



October 2016

## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 21/Bayfield River Bridge Replacement  
Site Number 12-188  
Highways 401, 4 and 21  
Structural Replacements  
GWP 3070-11-00, Assignment No. 3 (3011-E-0048)  
Ministry of Transportation, Ontario – West Region

**Submitted to:**

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REPORT



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LIST OF ABBREVIATIONS

LIST OF SYMBOLS

RECORD OF BOREHOLE SHEETS

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## **APPENDICES**

### **APPENDIX A**

Laboratory Test Data

### **APPENDIX B**

Geocres Report No. 40P12-9 Record of Borehole Sheets and Laboratory Results

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Laboratory Test Data

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Geocres Report No. 40P12-9 Record of Borehole Sheets and Laboratory Results



**PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT**

**HIGHWAY 21/BAYFIELD RIVER BRIDGE REPLACEMENT  
SITE NUMBER 12-188**

**HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
GWP 3070-11-00, ASSIGNMENT No. 3 (3011-E-0048)**

**MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**



## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder Associates) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the preliminary design and 30% detailed design work for GWPs 3030-11-00, 3054-11-00, 3053-11-00, 3070-11-00, 3059-11-00, and 3055-11-00. The project involves the preliminary design and 30% detailed design for ten (10) bridges and two (2) culverts, including improvements at five (5) Highway 401 interchanges.

This report addresses the replacement of the Highway 21 bridge over the Bayfield River (Site 12-188) as part of GWP 3070-11-00.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed bridge replacement by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P2-1132-0076-P01 dated September 10, 2012. The work was carried out in accordance with our Quality Control Plan for Foundations Engineering dated November 2012.



## **2.0 SITE DESCRIPTION**

### **2.1 General**

The Highway 21 bridge over the Bayfield River is located in the Town of Bayfield, Ontario as shown on the Key Plan, Figure 1.

For the purposes of this report, Highway 21 and the Bayfield River in the vicinity of the site are assumed to be oriented in north-south and east-west directions, respectively. This section of Highway 21 is currently a two lane, non-divided highway. The highway surface is at approximately elevation 185.7 metres at the bridge location. The existing bridge was constructed in 1949 and is a two-span, steel deck type structure. The area immediately surrounding the site downstream of the bridge, to the west, is occupied by commercial marina properties. Upstream of the bridge, to the east, the area consists of heavily treed river valley and flood plain. It is anticipated that the existing bridge will be demolished and replaced with a new bridge at the same location.

### **2.2 Site Geology**

This project lies within the physiographic region of southern Ontario known as the Huron Slope.<sup>1</sup> Geological mapping indicates that the predominant surficial material at the site is the St. Joseph clayey silt till.<sup>2</sup> Alluvial deposits of gravel, sand and silt and glaciofluvial outwash sand and gravel may also be present near the Bayfield River.

The rock formation in the area of the site is described as medium brown, microcrystalline limestone of the Dundee Formation which belongs to the Hamilton Group of Middle Devonian Age.<sup>3</sup> The bedrock surface is estimated, based on the available mapping, to be at about elevation 167 metres<sup>4</sup> or some 19 metres below highway surface at the site.

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<sup>1</sup> Chapman, L.J. and Putnam, D.F., 1984: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2.

<sup>2</sup> Cooper, A.J. and Fitzgerald, W.D., 1977: Quaternary Geology of the Goderich area, Southern Ontario; Ontario Geological Survey Prelim. Map P.1232, Geol. Ser., scale 1:50,000. Geology 1975, 1976.

<sup>3</sup> Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.

<sup>4</sup> Cooper, A.J., 1978: Bedrock Topography of the Goderich-Seaforth Area, Southern Ontario; Ontario Geological Survey Prelim. Map P.1974, Bedrock Topography Ser., Scale 1:50 000. Compilation 1977, 1978.



### 3.0 INVESTIGATION PROCEDURES

The field work for the current investigation was carried out between September 29 and December 19, 2014 during which time four boreholes, numbered 805 to 808, were drilled. Boreholes 1 to 6 drilled in 1999 by Agra for the foundation investigation for rehabilitation of the retaining wall located east of the north abutment (Geocres Report No. 40P12-9) have been used to supplement the current data. The Record of Borehole sheets and laboratory test results for the previous boreholes have been included in Appendix B.

The approximate locations of the current and previous boreholes are shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole locations, ground surface elevations at the borehole locations and borehole depths.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
805	4 825 794	369 498	185.7	17.4
806	4 825 784	369 508	185.8	18.9
807	4 825 730	369 568	185.9	19.9
808	4 825 711	369 589	185.8	15.7
1 (40P12-9)	4 825 793	369 529	174.3*	7.1
2 (40P12-9)	4 825 808	369 527	174.5*	6.4
3 (40P12-9)	4 825 814	369 525	174.7*	5.8
4 (40P12-9)	4 825 808	369 520	177.4	10.0
5 (40P12-9)	4 825 807	369 506	185.5	15.7
6 (40P12-9)	4 825 794	369 524	179.2	10.7

\* River bed elevation.

The current investigation was carried out using all-terrain and truck mounted drilling equipment supplied and operated by a specialist drilling contractors. In the boreholes, samples of the overburden were obtained at generally 0.75 or 1.5 metre intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures (ASTM D1586). Coring was carried out in borehole 806 using NQ-sized coring equipment.

The results of the SPT testing as presented on the Record of Borehole sheets and in Section 4 are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.). The samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes, including cobbles and boulders, are known to be present in the fill materials and native soils at the site as discussed in the text of this report.



The current boreholes were terminated between about 15.7 and 19.9 metres below the existing pavement surface. Groundwater conditions in the boreholes were observed throughout the drilling operations and a piezometer was installed in borehole 805 as indicated on the corresponding Record of Borehole sheets. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was monitored on a full-time basis by experienced Golder Associates staff members who also located the boreholes in the field, monitored the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in labelled containers and transported to our London laboratory for further examination and testing. Index and classification tests, consisting of water content determinations, Atterberg limits determinations and grain size distribution analyses, were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

The locations of the boreholes are shown on the Record of Borehole sheets and on Drawing 1, attached.





## **4.0 SUBSURFACE CONDITIONS**

### **4.1 Site Stratigraphy**

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the laboratory testing carried out on selected samples, are provided on the attached Record of Borehole sheets following the text of this report and in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets and Drawing 1 are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The boreholes drilled at the site generally encountered the existing pavement structure and fill materials, overlying layers of silt, sand, sand and gravel, sandy silt glacial till and silty clay to clayey silt glacial till.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on Drawing 1. Detailed descriptions of the subsurface conditions encountered in the boreholes are provided on the Record of Borehole sheets and are summarized in the following sections.

Materials described as clayey silt in the boreholes advanced for Geocres Report No. 40P12-9 have been classified as clayey silt till based of the results of laboratory testing and comparison of the stratigraphy in adjacent boreholes. In borehole 1 (40P12-9) the sandy gravel has been classified as sand and gravel. These classifications have been reflected in the following report sections and the inferred profile, Drawing 1.

Further, boreholes 1, 5 and 6 (40P12-P) are considered the most relevant to this assignment; thus, the other boreholes have not been included in the following discussion.

#### **4.1.1 Pavement Structure**

The existing pavement structure was encountered at the pavement surface at each of the current borehole locations. Asphaltic concrete, 180 millimetres thick, was encountered in boreholes 805 and 808 and 150 and 120 millimetres of Portland cement concrete was encountered in boreholes 806 and 807, respectively. The asphalt and concrete were underlain by 120 millimetres to 2.8 metres of granular base/subbase. Measured N values from the standard penetration testing in these materials in borehole 806 ranged from 5 to 10 blows per 0.3 metres. In borehole 807, a single N value of 22 blows per 0.3 metres was obtained in the pavement granulars. The corresponding samples had water contents of 8 to 9 per cent. A grain size distribution curve for a sample of the sand and gravel base material is shown in Figure A-1.

#### **4.1.2 Fill and Topsoil**

Fill materials were encountered beneath the pavement structures in the current boreholes and at ground surface in boreholes 5 (40P12-9) and 6 (40P12-9). The fill materials were variable, consisting of sand and gravel, sand, sandy silt and silty sand, clayey silt and organic silt. The fill materials were encountered between about elevations 179.2 and 185.5 metres and were about 3.7 to 10.2 metres thick. The fill had N values of 4 to 52 blows per 0.3 metres with water contents of about 4 to 30 per cent. Samples of the clayey fill had corresponding average plastic and liquid limits of about 11 and 26 per cent, respectively, based on three Atterberg limits determinations. These



data are shown on the Plasticity Chart, Figure A-7. Varying amounts of topsoil, wood fragments and cobbles were noted in the fill.

Grain size distribution curves for samples of the fill recovered from the standard penetration testing are provided on Figure A-1.

A layer of buried topsoil about 0.5 metres thick was encountered within the fill in borehole 806 at about elevation 177.3 metres. The buried topsoil had an N value of 13 blows per 0.3 metres with a water content of about 41 per cent.

About 0.6 metres of sediment described as sandy gravel was encountered in the Bayfield River in borehole 1(40P12-9).

#### **4.1.3 Sand and Gravel**

Layers of loose to very dense sand and gravel 0.8 to 5.0 metres thick were encountered in all of the current boreholes and in boreholes 1 (40P12-9) and 5 (40P12-9). The sand and gravel was encountered between elevations 171.1 and 177.3 metres.

Measured N values in the sand and gravel ranged from 8 to greater than 100 blows per 0.3 metres. Samples of the sand and gravel had water contents ranging from 5 to 21 per cent.

Grain size distribution curves for samples of the sand and gravel from the current and previous investigations are shown on Figure A-2 and on Figures 2 and 3 in Appendix B.

Cobbles and boulders should be expected in the sand and gravel strata.

#### **4.1.4 Silt**

Silt was encountered beneath the fill materials in boreholes 807 and 808 at elevations 177.2 and 178.5 metres, respectively, and beneath the sand and gravel in borehole 807 at elevation 173.1 metres. The upper silt layers were 1.1 to 1.2 metres thick and the lower silt layer was about 2.7 metres thick. Measured N values in the compact to very dense silt ranged from 11 to 59 blows per 0.3 metres and corresponding samples had water contents of 11 to 24 per cent.

Grain size distribution curves for samples of the silt are shown on Figure A-3.

#### **4.1.5 Sand**

A 2.1 metre thick layer of sand was encountered in borehole 6 (40P12-9) beneath the fill materials at elevation 175.5 metres, a layer of sand 0.6 metres thick was encountered beneath the lower silt in borehole 807 at elevation 170.3 metres and a layer of sand was encountered in borehole 5 (40P12-9) beneath the clayey silt till at elevation 171.5 metres. Borehole 5 (40P12-9) was terminated in the sand after exploring it for 1.7 metres. The loose to very dense sand had N values ranging from 7 to 75 blows per 0.3 metres. Samples of the sand from boreholes 807 and 5 (40P12-9) had water contents of about 18 to 22 per cent.



#### **4.1.6 Clayey Silt Glacial Till**

Clayey silt glacial till was encountered beneath the sand and gravel in boreholes 805, 806, 1 (40P12-9) and 5 (40P12-9), beneath the sand in borehole 807 and beneath a layer of silty clay till in borehole 805. The clayey silt till was encountered between elevations 169.6 and 174.2 metres and was 1.4 to 3.1 metres thick. Although not specifically encountered in the boreholes, cobbles and boulders should be expected in the clayey silt till.

Measured N values in the clayey silt till ranged from 11 to 106 blows per 0.3 metres indicating a stiff to hard consistency. A single laboratory vane test carried out on a sample of the clayey silt till from borehole 1 (40P12-9) yielded a shear strength of 35 kilopascals (kPa). Samples of the clayey silt till had water contents of 6 to 20 per cent. Ten Atterberg limits determinations were carried out on samples of the clayey silt till as part of the current and previous investigations. The clayey silt till had liquid and plastic limits ranging from 17 to 25 per cent and 10 to 15 per cent, respectively, and plasticity indices of 6 to 10 per cent, indicating a clay of low plasticity. The results of the Atterberg limits testing from the current investigation are shown on Figure A-7.

Grain size distribution curves for samples of the clayey silt till from the current investigation are shown in Figure A-4. Laboratory test results from the previous investigation are provided in Appendix B.

#### **4.1.7 Silty Clay Glacial Till**

Layers of silty clay glacial till, 1.4 and 2.3 metres thick, were encountered beneath the clayey silt glacial till in boreholes 805 and 806 at elevations 171.0 and 171.9 metres, respectively. Although not specifically encountered in the boreholes, cobbles and boulders should be expected in the silty clay till.

The silty clay till had N values of 18 and 38 blows per 0.3 metres indicating very stiff to hard consistency. Samples of the silty clay till had water contents of 17 and 23 per cent. Two Atterberg limits determinations were carried out on samples of the silty clay till, the results of which are shown on Figure A-7. The silty clay till samples had liquid limits of 33 and 44 per cent, plastic limits of 19 and 20 per cent, and plasticity indices of 13 and 25 per cent, indicating a clay of low to intermediate plasticity; however the liquid limit of 33 per cent is considered to be have been influenced, to some degree, by the silty sand and sandy silt pockets noted within the deposit. Grain size distribution curves for samples of the silty clay till are shown on Figure A-5.

#### **4.1.8 Sandy Silt Glacial Till**

Layers of very dense sandy silt glacial till were encountered beneath the silty clay till in borehole 806, beneath the clayey silt till in boreholes 807 and 6 (40P12-9) and beneath the sand and gravel in boreholes 808 and 1 (40P12-9). The sandy silt till was encountered between elevations 166.7 and 172.2 metres. All of these boreholes were terminated in the sandy silt till after exploring it for 0.7 to 2.8 metres. It should be noted that borehole 1 was terminated due to auger refusal at elevation 167.2 metres. No further information was provided on the Record of Borehole but it should be noted that this is near the mapped bedrock surface elevation as noted in Section 2.2. Cobbles and boulders were encountered in the sandy silt till.

Measured N values in the sandy silt till ranged from 56 to greater than 100 blows per 0.3 metres. Samples of the sandy silt till had water contents of 8 to 15 per cent.



Grain size distribution curves for samples of the sandy silt till from the current and previous investigations are shown on Figure A-6 and in Appendix B, respectively.

## 4.2 Groundwater Conditions

Groundwater conditions were observed in the current and previous boreholes during drilling and a piezometer was installed in borehole 805 as shown on the Record of Borehole sheets. The encountered and measured groundwater levels are summarized in the following tables.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Level	
		Depth (m)	Elevation (m)
805	185.7	4.8	180.9
806	185.8	8.4	177.4
807	185.9	10.7	175.2
808	185.8	10.4	175.4
1 (40P12-9)	174.3*	**	**
5 (40P12-9)	185.5	**	**
6 (40P12-9)	179.2	2.8	176.4

\* River bed elevation.

\*\* Groundwater level not established during drilling.

The encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions as the readings were taken only during the relatively short duration of the drilling programs.

Borehole	Measured Groundwater Elevation (m)	
	October 1, 2014 (following installation) 176.49	February 19, 2015 176.52
805		
5 (40P12-9)	October 13, 1999 175.5	

The water level in the Bayfield River was measured at elevation 176.5 metres on December 19, 2014 and by others at elevation 176.4 metres in September 1999.

Based on the measured and encountered groundwater levels, the groundwater level is expected to vary with the river water level and the adjacent Lake Huron and is inferred to be at elevation 176.5 metres. Groundwater levels should be expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring melt conditions.



## 5.0 MISCELLANEOUS

This investigation was carried out using equipment supplied and operated by Lantech Drilling, Aardvark Drilling and Henderson Drilling Inc., all Ontario Ministry of Environment and Climate Change licensed well contractors. The field operations were supervised by Mr. Michael Arthur and the field investigation manager, Mr. David J. Mitchell.

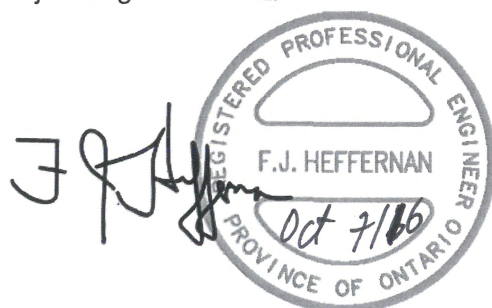
The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Ms. Nicole A. Gould, P.Eng., under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. This report was reviewed by Mr. Michael E. Beadle, P.Eng. an Associate and Senior Geotechnical Engineer with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

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**PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT**

**HIGHWAY 21/BAYFIELD RIVER BRIDGE REPLACEMENT  
SITE NUMBER 12-188**

**HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
GWP 3070-11-00, ASSIGNMENT No. 3 (3011-E-0048)**

**MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**





## **6.0 ENGINEERING RECOMMENDATIONS**

This section of the report provides our recommendations on the foundation aspects of the design of the replacement of the existing Highway 21 bridge over the Bayfield River (Site 12-188). The recommendations are based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

The existing bridge structure, constructed in 1949, is a two-span bridge over the Bayfield River in Bayfield, Ontario. The existing bridge is a steel deck truss type structure with two spans of 31 metres each and a width of 11.6 metres. The bridge deck was rehabilitated in 2000 and steel repairs were carried out on the truss elements in 2009 as well as construction of a steel sheet pile wall with tie backs and associated shoreline protection east of the north abutment.

Based on Department of Highways Ontario (DHO) Drawing No. D-2893 dated August 27, 1948, the north and south abutments were supported on shallow spread footings at elevations 174.0 and 173.4 metres. The central pier was constructed as two pedestals and an arch founded on shallow spread footings at elevation 173.7 metres. The founding soil conditions were not specified.

According to the MTO's Request for Proposal (RFP), this structure is to be replaced due to its advanced age, condition deficiency and structural obsolescence. The RFP indicates that this section of Highway 21 is to remain a two-lane cross section and it is anticipated that the Highway 21 grade and alignment will remain relatively unchanged.

Throughout preliminary design, various replacement options were considered. These options included various one and two span steel girder or deck truss alternatives as well as a single span half-through arch and a single span bowstring arch. Integral abutments may be considered, if appropriate. It is understood that consideration is being given to full or partial removal of the existing abutments and pier. At the time of this report, the preferred option consisted of a single-span girder/frame bridge based on aesthetics, future maintenance and initial construction cost. This option would include four foundation units where the foundations closest to the approaches would be subject to reaction uplift loads.

The proposed staging includes the construction of a temporary bridge structure to the east of the existing structure to carry traffic during demolition of the existing structure and construction of the new structure. It is expected that a two span, pre-fabricated truss bridge will be utilized for the temporary structure. The temporary alignment will require widening of the approach embankments to accommodate the traffic shift.

### **6.1 Bridge Foundations**

The subsurface soil conditions at the site typically consist of the existing pavement structure and fill materials overlying layers of silt, sand, sand and gravel, sandy silt glacial till and silty clay to clayey silt glacial till. Native soils were encountered between elevations 175.2 and 178.5 metres. The elevation of Highway 21 at the site is at



about elevation 185.7 metres at the bridge. The inferred groundwater level at the site is at about elevation 176.5 metres.

The abutments, retaining walls and reaction foundations for the replacement bridge and temporary structure may be supported on conventional shallow spread/strip footings depending on loading conditions. If integral abutments are selected, if the abutments are to be perched relatively high in the existing fill materials and reaction foundations are needed for the girder/frame structure deep foundations consisting of steel H-piles, concrete-filled steel tube piles or drilled shafts (caissons); however, it is expected that it may be difficult for some piles to penetrate the native soils at the site due to the presence of cobbles and boulders. Consideration could also be given to founding the piers and abutments on drilled shafts (caissons) or a combined drilled shaft and H-pile system whereby the H-pile serves as the structural reinforcement and the top portion of the drilled hole is backfilled with sand fill instead of concrete to allow flexure required for integral abutment design.

Construction for a central pier for the temporary by-pass structure could require construction of a coffer dam in the Bayfield River. It is expected that sheet piling will not effectively penetrate the sand and gravel and glacial till at the site, potentially preventing adequate toe embedment for lateral support of the protection system and for control of groundwater.

A comparison of foundation alternatives is presented in Table I following the text of this report. The relative costs are compared using the most economical foundation option (shallow foundations) as the base cost. The estimated relative costs are meant to provide an order of magnitude comparison amongst the alternatives and are not indicative of actual construction costs. Shallow foundations for conventional abutment design is the preferred technical alternative from a foundation engineering perspective.

### **6.1.1 Shallow Foundations**

#### ***Geotechnical Axial Resistance***

The abutments, temporary pier, girder/frame reaction structures and the related retaining walls for the replacement and temporary bridge may be founded on conventional spread/strip footings depending on loading conditions. Assuming the new footings are constructed at similar elevations as the existing footings, a factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kPa and a geotechnical reaction at Serviceability Limit States (SLS) of 400 kPa may be used for footings constructed on the compact to dense sand and gravel at or below elevation 176 metres at the abutments and at or below elevation 174 metres at the pier.

Alternatively, if portions of the existing abutments are to be left in place, the abutment footings may be founded on engineered fill constructed above the existing founding elevations on the existing compact to very dense sand and gravel fill that exists at and below elevation 180 metres in boreholes 806 and 807. The engineered fill should consist of a minimum of 1.5 metres of Ontario Provincial Standard Specifications (OPSS) Granular A placed on a properly prepared subgrade in maximum 300 millimetre thick loose lifts and compacted to at least 98 per cent of the standard Proctor maximum dry density. Abutment footings founded on engineered fill as described above may be designed using a factored geotechnical resistance at ULS of 600 kPa and a geotechnical reaction at SLS of 400 kPa. The effect of load inclination has not been included in these values. The SLS values correspond to an estimated total settlement of 25 millimetres. Practical footing widths may not be achievable, however, depending on final structural loads.





### ***Resistance to Lateral Forces***

Resistance to lateral forces/sliding between concrete footings and the native, undisturbed subsoil or engineered fill should be calculated in accordance with Section 6.7.5 of the Canadian Highway Bridge Design Code (CHBDC) 2006. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction and/or the engineered fill is constructed as described above, an angle of friction between the cast-in-place concrete and the founding soils of 35 degrees and corresponding unfactored coefficient of friction,  $\tan \delta$ , of 0.7 may be used.

### ***Frost Protection and Scour Protection***

All footings should be provided with a minimum of 1.2 metres of soil cover or thermal equivalent for frost protection. Footings should be adequately protected against scour as noted in Section 1.9.5.2 of the CHBDC 2006.

#### **6.1.2 Deep Foundations**

The abutments and other structure foundations for the girder/frame bridge could be founded on 324 millimetre outside diameter (OD) concrete-filled steel tube piles with 9.5 millimetre wall thicknesses, steel HP 310x110 (or heavier section) piles or drilled shafts (caissons). Integral abutments are typically founded on steel H-piles with lateral loads applied in the direction of the weak axis. Alternatively, integral abutments may be founded on concrete filled steel tube piles provided the additional resistance to lateral loads is adequately considered during structural design. Piles supporting integral abutments require pre-augering and placement of a corrugated steel pipe (CSP) liner filled with loose, uniform sand around the upper 3 metres of the pile to reduce resistance to lateral movement. Steel H-piles with the lateral loads in the direction of the strong axis and concrete filled steel tube piles may be used for conventional and semi-integral abutments. A combined drilled shaft and H-pile system may also be suitable whereby the H-pile serves as the structural reinforcement and the top portion of the drilled hole is backfilled with sand fill instead of concrete to allow flexure required for integral abutment design. This combination may be a practical approach to achieving structural goals of using integral abutment systems as well as for resisting vertical uplift and downward forces.

A central pier for the temporary by-pass bridge may be founded on steel tube piles, steel H-piles or drilled shafts as described above. For preliminary design purposes, the soil conditions at the location of a pier may be extrapolated from the borehole information available at the abutments, though additional investigation would be required for detailed design stages if a pier is to be constructed to define the thickness of recent river sediments in the immediate area of the pier. Defining the thicknesses of recent river sediments in the final pier location will assist with design and construction of temporary protections against scour and final temporary foundations.

Cobbles and boulders should be expected in the native soils at the site and the contractor should be prepared to deal with their presence.



### Geotechnical Axial Resistance – Driven Steel Piles

For design, the factored geotechnical axial resistances at ULS and geotechnical reactions at SLS for HP 310 x 110 (or heavier section) piles and concrete filled steel tube piles driven to refusal at or below the maximum anticipated elevations shown are provided in the following table. Values for SLS conditions are not provided since the founding condition is considered to be effectively unyielding as the bedrock is expected to be at a shallow depth below pile tips.

Location	Target Tip Elevation (m)	Factored Geotechnical Resistance at ULS (kN)	
		HP 310 x 110	324 mm OD x 9.5 mm concrete filled steel tube
South Abutment	166.0	1,800	1,400
North Abutment	168.0		

Although a target tip elevation is provided in the table above, it is anticipated that the tube piles will not penetrate as deeply as the H piles for similar impact hammer energies. Therefore, for this case, the resistance values provided for the steel tube piles are less than for the H piles on account of the reduced confinement at tip elevations. It is anticipated that the steel tube piles may not penetrate below elevations 175 m near the north abutment and 173 m near the south abutment. The actual pile penetration and pile set characteristics will be dependent, to some extent, upon the driving equipment selected by the contractor and the methods of pile tip protection against cobbles and boulders. The pile driving hammer should be selected such that the energy delivered to the piles is sufficient to achieve the termination criteria (ultimate geotechnical resistances and tip elevations) stated in the Contract Documents. It is recommended that, following the selection of the driving equipment, the piling contractor submit for review the proposed pile driving criteria based on the characteristics of the hammer and equipment intended for use.

The preferred bridge alternative includes foundations that may be subject to significant uplift forces. In general, driven steel H or tube piles should not be relied upon for uplift resistance unless these are constructed in pre-drilled holes. Friction resistance along the sides of the steel piles in uplift will be relatively nominal compared to anticipated uplift forces since use of pre-fabricated or built-up tips for protecting the piles during driving and the consequent enlargement of the tip dimensions will tend to displace the stiff and dense soils during driving such that normal forces between the ground and steel along the pile shaft may be reduced. Where H piles are installed within pre-bored holes, design of the piles for the length encased in structural concrete is more appropriately completed using drilled shaft (caisson) design methods presented below.



### Geotechnical Axial Resistance – Drilled Shafts (Caissons)

For preliminary design, the unfactored vertical load resistance (for downward or uplift directions) of drilled shafts (caissons) derived from skin friction, where the cast-in-place concrete is in direct contact with the soils (i.e., without a permanent steel liner), may be calculated using the following equation:

$$Q_s = \pi B \Delta z f_{SN}$$

where  $Q_s$  is the nominal skin friction in kilonewtons (kN),  $B$  is the shaft diameter in metres,  $\Delta z$  is the thickness of the soil layer over which resistance is calculated in metres and  $f_{SN}$  is the nominal unit skin friction in kPa. The upper 1.2 metres below the ground surface should be neglected to account for frost action. Any portion of the geotechnical resistance where the caisson is within fill materials above approximately elevation 180 m should also be neglected.

Assuming that caissons greater than 1 metre in diameter will be used, the component of the vertical load carrying capacity that may be derived from end bearing in the native soils may be calculated using the following equation:

$$Q_b = q_{BN} A_t$$

where  $Q_b$  is the toe resistance in kN,  $q_{BN}$  is the nominal unit base resistance in kPa and  $A_t$  is the cross-sectional area of the caisson in square metres. Provided that the base of the drilled shafts is appropriately cleaned of loose materials using airlift or cleaning bucket tools and appropriate tremie-concrete is used, caissons founded in the native soils may be designed using the nominal unit side and base resistances provided in the following table. The stratigraphy presented in the table below has been simplified for the purposes of this report.

Soil Type	Elevation (m)	$f_{SN}$ (kPa)	$q_{BN}$ (kPa)	Unit Weight (kN/m <sup>3</sup> )
Fill	Where applicable	-	-	19.0
South Abutment				
Existing embankment fill	175.0 to 180.0	20	NA	20.0
- Dense silt or sand and gravel	170.0 to 175.0	35	200	20.0
- Glacial till	167.5 to 170.0	120	1,000	21.0
- Glacial till	165.0 to 167.5	160	2,200	22.0
North Abutment				
Existing embankment fill	175.0 to 180.0	20	NA	20.0
- Dense sand and gravel	173.0 to 176.0	90	600	21.0
- Glacial till	170.0 to 173.0	120	1,000	21.0
- Glacial till	165.0 to 170.0	160	2,200	22.0



The ultimate resistance  $Q_u$  is the sum of  $Q_b$  and  $Q_s$ . A resistance factor of 0.5 should be applied to  $Q_u$  to obtain the factored axial resistance to downward (bearing) forces at ULS. A factor of 0.3 should be used to define the SLS resistance to downward vertical load resistance. The factored ULS and SLS uplift resistance values should be calculated using the equation and parameters provided above and a resistance factor of 0.3.

### ***Frost Protection and Scour Protection***

The pile caps should be provided with a minimum of 1.2 metres of soil cover or thermal equivalent for frost protection. Pile caps should be adequately protected against scour as noted in Section 1.9.5.3 of the CHBDC 2006.

### ***Downdrag Load (Negative Skin Friction)***

Considering the presence of compact to very dense granular soils and stiff to hard cohesive till, negative skin friction can be neglected.

### ***Resistance to Lateral Loads***

Lateral loading could be resisted fully or partially by the use of battered piles. In the case of integral abutments, the vertical piles must provide the resistance to the lateral loading. The stratigraphy presented in the table below has been simplified for the purposes of this report.

The horizontal reaction to the pile can be estimated using the following equations, undrained shear strengths, and ranges in subgrade reaction coefficient where:

$$\begin{aligned} k_h &= \text{coefficient of horizontal subgrade reaction (MPa/m)} &= n_h (z/d) &\text{for cohesionless soils} \\ & &= \frac{67 S_u}{d} &\text{for cohesive soils} \end{aligned}$$

where:

- $d$  = pile width or diameter (m)
- $n_h$  = constant of horizontal subgrade reaction (MPa/m)
- $S_u$  = undrained shear strength of the soil (MPa)
- $z$  = depth below ground surface grade (m)

The range in values reflects the variability in subsurface conditions as well as the two extremes of design; the requirement for flexibility if integral abutments are selected and the requirement for lateral support in the cases of non-integral abutments or pier foundations.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT**  
**HIGHWAY 21/BAYFIELD RIVER BRIDGE REPLACEMENT, SITE NUMBER 12-188**

Location	Soil Type	Elevation (m)	$n_h$ (MPa/m)	$S_u$ (kPa)
CSPs for integral abutments	Granular backfill	Where applicable	5 - 10	-
South Abutment	Compact to very dense sand and gravel	173.1 to 176.1	7 - 17	-
	Very dense silt	170.3 to 173.1	6 - 11	-
	Very dense sand	169.7 to 170.3	6 - 11	-
	Hard clayey silt till	166.7 to 169.7	-	400
	Very dense sandy silt till	166.0 to 166.7	9 - 12	-
North Abutment	Loose to very dense sand and gravel	173.3 to 176.0	3 - 10	-
	Very stiff to hard clayey silt till	171.9 to 173.3	-	300
	Very stiff to hard silty clay till	169.7 to 171.9	-	300
	Very dense sandy silt till	166.9 to 169.7	9 - 12	-

The preliminary lateral resistances for the various foundation options are summarized in the following table.

Pile Type	Lateral Resistance	
	Factored ULS (kN)	SLS (kN)
Integral abutments		
- HP 310 x 110, weak axis bending	100	95
- 324 mm OD x 9.5 mm tube	170	135
Semi-Integral or Conventional abutments		
- HP 310 x 110, strong axis bending	155	80
- 324 mm OD x 9.5 mm tube	110	60
Drilled Shaft, 1.2 metres diameter	600	300

The lateral resistances are based on Brom's Method from "Design and Construction of Driven Pile Foundations Workshop Manual – Volume 1, FHWA, Pub. No. FHWA H1 97-013, Revised November 1998". In the case of integral abutments, fixed head piles were assumed and a 3 metre sand filled CSP placed above elevation 180 metres with the horizontal load applied at the top of the CSP. Free-head conditions were assumed for semi-integral or conventional abutments, with the horizontal load for applied at the existing founding elevation. The SLS values are based on 10 millimetres of deflection at the ground surface. If integral abutments are selected for final design, the bridge length may be such that the horizontal deflections are greater than 10 millimetres. At the detail design stage, the structural engineer should liaise with the foundations engineer to select the appropriate lateral resistance values for the anticipated range of horizontal deflection.



Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor,  $R$ , as follows:

<i>Pile Spacing in Direction of Loading, <math>d</math> = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor <math>R</math></i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

## 6.2 Liquefaction Potential and Seismic Analysis

### 6.2.1 Seismic Parameters

The site is located near the Town of Clinton, Ontario. According to Table A.3.1.1 of the CHBDC 2006, the zonal acceleration ratio,  $A$ , applicable to this site is 0.00. The corresponding acceleration-related seismic zone,  $Z_a$  is 0.

The replacement bridge is in Seismic Performance Zone (SPZ) 1, based on a CHBDC classification as an "Emergency Route Bridge". Based on the site stratigraphy, the soil profile type is categorized as Type I with a seismic site response coefficient,  $S$ , of 1.0 based on the CHBDC criteria. Analysis of bridges in SPZ 1 is not a requirement of the CHBDC. However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1.

### 6.2.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.<sup>5</sup> The liquefaction potential is considered to be low based on the soil profile type, age of the deposits, relative density and the historically low regional seismicity; therefore, a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations and the effect of seismic forces on embankment stability is not considered warranted unless the structure is deemed to be a lifeline bridge.

<sup>5</sup> FHWA, 1997: "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.



### 6.3 Lateral Earth Pressures for Design

The lateral pressures acting on the bridge abutments and associated wing/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the freedom of lateral movement of the structure and the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutments, in accordance with the CHBDC 2006.

- Select, free-draining granular fill meeting the specifications of OPSS Granular A or Granular B Type III but with less than 5 per cent passing the No. 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with OPSS.PROV 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150 and 3190.100.
- A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment walls in accordance with CHBDC 2006 Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501.
- The granular fill may be placed either in a zone with a width equal to at least 1.2 metres behind the back of the stem (Case a from Commentary on CHBDC 2006 Figure C6.20) or within the wedge-shaped zone defined by a line drawn at a maximum 1 horizontal to 1 vertical extending up and back from the rear face of the footing (Case b from Commentary on CHBDC 2006 Figure C6.20).
- For Case a, the restrained case, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM).

Soil unit weight: 20 kN/m<sup>3</sup>

Coefficient of lateral earth pressure:  
At rest,  $K_0$  0.50

- For Case b, the unrestrained case, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	<u>GRANULAR A</u>	<u>GRANULAR B</u> <u>Type III</u>
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of lateral earth pressure:		
Active, $K_a$	0.27	0.31
At rest, $K_p$	3.7	3.3

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at rest earth pressures should be assumed for geotechnical design.



- For integral abutments, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.1 of the CHDBC 2006.

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. The lateral earth pressure coefficients should be adjusted if there is sloping ground at the back of the wall.

## **6.4 Construction Considerations**

### **6.4.1 Shallow Foundations**

The cleaned excavation base should be inspected by a Quality Verification Engineer (QVE) qualified in geotechnical engineering prior to placing the working slab. It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation is completed with the geotechnical personnel on site and the working slab placed immediately after footing inspection.

### **6.4.2 Deep Foundations**

Cobbles and boulders should be expected in the soils at the site and may impact operations for pile driving or drilled shafts. The existing soil anchors associated with the sheet pile wall east of the north abutment may also impact drilling operations and their locations should be accurately established prior to construction. A non-standard special provision (NSSP) should be added to the Contract Documents to alert the Contractor to the need for special procedures to deal with cobbles, boulders and other obstructions that may be encountered during pile installation. Slurry based or casing based construction methods will be necessary to advance drilled shafts into the sand and gravel, silt and sand deposits in order to maintain caisson stability.

Deep foundations should be installed and monitored in accordance with OPSS 903, as well as OPSD 3000.150, 3001.150, and SS103-11 (Pile Driving Control) for the driven piles. At a minimum, the H-piles and steel tube piles should be equipped with Type I driving shoes as shown in OPSD 3000.100 and 3001.100, respectively. However, it is preferable that pre-fabricated hardened steel driving shoes be used to better protect the piles from damage while driving through or into the soils containing cobbles and boulders.

## **6.5 Embankments**

It is anticipated that if raising of the embankments is required to accommodate repositioned abutments, the grade raise will be minimal. Further, it is understood that the highway cross section will remain two lanes; however, widening of the approach embankments to the east will be required to accommodate a temporary bridge alignment. For grade raising, if required, or embankment widening, the following should be considered.





All surficial topsoil, organic, loose, soft and/or otherwise deleterious materials should be stripped from areas of proposed embankment widenings. Prior to placement of embankment fill material, the exposed subgrade should be proofrolled under the direction of a geotechnical QVE. The embankment fills should consist of an approved granular borrow such as SSM, except for the top approximately 0.5 to 1.0 metre where pavement base and subbase materials will be placed. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts properly benched into the existing embankments and compacted. Upon completion of filling to the pavement subgrade level, the embankment side slopes should be trimmed to a final inclination of two horizontal to one vertical or flatter. Embankment modifications constructed in this manner are expected to have an adequate factor of safety against instability. Embankments greater than 8 metres in height should be constructed with a 2 metre wide bench at mid-height.

## **6.6 Excavations**

Excavations for the shallow foundations will penetrate the existing fill into the underlying sand and gravel. The groundwater level is expected to be at about elevation 176.5 metres, and will fluctuate seasonally and with the Bayfield River and Lake Huron water level. The excavations for spread footings for the abutments are expected to extend to or below the groundwater level. Groundwater flow from the native granular soils should be expected. It is anticipated that vacuum well points or the like will be required to control the groundwater and river water flows into the excavations in conjunction with properly filtered and constructed sumps. Surface water runoff should be directed away from the excavations at all times.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill materials at this site would be classified as Type 3 soils as would any cohesionless soils below the groundwater level. The native clayey materials, the sandy silt till and properly dewatered cohesionless soils would be classified as Type 2.

### **6.6.1 Temporary Roadway Protection and Cofferdams**

To support the sides of the excavation and permit the use of vertical cuts, temporary road protection systems may be required where space is restricted and will not permit open cuts. Cofferdams or other temporary shoring systems will be required for construction of any in-water piers and removal of the existing abutments which are adjacent to the waterline. The design and limits of the systems are to be determined by the contractor.

Temporary support systems could consist of soldier piles and lagging, where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds, or driven steel sheet piling. Support to the system(s) could be in the form of struts and walers or rakers and anchors. The support system must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as address the impact(s) of sloping ground behind the system.



Installation of vertical members may be affected by cobbles located in the existing embankment fill and cobbles and boulders which are present on the riverbed and present in the glacial till deposits.

If sheet pile cofferdams are used for in-water works, the sheets will need to extend through the sand and gravel and be terminated in the underlying silt or sandy silt till to provide an effective cut-off. Due to the presence of cobbles and boulders within the dense to very dense glacial till and river bed material (substrate), full penetration may not be achieved for all sheets. If sheet piles are terminated at a shallow depth, there may be inadequate embedment for lateral support of the protection system. Alternatives to a sheet pile cofferdam for in-water works include use of dual-chambered inflatable bladder cofferdams and temporary stone dams with an impermeable barrier on the waterside.



## **7.0 RECOMMENDATIONS FOR DETAIL DESIGN**

A Foundation Investigation and Design Report should be prepared during a future assignment to provide appropriate information for Detail Design. A standard MTO foundation investigation for a bridge structure is considered appropriate for the site. Specifically, a minimum of two boreholes should be advanced, one at each abutment, opposite to existing boreholes 806 and 807, and one at each pier location. Also, boreholes will be required at the location of any foundations associated with the temporary bridge alignment to better define the conditions of the existing embankment fill where the temporary foundations and new detour fill will abut or be within the boundaries of the existing fill as well as to define the anticipated relatively shallow river sediments at the location of a future temporary bridge pier. It should be noted that MTO's standard termination criteria of 100+ blows over a minimum distance of 3 metres was not achieved in either the 2014 or previous boreholes drilled at this site. Auger refusal was reported between elevation 167.2 to 168.9 metres in boreholes 1 to 4 of Geocres 40P12-9. This is near the bedrock surface elevation revealed by the geological mapping. Several feasible foundation options can be constructed within the explored depths. Depending on final design, it might be advantageous to prove the underlying bedrock. If bedrock is to be defined it should be cored in NQ size for a minimum depth of 3 metres and strength testing of the core carried out.


The recommendations given in this Preliminary Foundation Design Report should be expanded upon and updated in the Foundation Design Report for detail design in accordance with MTO's standard requirements for foundation engineering assignments. Detailed recommendations should be provided for foundations for the abutments, wingwalls and the central pier, if required. Embankment stability and settlement in the widened approaches should be evaluated. The discussion on temporary roadway protection should include lateral earth pressures and effect of ground conditions on shoring construction design.

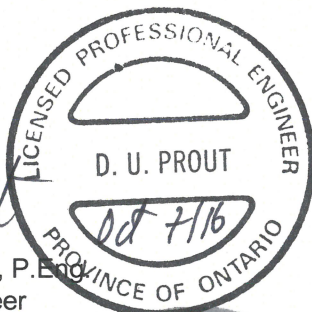



## 8.0 MISCELLANEOUS

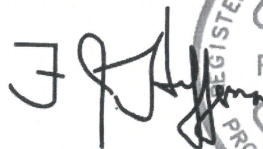
This report was prepared by Ms. Nicole A. Gould, P.Eng. under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. This report was reviewed by Mr. Michael E. Beadle, P.Eng. an Associate and Senior Geotechnical Engineer with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

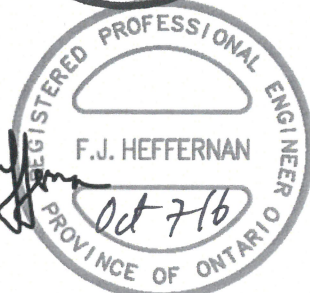
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TABLE I

**COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT**

Highway 21/Bayfield River Bridge  
 Highways 401, 4 and 21 Structural Replacements  
GWP 3070-11-00

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>RELATIVE COSTS<sup>1</sup></b>	<b>RISKS/ CONSEQUENCES</b>
Spread footings supported on compact to dense sand and gravel or engineered fill.	<ul style="list-style-type: none"> <li>• Feasible for abutments and central pier(s).</li> </ul>	<ul style="list-style-type: none"> <li>• Least expensive option.</li> <li>• Ease of construction for abutments.</li> </ul>	<ul style="list-style-type: none"> <li>• Not compatible with integral abutments.</li> <li>• More settlement expected than with deep foundations.</li> <li>• Larger work area required compared to caissons or driven piles.</li> <li>• Pier construction will require a cofferdam</li> </ul>	<ul style="list-style-type: none"> <li>• Low</li> </ul>	<ul style="list-style-type: none"> <li>• Relatively low risk.</li> <li>• Deeper excavations required if soil at founding elevation is unsuitable.</li> <li>• Dewatering requirements.</li> </ul>
End bearing steel H-pile or steel tube pile foundations.	<ul style="list-style-type: none"> <li>• Considered feasible for abutments and central pier</li> </ul>	<ul style="list-style-type: none"> <li>• High bearing resistance.</li> <li>• Negligible settlement.</li> <li>• Compatible with integral abutments.</li> </ul>	<ul style="list-style-type: none"> <li>• More expensive than shallow foundations.</li> <li>• Driving may be difficult due to presence of cobbles and boulders.</li> </ul>	<ul style="list-style-type: none"> <li>• Moderate</li> </ul>	<ul style="list-style-type: none"> <li>• Potential presence of cobbles and boulders could interfere with driving operations</li> </ul>

**COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT**

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>RELATIVE COSTS<sup>1</sup></b>	<b>RISKS/ CONSEQUENCES</b>
<ul style="list-style-type: none"> <li>Concrete caissons drilled into very stiff to hard clayey silt till or very dense sandy silt till.</li> </ul>	<ul style="list-style-type: none"> <li>Feasible for abutments and pier but not preferred.</li> </ul>	<ul style="list-style-type: none"> <li>Negligible settlement.</li> <li>Faster construction and less work space required compared to shallow foundations.</li> </ul>	<ul style="list-style-type: none"> <li>Not compatible with integral abutments.</li> <li>Drilling may be difficult due to presence of cobbles and boulders.</li> <li>Cannot be visually inspected at depth due to health and safety regulations.</li> </ul>	<ul style="list-style-type: none"> <li>High</li> </ul>	<ul style="list-style-type: none"> <li>Cleaning of base could be problematic or overlooked during construction.</li> </ul>

- NOTES:
- The estimated relative costs are intended to provide a comparison between alternatives rather than actual construction costs, and do not take into consideration additional costs such as traffic control, staging and shoring that may influence the total cost.
  - Table to be read in conjunction with accompanying report.

Prepared By: NG  
 Checked By: DUP

## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.)

#### Consistency

	<u>kPa</u>	<u>psf</u>
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

#### (b) Cohesive Soils

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
$SO_4$	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note:** 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

### I. General

$\pi$	3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

#### (a) Index Properties (continued)

w	water content
$w_l$	liquid limit
$w_p$	plastic limit
$I_p$	plasticity index $= (w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index $= (w - w_p) / I_p$
$I_C$	consistency index $= (w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

#### (b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

#### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_a$	coefficient of secondary consolidation
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction $= \tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 + \sigma_3)$
$S_t$	sensitivity

- Notes:**
- 1  $\tau = c' + \sigma' \tan \phi'$
  - 2 shear strength = (compressive strength)/2
  - \* density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density x acceleration due to gravity)



**RECORD OF BOREHOLE No 805**

1 OF 2

**METRIC**

PROJECT 12-1132-0076  
W.P. 3070-11-00 LOCATION N 4825793.9 , E 369497.5 ORIGINATED BY MA  
DIST HWY 21 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF  
DATUM GEODETIC DATE September 29 - October 1, 2014 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE									
185.74	PAVEMENT SURFACE							20	40	60	80	100						
0.00	ASPHALTIC CONCRETE																	
0.18	FILL, sand and gravel, crushed						Concrete											
0.43	FILL, clayey silt, some sand, trace to some gravel Stiff Brown						Sand Filter											
		1	SS	15														
		2	SS	9														
		3	SS	15														
		4	SS	9														
181.02	FILL, sand and gravel, some silt Compact Brown						Backfill											
		5	SS	8														
		6	SS	15														
		7	SS	19														
		8	SS	11														
180.56	FILL, clayey silt, some sand to sandy, trace to some gravel Stiff to very stiff Brown and grey						Bentonite											
		9	SS	9														
		10	SS	12														
		11	SS	18														
		12	SS	13														
177.51	FILL, clayey silt, some sand, trace gravel, with topsoil pockets and layers Stiff to very stiff Brown						Bentonite											
		13	SS	9														
		14	SS	8														
		15	SS	32														
		16	SS	100/100mm														
175.22	SAND AND GRAVEL, trace to some silt Loose to very dense Grey						Well Screen											
		17	SS	34														
		18	SS	100/100mm														
		19	SS	32														
		20	SS	100/100mm														
172.33	CLAYEY SILT TILL, sandy, trace to some gravel Hard Grey						Bentonite											
		21	SS	34														
		22	SS	34														
		23	SS	34														
		24	SS	34														
170.96																		
14.78																		

Continued Next Page

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

PROJECT <u>12-1132-0076</u>		<b>RECORD OF BOREHOLE No 805</b>		2 OF 2		<b>METRIC</b>	
W.P. <u>3070-11-00</u>		LOCATION <u>N 4825793.9 , E 369497.5</u>		ORIGINATED BY <u>MA</u>			
DIST <u>          </u> HWY <u>21</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM</u>		COMPILED BY <u>WDF</u>			
DATUM <u>GEODETIC</u>		DATE <u>September 29 - October 1, 2014</u>		CHECKED BY <u>          </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)							
								20   40   60   80   100	20   40   60   80   100	10   20   30	GR   SA   SI   CL						
169.59	SILTY CLAY TILL, trace to some sand, trace gravel, with silty sand partings Very stiff Grey		18	SS	18		170							2	10	38	50
16.15																	
168.37	CLAYEY SILT TILL, sandy, trace gravel Hard Grey		19	SS	106		169							3	31	49	17
17.37																	
	END OF BOREHOLE		20	SS	100/ 0mm												
	Groundwater encountered at about elev. 180.9m during drilling on Sept. 29, 2014.																
	Water level measured in Well at elev. 176.49m following installation on Oct. 1, 2014.																
	Water level measured in Well at elev. 176.52m following installation on Feb. 19, 2015.																

LDN\_MTO\_06 12-1132-0076-3001-R01.GPJ LDN\_MTO\_GDT 21/07/15

**RECORD OF BOREHOLE No 806**

1 OF 2

**METRIC**

PROJECT 12-1132-0076  
W.P. 3070-11-00 LOCATION N 4825783.7, E 369508.0 ORIGINATED BY MA  
DIST HWY 21 BOREHOLE TYPE POWER AUGER, HOLLOW STEM, TRICONE COMPILED BY WDF  
DATUM GEODETIC DATE September 30 - October 2, 2014 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			GR	SA	SI	CL
								20	40	60	80	100					20	40	60				
185.80	PAVEMENT SURFACE																						
0.00	CONCRETE																						
0.15	FILL, sand and gravel, crushed, trace silt, with pieces of concrete Loose Brown																						
			1	SS	9								○					33	55 (12)				
			2	SS	5								○										
			3	SS	10								○										
182.90	FILL, clayey silt, some sand, some gravel, with pieces of wood Firm Brown		4	SS	8								○										
182.14	FILL, sand and gravel, crushed, trace silt Compact to very dense Brown		5	SS	38								○					56	32 (12)				
			6	SS	52								○										
			7	SS	23								○										
180.01	FILL, clayey silt, some sand, trace gravel Very stiff Brown		8	SS	20								○	┌───┐				2	14 56 28				
			9	SS	28								○										
			10	SS	28								○										
			11	SS	25								○										
177.27	FILL, silty topsoil, some clay, with roots and organics, some sand, trace gravel Compact Brown and black		12	SS	13								○			41							
176.75	FILL, sand, fine, some silt Compact Brown		13	SS	14								○										
176.05	SAND AND GRAVEL, trace silt Compact to dense Grey		14	SS	20								○					72	23 (5)				
			15	SS	38								○										
			16	SS	37								○										
173.30	CLAYEY SILT TILL, some sand, trace gravel Very stiff Grey		17	SS	18								○										
171.93	SILTY CLAY TILL, trace sand, with sandy silt pockets Hard Grey		18	SS	38									┌───┐				0	4 47 49				
																44							

Continued Next Page

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

LDN\_MTO\_06 12-1132-0076-3001-R01.GPJ LDN\_MTO\_GDT 21/07/15

PROJECT <u>12-1132-0076</u>		<b>RECORD OF BOREHOLE No 806</b>		2 OF 2		<b>METRIC</b>	
W.P. <u>3070-11-00</u>		LOCATION <u>N 4825783.7 , E 369508.0</u>		ORIGINATED BY <u>MA</u>			
DIST <u>          </u> HWY <u>21</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM, TRICONE</u>		COMPILED BY <u>WDF</u>			
DATUM <u>GEODETIC</u>		DATE <u>September 30 - October 2, 2014</u>		CHECKED BY <u>          </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W <sub>p</sub>	W	W <sub>L</sub>		GR	SA	SI	CL	
								○ UNCONFINED	+	FIELD VANE	WATER CONTENT (%)									
						● QUICK TRIAXIAL	×	LAB VANE	20	40	60	80	100	10	20	30				
169.65	SILTY CLAY TILL, trace sand, with sandy silt pockets Hard Grey																			
16.15	SANDY SILT TILL, some gravel, trace to some clay, with clayey silt pockets, with cobbles and boulders Very dense Grey																			
			19	SS		100/ 50mm								○						
			20	RC																
166.90	END OF BOREHOLE																			
18.90	Groundwater encountered at about elev. 177.4m during drilling on Sept. 30, 2014.																			

LDN\_MTO\_06 12-1132-0076-3001-R01.GPJ LDN\_MTO\_GDT 21/07/15

**RECORD OF BOREHOLE No 807**

1 OF 2

**METRIC**

PROJECT 12-1132-0076  
W.P. 3070-11-00 LOCATION N 4825729.6 , E 369567.9 ORIGINATED BY DJM  
DIST HWY 21 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF  
DATUM GEODETIC DATE December 19, 2014 CHECKED BY

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	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE							w <sub>p</sub> w      w <sub>L</sub>		
185.86	PAVEMENT SURFACE					▽	20	40	60	80	100	10	20	30	GR   SA   SI   CL		
0.00	CONCRETE																
0.12	FILL, sand and gravel Compact Brown		1	SS	22		185										
184.49	FILL, sandy silt, some clay, trace to some gravel, with pockets and layers of clayey silt, sand and gravel Loose to compact Brown		2	SS	7		184										
1.37			3	SS	5		183										
			4	SS	8		182										
182.20	FILL, sand and gravel, crushed, trace to some silt, with silt layers and cobbles Compact Brown		5	SS	13		181										
3.66			6	SS	15		180										
			7	SS	29		179										
			8	SS	19		178										
177.17	SILT, trace to some sand, trace gravel, with roots Compact Brown		9	SS	11		177										
176.11	SAND AND GRAVEL, Compact to dense Brown		10	SS	17		176										
9.75		11	SS	33	175												
173.06	SILT, trace to some clay, some sand, trace gravel Dense to very dense Grey	12	SS	47	172												
12.80					171												

Continued Next Page

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

LDN\_MTO\_06 12-1132-0076-3001-R01.GPJ LDN\_MTO\_GDT 21/07/15

**RECORD OF BOREHOLE No 807**

2 OF 2

**METRIC**

PROJECT 12-1132-0076  
W.P. 3070-11-00 LOCATION N 4825729.6 , E 369567.9 ORIGINATED BY DJM  
DIST HWY 21 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF  
DATUM GEODETIC DATE December 19, 2014 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
							20 40 60 80 100													
170.32			13	SS	59															
15.54	SAND, fine to medium, trace silt Very dense Grey																			
169.71																				
16.15	CLAYEY SILT TILL, sandy, trace gravel Hard Grey		14	SS	51															
			15	SS	45															
166.66																				
19.20	SANDY SILT TILL, some gravel, with cobbles Very dense Grey																			
165.96			16	SS	100/ 100mm															
19.90	END OF BOREHOLE																			
	Groundwater encountered at about elev. 175.2m during drilling on Dec. 19, 2014.																			

PROJECT 12-1132-0076

W.P. 3070-11-00

LOCATION N 4825710.5 , E 369589.3

ORIGINATED BY MA

DIST HWY 21

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY WDF

DATUM    GEODETIC

DATE December 18, 2014

CHECKED BY

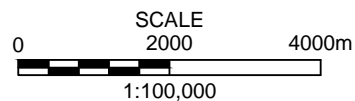
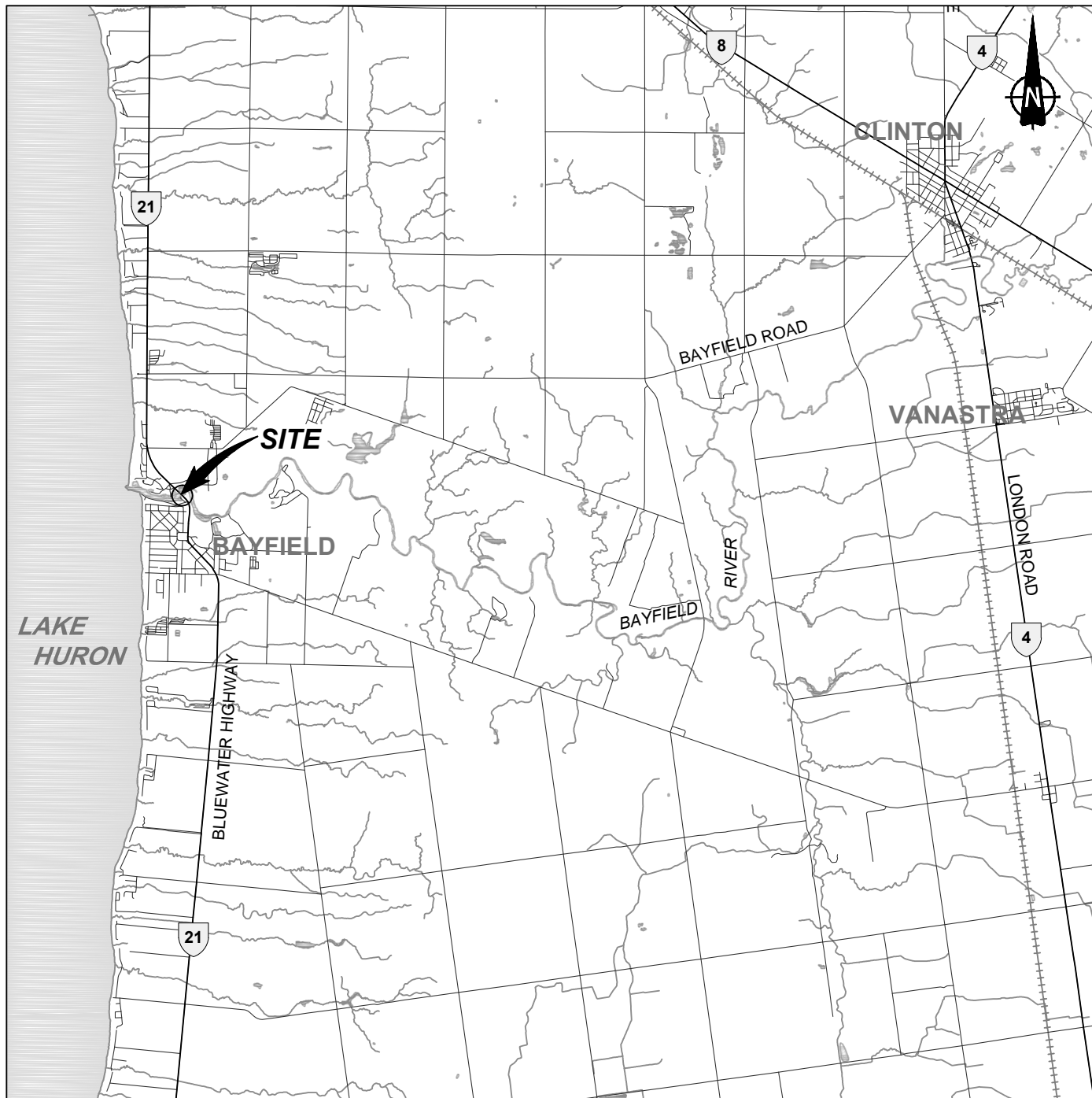
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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

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





## REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.

## NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

ALL LOCATIONS ARE APPROXIMATE.

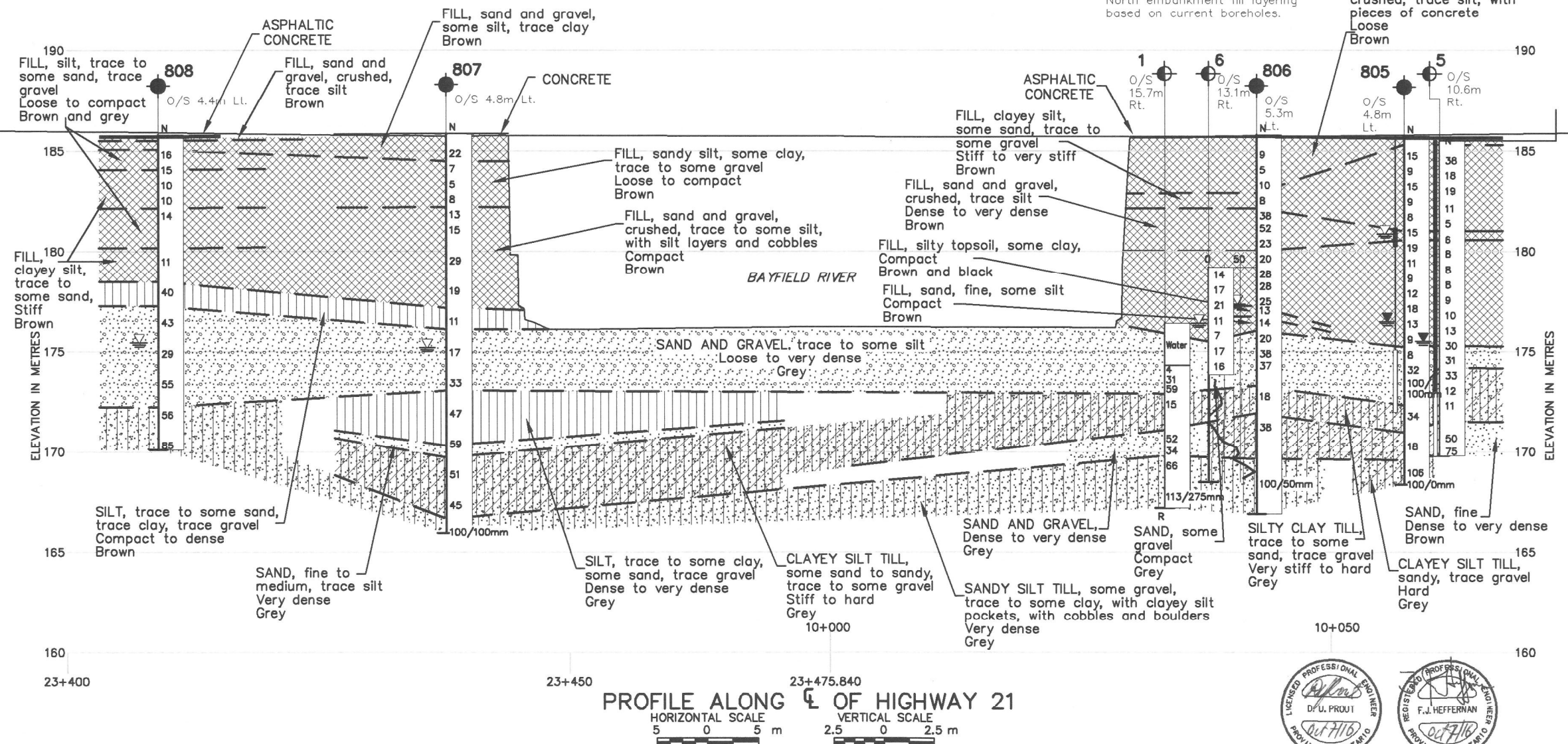
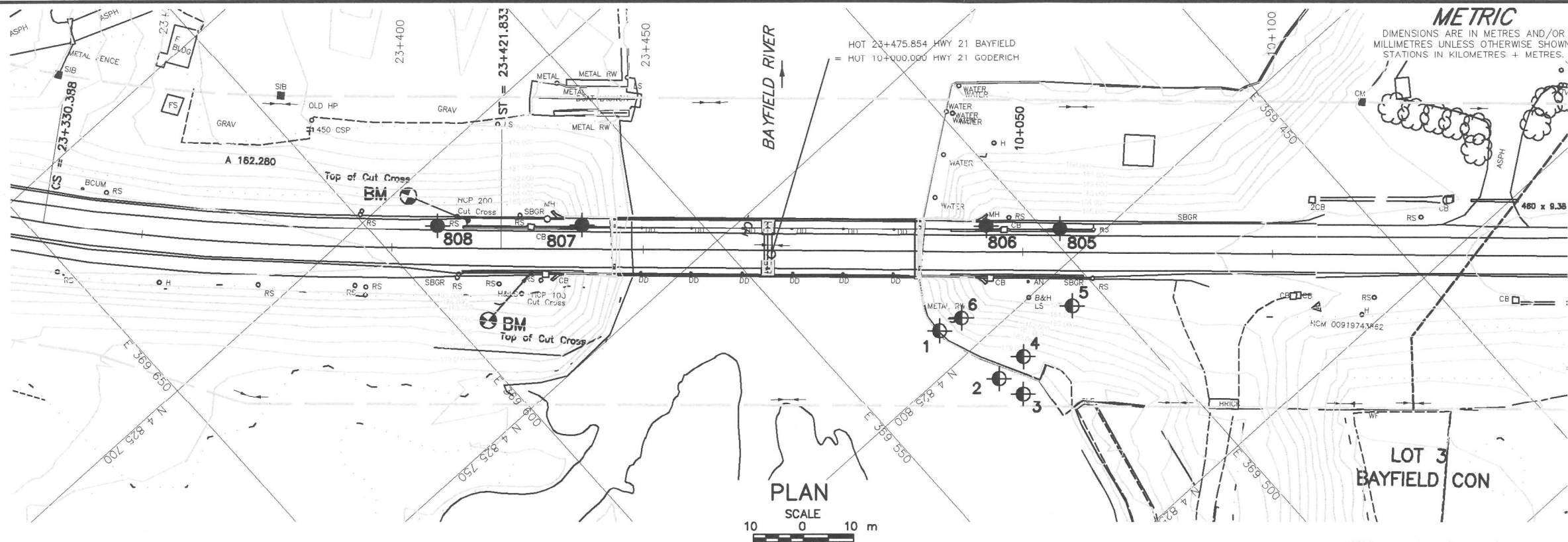
PROJECT HIGHWAY 21 / BAYFIELD RIVER BRIDGE REPLACEMENT  
SITE 12-195  
HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
GWP 3070-11-00

TITLE

## KEY PLAN



PROJECT No. 12-1132-0076			FILE No. 1211320076-3001-F01001	
CADD	WDF	Jan. 30/15	SCALE AS SHOWN	REV. 0
CHECK			<b>FIGURE 1</b>	



CONT No.  
 WP No. 3070-11-00

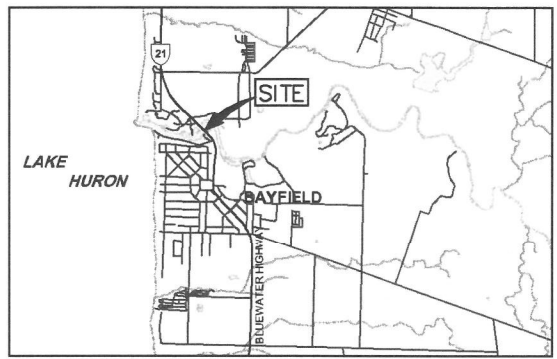


**BAYFIELD RIVER BRIDGE**  
 HIGHWAY 21, HIGHWAYS 401, 4 AND 21 STRUCTURAL  
 REPLACEMENTS  
 BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



**Golder Associates Ltd.**  
 LONDON, ONTARIO, CANADA



### LEGEND

- Borehole - Current Investigation
- Borehole - Geocres No. 40P12-9
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Standpipe
- R Refusal
- WL upon completion of drilling
- WL in Piezometer (Oct. 1, 2014)
- DRY Borehole dry during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
805	185.74	4 825 793.9	369 497.5
806	185.80	4 825 783.7	369 508.0
807	185.86	4 825 729.5	369 567.9
808	185.80	4 825 710.5	369 589.3
(Geocres No. 40P12-9)			
1	174.3	4 825 793.0	369 529.0
2	174.5	4 825 808.0	369 526.5
3	174.7	4 825 813.5	369 525.0
4	177.4	4 825 808.0	369 520.0
5	185.5	4 825 807.0	369 506.0
6	179.2	4 825 794.0	369 524.0
(Inferred from DWG. 1, 40P12-9)			

### NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.  
 The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

### REFERENCE

Base plans provided in digital format by Dillon.

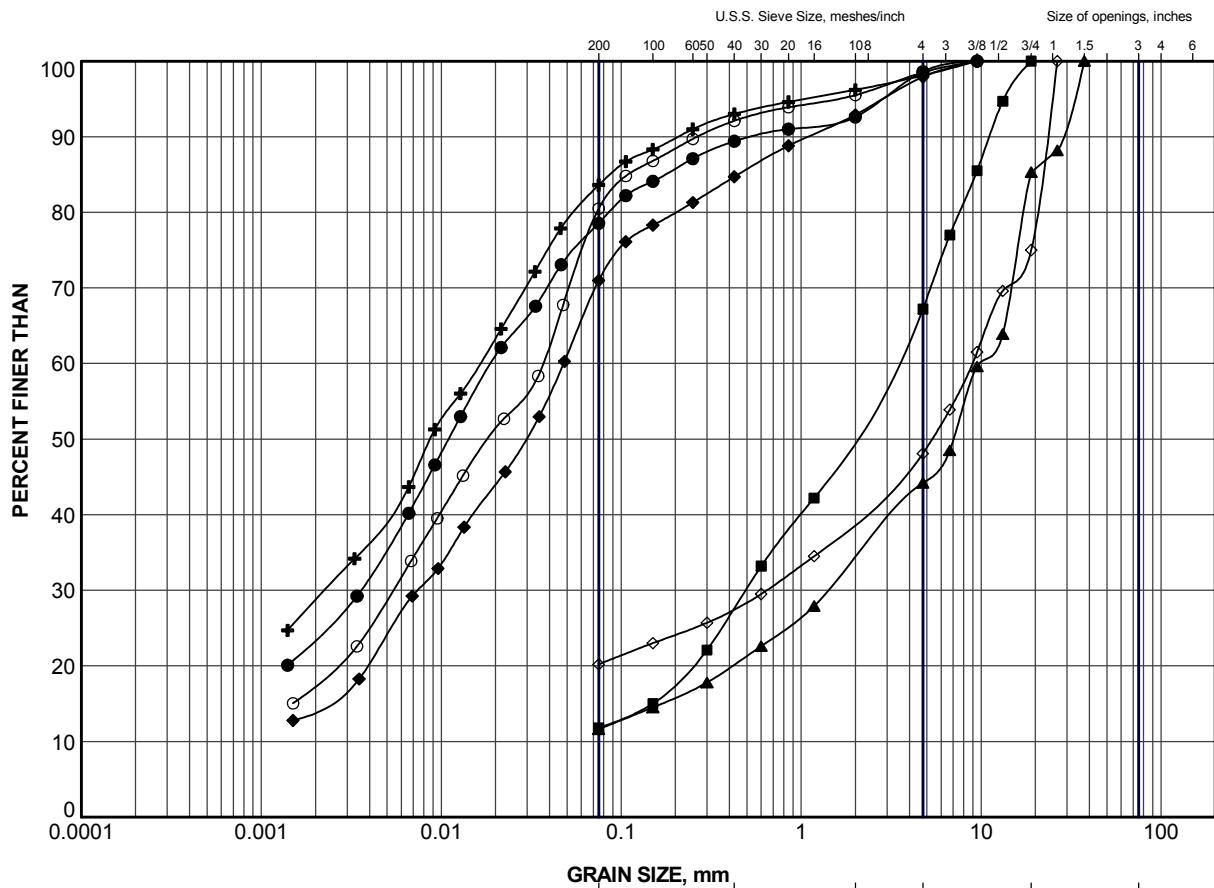
NO.	DATE	BY	REVISION
Geocres No. 40P12-34			
HWY.	21	PROJECT NO.	12-1132-0076
SUBM'D.	NAG	CHKD.	NAG
DRAWN:	LMK\WDF	CHKD.	DUP
DATE:	Jan. 30/15	APPD.	FJH
SITE:	12-188	DWG.	1





# APPENDIX A

## Laboratory Test Data



CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	805	9	178.7
■	806	1	184.8
▲	806	5	181.8
+	806	8	179.6
◆	807	4	182.6
◇	807	7	179.5
○	808	3	183.3

PROJECT  
HIGHWAY 21 / BAYFIELD RIVER BRIDGE REPLACEMENT  
HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
GWP 3070-11-00

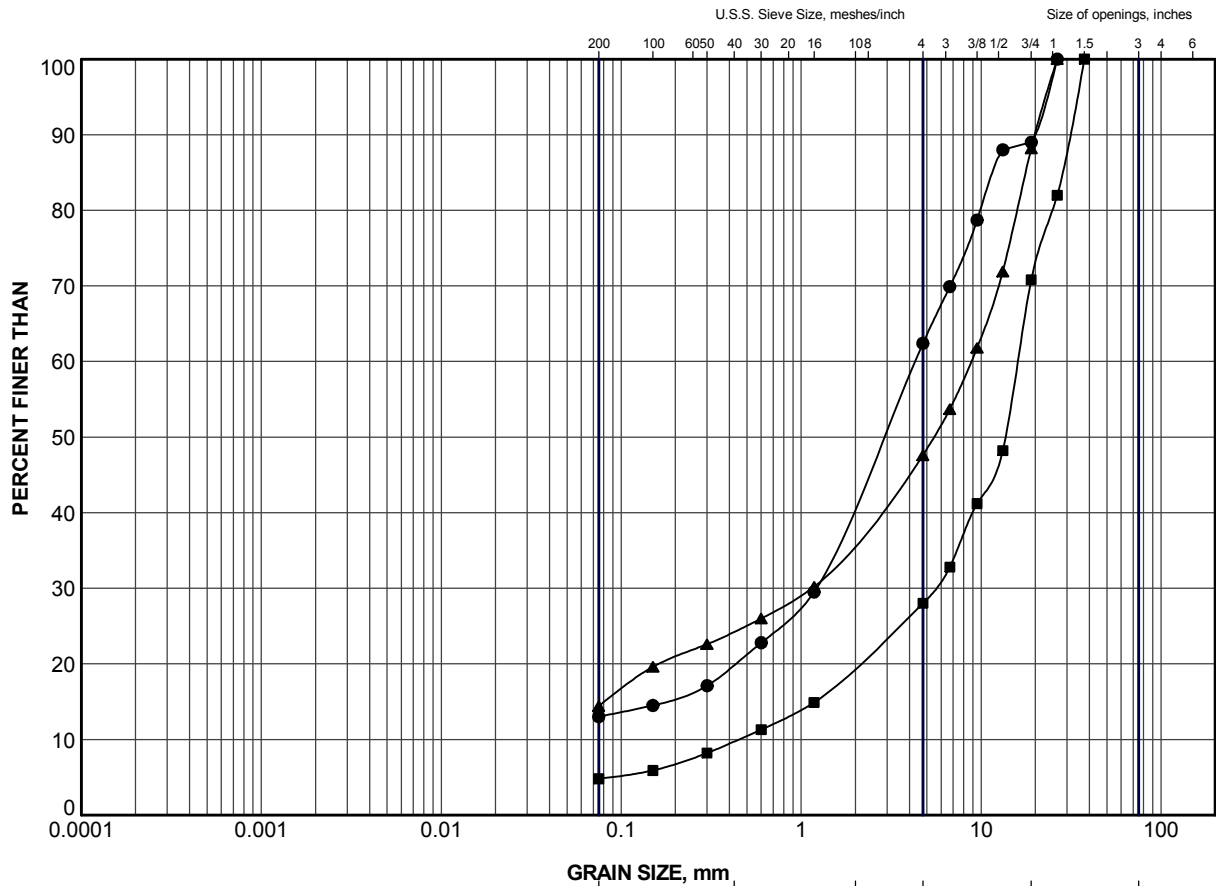
TITLE

## GRAIN SIZE DISTRIBUTION FILL



PROJECT No.	12-1132-0076	FILE No. 1211320076-3001-F010A1
DRAWN	WDF	Feb 09/15
CHECK		
SCALE	N/A	REV.

**FIGURE A-1**



CLAY AND SILT	SAND SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	805	14	174.8
■	806	14	175.7
▲	808	11	173.4

PROJECT  
**HIGHWAY 21 / BAYFIELD RIVER BRIDGE REPLACEMENT  
 HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
 GWP 3070-11-00**

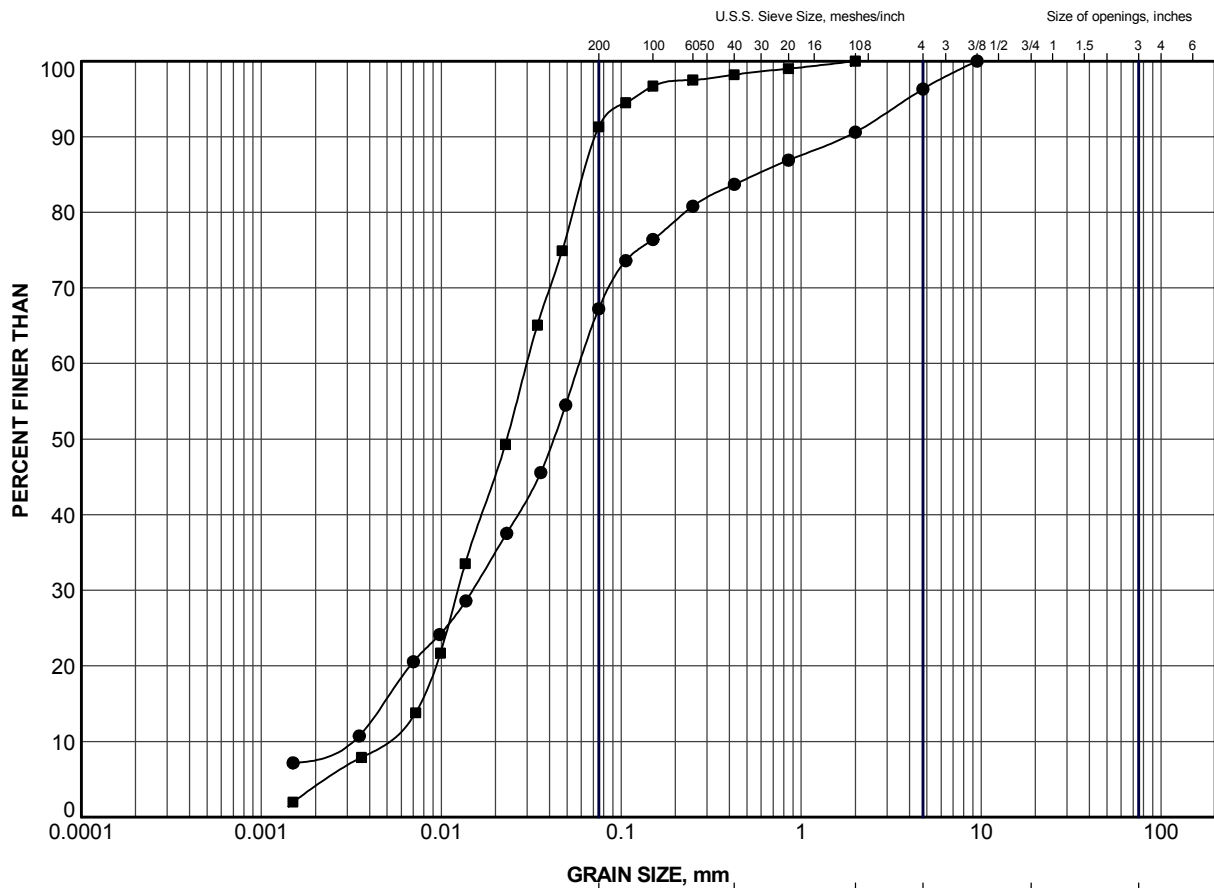
TITLE

## GRAIN SIZE DISTRIBUTION SAND AND GRAVEL



PROJECT No.	12-1132-0076	FILE No. 1211320076-3001-F010A2
DRAWN	WDF	Feb 09/15
CHECK		
SCALE	N/A	REV.


**FIGURE A-2**

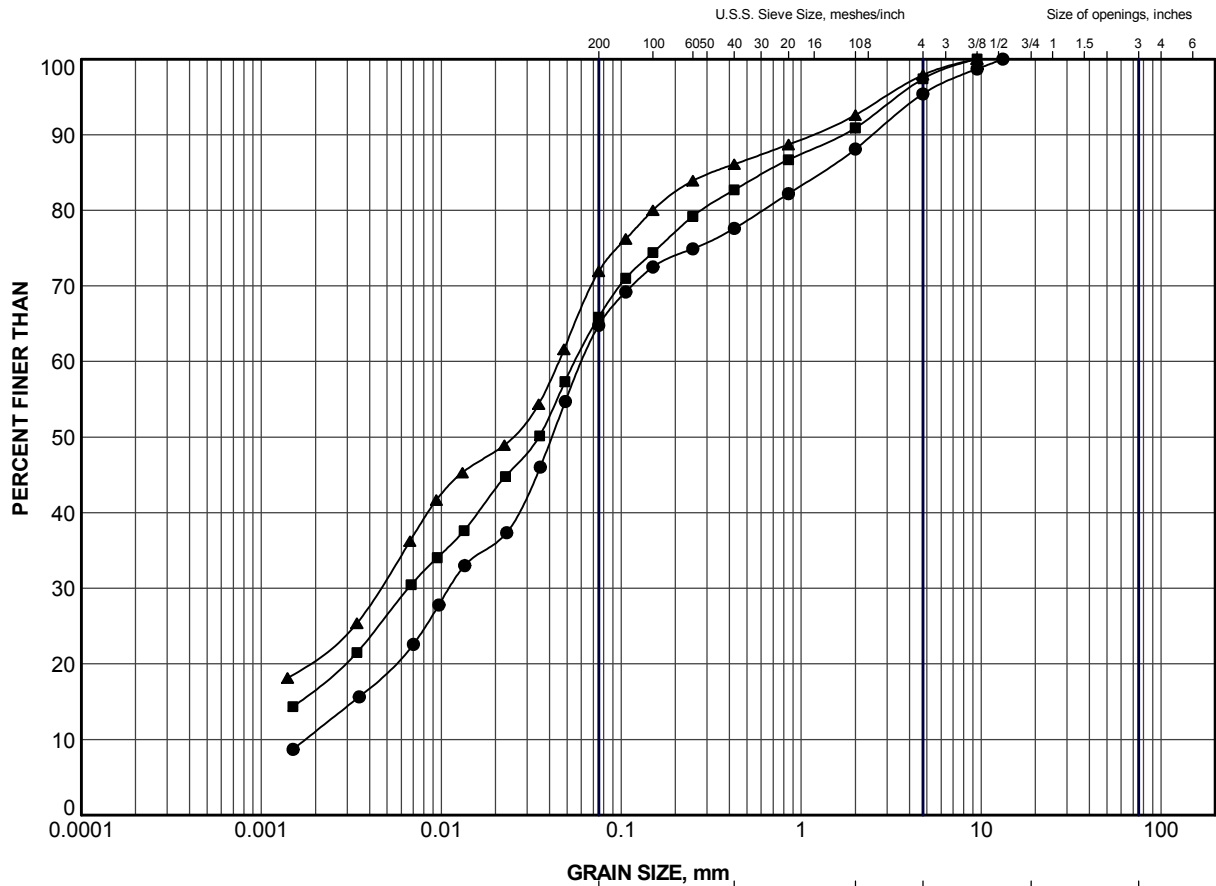


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	807	12	171.9
■	808	8	178.0

PROJECT HIGHWAY 21 / BAYFIELD RIVER BRIDGE REPLACEMENT HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS GWP 3070-11-00				
TITLE <b>GRAIN SIZE DISTRIBUTION</b> <b>SILT</b>				
	PROJECT No. 12-1132-0076		FILE No. 1211320076-3001-F010A3	
	DRAWN	WDF	Feb 09/15	SCALE N/A REV.
	CHECK			<b>FIGURE A-3</b>



CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	805	17	171.8
■	805	19	168.8
▲	807	15	167.3

PROJECT  
HIGHWAY 21 / BAYFIELD RIVER BRIDGE REPLACEMENT  
HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
GWP 3070-11-00

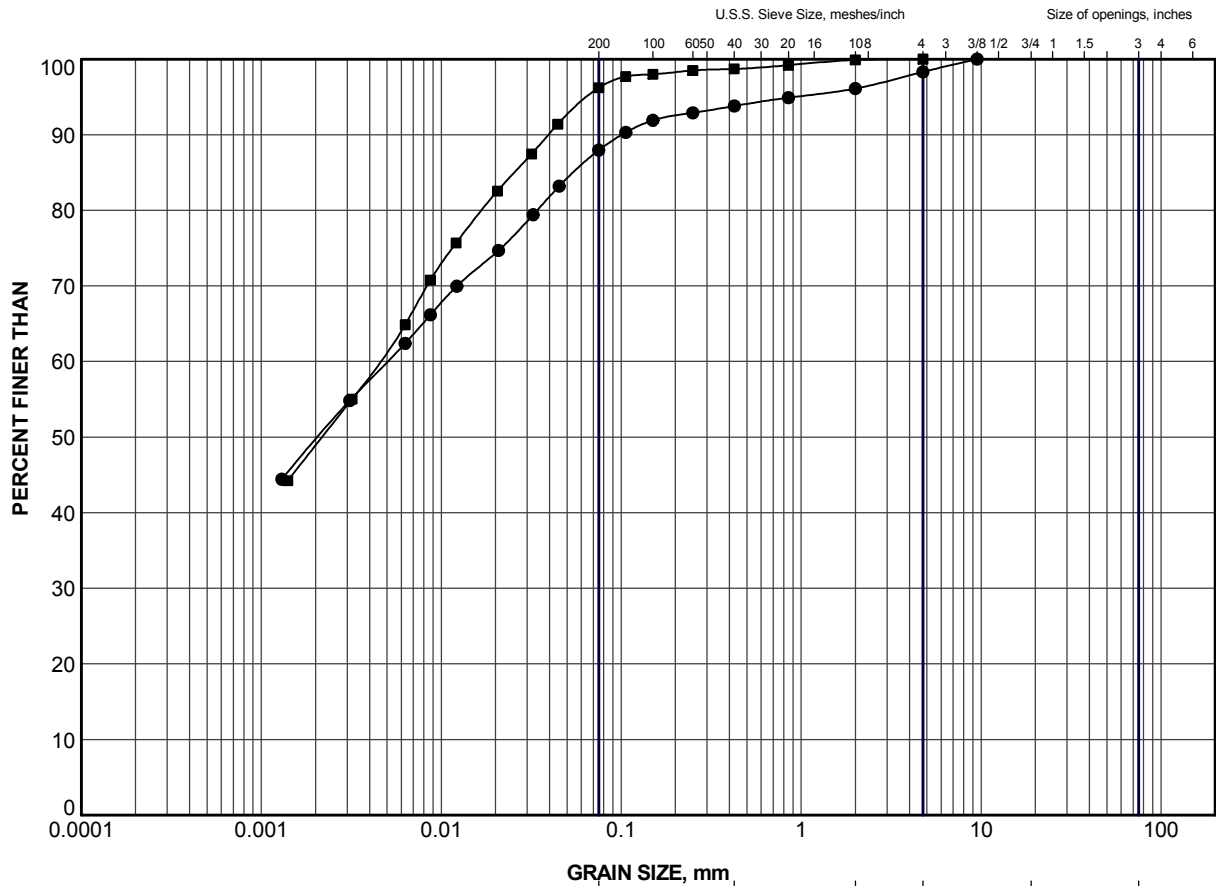
TITLE

## GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL



PROJECT No.	12-1132-0076	FILE No.	1211320076-3001-F010A4
DRAWN	WDF	Feb 09/15	SCALE N/A REV.
CHECK			

**FIGURE A-4**



CLAY AND SILT	GRAVEL SIZE, mm						Cobble Size
	fine	medium	coarse	fine	coarse		
	SAND SIZE			GRAVEL SIZE			

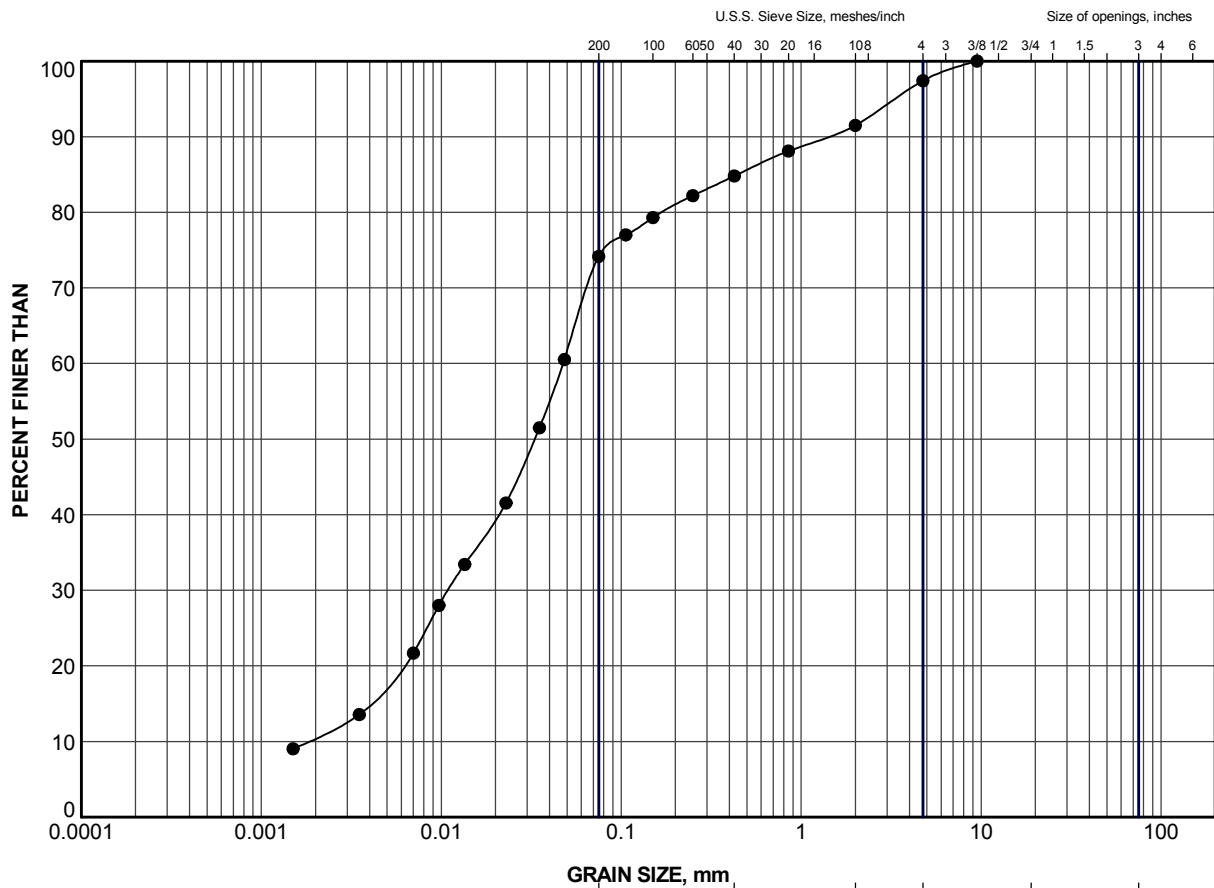
### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	805	18	170.3
■	806	18	171.2

PROJECT HIGHWAY 21 / BAYFIELD RIVER BRIDGE REPLACEMENT HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS GWP 3070-11-00			
TITLE <b>GRAIN SIZE DISTRIBUTION</b> <b>SILTY CLAY TILL</b>			
PROJECT No. 12-1132-0076		FILE No. 1211320076-3001-F010A5	
DRAWN	WDF	Feb 09/15	SCALE N/A REV.
CHECK			<b>FIGURE A-5</b>






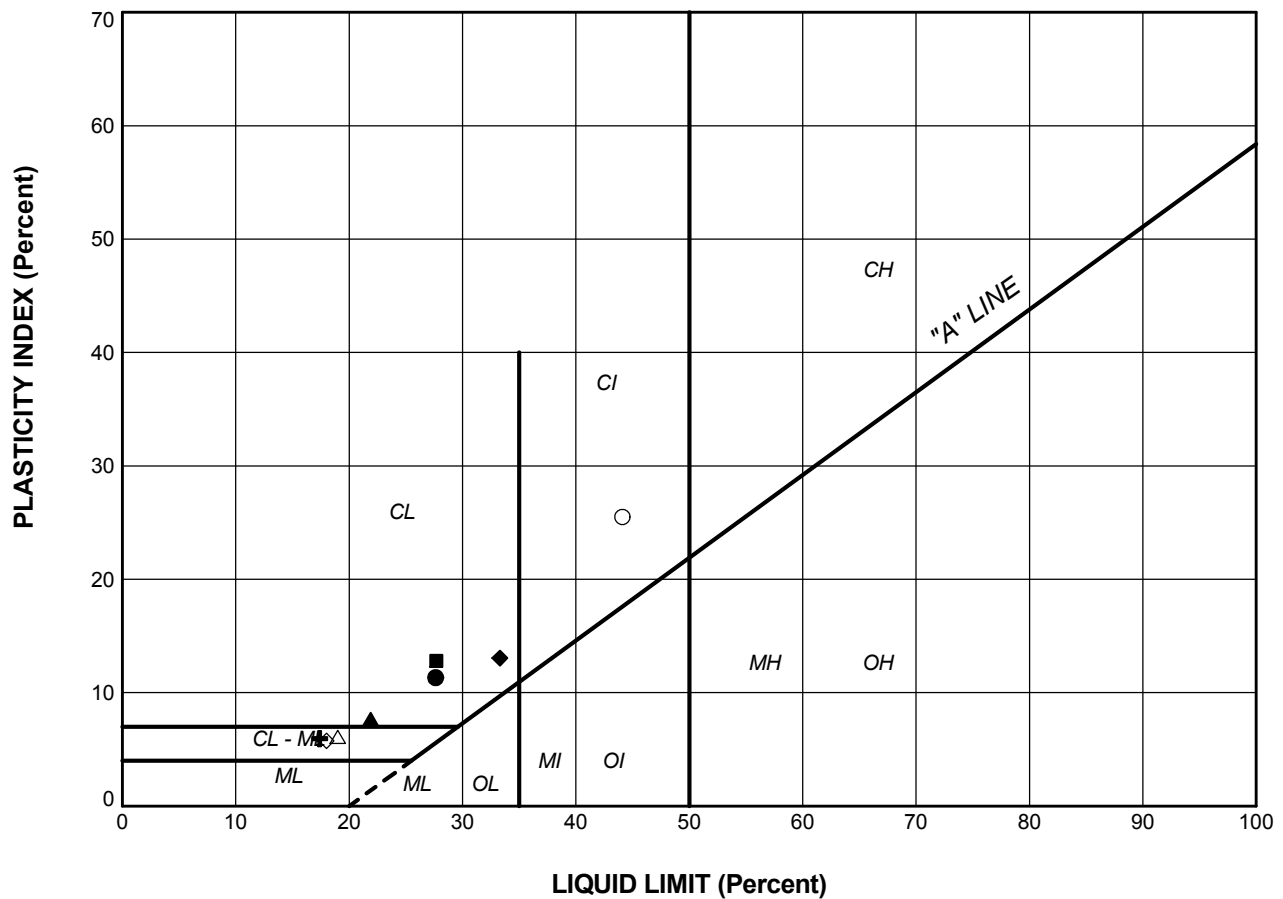


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	808	12	171.9

PROJECT HIGHWAY 21 / BAYFIELD RIVER BRIDGE REPLACEMENT HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS GWP 3070-11-00			
TITLE <h2 style="text-align: center;">GRAIN SIZE DISTRIBUTION</h2> <h3 style="text-align: center;">SANDY SILT TILL</h3>			
	PROJECT No. 12-1132-0076		FILE No. 1211320076-3001-F010A6
	DRAWN WDF	Feb 09/15	SCALE N/A REV.
	CHECK		
			<b>FIGURE A-6</b>

LDN\_MTO\_GSD\_GLDR\_LDN.GDT 09/02/15



### LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	805	9	27.6	16.3	11.3 (FILL)
■	806	8	27.7	14.9	12.8 (FILL)
▲	808	3	21.9	14.3	7.7 (FILL)
+	805	17	17.4	11.5	6.0
◆	805	18	33.3	20.3	13.1
◇	805	19	18.0	12.3	5.8
○	806	18	44.1	18.6	25.5
△	807	15	19.0	12.9	6.1

PROJECT  
 HIGHWAY 21 / BAYFIELD RIVER BRIDGE REPLACEMENT  
 HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS  
 GWP 3070-11-00

TITLE

## PLASTICITY CHART



PROJECT No.	12-1132-0076	FILE No.	1211320076-3001-F010A7
DRAWN	WDF	Feb 09/15	SCALE N/A REV.
CHECK			

**FIGURE A-7**



# APPENDIX B

## Geocres Report No. 40P12-9 Record of Borehole Sheets and Laboratory Results

# RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 348-98-00 LOCATION SITE No. 12-188, Bayfield River Bridge, Hwy. 21 ORIGINATED BY SEW  
 DIST 31 HWY 21 BOREHOLE TYPE H/S Auger with casing COMPILED BY SEW  
 DATUM Geodetic DATE 16.09.99 - 17.09.99 CHECKED BY EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N <sup>o</sup> VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
176.4	Water						20	40	60	80	100							
0.0																		
174.3	CHANNEL BED at 2.05m BELOW WATER LEVEL on 9/16/99						176											
2.1	Loose, grey SANDY GRAVEL SEDIMENT saturated		1	SS	4		175											
173.7																		
2.7	Dense to very dense, grey SANDY GRAVEL saturated		2	SS	31		174									67 27 (6)		
172.9			3	SS	59		173											
3.5	Stiff, light brown CLAYEY SILT, trace sand, trace to some gravel very moist		4	SS	15		172									20.5	0 7 63 30	
171.1			5	ST														
5.3	Very dense to dense, grey SAND and GRAVEL saturated		6	ST			171											
169.8			7	SS	52													
6.8	Very dense, grey/brown SILT TILL, some clay, gravel and sand layers moist		8	SS	34		170											
			9	SS	66		169									23.0	14 24 46 16	
167.2																		
9.2	AUGER REFUSAL at 9.2 m		10	SS13/275mm			168											

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

EXPRESS. 12-188 GPU EXPRESS GDT 13/12/99

# RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 348-98-00 LOCATION SITE No. 12-188, Bayfield River Bridge, Hwy. 21 ORIGINATED BY SEW  
 DIST 31 HWY 21 BOREHOLE TYPE H/S Auger with casing COMPILED BY SEW  
 DATUM Geodetic DATE 16.09.99 - 16.09.99 CHECKED BY EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	w <sub>p</sub>	w	w <sub>L</sub>		
178.4	Water													GR SA SI CL
0.0														
174.5	CHANNEL BED at 1.85m BELOW WATER LEVEL on 9/16/99													
1.9	Firm, grey CLAYEY SILT SEDIMENT, some sand, trace gravel, some wood fragments saturated		1	SS	6									
174.1														
2.3	Firm, grey CLAYEY SILT, trace to some gravel moist to very moist													
			2	SS	5									
			3	SS	7									
171.5														
4.9	Compact to dense, grey SAND and GRAVEL saturated		4	SS	57									
			5	CT	0									
			6	CT	14									
			7	CT	33									
			8	CT	30									
			9	CT	25									
169.5														
6.9	Very dense, grey SILT TILL, some clay, sand and gravel very moist		10	CT	50									
168.8														
7.6	Very dense, grey SAND and GRAVEL saturated		11	SS 58/170mm										
168.1														
8.3	AUGER REFUSAL at 8.3 m NOTE* Artesian condition rose to 0.8 m above water level of river for approximately 10 minutes after sample 4													

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# RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 348-98-00 LOCATION SITE No. 12-188, Bayfield River Bridge, Hwy. 21 ORIGINATED BY SEW  
 DIST 31 HWY 21 BOREHOLE TYPE H/S Auger COMPILED BY SEW  
 DATUM Geodetic DATE 20.09.99 - 20.09.99 CHECKED BY EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20	40							60	80	100
								○ UNCONFINED	● FIELD VANE							● QUICK TRIAXIAL	× LAB VANE	
177.4	Ground																	
0.0	Compact to loose, dark brown silt and gravel FILL, some fine sand and cobbles						177											
	moist		1	SS	8		176											
			2	SS	4		175											
175.0	wood inclusions						175											
2.5	saturated		3	SS	5		174											
174.7	Very dense, dark grey coarse SAND and GRAVEL saturated		4	SS	51		173											
2.7							172											
173.8	Silty, some cobbles		5	SS	50/125mm		171											
3.8							170											
173.0	Soft to firm, brown CLAYEY SILT, trace to some gravelly silt layers very moist		6	SS	8		169											
4.4			7	ST			168											
171.3	Dense, brown SILT TILL damp		8	SS	49													
6.1																		
170.9	Very dense, grey coarse SAND and GRAVEL saturated		9	SS	56													
6.6																		
168.2	Very dense, greyish brown SILT TILL, trace to some sand damp																	
9.3																		
167.4			10	SS	50/100mm													
10.0	AUGER REFUSAL at 10.0 m NOTE* Water level encountered at 176.4 m 3 hrs after completion																	

EXPRESS 12-188.GPJ EXPRESS GDT 13/12/99

# RECORD OF BOREHOLE No 5

1 OF 1

METRIC

W.P. 348-98-00 LOCATION SITE No. 12-188, Bayfield River Bridge, Hwy. 21 ORIGINATED BY SEW  
 DIST 31 HWY 21 BOREHOLE TYPE H/S Auger COMPILED BY SEW  
 DATUM Geodetic DATE 21.09.99 - 21.09.99 CHECKED BY EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
185.5	Ground																
0.0	Dense to compact, dark brown silty sand and gravel FILL, large asphalt pieces damp		1	SS	38		185										55 40 (5)
			2	SS	18		184										19.5
183.2	Compact, dark brown silt and sand FILL, some gravel, glass fragments damp		3	SS	19		183										22.8 28 56 (18)
2.3			4	SS	11		182										
181.7	Firm to stiff, greyish brown clay and silt FILL, occasional gravel, some dark grey staining moist		5	SS	5		181										18.5
3.8			6	SS	6		180										19.5
			7	SS	8		179										
			8	SS	8		178										
			9	SS	8		177										21.5
			10	SS	9		176										
			11	SS	10		175										20.5
			12	SS	13		174										18.5
175.3	Compact, dark grey SAND and GRAVEL		13	SS	30		173										
10.2	Silty Saturated		14	SS	31		172										
174.8			15	SS	33		171										21.0 10 18 47 25
10.7	Hard to stiff, brown CLAYEY SILT, trace sand and gravel moist		16	SS	12		170										
174.2			17	SS	11												20.2 0 2 66 42
11.3			18	ST													
171.5	Dense to very dense, brown fine SAND saturated		19	SS	50												
14.0			20	SS	75												
169.8	END OF BOREHOLE at 15.7 m NOTE: Water level recorded in piezometer at 10.0m on 10/13/99																
15.7																	

EXPRESS 12-188.GPJ EXPRESS.GDT 13/12/99



# RECORD OF BOREHOLE No 6

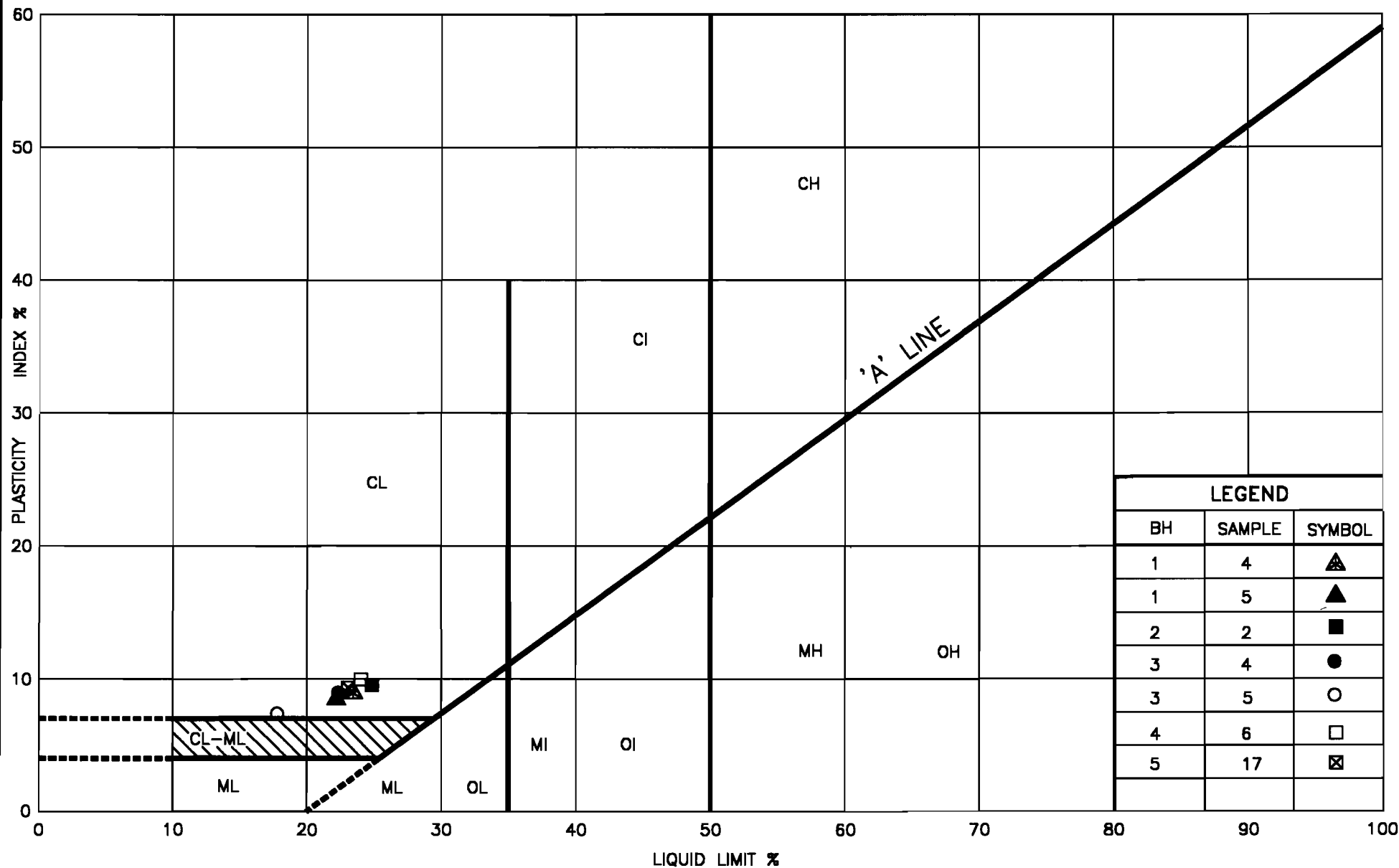
1 OF 1

METRIC

W.P. 348-98-00 LOCATION SITE No. 12-188, Bayfield River Bridge, Hwy. 21 ORIGINATED BY SEW  
 DIST 31 HWY 21 BOREHOLE TYPE Continuous SPT COMPILED BY SEW  
 DATUM Geodetic DATE 28.09.99 - 28.09.99 CHECKED BY EYC

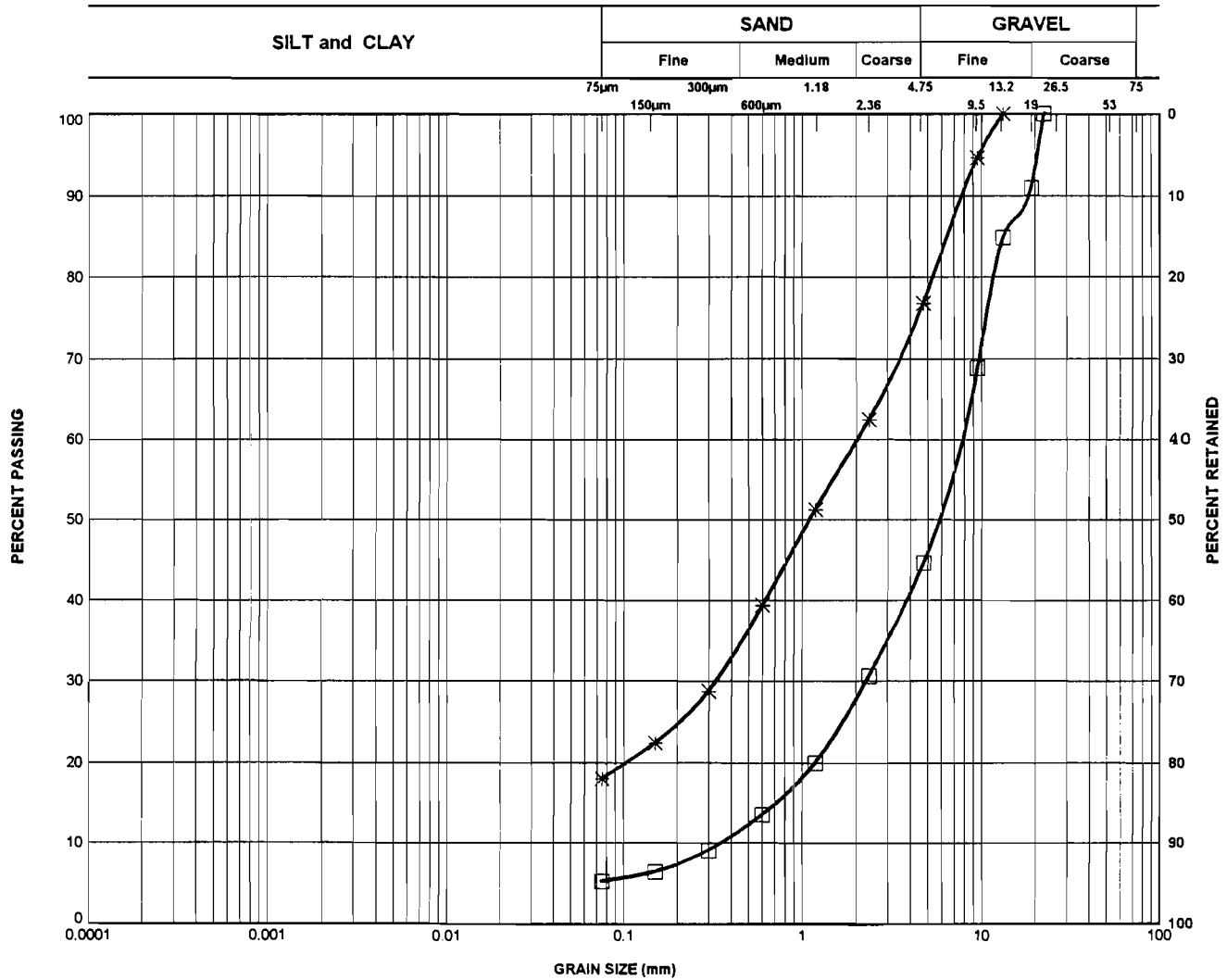
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
179.2	Ground							20 40 60 80 100						
0.0	Loose to compact, dark brown to black sandy silt FILL, some crushed gravel moist		1	SS	14		179							Dynamic cone penetration tests carried out
177.9			2	SS	17		178							
1.2	Compact, brown clayey silt FILL, trace gravel moist to very moist													
177.5			3	SS	21		177							
1.7	Compact, dark grey silt and fine sand FILL moist													
178.9														
2.3	Loose, olive organic silt FILL, some wood inclusions wet		4	SS	11		176							
178.5														
2.7	Loose, dark grey fine sand and silt FILL, frequent shell fragments, frequent woody inclusions saturated		5	SS	7		175							
175.5														
3.7	Compact, grey coarse to medium SAND, some gravel, frequent fine to medium sand layers saturated		6	SS	17		174							
			7	SS	16		173							
173.4			8	CT	12		172							
5.8	Soft to firm, grey CLAYEY SILT wet		9	CT	10		171							
			10	CT	16		170							
			11	CT	21		169							
			12	CT	21									
			13	CT	11									
			14	CT	1									
			15	CT	2									
171.1				16	CT	9								
8.1	Compact to very dense, grey SILT TILL, some clay, gravel and sand moist to damp			17	CT	15								
				18	CT	23								
		19		CT	44									
		20		CT	48									
		21		CT	40									
		22		CT	55									
		23		CT	74									
		24		CT	55									
168.5														
10.7	END OF BOREHOLE at 10.7 m NOTE* Water level at 176.4 m, 1hr after completion													

EXPRESS: 12-188.GPJ EXPRESS.GDT 13/12/99



# GRAIN SIZE DISTRIBUTION

## UNIFIED SOIL CLASSIFICATION SYSTEM

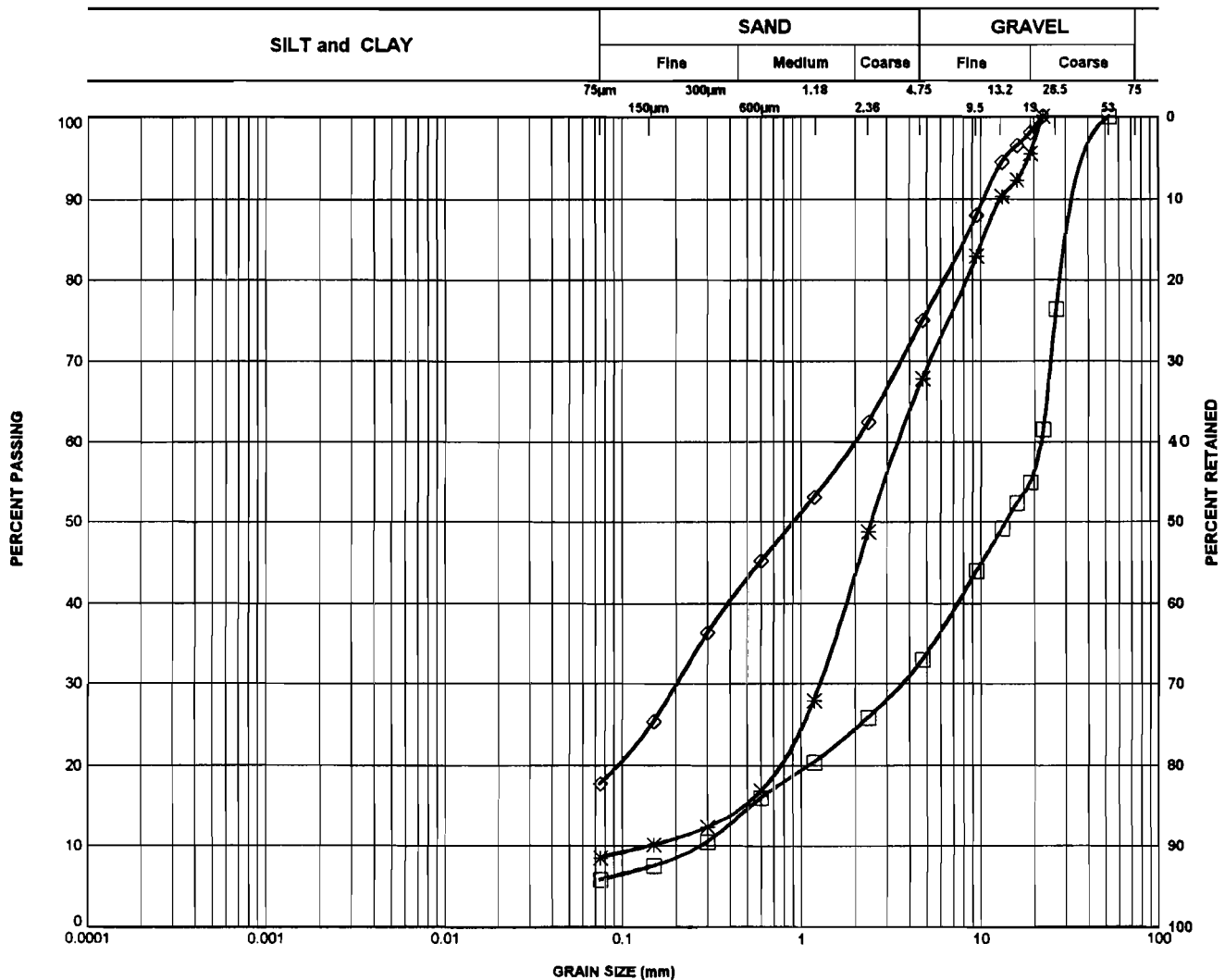


LEGEND		
SYMBOL	BOREHOLE	DEPTH (m)
□	5	0.8
*	5	2.4

**SAND and GRAVEL FILL**

# GRAIN SIZE DISTRIBUTION

## UNIFIED SOIL CLASSIFICATION SYSTEM



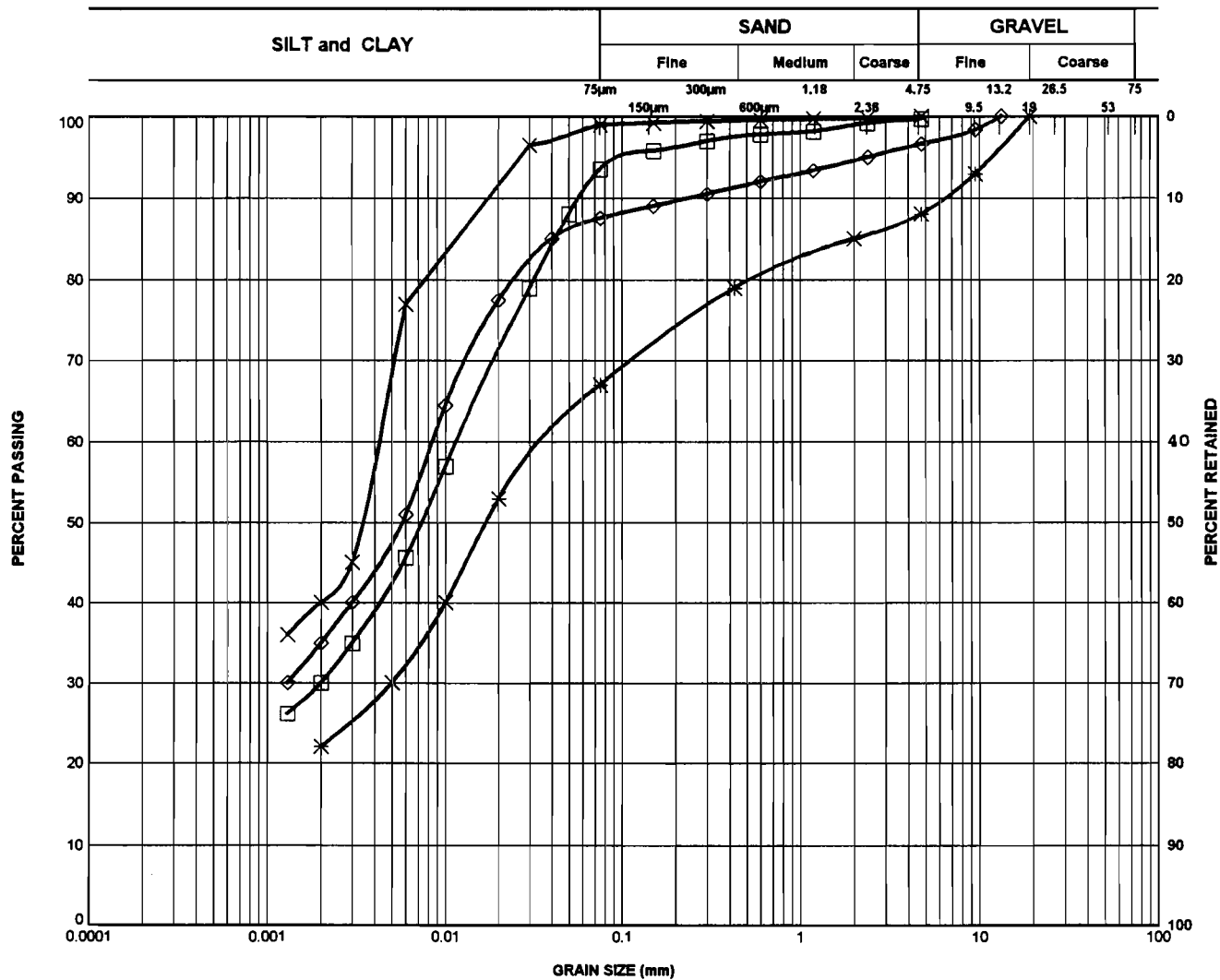
### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)
□	1	2.6
✱	3	5.4
◇	3	6.1

**SAND and GRAVEL**

# GRAIN SIZE DISTRIBUTION

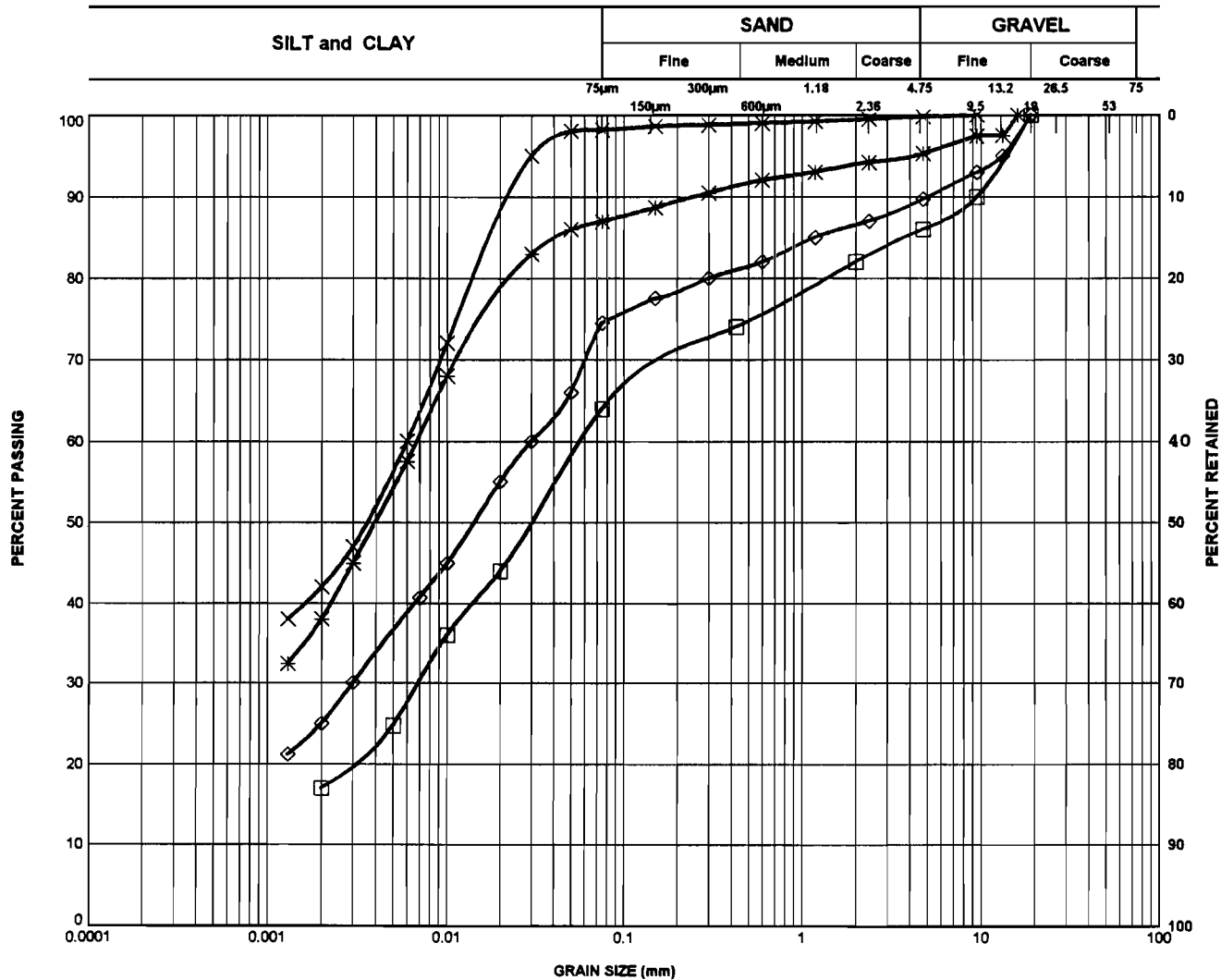
## UNIFIED SOIL CLASSIFICATION SYSTEM



**CLAYEY SILT**

# GRAIN SIZE DISTRIBUTION

## UNIFIED SOIL CLASSIFICATION SYSTEM



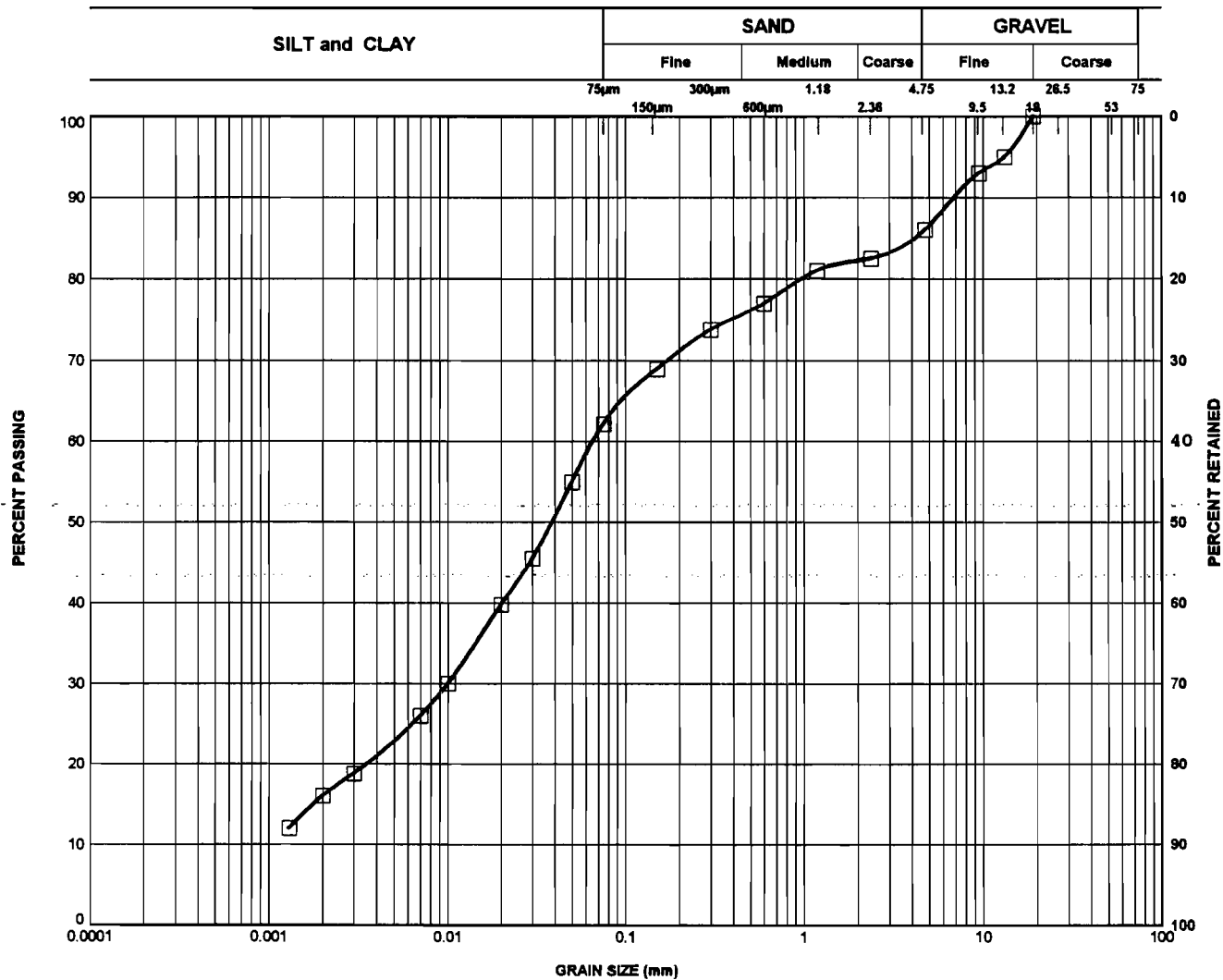
### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)
□	3	4.6
*	4	5.3
◇	5	11.5
×	5	13.0

**CLAYEY SILT**

# GRAIN SIZE DISTRIBUTION

## UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND		
SYMBOL	BOREHOLE	DEPTH (m)
□	1	6.9

**SILT TILL**

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