TULLOCH

GEOTECHNICAL REPORT

Potomac Bridge Replacement

Municipality of Huron Shores, Ontario

July 2023

TULLOCH Project # 22-0887

LICENSED PROFESSIONAL ENGINEER
E. K. GILES
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July 7th 2023
PROVINCE OF ONTARIO

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July 7, 2023
22-1011

Municipality of Huron Shores
7 Bridge Street
P.O. Box 460
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Attention: Natashia Roberts | CAO/Clerk

RE: Geotechnical Report for the Potomac River Bridge Replacement, Iron Bridge, Ontario

Dear Ms. Roberts,

Please find enclosed our Geotechnical Report for the proposed replacement of the Potomac River Bridge on Chiblow Lake Road in Iron Bridge, Ontario.

This report outlines the results of the geotechnical investigation and provides geotechnical recommendations for the proposed replacement of the existing structure.

We trust the enclosed is adequate for your current needs. If there is anything further that we can assist with, please contact us at your convenience.

Sincerely,

Erik Giles, P.Eng.
Geotechnical Engineer

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1. INTRODUCTION AND SCOPE

TULLOCH Engineering Inc. (TULLOCH) was retained by The Municipality of Huron Shores (Client) to complete a geotechnical investigation for the proposed bridge spanning the Potomac River on Chiblow Lake Road located in Iron Bridge, Ontario. A site location plan showing project location and site details including borehole locations can be seen in Appendix A.

The purpose of the geotechnical investigation was to evaluate the subsurface conditions at the abutments of the existing bridge structure and provide foundation engineering recommendations for its replacement.

This report provides the factual geotechnical investigation data and geotechnical design recommendations, which are based on the site investigation data, our understanding of the project scope and engineering experience. Common terminology used in this report can be found in Appendix B and specific terminology is referenced in table notes or in the report body.

2. REGIONAL GEOLOGY AND SITE

Based on a review of Bedrock Geology and Digital Northern Ontario Engineering Terrain Study (NOEGTS) mapping as published by the Ontario Geological Society (OGS), the general surficial geology of the area is comprised of a single landform type where the existing Potomac Bridge is located. Alluvial plain deposits exist surrounding the bridge site itself and the deposits are characteristic of sandy soils, with secondary materials comprised of gravelly soils (OGS 2005). Directly at, and to the west of the project site location, a bedrock knob is present in the NOEGTS mapping (OGS 2005). The Bedrock underlying the project site consists of conglomerate, sandstone, siltstone, and argillite host rock, with mafic and related intrusive rocks including mafic sills, mafic dikes and related granophyre (OGS 2011).

The existing bridge is a two-lane, load-posted structure that is located approximately 1 km north of the Town of Iron Bridge, Ontario in the Municipality of Huron Shores. The existing road surface at each approach of the Potomac Bridge is asphalt and the surface of the bridge is comprised of a timber deck with a surficial asphalt layer. A site photograph log of the drilling locations, the existing structure and the Potomac River can be seen in Appendix C.

3. SITE INVESTIGATION AND METHODOLOGY

The geotechnical investigation was completed from May 25th to 26th, 2022. The investigation campaign consisted of advancing two (2) boreholes (BH-22-01 and BH-22-02) at each abutment of the existing Potomac Bridge. Boreholes were advanced as close as possible to the abutments while allowing for traffic flow. Boreholes were advanced as close as possible to the bridge structure considering site logistics and the safety of the site crew including the locations of guardrails, accessibility and overhead and underground utilities. Due to the limited space for equipment, both boreholes were drilled along the east shoulder of the road at the north and south abutments.

Table 3-1 below shows a summary of the investigation. The locations of the boreholes are shown in the site plan in Appendix A. Please note that refusal depths do not necessarily indicate bedrock locations, while the bedrock surface may be inferred via auger or cone refusal during drilling operations, bedrock was not confirmed via coring due to a very dense soil layer at depth. For more details, borehole logs can be found in Appendix D.

Table 3-1: Summary of Borehole Information

Borehole No.	Borehole Location[1]		Borehole/Test Depth (mbgs)	Depth to Refusal (mbgs)
	Northing (m)	Easting (m)		
BH-22-01	5128473	329152	28.0	28.0
BH-22-02	5128440	329149	29.6	29.6

Note(s): [1] Borehole locations measured from handheld GPS only.

Boreholes were advanced using a CME 55 truck-mounted drill rig owned and operated by Landcore Drilling based in Chelmsford, Ontario. The boreholes were advanced using 200 mm Outside Diameter (OD), continuous flight, hollow stem augers, and NW Casing. The rig was equipped with standard soil sampling equipment. Due to the presence of heaving sands encountered at depths, to advance the borehole to the target depths wash-boring with NW casing was required.

In the overburden, soil samples were obtained using standard split spoon equipment in conjunction with Standard Penetration Tests (SPT) performed in accordance with ASTM D1586. Due to the presence of very dense granular material, wash boring techniques were utilized to advance the boreholes within the constraints of the project's schedule and budget. SPT sampling generally occurred at 0.76 m intervals in the upper 1.52 m of the borehole and at 1.5 m intervals thereafter. SPT sampling was conducted using an automatic hammer. Boreholes were advanced beyond the target depth of 10.0m due to the presence of poor soils and heaving sands as noted above in Table 3-1.

Dynamic Cone Penetrometer Tests (DCPT) were completed in both boreholes upon auger/casing refusal due to the dense heaving sands. The cone tests were completed to refusal on either the inferred bedrock surface or a very dense soil layer. SPT ‘N’ values were recorded at 0.3 m intervals throughout the test.

The drilling and soil sampling programs were directed by a TULLOCH representative, who logged the drilling operations and identified the soil samples as they were retrieved. The recovered soil samples were transported to TULLOCH’s CCIL Certified Laboratory in Sault Ste. Marie for detailed examination and testing. All samples will be stored at the laboratory for three (3) months and then disposed of unless directed otherwise. Detailed borehole logs and laboratory testing reports can be found in Appendix D and E, respectively.

4. LABORATORY TESTING PROGRAM

A geotechnical laboratory testing program was performed on representative samples at the bridge location in accordance with ASTM standards. Testing was completed at TULLOCH’s laboratory in Sault Ste. Marie. Corrosivity testing was completed by Testmark Laboratories in Garson, Ontario. Table 4-1 provides a list of the testing program. Detailed laboratory reports for grain size, Atterberg limits, natural water content and corrosivity can be found in Appendix E.

Table 4-1: Summary of Soil Laboratory Testing Program

Test	Number of Tests	ASTM Standards
Natural Water Content	11	ASTM D2216
Sieve & Hydrometer Analysis	7	ASTM D422
Atterberg Limits	2	ASTM D4318
Corrosivity Suite^{-1}	4	Various

Note(s):[1] Subcontracted Test

5. SUBSURFACE CONDITIONS

Subsurface conditions encountered at the Potomac Bridge site are summarized below. Detailed borehole logs and laboratory testing summaries can be seen in Appendix D and E, respectively. It should be noted that the soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The subsurface conditions encountered at the existing bridge structure were comprised of a granular road base overlying sandy silt to clayey silt, sand to gravelly sand, clayey silt to sandy silt, and sand, trace gravel. Soil layers encountered in the investigation are described below in the order they were encountered from ground surface.

5.1 SANDY TOPSOIL

At both drilling locations, a thin layer of topsoil was encountered at the surface ranging between 100 to 200 mm in thickness. The material was sandy, brown and contained organics and rootlets.

5.2 FILL – (SW) – Gravelly SAND to SAND

Beneath the topsoil, in both boreholes, an existing granular fill was encountered to an approximate depth of approximately 2.3 m below ground surface (mbgs). The material was brown in colour, non-cohesive and field moisture observations of recovered split-spoon samples indicated the material was moist. Material density for the fill ranged from loose to compact with SPT ‘N’ values ranging from 6 to 19 blows per 30 cm of sampler advancement and averaging 12 blows. It is noted that a gravel layer was encountered from 2.3 mbgs to 3.1 mbgs in BH-22-02, the gravel was brown, wet, and compact with SPT ‘N’ of 22 blows per 30 cm. The gravel layer may have been remnants of a previous fill placed prior to the current abutment fill for the bridge.

Laboratory testing results of the fill yielded an average natural water content of 3.5%.

Particle size distribution tests were conducted on two (2) representative samples of the existing fill with a material breakdown shown below in Table 5-1

Table 5-1: Fill Particle Size Distribution

Borehole/ Sample	Gravel (%)	Sand (%)	Silt/Clay (%)
BH-22-01/SS2	15	80	5
BH-22-02/SS2	28	63	9

5.3 (ML) Sandy SILT to CLAYEY SILT

Beneath the granular base/fill in both boreholes, a layer of sandy silt to clayey silt was encountered from a depth of 2.3 mbgs and 3.1 mbgs to approximately 4.6 mbgs and 4.8 mbgs in boreholes BH-22-01 and -02, respectively. The silt was then encountered again in both boreholes at a greater depth of approximately 12.2 mbgs.

The sandy silt to clayey silt was found to be grey to brown in colour. The behaviour of the material was typically cohesionless however some layers were found to be slightly cohesive with low plasticity based on tactile examination in field. Field observations of retrieved split-spoon samples indicated a moist to wet material. Material density for the sandy silt to clayey silt ranged from very soft to soft for cohesive samples and very loose for cohesionless samples with SPT ‘N’ values ranging from 1 to 4 blows per 30 cm of sampler advancement and averaging 3 blows.

Laboratory testing results of the sandy silt to clayey silt yielded an average natural water content of 31% for the upper silt layer and 57% for the lower silt layers indicating an increase in fines content with depth.

Particle size distribution tests were conducted on two (2) representative samples of the existing fill with a material breakdown shown below in Table 5-2.

Table 5-2: Sandy SILT to CLAYEY SILT Particle Size Distribution

Borehole/ Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH-22-01/SS5	0	4	78	18
BH-22-02/SS6A	1	29	56	14
BH22-02/SS9	0	1	85	14

Atterberg limits testing was conducted on select samples of the deposit and the tested samples indicated that the material was determined to be non-plastic.

5.4 (SW) - SAND

Beneath the silt to clayey silt in both boreholes a typically well graded sand with trace gravel was encountered. Sampling was terminated within this deposit in both boreholes with sampling continuing to approximately 25.0 mbgs and 18.9 mbgs in Boreholes BH-22-01 and -02, respectively. DCPTs were completed beyond the sampling depths due to the dense sand binding the augers. The DCPT testing was terminated at refusal depths of approximately 27.7 mbgs and 29.3 mbgs in boreholes BH-22-01 and -02 respectively.

The sand was found to be brown in colour, displayed non-cohesive behaviour, and field moisture observations indicated it ranged from and moist to wet with free standing water observed in retrieved soil samples. Material density for the sand ranged from compact to very dense with SPT ‘N’ values ranging from 15 to over 50 blows per 30 cm of sampler advancement and averaging 31 blows.

The DCPT yielded blows ranging from 3 to 83 blows per 30 cm and averaging 43 blows. It is noted that at approximately 26.2 mbgs and 27.4 mbgs in Boreholes BH-22-01 and -02, the material consistency became very dense with blows typically exceeding 100 blows per 30 cm of probe advancement. The observed high DCPT blows prior to termination likely indicated a very dense layer possibly containing cobbles and boulders or possible bedrock. However, based on infield observation it is likely bedrock was not encountered based on the refusal conditions of the DCPT advancement.

Laboratory testing results of the sand, trace gravel yielded an average natural water content of 22%.

Gradation testing was conducted on representative samples of the sand, trace gravel and yielded an average particle size distribution of 2% gravel, 97% sand, 1% fines.

Particle size distribution tests were conducted on two (2) representative samples of the existing fill with a material breakdown shown below in Table 5-3.

Table 5-3: SAND Particle Size Distribution

Borehole/ Sample	Gravel (%)	Sand (%)	Silt/Clay (%)
BH-22-01/SS13	2	96	2
BH-22-02/SS12	1	98	1

5.5 Groundwater Conditions

Water level measurements were taken in the open boreholes upon completion of the drilling. Table 5-4 provides a summary of the groundwater level readings obtained at the time of the investigation on May 25th and 26th in boreholes BH-22-01 and BH-22-02 respectively. It should be noted that due to the use of wash boring techniques, the water level may be elevated from injected drilling water at the time of measurement and may not represent stable conditions. Groundwater level is subject to seasonal fluctuations with high levels occurring during wet weather conditions in the spring and fall and lower levels during dry weather conditions. As such additional precautions should be taken for groundwater management if necessary.

Table 5-4: Water Level Readings Summary

Date of Measurement	Groundwater Levels (mbgs)	
	BH-22-01	BH-22-02
May 25th, 2022	2.8	-
May 26th, 2022	-	2.6

Given the presence of relatively permeable soils at the encountered groundwater level, it is likely that the water level of the Potomac River will significantly impact the groundwater levels experienced during construction.

6. GEOTECHNICAL RECOMMENDATIONS

6.1 General

The existing Potomac Bridge structure consists of a timber deck that has been paved. The deck of the bridge is supported on historic wood timber piles over five even spans (5) spans across the Potomac River. Each span is supported by a row of five (5) timber piles with wooden cross supports. The piles are capped by large wooden square cut beams that support the wooden deck. Based on visual inspection, the abutments of the bridge consist of wooden cribbing with granular backfill supported laterally by timber piles set in a similar fashion to the in-water spans. Lateral movement was noted on some of the wooden piles, indicating movement or possible failure over time. The bridge spans approximately 23 m across the river south of the intersection of Chiblow Lake Road and Allen Road. The founding elevations of the existing piles are unknown.

Based on the geotechnical investigation, the soil stratigraphy and groundwater conditions were generally similar at both borehole locations at each abutment of the bridge consisting of existing fill overlaying poor quality silts overlaying compact to dense well graded sands. Based on the investigation data it is unlikely that bedrock was encountered despite DCPT refusal in both boreholes. However, the refusal conditions observed indicated a very dense material layer at the termination depth of both boreholes as indicated above in Table 3-1.

The following sections of the report provide our interpretation of the available geotechnical data and geotechnical recommendations and it is intended for the guidance of the design engineer. Where comments are made regarding construction, they are provided only to highlight any aspects that could affect the design of the project. Contractors bidding on or undertaking the construction should make their own interpretation of the provided subsurface information with respect to their planned construction methods, equipment selection, scheduling, and the like. In addition, the following are to be noted:

- It is the Contractor’s responsibility to manage the water in the existing river as well as the groundwater during construction.
- The design and recommendations for a detour road and/or temporary bridge (if required) during the replacement of the existing Potomac Bridge are not addressed in this report.

The final construction staging and heavy lifts on this site should be reviewed by a qualified engineer prior to executing the work to ensure that a bearing failure of the riverbank is avoided. If required, TULLOCH can provide assistance during construction to verify the geotechnical conditions and approve any design changes at the time of construction.

6.2 Geotechnical Soil Parameters

The following soil parameters were used for the design of foundations for this report based on the geotechnical investigation. Based on the above-mentioned factual information, the native geological soil stratum was broken up into three (3) main groupings including an upper sandy silt to silt, a compact sand, and a lower dense sand. Table 6-1 shown below summarizes the parameters which were conservatively estimated based on the findings of the drilling investigation in conjunction with CFEM and the CHDBC. While some material samples of the upper silt indicated slight cohesion based on gradation and Atterberg results it has been assumed these soils will behave in a cohesionless manner, as such appropriate geotechnical properties are given in the table below.

Table 6-1: Geotechnical Design Loads for Driven Piles

Soil type	Unit Weight (kN/m^3)	Friction Angle (°)
Loose Sandy Silt to silt	17.5	28
Sand, Trace Gravel	20.	33
Very Dense Sand, Trace Gravel	21	35

Generally, subsurface conditions were found to be relatively consistent on both sides of the river and therefore, the same stratigraphic profile can be used for each abutment. Please note however that the dense lower sand was encountered approximately 1.0 m higher at BH-22-02 and which may affect deep foundation refusal. For further detailed stratigraphic information please refer to the borehole logs in Appendix A.

6.3 Foundations

A satisfactory foundation in terms of Limit State Design requires that the foundation can withstand the following two conditions. The first is the factored geotechnical resistance of the foundation to withstand the imposed factored Ultimate Limit State loading conditions – (ULS). The second is the ability to limit deformation/settlement to an acceptable magnitude under the Service Limit State loads – (SLS). The following section will outline geotechnical recommendations for foundation design for the Potomac River Bridge replacement.

6.3.1 Shallow Foundations

Spread footing foundations are not recommended at this site, namely due to the following considerations:

- The existing foundation system is timber piles with unknown founding elevation. In general, founding elevation for a new bridge at the same location should be at or below the existing founding elevation to prevent foundations from being placed on disturbed soils.
- Competent bearing stratum for spread footings is at considerable depth (approximately 12 mbgs) below the soft/loose silt stratum which would make excavations and footings unpractical or infeasible. Organics were also encountered within recovered split-spoon samples in the upper silt materials which are not considered suitable for bearing.
- Loose silty soils within the upper stratum at each boring location are susceptible to scour and undermining which could lead to reduction in capacity due to soil loss beneath the founding depth.

6.3.2 Deep Foundations - Driven Piles

A deep foundation system consisting of driven piles are the preferred option to support the new bridge abutments. Based on the investigation dense sands were not encountered until approximately 18 mbgs. The drilling and sampling were terminated in the very dense sand layer at approximately 26.5 to 27.5 mbgs for each approach. Piles driven into this very dense material will likely reach refusal. The presence of bedrock was not confirmed due to the significant depth and the very dense layer at depth. Piles should be driven into the very dense sands until refusal or a minimum embedment of 3.0 m if the geotechnical resistance has been achieved via the set conditions of the pile and/or verified via PDA testing.

There is a possibility of the presence of boulders that may make advancing the piles difficult as indicated by the very high DCPT blow counts at depth. Care should be taken not to damage driven piles. Driving shoes and/or rock points should be implemented for driven piles on this site.

The pile lengths shown are to be measured from the underside of the pile. The recommended geotechnical capacities in this report are conservatively based on the soil conditions encountered within the depths explored by the boreholes and considering the general range of pile loads expected for a short multi-span bridge. If greater load capacities are required, TULLOCH should be contacted for further assessment.

In TULLOCH’s opinion, the site conditions do not warrant the use of other deep foundation types such as drilled shafts or micropiles, due to prohibitive cost and therefore, will not be discussed further. The following sections summarize the feasible foundation types and the corresponding design geotechnical capacities.

6.3.2.1 Deep Foundation - Driven Steel Piles

Driven HP 310 X 110 or closed-end steel HSS pipe piles, of 304- and 324-mm diameter, driven into the very dense sand, layer encountered at the bottom of the boreholes are considered feasible options for the structure. Table 6-2 lists the estimated geotechnical capacities for these piles based on a minimum pile length of 21 m. End bearing was considered for piling conditions in each report but should be verified during driving operations via PDA testing. In general, given the site conditions, it is expected that the use of closed-end pipe piles will be advantageous and allow the required capacities to be achieved using shorter pile lengths than H-piles, however, H piles are also considered feasible for design.

While feasible the use of H-piles as friction piles in granular soils sometimes are known to result in significant length over-runs, furthermore, damage could be caused by potential boulders attempting to drive the H-Piles to depth, PDA testing should be employed during driving to help determine the set strength of the piles during installation.

The provided capacities represent estimates based on conventional analytical solutions for side friction and end bearing components and using a limit state design resistance factor of 0.4 in compression and 0.3 in tension. Larger capacities may be achievable using dynamic analyses (e.g., PDA testing) that would allow for in situ verification of pile capacities and allow the use of a greater resistance factor (0.5) under the limit state design approach.

Prior to driving piles, a wave equation (WEA) drivability analysis should be performed to assess the driving stresses and the anticipated penetration resistance required to develop the required pile capacity. This analysis considers the complete hammer-pile-soil system. Dynamic testing (PDA Testing) on a number of piles should be performed near the beginning of the pile driving phase of construction to confirm the pile capacities.

In addition, all piles should be inspected by TULLOCH personnel during installation to check for plumbness, set, internal damage, etc. All damaged piles should be rejected and if the damage is considered to be minor, the pile can be dynamically tested to determine the available pile capacity. The required penetration resistance will vary depending on the pile driving hammer, pile cross section and design loads. A Quality Control/Assurance program should be completed and pile driving techniques and equipment should be submitted for review by a qualified Engineer prior to work commencement. Pile driving techniques should follow OPSS 903.

Piles in groups should be spaced no closer than 3 pile diameters. All piles in a group should be checked for heaving during the driving of the adjacent piles, especially for displacement piles like closed-end pipe piles. Piles should be re-tapped in order to confirm the set after adjacent piles have been driven as per OPSS 903. A pre-fabricated driving shoe should be provided at the pile toe to protect the tip from damage and to assist in seating of the pile on/in the dense soils. In order to achieve the geotechnical resistance below an anticipated set criteria of approximately 5 – 10 blows per 25mm is anticipated. However, the contractor should submit driving criteria including driving methodology and equipment to the engineer prior to construction to confirm the methodology and final set criteria. Furthermore, in field monitoring of the pile driving in combination with PDA testing should be conducted to confirm the geotechnical resistance.

Table 6-2: Geotechnical Design Loads for Driven Piles

Diameter (mm)	Total Length (m)	Axial Compression	
		Factored ULS (kN) Compression/Tension	SLS (kN)[3]
HP310 x 110	20	650/500	Not Applicable
	25	875/650	Not Applicable
	30	1100/925	Not Applicable
305 mm (12”) Pipe Pile, Closed end	20	500/400	Not Applicable
	25	600/450	Not Applicable
	30	700/500	Not Applicable
324 mm (12 ¾”) Pipe Pile, Closed End	20	525/350	Not Applicable
	25	625/450	Not Applicable
	30	700/525	Not Applicable

Note(s): Resistance factor of 0.4 and 0.3 was used for the ULS Design Loads in compression and tension respectively; The estimated SLS corresponding to 25 mm settlement for a group of piles does not govern the design.

Negative skin friction or down drag forces have not been accounted for in the above values, as significant grade raises are not expected as part of the bridge replacement. Should the site conditions vary from these assumptions TULLOCH should be informed and corrected factored values may be provided.

6.3.3 Pile Settlements

The settlement of a single pile can be estimated using elastic theory and estimated soil compressibility parameters. Generally, however, the serviceability limit state of an individual pile is not expected to govern the design of piled foundations at this site. Pile groups of up to 25 piles spaced at least 3 pile diameters apart should not settle more than 25 mm at service load. The designer should contact TULLOCH for guidance if larger pile groups or closer spaced piles are required.

6.3.4 Resistance to Lateral Loads

Lateral Loading may be resisted partially or fully through the use of battered piles. In the case of integral abutments, vertical piles may be required to provide lateral resistance as well as axial. Should this be required, the modulus of horizontal subgrade reaction, k_h can be estimated using the following equations and ranges in subgrade reaction coefficient where:

$$k_h = n_h\,(z/d) \qquad \text{for cohesionless soils}$$

where:

d = pile width or diameter (m)

n_h = constant of horizontal subgrade reaction (MPa/m)

z = depth below ground surface grade (m)

Table 6-3 shown below contains $n_h$values that can be used.

Table 6-3: Horizontal Subgrade Reaction for Pile Lateral Loading

Soil Classification	n_h (MPa/m)
Loose sand/silt	1-2
Compact sand/silt	3-5
Dense sand	9-12

Group action for lateral loading should be considered when pile spacing in the direction of the loading is less than six (6) to eight (8) pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (k_h)in the direction of the loading by a reduction factor R as shown below in Table 6-4.

Table 6-4: Pile Spacing vs. Reduction Factor

Pile Spacing in direction of Loading, d = Pile Diameter	Subgrade Reaction Reduction Factor R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

While calculating lateral resistance, any portion of the pile within the frost depth, unless accounted for by the pile cap, should be excluded from contributing to the lateral capacity of the pile.

7. OTHER DESIGN AND CONSTRUCTION CONSIDERATIONS

7.1 Abutment Backfill

Backfill for abutments should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular B (OPSS 1010). Based on the gradation testing of the existing fill all samples met the OPSS 1010 envelope for a Granular B Type I material, as such the existing sand and gravel fill materials can be reused as backfill provided, they are free of organics, deleterious materials, and particle sizes greater than 150 mm, and continue to conform to the Granular B Type I specifications. The native clayey silts are not suitable backfill materials due to their fines content, poor bearing capacity and frost susceptibility. For abutment backfilling, the native soils and existing fill materials should be removed behind the abutment walls within a wedge-shaped zone extending from 1.2 m behind the base of the abutments and rising upward at an inclination of 1.0 vertical to 1.5 horizontal, according to OPSD 3101.150. A suitable frost taper is required for imported abutment backfill materials within the frost zone (as shown in OPSD 3101.150).

All granular backfill materials should be placed in thin lifts, not to exceed 300 mm in uncompacted thickness. Lifts should be compacted to 95% of the material's SPMDD. For backfilling beneath the pavement area, the degree of compaction of the fill materials within 1.5 m of pavement subgrade should be increased to 98% SPMDD. The minimal additional compaction effort will lead to better pavement performance. The Granular A base and Granular B sub-base courses below pavement (OPSS 1010) should be compacted to not less than 100% of the material's SPMDD. It is recommended that a non-woven geotextile be placed between the native silts and backfill materials. Furthermore, in order to achieve compactness of materials a bi-axial geogrid may be required.

The use of heavy compaction equipment should be avoided immediately adjacent to abutments. At no time should the fill levels on either side of the structure differ by more than 300 mm to avoid applying unbalanced loads on the structure, which could cause lateral displacement (dislodging) and/or damage.

7.2 Lateral Earth Pressure

The lateral earth pressures acting against abutments, culvert walls, and wing walls should be calculated according to the Canadian Highway Bridge Design Code (CHBDC 2014 CAN/CSA-S6-14). The general equation for calculating lateral earth pressure is:

$$P = k\,(\gamma h + q)$$

where

P = lateral earth pressure in kPa acting at depth;
k = earth pressure coefficient;
γ = bulk unit weight in kN/m^3;
h = depth to point of interest in m;
q= surcharge near wall in kPa.

Longitudinal drains and where applicable weep holes should be installed to provide positive drainage of the granular backfill. Unless drainage provisions are made or remain functional over the design life of the structure, hydrostatic pressure will have to be added to the lateral pressure. In accordance with Clause 6.12.3 of the CHBDC (S6-14), a minimum earth pressure of 12 kPa should be assumed to act on abutments, culvert walls and wing walls at the surface to account for compaction induced earth pressures. The properties listed in Table 7-1 can be used during design for the backfill materials.

Table 7-1: Recommended Lateral Earth Pressure Parameters

Material	Unit Weight (kN/m^3)	Effective Friction Angle (ϕ’)	Coefficient of Lateral Earth Pressure[1]	
			K’$_a$	K’$_o$
Granular A	22	38	0.24	0.38
Granular B, Type I	21	34	0.28	0.44
Granular B, Type II	21	36	0.26	0.41
Rock fill	21	42	0.20	0.33

Note(s): [1]K’a = coefficient of active earth pressure on a laterally unrestrained (non-rigid) structure; K’o = coefficient of earth pressure at rest for soil loading on a laterally restrained structure.

The above earth pressure coefficients assume a horizontal backfill condition, vertical back-face of the retaining structure and smooth soil-wall interface only. If the design includes a sloping ground surface in front of the retaining structure, a backfill inclination, or an inclined back-face of the retaining wall, the earth-pressure coefficients will require modification. TULLOCH should be contacted to provide appropriate coefficients for these conditions.

7.3 Frost Protection

The estimated frost penetration depth at the site is 1.8 m as per OPSD 3090.101. Frost tapers should be implemented for backfilling behind the bridge abutment or any trenches underlying areas (e.g., pavements) where differential frost heaving will not be acceptable, as recommended in the applicable OPSD (e.g., OPSD 3101.150). If the minimum soil cover cannot be achieved equivalent insulation should be used as the native upper silty soils on site are considered highly frost susceptible. Frost tapers should be implemented for backfilling behind the bridge abutment or any trenches underlying areas (e.g., pavements) where differential frost heaving will not be acceptable, as recommended in the applicable OPSD (e.g., OPSD 3101.150).

7.3.1 Adfreeze Stress

For steel pile foundations exposed to frost action, an adfreeze uplift stress of 100 kPa (unfactored) recommended by the Canadian Foundation Engineering Manual (CFEM) (2006) should be used in the pile design for the soil-pile contact area within the design frost depth of 1.8 m. If granular soils are used as backfill materials against the pile/foundation pole below the ground surface, then adfreeze adhesion values can be as high as 150 kPa, as discussed in the CFEM.

7.4 Erosion Protection

The native silty soils encountered on site are highly erodible and scour prone. Erosion and scour protection should be provided along the slopes and sides of the bridge abutments. Consideration should be given to appropriately sized rip-rap on the road embankments near the bridge abutments within the flood plain of the river as determined by hydraulic analysis that is considered outside the scope of this report. Alternatively, if sheet piling is installed it may be left in place to act as scour protection after the construction is complete. The erosion/scour protection should be designed by a hydraulic engineer.

7.5 Excavations and Groundwater Control

Trench excavations, if required, must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA), Regulation 213/91, as well as the specifications of OPSS 539 and OPSS 902. In accordance with OHSA, the native soils at the site can be classified as Type 4 material below the groundwater level. As such, excavations must be sloped at 3H:1V from the base of excavation unless the materials are dewatered or supported. If dewatered, the soils should behave as Type 3 material and trench excavations can be sloped at 1H:1V from the base of the excavation.

Excavations that extend below the groundwater table will require control measures. Although the silty soils on site are relatively low permeability soils, they can become easily disturbed. Sump and pump techniques may be enough to de-water excavations that do not extend significantly below the groundwater table, i.e., less than approximately 0.5 m. However, well points or other active groundwater control techniques or a groundwater cut-off (e.g., sheet pile cofferdam) system will need to be installed to dewater the site if deeper excavations below the water table are attempted. The actual dewatering methods should be established at the contractor’s discretion within the context of a performance specification for the project. It should be noted that, under the Ontario Water Resources Act, the Water Taking and Transfer Regulation 387/04, for the purpose of construction site dewatering an Environmental Activity Sector Registry (EASR) registration is required if the dewatering discharge is greater than 50,000 L/day but less than or equal to 400,000 L/day. If water taking is greater than 400,000 L/day a Permit to Take Water (PTTW) is required from the Ministry of the Environment, Conservation and Parks. Reference is also given to OPSS 517 and 518 which pertain to construction dewatering. At this site, an EASR registration will be required if active de-watering is implemented and a PTTW may need to be pursued depending on the groundwater control method selected. A hydrogeological study will likely have to be completed to assess dewatering volume, zone of influence and impact on any adjacent properties.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during normal dry weather conditions. The local groundwater conditions of the site will fluctuate with the levels of the Potomac River.

The excavation of the soils at the site should be achievable using conventional excavating equipment. During excavation, care should be made to minimize disturbance of the soft upper silty soils. This material will become easily disturbed and will also degrade with prolonged exposure to the elements.

7.5.1 Shoring/Support Systems for Excavation

Shoring/Sheet piling may be an effective way to create hydraulic isolation and support of excavations for abutments/pile caps. For general guidance, the design of the shoring system may be completed using the apparent earth pressure diagrams for braced excavations in granular soils, as discussed in Section 26.10.3 of the CFEM (4th Edition, 2006) shown below in Figure 7-1. The excavation is expected to extend native clayey silts and loose gravelly sands. A bulk unit weight of soils of 18.9 kN/m^3 and an active earth pressure coefficient of 0.41 may be used for the clayey silts, and a bulk unit weight of soils of 17.7 kN/m^3 and an active earth pressure coefficient of 0.36 may be used for the loose gravelly. A uniform surcharge pressure should be added to the earth pressure diagram to account for construction surcharge loads adjacent to the excavations. A surcharge of 12 kPa is typically used for light construction loads; heavier loads can be assessed individually using elastic stress distribution solutions.

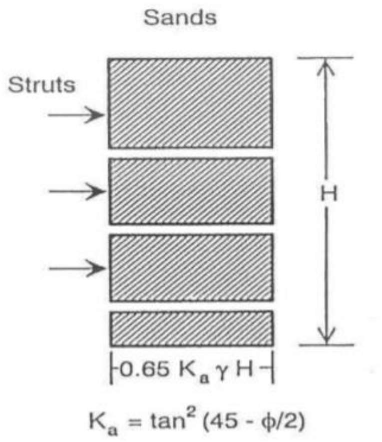

Figure 7-1: Pressure Diagram for Sand non-cohesive Silts

7.6 Re-Use of Excavated Soil

Further to the discussion provided in Section 7.1, the existing sand and gravel fill materials may be reused as backfill provided they are stockpiled separately, are consistently within ± 2% of optimum moisture content during compaction activities and any oversized cobbles and boulders are removed as well as any other deleterious material. The material should be inspected and tested prior to re-use. The excavated native silts are not suitable as fill materials due to their frost-susceptibility and relatively high water content. The suitability for reuse of any existing soils should be verified on site during construction by a qualified geotechnical engineer. Geo-environmental testing for the disposal of excess soils was not part of TULLOCH's scope and has not been addressed in this report.

7.7 Approach Reinstatement

Backfill to the abutments and wingwalls should be completed to the underside of the pavement structure, as recommended in the previous sections. The reinstated pavement structure of the bridge approaches should match the existing adjacent pavement structure at a minimum and comply with the current engineering standards for The Municipality of Huron Shores TULLOCH can provide a flexible pavement design for the site if required by the Client.

The pavement subgrade should be approved by a qualified geotechnical engineer. The subbase and base courses should be compacted to a minimum of 100% SPMDD. The pavement subgrade and the surface should have at least 2% crossfall towards the pavement edges for effective drainage.

Additionally, periodic maintenance for the road surface may be required at the approaches to account for any differential settlement of the abutment backfill and the new bridge. Consideration should be given to operating the crossing as a gravelled surface for approximately one (1) year after construction to allow initial settlement to occur. Ideally, the approach settlement should be monitored to confirm a suitable duration prior to placing the final asphalt layers.

7.8 Seismic Design Considerations

7.8.1 Site Classification for Seismic Response

The encountered soils at the site are, in accordance with Section 4.4.3.2 of the CHBDC S6-14, the shallow clayey/sandy silts and loose gravelly sand on site are generally characterized as soft soils (SPT $\tilde{N}_{60}$ ≤15), or Class E which should be applied to all shallow construction and foundations. However, the design of the deep foundations founded on the very dense sand and trace gravel which would be characterized as very dense soils ($\tilde{N}_{60}$ >50) allow for Site Class C. Consideration can be given to performing in situ geophysical testing (e.g., MASW testing) which may help confirm the higher of the two site classifications (Site Class C) considering the average soil properties in the upper 30 m at the site.

As mentioned above, the structural engineer should ensure that the structural design incorporates deflections and specified loading resulting from potential earthquake effects.

Peak Ground Acceleration (PGA) for the area in the site based on the NBCC seismic hazard values for Site Class E are 0.091g for a 2% exceedance over the course of 50 years.

7.9 Soil Corrosivity

Representative testing was completed for soil corrosivity and sulphate concentrations for the bridge location. The results of the testing are shown below in Table 7-2. Samples were tested at TestMark laboratories based in Garson, Ontario.

Table 7-2: Soil Corrosivity Results

Borehole No. / Sample No.	Depth (m)	Resistivity (Ω cm)	pH	Redox Potential (mV)	Sulphide (µg/g)	Sulphate (µg/g)	Chloride (µg/g)
BH-22-01	4.6	10400	5.25	322	<0.3[1]	73	3.4
BH-22-02	15.2	14500	7.84	310	<0.3[1]	<2[1]	0.4

Note(s):[1]Reported values below the testing limit

The results of the chemical testing were assessed in reference to the AWWA C-105 Standard from ANSI/AWWA Corrosivity Rating System. A score greater than 10 indicates the requirement of corrosion protective measures for buried cast iron alloys. All the tested samples analyzed for BH-22-01 and BH-22-02 scored a ranking of 2 which is below the threshold.

In addition, chloride ions can lead to corrosion of steel. Typically, soils with chloride concentrations greater than 500 µg/g are considered corrosive. As noted in Table 7-2, chloride concentrations are less than 500 µg/g in the tested samples.

The concentration of sulphate indicates the degree of sulphate attack for concrete buried at the site. As shown in Table 7-2, the sulphate concentrations are less than 1000 µg/g indicating a low degree of sulphate attack. Type GU Portland cement should be suitable for use at this site. The test results presented in Table 7-2 may be used to aid in the selection of any coatings and corrosion protection systems for buried steel infrastructure. For detailed chemical results please see Appendix E.

8. CLOSURE

This geotechnical report has been prepared by TULLOCH for the exclusive use of The Municipality Huron Shores and their authorized agents for the replacement of the Potomac River on Chiblow Lake Road in Iron Bridge, Ontario. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering, for the above noted location. Classification and identification of soils, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Please refer to Appendix F, Notice to Reader, which pertains to this report.

We trust that the information in this report will be sufficient to allow The Municipality of Dysart et al. to proceed with the Feasibility Study for the Redstone Brook Bridge. Should further elaboration be required for any portion of this project, we would be pleased to assist.

Kelvin Cheung, P.Eng.
Geotechnical Engineer

Reviewed By:
Erik Giles, P.Eng.
Geotechnical Engineer

REFERENCES

Canadian Geotechnical Foundation Engineering Design Manual 4th Edition, 2006.

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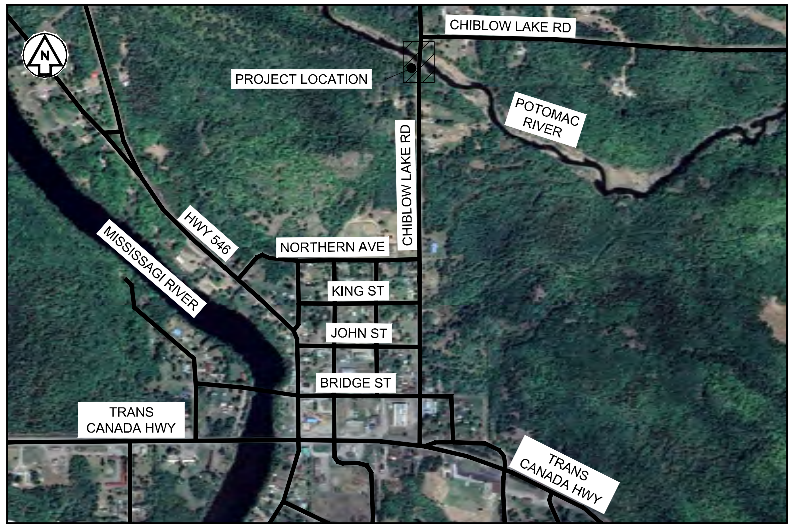
APPENDIX A

KEY LOCATION PLAN & SITE PLAN LOCATION

PLAN

SCALE: 1:500



PROJECT LOCATION

N.T.S

BOREHOLE LOCATIONS

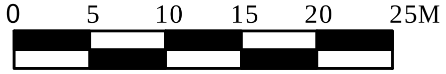
BOREHOLES	EASTING	NORTHING
BH-22-01	329 152	5 128 473
BH-22-02	329 149	5 128 440

LEGEND:

BH-22-01 ⊕ BOREHOLE INVESTIGATIONS

NOTES:

1. CO-ORDINATES ARE IN UTM ZONE 17 (NAD83 CSRS).



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No.	DATE	BY	ISSUES / REVISIONS
0	2022-06-16	KK	ISSUED FOR FINAL REPORT
A	2022-06-16	KK	ISSUED FOR DRAFT REPORT

TULLOCH

DRAWING:
BOREHOLE LOCATION
PLAN

CLIENT:
MUNICIPALITY OF
HURON SHORES

PROJECT:
POTOMAC BRIDGE REPLACEMENT
GEOTECHNICAL INVESTIGATION

DRAWN BY: K.KORTEKAAS	CHECKED BY: E. GILES	DESIGNED BY: S. deBORTOLI
APPROVED BY: E.GILES	SCALE: AS NOTED	DATE: 2022-06-16
DRAWING No. 22-0887-20-2050-01		REVISION No. 0

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APPENDIX B

TERMINOLOGY

ABBREVIATIONS, TERMINOLOGY AND PRINCIPAL SYMBOLS USED IN REPORT AND BOREHOLE LOGS

BOREHOLES AND TEST PIT LOGS

Soils

AA	Auger Sample	w	Water Content
SS	Split Spoon	wP	Plastic Limit
TO	Tin-walled Tube	wL	Liquid Limit
TP	Thin-walled Piston	V(FV)	Field Vane
WS	Washed Sample	OR	Organic Content
SC	Soil Core	GR	Gravel
BS	Block Sample	SA	Sand
WH	Weight of rods & hammer	SI	Silt
WR	Weight of rods	CL	Clay

Bedrock

TCR	Total Core Recover	VN	Vein
SCR	Solid Core Recovery	CO	Contact
FI	Fracture frequency index	KV	Karstic void
HQ	Rock Core (63.5 mm dia.)	MB	Mechanical Break
NQ	Rock Core (47.6 mm dia.)	PL	Planar
BQ	Rock Core (36.5 mm dia.)	CU	Curved
JN	Joint	UN	Undulating
FLT	Fault	IR	Irregular
SH	Shear	SM	Smooth
K	Slikensided	SR	Slightly Rough
BD	Bedding	R	Rough
FO	Foliation	VR	Very rough

IN SITU SOIL TESTING

Standard Penetration Test (SPT) "N" value. The number of blows required to drive a 51 mm OD split barrel sampler into the soil a distance of 300 mm with a 63.5kg weight free falling a distance of 760 mm after an initial penetration of 150 mm has been achieved.

Dynamic Cone Penetration Test (DCPT) is the number of blows required to drive a cone with a 60 degree apex attached to "A" size drill rods continuously into the soil for each 300 mm penetration with a 63.5 kg weight free falling a distance of 760 mm.

Cone Penetration Test (CPT) is an electronic cone point with a 10 cm base area with a 60 degree apex pushed through the soil at a penetration rate of 2cm/s.

Field Vane Test (FVT) consists of a vane blade, a set of rods and torque measuring apparatus used to determine the undrained shear strength of cohesive soils.

SOIL DESCRIPTIONS

The soil descriptions and classifications are based on an expanded Unified Soil Classification System (USCS). The USCS classifies soils on the basis of engineering properties. The system divides soils into three major categories; coarse grained, fine grained and highly organic soils. The soil is then subdivided based on either gradation or plasticity characteristics. The classification excludes particles larger than 75 mm. To aid in quantifying material amounts by weight within the respective grain size fractions the following terms have been included to expand the USCS:

Soil Classification	
Clay	<0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.075 to 4.75 mm
Gravel	4.751o 75 mm
Cobbles	75 to 200 mm
Boulders	>200 mm

Terminology	Proportion
“trace”, sand, etc.	1%to 10%
"some"	10% to 20%
Sandy, Gravelly, etc.	20% to 35%
“and”	>35%
Ex., SAND, SILT, etc.	>35%

Notes:

1. Soil properties, such as strength, gradation, plasticity, structure, etc., dictate the soils engineering behaviour over the grain size fractions;
2. With the exception of soil samples tested for grain size distribution or plasticity, all soil samples have been classified based on visual and tactile observations and is therefore an approximate description.

The following table outlines the qualitative terms used to describe the relative density condition of cohesionless soil:

Cohesionless Soils

Compactness	SPT “N” Value (blows/30cm)
Very Loose	0 to 4
Loose	5 to 10
Compact	11 to 30
Dense	31 to 50
Very Dense	>50

The following table outlines the qualitative terms used to describe the consistency of cohesive soils related to undrained shear strength and SPT, N-Index:

Cohesive Soils

Consistency	Undrained Shear Strength (kPa)	SPT “N” Value (blows/30 cm)
Very Soft	<12.5	< 2
Soft	12.5 to 25	2 to 4
Firm	25 to 50	5 to 8
Stiff	50 to 100	9 to 15
Very Stiff	100 to 200	16 to 30
Hard	> 200	>30

Note: Utilizing the SPT, “N” value to correlate the consistency and undrained shear strength of cohesive soils is very approximate and needs to be used with caution.

Particle Sizes

Constituent	Description	Size (mm)	Size (in)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	< 0.075	< (200)

ROCK CORING

Rock Quality Designation (RQD) is an indirect measure of the number of fractures within a rock mass, Deere et al. (1967). It is the sum of sound pieces of rock core equal to or greater than 100 mm recovered from the core run, divided by the total length of the core run, expressed as a percentage. If the core section is broken due to mechanical or handling, the pieces are fitted together and if 100 mm or greater included in the total sum.

Intact Rock Strength

Intact Strength (Mpa)	Description
< 1	Extremely low strength
1-5	Very low strength
5-25	Low strength
25-50	Medium strength
50-100	High strength
100-250	Very high strength
>250	Extremely high strength

Rock Mass Quality

RQD Classification	RQD Value (%)
Very Poor Quality	<25
Poor Quality	25 to 50
Fair Qualty	50 to 75
Good Quality	75 to 90
Excellent Quality	90 to 100

Rock Mass Weathering

Term	Description
Unweathered (Fresh)	No visible sign of material weathering to discoloration on major discontinuity surfaces.
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity of surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than its fresh condition.
Moderatly Weathered	Less than half the rock material is decomposed and/or disintegrates to soil. Fresh or discolored rock is present either as a continuous frame work of as core stones.
Highly Weathered	More than half the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a discontinuous frame work or as core stones.
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is largely intact.
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

Joint and Foliation Spacing

Description	Spacing
Very Wide	Greater than 3 m
Wide	1 m to 3 m
Moderately Close	0.3 m to 1 m
Close	50 mm to 300 mm
Very Close	Less than 50 mm

Bedding Thickness

Description	Spacing
Very thick	Greater than 2 m
Thick	0.6 m to 2 m
Medium	0.2 m to 0.6 m
Thin	60 mm to 0.2 m
Very thin	20 mm to 60 mm
Laminated	6 to 20 mm
Thinly Laminated	Less than 6 mm

SYMBOLS

General

w_N	Natural water content within the soil sample
γ	Unit weight
γ'	Effective unit weight
γ_D	Dry unit weight
γ_{SAT}	Saturated unit weight
ρ	Density
ρ_s	Density of solid particles
ρ_w	Density of water
ρ_D	Dry density
ρ_{SAT}	Saturated density
e	Void ratio
n	Porosity
S	Degree of saturation
E_{50}	Fifty percent secant modulus

Consistency

w_L	Liquid Limit
w_P	Plastric Limit
I_P	Plasticity Index
w_S	Shrinkage limit
I_L	Liquidity index
I_C	Consistency index
e_{max}	Void ratio in loosest state
e_{min}	Void ratio in densest state
I_D	Density index (formerly relative density)

Shear Strength

S_u	Undrained shear strength parameter (total stress)
c'	Effective cohesion intercept
ϕ'	Effective friction angle
τ_R	Peak shear strength
τ_R	Residual shear strength
δ	Angle of interface friction
μ	Coefficient of friction = tan ϕ'

Consolidation

C_c	Compression index (normally consolidated range)
C_r	Recompression index (over consolidated range)
m_v	Coefficient of volume change
c_v	Coefficient of consolidation
T_v	Time factor (vertical direction)
U	Degree of consolidation
σ_v'	Effictive overburden pressure
OCR	Overconsolidation ratio

APPENDIX C

PHOTOGRAPH LOG

Photo 1: Drill rig setup at BH-22-01

Photo 2: View from Drill Rig set up at BH-22-01 and view of the east side of the existing bridge

CLIENT
The Municipality of Huron Shores

PROJECT
22-0887 Potomac Bridge Replacement, Geotechnical Investigation

CONSULTANT
TULLOCH ENGINEERING

YYYY-MM-DD	2022-07-28
PREPARED	KC
DESIGNED	
REVIEWED	EG
APPROVED	EG

TITLE
Site Photograph Log

PROJECT NO.	Phase/Task	Rev.	FIGURE
22-0887	101		**1**

Photo 3: View from bridge deck facing southeast

Photo 4: View from bridge deck facing west

CLIENT
The Municipality of Huron Shores

PROJECT
22-0887 Potomac Bridge Replacement, Geotechnical Investigation

CONSULTANT
TULLOCH ENGINEERING

YYYY-MM-DD	2022-07-28
PREPARED	KC
DESIGNED	
REVIEWED	EG
APPROVED	EG

TITLE
Site Photograph Log

PROJECT NO.	Phase/Task	Rev.	FIGURE
22-0887	101		**2**

Photo 5: View of existing bridge deck and foundation from north abutment

Photo 6: View of the south bridge approach and abutment from the bridge deck

CLIENT
The Municipality of Huron Shores

PROJECT
22-0887 Potomac Bridge Replacement, Geotechnical Investigation

CONSULTANT
TULLOCH ENGINEERING

YYYY-MM-DD 2022-07-28
PREPARED KC
DESIGNED
REVIEWED EG
APPROVED EG

TITLE
Site Photograph Log

PROJECT NO. **22-0887**
Phase/Task 101
Rev.
FIGURE **3**

Photo 7: BH-22-01 location borehole backfilled

Photo 8: Completed BH-22-02 and backfilled

CLIENT
The Municipality of Huron Shores

PROJECT
22-0887 Potomac Bridge Replacement, Geotechnical Investigation

CONSULTANT
TULLOCH
ENGINEERING

YYYY-MM-DD	2022-07-28
PREPARED	KC
DESIGNED	
REVIEWED	EG
APPROVED	EG

TITLE
Site Photograph Log

PROJECT NO.	Phase/Task	Rev.	FIGURE
22-0887	101		**4**

APPENDIX D

BOREHOLE LOGS

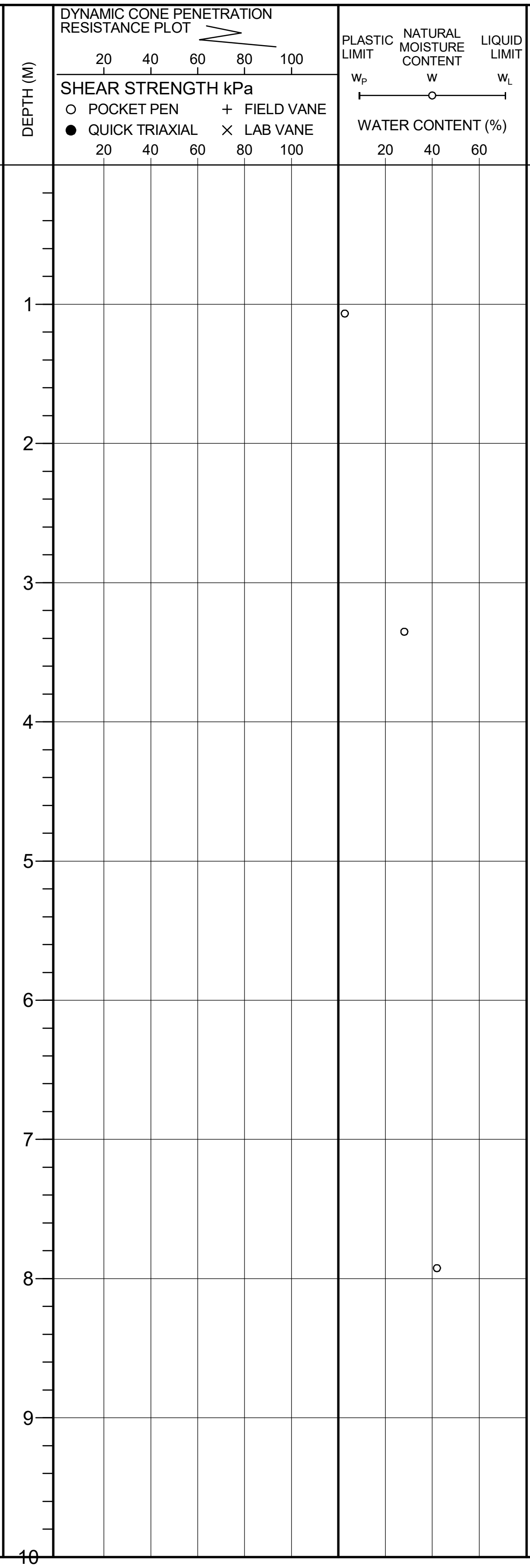
TULLOCH ENGINEERING

RECORD OF BOREHOLE No BH-22-01

1 OF 3 **METRIC**

JOB NUMBER 22-0887 LOCATION NE Bridge Abutment, Iron Bridge ORIGINATED BY SdB
CLIENT Huron Shores DATUM GS BOREHOLE TYPE HSA/Washbore COMPILED BY KC
DRILLER Landcore DATE 2022.05.25 NORTHING 5128473 EASTING 329152 CHECKED BY EG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.00	TOPSOIL, sandy, rootlets, brown, moist, compact						
0.23	FILL - (SW) SAND, Some gravel, trace non-plastic fines, brown, moist, non-cohesive, compact to loose	1A/B	SS	19	67		
		2	SS	7	46		15 80 (5)
		3	SS	15	42		
2.29	(ML) CLAYEY SILT, trace gravel, trace sand, brown-grey, moist, cohesive, very soft	4	SS	1	33	∇	No recovery on intial spoon, 2nd spoon pushed
3.05	(ML) SILT some clay, grey, oxidation present, wet, non-cohesive, very loose	5	SS	3	71		Too silty to attempt vane 0 4 78 18 Atterberg limit taken in SS5 - non-plastic result
4.57	(SP) SAND, medium grained, some subrounded gravel, trace silt, trace organics, brown to grey, wet, non-cohesive, wet, loose	6	SS	2	67		
	wood chips found in SS7 spoon sample	7	SS	3	71		
6.71	(SP) Gravelly SAND, fine to medium grained, subrounded gravel, some silt, trace organics, brown, wet, non-cohesive, very loose						
		8	SS	3	100		Switched to DCP tip in augers due heaving sands in augers
	trace organics/ wood pieces recovered in SS9	9	SS	2	100		



Continued Next Page

200 + : Numbers refer to Field Vane Over Limit

$+^3, \times^3$: Numbers refer to Sensitivity

$○^{3\%}$ STRAIN AT FAILURE

TULLOCH ENGINEERING

RECORD OF BOREHOLE No BH-22-01

2 OF 3

METRIC

JOB NUMBER 22-0887 LOCATION NE Bridge Abutment, Iron Bridge ORIGINATED BY SdB
CLIENT Huron Shores DATUM GS BOREHOLE TYPE HSA/Washbore COMPILED BY KC
DRILLER Landcore DATE 2022.05.25 NORTHING 5128473 EASTING 329152 CHECKED BY EG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)	GROUND WATER CONDITIONS	DEPTH (M)	WATER CONTENT (%)	REMARKS	GR	SA	SI	CL
	(SP) Gravelly SAND, fine to medium grained, subrounded gravel, some silt, trace organics, brown, wet, non-cohesive, very loose *(continued)*												
	organics no longer present in SS10	10A/B	SS	3	100		11		Heaving sand in auger, vane could not turn				
11.02	(ML) CLAYEY SILT, trace to some sand, dark brown, moist, cohesive, very soft												
12.19	(SW) SAND, trace gravel, trace to some silt, dark brown, moist to wet, non-cohesive, compact to very dense	11	SS	20	100		12						
							13						
	grey-brown, material becoming wet	12	SS	25	100		14		Switched to casing to washbore due to spoon getting stuck and heaving sand material. Groundwater at 2.67 mbgs before switch.				
	red, white and black sand grains in SS13	13	SS	15	17		15	○		2	96	(2)	
							16						
							17						
	increasing material density in SS 14	14	SS	42	8		18		Low recovery, 2nd attempt yielded full recovery				
							19						
							20						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ POCKET PEN + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L: WATER CONTENT (%) 20 40 60

Continued Next Page

200 + : Numbers refer to Field Vane Over Limit

$+^3, \times^3$: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

TULLOCH ENGINEERING

RECORD OF BOREHOLE No BH-22-01

3 OF 3

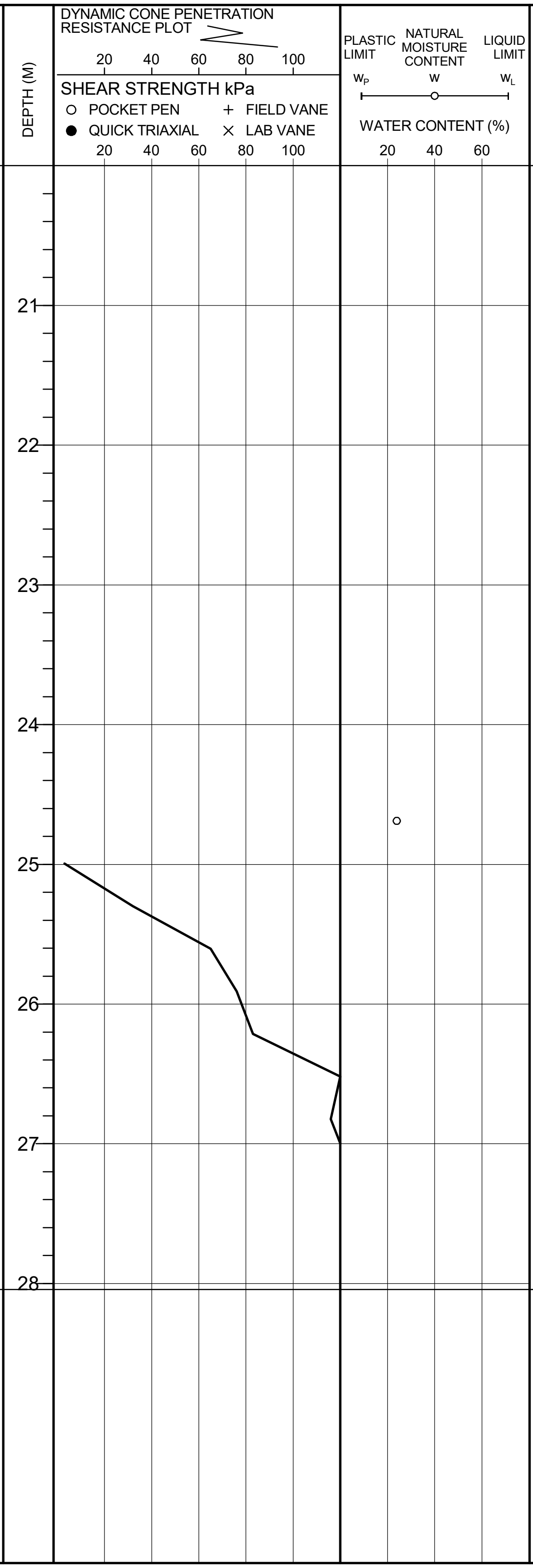
METRIC

JOB NUMBER 22-0887 LOCATION NE Bridge Abutment, Iron Bridge ORIGINATED BY SdB

CLIENT Huron Shores DATUM GS BOREHOLE TYPE HSA/Washbore COMPILED BY KC

DRILLER Landcore DATE 2022.05.25 NORTHING 5128473 EASTING 329152 CHECKED BY EG

SOIL PROFILE			SAMPLES						REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)	GROUND WATER CONDITIONS	DEPTH (M)	
	(SW) SAND, trace gravel, trace to some silt, dark brown, moist to wet, non-cohesive, compact to very dense *(continued)*								
			15	SS	42	0			No recovery, 2nd attempt yielded good recovery
			16	SS	58	29			
24.99	DCPT Advancement								
28.04	END OF BOREHOLE - DCPT Refusal Note: Water encountered at 2.8 mbgs after completion of investigation. Note that ground water may not be stabilized upon completion of investigation. Borehole caved at 7.3 mbgs after casing was pulled.								



200 + : Numbers refer to Field Vane Over Limit

$+^3, \times^3$: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

TULLOCH ENGINEERING

RECORD OF BOREHOLE No BH-22-02

1 OF 3

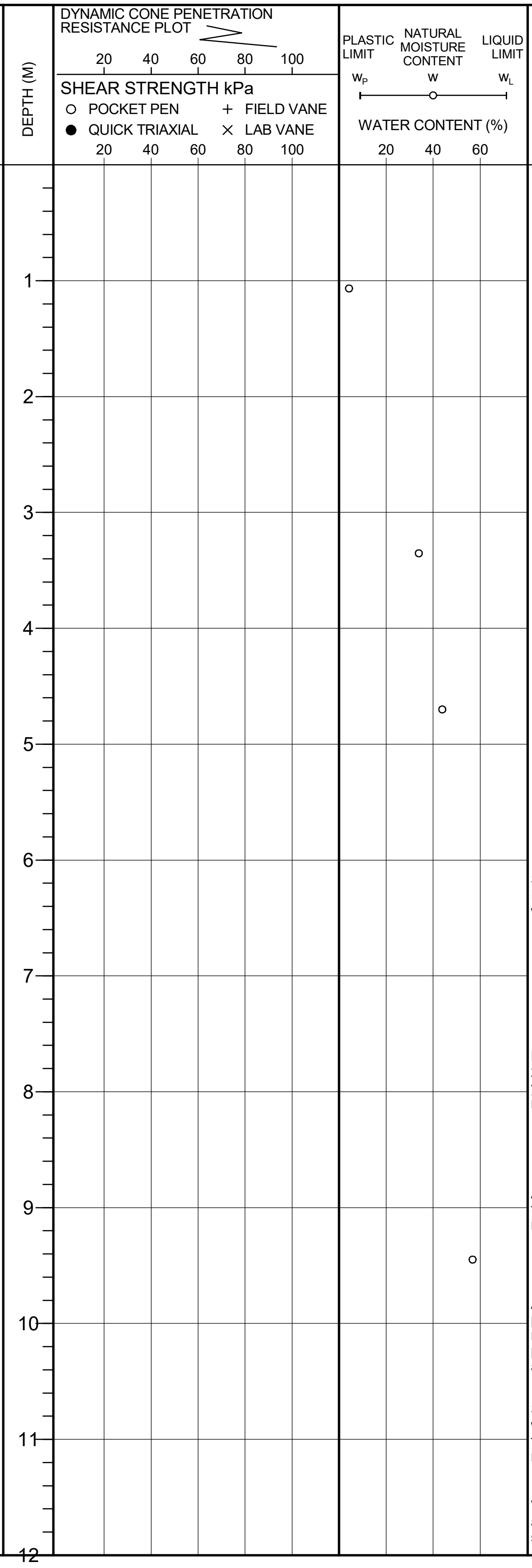
METRIC

JOB NUMBER 22-0887 LOCATION South Bridge Abutment, Iron Bridge ORIGINATED BY SdB

CLIENT Huron Shores DATUM GS BOREHOLE TYPE HSA/Washbore COMPILED BY KC

DRILLER Landcore DATE 2022.05.26 NORTHING 5128440 EASTING 329149 CHECKED BY EG

SOIL PROFILE		SAMPLES					GRAIN SIZE DISTRIBUTION (%)	REMARKS
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)	GROUND WATER CONDITIONS	GR SA SI CL	
0.00 0.10	TOPSOIL, sandy, some gravel, rootlets, brown, moist, compact FILL - (SW) Gravelly SAND, trace silt, trace organics subrounded, brown, moist, non-cohesive, compact to loose	1A/B	SS	12	54			
		2	SS	12	54		28 63 (9)	
	trace wood debris recovered in SS3	3	SS	6	50			
2.29	(GP) GRAVEL, some sand, angular to rounded, brown, wet, non-cohesive, compact	4	SS	22	42	∇		
3.05	(ML) SILT to SANDY SILT, some clay, trace organics, grey-brown, wet, slightly cohesive, low plasticity, soft	5	SS	4	83		1 29 56 14	
4.78	(SW) SAND, trace gravel, trace plastic fines, brown, wet, non-cohesive, loose	6A/B	SS	9	92			
6.10	(SP) SAND, medium grained, some silt, trace woody debris, grey-brown, wet, non-cohesive, compact	7	SS	10	63			Switched from HSA to washboring
7.62	(ML) Sandy SILT to SILT, some clay, brown, wet, slightly cohesive, low plasticity, soft	8	SS	4	0			Low recovery, 2nd attempt yielded full recovery Atterberg limit taken in SS9 - non-plastic result
9.14	(ML) SILT, some clay, trace sand, grey-brown, no -cohesive, wet, very loose	9	SS	WH	100		0 1 85 14	Attempted to push vane, silty material prevented advancement
		10	SS	WH	100			Sandy return water washboring from 10.7 to 12.2 mbgs
		1	VANE					Vane test attempted, no shear



Continued Next Page

200 + : Numbers refer to Field Vane Over Limit

+3, ×3 : Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

TULLOCH ENGINEERING

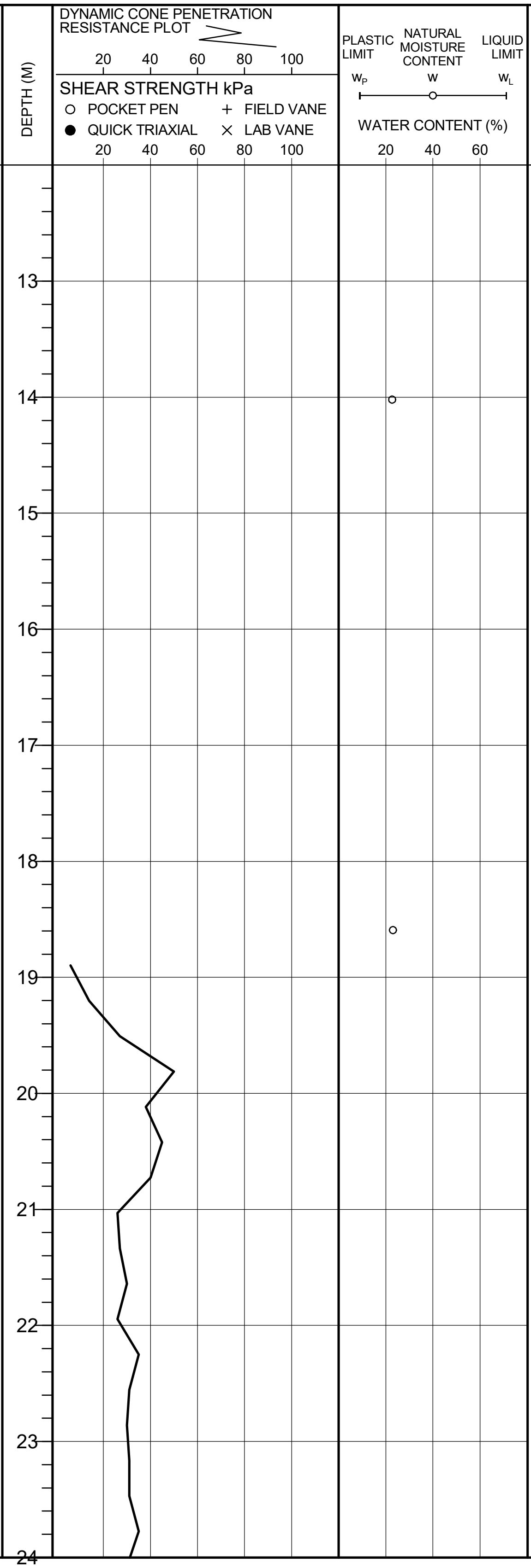
RECORD OF BOREHOLE No BH-22-02

2 OF 3

METRIC

JOB NUMBER 22-0887 LOCATION South Bridge Abutment, Iron Bridge ORIGINATED BY SdB
CLIENT Huron Shores DATUM GS BOREHOLE TYPE HSA/Washbore COMPILED BY KC
DRILLER Landcore DATE 2022.05.26 NORTHING 5128440 EASTING 329149 CHECKED BY EG

SOIL PROFILE			SAMPLES						REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)	GROUND WATER CONDITIONS	DEPTH (M)	GR	SA	SI	CL
12.19	(SW) SAND, medium to coarse grained, trace gravel, trace non-plastic fines, brown, wet, non-cohesive, compact to dense		11	SS	17	25						
			12	SS	22	50		14	1	98	(1)	
			13	SS	27	71						
			14	SS	33	100			Casing seized in dense sand, snapping the casing sub			
18.90	DCPT Advancement											



Continued Next Page

200 + : Numbers refer to Field Vane Over Limit

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

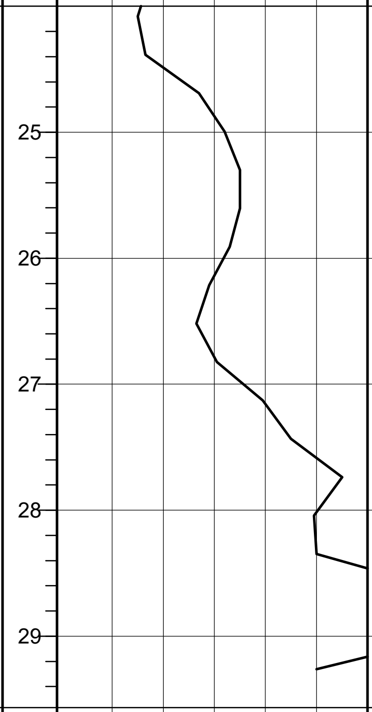
TULLOCH
ENGINEERING

RECORD OF BOREHOLE No BH-22-02

3 OF 3

METRIC

JOB NUMBER 22-0887 LOCATION South Bridge Abutment, Iron Bridge ORIGINATED BY SdB
CLIENT Huron Shores DATUM GS BOREHOLE TYPE HSA/Washbore COMPILED BY KC
DRILLER Landcore DATE 2022.05.26 NORTHING 5128440 EASTING 329149 CHECKED BY EG

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	RECOVERY RATIO (%)	GROUND WATER CONDITIONS	DEPTH (M)	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ POCKET PEN + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 20 40 60	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	DCPT Advancement *(continued)*										
29.57	END OF BOREHOLE - DCPT Refusal Note: Water encountered at 2.6 mbgs after completion of investigation. Note that ground water may not be stabilized upon completion of investigation. Borehole caved at 4.6 mbgs after casing was pulled.										

$^{200}+$: Numbers refer to Field Vane Over Limit

$+^{3}, \times^{3}$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

APPENDIX E

LABORATORY REPORTS

WATER CONTENT TEST

TEST METHOD: LS 701 / ASTM C 566 / D 2216

CONTRACT NO: 22-0887
DATE SAMPLED: 2022-05-24

PROJECT: Potomac Bridge GI
SOURCE: Borehole

DATE TESTED: 2022-07-05
TESTED BY: S. Campbell

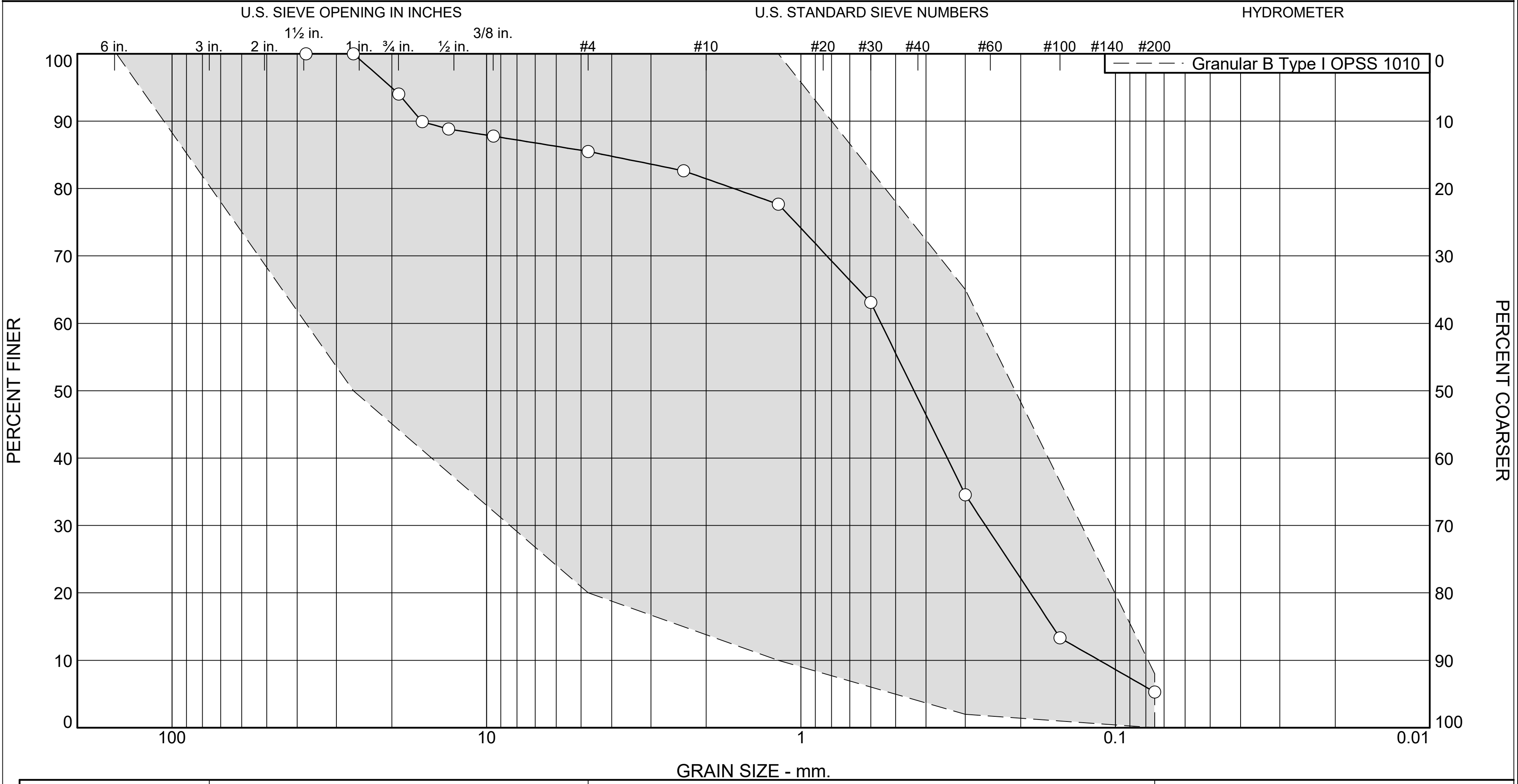
			Gross (inc. Tare) (g)				
Tare ID	**Sample ID**	**Depth (m)**	**Wet Weight**	**Dry Weight**	**TARE**	**Mass Lost**	**Water %**
	BH-22-01 SS02	0.8 to 1.4	611.92	602.90	271.88	9.02	2.7%
	BH-22-01 SS05	3.0 to 3.7	1135.00	934.69	221.89	200.31	28.1%
	BH-22-01 SS08	7.6 to 8.2	72.92	55.44	13.86	17.48	42.0%
	BH-22-01 SS13	15.2 to 15.8	313.77	299.05	219.60	14.72	18.5%
	BH-22-01 SS16	24.4 to 25.0	70.94	60.46	16.68	10.48	23.9%

REMARKS:

CLIENT: Township of Iron Bridge

COPIES TO:

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt
○ 0.0	5.9	8.6	4.1	32.5	43.6	5.3

Identification	Date Sampled	Date Received	Date Tested
Source of Sample: BH-22-01 Depth: 0.8m - 1.4m Sample Number: SS02	May 24/22		July 6/22

Client Township of Iron Bridge

Project Potomac Bridge GI

Project No. 22-0887

Figure

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○ F.M.=2.61

Tested By: L. Roach **Checked By:** T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2022-07-12

Client: Township of Iron Bridge
Project: Potomac Bridge GI
Project Number: 22-0887
Location: BH-22-01
Depth: 0.8m - 1.4m **Sample Number:** SS02
Date Sampled: May 24/22 **Date Tested:** July 6/22
Tested by: L. Roach **Checked by:** T. Linley
Material specification: Granular B Type I OPSS 1010

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained	Lower Spec. Limit, %	Upper Spec. Limit, %	Deviation From Spec., %
602.90	271.88	37.5mm	0.00	0.00	100.0	0.0			
		26.5mm	0.00	0.00	100.0	0.0	50.0	100.0	
		19mm	19.80	0.00	94.0	6.0			
		16mm	13.60	0.00	89.9	10.1			
		13.2mm	3.60	0.00	88.8	11.2			
		9.5mm	3.50	0.00	87.8	12.2			
		#4	7.50	0.00	85.5	14.5	20.0	100.0	
		#8	9.50	0.00	82.6	17.4			
		#16	16.40	0.00	77.7	22.3	10.0	100.0	
		#30	48.30	0.00	63.1	36.9			
		#50	94.50	0.00	34.5	65.5	2.0	65.0	
		#100	70.20	0.00	13.3	86.7			
		#200	26.60	0.00	5.3	94.7	0.0	8.0	

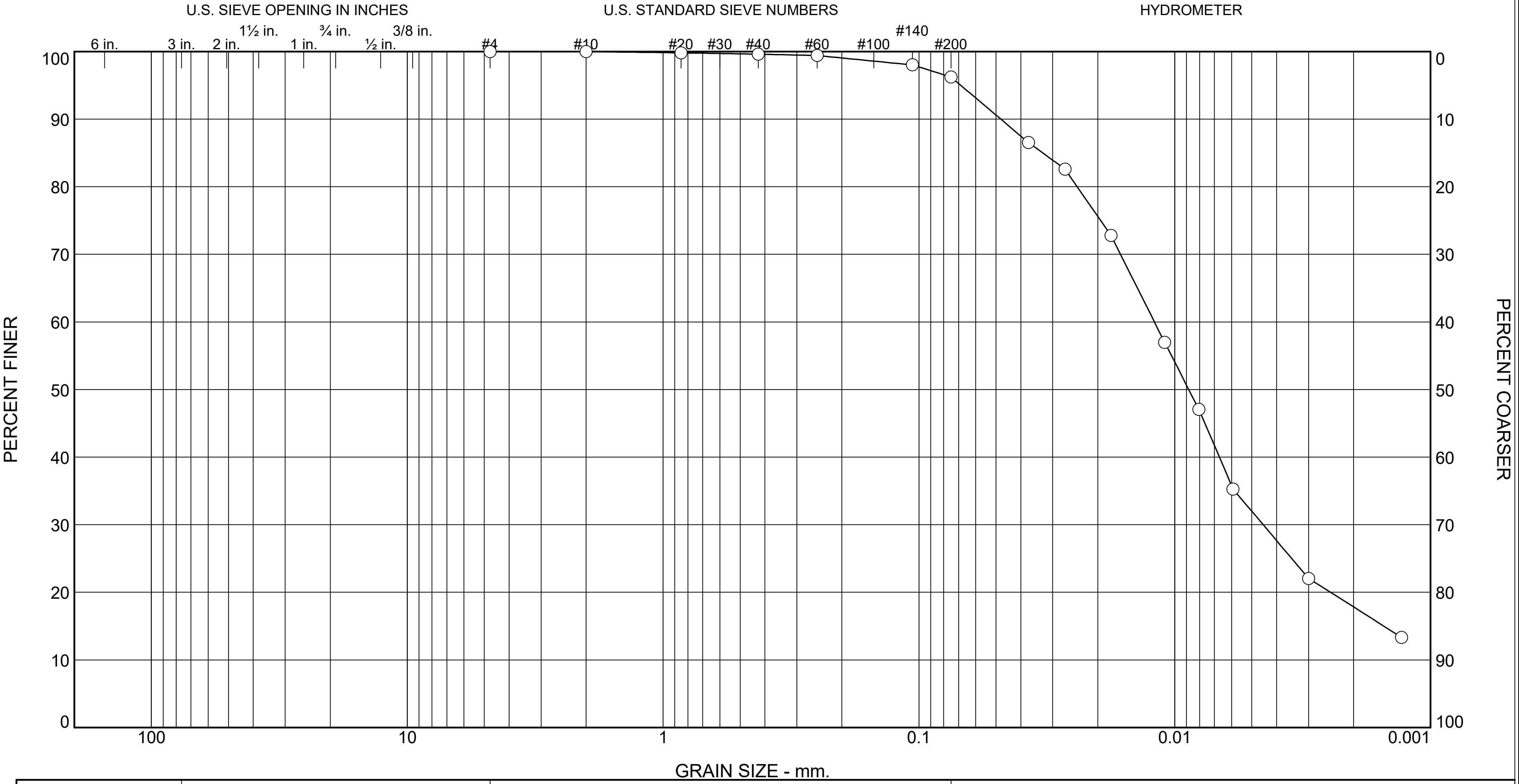
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	5.9	8.6	14.5	4.1	32.5	43.6	80.2			5.3

D_{5}	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.1126	0.1584	0.1865	0.2587	0.3426	0.4367	0.5567	1.6336	4.2057	16.0604	20.0661

Fineness Modulus	C_{u}	C_{c}
2.61	4.95	1.07

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.0	0.0	0.4	3.4	78.4	17.8

Identification	Date Sampled	Date Received	Date Tested
Source of Sample: BH-22-01 Depth: 3.0m - 3.7m Sample Number: SS05	May 24/22		July 5/22

Client Township of Iron Bridge

Project Potomac Bridge GI

Project No. 22-0887

Figure

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○ F.M.=0.02

Tested By: T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2022-07-12

Client: Township of Iron Bridge
Project: Potomac Bridge GI
Project Number: 22-0887
Location: BH-22-01
Depth: 3.0m - 3.7m **Sample Number:** SS05
Date Sampled: May 24/22 **Date Tested:** July 5/22
Tested by: T. Linley

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained
934.69	221.89	#4	0.00	0.00	100.0	0.0
		#10	0.00	0.00	100.0	0.0
50.41	0.00	#20	0.10	0.00	99.8	0.2
		#40	0.10	0.00	99.6	0.4
		#60	0.10	0.00	99.4	0.6
		#140	0.70	0.00	98.0	2.0
		#200	0.90	0.00	96.2	3.8

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 100.0
Weight of hydrometer sample =50.41
Automatic temperature correction
Composite correction (fluid density and meniscus height) at 20 deg. C = -5
Meniscus correction only = -1.0
Specific gravity of solids = 2.70
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - .164 **x Rm**

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer	Percent Retained
1.00	24.5	48.0	44.1	0.0127	47.0	8.6	0.0373	86.5	13.5
2.00	24.5	46.0	42.1	0.0127	45.0	8.9	0.0269	82.6	17.4
5.00	24.5	41.0	37.1	0.0127	40.0	9.7	0.0178	72.8	27.2
15.00	24.3	33.0	29.0	0.0128	32.0	11.0	0.0110	57.0	43.0
30.00	24.1	28.0	24.0	0.0128	27.0	11.9	0.0080	47.0	53.0
60.00	24.1	22.0	18.0	0.0128	21.0	12.9	0.0059	35.3	64.7
250.00	24.9	15.0	11.2	0.0127	14.0	14.0	0.0030	22.1	77.9
1440.00	23.5	11.0	6.8	0.0129	10.0	14.7	0.0013	13.3	86.7

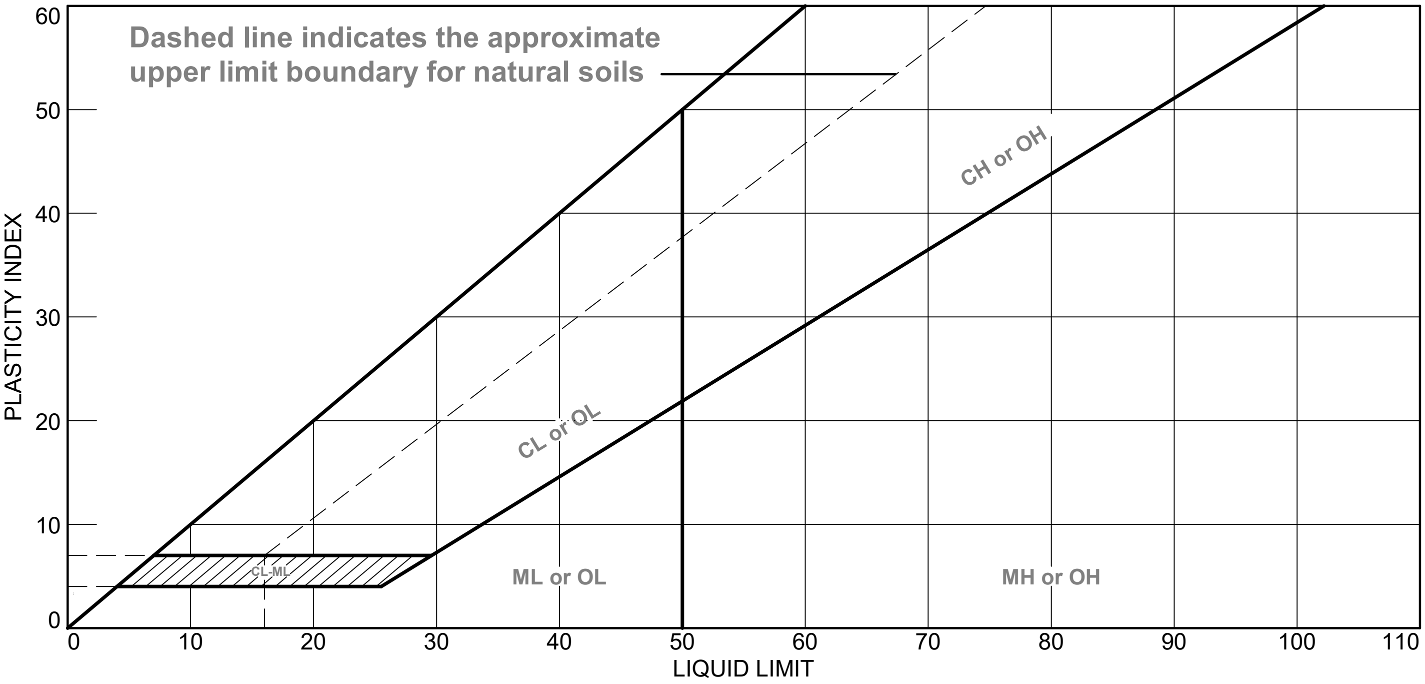
Fractional Components

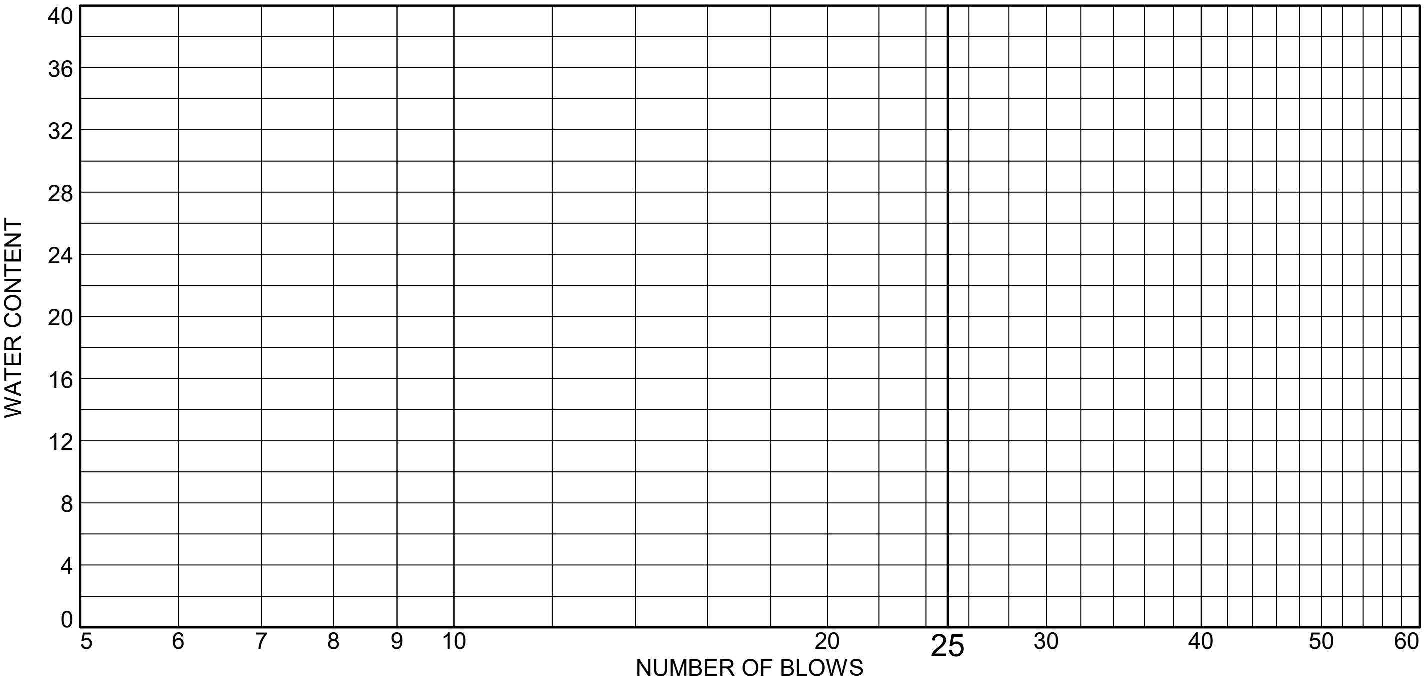
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	0.0	0.4	3.4	3.8	78.4	17.8	96.2

D_{5}	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
		0.0015	0.0025	0.0045	0.0067	0.0088	0.0120	0.0241	0.0328	0.0479	0.0686

Fineness Modulus
0.02

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●					99.6	96.2	

Project No. 22-0887 **Client:** Township of Iron Bridge

Project: Potomac Bridge GI

Source of Sample: BH-22-01 **Depth:** 3.0m - 3.7m
Sample Number: SS05

Remarks:

● Atterberg attempted, material determined to be non-plastic

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Figure

GRAIN SIZE DISTRIBUTION TEST DATA

2022-07-12

Client: Township of Iron Bridge
Project: Potomac Bridge GI
Project Number: 22-0887
Location: BH-22-01
Depth: 15.2m - 15.8m **Sample Number:** SS13
Date Sampled: May 24/22 **Date Tested:** July 5/22
Tested by: L. Roach **Checked by:** T. Linley

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained
299.10	219.60	13.2mm	0.00	0.00	100.0	0.0
		9.5mm	0.00	0.00	100.0	0.0
		#4	1.80	0.00	97.7	2.3
		#8	1.60	0.00	95.7	4.3
		#16	13.90	0.00	78.2	21.8
		#30	43.20	0.00	23.9	76.1
		#50	12.30	0.00	8.4	91.6
		#100	4.70	0.00	2.5	97.5
		#200	0.40	0.00	2.0	98.0

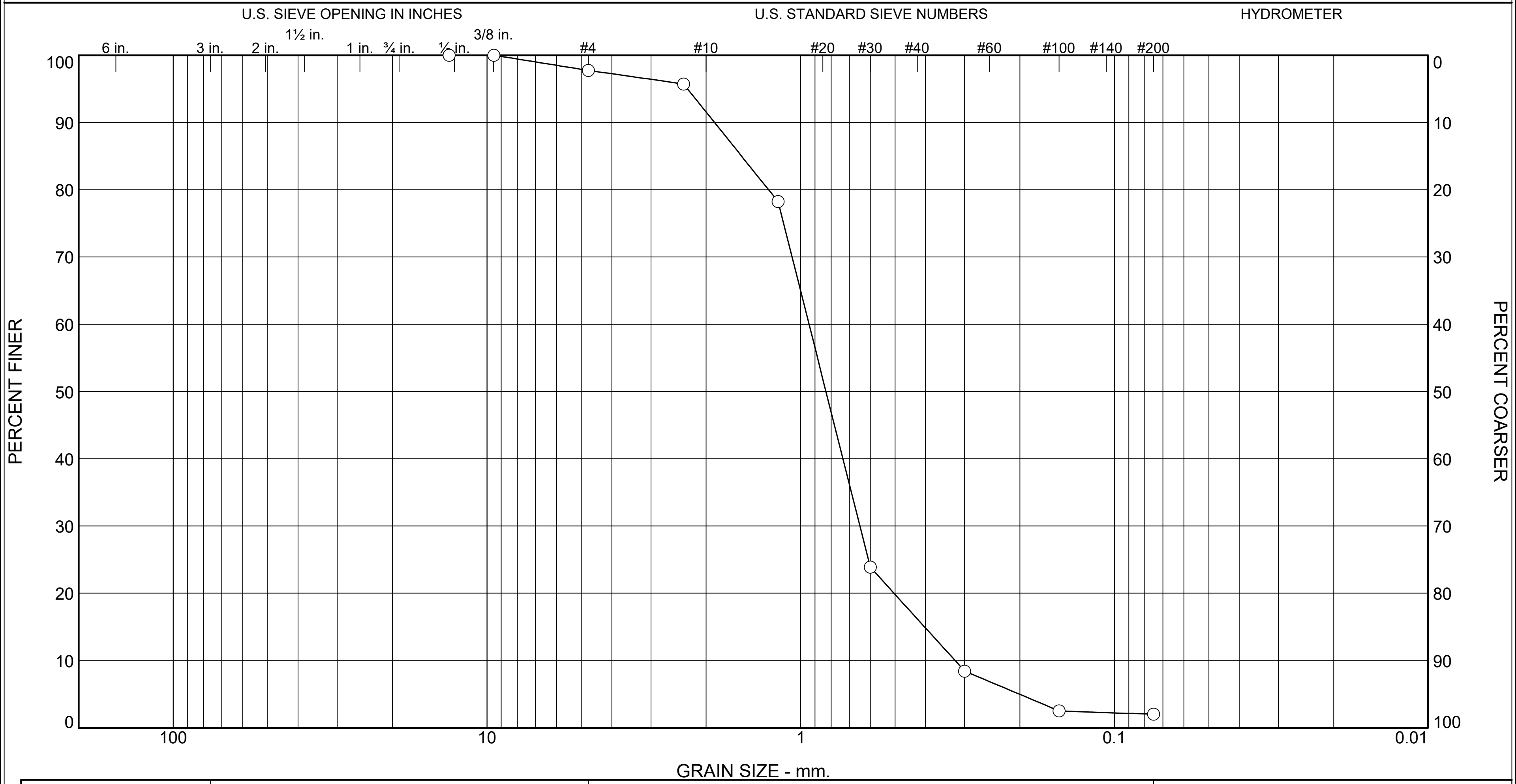
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	2.3	2.3	6.2	75.3	14.2	95.7			2.0

D_{5}	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.2007	0.3219	0.4027	0.5038	0.6473	0.7331	0.8303	0.9404	1.2653	1.5427	1.8809	2.2933

Fineness Modulus	C_u	C_c
2.93	2.92	1.38

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines
		Coarse	Fine	Coarse	Medium	Fine	Silt
○	0.0	0.0	2.3	6.2	75.3	14.2	2.0

Identification	Date Sampled	Date Received	Date Tested
Source of Sample: BH-22-01 Depth: 15.2m - 15.8m Sample Number: SS13	May 24/22		July 5/22

○ F.M.=2.93

Client Township of Iron Bridge

Project Potomac Bridge GI

Project No. 22-0887

Figure

TULLOCH ENGINEERING

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Tested By: L. Roach **Checked By:** T. Linley

TULLOCH ENGINEERING

CSA A283 Certified Laboratory for Concrete Testing
CCIL Certified Laboratory for Aggregates and Asphalt Testing
CSA/CCIL Certified Technicians

Canadian Council of Independent Laboratories
Conseil canadi des laboratoir indépendants

WATER CONTENT TEST

TEST METHOD: LS 701 / ASTM C 566 / D 2216

CONTRACT NO: 22-0887

DATE SAMPLED: 2022-05-24

PROJECT: Potomac Bridge GI

SOURCE: Borehole

DATE TESTED: 2022-07-05

TESTED BY: S. Campbell

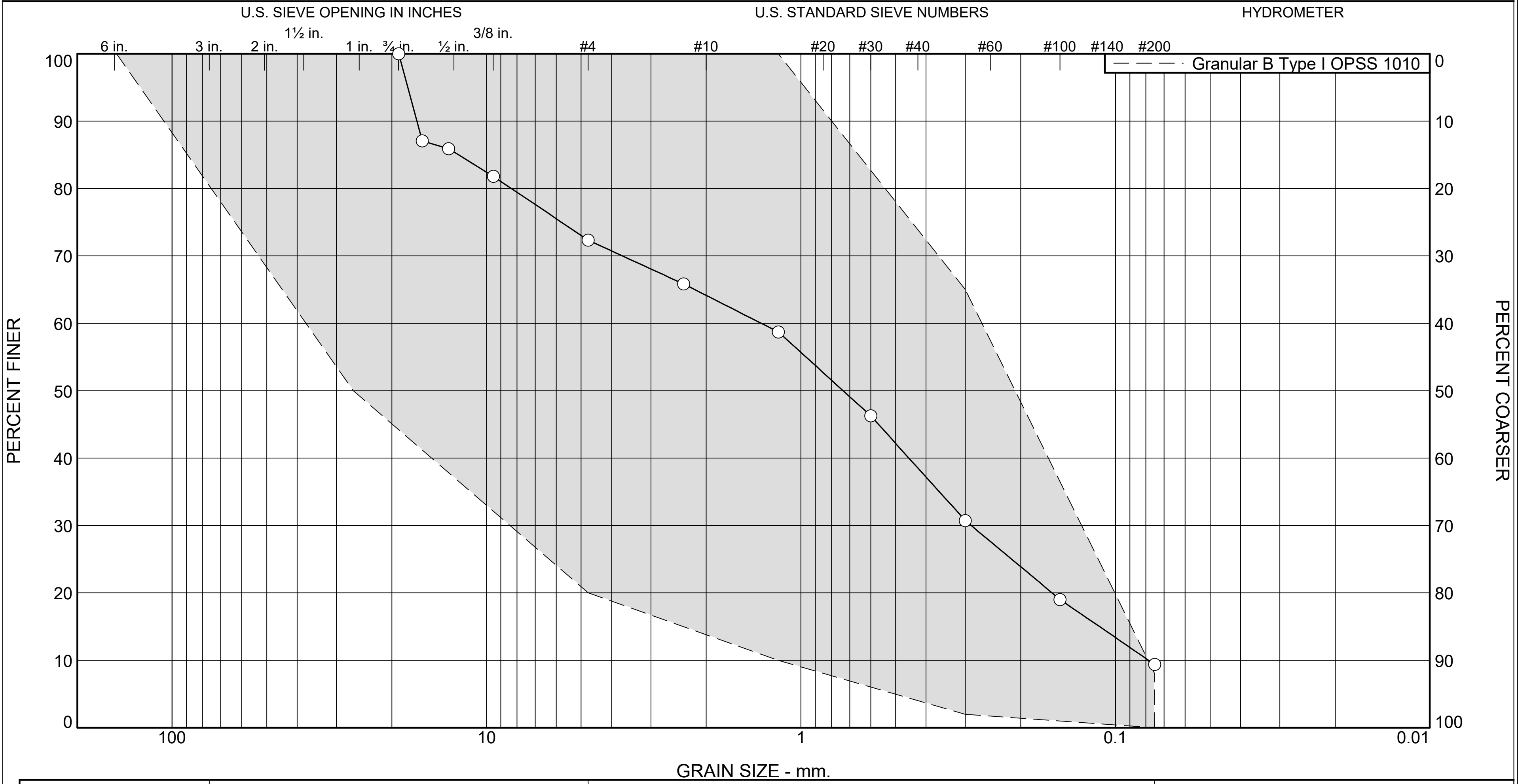
			Gross (inc. Tare) (g)				
Tare ID	**Sample ID**	**Depth (m)**	**Wet Weight**	**Dry Weight**	**TARE**	**Mass Lost**	**Water %**
	BH-22-02 SS02	0.8 to 1.4	626.45	609.86	217.73	16.59	4.2%
	BH-22-02 SS05	3.0 to 3.7	87.00	70.87	23.35	16.13	33.9%
	BH-22-02 SS06	4.6 to 5.2	599.87	478.51	201.84	121.36	43.9%
	BH-22-02 SS09	9.1 to 9.8	1449.58	1060.60	375.20	388.98	56.8%
	BH-22-02 SS12	13.7 to 14.3	816.14	706.33	221.60	109.81	22.7%
	BH-22-02 SS14	18.3 to 18.9	135.34	113.80	20.08	21.54	23.0%

REMARKS:

CLIENT: Township of Iron Bridge

COPIES TO:

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines
		Coarse	Fine	Coarse	Medium	Fine	Silt
○	0.0	0.0	27.7	8.2	25.6	29.1	9.4

Identification	Date Sampled	Date Received	Date Tested
Source of Sample: BH-22-02 Depth: 0.8m - 1.4m Sample Number: SS02	May 24/22		July 6/22

Client Township of Iron Bridge

Project Potomac Bridge GI

Project No. 22-0887

Figure

TULLOCH ENGINEERING

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○ F.M.=3.25

Tested By: L. Roach **Checked By:** T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2022-07-12

Client: Township of Iron Bridge
Project: Potomac Bridge GI
Project Number: 22-0887
Location: BH-22-02
Depth: 0.8m - 1.4m **Sample Number:** SS02
Date Sampled: May 24/22 **Date Tested:** July 6/22
Tested by: L. Roach **Checked by:** T. Linley
Material specification: Granular B Type I OPSS 1010

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained	Lower Spec. Limit, %	Upper Spec. Limit, %	Deviation From Spec., %
609.90	217.70	19mm	0.00	0.00	100.0	0.0			
		16mm	50.80	0.00	87.0	13.0			
		13.2mm	4.50	0.00	85.9	14.1			
		9.5mm	16.10	0.00	81.8	18.2			
		#4	37.10	0.00	72.3	27.7	20.0	100.0	
		#8	25.50	0.00	65.8	34.2			
		#16	27.90	0.00	58.7	41.3	10.0	100.0	
		#30	48.80	0.00	46.3	53.7			
		#50	61.00	0.00	30.7	69.3	2.0	65.0	
		#100	45.90	0.00	19.0	81.0			
		#200	37.80	0.00	9.4	90.6	0.0	8.0	*+1.4*

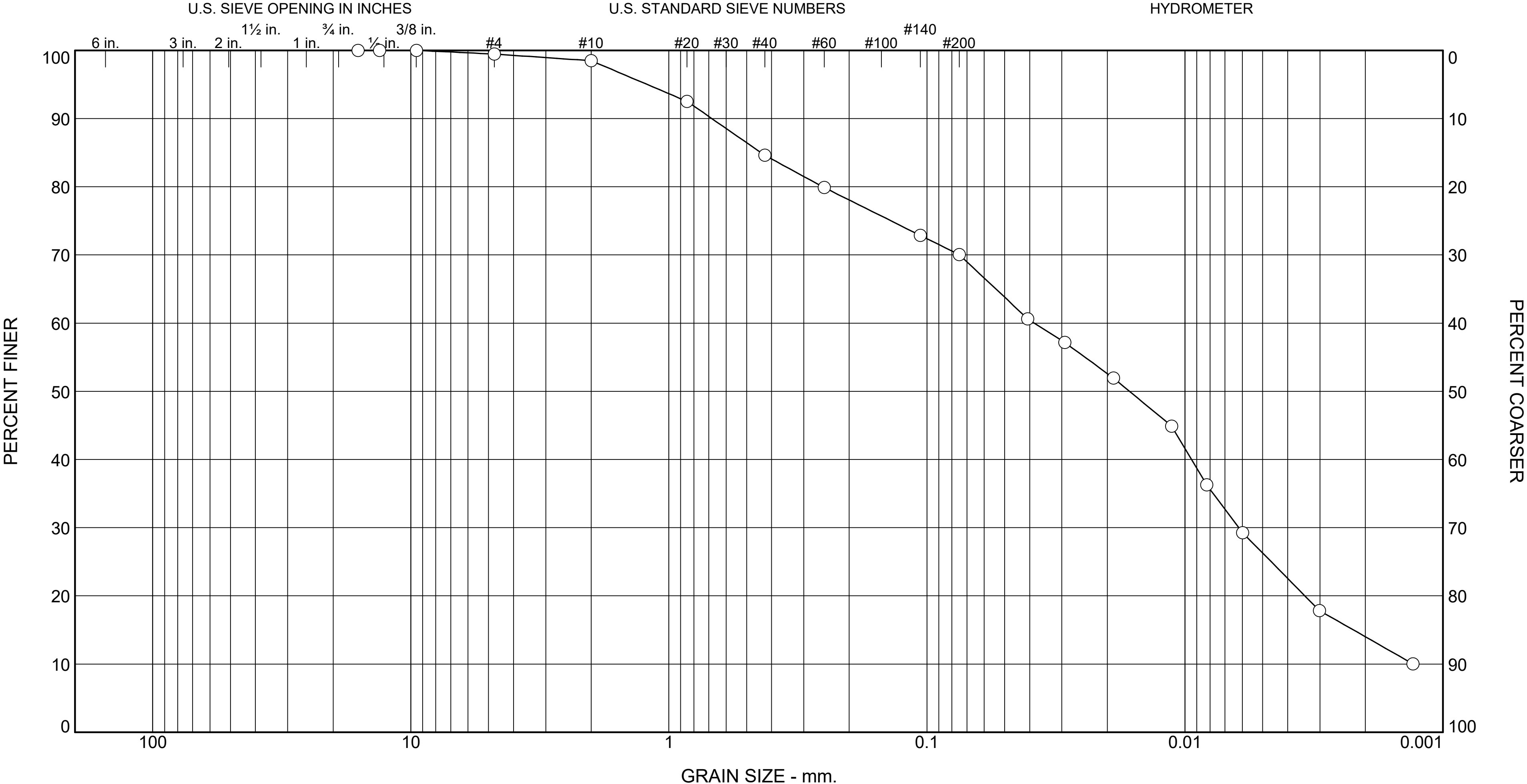
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	27.7	27.7	8.2	25.6	29.1	62.9			9.4

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.0784	0.1123	0.1590	0.2874	0.4536	0.7346	1.3367	8.3292	12.2816	16.6392	17.7805

Fineness Modulus	C_u	C_c
3.25	17.05	0.79

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.5	1.0	13.9	14.6	56.0	14.0

Identification	Date Sampled	Date Received	Date Tested
Source of Sample: BH-22-02 Depth: 4.6m - 5.2m Sample Number: SS06A	May 24/22		July 5/22

Client Township of Iron Bridge

Project Potomac Bridge GI

Project No. 22-0887

Figure

TULLOCH ENGINEERING

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Daren.Stadnisky@TULLOCH.ca

○ F.M.=0.61

Tested By: T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2022-07-12

Client: Township of Iron Bridge
Project: Potomac Bridge GI
Project Number: 22-0887
Location: BH-22-02
Depth: 4.6m - 5.2m **Sample Number:** SS06A
Date Sampled: May 24/22 **Date Tested:** July 5/22
Tested by: T. Linley

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained
478.50	201.80	16mm	0.00	0.00	100.0	0.0
		13.2mm	0.00	0.00	100.0	0.0
		9.5mm.	0.00	0.00	100.0	0.0
		#4	1.50	0.00	99.5	0.5
		#10	2.70	0.00	98.5	1.5
56.10	0.00	#20	3.40	0.00	92.5	7.5
		#40	4.50	0.00	84.6	15.4
		#60	2.70	0.00	79.9	20.1
		#140	4.00	0.00	72.9	27.1
		#200	1.60	0.00	70.0	30.0

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 98.5
Weight of hydrometer sample =56.1
Automatic temperature correction
Composite correction (fluid density and meniscus height) at 20 deg. C = -5
Meniscus correction only = -1.0
Specific gravity of solids = 2.70
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 **-** .164 **x Rm**

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer	Percent Retained
1.00	23.9	39.0	34.9	0.0128	38.0	10.1	0.0407	60.6	39.4
2.00	23.9	37.0	32.9	0.0128	36.0	10.4	0.0292	57.1	42.9
5.00	23.9	34.0	29.9	0.0128	33.0	10.9	0.0189	51.9	48.1
15.00	23.7	30.0	25.9	0.0129	29.0	11.5	0.0113	44.9	55.1
30.00	23.8	25.0	20.9	0.0128	24.0	12.4	0.0082	36.3	63.7
60.00	23.7	21.0	16.9	0.0129	20.0	13.0	0.0060	29.3	70.7
250.00	25.0	14.0	10.3	0.0127	13.0	14.2	0.0030	17.8	82.2
1440.00	23.4	10.0	5.8	0.0129	9.0	14.8	0.0013	10.0	90.0

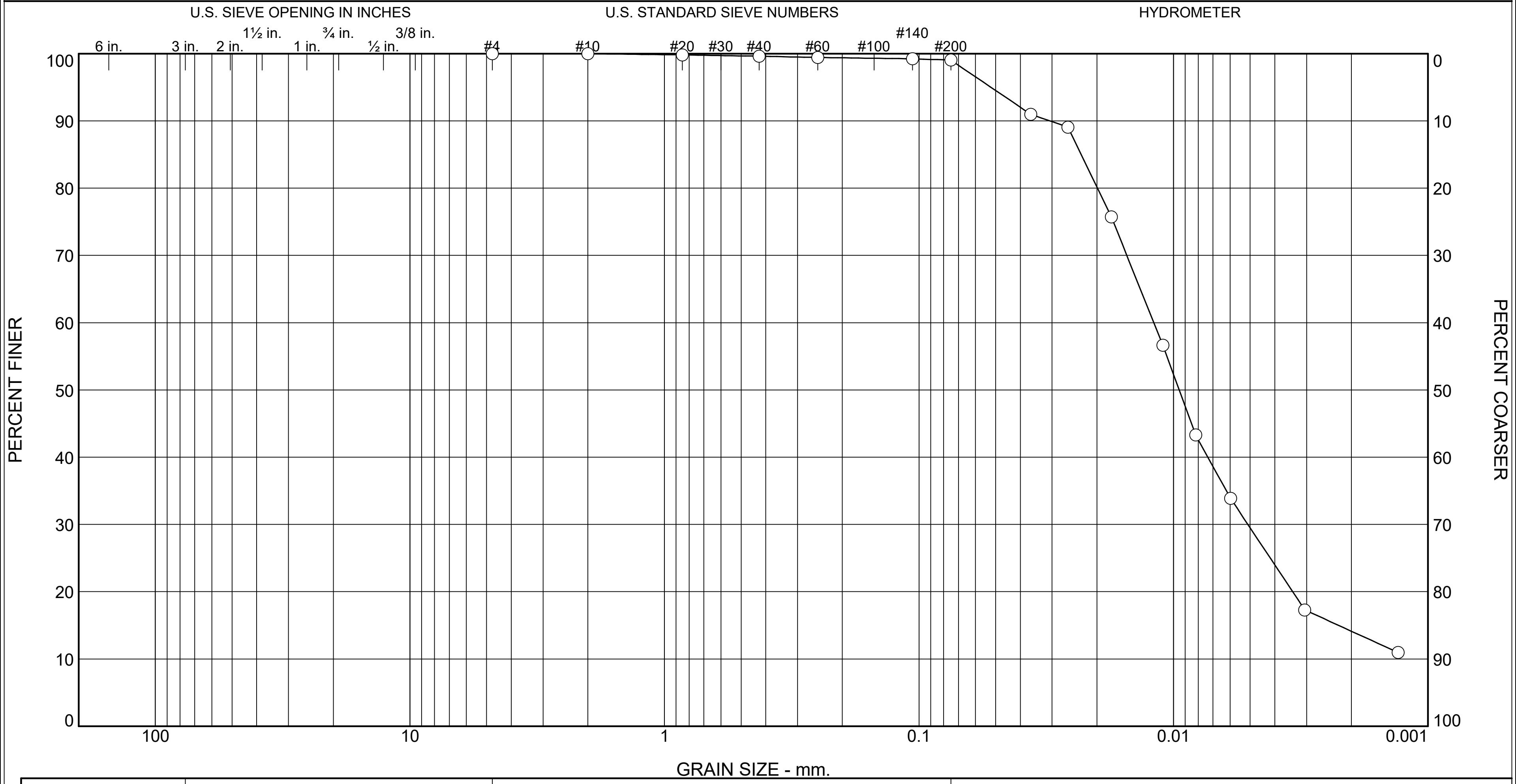
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.5	0.5	1.0	13.9	14.6	29.5	56.0	14.0	70.0

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
		0.0022	0.0034	0.0062	0.0094	0.0164	0.0383	0.2535	0.4396	0.6818	1.2140

Fineness Modulus
0.61

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.0	0.0	0.4	0.6	84.9	14.1

Identification	Date Sampled	Date Received	Date Tested
Source of Sample: BH-22-02 Depth: 9.1m - 9.8m Sample Number: SS09	May 24/22		July 5/22

Client Township of Iron Bridge

Project Potomac Bridge GI

Project No. 22-0887

Figure

TULLOCH ENGINEERING

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○ F.M.=0.02

Tested By: T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2022-07-12

Client: Township of Iron Bridge
Project: Potomac Bridge GI
Project Number: 22-0887
Location: BH-22-02
Depth: 9.1m - 9.8m **Sample Number:** SS09
Date Sampled: May 24/22 **Date Tested:** July 5/22
Tested by: T. Linley

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained
1060.60	375.20	#4	0.00	0.00	100.0	0.0
		#10	0.00	0.00	100.0	0.0
51.90	0.00	#20	0.10	0.00	99.8	0.2
		#40	0.10	0.00	99.6	0.4
		#60	0.10	0.00	99.4	0.6
		#140	0.10	0.00	99.2	0.8
		#200	0.10	0.00	99.0	1.0

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 100.0
Weight of hydrometer sample =51.9
Automatic temperature correction
Composite correction (fluid density and meniscus height) at 20 deg. C = -5
Meniscus correction only = -1.0
Specific gravity of solids = 2.70
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - .164 **x Rm**

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer	Percent Retained
1.00	23.3	52.0	47.7	0.0129	51.0	7.9	0.0364	91.0	9.0
2.00	23.3	51.0	46.7	0.0129	50.0	8.1	0.0260	89.1	10.9
5.00	23.3	44.0	39.7	0.0129	43.0	9.2	0.0176	75.7	24.3
15.00	23.2	34.0	29.7	0.0129	33.0	10.9	0.0110	56.6	43.4
30.00	23.2	27.0	22.7	0.0129	26.0	12.0	0.0082	43.3	56.7
60.00	23.4	22.0	17.8	0.0129	21.0	12.9	0.0060	33.9	66.1
250.00	24.4	13.0	9.1	0.0127	12.0	14.3	0.0031	17.3	82.7
1440.00	23.3	10.0	5.7	0.0129	9.0	14.8	0.0013	10.9	89.1

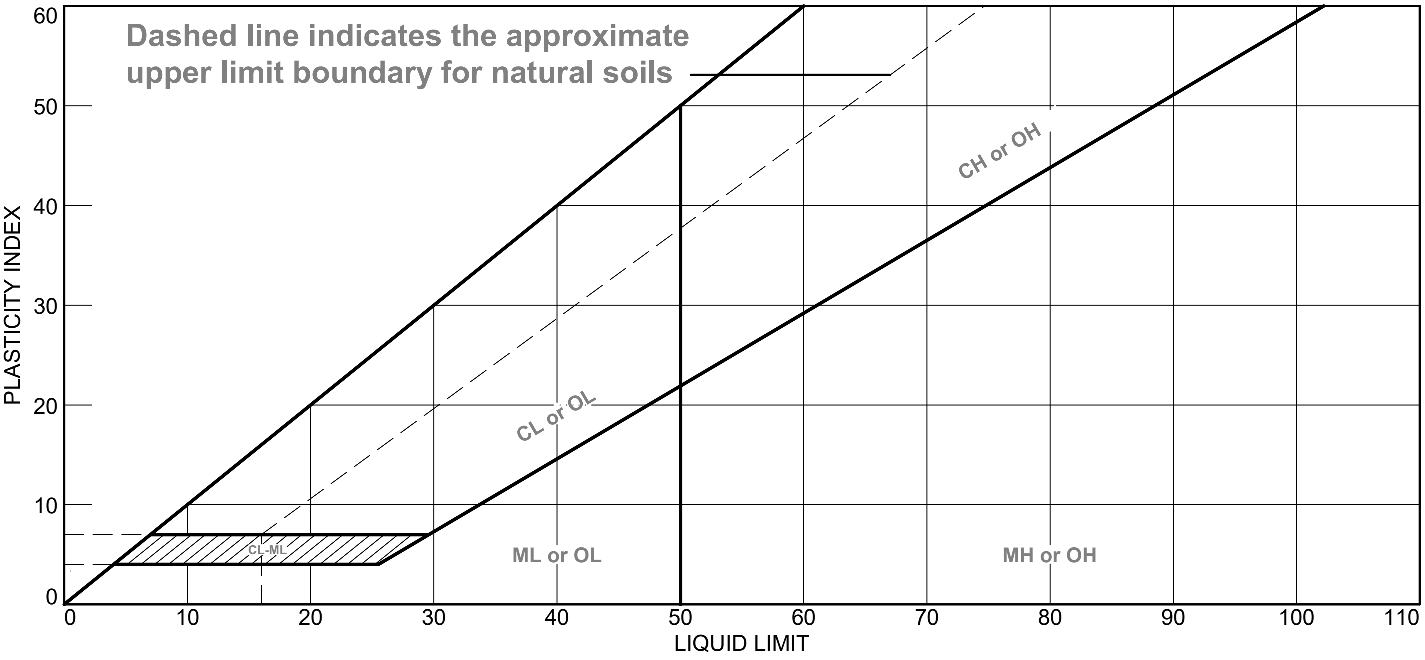
Fractional Components

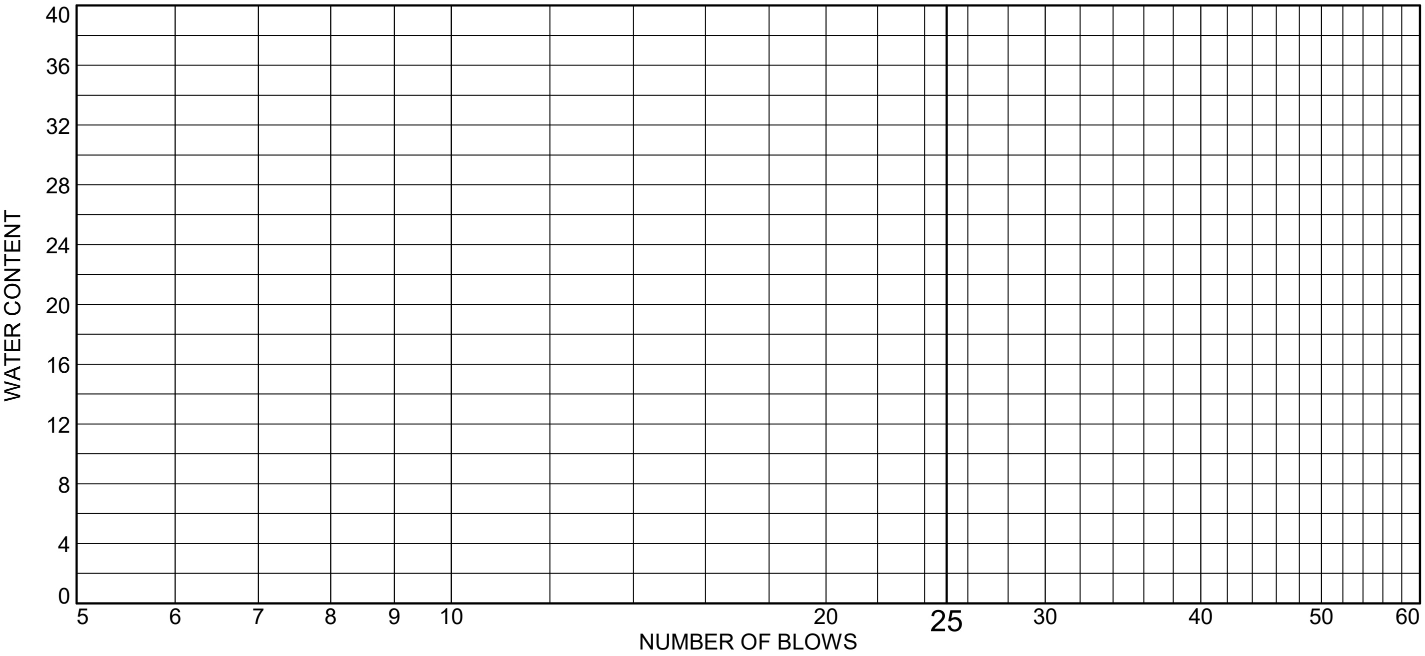
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	0.0	0.4	0.6	1.0	84.9	14.1	99.0

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
		0.0022	0.0034	0.0051	0.0073	0.0095	0.0120	0.0199	0.0230	0.0306	0.0522

Fineness Modulus
0.02

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●					99.6	99.0	

Project No. 22-0887 **Client:** Township of Iron Bridge

Project: Potomac Bridge GI

Source of Sample: BH-22-02 **Depth:** 9.1m - 9.8m
Sample Number: SS09

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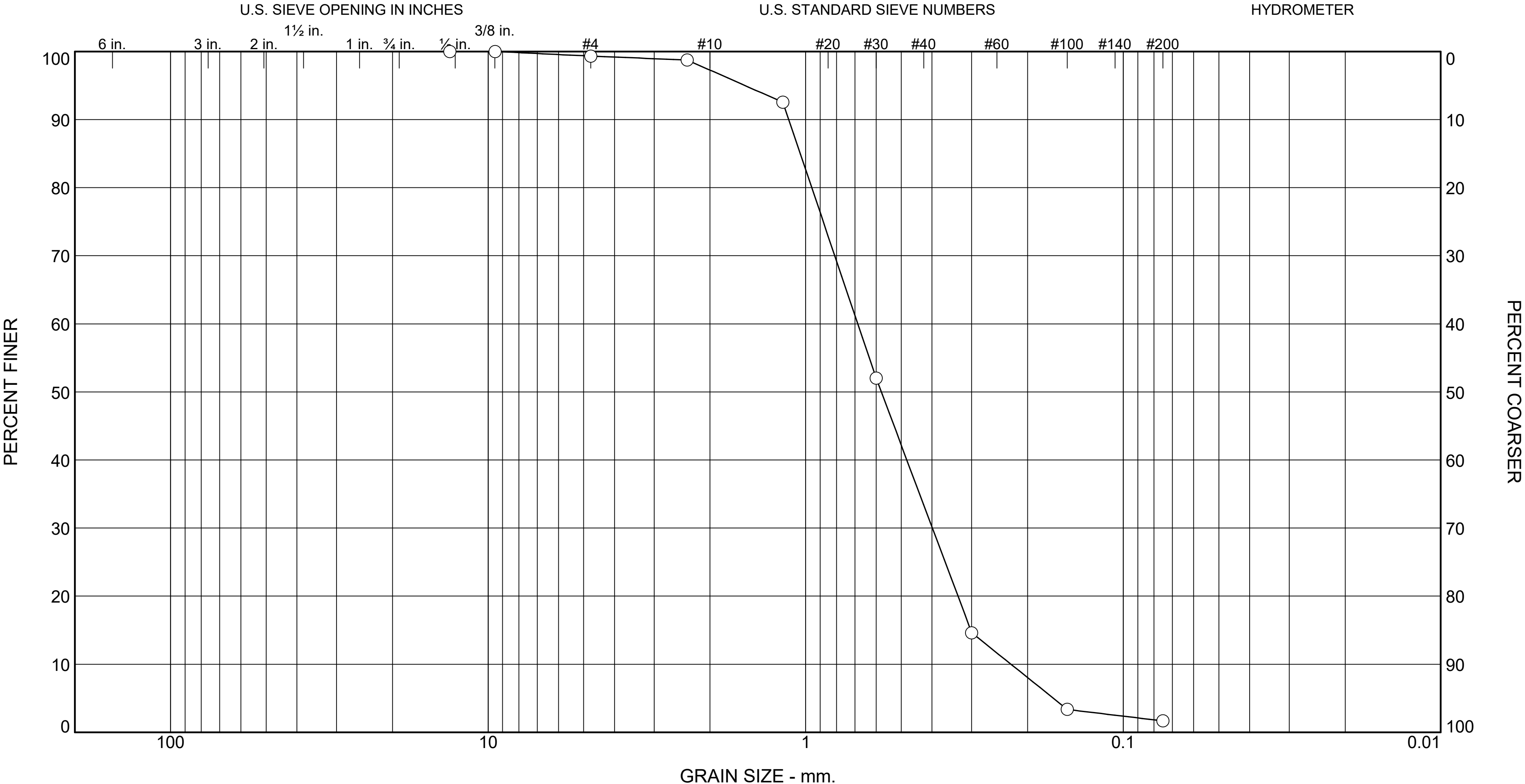
Remarks:

- ● Atterberg attempted, material determined to be non-plastic

Figure

Tested By: T. Linley

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines
		Coarse	Fine	Coarse	Medium	Fine	Silt
○	0.0	0.0	0.7	2.0	63.9	31.7	1.7

Identification	Date Sampled	Date Received	Date Tested
Source of Sample: BH-22-02 Depth: 13.7m - 14.3m Sample Number: SS12	May 24/22		July 6/22

Client Township of Iron Bridge

Project Potomac Bridge GI

Project No. 22-0887

Figure

TULLOCH ENGINEERING

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○ F.M.=2.39

Tested By: L. Roach **Checked By:** T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2022-07-12

Client: Township of Iron Bridge
Project: Potomac Bridge GI
Project Number: 22-0887
Location: BH-22-02
Depth: 13.7m - 14.3m **Sample Number:** SS12
Date Sampled: May 24/22 **Date Tested:** July 6/22
Tested by: L. Roach **Checked by:** T. Linley

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Percent Retained
706.30	221.60	13.2mm	0.00	0.00	100.0	0.0
		9.5mm	0.00	0.00	100.0	0.0
		#4	3.30	0.00	99.3	0.7
		#8	2.80	0.00	98.7	1.3
		#16	30.00	0.00	92.6	7.4
		#30	196.40	0.00	52.0	48.0
		#50	181.30	0.00	14.6	85.4
		#100	54.60	0.00	3.4	96.6
		#200	8.20	0.00	1.7	98.3

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.7	0.7	2.0	63.9	31.7	97.6			1.7

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1659	0.2257	0.3021	0.3314	0.3989	0.4801	0.5778	0.6853	0.9570	1.0402	1.1308	1.5522

Fineness Modulus	C_u	C_c
2.39	3.04	1.03

CERTIFICATE OF ANALYSIS

Client:	Erik Giles	Work Order Number:	468726
Company:	Tulloch Engineering-Sault Ste. Marie	PO #:	
Address:	71 Black Road Sault Ste. Marie, ON, P6B 0A3	Regulation: Project #:	Information not provided
Phone/Fax:	(705) 949-1457 / (705) 949-9606	DWS #:	
Email:	erik.giles@tulloch.ca	Sampled By:	Hannah Logan
Date Order Received:	7/7/2022	Analysis Started:	7/11/2022
Arrival Temperature:	12.5 °C	Analysis Completed:	7/19/2022

WORK ORDER SUMMARY

ANALYSES WERE PERFORMED ON THE FOLLOWING SAMPLES. THE RESULTS RELATE ONLY TO THE ITEMS TESTED.

Sample Description	Lab ID	Matrix	Type	Comments	Date Collected	Time Collected
BH-22-01 SS06	1774343	Soil	None		5/24/2022	
BH-22-02 SS13	1774344	Soil	None		5/26/2022	

METHODS AND INSTRUMENTATION

THE FOLLOWING METHODS WERE USED FOR YOUR SAMPLE(S):

Method	Lab	Description	Reference
Anions Soil (A5)	Garson	Determination of Anions in Soil	Modified from SW846-9056A
Cond Soil (R12)	Garson	Determination of conductivity in soil (1:2)	Modified from EPA SW846-9050A
Moisture (A99)	Garson	Determination of Percent Moisture	In-House
pH Soil (A2.0)	Garson	Determination of soil pH by Ion Selective Electrode	Modified from EPA SW-846 9045D
RedOx - Soil (T06)	Mississauga	Determination of RedOx Potential of Soil	Modified from APHA-2580B
Resistivity Soil (R12)	Garson	Determination of Resistivity in Soil (1:2)	Modified from Carter 18.3
Sulphide/S (R98)	Garson	Determination of Sulphide in Soil	In-House

REPORT COMMENTS

Tulloch Contract 22-0887
Potomac Bridge G1

CERTIFICATE OF ANALYSIS

Tulloch Engineering-Sault Ste. Marie

Work Order Number: 468726

This report has been approved by:

Mahesh Patel.

Mahesh Patel, B.Sc.
Laboratory Director

CERTIFICATE OF ANALYSIS

Tulloch Engineering-Sault Ste. Marie

Work Order Number: 468726

WORK ORDER RESULTS

Sample Description	BH - 22 - 01 SS06		BH - 22 - 02 SS13		
Sample Date	5/24/2022 12:00 AM		5/26/2022 12:00 AM		
Lab ID	1774343		1774344		
Anions	**Result**	**MDL**	**Result**	**MDL**	**Units**
Bromide	<0.2	0.2	<0.2 [<0.2]	0.2	µg/g
Chloride	3.4	0.4	0.9 [0.9]	0.4	µg/g
Fluoride	<0.2	0.2	0.6 [<0.2]	0.2	µg/g
Nitrate (as N)	<0.2	0.2	5.3 [5.3]	0.2	µg/g
Nitrite (as N)	<0.1	0.1	<0.1 [<0.1]	0.1	µg/g
Sulphate	73	2	<2 [<2]	2	µg/g

Sample Description	BH - 22 - 01 SS06		BH - 22 - 02 SS13		
Sample Date	5/24/2022 12:00 AM		5/26/2022 12:00 AM		
Lab ID	1774343		1774344		
General Chemistry	**Result**	**MDL**	**Result**	**MDL**	**Units**
% Moisture	22.0	0.1	17.8	0.1	%
Conductivity	96	1	69	1	µS/cm
pH	5.25	N/A	7.84 [7.84]	N/A	pH
RedOx (vs. S.H.E.)	322	N/A	310 [308]	N/A	mV
Resistivity	10400	N/A	14500	N/A	ohm-cm
Sulphide	<0.3	0.3	<0.3 [<0.3]	0.3	µg/g

LEGEND

Dates: Dates are formatted as mm/dd/year throughout this report.

MDL: Method detection limit or minimum reporting limit.

[ ]: Results for laboratory replicates are shown in square brackets immediately below the associated sample result for ease of comparison.

Quality Control: All associated Quality Control data is available on request.

LCL: Lower Control Limit.

UCL: Upper Control Limit.

QAQCID: This is a unique reference to the quality control data set used to generate the reported value. Contact our lab for this information, as it is traceable through our LIMS.

Field Data: Reports containing Field Parameters represent data that has been collected and provided by the client. Testmark is not responsible for the validity of this data which may be used in subsequent calculations.

Sample Condition Deviations: A noted sample condition deviation may affect the validity of the result. Results apply to the sample(s) as received.

Reproduction of Report: Report shall not be reproduced, except in full, without the approval of Testmark Laboratories Ltd.

ICPMS Dustfall Insoluble: The ICPMS Dustfall Insoluble Portion method analyzes only the particulate matter from the Dustfall Sampler which is retained on the analysis filter during the Dustfall method.

CERTIFICATE OF ANALYSIS

QUALITY CONTROL DATA

THIS SECTION REPORTS QC RESULTS ASSOCIATED WITH THE TEST BATCH; THESE ARE NOT YOUR SAMPLE RESULTS.
QAQC details include only values where sufficient sample data allowed measurement.

Anions

Blank: LRB-6 (Blank) (6)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Bromide	0.2	µg/g	0	<0.2	0.6	20220715.A5M
Chloride	0.4	µg/g	0	<0.4	1.2	20220715.A5M
Fluoride	0.02	µg/g	0	<0.02	0.6	20220715.A5M
Nitrate (as N)	0.2	µg/g	0	<0.2	0.6	20220715.A5M
Nitrite (as N)	0.1	µg/g	0	<0.1	0.18	20220715.A5M
Sulphate	0.4	µg/g	0	<0.4	6	20220715.A5M

Positive Control: LFB-5 (0.1/0.02/0.002 mg/g equiv) (5)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Bromide	N/A	%	80	90	120	20220715.A5M
Chloride	N/A	%	80	92.8	120	20220715.A5M
Fluoride	N/A	%	80	82.4	120	20220715.A5M
Nitrate (as N)	N/A	%	80	98	120	20220715.A5M
Nitrite (as N)	N/A	%	80	111	120	20220715.A5M
Sulphate	N/A	%	80	101	120	20220715.A5M

Positive Control: LFB-7 (0.2/0.1/0.02 mg/g equiv) (7)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Bromide	N/A	%	80	97.7	120	20220715.A5M
Chloride	N/A	%	80	99.6	120	20220715.A5M
Fluoride	N/A	%	80	97	120	20220715.A5M
Nitrate (as N)	N/A	%	80	95.4	120	20220715.A5M
Nitrite (as N)	N/A	%	80	94.1	120	20220715.A5M
Sulphate	N/A	%	80	104	120	20220715.A5M

CERTIFICATE OF ANALYSIS

Sample Replicate: % RPD (8)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Nitrate (as N)	N/A	%	0	0	35	20220715.A5M

Sample Spike: MFS-9 (Sample Spike) (9)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Bromide	N/A	% Rec	75	97.4	125	20220715.A5M
Chloride	N/A	% Rec	75	92.4	125	20220715.A5M
Fluoride	N/A	% Rec	75	92.6	125	20220715.A5M
Nitrate (as N)	N/A	% Rec	75	96.8	125	20220715.A5M
Nitrite (as N)	N/A	% Rec	75	95.3	125	20220715.A5M
Sulphate	N/A	% Rec	75	99.9	125	20220715.A5M

General Chemistry

Calibration Check: Lab Control Sample (2)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Conductivity	N/A	%	475	502	525	20220713.TM-G.R12B

Method Blank: Method Blank (1)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Conductivity	1	µS/cm	0	<1	5	20220713.TM-G.R12B

Positive Control: LCS (pH 8) (2)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
pH	N/A	pH	7.8	7.96	8.2	20220714.TM-G.R2B

Positive Control: LFB-7 (7)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Sulphide	0.05	µg/g	0.24	0.342	0.36	20220719.R98B

Positive Control: LRB-6 (Blank) (6)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Sulphide	0.02	µg/g	0	<0.02	0.06	20220719.R98B

CERTIFICATE OF ANALYSIS

Positive Control: ORP Control 240 (7)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
RedOx (vs. S.H.E.)	N/A	mV	220	240	260	20220714.TM-M.A6B

Sample Replicate: % RPD (3)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
pH	N/A	pH	0	0	0.3	20220714.TM-G.R2B

Sample Replicate: % RPD (8)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
Conductivity	N/A	%	0	2	10	20220713.TM-G.R12B

Sample Replicate: % RPD (9)

Parameter	MDL	Units	LCL	Result	UCL	QAQCID
RedOx (vs. S.H.E.)	N/A	%	0	0.6	10	20220714.TM-M.A6B

CERTIFICATE OF ANALYSIS

Tulloch Engineering-Sault Ste. Marie

Work Order Number: 468726

THIS INDEX SHOWS HOW YOUR SAMPLES ARE ASSOCIATED TO THE CONTROLS INCLUDED IN THE IDENTIFIED BATCHES.

Sample Description	Lab ID	Method	QAQCID	Prep QAQCID
BH - 22 - 01 SS06	1774343	Anions Soil (A5)	20220715.A5M	
BH - 22 - 01 SS06	1774343	Cond Soil (R12)	20220713.TM-G.R12B	
BH - 22 - 01 SS06	1774343	Moisture (A99)	20220711.TM-G.A99C	
BH - 22 - 01 SS06	1774343	pH Soil (A2.0)	20220714.TM-G.R2B	
BH - 22 - 01 SS06	1774343	RedOx - Soil (T06)	20220714.TM-M.A6B	
BH - 22 - 01 SS06	1774343	Resistivity Soil (R12)	20220714.TM-G.R12B	
BH - 22 - 01 SS06	1774343	Sulphide/S (R98)	20220719.R98B	
BH - 22 - 02 SS13	1774344	Anions Soil (A5)	20220715.A5M	
BH - 22 - 02 SS13	1774344	Cond Soil (R12)	20220713.TM-G.R12B	
BH - 22 - 02 SS13	1774344	Moisture (A99)	20220711.TM-G.A99C	
BH - 22 - 02 SS13	1774344	pH Soil (A2.0)	20220714.TM-G.R2B	
BH - 22 - 02 SS13	1774344	RedOx - Soil (T06)	20220714.TM-M.A6B	
BH - 22 - 02 SS13	1774344	Resistivity Soil (R12)	20220714.TM-G.R12B	
BH - 22 - 02 SS13	1774344	Sulphide/S (R98)	20220719.R98B	
BH - 22 - 02 SS13	1774344r	Anions Soil (A5)	20220715.A5M	
BH - 22 - 02 SS13	1774344r	pH Soil (A2.0)	20220714.TM-G.R2B	
BH - 22 - 02 SS13	1774344r	RedOx - Soil (T06)	20220714.TM-M.A6B	
BH - 22 - 02 SS13	1774344r	Sulphide/S (R98)	20220719.R98B	

APPENDIX F

NOTICE TO READER

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This Report has been prepared by TULLOCH Engineering Inc. (‘TULLOCH’) for the sole and exclusive use of the Municipality of Huron Shores (the ‘Client’) for foundation design for the replacement of the Potomac River Bridge in, Huron Shores, Ontario (the ‘Site’). The Report shall not be used for any other purpose, or provided to, relied upon or used by any third party without the express written consent of TULLOCH.

A limited number of boreholes were advanced at the Site; and as such, the information collected and presented herein applies to the borehole locations only. The subsurface conditions between boreholes can change and accordingly any use of the data contained in this Report should take into consideration the nature of the materials and potential variation between boreholes.

This Report contains opinions, conclusions and recommendations made by TULLOCH using professional judgment and reasonable care for the purpose of foundation design for the Development. Use of or reliance on this report by the Client is subject to the following conditions:

a) the report being read in the context of and subject to the terms of the Engineering Services Agreement for the Work, including any methodologies, procedures, techniques, assumptions and other relevant terms or conditions specified or agreed therein.
b) the report being read in its entirety. TULLOCH is not responsible for the use of portions of the report without reference to the entire report.
c) the conditions of the site may change over time or may have already changed due to natural forces or human intervention, and TULLOCH takes no responsibility for the impact that such changes may have on the accuracy or validity of the observations, conclusions and recommendations set out in this report.
d) the classification of soils and rocks in this report is based on commonly accepted methods. However, the classification of geologic materials and the boundaries between subsurface layers involves judgement. Boundaries between different soils layers may also be transitional rather than abrupt. TULLOCH does not warrant or guarantee the exactness of these descriptions and boundaries.
e) the subsurface conditions must be verified by a qualified geotechnical engineer during construction to ensure that the borehole data presented herein is representative of the actual site conditions so that the design recommendations contained herein remain valid; and
f) the report is based on information made available to TULLOCH by the Client or by certain third parties; and unless stated otherwise in the Agreement, TULLOCH has not verified the accuracy, completeness or validity of such information, makes no representation regarding its accuracy and hereby disclaims any liability in connection therewith.

This report has been prepared with the degree of care, skill and diligence normally provided by engineers in the performance of comparable services for projects of similar nature. The scope of this report includes foundation engineering design only and it specifically excludes investigation, detection, prevention and assessment of the presence of subsurface contaminants. No conclusions or inferences should be drawn regarding contamination at the site including but not limited to molds, fungi, spores, bacteria, viruses, soil gases such as Radon, PCBs, petroleum hydrocarbons, inorganic and volatile organic compounds, polycyclic aromatic hydrocarbons and or any by products thereof.