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PRELIMINARY REPORT

Foundation Investigation and Design
G.W.P. 248-99-00
Highway 62
Beaver Creek Bridge Replacement
Site No. 11-034

REPORT NO. NO11686

GEOCRES NO. : 31C-172

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REPORT NO. NO11686

**PRELIMINARY REPORT –
FOUNDATION INVESTIGATION AND DESIGN**

TO

**Totten Sims Hubicki
300 Water Street
Whitby, Ontario
L1N 9J2**

ON

G.W.P. 248-99-00

**Highway 62 - Beaver Creek
Bridge Replacement
Site No. 11-034**

Township of Tudor

Bancroft District

**Ministry of Transportation
Ontario**

Geocres No. 31C-172

August 22, 2006

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PRELIMINARY FOUNDATION INVESTIGATION REPORT

for

Beaver Creek Bridge Replacement

Site No. 11-034

Township of Tudor, Ontario

G.W.P. 248-99-00

Highway 62

Bancroft District

1.0 INTRODUCTION

This report presents the results of a foundation investigation carried out for the preliminary design of a replacement structure for the Beaver Creek Bridge located along Highway 62 at Station 28+015, Township of Tudor, approximately 35 km south of Bancroft, Ontario.

The work was carried out under Agreement No. 4005-A-000310 and in general accordance with our proposal dated May 23, 2003. Authorization to proceed was provided by Ms. Brenda Jamieson, P.Eng., of Totten Sims Hubicki Associates (TSH), the prime TPM consultant to the Ministry of Transportation, Eastern Region for this project.

This report contains the factual information obtained from the field and laboratory investigation.

2.0 SITE DESCRIPTION AND GEOLOGY

The project site is located on Highway 62 approximately 35 km south of Bancroft. The centreline of Beaver Creek is located at approximate Station 28+015 Township of Tudor. The centerline of the proposed replacement bridge is located approximately 15 m east of the centerline of the existing structure. The site location is shown on the Key Plan portion of Drawing No. 11686-2 in Appendix A.

The project site is within an area identified by Chapman and Putnam as the Algonquin Highlands. The region is characterized by frequent outcrops of bare rock, generally shallow soil, frequent swamps and bogs, and rough relief. The thickness of soil over bedrock can vary greatly over short distances and the valleys are frequently floored with outwash sand and gravel.

Photographs of the site are provided in Appendix C.

Beaver Creek flows from east to west and is approximately 35 m in width at the centreline of the proposed realignment. Water depths were estimated to be less than 2 m at the time of the investigation. The surveyed water level at the time of the investigation was 302.4 m Geodetic.

The area immediately surrounding the structure is relatively flat and roadside ditches are filled with cattails and other vegetation. The water level in Beaver Creek was approximately 2.2 m below the top of pavement on the existing bridge at the time of the field investigation. The water in Beaver Creek was observed to be slow moving at the time of the investigation. The natural channel slopes are relatively flat and vegetated. No evidence of active erosion was observed.

Highway 62 has a two lane rural cross-section within the study limits.

Historical contract drawings from 1937 indicate that the existing bridge structure consists of a three-span structure with concrete deck and steel frame supported on timber piles bearing on "possibly hardpan". The geological cross-section indicates that the soil conditions at the site consisted of approximately 1.2 m of soft muck over approximately 4 m to 5.5 m of fine white silty sand over "possibly hardpan". The term "hardpan" which was generally applied to a hard often clayey layer of cemented soil particles or to a clayey layer of glacial till is no longer in general use. A copy of the general drawing from 1937 with the geological cross-section is provided in Appendix C.

The existing approach fills are approximately 3 m in height. The sides are sloped at approximately 2H:1V and vegetated. The forward face of the abutment is exposed with little earth cover.

It is apparent on site that some structural rehabilitation has occurred since construction of the original bridge (see photograph in Appendix C). No evidence of embankment settlement or instability at the existing creek crossing was noted in the field.

3.0 INVESTIGATION PROCEDURES

3.1 Field Program

The field work for this investigation was carried out between November 2005 and January 2006. The subsurface conditions were investigated through a borehole drilling program. A total of five (5) boreholes, numbered 05-1 through 05-4 and 06-3, were advanced at select locations.

Boreholes 05-2 and 05-3 were drilled within the proposed south and north abutments of the replacement bridge respectively. Boreholes 05-1 and 05-4 were drilled approximately 20 m behind the south and north abutments, respectively. Refusal was not reached at the north abutment (Borehole 05-3) during the drilling in December 2005 due to limitations with the drilling equipment on site at that time. Jacques Whitford returned to the site on January 17, 2006, and advanced Borehole 06-3 to refusal at a location 0.5 m east of Borehole 05-3.

The locations of the proposed new abutments are within the existing creek. The boreholes at the proposed abutment locations were drilled from a 3 m by 3 m raft using portable drilling equipment equipped with casing. A one-third weight hammer was used for carrying out standard penetration testing (SPT) within the upper 1.5 m to 3 m. Below this depth a standard full-weight hammer was used. The SPT N-values shown on the borehole records have been corrected to account for the one-third weight hammer, where appropriate, by dividing the number of blows by three. Soil samples were generally retrieved at 0.61 m intervals by a split spoon sampler. The wet and loose soil conditions observed at depth prevented standard penetration testing at depth. Furthermore the SPT results become less dependable with depth below the water table, as water flow into the casing may have disturbed the soils prior to insertion of the sampling spoon. Below a depth of 14.6 m in Borehole 05-2 and 12.2 m in Borehole 05-3, the boreholes were advanced using a dynamic cone penetration test due to the groundwater and soil conditions which were encountered. Borehole 06-3 was advanced from surface using a dynamic cone penetration test. In accordance with the Terms of Reference the abutment boreholes were terminated on refusal, defined as greater than 100 blows per 300 mm of penetration which was reached at a depth of 17.9 m in Borehole 05-2 and a depth of 20.1 m in Borehole 06-3.

Standpipes were installed in Boreholes 05-1 and 05-4. The standpipes consisted of slotted flexible poly-tube with a diameter of 25 mm. Groundwater levels were measured in the standpipes on December 1, 2005.

Boreholes were abandoned in accordance with the requirements of MOE Regulation 903. Prior to completing the investigation, the boreholes located on land were backfilled with a cement/bentonite mixture. The boreholes within the creek caved in as the casing was withdrawn.

The subsurface conditions are described in detail in the Borehole Records presented in Appendix B. All soil samples recovered were identified in the field, stored in moisture proof containers and were returned to our laboratory for detailed classification and testing.

Borehole locations were established in the field by Jacques Whitford personnel relative to stations and offsets from the centreline of the road. The ground surface elevations at the borehole locations were referenced to the top of pavement elevation along the centreline at the north end of the existing bridge structure. This feature has a geodetic elevation of 304.628 m.

3.2 Laboratory Testing

All samples returned to the laboratory were subjected to detailed visual classification by a geotechnical engineer. Selected samples were tested for moisture content, grain size distribution and organic content. All soil samples will be stored for a period of twelve months after issuance of the final report. Unless otherwise directed, the stored samples will be disposed of after this period.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix B. An explanation of the symbols and terms used to describe the Borehole Records is also provided. A borehole location plan is shown on Drawing 11686-2 along with a Stratigraphic Plot (Appendix A). A detailed description of the subsurface conditions encountered is given below.

4.1 Silty Sand (SM / SM-SP)

A layer of silty sand was observed at the surface in Boreholes 05-1, 05-2 and 05-3. This silty sand layer contained trace gravel, trace to some organic matter at some locations and seams of sandy silt and silty clay at Borehole 05-2. The thickness of the deposit ranged from 0.6 m at Borehole 05-1 to 1.9 m at Borehole 05-3. The elevation of the base of this unit ranged from 299.1 m to 302.0 m Geodetic.

Standard Penetration tests in the silty sand, trace gravel yielded SPT N-values ranging from 1 to 9 indicating very loose to loose conditions.

The natural moisture content of the five samples tested ranged from 21% to 75% with an average of 50%. The results of two grain-size distribution analyses carried out on representative samples of the silty sand, trace gravel, trace to some organic matter indicated that it contained 0% to 4% gravel, 54% to 90% sand, and 10% to 42% silt and clay sized particles, see Figures 1 and 2, Appendix B.

The organic content of this layer was determined to be 3.3% and 3.4% for samples from Borehole 05-1 and 05-3 respectively.

4.2 Sandy Silt (ML)

A deposit of sandy silt was encountered beneath the silty sand in Boreholes 05-1, 05-2 and 05-3 and at surface in Borehole 05-4. The sandy silt deposit contained seams of silty sand, silt and silty clay. Boreholes 05-1 and 05-4 were terminated within the sandy silt deposit. The base of the sandy silt was encountered at a depth of 14.3 m below the water surface (13.6 m below the river creek bottom) in Borehole 05-2. The base of the sandy silt deposit was inferred to be at a depth of 14.3 m below the water surface (12.9 m below the river creek bottom) in Borehole 05-3 based on observations of the resistance during the dynamic cone penetration test. Where penetrated, the thickness of this deposit, was approximately 11 m to 12 m. The elevation of the base of this unit was 288.1 m and 288.3 m in boreholes 05-2 and 05-3 respectively.

Standard Penetration tests in the sandy silt yielded SPT N-values ranging from 2 to 19, indicating very loose to compact conditions. The average SPT N-value was 8 indicating that the deposit was generally loose.

The natural moisture content of the nine samples tested ranged from 23% to 30% with an average of 25%. The results of nine grain-size distribution analyses carried out on representative samples of the sandy silt, indicated that it contained 0% gravel, 0% to 64% sand, 30% to 95% silt and 5% to 10% clay sized particles, see Figures 3 and 4, Appendix B.

4.3 Sandy Silt, Some Gravel, Occasional Cobbles: TILL (SM)

Sandy silt with some gravel was observed in the bottom half of split spoon sample SS-13 in Borehole 05-2. The origin of the deposit was inferred to be a glacial till. The presence of cobbles within the deposit was inferred based on the uneven rate of penetration of the dynamic cone. The sandy silt, some gravel, occasional cobble (till) deposit was not fully penetrated with sampling, however, dynamic cone refusal (>100 blows per 300 mm) was reached at depths of 3.6 m and 5.7 m into the deposit at Boreholes 05-2 and 06-3, respectively. The elevation at refusal was 284.5 m and 282.3 m in Boreholes 05-2 and 05-3 respectively. The refusal of the dynamic cone was not abrupt in either Borehole 05-2 or 06-3. In Borehole 05-2, refusal consisted of 100 blows for 225 mm of penetration. In Borehole 06-3, the cone still appeared to be penetrating after 100 blows so driving of the cone was continued for another 100 blows. The penetration was 275 mm for the final 200 blows. It is possible that refusal was reached on cobbles, boulders or bedrock.

4.4 Groundwater

The water level in Beaver Creek at the proposed bridge location was 302.4 m on December 1, 2005. Boreholes 05-2, 05-3 and 06-3 were drilled in the creek where water depths were 0.7 m, 1.4 m and 1.2 m respectively. The elevation of the groundwater in the standpipes installed in boreholes 05-1 and 05-4 was measured to be 302.5 m and 302.4 m respectively. The water levels in the boreholes drilled through the proposed approach fills (Boreholes 05-1 and 05-4) were approximately equal to the water level in the adjacent creek.

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

5.0 CLOSURE

A subsurface investigation is a limited sampling of a site. The subsurface conditions provided herein are based on information gathered at 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.

Yours very truly,

JACQUES WHITFORD LIMITED

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



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



PRELIMINARY FOUNDATION DESIGN REPORT

for

Beaver Creek Bridge Replacement
Site No. 11-034
Township of Tudor, Ontario
W.P. 248-99-00
Highway 62
Bancroft District

6.0 DISCUSSION

6.1 Proposed Development

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

It is understood that the Ministry of Transportation of Ontario (MTO) plans to replace the existing Beaver Creek Bridge (Site No. 11-034). Based on historical contract drawings, the existing structure was constructed around 1938 and consists of a 25 m long three span (8.4 m, 8.2 m and 8.4 m) slab-on-girder structure. It has a concrete deck and steel girders supported on timber piles. The 1937 General Drawing (copy provided in Appendix C) indicates that the piles are approximately 10 m in length and are driven into hardpan. It is apparent that there has been some structural alteration since construction of the original bridge. Photographs are presented in Appendix C.

The preferred design option for the bridge replacement involves reconstructing the Beaver Creek Structure on a new alignment approximately 15 m to the east. It is understood that this option would improve highway geometry and facilitate single stage construction. A separate detour structure will not be necessary. Temporary roadway protection will be required during construction of the new abutments due to the proximity to the existing roadway.

The proposed replacement structure consists of a 26 m long single span concrete box girder structure with concrete deck. The width of the deck will be 14.14 m and will allow space for two 3.5 m wide lanes and two 3.0 m wide shoulders. The proposed design includes 2H:1V foreslopes and sideslopes. The approach embankments will be up to 2.9 m above existing grades along the proposed alignment.

6.2 Soil Summary

The native soil conditions at this site consist of a deep deposit of non-cohesive materials generally consisting of silty sand with a trace of organics, over sandy silt, some gravel, occasional cobbles (till). Although the SPT N-values suggest very loose to compact conditions in the sandy silt, it is likely that the lower N-values observed are a reflection of the groundwater conditions. For preliminary design purposes, the sandy silt soils will be considered to be compact, with a minimum design N-value of 12. For preliminary design purposes, the native non-cohesive soils at this site have been considered to have a unit weight of 19.0 kN/m^3 and a minimum angle of internal friction of 29 degrees.

6.3 Foundation Options

Spread footing foundations are not feasible at this site due to low bearing resistance and construction difficulties associated with foundation unwatering.

The following table compares the available foundation options considered for the bridge structure:

Table 6.1: Foundation Comparison

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Driven H-piles in till	<ul style="list-style-type: none"> readily incorporated into integral abutment design moderate geotechnical resistance 	<ul style="list-style-type: none"> anticipated length of 20 m 	Moderate	
Driven H-piles on bedrock	<ul style="list-style-type: none"> readily incorporated into integral abutment design high geotechnical resistance 	<ul style="list-style-type: none"> anticipated length of 25 m 	Moderate	<ul style="list-style-type: none"> bedrock depth not yet confirmed by coring. Pile length may need to be increased.
Caissons	<ul style="list-style-type: none"> high geotechnical resistance on bedrock allows for semi-integral abutment design 	<ul style="list-style-type: none"> require tremie concrete require cased holes incompatible with integral abutment design 	High	<ul style="list-style-type: none"> base instability in saturated sands may require use of drilling mud / extra cost bedrock depth not yet confirmed by coring. Caisson length may need to be increased

For preliminary design purposes, it is recommended that the replacement bridge be founded on H-piles driven to till. Should the detailed design geotechnical investigation confirm that bedrock is within a practical driving depth, higher geotechnical resistances will be available for the final design.

Due to the pervious nature of the soils at this site, the preliminary design should include dewatering for the foundation construction by well points. Shoring will be required to cut off infiltration from the creek.

7.0 PRELIMINARY RECOMMENDATIONS

7.1 Structure Foundations

Axial Resistance

The bridge structure may be supported on steel H-piles driven to till. The estimated pile tip elevation is 280 m for both the north and south abutments. It is noted dynamic cone refusal was reached at elevations 284.5 m and 282.3 m at the south and north abutments respectively.

The following geotechnical parameters are recommended for the design of single HP 310 x 110 piles:

Table 7.1: Recommended Pile Design Parameters for HP 310 x 110 Piles

Founding Material	Estimated Pile Tip Elevation (m)	Factored Axial Geotechnical Resistance at ULS (kN)	Unfactored Geotechnical Resistance at SLS (kN)
Till	280.0	600	500
Bedrock	280.0	2,000	N/A

Note that the pile tip elevation in the above table has been estimated as approximately 2 m below the deepest elevation at which dynamic cone refusal was encountered in the boreholes. See Section 8.0 for recommendations on further investigation.

A geotechnical resistance factor of 0.4 has been applied to generate the factored axial Geotechnical Resistance at ULS for piles driven into till.

Should the detailed design geotechnical investigation confirm that bedrock is within a practical driving depth, higher geotechnical resistance for piles on bedrock will be available for final design. Previous experience in the Algonquin Highlands has consistently revealed high strength rock where the Geotechnical Resistance of the rock would exceed the Structural Resistance of the pile. The above Factored Axial Resistance at ULS for piles on bedrock corresponds to the factored Structural Resistance. Note however, rock coring and rock testing will be required as part of the final design investigation to confirm this design assumption.

The top of the pile is expected to settle less than 10 mm at the SLS value for piles in till.

The pile tip for piles set on bedrock is not anticipated to settle. The elastic compression of the pile under the ULS loading is expected to be minimal, therefore the factored resistance at serviceability limit states (SLS) does not apply.

Downdrag forces are not anticipated at this site.

Lateral Resistance

For preliminary design purposes, passive lateral resistance for vertical piles should be calculated as per the non-cohesive approach of Section C6.8.7.1 (a) Static Analysis and C6.8.7.2 Static Analysis of the CHBDC using the following unfactored geotechnical soil parameters:

Table 7.2: Recommended Lateral Pile Design Parameters (Non-Cohesive Approach)

Parameter	OPSS Granular B Type I	Silty Sand	Sandy Silt
Bulk Unit Weight, kN/m ³	21.2	19.0	19.0
Effective Friction Angle, degrees	35	30	29
Coefficient of Passive Earth Pressure	3.7	3.0	2.9

Lateral Deflections

The coefficient of horizontal subgrade reaction, which may be used for deflection calculations, may be estimated for cohesionless soils using Terzaghi's method (1955) as follows:

$$k_s = n_h z/d$$

where

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

n_h = coefficient related to soil compactness

z = depth

d = pile diameter

All pile bending movements and horizontal pile deflections will occur within the upper soils.

The soil compactness, based on the SPT N-values, is generally loose to compact within the upper soils. Therefore, an n_h value of 3,000 kN/m³ is recommended for design calculations for those soils. Below elevation 288 m the soil is compact to dense and a n_h value of 10,000 kN/m³ is recommended.

Group Effects on Lateral Deflections

If piles are spaced at less than 8 pile diameters, center to center, parallel to the direction of lateral load, or less than 4 pile diameters, center to center, perpendicular to the lateral load, group effects will need to be considered and the lateral load at a specific deflection may need to be decreased.

The nature of pile-soil-pile interaction is complex, however is generally broken down into the following main components:

- alteration of the soil state due to pile installation and the potential overlap of the alterations when nearby piles are driven; and,
- superposition of strains and alterations of the soil failure zones when nearby piles are simultaneously loaded.

Studies (Reese, Isenhower and Wang, 2006) have reported the following reduction between single piles and pile groups.

- Condition No. 1: Load is parallel to pile spacing

Pile Spacing c/c	Trailing Pile Group Pile Efficiency, e_T	Lead Pile Group Pile Efficiency, e_L
7d	1.0	1.0
4d	0.8	1.0
3d	0.7	0.9
2d	0.6	0.8

- Condition No. 2: Load is perpendicular to pile spacing

Pile Spacing c/c	Group Pile Efficiency, e_P
4d	1.0
3d	0.9
2d	0.75

Where piles are on a skew to each other relative to the direction of load the Group Pile Efficiency may be calculated based on

$$e_s = (e_B^2 \cos^2 \alpha + e_p^2 \sin^2 \alpha)^{1/2}$$

where

e_B = either e_T or e_L from above

α = angle between direction of loading and the skew

Note that when piles are more than 3.3 pile diameters apart perpendicular to the direction of the load, the skew correction is not necessary. The lateral load at a specific deflection for each individual pile must consider the interaction of all piles within the group.

The reduction factor applied to a pile is the product of the efficiencies of all of the interactions of piles within that pile group.

Tensile Resistance

Resistance to tensile loads should be calculated based on the shaft resistance of the piles in accordance with Section 6.8.5 of the CHBDC. For this site, the soils primarily consist of a silt and sand mix and therefore the following parameters may be used for preliminary design purposes:

Submerged Unit Weight	9.2 kN/m ³
Effective Friction Angle	29 degrees
β Coefficient	0.3
Resistance Factor	0.3
Design Critical Depth	6.2 m

The following values have been calculated based on the above recommended parameters:

Table 7.3: Recommended Tensile Pile Design Parameters

Pile Type	Pile Tip Elevation (m)	Factored Geotechnical Resistance (Tension) at ULS (kN)
HP 310 x 110	280.0	400

The factored geotechnical resistance (tension) at ULS provided above includes a resistance factor of 0.3 but does not include the weight of the pile.

Pile Notes

Pile tips should be reinforced as per OPSS-3000.100, Type I.

7.2 Earth Pressure Design

The abutments and retaining walls (if required) 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 unfactored soil parameters in Table 7.4 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.

The total active and passive thrusts can be calculated using the following equations:

$$P_A = \frac{1}{2} K_a \gamma H^2$$

$$P_P = \frac{1}{2} K_p \gamma H^2$$

Where H is the height of the wall. Values for K_a , K_p and γ are provided below. The thrust acts at a point one third up the height of the wall.

Table 7.4: Recommended Lateral Earth Pressure Parameters

Parameter	OPSS Granular B, Type I and III	OPSS Granular A and Granular B Type II
Total Unit Weight, γ (kN/m ³)	21.2	22.0
Effective Friction Angle	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 3102.100 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 3101.150 using a depth of frost penetration, f , of 1.8 m.

7.3 Seismic Design Considerations

7.3.1 Zonal Acceleration Ratio

Table A3.1.7 of the CHBDC indicates that the Zonal Acceleration Ratio for Bancroft, which is 35 km north of the site, is 0.10. Reference is made to Section C4.6.4 of the CHBDC for the calculation of seismic forces on abutments and retaining walls.

7.3.2 Soil Profile Type

It is recommended that Soil Profile I as defined in CHBDC Section 4.4.6 be used in the seismic design of this site.

7.3.3 Liquefaction of Foundation Soils

An assessment of the potential for liquefaction of the foundation soils was carried out using the procedures outlined in CHBDC, Section C4.6.2. Liquefaction of Foundation Soils. Specifically, the Seed and Idriss (1971) simplified procedure was used to assess the upper silty sand layer and the Chinese criteria was used to assess the lower sandy silt deposit.

The results of this assessment revealed that the upper silty sand would be classified as liquefiable under a 0.1 g earthquake. In addition, the sandy silt at some locations meet the criteria to be classified as potentially liquefiable.

Although the Standard Penetration Test N-value may be useful as a preliminary indicator on the potential for soil liquefaction, it should be used for this purpose with great caution. Some studies have indicated that soils with an average N-value of 12 could have standard deviations in the measured values of 6, where other test methods show no appreciable changes in density. As well, the drilling techniques used in Ontario easily disturb native soils, frequently producing falsely low N-values in clean native sands below the water table.

As a preliminary assessment, the soils at this site are not considered liquefiable under a 0.1 g earthquake, however, as part of the final foundation investigation it is recommended that a static Cone Penetration (CPT) investigation be carried out to further assess the liquefaction potential.

If the potential for liquefaction is confirmed during the detailed design stage, soil improvement techniques such as densification or the installation of wick drains may be required. Alternatively, the structure will need to be designed to withstand potentially large ground movements.

7.3.4 Seismic Forces on Abutments and Retaining Walls

Abutments and retaining walls should be designed to resist the earth pressures produced under earthquake conditions. CHBDC Clause 4.6.4 recommends the use of the combined coefficients of static and seismic earth pressure, referred to as K_{AE} for active conditions and K_{PE} for passive conditions, for routine design purposes.

The total active and passive thrusts under earthquake conditions can be calculated using the following equations:

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v)$$

$$P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v)$$

where;

K_{AE} = active earth pressure coefficient (combined static and seismic)

K_{PE} = passive earth pressure coefficient (combined static and seismic)

H = height of wall

k_h = horizontal acceleration coefficient

k_v = vertical acceleration coefficient

γ = total unit weight

For this site, the following preliminary design parameters were used to develop the recommended K_{AE} and K_{PE} values.

- Zonal Acceleration Ratio, A 0.1
- Horizontal Acceleration Coefficient, k_h 0.05
- Vertical Acceleration Coefficient, k_v 0.033
- Vertical back of wall
- For yielding abutments or walls

The above k_h value corresponds to $\frac{1}{2}$ of the A value, and the k_v value corresponds to 0.67 of the k_h value. The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

Table 7.5: Combined Coefficients of Static and Seismic Earth Pressure

Parameter	OPSS Granular B Type I & Granular B Type III	OPSS Granular A & Granular B Type II
	Horizontal Backslope	Horizontal Backslope
Total Unit Weight, γ (kN/m ³)	21.2	22.0
Effective Friction Angle	32 degrees	35 degrees
Active Earth Pressure (K_{AE})	0.34	0.30
Height of application of P_{AE} from base as ratio of wall height (H)	0.349	0.350
Passive Earth Pressure (K_{PE})	3.16	3.59
Height of application of P_{PE} from base as ratio of wall height (H)	0.316	0.316

It is noted that the combined coefficients of static and seismic earth pressure presented in Table 7.5 deviate only slightly from the static coefficients presented in Table 7.4. This is due to the low zonal acceleration ratio at this site.

7.4 Embankment Design

The existing embankments are constructed at approximately 2H:1V and exhibit no signs of instability. The 1937 General Arrangement indicates that the forward slopes were originally constructed of rock fill at 1H:1V.

7.4.1 New Alignment

Embankment side slopes on the new alignment should be constructed no steeper than 2H:1V. Embankment fill should consist of OPSS Select Subgrade Material or clean granular fill such as OPSS Granular B. The use of rock fill could also be considered. Rockfill should be sloped to be no steeper than 1.25H:1V generally and 1.5H:1V where the fill extends below water level.

Settlement of the underlying soil has been analyzed using elastic theory. Stress distribution was assessed based on a Boussinesq distribution. As much as 3.2 m of fill will be required at some locations to achieve design grades at the approaches on the new alignment (approximate El. 305.6 m). This will induce as much as 35 mm of settlement in the underlying native materials.

Self settlement of the embankment fill of as much as 10 mm for 3.2 m of fill will also occur.

Due to the non-cohesive nature of the native materials, it is anticipated that settlement will occur rapidly. Post construction settlements of the underlying soils will be less than 5 mm. Self settlement of the fill will be virtually complete at the completion of construction.

7.4.2 Existing Alignment

The construction of the roadway embankment along the proposed new alignment 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. Historical documents indicate that the timber piles supporting the existing structure extend through the silty sand deposits to the "possible hardpan". No settlement of the existing timber piles is expected since they are shown to bear on the underlying hard layer.

7.5 Dewatering

It is anticipated that the excavation for the abutments will extend to elevations ranging from 299.5 m to 300.5 m. The water level in Beaver Creek at the time of the investigation was 302.4 m. Shoring or a coffer dam and piping system will be required for excavations that extend below the water level in the creek. Given the high permeability of the soils at this site, it is recommended that for preliminary design purposes, shoring be placed on all four sides of each abutment even if the span is lengthened to move the abutments back out of the creek.

Design of shoring will need to account for basal heave due to flow of water around (i.e. beneath) the sheet piling. It is recommended that the contract include a Non Standard Special Provision for dewatering with well points.

7.6 Erosion Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment forward slopes. The water in Beaver Creek was observed to be slow moving in November 2005. 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.

At other locations, 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.

7.7 Frost Protection

Pile caps, retaining walls and spread footings should be provided with the equivalent of 1.8 m of earth cover for frost protection.

7.8 Other Construction Considerations

Site Grading and Preparation

Although not observed in the boreholes drilled for the present preliminary investigation, the 1937 General Arrangement indicates the presence of a layer of soft muck.

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 and compacted in accordance with SP105S10.

It is anticipated that the soil conditions exposed at the design underside of abutment elevation will consist of loose, saturated silty sand to sandy silt. This material will be readily disturbed and not conducive to supporting construction equipment. It is recommended that a construction pad at least 500 mm in thickness be constructed. The pad should be built of compacted OPSS Granular A or Granular B Type II placed on a Class II non-woven filter fabric.

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 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.

Encroachment of excavations into the forward 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 (Drawing No. NO11686-3 in Appendix D).

Shoring design should meet the requirements of Performance Level 2 as per OPSS 539 and should consider sloping backfill and traffic loading.

8.0 FUTURE INVESTIGATIONS

Additional investigation is required at this site during the detailed design. The investigation should be carried out to satisfy MTO Foundation protocols and should include the following:

- Bedrock coring to confirm the depth, nature, and strength of the bedrock.
- At least two static cone penetration (CPT) tests using a seismic cone should be carried out to confirm that liquefaction is not a significant issue at this site.
- Representative soil samples should be acquired and submitted for determination of pH, resistivity, chloride and water soluble sulphate content. This information should be assessed and recommendations provided concerning cement type and corrosion potential.
- Insitu hydraulic conductivity testing should be carried out at the abutments to allow further evaluation of unwatering requirements

9.0 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 LIMITED

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



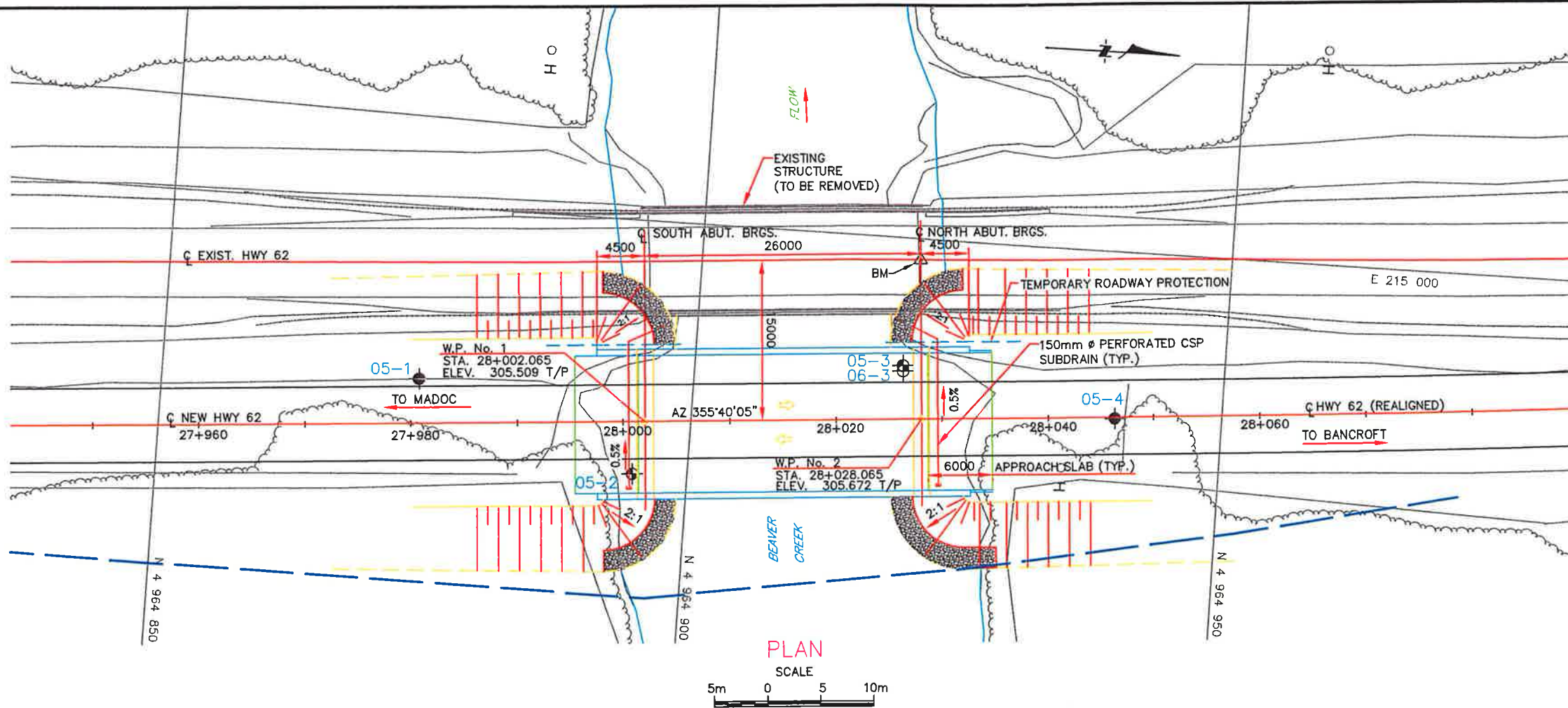
Fred Griffiths, Ph.D., P.Eng.
Designated Principal MTO Foundation Contact



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APPENDIX A

Borehole Location Plans and Profile Plots



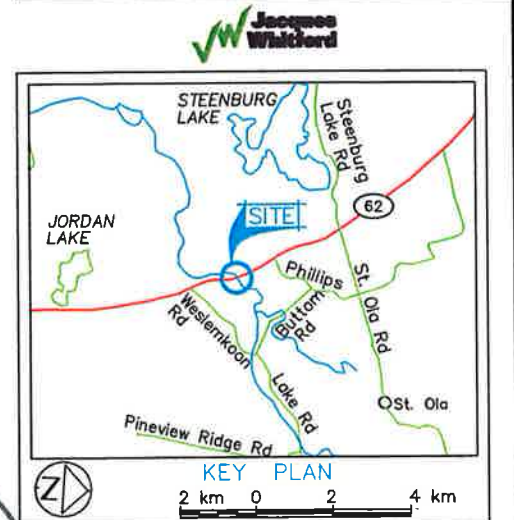
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 248-99-00

HIGHWAY 62
STATION 27+980 TO 28+050
BORE HOLE LOCATIONS & SOIL STRATA

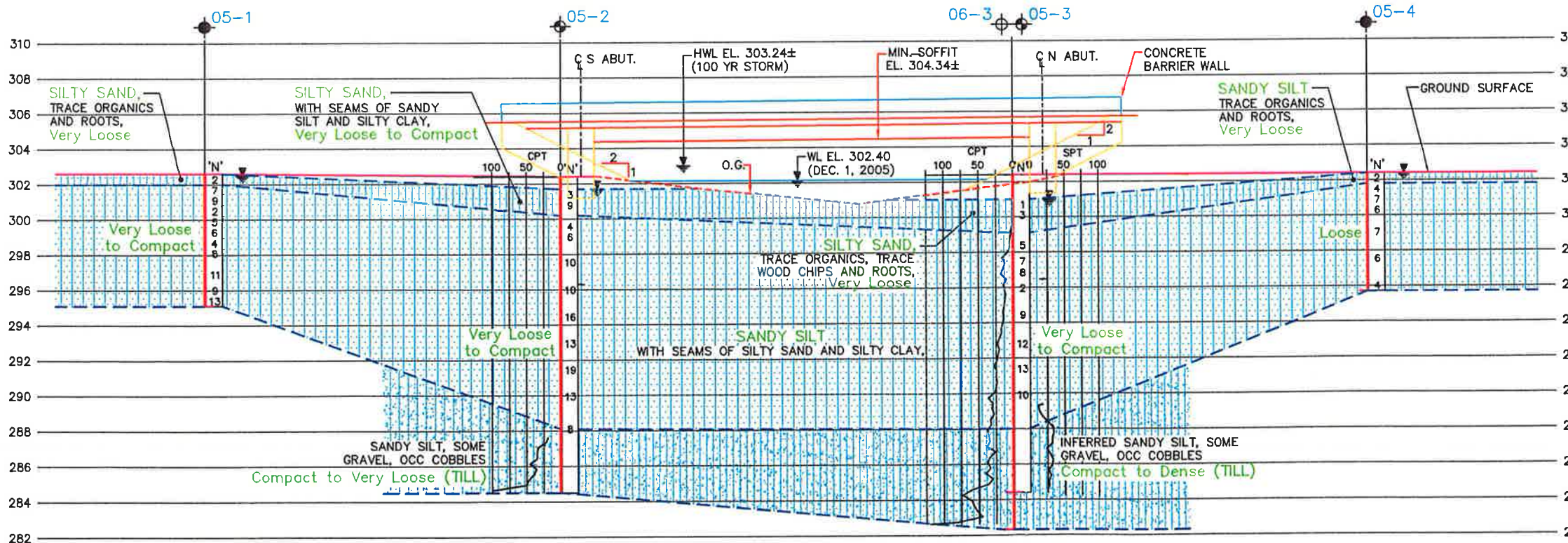


SHEET
1



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 Dec 2005
- ↓ WL in Piezometer
- ⚬ Piezometer
- △ Benchmark (CL of N Abutment)
Elev = 304.6 m Geodetic
Reference: TSH profile plate



PROFILE ALONG CL NEW HIGHWAY 62

No	ELEVATION	COORDINATES	
		NORTH	EAST
05-1	302.6	4 964 874.3	215 016.4
05-2	302.4	4 964 894.9	215 023.9
05-3	302.4	4 964 919.6	215 012.1
05-4	302.4	4 964 939.9	215 015.7
06-3	302.4	4 964 919.6	215 012.6

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.

REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No 31C-172			
HWY No 62	DIST 43		
SUBMIT FJG	CHECKED	DATE 2006-08-21	SITE 11-034
DRAWN GBB	CHECKED	APPROVED	DWG NO 11686-2

APPENDIX B

Symbols and Terms Used on Borehole Records

Borehole Records

Grain Size Distribution Test Results

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 vegetative growth
<i>Peat</i>	- mixture 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>	- material 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 in thickness
<i>Seam</i>	- 2 mm to 75 mm in thickness
<i>Parting</i>	- < 2 mm in thickness

Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488). The classification excludes particles larger than 76 mm (3 inches). The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

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%
<i>Frequent</i>	> 20%

Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test N-Value (also known as N-Index). A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

Terminology describing consistency of cohesive soils:

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

Consistency	Undrained Shear Strength	
	kips/sq.ft.	kPa
<i>Very Soft</i>	<0.25	<12.5
<i>Soft</i>	0.25 - 0.5	12.5 - 25
<i>Firm</i>	0.5 - 1.0	25 - 50
<i>Stiff</i>	1.0 - 2.0	50 - 100
<i>Very Stiff</i>	2.0 - 4.0	100 - 200
<i>Hard</i>	>4.0	>200

ROCK DESCRIPTION

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	<i>Very Poor</i>
25-50	<i>Poor</i>
50-75	<i>Fair</i>
75-90	<i>Good</i>
90-100	<i>Excellent</i>

Rock quality classification is based on a modified core recovery percentage (RQD) 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. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

Terminology describing rock mass:

Spacing (mm)	Joint Classification	Bedding, Laminations, Bands
> 6000	<i>Extremely Wide</i>	-
2000-6000	<i>Very Wide</i>	<i>Very Thick</i>
600-2000	<i>Wide</i>	<i>Thick</i>
200-600	<i>Moderate</i>	<i>Medium</i>
60-200	<i>Close</i>	<i>Thin</i>
20-60	<i>Very Close</i>	<i>Very Thin</i>
<20	<i>Extremely Close</i>	<i>Laminated</i>
<6	-	<i>Thinly Laminated</i>

Terminology describing rock strength:

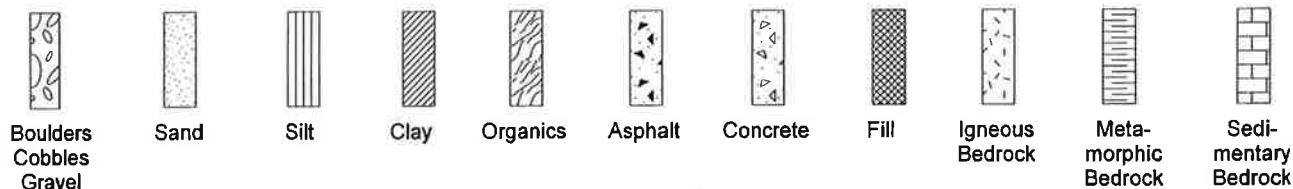
Strength Classification	Unconfined Compressive Strength (MPa)
<i>Extremely Weak</i>	< 1
<i>Very Weak</i>	1 – 5
<i>Weak</i>	5 – 25
<i>Medium Strong</i>	25 – 50
<i>Strong</i>	50 – 100
<i>Very Strong</i>	100 – 250
<i>Extremely Strong</i>	> 250

Terminology describing rock weathering:

Term	Description
<i>Fresh</i>	No visible signs of rock weathering. Slight discolouration along major discontinuities
<i>Slightly Weathered</i>	Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.
<i>Moderately Weathered</i>	Less than half the rock is decomposed and/or disintegrated into soil.
<i>Highly Weathered</i>	More than half the rock is decomposed and/or disintegrated into soil.
<i>Completely Weathered</i>	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



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 coring bits.

WATER LEVEL MEASUREMENT



measured in standpipe,
piezometer, or well



inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE / RQD





Numbers in this column are the field 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). Some design methods make use of N value corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log. RQD is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to A size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (305 mm) into the soil. The DCPT is used as a probe to assess soil variability. Soil type may be inferred from adjacent boreholes and test pits.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	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

RECORD OF BOREHOLE No BH05-1

1 OF 1

METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4964874.3, E215016.4 ORIGINATED BY JD
DIST 43 HWY 62 BOREHOLE TYPE Portable, Cased, Spillspoons COMPILED BY JD
DATUM Geodetic DATE 28.11.05 - 28.11.05 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE															
302.6	Tall Grass						20	40	60	80	100		10	20	30								
0.0	SILTY SAND, trace organics, trace roots, very loose, dark brown		1	SS	2											66.5		4	54	40	2		
302.0	(SM)															○							
0.6	SANDY SILT with seams of silty sand and silty clay, very loose to compact, brownish grey to grey		2	SS	7																		
			3	SS	9																		
			4	SS	2																		
	(ML)		5	SS	5										○			0	25	65	10		
			6	SS	6																		
			7	SS	4																		
			8	SS	8																		
			9	SS	11										○			0	0	95	5		
	(ML)		10	SS	9																		
			11	SS	13																		
295.2	End of Borehole																						
7.4	Standpipe Installed (25 mm diameter slotted flexible polytube) Water level measured December 1, 2005																						

MITO 11686MT0.GPJ ON MOT.GDT 27/06/06

1 OF 2

METRIC

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

MTO 11685MTO.GPJ ON MOT.GPT 27/06/06

2 OF 2

METRIC

W.P.	248-99-00	LOCATION	Beaver Creek Bridge, Site No. 11-034, N4964894.9, E215023.9	ORIGINATED BY	JD
DIST	43	HWY	62	BOREHOLE TYPE	Portable, Cased, Spillspoons
DATUM	Geodetic	DATE	29.11.05 - 29.11.05	CHECKED BY	PC

[illegible]

MTD 11686MTD.GPJ ON MOT.GDT 27/06/06

+ 3, × 3 Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH05-3

1 OF 2

METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4964919.6, E215012.1 ORIGINATED BY JD
DIST 43 HWY 62 BOREHOLE TYPE Portable, Cased, Splitterspoons COMPILED BY JD
DATUM Geodetic DATE 30.11.05 - 30.11.05 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	× FIELD VANE							
								● QUICK TRIAXIAL	× LAB VANE							
302.4	Water							20 40 60 80 100	10 20 30							
0.0	Beaver Creek															
301.0																
1.4	SILTY SAND, some organics, trace wood chips, trace roots, very loose, brown		1	SS	1								52.6			
	(SP - SM)		2	SS	3								74.8			
			3	GS												
			4	ST												
299.1			5	SS	5									0 64 30 6		
3.3	SANDY SILT with seams of silty sand and silty clay, very loose to compact, grey		6	GS												
	(SM)		7	SS	7											
			8	SS	8											
			9	SS	2											
			10	GS												
			11	SS	9									0 22 71 7		
	(ML)															
			12	SS	12											

Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH05-3

2 OF 2

METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4964919.6, E215012.1 ORIGINATED BY
DIST 43 HWY 62 BOREHOLE TYPE Portable, Cased, Splitspoons COMPILED BY
DATUM Geodetic DATE 30.11.05 - 30.11.05 CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
	SANDY SILT with seams of silty sand and silty clay, very loose to compact, grey		13	SS	13								
	(ML)		14	SS	10								
288.1						291							
14.3	Inferred sandy silt, some gravel, occasional cobbles, compact to dense, grey (TILL)					290							0 3 92 5
						289							
						288							
						287							
						286							
						285							
284.4													
18.0	End of Borehole												

RECORD OF BOREHOLE No BH05-4

1 OF 1

METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4964939.9, E215015.7 ORIGINATED BY JD
DIST 43 HWY 62 BOREHOLE TYPE Portable, Cased, Splitspoons COMPILED BY JD
DATUM Geodetic DATE 30.11.05 - 01.12.05 CHECKED BY PL

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		
302.4	Tall Grass													
302.2	SANDY SILT, trace organics, trace roots, very loose, dark brown		1	SS	2		302							
0.2	SANDY SILT, very loose, grey to brown													
301.8	SANDY SILT with seams of silty sand and silty clay, loose, grey		2	SS	4									
0.6														
	(ML)		3	SS	7		301							0 0 92 8
			4	SS	6									
							300							
			5	SS	7		299							
							298							
			6	SS	6									0 53 41 6
	(SM)													
							297							
			7	SS	4		296							
295.7	End of Borehole													
6.7	Standpipe Installed (25 mm diameter slotted flexible polytube) Water level measured December 1, 2005													

MTO 11695MTO.GPJ ON MOT.GDT 27/06/06

1 OF 2

METRIC

SOIL PROFILE	SAMPLES		DYNAMIC CONE PENETRATION			
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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH06-3

2 OF 2

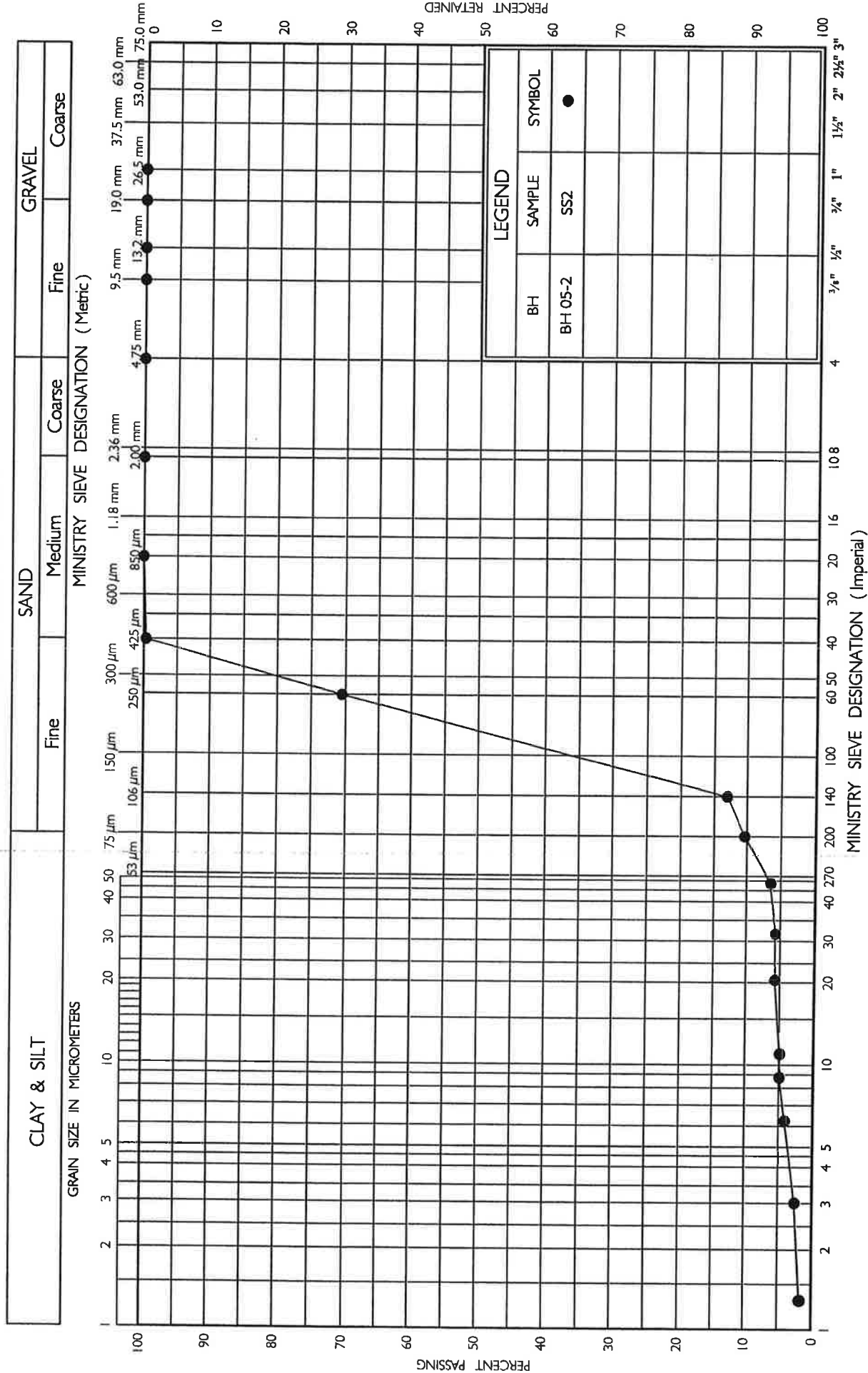
METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4964919.6, E215012.6 ORIGINATED BY JD
 DIST 43 HWY 62 BOREHOLE TYPE Portable, Dynamic Cone Penetration Test COMPILED BY JD
 DATUM Geodetic DATE 17.01.06 - 17.01.06 CHECKED BY PL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	× FIELD VANE	● QUICK TRIAXIAL						
	Inferred SANDY SILT with seams of silty sand and silty clay, very loose to compact, grey						20 40 60 80 100									
							291									
							290									
							289									
288.0							288									
14.4	Inferred sandy silt, some gravel, occasional cobbles, compact to dense, grey (TILL)						287									
							286									
							285									
							284									
							283									
282.3																
20.1	End of Borehole															
	Refusal															

MTO 11686MTO.GPJ ON MOT.GDT 27/06/06

UNIFIED SOIL CLASSIFICATION SYSTEM



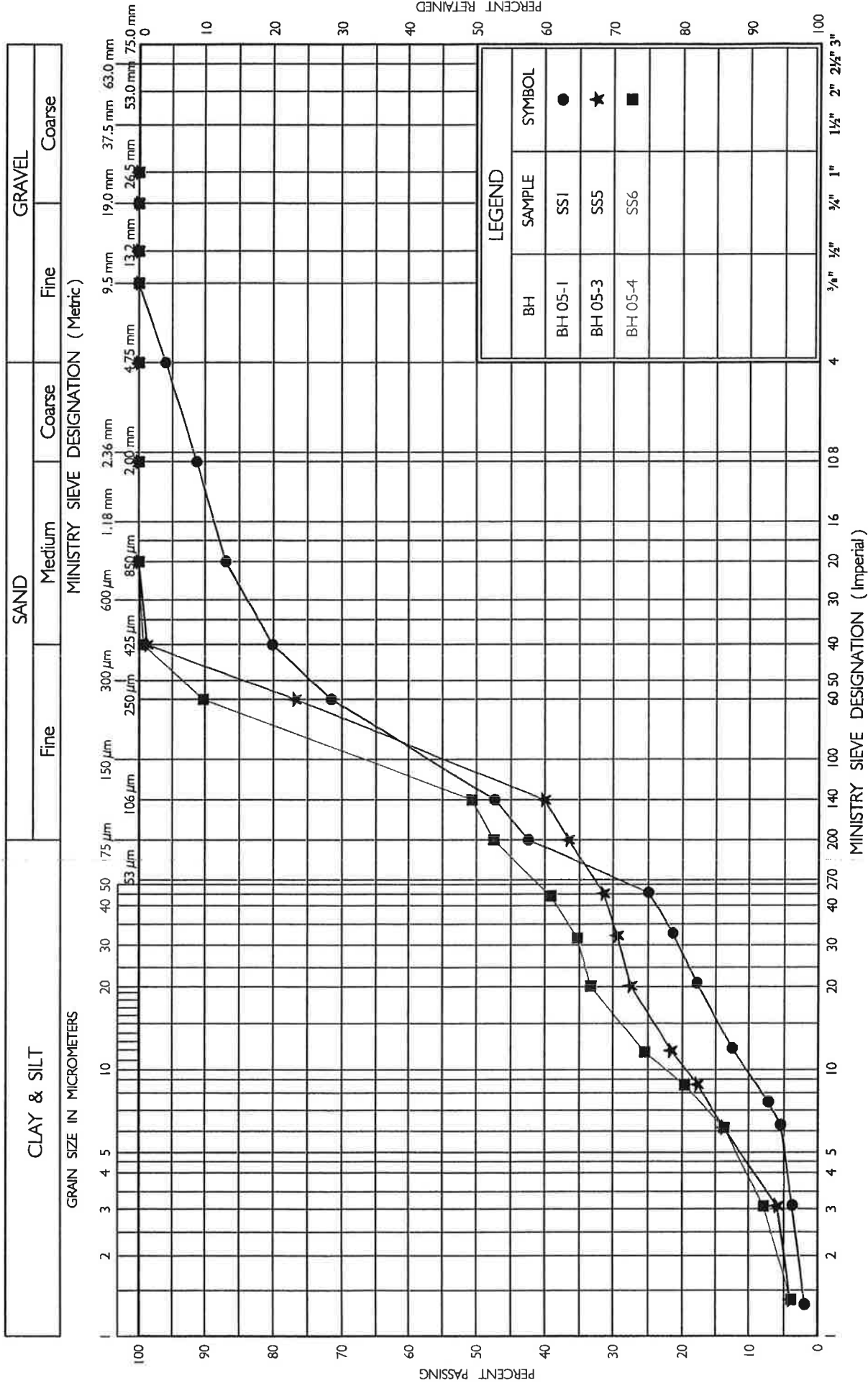
GRAIN SIZE DISTRIBUTION

POORLY GRADED SAND WITH SILT, SP-SM

FIG No 1

W P 248-99-00

UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM

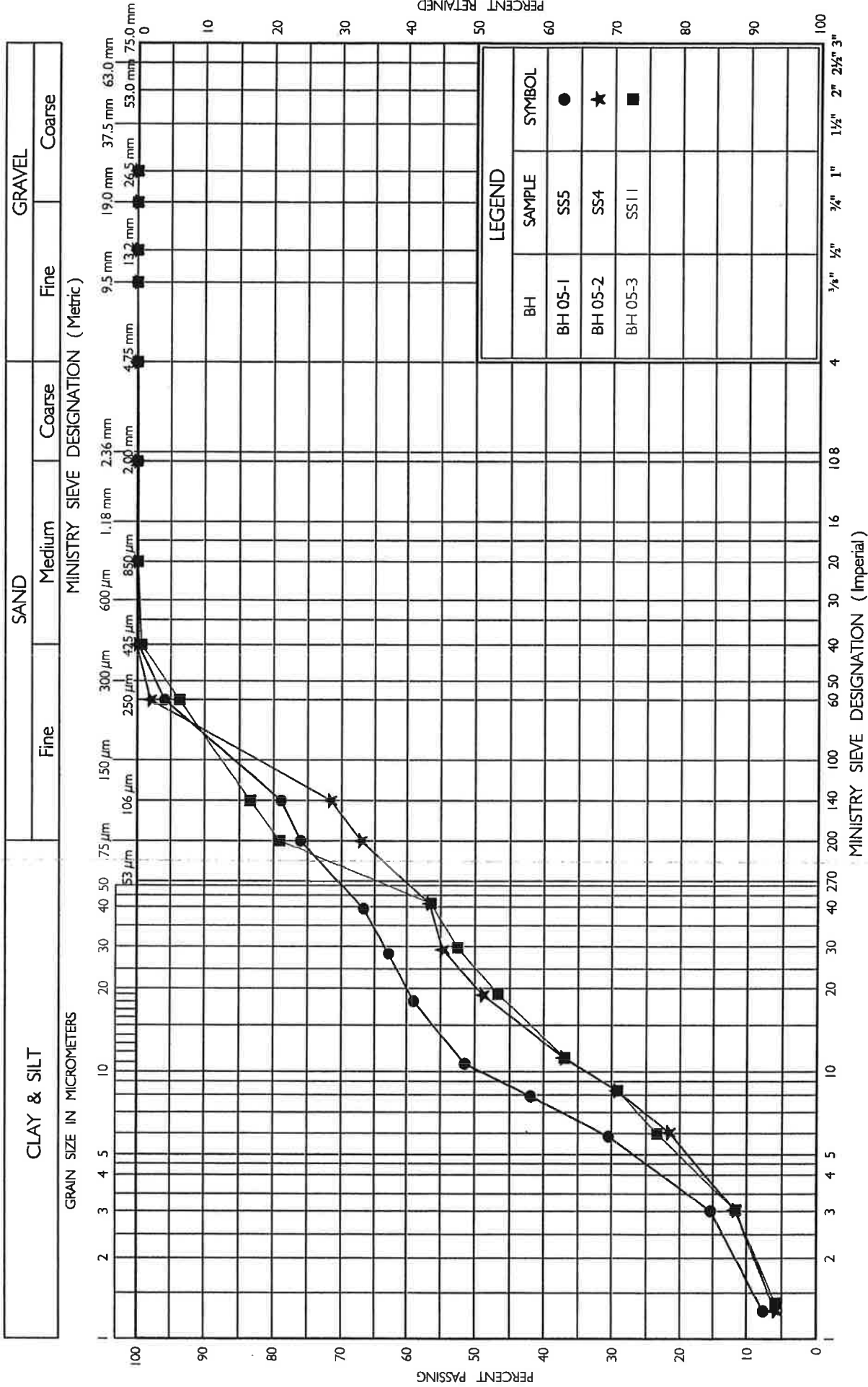


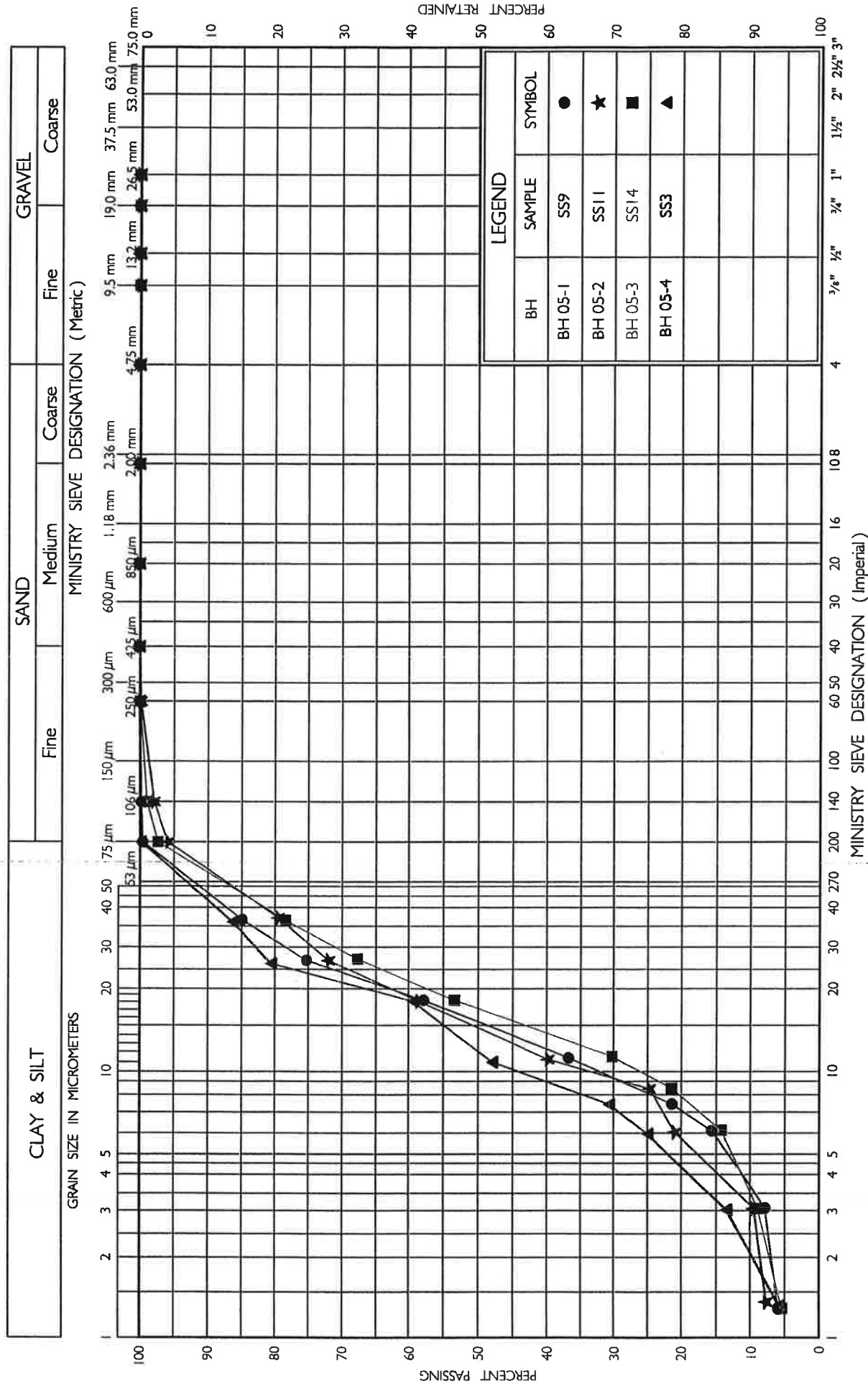
FIG No 3

W P 248-99-00

GRAIN SIZE DISTRIBUTION

SANDY SILT, ML

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

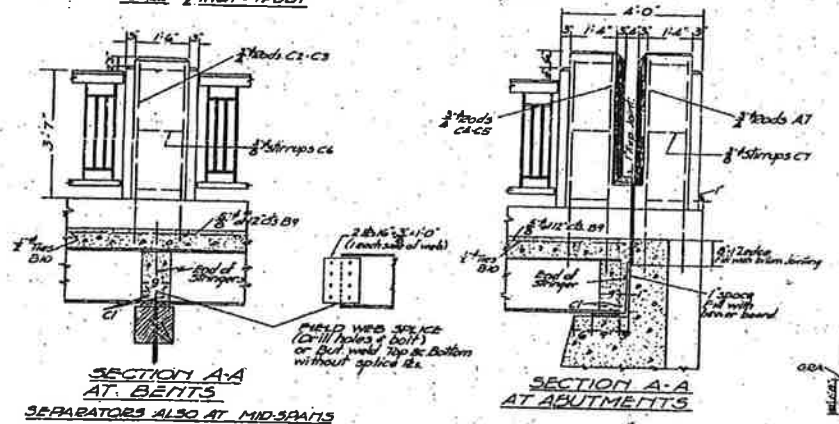
SANDY SILT, ML

FIG No 4

W P 248-99-00

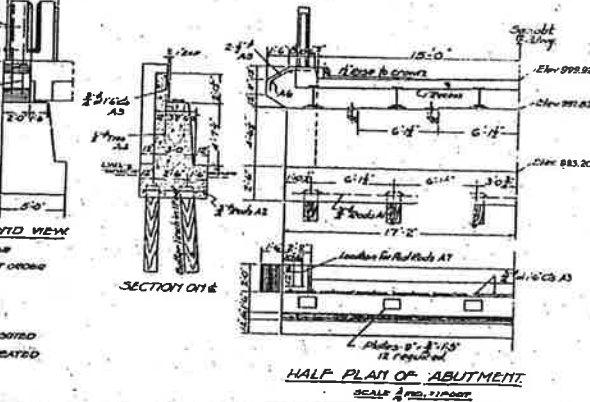
APPENDIX C

Historical Documents
Site Photographs



STRUCTURAL STEEL				PLAN & SECTION
ITEM	NUMBER	SIZE	LENGTH	REMARKS
FLOOR BEAMS	6	8W*10	27'-0"	Notes for Steel Joist Connections on the sides
	12	"	27'-10"	
SCAFFOLD ELS	4	9*12	1'-3"	
	4	9*12	1'-3"	
	4	9*12	1'-3"	
	12	9*12	1'-3"	
HARDWARE				
DRIFT PINS	22	1/4"	2'-0"	Welding nuts Headings
BOLTS	32	1/2"	1'-1"	1/4" light
	10	1/2"	1'-0"	Welding 1/2"
				connections with steel bolts
R.R. SPINDS	50	1/2"	6"	

CREOSOTED TIMBER			
ITEM	NUMBER	SIZE	LENGTH
PILE CAPS	2	12"x12"	34'-0"
Alternative -	4	" "	18'-0"
SWAY BRACED	5	3"x10"	14'-0"
	4	3"x10"	16'-0"
PILES	22		30'-0"
	22		30'-0"



Hand-drawn technical drawings for a road drainage system.

The top drawing is titled "HALF ELEVATION BENTS". It shows a cross-section of a road with a 20% slope, a 6:1 side slope, and a 6:1 backfill slope. The drawing includes labels for "Road", "Slope", "Backfill", "Level", and "Bent".

The bottom drawing is titled "DETAIL OF ROADWAY DRAIN (IF REQUIRED)". It shows a cross-section of a road with a 20% slope, a 6:1 side slope, and a 6:1 backfill slope. The drawing includes labels for "Road", "Slope", "Backfill", "Level", and "Bent".

PROPOSED BRIDGE
OVER
BEAVER CREEK
THE KING'S HIGHWAY #18 BELLEVILLE TO BAYVIEW
COUNTY - HASTINGS STRUCTURE #78
TOWNSHIP - TUDOR LOT 16 - CON XVIII
APPROVED

BRIDGE ENGINEER CHIEF ENGINEER

DEPUTY MINISTER

DEPARTMENT OF HIGHWAYS, ONT.
(CONSULTED 3/27/57 15 B 133)

DESIGN	LL	DETAIL	CHANCE	ILL	STRAIGHT	ENROLL
CHANCE	ILL	STRAIGHT	ENROLL	ENROLL	ENROLL	ENROLL

CONTRACT #736 5/1
DRAWING # D2454-1



Photo 1: Highway 62 - Beaver Creek Bridge looking north-west.



Photo 2: Highway 62 – Beaver Creek Bridge looking south.



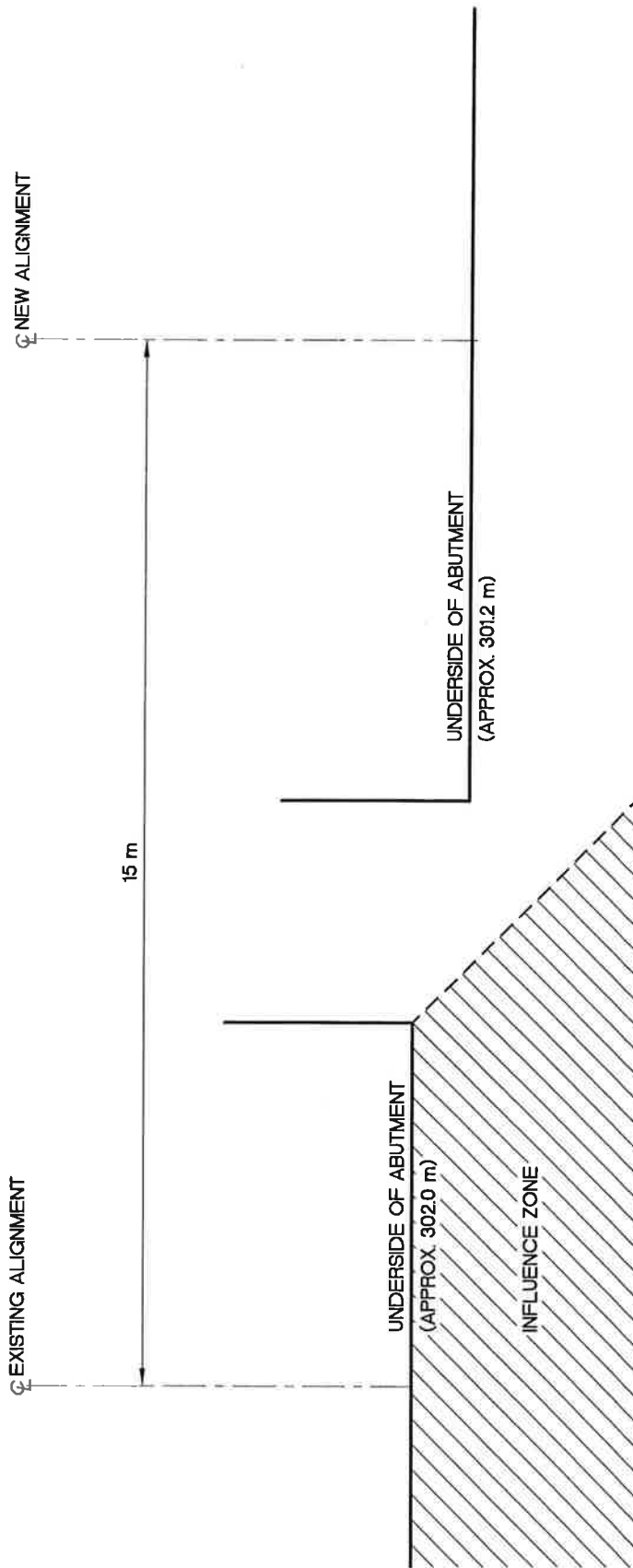
Photo 3: Highway 62 – Beaver Creek Bridge looking northeast



Photo 4: Highway 62 – Beaver Creek Bridge North Abutment looking west.

APPENDIX D


Cross-Section Showing Influence Zone



NOTE:

DIMENSIONS AND ELEVATIONS ARE
FOR ILLUSTRATION PURPOSES ONLY.

TOTTEN SIMS HUBICKI
GWP 248-99-00
HIGHWAY 62, BEAVER CREEK BRIDGE
CROSS-SECTION SHOWING INFLUENCE ZONE
TOWNSHIP OF TUDOR, ONTARIO

Scale: N.T.S.	Job No.: NO11686	Dwg. No.: NO11686-3	
	Date: 06/08/21	Dwn. by: GBB	

Appd.:
7/2/21