the shallow foundation be designed. In any case, the contractor's proposed scheme shall be submitted for review.

Caisson Construction

As mentioned previously, caisson construction within augered holes penetrating the cohesionless heterogeneous mixture of gravel, sand and silt 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. A NSSP should be included in the contract documents so that the identification of this potential condition is given to the Contractor. 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.

Environmental Considerations

In stream and shoreline construction can increase sediment depositions to a waterbody. Special environmental construction precautions such as silt fencing shall be used to mitigate damage to the environment.

Temporary Slopes

Temporary excavation slopes within the fill material or native material shall not be steeper than 1.5H:1V.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer utilizing equipment owned and operated by Malone's Soil Samples and Master Soils Investigation. Logging of 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 Mr. M.S. Devata, Chief Foundation Engineer.



A handwritten signature in black ink, appearing to read 'T. Sangiuliano'.

T. Sangiuliano, P.Eng.

Foundation Engineer

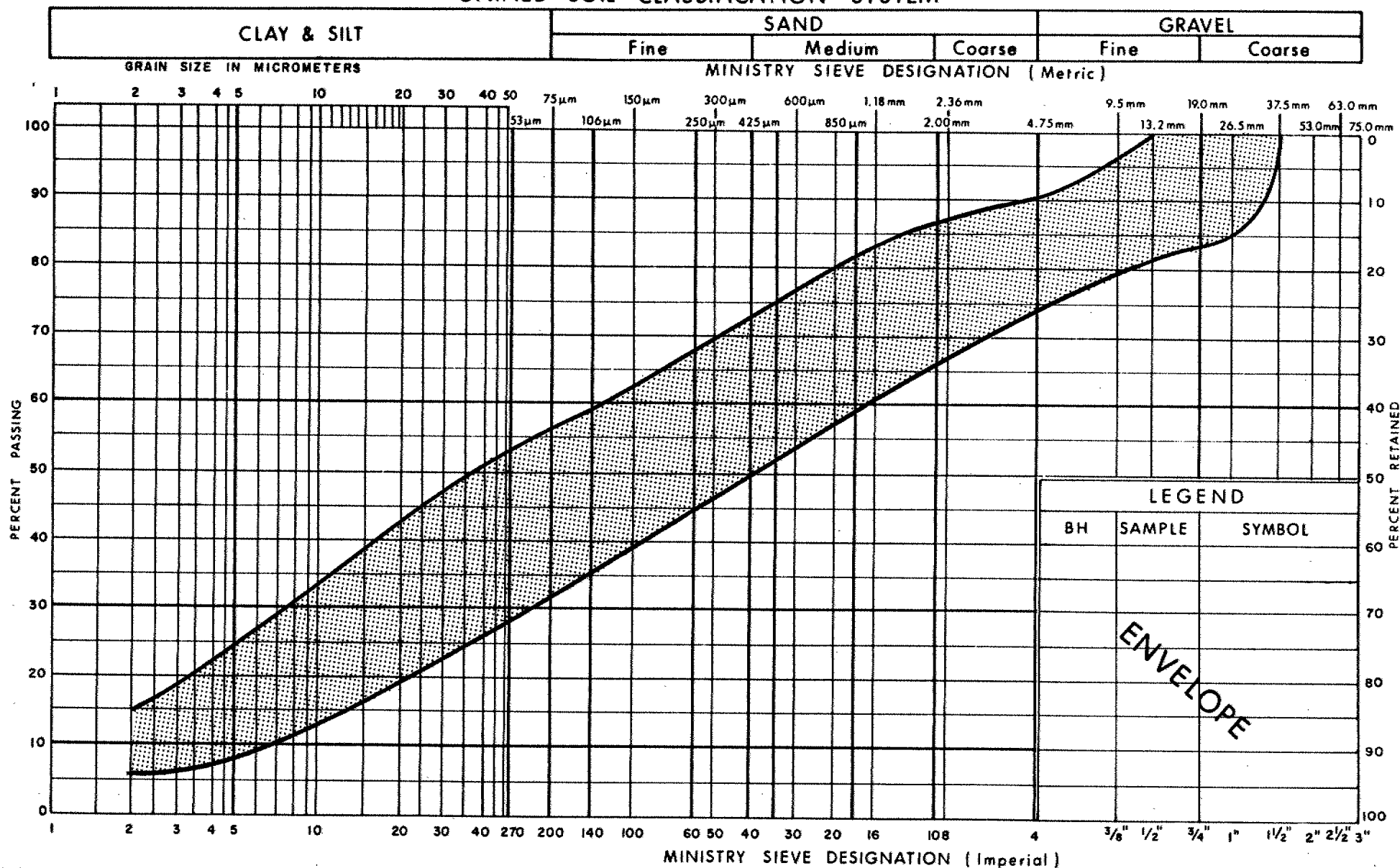
A handwritten signature in black ink, appearing to read 'M.S. Devata'.

M.S. Devata, P.Eng.

Chief Foundation Engineer

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM

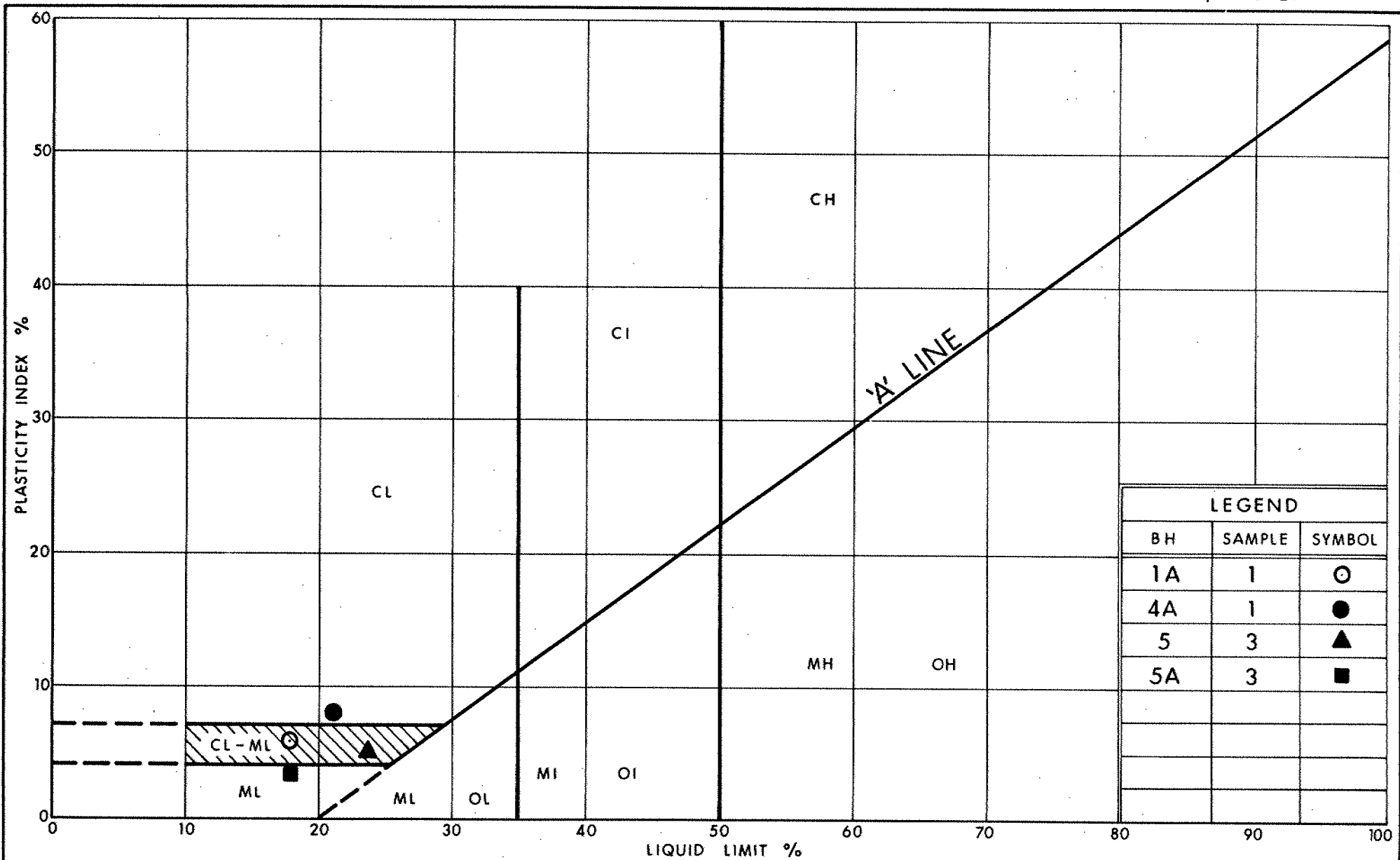


Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
 IRREGULAR MIXTURE OF SILT TO CLAYEY SILT,
 SAND & GRAVEL (FILL MATERIAL)

FIG No 1

W P 533 - 91 - 03



Ministry of
Transportation

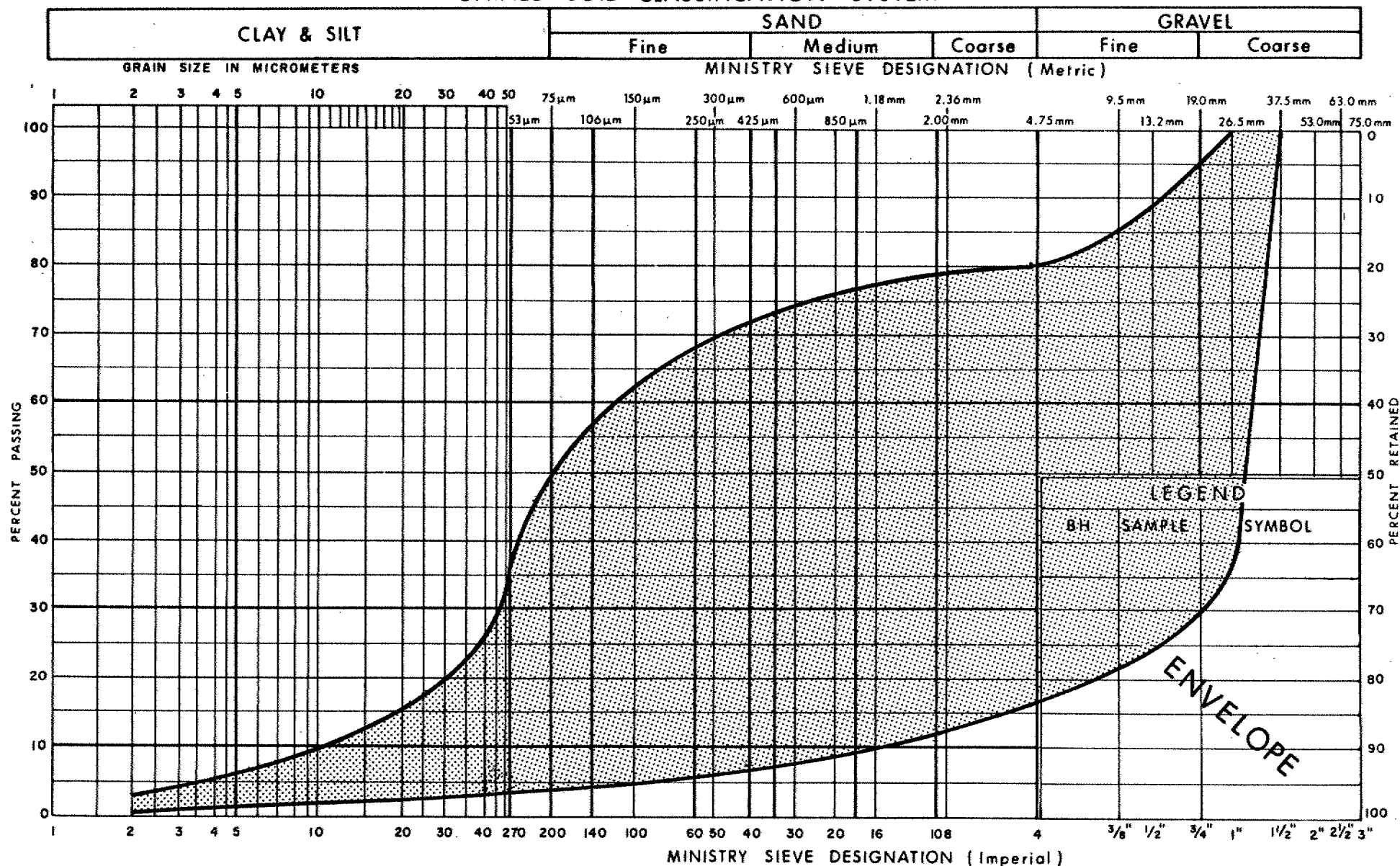
Ontario

PLASTICITY CHART IRREGULAR MIXTURE OF SILT TO CLAYEY SILT, SAND & GRAVEL (FILL MATERIAL)

FIG No 2

W P 533-91-03

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
 HETEROGENEOUS MIXTURE OF
 GRAVEL, SAND & SILT (Glacial Till)

FIG No 3

W P 533-91-03

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 (RQD), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

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

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 780.6; E 243 231.8 ORIGINATED BY TS
 DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core COMPILED BY TS
 DATUM Geodetic DATE 93 02 05-08 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
304.0	Ground Surface																
0.0	Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till)		1	SS	18												
	Brown		2	SS	29												
	Compact																
	Dense to Very Dense		3	SS	46		302										53 37 (10)
			4	SS	60	/13cm											
300.2			5	SS	60	/8cm											
3.8	Dolostone Bedrock Weak, Slightly Weathered to Unweathered		6	RC	REC 100%		300										RQD = 88%
			7	RC	REC 100%		298										RQD = 85%
286.5																	
7.5	End of Borehole																
	• 93 02 09																

RECORD OF BOREHOLE No 1A

1 OF 1

METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 801.5; E 243 233.4 ORIGINATED BY TS
 DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core COMPILED BY TS
 DATUM Geodetic DATE 93 02 09 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
311.6	Roadway Surface																
0.0	Asphalt					*											
311.2																	
0.4																	
	Irregular Mixture of Silt, Sand and Gravel (Fill Material)		1	SS	10		310									22.7	9 45 36 10
	Brown, Loose to Dense		2	SS	5		308										19 49 22 10
			3	SS	45												
			4	SS	45		306										
304.0																	
7.6	Black, trace Organics		5	SS	8		304										
	Heterogenous Mixture of Gravel, Sand and Silt (Glacial Till)		6	SS	22		302										
	Brown, Loose to Compact																
301.5																	
10.1	Dolostone Bedrock		8	RC	REC 100%		300										RQD = 33%
299.7																	
11.9	End of Borehole																
	* GWL not established																

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 793.5; E 243 260.5 ORIGINATED BY TS
 DIST 3 HWY 5 BOREHOLE TYPE NW Casing, NX Core COMPILED BY TS
 DATUM Geodetic DATE 93 02 15-16 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
303.7	Water Surface																
0.0	Water																
302.8																	
0.8	Het. Mixt. of Boulders, Cobbles, Gravel, Sand and Silt (Glacial Till)		1	SS	80												
302.2	Grey, Very Dense		2	SS	90												
1.5	Dolostone Bedrock																
	Weak, Unweathered		3	RC	REC 92%												RQD = 38%
			4	RC	REC 100%												RQD = 60%
299.4																	
4.3	End of Borehole																

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 780.0; E 243 280.5 ORIGINATED BY TS
 DIST 3 HWY 6 BOREHOLE TYPE NW Casing, NX Core COMPILED BY TS
 DATUM Geodetic DATE 93 02 16 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
303.6	Water Surface																
303.9	Water																
0.4	Heterogeneous Mixture of Boulders, Cobbles, Gravel, Sand and Silt (Glacial Till)		1	RC	REC 40%		302										RQD = 0%
301.9	Grey, Very Dense																
1.7	Dolostone Bedrock Weak, Unweathered		2	RC	REC 100%												RQD = 18%
			3	RC	REC 100%		300										RQD = 0%
298.7																	
4.9	End of Borehole																

RECORD OF BOREHOLE No 3A

1 OF 1

METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 791.0; E 243 284.0 ORIGINATED BY TS
 DIST 3 HWY 6 BOREHOLE TYPE NW Casing, NX Core COMPILED BY TS
 DATUM Geodetic DATE 93 02 16 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
303.7	Water Surface							20	40	60	80	100					
0.0	Water																
303.0																	
0.7	Heterogeneous Mixture of Boulders, Cobbles, Gravel, Sand and Silt (Glacial Till)		1	SS	8												
301.8	Trace Organics Grey, Loose to Very Dense		2	SS	95	/23cm											
1.9	Dolostone Bedrock Weak, Unweathered		3	RC	REC 92%		302										RQD = 33%
299.9			4	RC	REC 100%		300										RQD = 54%
3.8	End of Borehole																

RECORD OF BOREHOLE No 4

1 OF 1 METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 787.0; E 243 307.0 ORIGINATED BY TS
 DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core COMPILED BY TS
 DATUM Geodetic DATE 93 02 08 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
306.8	Ground Surface																
0.0	Irregular Mixture of Boulders, Cobbles, Gravel, Sand and Silt (Fill Material) Brown, Compact		1	SS	70	/28cm											
303.9			2	SS	18												
3.0			3	SS	80	/3cm											
	Dolomite Bedrock		5	RC	REC 67%	/3cm											RQD = 67%
	Slightly Weathered to Unweathered		6	RC	REC 87%												RQD = 8%
	Weak																
	Medium Strong		7	RC	REC 100%												RQD = 0%
299.1																	
7.8	End of Borehole																
	* 93 02 10																

RECORD OF BOREHOLE No 4A

1 OF 1

METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 776.5; E 243 304.0 ORIGINATED BY TS
 DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core COMPILED BY TS
 DATUM Geodetic DATE 93 02 08-09 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W _P	W	W _L		
306.8	Ground Surface																
0.0	Irregular Mixture of Silt to Clayey Silt, Sand and Gravel (Fill Material) Brown, Loose to Compact / Soft to Firm		1	SS	10												
			2	SS	5												
			3	SS	8												
303.1			4	SS	50												
3.7	Dolostone Bedrock Slightly Weathered to Unweathered Weak ----- Medium Strong		5	RC	REC 100%												RQD = 34%
			6	RC	REC 100%												RQD = 10%
299.9																	
6.9	End of Borehole • 93 02 10																

RECORD OF BOREHOLE No 5

1 OF 1

METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 773.5; E 243 322.0 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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
307.6	Ground Surface																
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material) Brown, Loose to Compact		1	SS	9		306										
			2	SS	14												
			3	SS	13												
303.8							304										27 40 27 6
3.8	Black, trace Organics		4	SS	9												
302.9	Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till) Brown, Loose to Compact		5	SS	60												
4.7			6	SS	60												
	Dolostone Bedrock Weak, Unweathered		7	RC	REC 88%		302										RQD = 36%
			8	RC	REC 100%		300										RQD = 38%
299.2																	
8.4	End of Borehole • 93 02 08																

RECORD OF BOREHOLE No 5A

1 OF 1

METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 787.5; E 243 331.3 ORIGINATED BY TS
 DIST 3 HWY 6 BOREHOLE TYPE HS Auger, NW Casing, NX Core COMPILED BY TS
 DATUM Geodetic DATE 93 02 10 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
313.3	Ground Surface																
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material) Brown, Loose to Dense		1	SS	6		312										10 41 38 11
			2	SS	2		310										
			3	SS	15		308										26 35 28 11
			4	SS	34		306										
305.7			5	SS	10		304										
7.6	Block, trace Organics Brown Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till) Compact		6	SS	23		302										RQD = 28%
302.6			7	SS	80	/13cm											
10.7	Dolostone Bedrock Weak, Slightly Weathered to Unweathered		8	RC	REC 100%												
301.0																	
12.3	End of Borehole • 93 02 11																

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 780.5; E 243 350.0 ORIGINATED BY TS
DIST 3 HWY 6 BOREHOLE TYPE HS Auger COMPILED BY TS
DATUM Geodetic DATE 93 02 11 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
313.6	Roadway Surface																
0.0	Irregular Mixture of Silt, Sand and Gravel (Fill Material) Brown, Loose to Compact		1	SS	6	DRY *	312										
			2	SS	8		310										26 30 35 9
			3	SS	23		308										
			4	SS	16		306										
			5	SS	26		304										
			6	SS	14												
302.9																	
10.7	Dolostone Bedrock		7	SS	56												
302.3	Weak, Slightly Weathered																
11.3	End of Borehole																
	* 93 02 12																

RECORD OF BOREHOLE No 7

1 OF 1

METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 783.5; E 243 206.9 ORIGINATED BY TS
 DIST 3 HWY 6 BOREHOLE TYPE HS Auger COMPILED BY TS
 DATUM Ceodetic DATE 93 02 08 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W _p	W	W _L		
304.4	Ground Surface																
0.0																	
	Black, trace Organics		1	SS	9												
	Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till)		2	SS	13												
	Brown, Loose to Dense		3	SS	48												
			4	SS	60	/3cm											
289.8			5	SS	60	/3cm											
4.6	Dolostone Bedrock Weak, Slightly Weathered to Unweathered																
298.2			6	SS	60	/10cm											
6.2	End of Borehole																
	• 93 02 08																

RECORD OF BOREHOLE No 8

1 OF 1 METRIC

W.P. 533-91-03 LOCATION Co-ords: N 4 820 801.0; E 243 216.0 ORIGINATED BY TS
DIST 3 HWY 6 BOREHOLE TYPE HS Auger COMPILED BY TS
DATUM Geodetic DATE 93 02 11 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									

311.3	Roadway Surface															
0.0	Irregular Mixture of Silt to Clayey Silt, Sand and Gravel (Fill Material) Brown, Compact to Dense/ Soft to Firm					*										
			1	SS	7		310									24 38 30 8
			2	SS	3		308									
			3	SS	44		306									48 41 9 2
			4	SS	23		304									
303.7			5	SS	20		302									20 28 49 3
7.6	Block, Organic		6	SS	44											20 57 21 2
	Heterogeneous Mixture of Gravel, Sand and Silt (Glacial Till)		7	SS	60	/13cm										
300.6	Grey, Compact to Dense															
10.7	Dolostone Bedrock															
300.1	Weak, Slightly Weathered															
11.2	End of Borehole															
	* G.W.L. not established															

ROCK CORE DESCRIPTION **WP 533-91-03**

Page 1 of 3

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1B	6	4.42-5.94	100	88	4.42-7.47	DOLOSTONE (with stylolites, abundant small vugs, and larger vugs up to 6 cm in diameter, commonly containing calcite crystals), very pale orange to pale yellowish brown (4.42-7.32 m) and dark yellowish brown to pale yellowish brown (7.32-7.47 m); medium grained; weak (4.42-7.32 m) to medium strong (7.32-7.47 m); unweathered to slightly weathered; fractures wide to very close spaced, flat to dipping, undulating to planar, smooth to rough.
	7	5.94-7.47	100	85		
1C	8	10.36-11.89	100	33	10.36-10.46	OVERBURDEN (till).
					10.46-11.89	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 moderate to very close spaced, flat to dipping, undulating to planar, smooth to rough.
2	3	1.54-2.84	92	38	1.54-4.29	DOLOSTONE (with stylolites, abundant small vugs, and some larger vugs up to 3 cm in diameter), very pale orange to pale yellowish brown; medium grained; weak; unweathered to slightly weathered; fractures moderate to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.
	4	2.84-4.29	100	60		

*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 533-91-03**

Page 2 of 3

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
3	1	0.36-1.88	40	0	0.36-1.73	OVERBURDEN (till).
	2	1.88-3.40	100	18	1.73-4.93	DOLOSTONE (with stylolites, abundant small vugs, and larger vugs up to at least 5 cm in diameter, commonly containing calcite and sphalerite crystals), very pale orange to pale yellowish brown (1.73-3.71 m) and dark yellowish brown to pale yellowish brown (3.71-4.93 m); medium grained; weak (1.73-3.71 m) to medium strong (3.71-4.93 m); unweathered to slightly weathered; fractures close to very close spaced, flat to near vertical, undulating to planar, smooth to rough.
	3	3.40-4.93	100	0		
3A	3	1.88-2.80	92	33	1.88-3.81	DOLOSTONE (with stylolites, abundant small vugs, and some larger vugs up to 1 cm in diameter), very pale orange to pale yellowish brown; medium grained; weak; unweathered to slightly weathered; fractures close to very close spaced, flat to near vertical, undulating to planar, smooth to rough.
	4	2.80-3.81	100	54		
4	5	3.81-4.72	67	67	3.81-7.77	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 (3.81-6.27 m) and dark yellowish brown to pale yellowish brown (6.27-7.77 m); medium grained; weak (3.81-6.27 m) to medium strong (6.27-7.77 m); unweathered to slightly weathered; fractures moderate to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.
	6	4.72-6.25	87	8		
	7	6.25-7.77	100	0		

*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 533-91-03

Page 3 of 3

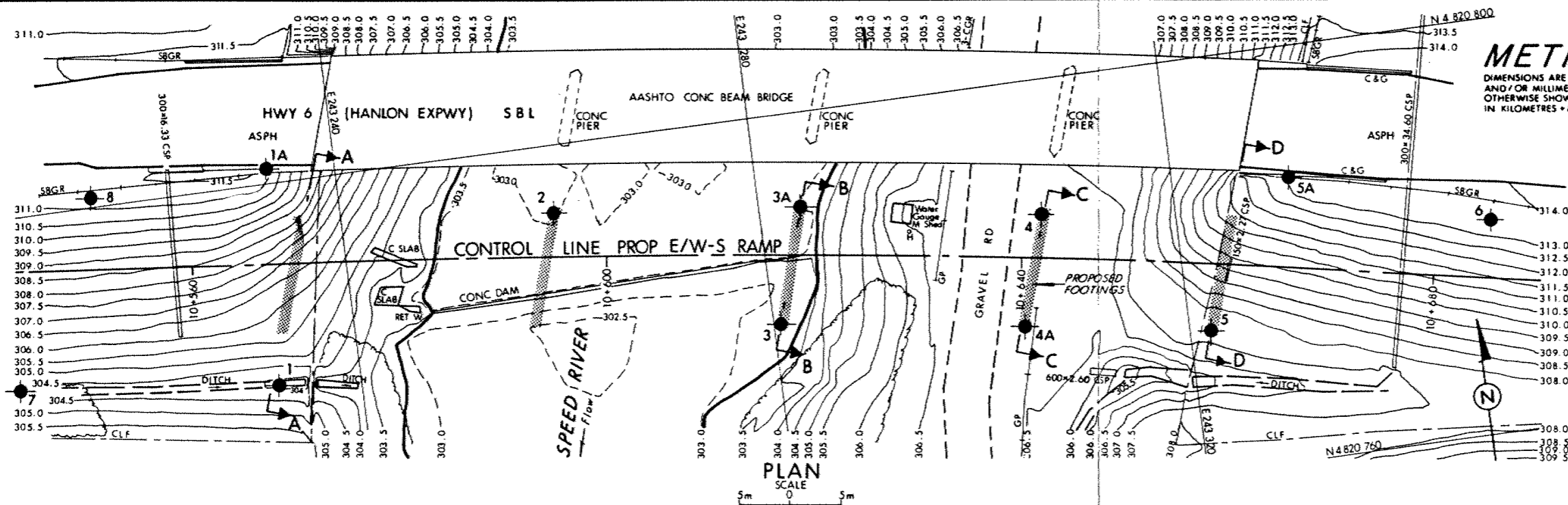
CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
4A	5	3.83-5.36	100	34	3.83-6.88	DOLOSTONE (with stylolites, abundant small vugs, and larger vugs up to 5 cm in diameter, commonly containing calcite and sphalerite crystals), very pale orange to pale yellowish brown (3.83-6.43 m) and dark yellowish brown to pale yellowish brown (6.43-6.88 m); medium grained; weak (3.83-6.43 m) to medium strong (6.43-6.88 m); unweathered to slightly weathered; fractures close to very close spaced, flat to near vertical, undulating to planar, smooth to rough.
	6	5.36-6.88	100	10		
5	7	5.36-6.88	88	36	5.36-8.41	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 (5.36-8.23 m) and dark yellowish brown to pale yellowish brown (8.23-8.41 m); medium grained; weak (5.36-8.23 m) to medium strong (8.23-8.41 m); unweathered to slightly weathered; fractures moderate to very close spaced, flat to near vertical, undulating to planar, smooth to rough.
	8	6.88-8.41	100	38		
5C	8	10.80-12.32	100	28	10.80-12.32	DOLOSTONE (with stylolites and abundant small vugs), very pale orange to pale yellowish brown; medium grained; weak; unweathered to slightly weathered; fractures moderate to very close spaced, flat to near vertical, undulating to planar, smooth to rough.

*CR = CORE RECOVERY

*RQD = ROCK QUALITY DESIGNATION

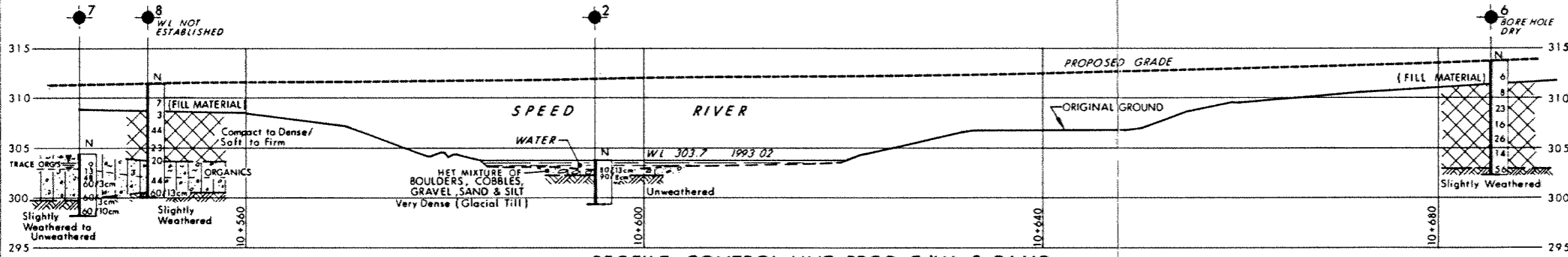
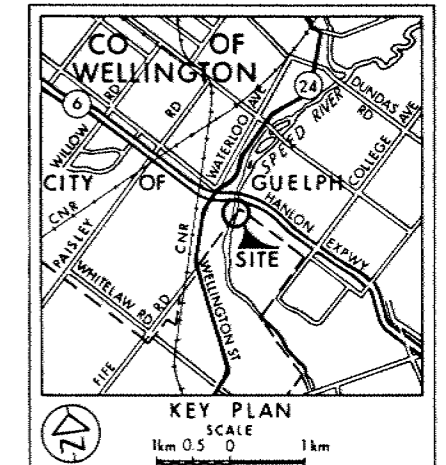
(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section

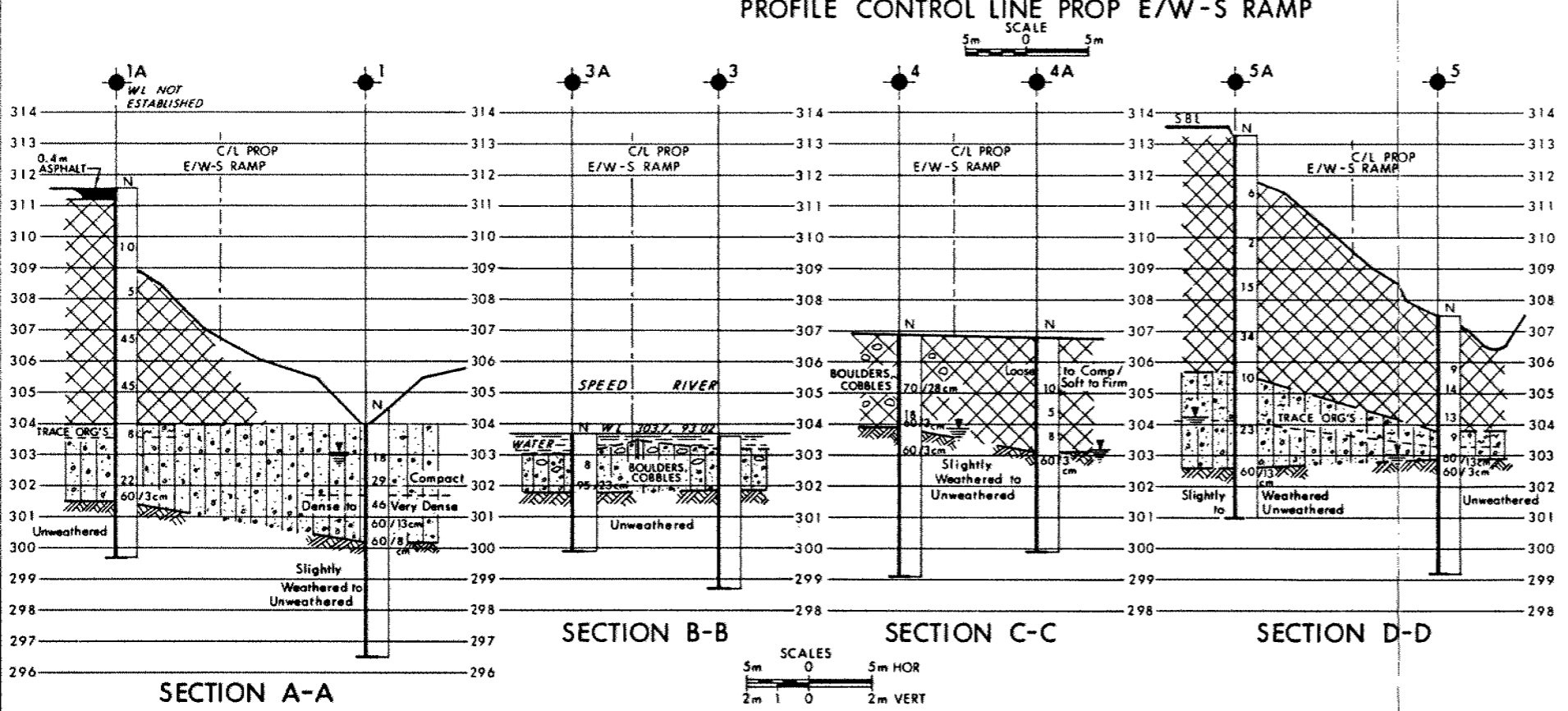


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 533-91-03
SPEED RIV & PROP E/W-S RAMP
(INTERCHANGE AT HWY 6 & WELLINGTON ST)
BORE HOLE LOCATIONS & SOIL STRATA



- LEGEND**
- Bore Hole
 - ⊕ Dynamic Cone Penetration Test (Cone)
 - ⊕ Bore Hole & Cone
 - N Blows/0.3m (Std Pen Test, 475 J/blow)
 - CONE Blows/0.3m (60° Cone, 475 J/blow)
 - W.L. at time of investigation 1993 02



SOIL STRATIGRAPHY LEGEND

- IRREGULAR MIXTURE OF SILT TO CLAYEY SILT, SAND & GRAVEL (FILL MATERIAL) Loose to Dense
- HETEROGENEOUS MIXTURE OF GRAVEL, SAND & SILT (Glacial Till) Loose to Very Dense
- DOLOSTONE BEDROCK

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
1			

Geocres No 40P9-32

HWY No	PROJECT	DIST
6	HANLON EXPWY	3

SUBMITTAL	CHECKED	DATE	SITE
TS	TS	1993 06 18	35-578

DRAWN	CHECKED	APPROVED	DWG
RS	TS		5339103-A

