



**FOUNDATION INVESTIGATION  
AND DESIGN REPORT  
BLANCHE RIVER BRIDGE  
REHABILITATION  
HIGHWAY 65  
NEW LISKEARD AREA  
SITE NO.: 47-047  
AGREEMENT NO.: 5009-E-0073  
GWP: 166-98-00  
GEOCRES NO.: 31M-092**

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**FOUNDATION INVESTIGATION AND DESIGN REPORT  
REHABILITATION OF BLANCHE RIVER BRIDGE  
HIGHWAY 65, NEW LISKEARD AREA  
SITE NO. : 47 - 047  
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**PART 1: FACTUAL INFORMATION**

**1. INTRODUCTION**

DST Consulting Engineers Inc. has been subcontracted by Genivar which was retained by the Ministry of Transportation (MTO), Northeastern Region, to conduct a geotechnical investigation for a roadway protection system at Blanche River Bridge, Highway 65, New Liskeard Area. This work was carried out under Agreement No.: 5009-E-0073, Detail Design Service for Rehabilitation of Blanche River Bridge, Site No. 47 – 04.

This report addresses the field investigation, laboratory test program, factual report on conditions (Part 1) and assessment of the suitability of the subsurface conditions to provide temporary support during rehabilitation of Blanche River Bridge (Part 2). The nature of the temporary works is understood to be vertical excavations near the abutments up to 2 m deep. Details of the works are yet unknown.

## 2. SITE DESCRIPTION

The site, Blanche River Bridge is located on Highway 65, about 19.2km east of the junction of Highway 11B, Township of Casey, New Liskeard Area. The structural site number is 47-047.

The Blanche River Bridge is a three-span steel low truss superstructure that supports a structural steel floor system and asphalt that covers the concrete deck. A sidewalk supported by triangular steel brackets is present on the outside of the south truss. The total length of the bridge is 105.6 m with width of 9.15 m. It was constructed in 1960. The structure consists of concrete abutments and two massive piers supported on wooden and steel piles.

Approach slabs of approximately 0.25 m (10 inch) thick and 6 m (20 ft) long are at both ends of the bridge. Guardrails are installed on the embankment. Steep slopes at the embankment are noticeable (Figure 2.1, 2.4).



Figure 2.1 Blanche River Bridge (Looking Southwest)



Figure 2.2 Blanche River Bridge (Looking East)



Figure 2.3 Blanche River Bridge (Looking Northeast)





Figure 2.4 Bridge Approach Embankment, Blanche River Bridge



### **3. INVESTIGATION PROCEDURES AND LABORATORY TESTING**

Site work was carried out in the period between April 30, 2011 and May 7, 2011 utilizing a CME 750 drill rig operated by DST personnel.

A total of six (6) drilled boreholes using hollow stem auger was put down for the purpose of design analysis for roadway protection system at Blanche River Bridge. Three (3) boreholes were advanced at each end of the bridge and drilled up to 20 m depth from the existing ground surface. The number of boreholes, depths and locations of boreholes were chosen according to the given minimum specification in Request for Proposal (RFP) issued by MTO. Borehole locations and stratigraphic sections are shown on the Borehole Location Plans, (Drawings 1 to 3).

The borehole stationing numbers are shown in the Table 3.1. The ground surface elevations at the boreholes locations were surveyed by DST personnel. A station selected on the bridge at the southeastern corner of the sidewalk was assigned as temporary benchmark with elevation of 100.0 m (Drawing 1). Subsequently, this benchmark elevation was surveyed by Geniver's survey team. Boreholes elevations were referred to the benchmark elevations surveyed by Genivar. Table 3.1 summarizes the detail of borehole locations and depths.

Table 3.1 Detail of borehole locations

Borehole ID	Station	Borehole Top Elevation (m)	Depth (m)	Offset (m)
BH1	17+121	183.8	20.1	4.5 Rt
BH2	17+132	183.5	20.1	4.5 Rt
BH3	17+132	183.5	20.1	4.5 Lt
BH4	17+250	183.5	20.1	4.5 Rt
BH5	17+250	183.5	20.0	4.5 Lt
BH6	17+262	183.5	20.0	4.5 Rt

The fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, performed sampling, in-situ testing and logged the boreholes. Standard penetration testing (SPT) were performed in the boreholes. Undisturbed soil samples were collected using Shelby tubes

for laboratory consolidation tests. The soil samples collected during drilling were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analyses.

Classification and index tests were subsequently performed in the laboratory on samples collected from the boreholes to aid in the selection of engineering properties. Laboratory tests included moisture contents, particle size analyses and Atterberg limits including plastic and liquid limits. Laboratory one-dimensional consolidation testing was carried out on the selected undisturbed soil samples. A total of ninety three (93) moisture contents, eleven (11) sieve analyses, eight (8) particle size analyses (hydrometer test), twelve (12) Atterberg limit tests and three (3) consolidation tests have been carried out for this assignment. Laboratory test results are presented in the boreholes Logs (Enclosures 1 to 6), and Plots (Enclosures 7 to 11).

#### **4. DESCRIPTION OF SUBSURFACE CONDITIONS**

The subsurface conditions are presented based on the information obtained during field and laboratory testing.

The generalized stratigraphy of the existing embankment and underlying soils, based on the conditions encountered in boreholes, consists of surfacing (hot mix asphalt) overlying a fill of sand and crushed gravel (base layer), underlain by sand fill (sub-base layer) which is underlain again by clay and sand layers before encountering a thick varved clay layer. During drilling, cobble sized rock fragments were encountered in the fill sand (sub-base layer) and hindered for drilling work.

Boreholes 1, 2 and 3 are drilled at the western end of the bridge and Boreholes 4, 5 and 6 are drilled at the eastern end of the bridge.

##### **4.1 Asphalt**

Asphalt was encountered in the boreholes drilled on the embankment. The thickness of the asphalt was approximately 70 to 90 mm.

##### **4.2 Embankment Fill**

The upper fill material (base layer) was identified below the asphalt as “sand and crushed gravel”, based on the main fractions of the material. The thickness of the sand and gravel layer was about 0.1 m. Gradation analysis conducted on samples from Borehole 5 indicated gravel, sand, and fine contents of approximately 27 %, 70 % and 3 % respectively and it meets the gradation requirement for Granular A.

The lower fill material (sub-base layer), thickness of about 3.8 m to 6.7 m, was encountered in all boreholes. SPT values range from 5 to >100 blows/ 300mm and indicate the compactness condition as loose to very dense. The moisture contents varied from 2 to 25 %. Gradation analyses conducted on samples from boreholes indicated gravel, sand and fines contents of approximately 0 – 34 %, 58 - 97 % and 1 – 29 % respectively. According to the granular gradations, the material identified in the embankment fill can be classified as “Granular B Type 1” (SP110S13, Table 2) except at Borehole 2 and 3 which contain higher fines contents. Some cobbles, clay and wood were found in the fill layer during the drilling work. The cobbles are inferred from drilling observations and there is a possibility that the observations could also indicate boulders.

Grain size distributions of the fill material are reported in borehole logs (Enclosures 1 to 6) and plots (Enclosures 7 and 8).

#### **4.3     Clay and Sand Interbedded layers**

Interbedded layers of clay and sand were indentified in Borehole 1, 2, 3, and 5. The thickness of individual clay and sand layers range from 1 to 5 m.

In the sand layer, SPT values range from 3 to >100 and can be classified as very loose to very dense sand. Gradation analyses conducted on sand samples from Borehole 1 and 4 indicate gravel, sand, and fines contents of approximately 0 – 4 %, 87 – 88 % and 9 - 12 % respectively. Moisture content of tested sand samples from Boreholes 1, 3, 4 and 5 resulted in the range of 17 – 27 %.

Clay was found firm to very stiff in consistency. Gradation analyses conducted on clay samples from Boreholes 1, 2 and 3 indicate sand, silt and clay contents of approximately 29 - 38 %, 40 - 52 % and 19 - 21 % respectively. Atterberg limits tests results indicate low plasticity with liquid limit and plasticity index of 25 – 33 % and 11 – 13 %. Moisture contents of tested clay samples from Boreholes 1, 2, 3, and 4 range from 23 % to 31 %.

Pieces of wood within a sand layer was found in Borehole 5 and it was approximately 1.4 m in thick. Organic material was found in Boreholes 6 and its thickness was approximately 0.2 m.

#### **4.4     Varved Clay**

Varved clay was found in all boreholes at a depth of approximately 4.2 m to 10.7 m. No non-plastic varves were apparent. Varved clay is firm to very stiff in consistency based on field vane tests. The stratum extends to the borehole termination depth, which is approximately 20 m below the existing ground level. Atterberg limits results indicate intermediate to high plasticity with liquid limit and plasticity index of 45 – 63 % and 24 – 38 %. Gradation analyses conducted on samples from boreholes indicated silt and clay contents of approximately 25 – 45 % and 55 – 75 % respectively. Moisture contents of samples ranged from 21 - 80 %.

The results of field vane tests are recorded on the borehole logs (Boreholes 1 to 6, drilled through the embankment), and typically in the range of 25 to 50 kPa. The upper zone of the clay is stiff to very stiff representing an overconsolidated crust, and it is likely that this zone has a fissured structure.

Atterberg limits test and grain size distribution test results are reported in boreholes logs (Enclosures 1 to 6) and plots (Enclosure 7- 11). Particle size distribution test results are tabulated in Table 4.3.

Three (3) oedometer consolidation tests were carried out. Test results are reported in Enclosure 12 - 17. Following consolidation parameters are estimated from the tests, which were sampled to represent clay varves as much as possible.

Table 4.1 Oedometer consolidation tests results

Borehole No.	Sample Depth, m	Preconsolidation Pressure, kPa	Compression Coefficient, C <sub>c</sub>	Recompression Coefficient, C <sub>r</sub>
BH3	10.7	118	0.356	0.044
BH4	10.7	180	1.620	0.072
BH4	18.3	210	1.101	0.079

Table 4.2 Descriptive Terms used for Soil Type Classification

Descriptive Term	Example	Percent by Mass of Sample
And (with two major soil types)	Sand and Gravel	40 – 60
Adjective (--ty)	Silty	30 – 40
With	Silt with fine sand	20 – 30
Some	Silt, some fine sand	10 – 20
Trace	Sand, trace of gravel	0 – 10

Table 4.3 Particle size distribution test results

Borehole No.	Sample Depth, m	Soil Description	Grain Sizes, %			
			Gravel	Sand	Silt	Clay
BH1	6.10	Silt, sandy with clay	0	38	41	21
BH1	7.60	Sand, some silt and clay	0	88	12	
BH1	18.30	Clay and silt	0	0	44	56
BH2	5.30	Sand, some silt and clay	0	82	18	
BH2	7.60	Silt, sandy, with clay	0	37	40	23
BH3	3.60	Sand, with silt and clay, trace gravel	2	77	21	
BH3	4.60	Sand, with silt and clay	0	71	29	
BH3	6.10	Silt, with sand, some clay	0	29	52	19
BH3	12.20	Clay, silty	0	0	37	63
BH4	0.75	Sand, gravelly, trace silt and clay	32	58	10	
BH4	2.30	Sand, some gravel and trace silt and clay	20	71	9	
BH4	7.60	Sand, trace silt, clay and gravel	4	87	9	
BH4	12.20	Clay, silty	0	0	34	66
BH5	0.20	Sand, with gravel, trace silt and clay	27	70	3	
BH5	3.80	Sand, trace silt, clay and gravel	2	91	7	



Borehole No.	Sample Depth, m	Soil Description	Grain Sizes, %			
			Gravel	Sand	Silt	Clay
BH5	12.20	Clay, with silt	0	0	25	75
BH6	2.30	Sand, gravelly, trace silt and clay	34	59	7	
BH6	3.10	Sand, trace gravel, silt and clay	2	97	1	
BH6	16.80	Clay and silt	0	0	45	55

#### 4.5 Groundwater

The groundwater table was identified during the field investigation. Five (5) temporary water standpipes were installed in the drilled boreholes and monitored for a few hours. The estimated depth of groundwater table at the time of investigation as estimated from the water standpipe data and soil observations is given Table 4.2. The groundwater table at the site can be expected to vary with season and precipitation events as well as river levels.

Table 4.4 Probable depth of water table at boreholes

Borehole ID	Borehole elevation (m)	Estimated ground water table elevation (m)	Measured highest ground water table elevation (m)
BH1	183.8	178.8*	-
BH2	183.5	180.5	180.5
BH3	183.5	182.3	182.3
BH4	183.5	179.3	179.3
BH5	183.5	179.4	179.4
BH6	183.5	179.3	179.3

\*based on observation during drilling. Actual level is likely above river level.

The high and low river levels recorded on the drawings (dated March 1959) are 181 m and 178.3 m respectively. At the time of investigation, the level was recorded at 179.0 m. The groundwater table near the abutments is expected to be slightly above and affected by the river level.

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**PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS**

**5. PROJECT DESCRIPTION**

DST Consulting Engineers Inc. (DST) has been subcontracted by Genivar which was retained by Ministry of Transportation (MTO), Northeastern Region, to conduct a geotechnical investigation for the rehabilitation of Blanche River Bridge, Highway 65, Township of Casey, New Liskeard Area.

The site, Blanche River Bridge is located on Highway 65, about 19.2 km east from the junction of Highway 11B, Township of Casey, New Liskeard Area. The structural site number is 47-047.

The purpose of this report is to assess the suitability of the subsurface conditions to provide temporary support during rehabilitation of Blanche River Bridge. The nature of the temporary works is understood to be vertical excavations near the abutments. Details of the works are yet unknown.

Subsurface conditions are reported in Part 1 of this report. Detailed design and selection of appropriate geotechnical parameters will be the responsibility of the contractor's design engineer. For the purpose of this report, it is assumed (as provided by prime Service Provider Genivar) that excavations will not extend more than 2 m below the base of the existing road surface, or approximately elevation 181.5 m

**5.1 Roadway Protection System**

The roadway protection systems (shoring system) are required for the excavation work of bridge rehabilitation where conditions may not be feasible for cut slopes. The road protection system should be designed based on the excavation depth and geometry, soil/groundwater conditions, and benefits of the protection system, meeting safety requirements and conformance with OPSS 539 (Special Provision No. 105S19, March 2005), Canadian Highway Bridge Design Code (CHBDC), and the Ontario Occupational Health and Safety Act.

In the following sections, suitable temporary protection systems for road work are discussed. Also discussed, including in subsequent sections, are advantages/disadvantages, deformations with

respect to performance levels, general design guidelines and construction considerations, anchor support requirements and monitoring requirements. Conceptual shoring locations and performance level recommendations are outside of DST's scope of work.

#### 5.1.1 Temporary Protection Systems

Practical options for temporary roadway protection systems are sheet piles walls and H piles with lagging walls. Both of these are effective in a variety of soil and groundwater conditions, and are routinely used for excavations, and there are experienced contractors available to install these. Subsurface conditions at this site are suitable for both of these, albeit with limitations as discussed below. Schematic diagrams of temporary protection systems are shown in Drawing 4.

Sheet pile walls allow vertical excavation with small deformation of the earth structure, the magnitude which depends on the design and construction details. At this site, for excavation extending below the groundwater table, it is likely that the sheet piles can extend into the clay to form an effective seal for groundwater control. Difficulties with installation may occur where occasional boulders are encountered in the fill, requiring their removal before further driving.

Alternatively, an H-pile with lagging wall can be used as a vertical temporary shoring system. The H-piles are installed and lagging is inserted between installed H-piles during excavation. This requires the soil to stand unsupported for a time, and therefore may not prove feasible for granular soils below the groundwater table. Pile resistance for deeper excavations is often increased by installing the piles into bored and grouted holes. Space between the excavation and lagging must be suitably backfilled and drained. Lagging wall material can be selected as wood (timber), steel or concrete. The advantage of this system is it will allow for increasing or decreasing the depth of lagging walls.

Supports will typically be required where the shoring supports more than 2 or 3 m of soil. This can be accomplished with strut or raker supports (supported either against other structures or the ground) or tie-backs (to dead men or soil anchors).

If the shoring system is to be removed, measures must be implemented to ensure that any resulting voids are adequately filled without detrimentally loosening the retained or backfilled soil in order to meet performance level requirements. Similarly, the method must not result in loose zones of soil that would not meet performance requirements.

### 5.1.2 Requirement for the Protection System Design and Construction

Design and construction specification for a roadway protection system should be prepared in accordance with OPSS 539 (special provision no. 105S19, March 2005). Descriptions of major points of the standard OPSS 539 are shown below.

The contractor is required to submit three (3) copies of working drawings to the contract administrator at least one (1) week before commencement of construction of protection system. Submission should bear seals and signatures of the Design Engineer and Design Checking Engineer. The contractor should have a copy of the stamped working drawings at the site during protection system construction. Working drawings should include plans, elevations, design details, design criteria, materials, installation procedures, monitoring method, removal of protection system.

Design engineer and design checking engineer should have a minimum of five (5) years experience in designing protection systems of a similar nature and scope to the required work. Design checking engineer should not be same person as design engineer. Supervisory personal of protection system construction should have experience in the method of constructing protection systems within the preceding five (5) years on projects of similar nature and scope to the required work. The quality verification engineer should have a minimum five (5) years of experience in the design of comparable protection systems, or alternatively with demonstrated expertise through providing satisfactory quality verification services for a minimum of two (2) projects in which the work was of similar scope to that in the contract.

Excavation depth less than or equal to 3 metres and surcharge loading (vehicular traffic, construction equipment, materials, etc.) is beyond a horizontal distance defined by a 1H : 2V line projected from the dredge line at the face of the protection system to the roadway surface, contractor should submit to the contract administrator a certificate of conformance sealed and signed by the quality verification engineer following the installation of protection system to the dredge line.

If excavation depth is less than three (3) metres and surcharge traffic load is within a horizontal distance defined by a 1H : 2V line projected from the dredge line at the face of the protection system to the roadway surface, or excavation depth exceeding three (3) metres, the contractor should submit to the contract administrator a certificate of conformance sealed and signed by the quality verification engineer upon completion of each of the following operations, prior to commencement of each subsequent operation:

- 1) Layout and extent of protection system
- 2) Piling
- 3) Installation of protection system including exaction to dredge line
- 4) Removal and management (in accordance with OPSS 180 and as specified in the contract)

Upon completion of the operation of a protection system and removal of the system, the contractor should submit to the contract administrator a final certificate of conformance sealed and signed by the quality verification engineer. The certificates of conformance should state that the materials and work have been supplied and installed in general conformance with the working drawings.

If there are amendments to the protection systems, work should not proceed until the contractor has received, sealed and signed an approval to proceed from the original design engineer and design checking engineer and has submitted a copy of the approval to the contract administrator. Amendments to the protection system should be submitted to the contract administrator on revised working drawings/details bearing the seal and signature of the original design engineer and design checking engineer.

Prior to commencing the work, the contractor should submit to the contract administrator, a condition survey of property and structures that may be affected by the work. The survey should include, but not be limited to, the locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as walls abutting the site within a horizontal distance of  $2H_w$  from the face of the protection system, where  $H_w$  is the height of the wall from the ground surface to the dredge line.

The protection system should be designed for the performance level specified in the contract document. If performance level is not specified in the contract, protection system should be assigned an appropriate performance level for design by the design engineer. Contract administrator should review the performance level selected at the time of submission of the specified working drawings. The contractor should be responsible for the complete detailed design of the protection system needed to fulfil the requirements specified in the contract drawings.



The performance levels for protection systems are as follows;

Performance Level	Maximum Angular Distortion	Maximum Horizontal Displacement
1a	1 : 1000	5 mm
1b	1 : 1000	10 mm
2	1 : 200	25 mm
3	1 : 100	50 mm

Where : Angular Distortion =  $\pm \Delta/H$

$\Delta$  = Horizontal displacement (mm) at height H

H = Height (mm) above dredge line to point of measurement or height above the nearest system restraining support.

When Performance Level 1a is specified, the bracing system should be preloaded. The effects of the preload should not cause damage to adjacent facilities. The protection systems with a face within a horizontal distance of  $1/3 H$  of any part of a structure foundation should be designed for Performance Level 1a.

The geotechnical/foundation portion of the design should be based on a method published in AASHTO Guide Design Specification for Bridge Temporary Works and should be in general conformance with the CAN/CSA-S6-00 Canadian Highway Bridge Design Code (CHBDC). Design method not meeting this design specification may be used on a particular contract only if prequalified by the Owner. A protection system should be designed to provide protection for excavations as required by the Ontario Occupational Health and Safety Act, at the locations specified in the contract, and at any other location where the stability, safety or function of an existing structure and/or utility may be impaired by construction work.

Design assumptions should accurately represent the subsurface conditions prevalent at the site, and should be specific to the type of protection system used. The design should address the subsurface conditions at the project site reported in the Foundation Investigation Report.

Vertical and horizontal design loadings used should represent existing conditions and be accepted

design practice. Future loadings that are known and may affect the protection system during its useful life should be considered.

Material used should meet OPSS, ISO/IEC specifications. Wood should be accordance with OPSS 1601, concrete should be accordance with OPSS 1350. Structural steel used should be tested at the Canadian National Accreditation Body accredited laboratory. Design engineer may consider using other suitable materials when sufficient information is available to quantify the allowable design loads or when the manufacturer's recommendations as to load carrying capacities are supported by test results from an independent organization accredited by the Standards Council of Canada.

When used proprietary shoring and patented shoring system or accessories, the contractor should follow the manufacturers' recommendations for load carrying capacity. The recommended load carrying capacities should be supported by test results from an accredited testing laboratory approved by the Owner.

Protection systems should be built according to the specifications and the stamped working drawings. Concrete construction should be accordance with OPSS 904. Structural steel construction should be accordance with OPSS 906. Piling should be accordance with OPSS 903. Prestressed anchors should be supplied, installed and stressed according to the contract documents. The protection system should be protected from the detrimental effects of rain and frost action. Loss of soil from behind the shoring should be prevented during and following the installation of the lagging. The contractor should carry out dewatering as required to facilitate the installation of the protection system. Concrete should be placed in the dry unless otherwise specified in the contract. Where cofferdams are used, they should be sealed sufficiently to permit concrete to be placed in the dry. When concrete cannot be placed in the dry condition, tremie techniques should be employed according to OPSS 904.

Protection systems should be removed from the right-of-way unless otherwise specified in the contract that the protection system may be left in place. The contractor should obtain approval from the Ministry of Environment and other approving authorities when all or any portion of the protection system is to be left in place. Where piles are left in place, the top should be removed to at least 1.2m below the finished grade or ground level or at least 0.6 m below the stream bed. Method and sequence of removal should not damage new work, existing work and the facility being protected. The area remaining disturbed after removal of the protection system should be restored to as close to its original condition as possible.

During Blanche river bridge rehabilitation, road way protection will be required to retain one lane traffic for excavating to expose top of the bridge abutments. Excavation of maximum 2 m below road surface is expected for the bridge rehabilitation. The excavation will be close to the abutment foundation. Performance Level 1b would be reasonable if applied to movements of the pavement surface only, however this may be insufficient for structures or buried facilities for which performance levels should be established by the design engineers.

### 5.1.3 Geotechnical Parameters for Protection Systems Design Analysis

In designing the protection system, stability and lateral deformation of protection systems should be analysed with the site geotechnical condition. In this section, geotechnical design parameters are discussed.

At the Blanche River bridge site, there are four major soil layers. The layers are fill layer, clay layer, sand layer and varved clay layer. Fill is overlying interlayers of sand and clay before encountering the varved clay layer. The thickness of fill layer is 4.0 m to 6.9 m, clay layer is 0.6 m to 3.4 m, sand layer is 1 m to 1.6 m and varved clay layer is > 9.4 m. Soil conditions at the East and West ends of the approach embankments are varied. Cobble or rock boulders are encountered in the fill layer. Cobble and rock boulders in the fill layer may interfere in the piling or excavation works.

Varved clay layer is firm to very stiff in consistency. Most of the undrained shear strengths tested in the varved clay layer fall between 25 kPa to 60 kPa. Soft soil condition may be expected at the depth of 4.2 m to 10.7 m (elevation 179.3 to 172.8). Protection system design should be considered for this soft soil condition. For shoring system design analysis, designer should refer to the boreholes logs for detailed sub-surface conditions at the location.

Typical geotechnical design parameters of soil layers are shown in Table 5.1. Whereas the parameters are suitable for typical design applications, design-specific adjustments may be appropriate depending on such factors as performance level and construction method. The design engineer should select appropriate parameters for the final design. Undrained shear strength were obtained from the field vane shear tests directly. Drained internal friction angles for the granular soils were estimated from standard penetration tests applying empirical correlations proposed by Wolff (1989). Drained internal friction angle of cohesive soil was estimated from the plasticity index applying empirical correlations (Bjerrum and Simons, 1960; Kenney, 1959). The consolidation test results provide deformation characteristics for certain zones of the clay.

Table 5.1 Recommended soil parameters for protection system analysis

Soil Type	Thickness, m	Unit Weight, $\text{kN/m}^3$	Undrained Shear Strength, kPa	Drained Internal Frictional angle, $\phi$
Fill (Sand with Some Cobble & Boulder)	4.0 – 6.9	20 – 23 (21)	-	28 – >50 (28)
Clay	0.6 – 3.4	17 – 18 (18)	88 – 110 (88)	30 – 32 (30)
Sand	1.0 – 1.6	20 – 23 (21)	-	28 – >47 (28)
Varved Clay	> 9.4	17 – 18 (17)	25 – 72 (25)	23 – 30 (23)

Value in brackets is recommended value.

#### 5.1.4 Guideline for Protection System Design Work

Design of protection system should meet the specified/required performance level and designed to provide protection for excavations as required by Occupational Health and Safety Act.

Geotechnical portion of the shoring system design should be based on a method published in AASHTO guide design specification for bridge temporary works and in general conformance with the CAN/CSA-S6-00 Canadian Highway Bridge Design Code (CHBDC).

Design assumptions should represent the subsurface conditions at the site. The design should address the subsurface conditions at the project site. Vertical and horizontal design loading used should represent the existing site conditions and accepted design practice. Future loadings on the protection system during design life should be included in the design loading used.

Materials used should meet OPSS, ISO/IEC specifications. Wood should be accordance with OPSS 1601, concrete should be accordance with OPSS 1350. Structural steel material should be tested at the Canadian National Accreditation Body accredited laboratory.

In designing a sheet pile protection system, required installation depths and sheet piles material specification should be analysed for the required protection. Lateral earth pressures, resistant to

should be analysed. Vertical and lateral deformation should be analysed and recommendations provided for a suitable monitoring program and maximum allowable deformations. Construction procedures of the protection system should be included in the design.

If designing an H-pile with lagging wall protection system, required H pile depth, size and wall material should be specified for the required protection. Lateral earth pressures should be analysed. Vertical and lateral deformation should be analysed and recommendations provided for a suitable monitoring program and maximum allowable deformations. Construction procedures of the protection system should be included in the design.

#### 5.1.5 General Construction Considerations

Prior to construction, a proposed protection system should be verified by the contract administrator for required protection. The contractor should submit a condition survey report of property and structures that may affect by the work to the contract administrator.

Protection system should be constructed in accordance with specifications provided by the protection system designer. Piling should be in accordance with OPSS 903. Concrete construction should be accordance with OPSS 904. Structural steel construction should be in accordance with OPSS 903. Loss of soil from behind the shoring should be protected against.

During protection system construction, the deformations of the shoring as well and nearby facilities to be protected should be monitored and compared to allowable deformation limits. Subsequent monitoring of protection system should be carried out with suitable frequency. In the event of deformation of the shoring system approaching the allowable limit, the contract administrator should be informed and remediation measures should be carried out immediately.

Construction of protection systems should be supervised by an experienced supervisory person. Quality verification engineer should submit certificate of conformance of protection system to contract administrator.

Traffic should be controlled during protection system construction. Constructed protection system should be checked by quality verification engineer before traffic is allowed.

Any loss of soil from behind the shoring should be prevented during installation of the protection system. If required, backfill should be used and compacted behind the shoring wall. The backfilling material could be Granular A material or as otherwise approved by the design engineer. The

protection system should be protected from the weather action (precipitation, frost, etc) as well as any water flows.

Any amendment in the protection system, construction should not proceed without approval from the contract administrator. Designer should certify the design amendments and submit to the contract administrator.

Temporary protection system should be removed from the right of way after construction work. If protection system is to be left in place, contractor should obtain approval from the Ministry of Environment and other approving authorities. Where piles are left in place, the top should be removed at least 1.2m below the finished grade or ground level or 0.6 m below the stream bed.

Method and sequence of removal should not damage new work, existing work or the facility being protected. Any area remaining disturbed after removal of the protection system should be restored to original condition.

#### 5.1.6 Monitoring

Excavation work at bridge rehabilitation should be monitored frequently for early warning of possible non-performance and the need for any remedial measures. The design engineer for temporary protection systems should propose monitoring instrument types, locations, monitoring frequency and allowable limits of deformations. The minimum monitoring requirement should be survey measurements of markers at shoring walls for horizontal displacements.

Monitoring of shoring system and ground movement should be done by a registered Ontario land surveyor or site engineer. All the test results, observations and records, including the construction survey taken during construction and operation of the protection system should be available on the site for review by the contract administrator. If the movement of protection system approaches the allowable limit, the engineer must notify the contract administrator immediately and suitable measures should be taken to ensure stability of the protection system and ensure movement does not exceed the performance level specified.

If excavation is less or equal than 3 metres in height, readings should be taken at the top of protection system. During installation of protection system, readings should be taken at each construction stages of the installation. After protection system installation, readings should be taken bi-weekly. More frequent reading may be required depending on performance level and risk.



If excavation is more than 3 metres, readings should be taken at the top of protection system, at each restrain point, at the dredge line and halfway between the restraint points. During installation of the protection system, readings should be taken at each construction stages of installation. After protection system installation, readings should be taken weekly. More frequent reading may be required depending on performance level and risk.

For Performance Level 1b of the road protection system, the maximum allowable horizontal displacement is less than 10 mm or 1% of vertical height whichever is lower.

## 6. REFERENCES

*Canadian Highway Bridge Design Code* (2006), CAN/CSA-S6-06, A National Standard of Canada, Canadian standards Association.

*Fundamentals of Geotechnical Engineering, Third Edition* (2008), Braja M. Das.

*Canadian Foundation Engineering Manual, 4<sup>th</sup> Edition, 2006*, Canadian Geotechnical Society.

*OPSS 539, November 2009, Construction Specification for Temporary Protection Systems*, Ontario Provincial Standard Specification.

*SSP105S19, Special Provision Number 539, November 2003*, Construction Specification for Protection Systems.

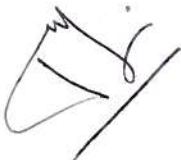
*Thomas F. Wolff (1995)*, Spreadsheet Applications in Geotechnical Engineering.

## 7. LIMITATION OF REPORT

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix 'A', and this forms an integral part of this report.

For DST CONSULTING ENGINEERS INC.

Prepared by:



Tun Lwin, P.Geo., M.Eng., M.Sc.  
Geotechnical Specialist

Reviewed by:



Dr. M W Bo, PhD., P. Eng, P.Geo, Int PE,  
C.Geol, C. Eng, Eur Geol, Eur Eng  
Senior Principal / Director (GeoServices)

Reviewed by:



Mike Fabius, P. Eng.  
Senior Principal

**APPENDIX 'A'**  
**LIMITATIONS OF REPORT**

# **LIMITATIONS OF REPORT**

## **GEOTECHNICAL STUDIES**

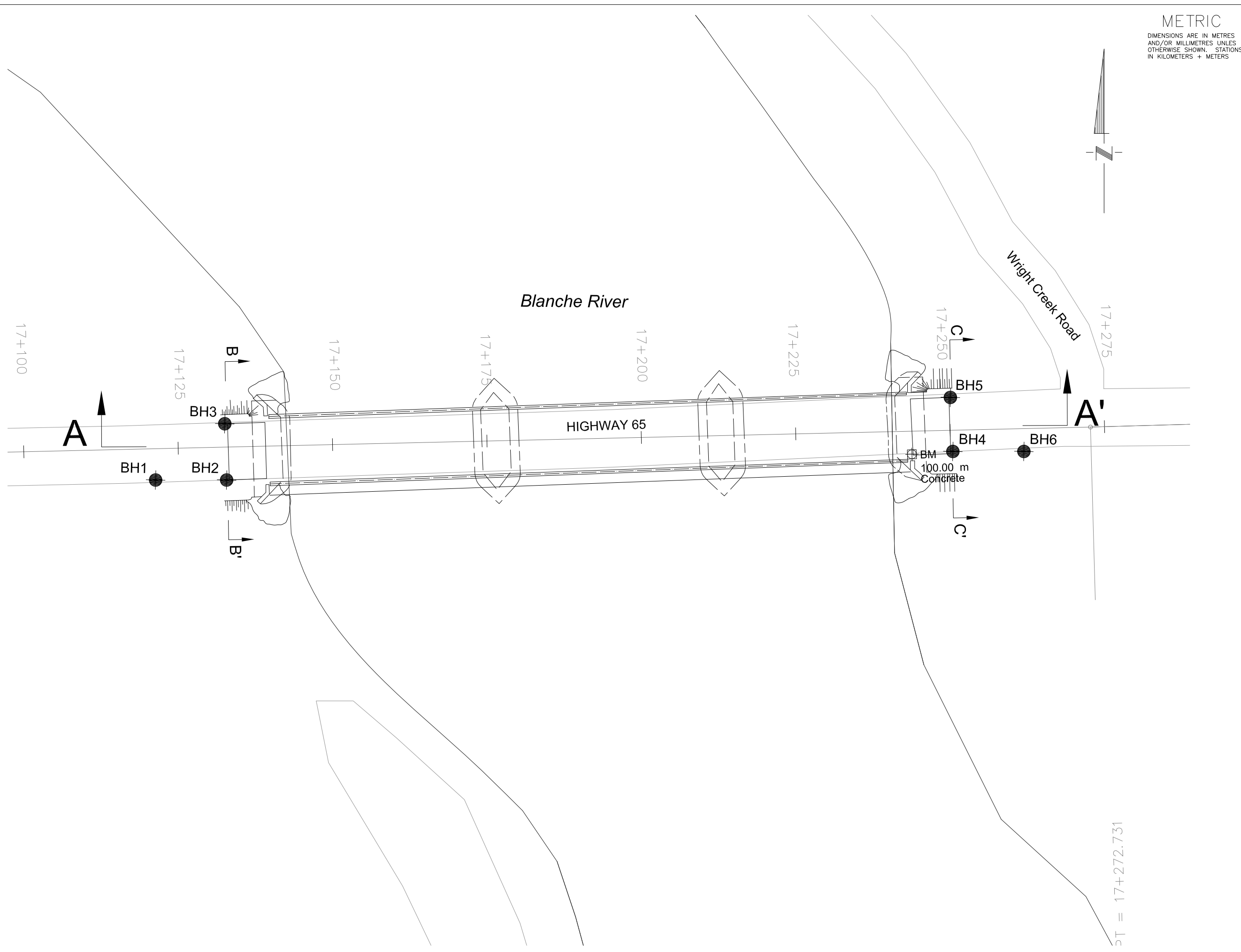
The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that DST Consulting Engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavation, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and DST Consulting Engineers Inc. cannot warranty their accuracy. Similarly, DST cannot warranty the accuracy of information supplied by the client.



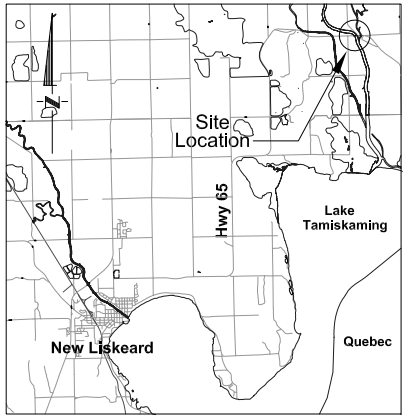
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DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLES  
OTHERWISE SHOWN. STATIONS  
IN KILOMETERS + METERS

CONT No 5009-E-0073  
WP No 166-98-00  
Site No 47-047  
Geocres No 31M-92



BRIDGE REHABILITATION  
AT BLANCHE RIVER  
Highway 65 – Casey Twp.  
Borehole Location Plan

SHEET



KEY PLAN  
0 12  
SCALE IN KILOMETRES

LEGEND

- Borehole/Hand Auger
- Borehole with DCPT
- Dynamic Cone Penetration Test (DCPT)
- Rock Probe
- Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Water level at time of Investigation.
- Benchmark

- |          |               |
|----------|---------------|
| Fill     | Sand          |
| Organics | Silt          |
| Topsoil  | Clay          |
| Till     | Sand & Gravel |
| Bedrock  | Boulders      |

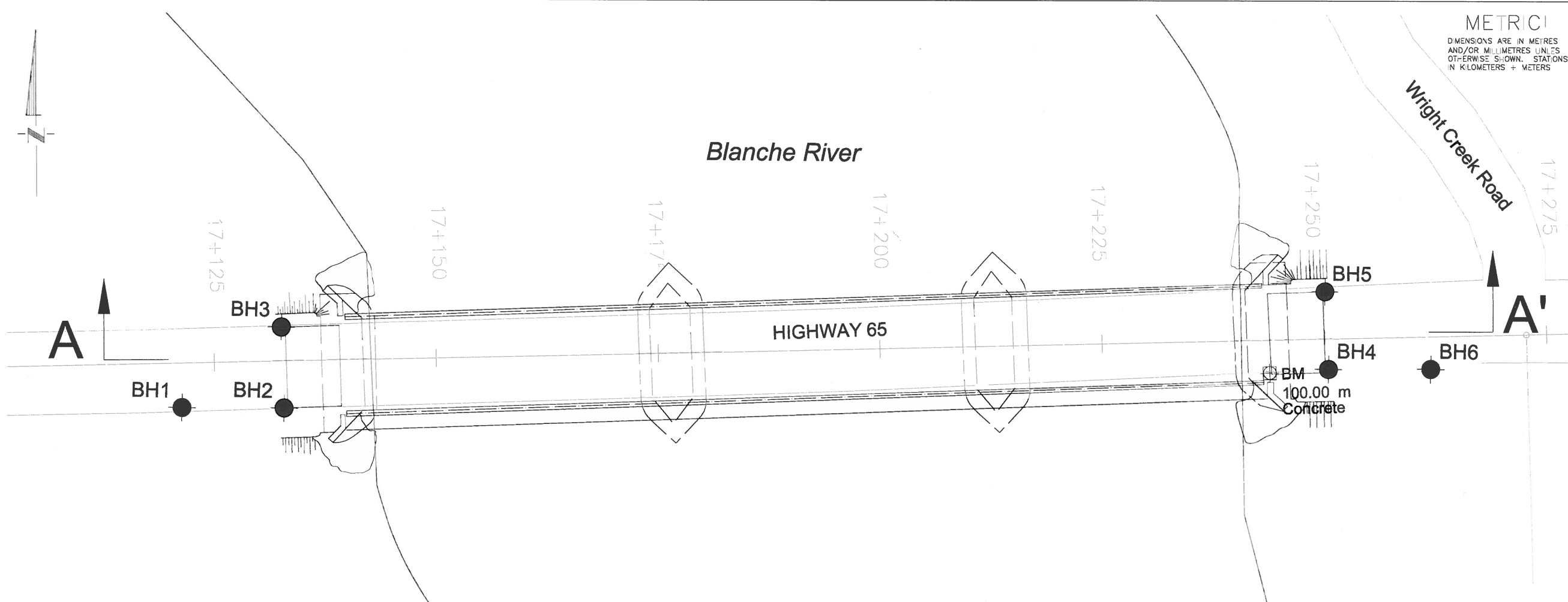
No.	Elevation	Northing	Easting	Station	Offset
BH1	183.763	5273961	609531	17+121	4.5 RT
BH2	183.523	5273961	609542	17+132	4.5 RT
BH3	183.533	5273970	609542	17+132	4.5 LT
BH4	183.453	5273966	609660	17+250	4.5 RT
BH5	183.463	5273975	609659	17+250	4.5 LT
BH6	183.458	5273966	609671	17+262	4.5 RT


NOTE:  
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

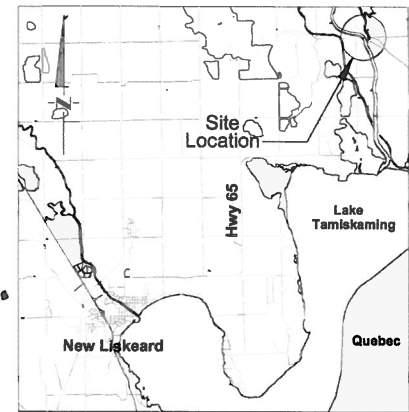
DST  
consulting engineers

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Fx: (807) 623-1792  
Email: thunderbay@dstgroup.com





CONT No	5009-E-0073	
GWP No		
WP No	166-98-00	
Site No	47-047	
BRIDGE REHABILITATION AT BLANCHE RIVER Highway 65 - Casey Twp. Borehole Locations & Stratigraphy		SHEET

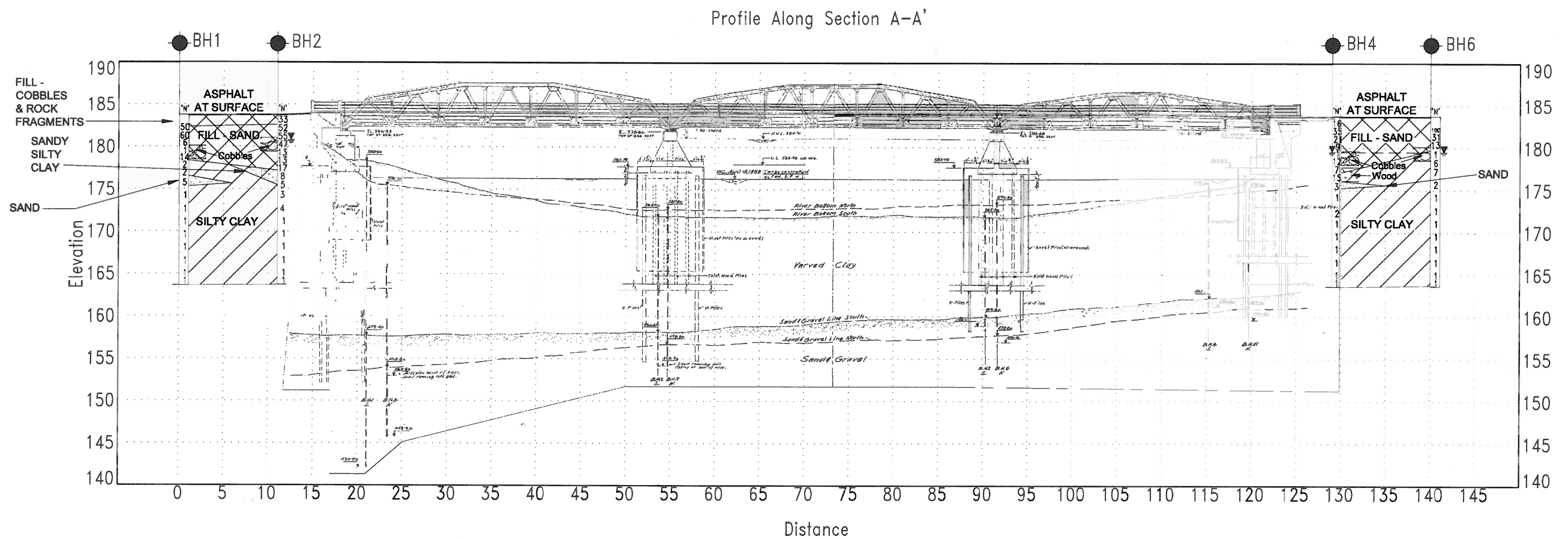


KEY PLAN  
0 12  
SCALE IN KILOMETRES

**LEGEND**

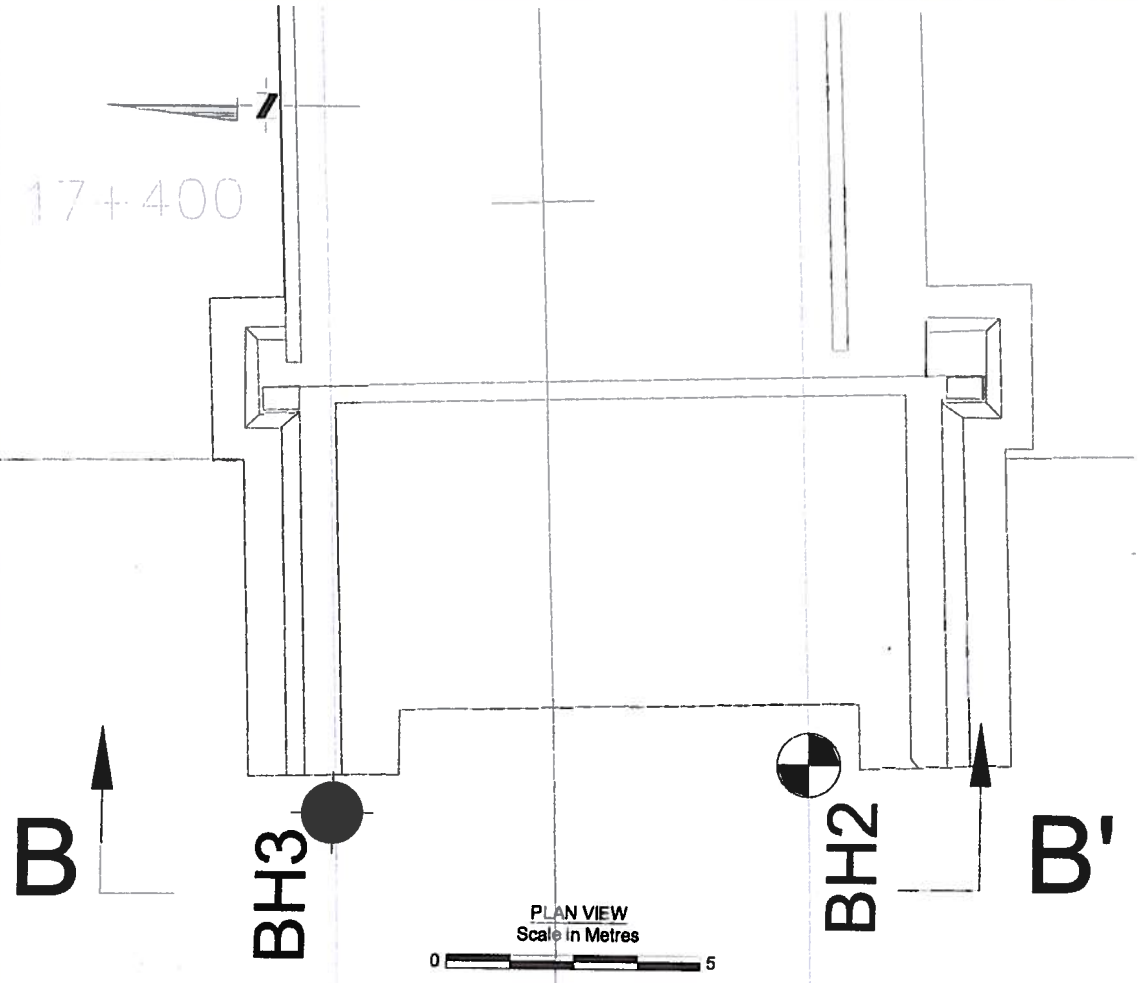
- Borehole/Hand Auger
- Borehole with DCPT
- Dynamic Cone Penetration Test (DCPT)
- Rock Probe
- Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Water level at time of investigation
- Benchmark
- Fill
- Organics
- Topsoil
- Till
- Bedrock
- Sand
- Silt
- Clay
- Sand & Gravel
- Boulders

No.	Elevation	Northing	Easting	Station	Offset
BH1	183.783	5273961	609531	17+121	4.5 RT
BH2	183.523	5273961	609542	17+132	4.5 RT
BH3	183.533	5273970	609542	17+132	4.5 LT
BH4	183.463	5273966	609660	17+250	4.5 RT
BH5	183.463	5273975	609650	17+250	4.5 LT
BH6	183.458	5273966	609671	17+262	4.5 RT

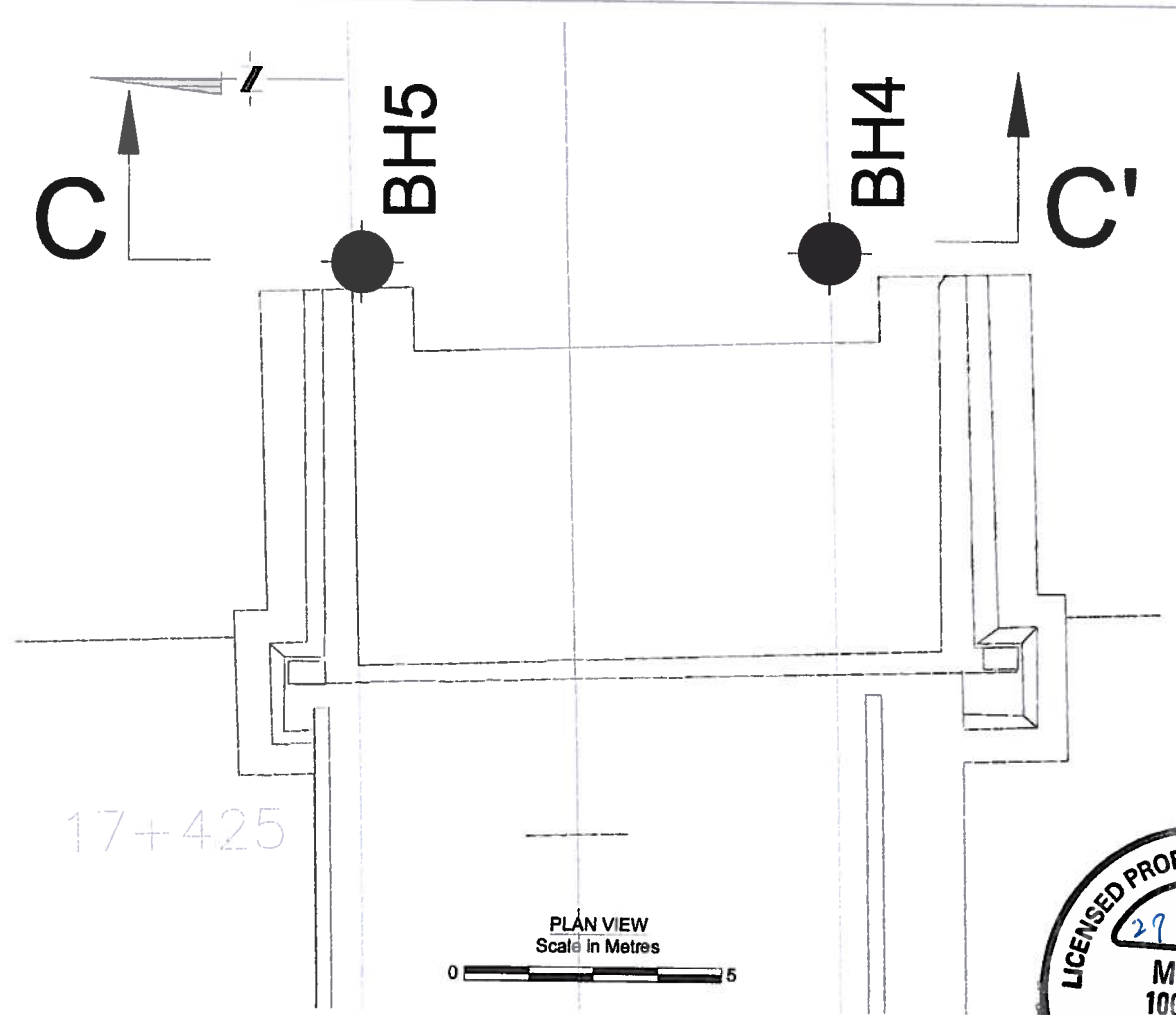
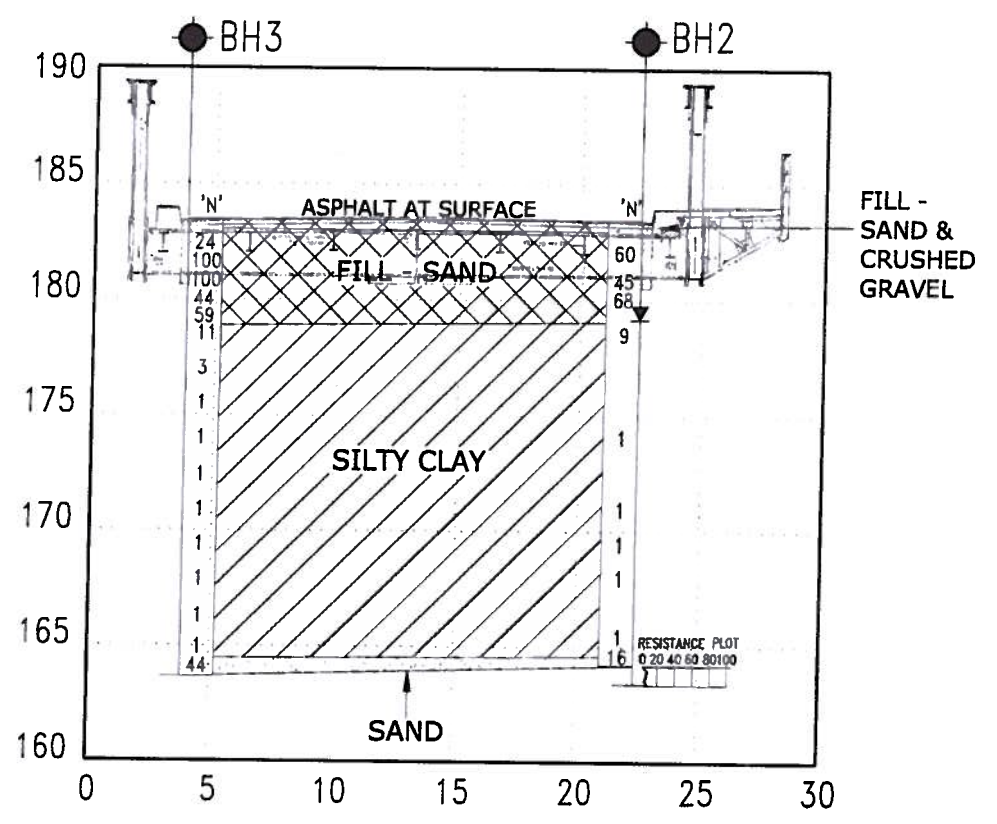


**NOTE:**  
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

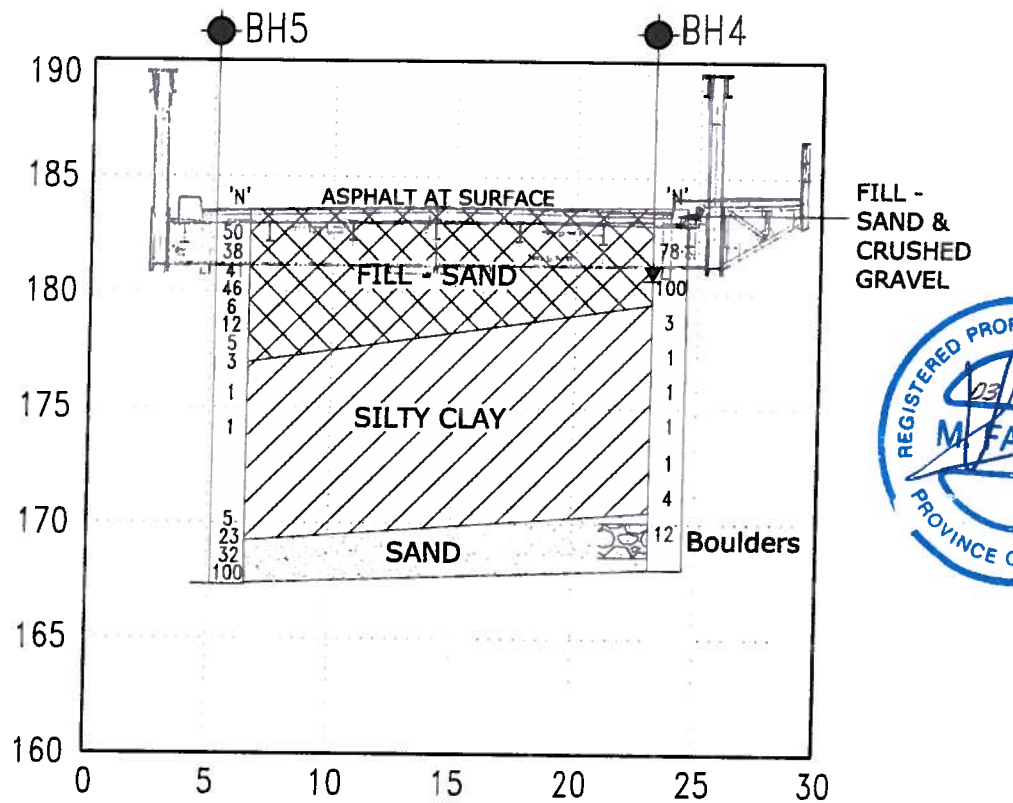

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 Thunder Bay, ON P7B 5V5  
 Ph: (807) 623-2929  
 Fax: (807) 623-1792  
 Email: thunderbay@dstgroup.com



Profile Along Section B-B'



Profile Along Section C-C'

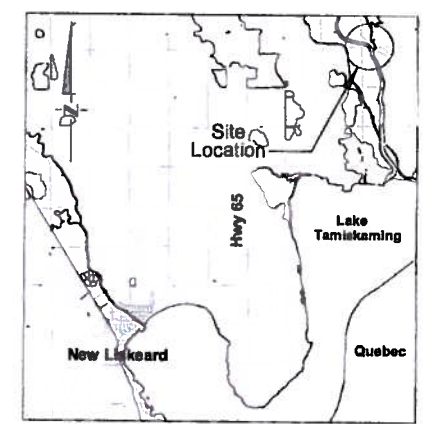


METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES & METERS

CONT No	5009-E-0073
WP No	166-98-00
Site No	47-048
Geocres No	31M-93

BRIDGE REHABILITATION  
AT WRIGHT CREEK  
Highway 65 - Casey Twp.  
Borehole Locations & Stratigraphy

SHEET



KEY PLAN  
0 12  
SCALE IN KILOMETRES



**LEGEND**

- Borehole/Hand Auger
- Borehole with DCPT
- Dynamic Cone Penetration Test (DCPT)
- Rock Probe
- Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Water level at time of investigation.
- Benchmark

Fill	Sand
Organics	Silt
Topsoil	Clay
Till	Sand & Gravel
Bedrock	Boulders

No.	Elevation	Northing	Easting	Station	Offset
BH1	183.474	5273886	609788	17+378	4.5 m RT
BH2	183.484	5273886	609798	17+388	4.5 m RT
BH3	183.424	5273877	609797	17+388	4.5 m LT
BH4	183.884	5273880	609846	17+436	4.5 m RT
BH5	183.514	5273876	609846	17+436	4.5 m LT
BH6	183.584	5273879	609856	17+447	4.5 m LT
BH7	179.094	5273881	609802	17+393	12.0 m RT
BH8	180.094	5273885	609802	17+393	12.0 m LT
BH9	182.234	5273882	609841	17+432	12.0 m RT
BH10	181.884	5273885	609840	17+432	12.0 m LT



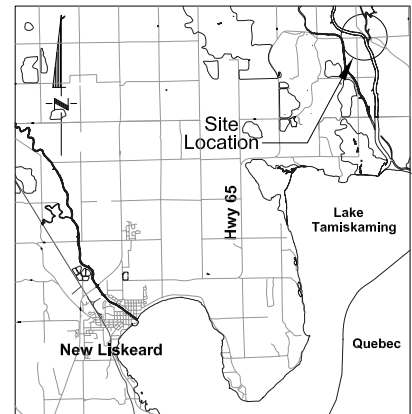
NOTE:  
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METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETERS + METERS

CONT No 5009-E-0073	
GWP No 5009-0000	
WP No 166-98-00	
Site No 47-047	
BRIDGE REHABILITATION AT BLANCHE RIVER Highway 65 – Casey Twp. SHORING WALL EXAMPLES	SHEET



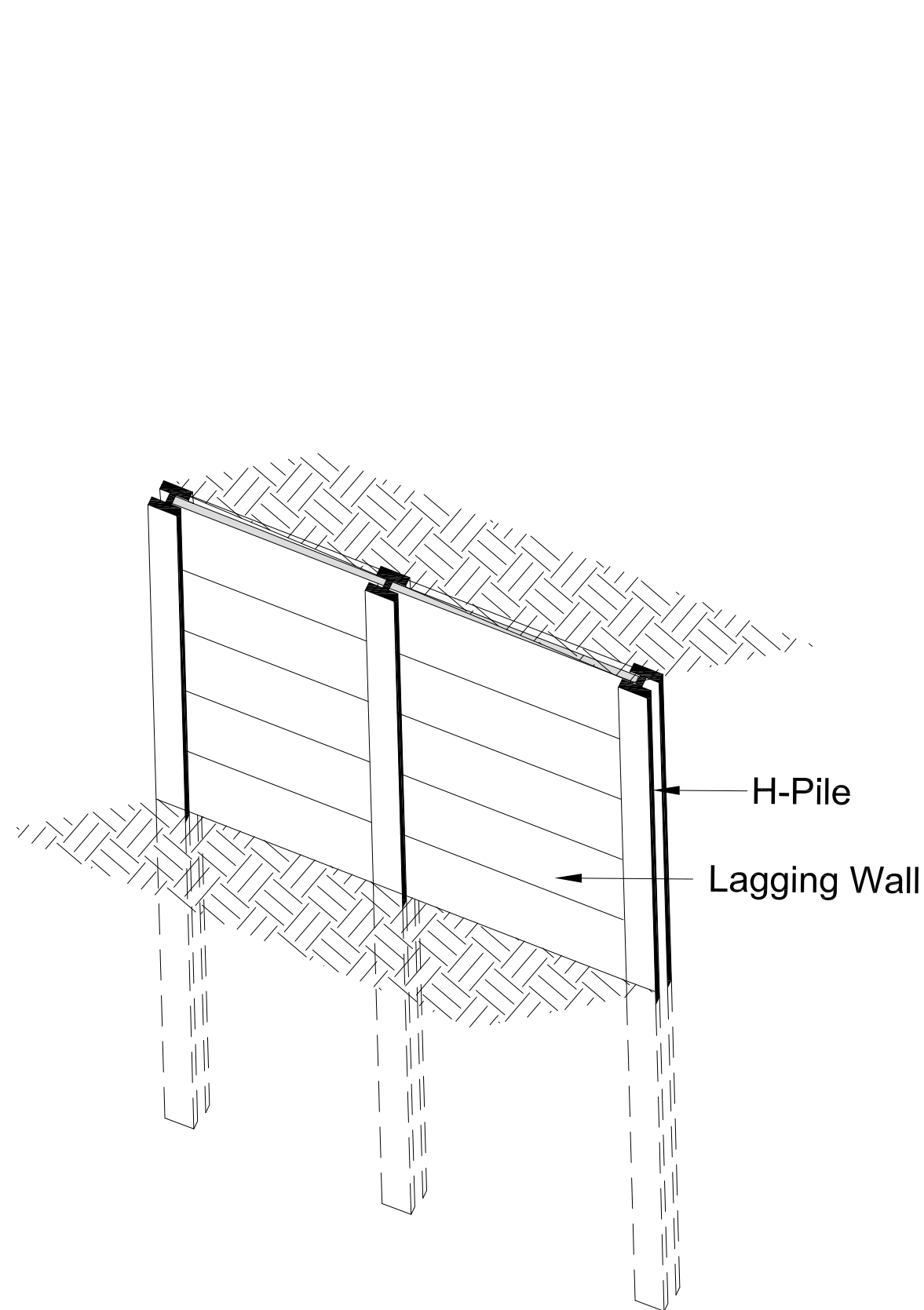
LEGEND					
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	Borehole with DCPT				
	Dynamic Cone Penetration Test (DCPT)				
	Rock Probe				
	Blows/0.3m (Std. Pen Test, 475 J/Blow)				
	Water level at time of investigation.				
	Benchmark				
	Fill		Sand		
	Organics		Silt		
	Topsoil		Clay		
	Till		Sand & Gravel		
	Bedrock		Boulders		
No.	Elevation	Northing	Easting	Station	Offset
BH1	183.763	5273961	609531	17+121	4.5 RT
BH2	183.523	5273961	609542	17+132	4.5 RT
BH3	183.533	5273970	609542	17+132	4.5 LT
BH4	183.453	5273966	609660	17+250	4.5 RT
BH5	183.463	5273975	609659	17+250	4.5 LT
BH6	183.458	5273966	609671	17+262	4.5 RT

NOTE:  
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

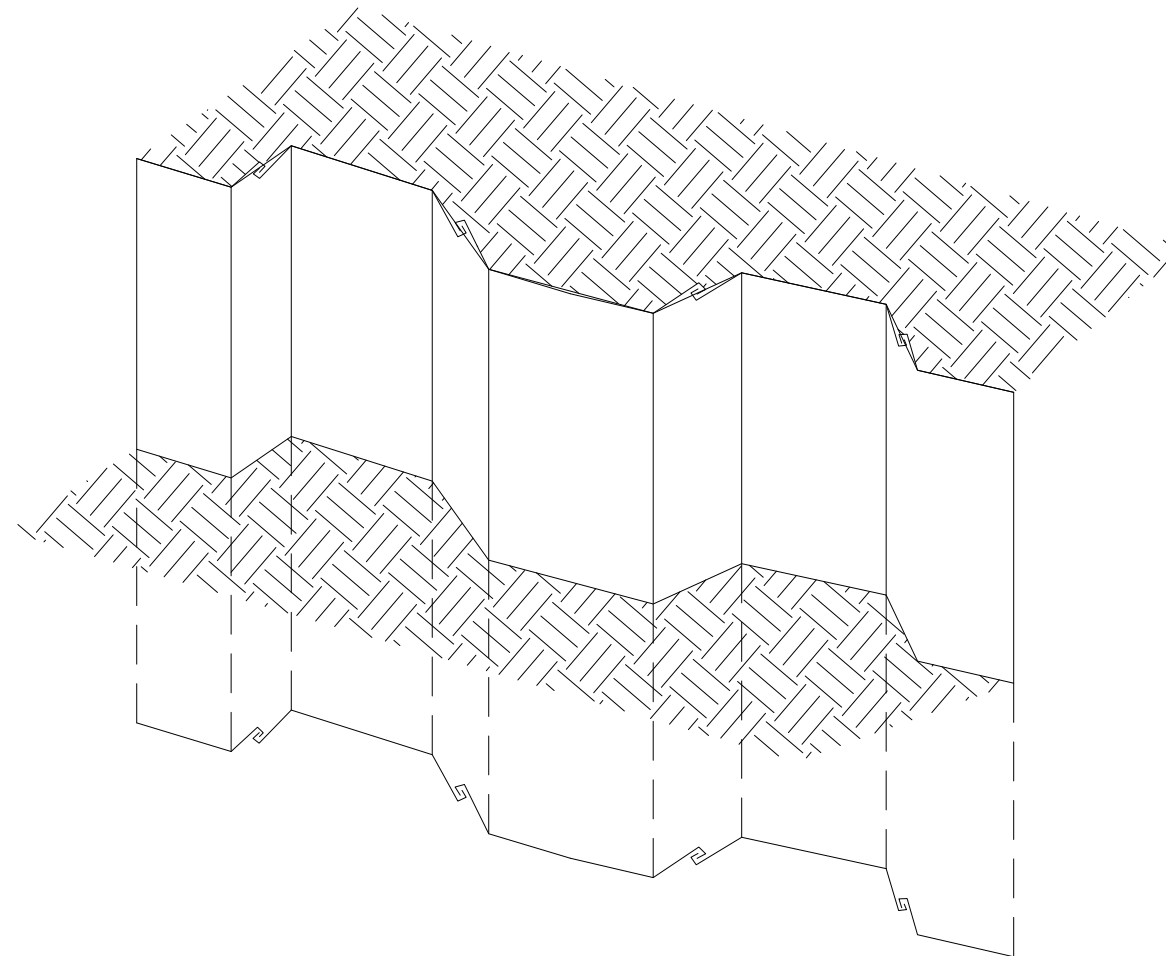


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Email: thunderbay@dstgroup.com



H-Pile with Lagging Shoring Wall



Sheet Pile Wall

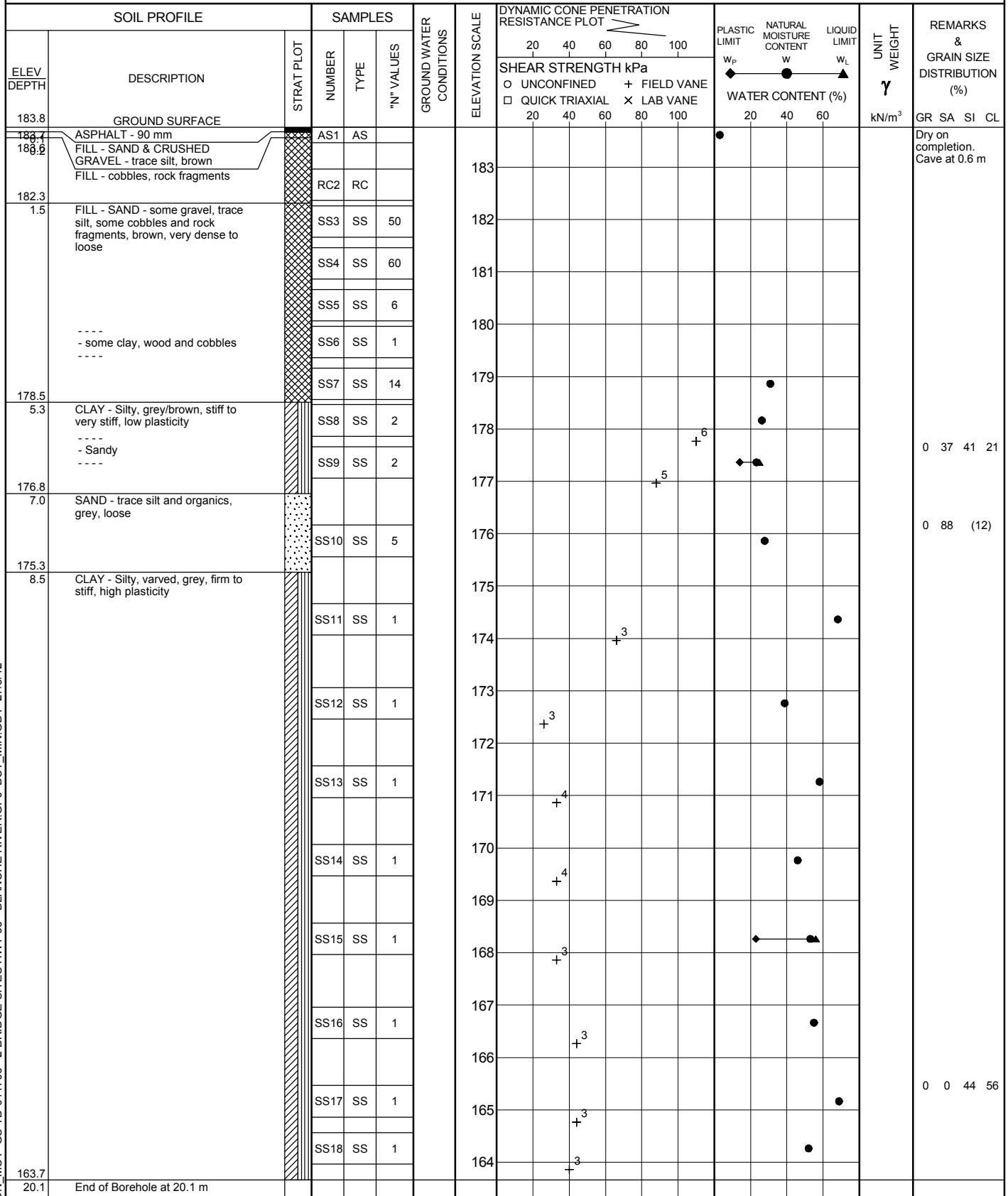
# **ENCLOSURES**

# RECORD OF BOREHOLE No BH1

1 OF 1

METRIC

W.P. # 5009-E-0073 LOCATION Blanche River: STA. 17+121, 4.5 m RT (5273961 m N, 609531 m E) ORIGINATED BY PR  
DIST HWY 65 BOREHOLE TYPE Hollow Stem Auger COMPILED BY ML  
DATUM NAD83 - UTM Zone DATE 2011 05 06 CHECKED BY TL/MWB



ON MOT CS-TB-011705 - 2 BRIDGE SITES HWY 65 - BLANCHE RIVER.GPJ DST\_MIN.GDT 27/3/12

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ENCLOSURE 1

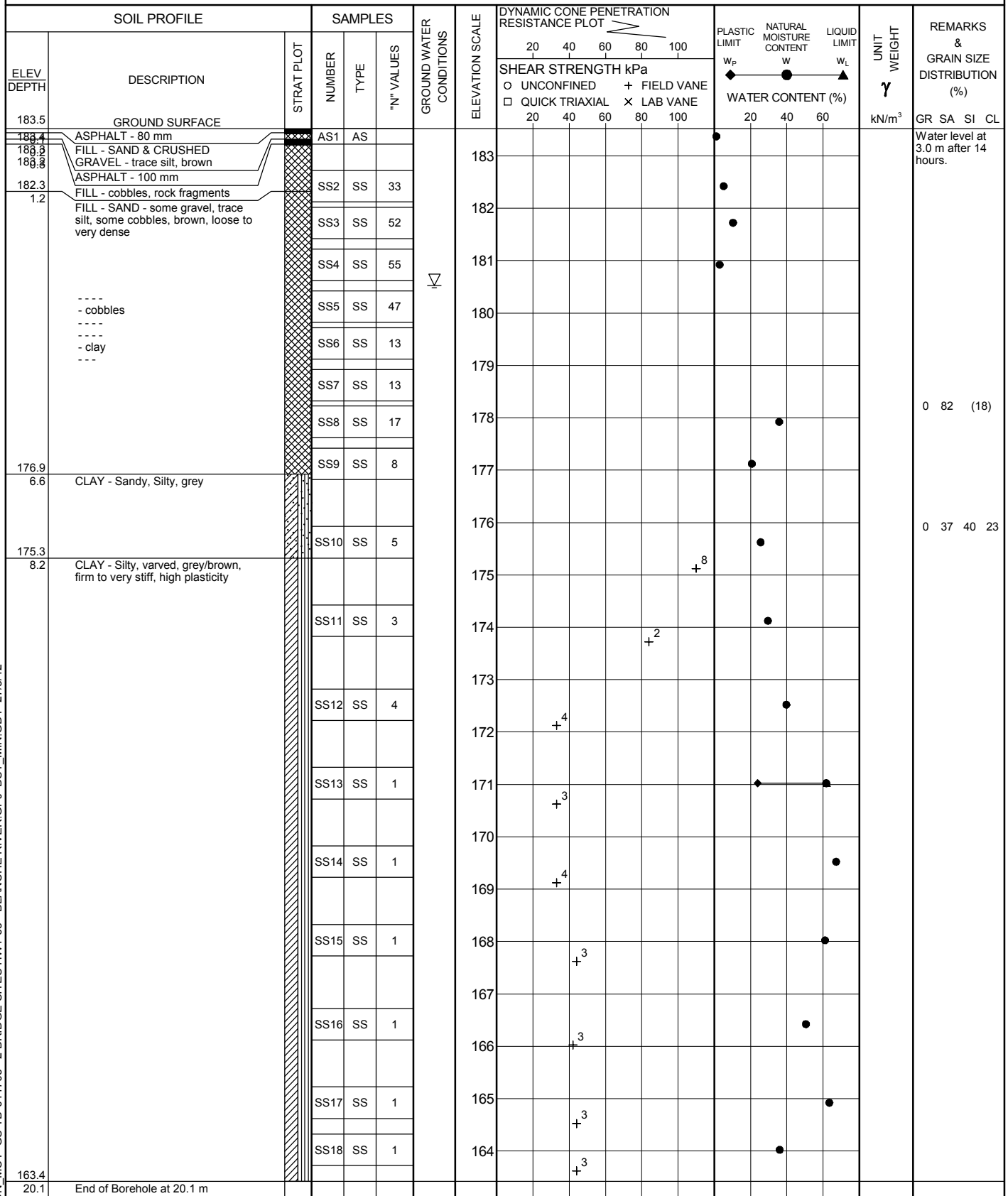


# RECORD OF BOREHOLE No BH2

1 OF 1

METRIC

W.P. # 5009-E-0073 LOCATION Blanche River: STA. 17+132, 4.5 m RT (5273961 m N, 609542 m E) ORIGINATED BY PR  
DIST HWY 65 BOREHOLE TYPE Hollow Stem Auger COMPILED BY ML  
DATUM NAD83 - UTM Zone DATE 2011 05 03 CHECKED BY TL/MWB



ON MOT CS-TB-011705 - 2 BRIDGE SITES HWY 65 - BLANCHE RIVER.GPJ DST\_MIN.GDT 27/3/12

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

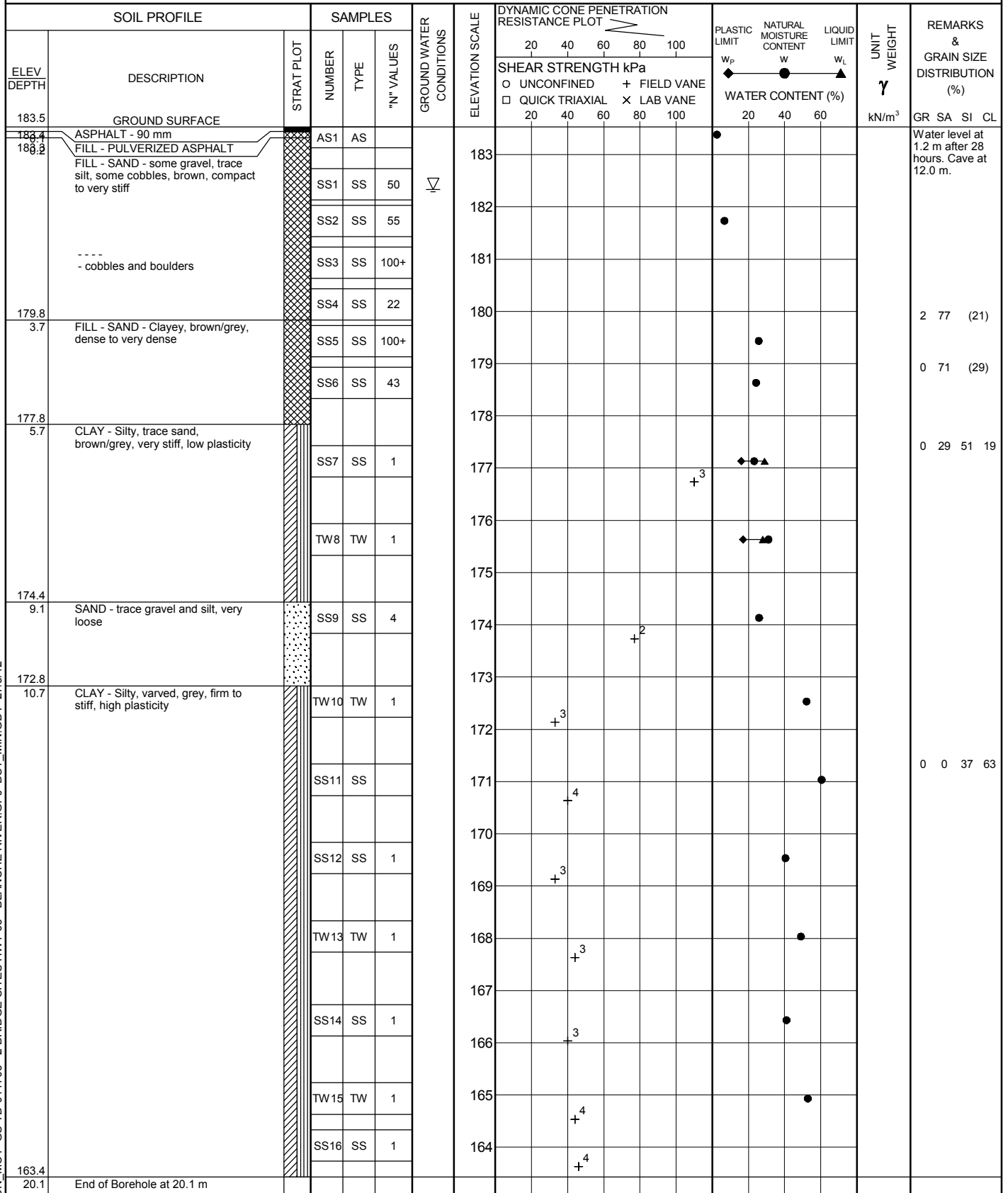
ENCLOSURE 2

# RECORD OF BOREHOLE No BH3

1 OF 1

METRIC

W.P. # 5009-E-0073 LOCATION Blanche River: STA. 17+132, 4.5 m LT (5273970 m N, 609542 m E) ORIGINATED BY PR  
DIST HWY 65 BOREHOLE TYPE Hollow Stem Auger COMPILED BY ML  
DATUM NAD83 - UTM Zone DATE 2011 05 04 CHECKED BY TL/MWB



+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

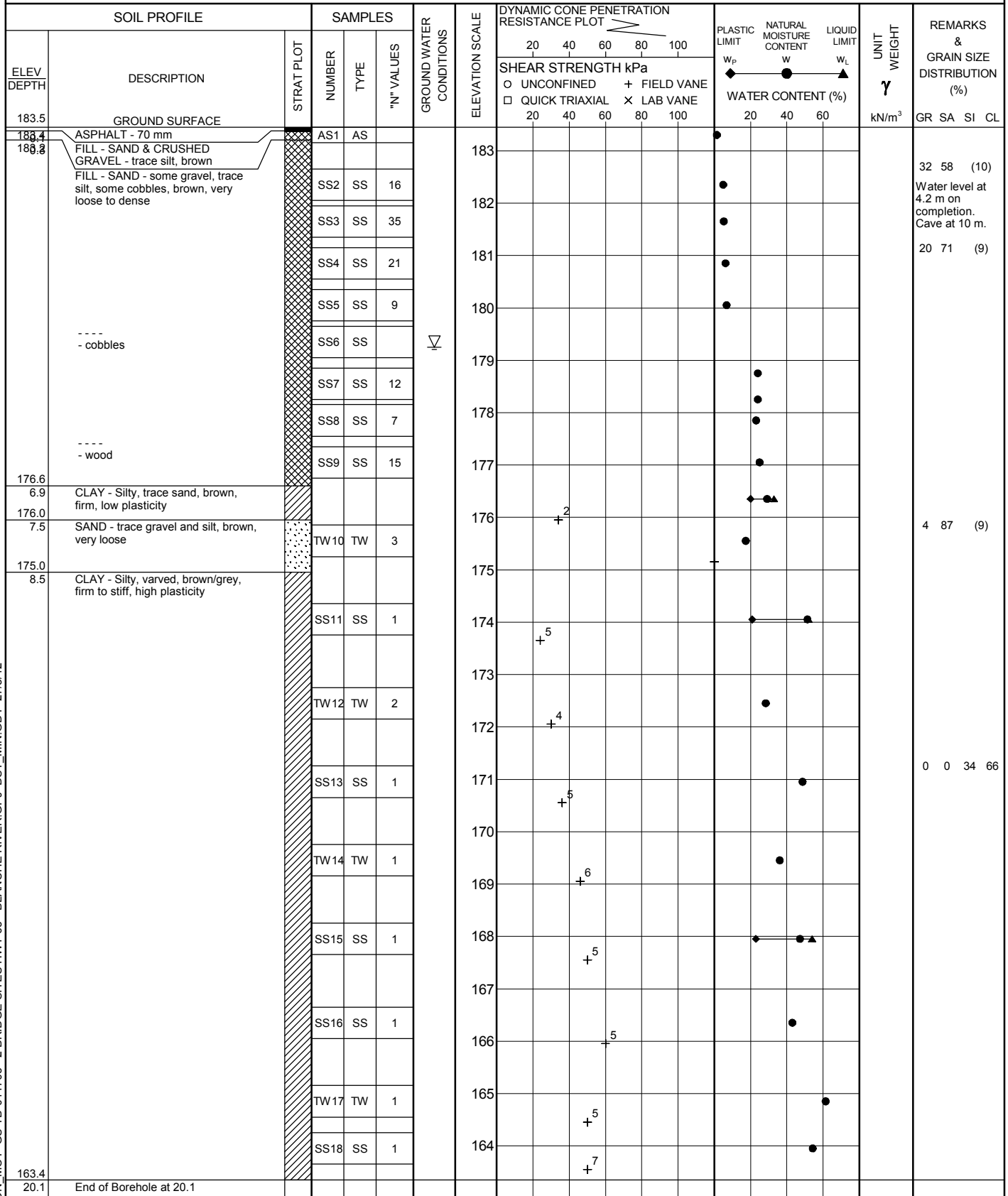
ENCLOSURE 3

# RECORD OF BOREHOLE No BH4

1 OF 1

METRIC

W.P. # 5009-E-0073 LOCATION Blanche River: STA. 17+250, 4.5 m RT (5273966 m N, 609660 m E) ORIGINATED BY PR  
DIST HWY 65 BOREHOLE TYPE Hollow Stem Auger COMPILED BY ML  
DATUM NAD83 - UTM Zone DATE 2011 05 01 CHECKED BY TL/MWB



ON MOT CS-TB-011705 - 2 BRIDGE SITES HWY 65 - BLANCHE RIVER.GPJ DST\_MIN.GDT 27/3/12

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ENCLOSURE 4

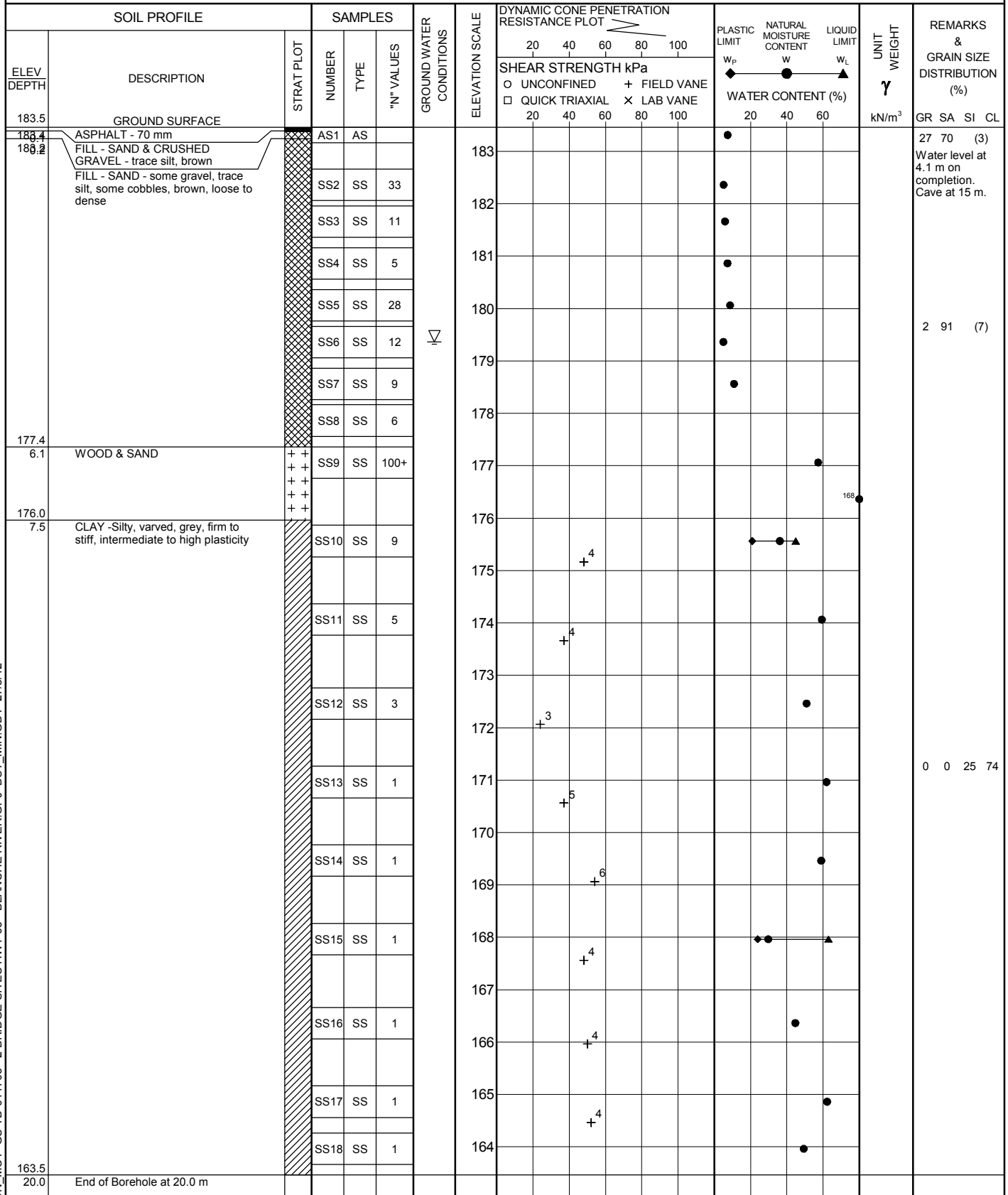


# RECORD OF BOREHOLE No BH5

1 OF 1

METRIC

W.P. # 5009-E-0073 LOCATION Blanche River: STA. 17+250, 4.5 m LT (5273975 m N, 609659 m E) ORIGINATED BY PR  
DIST HWY 65 BOREHOLE TYPE Hollow Stem Auger COMPILED BY ML  
DATUM NAD83 - UTM Zone DATE 2011 05 02 CHECKED BY TL/MWB



ON MOT GS-TB-011705 - 2 BRIDGE SITES HWY 65 - BLANCHE RIVER.GPJ DST\_MIN.GDT 27/3/12

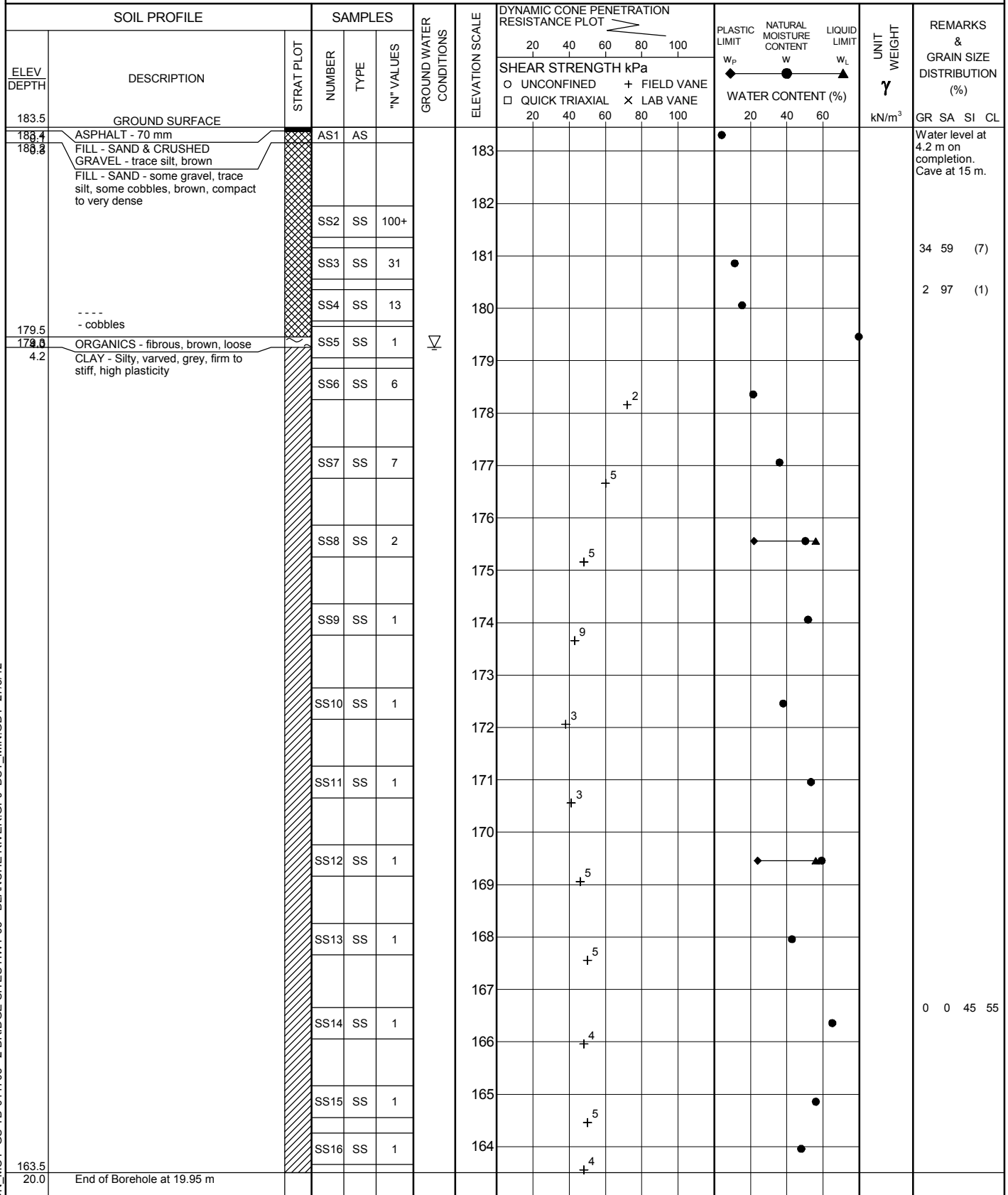
+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH6

1 OF 1

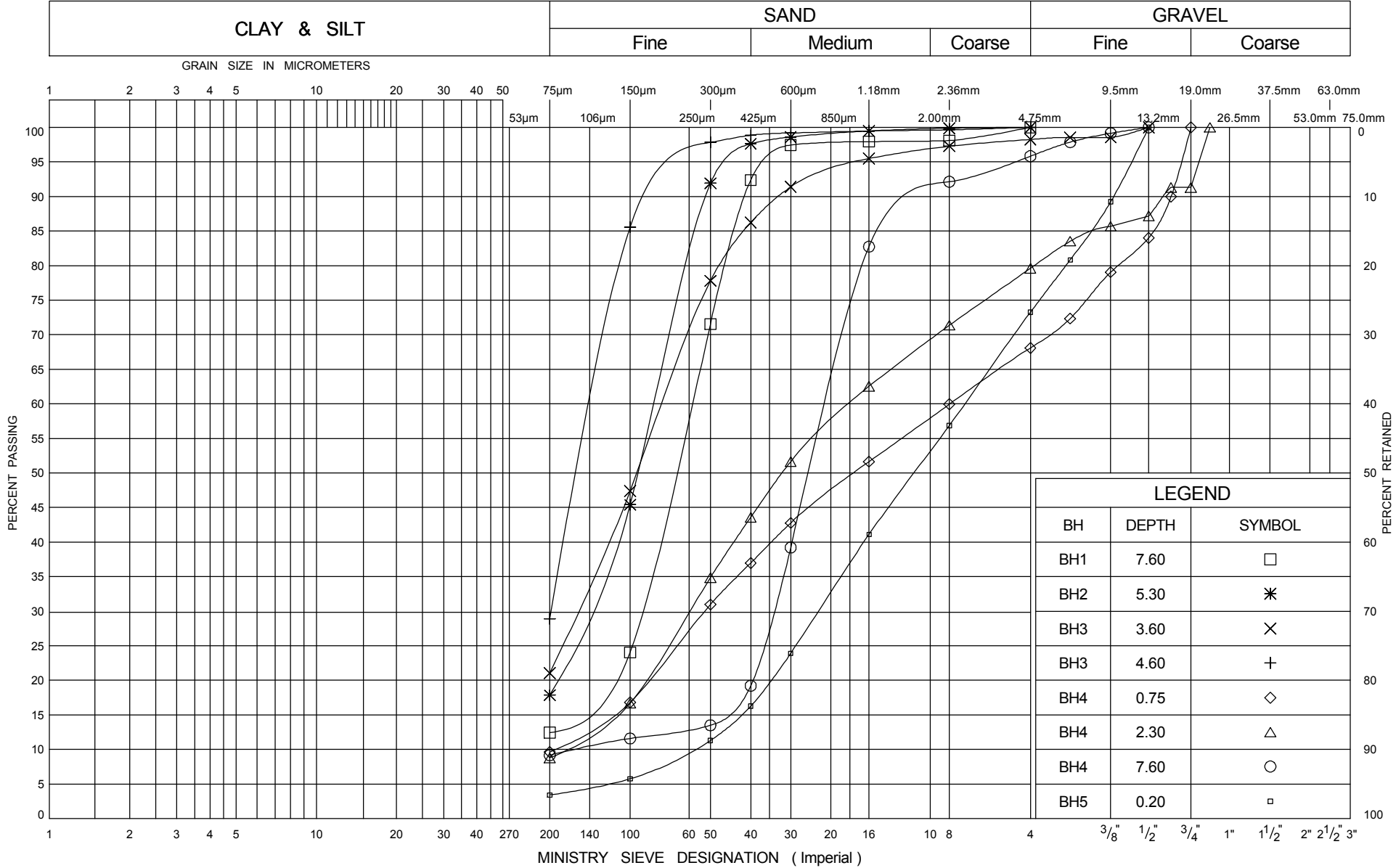
METRIC

W.P. # 5009-E-0073 LOCATION Blanche River: STA. 17+262, 4.5 m RT (5273966 m N, 609671 m E) ORIGINATED BY PR  
DIST HWY 65 BOREHOLE TYPE Hollow Stem Auger COMPILED BY ML  
DATUM NAD83 - UTM Zone DATE 2011 04 30 CHECKED BY TL/MWB



+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

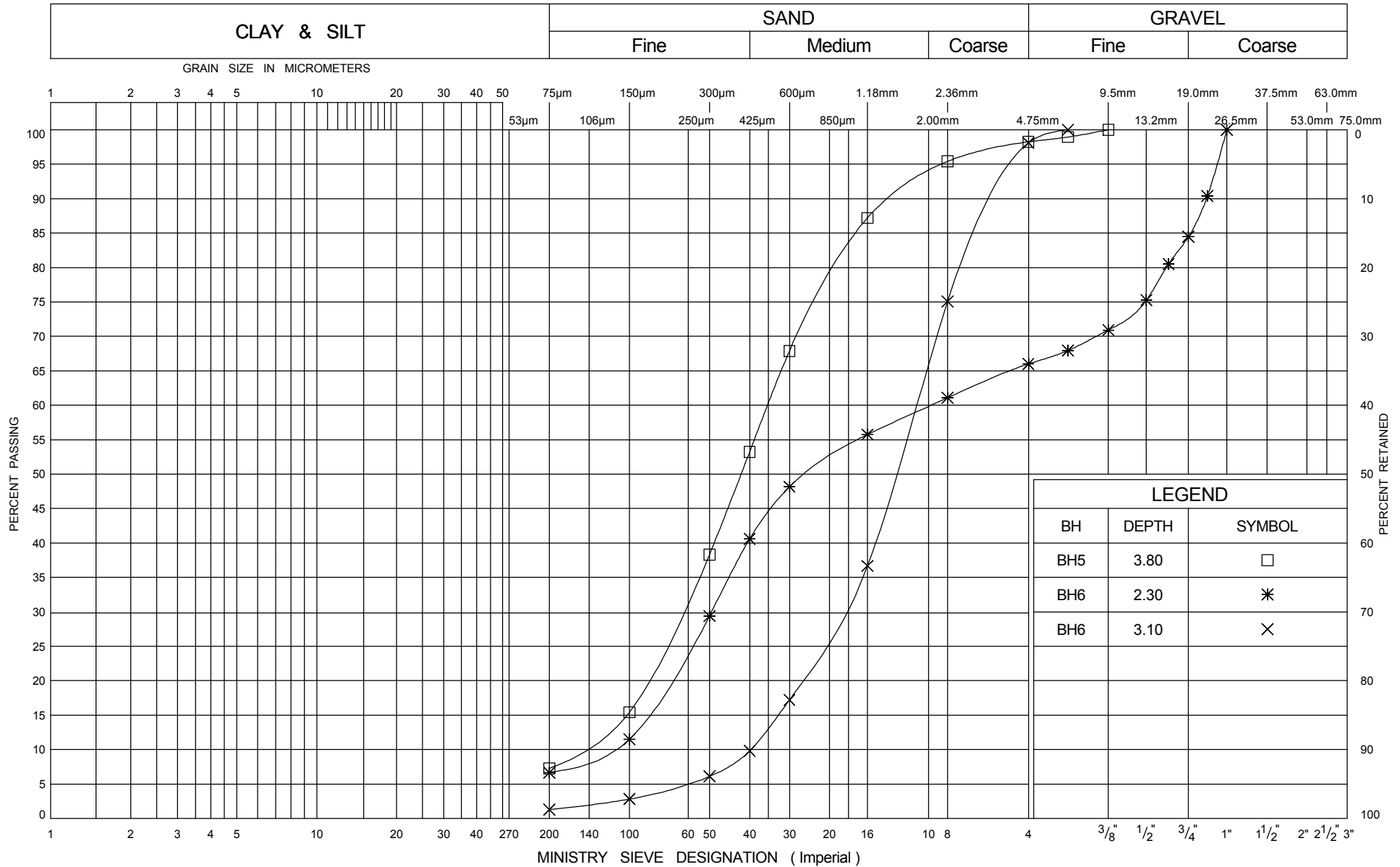
ENCLOSURE 7

W P # 5009-E-0073

HWY 65 - BLANCHE RIVER



# UNIFIED SOIL CLASSIFICATION SYSTEM



## GRAIN SIZE DISTRIBUTION

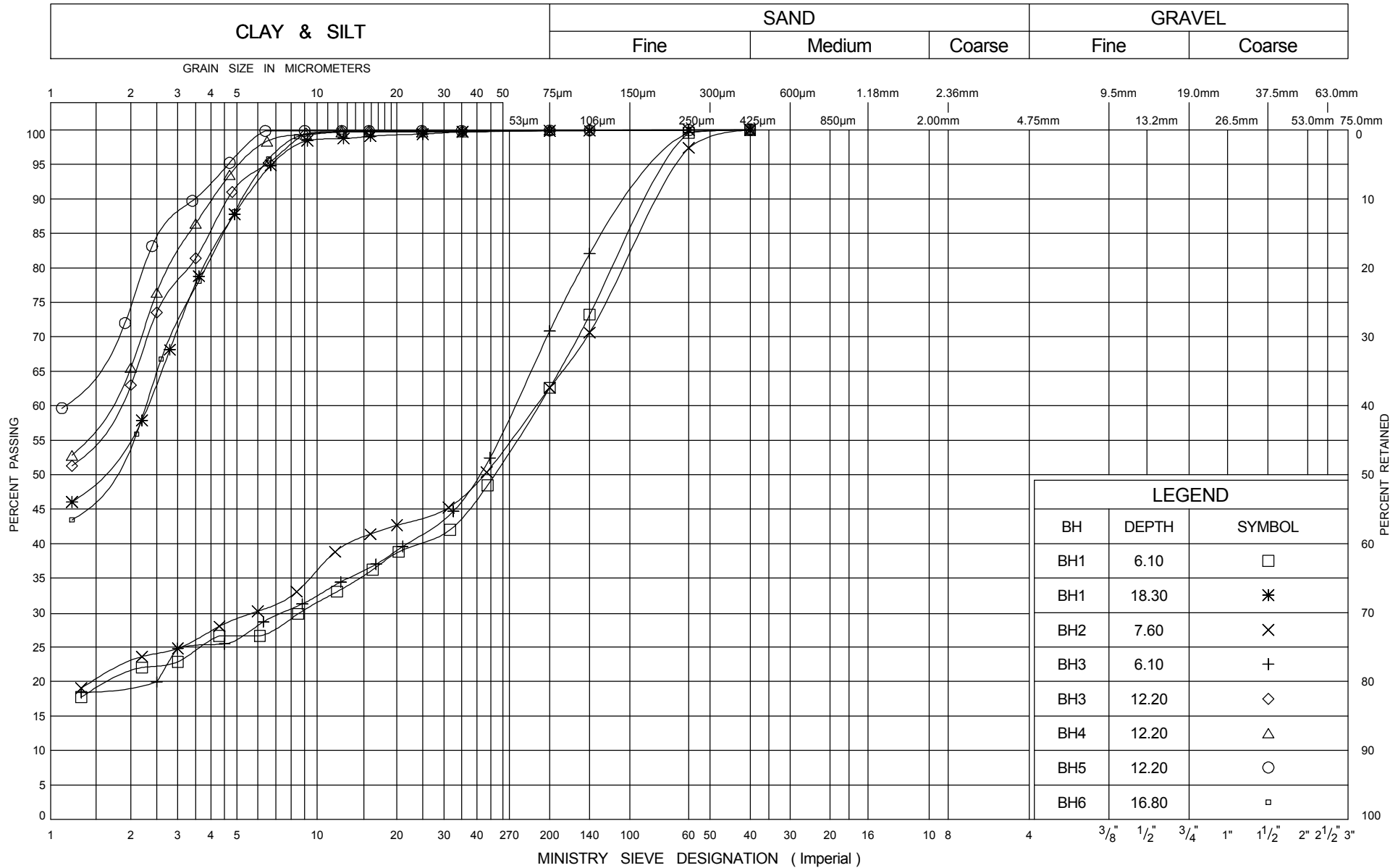
ENCLOSURE 8

W P # 5009-E-0073

HWY 65 - BLANCHE RIVER



# UNIFIED SOIL CLASSIFICATION SYSTEM



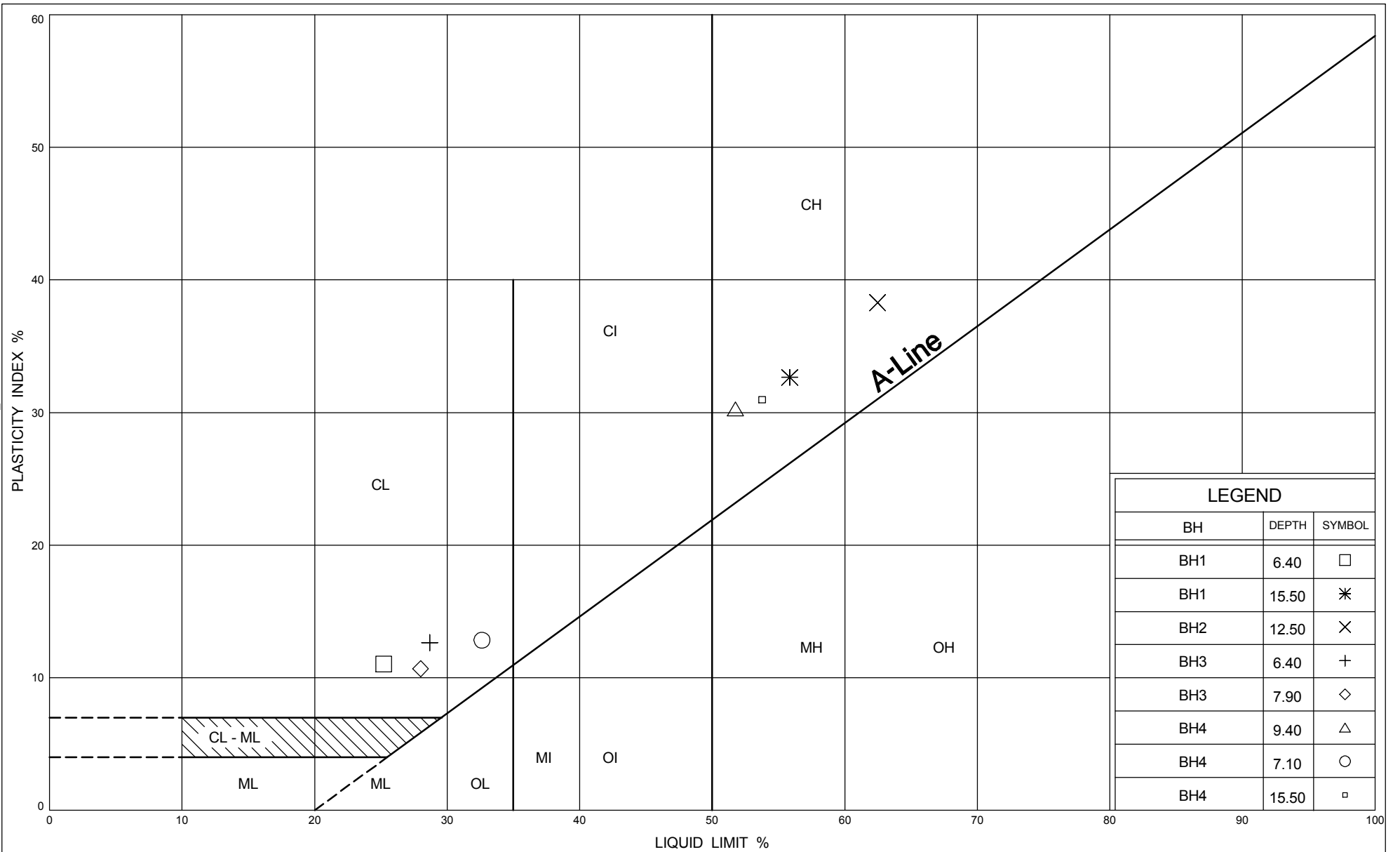
## GRAIN SIZE DISTRIBUTION



ENCLOSURE 9

W P # 5009-E-0073

HWY 65 - BLANCHE RIVER



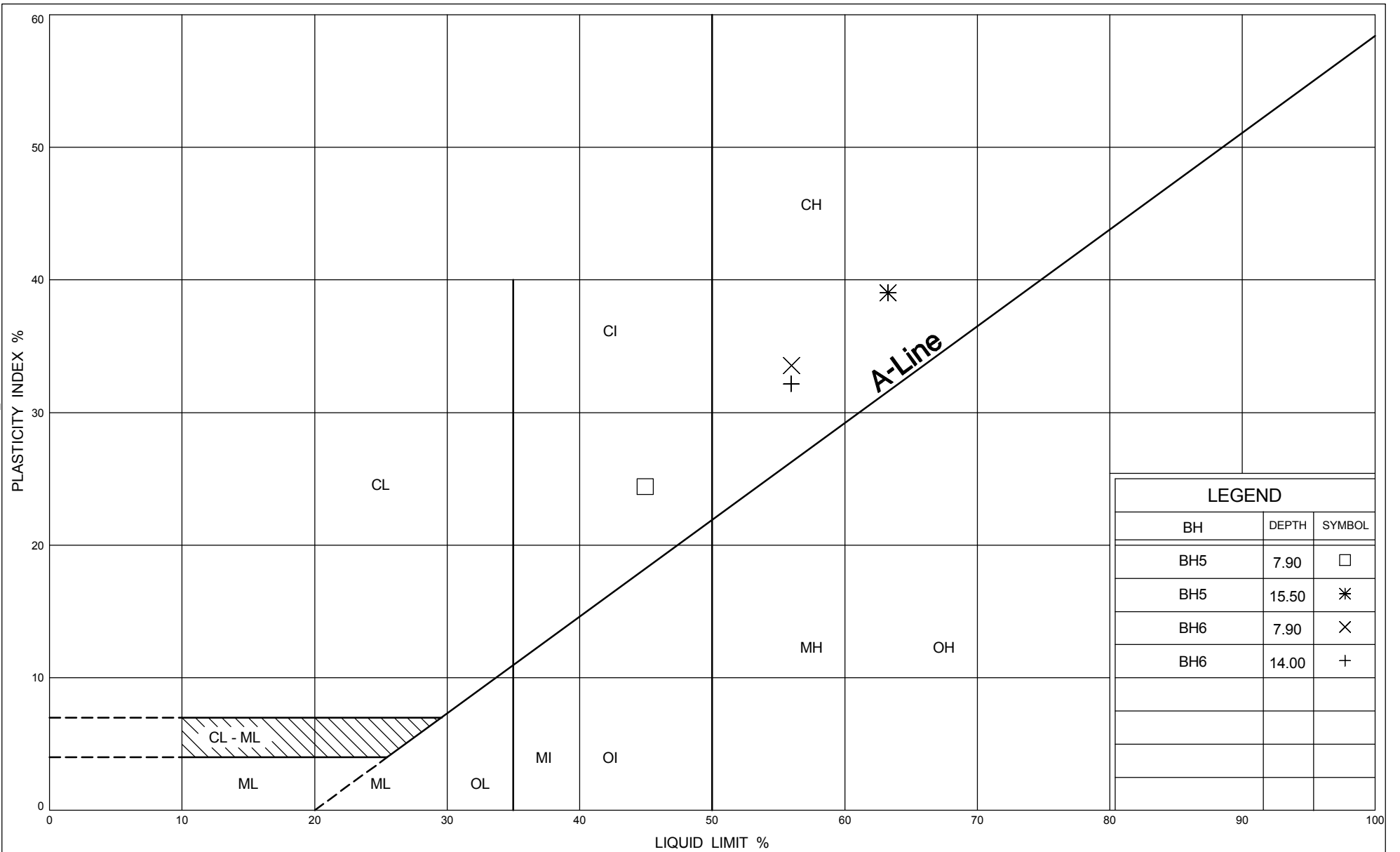
## PLASTICITY CHART

ENCLOSURE 10

W P # 5009-E-0073

HWY 65 - BLANCHE RIVER





## PLASTICITY CHART

ENCLOSURE 11

W P # 5009-E-0073

HWY 65 - BLANCHE RIVER

RESULTS OF ONE-DIMENSIONAL CONSOLIDATION TEST

Project : Blanche River Bridge Rehabilitation

Borehole No. : 3 Date of Test : May 24, 2011  
Sample No. : - Tested By : Bruno  
Sample Depth, m : 10.70 Job No. : GS-TB-011705  
Sample Description : Grey Silty Clay  
  
Test Method : ASTM D 2435-04  
Consolidation Type : Fixed Ring Oedometer  
Condition of Test : Vertical trimmed/ horizontal trimmed/ Remoulded/ other

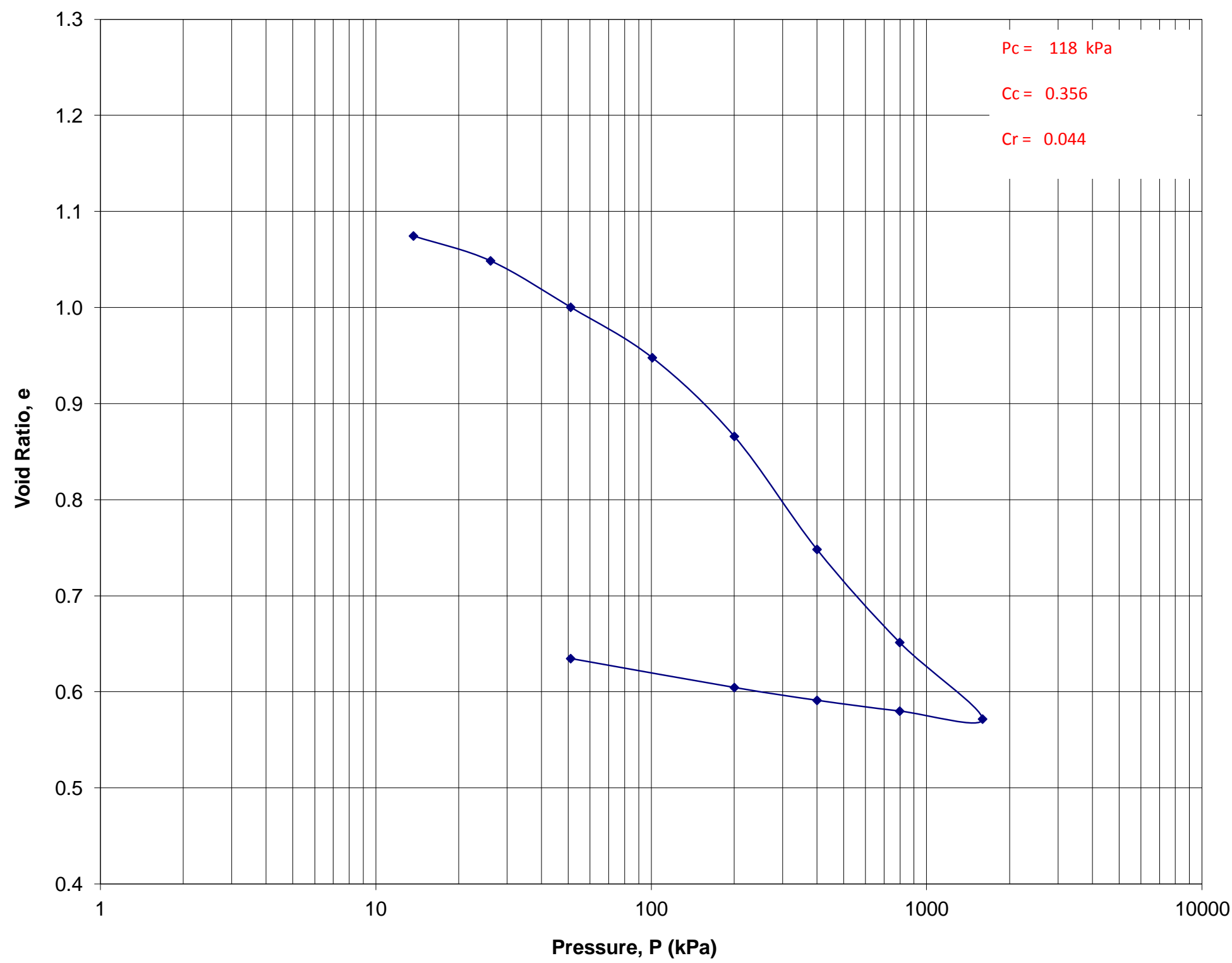
Stage		Initial	Final
Diameter of Sample	(mm)	50.06	50.06
Height of Sample	(mm)	20.00	16.40
Volume of Sample	(cm <sup>3</sup> )	39.36	32.28
Mass of Ring + Wet Soil	(g)	139.21	132.91
Mass of Ring + Dry Soil	(g)	119.43	119.43
Mass of Ring	(g)	68.90	68.90
Mass of Wet Soil	(g)	70.31	64.01
Mass of Dry Soil	(g)	50.53	50.53
Mass of Moisture	(g)	19.78	13.48
Moisture Content	(%)	39.15	26.68
Bulk Density	(Mg/m <sup>3</sup> )	1.79	1.98
Dry Density	(Mg/m <sup>3</sup> )	1.28	1.57
Specific Gravity (tested/ assumed)		2.70	2.70
Void Ratio		1.100	0.722
Degree of Saturation	(%)	95.92	99.59

Inc. No	Load (kPa)	Change in Ht. (mm)	Voids Ratio	t <sub>90</sub> (min)	C <sub>v</sub> (m <sup>2</sup> /yr)	M <sub>v</sub> (m <sup>2</sup> /MN)	K <sub>v</sub> (x 10 <sup>-9</sup> m/s)
1	14	0.246	1.074			0.898	
2	26	0.493	1.049	1.82	47.217	1.020	14.928
3	51	0.953	1.000	1.44	57.549	0.969	17.295
4	101	1.453	0.948	2.25	35.020	0.541	5.869
5	200	2.233	0.866	6.00	12.253	0.441	1.676
6	400	3.353	0.748	12.39	5.329	0.337	0.557
7	798	4.275	0.651	7.00	8.346	0.147	0.381
8	1596	5.034	0.572	4.40	11.935	0.064	0.236
9	798	4.956	0.580				
10	400	4.849	0.591				
11	200	4.722	0.604				
12	51	4.435	0.635				

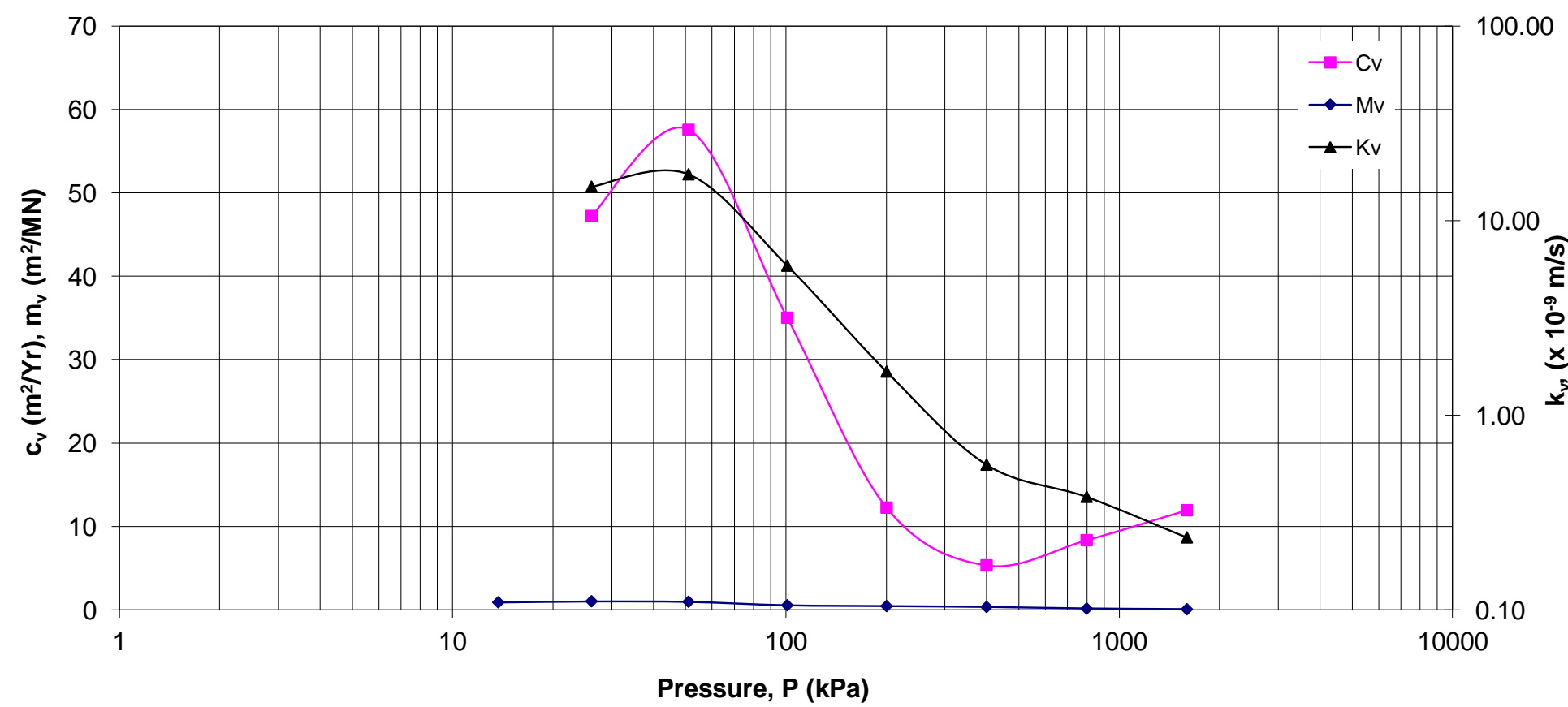


ONE-DIMENSIONAL CONSOLIDATION TEST  
PRESSURE VS. VOID RATIO CURVE

Borehole No.: 3  
Sample No. : -  
Depth : 10.7 m



PRESSURE VS.  $C_v$ ,  $M_v$  &  $K_v$  CURVE



RESULTS OF ONE-DIMENSIONAL CONSOLIDATION TEST

Project : Blanche River Bridge Rehabilitation

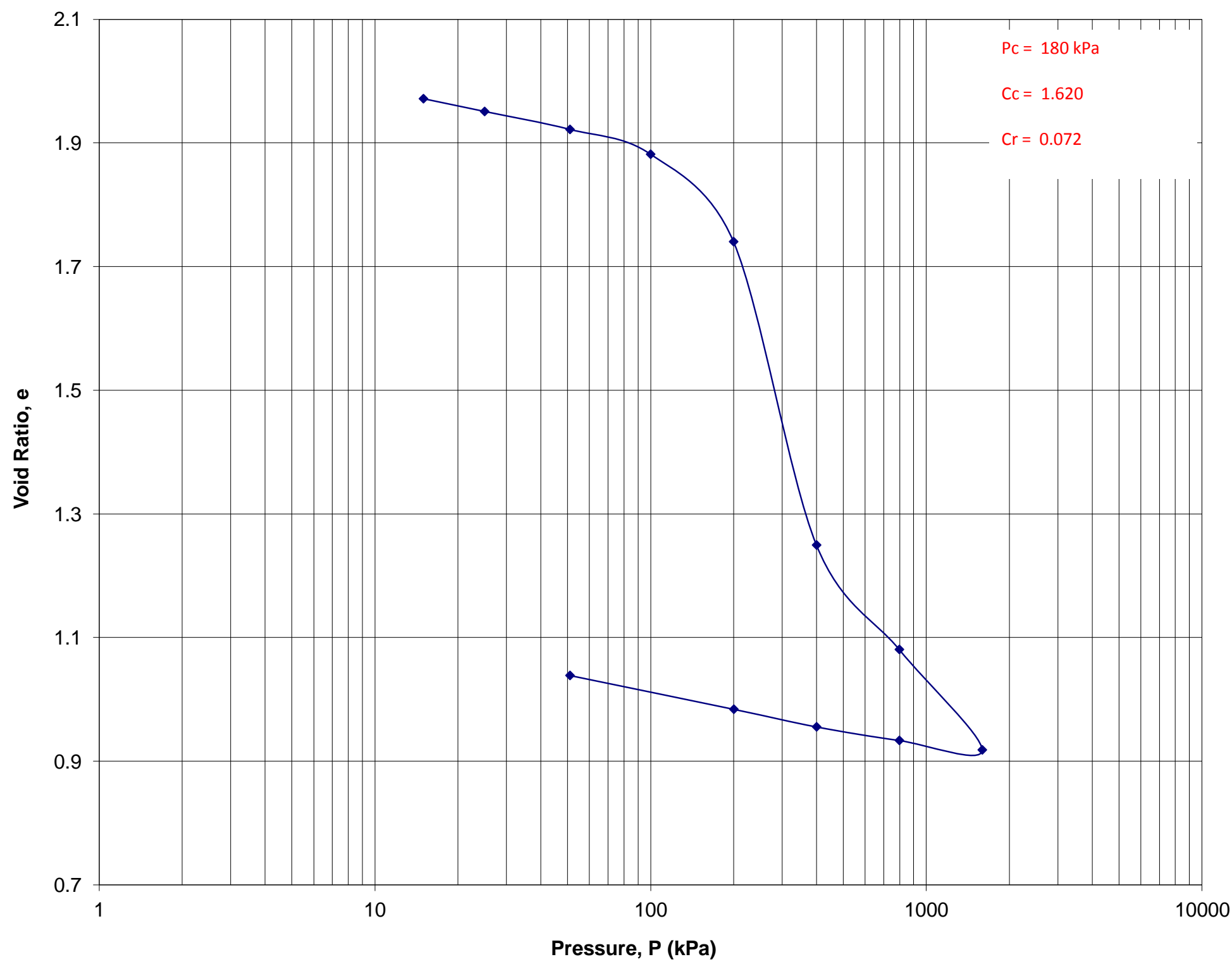
Borehole No. : 4 Date of Test : May 26, 2011  
Sample No. : - Tested By : Bruno  
Sample Depth, m : 10.70 Job No. : GS-TB-011705  
Sample Description : Grey Silty Clay  
  
Test Method : ASTM D 2435-04  
Consolidation Type : Fixed Ring Oedometer  
Condition of Test : Vertical trimmed/ horizontal trimmed/ Remoulded/ other

Stage		Initial	Final
Diameter of Sample	(mm)	50.06	50.06
Height of Sample	(mm)	20.00	13.55
Volume of Sample	(cm <sup>3</sup> )	39.36	26.67
Mass of Ring + Wet Soil	(g)	129.31	117.70
Mass of Ring + Dry Soil	(g)	104.00	104.00
Mass of Ring	(g)	68.89	68.89
Mass of Wet Soil	(g)	60.42	48.81
Mass of Dry Soil	(g)	35.11	35.11
Mass of Moisture	(g)	25.31	13.70
Moisture Content	(%)	72.08	39.01
Bulk Density	(Mg/m <sup>3</sup> )	1.53	1.83
Dry Density	(Mg/m <sup>3</sup> )	0.89	1.32
Specific Gravity (tested/ assumed)		2.68	2.68
Void Ratio		2.009	1.039
Degree of Saturation	(%)	96.29	100.80

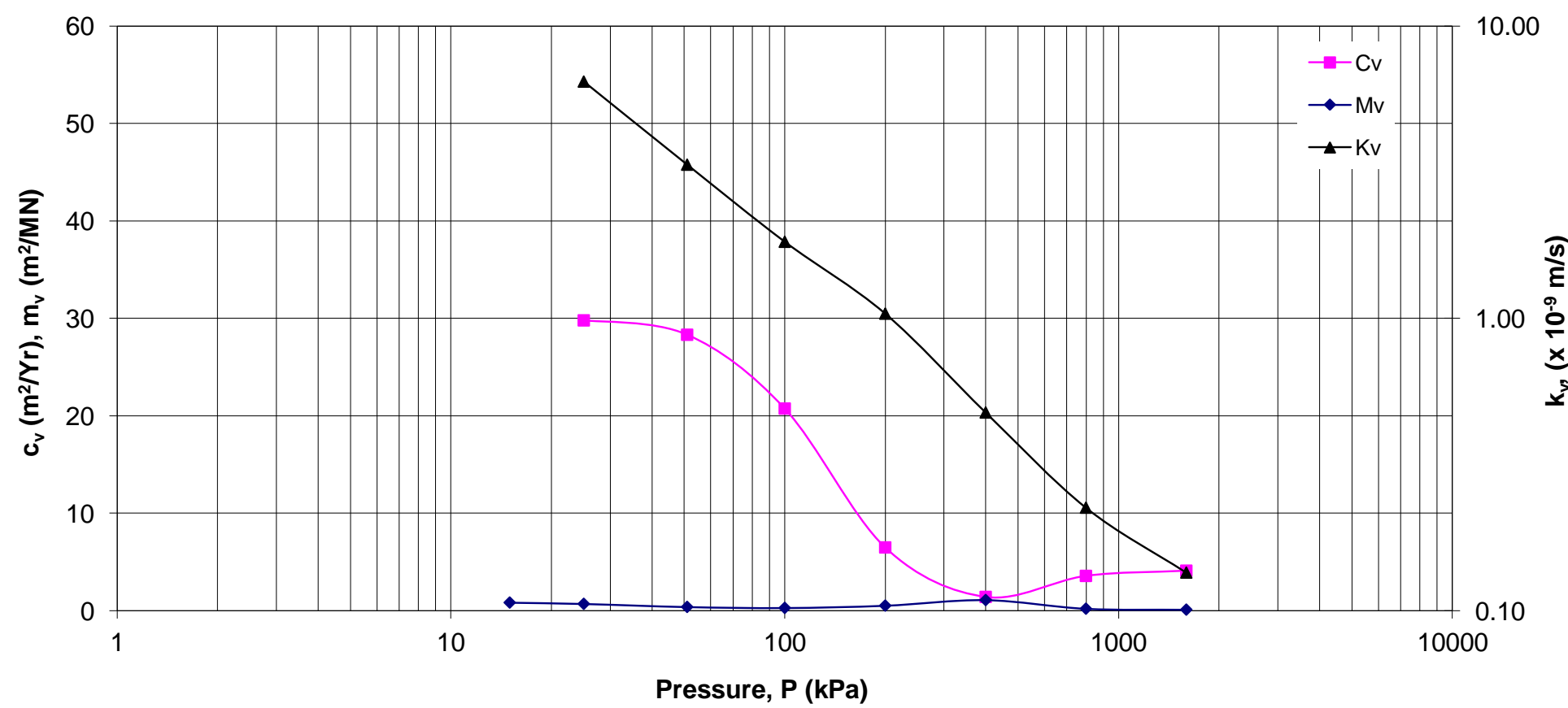
Inc. No	Load (kPa)	Change in Ht. (mm)	Voids Ratio	t <sub>90</sub> (min)	C <sub>v</sub> (m <sup>2</sup> /yr)	M <sub>v</sub> (m <sup>2</sup> /MN)	K <sub>v</sub> (x 10 <sup>-9</sup> m/s)
1	15	0.249	1.972			0.830	
2	25	0.386	1.951	2.90	29.790	0.699	6.455
3	51	0.579	1.922	3.00	28.316	0.382	3.355
4	100	0.846	1.882	4.00	20.740	0.284	1.828
5	200	1.786	1.740	12.00	6.487	0.516	1.038
6	400	5.047	1.250	43.56	1.408	1.091	0.476
7	798	6.170	1.081	12.96	3.564	0.204	0.225
8	1596	7.249	0.918	9.61	4.099	0.106	0.135
9	798	7.148	0.934				
10	400	7.002	0.956				
11	200	6.812	0.984				
12	51	6.449	1.039				

ONE-DIMENSIONAL CONSOLIDATION TEST  
PRESSURE VS. VOID RATIO CURVE

Borehole No.: 4  
Sample No. :  
Depth : 10.7 m



PRESSURE VS.  $C_v$ ,  $M_v$  &  $K_v$  CURVE



RESULTS OF ONE-DIMENSIONAL CONSOLIDATION TEST

Project : Blanche River Bridge Rehabilitation

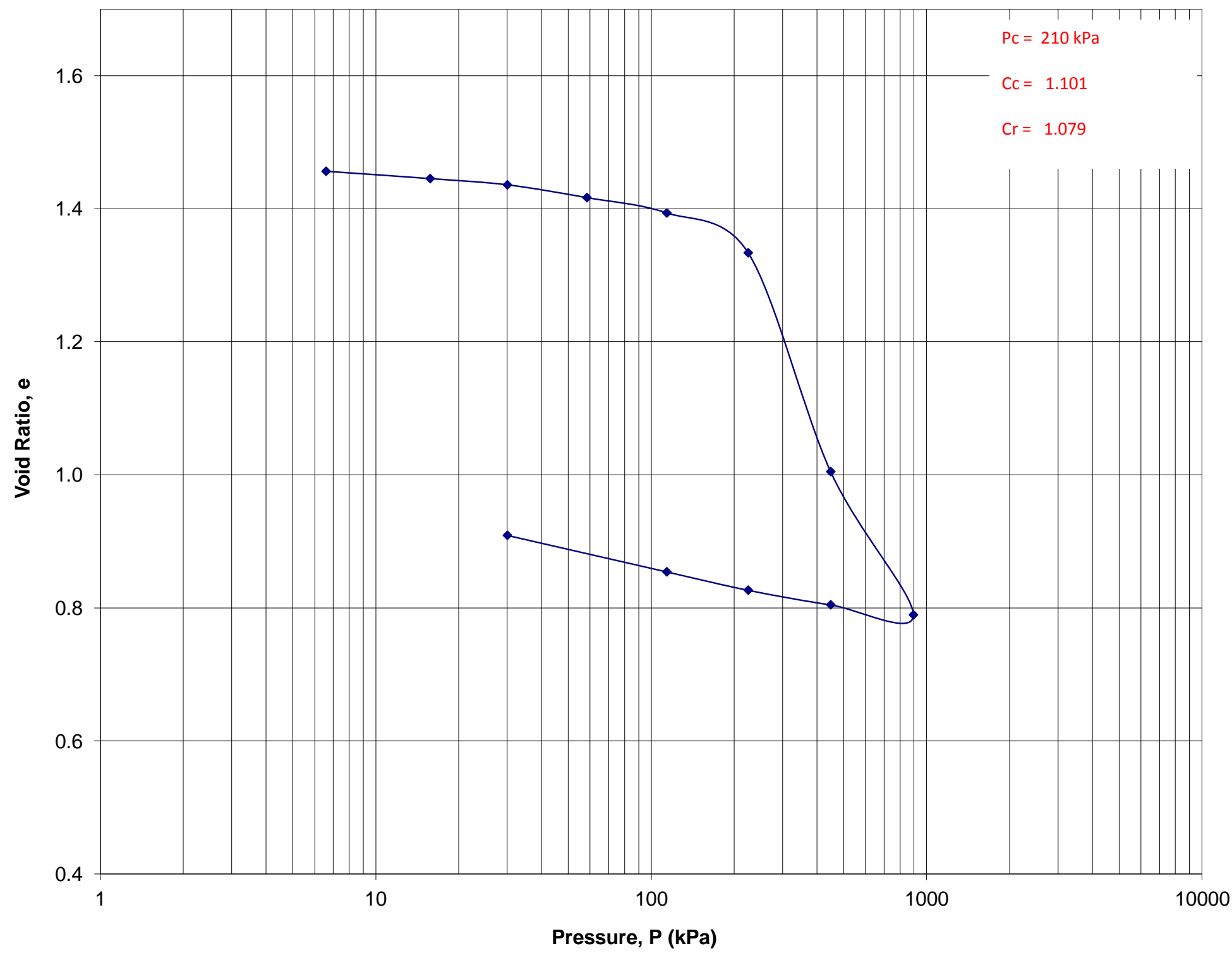
Borehole No. : 4 Date of Test : May 31, 2011  
Sample No. : - Tested By : Bruno  
Sample Depth, m : 18.30 Job No. : GS-TB-011705  
Sample Description : Grey Silty Clay  
  
Test Method : ASTM D 2435-04  
Consolidation Type : Fixed Ring Oedometer  
Condition of Test : Vertical trimmed/ horizontal trimmed/ Remoulded/ other

Stage		Initial	Final
Diameter of Sample	(mm)	63.60	63.60
Height of Sample	(mm)	19.06	14.76
Volume of Sample	(cm <sup>3</sup> )	60.55	46.90
Mass of Ring + Wet Soil	(g)	175.79	164.05
Mass of Ring + Dry Soil	(g)	141.79	141.79
Mass of Ring	(g)	75.95	75.95
Mass of Wet Soil	(g)	99.83	88.10
Mass of Dry Soil	(g)	65.84	65.84
Mass of Moisture	(g)	34.00	22.26
Moisture Content	(%)	51.64	33.81
Bulk Density	(Mg/m <sup>3</sup> )	1.65	1.88
Dry Density	(Mg/m <sup>3</sup> )	1.09	1.40
Specific Gravity (tested/ assumed)		2.68	2.68
Void Ratio		1.465	0.909
Degree of Saturation	(%)	94	100

Inc. No	Load (kPa)	Change in Ht. (mm)	Voids Ratio	t <sub>90</sub> (min)	C <sub>v</sub> (m <sup>2</sup> /yr)	M <sub>v</sub> (m <sup>2</sup> /MN)	K <sub>v</sub> (x 10 <sup>-9</sup> m/s)
1	7	0.066	1.456	2.25		0.525	
2	16	0.151	1.445	3.06	26.174	0.491	3.988
3	30	0.224	1.436	3.80	20.888	0.269	1.741
4	58	0.372	1.417	4.00	19.625	0.280	1.706
5	114	0.550	1.394	3.24	23.809	0.173	1.277
6	225	1.013	1.334	8.42	8.848	0.231	0.634
7	449	3.557	1.005	38.67	1.623	0.734	0.369
8	895	5.222	0.790	27.57	1.741	0.269	0.145
9	449	5.105	0.805				
10	225	4.935	0.827				
11	114	4.722	0.854				
12	30	4.298	0.909				

ONE-DIMENSIONAL CONSOLIDATION TEST  
PRESSURE VS. VOID RATIO CURVE

Borehole No.:	4
Sample No. :	-
Depth :	18.3 m



PRESSURE VS.  $C_v$ ,  $M_v$  &  $K_v$  CURVE

