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**PRELIMINARY FOUNDATION INVESTIGATION
AND DESIGN REPORT**

**GWP 731 - 93 - 00
HIGHWAY 62, PALLISER CREEK BRIDGE WIDENING
COUNTY OF HASTINGS
DISTRICT 43, BANCROFT**

MINISTRY OF TRANSPORTATION ONTARIO

PROJECT NO. ONO11632

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**G.W.P. 731-93-00
HIGHWAY 62, PALLISER CREEK BRIDGE WIDENING
COUNTY OF HASTINGS
DISTRICT 43, BANCROFT**

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

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FOUNDATION INVESTIGATION REPORT

for

**G.W.P. 731-93-00
Highway 62, Palliser Creek Bridge
County of Hastings
District 43, Bancroft**

1.0 INTRODUCTION

This report presents the results of a foundation investigation carried out for the proposed widening of the existing Palliser Creek Bridge on Highway 62, north of the city of Belleville. The widening is understood to be approximately 7.0 m in width and will be to the west of the existing structure. It is also understood that the widened portion will need to be supported by on a new substructure.

The foundation investigation was carried out in general accordance with our proposal number ONO 00765 dated December 20, 2002. Authorization to proceed was provided by the Ministry of Transportation of Ontario (MTO) under Agreement Number 4005-A-000109 with Totten Sims Hubicki Limited (TSH), the Prime Consultant for this project.

This report has been prepared specifically and solely for the project described herein. It contains factual information pertaining to the subsurface conditions which was obtained as part of this investigation.

2.0 SITE DESCRIPTION AND GEOLOGY

The subject site is within the limits of MTO project G.W.P. 731-93-00 (Highway 62 north of the City of Belleville). The site location is shown on the Key Plan provided in Appendix 1 (Drawing No. 11632-1). It is noted that for project orientation purposes, Highway 62 will be assumed to run north-south at the Palliser Creek Bridge, with chainage increasing from south to north.

Physiographically, the Palliser Creek Crossing is located within the eastern most limit of the Trent Embayment of the Iroquois Plain. This region is characterized by a drumlin and clay flat landscape. Lacustrine soil deposits of various types are present in the region, however, stratified silts are anticipated at the Palliser Creek Crossing.



Palliser Creek flows from west to east and is approximately 11 m in width. Water depths were less than 1 m at the time of the investigation.

The banks of the creek to the east and west of the existing structure were steeply sloped for approximately 1 m above water level and then very gradually sloped upwards away from the creek. Ground surface was vegetated with grass. Drainage in the area consisted of overland flow directed towards the creek.

A plan view and cross sections are shown on Drawing No. 11632-1, provided in Appendix 1.

3.0 PROCEDURE

3.1 Field Investigation

The site soil conditions were investigated with a borehole drilling investigation and laboratory testing program. The drilling was carried out using a track-mounted CME-55 drill rig on March 11, 2003.

A total of five (5) boreholes, designated as BH03-1 through BH03-5, were put down during the field investigation. Boreholes BH03-1 and BH03-2 were located on the south side of the creek and Boreholes BH03-3, BH03-4 and BH03-5 were put down on the north side of the creek.

The boreholes were advanced through the overburden using hollow stem augers. The subsurface conditions were identified in the field by Jacques, Whitford and Associates Limited (JWA) personnel from samples obtained while carrying out Standard Penetration Tests (SPT) (ASTM D1586) at regular intervals (760 mm). The recovered soil samples were stored in moisture proof containers and returned to our laboratory. Bedrock samples were obtained by coring with N-sized coring equipment. The subsurface conditions are described in detail in the Borehole Records presented in Appendix 2.

The undrained shear strength of the cohesive soils encountered at the site were measured using a field vane test apparatus and an MTO sized vane. The vane tests were carried out through the hollow stem augers into the drilled holes. After completion of each test, the blades of the vane were washed to remove any accumulated soil clumps to ensure the minimum resistance to insertion during the following test.

Standpipes were installed in Boreholes BH03-1 to BH03-4. Prior to completing the investigation, the boreholes were grouted with a cement/bentonite mix.



3.2 Survey

Borehole locations were established in the field by JWA personnel relative to existing site features. The ground surface elevations at the borehole locations were surveyed relative to the pavement elevation at centreline at Station 16+600 which has an Elevation of 105.0 m Geodetic.

3.3 Laboratory Testing

All samples returned to the laboratory were subjected to detailed visual classification by a geotechnical engineer. Routine testing, consisting of moisture content testing, Atterberg limit determination, and grain size distribution analysis was carried out on at least 25% of the collected samples. One representative soil sample was submitted for pH, sulphate and resistivity testing to assess the potential for corrosion of buried steel and the potential for sulphate attack on buried concrete.

The bedrock core samples were examined in the laboratory by a Geological Engineer. Selected samples were subject to Unconfined Compressive Strength Testing.

No complex testing was deemed to be necessary based on the soil conditions.

All soil and bedrock samples will be stored for a period of one year after issuance of the final report. Unless otherwise directed, the stored samples will be disposed of after this period.

4.0 SUBSURFACE CONDITIONS

4.1 Subsurface Profile

The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix 2. An explanation of the symbols and terms used to describe the Borehole Records is also provided. In general, the subsurface profile consists of a thin topsoil layer, overlying a fill layer, on top of native clay, over till underlain by bedrock.

A borehole location plan and stratigraphic section of the soils encountered within the boreholes are provided on Drawing ONO11632-1.

4.1.1 Fill: Silty Sand, Trace Gravel to Gravelly

Granular fill was encountered in all of the Boreholes. It consisted of a brown silty sand, trace gravel to gravelly. The fill was observed to extend to a maximum depth of approximately 1.5 m (Elevation 101.5 m) at Borehole BH03-1. The moisture content of the 5 samples of fill tested ranged from 12% to 31% and averaged 24%. SPT 'N' values indicate the fill was generally compact. The pavement structure was observed in Borehole BH03-5 to consist of 180 mm of asphalt over 180 mm of gravelly sand base over sand, some gravel to the termination depth of the borehole at 1.5 m below ground surface.

4.1.2 Clay, with Silt, Trace Sand

A layer of clay with silt, trace sand was observed in Boreholes BH03-1 through BH03-4 beneath the granular fill. The upper portion of the deposit corresponds to the crust and was observed to have a greater stiffness. The clay, with silt, trace sand layer varied significantly in thickness and extended to a depth of as much as 5.2 m below ground surface at Borehole BH03-1. The undrained shear strength of the clay with silt trace sand was consistently between 100 kPa and 110 kPa. This material can be classified as very stiff. The moisture content of the 18 samples tested ranged from 24% to 48% with an average of 33%. Hydrometer analysis carried out on 5 representative samples determined that the clay portion ranged from 62% to 77% with an average of 71%; the silt portion ranged from 22% to 39% with an average of 27%; and the sand content ranged from 0% to 5% with an average of 2%. One Atterberg limit test carried out on the clay, with silt, trace sand sample from BH03-4 showed a liquid limit of 59%, a plastic limit of 21% and a plasticity index of 38%. The results of the grain size distribution and Atterberg Limit Testing are shown on the Figures in Appendix 2.

4.1.3 Gravelly Sand, some Silt (TILL)

A till deposit was observed beneath the clay unit in Boreholes BH03-1 through BH03-4. The thickness of this unit was as great as 1.6 m in Borehole BH03-2. The layer was observed to be a gravelly sand with some silt. The moisture content of the 6 samples tested ranged from 8% to 28% with an average of 16%. SPT 'N' values ranged from 21 to 37 and averaged 30, suggesting a compact to dense state.



4.1.4 Bedrock

Bedrock was proven by coring in Boreholes BH03-2 and BH03-3. The bedrock was observed to consist of limestone. Core recoveries ranged from 75% to 100% and averaged 89%. RQD values on the recovered cores ranged from 29% to 100% with a weighted average of 77%. This indicates a fair to excellent rock quality. Rock core descriptions are provided in Appendix 2. The Geodetic elevation of bedrock encountered at the boreholes is presented below:

Location	Bedrock Elevation (Geodetic)
BH03-1	96.83 m(inferred)
BH03-2	97.04 m
BH03-3	97.52 m
BH03-4	97.20 m(inferred)

Unconfined compressive strength testing was carried out on 3 samples with the results ranging from 86 MPa to 113 MPa.

The results of the Unconfined Compressive Strength Testing are presented below:

Location	Elevation	UCS, MPa
BH03-3	96.74	113
BH03-3	95.21	86
BH03-3	94.6	95

4.2 Groundwater

Groundwater levels were measured within the standpipes on March 11, 2003. The measured water levels are summarized in the table below. The water level in Palliser Creek was also measured on March 11, 2003.

Location	Ground Surface Elevation (m)	Depth to Groundwater (m)	Groundwater Elevation (m)
BH03-1	102.93	1.16	101.77
BH03-2	102.44	0.76	101.68
BH03-3	102.22	0.67	101.55
BH03-4	102.40	0.81	101.59
Creek			101.42

Fluctuations in the groundwater level due to seasonal variations or in response to a particular precipitation event should be anticipated.

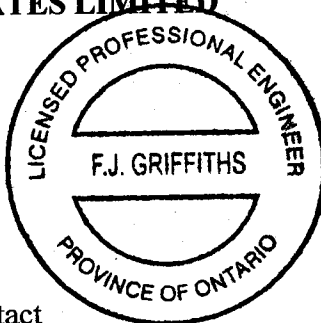
The depth to groundwater as shown in the table above is influenced by the distance from the edge of Palliser Creek and topography. Groundwater flow is interpreted to be toward the creek.

5.0 CLOSURE

A subsurface investigation is a limited sampling of a site. The subsurface conditions given herein are based on information gathered at the specific borehole locations. 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.

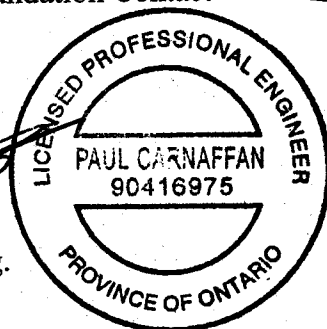
Yours very truly,

JACQUES, WHITFORD AND ASSOCIATES LIMITED



Fred J. Griffiths, Ph.D., P.Eng.

Designated Principal MTO Foundation Contact



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



FOUNDATION DESIGN REPORT

for

G.W.P. 731-93-00

Highway 62, Palliser Creek Bridge

County of Hastings

District 43, Bancroft

6.0 DISCUSSION

6.1 Proposed Development

It is understood that the Ministry of Transportation of Ontario (MTO) plans to widen the existing Palliser Creek Crossing. The existing structure was built in 1963 and consists of a single span concrete structure, 14 m in length, supported on H-piles. It is understood that the H-piles were driven to bedrock and that the load capacity of the piles is 65 tons (578 kN), as noted in the Structural Design Report of November 25, 2002. Retaining walls wrapping around the approach fills are present at all four corners of the abutments.

The widening is understood to be 7.075 m in width and will be to the west of the existing structure. It is understood that the existing deck expansion joints are to be modified by using semi-integral construction. It is also understood that the widened portion will need to be supported by new abutments and foundations. The approach fills will be widened to match the expanded bridge deck. As much as 3.5 m of fill will be required. Embankment side slopes of 3H:1V are proposed. The proposed General Arrangement for the bridge shows new retaining walls wrapping back from the stream and no forward slope. It is understood that it is not proposed to modify the horizontal alignment and that the vertical alignment will be modified only slightly from El. 104.9 m and 105.0 m Geodetic at the south and north abutments respectively.

This work is a component of G.W.P. 731-93-00.

It is noted that, for project orientation purposes, Highway 62 will be assumed to run south-north at the Palliser Creek Bridge site, with chainage increasing from south to north.

6.2 Foundation Options

The use of spread footing foundations to support the widening would result in differential performance with the existing structure and is not recommended. Bedrock was encountered in the boreholes at between El. 97.0 m and 97.5 m Geodetic, or approximately 7.5 m below the finished grade.

The following table compares the available deep foundation options considered for this site:

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
H-piles	<ul style="list-style-type: none">- readily installed on a batter- rugged member- same support as existing	<ul style="list-style-type: none">-	low	-
Pipe-piles	<ul style="list-style-type: none">- can be installed on a batter	<ul style="list-style-type: none">- prone to damage during driving where rock is shallow- different support from existing	low	Damage to piles during driving resulting in need for additional piles.
Caissons	<ul style="list-style-type: none">- large capacity	<ul style="list-style-type: none">- difficult to batter- require tremie concrete- require cased holes- different support from existing	high	-

Based on a review of these options, it is recommended that the new abutments and retaining walls be founded on H-piles driven to bedrock.



6.3 Slope Stability

Based on the drilling investigation carried out in March 2003, the following soil parameters are appropriate for this site and have been utilized in the global stability analyses and determination of geotechnical design parameters presented within this report:

Soil Type\Parameter	Unit Weight	Angle of Internal Friction	Effective Cohesion	Undrained Shear Strength
Granular Fill	21 kN/m ³	35 °	0	
Earth Fill or SSM	19 kN/m ³	32 °	0	
Native Clay, with Silt	18 kN/m ³	27 °	10 kPa	100 kPa
Native Gravelly Sand, some Silt (Till)	20 kN/m ³	32 °	0	

The groundwater level has a significant impact on slope stability. Based on our laboratory measurements of water content, as well as the ground water levels measured in the standpipes installed at the site, it can be concluded that the groundwater level could reach as high as Elevation 102 m Geodetic.

It is understood that approach slabs will be utilized, therefore, in accordance with the CHBDC Section 6.9.5, the surcharge load equivalent to an additional fill height of 800 mm has not been applied to the embankment for the slope stability analyses.

The cross section at 16+560 has the greatest side-slope height at approximately 3.5 m. This cross-section represents the worst case for this site. An embankment side-slope of 2H:1V was assessed for Station 16+560.

The stability of the approach embankments at these locations were analyzed utilizing a computer analysis program called SLOPE/W, produced by Geo-Slope International. The program uses the Bishop's Modified Method of analysis. Potential slope failure surfaces were considered together with the soil parameters listed above.

The analyses yielded safety factors above 1.3 which is considered an appropriate minimum factor of safety for static loading in the finished condition. Graphical representations of the slope stability analysis output which show slope height, geometry, foundations, subsurface and groundwater conditions, and soil parameters are presented in Appendix 3 for both effective and total stress conditions.

6.4 Impact to Existing Structure and Embankment

The construction of the widened roadway will result in additional settlement of the existing embankment. It is estimated that settlement at the existing edge of shoulder will be less than 5 mm. Addition of downdrag forces to the existing piles has been considered, however, given the very small settlement values calculated, additional downdrag forces in the existing piles will be minimal and can be ignored.

It is understood that the piles supporting the existing abutment are either vertical or battered forward. Therefore, no conflicts between new piles and existing abutment piles are envisioned. The General Arrangement prepared for the Preliminary Design Study indicates that the existing retaining walls on the west side of the structure are supported on piles. The existing retaining walls are at an angle to the proposed abutment extension, thus it is possible that some of the existing piles will not be below the proposed abutment. Existing piles should be left in place where they do not conflict with the proposed development in order to minimize disturbance of the soils. These piles could be cut off after demolition of the retaining wall. Where existing piles conflict with the proposed development, they should be removed. The existing piles are in the order of 3 m to 4 m in length. Equipment capable of lifting approximately 150 kN plus the weight of the pile would be required to extract the piles. There will be some impact to the site soils caused by extracting the existing piles however, the axial capacity for new piles will not be affected as they will be end-bearing piles.

6.5 Dewatering

Water infiltration into temporary construction excavations is likely to be heavy where excavations are required below creek level. Sheet-pile cutoff walls may be required. Alternatively, Palliser Creek water could be controlled with temporary coffer dams and pumping. Department of Fisheries and Oceans requirements may need to be satisfied.

Groundwater infiltration from the clay layer is expected to be minimal, however, potential basal instability for temporary excavations due to groundwater conditions in the underlying till must be considered.

Environmental mitigation measures will have to be designed to limit sedimentation.



7.0 PRELIMINARY RECOMMENDATIONS

7.1 Structure Foundations

Axial Resistance

All structure foundations for abutments and retaining walls may be supported on end-bearing steel H-piles driven to bedrock. The following design parameters are recommended:

Location	Estimated Tip Elevation, Geodetic
South Abutment and Retaining Wall	97.0 m
North Abutment and Retaining Wall	97.5 m

It is noted that adequate geotechnical resistance is available from the bedrock at this site. Therefore the factored structural resistance of the pile will govern. The following values are recommended for the design of pile foundations and include a resistance factor of 0.5.

Pile Type	Factored Axial Resistance at ULS
HP 310 x 7 9	1450 kN
HP 310 x 110	2000 kN

Given the very short pile length, elastic deformation of the piles under loading will be small, therefore, SLS values will not apply at this site.

Downdrag forces induced as a result of consolidation of the clay under the widened approach fills must be considered. Settlement of the soils below the pile cap of as much as 12 mm to 20 mm is anticipated. As the piles will be end-bearing on bedrock and are not anticipated to compress significantly, the neutral plane will be near the pile toe. Application of the downdrag force should be as described in Section 6.8.4 of the CHBDC. The predicted downdrag force has been estimated based on unit negative skin friction equal to 37 kPa corresponding to an undrained shear strength of 100 kPa for the clay at this site. The unit negative skin friction has been applied to the perimeter of a box around the pile, i.e. the perimeter length of the web of the H-pile has not been included. The unit skin friction has been applied to a pile length of 2.9 m (underside of pile cap at El. 99.9 m, pile toe at El. 97.0 m). The following table provides estimates of downdrag:

Pile Type	Unfactored Downdrag Force
HP 310x79	130 kN
HP 310x110	135 kN

Lateral Resistance

Passive lateral resistance for vertical piles should be calculated as per C6.8.7.2 (Static Analysis i.e. Brom's method) of the CHBDC using the following unfactored geotechnical soil parameters:

Parameter	OPSS Granular B Type I	Clay	Till
Bulk Unit Weight, kN/m^3	21.2	18	20
Effective Friction Angle, degrees	35	-	32
Coefficient of Passive Earth Pressure	3.7	-	3.3
Design Undrained Shear Strength kPa	-	100	-

Lateral resistance within the Granular B Type I and the till should be calculated using the non-cohesive approach and within the clay using the cohesive approach, as defined in the CHBDC.

Lateral Deflections

The coefficient of horizontal subgrade reaction, which may be used for deflection calculations, may be estimated for cohesive soils as follows:

$$k_s = 67 C_u / d$$

where k_s = the coefficient of horizontal subgrade reaction (force per volume)

C_u = undrained shear strength of the soil = 100 kPa for this application

d = pile diameter

As the till layer under the clay unit at this site is of limited length, it is recommended that a uniform coefficient be assumed full depth. A design value of 100 kPa should be used for the undrained shear strength of the clay soil.



A reduction in the coefficient of horizontal subgrade reaction should be applied to account for pile spacing. The following table indicates the appropriate corrections as a function of pile spacing in the direction of loading. Pile spacing in the direction normal to the direction of loading has little to no effect provided it is greater than 2.5 d, where d is the width of the pile.

Spacing	k group
8d	1.0 k_s
6d	0.70 k_s
4d	0.40 k_s
3d	0.25 k_s

Tensile Resistance

Resistance to tensile loads should be calculated based on the shaft resistance of the piles in accordance with Section 6.8.5. A ULS Resistance Factor of 0.3 has been applied to derive the following values for use in design:

Pile Type	Factored Geotechnical Resistance (Tension) at ULS
HP 310 x 79	19 kN
HP 310 x 110	20 kN

Pile Notes

Piles should be equipped with driving shoes as per OPSD 3301.000 Type I. Piles should be driven to rock.

7.2 Earth Pressure Design

The abutments and retaining walls should be backfilled with free-draining material such as OPSS Granular B Type II or OPSS Granular A to prevent hydrostatic pressure build-up.

Computation of earth pressures should be in accordance with Section 6.9 of the CHBDC. For abutments or retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. For a structure with a horizontal backfill, the following unfactored soil parameters may be used for design. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.9.3 of the CHBDC.

Parameter	OPSS Granular B, Type II	OPSS Granular A
Bulk Unit Weight, γ (kN/m ³)	21.2	22
Effective Friction Angle, ϕ	32 degrees	35 degrees
Coefficient of Active Earth Pressure (K_a)	0.31	0.27
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43
Coefficient of Passive Earth Pressure (K_p)	3.2	3.7

Compaction of the granular backfill near the walls should be carried out using hand-operated equipment to prevent over stressing the abutment walls.

Drainage should be provided behind vertical walls to prevent hydrostatic pressure build-up. Drainage should be provided by installing a 100 mm diameter subdrain wrapped in geotextile. The subdrain should be installed as per OPSD 3501 and should provide positive drainage to a frost-free outlet. In addition, weep holes through the wall should be provided at regularly spaced intervals. Granular backfill should be designed as per OPSD 3501 using a depth of frost penetration, f , of 1.5 m.

Table A3.1.7 of the CHBDC indicates that the Zonal Acceleration Ratio for Belleville, which is 11 km south of the site, is 0.05. Reference is made to Section C4.6.4 of the CHBDC for the calculation of seismic forces on abutments and retaining walls.

It is recommended that Soil Profile I as defined in CHBDC Section 4.4.6.2 be used in the seismic design of this site.

7.3 Embankment Design

Embankment widenings should be constructed with benched slopes as per OPSD 208.010. Embankment side slopes should be constructed no steeper than 2H:IV. Embankment fill should consist of OPSS Select Subgrade Material.

Settlement of the underlying soil has been carried out using consolidation theory and the tangent modulus method. The 2.9 m thick clay unit was analyzed as four layers. Stress distribution was assessed based on a Boussinesq distribution.



As much as 3.5 m of fill will be required at some locations to achieve design grades at the approaches to the structure. This will induce as much as 25 to 30 mm of settlement in the underlying native materials. Due to the generally overconsolidated nature of these materials, it is anticipated that settlement will occur rapidly. Post construction settlements of the underlying soils will be less than 25 mm. Self settlement of the embankment fill of as much as 10 mm for 3.5 m of fill will occur. This settlement will be complete at the completion of construction.

7.4 Dewatering

It is anticipated that the excavation for the foundations of the abutments and retaining walls will be to approximate elevation 99.9 m or as much as 1.5 m below creek level. Shoring or a coffer dam and piping system will be required for the excavations.

Based on the soil profile at BH03-2, basal heave is a potential temporary construction concern. A Factor of Safety of less than 1.0 against basal heave due to artesian pressures in the underlying till has been generated by dividing the total weight of the soil to remain in place below the abutment excavation and above the till by the pore pressure in the till assuming a groundwater level of 101.4 m.

$$FS = \frac{(1.3 \times 18)}{(2.8 \times 9.8)} = \frac{23.4}{27.4} < 1.0$$

This suggests that the base of the excavation could heave during abutment construction. Design of shoring will need to account for basal heave. It is recommended that dewatering from the underlying till be carried out by installing a well within the sheet piled area. The well should be installed to be screened in the till unit and should be pumped to maintain the groundwater level in the till at a level no higher than Elevation 100.3 m for the south abutment and 100.6 m for the north abutment. It is recommended that the contract include a Non Standard Special Provision for dewatering.

7.5 Erosion Protection

The Structural Design Report of November 25, 2002, notes that erosion is occurring on the north bank of Palliser Creek upstream of the existing wing wall.

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. The creek slopes within 3 m of the structures should be surfaced with rip rap at least 300 mm thick placed on a Class II non-woven filter fabric. Where embankment construction includes earth fill, normal slope vegetation should be established as soon as possible after completion of the embankment fills in order to control surficial erosion.

The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site

7.6 Frost Protection

Pile caps should be provided with the equivalent of 1.5 m of earth cover for frost protection.

7.7 Other Construction Considerations

Site Grading and Preparation

All organic soils and other deleterious materials must be removed from beneath the proposed foundation units. Where deleterious materials are encountered, the material should be excavated, wasted and replaced. The lateral extent of such excavation should include all deleterious material within the embankment footprint.

Surficial vegetation, rootmat and topsoil should be removed beneath the approach embankments. Stripping of deleterious materials should be inspected by geotechnical personnel to ensure that all unsuitable materials are removed prior to placement of embankment fill. The exposed subgrade surface should be surface-compacted using suitable compaction equipment to 95% of Standard Proctor maximum dry density where applicable.

Where required for grading purposes, fill should consist of Select Subgrade Material (SSM), placed in lifts no greater than 300 mm and compacted to at least 95% Standard Proctor Maximum Dry Density (SPMDD).

Site preparation should be carried out in accordance with the requirements of *SP 902S01 Excavation and Backfilling - Structures*.

Excavation

Earth excavation should be carried out in accordance with OPSS-206.07.03. Side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations. The soils to be excavated for the proposed foundations should be considered as a Type 2 to 3 soil. Above the creek and ground water level, temporary cut slopes should be no steeper than 1 horizontal to 1 vertical from the base of the excavation. For excavations below creek and groundwater levels, shoring will be required. Shoring design should meet the requirements of Performance Level 2 as per SP539501.



Encroachment of excavations into the foreward and side slopes of the existing structure will require special attention. Excavations will not be permitted within the influence zone of the existing abutments. The influence zone includes all materials below an imaginary line drawn at an angle of 1 horizontal to 1 vertical downward and away from the vertical edges of the abutments.

Cement Type and Corrosion Protection

A representative soil sample was submitted to Paracel Laboratories in Ottawa, Ontario, for analysis of pH, resistivity and water soluble sulphate, in order to determine cement type and reinforcing steel protection requirements. The results are presented in the table below.

Location	Borehole	Sample	pH	Resistivity	Soluble Sulphate	Chloride
North Abutment	BH 03-3	5	7.85	1800 ohm·cm	40 µg/g	270 µg/g

The soluble sulphate results indicate that a Type 10 Portland cement would be suitable for use in concrete mixtures at this site. The chloride, pH and resistivity results should be considered by the structural designer when designing corrosion protection system.

8.0 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. 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 recommendations.

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 AND ASSOCIATES LIMITED



Fred Griffiths, Ph.D., P.Eng.
Designated Principal MTO Foundation Contact



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



P:\2004\10000\11632\Prelim Report 2004 04 22.doc



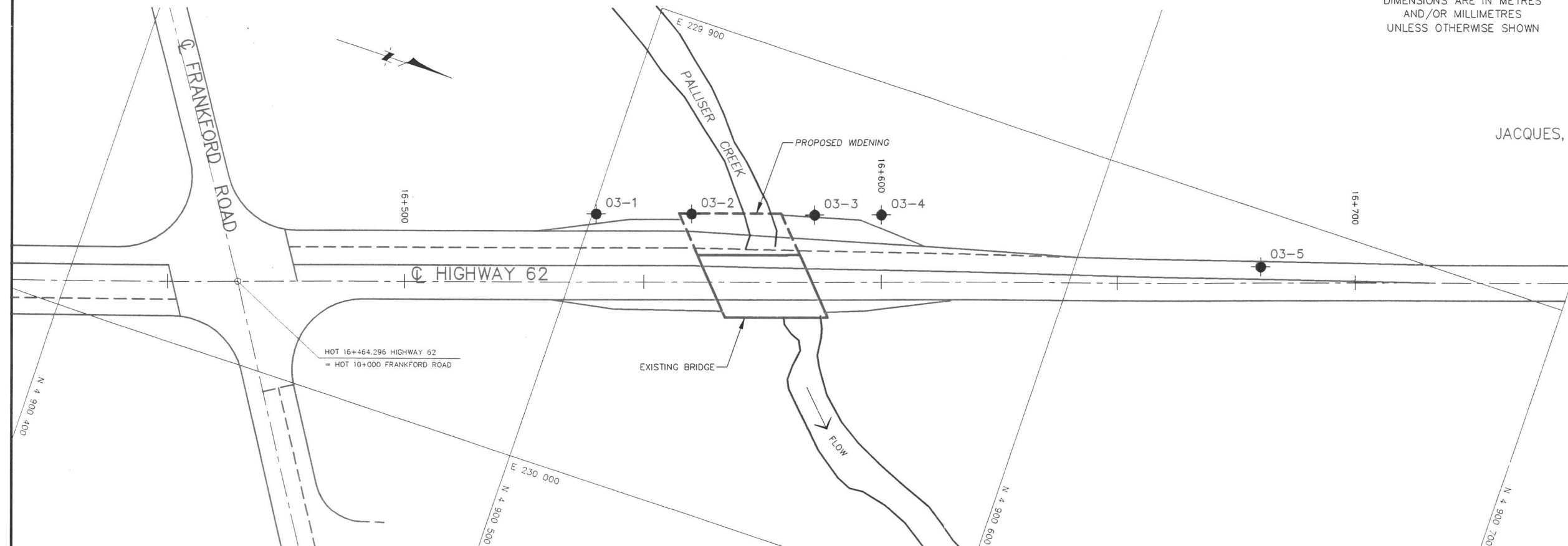
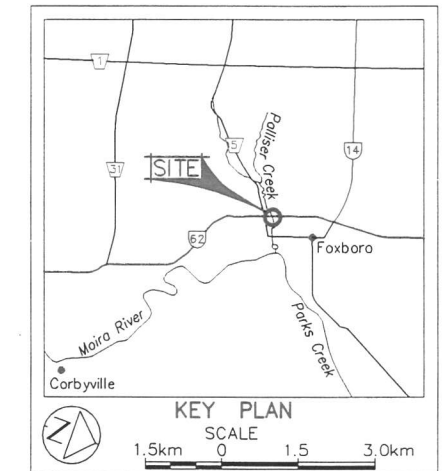
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AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 731-93-00
PALLISER CREEK - PRELIMINARY
FOUNDATION INVESTIGATION
HIGHWAY 62
BORE HOLE LOCATIONS & SOIL STRATA

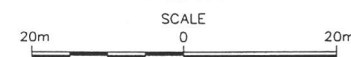


SHEET
1

JACQUES, WHITFORD & ASSOCIATES LIMITED



PLAN



LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60' Cone, 475 J/blow)
- WL at time of investigation 97 07
- WL in Piezometer
- Piezometer

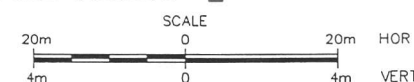
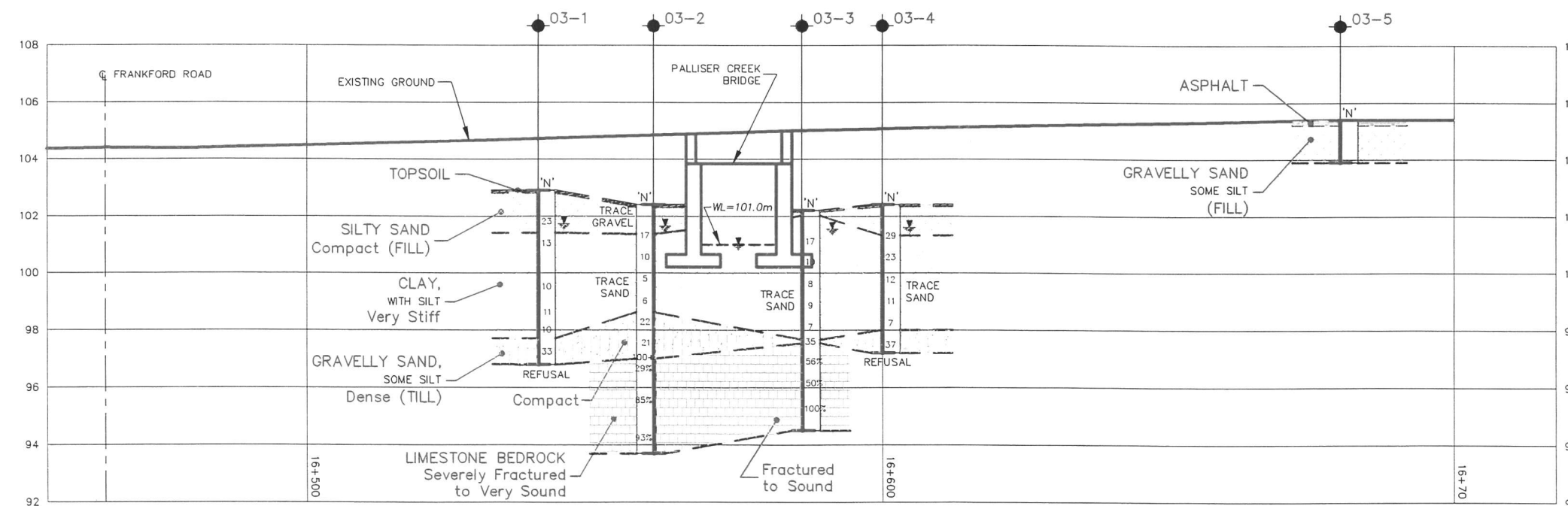
No	ELEVATION	CHAINAGE	OFFSET FROM CL
03-1	102.9	16+540	14.4 LT
03-2	102.4	16+560	14.4 LT
03-3	102.2	16+586	14.0 LT
03-4	102.4	16+600	14.2 LT
03-5	105.4	16+680	3.4 LT

NOTE

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

PROFILE ALONG C HIGHWAY 62



REV	DATE	BY	DESCRIPTION
1	2003-06-18	GBB	APPROVED

GEOCRES No

HWY No 62

SUBM'D FRG CHECKED DATE 2003-06-18 SITE

DRAWN GBB CHECKED APPROVED DWG 11632-1

11632-1 WP 731-93-00

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	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	<i>N</i> Value	Compactness %
<i>Very Loose</i>	<4	<15
<i>Loose</i>	4-10	15-35
<i>Compact</i>	10-30	35-65
<i>Dense</i>	30-50	65-85
<i>Very Dense</i>	>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	
<i>Very Soft</i>	<0.25	<12.5	<2
<i>Soft</i>	0.25-0.5	12.5-25	2-4
<i>Firm</i>	0.5-1.0	25-50	4-8
<i>Stiff</i>	1.0-2.0	50-100	8-15
<i>Very Stiff</i>	2.0-4.0	100-200	15-30
<i>Hard</i>	>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	<i>Very Thick</i>	<i>Very Wide</i>
600-2000	<i>Thick</i>	<i>Wide</i>
200-600	<i>Medium</i>	<i>Moderate</i>
60-200	<i>Thin</i>	<i>Close</i>
20-60	<i>Very Thin</i>	<i>Very Close</i>
<20	<i>Laminated</i>	<i>Extremely Close</i>
<6	<i>Thinly Laminated</i>	

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

Terminology describing weathering:

<i>Slight</i>	-	Weathering limited to the surface of major discontinuities. Typically iron stained.
<i>Moderate</i>	-	Weathering extends throughout rock mass. Rock is not friable.

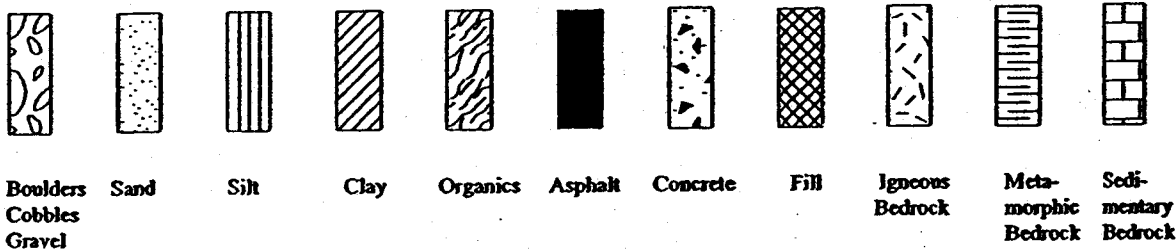


High

Weathering extends throughout rock mass. Rock is friable.

STRATA PLOT

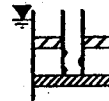
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



WATER LEVEL MEASUREMENT



Borehole or
Standpipe



Piezometer

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)	BS	Bulk sample
ST	Shelby tube or thin wall tube	WS	Wash sample
PS	Piston 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	H	Hydrometer analysis
G _s	Specific gravity of soil particles	̄a	Unit weight
k	Permeability (cm/sec)	C	Consolidation
↓	Single packer permeability test; test interval from depth shown to bottom of borehole	CD	Consolidated drained triaxial
↓	Double packer permeability test; test interval as indicated	CU	Consolidated undrained triaxial with pore pressure measurements
○	Falling head permeability test using casing	UU	Unconsolidated undrained triaxial
↓	Falling head permeability test using well point or piezometer	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)



Ecological Paper / Papier Écologique

1 OF 1

METRIC

ORIGINATED BY **DF**

COMPILED BY **AF**

CHECKED BY FG

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 03-2

1 OF 1

METRIC

W.P. 731-93-00 LOCATION Palliser Creek, 16+560, 14.4 Lt, Sidney Township, Ontario ORIGINATED BY DF
DIST 43 HWY # 62 BOREHOLE TYPE Hollow Stem Augers with SplitSpoons COMPILED BY AF
DATUM Geodetic DATE 03.10.03 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
102.4 0.1	80 mm TOPSOIL Silty sand, trace gravel, compact, brown (FILL)		1	GS			102							
101.4 1.0	CLAY, with silt, trace sand, very stiff, brown		2	SS	17		101							
			3	SS	10		100							
			4	SS	5									
99.2 3.2	CLAY, with silt, very stiff, grey		5	SS	6		99							
98.6 3.8	Gravelly sand, some silt, compact, grey (TILL)		6	SS	22		98							
			7	SS	21									
97.0 5.4	BEDROCK Limestone, severely fractured to very sound		8	SS	100+		97							
			9	NQ	29 %		96							
			10	NQ	85 %		95							
			11	NQ	93 %		94							
93.7 8.8	End of Borehole Standpipe Installed Water Level Measured on March 11, 2003													

MTO 11632B.GPJ ON MOT.GDT 17-06-03

RECORD OF BOREHOLE No BH 03-3

1 OF 1

METRIC

W.P. 731-93-00 LOCATION Palliser Creek, 16+586, 14.0 Lt, Sidney Township, Ontario ORIGINATED BY DF
DIST 43 HWY #62 BOREHOLE TYPE Hollow Stem Augers with Split Spoons COMPILED BY AF
DATUM Geodetic DATE 03.11.03 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								WATER CONTENT (%)				
102.2						20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L			
100.0	50 mm TOPSOIL						○ UNCONFINED	× FIELD VANE				
102.0	Silty sand, brown (FILL)						● QUICK TRIAXIAL	× LAB VANE				
102.0	CLAY with silt, trace sand, very stiff, brown to grey		1	GS							47.8	
			2	SS	17							
			3	SS	10							
			4	SS	8							
			5	SS	9							
98.4	CLAY with silt, very stiff, grey		6	SS	7							
97.7			7	SS	35							
97.6	Gravelly sand, trace silt, dense, grey (TILL)		8	NQ	56 %							
4.7	BEDROCK Limestone, fractured to sound		9	NQ	50 %							
			10	NQ	100 %							
94.5	End of Borehole											
7.8	Standpipe Installed											
	Water Level Measured on March 11, 2003											





MT0 11632B.GPJ ON MOT.GDT 11-06-03

RECORD OF BOREHOLE No BH 03-4

1 OF 1

METRIC

W.P. 731-93-00 LOCATION Palliser Creek, 16+600, 14.2 Lt, Sidney Township, Ontario ORIGINATED BY DF
 DIST 43 HWY # 62 BOREHOLE TYPE Hollow Stem Augers with Split Spoons COMPILED BY AF
 DATUM Geodetic DATE 03.11.03 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
102.4							20	40	60	80	100							
100.7	50 mm TOPSOIL Silty sand, compact, brown (FILL)		1	GS														
101.3	CLAY, with silt, very stiff, brown		2	SS	29													
100.6	CLAY, with silt, trace sand, very stiff, grey		3	SS	23													
			4	SS	12													
			5	SS	11													
			6	SS	7													
98.0	Gravelly sand, some silt, dense, grey (TILL)		7	SS	37													
97.2	End of Borehole Auger Refusal on Inferred Bedrock Standpipe Installed Water Level Measured on March 11, 2003																	

RECORD OF BOREHOLE No BH 03-5

1 OF 1

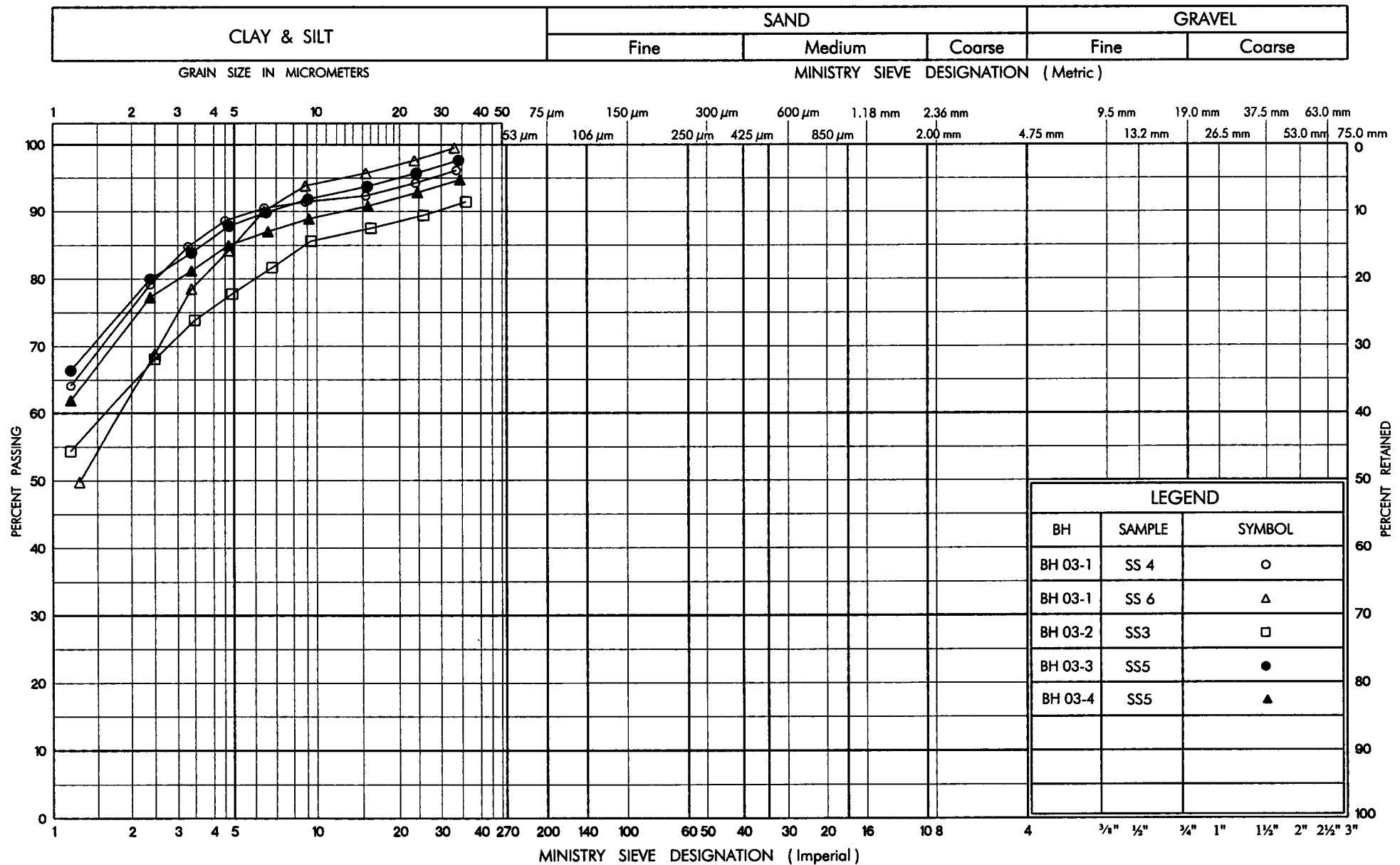
METRIC

W.P. 731-93-00 LOCATION Palliser Creek, 16+680, 3.4 Lt, Sidney Township, Ontario ORIGINATED BY DF
DIST 43 HWY # 62 BOREHOLE TYPE Solid Stem Auger COMPILED BY AF
DATUM Geodetic DATE 03.11.03 CHECKED BY FG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W		
							20	40	60	80	100					
105.4	180mm ASPHALT															
105.2	Gravelly sand, some silt, brown (FILL)		1	GS												
105.1	Sand, some gravel, some silt, brown (FILL)															
103.9	End of Borehole															
1.5																

MTO 11632B.GPJ ON_MOT.GDT 11-06-03

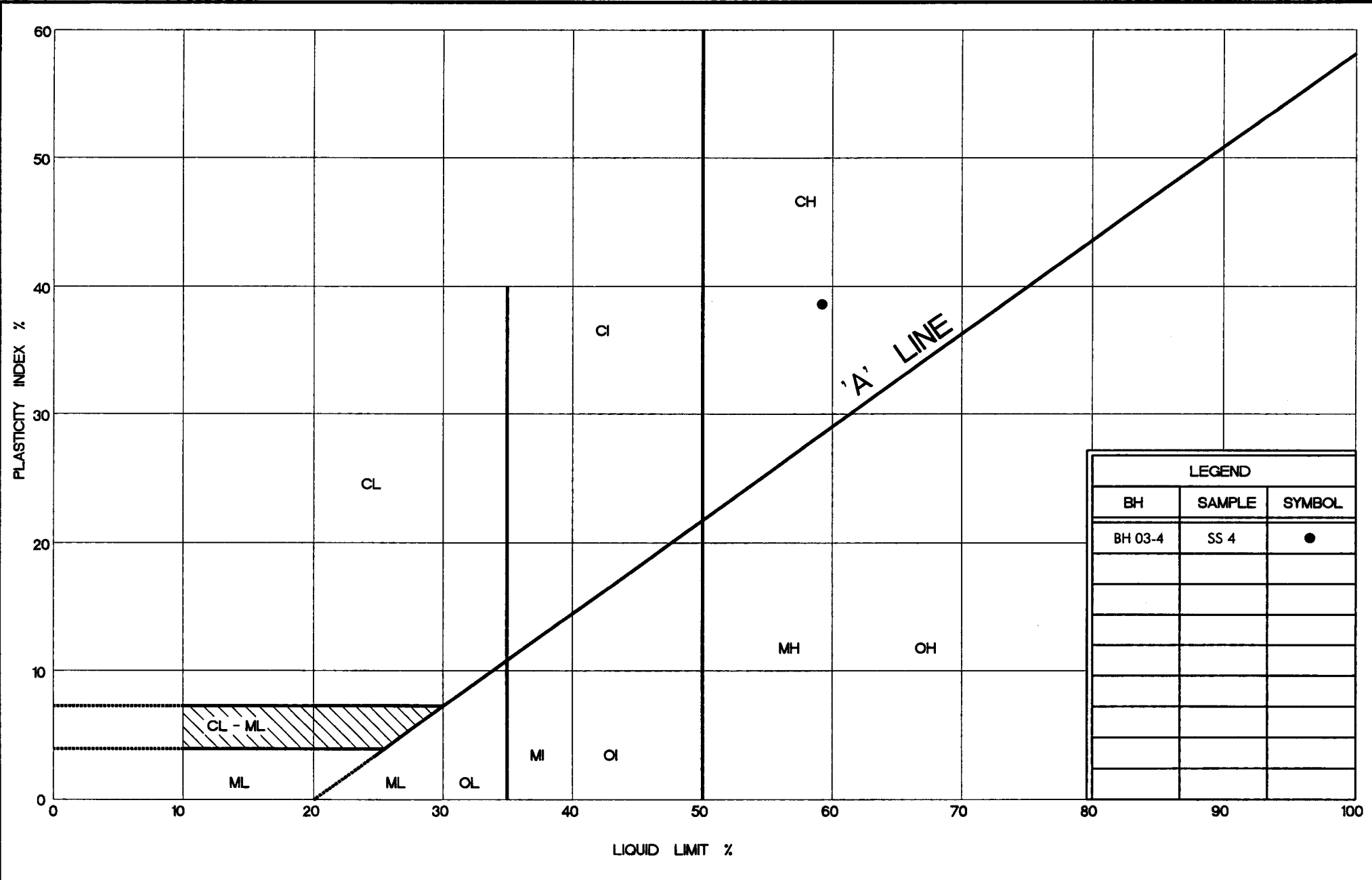
UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION
CLAY

FIG No 1

W P 731-93-00



ROCK CORE DESCRIPTION

GWP 731 - 93 - 00

CORE RECOVERY					CORE DESCRIPTION
BH #	RC #	DEPTH (m)	CR %	RQD %	
03 - 2	9	5.44 to 6.12	80	29	LIMESTONE - grey, occasional layers of oolitic limestone and shale. Unweathered, bedding thin to very thin horizontal to 45°
03 - 2	10	6.12 to 7.65	97	85	LIMESTONE - grey, occasional layers of oolitic limestone and dark grey shale, unweathered, bedding thin to very thin, horizontal to 45°
03 - 2	11	7.65 to 8.76	100	93	LIMESTONE - grey, occasional layers of oolitic limestone and dark grey shale, unweathered, bedding thin to very thin, horizontal to 45°
03 - 3	8	4.70 to 5.92	81	56	LIMESTONE - grey, occasional layers of oolitic limestone and dark grey shale, unweathered, bedding thin to very thin, horizontal to 45°
03 - 3	9	5.92 to 6.22	75	50	LIMESTONE - grey, occasional layers of oolitic limestone and dark grey shale, unweathered, bedding thin to very thin, horizontal to 45°
03 - 3	10	6.22 to 7.75	100	100	LIMESTONE - grey, occasional layers of oolitic limestone and dark grey shale, unweathered, bedding thin to very thin, horizontal to 45°

CR = Core Recovery
RQD = Rock Quality Designation

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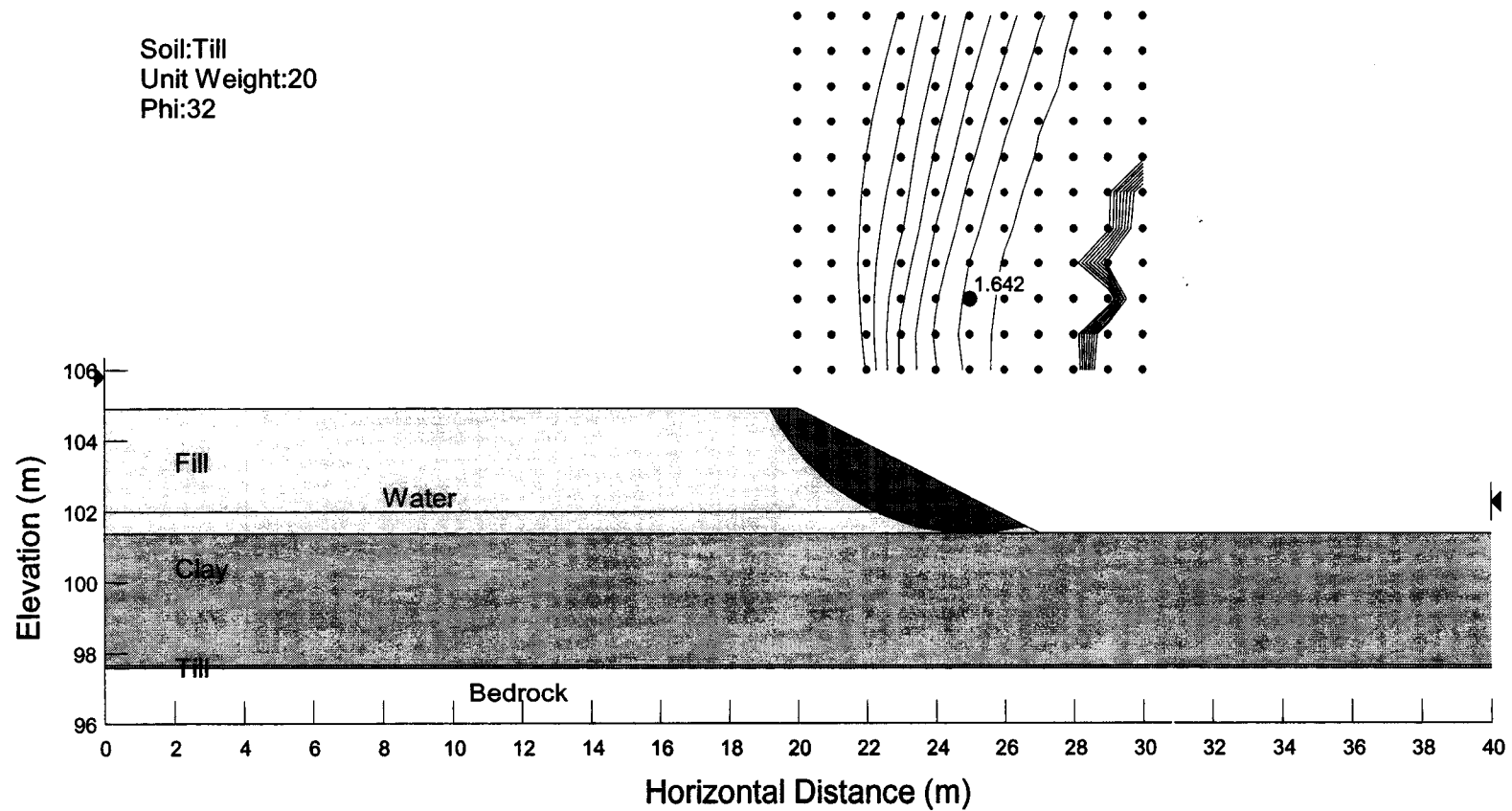


ONO 11632
Hwy 62 Palliser Creek Bridge Widening

Soil:Fill
Unit Weight:21
Phi:35

Soil:Clay
Unit Weight:18
Cohesion:10
Phi:27

Soil:Till
Unit Weight:20
Phi:32



ONO 11632
Hwy 62 Palliser Creek Bridge Widening

Soil:Fill
Unit Weight:21
Phi:35

Soil:Clay
Unit Weight:18
Cohesion:100

Soil:Till
Unit Weight:20
Phi:32

