



**Submitted To AECOM Canada Ltd.
189 Wyld Street Suite 103, North Bay, Ontario P1B 1Z2
On Behalf of the Ontario Ministry of Transportation**

**Highway 144 Rehabilitation - GWP 5580-04-00
Bridge Rehabilitation – Site No. 46-051
Whitson Creek Bridge**

FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT

Date: March 21, 2014
Ref. N^o: 12/11/12218

Geocres No. 41I-304

LVM | MERLEX

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Final Foundation Investigation and Design Report

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LVM inc.'s subcontractors who may have accomplished work either on site or in laboratory are duly qualified as stated in our Quality Manual's procurement procedure. Should you require any further information, please contact your Project Manager."

Client:

AECOM Canada Ltd.

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Attention: **Mr. Al Rose**

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1 INTRODUCTION

LVM | Merlex Ltd. has been retained by AECOM Canada Ltd., on behalf of the Ministry of Transportation of Ontario (MTO), to carry out a foundation investigation to supply subsurface data for the design of a protection system to be implemented at the Whitson Creek Bridge during the proposed rehabilitation. The bridge is located on Highway 144, some 20 km north of Highway 17, in the Township of Balfour. The existing bridge is a single span concrete girder structure some 25 m in length.

The foundation investigation location was specified by the MTO. The terms of reference for the scope of work are outlined in LVM | Merlex Ltd.'s Proposal for Foundation Engineering P-12-103, dated July 5, 2012. The purpose of this investigation was to determine the subsurface conditions in the area of the bridge approaches in order to provide design recommendations for a protection system to be implemented during rehabilitation activities. LVM | Merlex Ltd. investigated the foundation area by the drilling of boreholes, carrying out in-situ tests, and performing laboratory testing on select samples.

2 SITE DESCRIPTION

The Whitson Creek Bridge is located on Highway 144, between Stations 12+419.5 to 12+444.5, Township of Balfour (Site No. 46-051). The topography at the site is generally of low relief. The existing highway embankment currently supports two undivided lanes of highway, locally running in an east-west direction. Whitson Creek flows from north to south at the bridge location. A visual review of the highway at the east and west approaches indicates that, in general, the approaches are in poor condition.

The existing 25 m single span concrete girder bridge was constructed in 1961 and rehabilitated in 1995 on the existing highway alignment. It is understood that the structure is in poor condition.

Infrastructure at the bridge location consists of overhead wires on the north and south sides of the highway.

2.1 SITE PHYSIOGRAPHY AND SURFICIAL GEOLOGY

This project is located in the Geomorphic Sub-province known as the Eastern Sandy Uplands. The topography along this section of Highway 144 is generally flat to slightly rolling. Within the specific project area overburden consists primarily of sands with silts overlying silty clays overlying bedrock.

Bedrock in the area, as indicated on OGS Map 2506, is of the Middle Precambrian Animikie Group which consists of sandstone, shale, argillite, iron formation, tuff, basalt, and limestone.

3 INVESTIGATION PROCEDURES

The field work for this investigation was carried out during the period of August 19th to 21st, 2013, during which time four (4) sampled boreholes were advanced. Two boreholes were advanced at each end of the bridge: one through the existing approach slab and the second a short distance beyond the end of the approach slab.

The field investigation was carried out using a truck mounted CME drilling rig equipped with hollow stem augers, standard augers, and routine geotechnical sampling equipment. Prior to mobilizing the auger drill to the site, the concrete approach slabs were core drilled, where required, with an electric core drill. Soil samples were obtained at the borehole locations at regular intervals of depth using the standard 50 mm O.D. split spoon sampler advanced in accordance with the Standard Penetration Test (SPT) procedures (ASTM D-1586). The SPT method involves advancing a 50 mm O.D. split spoon sampler with the force of a 63.5 kg hammer freely dropping 760 mm mounted in a trip (automatic) hammer. The number of blows per 300 mm penetration was recorded as the “N” value. When cohesive deposits were encountered, the in-situ strength was measured using an “N” size field vane, vane collar, and calibrated torque meter. When shallow refusal was encountered, NQ size diamond coring equipment was used to determine the nature of shallow refusal. All samples taken during this investigation were stored in labeled airtight containers for transport to our North Bay laboratory for visual examination and select laboratory testing.

Groundwater conditions in the open boreholes were observed during the advancement of and immediately following, completion of the individual boreholes. All open boreholes were backfilled upon completion with compacted auger cuttings in the general order they were removed and, where necessary, bentonite pellet backfill was added to the boreholes to bring them up to grade. At the borehole(s) through the embankment, the upper portion of the hole, where necessary, was backfilled with an asphalt cold patch to seal the existing asphalt surface.

The field work for this investigation was under the full time direction of a senior member of the LVM | Merlex engineering staff, who was responsible for locating the boreholes, clearing the borehole locations of underground services, in-situ sampling and testing operations, logging of the boreholes, labeling and preparation of samples for transport to our North Bay laboratory, plus overall drill supervision. All samples received a visual confirmatory inspection in our laboratory. Laboratory testing of select samples included routine testing for natural moisture content determination and particle size analysis, an Atterberg Limits Testing, as well as specific gravity testing. The results of the laboratory testing are presented on the individual Record of Borehole Sheets (Appendix 2), with a summary of results presented on the laboratory sheets in Appendix C (Figures Nos. L-1 to L-6).

The location of the individual boreholes were determined in the field using highway chainage (established by others) and offset relative to highway centerline. The MTO co-ordinates,

northing and easting, were then established for the boring locations. Elevations contained in this report are referenced to a geodetic datum.

4 SUBSURFACE CONDITIONS

Details of the subsurface conditions revealed by the investigation program are presented on the enclosed Record of Borehole Logs (Appendix 2) and on Drawing No. 2 (Appendix 3). Please note that stratigraphic delineation presented on the borehole logs and soil strata plot are the results of non-continuous sampling, response to drilling progress, the results of SPT and field observations. Typically such boundaries represent transitions from one zone to another and are not an exact demarcation of specific geological unit. Additional consideration should be given to the fact that subsurface conditions may vary markedly between adjacent boreholes and beyond any specific boring location, and are shown on the drawings for illustration purposes only.

4.1 WHITSON CREEK BRIDGE

A plan and profile illustrating the borehole locations and stratigraphic sequences is shown on Drawing No. 2, Appendix 3. During the course of the exploration program, four (4) sampled boreholes were put down at this site, as follows;

- Borehole No. 1 was advanced to the east of the east approach slab right of centerline;
- Borehole No. 2 was advanced behind the west abutment right of centerline;
- Borehole No. 3 was advanced behind the east abutment to the left of centerline, and
- Borehole No. 4 was advanced to the west of the west approach slab, left of centerline.

At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 4 were recorded at 268.5, 268.8, 268.6, and 268.9 m, respectively.

4.1.1 Pavement Structure

At surface at Borehole Nos. 1 and 4, a pavement structure consisting of 100 to 150 mm of asphalt and 200 to 300 mm crushed gravel was penetrated. At Borehole Nos. 2 and 3, a pavement structure consisting of 75 to 100 mm of asphalt overlying a concrete approach slab some 250 to 300 mm thick was encountered. A layer of crushed gravel some 250 to 300 mm thick was encountered underlying the concrete approach slab at Borehole Nos. 2 and 3.

4.1.2 Embankment Fill

Underlying the pavement structure at Borehole Nos. 1 to 4, a deposit of fill consisting of brown sand and gravel trace silt was penetrated. The natural moisture content measured on samples of this deposit was in the order of 2 to 6%. Gradation analyses were carried out on eight (8) samples of this deposit, the results of which indicated 29 to 48% gravel size particles, 46 to 60% sand size particles, and 6 to 8% silt and clay size particles (Figure No. L-1, Appendix 3). Based on SPT 'N' values of 7 to 78 blows per 300 mm penetration, the compactness of this deposit was described as loose to very dense, generally dense. This deposit was encountered

to depths of 3.4, 4.4, 5.2, and 3.0 m below grade at Borehole Nos. 1 to 4, respectively (elevations 265.1, 264.4, 263.4, and 265.9 m, respectively).

4.1.3 Sand Fill

Underlying the embankment fill at Borehole Nos. 2 and 3, a deposit of fill described as brown to grey sand some silt was penetrated. The natural moisture content measured on samples of this deposit was in the order of 8 to 25%, indicating a moist to wet moisture condition relative to optimum moisture content. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 0% gravel size particles, 89% sand size particles, and 11% silt and clay size particles (Figure No. L-2, Appendix 3). Based on SPT 'N' values of 1 to 17 blows per 300 mm penetration, the compactness of this deposit was described as very loose to compact, generally loose. This deposit was encountered to a depth of 8.1 m below grade at Borehole Nos. 2 (elevation 260.7 m). Auger refusal was encountered in this deposit at a depth of 10.1 m below grade at Borehole No. 3 (elevation 258.5 m).

4.1.4 Mixed Fill

Underlying the embankment fill at Borehole No. 4, a deposit of fill described as a mix of cobble and boulder sizes mixed with a grey sand with silt was penetrated. Pieces of wood and concrete were encountered in this deposit. The natural moisture content measured on samples of this deposit was in the order of 21%, indicating a wet condition relative to optimum moisture content. This deposit was encountered to a depth of 4.4 m below grade (elevation 264.5 m).

4.1.5 Sand

Underlying the embankment fill at Borehole No. 1, a deposit of grey sand with silt was penetrated. The natural moisture content measured on samples of this deposit was in the order of 9 to 31%, indicating a moist to wet moisture condition, relative to optimum moisture content. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 0% gravel size particles, 75% sand size particles, and 25% silt and clay size particles (Figure No. L-3, Appendix 3). Based on STP 'N' values of 2 blows per 300 mm penetration, this deposit was described as very loose. This deposit was encountered to a depth of 6.7 m below grade (elevation 261.8 m).

4.1.6 Silt

Underlying the fill at Borehole No. 4, a deposit of grey silt trace clay was penetrated. The natural moisture content measured on a sample of this deposit was in the order of 20%, indicating a wet moisture condition, relative to optimum moisture content. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 0% gravel size particles, 0% sand size particles, 95% silt size particles, and 5% clay size particles (Figure No. L-4, Appendix 3). Based on STP 'N' values of 28 blows per 300 mm penetration, this deposit was described as compact. This deposit was encountered to a depth of 5.8 m below grade (elevation 263.1 m).

4.1.7 Silty Clay

Underlying the sand at Borehole No. 1 and underlying the silt at Borehole No. 4, a deposit of grey silty clay was penetrated. The natural moisture content measured on samples of this deposit was in the order of 36 to 51%, indicating a wet moisture condition relative to optimum moisture content. Based on in situ shear strength of 64 to 88 kPa, the consistency of this deposit was described as stiff. Auger refusal was encountered in this deposit at depths of 11.4 and 8.8 m below grade at Borehole Nos. 1 and 4, respectively (elevations 257.1 and 260.1 m, respectively).

4.1.8 Concrete

A layer of concrete was encountered below the sand fill at Borehole No. 2. This concrete is likely part of the abutment footing. The borehole was terminated at a depth of 9.3 m below grade at Borehole No. 2 (elevation 259.5 m).

4.1.9 Bedrock

Underlying the above described silty clay at Borehole No. 4, bedrock was proven by diamond core drilling. The bedrock was described as black slate. Based on Rock Quality Designation (RQD) values of 74 to 98% the bedrock was described as fair to excellent quality. Sampling in the bedrock was terminated at a depth of 12.2 m below grade (elevation 256.7 m). It should be noted that, when encountered, the underlying bedrock surfaces in this area are very erratic in nature, varying substantially in elevation over short horizontal distances.

4.2 GROUNDWATER DATA

Measurements of the groundwater table and cave-in levels were undertaken, where possible, in the open boreholes during the advance of the individual borings and upon completion. Piezometers were installed at Borehole Nos. 1 and 2 to determine stabilized water levels. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix B).

The groundwater levels in Borehole Nos. 1 and 2 were measured at elevations between 261.8 to 262.2 m, some two days after completion of the field program. Borehole No. 3 was dry immediately following completion of the borehole. The groundwater was encountered at a depth of 0.5 m below grade at Borehole No. 4 immediately following completion of coring, however this water level was not stabilized and was likely elevated due to the water used during coring operations. The water level in Whitson Creek was measured at elevation 262.7 m in August 2013.

The groundwater and river water levels will fluctuate seasonally/yearly.

5 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

A foundation investigation was carried out for the design of a protection system for the proposed bridge rehabilitation and conversion to semi integral abutments, at the Whitson Creek Bridge. The bridge is located approximately between Stations 12+419.5 to 12+444.5, in the Township of Balfour, and is identified as Site No. 46-051. The existing bridge is a single span, concrete girder structure, some 25 m in length.

The existing highway, at the bridge location, supports two undivided lanes of traffic, running in an east-west direction. Based on data from this foundation investigation, the embankment supporting the existing pavement structure at this site has been constructed with a granular fill overlying native sands, silts, and silty clays.

Based on the historical General Arrangement Drawing D-4476/1, the Whitson Creek Bridge abutments are founded on spread footings supported on bedrock (see Appendix 4). The conversion to semi-integral abutments will require the removal of the ballast walls. It is anticipated that, to carry out the bridge rehabilitation and convert the Whitson Creek Bridge to a semi-integral abutment, an excavation some 1 m deep will be required behind the existing abutments. As such, a protection system will be required at the east and west abutments of the bridge to support an excavation some 1 m deep behind the abutments and maintain an active lane of traffic. Based on data from this foundation investigation, the fill behind the abutments supporting the approach slabs and pavement structure consists of granular fills.

5.2 EXCAVATION AND DEWATERING

The fill below the pavement structure and approach slabs is considered a Type 3 soil in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. As such, to remain stable above the water table, side walls of temporary open excavations would have to be cut back to an angle of 1H:1V. A 1 m deep excavation (i.e. to elevations 267.6 to 267.8 m) will be required to the rear of the abutments (east and west respectively) to allow the rehabilitation work to be carried out on the ballast wall. The existing width of the approach is insufficient to allow the construction of a 1H:1V slope parallel to the active traffic lane. As such, a vertical excavation adjacent to the active traffic lane will be required and a protection system, installed perpendicular to the abutments, will be needed to support the active traffic lane. Conceptual shoring locations are illustrated on Figure No. SK-4, Appendix 5. Traffic control limiting traffic flow to one lane will be required during excavation and construction operations.

Excavations must be maintained in a dewatered condition during excavation and foundation construction. The water level in the boreholes was recorded at elevations 261.8 to 262.2 m. This water level is below the anticipated depth of excavation (elevations 267.6 m behind the east approach and 267.8 m at the west approach), as such, it is not anticipated that the groundwater table will be encountered during the shallow excavations at the abutments. If a

deeper excavation is required to be advanced below the prevailing groundwater table (estimated at elevation 262.2 m), then groundwater control in accordance with OPSS 517 will have to be carried out.

5.3 PROTECTION SYSTEM

The results of this investigation indicated that, underlying the pavement structure and approach slabs, a granular fill consisting of sands and gravels, in a generally dense state of compactness, was encountered overlying sand fill behind the abutments. Sands and silts were encountered beyond the approach slab. It should also be noted that a layer of fill consisting of cobble/boulder size rock obstructions mixed with sand and silts, containing pieces of concrete was encountered at BH No. 4, at an elevation of 265.6 m.

The required depth of anticipated excavation, directly behind the abutments, will be relatively shallow, in the order of 1.0 m (elevations 267.8 at the west abutment and 267.6 m at the east abutment). In consideration of the anticipated soil conditions, the use of sheet piles of sufficiently robust cross section could be used for a protection system. In order to fix the sheet toe, the sheeting should be driven to a depth of a minimum of 0.5 m below the required depth of excavation. This is in the granular fill deposit. Considering the limited depth of excavation and provided a sheet pile of sufficiently robust section is used, a whaler and raker may not be required if the top of the sheet pile wall is fixed to the existing approach slab. If fixing the sheet pile walls to the approach slab is not possible, a whaler with raker or a tieback system would have to be installed. If tiebacks are required, the resistance (R) for grouted anchors, located outside the active failure wedge, in cohesionless soils can be estimated from the following equation as supplied in the Canadian Foundation Manual (4th Edition):

$$R = \sigma'_z A_s L_s \alpha_g \quad \text{Where: } \sigma'_z = \text{effective vertical stress at the midpoint of the load carrying length}$$

A_s = effective unit surface area of the anchor

L_s = effective embedment length of the anchor

α_g = anchorage coefficient use 1.0 for granular backfill

Unless the pull-out resistance (capacity) of the anchor is proven with a load test program, the allowable anchor load (as suggested by the Canadian Foundation Engineering Manual, 4th Edition), is commonly obtained by dividing the computed capacity of the anchor by a factor of safety of 3. Alternatively, proprietary anchor systems can be used.

If excavation to a greater depth than the anticipated 1 m is required, then the shoring system would have to be advanced to a greater depth. Based on the results of this investigation, the embankment fill, sand fill, and native materials generally do not contain obstructions. However, a layer of fill consisting of a mix of cobble/boulder size rock and sands and silts was encountered at a depth of 3.0 m below grade at Borehole No. 4. As obstructions were not

encountered at the approach boreholes, except at Borehole No. 4, sheet piles are also considered acceptable for use in deeper excavations. However, if a cobble/small boulder size rock is encountered during driving of a sheet section, the individual section could be left high and the cobble/small boulder removed during excavation advance, followed by continued driving of the sheet.

Considering the cohesionless nature of the embankment fills (granular pavement structure over rock fill and granular fills) a rectangular apparent pressure distribution over the height of the cut would be appropriate for design of the temporary shoring. The width of the apparent rectangular pressure distribution, over the height of excavation, can be considered equal to $0.65 \cdot K_a \cdot \gamma \cdot H$, where:

K_a = active earth pressure,

γ = unit weight, and

H = height of wall above the base of excavation.

The protection system can be designed using the lateral earth pressure parameters provided in section 5.4 Lateral Earth Pressures.

The temporary protection system should be designed and constructed to comply with OPSS 539. In consideration of the location of the protection system and traffic volume, a performance level 2 is considered appropriate.

5.4 LATERAL EARTH PRESSURES

Lateral earth pressures should be computed in accordance with the Canadian Highway Bridge Design Code (CHBDC). The design parameters for the fill and native materials are as follows:

PARAMETER	GRANULAR B TYPE I	EMBANKMENT FILL/ GRANULAR A	SAND FILL/SSM	NATIVE SANDS	NATIVE SILT	SILTY CLAY
Unit Weight (kN/m^3)	21.2	22.8	18.0	17.5	18.5	16.5
Angle of Internal Friction	31°	34	29°	28°	29°	-
Coefficient of Active Earth Pressure (K_a)	0.32	0.28	0.35	0.36	0.35	-
Coefficient of Passive Earth Pressure (K_p)	3.12	3.54	2.88	2.77	2.88	-
Coefficient of Earth Pressure at Rest (K_o)	0.48	0.44	0.52	0.53	0.52	-

For rigid structures, the “at-rest” condition (K_o) applies. For flexible structures the “active” condition (K_a) applies.

5.5 BACKFILL AND COMPACTION

The existing backfill at the abutments was generally in a compact condition. Prior to backfilling the excavation the existing subgrade should be proofrolled with a minimum of five overlapping passes of a hand operated vibratory compactor with a minimum weight of 400 kg (or a centrifugal force of 50 kN). Backfilling should be carried out in accordance with OPSS 902 and compaction should be carried out in accordance with OPSS 501.

5.6 CONSTRUCTION CONCERNS

Considering the relatively shallow depth of expected excavations and nature of the approach fill and native materials, no major construction concerns are anticipated if the works are carried out in general conformance to that discussed herein.

6 STATEMENT OF LIMITATIONS

The design recommendations given in this geotechnical report are applicable only to the project described in the text and only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known, in our analysis certain assumptions had to be made. The actual conditions may however, vary from those assumed, in which case changes and modifications may be required to our geotechnical recommendations. We recommend, therefore, that we be retained and provided the opportunity during the design stage to review the design drawings, site survey information, proposed elevations, etc. to verify that they are consistent with our recommendations or the assumptions made in our analysis. It is further recommended that we be retained to review the final design drawings and specifications relative to the geotechnical recommendations.

If, during construction, conditions in the field vary from those assumed at the design stage, an engineer from this office must be notified immediately.

Proper subgrade preparation, groundwater control, compaction, etc. are all critical aspects of the bearing capacity of native soils. It must be noted that different aspects of the geotechnical design are based on the assumption that LVM | Merlex Ltd. will be retained during site preparation and construction of the proposed works to ensure that both the geotechnical site characteristics and the construction operations/techniques are consistent with our recommendations. Should LVM | Merlex Ltd. not be involved during the full construction phase, our liability is strictly limited to the factual information contained herein only.

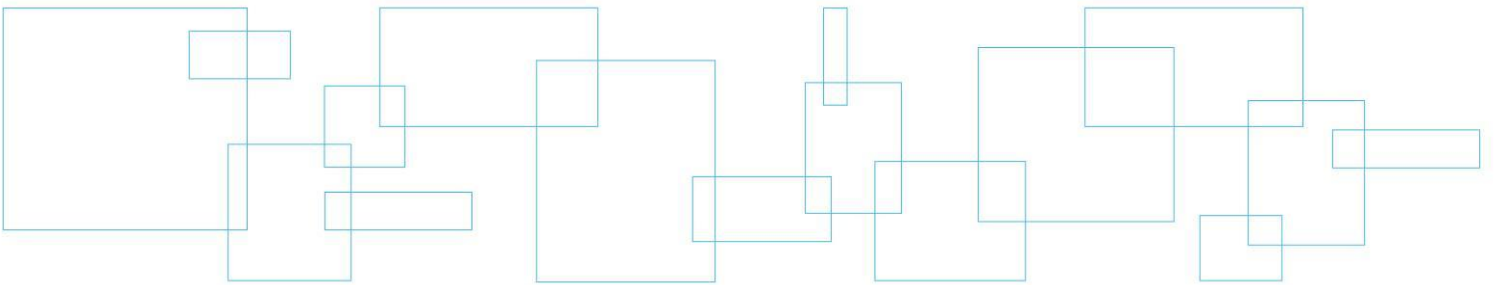
The comments in this report are intended solely for the guidance of the design engineer and address the geotechnical conditions only. The number of boreholes required to determine the localized conditions between boreholes directly affecting construction costs, equipment, scheduling, etc. would in fact be greater than what has been carried out for design purposes. Therefore, contractors bidding on this project or undertaking this work should make their own interpretations of the factual borehole results and carry out further work as they deem necessary to assess the scope of the project.

Section 5 of this reported is intended for the use of the client and the design team only and is not intended to be included in the tender documents. Inclusion of the factual information (Sections 1 to 5 inclusive) in the tender documents is furnished merely for the general information of bidders and is not in any way warranted or guaranteed by or on behalf of the owner or the owner's consultants and its subconsultants or the consultants' or subconsultants' employees, and neither the owner nor its consultants or its employees shall be liable for any representations negligent or otherwise contained in the documents.

Appendix 1 Key Plan

Drawing No. 1

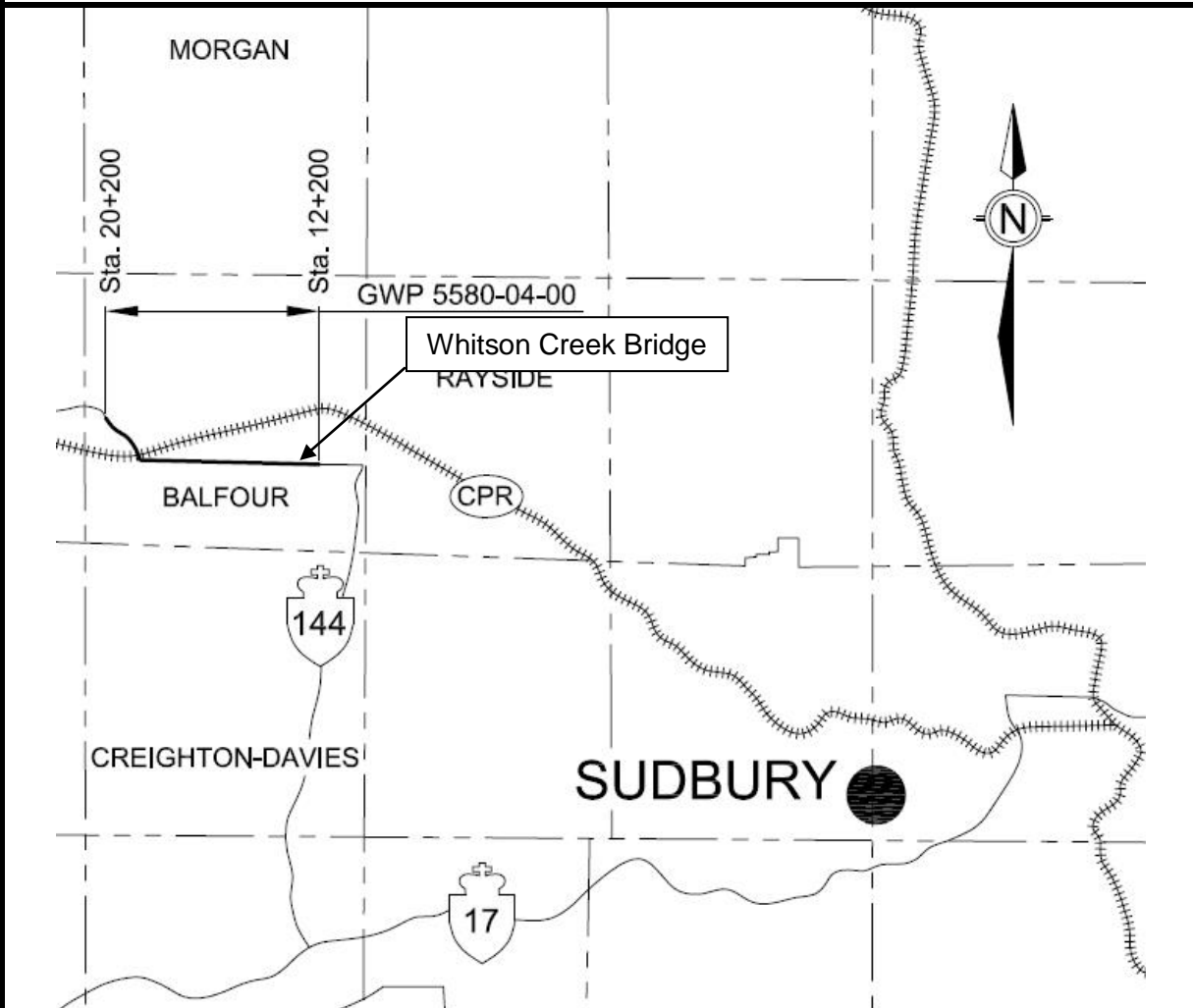
Key Plan



KEY PLAN

Drawing No. 1

NOT TO SCALE



**FINAL
FOUNDATION INVESTIGATION
AND DESIGN REPORT**

GWP 5580-04-00

Highway 144

Whitson Creek

LVM | MERLEX

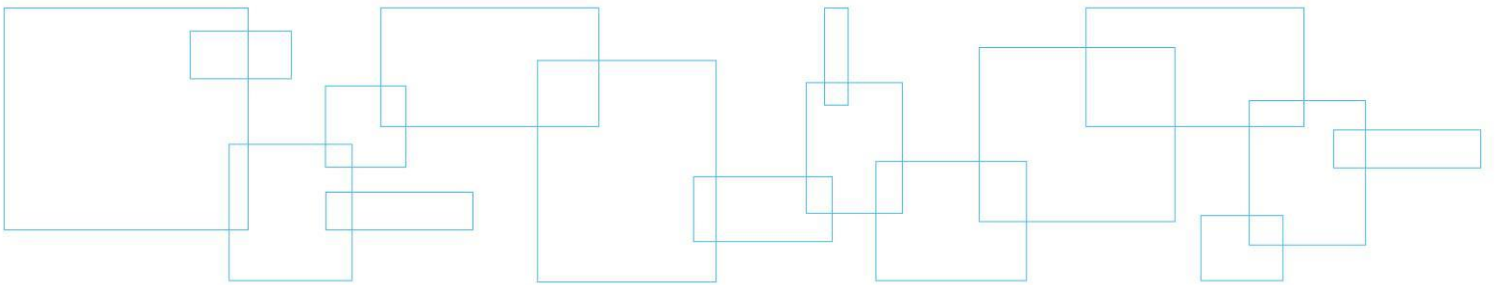
Reference No: 12/11/12218

March 2014

Appendix 2 Subsurface Data

Enclosure No. 1
Enclosure Nos. 2 to 5

List of Abbreviations and Symbols
Record of Borehole Sheet



LIST OF ABBREVIATIONS & DESCRIPTION OF TERMS

The abbreviations and terms, used to describe retrieved samples and commonly employed on the borehole logs, on the figures and in the report are as follows:

1. ABBREVIATIONS

AS	Auger Sample
CS	Chunk Sample
DS	Denison type sample
FS	Foil Sample
NFP	No Further Progress
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
RC	Rock core with size & percentage of recovery
SS	Split Spoon
ST	Slotted Tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash Sample
Rec	% recovery from individual run of rock core
RQD	Rock quality designation (%)

2. PENETRATION RESISTANCE/"N"

Dynamic Cone Penetration Test (DCPT):

A continuous profile showing the number of blows for each 300 mm of penetration of a 50 mm diameter 60° cone attached to AW rod driven by a 63 kg hammer falling 760 mm.

Plotted as —●—●—●—●—

Standard Penetration Test (SPT) or "N" Values

The number of blows of a 63 kg hammer falling 760 mm required to advance a 50 mm O.D. drive open sampler 300 mm.

3. SOIL DESCRIPTION

a) *Cohesionless Soils:*

"N" (blows/0.3 m)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

b) *Cohesive Soils:*

Undrained Shear Strength (kPa)	Consistency
Less than 12	very soft
12 to 25	soft
25 to 50	firm
50 to 100	stiff
100 to 200	very stiff
over 200	hard

3. SOIL DESCRIPTION (Cont'd)

c) *Cohesive Soils:*

RQD (%)	Classification
Less than 25	Very poor quality
25 to 50	Poor quality
50 to 75	Fair quality
75 to 90	Good quality
90 to 100	Excellent quality

d) *Method of Determination of Undrained Shear Strength of Cohesive Soils:*

- + 3.2 - Field Vane test in borehole.
The number denotes the sensitivity to remoulding.
- D - Laboratory Vane Test
- " - Compression test in laboratory

For a saturated cohesive soil the undrained shear strength is taken as one-half of the undrained compressive strength.

e) *Soil Moisture:*

Moisture	Described as
Dry	Below optimum moisture content
Moist	Near optimum moisture content
Wet	Above optimum moisture content

4. TERMINOLOGY

Terminology used for describing soil strata is based on the proportion of individual particle sizes present in the samples (please note that, with the exception of those samples subject to a grain-size analysis, all samples were classified visually and the accuracy of visual examination is not sufficient to determine exact grain sizing):

Trace, or occasional	Less than 10%
Some	10 to 20%
With	20 to 30%
Adjective (i.e. silty or sandy)	30 to 40%
And (i.e. sand and gravel)	40 to 60%

Terminology for cobbles and boulders is based on auger response and field observations:

Occasional	Obstructions encountered in borehole, however advance is not impeded
Numerous	Obstructions are essentially continuous over drilled length

SAMPLE DESCRIPTION NOTES:

1. **FILL:** The term fill is used to designate all man-made deposits of natural soil and/or waste materials. The reader is cautioned that fill materials can be very heterogeneous in nature and variable in depth, density and degree of compaction. Fill materials can be expected to contain organics, waste materials, construction materials, shot rock, rip-rap, and/or larger obstructions such as boulders, concrete foundations, slabs, abandoned tanks, etc.; none of which may have been encountered in the borehole. The description of the material penetrated in the borehole therefore may not be applicable as a general description of the fill material on the site as boreholes cannot accurately define the nature of fill material. During the boring and sampling process, retrieved samples may have certain characteristics that identify them as 'fill'. Fill materials (or possible fill materials) will be designated on the Borehole Logs. If fill material is identified on the site, it is highly recommended that testpits be put down to delineate the nature of the fill material. However, even through the use of testpits defining the true nature and composition of the fill material cannot be guaranteed. Fill deposits often contain pockets or seams of organics, organically contaminated soils or other deleterious material that can cause settlement or result in the production of methane gas. It should be noted that the origins and history of fill material is frequently very vague or non-existent. Often fill material may be contaminated beyond environmental guidelines and the material will have to be disposed of at a designated site (i.e. registered landfill). Unless requested or stated otherwise in this report, fill material on this site has not been tested for contaminants however, environmental testing of the fill material can be carried out at your request. Detection of underground storage tanks cannot be determined with conventional geotechnical procedures.
2. **TILL:** The term till indicates a material that is an unstratified, glacial deposit, heterogeneous in nature and, as such, may consist of mixtures and pockets of clay, silt, sand, gravel, cobbles and/or boulders. These heterogeneous deposits originate from a geological process associated with glaciation. It must be noted that due to the highly heterogeneous nature of till deposits, the description of the deposit on the borehole log may only be applicable to a very limited area and therefore, caution must be exercised when dealing with a till deposit. When excavating in till, contractors may encounter cobbles/boulders or possibly bedrock even if they are not indicated on the borehole logs. It must be appreciated that conventional geotechnical sampling equipment does not identify the nature or size of any obstruction.
3. **BEDROCK:** Auger refusal may be due to the presence of bedrock, but possibly could also be due to the presence of very dense underlying deposits, boulders or other large obstructions. Auger refusal is defined as the point at which an auger can no longer be practically advanced. It must be appreciated that conventional geotechnical sampling equipment does not differentiate between nature and size of obstructions that prevent further penetration of the boring below grade. Bedrock indicated on the borehole logs will be labeled 'possibly' or 'probable' etc. based on the response of the boring and sampling equipment, surrounding topography, etc. Bedrock can be proven at individual borehole locations, at your request, by diamond core drilling operations or, possibly, by testpits. It must also be appreciated that bedrock surfaces can be, and most times are, very erratic in nature (i.e. sheer drops, isolated rock knobs, etc.) and caution must be used when interpreting subsurface conditions between boreholes. A bedrock profile can be more accurately estimated, at the clients' request, through a series of closely positioned unsampled auger probes combined with core drilling.
4. **GROUNDWATER:** Although the groundwater table may have been encountered during this investigation and the elevation noted in the report and/or on the record of boreholes, it must be appreciated that the elevation of the groundwater table will fluctuate based upon seasonal conditions, localized changes, erratic changes in the underlying soil profile between boreholes, underlying soil layers with highly variable permeabilities, etc. These conditions may affect the design and type and nature of dewatering procedures. Cave-in levels recorded in borings give a general indication of the groundwater level in cohesionless soils however, it must be noted that cave-in levels may also be due to the relative density of the deposit, drilling operations etc.

METRIC

RECORD OF BOREHOLE NO. 1



REFERENCE 12/11/12218 DATUM Geodetic LOCATION N 5158833.8 E 288452.9 - Station 12+401.7 Balfour Township ORIGINATED BY JL

PROJECT GWP 5580-04-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT

CLIENT AECOM DATE (Started) 2013 August 19 TIME CHECKED BY MAM

DATE (Completed) 2013 August 19 (Completed) 5:00:00 PM

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	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40					
268.5	Ground Surface													
0.0	150 mm Asphalt 300 mm Crushed Gravel FILL - sand and gravel trace silt brown, dry (compact/very dense)		1	SS	72									
			2	SS	69									
			3	SS	17									
			4	SS	41									
265.1			5	SS	19									
3.4	SAND - with silt brown, dry (very loose) moist		6	SS	2									
			7	SS	2									
			8	SS	2									
261.8														
6.7	SILTY CLAY grey, wet 6 mm silt varves at 25 mm spacing (stiff)		9	SS	WH									
			10	SS	PM									
			11	SS	PM									
257.1														
11.4	Auger Refusal End of Borehole													
COMMENTS							+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		WATER LEVEL RECORDS Date (dd/mm/yy)/Time Water Depth (m) Cave In (m) 1) 13/8/19 5:00:00 PM 6.7 ▽ - <input checked="" type="checkbox"/> 2) 13/8/21 5:05:00 PM 6.7 ▽ - <input checked="" type="checkbox"/> 3) 13/8/22 8:40:00 AM 6.7 ▽ - <input checked="" type="checkbox"/>					

The stratification lines represent approximate boundaries. The transition may be gradual.



METRIC

RECORD OF BOREHOLE NO. 2



REFERENCE 12/11/12218 DATUM Geodetic LOCATION N 5158834.6 E 288407.6 - Station 12+447 Balfour Township ORIGINATED BY JL

PROJECT GWP 5580-04-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT

CLIENT AECOM DATE (Started) 2013 August 20 TIME CHECKED BY MAM

DATE (Completed) 2013 August 20 (Completed)

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	w_p	w	w_L		
268.8	Ground Surface																
0.0	100 mm Asphalt		1	SS	29												
268.2	250 mm Concrete																
	300 mm Crushed Gravel																
0.6	FILL - sand and gravel trace silt		2	SS	30												48 46 (6)
	brown, dry																
	(compact/dense)		3	SS	27												
			4	SS	50												39 55 (6)
			5	SS	18												
264.4	FILL - sand some silt		6	SS	45												37 54 (9)
	brown, dry																
	(loose/compact)																
			7	SS	17												0 89 (11)
			8	SS	8												
	grey, wet																
260.7	creosote treated wood in tip		9	SS	15												
8.1	CONCRETE																
	(probably footing)																
259.5																	
9.3	End of Borehole																

COMMENTS		WATER LEVEL RECORDS	
The stratification lines represent approximate boundaries. The transition may be gradual.		+ 3, x 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE	
		Date (dd/mm/yy)/Time	Water Depth (m)
		1) 13/8/20 3:30:00 PM	4.1
		2) 13/8/21 5:00:00 PM	6.6
		3) 13/8/22 8:35:00 AM	6.6

MEL-GEO 12218 - BOREHOL LOGS.GPJ MEL-GEO.GDT 14/3/21

METRIC

RECORD OF BOREHOLE NO. 3



REFERENCE 12/11/12218 DATUM Geodetic LOCATION N 5158829.0 E 288438.5 - Station 12+416 Balfour Township ORIGINATED BY JL

PROJECT GWP 5580-04-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT

CLIENT AECOM DATE (Started) 2013 August 21 TIME

DATE (Completed) 2013 August 21 (Completed) 11:30:00 AM CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	w_p	w	w_L		
268.6	Ground Surface																
0.0	75 mm Asphalt 300 mm Concrete 250 mm Crushed Gravel		1	SS	27												
268.0	FILL - gravelly sand trace silt brown, dry (loose/very dense)		2	SS	17											33 60 (7)	
0.6			3	SS	7												
			4	SS	29												
			5	SS	42											32 60 (8)	
			6	SS	35												
			7	SS	61											29 63 (8)	
263.4	FILL - sand some silt grey, moist (very loose/loose)		8	SS	6												
5.2			9	SS	5												
			10	SS	1												
258.5	Auger Refusal End of Borehole																
10.1																	

COMMENTS		WATER LEVEL RECORDS	
+ 3, \times 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		Date (dd/mm/yy)/Time	Water Depth (m)
		1) 13/8/21 11:30:00 AM	DRY ∇ 6.4 ∇
		2)	- ∇ -
		3)	- ∇ -

The stratification lines represent approximate boundaries. The transition may be gradual.

MEL-GEO 12218 - BOREHOL LOGS.GPJ MEL-GEO.GDT 14/3/21

METRIC

RECORD OF BOREHOLE NO. 4



REFERENCE 12/11/12218 DATUM Geodetic LOCATION N 5158830.0 E 288397.5 - Station 12+457 Balfour Township ORIGINATED BY JL

PROJECT GWP 5580-04-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT

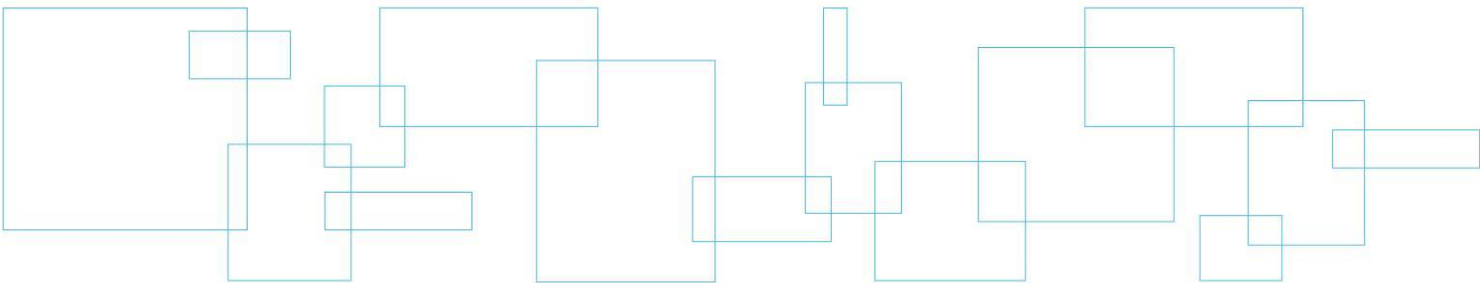
CLIENT AECOM DATE (Started) 2013 August 21 TIME 2013 August 21 (Completed) 8:10:00 PM CHECKED BY MAM

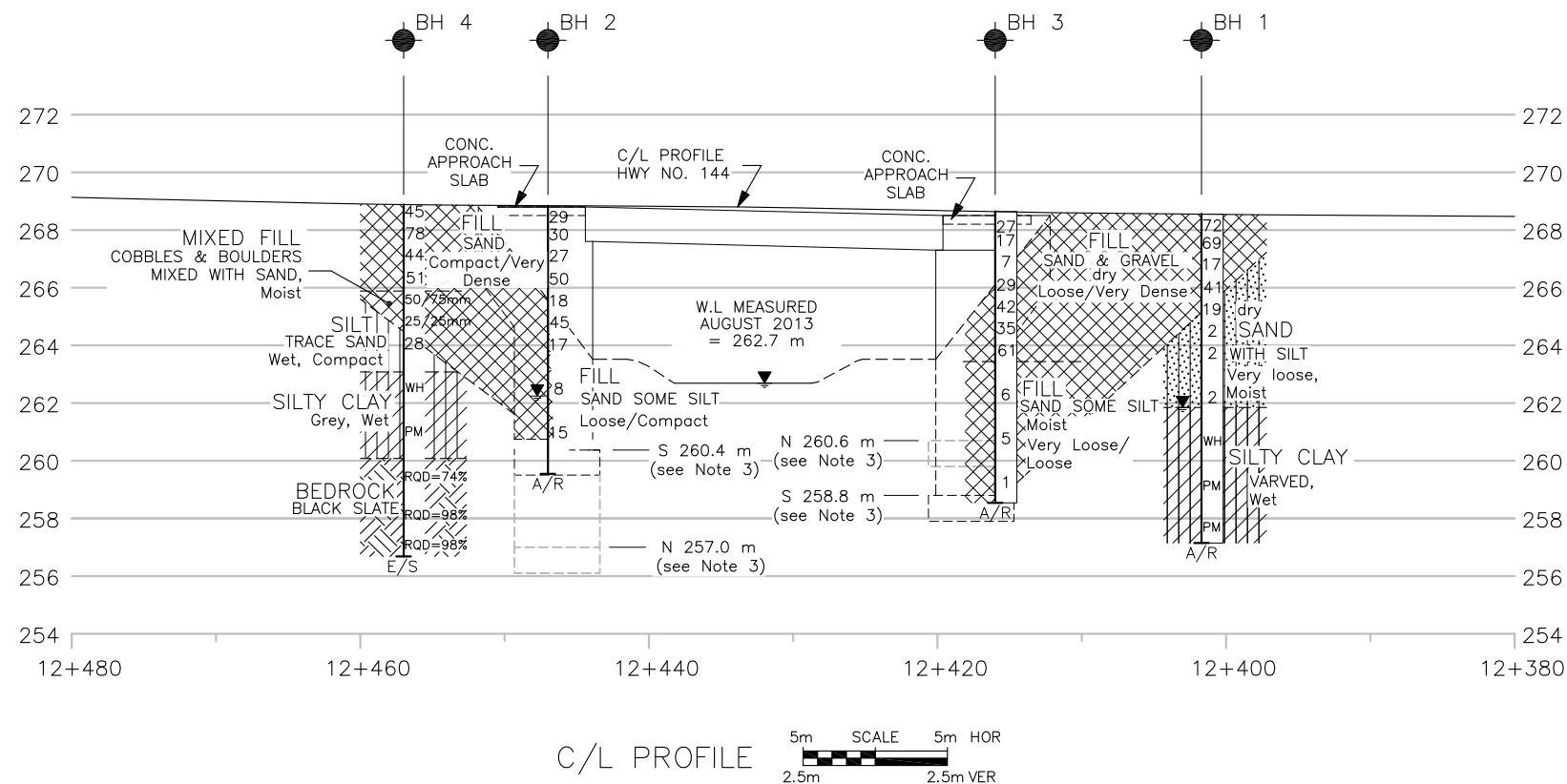
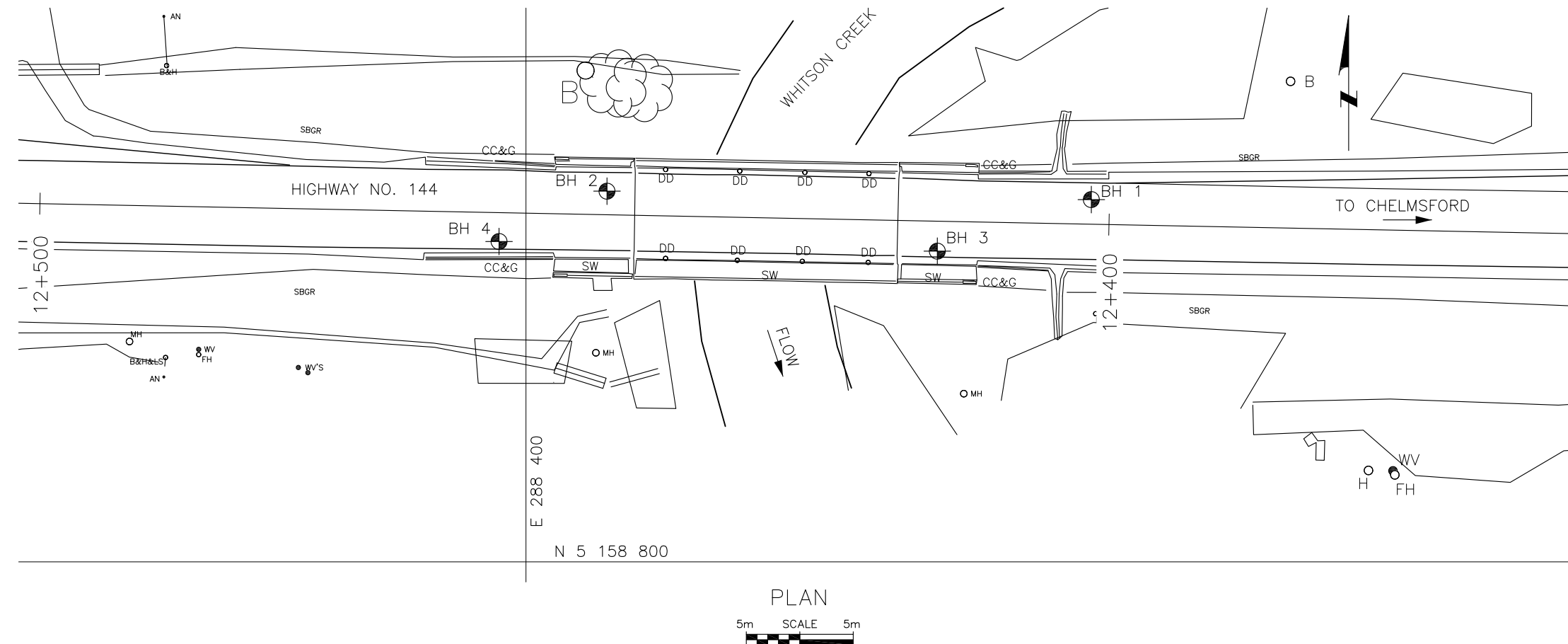
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40					
268.9	Ground Surface													
0.0	100 mm Asphalt 200 mm Crushed Gravel FILL - sand and gravel trace silt brown, dry (dense/very dense)		1	SS	45									
			2	SS	78									
			3	SS	44									
			4	SS	51									
265.9														
3.0	FILL - cobbles/boulder size rock mixed with sand with silt grey, wet		5	SS	50/75 mm									
			6	SS	25/25 mm									
264.5	pieces of wood and concrete													
4.4	SILT trace sand grey, wet (compact)		7	SS	28									
263.1														
5.8	SILTY CLAY grey, wet (stiff)		8	SS	WH									
			9	SS	PM									
260.1														
8.8	BEDROCK - black slate fair to excellent to quality		10	RC	Rec= 100% RQD= 74%									
			11	RC	Rec= 100% RQD= 98%									
			12	RC	Rec= 100% RQD= 98%									
256.7														
12.2	End of Borehole													
COMMENTS Note: Groundwater level in borehole at 0.5 m depth below grade upon completion. Water level NOT Stabilized. The stratification lines represent approximate boundaries. The transition may be gradual.								+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		WATER LEVEL RECORDS Date (dd/mm/yy)/Time Water Depth (m) Cave In (m) 1) - - 2) - - 3) - -				


MEL-GEO 12218 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/21

Appendix 3 Lab Data

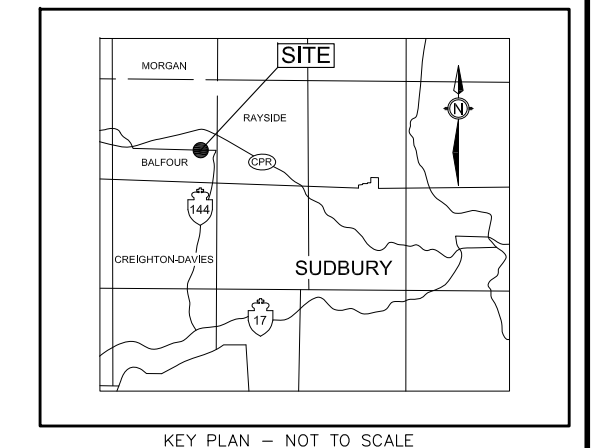
Drawing No. 2: Borehole Location and Soil Strata
Figure Nos. L-1 to L-4: Grain Size Distribution Curves
Figure No. L-5: Shear Strength Summary Chart
Figure No. L-6: Lab Test Summary Sheet









CONT. No. XXXX-XXXX GWP. No. 5580-04-00	<div style="text-align: center;">  </div> <div style="text-align: center;"> DRAWING 2 </div>
--	--

HWY 144
WHITSON CREEK BRIDGE (SITE 46-051)
BALFOUR TOWNSHIP
BOREHOLE LOCATIONS & SOIL STRATA



LEGEND

- | | | | |
|---|--|---|--------------------------------------|
|  | Borehole |  | Dynamic Cone Penetration Test (DCPT) |
|  | Borehole w/ DCPT | | |
| N | Blows/0.3 m (Std Pen Test, 475 J/blow) | | |
| DCPT | Blows/0.3 m (60° Cone, 475 J/blow) | | |
|  | Water Level at Time of Investigation | | |
| A/R | Auger Refusal | | |
| E/S | End of Sampling | | |

Borehole No.	Elev.	O/S	Co-ordinates	
			Northerly	Easterly
Borehole No. 1	268.5	2.3 m Rt	5158833.8	288452.9
Borehole No. 2	268.8	2.2 m Rt	5158834.6	288407.6
Borehole No. 3	268.6	2.8 m Lt	5158829.0	288438.5
Borehole No. 4	268.9	2.7 m Lt	5158830.0	288397.5

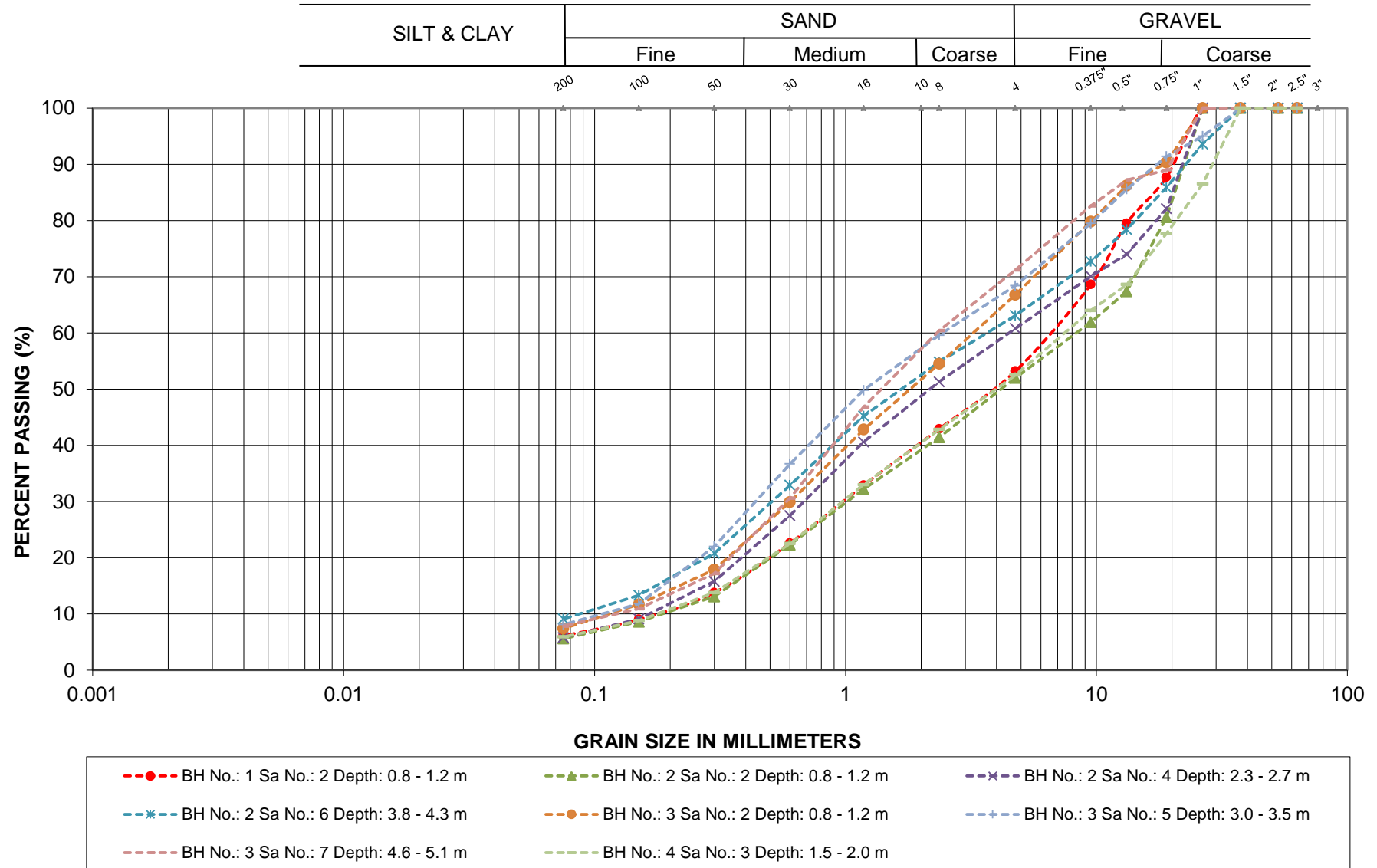
NOTE 1: This drawing is for subsurface information only. Surface details and features are for conceptual illustration. The proposed structure location is shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

NOTE 2: The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design only.

NOTE 3: Top of footing elevations illustrated in the profile were derived from elevations shown in the General Arrangement Drawing D-4476/1 dated March 1960.

REVISIONS	DATE	BY	DESCRIPTION
	Mar 2014	IK	REVISION 1
HWY NO. 144 – BALFOUR TOWNSHIP GEOCRE NO.: 411–304 L V M REF. NO.: 12/11/12218 DRAWN: RG CHECKED: AT DATE: FEBRUARY 2014			

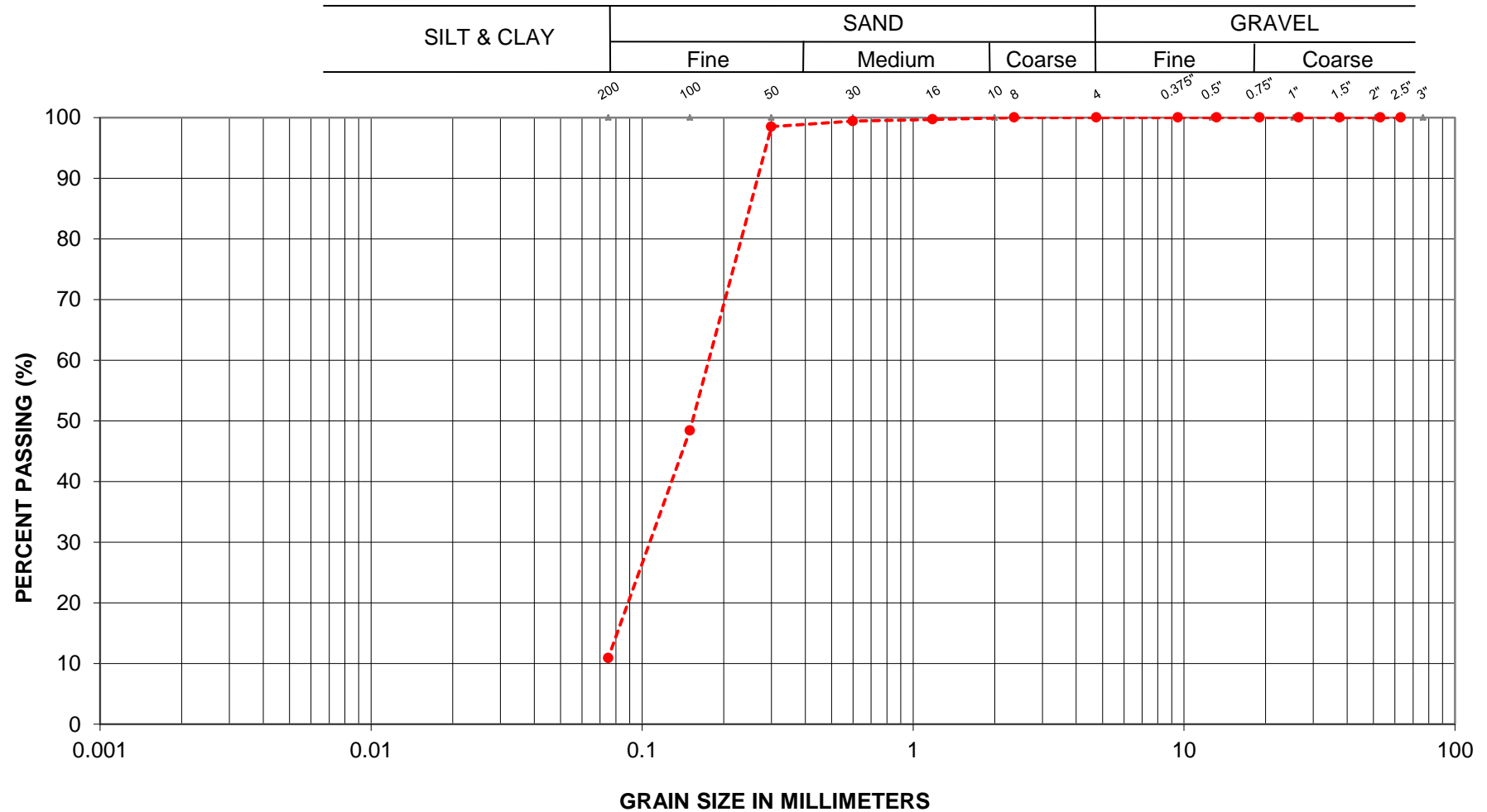
GRAIN SIZE ANALYSIS



G.W.P.: 5580-04-00
LOCATION: Hwy 144, Whitson Creek Culvert

EMBANKMENT FILL

GRAIN SIZE ANALYSIS

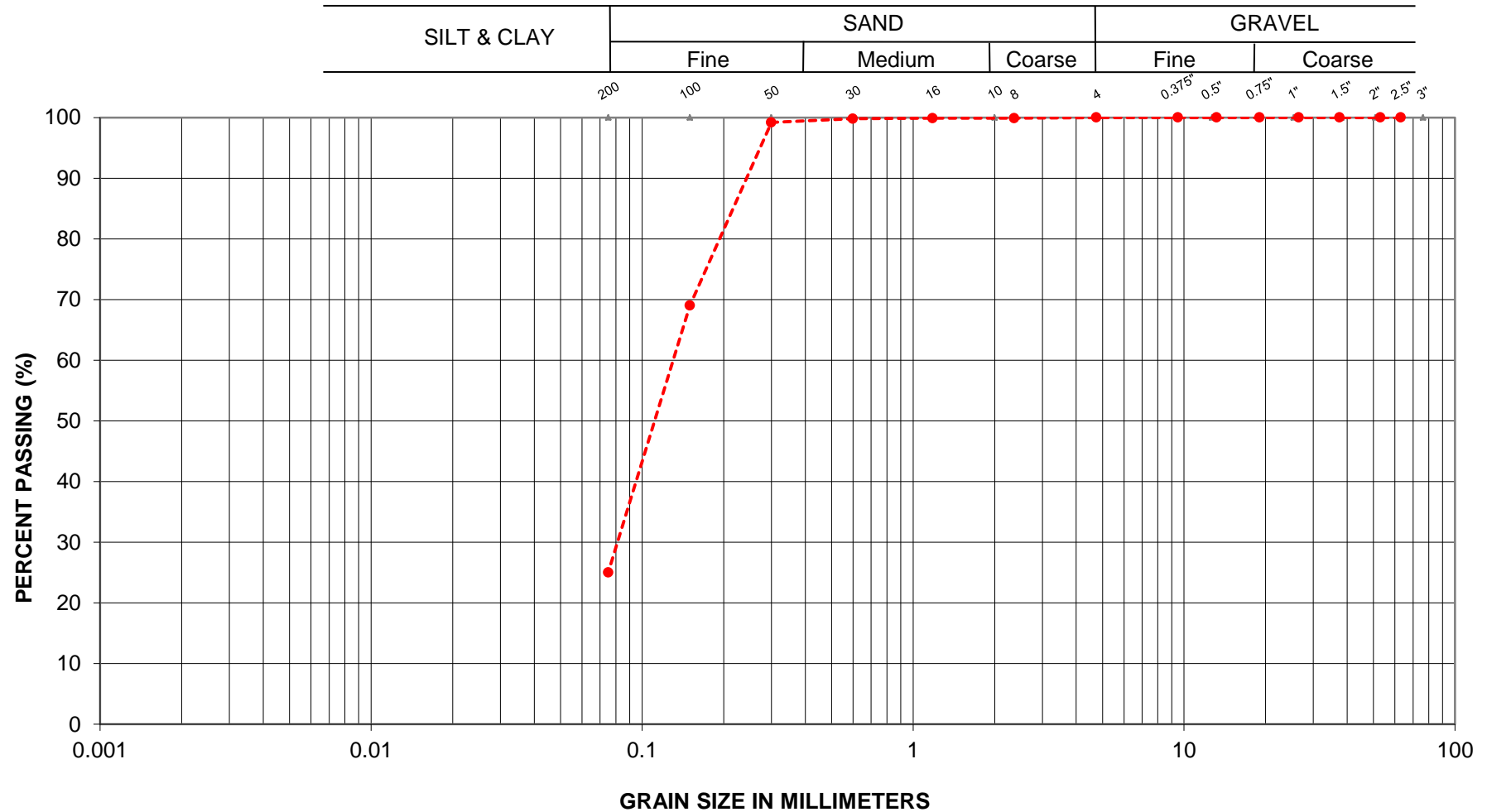


---●--- BH No.: 2 Sa No.: 7 Depth: 4.6 - 5.1 m

G.W.P.: 5580-04-00
LOCATION: Hwy 144, Whitson Creek Culvert

SAND FILL

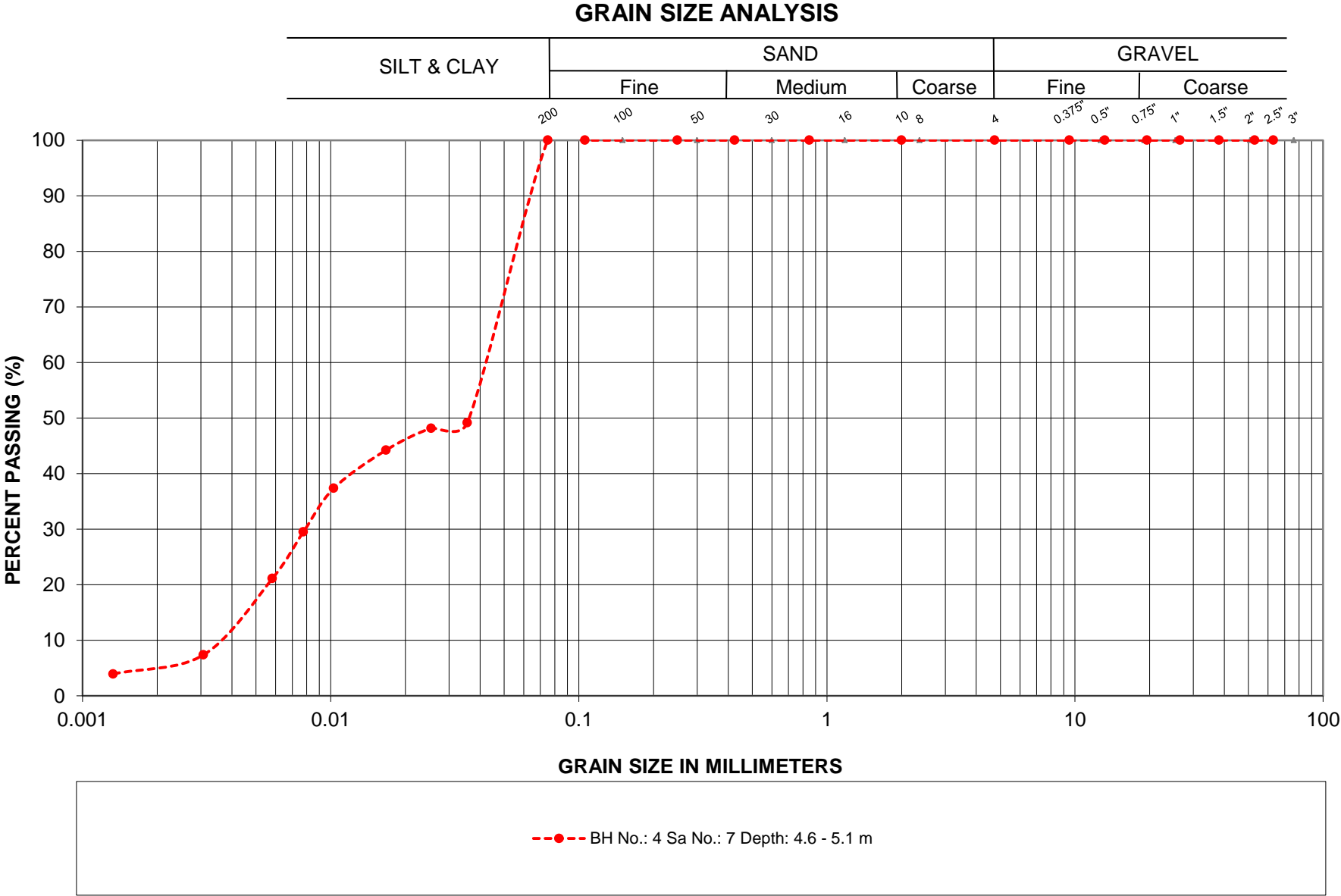
GRAIN SIZE ANALYSIS



---●--- BH No.: 1 Sa No.: 6 Depth: 3.8 - 4.3 m

G.W.P.: 5580-04-00
LOCATION: Hwy 144, Whitson Creek Culvert

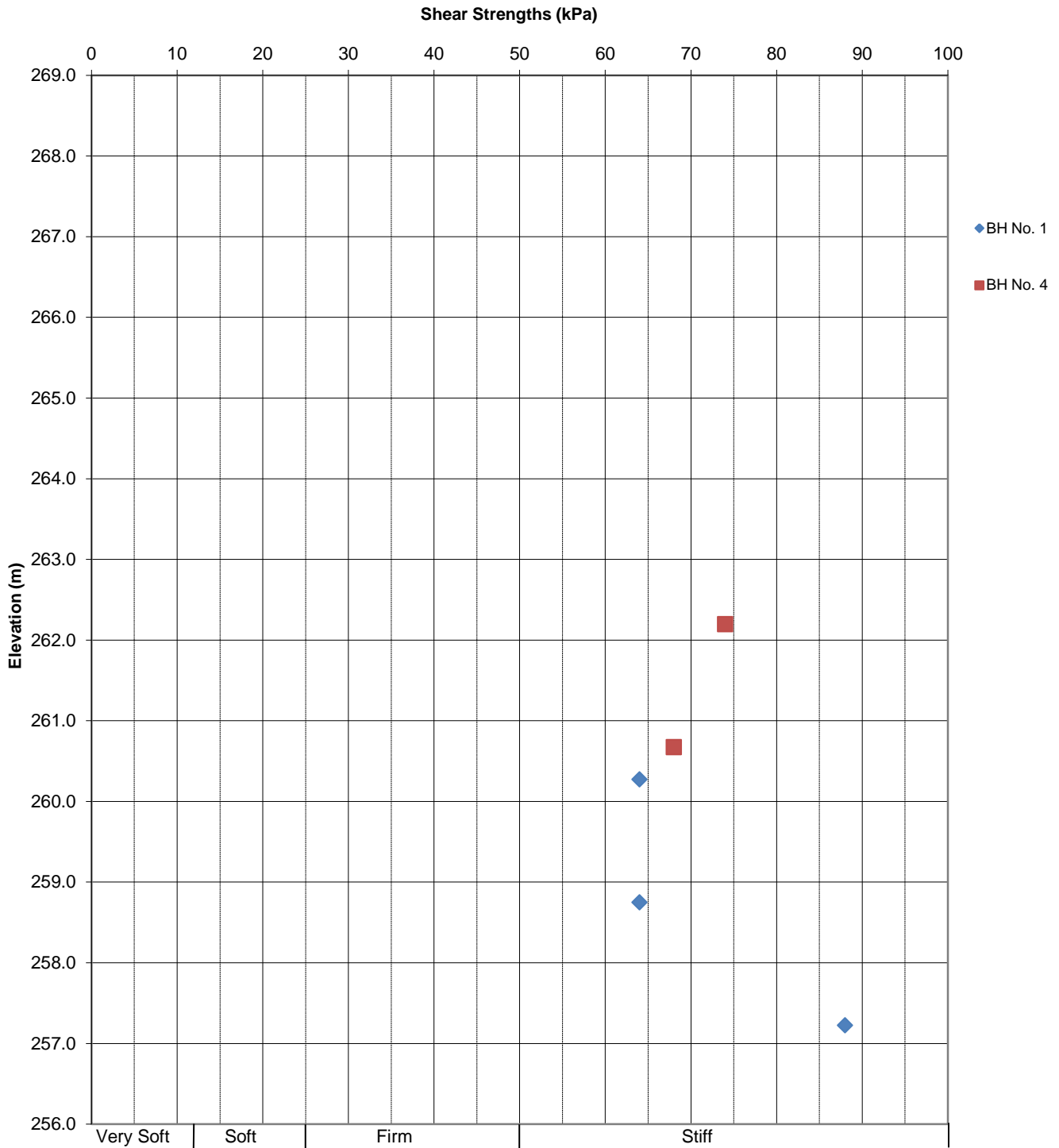
SAND



G.W.P.: 5580-04-00
LOCATION: Hwy 144, Whitson Creek Culvert

SILT

In-Situ Shear Strengths vs. Depth



Laboratory Tests - Summary Sheet

Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Unit Weight (kN/m3)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
1	1	0.2					2.0				72			
	2	0.8	47	47	6		2.1				69			
	3	1.5					2.8				17			
	4	2.3					4.7				41			
	5a	3.1					3.8				19			
	5b	3.1					8.6				19			
	6	3.8	0	75	25		16.6				2			
	7	4.6					17.9				2			
	8	6.1					30.8				2			
	9	7.6					51.3				WH			
	10	9.1					41.9				PM			
	11	10.7					35.7				PM			
2	1	0.0					3.6				29			
	2	0.8	48	46	6		1.5				30			
	3	1.5					1.6				27			
	4	2.3	39	55	6		2.5				50			
	5	3.1					3.4				18			
	6	3.8	37	54	9		4.6				45			
	7	4.6	0	89	11		7.7				17			
	8	6.1					21.6				8			
	9	7.6					21.0				15			
3	1	0.3					3.3				27			
	2	0.8	33	60	7		2.6				17			
	3	1.5					2.2				7			
	4	2.3					4.5				29			

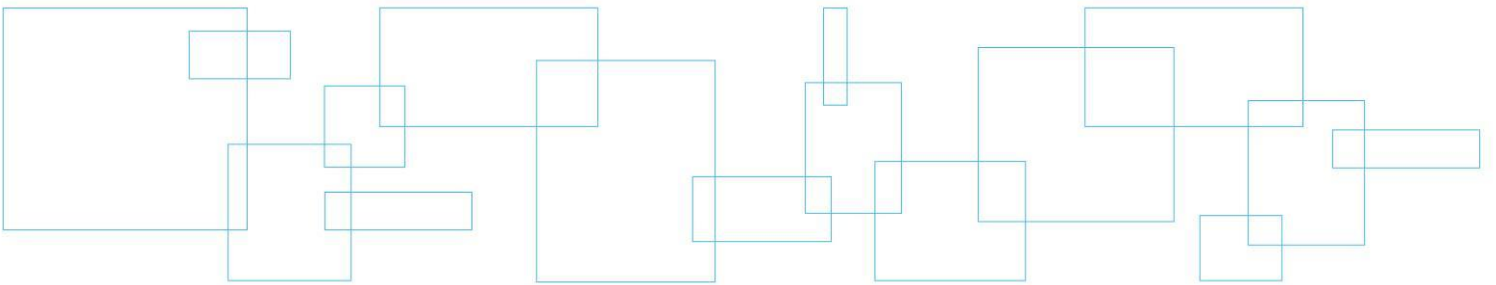
Laboratory Tests - Summary Sheet

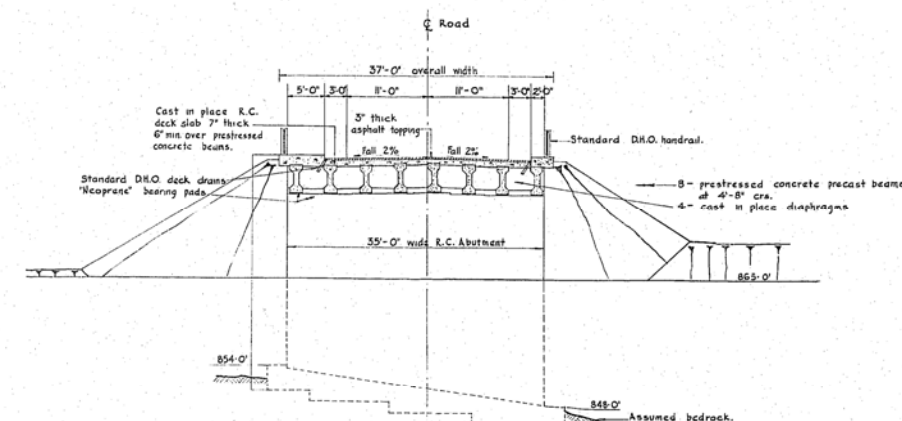
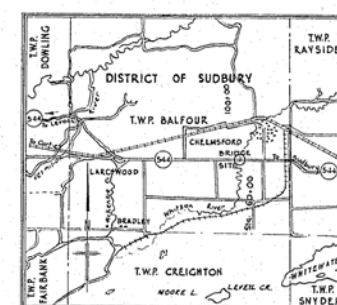
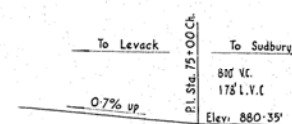
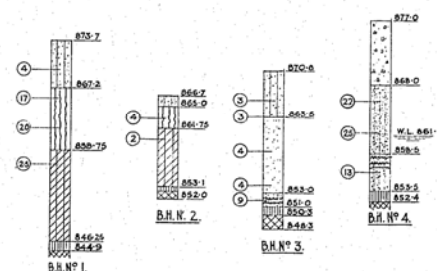
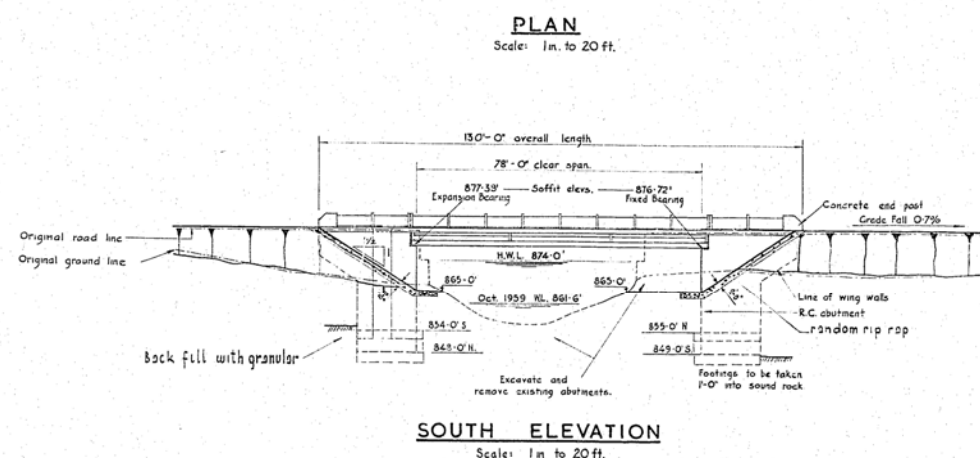
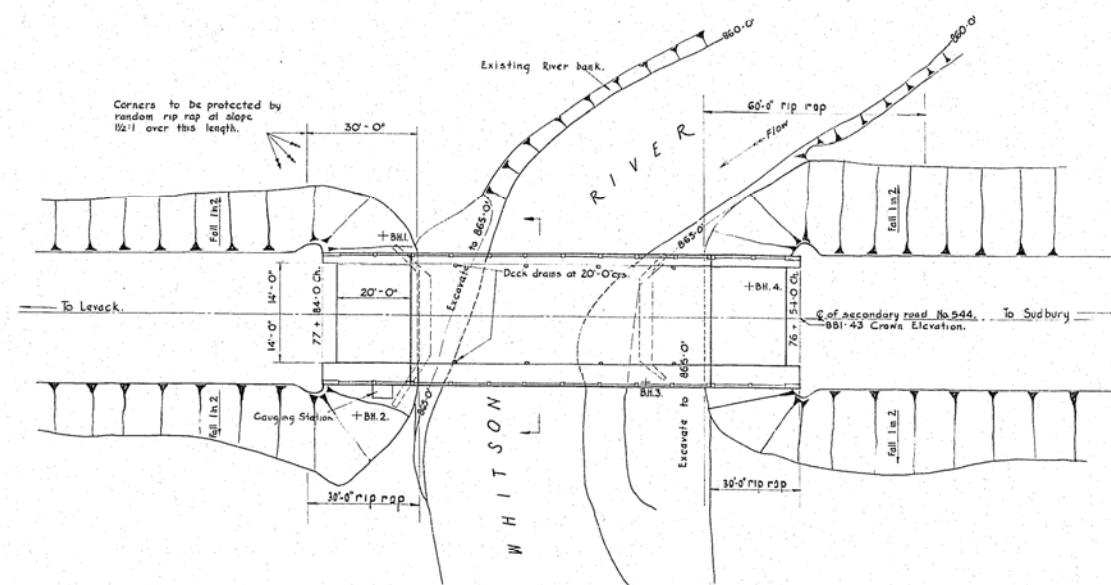
[illegible]

Appendix 4 Historical Data

Enclosure Nos. 6:

Historical Drawings





CROSS SECTION
Scale: 1 in to 10 ft

GENERAL NOTES

NOTE TO DISTRICT ENGINEERS:-
Concrete work on this structure must not be commenced until monuments to fix control points have been erected and checked by the District Engineer.

NOTE TO CONTRACTOR:-
Structure to be built in accordance with the latest Form N° 9 and the Special Provisions, extra copies of which may be obtained from the District Engineer.

All Construction Joints must be approved by the Bridge Engineer.

CONCRETE DETAILS:-

Concrete to Footings, Abutments & Wing Walls, Deck Slab, Diaphragms, Sidewalks & Curbs to have a minimum compressive strength of 3000 p.s.i. at 28 days. Concrete to prestressed beams to have a minimum compressive strength of 5000 p.s.i. at 28 days and a minimum compressive strength of 4000 p.s.i. at transfer of prestress.

An approved admixture supplied by the Department will be added to all concrete as specified by the Materials and Research Section, D.H.O.

Maximum aggregate size in footings and abutments below beam seatings $1\frac{1}{2}$ " in all other work, $3\frac{1}{4}$ "

REINFORCING STEEL:-
Clear cover in footings and abutments to be 2" except where noted otherwise.
Clear cover in Deck Slab, Diaphragms, Sidewalks & Curbs to be 1".
Clear cover in Prestressed Beams to be 1".

CONSTRUCTION NOTES:-
All exposed edges to be chamfered as shown on the drawings.
No concrete to be placed above bridge seat elevations until
concrete in deck slab has been placed.

The General Contractor shall be responsible for finishing the bridge seats dead level to the specified elevations with a tolerance of plus or minus 1/8 inch. If they are cast too high they shall be bush hammered down by the General Contractor. If they are cast too low the General Contractor shall provide full bearing shims to bring them up to the correct elevations. The use of grout is prohibited.

Footings to be taken 1'-0" minimum into sound rock.
Drainage ditches at foot of existing embankments to be maintained.
No concrete to be placed in ballast wall until deck has been replaced.

J.G.G.	ENCL. ON ABUT. SEAT REVISED. (MAY 31/64 BY J.G.G. TOP PORTION CHANGING STA. ENDS)
J.G.G.	-15 ADDED TO SCHEDULE.
J.M.	Gauging Station Added
P.B.H.	Note Added to Construction Notes
P.B.H.	End Posts Corrected
	DESCRIPTION

SCHEDULE OF DRAWINGS.

- D-4476/1 General Arrangement.
- D-4476/2 Abutments - Concrete Details.
- D-4476/3 West Abutment - Reinforcing.
- D-4476/4 East Abutment - Reinforcing.
- D-4476/5 Prestressed Concrete Beams - Details.
- D-4476/6 Deck Slab
- D-4476/7 Reinforcing Steel Schedule 1.
- D-4476/8 Reinforcing Steel Schedule 2.
- D-4476/9 Details of Steel Handrail Panels.
- D-4476/11 Details of Steel End Post for Handrail Panels.
- BD 4-1 Standard Drain Pipe for Bridge Floors
- D-4476/10 Reinforcing Steel Schedule 3.
- D-4476/12 Detail of Interior Steel Post for Handrail Panels
- D-4476/13 Gauging Station
- D-4476/15 Angle Setting for Standard Drain Pipe

WP. 252/59

SIR ALEXANDER GIBB & PARTNERS
CONSULTING ENGINEERS

SIR A.C. & P. DRG.N
3550/C/4

DEPARTMENT OF HIGHWAYS: ONTARIO
BRIDGE OFFICE: -TORONTO

WHITSON RIVER BRIDGE

THE KING'S HIGHWAY No. 544 (Secondary). DIST. No. 17
CO.
TWP. BALFOUR LOT 3 & 4 CON. 1 & 2

GENERAL ARRANGEMENT

APPROVED

M. Long

BRIDGE ENGINEER				DESIGN ENGINEER			
DESIGN	J.D.	CHECK	J.R.L.	CONTRACT			GR 100

DRAWING	J. B.	CHECK	J. K. L.	NUMBERS		60-
TRACING	J. M.	CHECK	J. B.	LOADING	DRAWING	D-1176/1

DATE	MARCH 1960	H20-S16	NUMBER	D-44/6/1
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TWP# 618-51-1-A

Appendix 5 Design Data

Table A: Comparison of Shoring Alternatives
Figure No. SK-4: Conceptual Shoring Locations

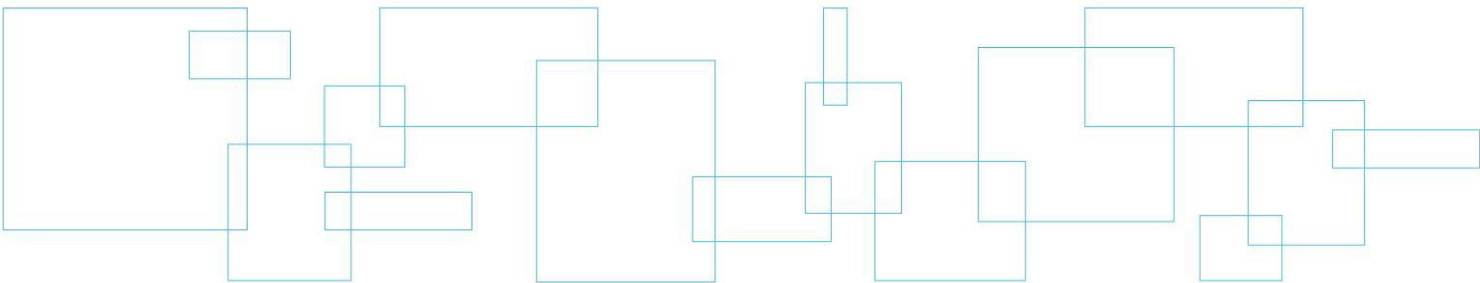
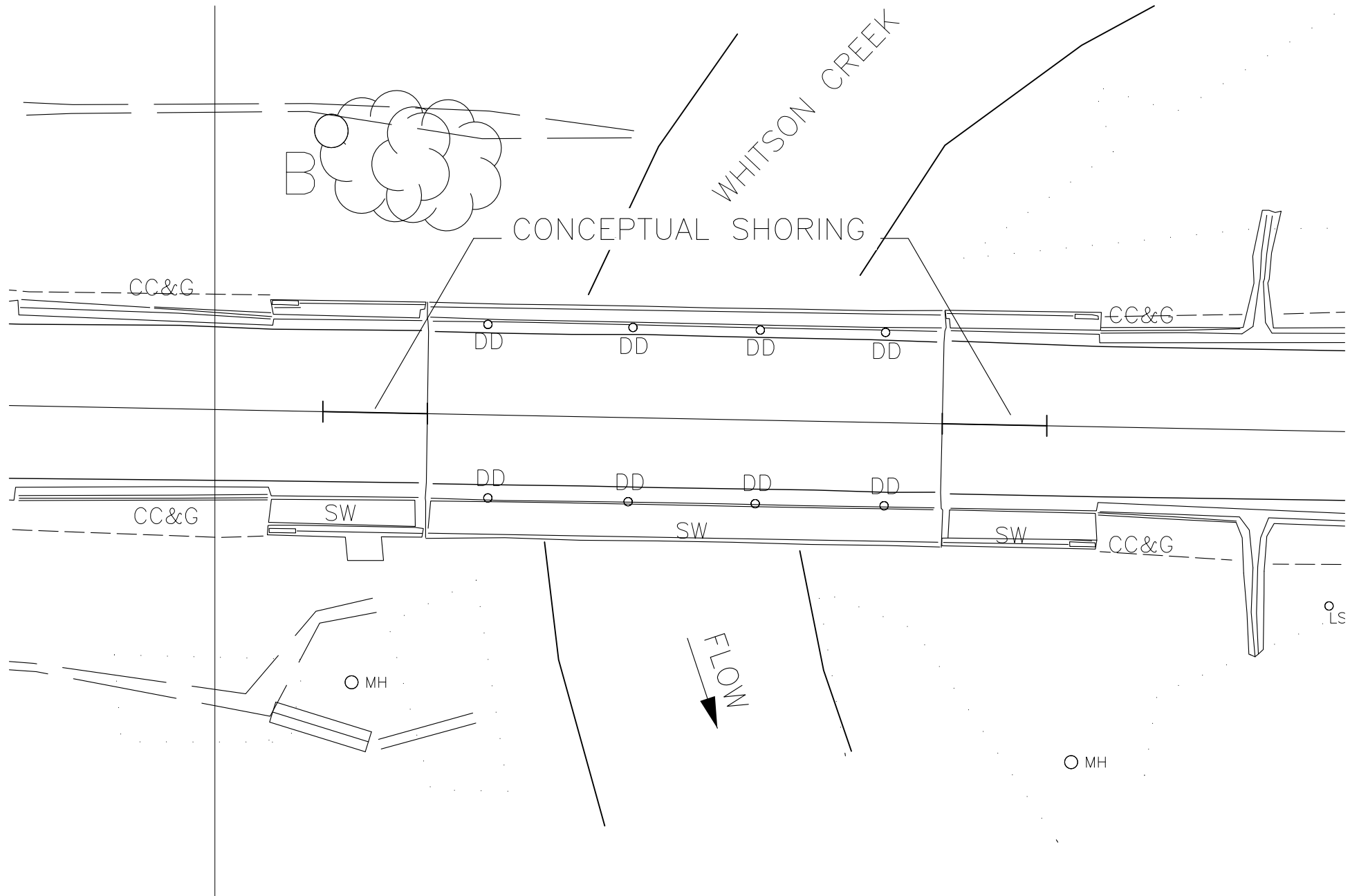


Table A – Protection Systems

Retaining System	Depth Range (m)	Advantages	Disadvantages	Remarks	Estimated Costs
Wood Sheeting	1.5 – 5	-Low cost, -Easily installed in good ground conditions	-Limited by soil conditions, -Limited depth of installation, -Low strength, -discontinuous	Considered for protection system	\$ 650/m ²
Steel Sheet Piles	5 – 21	-High strength, continuous, -Readily available	-Limited by soil conditions (i.e. obstructions)	Recommended for shallow excavations at this site (i.e. 2 m or less)	\$ 650/m ²
Pre-cast concrete panels	3 – 10	-Durable -Assists in minimizing seepage	-Limited depths -Can be damaged by driving -Limited by soil conditions (i.e. obstructions)	Not considered due to limited depth required and higher costs	
Soldier piles With lagging	5 – 25	-Easy installation -Readily available -Adaptable to various ground conditions	-Pre-drilling may be required -Possible ground loss	Considered for deep excavations at this site	\$ 725/m ²
Tangent/ Secant/ Staggered Drilled Piles	10 – 18	-Readily available -Adaptable to various ground conditions	-Possible ground loss and/or seepage -Poor alignment tolerance	Not Considered due to limited depths required and higher costs	
Concrete Diaphragm	10 – 30	-High Strength -Durable -Can be permanent	-High cost -Requires specialized equipment/control	Not Considered due to limited depths required and higher costs	
Micropiles with reinforced shotcrete face		-Can be installed in various ground conditions -High strength -Good tolerance	-High Cost -Requires specialized equipment	Considered for deep excavations at this site	\$ 900/m ²



HWY 144 - Township of Balfour
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FIGURE SK-4