DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 316-191

DIST. 9 REGION

W.P. No. 45-74-05

CONT. No. 83-15

W. O. No.

STR. SITE No. 3-358

HWY. No. 16W

LOCATION Mud Creek Bridge

No. of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

memorandum

Ontario

To: Mr. T. C. Kingsland,
Head,
Structural Section,
Eastern Region

Date: 82 01 08

CONT 83-15

From: Pavement and Foundation Design Section,
Room 315, Central Building

Re: Foundation Investigation and Design Report by Warnock Hersey Professional Services Ltd., W. P. 145-74-05, Site 3-358
Highway 16N - Mud Creek, District 9, Ottawa

The M. T. C. retained the services of geotechnical consultants, Warnock Hersey Professional Services Ltd., for the above-mentioned project. The final foundation report was received by this office and subsequently reviewed for technical content. Our comments are as follows:

Artesian conditions were encountered once the borehole penetrated into the sand and gravel glacial till stratum. Some means of dissipating the artesian flow pressures and preventing piping and erosion is required. In our opinion, the most practical solution is the provision of a granular filter drainage blanket below the pile cap. The thickness of the blanket should be approximately 450 mm. If this blanket is situated below high water level, this should be protected with rip rap against erosion by flooding.

We feel that the other recommendations contained in the report are adequately covered for your requirements. Should you require further clarification on any additional information, please feel free to contact us.

M. Devata, P. Eng.,
Senior Foundations Engineer

MD/bpd

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JAN 11 1982

FOUNDATION INVESTIGATION REPORT

for

W.P.145-74-05 Str.Site: 3-358

HIGHWAY 16N - MUD CREEK

District 9 (Ottawa) Eastern Region

Distribution:

13 copies: Ministry of Transportation and Communications, Downsview

2 copies: Warnock Hersey Professional Services Limited

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GEOCRES No 31G-191

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FOUNDATION INVESTIGATION REPORT
for
W.P. 145-74-05
Structure Site 3-358
Proposed Crossing at Mud Creek and Hwy. 16N
District of (Ottawa), Eastern Region

1. INTRODUCTION

Warnock Hersey Professional Services Limited have been retained by the Ontario Ministry of Transportation and Communications, under Agreement No. 4242-9081-109, to provide geotechnical engineering services in connection with the above project. The terms of reference were to carry out a site investigation of sufficient scope to provide recommendations for the design and construction of a new bridge at this site.

The request was received on 1981 11 12. A letter dated 1981 11 27 was submitted upon completion of the field work, with some preliminary recommendations. This report describes the field work and soil conditions in greater detail. Recommendations are given for the design and construction of foundations and embankments of the proposed structure and its approaches.

2. DESCRIPTION OF SITE AND GEOLOGY

The site is located in North Gower Township, some 35 kilometers south of Ottawa. Specifically, the proposed structure site is located off 2nd Line Road between existing Hwy. 16 and Carsonby. At this location, Mud Creek is approximately 5 to 7 m wide and about 1 m deep. The flow is in a northerly direction.

The area is designated physiographically as the North Gower Drumlin Field. Air photos of the area and ground observations show the presence of buried drumlins in the vicinity. These drumlins are oriented more or less north-south. A mantle of marine soil is draped over these drumlins and other similar glacial inorganic land forms.

2. DESCRIPTION OF SITE AND GEOLOGY (Cont'd)

The marine soil originated with the inundation of the area by the Champlain Sea. Therefore silts and Leda clays are common in this area.

At this site, Mud Creek meanders on a relatively flat-lying plain bordered in places by glacial landforms such as moraine remnants. The proposed alignment of Hwy. 16N runs parallel to and about 50 m west of existing 2nd Line Road. This township road crosses Mud Creek with a single span steel beam structure (structure site 3-143). At the south approach bearing ledge, the concrete beneath the steel beam bearing plates is cracked in a semicircular fashion, and, at one corner, the concrete has spalled, exposing the reinforcing. The approaches to this structure are about 3 metres in height and show no sign of excessive or differential settlement, nor instability.

3. FIELD AND LABORATORY WORK

The field work consisted of the drilling of two sampled boreholes, one on each side of the creek, supplemented by two additional dynamic cone penetration tests and one continuous vane shear strength hole. The investigation was carried out by means of a muskeg vehicle mounted continuous flight auger machine, equipped with hollow stem augers (item 5.2.i of MTC Supply Contract No. S-80-3298). The location of the bore and cone holes is shown on drawing 1457405-A in the Appendix. All bore and cone holes were terminated within a very competent glacial till deposit.

3. FIELD AND LABORATORY WORK (Cont'd)

Artesian flow was encountered at the north approach borehole (BH1) once the till was penetrated. It was thought prudent to terminate the drilling within the till in order to control the groundwater flow.

Standard Penetration Resistance tests were carried out in the hard or very dense materials as well as in the non-cohesive zones of the stratigraphy. Elsewhere, 51 mm thin-walled steel tube samplers were pushed in manually or hydraulically to recover relatively undisturbed samples of the cohesive soil strata. Some silt samples recovered by this procedure began to "flow" and were transferred into sealed glass jars. Where feasible, laboratory size vane shear tests were carried out at the tip of each tube in the field immediately upon recovery of the sample, prior to capping and sealing of the tubes.

After recovery of each thin-wall tube sample, standard M.T.C. "A" size vane tests were carried out some 457 mm below the last sample. The undrained field vane shear strengths were noted. After initial failure, the vane was rotated six times quickly to remould the soil, care being taken to ensure the vane and rod asssembly did not sink into the soil when this was being done. After a 60 second set-up period, the remoulded shear strength was measured. The ratio of the undisturbed to remoulded undrained shear strength was recorded as the sensitivity of the soil to the nearest whole number.

Upon completion of each borehole, water level readings were taken in the uncased holes. At the location of Borehole 1, the artesian flow was estimated by visual means and the head was estimated by adding augers to serve as a standpipe. Due to leakage between the auger joints, the artesian head could not be accurately measured, hence, the reported value is a best estimate.

3. FIELD AND LABORATORY WORK (Cont'd)

All soil samples were protected from freezing during the investigation. They were transported each evening to a warm room and stored there for ultimate transportation by car to our laboratories in Toronto. The samples were placed inside the car both for warmth and lessening of disturbance during transportation.

In the laboratory, the soil samples were visually examined and assigned for further testing. The laboratory testing consisted of:

Natural Moisture Contents
Atterberg Limits (on undried samples)
Grain Size Distributions

The results of these tests are plotted on individual records of boreholes as well as on the Figures in the Appendix.

The samples remaining after testing were transferred into sealed glass jars and will be stored for future examination. All thin-walled tube samples were extruded; unused portions were dissected longitudinally for close examination of the sedimentation characteristics. The dissected specimens are stored in sealed glass containers.

4. SUBSOIL CONDITIONS

4.1 General

The soil conditions across the site are fairly consistent. Briefly, the soil stratigraphy consists of a surficial slightly organic layer followed by a highly plastic, sensitive clay, overlying a silt deposit. The silt stratum, which contains clay seams and silty clay zones is underlain by a very dense sand, gravel glacial till of undetermined thickness. The details follow:

4.2 Surficial Soils

Surficial soils consist mainly of topsoil and some organically contaminated sandy silt. The maximum depth of this material was 1.4 metres at the southern approach area. The material is not highly compressible and exhibits a moisture content of about 25 percent. One N value in this material was 8 blows/0.3 m indicating it to be of loose compactness.

4.3 Clay

Underlying the topsoil, or the organically contaminated sandy silt, there is a 2.5 to 5 metre thick deposit of clay, the upper 2.5 metres of which is desiccated. The bottom of the clay stratum is located at about elevation 85.

The upper desiccated zone exhibits a characteristic mottled brown colouring and contains root hairs. Due to the fissured nature of the desiccated zone, samples could not be suitably trimmed for triaxial testing. However, field vane and undrained strength values exceeded 50 kPa, indicating this material to be of generally stiff consistency. N values of 15 and 8 blows/0.3 m were recorded in this zone.

Below the desiccated zone, the clay exhibits a characteristic grey colour, with a trace of black mottling typical of leda clays. The clay is homogeneous in texture and sensitive to remoulding. Typical physical properties of the clay are shown in Table 1.

Table 1 Physical Properties of Clay at Mud Creek Site

	Range	Average
Moisture Content (%)	82-85	83
Liquid Limit (%)	50-75	65
Plastic Limit (%)	22-31	27
Plasticity Index (%)	32-46	38
Liquidity Index		1.4
Field (vane) Undrained Shear Strength (kPa)	31-57	37

The Atterberg limits are plotted on Figure 1 and show the material to be highly plastic.

Based on the field vane tests, particularly the continuous vanes at Borehole 3, the consistency of the clay may be described as firm to stiff, based on an average undrained shear strength of about 37 kPa.

Some hand probings were made in this deposit at the bottom of Mud Creek. These probings show the upper one metre of the soil to be quite soft, (estimated shear strengths in the order of 10 kPa).

4.4 Silt

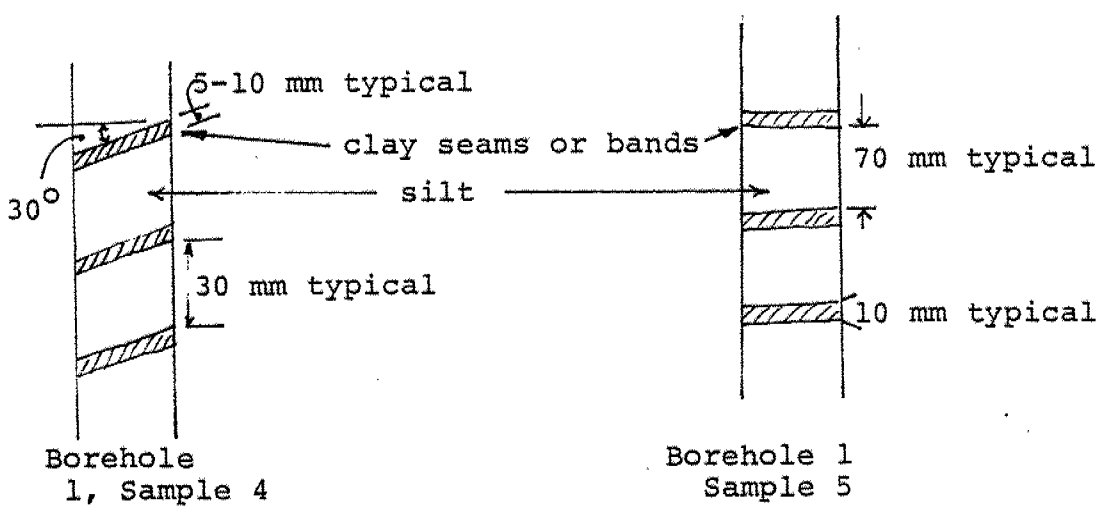
At about elevation 85±, the clay is underlain by a silt deposit which extends for a thickness of 3.5 to 6 metres to a till stratum.

The silt is grey in colour and extremely dilatant. One grain size disbribution curve is shown on Figure 2. The silt has trace amounts of fine sand and clay.

Field vane tests were carried out in this deposit in the belief that the material was cohesive. It was believed that the predominant deposit was clay, with silt zones. In fact, the opposite was revealed when all the recovered soil samples were examined in the laboratory and tested for verification of classification.

At the clay-silt strata interface, the silt exhibits slight plasticity and in fact behaves as a silty clay of low plasticity would behave. With increasing depth, however, the silt character dominates the profile.

The distinguishing characteristic of this deposit is the presence of clay bands or seams at periodic intervals. A sketch of two samples is given below for illustration purposes. The clay bands are typically about 5 to 10 mm in thickness. The clay is very stiff in consistency, smooth in fabric and cubically fractured in structure. It is extremely brittle.



4.4 Silt (Cont'd)

The natural moisture content of the silt ranged from 23 to 29 percent, with an average of 26 percent. The moisture content of a clay seam from this silt was measured to be 74 percent.

Based on N values of 1 blow/0.3 m, the silt is considered to be in a very loose state of density.

4.5 Sand and Gravel (Glacial Till)

The silt deposit is underlain by a sand and gravel deposit of glacial origin. All bore, cone and vane test holes were terminated in this stratum whose full extent was not determined. From other soils investigations in this area, it is believed the till deposit is fairly extensive in depth.

A typical grain size distribution of this deposit is shown on Figure 3. The material is fairly coarse but does contain trace amounts of fines. The sand and fine gravel sizes are sharp and angular and consist of orthoclase feldspars, quartz, calcite and limestone particles. Hence, the glacial till is basal, derived from the local rock in the area.

The N values in this deposit were over 69 blows/0.3. The dynamic cone penetration tests met refusal in this deposit between elevations 76 and 79. Therefore, it is suspected the till contains zones of either reworked material or silt lenses. For example, note the cone test profile at Borehole 1 where the blow count dropped, from about elevation 77 down to elevation 76.5, before picking up agin.

Based on the N values and results of the cone tests, the sand and gravel deposit is considered to be in a very dense state of compaction, with locally loose to compact zones.

5. Groundwater Conditions

The phreatic level at the site was found to correspond more or less to the creek level. However, at Borehole 1, an artesian condition was noted once the borehole penetrated the sand and gravel glacial till stratum. The head of the artesian flow was estimated at about 2 metres above the ground surface.

6. DISCUSSION AND RECOMMENDATIONS

6.1 General

It is proposed to construct a single span structure on the new Hwy. 16N alignment. The structure span will be 11 metres on a roadway width of 12.5 metres. In conjunction with the construction of this new structure, it is also proposed to realign the creek so as to eliminate a long meander. A plan of the proposed stream diversion is shown on the left side of Drawing 1457405-A. According to these proposals, therefore, a significant portion of the south approach fill will be located in the bypassed channel of Mud Creek.

The investigation has revealed the presence of a surficial slightly organic sandy silt soil or topsoil, overlying a sequence of clay, silt and sand and gravel strata. The upper half of the clay deposit is desiccated. The silt is considered to be in a loose state of relative density. The sand and gravel below the silt is a glacial till, generally in a very dense state but with potentially weaker zones or pockets. The till is the source of artesian water of low head.

6.2 Foundation Design

It was originally indicated in our preliminary letter report that it might be possible to found the structure on spread footings so as to take advantage of the desiccated clay zone. At the time that recommendation was made, an E-plan of the site was not available. Therefore, it was not known that the proposed geometrics would not favour such a design. As can be seen from Drawing 1457405-A, the south abutment straddles the creek. Hence, the most practical foundation alternative is a deep foundation.

The existence of potentially weaker zones within the till stratum preclude the use of timber piles as these would not be able to penetrate more than a metre or so into the dense till zone. Therefore, steel-H piles are recommended. The steel H-piles should be reinforced at the tip as per MTC Standard SS3-1

6.2 Foundation Design (Cont'd)

since boulders are reported to be present in the till sampled at the nearby township bridge location. Also, it will be necessary to apply fairly large energies per blow to cause the pile tips to penetrate below the potentially weaker zones of the till.

From the soil stratigraphy and records of nearby structures, it is recommended that tip reinforced steel H piles be driven into the glacial till deposit to or below elevation 76. The design capacities of the pile selected may be assumed to be directly proportional to the cross-sectional area of the section chosen. For example, for 310 HP @ 110 steel piles the following design values are recommended:

Safe capacity	1,150 kN
Factored capacity at U.L.S.	1,600 kN
Capacity at S.L.S. Type II	1,110 kN

6.3 Approach Embankments

The approach embankment fill heights are nominal. The profile grade is at about elevation 91 and results in maximum heights of fill of about 4 metres at Station 22 + 940 and station 22 + 970 at the south approach, and about one metre at the north approach to sta. 23 + 010.

6.3.1 Stability

The undrained shear strength of the desiccated and undesiccated clay is in the order of 50 and 35 kPa respectively. Hence, fills up to 5 metres in height will be stable with 2:1 side slopes. At the present site, no instability is forseen for the proposed profile grade. However, care should be taken during construction, particularly at the south approach along the by-passed channel, to prevent failures by following certain construction procedures as outlined in detail in Section 6.7.3 of this report.

6.3.2 Settlement

The existing road fill (2nd Line Rd., Twp. of North Gower) is performing well. Therefore, settlements of new fills built on the desiccated clay are expected to be minor. However, at the south approach, new fill will straddle the by-passed creek channel. Therefore, differential settlement in a transverse direction is likely to occur between portions of fill in the channel and portions outside of the channel.

In order to limit and control differential settlements, the soft clay in the creek bottom should be excavated to a depth of at least 1 metre below creek bed. The backfill should consist of non-cohesive granular material to at least the original creek bed level. All new fill, up to elevation 88, should be granular and should be compacted to 95 percent of the standard proctor density of the material used. Above this level, any earth fill would be acceptable.

The new fill should be allowed to consolidate the soil below for as long a period as scheduling permits. In any event, paving of the approach embankment between sta. 22 + 890 and the south abutment should be delayed until monitoring of settlements indicates it to be advisable to do so.

The silt below elevation 84 to 85, although considered to be in a loose state, will quickly consolidate. Hence, a major portion of the ultimate settlements should be realized within a period of 6 months to a year.

Surcharging of the south approach fill will hasten consolidation. Stability is not a problem within a confined channel valley. Therefore, the channel portion of the fill could be built up to elevation 92 and allowed to settle for a minimum period of 3 months prior to regrading back to profile grade. The excess material could be used to build the adjoining fill along the creek banks. Care should be taken at station 22 + 890 not to surcharge the fill, as instability is likely to occur perpendicular to the alignment along the westerly arm of the bypassed channel.

6.4 Earth Pressures

The closed abutments for the single span structue will be subjected to earth pressures from the adjoining fills. These may be calculated in accordance with the provisions of subsection 6.6.1.2.2. of the O.H.B.D.C.

Along the south abutment, the earth pressure will vary from west to east, i.e. non-uniform conditions will prevail, as the height of fill will vary from about 5 metres at the centre of the abutment to about 1 metre at the western extremity. Therefore, it would be advisable to either step down the south abutment footing, or alternatively, to create more uniform earth pressure conditions by subexcavating a 1.5 m wide channel parallel to and behind the west half of the south abutment and backfilling it with the approach fill soil. If the approach fill material is granular, a clay seal should be provided at the western face of such a channel to prevent seepage of the diverted creek water into the granular fill behind the south abutment.

6.5 Creek Diversion

The proposed creek diversion reduces the flow length from about 220 metres to 80 metres between the exit and entry points. Such a reduction in flow length will be accompanied by a doubling of the flow velocity (since gradient is now more than doubled) and a quadrupling of the erosive power of the creek. This factor must be takn into consideration in the design of the new creek channel. Either energy dissipators should be provided along the new channel, or the channel bottom widened to provide a greater cross sectional area for stream flow.

Along the existing creek channel, erosion is occurring at a slow pace. Roots of large trees are exposed. The soil being eroded is the desiccated clay. This clay is fissured, as noted previously. Hence, erosion is occurring in a quantum manner. This is evidenced by the presence of gravel-sized clay nodules in the stream bed, obviously the remnant of larger pieces of clay bank toppled into the stream and then worked at by the water current.

6.5 Creek Diversion (Cont'd)

The proposed diversion invert grade is at elevation 87.2, very close to the boundary between the desiccated clay and the undesiccated clay. For all practical purposes, it may be assumed the diverted creek channel bed will be located in the firm clay deposit. In order to preserve stability, the new channel side slopes should be excavated to no steeper than 1-1/2:1 side slopes, if artificial side and bottom treatments are considered, or 2:1 for natural slope treatment. Economics of savings in earth volume versus expenses of artificial treatment will dictate the choice of side slopes.

Several commercial products using synthetic fabrics, permeable mats with concrete block facings and the like, are available to line the channel. Whatever the lining chosen, suitable filters should be provided between the native clay and the lining material, particularly if gabions or gravel blankets or other loose materials are to be used.

The channel bottom may be left unlined provided bed flow velocities are not in excess of current values. Otherwise, energy dissipators, or weir-type drop structures, should be provided to control bed erosion.

6.6 Alternative Alignment

The current centre line of the proposed alignment (Line C, Hwy. 16N) results in undesirable approach characteristics south of the south abutment. If feasible, the alignment should be moved towards the east about halfway between the present alignment and 2nd Line Rd. If future four-laning is being contemplated, two choices are available. Either the township road can be annexed or the current proposed Line C used for the future 2 lanes. In the latter event, the creek channel could be surcharged and preloaded for an extensive period of time.

It is our belief that the existing pool of water on the north side of the creek in the vicinity of our suggested realignment is shallow and not a design factor in the adoption of the suggested

6.6 Alternative Alighment

realignment. The soil conditions will be better along the suggested realignment.

Further careful testing of the desiccated zone is recommended to establish the feasibility of building perched abutments on compacted fill. It may be feasible to design a simply supported single span structure which can tolerate some settlement at the approaches.

6.7 Construction Considerations

6.7.1 General

The recommendations provided in this report are based on data collected at and interpreted from a limited site investigation. If soil, rock or groundwater conditions different from those described or assumed in this report are uncovered during construction, we should be consulted at once to determine and advise if modifications from a geotechnical viewpoint are necessary to the design or construction aspects of the project, and so to ensure its coqtinued economy and safety.

6.7.2 Excavations and Dewatering

Temporary excavations in the desiccated clay will stand for normal construction periods, at 1:1 side slopes. Inflow of groundwater from the clay is expected to be minimal. Pumping from sumps will be sufficient to handle minor seepage and surface inflows into excavations within the clay.

Since the clay is sensitive to remoulding, and a difficult construction material to work with, provisions should be made for the application of a 150 mm sand cushion for workmen to walk upon.

6.7.3 Approach Fills

Surficial organics should be removed from the plan limits of the approach fills. The existing trees and roots should be cleared and grubbed from along the banks of the creek prior to placing of new fill. In order to preserve stability of the channel in-fill material, filling should commence in a direction parallel to the creek course, starting at the base of the creek. The upper 1 metre soft material of the creek bed should be excavated and wasted. The channel infilling should proceed in small lifts in a longitudinal direction.

7. CONCLUSIONS

The site is not suitable for spread footing type foundations. Steel-H piles with reinforced tips are recommended as the deep foundation for the proposed structure.

Due to anticipated differential settlement problems along the current south approach alignment, surcharging and pre-loading is recommended. Alternatively, consideration should be given to shifting the present alignment 25 metres to the east. On this revised alignment, it may be feasible to utilize the strength of the desiccated clay to support perched footings on compacted fill at nominal bearing pressures.

The creek channel revision implies greater erosion potential due to increased flow velocities. Hence energy dissipators should be provided, or the channel geometry modified, to limit flow velocities to or below current values.

Respectfully submitted,

Warnock Hersey Professional Services Ltd.

C. Mirza, P. Eng.
Manager,
Geotechnical Services

Encl.

A P P E N D I X

Office Record of Boreholes 1 to 5
Figures 1 to 3
Drawing 1457405-A

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:

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
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 and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1

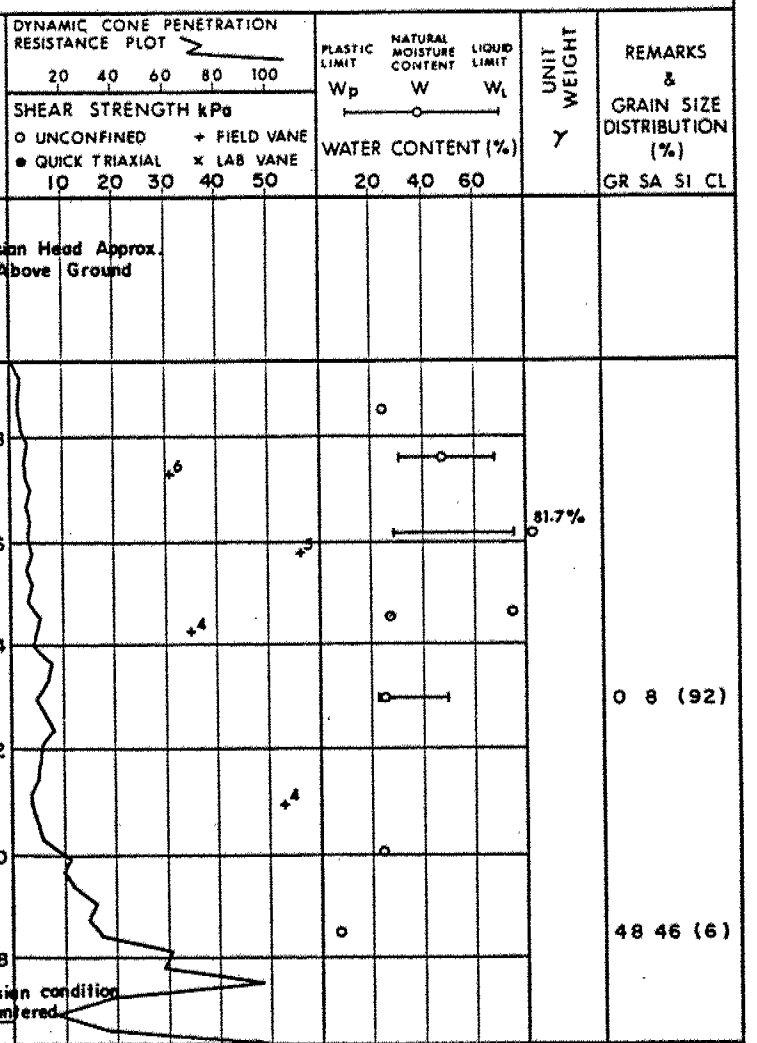
METRIC

W P 145-74-05 LOCATION STA 22+965.0 %s 10.0m Rt of ℄ Hwy 16 N ORIGINATED BY CM

DIST 9 HWY 16N BOREHOLE TYPE Hollow Stem Auger and Cone Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 24 and 25 CHECKED BY CM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Artesian Head Approx. 2m Above Ground				
89.4	Ground Surface				
0.0 / 88.0	TOPSOIL and Fine SANDY SILT Slightly Organic, trace of Clay	1	SS	8	
1.4 / 87.0	Desiccated Stiff to Firm	2	SS	3	
2.4 / 85.4	CLAY, Sensitive Firm to Stiff Grey	3	TW	PM	
4.0	SILT, Very Loose with thin Very Stiff Clay bands at 30 to 70 mm intervals Becoming Soft cohesive Silt with depth	4	TW	PM	
		5	TW	PH	0 8 (92)
		6	SS	1	
79.7		7	SS	1	
9.7	SAND AND GRAVEL with some Silt and Clay (Glacial Till) Very Dense	8	SS	96	48 46 (6)
76.8		9	SS	69	
76.3	END OF BOREHOLE				
13.1	END OF CONE TEST				



+3, x5 : Numbers refer to Sensitivity

15 20 10 5 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION–ENGINEERING MATERIALS OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 2 METRIC

W P 145-74-05 LOCATION STA 9+980.0 ℄ CREEK DIVERSION ORIGINATED BY C M

DIST 9 HWY 16 N BOREHOLE TYPE Hollow Stem Auger and Cone Test COMPILED BY B D

DATUM Geodetic DATE 1981 11 25 CHECKED BY C M

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.2	Ground Surface							
0.0 / 89.6	TOPSOIL				*	90		
0.6	Desiccated, Stiff Mottled Brown–Grey	1	SS	15				
		2	SS	6		88		
87.7 / 2.5	CLAY, Sensitive Firm to Stiff Grey	3	SS	1				84.8%
85.6 / 4.6	SILT, Very Loose with thin Clay bands at 70 mm intervals Becoming Soft cohesive Silt with depth	4	TW	PH		86		
		5	TW	PM		84		
		6	TW	PM				
82.0 / 8.2	END OF BOREHOLE Probably SILT	6A	SS	PH				
80.5 / 9.7	Probably SAND and GRAVEL (Glacial Till)					80		
79.4 / 10.8	END OF CONE TEST							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, x LAB VANE; 10 20 30 40 50

PLASTIC LIMIT Wp — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60

Cone Test Below Augers

100/15 cm

* Water Level Not Established

$+^3$, x^5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 3

METRIC

W P *145-74-05 LOCATION STA 22+987.0 %s 3.0 m Lt of ℄ Hwy 16N ORIGINATED BY C M

DIST 9 HWY 16 N BOREHOLE TYPE Augers ; Continuous Vanes and Cone Test COMPILED BY B D

DATUM Geodetic DATE 1981 11 25 CHECKED BY C M

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, x LAB VANE	WATER CONTENT (%)		GR SA SI CL
90.2	Ground Surface										
0.0	Desiccated Very Stiff						90				
87.0							88				
3.2	CLAY, Sensitive Firm to Stiff Grey						86				
85.2											
5.0	SILT, Probably banded with Clay Seams						84				
83.8											
6.4	END OF VANE TEST										
	Probably SILT						82				
81.1											
9.1	Probably SAND and GRAVEL (Glacial Till)						80				
79.8											
10.4	END OF CONE TEST										

Cone Test Below Augers

50/5 cm and Bouncing

+3, x5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 4 METRIC

W P 145-74-05 LOCATION STA 9+935.0 ℄ CREEK DIVERSION ORIGINATED BY CM

DIST 9 HWY 16 N BOREHOLE TYPE Cone Penetration Test COMPILED BY BD

DATUM ⊥ Geodetic DATE 1981 11 25 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, x LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						GR SA SI CL
90.0	Ground Surface										
0.0	Probably CLAY						88, 86				
85.3											
4.7	Probably SILT						84, 82				
81.5											
8.5	Probably SAND and GRAVEL (Glacial Till)										
80.3											
9.7	END OF CONE TEST							60/23 cm and Bouncing			

+³, x⁵ : Numbers refer to Sensitivity

15 ⟡ 5 (%) STRAIN AT FAILURE (20, 10)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 5

METRIC

W P 145-74-05 LOCATION STA 22+949.0 % 5.5m Rt of ℄ Hwy 16N ORIGINATED BY CM

DIST 9 HWY 16N BOREHOLE TYPE Cone Penetration Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 25 CHECKED BY CM

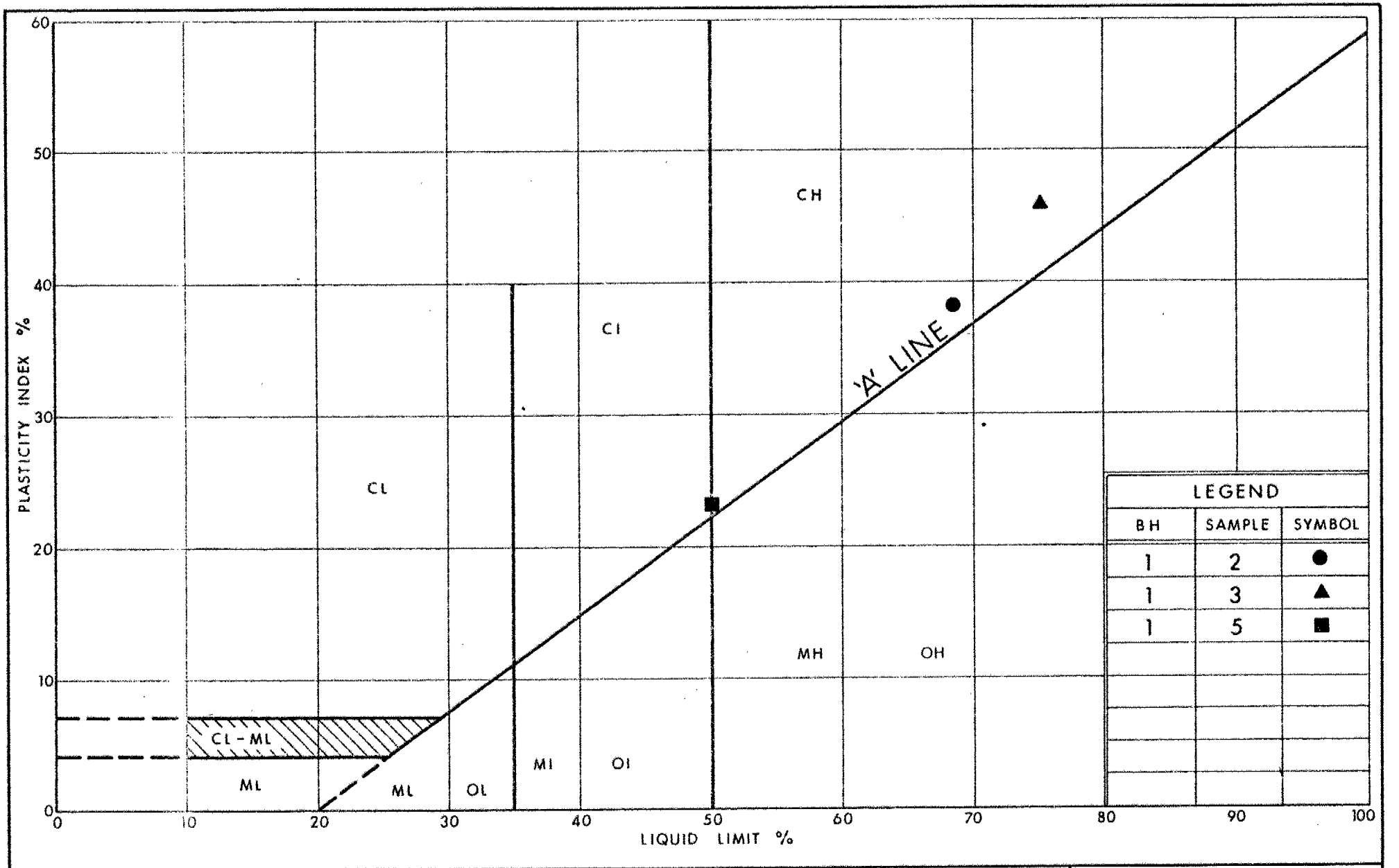
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH: O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, x LAB VANE	WATER CONTENT (%)		GR SA SI CL
89.5	Ground Surface										
0.0	Probably CLAY						88, 86				
84.7											
4.8	Probably SILT						84, 82, 80				
79.0											
6.5	Probably SAND and GRAVEL (Glacial Till)						78				
77.4											
12.1	END OF CONE TEST							63/20 cm and Bouncing			

+3, x5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Oct 75, FF-S-21

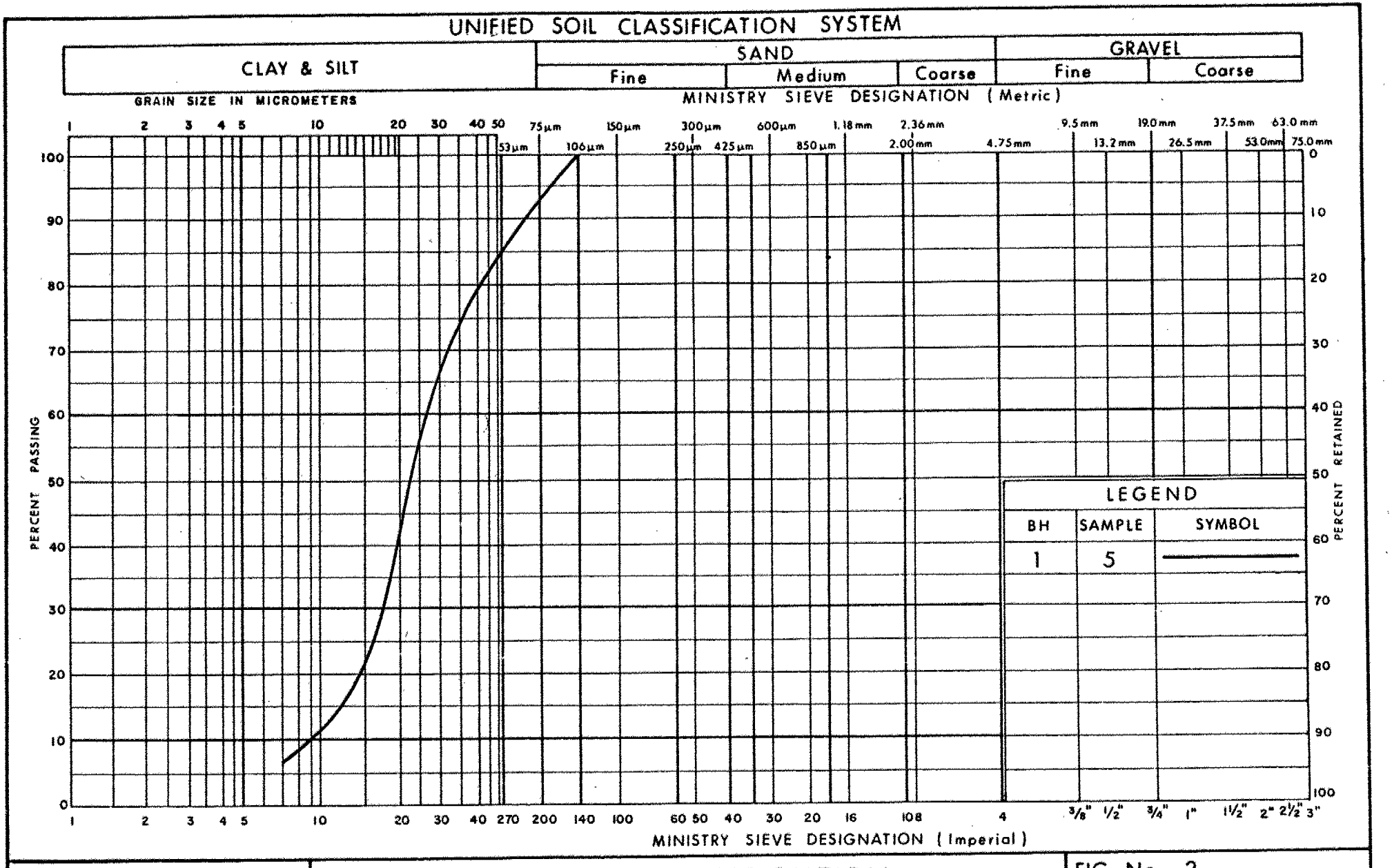


Ministry of Transportation and Communications
Ontario

PLASTICITY CHART
CLAY

FIG No 1
W P 145-74-05
Mud Creek

78 12 M



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GRAIN SIZE DISTRIBUTION

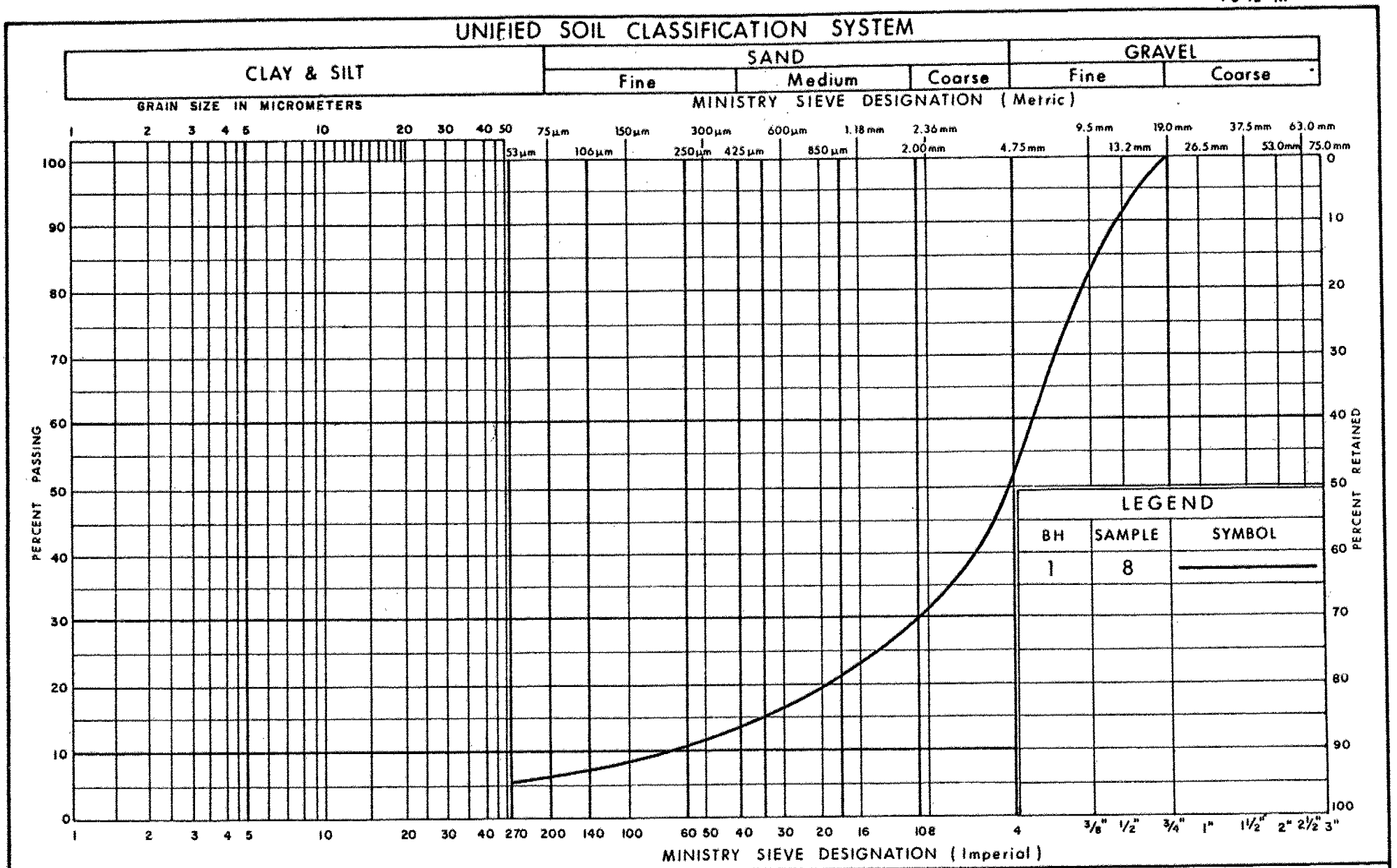
SILT

FIG No 2

W P 145-74-05

Mud Creek

78 12 M



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Ontario

GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL (Glacial Till)

FIG No 3
W P 145-74-05
Mud Creek

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CONT No
WP No 145-74-05

MUD CREEK

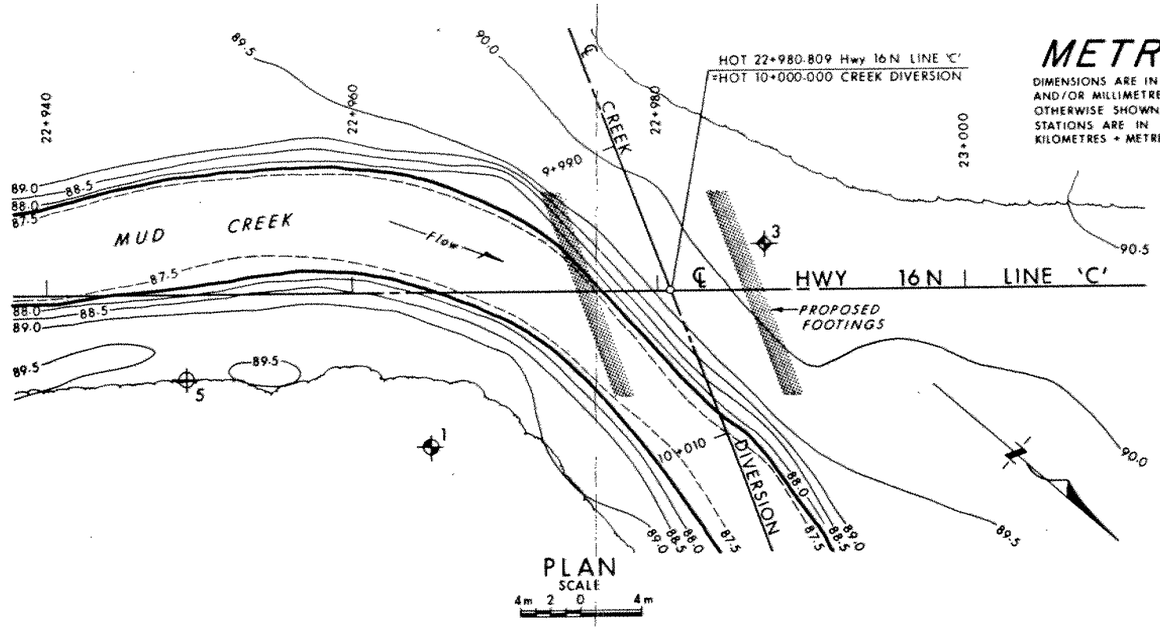
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

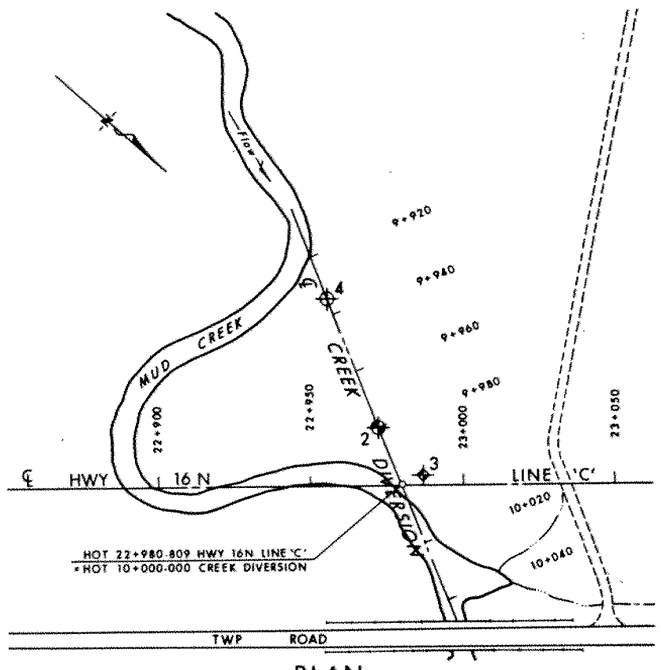
Warnock Hersey Professional Services Ltd.

METRIC

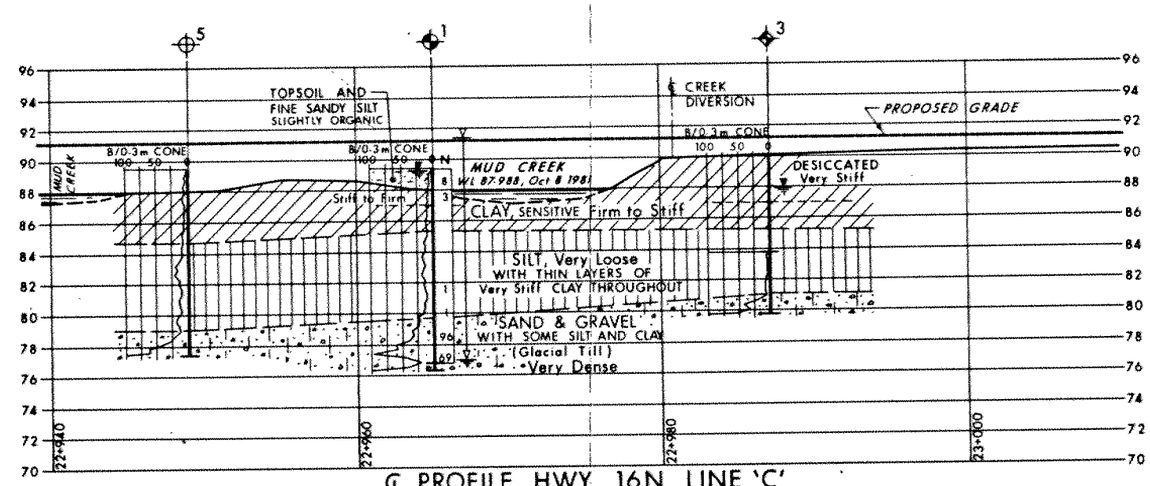
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.



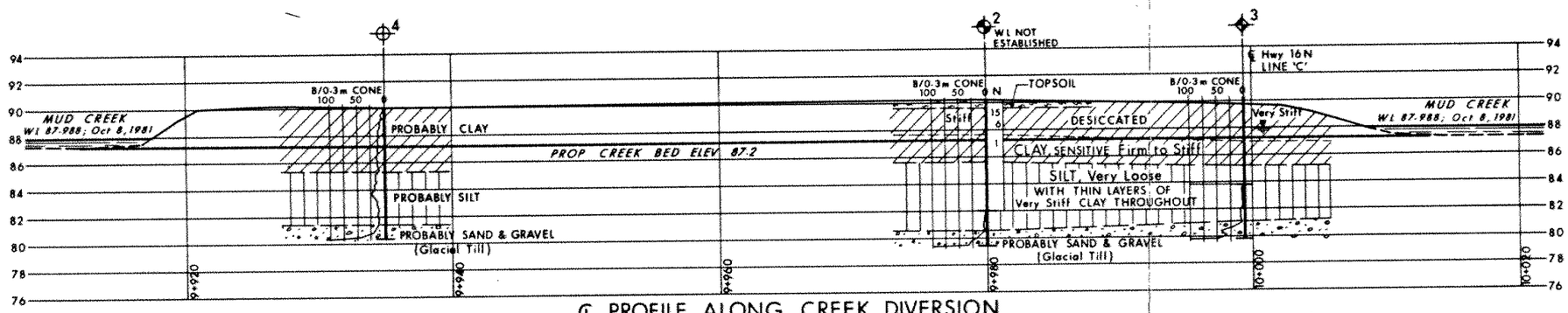
PLAN
SCALE
4m 2 0 4m



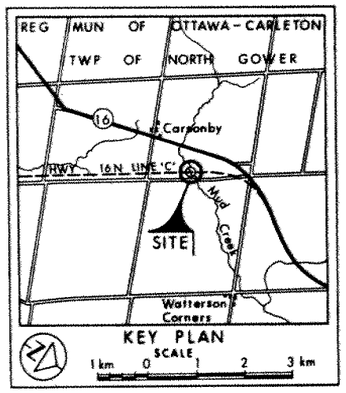
PLAN
SCALE
20m 10 0 20m



℄ PROFILE HWY 16N LINE 'C'
SCALE
4m 2 0 4m



℄ PROFILE ALONG CREEK DIVERSION
SCALE
4m 2 0 4m



KEY PLAN
SCALE

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 1981 11
  W L Not Established in BH# 2
- Vane Test & Cone
- Head
- ARTESIAN CONDITIONS Encountered

No	ELEVATION	Hwy 16N LINE 'C' STATION	OFFSET
1	89.4	22+965.0	10.0 m Rt
3	90.2	22+987.0	3.0 m Lt
5	89.5	22+949.0	5.5 m Rt
		MUD CREEK DIVERSION STATION	OFFSET
2	90.2	9+980.0	℄
4	90.0	9+935.0	℄

NOTE

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

REVISIONS

DATE	BY	DESCRIPTION

Geocres No

HWY No 16N			DIST 9
SUBM'D C M	CHECKED	DATE 1981 12 21	SITE 3-358
DRAWN J T	CHECKED	APPROVED	DWG 1457405-A

REF No E-7003-1; 1981 11

ENGINEERING MATERIALS OFFICE
PAVEMENT & FOUNDATION DESIGN SECTION

WP 145-74-05 DIST 9

HWY 16N STR SITE 3-358

Mud Creek Structure

CONT 83-15

DISTRIBUTION

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GEOCRES 31G-191 DATE

FOUNDATION INVESTIGATION REPORT (ADDENDUM)

For

W.P. 145-74-05, Site 3-358
Mud Creek Structure
Hwy. 16N, District 9, Ottawa

INTRODUCTION

This addendum summarizes the results of a foundation investigation required for a proposed structure at the above-mentioned site.

Additional fieldwork was carried out on 82 12 08 (B.H. 101 & 102). Data from previous fieldwork (B.H. 1 to 5) conducted at this site between 81 11 24-25 has been incorporated into this addendum. Total fieldwork consisted of 2 sampled boreholes, 2 dynamic cone penetration tests, 1 continuous vanes test, and 3 sampled borehole with accompanying cone penetration test.

This addendum provides additional data and recommendations required for the design of the proposed structure. However, information contained in the original foundation investigation and design report (dated 81 12 24) remains valid.

SUBSURFACE CONDITIONS

General

Refer to the Appendix Record of Borehole Sheets and updated Drawing 1457405-A.

Extending from the ground surface downwards, subsurface conditions at this site consist of a surficial slightly organic layer of 1.3 to 1.5 metres in thickness. This layer is followed by a 3.0 to 4.5 m stratum of sensitive clay of high plasticity. The clay strata is underlain by a 4.1 m to 4.3 m thick silt stratum containing thin clay seams.

.../2

Glacial till of undetermined thickness underlies the silt stratum. Auger refusal on what is probably boulders in the glacial till stratum was encountered at about 13.2 metre depth (elevation 77.2) at the north abutment location and at approximately 12.0 metre depth (elevation 78.0) at the south abutment location.

Surficial Soils

Surficial soils consist mainly of topsoil and some organically contaminated silty sand. The maximum depth of this material was 1.5 metres at the north abutment location. The denseness of this material can be assessed as being loose throughout.

Clay

Underlying the topsoil and silty sand, all borings encountered a 3.0 to 4.5 m thick deposit of clay, the upper 1.5 metres of which is desiccated.

The desiccated clay zone exhibits a mottled brown to grey colour and some degree of fissuring.

In situ vane tests as well as an undrained quick triaxial test gave shear strength values in excess of 70 kPa, indicating a generally stiff consistency. Natural moisture content determinations on the desiccated clay gave an average value of 40 percent.

Below the desiccated zone, the colour of the clay changes to grey and its consistency is reduced to firm. The clay is homogeneous in texture and sensitive to remoulding. Physical properties of this material, as determined from field and laboratory tests, are summarized below:

.../3

		Range	Average
Natural Moisture Content	(w) %	82-85	83
Liquid Limit	(w_L)%	50-75	65
Plastic Limit	(w_p)%	22-31	27
Plasticity Index	(I_p)%	32-46	38
Shear Strength	(kPa)		
- field vane (undisturbed)		31-57	37

A plot of plasticity index vs. liquid limit (Fig. 1) indicates the clay to be of high plasticity.

Silt

The clay stratum is underlain by a silt deposit which ranges from 4.1 to 4.3 m in thickness. The silt is grey in colour and very dilantent. A typical grain-size distribution curve is shown in Figure 2.

Present within the silt deposit are clay bands which appear at periodic intervals, and which are very stiff in consistency. These bands are typically about 5 to 10 mm in thickness, and occur at intervals of 30 to 70 mm.

The natural moisture content of the silt averaged approximately 26 percent, and based on N values of 1 to 2 blows/0.3 m is considered to be in a very loose state throughout.

Increasing amounts of sand and gravel occur within the silt stratum at approximate elevation 80.5.

Standard Penetration testing within this layer measured 'N' values of 3 to 9 blows/0.3 m, indicating a very loose to loose state of denseness.

Glacial Till

Beneath the silt stratum, borings encountered glacial till of undetermined thickness, with all borings being terminated after penetrating 0.7 to 2.7 m into the glacial till. The till consists of a silty clay of low plasticity, sand and gravel. Standard Penetration 'N' values indicate a hard consistency within the clay, and a compact to very dense state within the sand and gravel portions of the till stratum.

Borehole 101 at the north abutment location encountered auger refusal on probable boulders within the glacial till at about 13.2 metre depth (elevation 77.2). Borehole 102, at the south abutment location, net refusal to augering at 12.0 metre depth (elevation 78).

Groundwater

Groundwater levels at the site were found to correspond closely to the water level of the adjacent Mud Creek. However, at B.H. 102, groundwater was encountered once the borehole penetrated the sand and gravel layer, and appeared to stabilize at approximately creek level. In addition, at B.H. 1 south of Mud Creek, an artesian condition was noted upon penetration of the sand and gravel layer, and was estimated at approximately 2 metres above ground surface.

DISCUSSION AND RECOMMENDATIONS

It is proposed to construct a single span structure over the realigned Mud Creek on the new Hwy. 16N alignment. The structure span will be 16 metres by 13.5 metres. Abutment foundations may be supported on Steel H piles equipped with reinforced tips and driven in accordance with M.T.C. Standards SS103-10, 11. The following design values are recommended:

Pile Type	Factored Capacity at U.L.S.	Capacity at S.L.S. Type II
310 HP 110	1600 kN	1150 kN
310 HP 79	1150 kN	820 kN

For estimation purposes, it may be assumed that the recommended pile capacities will be achieved at or below elevation 76.0 metres.

Since artesian conditions were encountered upon penetration into the sand and gravel (glacial till stratum), a means of dissipating artesian flow pressures and the prevention of piping and erosion will be required. It is recommended that a granular filter drainage blanket be placed below the pile cap. The thickness of the blanket should be approximately 450 mm. Since the drainage blanket will be situated below high water level, protection with rip-rap against erosion by flooding should be provided.

Earth pressures against the abutment walls may be computed as per Subsection 6.6.1.2.2. of the O.H.B.D.C. Manual.

For frost protection purposes, all pile caps should be provided with a minimum of 2 metres of earth cover.

All topsoil and surficial organic material should be removed from the plan limits of the approach fills.

In addition, refer to the original investigation and design report for this W.P. for additional discussion and recommendations.

MISCELLANEOUS

The fieldwork for the addendum portion of this project was carried out under the supervision of Mr. K. Chak, Trainee Engineer, utilizing equipment owned and operated by F.E. Johnston Drilling Ltd., Ottawa. The report was written by Mr. Chak and reviewed by Mr. M. Devata, Senior Foundations Engineer.

K.D. Chak
Trainee Engineer

M. Devata, P. Eng.
Senior Foundations Engineer

APPENDIX

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 1

METRIC

W P 145-74-05 LOCATION STA 22+965.0 % 10.0m Rt of ℄ Hwy 16 N ORIGINATED BY CM

DIST 9 HWY 16N BOREHOLE TYPE Hollow Stem Auger and Cone Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 24 and 25 CHECKED BY CM

SOIL PROFILE ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (10 20 30 40 50)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
						Artesian Head Approx. 2m Above Ground					
89.4	Ground Surface										
0.0	TOPSOIL and Fine SANDY SILT Slightly Organic, trace of Clay		1	SS	8						
88.0							88				
1.4	Desiccated Stiff to Firm		2	SS	3			+6			
87.0											
2.4	CLAY, Sensitive Firm to Stiff Grey		3	TW	PM		86	+5		81.7%	
85.4											
4.0	SILT, Very Loose with thin Very Stiff Clay bands at 30 to 70 mm intervals Becoming Soft cohesive Silt with depth		4	TW	PM		84	+4			
			5	TW	PH						0 8 (92)
			6	SS	1		82	+4			
79.7			7	SS	1		80				
9.7	SAND AND GRAVEL with some Silt and Clay (Glacial Till) Very Dense		8	SS	96		78				48 46 (6)
76.8			9	SS	69	Artesian condition Encountered					
76.3	END OF BOREHOLE										
13.1	END OF CONE TEST							100/25 cm Cone Refusal at 13.1m depth			

+3, ×5 : Numbers refer to Sensitivity

15 ⟡ 5 (20, 10) (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL LABORATORY

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 2 — METRIC

W P 145-74-05 | LOCATION STA 9+971·8 % 22·5 m Rt ℄ Creek Diversion | ORIGINATED BY C M
DIST 9 HWY 16 N | BOREHOLE TYPE Hollow Stem Auger and Cone Test | COMPILED BY B D
DATUM Geodetic | DATE 1981 11 25 | CHECKED BY C M

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES
90·2	Ground Surface			
0·0 / 89·6	TOPSOIL			
0·6	Desiccated, Stiff Mottled Brown – Grey	1	SS	15
		2	SS	6
87·7 / 2·5	CLAY, Sensitive Firm to Stiff Grey	3	SS	1
85·6 / 4·6	SILT, Very Loose with thin Clay bands at 70 mm intervals Becoming Soft cohesive Silt with depth	4	TW	PH
		5	TW	PM
		6	TW	PM
82·0		6A	SS	PH
8·2	END OF BOREHOLE Probably SILT			
80·5 / 9·7	Probably SAND and GRAVEL (Glacial Till)			
79·4 / 10·8	END OF CONE TEST			

* Water Level Not Established

GROUND WATER CONDITIONS: *

ELEVATION SCALE: 90, 88, 86, 84, 80

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE: 10 20 30 40 50

PLASTIC LIMIT Wp — NATURAL MOISTURE CONTENT W — LIQUID LIMIT WL

WATER CONTENT (%): 20 40 60

84·8%

Cone Test Below Augers

100/15 cm

UNIT WEIGHT γ

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

+3, ×5 : Numbers refer to Sensitivity

15 20 10 5 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 3

METRIC

W P 145-74-05 LOCATION STA 22+987.0 %s 3.0 m Lt of ℄ Hwy 16N ORIGINATED BY C M

DIST 9 HWY 16 N BOREHOLE TYPE Augers ; Continuous Vanes and Cone Test COMPILED BY B D

DATUM Geodetic DATE 1981 11 25 CHECKED BY C M

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (10 20 30 40 50)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT Wl — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						GR SA SI CL
90.2	Ground Surface										
0.0	Desiccated Very Stiff						90				
87.0							88				
3.2	CLAY, Sensitive Firm to Stiff Grey						86				
85.2											
5.0	SILT, Probably banded with Clay Seams						84				
83.8											
6.4	END OF VANE TEST										
	Probably SILT						82	Cone Test Below Augers			
81.1											
9.1	Probably SAND and GRAVEL (Glacial Till)						80				
79.8											
10.4	END OF CONE TEST							50/5 cm and Bouncing			

+3, x5 : Numbers refer to Sensitivity

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

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 4

METRIC

W P 145-74-05 LOCATION STA 9+922.5 % 9.8 m Rt ℄ Creek Diversion ORIGINATED BY CM

DIST 9 HWY 16 N BOREHOLE TYPE Cone Penetration Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 25 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
90.0	Ground Surface										
0.0	Probably CLAY						88, 86				
85.3 / 4.7	Probably SILT						84, 82				
81.5 / 8.5	Probably SAND and GRAVEL (Glacial Till)										
80.3 / 9.7	END OF CONE TEST							60/23 cm and Bouncing			

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

15 ⊕ 20 5 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 5

METRIC

W P 145-74-05 LOCATION STA 22+949.0 % 5.5m Rt of ℄ Hwy 16N ORIGINATED BY CM

DIST 9 HWY 16N BOREHOLE TYPE Cone Penetration Test COMPILED BY BD

DATUM Geodetic DATE 1981 11 25 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
89.5	Ground Surface										
0.0	Probably CLAY						88, 86				
84.7											
4.8	Probably SILT						84, 82, 80				
79.0											
6.5	Probably SAND and GRAVEL (Glacial Till)						78				
77.4											
12.1	END OF CONE TEST							63/20 cm and Bouncing			

+3, x5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 101

METRIC

W P 145-74-05 LOCATION Sta. 23 + 005.0 o/s 5.5 m Lt. ℄ Hwy. 16N ORIGINATED BY KC

DIST 9 HWY 16N BOREHOLE TYPE Hollow Stem Auger COMPILED BY KC

DATUM Geodetic DATE 82 12 08 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.4	Ground Surface										
0.0	Desiccated Stiff to Firm		1	TW	PH		90				
88.0							88				
2.4	Clay, Sensitive Firm Grey		2	TW	PM						
86.4							86				
4.0	Silt, Very Loose with thin Very Stiff Clay bands at 30 mm to 70 mm intervals Becoming Soft cohesive Silt with depth		3	TW	PH						
			4	TW	PH		84				
			5	SS	2		82				
	increasing Sand & Gravel		6	SS	3						
80.0							80				
10.4	Silty Clay of low plasticity with Sand, trace of Gravel (Glacial Till) Hard Grey		7	SS	100/	15 cm					
77.2			8	SS	100/	18 cm	78				
13.2	End of Borehole Refusal to Augering Probable Boulders										

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

15 ⊹ 5 (%) STRAIN AT FAILURE (20, 10)

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 102 METRIC

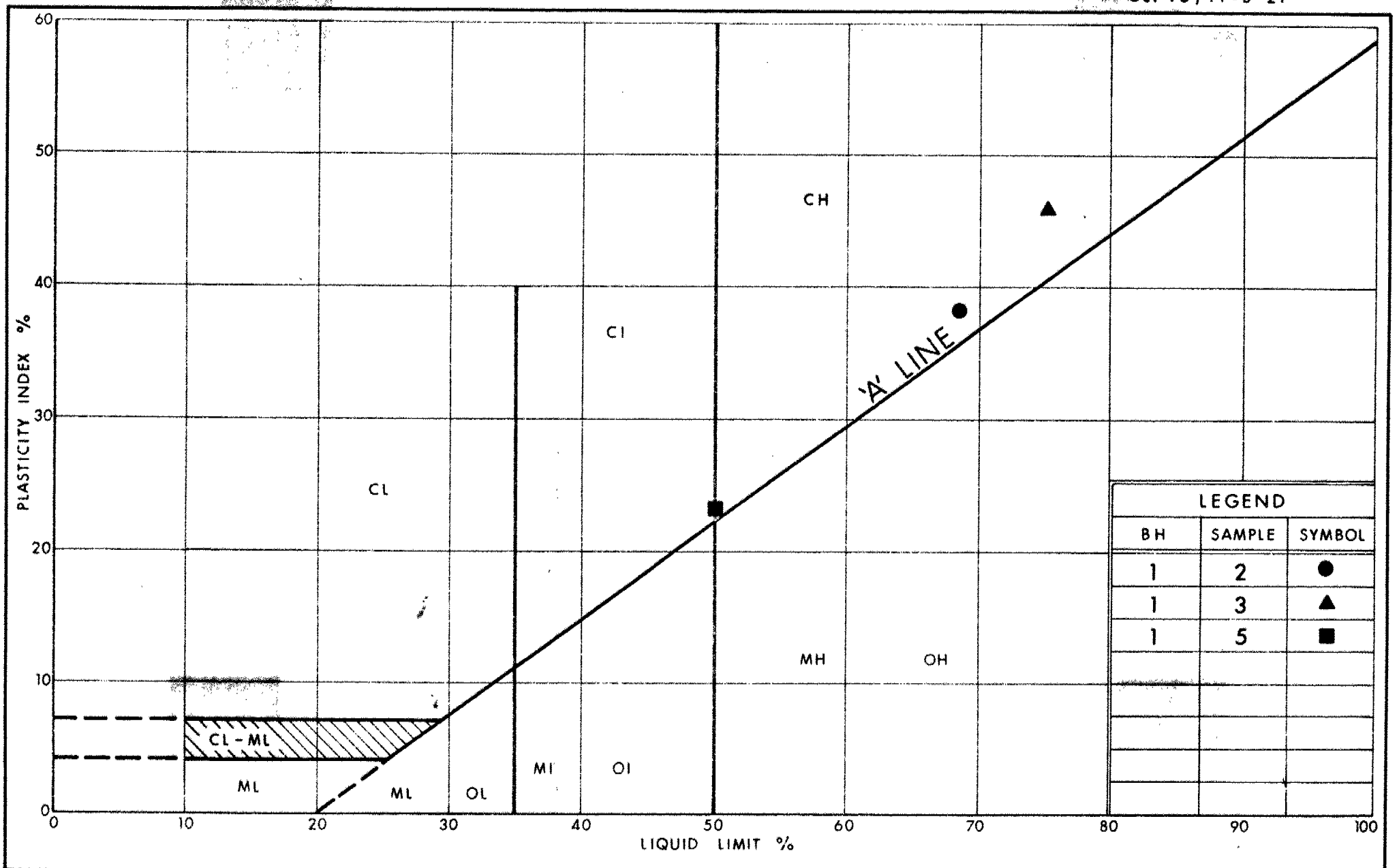
W P 145-74-05 LOCATION Sta. 22 + 988.0 o/s 5.5 m Rt. ℄ Hwy. 16N ORIGINATED BY KC
DIST 9 HWY 16N BOREHOLE TYPE Hollow Stem Auger COMPILED BY KC
DATUM Geodetic DATE 82 12 08 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, x LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.0	Ground Surface										
0.0	Topsoil and Fine Sandy Silt, slightly*		1	SS	6						
89.1			2	SS	10			+7			
0.9	Desiccated Stiff to Firm						88				
87.6			3	SS	2	▼					
2.4	Clay, Sensitive Firm Grey						86				
85.3											
4.7	Silt, Very Loose with thin Very Stiff Clay Bands at 30 mm to 70 mm intervals Becoming Soft cohesive Silt with Depth						84				
			4	SS	2		82				
			5	SS	9						
79.9	Increasing Sand & Gravel						80				
10.1	Sand & Gravel (Glacial Till) Compact to Very Dense		6	SS	27						
78.0											
12.0	End of Borehole Refusal to Augering Probable Boulders										

*organic
Trace of Clay

+3, x5 : Numbers refer to Sensitivity

15 20 10 5 (%) STRAIN AT FAILURE



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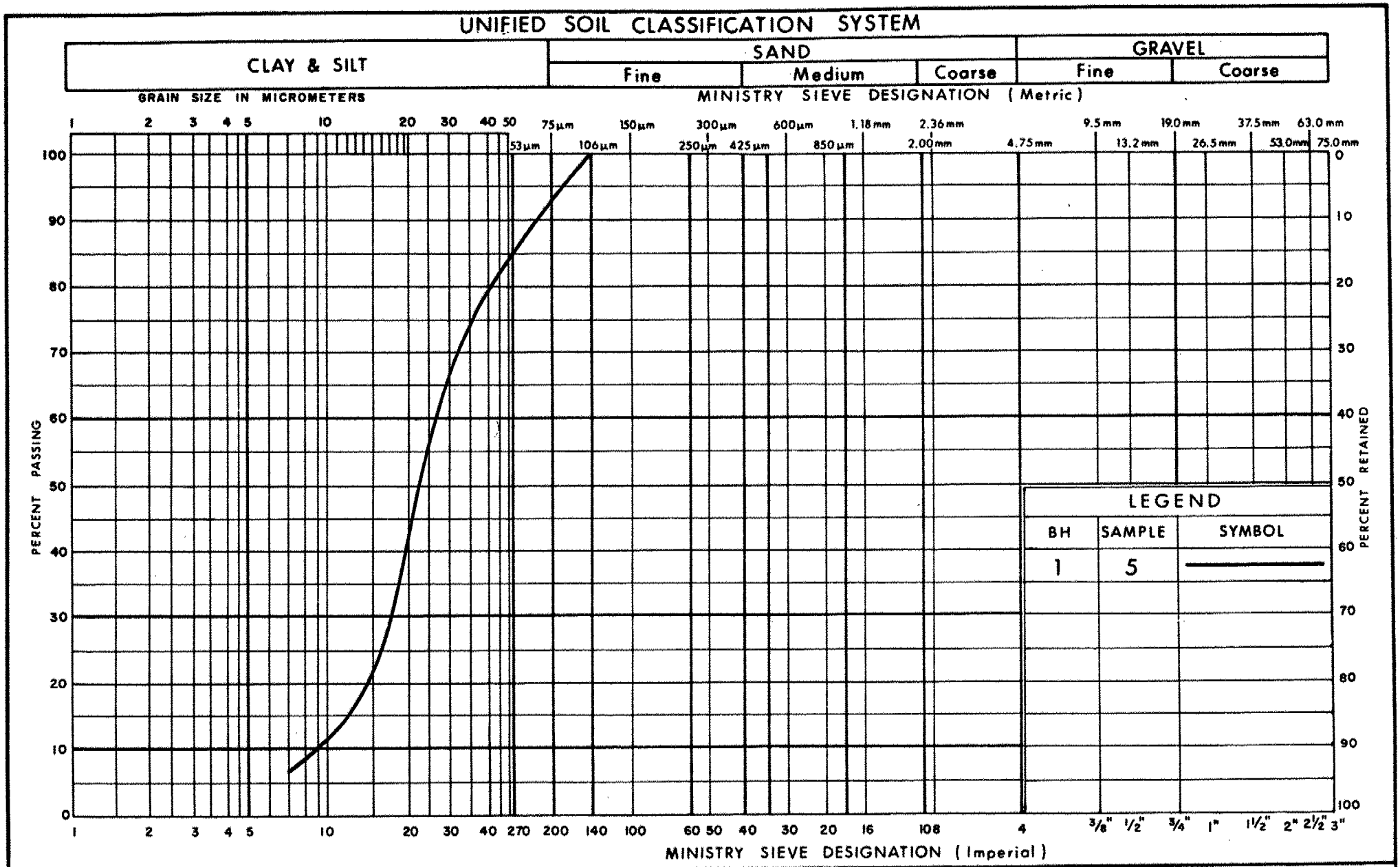
Ontario

PLASTICITY CHART
CLAY

FIG No 1
W P 145-74-05
Mud Creek

78 12 M



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Ontario

GRAIN SIZE DISTRIBUTION

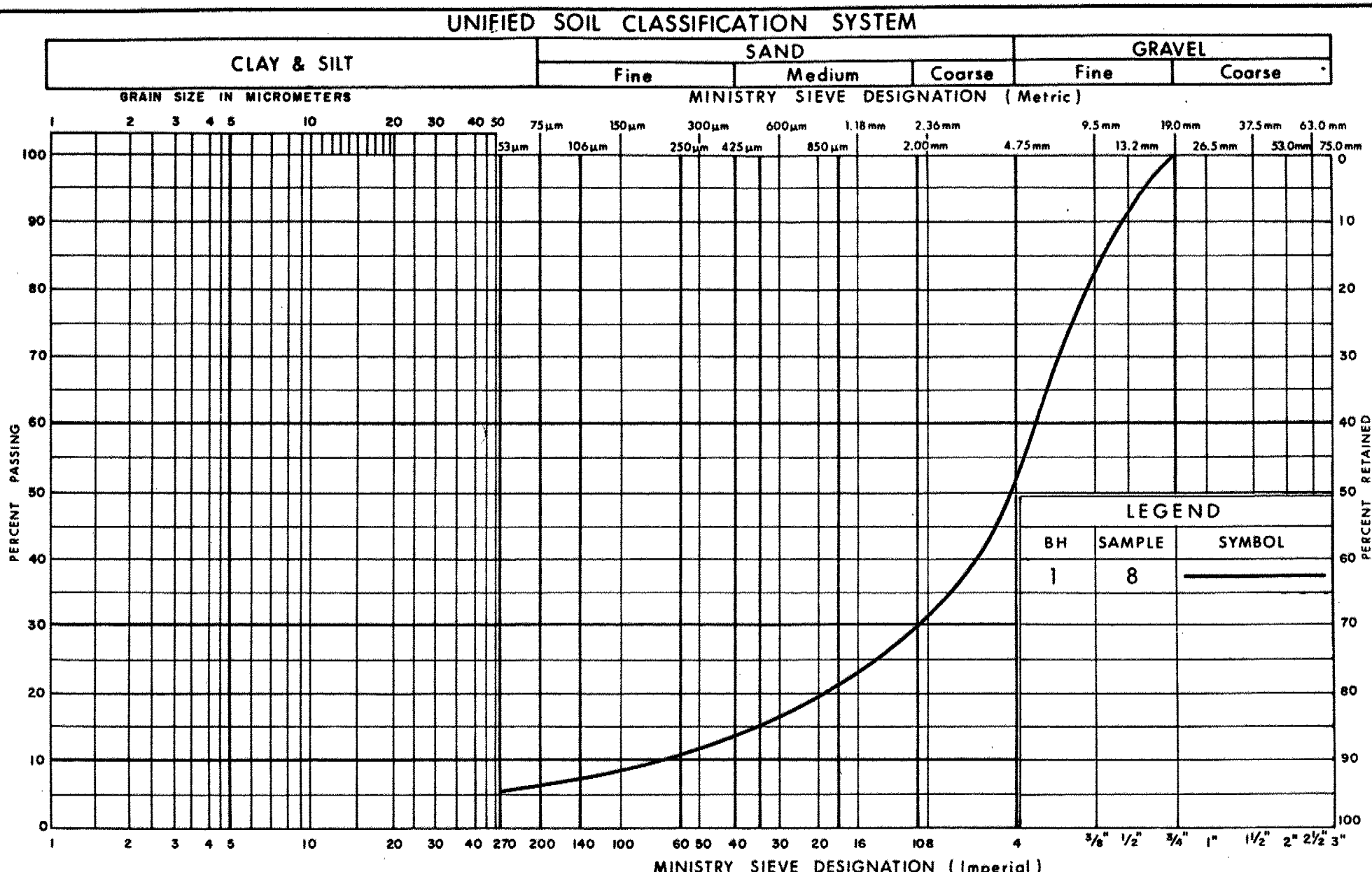
SILT

FIG No 2

W P 145 – 74 – 05

Mud Creek

78 12 M



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GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL (Glacial Till)

FIG No 3
W P 145-74-05
Mud Creek

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:

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
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 AND COMMUNICATIONS, ONTARIO 08-MT-308M 6-78

CONT No
WP No 145-74-05

MUD CREEK

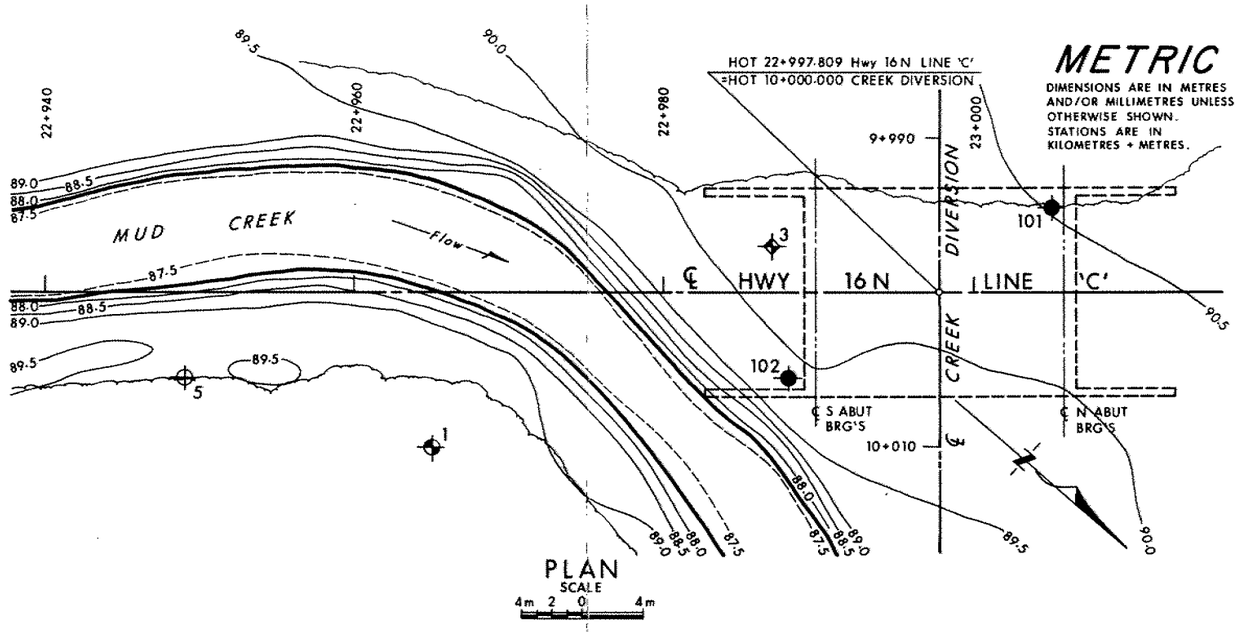
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

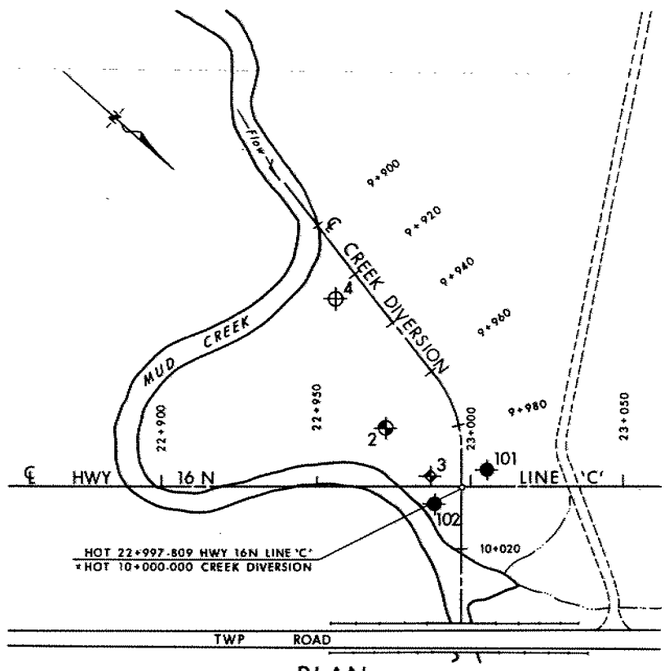
Warnock Hersey Professional Services Ltd.

METRIC

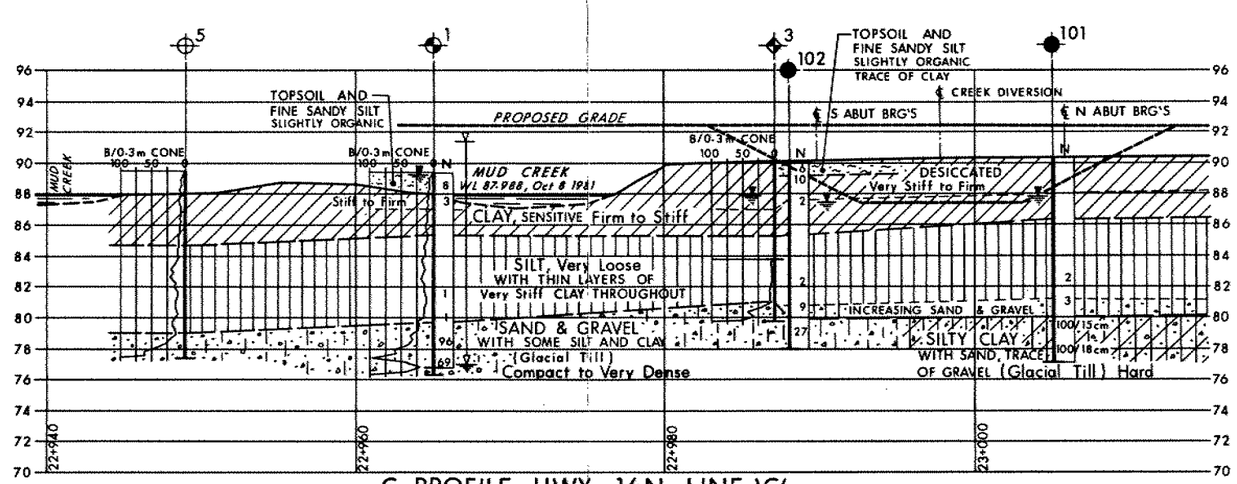
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.



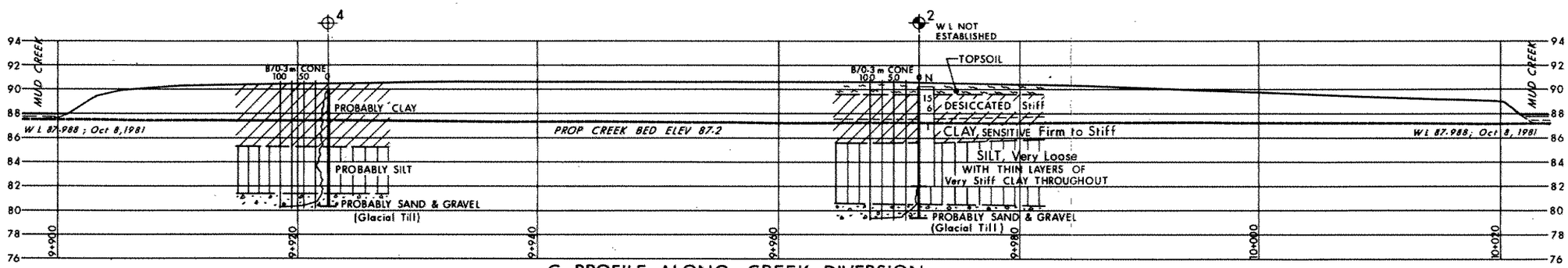
PLAN
SCALE
4m 2 0 4m



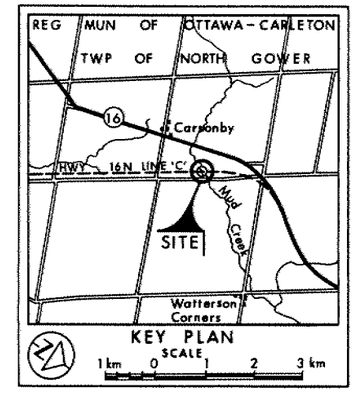
PLAN
SCALE
20m 10 0 20m



℄ PROFILE HWY 16N LINE 'C'
SCALE
4m 2 0 4m



℄ PROFILE ALONG CREEK DIVERSION
SCALE
4m 2 0 4m



KEY PLAN
SCALE

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 1981 11
  WL for BH 101 & 102 ; 1982 12
- Vane Test & Cone
- Head
- ARTESIAN CONDITIONS Encountered

No	ELEVATION	Hwy 16N STATION	LINE 'C' OFFSET
1	89.4	22+965.0	10.0 m Rt
3	90.2	22+987.0	3.0 m Lt
5	89.5	22+949.0	5.5 m Rt
101	90.4	23+005.0	5.5 m Lt
102	90.0	22+988.0	5.5 m Rt

		MUD CREEK DIVERSION STATION	OFFSET
2	90.2	9+971.8	22.5 m Rt
4	90.0	9+922.5	9.8 m Rt

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

REVISIONS		
83 03 4	G P	BOREHOLES 101 & 102 ADDED ON PLAN & PROFILE
DATE	BY	DESCRIPTION

Geocres No 31G-191

REF No E-7003-1 ; 1981 11

HWY No 16N			DIST 9
SUBM'D C M	CHECKED	DATE 1981 12 21	SITE 3-358
DRAWN J T	CHECKED	APPROVED	DWG 1457405-A

DOCUMENT MICROFILM[redacted] IDENTIFICATION

GEOCRES No. 31G5-191

DIST. 42 REGION

W.P. No.

CONT. No. 98-609/616

W. O. No.

STR. SITE No.

HWY. No. 417

LOCATION West Transitway
AT RICHMOND RD

No OF PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

GEOCRES No:
31G5-191

GEOTECHNICAL INVESTIGATION REPORT

PROPOSED WEST TRANSITWAY AT RICHMOND ROAD
OTTAWA, ONTARIO

PROJECT NO. 10923

REPORT TO

MORRISON HERSHFIELD LIMITED

ON

GEOTECHNICAL INVESTIGATION
PROPOSED WEST TRANSITWAY AT RICHMOND ROAD
OTTAWA, ONTARIO

Jacques Whitford Limited
2781 Lancaster Road, Suite 200
Ottawa, ON K1B 1A7
Tel:(613)738-0708
Fax:(613)738-0721

February 16, 1998

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Appendix 1 Key Plan
Borehole Location Plan

Appendix 2 Symbols and Terms Used on Borehole and Test Pit Records
Borehole Records

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the design of the proposed Regional Municipality of Ottawa-Carleton (RMOC) West Transitway from Stations 6+740 to 7+070. This section of the West Transitway project includes the crossing of Richmond Road, construction of a new N-W ramp to the Queensway, and construction of 330 m of Transitway, beginning in the City of Nepean and ending in the City of Ottawa. The work was carried out in accordance with our proposal dated May 1, 1997. Authorization to proceed with the investigation was provided by Mr. John Douglas of Morrison Hershfield Limited.

This report has been prepared specifically and solely for the proposed work described herein. It contains all of our findings and includes the necessary geotechnical recommendations for the design of the section of the West Transitway described above, plus the associated ramps, structures and underground services.

2.0 BACKGROUND

As part of the 1994 Functional Design of the West Transitway, Jacques, Whitford Limited (JWL) prepared a general overall assessment of the geotechnical and environmental considerations based on the then existing information, supplemented by a limited number of boreholes.

The current investigation was carried out to obtain the necessary geotechnical information for the preliminary and detail design of the following section of the West Transitway:

- Transitway (Station 6+740 to 7+070) and associated works
- Richmond Road Bridge
- New N-W Ramp, approximately 250 m in length from Richmond Road to the Queensway
- N-W Ramp Bridge

Based on the Functional Design and discussions with Morrison Hershfield, the associated works is to include the following:

- Removal of the existing N-W ramp, approximately 350 m in length
- Storm sewer installation along the Transitway alignment
- Retaining walls associated with the realigned N-W ramp, between the West Transitway and the Queensway, as well as in front of the abutments for the Richmond Road bridge structure.

3.0 SCOPE OF WORK

The scope of work for the geotechnical investigation included the following:

- A field and laboratory investigation of the soil, bedrock and groundwater conditions at the site in order to provide the necessary foundation design parameters (as per OHBDC), and the pavement structure design for the preliminary and detail design of the section of the West Transitway described herein.
- A geotechnical investigation report complete with borehole records, all the findings of the investigation, and the necessary geotechnical recommendations for the preliminary and detail design of the proposed project, as well as for the preparation of the tender documents.

4.0 METHODOLOGY

4.1 Geotechnical Investigation

Prior to the start of the field investigation, a plan showing the proposed borehole locations was submitted to Morrison Hershfield Limited for approval. The borehole location plan was subsequently provided to the Regional Municipality of Ottawa-Carleton (RMOC) and the Ministry of Transportation of Ontario (MTO) for their review and approval. The borehole locations were laid out in the field after the required work permit was issued by MTO. The appropriate utility agencies were notified and the borehole locations were cleared of underground utilities.

A site meeting was held with representatives from Morrison Hershfield, the RMOC and MTO to review the proposed borehole locations. MTO expressed concerns regarding disruptions to traffic, traffic control safety, site access and drilling through existing MTO pavement structures. Many of the borehole locations were adjusted in order to comply with the work restrictions set out by MTO and due to the presence of underground utilities. Proposed Boreholes M-14, M-15 and M-17 were cancelled due to MTO restrictions. Borehole M-21 was drilled by Golder Associates for the adjacent West Transitway section. An agreement was made with the consulting team for the adjacent Transitway section to share borehole information at this location.

A total of nineteen (19) soil borings were put down during the field investigation. The borings were located as follows:

Element	No. of Borings	Boring Designations
Richmond Road bridge	5 Boreholes 2 Probeholes	M-1 to M-4 and M-21 M-20 and M-22
N-W ramp bridge	3 Boreholes	M-5 to M-7
N-W ramp	6 Boreholes	M-8 to M-13
Transitway, stn. 6+740 to 7+070	3 Boreholes	M-16, M-18 and M-19

The boreholes were advanced using a truck mounted CME 55 power auger drill rig suitably equipped for soil and bedrock sampling. Soil samples were recovered from the split-spoon sampler while carrying out Standard Penetration Tests (SPT). The subsurface conditions were identified in the field by our personnel and all samples were returned to our laboratory for detailed classification and testing. The subsurface conditions are described in detailed in the Borehole Records presented in Appendix 1. Standpipes were installed in Boreholes M-5 through M-8, M-10, M-11 and M-16. A monitoring well was installed in borehole M-19 to allow for borehole permeability testing to be carried out.

Prior to completing the investigation, the boreholes were backfilled by replacing (and tamping in layers) the augered material. The asphalt surface, where present, was reinstated to match the level of the existing surface with at least 100 mm of cold mix asphalt.

All soil samples will be stored for a period of three months after issuance of the final report. Unless otherwise directed, the stored samples will be disposed of after this period.

5.0 INVESTIGATION RESULTS

5.1 Surface Conditions

General

The investigated section of the proposed West Transitway and associated works lies entirely within the existing Richmond Road - Hwy 417 (Queensway) WBL interchange area. The majority of the ground surface presently consists of maintained grass or existing pavement structure. A large, heavily treed area is present east of Richmond Road between Stations 6+920 and 7+040 (approximate). Exposed bedrock is evident in the ditch along the north side of the Hwy 417 WBL between Stations 6+890 and 6+980 (approximate). The slopes of the Richmond Road overpass approach fill are covered with grass and small bushes.

Existing Pavement Structures

Boreholes were put down within the existing pavement structure of Richmond Road and the gore area between the N-W ramp and the S-W ramp acceleration lane. Boreholes M-14, M-15 and M-17, all located within the N-W ramp or N-W ramp acceleration lane, were cancelled because MTO would not approve any borehole drilling within these pavement areas.

Based on Boreholes M-12 and M-13, the existing pavement structure within the gore area between the N-W ramp and the S-W ramp includes approximately 175 mm of asphaltic concrete. In this area, the subgrade soils are variable, ranging from silty clay to sand and gravel. Based on the Ministry of Transportation of Ontario classification, the subgrade soils range from low susceptibility to frost heave (LSFH) to high susceptibility to frost heave (HSFH).

Based on Boreholes M-1 though M-4 and M-21, the existing pavement structure within Richmond Road in the vicinity of the proposed Richmond Road bridge includes approximately 300 mm of asphaltic concrete. In this area, the subgrade soils consist of granular approach fill. The approach fill is a heterogeneous mixture of silty clay, silt, sand and gravel. Based on the Ministry of Transportation of Ontario classification, the subgrade soils would be considered as having low to medium susceptibility to frost heave (LSFH to MSFH).

5.2 Subsurface Conditions

The geology in the vicinity of the Richmond Road - Hwy 417 interchange is complex and varied. The two most significant geological features with respect to construction of the proposed West Transitway project are:

1. A dramatic change in the depth to bedrock, associated with a geological contact or discontinuity, occurs directly beneath Richmond Road. The bedrock surface elevation varies from 65.3 m beneath the northbound lanes to 47.9 m beneath the southbound lanes. In addition, the higher bedrock to the east of the discontinuity consists of sandstone with shaly partings, whereas the bedrock to the west consists of dolomite with shale seams. The discontinuity in the bedrock surface elevation also leads to very different overburden soil conditions.

2. The presence of loose, high permeability sand layers. These conditions can lead to sand and water flowing into open boreholes or excavations if control measures are not taken. The condition is commonly referred to as *running sands*. Drilling fluids and casing were required for drilling though these conditions.

The primary soil types encountered during the drilling investigation are described below.

5.2.1 Fill

A surficial layer of fill material was found at most borehole locations. In most cases, the fill is associated with the construction of the existing Richmond Road - Hwy 417 interchange. Typically the fill material immediately adjacent to paved areas consisted of sand and gravel. Within the Richmond Road approach fill, the material varied from silt and sand to silt and clay.

The moisture content of seven samples tested ranged from 7.4 % to 25.3 %, with an average of 17.4 %. The thickness of the fill deposit varied from approximately 1.5 m to 6.0 m beneath parts of Richmond Road.

5.2.2 Silt and Clay

Deposits of silt and clay were encountered in most of the deeper boreholes (M-1, M-3, M-4, M-5, M-6, M-7 and M-21) which are all directly above or west of the geological discontinuity discussed above. The deposits consisted of varying amounts of silt and clay, varying from clayey silt to silty clay, occasionally with trace amounts of sand and/or gravel. In many cases, the silt and clay deposits alternated with sand layers. The moisture contents of eight samples tested ranged from 16 % to 37 %, with an average of 28.6 %.

5.2.3 Sand / Sand and Gravel / Silty Sand

Deposits of sand, sand and gravel or silty sand were encountered in almost all boreholes. Some sand deposits were confined by layers of silt and clay. The relative density of the sand varied from loose to dense. Running sands were encountered in most boreholes where sand layers were encountered beneath the water table. The moisture contents of twelve samples tested ranged from 4 % to 23 %, with an average of 12.7 %. Grain size distribution tests were carried out on four samples. The results are summarized in the table below:

Borehole	Sample	Depth (m)	% Gravel	% Sand	% Silt and Clay
M-6	SS-7	11	0	57.7	42.3
M-6	SS-10	15.5	8.4	60.3	31.3
M-7	SS-11	15.3	0	96.2	3.8
M-7	SS-11	15.7	0	70.7	29.3

5.2.4 Glacial Till

A glacial till deposit consisting of a heterogeneous mixture of silt and sand with trace clay and gravel was encountered near the bottom of Boreholes M-4, M-5, M-19 and M-21, directly above the bedrock. In Borehole M-4, it was noted that the till deposit was considerably thicker and that a silt layer was confined within the till. In general, the till was in a compact state. Frequent boulders were encountered in the till in Borehole M-21. The moisture contents of two samples tested were 7.2 % to 19.5 %.

5.2.5 Bedrock

The bedrock conditions vary significantly across the site. The elevation of the bedrock surface drops off by over 17 m across the width of Richmond Road. In addition, the bedrock on the east side of the drop off is sandstone with shale seams and partings. To the west of the drop off, the bedrock is dolomite with shaly partings.

Exposed bedrock is evident in the ditch on the north side of the S-W ramp acceleration lane, to the east of the existing Richmond Road structure. The exposed bedrock appears to be relatively flat.

The above information suggests that a fault may lie beneath Richmond Road.

The core samples recovered from Boreholes M-1, M-2, M-4, M-18, M-19 and M-21 consisted of grey, sandstone with grey and/or green shale seams and partings. The rock quality designation (RQD) of the recovered sandstone core samples varied from 0 % to 66 %, with an average of 32 %, indicating that the bedrock is typically in a severely fractured condition. Based on geological maps, the bedrock is likely part of the Rockliffe formation.

The core samples recovered from Boreholes M-3, M-5, M-6, and M-7 consisted of medium to dark grey, dolomite with shaly partings. The rock quality designation (RQD) of the recovered core samples varied from 18 % to 92 %, with an average of 45 %. The dolomite is typically in a fractured condition. Based on geological maps of the area, the rock mass is likely part of the Oxford Formation.

5.3 Groundwater Conditions

The groundwater level was measured in the standpipes and monitoring well on August 22, 1997, and again on August 28, 1997. The depths to groundwater, measured from ground surface are presented in the table below:

Groundwater Level Readings					Approx TWY Grade
Borehole	August 22, 1997		August 28, 1997		
	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)	
M-5	7.2	62.3	7.2	62.3	± 65.0
M-6	dry to 3.9	<63.8	dry to 3.9	<63.8	± 65.1
M-7	4.7	61.8	dry to 4.9	<61.6	± 65.2
M-8	dry to 3.0	<62.6	dry to 3.0	<62.6	± 65.25
M-10	dry to 2.4	<63.8	dry to 2.4	<63.8	± 65.5
M-11	dry to 2.9	<63.2	dry to 2.9	<63.2	± 65.7
M-16	4.4	62.4	4.4	62.4	± 65.5
M-19	2.8	65.5	2.8	65.5	± 64.0

5.4 Environmental Conditions

No visual or olfactory evidence of hydrocarbon contamination was encountered during the field investigation. Environmental testing was carried out on select samples of existing granular fill material. The results of the environmental testing are presented in a separate report.

6.0 RECOMMENDATIONS

6.1 West Transitway Alignment, Stn. 6+740 to 7+070

Based on the functional design, the proposed West Transitway alignment between Stations 6+740 and 7+070 will be located within a cut section and will pass beneath Richmond Road and the proposed N-W ramp. The proposed finished grades for the West Transitway are approximately as follows:

Station	Finished Grade (m)
6+740	65.5
6+800	65.2
6+850	65
6+900	64.9
6+950	64.7
7+000	64.3
7+070	63.8

Based on the proposed grades and the borehole information, the West Transitway will be constructed within a rock cut approximately between Stations 6+840 and 7+070. The proposed finished grade of the West Transitway is close to the bedrock surface at Station 7+070. The bedrock drops off steeply to the west of Station 6+840, directly beneath Richmond Road.

6.1.1 Subgrade Preparation

It is anticipated that the full length of the West Transitway alignment for this project will be located within a cut section. In all areas, the cut material should be wasted or used as landscaping fill. Existing surficial organic topsoil will be removed as part of the general cut excavation, therefore stripping is only required if preservation of either the topsoil or the underlying material to be excavated is proposed.

An appropriate frost taper should be incorporated into the transition between the bedrock subgrade and the adjacent overburden subgrade sections to minimize differential frost action. The transition point treatments should be carried out using the applicable OPSD 205 series drawings, using a "t" depth of 1.2 m.

Where the exposed subgrade consists of existing fill material, the exposed subgrade should be proof rolled with a heavy vibratory roller under the direct supervision of experienced geotechnical personnel. Soft areas revealed during the proof rolling should be removed and replaced with approved subgrade fill. In addition, where the exposed subgrade consists of native sand and gravel, the subgrade surface should be compacted prior to placement of the granular subbase/base. Bedrock subgrade should be free of loose rock material.

Full-time inspection of the surface compaction should be carried out by qualified geotechnical personnel. If required for leveling purposes or for construction of transition point treatments, subgrade fill should consist of OPSS Select Subgrade Material (SSM), placed and compacted in lifts no greater than 300 mm thick, to 98 % Standard Proctor Maximum Dry Density (SPMDD).

6.1.2 Pavement Structure

In accordance with information provided in the RMOC Transitway Design Manual, it is understood that the projected Transitway traffic is about 1,700 articulated buses per day in each direction. In carrying out the design, the AASHTO Axle Equivalent Factor, F_A, per vehicle used was 5.0 which reflects an average of 40% of the maximum design passenger capacity of the Orion - IKARUS articulated bus. Assuming a 20 year cumulative traffic load period and anticipating a 1.5% yearly growth, the design number of ESALs used in the pavement design analysis was 72 million. The following recommended pavement structures are based on the above design assumptions using the AASHTO design method.

Station		Anticipated Subgrade	Recommended Pavement Structure	
From	To			
6+740	6+840	Sand & Gravel or Fill	40 mm	OPSS HL1
			100 mm	OPSS Heavy Duty Binder (2 lifts)[2]
			225 mm	OPSS Granular A
			850 mm	OPSS Granular B, Type II
6+840	7+070	Bedrock	40 mm	OPSS HL1
			100 mm	OPSS Heavy Duty Binder (2 lifts)
			225 mm	OPSS Granular A
			150 mm [1]	OPSS Granular B, Type II
			300 mm	rock shatter

Note: [1] The OPSS Granular B, Type II is recommended over the bedrock surface to improve the internal drainage characteristics of the pavement structure. In addition, this layer will provide additional cushion over the rock shatter to smooth out variances on the rock shatter surface which frequently reflect through to the pavement surface which affects both the ride comfort and the local pavement life.

[2] Heavy Duty Binder (HDB) is recommended, instead of the HL8 identified in the Functional Reports to provide more resistance to rutting. The articulated buses are one of the heaviest and most damaging vehicles on Ontario roadways, and the use of HDB is considered warranted, especially when considering the low price premium when compared to HL8.

It is recommended that the rock shatter be designed to extend laterally to 500 mm behind the roadway curbs.

The transition between rock cut to earth cut should be constructed in accordance with OPSD-205.05. It is recommended that a transition treatment depth, t, of 1.5 m be used in the transitions.

It is understood that the RMOC is presently using performance graded asphalt for all new construction projects. It is recommended that the liquid asphalt consist of PG 58-34.

6.1.3 Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the subbase material in a dry condition. Failure to provide adequate internal drainage under conditions of heavy wheel loading can result in the reduction of the load carrying capacity of the pavement structure.

This section of the West Transitway is within a cut, and therefore provision of longitudinal subdrains, as detailed in Drawings TD-B-03 and 04 in the Transitway Design Manual or alternatively as detailed in the RMOC standard drawing J-3.13 should be incorporated in this project. These subdrains should be placed throughout this cut section, regardless of subgrade type with the exception of rock subgrade which includes both rock shatter and ditches where the shatter is carried through to the full width of the ditch base as shown in OPSD-201.01.

Within urban sections, it is noted that subdrains have not historically been placed in areas where rock shatter is present for reasons that the rock shatter is usually graded and that any water directed towards the shatter is drained towards low points in the profile where it is then collected and disposed of. However, areas in rock cuts have frequently been poor performing at the ends, and sometimes throughout. Based on the proposed transitway profile, between Station 6+850 to 7+000, which is within the rock cut area, the grade change represents a 0.5% fall which is not considered sufficient to ensure adequate internal drainage within rock shatter.

6.2 New N-W Ramp

To accommodate the proposed Transitway, the existing N-W ramp will be demolished and replaced with a new N-W ramp which will include a bridge structure over the proposed Transitway and retaining walls between the bridge and Hwy 417. The proposed ramp is to be located to the south and east of the existing ramp. The ramp will link Richmond Road (elev. = 72.2 m) with the west bound lanes of the Hwy. 417 (elev. = 66.4 m) . It is anticipated that the proposed retaining walls and approach fill may be up to 5 m in height.

The site of the new N-W ramp was investigated by drilling nine boreholes, including the three boreholes put down for the N-W ramp structure. The three boreholes for the proposed structure were put down to bedrock. The six boreholes for the ramp pavement structure and approach fill were put down to a depth of 3.1 m (10 feet).

6.2.1 New Ramp Structure

The proposed N-W ramp will include a single lane bridge with a central pier support, located between the east and west bound lanes of the West Transitway. The soil conditions in the vicinity of the proposed bridge abutments consist of existing fill over alternating layers of sand, silt and clay, sand and gravel and glacial till overlying bedrock. The depth to bedrock varies from approximately 22.1 m (elev. = 47.4 m) to 24.4 m (elev. = 42.1 m).

The bridge abutments may be supported on driven H-piles end bearing on bedrock, as per the functional design. Due to the complex geology at this site, driven steel H-piles may encounter boulders or an uneven bedrock surface. It should be anticipated that pile lengths may vary significantly even within the same founding unit. Steel H-piles should be equipped with reinforced tips to facilitate pile penetration through boulders and to help seat the piles on uneven bedrock surfaces. The following table summarizes the bedrock elevations at the approximate abutment and pier locations:

Location	Borehole	Bedrock Elevation (m)
North Abutment	M-5	47.4
Central Pier	M-6	44.4
South Abutment	M-7	42.1

The following design parameters are recommended for vertical steel piles driven to refusal on the bedrock.

Pile Type	Factored Capacity at U.L.S. (kN)	Capacity at S.L.S (kN)
HP310x110	1600	1150
HP310x94	1300	960
HP310x79	1150	890

The above capacities are axial to the pile alignment and can be used for both vertical and battered piles. It is recommended that battered piles be equipped with pile points to avoid damaging the pile tips.

The lateral resistance of a single vertical pile should be computed in accordance with the following paper:

> Broms, B.B., 1965. "Design of Laterally Loaded Piles", J. of Soil Mech. and Found. Div., ASCE, Vol. 91, SM3: 79-99.

The following unfactored design parameters are recommended to be used for calculation of the ULS lateral resistance of vertical piles:

Location	Elevation	γ	q_u	ϕ
M-5	69.5 - 64.5	20 kN/m^3	0	30°
	64.5 - 60.2	19 kN/m^3	150 kPa	0
	60.2 - 58.8	20 kN/m^3	0	30°
	58.8 - 57.3	19 kN/m^3	150 kPa	0
	57.3 - 51.0	21 kN/m^3	0	32°
	51.0 - 47.4	21 kN/m^3	0	30°
M-6	67.7 - 64.6	20 kN/m^3	0	30°
	64.6 - 62.8	19 kN/m^3	150 kPa	0
	62.8 - 59.9	20 kN/m^3	0	30°
	59.9 - 58.5	19 kN/m^3	150 kPa	0
	58.5 - 55.0	21 kN/m^3	0	32°
	55.0 - 54.0	19 kN/m^3	150 kPa	0
	54.0 - 44.4	21 kN/m^3	0	30°
M-7	66.5 - 65.0	20 kN/m^3	0	30°
	65.0 - 60.4	21.5 kN/m^3	0	34°
	60.4 - 58.9	19 kN/m^3	150 kPa	0
	58.9 - 51.0	21.5 kN/m^3	0	34°
	51.0 - 42.1	21 kN/m^3	0	30°

For the assessment of the SLS lateral resistance of vertical piles, it is recommended that the horizontal modulus of subgrade reaction approach be carried out. Should this type of analysis be proposed, the pile layout should be reviewed by the geotechnical engineer in order that the appropriate soil/structure stiffness parameters be developed to assess the SLS lateral resistance.

If the approach fills are to be placed after pile installation, downdrag forces on piles will need to be considered in the design. Based on the bedrock depths at the boreholes, the following non-factored downdrag loads would apply:

North Abutment	370 kN
Centre Pier	530 kN
South Abutment	650 kN

The down drag loads and live load do not combine, and therefore two separate loading cases must be considered: permanent load plus down drag load, but no live load; and permanent load and live load, but no down drag load.

The above steel piles should be driven with a pile hammer delivering an energy of at least 3.5 J/mm^2 of steel cross-sectional area. The refusal criteria for piles driven with this energy has two components, as follows:

1. 20 blows for the last 25 mm of penetration; **and**
2. A total of 50 blows for not more than 100 mm of penetration.

In cases where piles do not penetrate the glacial till stratum, the pile capacity should be controlled in the field using current MTO pile driving standards. Attempts should be made in all cases to drive the piles to the bedrock surface. All pile driving activities should be inspected by qualified geotechnical personnel and the pile capacities controlled in the field using MTO pile driving standards (MTO Standards SS103-10 or SS103-11). Furthermore, at least one conventional pile load test combined with the use of a dynamic pile analyzer is recommended for this project. The proposed test pile(s) should be loaded to twice the S.L.S. Type II bearing capacity. It is recommended that the pile analyzer be used to monitor at least ten (10) piles with a minimum of two (2) piles per founding unit during pile installation.

Pile caps should be protected from frost action with a minimum soil cover of at least 1.8 m or equivalent insulation.

6.2.2 Ramp Fill and Retaining Wall

Based on the functional design, the majority of the ramp fill south and west of the N-W Ramp Bridge will be contained between two retaining walls. The maximum height of the retaining walls is expected to be approximately 5 m. The southwest retaining wall is proposed to be a reinforced earth (RECO) type wall, while the southeast wall may be a combination of RECO wall, reinforced concrete cantilever wall, and a modified concrete toe wall.

Design parameters for reinforced earth retaining walls are given in Section 6.4.

In the long term, settlement of ramp fill is expected to be no greater than 25 mm.

6.2.3 Pavement Structure

The recommended pavement structure for the proposed N-W ramp based on the MTO Routine Design Method, the estimated AADT, and the existing subgrade conditions is:

40 mm	OPSS HL1
90 mm	OPSS HL8
150 mm	OPSS Granular A
450 mm	OPSS Granular B, Type II

6.3 Richmond Road Structure

A bridge is required to carry traffic on Richmond Road over the proposed West Transitway. The functional design incorporated a split structure (northbound and southbound lane structures) with pile-supported integral abutments. A central pier may be included in the design. Due to the complex soil and bedrock conditions at the Richmond Road structure location, several alternative foundation types have been considered for the support of the bridge abutments. Design parameters for each foundation type are discussed in the sections below.

It is noted that a combination of the foundation support systems discussed below will likely be required due to the bedrock conditions at the site.

6.3.1 Abutments - Spread Footings on Bedrock

Bridge abutments may be founded on clean, level bedrock using conventional spread footings. Based on the borehole information, it is anticipated that spread footings will be used for the north abutment of the eastern structure (northeast abutment). The bedrock surface elevation in the vicinity of this abutment is approximately 65 to 66 m. Based on the functional design the finished grade of the West Transitway will be approximately 65 m. The anticipated founding elevation for the northeast abutment is therefore 64 m. The bedrock beneath the northeast abutment consists of severely fractured sandstone with shale seams and partings.

The bedrock surface elevation beneath the proposed southwest abutment is too deep (approximately 45 to 49 m) for spread footings to be practical. The bedrock surface elevation beneath the northwest and southeast abutments dropped off sharply, prohibiting the use of spread footings. The approximate variation in the bedrock surface elevation at each of the proposed abutment locations is presented in the following table:

Abutment Location	Bedrock Surface Elevation (m)		Variation in Bedrock Surface Elevation (m)
	Minimum	Maximum	
Northeast	65	66 (estimated)	1
Northwest	51	62	11
Southeast	55	64	9
Southwest	45	49	4

Negligible settlement is expected for spread footings founded on bedrock, therefore serviceability limit states (S.L.S.) does not apply. The following design parameters may be used for the design of spread footings founded on clean, level bedrock:

Material	Factored Capacity at U.L.S. (kPa)
Sandstone	1,500
Dolomite	1,750

The bearing surface should be free of overburden soils and loose rock fragments and should be no steeper than 5 horizontal to 1 vertical. A minimum of 1.0 m of soil cover or equivalent insulation should be provided for frost protection.

6.3.2 Pile-Supported Integral Abutments

The bridge structure could be supported on end bearing steel H-piles that are free to move laterally within the upper part. The piles would extend up through the approach fill to just underneath the bridge structure. Retaining walls would be constructed parallel to the Transitway to retain the approach fill. Lateral movement of the piles would be accommodated by installing C.S.P.'s around the pile within the approach fill.

Geotechnical design recommendations for retaining walls are provided in section 6.4.

Due to the complex geology at this site, driven steel H-piles may encounter boulders or an uneven bedrock surface. It should be anticipated that pile lengths may vary significantly even within the same founding unit. Steel H-piles should be equipped with reinforced tips to facilitate pile penetration through boulders and to help seat the piles on uneven bedrock surfaces

Due to the uneven bedrock surface and potential for refusal on dislodged bedrock pieces the pile capacities presented below are based on a practical refusal criteria on boulders or dislodged rock pieces. The following

design parameters are recommended for vertical steel piles driven to practical refusal on the bedrock.

Pile Type	Factored Capacity at U.L.S. (kN)	Capacity at S.L.S (kN)
HP310x110	1000	760
HP310x94	825	640
HP310x79	700	540

It is noted that down drag forces need not be considered at this structure location.
The above steel piles should be driven with a pile hammer delivering an energy of at least 3.5 J/mm^2 of steel cross-sectional area. The refusal criteria (practical refusal) for piles driven with this energy has two components, as follows:

1. 5 blows for the last 25 mm of penetration; **and**
2. A total of 50 blows for not more than 300 mm of penetration.

All pile driving activities should be inspected by qualified geotechnical personnel and the pile capacities controlled in the field using MTO pile driving standards (MTO Standards SS103-10 or SS103-11). Furthermore, at least one conventional pile load test combined with the use of a dynamic pile analyzer is recommended for this project. The proposed test pile(s) should be loaded to twice the S.L.S. Type II bearing capacity. It is recommended that the pile analyzer be used to monitor at least ten (10) piles with a minimum of two (2) piles per founding unit during pile installation.

6.3.3 Perched Abutments

The bridge structure may be supported on spread footings founded on reinforced earth fill. The reinforced earth would be placed as part of the backfill with a retaining wall along the West Transitway.

The fill material for the reinforced earth embankment should consist of OPSS Granular A material placed and compacted in lifts no greater than 300 mm thick, and each lift compacted to 100% Standard Proctor Maximum Dry Density.

Assuming a minimum thickness Granular A pad of 3 m and a footing width ranging from 2.0 m to 4.0 m, the following bearing pressures are recommended:

SUGGESTED RESISTANCE VALUES ON COMPACTED GRANULAR A
Internal Stability of Reinforced Earth May Require Reduced Values Designer to Confirm

Bridge Seat Width	U.L.S. Factored Bearing Resistance (kPa)	S.L.S Bearing Resistance (kPa)
2.0	300	250
2.5	315	250
3.0	325	250
3.5	340	250
4.0	350	250

It is noted that the above SLS resistance will restrict settlements within the reinforced fill to a range of 5 mm to 10 mm. Where the overburden is deepest, the anticipated post bridge seat placement settlemen below the reinforced earth wall is estimated to be 15 to 20 mm. Therefore, it is anticipated that the post bridge seat placement differential settlement could be in the order of about 15 mm.

6.3.4 Abutments - Caisson Wall

It is understood that consideration is being given to using a caisson wall to act as both the abutment support and as the retaining wall along the edge of the West Transitway beneath Richmond Road. The caisson wall approach would likely consist of every second caisson socketed into the bedrock with the in between caissons acting strictly in the lateral direction (i.e. filling in the face of the retaining wall).

It should be noted that due to the flowing sand conditions, drilling of caissons may prove difficult and will definitively require drilling fluid and the use of casing during drilling. Tremie techniques will be required for the concrete placement.

The compressive load carrying capacity of socketed caissons can be calculated from either end bearing or from bond capacity between the concrete and the rock face within the socket. Socketed caissons may be designed based on the following parameters:

- The capacity based on end bearing alone using a factored bearing capacity at U.L.S. of 2000 kPa, or

- The capacity based on bond stress alone using a factored ultimate concrete/rock bond stress of 750 kPa.

For design based on end bearing or bond, a minimum socket length of 1 m or equal to one (1) caisson diameter, whichever is greater, is recommended. Concrete for caisson sockets should have a minimum compressive strength of 30 Mpa. The base of the sockets should be well cleaned before concreting regardless whether the design is based on end bearing or bond.

The horizontal earth pressures which would act on the back of the caisson retaining wall should be calculated using the following parameters, which reflect the typical nature of the existing embankment fill at the site.

Material Type: Existing Embankment Fill	
Bulk Unit Weight Embankment Fill:	20 kN/m^3
Bulk Unit Weight of Water:	9.807 kN/m^3
Unfactored Angle of Internal Friction:	29°
Unfactored Coefficient of Active Earth Pressure:	0.35
Unfactored Coefficient of Earth Pressure at Rest:	0.52

The caisson retaining wall option does not include the provision for a free draining granular backfill with outlet drains, therefore the portion of the retaining wall below elevation 67.0 m, corresponding to existing surrounding grades, should include the addition of hydrostatic pressures over and above the earth pressures.

The horizontal capacity of the caisson retaining wall are dependent upon their embedment lengths in both the overburden and bedrock, the caisson diameters and stiffness, spacing, and loading. Horizontal design parameters and resistance will be provided once this design option is further developed.

Should the option be further developed, it is recommended that the facing of the retaining wall below elevation 68.5 m be protected against frost action, to prevent water from freezing behind the wall. The proposed elevation of 68.5 is preliminary and will need to be reviewed once the design details are further developed. The recommended insulation would be 75 mm thick sheets of extruded polystyrene such as DOW product styrofoam SM.

In order to prevent surface water from attaining an impermeable membrane design or a subddrain should be incorporated directly beneath the roadway granulars adjacent to the caisson wall.

6.3.5 Richmond Road Pavement Reconstruction

As part of the West Transitway construction project, it is anticipated that portions of the Richmond Road approaches will need to be reconstructed. The following recommendations will apply to reconstruction of the pavement structure within these areas.

Based on information from the borehole investigation, the approaches presently have the following approximate pavement structure:

Asphalt	230 mm
Base and Subbase	380 mm

The design criteria for the new pavement structure includes the following:

- Traffic
- Ensure that the bottom of the subbase is lower than the adjacent existing subbase.

Based on the above two points, the following pavement structure is recommended:

40 mm	OPSS HL1
100mm	OPSS HL8 (two lifts)
150 mm	OPSS Granular A
450 mm	OPSS Granular B, Type II

The provision of longitudinal subdrains, such as outlined in RMOC standard drawing J-3.13, is recommended to ensure efficient internal drainage of the pavement structure.

Where the new pavement structure meets that existing, the transitions between the underside of the subbase materials should be constructed at a grade no steeper than 10H : 1V.

Where the new asphalt abuts existing asphalt, the construction should be stepped such that all existing asphalt within 300 mm of the construction excavation is sawcut full depth and removed. In addition, it is recommended that the asphalt immediately beyond the full depth sawcut be milled 40 mm deep for a minimum width of 300 mm adjacent to the sawcut in order to provide for a step joint within the asphalt.

6.4 Retaining Walls

Retaining walls will likely be required along the edges of the approach fill for the new N-W ramp and in front of the abutments for both the N-W Ramp structure and the Richmond Road Structure. Based on the functional design, all retaining walls will likely consist of reinforced earth retaining walls, except for those sections of the southeast wall of the N-W Ramp embankment which could be a reinforced concrete cantilever retaining wall (on spread footings or pile foundations) or modified toe wall.

The following recommendations apply to vertical Reinforced Earth and concrete retaining walls.

- A frost depth of 1.8 m should be used for design purposes. Retaining wall footings should have a minimum soil cover of 1.8 m or equivalent insulation to protect against frost action. In the case of reinforced earth retaining walls, the leveling pad should be protected against frost heaving by ensuring that any underlying soil within the frost penetration zone is subexcavated and replaced with a drained non-frost susceptible soil.

- The retaining walls should be backfilled with an approved free-draining granular material and the material drained at the base of the wall. The source of the granular material should be approved by a geotechnical engineer prior to delivery to the site. Particles greater than 75 mm nominal diameter should not be placed in direct contact with the back of the wall.

- The minimum zone of the free draining granular backfill should form a 45° wedge above a line projected upwards from the heel of the retaining wall. In the case of a RECO type wall, the heel of the retaining wall is defined as the lower back of the reinforced earth mass.

- Design parameters for conventional reinforced earth retaining walls with granular backfill are provided in the following table. Serviceability Limit States (SLS) do not apply for the coefficient of active earth pressure and coefficient of sliding.

Material Properties (OPSS Granular B Type I)		
Bulk Unit Weight		20.5 kN/m^3
Unfactored Angle of Internal Friction		33°
Unfactored Coefficients of Active Earth Pressure, K_a		
Ground surface behind wall: Level ground surface		0.30
Unfactored Coefficient of Sliding (Reinforced Earthwall, Granular B, TypeI)		
Sliding of Reinforced Earth Block Sliding of Concrete Footing		0.55 0.45
Design Bearing Resistance - Base of Earth Block or Concrete Footing		
Bearing Material	U.L.S (factored)	S.L.S. (25mm settlement)
Bedrock	1500 kPa	N/A
Overburden	225 kPa	150 kPa

6.5 Cut Sections

6.5.1 Overburden

Temporary Cuts

Excavations for the proposed roadway subgrade are expected to be as deep as 8.5m. The overburden soils anticipated are topsoil, silty clay and sandy/silty clay sand; and should be considered as a Type 3 soil with respect to the Occupational Health and Safety Act, under dry conditions or when proper dewatering has been carried out using side slopes no steeper than 1H:1V from the bottom of the excavation. Without proper dewatering, the sandy/silty sand materials under the water table should be considered as a Type 4 soil. Open cut excavation side slopes in this case should be 3H:1V or flatter. Temporary cut slopes should be regularly inspected during construction and flattened if required. Shoring techniques discussed below in Section 6.6 will be required if steeper temporary cut slopes are required.

The deepest excavation is anticipated at the Richmond Road structure where both overburden and bedrock will be excavated beneath the existing groundwater table. Dewatering using sumps and pumping from outside of the excavation or using well points will likely be required during construction.

Permanent Cuts

Permanent cuts can be drained using conventional design such as slope interceptor drains, pavement subdrains, and drains behind retaining walls. Drawdown of the groundwater table beneath the adjacent Highway 417 and beneath nearby structures located within approximately 500 m from the construction is anticipated. However, the total settlement of the supporting subgrade materials as a result of this groundwater drawdown is predicted to be less than 10 mm in the immediate vicinity of the cut and reduces with distance from the excavation. Differential settlement within the affected area will be less. Permanent open cut slopes should be protected to prevent surficial instability and erosion using gravel blankets and standard RMOC landscaping techniques. Permanent open cut slopes may be constructed at 2.5H:1V or flatter. Steeper slopes will require internal reinforcement or external support.

Subgrade Preparation

Silty clay or loose sand/silty sand subgrade is anticipated for all cut sections from Station 6+740 to 6+870. It is recommended that the exposed subsoil be inspected by a geotechnical engineer prior to undertaking any surface treatment. As directed, the exposed subgrade should be proof rolled and soft areas subexcavated as required.

The Contractor should not be permitted to have construction equipment or vehicles directly on or within 1m above or below the final earth cut subgrade. The Contractor could construct and maintain haul roads from an interim subgrade at least 1m from the final earth cut subgrade, or could use the top of the OPSS Granular "B" as a haul road, as directed by the Engineer.

6.5.2 Bedrock

Based on the current profile and available information, bedrock excavation is anticipated between 6+860 to 7+070.

Bedrock may be excavated at slopes near vertical for temporary cuts, however, permanent bedrock cuts should be sloped no steeper than 4 vertical to 1 horizontal. It is noted that the quality of the bedrock encountered at the borehole locations was generally poor. Scaling operations must be restricted. Overly aggressive scaling will result in considerable over-excavation of this material.

A space of at least 2 m width should be maintained in front of the toe of permanent rock slopes. Some rock spalling should be anticipated from rock faces.

Where retaining wall foundations or sideslopes are to be constructed above the rock benches, the bench width should be at least equal to the height of the rock face, to a maximum bench width of 2 m.

Excavation in bedrock will likely require drilling and blasting techniques. A pre-blast survey should be carried out prior to construction to include all structures located within a minimum 500 m distance. Vibration control will be required in accordance with City Bylaws and should be monitored using seismographs. The stability of overburden cut slopes or retaining walls subjected to the acceleration due to blasting should be examined. The loose to very loose sand/silty sand soils may be prone to liquefaction if subject to significant vibration.

6.6 Temporary Shoring

The following temporary shoring methods may be considered for the proposed cut at the Richmond Road structure. Based on the subsurface conditions and estimated permeabilities obtained in this investigation, and based on our understanding of the proposed development, it is considered that conventional temporary shoring systems such as sheetpiles, or soldier piles and timber lagging system are feasible. As discussed above, depending on the actual groundwater levels at the time of the construction, groundwater control may be required.

The suitability of each system based on subsurface conditions and physical constraints at particular locations should be examined during the final design phase. The design of the shoring system is the responsibility of the contractor. The contractor designed shoring should be reviewed by the Geotechnical Engineer to confirm that the geotechnical conditions have been properly interpreted. The following outlines alternative shoring systems which may be considered.

Sheetpiles

Sheetpile walls may be considered at locations where the bedrock is deep enough to allow sufficient penetration to provide lateral stability of the sheetpiles and stability of the excavation bottom. The overburden within the major cut section consists of loose sand/silty sand materials. A preliminary assessment for functional design purposes may be based on the assumption that the sheetpile penetration below the excavation bottom should be equal to the retained height in order to achieve lateral and bottom stability.

In this case, minor dewatering inside the sheetpile walls will be required. Conventional sump and pumping techniques are likely sufficient.

Soldier Piles and Timber Lagging

A shoring system consisting of soldier piles and timber lagging also requires sufficient penetration of the soldier piles to develop lateral stability. Where the bedrock is shallow, the soldier piles may be grouted in pre-drilled holes in the bedrock to provide lateral stability.

Based on the loose sand/silty sand overburden and the relatively high groundwater table encountered within the area of the proposed cut, it is expected that dewatering from outside the excavation by well point or other method may be required to control the side slope prior to the installation of the timber lagging, and also to prevent excavation bottom instability (boiling).

6.7 Storm Sewer Installation

It is understood that a 1.2 m diameter storm sewer is to be installed beneath the proposed Transitway alignment. It is further understood that the bottom of the service trench will be approximately 5 m and 2.5 m below the finished Transitway grade at Stations 6+600 and 7+070, respectively.

Where retaining walls are present, it is recommended that the sewer alignment be maintained a lateral distance, away from the wall foundation, that is greater than the vertical excavation depth below the wall foundation level, plus 1.0 metre. In areas where the retaining walls are founded on bedrock, this offset

requirement can be reduced by half. If this offset cannot be maintained, the actual proposed geometric details should be reviewed by the geotechnical consultant to establish the construction restrictions which will be required to avoid undermining the foundations.

If the nearby retaining walls are to be constructed after sewer installation, it is recommended that the sewer be offset from the wall foundation a distance equal to or greater than the vertical excavation depth below the proposed wall foundation to avoid foundation stresses being exerted on the sewer. In rock subgrade, the recommended offset can be reduced to 1.0 m.

Excavation for the sewer trench east of Richmond Road will require bedrock excavation. Drilling and blasting techniques will be required. Open cut excavations within bedrock may be made near vertical provided all loose rock material is removed from the exposed rock faces prior to worker entry into the excavation. The excavation walls should be inspected daily for signs of rock fall hazard and remedial measures taken as required.

Within Transitway cut sections, temporary excavations for storm sewers in overburden soils could be carried out in an open cut provided the nearest edge of the sewer excavation is located at least equal to the depth of excavation from the toe of the nearest excavation slope. Plans for excavations located within this distance should be reviewed by a geotechnical engineer prior to construction.

Temporary excavations in the overburden soils may be made at near vertical slopes provided the depth of excavation is less than 1.2 m. Deeper excavations should be supported or flattened in accordance with the guidelines set out in the Occupational Health and Safety Act (OHSA).

Groundwater may be encountered during excavation for installation of the storm sewer. Based on preliminary information, the bottom of the sewer excavation may be as low as 60.5 m elevation. At the time of the geotechnical investigation, the groundwater table elevation was approximately 62 m. Flowing sand conditions were encountered at approximately 60.3 m in several of the boreholes. It is recommended that the groundwater levels be verified prior to the start of construction and that all excavations that extend below the static water table be dewatered using a combination of standard sump and pump techniques and a series of wells or sump pits located outside of the sewer trench excavation.

Dewatering from outside of the excavation will prevent the sand deposits from flowing into the excavation and thereby undermining the excavation side walls.

Pipe bedding should consist of an approved OPSS Granular A material placed and compacted in accordance with the pipe design requirements. Trench backfill, consisting of an approved OPSS Granular B, Type I material or Select Subgrade Material, should be placed above the pipe bedding to a depth of 1.5 m beneath the finished road grade. The Granular B or SSM should be placed in lifts and

compacted to at least 95 % of Standard Proctor maximum dry density. The material used within the upper 1.5 m should be similar to that exposed in the trench walls.

Excavated bedrock may be used as backfill if all pieces larger than 150 mm are removed. Rockfill should be placed in 500 mm lifts, with each lift being compacted. The upper rockfill should be well graded or covered with a geotextile filter barrier to prevent soils from percolating through.

6.8 General Construction Recommendations

6.8.1 Cement Type and Corrosion Potential

Five (5) soil samples were submitted to Accutest Laboratories in Nepean, Ontario for analysis of pH and water soluble sulphate and chloride, in order to determine cement type and reinforcing steel protection requirements. The results of the testing are summarized in the table below:

Borehole	Sample	Depth	pH	Sulphate	Chloride
M-1	SS-4	6.4 m	7.4	0.07%	0.129%
M-1	SS-9	15.6 m	8.0	0.04%	0.024%
M-4	SS-4	6.4 m	8.4	0.04%	0.039%
M-16	SS-11	7.9 m	8.3	0.06%	0.019%
M-19	SS-3	1.8 m	7.9	0.04%	0.003%

The water soluble sulphate results are all below 0.10 %, indicating that a negligible degree of sulphate attack is expected for concrete in contact with the site soils. Therefore, a normal Type 10 Portland cement should be suitable for use in concrete at this site.

The soil pH level and concentration of water soluble chloride give an indication to the level of potential attack on buried steel objects such as reinforcing steel. The pH level is within the normally acceptable range of 5.8 to 9, indicating no special corrosion potential problems. Several of the water soluble chloride concentration results are above 0.025%, indicating an environment potentially favourable for the corrosion of buried steel.

It is noted that although most of the water soluble chloride ion test results are below or only marginally above 0.025 %, winter deicing practices along the Transitway typically result in high levels of chloride being introduced into the near surface soils and into the cover concrete. Consideration should be given to

providing special corrosion protection to buried steel located in close proximity to the Transitway lanes.

6.8.2 Assessment of Excavated Material

The overburden material encountered throughout the investigation does not generally meet the requirements for OPSS Select Subgrade Material. However, the silty sand and portions of the silty clay could be used as OPSS Earth Borrow depending upon the time of the construction season and construction techniques. Therefore, consideration could be given to using selected site cut materials in fill sections provided it can be compacted to at least 95 percent of Standard Proctor maximum dry density. To achieve this compaction, site selection of excavated cut material may be required at the time of construction. For design purposes, it is suggested that 50% of the material excavated after stripping would be acceptable for road subgrade fill placement. The actual amount will vary significantly depending on the weather during cut and fill operations as well as the construction techniques. Where encountered, unacceptable materials may need to be wasted or used for berm construction or other landscaping purposes.

6.9 Proposed Stormwater Management Pond

It is understood that a stormwater management pond is being designed on behalf of the RMOC by others. It is also understood that a separate geotechnical report will be prepared to address the pond design requirements. The following recommendations relate only to items of concern as they relate to the nearby transitway cut.

The area being considered for the pond is located north of the Transitway, east of Richmond Road, and specifically just east of the E-N/S Ramp. In this area, the overburden soils are shallow (≈ 2 m to bedrock) and are generally quite permeable. The following is recommended:

- The underlying soils can support berms with sideslopes as steep as 2.5 H:1V with no risk of rotational failure within the overburden. The actual berm sideslopes will need to reflect the construction materials, their permeability and the proposed construction heights.

- It is recommended that the toe of the berms be offset from the transitway cut slope or the heel of retaining wall (if applicable) by at least 2.0 m and at least a distance equal to the lesser of the following:
  - The depth of the local overburden
  - The depth of the local transitway cut.

- The pond should be designed to restrict inflow into the overburden.

6.10 Recreational Pathways

For the design of recreational pathways, RMOC standard drawing J-1.2 may be used.

In areas where the subgrade type changes from rock to earth subgrade, it is recommended that transition treatments, such as defined in OPSD-205.05, be incorporated in the design using a "t" depth of 1.2 m.

If the pathway is to be located at the base of a slope, it is recommended that a single longitudinal subdrain be installed on the slope side of the pathway to prevent surface water from collecting within the granulars.

The subgrade of the pathway should be prepared by compacting the exposed surface to at least 95% of the Standard Proctor Maximum Dry Density (SPMDD).

7.0 CLOSURE

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations and can only be extrapolated to an undefined limited area around these locations. The extent of the limited area depends on the soil and groundwater conditions, as well as the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above conclusions.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Yours very truly,

JACQUES, WHITFORD LIMITED

LICENSED PROFESSIONAL ENGINEER
P. CARNAFFAN
PROVINCE OF ONTARIO

Paul Carnaffan, M.Eng., P.Eng.

LICENSED PROFESSIONAL ENGINEER
J. G. A. R. HACHE
PROVINCE OF ONTARIO

J.G.A. Raymond Haché, M.Sc., P.Eng.

APPENDIX 1

KEY PLAN

1 : 20 000

DRAWING No. 10923-1

NOTE:

BH DESIGNATIONS M-14, M-15, M-17 WERE NOT USED.
(PROPOSED BOREHOLE LOCATIONS WERE CANCELLED).

BAYSHORE SHOPPING CENTRE

EXISTING RAMP TO BE REMOVED

PROPOSED WEST TRANSITWAY ALIGNMENT

RECO WALL

N-W RAMP

S-W RAMP

REALIGNED

RICHMOND ROAD

HWY 417 W.B.L.

HWY 417 E.B.L.

TRANSITWAY TYPICAL SECTION PLATE 118

M-19 M-18 BH 94-5 BH 94-6 M-5 M-1 M-20 M-2 BH 6C M-3 M-6 M-21 M-4 M-22 BH 6 BH 6B BH 89-4 M-7 M-8 BH 3 M-16 M-10 M-9 M-11 M-12 M-13

LEGEND:

M-16 BOREHOLE LOCATION/DESIGNATION

Jacques Whitford	REFERENCE: BASE PLAN PROVIDED BY MORRISON HERSHFIELD LIMITED	SCALE: 1 : 1000 DATE: 97/09/12 DWN. BY: GBB APP'D BY:	MORRISON HERSHFIELD LIMITED PROPOSED WEST TRANSITWAY OTTAWA, ONTARIO	BOREHOLE LOCATION PLAN	DRAWING No.: 10923-2

APPENDIX 2

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Topsoil	-	mixture of soil and humus capable of supporting good vegetative growth
Peat	-	fibrous aggregate of visible and invisible fragments of decayed organic matter
Till	-	unstratified glacial deposit which may range from clay to boulders
Fill	-	any materials below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	-	having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure
Varved	-	composed of regular alternating layers of silt and clay
Stratified	-	composed of alternating successions of different soil types, e.g. silt and sand
Layer	-	>75 mm
Seam	-	2 mm to 75 mm
Parting	-	< 2 mm
Well Graded	-	having wide range in grain sizes and substantial amounts of all intermediate particle sizes
Uniformly Graded	-	predominantly of one grain size

Terminology describing soils on the basis of grain size and plasticity is based on the Unified Soil Classification System (USCS) (ASTM D-2488). The classification excludes particles larger than 76 mm (3 inches). This system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%

The standard terminology to describe cohesionless soils includes the compactness (formerly "relative density"), as determined by laboratory test or by the Standard Penetration Test 'N' - value.

Relative Density	'N' Value	Compactness %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests, or occasionally by standard penetration tests.

Consistency	Undrained Shear Strength		'N' Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25-0.5	12.5-25	2-4
Firm	0.5-1.0	25-50	4-8
Stiff	1.0-2.0	50-100	8-15
Very Stiff	2.0-4.0	100-200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Rock Quality Designation (RQD)

The classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures.

RQD	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

Terminology describing rock mass:

Spacing (mm)	Bedding, Laminations, Bands	Discontinuities
2000-6000	*Very Thick*	*Very Wide*
600-2000	*Thick*	*Wide*
200-600	*Medium*	*Moderate*
60-200	*Thin*	*Close*
20-60	*Very Thin*	*Very Close*
<20	*Laminated*	*Extremely Close*
<6	*Thinly Laminated*	

Strength Classification	Uniaxial Compressive Strength (MPa)
Very Low	1-25
Low	25-50
Medium	50-100
High	100-200
Very High	>200

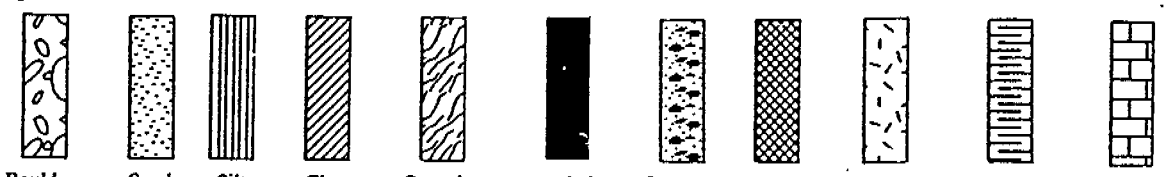
Terminology describing weathering:

Slight - Weathering limited to the surface of major discontinuities. Typically iron stained.

Moderate	-	Weathering extends throughout rock mass. Rock is not friable.
High	-	Weathering extends throughout rock mass. Rock is friable.

STRATA PLOT

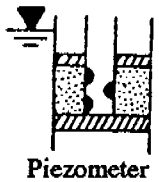
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



Boulders Cobbles Gravel | Sand | Silt | Clay | Organics | Asphalt | Concrete | Fill | Igneous Bedrock | Meta-morphic Bedrock | Sedi-mentary Bedrock

WATER LEVEL MEASUREMENT

Borehole or Standpipe



Piezometer

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond drilling bits.

N - VALUE

Numbers in this column are the results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and 'N' values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75).

OTHER TESTS

S	Sieve analysis
G_s	Specific gravity of soil particles
k	Permeability (cm/sec)
	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer
H	Hydrometer analysis
γ	Unit weight
C	Consolidation
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct shear
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$; the index corrected to a reference diameter of 50 mm)

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-17
WATER LEVEL:

BOREHOLE No.: M-1
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (mm)	N-VALUE OR RQD
	72.39					
0	72.1	300 mm ASPHALT				
		Compact, brown, sandy silt, trace clay, trace gravel : FILL				
1						
2			SS	1	300	24
3			SS	2	25	21
4	67.9					
		Very stiff, grey, silt and clay, trace sand : FILL	SS	3	460	8
5	67.3	Loose, dark brown, SILT, trace organics				
6	66.3	Compact, grey, SILT, some sand, some clay	SS	4	560	10
7	64.8					
		Stiff, grey, SILT and CLAY	SS	5	610	5
8						
9			SS	6	610	7
		- sand seams				
10						

STRATA PLOT / WATER LEVEL

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

Continued Next Page

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-17 WATER LEVEL:

BOREHOLE No.: M-1
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
10–12	60.3	Stiff, grey, SILT and CLAY			SS	7	610	7
12–15	57.3	Dense, grey to brown, SAND, some silt			SS	8	510	30
15–18	54.2	Compact, grey to brown, SILT, some sand, some clay			SS	9	560	20
18–20		Dense, grey to brown, GRAVELLY SAND, some silt			SS	10	155	48

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200
WATER CONTENT & ATTERBERG LIMITS: W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

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JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-07-17 WATER LEVEL

BOREHOLE No. M-1

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
20		Dense, grey to brown, GRAVELLY SAND, some silt						
21	50.8							
22		Severely fractured to fractured, grey, sandstone with shale seams: BEDROCK			NQ	11	100%	35%
23	49.2				NQ	12	100%	60%
		End of Borehole						
24								
25								
26								
27								
28								
29								
30								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

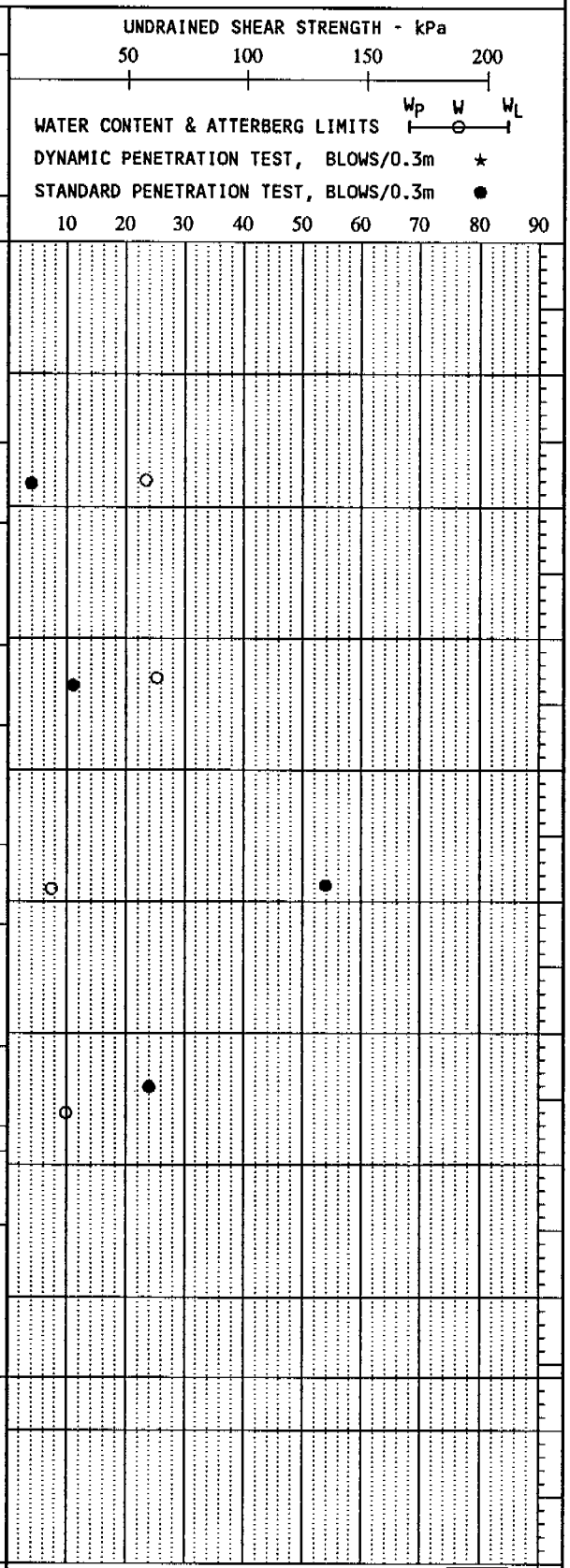
JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-15 WATER LEVEL:

BOREHOLE No.: M-2
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
	72.21							
0	72.0	250 mm ASPHALT						
		Very stiff, brown, silt and clay, trace sand : FILL						
1								
2					SS	1	510	4
3					SS	2	460	11
4								
5	67.3	Very dense, brown, silty sand, some gravel : FILL			SS	3	400	54
6	66.1							
	65.7	Hard, brown, SILTY CLAY			SS	4	580	24
	65.3	Compact, brown, SAND						
7		Severely fractured, grey, sandstone with shale seams: BEDROCK			NQ	5	100%	0%
8					NQ	6	100%	37%
	63.6							
9		End of Borehole						
10								



Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-16
WATER LEVEL:

BOREHOLE No.: M-3
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	72.46							
	72.2	300 mm ASPHALT						
		Compact, brown to grey, silty sand : FILL						
1								
2					SS	1	300	11
3	69.4							
		Dense, dark grey, sand and gravel, some silt, some asphalt: FILL			SS	2	155	Ref
4	67.9							
		Loose to compact, greyish brown, SILTY SAND, some gravel			SS	3	0	8
5								
6					SS	4	360	31
7	64.9							
		Compact, brownish grey, SILT, trace clay and sand			SS	5	610	24
8								
9	63.2							
	63.0	Loose to compact, brownish grey, SAND			SS	6	460	48
10								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

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JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-16 WATER LEVEL:

BOREHOLE No.: M-3
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
10		Very stiff, grey, SILT and CLAY						
11					SS	7	610	8
12					SS	8	530	7
13								
14								
15	57.3							
	57.1	Very loose, brown and grey, SAND			SS	9	500	19
16		Hard, grey, SILT and CLAY, trace sand						
17								
18	54.1							
		Compact, grey, SAND, trace silt			SS	10	610	-
19								
20								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

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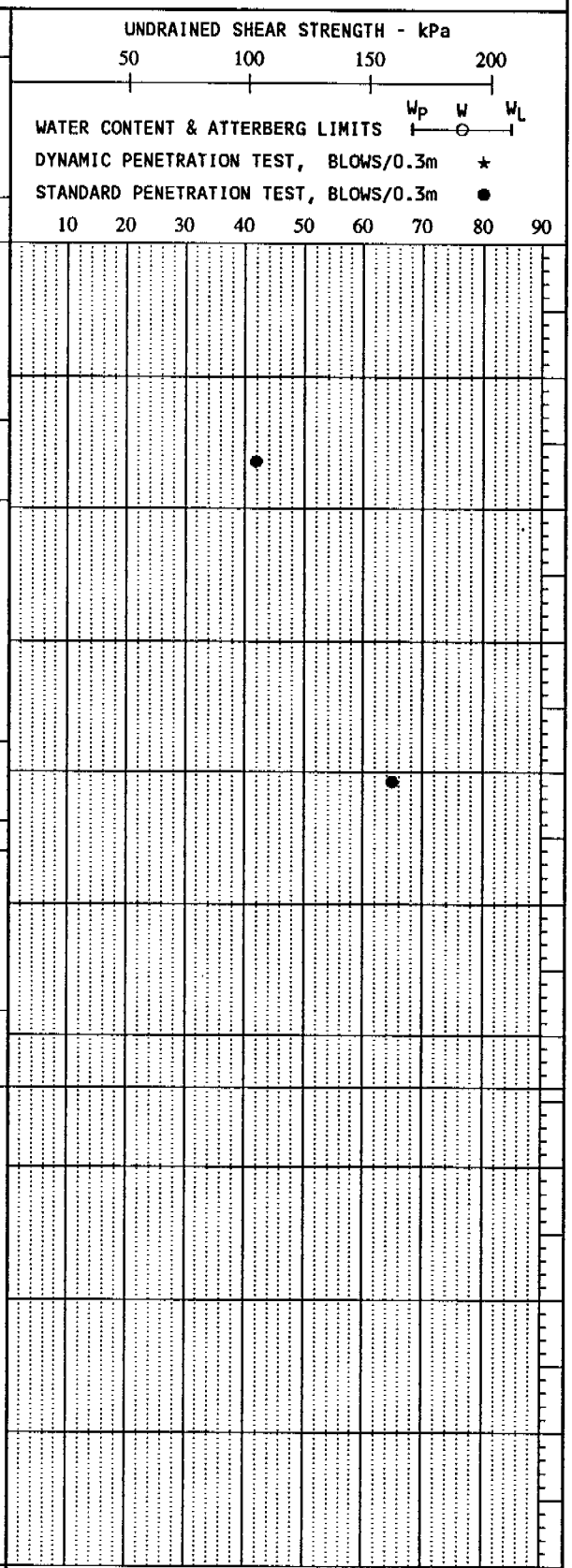
JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-16
WATER LEVEL:

BOREHOLE No.: M-3
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
20		Compact, grey, SAND, trace silt						
21	51.2	Grey, SILT, trace to some clay			SS	11	610	42
22								
23	48.7							
24	47.9	Dense to very dense, grey, SAND, trace gravel and silt			SS	12	460	65
25		Fractured to very sound, dolomite with shaly partings: BEDROCK			NQ	13	100%	92%
26	46.1				NQ	14	100%	69%
27		End of Borehole Ref = >50 blows for 150mm						
28								
29								
30								



Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

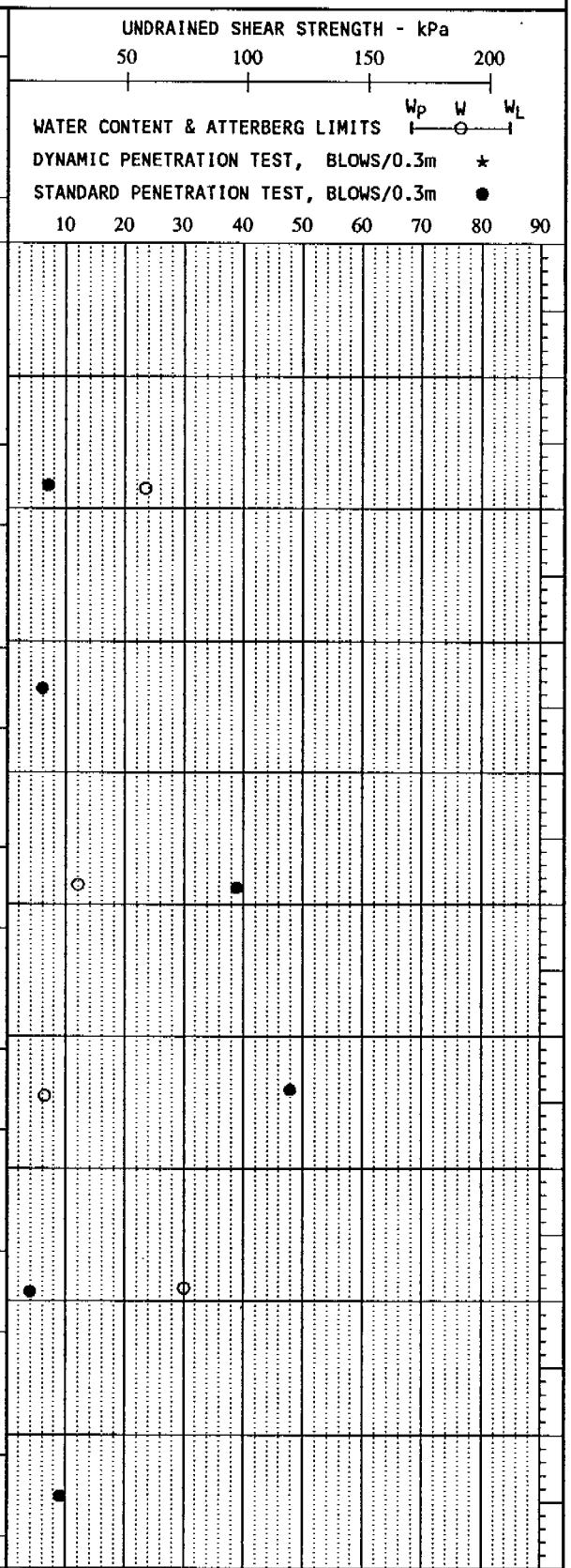
JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-15
WATER LEVEL:

BOREHOLE No.: M-4
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
	72.40							
0	72.1	300 mm ASPHALT						
		Very stiff to hard, brown, silty clay, trace sand and gravel : FILL						
1								
2					SS	1	250	7
3					SS	2	430	6
4	67.8							
		Dense, brown, sand, trace gravel : FILL			SS	3	410	39
5								
6	66.3							
		Dense, grey, SAND, trace silt, trace gravel			SS	4	460	48
7	64.8							
		Very stiff, grey, SILTY CLAY			SS	5	510	4
8								
9					SS	6	610	9
10								



Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

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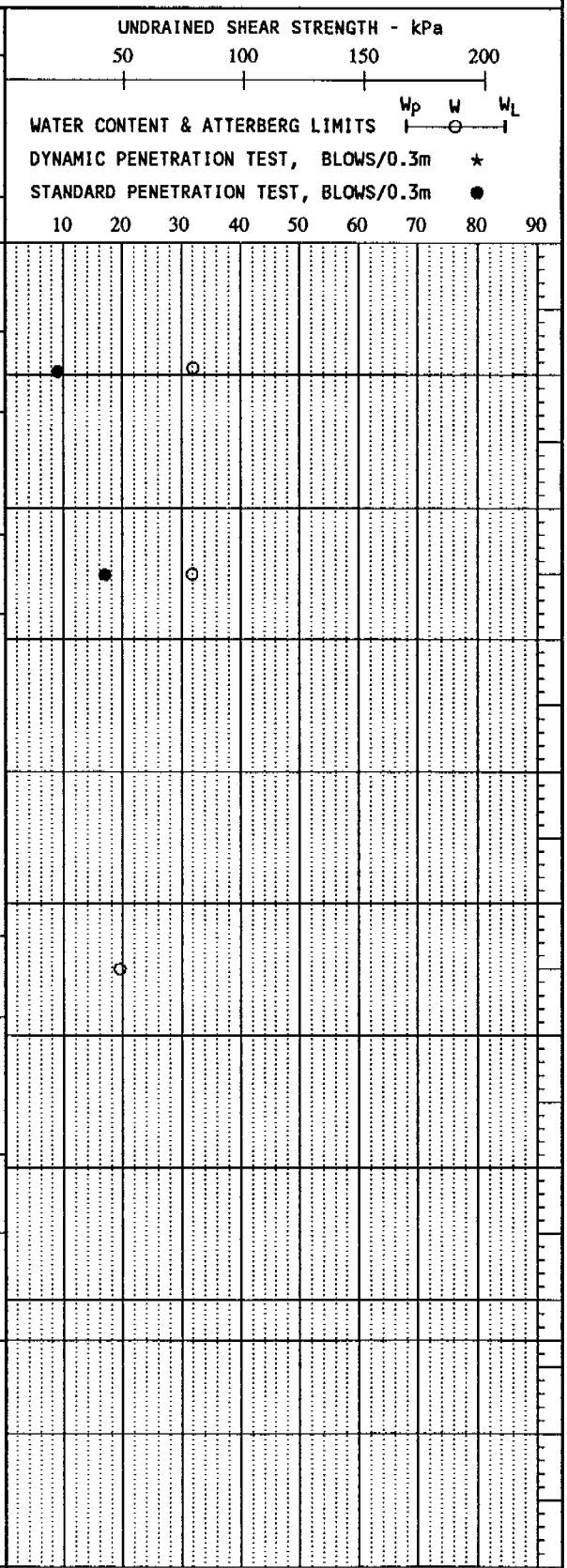
JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-15
WATER LEVEL:

BOREHOLE No.: M-4
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
10–12	60.2	Very stiff, grey, SILTY CLAY			SS	7	610	9
12–15	57.2	Very stiff, grey, SILTY CLAY, trace sand, trace gravel			SS	8	460	17
15–16.5	55.5	Very dense, grey, silt and sand, some clay, some gravel : TILL			SS	9	-	Ref
17–18.4	54.1	Fractured to very severely fractured, grey, sandstone with greenish-grey shale seams: BEDROCK			NQ	10	100%	66%
					NQ	11	92%	0%
		End of Borehole Ref = >50 blows for 150mm						



Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

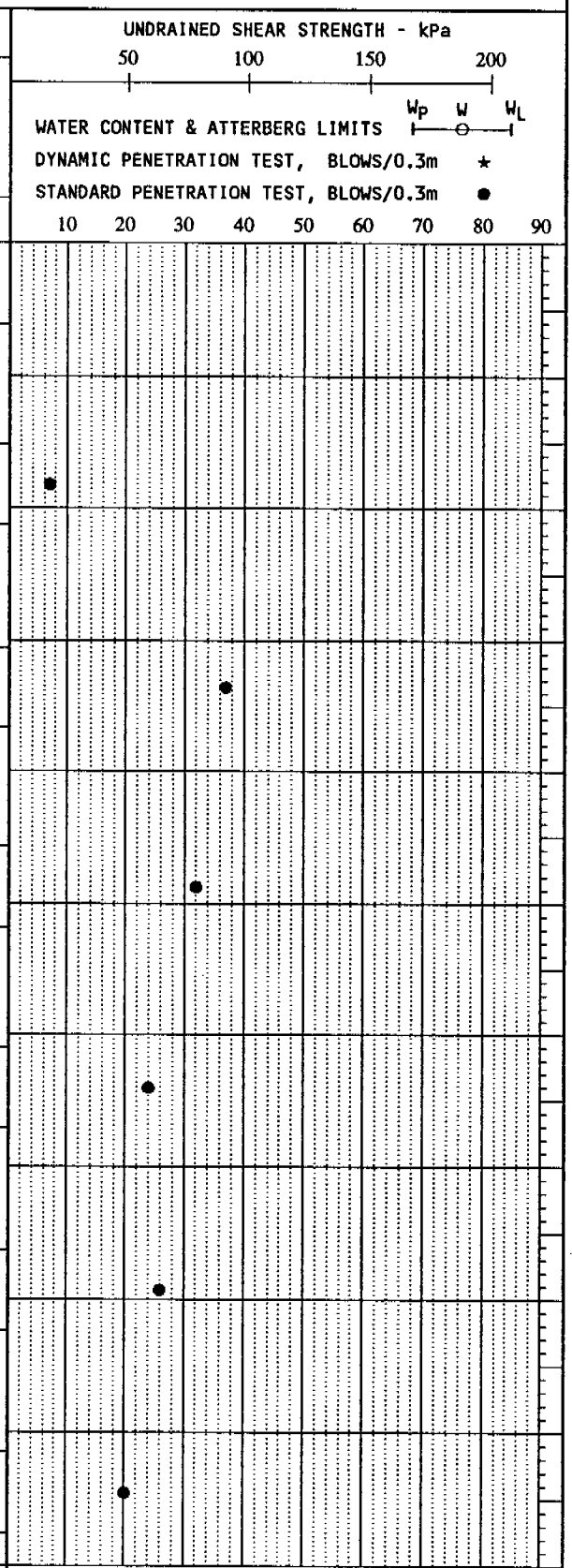
JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-30 WATER LEVEL: 97-08-28

BOREHOLE No.: M-5
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	69.47							
	69.0	Brown, sand, trace silt : FILL			BS	1	-	-
1		Stiff, grey, clayey silt, trace sand, trace gravel: FILL						
2					SS	2	0	7
3	66.5							
		Dense, grey, sandy silt, trace gravel : FILL			SS	4	410	37
4	64.8							
5	64.5	Compact, brown, SAND			SS	5	380	32
		Very stiff, brownish-grey, SILT and CLAY, trace sand						
6	62.9				SS	6	610	24
	62.8	Loose, grey, SAND						
7		Very stiff, brownish-grey, SILT and CLAY, trace sand		▼				
8					SS	7	280	26
9	60.2							
		Loose, grey, SAND			SS	8	480	20
10								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe
◄ Proposed Pipe Invert

■ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

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JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-30 WATER LEVEL: 97-08-28

BOREHOLE No.: M-5
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD
10	58.8	Loose, grey, SAND						
11	57.3	Very stiff, grey, CLAYEY SILT, trace sand			SS	9	560	8
12		Dense, grey, sand and silt, some gravel, trace clay : TILL			SS	10	460	33
13								
14					SS	11	530	25
15								
16					SS	12	560	Ref
17								
18	51.0				SS	13	610	Ref
19		Dense, grey, SILT, trace clay, trace sand			SS	14	460	39
20								

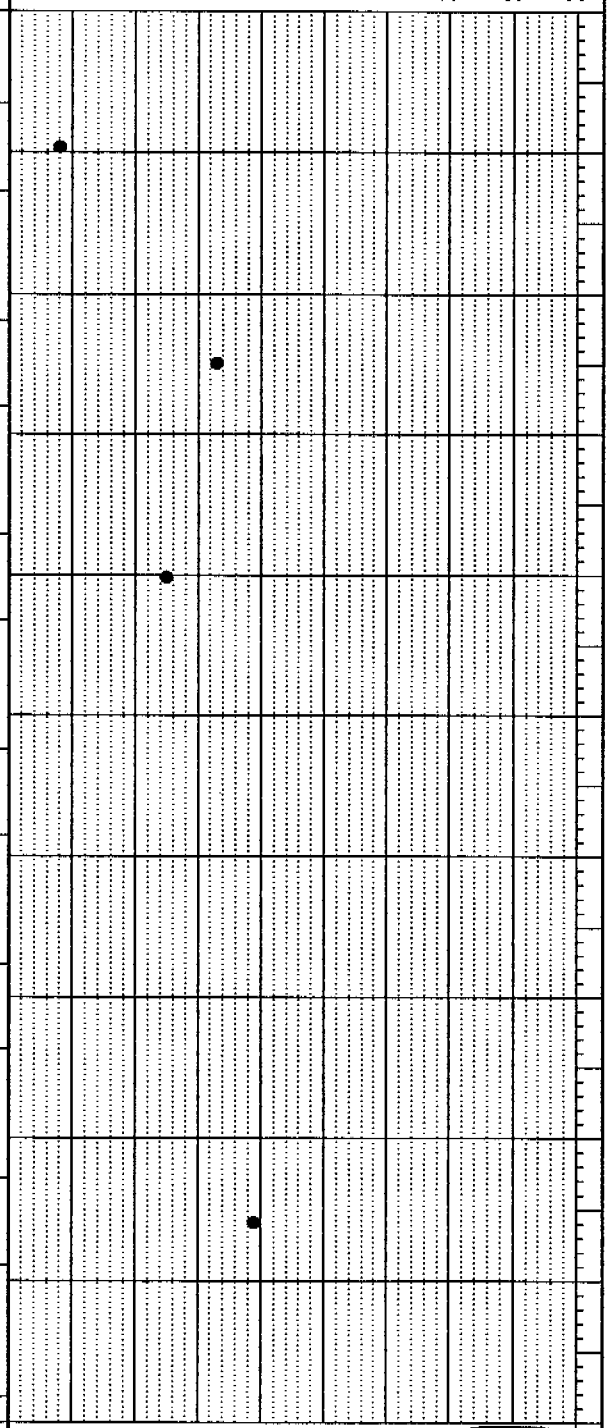
UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90



Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

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JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-07-30 WATER LEVEL: 97-08-28

BOREHOLE No.: M-5
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
20		Dense, grey, SILT, trace clay, trace sand			SS	15	460	20
21	48.2							
	47.8	Dense, grey, SAND			SS	16	510	78%
22	47.4	Vey dense, grey, silt, some sand, trace clay, trace gravel: TILL						
23	45.9	Fractured, medium to dark grey, dolomite with shaly partings: BEDROCK			NQ	17	90%	52%
24		End of Borehole Standpipe Installed Ref = >50 blows for 150mm						
25								
26								
27								
28								
29								
30								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

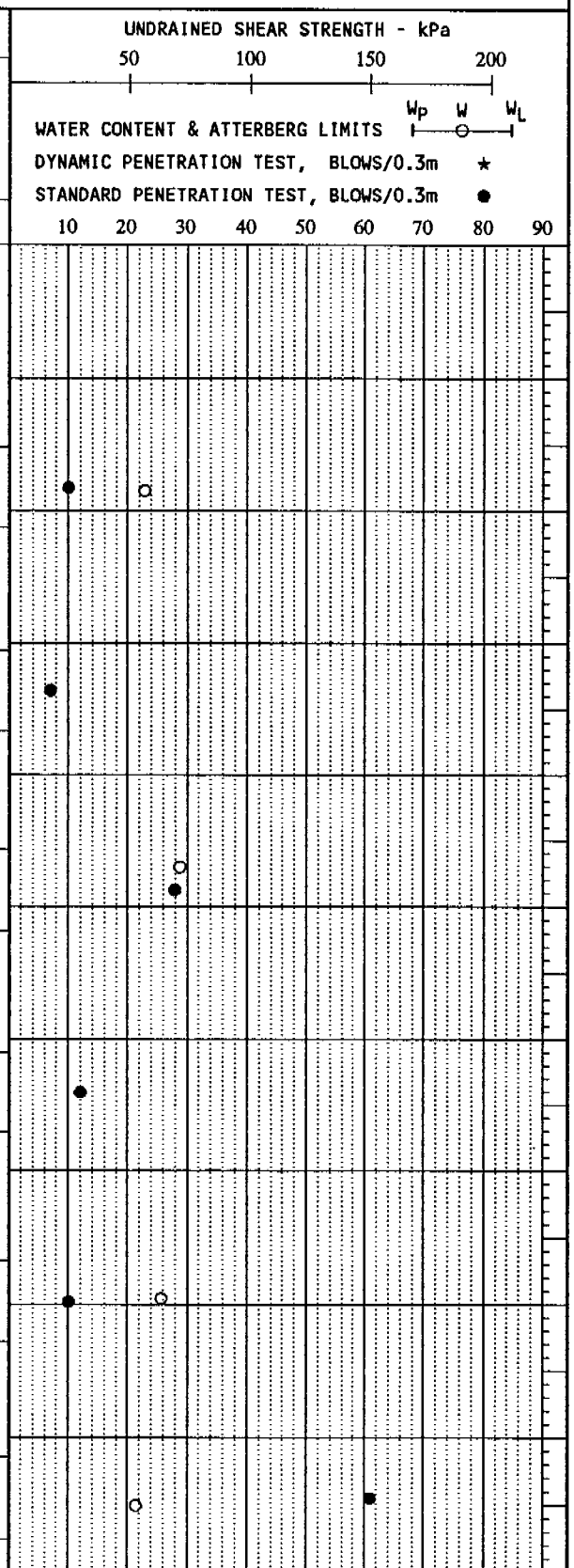
JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING 97-07-31 WATER LEVEL 97-08-28

BOREHOLE No. M-6
PROJECT No. 10923
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD
	67.65							
0		Compact, brown, silt and sand, trace clay: FILL						
1								
2					SS	1	610	10
3	64.6							
		Stiff to very stiff, grey, SILTY CLAY, trace sand			SS	2	530	7
4								
	62.8				SS	3	460	28
5		Dense, grey, SAND						
6	61.6							
	61.5	Very stiff, grey, SILTY CLAY, trace sand			SS	4		12
7		Loose to compact, grey, SAND						
	59.9							
8		Very stiff, grey, SILTY CLAY, trace sand			SS	5		10
9	58.5							
	58.1	Dense, grey, SAND			SS	6		61
10								



Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

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Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

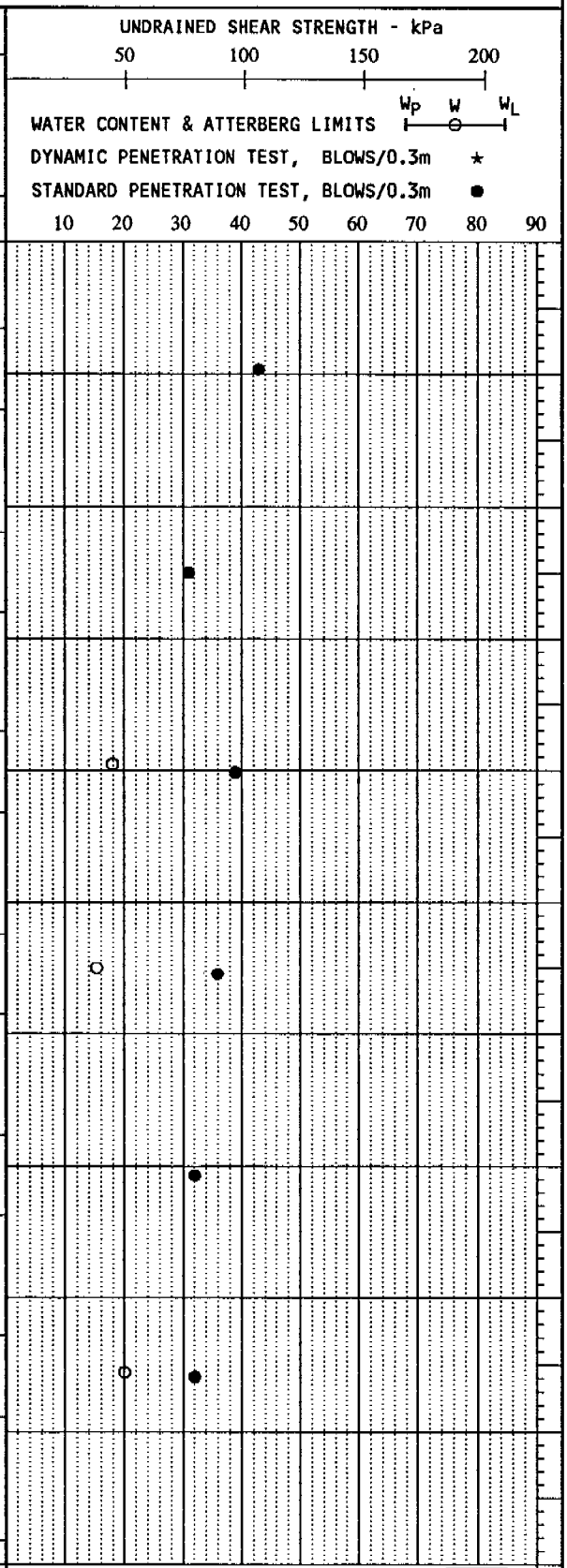
DATES: BORING 97-07-31 WATER LEVEL 97-08-28

BOREHOLE No. M-6

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD
10		Dense, grey, SILTY SAND, trace clay						
11					SS	7	610	43
12	55.5							
	55.0	Compact, grey, SAND			SS	8	510	31
13	54.0	Very stiff, grey, CLAYEY SILT, trace sand						
14		Dense, grey, SILT and SAND, trace clay, trace gravel			SS	9	410	39
15	52.4							
		Compact to dense, grey, SILTY SAND, trace gravel			SS	10	510	36
16								
17					SS	11	430	32
18								
					SS	12	480	32
19								
20								



Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

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JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING 97-07-31 WATER LEVEL 97-08-28

BOREHOLE No. M-6
PROJECT No. 10923
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD
20		Compact to dense, grey, SILTY SAND, trace gravel			SS	13	-	Ref
21								
22					SS	14	430	13
23	44.4				SS	15	230	Ref
24	42.9	Very severely fractured, medium to dark grey, dolomite with shaly partings: BEDROCK			NQ	16	95%	18%
25		End of Borehole Stanpipe Installed Standpipe blocked @ 3.9 m. Dry to 3.9 m. Ref = >50 blows for 150mm						
26								
27								
28								
29								
30								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200
WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

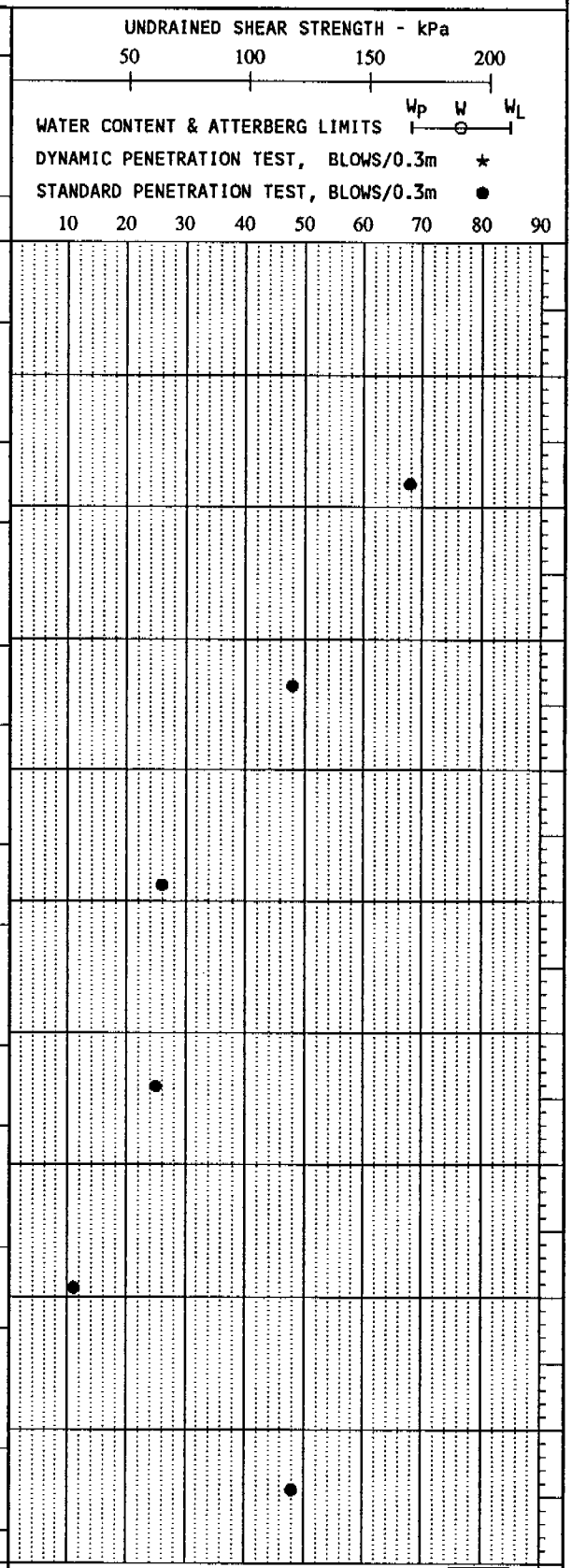
JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING 97-08-01 WATER LEVEL 97-08-28

BOREHOLE No. M-7
PROJECT No. 10923
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	66.51							
		Very dense, brown, sand and gravel : FILL			SS	1	50	Ref
1	65.0							
		Very dense to compact, brown, SAND and GRAVEL			SS	2	155	68
2								
3					SS	3	100	48
4								
5					SS	4	100	26
6	60.4							
		Very stiff, grey, SILT and CLAY, trace sand			SS	5	510	25
7	58.9							
		Compact to very dense, brownish-grey to grey, SAND			SS	6	560	11
8								
9					SS	7	460	48
10								



Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

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JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING 97-08-01 WATER LEVEL 97-08-28

BOREHOLE No. M-7
PROJECT No. 10923
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
10		Compact to very dense, brownish-grey to grey, SAND						
11		- some silt			SS	8	410	66
12	54.1							
		Compact to very dense, grey, SAND, trace to some silt, occasional silty sand seams			SS	9	480	31
13								
14					SS	10	610	66
15								
					SS	11	610	21
16								
17								
18					SS	12	610	16
19	47.3							
		Compact, grey, SAND			SS	13	610	17
20								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200
WATER CONTENT & ATTERBERG LIMITS: W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

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JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-08-01 WATER LEVEL 97-08-28

BOREHOLE No. M-7

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
20		Compact, grey, SAND						
21								
22								
23					BS	14	-	-
24	42.1							
25		Fractured, medium to dark grey, dolomite with occasional shaly partings: BEDROCK			NQ	15	98%	75%
26	40.6	End of Borehole Standpipe Installed Standpipe dry to 4.9m on 97-08-28 Ref = >50 blows for 150mm						
27								
28								
29								
30								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-08-01 WATER LEVEL 97-08-28

BOREHOLE No. M-8

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD
0	65.57							
		Compact to dense, brown, SAND and GRAVEL			SS	1	300	40
1					SS	2	0	26
2					SS	3	100	21
3	62.5				SS	4	0	43
		End of Borehole Standpipe Installed Standpipe dry on 97-08-28						

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◀ Proposed Pipe Invert

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-08-14 WATER LEVEL

BOREHOLE No. M-9

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD
0	65.89							
		Compact, brown, sand, trace gravel, trace silt: FILL			SS	1	200	32
	64.4							
		Compact, brown, SAND, some gravel, trace silt			SS	2	180	14
	63.4							
		Dense, brown, SAND and GRAVEL			SS	3	130	34
	62.8							
		End of Borehole						

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited

LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-08-14 WATER LEVEL 97-08-28

BOREHOLE No. M-10

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD
0	66.21							
	65.7	Grey, sand and gravel (crushed stone) : FILL			BS	1	-	-
1	64.7	Dense, grey, gravel and sand (crushed stone) : FILL			SS	2	200	33
2		Compact, brown, SAND, some gravel			SS	3	130	40
3	63.2				SS	4	230	16
		End of Borehole Standpipe Installed Standpipe dry to 2.4m on 97-08-28						

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-08-14 WATER LEVEL 97-08-28

BOREHOLE No. M-11

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	66.10	Grey, gravel and sand (crushed stone) : FILL			BS	1	-	-
1					SS	2	0	Ref
	64.6	Compact, brown, SAND, some gravel, trace silt			SS	3	510	21
2	63.7	Firm, brown to grey, SILT and CLAY, some gravel, trace sand			SS	4	330	15
3	63.1	End of Borehole Standpipe Installed Standpipe dry on 97-08-28 Ref = >50 blows for 150mm						
4								
5								
6								
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited

LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING: 97-08-14 WATER LEVEL:

BOREHOLE No.: M-12

PROJECT No.: 10923

DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	66.19							
		Compact, grey, sand and gravel: FILL			BS	1	-	-
1					SS	2	130	26
	64.7							
		Stiff, brown, SILTY CLAY, trace sand			SS	3	430	7
2					SS	4	380	6
3	63.1							
		End of Borehole						
4								
5								
6								
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-08-14 WATER LEVEL

BOREHOLE No. M-13

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	66.39							
		Grey, sand and gravel (crushed stone) : FILL			BS	1	-	-
	65.5							
1		Stiff brown to grey, CLAYEY SILT, trace sand			SS	2	300	18
2					SS	3	480	8
3	63.3				SS	4	430	7
		End of Borehole						
4								
5								
6								
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa

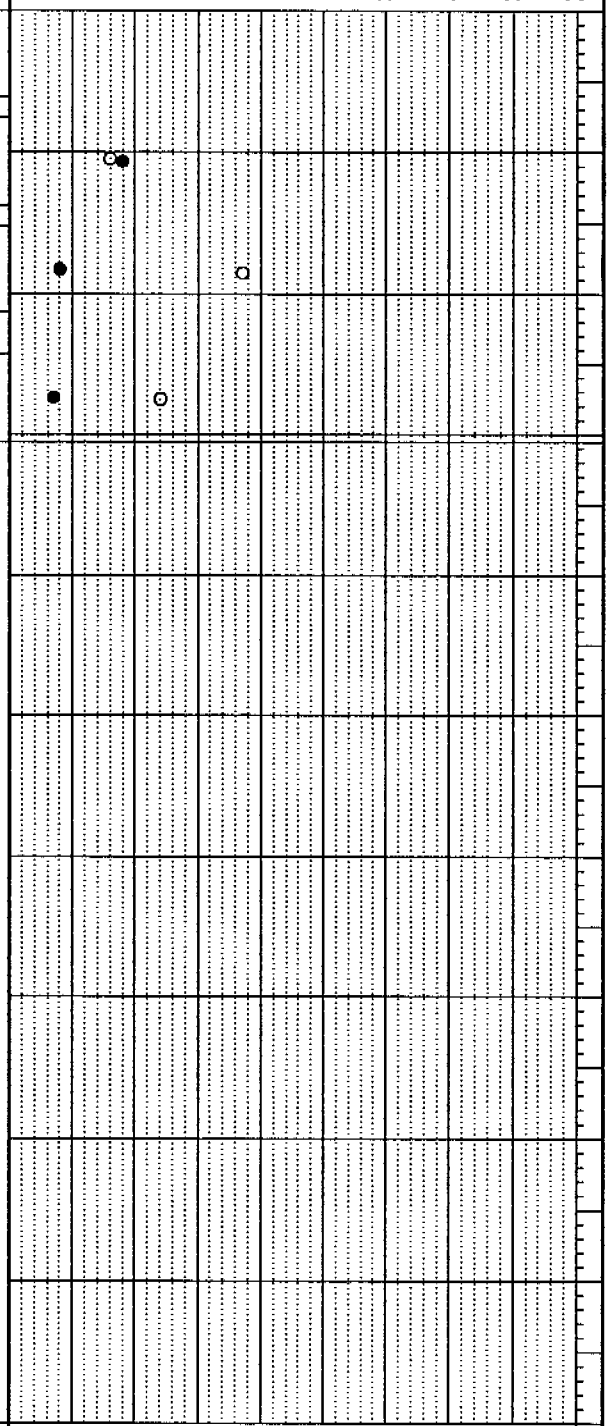
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90



Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

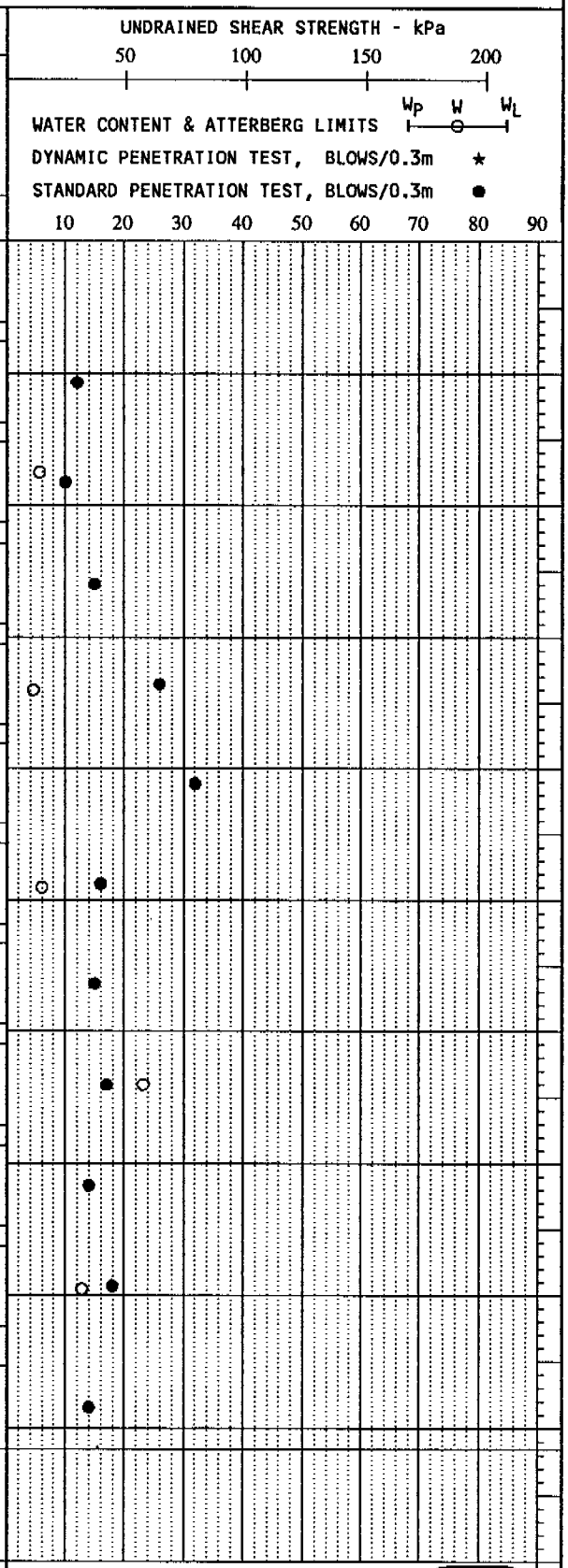
JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING 97-08-07 WATER LEVEL 97-08-28

BOREHOLE No. M-16
PROJECT No. 10923
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	66.80					
		Brown, sand and gravel, some silt : FILL	BS	1	-	-
1			SS	2	25	12
	65.3					
		Compact, brown to grey, SAND and GRAVEL	SS	3	230	10
2			SS	4	75	15
3			SS	5	155	26
4			SS	6	50	32
5			SS	7	75	16
			SS	8	180	15
6	60.7					
		Compact, brown, SAND, trace silt	SS	9	50	17
7			SS	10	0	14
	59.2					
		Compact, grey, SAND and GRAVEL	SS	11	75	18
8						
			SS	12	0	14
9	57.7					
		End of Borehole Standpipe installed				
10						



Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited

LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING: 97-08-07 WATER LEVEL:

BOREHOLE No.: M-18

PROJECT No.: 10923

DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY	N-VALUE OR RQD
	67.21						mm	
0	66.7	Dark brown to brown, SILT and SAND, trace organics			BS	1	-	-
1	66.2	Very dense, brown, SAND, some gravel			SS	2	0	Ref
		Fractured, grey, sandstone, with frequent shaly partings: BEDROCK			NQ	3	100%	58%
2	64.4				NQ	4	100%	50%
3		End of Borehole Ref = >50 blows for 150mm						
4								
5								
6								
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
BOREHOLE No.: M-19
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
PROJECT No.: 10923
DATES: BORING: 97-08-07
WATER LEVEL: 97-08-28
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	68.27							
	67.8	Dark brown, SILT and SAND, trace organics			BS	1	-	-
1	66.8	Dense, olive brown, SAND, some gravel			SS	2	510	52
2	65.8	Compact to dense, grey, sand and silt, some gravel and clay : TILL			SS	3	610	13
					SS	4	0	Ref
3		Very severely fractuerd to severely fractured, grey, sandstone with frequent shaly partings: BEDROCK			NQ	5	38%	0%
4	63.5				NQ	6	93%	44%
5		End of Borehole Monitoring well installed Ref = >50 blows for 150mm						
6								
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

PROBE HOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-08-12 WATER LEVEL

BOREHOLE No. M-20

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY	SAMPLES N-VALUE OR RQD
	72.32						mm	
0		Solid auger probe through overburden to define inferred bedrock depth at 10.4 m						
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Continued Next Page

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

PROBE HOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-08-12 WATER LEVEL

BOREHOLE No. M-20

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR RQD
10								
	61.9							
		End of Probe Hole Auger refusal on inferred bedrock						
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited
LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON
DATES: BORING: 97-08-12
WATER LEVEL:

BOREHOLE No.: M-21
PROJECT No.: 10923
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	72.43							
	72.2	225 mm ASPHALT						
		Grey, sand and gravel, some silt: FILL			BS	1	-	-
1								
	70.8							
		Brown, sand, pockets of silty clay : FILL			SS	2	325	12
2								
3	69.4							
		Firm to stiff, brown, silty clay, trace to some sand : FILL			SS	3	275	2
4								
5	67.4				SS	4	525	19
		Compact, dark brown to black, sand, some silt, trace organics : FILL						
6	66.3							
		Very stiff, olive grey, CLAYEY SILT, trace to some sand			SS	5	-	13
7								
8								
9								
	62.7				SS	6	610	9
		Compact, grey, SAND						
10								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level
Groundwater Level Measured in Standpipe
Proposed Pipe Invert

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

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JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-08-12 WATER LEVEL

BOREHOLE No. M-21

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
10		Compact, grey, SAND						
11								
12	60.2							
	59.8	Grey, SILT and CLAY			SS	7	610	21
		Compact, grey, SAND						
13								
14								
15	57.2							
		Compact, grey, silt, trace sand, trace gravel, trace clay : TILL			SS	8	450	21
16								
17		- boulders						
18		- boulders						
19								
20								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Continued Next Page

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

DRAFT BOREHOLE RECORD

CLIENT: Morrison Hershfield Limited

LOCATION: West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING: 97-08-12

WATER LEVEL:

BOREHOLE No.: M-21

PROJECT No.: 10923

DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
20		Compact, grey, silt, trace sand, trace gravel, trace clay, frequent boulders: TILL						
21								
22								
23	49.4	Very severely fractured, grey sandstone with shaly partings: BEDROCK			NQ	9	100%	0%
					NQ	10	70%	0%
24	48.3	End of Borehole						
25								
26								
27								
28								
29								
30								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

PROBE HOLE RECORD

CLIENT Morrison Hershfield Limited

LOCATION West Transitway Project, Stn. 6+740 to 7+070, Ottawa, ON

DATES: BORING 97-08-12 WATER LEVEL

BOREHOLE No. M-22

PROJECT No. 10923

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR RQD
0	72.45							
4		Solid auger probe through overburden to define inferred bedrock depth at 8.3 m						
8	64.2							
		End of Probe Hole Auger refusal on inferred bedrock						

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Proposed Pipe Invert

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

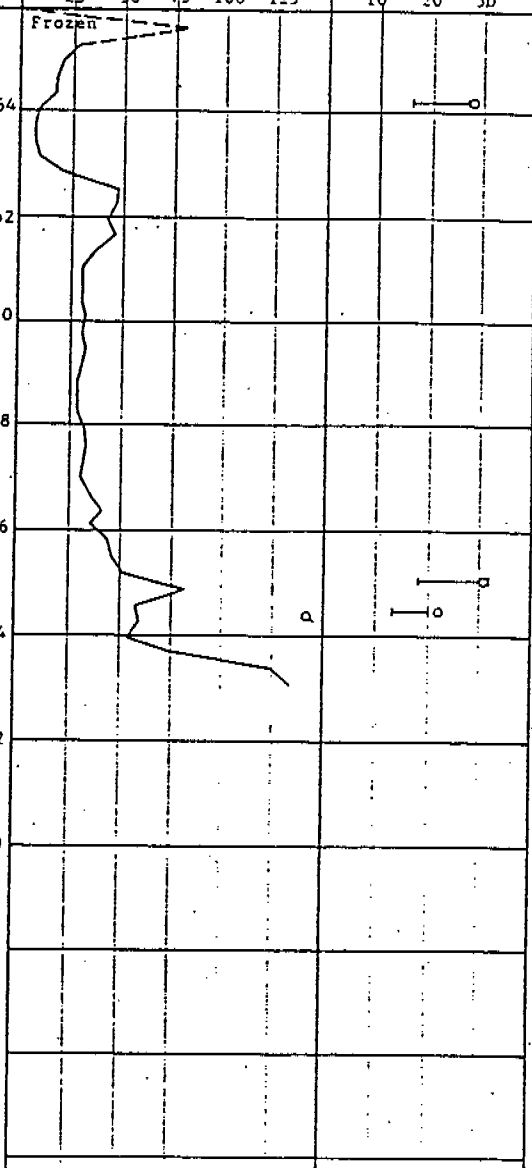
RECORD OF BOREHOLE No 3 (Formerly: WP 909-64) METRIC

W P 124-87-01 LOCATION Co-Ords N 5022 875.0; E 359 183.5 ORIGINATED BY PLW

DIST 9 HWY 417 BOREHOLE TYPE Washboring & Diamond Drilling COMPILED BY [illegible]

DATUM Geodetic DATE 1966 03 10 CHECKED BY TCK

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.9	Ground Level								
0.0	Clayey Silt with Interbedded Sandy Silt Soft to Firm		1	SS	4	*			
			2	TW	PM				
64.1 / 1.8			3	SS	14		64	19.8	
			4	SS	31				
	Silty Sand to Sand Some Gravel Compact to Very Dense		5	SS	18		62		
			6	SS	26				2 96 (2)
			7	SS	12		60		
			8	SS	11		58		51 49 0 0
			9	SS	62				
55.5 / 10.4	Clayey Silt with Interbedded Sandy Silt		10	SS	31		56		
54.3 / 11.6	Very Stiff to Hard		11	TW	PM			20.6	
			12	SS	35		54		
	Silty Sand to Sand Dense to Very Dense		13	SS	31		52		
			14	SS	56		50		1 58 33 8
			15	SS	48		48		1 90 7 2
44.8 / 21.1	Bedrock		16	RC	REC		46		
43.7	Shale Sound			AXT	100%		44		
22.2	END OF BOREHOLE								



* Hole caved in at Elev. 64.8m
Water Level Not Established

+³, x⁵ : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20, 10)

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 6

METRIC

W P 124-87-01 LOCATION Co-Ords N 5 022 884.2; E 359 214.1 ORIGINATED BY MS

DIST 9 HWY 417 BOREHOLE TYPE B-Casing, BX Rock Core COMPILED BY MS

DATUM Geodetic DATE 88 08 09 CHECKED BY TCK

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, • QUICK TRIAXIAL, + FIELD VANE, x LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL / WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
72.5	Ground Surface										
0.0 / 71.9	Concrete Slab		1	RC	-						
0.6 / 66.4	Sand Fill						72, 70, 68				
6.1 / 63.9	Silty Sand to Sand Compact		2 3	SS SS	10 23		66, 64		o		0 46 36 18
8.6 / 60.9	Clayey Silt with interbedded Silty Sand Stiff to very Stiff		4 5	SS SS	11 29		62				0 46 37 17
11.6 / 57.9	Silty Sand to Sand Trace of Clay Very Dense		6 7	SS SS	63 52		60		o		0 78 16 6
14.6 / 54.8	Bedrock Sandstone, Shale and Silty Sandstone		8 9	RC RC	REC 89% REC 87%		58, 56				RQD = 8% RQD = 8%
17.7	END OF BOREHOLE										

+3, x5 : Numbers refer to Sensitivity

15 20 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 6B

METRIC

W.P. 12[illegible]-87-01 LOCATION Co-Ords N 5022 887.9; E 359235.6 ORIGINATED BY MS

DIST 9 HWY [illegible] BOREHOLE TYPE H-S Auger, BX Rock Core COMPILED BY MS

DATUM Geodetic DATE 88 08 05 CHECKED BY TCK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
67.8	Ground Surface										
0.0	Sand and Gravel (Fill)					*					
66.6											
1.2	Bedrock		1	RC	REC	76%	66				RQD = 0%
65.4	Sandstone		2	RC	REC	65%					RQD = 0%
2.4	END OF BOREHOLE										

* Borehole Dry

+3, ×5 : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20, 10)

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 6C- METRIC

W P 124-87-01 LOCATION Co-Ords N 5022 897.4; E 359 321.2 ORIGINATED BY MS

DIST 9 HWY 417 BOREHOLE TYPE H-S Auger, 'B' Casing, BX Rock Core COMPILED BY MS

DATUM Geodetic DATE 88 08 05 CHECKED BY TCK

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	SAMPLE TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, x LAB VANE)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L (WATER CONTENT (%))	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
72.4 / 0.0	Ground Surface										
71.8	Asphalt					*	72				
0.6	Clayey Silt to Sand (Fill)		1	AS	-						
							70				
			2	SS	11						
			3	SS	60/15cm		68				
66.0											
6.4	Silty Sand to Sand		4	SS	39		66				
65.1											
7.3	Bedrock										
63.7	Sandstone		5	RC	REC 89%		64				RQD = 0%
8.7	END OF BOREHOLE										

*Water Level Not Established

+3, x5 : Numbers refer to Sensitivity

15 20 10 S (%) STRAIN AT FAILURE

Ontario Ministry of Transportation

RECORD OF BOREHOLE No 89-4 METRIC

W P 120-87-00 LOCATION Co-Ords N 5,022,890.0; E 359,174.0 (HWL C-5) ORIGINATED BY R.H.

DIST 9 HWY 417 @ 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY R.H.

DATUM Geodetic DATE July 11, 1989 CHECKED BY G.J.K.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, • QUICK TRIAXIAL, + FIELD VANE, x LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
67.6	Ground Surface										
0.0	Silty Sand, some gravel; FILL. Dark grey. Very loose to Compact		1	SS	26		67				
			2	SS	2		66				
64.9							65				
2.7	Silty Clay, some sand. Grey stiff		3	SS	4	Standpipe	64		o		
63.6											
4.0	Sand, trace silt, trace to some gravel. Grey to greyish brown. Loose to compact. -cobbles 4.0m to 4.6m; 5.0m to 6.1m. -Clayey silt zone at 7.8m		4	SS	11		63		o		11 84(5)
			5	SS	14		62				
			6	SS	5		61				
			7	SS	11		60				
57.8							59				
9.8	End of Borehole						58				

+3, x5 : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20, 10)

OFFICE REPORT ON SOIL EXPLORATION

JACQUES, WHITFORD LIMITED

BOREHOLE RECORD

94-2

Sta. 8+114 4.0m LT

CLIENT McCormick Rankin Consulting Engineers

LOCATION West Transitway

DATES: BORING 94-06-27 WATER LEVEL 94-07-13

PROJECT No. 10494

BOREHOLE No. 94-2

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	72.03							
	71.9	75 mm TOPSOIL & ROOTMAT			SS	1	600	10
	71.3	Compact, brown silty sand, trace clay: FILL			SS	2	100	50/100
1	71.1	Compact, brown SAND, some silt, trace gravel			RC	3	57%	10%
2		Very severely fractured to sound shaly limestone: BEDROCK			RC	4	100%	91%
3					RC	5	97%	97%
	68.6	End of Borehole - monitoring well installed						
4								
5								
6								
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Proposed Pipe Invert

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

BOREHOLE RECORD

94-5

Sta. 6+907 25.5 m LT

CLIENT McCormick Rankin Consulting Engineers

LOCATION West Transitway

DATES: BORING 94-06-28 WATER LEVEL ---

PROJECT No. 10494

BOREHOLE No. 94-5

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	67.40							
	66.8	Firm, brown silty clay, trace sand: FILL			SS	1	600	4
		End of Borehole Auger Refusal on Inferred Bedrock - standpipe installed						

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Proposed Pipe Invert

- ◘ Field Vane Test, kPa
- ☐ Remoulded Vane Test, kPa
- △ Pocket Penetrometer Test, kPa