

Geotechnical Investigation

Greenview Industrial Gateway

Highway 40, Municipal District of Greenview, AB

Municipal District of Greenview

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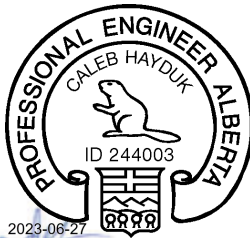
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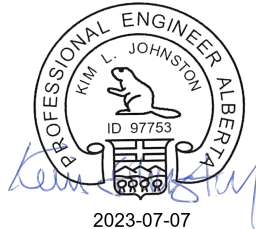
Municipal District of Greenview 02208360.000 (5946-001-00)

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Executive Summary

Englobe Corp. (Englobe) was mandated by the Municipal District of Greenview, through its subsidiary MPE Engineering Ltd., to conduct a geotechnical investigation for the proposed river intake and storage structures for the Greenview Industrial Gateway project located south of Grande Prairie, along Highway 40 in the Municipal District of Greenview, Alberta. There are two locations of interest, termed the River Site and the Pond Site.

The general stratigraphy differed at the two sites. At the River Site, sandstone was encountered either at the surface or underlying a thin sandy soil. The sandstone comprised the majority of the investigation depth however there were units of siltstone, mudstone, coal and bentonite identified between 7.4 and 9.4 metres below ground surface (mbgs). Two boreholes, BH23-01 and BH23-02, were advanced to a planned depth of 15 mbgs, while one borehole, BH23-03, was advanced upslope at the top of a hill to ascertain the depth of the soil cover further from the river and was only advanced to 4.0 mbgs until its contact with the sandstone surface.

At the Pond Site, the stratigraphy was comprised of thin topsoil that had been removed at the location of BH23-04 and was underlain by clay to the extent of the boreholes. Both pond boreholes, BH23-04 and BH23-05, were advanced to the planned depth of 15.7 mbgs.

One standpipe piezometer was installed at the River Site, in BH23-02 with a screened interval between 7.6 to 10.7 mbgs with a sand pack and bentonite seal. The 15-day water level in BH23-02 was found to be 5.97 mbgs. Two standpipes were installed at the Pond Site, in BH23-04 and BH23-05, with 10 ft screened intervals extending up from the base of 15.7 mbgs. The 14-day water levels for were 3.4 and 15.3 mbgs, respectively. The groundwater table is expected to fluctuate seasonally, with variation in the order of 1 m or more. Care must be exercised to ensure the groundwater table is considered during construction. Sloughing was not encountered in any of the boreholes.

A preliminary assessment of the feasibility of two wet well excavation methodologies has been undertaken. An open, sloped excavation will likely be feasible with excavation slopes viable somewhere between 1.5:1 and 2:1. The other option deemed feasible, is a vertical cut supported by shotcrete and anchors.

Several methodologies for cofferdam construction are considered, however due to the likely presence of shallow bedrock in the river, sheet pile or cut-off pile methods will be expensive. Therefore, an earth and rock fill cofferdam are considered the most practical.

Investigation at the Pond Site has found a stiff clay throughout the full depth investigated. This material is considered likely suitable for construction of dykes, but more testing is required to better characterize its properties. While the two boreholes advanced did find consistent stratigraphy, it should not be taken as indicative of consistent stratigraphy throughout the Pond Site and additional investigation should be conducted to assess the variability across the ponds and search for geologic units that could present problems for the design of the dykes.

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1 Introduction

Englobe Corp. (Englobe) was mandated by the Municipal District of Greenview, through its subsidiary MPE Engineering Ltd. (MPE), to conduct a geotechnical investigation for the proposed river intake and storage structures for the Greenview Industrial Gateway (GIG) project located south of Grande Prairie, along Highway 40 in the Municipal District of Greenview, Alberta. There are two locations of interest, termed the River Site and the Pond Site.

The objective of the work was to investigate ground conditions at the proposed water intake, assess the excavation stability, investigate cofferdam construction methodology, and conduct a preliminary investigation of the proposed pond locations.

1.1 Scope of Work

The scope of work for this evaluation included three components:

- A field program comprising a total of five boreholes;
- Testing of recovered soil samples in Englobe's laboratory; and,
- Engineering analysis/reporting.

2 Site Description and Methodology

2.1 Proposed Development

A water intake from the Smoky River and an associated wet well is planned for development at the River Site. The wet well will be excavated to an elevation of 566 m, which is 14 m below the current ground surface. At the Pond Site, a series of raw water retaining ponds are planned. The size and location of these ponds has not been finalized. A water transmission line is planned to connect the water intake and the ponds. The route and depth of this transmission line has not been finalized.

2.2 Site Description

The River Site is located east of Highway 40 approximately 45 km south of Grande Prairie, Alberta. The proposed water intake is located along the Smoky River approximately 1.0 km south of the CanFor 200 road and is currently occupied by an abandoned county road, a bedrock outcrop, and undeveloped forest. The Pond Site is located within the designated boundaries of the GIG, east of Highway 40 approximately 50 km south of Grande Prairie, Alberta and spans an area approximately 3 km east to west by 1 km north to south. It is predominantly undeveloped with a few pipeline rights-of-way, access

roads, operating and abandoned pumping stations, forested with recently cleared forest areas. Figures 1 through 4, included in Appendix A, show the location of the Sites and the borehole locations, respectively.

2.3 Field Investigation

The field program was carried out from January 14 to 16, 2023, using a drill rig contracted from Vals Drilling Ltd. The drill rig was equipped with solid stem continuous-flight augers as well as a 50 mm diamond tipped rock coring stem. Soil samples were retrieved at intervals of approximately 0.6 m and all recovered rock core was retained. The soil was classified and logged by Englobe's field representative, Mr. Caleb Hayduk. Standard Penetration Testing was generally performed at regular intervals of 1.5 m where applicable. Water levels were measured in the boreholes 14 days following completion of the drilling.

Subsequent to obtaining public service clearances, five boreholes (BH23-01 to BH23-05) were drilled at the Sites, as shown on Figures 2, 3 and 4. All boreholes were advanced between January 14 and 16, 2023, using a track-mounted rig equipped with continuous flight solid stem augers and 50 mm diameter rock coring equipment supplied and operated by Vals Drilling Ltd., under the continuous supervision of an Englobe representative.

Two boreholes (BH23-01 and BH23-02) at the River Site were planned to a depth of 15 metres below ground surface (mbgs) and were completed as full depth diamond core holes. A 50 mm standpipe piezometer was installed in BH23-02 with the screened interval from 7.6 to 10.7 mbgs and sand was placed to 0.3 m above the filtered length of the standpipe followed by cuttings. The piezometer was sealed with bentonite chips. One borehole, BH23-03, was completed at the River Site as a solid stem auger hole to assess the depth of soil above bedrock at the top of the river valley. Borehole BH23-03 was completed to 3.9 mbgs.

Two boreholes (BH23-04 and BH23-05) at the Pond Site were planned to a depth of 15 mbgs and were completed to the full depth as solid stem soil boreholes. Standpipe piezometers, 25 mm in diameter, were installed to the full depth in both Pond Site boreholes.

Soil samples were recovered at regular intervals of depth using a 50 mm O.D. split barrel sampler driven into the subsoil in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). In addition, disturbed soil samples were collected from the auger cuttings. The recovered soil samples were visually examined in the field and then preserved and transported to Englobe's Material Testing laboratory in Edmonton, Alberta, for further examination and testing. Borehole Logs are included in Appendix B.

All recovered rock core was collected, visually examined in the field then preserved and transported to Englobe's Material Testing laboratory in Edmonton, Alberta for further examination and testing. Rock core logs are included in Appendix B.

2.4 Laboratory Testing

In the laboratory, each soil sample was examined as to its visual and textural characteristics. Moisture content determinations (ASTM D2216) were carried out on all recovered samples. Additional testing was carried out on select recovered samples as detailed below:

- Three Hydrometer Analysis (ASTM D422-07);
- Four Atterberg Limits (ASTM D4318); and,
- Seven Unconfined Compressive Strength (UCS) (ASTM D2166/D2166M).

The geotechnical laboratory results are provided in Appendix C.

The number and type of tests conducted are summarized in Table 1.

Table 1: Summary of Laboratory Tests Completed

Name of Test	Number of Tests Completed	Sample Type	Laboratory	ASTMs Completed
Moisture Content	60	Soil	Englobe	ASTM D2216
Hydrometer	3	Soil	Englobe	ASTM D422-07
Atterberg Limits	4	Soil	Englobe	ASTM D4318
UCS	7	Rock	Englobe	ASTM D2166/D2166M

Summaries of testing results are briefly outlined in Tables 2, 3 and 4.

Table 2: Summary of Grain Size Distributions

Borehole No.	Depth (mbgs)	Grain Size Distribution Analyses (%)			
		Gravel	Sand	Fines	
				Silt	Clay
BH23-01	13.39	0.0	87.4	12.6	
BH23-02	4.80	0.0	90.6	9.4	
BH23-04	2.7	0.0	21.4	35.6	43.0
BH23-04	11.9	0.0	16.5	40.1	43.4
BH23-04	14.3	0.2	23.9	35.0	41.0

Table 3: Summary of Atterberg Limits Results

Borehole No.	Depth (mbgs)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unified Soil Classification
BH23-02	8.8	146	42	103	CH
BH23-04	4.3	47.0	14.3	33	CI
BH23-05	6.7	39.1	13.4	26	CI
BH23-05	8.2	42.9	15.9	27	CI

Table 4: Summary of UCS Results

Borehole No.	Depth (mbgs)	Peak UCS (kPa)
BH23-01	5.68	5560
BH23-01	8.65	2096
BH23-01	12.50	1633
BH23-02	5.30	Damaged
BH23-02	9.82	287
BH23-02	11.67	Damaged
BH23-02	14.61	3943

2.5 Borehole Summary

The borehole coordinates were surveyed using a handheld GPS device by Englobe. Table 5 summarizes the borehole coordinates and borehole depths.

Table 5: Summary of Borehole Coordinates, Elevations and Depths

Borehole No.	Coordinates ¹			Borehole Depth (mbgs)	Instrumentation
	Northing	Easting	Elevation		
BH23-01	6076278	398513.6	578.8	15.2	-
BH23-02	6076280	398488.2	580.2	15.2	Standpipe Piezometer
BH23-03	6076257	398450.6	583.0	3.9	-
BH23-04	6073095	393294.1	725.45	15.2	Standpipe Piezometer
BH23-05	6073374	394148.5	708.37	15.2	Standpipe Piezometer

¹ NAD83 (CSRS)/ UTM 12N

3 Soil Conditions

Detailed descriptions of the soil conditions in each borehole are presented in the Borehole Log Sheets provided in Appendix B. Classification and identification of soils have been based on commonly accepted methods employed in the practice of geotechnical engineering. The stratigraphic boundaries shown on the Borehole Log Sheets represent transitions between soil types rather than distinct lithological boundaries. The subsurface conditions often vary both with depth and laterally between individual borehole locations.

The borehole locations are indicated on the Site Plan (Figure 2), which is included in Appendix A. The general soil conditions are outlined briefly below.

3.1 The River Site

3.1.1 Surficial Geology

Englobe reviewed surface geology mapping published by the Alberta Geological Survey (AGS). According to the Surficial Geology Map of Alberta (Alberta Geological Survey, 2013a), the Site's surficial geology is classified as glaciolacustrine deposits. AGS defines these deposits as follows:

Fluvial Deposits: *Poorly to well sorted, stratified to massive sand, gravel, silt, clay and organic sediments occurring in channel and overbank deposits, in placed includes a significant component of colluvial deposits as these two units are inseparable at 1 million map.*

3.1.2 Bedrock Geology

Englobe reviewed the bedrock geology map (Alberta Geological Survey, 2013b), and the Site's bedrock geology was classified as belonging to the Kaskapau formation. The AGS defines the Kaskapau formation as follows:

Wapiti Formation: *Siltstone and mudstone with subordinate sandstone (thin to thick bedded) coal, and bentonite, fluvial and lacustrine.*

3.1.3 Field Investigation

Based upon the borehole investigation for this project, the soil conditions at the wet well location are generally comprised of a very thin layer of surficial soils underlain by sandstone, with layers of mudstone, bentonite, coal, and siltstone below 7 mbgs. Boreholes were not completed in the river to assess the subsurface conditions of the river intake. At this time, due to the narrowing of the river and the associated water flow rate, it is assumed that the riverbed is comprised of shallow alluvial or colluvial sediments overlying sandstone bedrock.

Assessments of Rock Quality Designation (RQD), Total Core Recovery (TCR), Solid Core Recovery (SCR) and Fracture Index were completed on recovered core samples and are presented on the borehole logs included in Appendix B. Unconfined compression tests (UCS) were performed on samples with enough intact length for the test to be conducted. Two samples were unable to be prepped for testing without damage. As such, the UCS tests should be considered to represent the upper bound of potential rock strength and not representative of the entire unit.

Joint orientations in the bedrock ranged from 0 degrees to 73 degrees from horizontal, however 77 of the 82 identified natural joints dipped less than 30 degrees from the horizontal suggesting the bedding is predominantly flat.

3.1.3.1 Soil - Fill

Road fill soils were encountered in BH23-02 and BH23-03. The road fill varied in thickness from 1.3 to 2.0 m and was encountered on the surface. Fill material was generally well graded coarse to medium gravel and sand at the surface with sandy clay with some gravel and some silt below.

3.1.3.2 Soil - Native

Natural soil was only encountered in BH23-03 underlying the road fill and extended to a depth of approximately 4.0 m. The natural soil was generally clay, with some silt, some sand and trace gravel.

3.1.3.3 Sandstone

Sandstone was encountered in BH23-01 and BH23-02 and is the inferred termination depth of BH23-03. Sandstone was the primary material encountered and for conciseness will be considered to extend the full length of the borehole. In general, the sandstone was highly weathered within the first 3.0 to 4.0 mbgs and slightly weathered to fresh below. Additionally, BH23-01, located nearer to the riverbank, exhibited a greater degree of weathering at the surface than did BH23-02.

The sandstone was medium to coarse grained, extremely weak to weak, grey, or orange-brown when oxidized. Two particle size distributions performed on sandstone samples indicated 87.4% to 90.6% sand, with the remaining proportion comprised of fines.

3.1.3.4 Siltstone

Siltstone was encountered in BH23-02 between 8.42 and 9.40 mbgs. The siltstone was fresh to slightly weathered, fine grained, extremely weak to weak and greyish brown.

3.1.3.5 Mudstone

Mudstone was encountered in BH23-01 and BH23-02 in two intervals that were between 0.2 and 0.3 m in thickness. In BH23-01, the mudstone was encountered at 7.42 and 7.72 mbgs, while in BH23-02, the mudstone was encountered at 7.40 and 8.15 mbgs.

3.1.3.6 Residual Sand

Residual sands were encountered in several intervals in BH23-01 and BH23-02 where weathering had fully degraded the rock structure. In general, these residual sands were medium to coarse, uniformly graded and between 0.1 and 0.3 m in thickness.

3.1.3.7 Coal

Coal was encountered in BH23-02 and is presumed to have occurred in BH23-01 although it was not recovered. Coal was encountered as a unit approximately 0.70 m thick at an elevation of 572.5 m between the two identified mudstone layers. Fine coal seams were also noted within the mudstone and siltstone at a scale of approximately 5 mm.

3.1.3.8 Bentonite

Bentonite was encountered in BH23-02 and is presumed to have occurred in BH23-01 although it was not recovered. An Atterberg test was conducted and found a liquid limit of 146% and a plastic limit of 42%. The bentonite was encountered at 8.83 mbgs and was approximately 0.025 m thick. In general the bentonite was dry, hard and white.

3.1.4 Groundwater Conditions

Sloughing was not noted in any of the boreholes. Water was present upon completion of drilling for BH23-02. Groundwater levels were measured after drilling and 15 days after standpipe installation.

Table 6 summarizes the groundwater measurements. These results are also recorded on the Borehole Logs provided in Appendix B.

Table 6: Groundwater Observations at the River Site

Borehole No.	Instrumentation	Groundwater Depth at Completion (mbgs)	Groundwater Depth after 15 Days (mbgs)	Sloughing or Seepage Depth (mbgs)
BH23-02	Standpipe Piezometer	-	5.97	-

Variation in the order of 1 m or more is possible, depending on the season. As such, groundwater conditions during construction may differ.

Groundwater readings were taken on January 30, 2023, approximately 15 days following standpipe piezometer installation. It is recommended that the standpipe piezometers be left in place to allow further groundwater level measurements as the groundwater table is expected to fluctuate seasonally depending on rainfall events and run-off conditions. The water levels in the monitoring wells must be recorded prior to the start of construction.

3.2 The Pond Site

3.2.1 Surficial Geology

Englobe reviewed surface geology mapping published by the Alberta Geological Survey (AGS). According to the Surficial Geology Map of Alberta (Alberta Geological Survey, 2013a), the Site's surficial geology is classified as glaciolacustrine deposits. AGS defines these deposits as follows:

Glaciolacustrine Deposits: A) *Offshore sediment, rhythmically laminated to massive fine sand, silt and clay, locally debris released from floating ice.* B) *Littoral and nearshore sediments massive to stratified, well-sorted silty sand, pebbly sand and minor gravel.*

3.2.2 Bedrock Geology

Englobe reviewed the bedrock geology map (Alberta Geological survey, 2013b), and the Site's bedrock geology was classified as belonging to the Kaskapau formation. The AGS defines the Kaskapau formation as follows:

Wapiti Formation: Siltstone and mudstone with subordinate sandstone (thin to thick bedded) coal, and bentonite, fluvial and lacustrine.

3.2.3 Field Investigation

3.2.3.1 Topsoil

Topsoil was encountered in BH23-05 at the surface. Borehole BH23-04 did not have topsoil however the forest had recently been cleared and the topsoil was likely removed at that time. In BH23-05, the topsoil was approximately 0.3 m thick and was a sandy organic clay which was amorphous.

3.2.3.2 Clay

Clay was encountered in both boreholes; at the surface in BH23-04 and underlying the topsoil in BH23-05. The clay extended to 15.7 mbgs in both boreholes. Generally, the clay contained 'and silt', was sandy, had trace gravel near the surface, and was moist, medium plastic, was stiff to very stiff with brown and grey mottling.

The SPT 'N' values ranged from 10 to 30 blows per 0.30 m of penetration. The moisture content varied from 14.1% to 29.7%. The moisture content in BH23-05 was generally higher than the moisture contents in BH23-04. Atterberg testing indicated that the clay till was medium plastic. The constituents ranged from 0.0% to 0.2% gravel, 16.5% to 23.9% sand, 35.0% to 40.1% silt, and 41.0% to 43.4% clay. Pockets with higher sand content were observed in BH23-04 below 11.0 mbgs.

3.2.4 Groundwater Conditions

Sloughing was not noted in either borehole. Standpipe piezometers were installed in both boreholes. Water was present upon completion of drilling for BH23-04. Groundwater levels were measured after drilling and 14 days after standpipe installation. Sand was placed to 0.3 m above the filtered length of the standpipe followed by cuttings. The piezometers were sealed with bentonite chips.

Table 7 summarizes the groundwater measurements after completion of drilling and 14 days after standpipe installation. These results are also recorded on the Borehole Logs provided in Appendix B.

Table 7: Groundwater Observations at the Pond Site

Borehole No.	Instrumentation	Groundwater Depth at Completion (mbgs)	Groundwater Depth after 14 Days (mbgs)	Sloughing or Seepage Depth (mbgs)
BH23-04	Standpipe Piezometer	4.8	3.44	Seepage below 11.0
BH23-05	Standpipe Piezometer	Dry	15.34	-

Variation in the order of 1 m or more is possible, depending on the season. As such, groundwater conditions during construction may differ.

Groundwater readings were taken on January 30, 2023, approximately 14 days following standpipe piezometer installation. It is recommended that the standpipe piezometers be left in place to allow further groundwater level measurements as the groundwater table is expected to fluctuate seasonally

depending on rainfall events and run-off conditions. The water levels in the monitoring wells must be recorded prior to the start of construction.

4 Geotechnical Analysis and Recommendations at the River Site

The geotechnical recommendations provided in this section are from the interpretation of the geotechnical data obtained during the investigation and recommendations for geotechnical aspects of the design of foundations and general site development. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. It must be noted that information regarding construction must be interpreted to accommodate any change for designs, construction methods, costs, and scheduling.

4.1 Wet Well

The sandstone, mudstone and siltstone are expected to have sufficient bearing capacity for the wet well foundations; however, design of the wet well will be dictated mainly by temporary and permanent lateral support requirements. An adaptive design approach is provided where the stability model can be updated as excavation progresses allowing excessive movement, if observed to be mitigated and/or conservative design relaxed appropriately.

Three suitable wet well support options are presented:

- Sloped excavation;
- Full-depth secant pile wall; and,
- Anchors with shotcrete facing.

A sloped excavation is considered the simplest option of the feasible options; however, it is also likely the most laborious even if relatively steep, 2H:1V, slopes are facilitated by ground strength characteristics. A sloped excavation feasibility is in part governed by the strength of the weakest layer, in this case bentonite, which could pose a risk.

Tangent pile wall shoring is considered marginally feasible and higher risk as the excavation will extend below the groundwater table potentially requiring extensive dewatering. Maintaining pile alignment over 19 m to avoid gaps between piles resulting in soil/rock erosion is less constructable than the secant pile option.

Anchors with shotcrete facing are considered feasible reinforcement from existing grade to the base of the excavation as no surficial soils were identified at the wet well location.

The full depth secant pile wall option has not been discussed with foundation contractors and may be difficult or prohibitively expensive to implement given the depth of the excavation. The depth of the excavation indicates that lateral pile support will be required in the form of anchors and walers in addition to secant piles.

One advantage of a secant pile wall is that degradation of the rock when exposed to air and seepage is limited compared to a shotcrete and anchor wall. Another advantage is the reduced number of rows of

anchors. A disadvantage of the secant pile option is the cost of mobilizing a piling rig to site versus the smaller equipment for anchor and shotcrete installation. Another disadvantage is that the secant pile wall is constructed blind while the rock can be observed and any significant joints, fractures, or changes in the quality (strength) of the rock may be logged with the shotcrete and anchor option.

It is considered advantageous to consider a circular wet well. The lateral earth pressures on a circular shaft forces the concrete piles into compression, taking advantage of the compressive strength of concrete and making it a more efficient design which does not generally require anchors. This configuration would require walers which would likely be reinforced cast-in-place concrete.

Preliminary discussion of the sloped excavation, full depth secant pile and anchor with shotcrete facing options are provided in the following sections. It should be noted that any excavation, whether sloped, or shotcrete and anchor supported, will likely have its stability governed by the weakest unit, in this case bentonite. Any excavation that exposes the bentonite unit will require consideration of additional oxidation, hydration, or weathering in order to preserve its strength.

4.1.1 Sloped Excavation

The intended goals of the slope stability analyses were to assess the in-situ stability of the natural slope, wet well excavation, and river intake construction.

The stability of slopes is typically represented as a Factor of Safety (FS). FS is the ratio of soil or rock shear strength to shear stress along a plane within the slope. An FS of 1.0 is defined as reaching limit equilibrium and in a state of imminent failure or deformation. An FS greater than 1.0 indicates that the slope is currently stable. For this project, an FS of 1.3 is recommended for global stability of the wet well structure, i.e., slope failure surfaces with an FS of 1.3 do not intercept the wet well and its foundations provided that the excavation will only be open temporarily and is not considered a permanent fixture.

The strength properties used were based on Generalized Hoek-Brown Criterion which is an empirical failure criterion which establishes the strength of a jointed rock mass in terms of major and minor principal stresses.

Exposure to the elements could cause significant weathering and wetting to previously unweathered rock units, especially the mudstone, coal, siltstone, and bentonite. Once exposed, these units should be sealed with a thin layer of shotcrete or other sealant to prevent water and air from degrading them.

4.1.1.1 Stability Cross Section

The cross section used for the stability analyses was selected based on the surveyed elevation of the boreholes, LiDAR provided by MPE and a 14 m deep excavation at the proposed location of the wet well.

4.1.1.2 Rock Parameters

Rock strength parameters were based on field RQD estimates, laboratory testing, and laboratory results obtained on samples collected during the investigation. Mohr-Coulomb strength values were derived from Hoek-Brown rock strength values which were calculated from field measurements of GSI, and laboratory UCS tests. The Mohr-Coulomb values were then refined by back analysis of the natural riverbank slope and an assumed factor of safety of 1.0. Values for bentonite, residual soil, and coal were based on field observations, prior experience, and relevant published literature. Strength parameters selected for the analyses contained within this report are shown in Table 8.

Table 8: Soil and Rock Strength Parameters

Property	Bulk Unit Weight (kN/m ³)	Effective Cohesion (kPa)	Effective Internal Friction Angle (°)
Sandstone	23.5	21	35
Siltstone	21	9	19
Mudstone	21	9	19
Coal	19	9	19
Bentonite	16	0	22
Residual Soils	21	0	25

4.1.1.3 Stability Model

The geotechnical modeling computer program, Slope/W, version 2021.4, by GeoStudio, was used to complete the limit equilibrium analyses for this project. The geometry, material properties, and groundwater conditions for the stability model were based on information discussed in the previous sections. The strength parameters (friction angle, cohesion, Hoek-Brown Criterion) were varied to review the impact on the FS. The piezometric surface utilized in the analyses was based upon the groundwater data collected from the standpipe piezometers and assumed dewatering activities.

4.1.1.4 Stability Analysis Results

For the analysis of the slope from upslope of the wet well to the river, several scenarios were analyzed, and two key scenarios are described in further detail below: slope with natural groundwater table, and slope with dewatering activities.

The assumed groundwater level for the stability analyses was based on the ground water reading of 5.9 mbgs taken on January 30, 2023, and no dewatering. The water level measured in the BH23-02 standpipe is indicative of water pressures near the mudstone, coal, and siltstone layers as that is where the piezometer was screened. There is likely groundwater present in fractures nearer the surface as evidenced by oxidation and weathering noted in the borehole logs. The water present in the upper layers could have an impact on slope stability and constructability but cannot be assessed without additional investigation. An additional scenario with dewatering activities occurring was also analyzed.

In all cases, the critical failure surface tended to favor the bentonite and siltstone layers and was relatively shallow on the slope face, rather than a more globalized failure surface passing through the toe or the base of the excavation. Several slope angles, benching configurations, intermediate slope angles, and other geometries were assessed for stability and constructability.

For the scenario of natural groundwater conditions, a 2H:1V slope was found to have an FS of 1.3 which is considered acceptable for temporary excavations. The critical failure surface was relatively shallow, daylighting with failure surface entry point on the face of the slope, with the toe of the surface daylighting through the siltstone layer. The critical slip surface is shown in white on Figure 5.

If dewatering activities are performed to push the groundwater surface away from the slope, a steeper slope is viable. An overall 1.5H:1V slope was assessed as having an FS of 1.3. In this scenario, the critical failure surface is shallower than the one with natural groundwater, with the critical surface entering in the lower half of the upper slope and exiting in the upper half the of the lower slope. The location of the toe of the critical surface still favors the siltstone and bentonite layer. This is shown on Figure 6.

It should be noted that the above analyses are sensitive to the groundwater levels and as such if the no-dewatering option is selected, the groundwater level should be closely monitored during the course of construction. Groundwater levels could fluctuate significantly throughout the year and the water level found during the investigation likely is not at the annual peak. Also, degradation of the rock surface in

the form of swelling, slaking, and ravelling while the excavation is open could block seepage and increase groundwater pressures in the slope.

4.1.1.5 Groundwater Investigation, Seepage into the Excavation, and Dewatering

Water bearing fractures were identified throughout the depth of both boreholes. The hydraulic conductivity of these fractures and therefore feasibility of pumping and dewatering methods for the wet well excavation were not assessed. A 50 mm diameter monitoring well was installed in BH23-02 which could be used to perform slug tests and other tests to assess the risk of water flow through the fractures near the base of the excavation and further refine understanding of the water pressures. Englobe recommends that this further testing be carried out prior to finalization of the wet well design. It could also be beneficial to investigate the water pressures and flow rates in the near surface fractures as these could also have an impact on water ingress to the excavation and stability of the slope.

Dewatering activities would likely be performed with several rows of horizontal drains, drilled at a slight upwards angle, and extending at least 10 m behind the slope. Spacing of the dewatering wells would be dependant on a number of factors, including ground permeability and water table recharge rates. A detailed design of the dewatering efforts should be completed by a qualified contractor. Figure 6 shows the assumed level of the dewatered water table. The cost of dewatering activities, which likely would only be needed uphill or on the side of the excavation furthest from the river, could possibly be offset by the reduction in material that needs to be excavated. Englobe has not assessed the feasibility or costs associated with dewatering at this time.

It is anticipated that dewatering activities will only be required for the duration of the excavation. Once backfill of the excavation begins, dewatering will no longer be required.

4.1.2 Wet Well with Full Depth Secant Piles

If an open cut excavation is not desirable, bored cast-in-place secant piles with approximately 0.2 m interlock will likely be either 914 mm or 1220 mm in diameter and may be implemented. Infill piles may be constructed with a reduced concrete strength of 5 MPa. The reinforced piles will likely be cast with an embedded H or W section. The piles should extend a minimum of 3 m below the finished grade of the bottom of the wet well for stability reasons.

The bending moments on the secant piles will be large so lateral pile analysis will be essential to the detailed design. Soil input parameters vary across software platforms and may be provided by Englobe upon request but the software LPILE is commonly used.

Instrumentation and monitoring during construction is recommended to allow for re-evaluation and updating of the design as excavation progresses. This is common practice for deep excavations. The proposed instrumentation for the full depth secant pile wall option would include:

- Three inclinometers installed within secant piles on the upslope side, with one extending a minimum of 10 m below the finished bottom of excavation elevation and two extending 5 m below the bottom of the excavation;
- Two inclinometers installed in secant piles on the downslope side of the wet well 1.5 m from face extending to a minimum of 5 m below the lowest excavation elevation; and,
- One inclinometer installed in each of the other two sides of the well.

A program for the frequency of reading of the inclinometers, additional monitoring devices if required, and actions to be taken by the engineer and contractor in the event of movement will be developed during detailed design. These recommendations may be modified during construction.

If movement is observed, depending on the rate or change in rate of movement daily readings may be recommended. Inclinometer movement will be used to revise the design as required to maintain stability

of the excavation. If movement is observed the space of anchors may be reduced or their length increased.

A program for the frequency of reading of the inclinometers and actions to be taken by the engineer and contractor in the event of movement will be developed during detailed design. These recommendations may be modified during construction.

4.1.3 Wet Well with Shotcrete and Anchors

Shotcrete and anchors excavation support would allow for top-down construction in tandem with excavation. Ultimate grout to rock bond strengths for anchors were developed from UCS testing and from rock quality designations for the rock logs and published values (Post-Tensioning Institute, 2014). Parameters chosen for the supports, i.e., rock bolts, are shown in Table 9 and Table 10. One unoptimized option for anchor and shotcrete facing is summarized in Table 10 below. The large number of rows of anchors is due to the relatively low grout-rock bond strength. This design includes 10 rows of anchors, all 12 m long. This configuration has a factor of safety of 1.5.

Table 9: Wet Well Anchor Parameters

Material	Ultimate Rock / Soil Grout Bond Strength (kPa)	Elevation at BH23-02
Sandstone	90	Surface to 572.77, Below 570.77
Mudstone	30	572.77 to 571.75
Siltstone	50	571.75 to 570.77

Table 10: Preliminary Design Anchor Properties, full depth 75 mm shotcrete and anchor option, assumes free anchor length of 3.0 m for all rows

Rock Unit	Ultimate Grout-Anchor Bond Stress (kPa)	Drill Hole Diameter (m)	Bond Length (m)	Horizontal Anchor Spacing (m)	Vertical Anchor Spacing (m)
Sandstone	90	0.075	8	1.3	1.0
Mudstone	30	0.075	8	1.3	1.0
Siltstone	50	0.075	8	1.3	1.0

If the planned structure is to be permanent, double corrosion protection will be needed. This could result in larger diameter holes being required for installation. Residual soils were identified during the field investigation. It is possible that these layers will collapse when the anchor drill bit is removed before it can be replaced with the anchor. To mitigate this, casing may be necessary to keep the boreholes open until grouting occurs.

General recommendations of anchors include:

- Staggering of bond lengths for horizontal or vertical spacings of less than 1.2 m;
- Anchors oriented 15 degrees below horizontal;
- Centralizers spaced 3.0 m on center or less;
- Strip drains installed prior to shotcrete placement at 1.2 m to 2.4 m spacing; and,
- Grouting of anchors as soon as they are drilled.

If test anchor capacities are less than anticipated based on pullout testing, design adjustments may be required. Design adjustments may include more anchors or larger diameter anchor boreholes, etc.

The proposed instrumentation for the anchor and shotcrete option would include:

- Two inclinometers located approximately 1.5 m from the excavation face on the upslope side, with one extending to a minimum of 10 m below the finished bottom of the excavation elevation and one extending 5 m below the bottom of the excavation;
- One inclinometer on the downslope side of the wet well 1.5 m from the excavation face extending to a minimum of 5 m below the lowest excavation elevation; and,

As for the secant pile option, a program for the frequency of reading of the inclinometers and actions to be taken by the engineer and contractor in the event of movement will be developed during the detailed design phase. These recommendations may be modified during construction.

If movement is observed, depending on the rate or change in rate, daily readings may be recommended. Inclinometer movement will be used to revise the design as required to maintain stability of the excavation. If movement is observed the spacing of anchors may be reduced or their length increased.

4.1.4 Foundation Movement - Serviceability Limit State

The wet well building may either be founded on top of the wet well walls, or on an isolated shallow foundation constructed outside the wet well. An independent foundation would serve to isolate the building from wet well movements and is recommended. If constructed on an isolated foundation comprised of piles, settlements of 10 mm to 15 mm may be expected.

For foundations constructed on top of (and integrated into) the wet well, settlement estimates become more complex. Settlement of the wet well secant pile wall from construction of the Pump House and Wet Well Building is expected to be less than 10 mm due to the depth of the well and the presence of anchors. Predicting the movement of the walls of the deep excavation is inexact. There may be inward movement of the secant piles over time. The foundation of the building should be designed to accommodate this movement. Such movement is difficult to estimate as it depends on the existing stress state of the rock, groundwater, rate of excavation, the delay between excavation and shotcrete placement for the anchor option, based on other deep excavations it could be up to 50 mm.

If the anchored shotcrete wall option is selected, a thickened shotcrete or cast-in-place footing with adequate reinforcing could be constructed integral with the top of the wet well wall or an independent footing constructed outside the wet well.

4.2 River Intake

The foundation conditions at the river intake were not physically investigated as part of the scope of work. However, it is assumed that the soils at the river intake comprise granular soils with poorly graded sand and gravel predominating, possibly underlain by bedrock. At this time, it is assumed that the river intake will be founded on shallow foundations overlain by ballast of either native fill or riprap.

If investigation in the river is not possible due to cost, it is recommended that investigation be undertaken during construction. It is expected that may be accomplished with an excavator or backhoe. Suitable options should be developed for several likely soil types in advance and the optimal design selected at the time of construction.

4.2.1 General

The cofferdam crest height should be set such that cofferdam flooding is unlikely to occur. The US Federal Emergency Agency (FEMA) provides recommended flood return periods for cofferdams. If the projected length of the construction period is one year, the cofferdam would need to accommodate a flood with a return period of 5 years. If the projected length of the construction period is two years, a

flood with a return period of 10 years would be used (FEMA, 2005). The estimated 5-year and 10-year flood levels at the site are 576.63m and 577.18 m, respectively (Northwest Hydraulic Consultants Ltd., 2023).

Ice jams only occurred in approximately 35%-40% of break ups between 2000 and 2021 at the Watino and Wapiti gauge locations. The 5- and 10-year return peak ice jam elevations are 576.4 and 577.1 respectively, which are in line with the peak water flood elevations. A one-year serviceable cofferdam could be suitable with a crest elevation of 576.63 m plus a suitable freeboard (Northwest Hydraulic Consultants Ltd., 2023).

4.2.2 Dam Safety Considerations

The cofferdam is regulated under the Water Act Alberta *Dam and Canal Safety Directive* published by the Alberta Government in 2018, Section 5.10 Design of temporary structures. This section outlines the regulatory requirements for information to be submitted to the regulatory authorities. This includes:

- a) Design drawing and specifications for cofferdams, weirs conduits, and other temporary structures;
- b) The consequence classification of the structures;
- c) Justification for the selected design criteria for the structures;
- d) Hydraulic analyses of the temporary structures that are used to control or divert water during construction, with supporting hydraulic data;
- e) Stability analyses of the temporary structures under design usual and unusual loading conditions;
- f) Seepage analyses of the temporary structures and details on seepage control during construction, including use of pumping;
- g) An operation, maintenance and surveillance manual for the temporary structure;
- h) An emergency management plan for the temporary structure; and,
- i) The removal of the temporary structures that are used to control or divert water during construction.

When the Consequence Classification is determined, it is then used to inform the level of detail and scrutiny to which the dam is designed. Based on the preliminary information available, the cofferdam will likely have a significant to high consequence classification based on the number of people working within the cofferdam.

In addition to regular dam safety considerations, more hazards need to be assessed and managed due to the cofferdam being constructed in a river. The flow of water applies a force to the dam that will affect the stability balance as well as cause erosion along the exposed surface. If the cofferdam is to be operational during the winter freeze up or the spring thaw, moving ice will have implications for the stability of the dam as well as the required slope armoring. A cofferdam should not be considered impermeable to water. While the majority of water will be prevented from entering the cofferdam, some seepage is to be expected and as such pumps should be on hand to manage it. It is possible that seepage cannot be managed with pumping and sumps, in which case sheet pile cut off walls, watering wells, or internal berms may be required to manage seepage.

The following discussion is to provide feasible options of cofferdam types, but design of the preferred methodology is beyond the scope of this report.

4.2.3 Cofferdam Types

Construction of waterway cofferdams can follow many methodologies such as an earth embankment with:

- Rock fill placed on river side, with impervious clay on the downstream side;
- Earth fill with sheet piles driven along the centerline to reduce seepage;
- Double walled sheet piles backfilled with granular; or,
- Cellular sheet pile walls backfilled with granular fill.

There is a risk of the riverbed being comprised of bedrock with no or minimal overlying riverbed material. If this is the case, cofferdams with sheet piles will likely not be feasible due to the difficulty in seating them adequately in the rock to prevent seepage or stability issues. Because of this, Englobe believes the earth fill cofferdam with rock and clay is the more feasible option.

If there is a significant thickness of cobbles and gravel, dredging may be necessary to avoid significant leakage under the cofferdam. If not, the clay fill will generally tend to infill any flow paths at the interface between the cofferdam and riverbed material. If dredging is not feasible and excessive leakage is observed, sheet piles may need to be driven after the cofferdam is in place.

The earth fill option would include pushing fill into the river from shore to create a berm extending above the water level and extending this along the cofferdam alignment. It may be necessary to start with rock fill along the upstream side of the cofferdam to limit the loss of soil by erosion, then place clay against and upstream of the rock fill to limit seepage through the cofferdam. A filter layer is generally not included in below-water placement; the clay will fill surficial voids in the rock fill. This method is commonly used for cofferdams in lakes and rivers and is likely the preferred method. With this methodology, cofferdam side slopes roughly between 6:1 to 7:1 could be expected; however, some studies have shown cofferdams can have fine grained fill slopes as flat as 20:1. There are many compounding factors that determine the stable slope angle of clay placed into free water, some of which can only be assessed after construction is completed. Because of this complexity, a detailed stability analysis that considers actual selected fill material, the flow rate of the river along with many other factors may need to be performed while the cofferdam is being constructed.

One option to reduce the overall footprint of the cofferdam would be to construct the cofferdam of rockfill with slopes approximately 2:1, and place clay on the upstream face. This would allow the flow of the river to move the clay into the rock fill's void spaces thereby reducing the overall permeability of the cofferdam.

The crest width of the cofferdam would largely be controlled by the working width of the equipment used to construct it. A working area of 6 m has been assumed to allow for access by construction equipment.

The earth fill and sheet pile cofferdam could be considered a backup plan for an earth cofferdam with excessive leakage. The feasibility of this option will be limited if there are cobbles or boulders or shallow bedrock in the riverbed material, which appears likely.

A double walled sheet pile cofferdam could be considered. The risk of sheet pile refusal makes a geotechnical investigation in the river necessary, increasing the cost of this option. The two sheet pile walls would be connected by steel ties / bracing and the space between walls backfilled with granular material. This type of wall relies on friction from the granular material to limit overturning therefore the quality of granular material is essential for a stable wall.

As for the double walled cofferdam option, a barge based geotechnical investigation or similar would be required. The cells would need to be filled with granular material. The advantage of cellular cofferdams over a double walled cofferdam with ties is that less bracing is required for cellular walls. Cellular cofferdams are generally more expensive than other options.

If seepage or boils are observed inside the cofferdam perimeter, they may be covered with sand and gravel to provide stabilization and filtration.

It is understood there is not a nearby borrow for construction materials. Imported granular material for the double wall or cellular fill, low to medium plastic, non-dispersive clay or clay till would be required for the low hydraulic conductivity ("impervious") portion of the earth cofferdam.

Based on the preliminary investigation, the material identified at the Pond Site should be suitable for the cofferdam impermeable core. Granular materials would likely still need to be imported for armoring and stability.

4.2.4 Environmental

As for environmental concerns, regardless of the above construction methodology, prior to construction of the embankment or sheet pile system, the area underneath the structure would have to be dredged, leading to siltation downstream. With the earth embankment option, there will likely be siltation during placement of the granular material, as well as erosion and subsequent siltation if the river scours the placement of granular material, as well as erosion and subsequent siltation if the river scours the granular fill (rip rap on the upstream and river side of the cofferdam should be used to mitigate this).

When sheet piles are employed, there is the concern of excessive noise and vibrations during pile driving that can disturb aquatic life.

With any cofferdam, pumping of water within the cofferdam will be required after the structure is built to expose the river bottom. The stability of the cofferdam should be checked against draw down during dewatering as it may cause sloughing of the downstream side of the cofferdam.

During dewatering, a biologist would likely be involved in fish capture and release, there will also be a continuous dewatering requirement during construction from leakage. Depending on project environment requirements, this water may have to be stored in a settling pond prior to release if it has excessive fines.

Breaching of the cofferdam should be controlled by initially pumping or siphoning water into the dry side of the cofferdam to limit damage to the intake and riverbank erosion. When the major works are complete, and the cofferdam is being decommissioned, there will be some additional siltation during removal of the embankment fill. As well, for sheet piles they are generally cut-off at the base, and the embedded steel is left in the ground.

4.3 Construction Considerations

The design parameters provided in the preceding sections assume that a qualified contractor with relevant design and construction experience completes the work and good construction practices are followed. As ground conditions may vary from the five boreholes completed, it is recommended that a geotechnical engineer be present during construction to confirm that the conditions encountered are similar to those assumed.

Recommendations and design parameters presented herein are provided on the assumption that Englobe will be retained to provide engineering design review and construction supervision. Englobe takes no liability for work performed where Englobe is not retained to provide adequate construction supervision services. Construction inspection should include:

- Inspection of the excavated bearing surface for the wet well and river intake;
- Full time monitoring of pile construction of the river intake;
- Full time monitoring of slopes and any reinforcement options selected for construction and interior excavation during excavation and any slope stability impacting activities;

- Full time instrumentation monitoring during construction including inclinometers and vibrating wire piezometers;
- Full time monitoring and compaction testing for earthworks;
- Strength testing for concrete and grout; and,
- Monitoring and inspection of anchor testing.

Once excavation is complete, monitoring of the slope and reading of the associated instruments could be reduced from daily to weekly. If any activities are performed that could impact slope stability, such as excavation of the base or slope, a change in dewatering effort, placement of weight near the crest of the excavation, slope stability monitoring frequency would be increased. Similarly for the cofferdam, once construction is complete geotechnical monitoring could be reduced from daily to weekly, although the contractor would still be responsible for their own full-time monitoring of the coffer dam during and after construction.

4.3.1 Excavation of Rock

UCS testing of rock core samples has indicated a wide range of strengths, which are summarized in Table 4. It is worth noting that the samples selected for these tests were generally perceived to be on the weaker end of the spectrum in order to conservatively estimate strength values for slope stability. Schmidt hammer tests were conducted on some of the relatively hard samples and found rebound numbers correlating to UCS results between 10 MPa and 30 MPa

There are several ways to estimate the “rippability” or “excavatability” based on a number of rock mass parameters such as UCS, point load strength, GSI, and shear wave velocity. The method proposed by Pettifer and Fookes, is based on UCS and block size, suggests excavation at the River Site could range from hard digging to hard ripping and blasting, Tsiambaos and Saroglou suggest an estimate based on GSI for both rock masses with a high point load strength (>3 MPa) and low point load strength (<3 MPa). Based on the borehole investigation, the rock mass at the River Site is estimated to have a GSI in the range of 30-60 which correlates to ripping or hammering (Tsiambaos and Saroglou 2009).

Caterpillar Inc. (Caterpillar) has some guidance on ground rippability with various versions of their products. Their analysis is based on seismic shear wave velocity, which was not assessed during this investigation. The right equipment can facilitate ripping in rock with seismic velocities as high as 3500 m/s (Caterpillar, 2000) which would correlate to a hard rock (NBCC, 2015). While the specifications for Caterpillar products are referenced here, it is assumed that other similar equipment would be able to facilitate a similar level of ripping effort.

Kirsten proposes a more comprehensive classification methodology in *A Classification System for Excavation in Natural Materials*. Factoring in rock mass, joint spacing, roughness, alteration, dip direction and RQD excavatability is broken down into 8 categories ranging from hand digging to blasting. Based on the results of the field investigation and laboratory testing, the rock strongest rock encountered at the wet well location would be categorized as either 6 or 7, which correlates to very to extremely hard ripping, which is viable with a D9H or D10 sized ripper.

It is expected that these very hard layers do not comprise the majority of the rock mass and it may be feasible to rip through softer materials and hammer through the harder, to avoid blasting. The distinction will require continuous assessment in the field as conditions change.

Selection of equipment will determine the possibility and potential ease with which excavation is carried out. It will be up to the contractor to look at the information provided and determine if the equipment they propose to use will be sufficient to allow for excavation or if blasting will be required.

5 Preliminary findings at the Pond Site

Very stiff clay tills were encountered in both locations, for the full depth of the borehole in the central and east side of the Pond Site. No information can be inferred about the west side of the Pond Site as the distance of the nearest borehole is significant.

The clay tills encountered in both locations are likely to be suitable both for impermeable clay liners and dam fill material due to the high SPT blow counts observed during drilling and the high clay content. Groundwater is potentially an issue in the central region, due to the high-water table encountered, and while the 14-day water levels in BH23-05, did not indicate significant presence of ground water, it should not be considered indicative of the rest of the pond as ground conditions will vary.

A preliminary estimate for a reworked clay liner consisting of the native clay tills encountered at the Pond Site could have a permeability as low as 1×10^{-7} m/s. However, without representative laboratory testing of both undisturbed and remoulded samples from across the pond footprints there is significant uncertainty in this value.

Stiff clays can often support slopes with a steepness on the order of 3H:1V to 5H:1V. However, many factors such as construction methodology, groundwater regime, dam configuration, level of water impounded among others can have a drastic impact on this value. In addition, only a small area was investigated, and material properties can vary significantly over even small distances. The values provided above as simply provided to illustrate a general idea of possible dam size and should not be relied upon for any detailed design.

If, upon further investigation, an insufficient volume of suitable clay is found to reliably construct clay cores or clay liners for the dams, a synthetic liner could be considered. It should be noted that synthetic liners come with their own challenges with regard to slope stability, the buoyancy of the liner, seepage through the dam, and cost among others.

If these ponds are to be greater than 2.5 m in height or contain more than 30 000 m³ of water, they will be dams regulated by the Water Act Alberta *Dam and Canal Safety Directive* published by the Alberta Government in 2018. This directive outlines numerous requirements for the safe construction, maintenance and operation of a dam. Dams must be authorized by the regulator prior to construction and the designs must be approved. Some key components of the process as outlined in the directive include a requirement for a Dam Safety Management Plan, a Consequence Classification that ranges from low to extreme, and an Emergency Management Plan. In addition to the *Dam and Canal Safety Directive*, other documents such as the Canadian Dam Association's *Dam Safety Guidelines 2007 (2013 Edition)* further outline best practices for dam safety management, operation, emergency preparedness, dam safety reviews, and the analysis and assessment of dam design. Dam design is often a multidisciplinary operation that requires a level of thorough detail commensurate with the risk level (Consequence Classification) of that dam.

6 Further Considerations

6.1 The River Site

Access will need to be created to the River Site. For this report, access was obtained with all terrain vehicles, passing along an oil lease road off of the Canfor 2000, across pasture, pipeline rights-of-way, a small creek, and along a decommissioned municipal road. Creating sufficient access for the equipment required at the River Site will likely require construction of an all-weather gravel roadway. Depending on anticipated life span and size of equipment it needs to support, this road may require a geotechnical investigation and design. Englobe can submit a proposal for this scope upon request.

6.2 Pond Site

Further investigation is required to adequately characterize the geological variability in the area. The proposed ponds have a combined perimeter of approximately 8.0 km. Boreholes should be completed to sufficient depth every 100 m along the perimeter of the proposed pond locations, with an additional borehole completed within the pond footprint at a spacing of one per hectare. This will allow for adequate characterization of the soil stratigraphy and should allow for the identification of any smaller scale anomalies that could cause geotechnical issues. Problematic findings could include but are not limited to soft clays, heavily over consolidated clays, silts, low fines content sands or gravels or near surface bedrock of any kind.

Additionally, groundwater monitoring wells should be installed to assess the natural groundwater regime, measure groundwater flow magnitude and direction. These wells will also be used to conduct in situ permeability testing to facilitate modeling of water seepage through and under the dykes.

The drilling program should be accompanied by an extensive laboratory testing program that includes Atterberg, hydrometer, grain size distribution, direct shear, hydraulic conductivity, shear vane and unconfined compressive strength testing. Groundwater monitoring wells should be installed to characterize the existing hydrogeological regime.

Englobe can provide a formal scope of work and cost estimate for the further investigation required for this project.

6.3 Water Transmission Line

The shortest path between the River Site and the Pond Site is approximately 6 km in a straight line. The specific route of the pipeline should be chosen with careful consideration of topography, other utilities in the path, geohazards such as slope instability along creeks, marshes and other environmentally sensitive ecosystems, among others. Once a suitable path is determined, a geotechnical investigation should be completed along the pathway to inform on risks associated with open cut or horizontal directional drilling installation of the water transmission line. Assessment of the water transmission line route was not part of this scope, however, Englobe can submit a proposal for this scope upon request.

7 Closing Remarks

The soil and rock boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling and must not be interpreted as exact planes of geological change. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design. In addition, the subsoil and groundwater conditions have been determined at the borehole locations only.

Any depths expressed as metres below ground surface (mbgs) take into account the original ground elevation at the time of the investigation. If fill material is added to the Site, or if soil is removed, care should be taken during the subsequent design phases to ensure that parameters from the correct elevations are used.

The recommended bearing capacity has been calculated by Englobe from the information obtained from the borehole data.

It is further noted that, depending on the time of year the fieldwork was completed, water levels are expected to vary, perhaps significantly, from those observed at the time of this investigation.

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- Tsiambaos, G. and Saroglou, H. 2009. Excavatability assessment of rock masses using the Geological Strength Index (GSI).

9 Statement of Limitations

This report (hereinafter, the “Report”) was prepared by Englobe Corp. (hereinafter the “Company”) and is provided for the sole and exclusive use and benefit of the Municipal District of Greenview (the “Client”). Ownership in and copyright for the contents of the Report belong to the Company.

No other person is authorized to rely on, use, copy, duplicate, reproduce or disseminate this Report, in whole or in part and for any reason whatsoever, without the express prior written consent of the Company. Any person using this Report, other than the person(s) to whom it is directly addressed, does so entirely at its own risk. The Company assumes no responsibility or liability in connection with decisions made or actions taken based on the Report, or the observations and/or comments contained within the Report. Others with interest in the site and/or subject matter of this Report should undertake their own investigations and studies to determine how or if they or their plans could be affected.

This Report should be considered in its entirety; selecting specific portions of the Report may result in the misinterpretation of the content.

The work performed by the Company was carried out in accordance with the terms and conditions specified in the Professional Services Agreement between the Company and the Client, in accordance with currently accepted engineering standards and practices and in a manner consistent with the level of skill, care and competence ordinarily exercised by members of the same profession currently practicing under similar conditions and like circumstances in the same jurisdiction in which the services were provided. Standards, guidelines and practices may change over time; those which were applied to produce this Report may be obsolete or unacceptable at a later date.

The findings, recommendations, suggestions, or opinions expressed in this Report reflect the Company's best professional judgment based on observations and/or information reasonably available at the time the work was performed, as appropriate for the scope, work schedule and budgetary constraints established by the Client. No other warranty or representation, expressed or implied, is included in this Report including, but not limited to, that the Report deals with all issues potentially applicable to the site and/or that the Report deals with any and all of the important features of the site, except as expressly provided in the scope of work.

This Report has been prepared for the specific site, development, building, design or building assessment objectives and/or purposes that were described to the Company by the Client. The applicability and reliability of the content of this Report, subject to the limitations provided herein, are only valid to the extent that there has been no material alteration or variation thereto, and the Company expressly disclaims any obligation to update the Report. However, the Company reserves the right to amend or supplement this Report based on additional information, documentation or evidence made available to it.

The Company makes no representation concerning the legal significance of its findings, nor as to the present or future value of the property, or its fitness for a particular purpose and hereby disclaims any responsibility or liability for consequential financial effects on transactions or property values, or requirements for follow-up actions and costs.

Since the passage of time, natural occurrences, and direct or indirect human intervention may affect the views, conclusions and recommendations (if any) provided in this Report, it is intended for immediate use.

The assessment should not be considered a comprehensive audit that covers and eliminates all present, past and future risks. The information presented in this Report is based on data collected during the completion of the monitoring conducted. The overall site conditions were extrapolated based on information collected at specific sampling locations. Professional judgement was exercised in gathering and analyzing data; however, no monitoring method can completely eliminate the possibility of obtaining

partially imprecise or incomplete information; it can only reduce the possibility to an acceptable level. Consequently, the actual site conditions between the sampling points may vary. In addition, analysis has been carried out only for the chemical and physical parameters identified, and it should not be inferred that other chemical species or physical conditions are not present.

It is recommended practice that the Company be retained during subsequent phases of the project, to confirm that the conditions throughout the site do not deviate materially from those encountered throughout the sampling program.

Any description of the site and its physical setting documented in this Report is presented for informational purposes only, to provide the reader a better understanding of the site and scope of work. Any topographic benchmarks and elevations are primarily to establish relative elevation differences between sampling locations and should not be used for other purposes such as grading, excavation, planning, development, or similar purposes.

Any results from laboratory or other subcontractors reported herein have been carried out by others, and the Company cannot warrant their accuracy.

This Report is based on the assumption that the design features relevant to our work will be in accordance with applicable codes, standards and guidelines of practice and constructed substantially in accordance with the Report. If there are any changes to the site development features, or there is any additional information that was not otherwise available at the time the work was performed, the Company should be retained to review the implications thereof to the contents of this Report. The design recommendations expressed in this Report are applicable only to the project described therein.

This Statement of Limitations forms an integral part of the Report.

Appendix A

Figures

Figure 1: Site Location

Figure 2: Aerial Photograph showing Borehole Locations and Access Routes

Figure 3: Detail 'A' Borehole Locations ("The River Site")

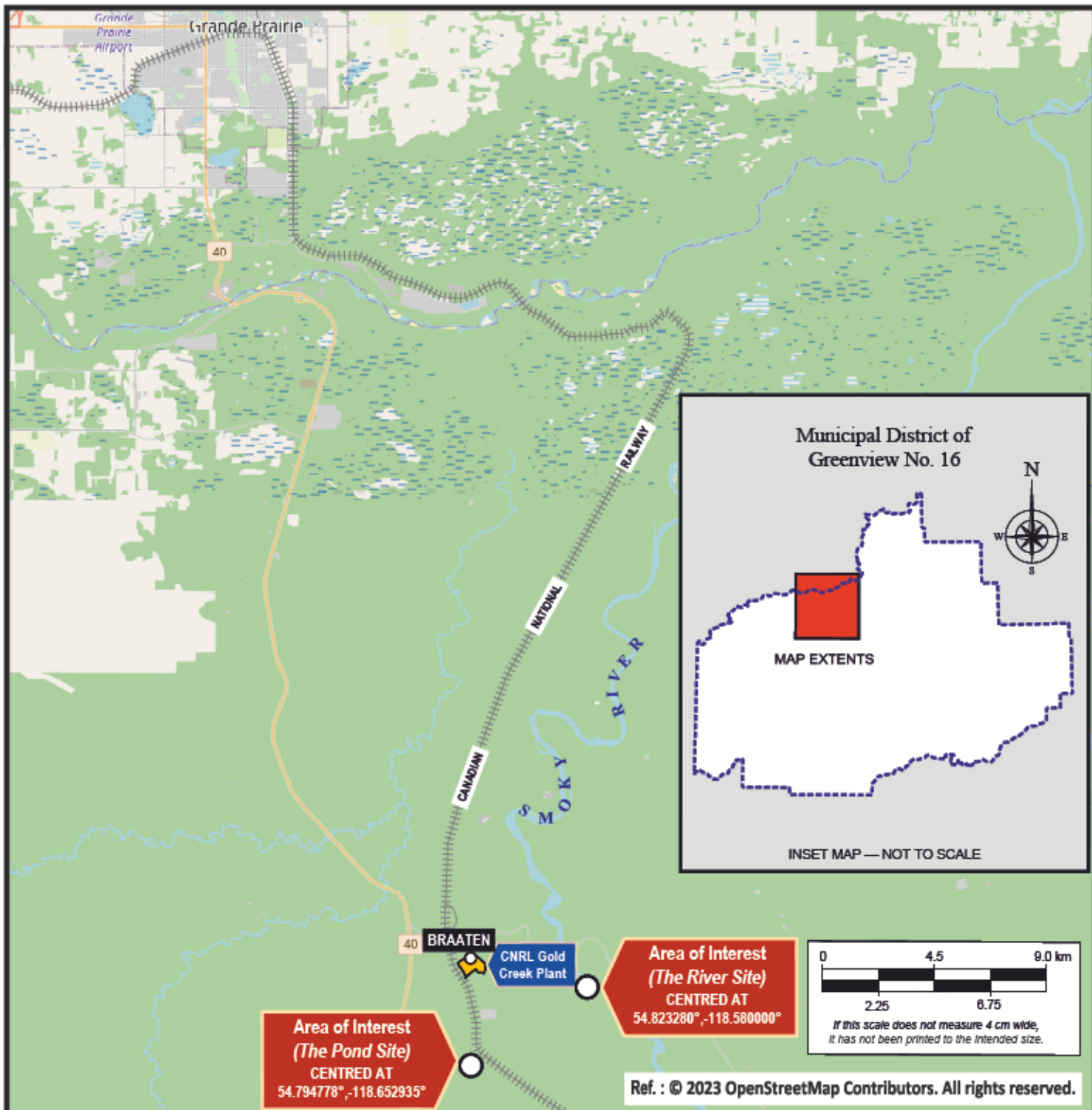
Figure 4: Detail 'B' Borehole Locations ("The Pond Site")

Figure 5: Excavation Stability Section - Natural Groundwater Conditions

Figure 6: Excavation Stability Section - Dewatered Groundwater Conditions



ENGLOBE



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**Municipal District of
GREENVIEW**

**GEOTECHNICAL INVESTIGATION
GREENVIEW INDUSTRIAL GATEWAY
3-14-67-5 W6M and 4-29-67-4 W6M near Braaten, Alberta**

SITE LOCATION



18114, 114 Avenue NW
Edmonton, Alberta
T5M 2Z5
780-481-1418

Discipline : Geotechnical		Prepared by : C. HAYDUK		Checked by : R. HUCULAK	
Scale : 1:225,000		Drawn by : D. WILSON		Approved by : P. GINGRAS	
Date : June 2023		Figure no. : FIGURE 1			
Layout : LS		Paper size : LETTER (8.5" x 11")		Registration no. : —	

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FILE: G:\141\CAD\Projects\02208360.000.02 5\02208360.000.02 5-GEO-PL.dwg DATE: 2023-06-27 TIME: 12:09:51 LAST SAVED BY: WilsDr



LEGEND

- Access Route
- Canadian National Railway
- Borehole
- Borehole / Piezometer

Location	Northing	Easting	Elevation (masl)
BH23-01	6076277.60	398513.58	578.85
BH23-02	6076279.83	398488.17	580.17
BH23-03	6076257.31	398450.58	583.81
BH23-04	6073095.26	393294.10	725.46
BH23-05	6073374.18	394148.46	708.37

Datum is NAD83(CSRs) / UTM zone 11N

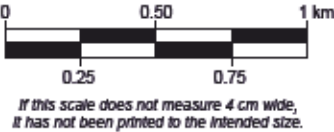
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No.	Version	Date	By	Check	Appr.	
Discipline : Geotechnical		Prepared by : C. HAYDUK		Checked by : R. HUCULAK		
Scale : 1:25,000		Drawn by : D. WILSON		Approved by : P. GINGRAS		
Date : June 2023		Figure no. :		FIGURE 2		
Layout : PL		Paper size : ANSI full bleed B (11.00 x 17.00 inches)		Registration no. : ---		
Resp.	Project	Otp	Project/ Disc	Phase/ Type	Electronic ref/ Drawing no.	Rev.
140	02208360.000.025	---	---	---	02208360.000.025-GEO-PL.dwg	---



Municipal District of
GREENVIEW



Englobe Corp.
16114, 114 Avenue NW
Edmonton, Alberta
T5M 2Z5
780-481-1416



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GEOTECHNICAL INVESTIGATION
GREENVIEW INDUSTRIAL GATEWAY
3-14-67-5 W6M and 4-29-67-4 W6M near Braaten, Alberta

Aerial Photograph showing
Borehole Locations and Access Routes

FILE: G:\141\CAD\Projects\02208360.000.02 5\02208360.000.02 5-GEO-PL.dwg DATE: 2023-06-27 TIME: 12:11:14 LAST SAVED BY: WilsDr



Ref. : Aerial Photograph ©2023 ESRI Community Maps Contributors. All rights reserved.



LEGEND

- Access Route
- Borehole

Location	Northing	Easting	Elevation (masl)
BH23-01	6076277.60	398513.58	578.85
BH23-02	6076279.83	398488.17	580.17
BH23-03	6076257.31	398450.58	583.81

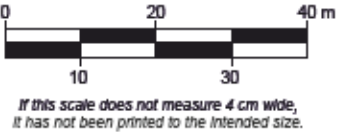
Datum is NAD83(CSRS) / UTM zone 11N

A	FINAL	2023-06-27	D.W.	R.H.	P.G.	
No.	Version	Date	By	Check	Appr.	
Discipline : Geotechnical		Prepared by : C. HAYDUK		Checked by : R. HUCULAK		
Scale : 1:1,000		Drawn by : D. WILSON		Approved by : P. GINGRAS		
Date : June 2023		Figure no. : FIGURE 3				
Layout : DETAIL 'A'		Paper size : ANSI full bleed B (11.00 x 17.00 inches)		Registration no. : ---		
Resp.	Project	Otp	Project/ Disc	Phase/ Type	Electronic ref/ Drawing no.	Rev.
140	02208360.000.025	---	---	---	02208360.000.025-GEO-PL.dwg	---



Englobe Corp.

16114, 114 Avenue NW
Edmonton, Alberta
T5M 2Z5
780-481-1416

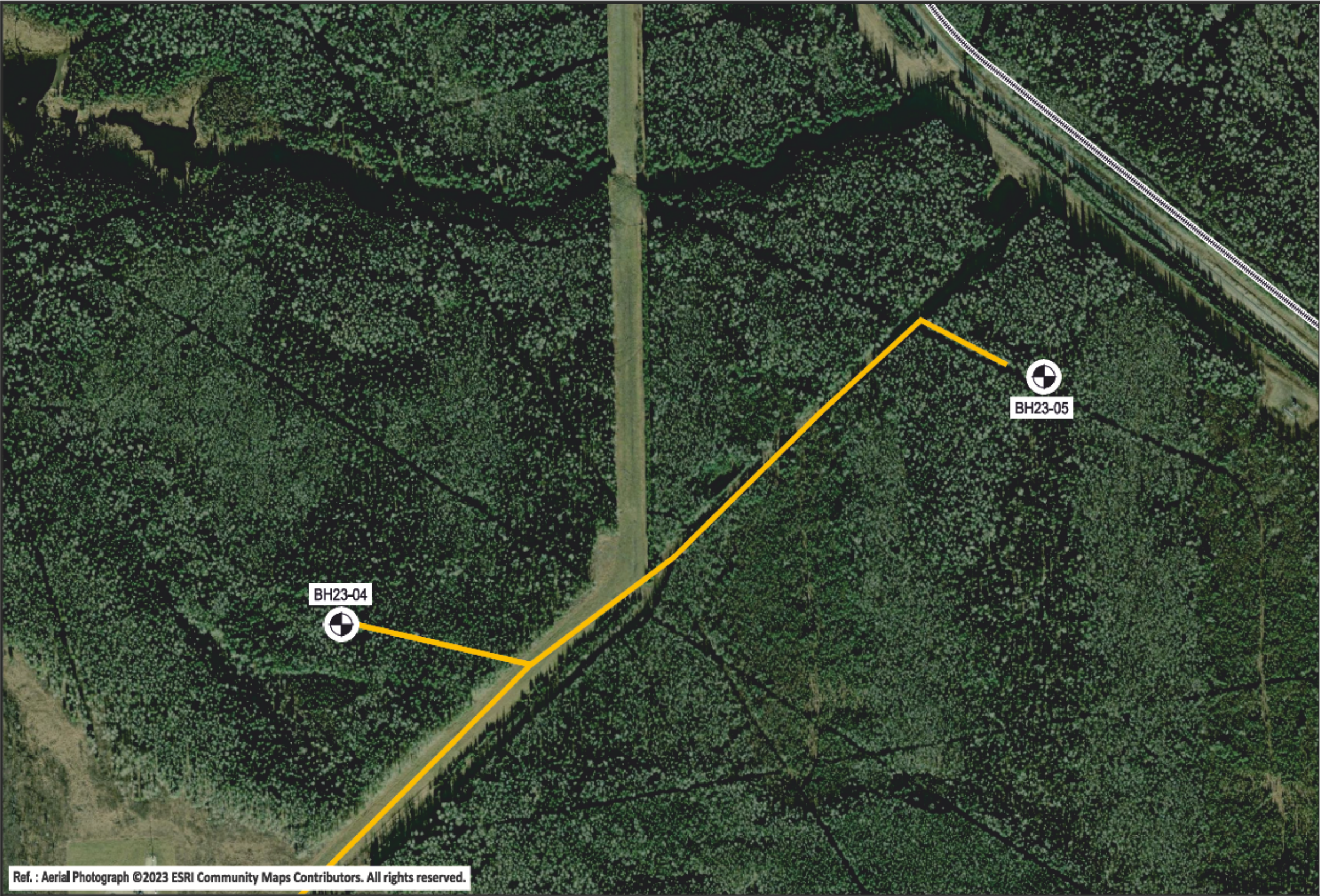


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GEOTECHNICAL INVESTIGATION
GREENVIEW INDUSTRIAL GATEWAY
3-14-67-5 W6M and 4-29-67-4 W6M near Braaten, Alberta

Detail 'A'
Borehole Locations ("The River Site")

FILE: G:\141\CAD\Projects\02208360.000.02\502208360.000.02-5-GEO-PL.dwg DATE: 2023-06-27 TIME: 12:11:35 LAST SAVED BY: WilsDr



Ref. : Aerial Photograph ©2023 ESRI Community Maps Contributors. All rights reserved.



LEGEND

- Access Route
- Canadian National Railway
- Borehole / Piezometer

Location	Northing	Easting	Elevation (masl)
BH23-03	6076257.31	398450.58	583.81
BH23-04	6073095.26	393294.10	725.46
BH23-05	6073374.18	394148.46	708.37

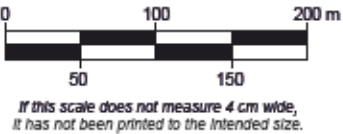
Datum is NAD83(CSRS) / UTM zone 11N

A	FINAL	2023-06-27	D.W.	R.H.	P.G.
No.	Version	Date	By	Check	Appr.
Discipline :	Geotechnical	Prepared by :	C. HAYDUK	Checked by :	R. HUCULAK
Scale :	1:5,000	Drawn by :	D. WILSON	Approved by :	P. GINGRAS
Date :	June 2023	Figure no. :	FIGURE 4		
Layout :	Paper size : ANSI full bleed B (11.00 x 17.00 inches)	Registration no. :	---		
Resp.	Project	Otp	Project/ Disc	Phase/ Type	Electronic ref/ Drawing no.
140	02208360.000.025	---	---	---	02208360.000.025-GEO-PL.dwg



Englobe Corp.

16114, 114 Avenue NW
Edmonton, Alberta
T5M 2Z5
780-481-1416



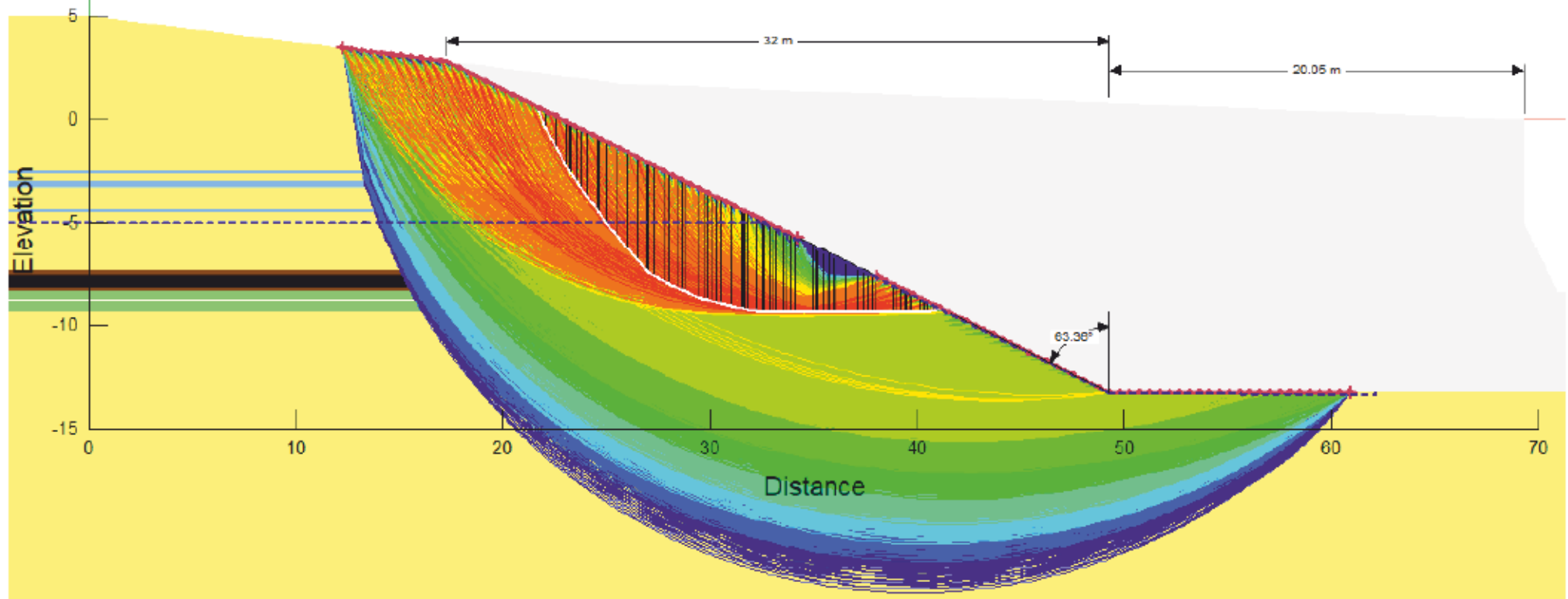
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GEOTECHNICAL INVESTIGATION
GREENVIEW INDUSTRIAL GATEWAY
3-14-67-5 W6M and 4-29-67-4 W6M near Braaten, Alberta

Detail 'B'
Borehole Locations ("The Pond Site")

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bentonite	16	0	22
	Coal	19	9.4	18.6
	Mudstone	21	9.4	18.6
	Residual Soils	18	0	25
	River sands	18	0	25
	Sandstone	23.5	21	35
	Siltstone	21	9.4	18.6

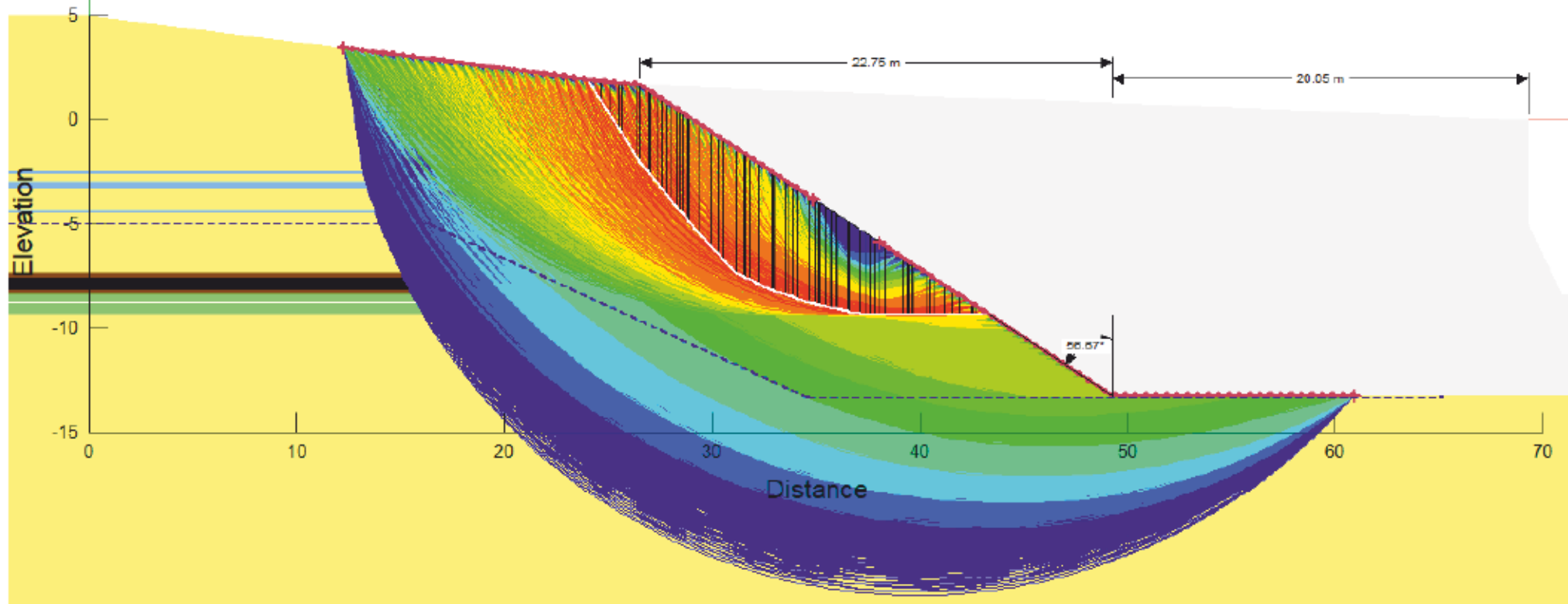
Factor of Safety
 1.360 - 1.460
 1.460 - 1.560
 1.560 - 1.660
 1.660 - 1.760
 1.760 - 1.860
 1.860 - 1.960
 1.960 - 2.060
 2.060 - 2.160
 2.160 - 2.260
 ≥ 2.260



Project:	Greenview Industrial Gateway		
Client:	Municipal District of Greenview		
Analysis Description:	Excavation Stability Section – Natural Groundwater Conditions		
Drawn:	CH	Approver:	KJ
Scale:	1:334	Job:	5946-001-00
		Title:	Figure 5

Color	Name	Unit Weight (kN/m ³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bentonite	16	0	22
	Coal	19	9.4	18.6
	Mudstone	21	9.4	18.6
	Residual Soils	18	0	25
	River sands	18	0	25
	Sandstone	23.5	21	35
	Siltstone	21	9.4	18.6

Factor of Safety
 1.372-1.472
 1.472-1.572
 1.572-1.672
 1.672-1.772
 1.772-1.872
 1.872-1.972
 1.972-2.072
 2.072-2.172
 2.172-2.272
 ≥ 2.272



Project:	Greenview Industrial Gateway		
Client:	Municipal District of Greenview		
Analysis Description:	Excavation Stability Section – Dewatered Groundwater Conditions		
Drawn:	CH	Approver:	KJ
Scale:	1:334	Job:	5946-001-00
		Title:	Figure 6

Appendix B

Rock Core Logs and Borehole Logs



eNGLOBE

DRILLING DATE: 14/01/2023 - 14/01/2023

COORDINATES: N: 8076278 m E: 398514 m

ORIGINATED BY: CH/KO

PROJECT: 5946-001-00

DATUM:

COMPILED BY: CH

ELEVATION: 578.8 m

DRILL RIG:

CHECKED BY: KJ

INCLINATION: 0° AZIMUTH: 0°

CONTRACTOR: Vals Drilling

REPORT DATE: 30/03/2023

DEPTH - feet	DEPTH - meters	DEPTH - m	DESCRIPTION	SYMBOL	RUN NO.	TOTAL CORE %	SOLID CORE %	RQD %	DISCONTINUITY DATA			STRENGTH	WEATHERING INDEX	NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
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RECORD OF DRILLHOLE No BH23-01

DRILLING DATE: 14/01/2023 - 14/01/2023

COORDINATES: N: 8076278 m E: 398514 m

ORIGINATED BY: CH/KO

PROJECT: 5946-001-00

DATUM:

COMPILED BY: CH

ELEVATION: 578.8 m

DRILL RIG:

CHECKED BY: KJ

INCLINATION: 0° AZIMUTH: 0°

CONTRACTOR: Vals Drilling

REPORT DATE: 30/03/2023

DEPTH - feet	DEPTH - meters	DEPTH - m	DESCRIPTION	SYMBOL	RUN NO.	TOTAL CORE %	SOLID CORE %	RQD %	DISCONTINUITY DATA			STRENGTH	WEATHERING INDEX	NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
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			dark brown laminations present, interbedded sand and siltstone Residual Soil - CLAY SANDSTONE fresh, 1 cm scale beds, medium to fine grained, weak, grey						2			5																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													

DRILLING DATE:14/01/2023 - 15/01/2023

COORDINATES: N: 6076280 m E: 398488 m

ORIGINATED BY: CH/KO

PROJECT: 5948-001-00

DATUM:

COMPILED BY: CH

ELEVATION: 580.2 m

DRILL RIG:

CHECKED BY: KJ

INCLINATION: 0° AZIMUTH: 0°

CONTRACTOR: Vals Drilling

REPORT DATE: 11/04/2023

[illegible]

DRILLING DATE:14/01/2023 - 15/01/2023

COORDINATES: N: 6076280 m E: 398488 m

ORIGINATED BY: CH/KO

PROJECT: 5948-001-00

DATUM:

COMPILED BY: CH

ELEVATION: 580.2 m

DRILL RIG:

CHECKED BY: KJ

INCLINATION: 0° AZIMUTH: 0°

CONTRACTOR: Vals Drilling

REPORT DATE: 11/04/2023

[illegible]

Greenview Industrial Gateway

N 6076257 E 398450.6

BOREHOLE NO: **BH23-03**

DATES: 01/15/2023 - 01/15/2023

ZONE: 11 N

PROJECT NO: 5946-001-00

ELEVATION: 593.81 m

SAMPLE STATE ☒ Remoulded☐ Intact☐ Core☐ Lost (no recovery)

SAMPLE TYPE AS: Auger Sample

ST: Shelby Tube

SS: Split Spoon

MA: Manual Sample

DC: Diamond Rock Core

DEPTH (m)	WET UNIT WEIGHT (kN/m ³) ▲					COMPRESSION STRENGTH (kPa)					SAMPLE STATE	SAMPLE TYPE and NUMBER	SPT (N)	USC	GROUNDWATER CONDITIONS	ELEVATION - m DEPTH - m	SOIL SYMBOL	SOIL DESCRIPTION	ELEVATION (m)
	17	18	19	20	21	100	200	300	400										
	WATER CONTENT and LIMITS (%)					STANDARD PENETRATION (N) ■													
	PLASTIC M.C. LIQUID																		
	20	40	60	80		20	40	60	80	100									
1																593.8	• •	GRAVEL and SAND - FILL well graded, medium to coarse grained, dry, compact, brown, trace rootlets	593
2																593.3		CLAY - FILL sandy, some silt, trace to some gravel, moist, firm, medium plastic, brown, frozen	592
3																591.8		CLAY some sand, some silt, trace gravel, moist, firm medium plastic, grey to black	591
4																589.9		At 3.0 m: Rocks in SPT spoon	590
5																4.0		END OF BOREHOLE - 3.95 m	589
6																		No standpipe installed, borehole terminated on bedrock contact	588
7																			587
8																			586
9																			585
10																			584
11																			583
12																			582
13																			581
14																			580
15																			579
16																			578
17																			577
																			576

REMARK: Borehole completed to target depth (bedrock contact)

LOGGED BY: CH/KO

COMPILED BY: CH

REVIEWED BY: KJ

REPORT DATE: 03/30/2023



Greenview Industrial Gateway

N 6073095 E 393294.1

BOREHOLE NO: BH23-04

DATES: 01/16/2023 - 01/16/2023

ZONE: 11 N

PROJECT NO: 5946-001-00

ELEVATION: 725.46 m

SAMPLE STATE  Remoulded Intact Core Lost (no recovery)

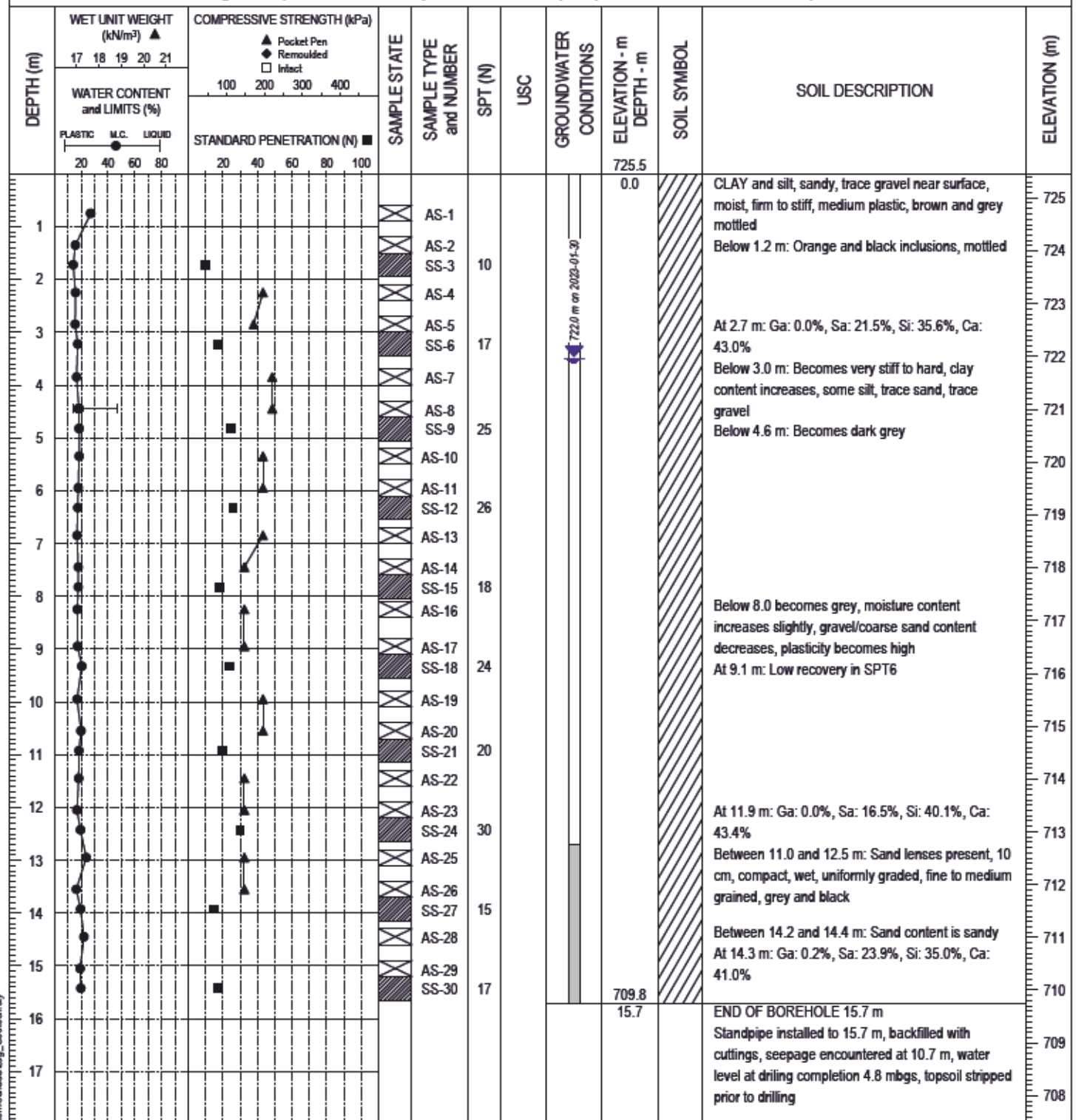
SAMPLE TYPE AS: Auger Sample

ST: Shelby Tube

SS: Split Spoon

MA: Manual Sample

DC: Diamond Rock Core



REMARK: Borehole completed to target depth

LOGGED BY: CH/KO

COMPILED BY: CH

REVIEWED BY: KJ

REPORT DATE: 03/30/2023



Greenview Industrial Gateway

N 6073374 E 394148.5

BOREHOLE NO: **BH23-05**

DATES: 01/16/2023 - 01/16/2023

ZONE: 11 N

PROJECT NO: 5946-001-00

ELEVATION: 708.37 m

SAMPLE STATE  Remoulded Intact Core Lost (no recovery)

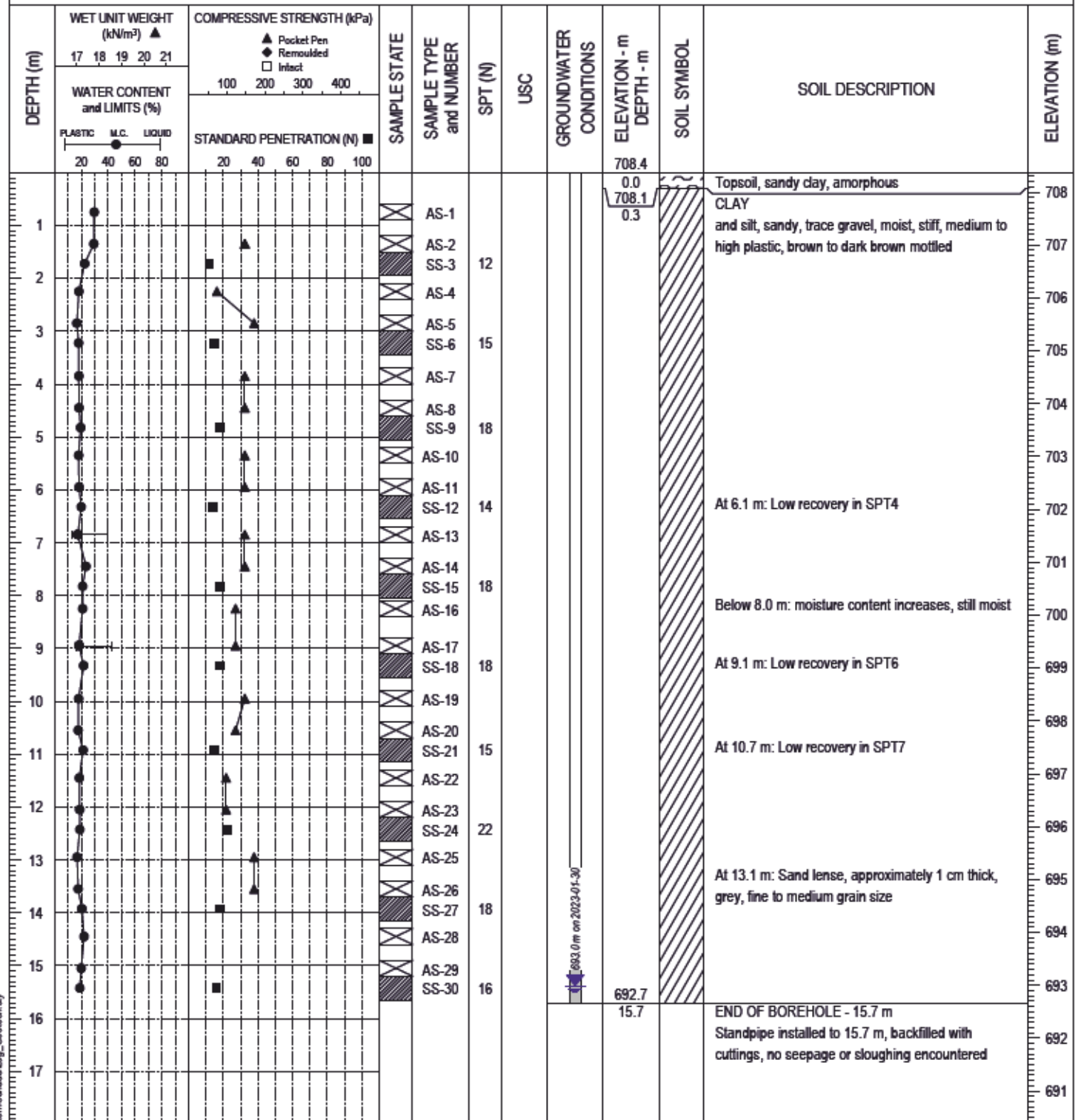
SAMPLE TYPE AS: Auger Sample

ST: Shelby Tube

SS: Split Spoon

MA: Manual Sample

DC: Diamond Rock Core



REMARK: Borehole completed to target depth

LOGGED BY: CH/KO

COMPILED BY: CH

REVIEWED BY: KJ

REPORT DATE: 03/30/2023



Appendix C

Geotechnical Laboratory Results



eNGLOBE



Moisture Content As Per ASTM D2217

Test Hole No.:	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04
Sample No.:	AU1	AU2	SPT1	AU3	AU4	SPT2	AU5	AU6	SPT3	AU7
Depth:	0.6m	1.2m	1.5m	2.1m	2.7m	3.0m	3.7m	4.3m	4.6m	5.2m
Tare No.:	185	1022	308	275	8A	276	311	310	14A	251
Wet Sample + Tare (g):	241.8	305.2	294.3	281.1	409.3	271.3	256.5	412.3	198.3	223.4
Dry Sample + Tare (g):	191.2	264.3	258.5	243.4	354.7	231.6	220.6	354.2	168.1	189.1
Weight of Tare (g):	4.4	4.3	4.5	4.4	4.4	4.3	4.4	4.2	4.5	4.2
Weight of Water (g):	50.6	40.9	35.8	37.7	54.6	39.7	35.9	58.1	30.2	34.3
Weight of Dry Soil (g):	186.8	260.0	254.0	239.0	350.3	227.3	216.2	350.0	163.6	184.9
Moisture Content (%):	27.1%	15.7%	14.1%	15.8%	15.6%	17.5%	16.6%	16.6%	18.5%	18.6%

Test Hole No.:	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04
Sample No.:	AU8	SPT4	AU9	AU10	SPT5	AU11	AU12	SPT6	AU13	AU14
Depth:	5.8m	6.1m	6.7m	7.3m	7.6m	8.2m	8.8m	9.1m	9.8m	10.4m
Tare No.:	170	239	2048	278	6A	24	161	204	287	280
Wet Sample + Tare (g):	214.3	279.3	202.3	278.2	222.1	234.6	244.4	63.1	255.4	233.7
Dry Sample + Tare (g):	182.4	238.3	173.5	236.6	189.1	200.8	208.9	53.1	219.0	195.6
Weight of Tare (g):	4.2	4.2	4.3	4.4	4.6	4.5	4.4	4.3	4.4	4.4
Weight of Water (g):	31.9	41.0	28.8	41.6	33.0	33.8	35.5	10.0	36.4	38.1
Weight of Dry Soil (g):	178.2	234.1	169.2	232.2	184.5	196.3	204.5	48.8	214.6	191.2
Moisture Content (%):	17.9%	17.5%	17.0%	17.9%	17.9%	17.2%	17.4%	20.5%	17.0%	19.9%

Test Hole No.:	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04
Sample No.:	SPT7	AU15	AU16	SPT8	AU17	AU18	SPT9	AU19	AU20	SPT10
Depth:	10.7m	11.5m	11.9m	12.2m	12.8m	13.1m	13.7m	14.3m	14.9m	15.2m
Tare No.:	226	10A	301	304	137	16A	147	7A	1008	201B
Wet Sample + Tare (g):	238.9	245.8	481.7	221.3	424.7	232.0	273.6	462.1	236.9	294.8
Dry Sample + Tare (g):	202.4	208.7	410.5	185.9	343.7	200.0	229.4	387.1	199.3	246.8
Weight of Tare (g):	4.4	4.4	4.4	4.5	4.4	4.5	4.4	4.5	4.4	4.5
Weight of Water (g):	36.5	37.1	71.2	35.4	81.0	32.0	44.2	75.0	37.6	48.0
Weight of Dry Soil (g):	198.0	204.3	406.1	181.4	339.3	195.5	225.0	382.6	194.9	242.3
Moisture Content (%):	18.4%	18.2%	17.5%	19.5%	23.9%	16.4%	19.6%	19.6%	19.3%	19.8%

Test Hole No.:										
Sample No.:										
Depth:										
Tare No.:										
Wet Sample + Tare (g):										
Dry Sample + Tare (g):										
Weight of Tare (g):										
Weight of Water (g):										
Weight of Dry Soil (g):										
Moisture Content (%):										

Comments: BH23-04 SPT6 contained low sample quantity, less than 100g



Reviewed By: Connor Carlson



Moisture Content As Per ASTM D2217

Test Hole No.:	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05
Sample No.:	AU1	AU2	SPT1	AU3	AU4	SPT2	AU5	AU6	SPT3	AU7
Depth:	0.6m	1.2m	1.5m	2.1m	2.7m	3.0m	3.7m	4.3m	4.6m	5.2m
Tare No.:	154	228	282	279	200A	1023	252	169	221	210K
Wet Sample + Tare (g):	230.3	276.9	258.1	281.1	232.8	284.0	267.7	233.4	305.1	257.6
Dry Sample + Tare (g):	178.6	215.1	211.4	238.5	200.1	241.4	227.1	198.0	256.0	218.9
Weight of Tare (g):	4.3	4.2	4.2	4.2	4.3	4.4	4.3	4.3	4.3	4.3
Weight of Water (g):	51.7	61.8	46.7	42.6	32.7	42.6	40.6	35.4	49.1	38.7
Weight of Dry Soil (g):	174.3	210.9	207.2	234.3	195.8	237.0	222.8	193.7	251.7	214.6
Moisture Content (%):	29.7%	29.3%	22.5%	18.2%	16.7%	18.0%	18.2%	18.3%	19.5%	18.0%

Test Hole No.:	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05
Sample No.:	AU8	SPT4	AU9	AU10	SPT5	AU11	AU12	SPT6	AU13	AU14
Depth:	5.8m	6.1m	6.7m	7.3m	7.6m	8.2m	8.8m	9.1m	9.8m	10.4m
Tare No.:	139	20	145	323	19A	12A	289	196	276	1004
Wet Sample + Tare (g):	257.3	112.3	400.2	240.6	227.0	410.0	288.0	39.1	323.9	235.2
Dry Sample + Tare (g):	218.0	94.3	341.6	195.8	188.3	341.3	244.0	32.9	275.1	200.5
Weight of Tare (g):	4.3	4.2	4.2	4.2	4.2	4.2	4.5	4.3	4.3	4.4
Weight of Water (g):	39.3	18.0	58.6	44.8	38.7	68.7	44.0	6.2	48.8	34.7
Weight of Dry Soil (g):	213.7	90.1	337.4	191.6	184.1	337.1	239.5	28.6	270.8	196.1
Moisture Content (%):	18.4%	20.0%	17.4%	23.4%	21.0%	20.4%	18.4%	21.7%	18.0%	17.7%

Test Hole No.:	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05
Sample No.:	SPT7	AU15	AU16	SPT8	AU17	AU18	SPT9	AU19	AU20	SPT10
Depth:	10.7m	11.5m	11.9m	12.2m	12.8m	13.1m	13.7m	14.3m	14.9m	15.2m
Tare No.:	326	172	21	191	189	155	321	1013	182	186
Wet Sample + Tare (g):	104.7	316.9	248.8	286.8	281.3	244.9	242.0	195.7	200.2	267.8
Dry Sample + Tare (g):	87.0	267.8	210.1	241.9	241.3	208.7	201.7	161.1	167.6	225.9
Weight of Tare (g):	4.3	4.2	4.2	4.2	4.3	4.2	4.3	4.3	4.3	4.4
Weight of Water (g):	17.7	49.1	38.7	44.9	40.0	36.2	40.3	34.6	32.6	41.9
Weight of Dry Soil (g):	82.7	263.6	205.9	237.7	237.0	204.5	197.4	156.8	163.3	221.5
Moisture Content (%):	21.4%	18.6%	18.8%	18.9%	16.9%	17.7%	20.4%	22.1%	20.0%	18.9%

Test Hole No.:										
Sample No.:										
Depth:										
Tare No.:										
Wet Sample + Tare (g):										
Dry Sample + Tare (g):										
Weight of Tare (g):										
Weight of Water (g):										
Weight of Dry Soil (g):										
Moisture Content (%):										

Comments: BH23-05 SPT6 contained low sample quantity, less than 100g



Reviewed By: Connor Carlson



Unconfined Compressive Strength of Cohesive Soil As Per ASTM D2186/D2186M

UCS Specimen	
Initial Diameter of Sample (mm):	59.5
Initial Height of Sample (mm):	135.4
Initial Area of Sample (mm ²):	2782
Initial Vol of Sample (mm ³):	376632
Length to Diameter Ratio:	2.28
Strain Rate (mm/80s):	0.877

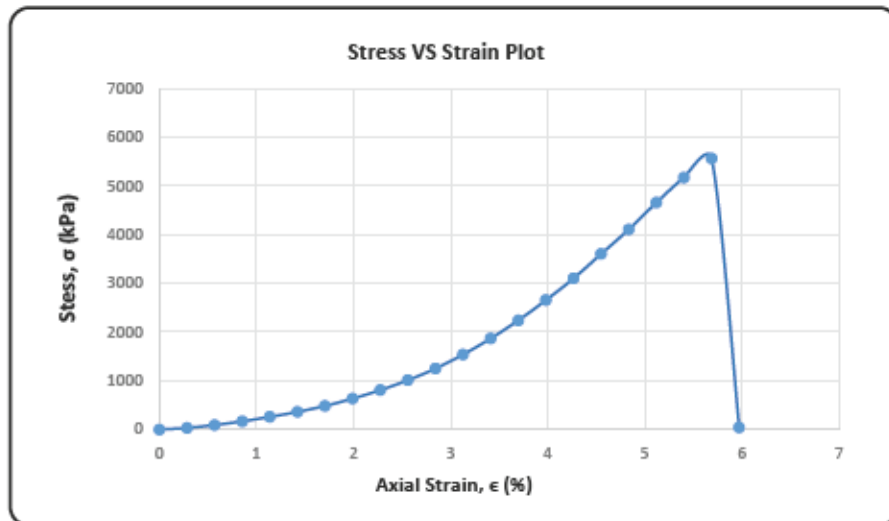
Moisture Content	
Tare No.:	2
Wet Weight + Tare (g):	854.7
Dry Weight + Tare (g):	789.4
Weight of Tare (g):	218.9
Moisture Content of Sample (%):	11.4%

Sample Density	
Wet Density of Sample (kg/m ³):	2251
Dry Density of Sample (kg/m ³):	2020
Wet Unit Wt. of Sample (kN/m ³):	N/A
Dry Unit Wt. of Sample (kN/m ³):	N/A

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
15	0.09	0.189
30	0.26	0.339
45	0.47	0.508
60	0.73	0.677
75	1.02	0.846
90	1.37	1.016
105	1.80	1.185
120	2.30	1.354
135	2.89	1.523
150	3.58	1.693

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
165	4.42	1.862
180	5.38	2.031
195	6.47	2.200
210	7.69	2.370
225	9.01	2.539
240	10.50	2.708
255	12.00	2.877
270	13.65	3.047
285	15.20	3.216
300	16.40	3.385

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
315	0.13	3.554



Peak Unconfined Compressive Strength: 5560.4 kPa

Sample Description: Gray Sandstone

Comments: _____

Connor C

Reviewed By: Connor C



Photographs



Fig No. 1: Sample Prior to Trim



Fig No. 2: Sample Before Break



Fig No. 3: Sample After Break



Fig No. 4: Sample Split

Comments: _____

Connor Carlson

Reviewed By: Connor Carlson



Unconfined Compressive Strength of Cohesive Soil As Per ASTM D2186/D2186M

UCS Specimen	
Initial Diameter of Sample (mm):	59
Initial Height of Sample (mm):	123.8
Initial Area of Sample (mm ²):	2735
Initial Vol of Sample (mm ³):	338602
Length to Diameter Ratio:	2.10
Strain Rate (mm/80s):	0.619

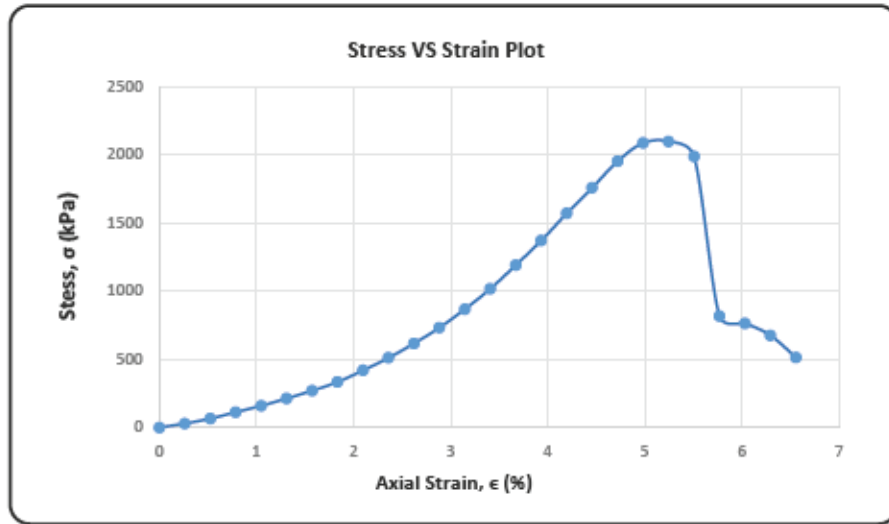
Moisture Content	
Tare No.:	1
Wet Weight + Tare (g):	883
Dry Weight + Tare (g):	811.6
Weight of Tare (g):	218.2
Moisture Content of Sample (%):	12.0%

Sample Density	
Wet Density of Sample (kg/m ³):	2224
Dry Density of Sample (kg/m ³):	1985
Wet Unit Wt. of Sample (kN/m ³):	N/A
Dry Unit Wt. of Sample (kN/m ³):	N/A

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
15	0.08	0.155
30	0.18	0.310
45	0.31	0.464
60	0.44	0.619
75	0.59	0.774
90	0.75	0.929
105	0.93	1.083
120	1.17	1.238
135	1.43	1.393
150	1.73	1.548

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
165	2.06	1.702
180	2.45	1.857
195	2.88	2.012
210	3.38	2.167
225	3.90	2.321
240	4.48	2.476
255	5.03	2.631
270	5.60	2.786
285	6.00	2.940
300	6.05	3.095

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
315	5.75	3.250
330	2.37	3.405
345	2.22	3.559
360	1.97	3.714
375	1.50	3.869



Peak Unconfined Compressive Strength: 2096 kPa

Sample Description: Gray Sandstone, Trace Silt

Comments: Pre-existing crack along upper face (vertical). Cracked along diagonal of pre-existing crack and appearance of vertical cracking.

Reviewed By: Connor C



Photographs



Fig No. 1: Sample Prior to Trim

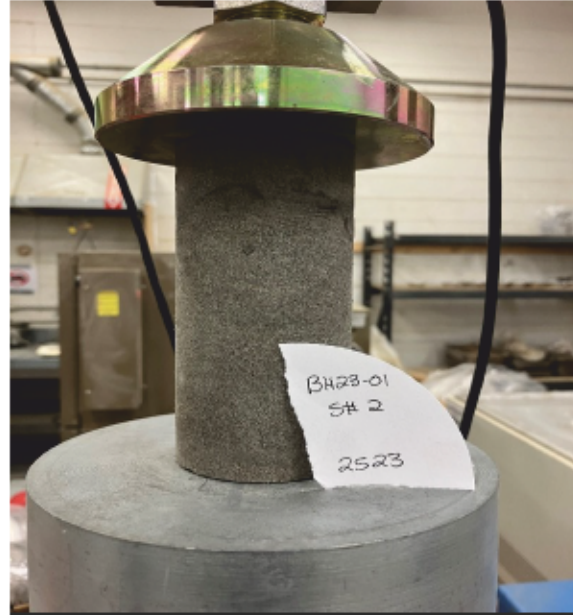


Fig No. 2: Sample Before Break

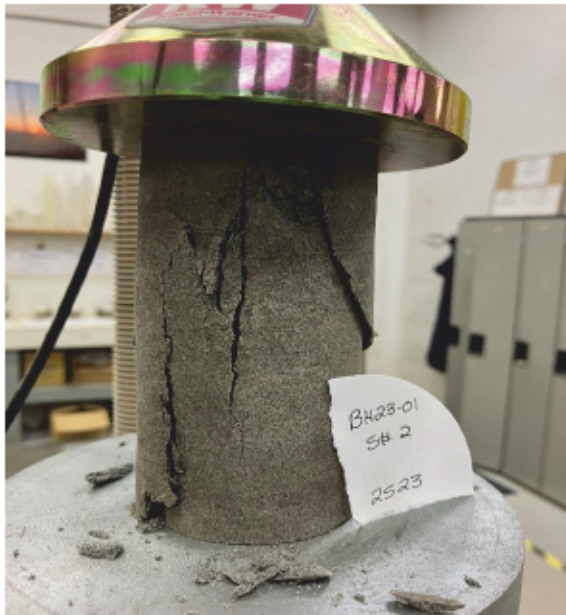


Fig No. 3: Sample After Break



Fig No. 4: Sample Split

Comments: _____

Reviewed By: Connor Carlson



Unconfined Compressive Strength of Cohesive Soil As Per ASTM D2166/D2168M

UCS Specimen	
Initial Diamter of Sample (mm):	58.5
Initial Height of Sample (mm):	135.1
Initial Area of Sample (mm ²):	2689
Initial Vol of Sample (mm ³):	363272
Length to Diameter Ratio:	2.31
Strain Rate (mm/80s):	0.8755

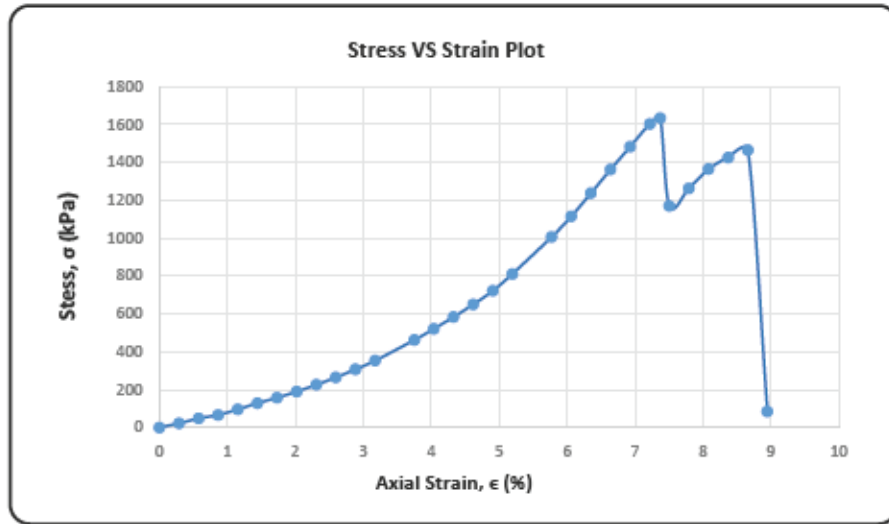
Moisture Content	
Tare No.:	8
Wet Weight + Tare (g):	829.1
Dry Weight + Tare (g):	760.9
Weight of Tare (g):	216.3
Moisture Content of Sample (%):	12.5%

Sample Density	
Wet Density of Sample (kg/m ³):	2210
Dry Density of Sample (kg/m ³):	1984
Wet Unit Wt. of Sample (kN/m ³):	N/A
Dry Unit Wt. of Sample (kN/m ³):	N/A

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
15	0.06	0.169
30	0.13	0.338
45	0.18	0.507
60	0.26	0.676
75	0.35	0.844
90	0.43	1.013
105	0.52	1.182
120	0.62	1.351
135	0.73	1.520
150	0.85	1.689

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
165	0.98	1.858
195	1.29	2.195
210	1.46	2.384
225	1.64	2.533
240	1.83	2.702
255	2.04	2.871
270	2.30	3.040
300	2.87	3.378
315	3.19	3.546
330	3.55	3.715

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
345	3.82	3.884
360	4.28	4.053
375	4.64	4.222
383	4.74	4.312
390	3.40	4.391
405	3.68	4.560
420	3.99	4.729
435	4.18	4.897
450	4.31	5.066
465	0.25	5.235



Peak Unconfined Compressive Strength: 1632.9 kPa

Sample Description: Gray Sandstone, Trace Silt

Comments: _____

Reviewed By: Connor Carlson



Photographs



Fig No. 1: Sample Prior to Trim

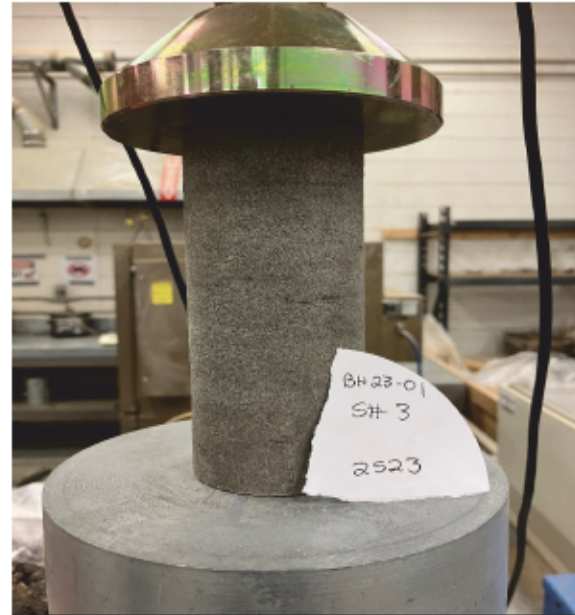


Fig No. 2: Sample Before Break

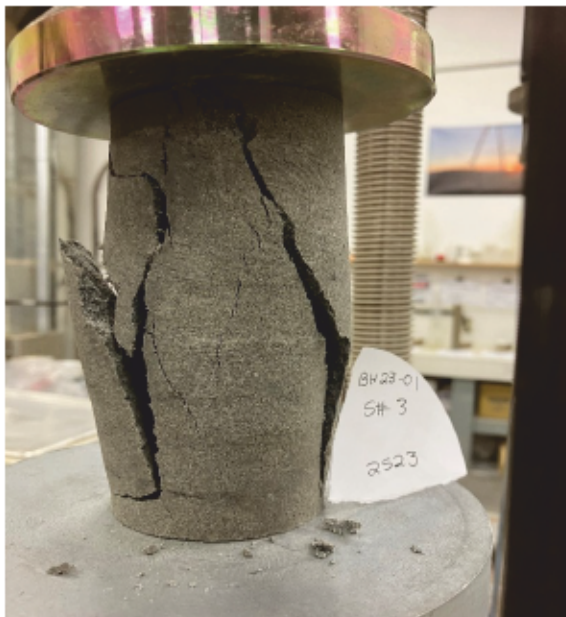


Fig No. 3: Sample After Break



Fig No. 4: Sample Split

Comments: _____

Reviewed By: Connor Carlson



Unconfined Compressive Strength of Cohesive Soil As Per ASTM D2166/D2166M

UCS Specimen	
Initial Diamter of Sample (mm):	58
Initial Height of Sample (mm):	116.5
Initial Area of Sample (mm ²):	2643
Initial Vol of Sample (mm ³):	307926
Length to Diameter Ratio:	2.01
Strain Rate (mm/80s):	0.5825

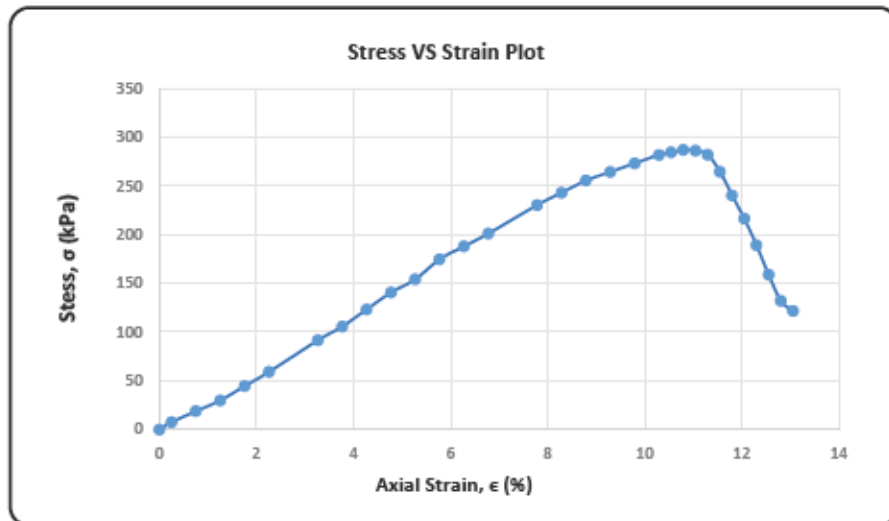
Moisture Content	
Tare No.:	4
Wet Weight + Tare (g):	802.4
Dry Weight + Tare (g):	736.3
Weight of Tare (g):	218.4
Moisture Content of Sample (%):	12.8%

Sample Density	
Wet Density of Sample (kg/m ³):	2021
Dry Density of Sample (kg/m ³):	1792
Wet Unit Wt. of Sample (kN/m ³):	N/A
Dry Unit Wt. of Sample (kN/m ³):	N/A

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
15	0.02	0.146
45	0.05	0.437
75	0.08	0.728
105	0.12	1.019
135	0.16	1.311
195	0.25	1.893
225	0.29	2.184
255	0.34	2.476
285	0.39	2.767
315	0.43	3.058

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
345	0.49	3.349
375	0.53	3.641
405	0.57	3.932
465	0.66	4.514
495	0.70	4.806
525	0.74	5.097
555	0.77	5.388
585	0.80	5.679
615	0.83	5.971
630	0.84	6.116

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
645	0.85	6.262
660	0.85	6.408
675	0.84	6.553
690	0.79	6.699
705	0.72	6.844
720	0.65	6.990
735	0.57	7.136
750	0.48	7.281
765	0.40	7.427
780	0.37	7.573



Peak Unconfined Compressive Strength: 286.9 kPa

Sample Description: Gray Sandstone, Trace Clay, Trace Silt

Comments: Pre-existing crack through center of sample prior to compression.

Reviewed By: Connor Carlson



Photographs



Fig No. 1: Sample Prior to Trim

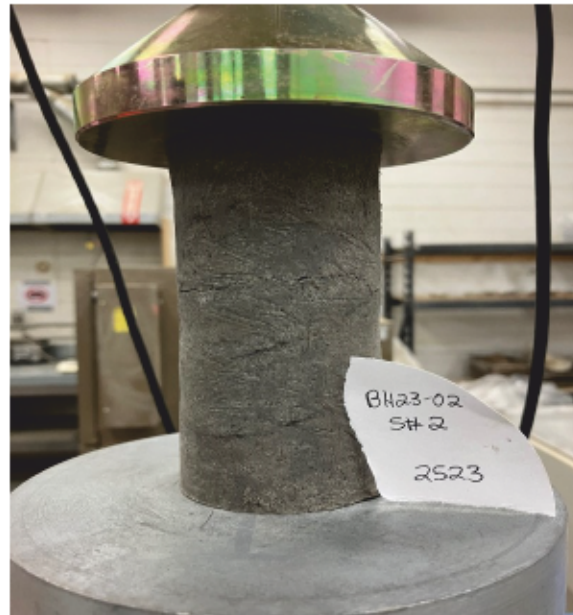


Fig No. 2: Sample Before Break with Crack

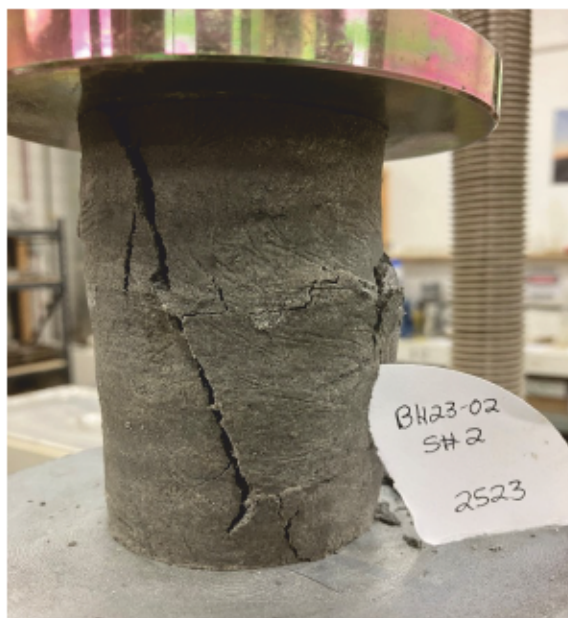


Fig No. 3: Sample After Break



Fig No. 4: Sample Split

Comments: _____

Connor Carlson

Reviewed By: Connor Carlson



Unconfined Compressive Strength of Cohesive Soil As Per ASTM D2186/D2186M

UCS Specimen	
Initial Diameter of Sample (mm):	58.5
Initial Height of Sample (mm):	124.2
Initial Area of Sample (mm ²):	2689
Initial Vol of Sample (mm ³):	333983
Length to Diameter Ratio:	2.12
Strain Rate (mm/80s):	0.621

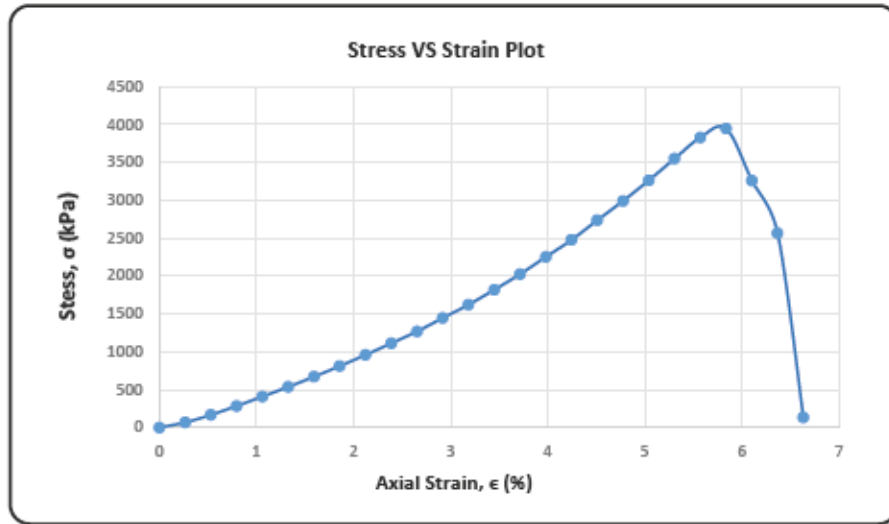
Moisture Content	
Tare No.:	2A
Wet Weight + Tare (g):	883.2
Dry Weight + Tare (g):	819.3
Weight of Tare (g):	218.9
Moisture Content of Sample (%):	10.6%

Sample Density	
Wet Density of Sample (kg/m ³):	2245
Dry Density of Sample (kg/m ³):	2029
Wet Unit Wt. of Sample (kN/m ³):	N/A
Dry Unit Wt. of Sample (kN/m ³):	N/A

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
15	0.18	0.155
30	0.45	0.311
45	0.77	0.466
60	1.11	0.621
75	1.46	0.776
90	1.83	0.932
105	2.22	1.087
120	2.63	1.242
135	3.06	1.397
150	3.50	1.553

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
165	4.00	1.708
180	4.50	1.863
195	5.06	2.018
210	5.65	2.174
225	6.30	2.329
240	6.95	2.484
255	7.70	2.639
270	8.44	2.795
285	9.23	2.950
300	10.07	3.105

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
315	10.89	3.260
330	11.28	3.416
345	9.33	3.571
360	7.36	3.726
375	0.38	3.881



Peak Unconfined Compressive Strength: 3943.1 kPa

Sample Description: Gray Sandstone Layered Siltstone

Comments: _____

Reviewed By: Connor Carlson



Photographs

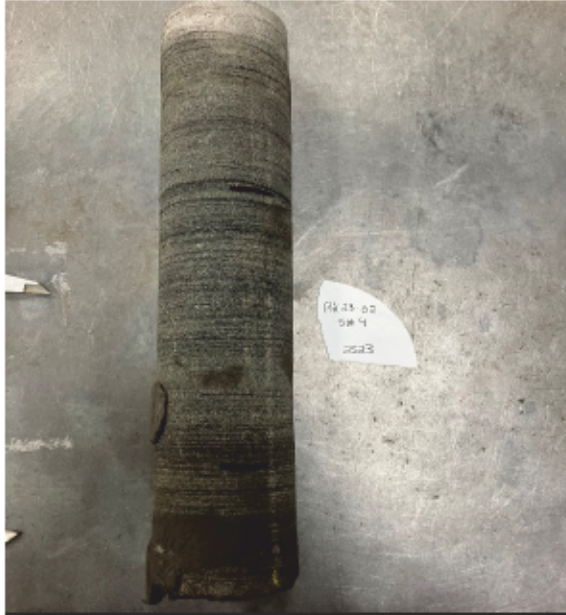


Fig No. 1: Sample Prior to Trim



Fig No. 2: Sample Before Break

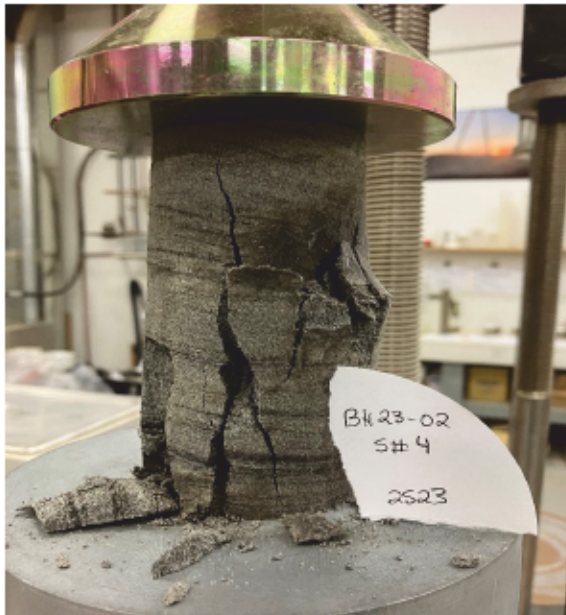


Fig No. 3: Sample After Break



Fig No. 4: Sample Split

Comments: _____

Reviewed By: Connor Carlson



Hydrometer Particle-Size Analysis of Soils As Per ASTM D422

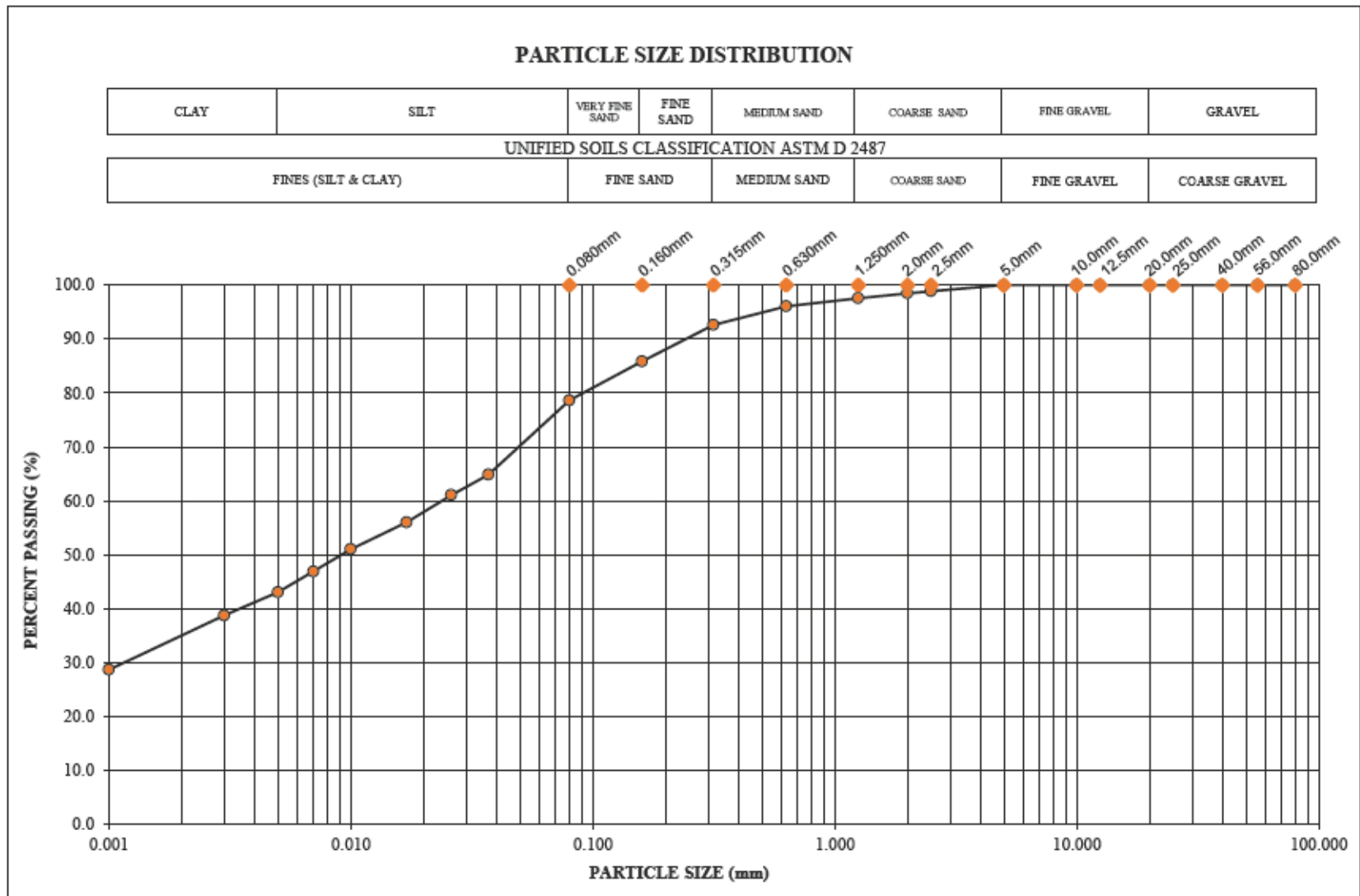
Sample Information	
BH No.:	23-04
Sample No.:	AU4
Depth (m/ft):	2.7m
In-Situ Moisture Cont. (%):	15.6%
Sand & Gravel Shape:	Angular
Dispersion Device:	30oz Mlt Cup
Length of Dispersion (min):	1 min

Grain Size Proportions (%)	
% Gravel (> 5.0mm):	0.0%
% Sand (80µm to 5.0mm):	21.4%
% Silt (5µm to 80µm):	35.6%
% Clay (< 5µm):	43.0%

Grain Size Analysis (Gravel & Sand)	
Sieve Size (µm)	Passing (%)
40 000	100.0%
25 000	100.0%
20 000	100.0%
12 500	100.0%
10 000	100.0%
5 000	100.0%
2 500	98.9%
2 000	98.5%
1 250	97.5%
630	96.1%
315	92.6%
160	85.8%
80	78.6%

Hydrometer Analysis (Silt & Clay)	
Sieve Size (µm)	Passing (%)
37	64.8%
26	61.1%
17	56.0%
10	51.0%
7	46.9%
5	43.0%
3	38.7%
1	28.7%

Comments



Reviewed By: Connor Carlson



Hydrometer Particle-Size Analysis of Soils As Per ASTM D422

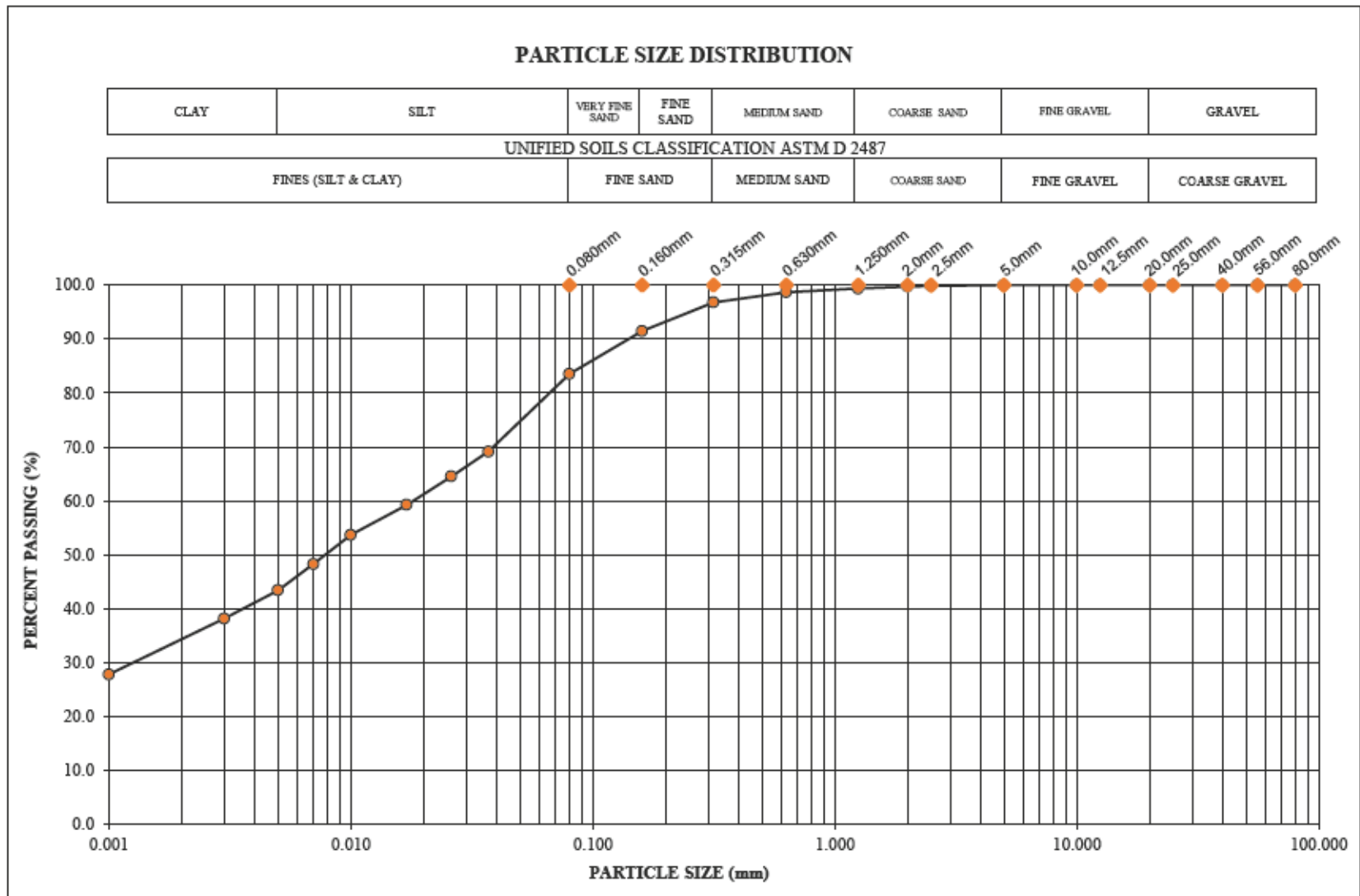
Sample Information	
BH No.:	23-04
Sample No.:	AU16
Depth (m/ft):	11.9m
In-Situ Moisture Cont. (%):	17.5%
Sand & Gravel Shape:	Angular
Dispersion Device:	30oz Mlt Cup
Length of Dispersion (min):	1 min

Grain Size Proportions (%)	
% Gravel (> 5.0mm):	0.0%
% Sand (80µm to 5.0mm):	16.5%
% Silt (5µm to 80µm):	40.1%
% Clay (< 5µm):	43.4%

Grain Size Analysis (Gravel & Sand)	
Sieve Size (µm)	Passing (%)
40 000	100.0%
25 000	100.0%
20 000	100.0%
12 500	100.0%
10 000	100.0%
5 000	100.0%
2 500	99.8%
2 000	99.7%
1 250	99.4%
630	98.7%
315	96.8%
160	91.5%
80	83.5%

Hydrometer Analysis (Silt & Clay)	
Sieve Size (µm)	Passing (%)
37	69.1%
26	64.5%
17	59.2%
10	53.6%
7	48.2%
5	43.4%
3	38.1%
1	27.8%

Comments



Reviewed By: Connor Carlson



Hydrometer Particle-Size Analysis of Soils As Per ASTM D422

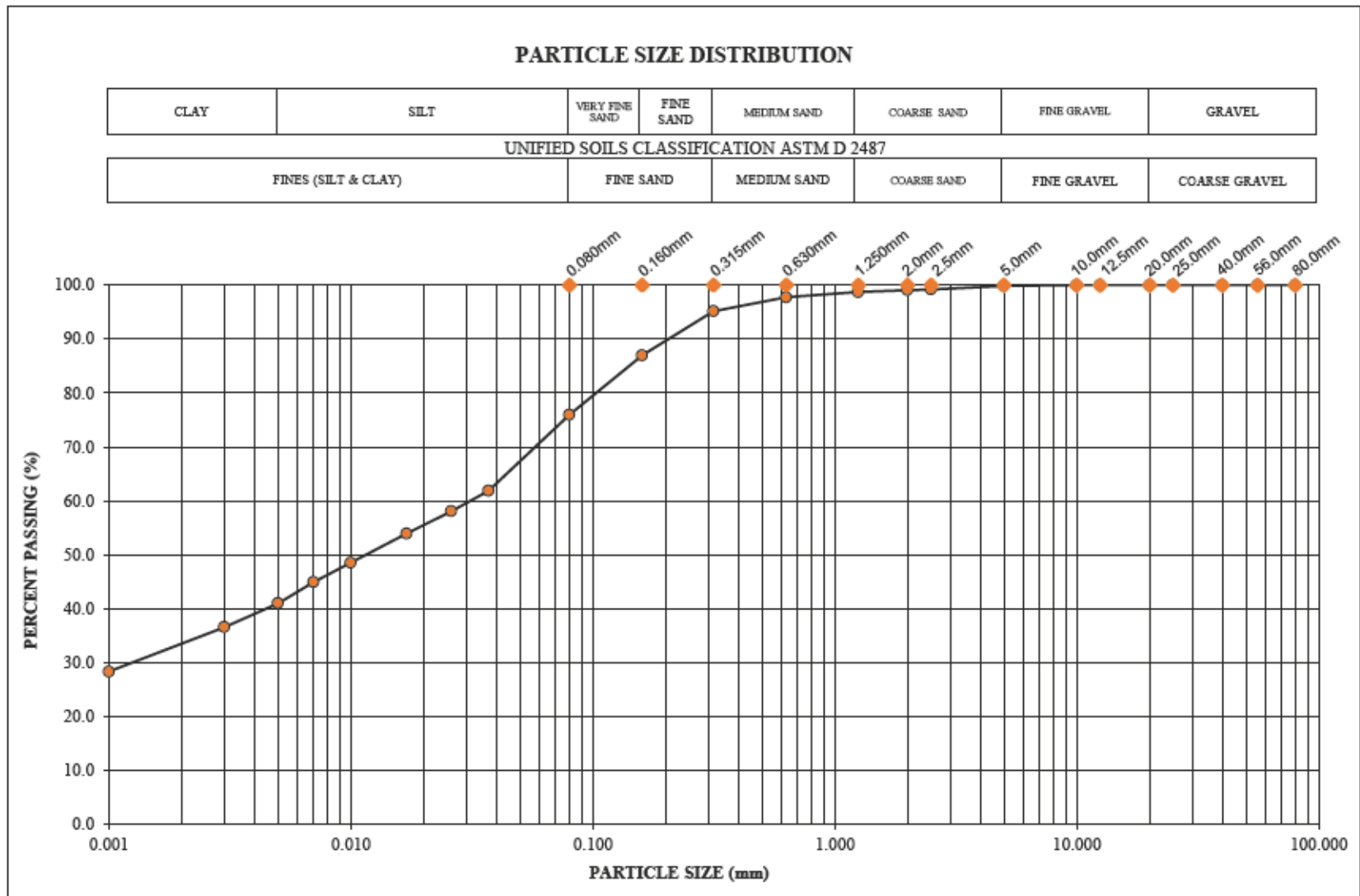
Sample Information	
BH No.:	23-04
Sample No.:	AU19
Depth (m/ft):	14.3m
In-Situ Moisture Cont. (%):	19.6%
Sand & Gravel Shape:	Angular
Dispersion Device:	30oz Mlt Cup
Length of Dispersion (min):	1 min

Grain Size Proportions (%)	
% Gravel (> 5.0mm):	0.2%
% Sand (80µm to 5.0mm):	23.9%
% Silt (5µm to 80µm):	35.0%
% Clay (< 5µm):	41.0%

Grain Size Analysis (Gravel & Sand)	
Sieve Size (µm)	Passing (%)
40 000	100.0%
25 000	100.0%
20 000	100.0%
12 500	100.0%
10 000	100.0%
5 000	99.8%
2 500	99.2%
2 000	99.0%
1 250	98.7%
630	97.7%
315	95.1%
160	87.0%
80	76.0%

Hydrometer Analysis (Silt & Clay)	
Sieve Size (µm)	Passing (%)
37	61.8%
26	58.1%
17	53.9%
10	48.5%
7	44.9%
5	41.0%
3	36.6%
1	28.3%

Comments



Reviewed By: Connor Carlson



Atterberg Limits As Per ASTM D4318

Liquid Limit - Hand Operated Method

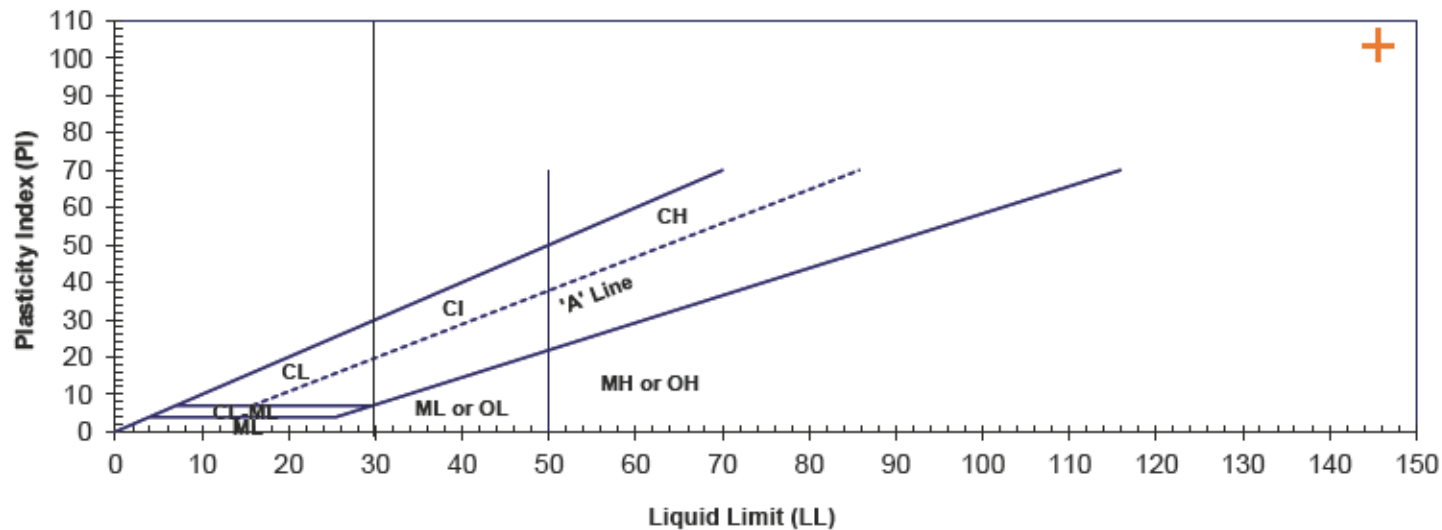
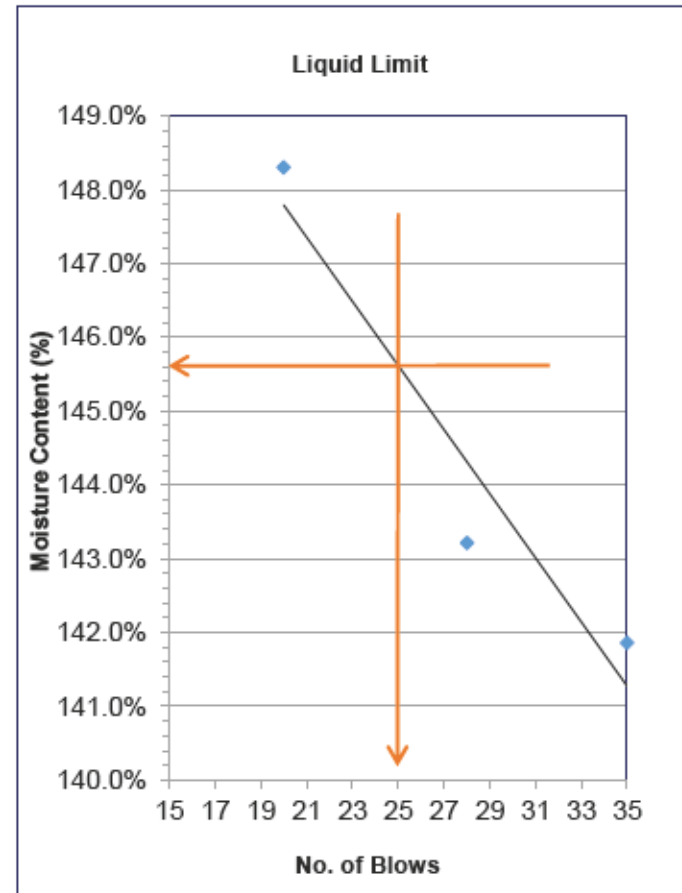
Borehole No.:	22-01	Sample No.:	S1	Depth:	9.0m
Tare No.:	26B	78	ZZ		
Weight of Tare (g):	29.45	28.81	29.43		
Weight of Wet Soil + Tare (g):	54.72	56.14	55.03		
Weight of Dry Soil + Tare (g):	39.84	40.11	39.74		
Weight of Water (g):	14.88	16.03	15.29		
Weight of Dry Soil (g):	10.39	11.30	10.31		
Moisture (%):	143.2%	141.9%	148.3%		
No. of Blows:	28	35	20		
Liquid Limit from Flow Curve			145.6		

Prep Method: Moist ☐ Dry ☒ Liquid Limit Test Method
Oversize: Pestle ☒ Grinder ☐ Method A ☒ Method B ☐

Plastic Limit - Manual Rolling Method

Tare No.:	45	57
Weight of Tare (g):	28.65	28.99
Weight of Wet Soil + Tare (g):	31.98	31.97
Weight of Dry Soil + Tare (g):	30.99	31.09
Weight of Water (g):	0.99	0.88
Weight of Dry Soil (g):	2.34	2.10
Moisture (%):	42.3%	41.9%
Plastic Limit		42.1

Grooving Tool: Plastic ☒ Metal ☐



Summary

Liquid Limit:	146	Material Description:	N/A
Plastic Limit:	42		
Plasticity Index:	103	Material Retained on 425um Sieve (%):	0.3%
Unified Soil Classification:	CH	As Received Moisture (%):	42.0%

Comments: _____

Reviewed By: Connor Carlson



Atterberg Limits As Per ASTM D4318

Liquid Limit - Hand Operated Method

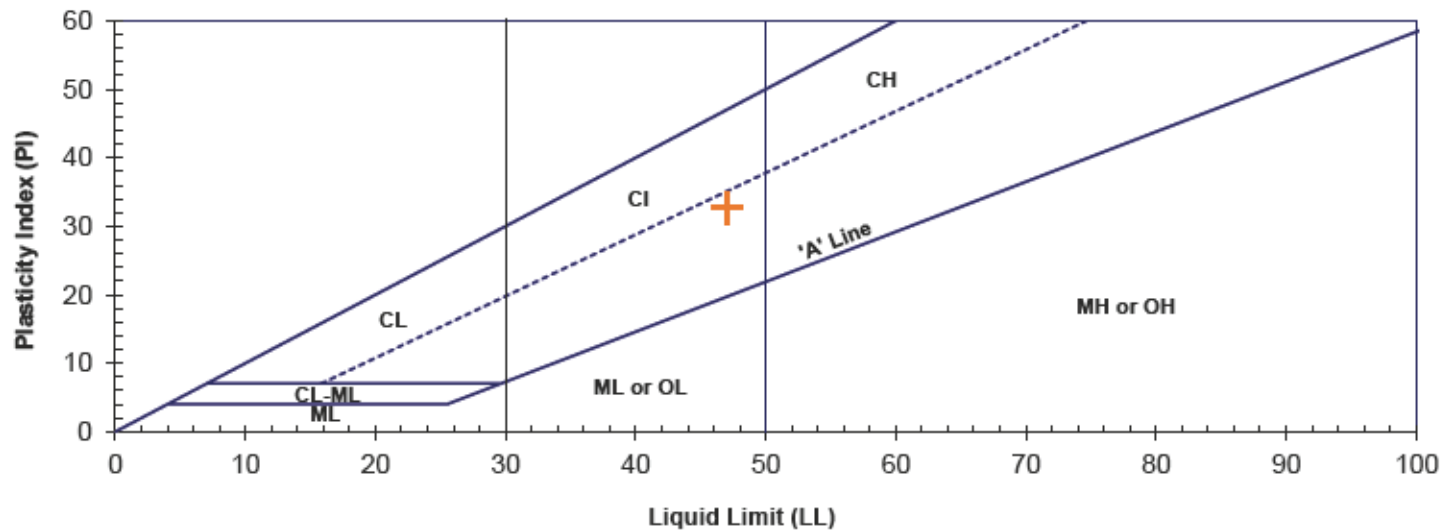
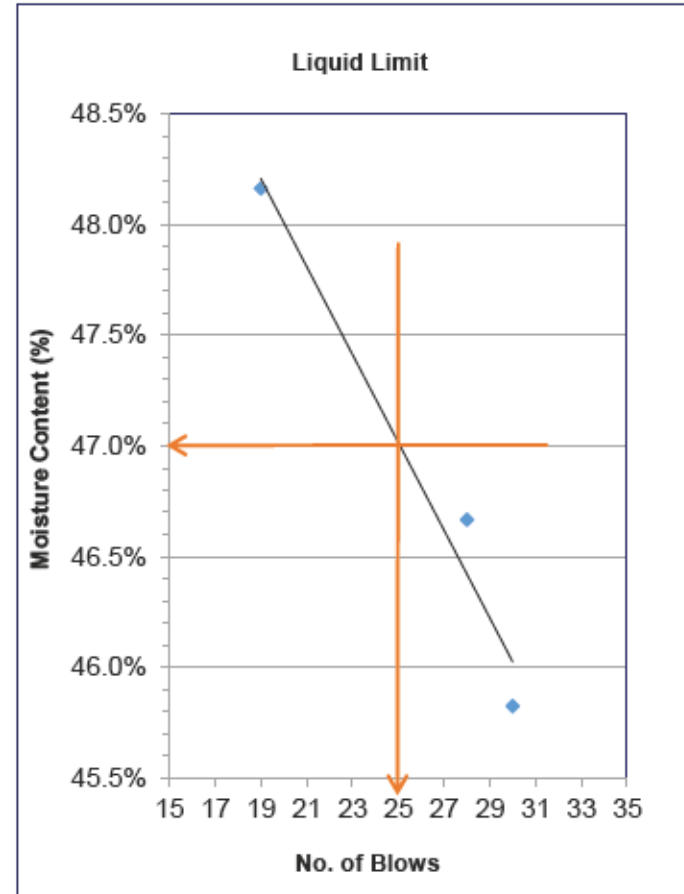
Borehole No.:	23-04	Sample No.:	AU6	Depth:	4.3m
Tare No.:	B11	B19	A27		
Weight of Tare (g):	20.69	20.72	20.79		
Weight of Wet Soil + Tare (g):	39.64	43.82	41.22		
Weight of Dry Soil + Tare (g):	33.48	36.47	34.80		
Weight of Water (g):	6.16	7.35	6.42		
Weight of Dry Soil (g):	12.79	15.75	14.01		
Moisture (%):	48.2%	46.7%	45.8%		
No. of Blows:	19	28	30		
Liquid Limit from Flow Curve	47.0				

Prep Method: Moist ☐ Dry ☒ Liquid Limit Test Method
Oversize: Pestle ☒ Grinder ☐ Method A ☒ Method B ☐

Plastic Limit - Manual Rolling Method

Tare No.:	B23	A33
Weight of Tare (g):	20.62	21.09
Weight of Wet Soil + Tare (g):	27.03	27.97
Weight of Dry Soil + Tare (g):	26.25	27.09
Weight of Water (g):	0.78	0.88
Weight of Dry Soil (g):	5.63	6.00
Moisture (%):	13.9%	14.7%
Plastic Limit	14.3	

Grooving Tool: Plastic ☒ Metal ☐



Summary

Liquid Limit:	47
Plastic Limit:	14
Plasticity Index:	33
Unified Soil Classification:	CI

Material Description: Clay, Some Silt, Trace Sand, Trace Gravel, Dark Brown, Oxidation, Coal Chips

Material Retained on 425um Sieve (%): 4.4%

As Received Moisture (%): 16.6%

Comments:

Reviewed By: Connor Carlson



Atterberg Limits As Per ASTM D4318

Liquid Limit - Hand Operated Method

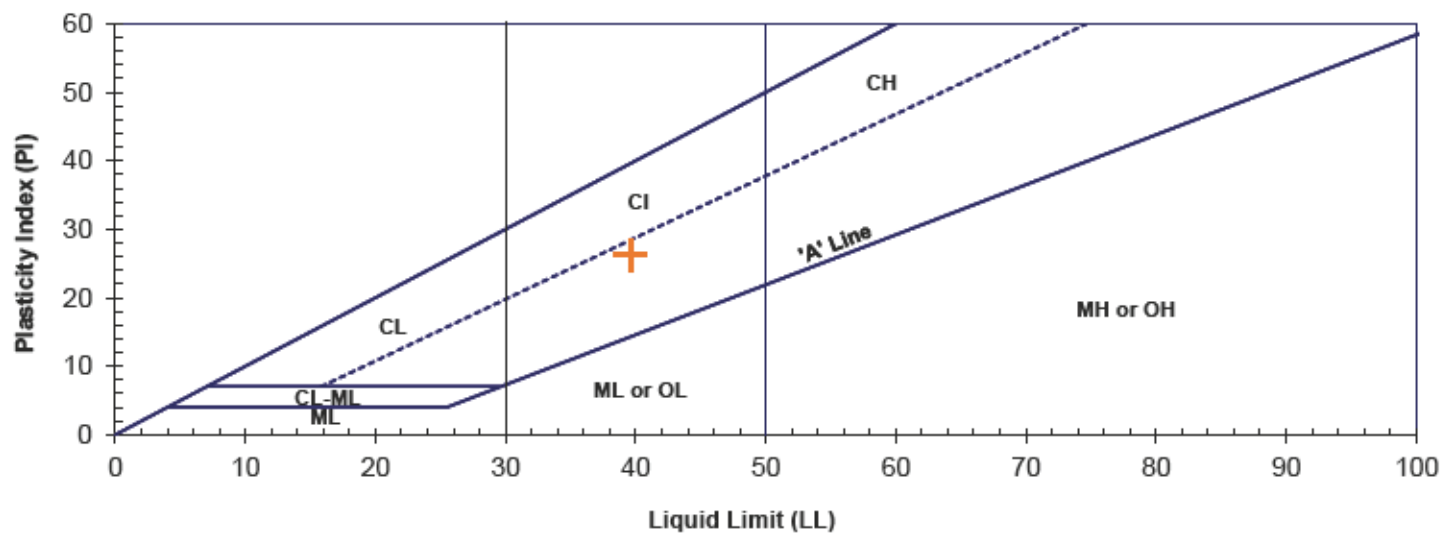
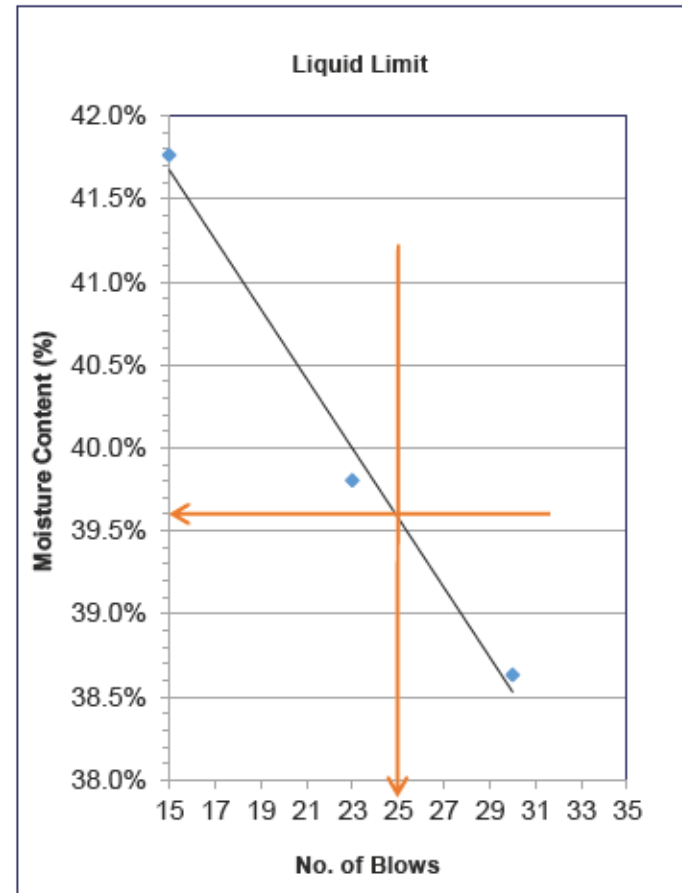
Borehole No.:	23-05	Sample No.:	AU9	Depth:	6.7m
Tare No.:	B24	A23	A29		
Weight of Tare (g):	20.68	20.69	21.00		
Weight of Wet Soil + Tare (g):	37.72	39.27	39.05		
Weight of Dry Soil + Tare (g):	32.70	33.98	34.02		
Weight of Water (g):	5.02	5.29	5.03		
Weight of Dry Soil (g):	12.02	13.29	13.02		
Moisture (%):	41.8%	39.8%	38.6%		
No. of Blows:	15	23	30		
Liquid Limit from Flow Curve	39.6				

Prep Method: Moist ☐ Dry ☒ Liquid Limit Test Method
Oversize: Pestle ☒ Grinder ☐ Method A ☒ Method B ☐

Plastic Limit - Manual Rolling Method

Tare No.:	A25	A13
Weight of Tare (g):	20.60	20.66
Weight of Wet Soil + Tare (g):	32.94	29.48
Weight of Dry Soil + Tare (g):	31.46	28.46
Weight of Water (g):	1.48	1.02
Weight of Dry Soil (g):	10.86	7.80
Moisture (%):	13.6%	13.1%
Plastic Limit	13.4	

Grooving Tool: Plastic ☒ Metal ☐



Summary

Liquid Limit:	40
Plastic Limit:	13
Plasticity Index:	26
Unified Soil Classification:	CI

Material Description: Clay, Trace Silt, Trace Sand, Trace Gravel, Dark Brown, Coal Chips

Material Retained on 425um Sieve (%): 2.9%

As Received Moisture (%): 17.4%

Comments:

Reviewed By: Connor Carlson



Atterberg Limits As Per ASTM D4318

Liquid Limit - Hand Operated Method

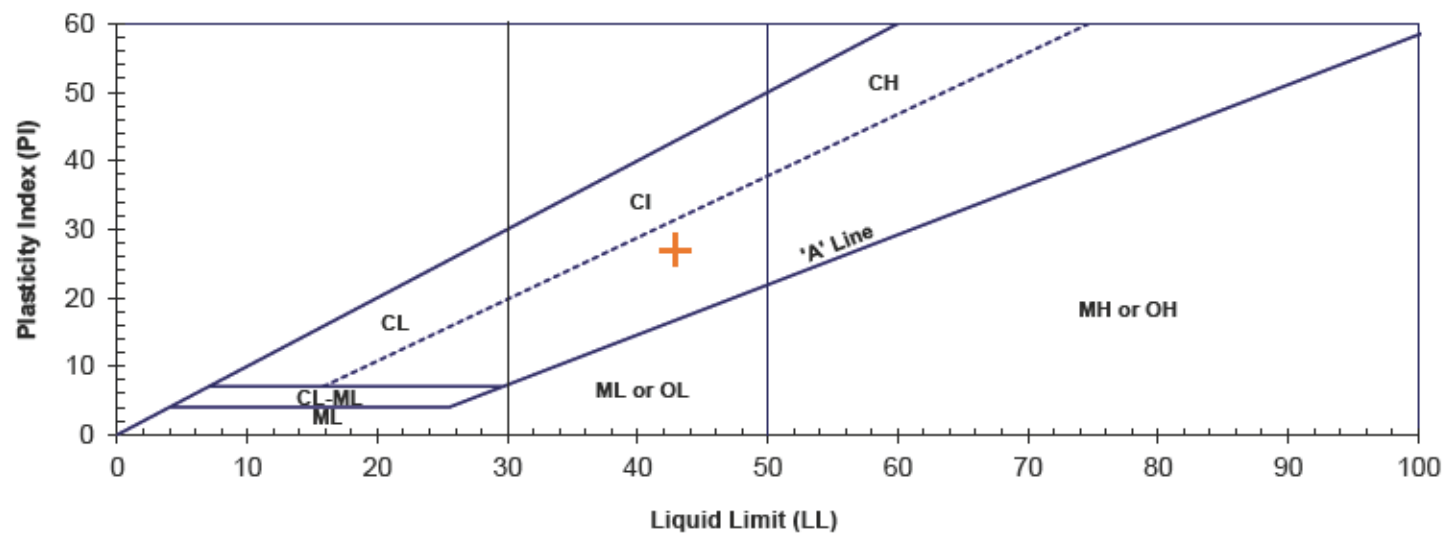
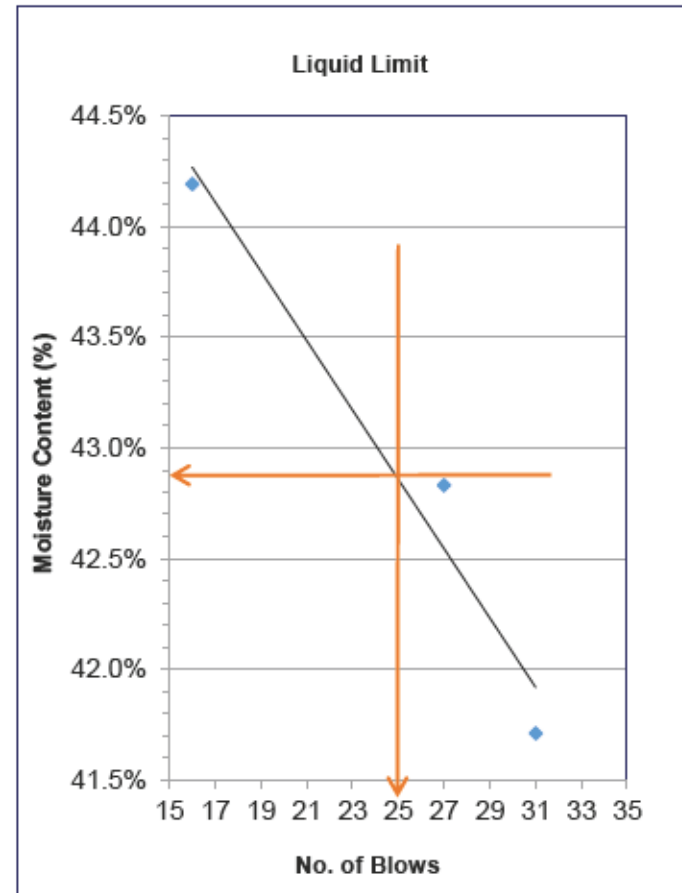
Borehole No.:	23-05	Sample No.:	AU11	Depth:	8.2m
Tare No.:	A30	A31	B18		
Weight of Tare (g):	20.73	20.73	20.80		
Weight of Wet Soil + Tare (g):	44.19	43.84	50.63		
Weight of Dry Soil + Tare (g):	37.00	36.91	41.85		
Weight of Water (g):	7.19	6.93	8.78		
Weight of Dry Soil (g):	16.27	16.18	21.05		
Moisture (%):	44.2%	42.8%	41.7%		
No. of Blows:	16	27	31		
Liquid Limit from Flow Curve	42.9				

Prep Method: Moist ☐ Dry ☒ Liquid Limit Test Method
Oversize: Pestle ☒ Grinder ☐ Method A ☒ Method B ☐

Plastic Limit - Manual Rolling Method

Tare No.:	B30	B17
Weight of Tare (g):	20.66	20.74
Weight of Wet Soil + Tare (g):	32.18	31.42
Weight of Dry Soil + Tare (g):	30.60	29.95
Weight of Water (g):	1.58	1.47
Weight of Dry Soil (g):	9.94	9.21
Moisture (%):	15.9%	16.0%
Plastic Limit	15.9	

Grooving Tool: Plastic ☒ Metal ☐



Summary

Liquid Limit:	43
Plastic Limit:	16
Plasticity Index:	27
Unified Soil Classification:	CI

Material Description: Clay, Trace Silt, Dark Brown

Material Retained on 425um Sieve (%): 0.6%

As Received Moisture (%): 20.4%

Comments:

Reviewed By: Connor Carlson



Sieve Analysis of Fine and Coarse Aggregates As Per ASTM C136/C136M

Moisture Content

Tare No.:	4B
Wet Wt. + Tare (g):	2452.6
Dry Wt. + Tare (g):	2264.6
Tare Weight (g):	627.4
Moisture Content (%):	11.5%

Sieve Analysis Sample

Tare No.:	S-1
Dry Wt. + Tare (g):	616.2
Wt. of Tare (g):	150.7
Dry Wt. of Sample (g):	465.5
Adjusted Sample Wt. (g):	417.6

Aggregate Analysis

Sieve Size (µm)	Weight Retained (g):	Cumulative Weight Passing (g):	Passing (%)
20 000	0.0	417.6	100.0
16 000	0.0	417.6	100.0
10 000	0.0	417.6	100.0
5 000	0.0	417.6	100.0
1 250	0.0	417.6	100.0
630	1.2	416.4	99.7
315	18.6	397.8	95.3
160	216.3	181.5	43.5
80	128.7	52.8	12.6

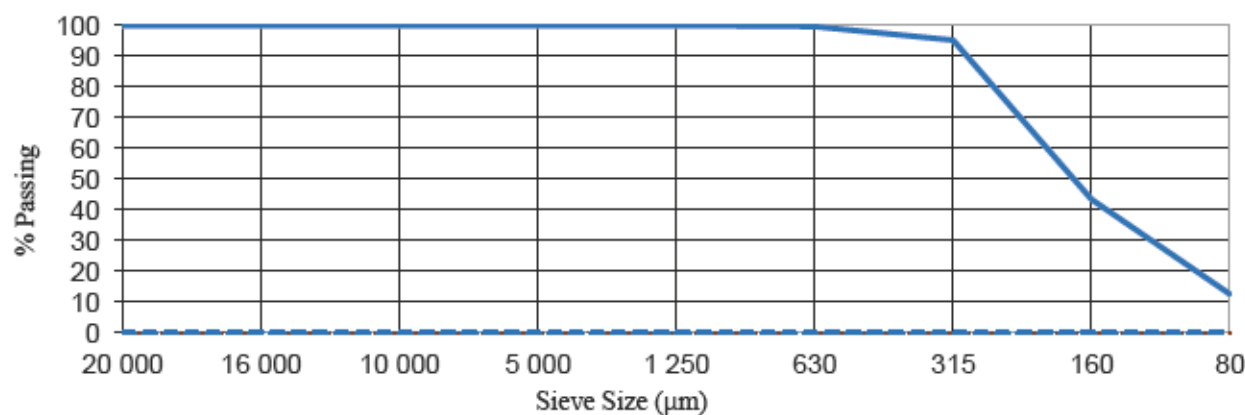
Pan:	10.7	Dry Wash + Tare (g):	526.5
Total:	375.5	Tare Wt. (g):	150.7

Specification

No Specification Given	
Min	Max
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A

Sample (g):	375.8
Loss (Max 0.5%):	-0.08%

Particle Size Distribution



Fracture Count in Accordance with ASTM D5821

Percentage Fracture: N/A

Comments: Borehole 23-01, S1, 13.39m

Reviewed By: Connor Carlson



Sieve Analysis of Fine and Coarse Aggregates As Per ASTM C136/C136M

Moisture Content

Tare No.:	ML
Wet Wt. + Tare (g):	1883.3
Dry Wt. + Tare (g):	1735.1
Tare Weight (g):	538.0
Moisture Content (%):	12.4%

Sieve Analysis Sample

Tare No.:	1F
Dry Wt. + Tare (g):	510.5
Wt. of Tare (g):	101.8
Dry Wt. of Sample (g):	408.7
Adjusted Sample Wt. (g):	363.7

Aggregate Analysis

Sieve Size (µm)	Weight Retained (g):	Cumulative Weight Passing (g):	Passing (%)
20 000	0.0	363.7	100.0
16 000	0.0	363.7	100.0
10 000	0.0	363.7	100.0
5 000	0.0	363.7	100.0
1 250	0.0	363.7	100.0
630	15.8	347.9	95.7
315	41.3	306.6	84.3
160	196.3	110.3	30.3
80	76.1	34.2	9.4

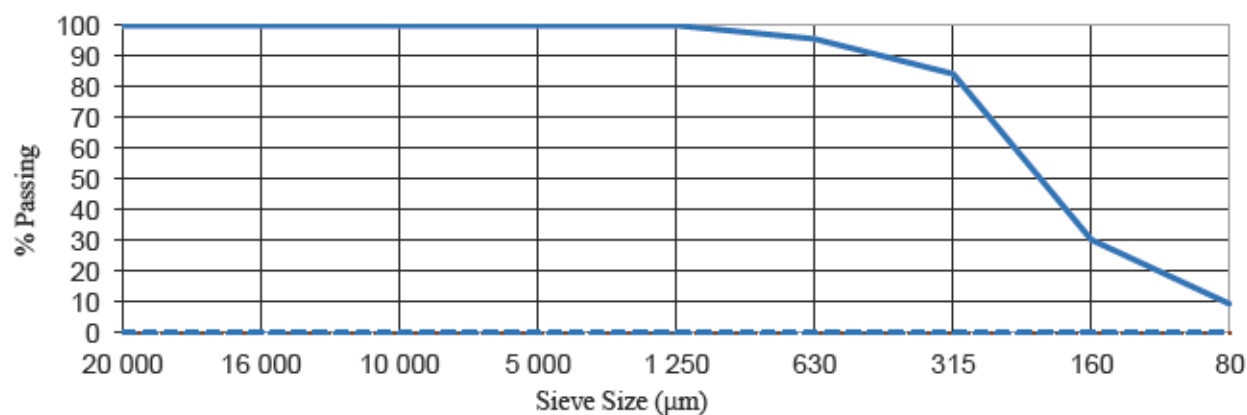
Pan:	10.0	Dry Wash + Tare (g):	440.8
Total:	339.5	Tare Wt. (g):	101.8

Specification

No Specification Given	
Min	Max
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A

Sample (g):	339.0
Loss (Max 0.5%):	0.15%

Particle Size Distribution



Fracture Count in Accordance with ASTM D5821

Percentage Fracture: N/A

Comments: Borehole 23-02, S1, 4.5m

Reviewed By: Connor Carlson



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