CP-008872 Queen Alexandra Housing Development - Preliminary Geotechnical Site Investigation Final Revision 1

The City of Edmonton

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1. Introduction

1.1 General

AECOM Canada Ltd. (AECOM) was retained by the City of Edmonton (CoE) to conduct a preliminary geotechnical site investigation to support the Queen Alexandra housing project. It is understood that the CoE intends to develop the three lots at 10820, 10824 and 10828 – 71 Avenue Northwest for the purposes of constructing a multi storey housing building. The CoE indicated this housing project could reach a height of up to 28 metres. At the time of writing of this report, the layout of the housing building had not yet been determined. The purpose of this preliminary geotechnical investigation was to determine the subsurface conditions to support the design of this multi storey housing building, identify potential geotechnical risks at this site, and provide design parameters for the foundation design. It is anticipated that more testholes will be required for the detailed design phase of this project. The testhole locations of the testholes drilled during this geotechnical investigation are illustrated on **Figure 1** in **Appendix A**. Testhole logs are included in **Appendix B** and laboratory test results are included in **Appendix C**.

1.2 Scope of work

The scope of work for this intrusive geotechnical investigation includes the following:

- Planning and co-ordination of the field drilling program, which included site reconnaissance, safety planning, utility coordination and clearances, coordination of AECOM subcontractors, and logistics planning (site access, mobilization, staging, and demobilization of equipment).
- Performing a geotechnical desktop study, which included a review of available geological maps.
- Executing the geotechnical field investigation, which included drilling three testholes within the site limits of the proposed housing building. These testholes were drilled to depths of between 6.5 and 15.5 metres below ground surface (mBGS).
- Installation of a standpipe piezometer in one testhole to monitor groundwater conditions.
- Measuring groundwater levels in the standpipe after completion of the field drilling program.
- Performing laboratory testing on soil samples for soil classification and to determine engineering properties of selected soil samples collected during the field investigation.
- Completing a geotechnical investigation report, which includes:
    - Description of geotechnical investigation methodology
    - Geological desktop study
    - Description of the subsurface conditions
    - General site recommendations and site suitability
    - Foundations recommendations, including radon has mitigation recommendations
    - Recommendations for pavement structures
    - Recommendations for further site investigation

2. Methodology

2.1 Safety Planning

The safety planning for this geotechnical investigation took into consideration AECOM and the CoE safety practices and procedures. The CoE Prime Contractor OH&S Orientation was completed prior to conducting field work. The Project Hazard Assessment for the site shared by the CoE was reviewed by AECOM. An AECOM Safe Work Plan was completed and submitted for review by the CoE. Daily Tailgate Meetings and Task Hazard Assessments were completed prior to all field operations which included utility locating, borehole drilling and groundwater monitoring. All safety planning complied with COVID-19 safety recommendation set by the government of Alberta and the CoE.

2.2 Site Reconnaissance

Prior to the commencement of the intrusive investigation, a site reconnaissance was conducted by AECOM on February 12, 2021 to assess general site access conditions, identify the suitability of the proposed testhole locations, and review the locations of the buried and overhead utilities. Utility coordination and clearances included contacting Alberta One-Call and using a private locator to clear borehole locations. Maverick Inspection Ltd. was contracted by AECOM to clear the borehole locations of utilities.

2.3 Surficial Geology

A surficial geological map (Map 601, Surficial Geology of Alberta, M.M Fenton, et. al, 2013.) provided by the Alberta Geological Survey was reviewed prior to conducting the geotechnical investigation. The surficial geology in the study area is expected to include primarily glaciolacustrine deposits. Glaciolacustrine deposits include either deposited sediments consisting of rhythmically fine sand, silt, clay, and till, or littoral sediments consisting of well-sorted silty sand, pebbly sand, and minor gravel.

2.4 Bedrock Geology

The bedrock geology in this study area is a part of the Horseshoe Canyon Formation (marked as KHC in Map 600, Bedrock Geology of Alberta, Prior G.J et al, 2013), which is comprised of fine-grained sandstone, interbedded with siltstone and bentonitic mudstone. The Horseshoe Canyon Formation was formerly known as the Edmonton formation. The bedrock is expected to be non-marine to locally marginal marine. Coal seams and bentonite beds of variable thickness are common throughout the formation.

2.5 Field Investigation

Three testholes were advanced within the site limits of the proposed Queen Alexandra Housing project. The three testholes, TH21-01, TH21-02, and TH21-03, were drilled to depths of 15.5 metres below ground surface (mBGS), 6.5 mBGS and 14.8 mBGS respectively, on February 19, 2021. The testholes were drilled with a 150 millimetre (mm) diameter solid stem auger using a truck mounted drill rig from Canadian Geological Drilling Ltd. One 25-milimetre (mm) diameter polyvinyl chloride (PVC) monitoring well was installed in testhole TH21-03 to monitor groundwater conditions.

Testholes were logged in the field and the soil was classified according to the Modified Unified Soil Classification System (MUSCS) for soils. Standard Penetration Tests (SPTs) were conducted at approximate 1.5 m intervals in all drilled testholes. Disturbed samples from all testholes were collected at regular intervals for laboratory testing. Undisturbed Shelby tube samples were also collected. Testhole Logs along with an Explanation of Field and Laboratory Test Data and the MUSCS for soils are provided in **Appendix B**.

Testhole locations were surveyed by the CoE after completion of drilling on March 05, 2021. The location of each testhole is presented on **Figure 1** in **Appendix A**. **Table 2-1** below summarizes the details pertaining to each testhole.

Table 2-1: Summary of Testhole Details

Testhole	Location	Depth (mBGS)	Coordinates Northing[1]	Coordinates Easting[1]	Elevation[1] (mASL)	Monitoring Well Installed (Y/N)
TH21-01	10828 – 71 Avenue	15.5	5930660.603	32463.782	668.975	N
TH21-02	10824 – 71 Avenue	6.5	5930650.623	32471.517	668.859	N
TH21-03	10820 – 71 Avenue	14.8	5930641.004	32479.672	669.002	Y

[1] Coordinates and elevation surveyed by CoE and presented in NAD83 3TM.
Elevations in this table are provided as Metres Above Sea Level (mASL).

2.6 Laboratory Testing Program

Soil samples collected during the site investigation were tested in AECOM's materials testing laboratory in Calgary, Alberta. The laboratory testing included the determination of moisture contents, Atterberg Limits, and grain size distributions, unconfined compression testing, and soil chemical properties. For soil chemical testing, selected samples were sent to ALS Environmental in Calgary for determination of pH, soluble sulphates, resistivity, and chloride contents. The test results are shown on the testhole logs, and are presented separately in **Appendix C**. Laboratory testing consists of the following:

Table 2-2: Summary of Laboratory Testing

Laboratory Test	Number of Tests	Data Location
Moisture content determination	47	Testhole Logs and Appendix C
Atterberg limits determination on selected soil samples	3	Testhole Logs and Appendix C
Grain Size Analysis on selected samples	2	Testhole Logs and Appendix C
Soil Chemical Testing	2	Testhole Logs and Appendix C
Unconfined Compression Testing	2	Testhole Logs and Appendix C

3. Subsurface Conditions

3.1 Organic Clay

Organic clay was encountered at ground surface in all testholes. The thickness of the organic clay varied from 400 mm to 500 mm. The organic clay was silty, contained trace fine grained sand, and trace gravel. The organic clay was also noted to contain trace rootlets and occasionally contained some silt laminations. The organic clay was moist and black in colour. Three moisture content tests were completed on the organic clay, and the results varied from 24.7% to 29.7%.

3.2 Clay (Lacustrine)

Clay was encountered below the organic clay in all testholes. The thickness of the clay varied from 5.2 m to 6.1 m. The clay was silty and contained trace fine grained sand. Some fine-grained sand and silt laminations were noted in the clay. The clay was noted to be of high plasticity, moist to wet, oxidized, and brown in colour. SPT N-values for the clay ranged from 4 to 9 blows per 300 mm of penetration, indicating the clay was soft to stiff. The moisture content of the clay varied from 27.6% to 38.3%. One Atterberg limits test was completed on the clay, and the results are summarized in **Table 3-1**.

Table 3-1: Summary of Atterberg Limits Test for Clay Lacustrine

Testhole	Sample Number	Depth (mBGS)	MUSCS	Moisture (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
TH21-02	6	4.55 – 5	CH	37.2	65.8	23.6	42.2

3.3 Clay Till

Clay till was encountered in testholes TH21-01 and TH21-03. The thickness of the clay till varied between 1.35 m to 3.7 m. The clay till was sandy to containing some sand, silty to containing trace silt, and occasionally contained trace gravel. Fine grained sand and silt laminations were noted within the clay till. The clay till was also noted to be of low to high plasticity, oxidized, moist, and either dark brown, brown or grey in colour.

SPT N-values for the clay till ranged from 5 to 19 blows per 300 mm of penetration, indicating the clay till was firm to very stiff. Moisture contents were determined on 10 clay till samples and the results varied from 17.7% to 33.2%. Two Atterberg Limits and two grain size analyses tests were completed on the clay till and the results are summarized in **Table 3-2** below.

Table 3-2: Summary of Atterberg Limits Test and Grain Size Analyses Test Results for Clay Till

Testhole	Sample Number	Depth (mBGS)	MUSCS	Moisture (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
TH21-01	10	6.85 - 7.0	CI-CL	22.8	0.5	25.9	46.3	27.3	31.1	15.6	15.5
TH21-03	11	7.55 - 7.7	CH	24.4	0.0	12.8	32.3	54.9	54.7	15.7	39.0

One unconfined compressive strength test was completed on the clay till and the unconfined compressive strength (Q_u) was 103.0 kPa and undrained shear (S_u) was 51.5 kPa.

3.4 Silt

Silt was encountered below the clay in testhole TH21-03 and was 1.15 m thick. The silt contained some clay and trace fine grained sand. The silt was noted to be oxidized, wet, and brown in colour. One moisture content was determined on the silt and was 33.1%.

3.5 Sand

Sand was encountered below the silt layer and within the clay till layer in testhole TH21-03. The thickness of the sand layer varied from 150 mm to 400 mm. The sand was silty to containing some silt and contained trace clay. The sand was noted to be fine grained, oxidized, saturated, and brown in colour. One SPT was completed in the sand layer and was 15, indicating the sand was compact. Two moisture contents tests were completed on the sand and were 17.6% and 23.4%.

3.6 Clay Shale

Clay shale was encountered below the clay till in testholes TH21-01 and TH21-03. The thickness of the clay shale layer varied between 2.2 m and 3.7 m. The clay shale layer thickness may be greater than 3.7 m, as the clay shale extended to the termination depth of TH21-01. The clay shale was silty and contained trace fine grained sand. The clay shale was also noted to contain some sand and silt laminations. The clay shale was noted to be of medium plasticity, poorly lithified and noted to be damp and grey in colour. SPT N-Values for the clay shale varied between 62 and 124 blows per 300 mm of penetration, indicating the clay shale was very hard. Moisture contents were determined on eight samples and varied between 17.1% and 26.1%. One unconfined compressive strength test was completed on the clay shale. The unconfined compressive strength (Q_u) was 204 kPa and undrained shear strength (S_u) was estimated to be 102 kPa.

3.7 Sandstone

Sandstone was encountered below the clay shale in testhole TH21-03 and was 0.7 m thick. The thickness of the sandstone layer may be greater than 0.7 m, as the sandstone extended to the termination depth of TH21-03. The sandstone was silty, contained trace silt laminations, was fine grained, and poorly lithified. The sandstone was damp and grey in colour. One SPT was completed in the sandstone and the SPT N-value was 95 blows per 300 mm of penetration, indicating the sandstone was very dense. One moisture content was determined on the sandstone sample and was 15.2%.

3.8 Groundwater

Groundwater levels were measured upon completion of drilling on February 19, 2020 and 12 days after on March 3, 2021. The results of the groundwater measurements are summarized in **Table 3-3**.

Table 3-3: Summary of Groundwater Measurements

Testhole	Testhole Elevation (mASL)	Depth of Standpipe (mBGS)	Upon Completion of Drilling February 19, 2021 (mBGS)	Groundwater Depth During Monitoring Event on March 3, 2021 (mBGS)	Groundwater Elevation During Monitoring Event on March 3, 2021 (mASL)
TH21-01	668.98	-	Trace groundwater at bottom of testhole	-	-
TH21-02	668.86	-	Trace groundwater at bottom of testhole	-	-
TH21-03	669.00	14.8	8.1	6.3	662.70

- No monitoring wells installed in testholes TH21-01 and TH21-02. The monitoring well in TH21-03 was screened in bedrock only.

Measured groundwater depths are also shown on the testhole logs in **Appendix B**. It should be noted that the groundwater levels in **Table 3-3** are relatively short term and may not be representative of stable groundwater conditions. Groundwater levels can vary in response to seasonal factors and precipitation. The groundwater conditions at the time of construction may vary from those recorded in this investigation.

Decommissioning of the standpipe piezometers was not included in the scope of this investigation. It is recommended that this standpipe be decommissioned in compliance with industry standards during construction.

3.9 Frost Susceptibility

The surficial soils encountered at this site consist of clay (CH) and organic clay (OL). The qualitative frost susceptibility of a soil is typically assessed using guidelines developed by Casagrande (1932) on the basis of the percentage by weight of the soil finer than 0.02 mm and plasticity index. This classification system has been adapted by the U.S. Army Corps of Engineers and the Canadian Foundation Engineering Manual (CFEM, 2006). Soils are classified as F1 through F4 in order of increasing frost susceptibility and loss of strength during thaw. The soil units encountered at the sites and their frost group classifications are summarized in **Table 3-4**.

Table 3-4: Frost Susceptibility

Soil Unit	USC	Finer than 0.02 mm (%)	Plasticity Index (%)	Frost Group
Clay	CH, OL	-	-	F3 - F4

Generally, the surficial soils at this site were classified in the F3-F4 frost group, which indicates the surficial soils are highly susceptible to frost.

3.10 Frost Penetration

The clay deposits at this site are highly susceptible to frost action. The depth of frost penetration for soils can be determined using the CFEM (4th Edition) guidelines. The depth of frost penetration for a 30-year return period corresponds to an estimated Design Freezing Index of 1996 degree Celsius days (°C-days). The depth of frost penetration for the soil encountered at the Queen Alexandra Housing site is summarized in **Table 3-5**.

Table 3-5: Frost Penetration Depth

Soil Unit	Frost Penetration Depth (m)
Clay	2.4[1]

[1] The Frost Penetration depth may be reduced by using insulation as designed by the insulation supplier or manufacturer.

The frost penetration depth provided above is based on a uniform soil type with no insulation cover. In areas covered with turf or snow cover, the depth of frost penetration will be less. Conversely, if well graded granular backfill is used, the depth of frost penetration will be greater. The depth of frost penetration is dependent on the in-situ moisture content, relative density, grain and pore sizes, and permeability of the soil. As a result, frost penetration is expected to vary across the site as the subsurface materials and temperatures vary. The depth of frost penetration will also increase in snow-cleared paved areas such as roads.

3.11 Soil Chemical Testing

Chemical testing was conducted on select samples to determine pH, resistivity, soluble chloride concentration and total sulphate ion content. The degree of corrosiveness and corrosion potential for sulphate attack are provided in **Table 3-6** below in accordance with the Handbook of Corrosion Engineering (Roberge, P. R., 2000) and the Canadian Standards Association Guidelines (CSA, 2018).

Table 3-6: Soil Chemistry Summary

Testhole	Sample Number	Depth (mBGS)	Resistivity (ohm-cm)	Chloride Concentration (mg/L)	Total Sulphate Ion Content (%)	pH	Corrosion Potential	Sulphate Attack
TH21-01	4	2.35 - 2.5	400	<20	0.250	7.80	Extremely Corrosive	Severe
TH21-03	13	9.05 - 9.2	1100	<20	0.086	8.10	Highly Corrosive	Low

Based on the above test results, the degree of corrosivity is expected to be highly corrosive to extremely corrosive at this site. The potential for sulphate attack in concrete is expected to be low to severe at this site.

3.12 Seismic Site classification

Based on criteria from the National Building Code of Canada, corrected SPT-N values and undrained shear (S_u) strength can be used to determine the seismic classification of a site. The seismic classification of a site is rated from A through F, in order of increasing seismic sensitivity. Sites classified in the A group consist of hard rock, while sites classified in the E group consist of soft soils. Testhole TH21-03 was used for the seismic site classification. The soil stratigraphy at this site consisted of clay, clay till and clay shale with occasional sand or silt layers. Testhole TH21-03 was not advanced to a depth of 30 mBGS; therefore, the following assumptions were made regarding the soil stratigraphy to determine seismic classification.

- The sandstone in testhole TH21-03 continues to a depth of 30 mBGS.
- The minimum SPT of 62 blows per 300 mm of penetration in Testhole TH21-01 was representative of the bedrock encountered.
- SPT tests spanning 2 layers will be representative of where a majority of the SPT test spans

The proposed project location is generally rated in the D category for seismic classification, indicating moderately high sensitivity to seismic activity. The site seismic classification could be confirmed with more certainty from a detailed geotechnical investigation with a testhole extending to at least 30 mBGS and completing seismic cone penetrations tests to measure the shear wave velocity versus depth. If highly weathered bedrock is present, a 30 mBGS testhole to confirm site seismic classification may not be necessary.

4. General Construction Recommendations

4.1 Site Suitability

The site is considered suitable for the proposed housing building provided that the geotechnical risks identified during this investigation are understood and recommendations in this report are followed. It is understood that the proposed housing building could reach a height of up to 28 metres. Shallow foundations founded within 8 mBGS at this site may be problematic for heavily loaded structures and the proposed housing building would likely need to be supported on deep foundations rather than shallow foundations. Based on the soil conditions encountered during this geotechnical investigation, the primary geotechnical risks with the proposed site include:

- The near surface clay encountered at this site was soft to firm, with the moisture content of this clay varying from 27.6% to 38.3%. This moisture content is considered high relative to the native clay till typically found in the Edmonton area. This clay will have a low bearing capacity if certain shallow foundation types are selected and will be prone to excessive consolidation settlement if a heavily loaded structure is constructed on this clay.
- The presence of wet to saturated sandy and silty soils was noted in TH21-03. These soil conditions typically are prone to sloughing. For mid to high rise housing construction, a deep excavation is typically required. Sloughing soils within the deep excavation may result in ground loss and induce settlement of nearby infrastructure if not controlled during construction.
- The presence of wet to saturated sand and silty soils may be problematic during installation of deep foundations, such as cast-in-place piles. (If continuous flight auger piles are used, sloughing soils may not be a problem.)
- The presence of high plasticity clay within the subsurface was noted in TH21-03, which may prone to swelling and shrinkage if exposed during construction for foundation placement.
- The presence of highly frost susceptible soils due the naturally high moisture content of the surficial clay.
- Potential for severe sulphate attack on concrete walls and foundations embedded within the ground surface.
- Perched groundwater was noted within the subsurface, which will require dewatering during construction and subsurface drainage may be required. Groundwater depth was measured at 6.3 mBGS and trace seepage was noted at 4.6 mBGS in TH21-02.
- Fill may have been placed at this site during demolition of the previous structure. This fill may include poorly compacted soil or include debris and deleterious materials, which is not suitable for a foundation base.

In order to mitigate the risks, the recommendations provided in this section should be followed. It should be noted that the recommendations provided in this report are preliminary and are subject to review and revision during the detailed design phase. At the time of submission of this geotechnical investigation report, specific details of the housing project such as building type, building size, foundation type, foundation elevation, and building loadings have not been yet known. Once this information is confirmed, a detailed geotechnical investigation is recommended. This section provides general construction recommendations. Foundation and pavement recommendations are discussed in **Sections 5** and **Section 6** respectively.

4.2 Site preparation – Building Area

Generally, site preparation should begin by removing all organic material and clay fill, as well as any deleterious material (such as fill debris, high plasticity clay) within the building plan area, exposing the underlying inorganic native clay. Following the initial site stripping and cutting to grade or foundation elevation, the exposed subgrade should be inspected by geotechnical to determine if competent foundation base is present. Based on the information from this investigation, it should be anticipated that a scarification depth of at least 150 mm will be required assuming a foundation depth at an elevation of 661 m or below. The scarified 150 mm layer below the foundation base should be moisture conditioned to between 0 and 2 percent above the optimum moisture content (OMC) and recompacted to 98 percent of the Standard Proctor maximum dry density (SPMDD). Following compaction, the areas should be proof-rolled to identify any loose or soft areas. Any soft areas should be over-excavated and backfilled and compacted to 98 percent SPMDD using general engineered fill of low to medium plasticity. Imported fill used for construction should be approved by the geotechnical engineer of record.

After completion of subgrade preparation, the building area should be backfilled using either a granular fill or imported low to medium plasticity clay fill. The fill material should be moisture conditioned as required and compacted to 98 percent SPMDD and placed in lifts of 150 mm compacted thickness.

Full-time monitoring will be required by experienced geotechnical personnel to ensure that suitable fill material is placed to the proper moisture content and compaction standards within the building area.

4.3 Trenching and Excavation

A deep excavation would likely be required if the proposed housing project will be a mid to high-rise structure. All excavations should be in accordance with the provisions of the Occupation Health and Safety Regulations (OHS). The excavation walls should be sloped or adequately shored. Given the surrounding developments at this site, shoring will likely be the methodology implemented to ensure a safe excavation. The appropriate side slopes that will be required will depend on the soil type, depth of excavation, drainage method, the amount of groundwater seeping into the excavation, and the time interval the excavation is left open.

The Alberta Occupational Health and Safety code (Section 442) classifies soils into three groups:

a) Hard and compact – hard in consistency, very dense, appears to be dry, no signs of water seepage, can be penetrated only with difficulty by a small, sharp object, and is extremely difficult to excavate with hand tools.

b) Likely to crack or crumble – has been excavated before, stiff in consistency, compact, damp appearance, signs of water seepage, can be penetrated with moderate difficulty with a small sharp object, and moderately difficult to excavate with hand tools.

c) Soft, sandy or loose – firm to very stiff in consistency, loose, appears to be wet, can be easily excavated with hand tools, becomes unstable when disturbed.

The OHS indicates that if an excavation contains more than one soil type, the soil type with the least stability will govern. Based on the testholes from this geotechnical investigation, the soils encountered at the site are classified as soft, sandy or loose. Part 32 of the Alberta OHS code indicates that excavations with this soil type must have slopes of the excavation sloped from the bottom of the excavation at an angle of not less than 45 degrees measured from the vertical. However, based on AECOM's experience with temporary cut slopes, the OHS code guidelines for sloped excavations may be too steep in certain situations. AECOM recommends that temporary cut slopes within excavations of less than 3.0 m in depth within clay or sand have side slopes cut no steeper than 1.5H:1V. Temporary cut slopes exceeding 3.0 m in excavations of up to a maximum depth of 5.0 m within clay or sand should have side slopes cut no steeper than 2.0H:1V. Excavations exceeding 5.0 m in depth should have a geotechnical slope analysis be completed to determine a safe slope inclination. Flatter short term cut slopes may be required in zones where groundwater seepage is encountered. Alternatively, shoring may be implemented if the excavation cannot be sloped.

If the excavation for the building construction will be sloped, the slopes should be checked regularly for signs of sloughing, especially if loose sand pockets are observed or after inclement weather conditions. It should be noted that sand and silt layers/pockets were encountered within the clay till in this investigation. The amount of time an excavation is left open should be minimized as stability decreases over time. If there are signs of movement, the side slopes should be unloaded by benching the upper portion of the crest of the slope to relieve overburden pressure. The temporary cut slopes should also be protected against surface runoff and heavy rainfall. Small earth falls from the side slopes are a potential source of danger to workers and must be guarded against.

Existing underground utilities in the excavation area should be exposed by hand digging or hydro-vacuumed. No mechanical excavation should be undertaken within 1 m of anticipated location of existing utilities.

Fill should only be placed over dry, clean, stiff, unfrozen soils. The site soils are susceptible to softening and deterioration if left exposed in an excavation; therefore, traffic on the excavation base should be minimized, and construction should commence immediately after the excavation is complete. The time the excavation is left open should be minimized.

Temporary surcharge loads, such as construction materials or excavated soil and spoil piles, should not be allowed within 1.5 m or a distance equal to the depth of the excavation, whichever is greater, of an unsupported excavated face. Vehicles delivering materials should be kept back from faces by at least 3.0 m or a distance equal to the depth of the excavation, whichever is greater, of an unsupported excavated face.

The method of excavation and safe support of excavations, selecting suitable slopes for excavations, selecting temporary shoring system, protection of the existing infrastructure and maintaining stability of the excavation slopes are the responsibility of the contractor.

4.4 Dewatering

Perched groundwater was measured during the geotechnical investigation to be at 6.3 mBGS. It should be noted that groundwater typically varies in response to seasonal factors and precipitation. The groundwater conditions at the time of construction may vary from those recorded in this investigation. Groundwater during construction will be encountered during construction by seepage from wet sand and silt seams and pockets through clay and clay till layers. Groundwater accumulations should be handled by sumps and wells, or combination of these methods such that water can be pumped away.

The contractor is responsible for temporary dewatering of the excavation during construction. The contractor will be responsible for maintaining stability of the slopes or shoring system as well as protection of any existing infrastructure located near the temporary excavations.

4.5 Suitability of Existing Soil for Fill

The excavation for the housing building foundations and construction of below grade elements will result in an excess of soil. Generally, the soil excavated from the footprint of the building will include organic clay and high plasticity clay and clay till. The surficial organic clay at this site is not considered suitable for use for fill. The existing high plasticity clay is also not considered to be suitable for establishing site grading and backfilling. This clay is excessively moist and will be difficult to compact. It is recommended low to medium plasticity clay fill be imported for grading and backfill.

The imported soil used for fill should be compacted to 98% SPMDD, and within ±2% of the OMC. Lifts of backfill material should not exceed 150 mm in compacted thickness. It is recommended that fill material be reviewed and inspected by a qualified geotechnical engineer during construction.

4.6 Structural Fill Placement

Structural fill should be used under foundations, or any other settlement sensitive structures. Structural fill should consist of well-graded crushed gravel with less than 10% fines (silt and clay), and a maximum particle size of 20 mm.

The structural fill should be compacted to 100% of the SPMDD, and within ±2% of the OMC. Lifts of backfilled material should not exceed 150 mm in compacted thickness. The compacted lift thickness may be increased to 200 mm depending on the quality of structural fill (low fines). This increase in lift thickness should be approved by the geotechnical engineer of record during construction. The structural fill should extend on each side of the foundation a minimum distance of 500 mm.

Structural fill should comply with the CoE Designation 3, Class 20, or approved equivalent. The gradation for the Designation 3, Class 20 is provided in **Table 4-1** Below.

Table 4-1 Recommended Gradation for Structural Fill (City of Edmonton, Complete Streets Design and Construction Standards, Aggregate Designation 3, Class 20)

Metric Sieve (mm)	Percent Passing by Mass (%)
20.0	100
16.0	84 - 95
12.5	60 - 90
10.0	50 - 84
5.0	37 - 62
2.0	26 - 50
1.25	19 - 43
0.63	14 - 34
0.40	11 - 28
0.315	10 - 25
0.160	6 - 18
0.080	2 - 10

4.7 Utility Installation

Utility services required for this housing building should be installed at a minimum depth of 2.4 mBGS to protect against frost. If utilities are founded within the frost penetration depth, insultation should be used to protect the utilities against frost. All utility trenches should be backfilled with low to medium plastic clay or clay till, as fine-grained soils offer better frost protection than granular soil.

5. Preliminary Foundation Recommendations

The soil conditions commencing at depths of 2 to 8 mBGS are generally favorable for pile foundations. However, the conditions are complicated by random occurrences of silt and saturated sand layers within the clay and clay till. The feasible pile types that could be considered include straight shaft cast-in-place piles and driven steel piles; however, the suitability of driven steel may not be feasible due to vibrations during pile installations. The vibrations may affect and damage the existing nearby structures at this site. Continuous flight auger-cast piles (CFA) may also be considered for the site. Based on the test borings, belled concrete piles are also considered to be suitable at the site, due to the presence of very hard clay shale at approximate depths of 12 mBGS and 12.5 mBGS. Construction of straight shaft concrete piles and belled piles will need to incorporate contingencies for proper installation including temporary steel casings, groundwater handling, and concrete by tremie methods.

Shallow footings and raft foundations could be feasible for lightly and moderately loaded structures, respectively, where the wet/soft clay is removed and the structure founded on competent clay till below 7 mBGS to 8 mBGS. The feasibility of shallow footings or raft foundations is expected to be limited to non-critical structures where some settlement and/or differential settlement could be tolerated. With the requirements of providing a sufficient soil cover for frost penetration for shallow and raft foundations, relatively deep excavations, dewatering and concrete form works, pile foundations are likely to be more cost effective than footings or rafts.

The final selection of foundations for the proposed housing building should be determined when the building type, building size, and foundation elevations are determined, and based on results from a detailed site investigation.

5.1 Raft Foundations

5.1.1 General

If raft foundations are selected to construct the housing building, it is recommended that raft foundations be founded at an elevation of at least 661 m (8 mBGS) or below. Raft foundations may be designed using a factored ultimate bearing capacity and subgrade reaction modulus values summarized in **Table 5-1.**

Table 5-1: Bearing Capacity and Subgrade Reaction for Raft Foundation

Raft Foundation Base Elevation (m)	Ultimate Bearing Capacity (kPa)	Factored Ultimate Bearing Capacity (kPa)[1]	Subgrade Reaction Modulus (kN/m^3)
661.0 - 653.5	250	125	10,000

[1] A resistance factor of 0.5 is applied Ultimate Limits State design

The total estimated settlement is expected to be less than 30 mm if the applied load on the raft does not exceed 125 kPa. For serviceability limits states design, the total settlement is expected to be less than 25 mm if the applied load does not exceed 120 kPa and subgrade preparation recommendations provided in this report are followed, with a minimum scarification depth of 150 mm. A major portion of the total settlement of the raft foundation will be due to the recompression of the base heave which would occur during the excavation. This settlement will mostly occur through loading during construction rather than long term settlement if founded at this depth, assuming the proposed housing building will have a basement and walls.

Differential settlements are typically 50% to 75% of the total settlement noted above if rafts are supported with relatively uniform subgrade soil. Differential settlements could be highly variable if the building structure is supported on more than one type of subgrade soils.

Rafts foundation slabs should be adequately reinforced to allow the structure to settle uniformly and maintain structural integrity. Flexible connections should be provided from the structure to all connected piping to accommodate differential settlements. It is anticipated that where pipe connections enter the building, additional settlement will occur due to the greater thickness of overlying backfill. It is recommended that fillcrete or lean mix concrete be placed beneath the piping within the trench zone at the entrance into the building excavation.

5.1.2 Subgrade Preparation and Protection

The presence of high plasticity clay within the subsurface complicates the subgrade requirements at this site for raft building foundations. Generally, high plasticity clay below building foundations should be removed to eliminate the risk of consolidation settlement of the building that could occur over several years after construction of the building is complete. However, this may not be reasonably practical in some instances where the termination depth of high plasticity clay is significant, or the presence of high plasticity clay is variable below the building foundation. Additionally, the amount of high plasticity clay that is required to be removed will depend on the foundation elevations, size of the building, and the building load. Raft foundations are generally suitable foundation types when compressible or weak soils (high plasticity clay) are present within the subsurface. It is recommended the replacement depth of high plasticity clay be determined during the detailed design phase, when the building size, type, and elevation of the foundation is known. If the foundation base if founded within 5 to 6 mBGS, significant removal of high plasticity clay will be required. Depending on the building information, consolidation testing may be recommended during the detailed design phase to assist in the decision to determine the replacement depth of high plasticity clay below the foundation. The replacement depth should also be confirmed during construction with a geotechnical inspection from the geotechnical engineer of record.

Once the depth of the removal of high plasticity clay is determined, the base of raft excavation should be thoroughly cleaned of all loosened or disturbed soil prior to pouring concrete. The prepared subgrade should be inspected by a qualified geotechnical engineer to confirm that the prepared subgrade is acceptable prior to pouring mud slab concrete. After completion of the inspection, a lean concrete pad (mud slab) about 75 mm to 100 mm thick is recommended to protect the bearing surface from disturbance during the time period between excavation completion and casting of the raft foundation. High plasticity clay and clay shale has the potential to swell if left exposed to weather conditions. A mud slab is therefore highly recommended to protect the exposed subgrade from weather. If a satisfactory bearing surface cannot be attained, a 150 mm thick layer of well graded 20 mm minus crushed gravel should be placed and compacted to a minimum of 100% of SPMDD.

5.1.3 Subgrade Friction

Friction between the subgrade and raft foundation can be calculated as follows:

$F = \sigma_v \tan(0.66\,\phi')$

Where:

F = Friction between base of building and subgrade
σ_v = Applied vertical stress below the foundation base
ϕ' = Internal friction angle (use 25° for clay)

5.1.4 Buoyant Uplift

Raft foundations may be prone uplift forces. Based on groundwater observations completed on March 3, 2021, the depth of the groundwater table was 6.3 mBGS (Elev. 662.7 m). However, it is possible that higher short-term water levels will be encountered after periods of increased precipitation. It is therefore recommended for a preliminary design groundwater level of 4.6 mBGS (Elev. 664.4 m) be used. Trace seepage was encountered in TH21-03 during the investigation at this depth.

The magnitude of hydrostatic uplift forces applied to below grade structures should be calculated, assuming that the groundwater table is at 4.6 mBGS (Elev. 664.4 m). The hydrostatic pressure may be calculated using the following equation:

$P_w = \gamma_w H_w$

Where:

P_w = Hydrostatic pressure (kPa)
γ_w = Unit weight of water (9.8 kN/m^3)
H_w = Depth below top of water table (m)

Buoyancy forces should be determined using the following equation:

$U = \gamma_w V_s$

Where:

U = Hydrostatic uplift force (kN)
γ_w = Unit weight of water (9.8 kN/m^3)
V_s = Volume of structure below the groundwater table (m^3)

Buoyant uplift forces may be resisted by the mass of the structure, or by extending the base of the raft beyond the walls of the structure (assuming the housing building will have below grade basement walls), such that the mass of the soils above the projection are used to resist uplift forces.
If an extended base is considered, uplift resistance due to the weight of the soil above the raft foundation may be determined as follows:

$R_{ss} = AWH\gamma'$

Where:

R_{ss} = Total allowable resistance due to weight of soil (kN)
A = Perimeter of reservoir walls (m)
W = Width of projected base slab beyond reservoir walls (m)
H = Height between top-of-slab and ground surface (m)
γ' = Submerged unit weight of soil (kN/m^3)

Uplift resistance due to shearing through the soil may be assumed to have a triangular distribution as determined by the following equation:

$R_s = (k_o\gamma' d\tan\varphi')/FS$

Where:

R_s = Allowable shearing resistance (kPa)
k_o = Coefficient of earth pressure at rest (0.5)
γ' = Submerged unit weight of soil (kN/m^3)
d = Depth below final ground level (m)
φ' = Friction angle of backfill (assume 20° for cohesive fill and 30° for granular fill)
FS = Factor of Safety (minimum of 2.0)

5.2 Cast-in-Place (CIP) Concrete Piles

5.2.1 CIP Concrete Pile Design Parameters

Straight shaft drilled CIP concrete friction or end bearing piles are another foundation alternative considered suitable for the proposed housing building. The use of casing may be required for cast-in-place concrete piles due to presence of water bearing sand layers within the clay till and a silt layer overlying the clay till.

The ultimate capacity of straight shaft CIP concrete piles may be determined from the following equation:

$Q_u = q_s P_s L + q_t A_t$

Where:

Q_u = ultimate capacity of the pile (kN)
q_s = ultimate skin friction between the pile and soil (kPa)
q_t = ultimate end bearing (kPa)
P_s = perimeter of the pile section (m)
= π x d, where π is 3.14 and d is the diameter of the pile in metres
L = effective pile embedment length (accounting for depth of frost, height of fill, etc.)
A_t = cross sectional area of the pile (m^2)
= $\pi\ d^2/4$, where π is 3.14 and d is the diameter of the pile

For limit states design, a resistance factor of 0.4 should be applied on the ultimate pile load capacity to obtain the factored pile load capacity. For working stress design, a factor of safety of 2 and 3 should be applied on ultimate skin friction and ultimate end bearing, respectively, to obtain allowable skin friction and allowable end bearing.

The axial capacity of CIP piles may be determined using parameters provided in **Table 5-2** and the above equation.

Table 5-2: Ultimate Design Parameters for CIP Concrete Piles

Elevation (m)	Soil Type	Ultimate Skin Friction (kPa)[1]
669.0 - 666.7	Clay (within Frost Depth)	-
666.7 - 661.0	Clay, Clay Till	30
661.0 - 657.0	Clay Till	40
657.0 - 653.5	Clay Shale	100

[1]*A resistance factor of 0.4 should be applied to determine the factored ultimate skin friction in compression for limit states*

The pile design parameters in **Table 5-2** are considered applicable for downward (compressive) static loads. All piles should have a minimum diameter of 400 mm.

Belled piles (end bearing piles) may be founded a minimum 1.5 m within the clay shale (below a depth of 12 m). The ultimate bearing pressure at this depth can be taken as 1800 kPa. For Ultimate Limit States (ULS) design, a resistance factor of 0.4 should be applied to the ultimate bearing pressure for belled piles to obtain the factored end bearing pressure. The design may consider end bearing (for belled piles) in addition to shaft friction as provided above in order to determine the total pile capacity. However, the shaft friction should be neglected for a distance of one shaft diameter above the top of the bell.

It is recommended to use a bell diameter that is a minimum of 2 and a maximum of 3 times the shaft diameter. The ratio of the depth to bell base and bell diameter should be a minimum of 2.5.

Pile bells cannot be formed within a sloughing silt or sand layer. Therefore, in order to ensure adequate support for the roof of a bell where wet sloughing layers are encountered, the minimum distance from the underside of a sloughing layer to the top of the roof of a bell should be 0.6 m.

5.2.2 CIP Concrete Pile Design and Construction Recommendations

The subsurface stratigraphy at the site generally consists of clay, overlying clay till with occasional sand layers, underlain by clay shale at an approximate elevation of 657 m. The groundwater was recorded at 662.7 m, however, the water level is expected to fluctuate seasonally. The sand layers are expected to be saturated and slough into the pile installation holes. Due to the presence of wet and saturated sand and silt layers, sloughing of overburden soils should be expected in the pile installation hole; therefore, the contractor should be prepared to control seepage and sloughing and maintaining clean pile holes by using a full-length temporary casing. The casing should be properly seated on/into the clay shale at elevation 657 m to seal the pile hole and reduce seepage and sloughing. The overburden thickness at the pile locations may be variable; therefore, the contractor should have sufficient length of casing available on site.

The following recommendations should be considered when designing and constructing the CIP concrete piles:

- Skin friction should be neglected within either the zone of seasonal frost penetration to account for the effects of soil desiccation and frost heave or the depth of fill if present, whichever is greater. (Fill may have been placed at this site during demolition of the previous structure.)
- Piles should be founded at a sufficient depth to resist uplift pressures due to frost. An uplift adfreeze pressure of 65 kPa for fine grained soils frozen to concrete should be considered for the maximum frost penetration depth of 2.4 m. The minimum embedment depth to resist uplift due to frost will be a function of the pile shape, pile size and the applied dead load on the pile. For example, ignoring the effects of self-weight of the pile and applied dead load on the pile, a 400 mm diameter CIP concrete pile will require installation to approximately 8 mBGS to adequately resist uplift pressures due to frost.
- Shaft resistance of CIP concrete piles should be designed using the site-specific design parameters provided in **Table 5-2**.
- A minimum pile spacing of 3 times the shaft diameter is recommended for straight shaft piles.
- A minimum pile spacing of 3 times the bell diameter is recommended for belled piles. In addition, a minimum edge-to-edge spacing of about 0.5 m is recommended in the case of belled piles to reduce potential construction problems.
- Piles within three shaft diameters should not be drilled or poured consecutively within the same 48-hour period to allow the concrete in the adjacent piles to set.
- Concrete piles must be reinforced for the full length of the pile. Belled piles subject to uplift loads including frost should have reinforcement, which extends to the base of the pile.
- The contractor should be prepared to control seepage and sloughing and maintain clean pile holes. Temporary steel casing may be required to prevent excessive seepage and sloughing into the pile holes during excavation and pouring of concrete. Based on observations provided on the testhole logs, silt and sand lenses and corresponding seepage may be encountered at any depth. The contractor should bring enough casing to case the entire pile hole should the need arise.
- The contractor should evaluate means and methods to install/extract casing.
- The foundation contract should have provisions for lengthening the pile, casing, and steel cage if required due to site subsurface conditions.
- End bearing of CIP piles may only be considered if bases can be thoroughly cleaned of all loosened material and dewatered prior to pouring concrete. The base should be inspected by qualified personnel. End bearing will not be applicable if pile bases are not properly cleaned and inspected prior to placement of concrete.
- To avoid segregation of the concrete, a tremie tube should be used when placing concrete. The tremie tube should be watertight, and the outlet of the tremie tube should be at least 1 m below the concrete surface during pouring.
- Concrete should be poured immediately after drilling of the pile hole to reduce the risk of groundwater seepage and soil sloughing.
- Monitoring of the pile installation by qualified personnel is recommended to verify that the piles are installed in accordance with design assumptions. Inspection should be carried out before casting the pile.

5.2.3 Pile Caps

Pile caps and grade beams are usually required to transfer the loads onto the tops of the piles. If the bases of the pile caps and grade beams are located within the frost penetration depth, precautions should be taken to prevent heaving of the pile cap due to frost. The recommended construction procedure for reducing heave effect under the pile cap involves placement of crushable non-degradable void filler (such as Beaver Plastic Frost Cushion or equivalent) of at least 150 mm thickness under the pile cap. The pile should be designed to withstand the upward heave forces equal to the crushing strength of the void form.

The void form is not required if pile caps and grade beams are located with a minimum soil cover of 1.5 m along the exterior perimeter of heated buildings for protection against frost heave.

5.2.4 Lateral Loading

Vertical piles will be subjected to horizontal loads in addition to vertical loads; their lateral capacity should be checked by a proper analysis (i.e. LPile Analysis). Short term lateral loads may be imposed by construction, by seismic forces or by wind. Long term forces may be those acting on supports of an above ground conveyance structure at bends and intermediate supports.

Design of laterally loaded piles is generally governed by Serviceability Limit States limiting the top of pile movement to within tolerable limits.

Lateral load capacity of piles will depend upon the pile stiffness and geotechnical engineering properties of the native soil or fill material within the upper few metres of the pile. Lateral pile capacity can be determined using commercially available software such as LPile. The analysis using this software provides estimates of the lateral displacements, bending moments, shear forces and soil reaction along the depth of the piles, and it requires input pertaining to soil properties, pile properties, and applied loads on the pile.

Lateral pile capacity can also be calculated in structural analysis using horizontal subgrade modulus to determine spring constants along the depth of the soil. This assumes a linear relationship between load and displacement. The soil response is modelled by linear springs represented by the horizontal subgrade modulus (ks). The subgrade reaction modulus for lateral pile deflections should only be used when the expected pile deflection is less than 1% of the pile diameter, as recommended by the Canadian Foundation Engineering Manual (4th Edition). P (Static Soil Reaction) – Y (Pile Deflection) method may be used if larger deflections are expected for lateral static, cyclic or even transient loads. This section includes lateral pile capacities using the subgrade reaction method only.

If lateral deflections are the limiting factor in the overall pile design, it is recommended to conduct full-scale lateral pile load tests to verify the horizontal subgrade modulus value for this site.

For cohesive soils (clay, clay till and clay shale) k_s can be estimated using the following equation:

$k_s = 67\ S_u /D$

Where:

S_u = undrained shear strength of the soil (kN/m^2); and
D = pile diameter (m)

The undrained shear strengths to be used in determining the horizontal subgrade modulus (ks) were estimated based on field SPT test results and are summarized in **Table 5-3**.

Table 5-3: Undrained Shear Strength of Soil Units

Soil Type	Elevation (m)	Undrained Shear Strength, S_u (kPa)
Clay	669.0 - 661.0	20 to 40
Clay Till	661.0 - 657.0	60 to 80
Clay Shale	Below 657.0	200

5.2.5 Tension Loading

The piles will be subject to uplift forces due to frost heave, tensile forces due to lateral loading, overturning movements due to wind, etc. The piles should be designed to resist these uplift forces. The resistance to uplift will be provided by pile self-weight, applied dead loads, and uplift skin resistance. Factors such as seasonal frost depth, heating and insulation, and soil type should be taken into account while designing the pile against uplift.

The resistance to uplift may be calculated using ultimate skin friction parameters provided in **Table 5-2** of this report. A resistance factor of 0.3 should be applied on ultimate parameters to obtain factored uplift parameters. This resistance factor is in accordance with the CFEM (2006).

5.2.6 Frost design considerations for Cast-in-Place Piles

All foundations are expected to be for a heated structure. For piles that are placed outside the area of a heated building, some precautions should be taken to avoid frost heaving and frost jacking of piles. Frost heave on the underside of pile caps/grade beams and adhesion freezing forces (adfreeze) along the pile shaft and sides of pile caps/grade beams within the seasonal frost zone should be considered in pile design if founded within the frost depth. The proposed housing building will likely include a heated basement or below grade parkade. CIP piles will therefore likely be installed below the seasonal frost penetration depth. Assuming a pile length of at least 8 metres and pile diameter of 400 mm, adhesion freezing forces (adfreeze) may be neglected. However, this should be determined once the details of the housing building are known, such as the depth of the basement.

5.3 Grade Supported Floor Slab

If a grade supported floor slab is to be considered, recommendations for subgrade preparation have been provided in **Section 4.2**. The recommended subgrade preparation and the possible placement of low to medium plasticity engineered clay fill may still result in floor movements of approximately 15 to 25 mm or greater, depending on the depth and quality of fill placement and compaction. Using granular fill can reduce the floor movements. The use of high plasticity clay soil as engineered fill within the buildings is not recommended due to potential of swelling with increasing moisture content.

The above noted movements are typically gradual but can often results in floor cracking or distortion with time. This movement can be reduced by placement of low to medium plasticity clay fill or granular fill to provide more uninform subgrade condition and reduce the risk of slab differential movement.

The clay subgrade possesses a high potential for volume change if allowed to remain in contact with water for extended periods of time. Measures should be taken to ensure water is not allowed to pond on the subgrade during and after construction as detrimental swelling may occur. It is also recommended that the exposed subgrade is not allowed to dry out during construction prior to slab placement.

Slab-on-grade floors should rest on at least 300 mm thick of compacted structural fill as specified in **Section 4.6**. The structural fill should be compacted to 100 percent of SPMDD and placed in lifts not exceeding 150 mm in compacted thickness. For the structural design purposes the compacted structural fill and clay soils underneath a subgrade modulus of 20 MPa/m can be used.

The floor slab should be reinforced along with proper joints to be provided to prevent shrinkage cracks.

If possible, water lines should not be placed beneath slab-on-grade floors. Wastewater lines should be of rigid plastic with cemented joints. Wastewater lines with butt joints and flexible rubber connections should not be permitted.

Non-load bearing partitions resting on slab-on-grade floors should be designed such that floor movements can be accommodated. An allowance of 15 mm - 30 mm should be considered for the swelling potential of the underlying clay soils. For interior walls that do not have some flexibility, consideration should be given to supporting these walls on independent foundations.

5.4 Lateral Earth Pressures

Buried structures resisting lateral earth pressures such as foundation walls and below grade elements should be designed to resist lateral earth pressures in at-rest conditions. The earth pressure acting on below grade structures depends on many factors including the structure stiffness, the construction methodology, the extent and direction of any movement of the soil, the nature and extent of backfill, and the groundwater conditions. For rigid walls such as foundation walls, the at-rest earth pressure co-efficient (K_o) should be used.
The lateral earth pressure can be calculated using the following equation:

$$P = K(\gamma' H + q) + \gamma_w H_w$$

Where:

P	=	lateral earth pressure (kPa);
K_o	=	at rest coefficient of earth pressure using K_o from **Table 5-4**;
γ	=	bulk unit weight of backfill free draining gravel (21 kN/m^3);
H	=	depth below final design grade (m);
q	=	any surcharge pressure at ground level (kPa);
γ'	=	effective unit weight of backfill soil below groundwater level (11.2 kN/m^3);
H_w	=	height of groundwater above the foundation base to top of wall (m); and,
γ_w	=	unit weight of water (9.81 kN/m^3)

Compaction of backfill material behind walls should be done in a controlled manner to avoid higher earth pressures against the sides of the foundation wall. A minimum surcharge of 12 kPa should be included in the design to account for compaction induced pressures.

Where traffic or other live loads may operate near the rigid wall, the horizontal pressure due to the live load should be superimposed on the static earth pressures.

The equation above assumes the use of native or imported granular fill compacted to approximately 95% of SPMDD and horizontal ground behind the buried wall. If the ground surface slopes away from the wall, design coefficient of at rest earth pressure should be re-evaluated.

The parameters required for calculation of the lateral earth pressure assuming horizontal ground surface behind the wall are summarized in **Table 5-4**.

Table 5-4: Lateral Earth Pressure Coefficients for the Foundation Walls

Backfill Type	γ (kN/m^3)	Friction Angle, Φ	Coefficient of at-rest earth pressure, K_o
General Engineered Fill (Low to Medium plasticity clay)	18	25	0.577
Structural Fill	21	34	0.593
Clay	18	25	0.577
Clay Till (Low to Medium Plasticity)	18	26	0.562
Clay Till (High Plasticity)	18	23	0.609

5.5 Subsurface Drainage

If foundations are founded below the groundwater table, placement of a sub-drain (weeping tile system) below the base of foundation will be required to provide drainage and reduce potential adfreeze forces. The design groundwater level should be taken as 4.6 mBGS (Elev. 664.4 m). The design water level should be confirmed during the detailed design phase with additional groundwater readings over several different seasons. The drainage system must maintain the groundwater level at or below the base of the foundation.

Permanent structures founded below the groundwater table should either be designed to resist the potential hydraulic uplift pressures, or alternatively should have a subsurface drainage system below the foundation or around the perimeter walls to drain water away from the foundations.

A higher groundwater table would be expected during spring and upon melting of snow. A subsurface drainage system may be provided to prevent buildup of hydrostatic uplift pressures on the base of the foundation during periods of high groundwater. The recommended approach for permanent subsurface drainage where required is to provide a gravel drainage layer around the perimeter walls and below the base of foundation to collect water. The subgrade should be sloped to drain subsurface water towards permanent drains and sumps. The collected water should be directed to the site drainage system or to a sump for collection and discharge. A minimum thickness of between 300 mm and 1000 mm of free draining gravel with less than 5% passing sieve No. 200 should be used under the base of foundations and behind the walls, respectively. It is recommended that a non-woven geotextile be placed directly over the prepared subgrade and at the interface around perimeter wall drainage layer to provide separation between the subgrade and drainage gravel layer and to prevent clogging of the gravel. It is recommended that further monitoring of groundwater levels to be carried out after completion of the site grading works to measure the depth of groundwater below the finished grade.

5.6 Sulphate Attack and Corrosion

The potential for sulphate attack on concrete in contact with subsurface soils or groundwater at this site was rated as low to severe (**Table 3-6**) at this site. It is highly recommended additional sulphate testing be completed on imported fill used for construction at this site. As per **Table 3** of CSA23.1-14, Portland cement concrete in contact with the soil at this project site would fall under exposure Class (S-2). "S-2" has a severe degree of exposure to sulphate attack and would require the use of CSA Type HS or HSb or HSLb or HSe (regular or blended high sulphate-resistant hydraulic cement). Following the guidelines of **Table 2** of CSA A23.1-14, we recommend concrete should have a maximum water to cementing materials ratio of 0.45, minimum compressive strength of 32 MPa, and incorporate appropriate air entrainment.

Table 5-5: Requirements for Concrete Subjected to Sulphate Attack

Class of Exposure	Degree of Exposure	Water-Soluble Sulphate (SO4) in Soil Sample, %	Sulphate (SO4) in Groundwater Samples, mg/L	Minimum Specified Compressive Strength, MPa	Maximum Water / Cementing Materials Ratio	Cementing Materials to be Used	Air Content Category
S-1	Very Severe	Over 2.0	Over 10,000	35 at 56 days	0.40	HS, HSb, HSLb or HSe	1 or 2
S-2	Severe	0.20 – 2.0	1,500 – 10,000	32 at 56 days	0.45	HS, HSb, HSLb or HSe	1 or 2
S-3	Moderate	0.10 – 0.20	150 – 1,500	30 at 56 days	0.50	MS, MSb, MSe, MSLb, LH, LHb, HS, HSb, HSLb, HSe	1 or 2

The recommendations stated above for the subsurface concrete may require further addition and/ or modifications due to structural, durability, service life, or other considerations which are beyond the geotechnical scope.

Measured resistivity value of the soil was 400 ohm-cm and 1100 ohm-cm as shown in **Table 3-6**, which indicates the subsurface soil is expected to be extremely corrosive to highly corrosive. It is therefore recommended that all metals, if any, in contact with subsurface soils should be designed for a corrosive environment.

5.7 Radon Gas Mitigation Recommendations

The National Building Code (2019 Alberta Edition) has requirements for Radon gas control for newly constructed buildings. Radon gas is a radioactive gas that originates from the ground surface and poses several health risks to humans if exposed to it in high concentrations. Radon gas may travel through bedrock, soil, and groundwater.

Radon gas emission from the surface is very common throughout various regions of Canada but is especially common in Alberta. The City of Edmonton is located in an area with a high hazard rating for Radon gas potential. It is therefore highly recommended that the recommendations from this report and the National Building Code be followed to limit the amount of Radon gas that is able to enter the proposed Queen Alexandra Housing buildings in the project area.

Radon gas may enter buildings by various routes, but primarily enters buildings through the foundations or floor slabs of a building. In particular, Radon gas may enter through openings or cracks in the foundations, conduits or pipes, sumps, or through windows and doors. The following requirements were outlined in the building code to reduce the amount of Radon gas entering the building foundations:

- A Polyethylene soil gas barrier is required under the slab between the ground and the building
- The Slab perimeter must be sealed to the walls

- All penetrations through the slab must be sealed
- Granular fill and perforated pipes are required underneath the slab of the building
- A rough-in for Radon extraction to either performance or prescriptive requirements must be installed

For radon gas collection systems located below floor slabs (non-grade supported), it is recommended that the radon gas collection be surrounded by at least 100 mm thick washed gravel, as specified in **Table 5-6**.

Table 5-6: Recommended Gradation for Radon Gas Collection (City of Edmonton, Complete Streets Design and Construction Standards, Aggregate Designation 6, Class 20)

Metric Sieve (mm)	Percentage Passing by Mass
20	100
14	90 to100
10	45 to 75
5.0	0 to 15
2.5	0 to 5

It is also recommended that a non-woven geotextile filter fabric be placed at the interface between the granular fill and the subgrade to prevent migration of fines within the granular fill.

The above gravel is not intended for structural fill or to be used as a levelling course base for floor slabs. In situations where slab on grade or grade supported floor slabs are used, the structural and building designer will need to design the radon gas collection system to prevent loadings being placed directly on the radon collection system.

5.8 Surface Site Drainage

The final site grade should be properly graded to direct water away from the building and building foundations. A minimum grade of between 2% and 3% should be maintained around the building structure. Ponding of water near building foundations may result in subgrade softening and instability/failure of the overlying structure. Additionally, excess moisture near the building may result in frost heave.

6. Pavement Recommendations

The Queen Alexandra Housing project may consist of light-duty and heavy-duty pavement structures. At the time of writing this report, the preferred pavement structure type and anticipated traffic loading has not yet been known. This section includes recommendations for a light-duty and a heavy-duty pavement structure.

6.1 Subgrade Preparation – Pavement Area

Subgrade preparation at this site is recommended prior to placement of gravel and asphalt pavement. Subgrade preparation should consist of stripping all organic material, and frozen subgrade from the existing grade to expose a competent unfrozen bearing stratum. Other soft, excessively moist, or deleterious materials should be removed as well. The near surface soil encountered during the investigation includes organic clay and high plasticity clay. This type of clay is poor for pavement structures, as these soils have the potential to swell and are typically more frost susceptible.

Following the stripping of the surficial organic clay at this site, an additional 150 mm of the existing high plasticity clay should be removed and replaced with medium to low plasticity clay fill. Prior to placement of low to medium plasticity clay fill, the exposed sub-grade at the bottom of the replacement should be moisture conditioned to within ±2% of the OMC, and compacted to 98% of the SPMDD. The final subgrade should be proof-rolled to identify any loose or soft areas. Soft areas should be over-excavated and backfilled with low to medium plastic clay fill and compacted to 98% SPMDD and within ±2% of the OMC, or as recommended by the City of Edmonton design and constructions standards.

6.2 Fill Placement, Compaction, and Grading

If fills are used to establish site grading, these fills should consist of low to medium plasticity clay or well-graded, granular soils. The fill for the proposed pavement should be compacted to 98% of the SPMDD and within ±2% of the OMC. Fills should be free of organics, deleterious and frozen materials. Granular fill for the base course should be compacted to 100% of SPMDD at the OMC. A layer of non-woven geotextile fabric is recommended between granular fill and the existing clay soil at this site to prevent migration of fines from traffic that may cause pumping of the clay subgrade. Placement of the road fill should not be completed during winter months. The final subgrade should be crowned or sloped to promote positive drainage.

6.3 Pavement Structure Design

The preliminary pavement design provided in this report was based methodology from the Alberta Transportation Pavement Design Manual (1997), which is based on design information from the American Association of State Highway and Transportation Officials (ASSHTO). The pavement design parameters were obtained from the CoE Complete Streets Design and Construction Standards and Alberta Transportation Pavement Manual. The primary design parameters used for the pavement structure design include the Equivalent Single Axle Loading (ESAL) value and subgrade modulus (M_R).

Traffic loading for light-duty and heavy-duty pavement design in this report was obtained from **Table 1.2.5** of the City of Edmonton Complete Streets design and Construction Standards. The light-duty traffic loading was assumed as a Residential Minor Collector roadway truck route with no bus. The heavy-duty traffic loading was assumed as a Residential Major connector with Truck and Bus Route from.

A summary of the pavement design parameters for heavy and light duty pavement structures are provided in **Table 6-1** and **Table 6-2** below respectively.

Table 6-1: Light Duty Pavement Design Parameters

Parameters	Design Values	Remarks
Subgrade Modulus (M_R)	30 MPa	Value estimated based on subgrade conditions. The City of Edmonton 2018, Complete Streets Design and Construction Standards, Table 1.2.1 recommended subgrade modulus of 30 MPa for CI and CH soils.
Traffic Loading	1.8 x 10^5 ESALs	From CoE Complete Street Design and Construction Standards for Residential Minor Collector (Truck Route with No Bus)
Design Life	20 years	From Alberta Transportation Pavement Design Manual
Reliability	85%	From CoE Complete Street Design and Construction Standards
Initial Serviceability	4.2	From Alberta Transportation Pavement Design Manual
Terminal Serviceability	2.5	From Alberta Transportation Pavement Design Manual
Standard Deviation	0.45	From Alberta Transportation Pavement Design Manual
Structural Number	80 mm	Minimum Required Structural Number

Table 6-2: Heavy Duty Pavement Design Parameters

Parameters	Design Values	Remarks
Subgrade Modulus (M_R)	30 MPa	Value estimated based on subgrade conditions. The City of Edmonton 2018, Complete Streets Design and Construction Standards, Table 1.2.1 recommended subgrade modulus of 30 MPa for CI and CH soils.
Traffic Loading	3.6 x 10^5 ESALs	Estimated using CoE Complete Street Design and Construction Standards Table 1.2.5
Design Life	20 years	From Alberta Transportation Pavement Design Manual
Reliability	85%	From CoE Complete Street Design and Construction Standards
Initial Serviceability	4.2	From Alberta Transportation Pavement Design Manual
Terminal Serviceability	2.5	From Alberta Transportation Pavement Design Manual
Standard Deviation	0.45	From Alberta Transportation Pavement Design Manual
Structural Number	87 mm	Minimum Required Structural Number

Due to presence of firm high plasticity clay at this site near the surface, a conservative subgrade modulus of 30 MPa was selected for the design of the light and heavy duty pavement structure. A figure illustrating the required structural number in relation to the subgrade modulus and traffic loading is provided in **Appendix D**.

The result of this geotechnical investigation indicates that the ground conditions are suitable for the light-duty and heavy-duty pavement structures provided proper subgrade preparation is undertaken. **Table 6-3** and **Table 6-4** below summarize the recommended pavement structures for light-duty and heavy-duty pavement structures respectively.

Table 6-3: Light-Duty Pavement Structure

Description	Pavement Structure Material	Pavement Structure Thickness (mm)	Remarks
Light-Duty Pavement Structure	Asphalt Concrete Pavement	100	20 mm-B Asphalt should be placed in two layers. The first layer should be 60 mm and compacted to 94% of Maximum Theoretical Density (MTD) followed by placement of 10mm-LT of 40 mm compacted to 94% of MTD (CoE Complete Streets Design and Construction Standards).
	Crushed Granular Base Course over Non-woven geotextile	300	Designation 3, Class 20 granular material compacted to 100% of SPMDD within ±3% of OMC.
	Prepared Subgrade	150	Refer to **Section 6.1** for Subgrade Preparation. If the existing exposed subgrade cannot be compacted to 98% of SPMDD, additional subgrade preparation may be required.
Total Pavement Structure above prepared subgrade		**400**	

Table 6-4: Heavy-Duty Pavement Structure

Description	Pavement Structure Material	Pavement Structure Thickness (mm)	Remarks
Heavy-Duty Pavement Structure	Asphalt Concrete Pavement	100	20 mm-B Asphalt should be placed in two layers. The first layer should be 60 mm and compacted to 94% of Maximum Theoretical Density (MTD) followed by placement of 10 mm-LT 40 mm compacted to 94% of MTD (CoE Complete Streets Design and Construction Standards)
	Crushed Granular Base Course over Non-woven geotextile	335	Designation 3, Class 20 granular material compacted to 100% of SPMDD within ±3% of OMC
	Prepared Subgrade	150	Refer to **Section 6.1** for Subgrade Preparation. If the existing exposed subgrade cannot be compacted to 98% of SPMDD, additional subgrade preparation may be required.
Total Pavement Structure above prepared subgrade		**435**	

The crushed granular base course should be Designation 3, Class 20 granular material in accordance with CoE Complete Streets Design and Construction Standards.

A non-woven filter fabric should be provided between the base of the granular fill and subgrade to prevent migration of fine materials into the granular fill.

7. Conclusion

The site is considered suitable for the proposed housing building provided that the geotechnical risks identified during this investigation are understood and recommendations in this report are followed. At the time of submission of this geotechnical report, the building type, building size, foundation type, and foundation elevations have not been yet known. Depending on the size of the building and the foundation depth, a raft or deep foundations are likely the most suitable foundation type for this project. A detailed site investigation is recommended to confirm the bearing capacity and estimate the settlement of the selected foundation type. A detailed geotechnical investigation may also be an opportunity to confirm the site seismic classification of this site with a 30 mBGS testhole and seismic cone penetration test to measure the shear wave velocity profile versus depth. The detailed site investigation should be completed once the building details are confirmed.

8. References

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Appendix A

Testhole Location Plan

Issue Status: FINAL

Queen Alexandra Housing
Preliminary Geotechnical Investigation
City of Edmonton
Project No.: 60652243 Date: 2021-04-06

TESTHOLE LOCATION PLAN

AECOM

Figure 1

ANSI B 279.4mm x 431.8mm
Approved:
Checked:
Designer:
Project Management Initials:
Last saved by: EROSC(2021-04-06) Last Plotted: 2021-04-06
Filename: L:\DCS\PROJECTS\ENV\60652243_COFEDMQUNALEXH\900_CAD_GIS\910_CAD\30-FIGURES\B\0060652243-FIG-00-0000-B-0001.DWG

Appendix B

General Statement; Normal Variability of Subsurface Conditions;
Explanation of Field and Laboratory Test Data;
Modified Unified Soil Classification System;
Testhole Logs

AECOM Canada Ltd.
General Statement; Normal Variability Of Subsurface Conditions

The scope of the investigation presented herein is limited to an investigation of the subsurface conditions as to suitability of the site for the proposed project. This report has been prepared to aid in the general evaluation of the site and to assist the design engineer in the conceptual design for the area. The description of the project presented in this report represents the understanding by the geotechnical engineer of the significant aspects of the project relevant to the design and construction of the subdivision, infrastructure and similar. In the event of any changes in the basic design or location of the structures, as outlined in this report or plan, AECOM should be given the opportunity to review the changes and to modify or reaffirm in writing the conclusions and recommendations of this report.

The analysis and recommendations represented in this report are based on the data obtained from the test holes drilled at the locations indicated on the site plans and from other information discussed herein. This report is based on the assumption that the subsurface conditions everywhere on the site are not significantly different from those encountered at the test locations. However, variations in soil conditions may exist between the test holes and, also, general groundwater levels and condition may fluctuate from time to time. The nature and extent of the variations may not become evident until construction. If subsurface conditions, different from those encountered in the test holes are observed or encountered during construction or appear to be present beneath or beyond the excavation, AECOM should be advised at once so that the conditions can be observed and reviewed and the recommendations reconsidered where necessary.

Since it is possible for conditions to vary from those identified at the test locations and from those assumed in the analysis and preparation of recommendations, a contingency fund should be included in the construction budget to allow for the possibility of variations which may result in modifications of the design and construction procedures.

EXPLANATION OF FIELD & LABORATORY TEST DATA

The field and laboratory test results, as shown for each hole, are described below.

1. NATURAL MOISTURE CONTENT

The relationship between the natural moisture content and depth is significant in determining the subsurface moisture conditions. The Atterberg Limits for a sample should be compared to its natural moisture content and plotted on the Plasticity Chart in order to determine the soil classification.

2. SOIL PROFILE AND DESCRIPTION

Each soil stratum is classified and described noting any special conditions. The Modified Unified Classification System (MUCS) is used. The soil profile refers to the existing ground level at the time the hole was done. Where available, the ground elevation is shown. The soil symbols used are shown in detail on the soil classification chart.

3. TESTS ON SOIL SAMPLES

Laboratory and field tests are identified by the following and are on the logs:

N - Standard Penetration Test (SPT) Blow Count. The SPT is conducted in the field to assess the in-situ consistency of cohesive soils and the relative density of non-cohesive soils. The N value recorded is the number of blows from a 63.5 kg hammer dropped 760 mm which is required to drive a 51 mm split spoon sampler 300 mm into the soil.

SO_4 - Water Soluble Sulphate Content. Expressed in percent. Conducted primarily to determine requirements for the use of sulphate resistant cement. Further details on the water-soluble sulphate content are given in Section 6.

γ_D - Dry Unit Weight. Usually expressed in kN/m^3.

γ_T - Total Unit Weight. Usually expressed in kN/m^3.

Q_U - Unconfined Compressive Strength. Usually expressed in kPa and may be used in determining allowable bearing capacity of the soil.

C_U - Undrained Shear Strength. Usually expressed in kPa. This value is determined by either a direct shear test or by an unconfined compression test and may also be used in determining the allowable bearing capacity of the soil.

C_{PEN} - Pocket Penetrometer Reading. Usually expressed in kPa. Estimate of the undrained shear strength as determined by a pocket penetrometer.

The following tests may also be performed on selected soil samples and the results are given on separate sheets enclosed with the logs:

- Grain Size Analysis
- Standard or Modified Proctor Compaction Test
- California Bearing Ratio Test
- Direct Shear Test
- Permeability Test
- Consolidation Test
- Triaxial Test

4. SOIL DENSITY AND CONSISTENCY

The SPT test described above may be used to estimate the consistency of cohesive soils and the density of cohesionless soils. These approximate relationships are summarized in the following tables:

Table 1 Cohesive Soils

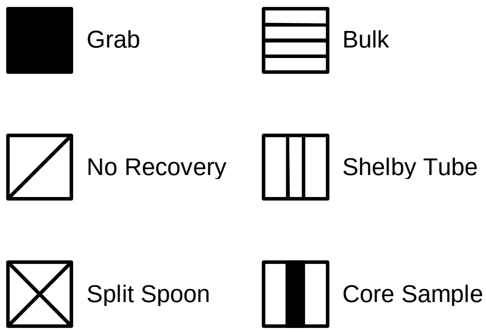
N	Consistency	C_u (kPa) approx.
0 - 1	Very Soft	<10
1 - 4	Soft	10 - 25
4 - 8	Firm	25 - 50
8 - 15	Stiff	50 - 100
15 - 30	Very Stiff	100 - 200
30 - 60	Hard	200 - 300
>60	Very Hard	>300

Table 2 Cohesionless Soils

N	Density
0 - 5	Very Loose
5 - 10	Loose
10 - 30	Compact
30 - 50	Dense
>50	Very Dense

5. SAMPLE CONDITION AND TYPE

The depth, type, and condition of samples are indicated on the logs by the following symbols:



6. WATER SOLUBLE SULPHATE CONCENTRATION

The following table, from CSA Standard A23.1-14, indicates the requirements for concrete subjected to sulphate attack based upon the percentage of water-soluble sulphate as presented on the logs. CSA Standard A23.1-14 should be read in conjunction with the table.

Table 3 Requirements for Concrete Subjected to Sulphate Attack*

Class of exposure	Degree of exposure	Water-soluble sulphate (SO_4)† in soil sample, %	Sulphate (SO_4) in groundwater samples, mg/L‡	Water soluble sulphate (SO_4) in recycled aggregate sample, %	Cementing materials to be used§††	Performance requirements§,§§		
						Maximum expansion when tested using CSA A3004-C8 Procedure A at 23 °C, %		Maximum expansion when tested using CSA A3004-C8 Procedure B at 5 °C, % †††
						At 6 months	At 12 months††	At 18 months‡‡
S-1	Very severe	> 2.0	> 10 000	> 2.0	HS** ,HSb, HSLb*** or HSe	0.05	0.10	0.10
S-2	Severe	0.20–2.0	1500–10 000	0.60–2.0	HS**, HSb, HSLb*** or HSe	0.05	0.10	0.10
S-3	Moderate (including seawater exposure*)	0.10–0.20	150–1500	0.20–0.60	MS, MSb, MSe, MSLb***, LH, LHb, HS**, HSb, HSLb*** or HSe	0.10		0.10

*For sea water exposure, also see Clause 4.1.1.5.
†In accordance with CSA A23.2-3B.
‡In accordance with CSA A23.2-2B.
§Where combinations of supplementary cementing materials and portland or blended hydraulic cements are to be used in the concrete mix design instead of the cementing materials listed, and provided they meet the performance requirements demonstrating equivalent performance against sulphate exposure, they shall be designated as MS equivalent (MSe) or HS equivalent (HSe) in the relevant sulphate exposures (see Clauses 4.1.1.6.2, 4.2.1.1, and 4.2.1.3, and 4.2.1.4).
**Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates, including seawater. See Clause 4.1.1.6.3.
††The requirement for testing at 5 °C does not apply to MS, HS, MSb, HSb, and MSe and HSe combinations made without portland limestone cement.
‡‡ If the increase in expansion between 12 and 18 months exceeds 0.03%, the sulphate expansion at 24 months shall not exceed 0.10% in order for the cement to be deemed to have passed the sulphate resistance requirement.
§§For demonstrating equivalent performance, use the testing frequency in Table 1 of CSA A3004-A1 and see the applicable notes to Table A3 in A3001 with regard to re-establishing compliance if the composition of the cementing materials used to establish compliance changes.

***Where MSLb or HSLb cements are proposed for use, or where MSe or HSe combinations include Portland-limestone cement, they must also contain a minimum of 25% Type F fly ash or 40% slag or 15% metakaolin (meeting Type N pozzolan requirements) or a combination of 5% Type SF silica fume with 25% slag or a combination of 5% Type SF silica fume with 20% Type F fly ash. For some proposed MSLb, HSLb, and MSe or HSe combinations that include Portland-limestone cement, higher SCM replacement levels may be required to meet the A3004-C8 Procedure B expansion limits. Due to the 18-month test period, SCM replacements higher than the identified minimum levels should also be tested. In addition, sulphate resistance testing shall be run on MSLb and HSLb cement and MSe or HSe combinations that include Portland-limestone cement at both 23 °C and 5 °C as specified in the table.
†††If the expansion is greater than 0.05% at 6 months but less than 0.10% at 1 year, the cementing materials combination under test shall be considered to have passed.

7. SOIL CORROSIVITY

The following table, from the Handbook of Corrosion Engineering (Roberge, 1999) indicates the corrosivity rating can be obtained from the soil resistivity, presented on the logs.

Table 4 Corrosivity Ratings Based on Soil Resistivity

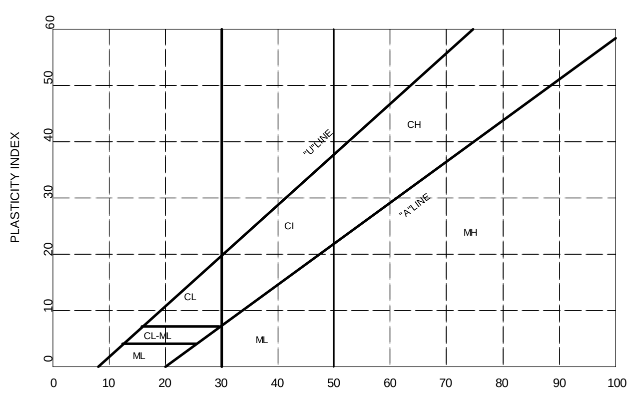
Soil Resistivity (ohm-cm)	Corrosivity Rating
>20,000	Essentially non-corrosive
10,000 – 20,000	Mildly corrosive
5,000 – 10,000	Moderately corrosive
3,000 – 5,000	Corrosive
1,000 – 3,000	Highly corrosive
<1,000	Extremely corrosive

8. GROUNDWATER TABLE

The groundwater table is indicated by the equilibrium level of water in a standpipe installed in a testhole or test pit. This level is generally taken at least 24 hours after installation of the standpipe. The groundwater level is subject to seasonal variations and is usually highest in the spring. The symbol on the logs indicating the groundwater level is an inverted solid triangle (▼).

AECOM

MAJOR DIVISION			LOG SYMBOLS	UCS	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS	GRAVELS (MORE THAN HALF COARSE GRAINS LARGER THAN 4.75 mm)	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4 \quad C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
				GP	POORLY GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE W_p LESS THAN 4
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE W_p MORE THAN 7
	SANDS (MORE THAN HALF COARSE GRAINS SMALLER THAN 4.75 mm)	CLEAN SANDS (LITTLE R NO FINES)		SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 6 \quad C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
				SP	POORLY GRADED SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		SANDS WITH FINES		SM	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE W_p LESS THAN 4
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE W_p MORE THAN 7
FINE GRAINED SOILS	SILTS (BELOW 'A' LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 50$		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)	
		$W_L > 50$		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS		
	CLAYS (ABOVE 'A' LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 30$		CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS	WHENEVER THE NATURE OF THE FINE CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER 'F'. E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
		$30 < W_L < 50$		CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50$		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS (BELOW 'A' LINE)	$W_L < 50$		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY		
		$W_L > 50$		OH	ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS				Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE	
BEDROCK				BR	SEE REPORT DESCRIPTION		
FILL				FILL	SEE REPORT DESCRIPTION		



SOIL COMPONENTS

FRACTION		SIEVE SIZE (mm) PASSING	SIEVE SIZE (mm) RETAINED	DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS PERCENT	IDENTIFIER
GRAVEL	COARSE	75	19	50 – 35	AND
	FINE	19	4.75		
SAND	COARSE	4.75	2.00	35 – 20	_____Y
	MEDIUM	2.00	0.425		
	FINE	0.425	0.080	20 – 10	SOME
SILT (non-plastic) or CLAY (plastic)		0.080		10 – 1	TRACE

OVERSIZE MATERIALS

ROUNDED OR SUB-ROUNDED	ANGULAR
COBBLES 75 mm TO 300 mm BOULDERS >300 mm	ROCK FRAGMENTS ROCKS > 0.75 m3 IN VOLUME

NOTE:
1. BOUNDARY CLASSIFICATION POSSESSING CHARACTERISTICS OF TWO GROUPS ARE GIVEN GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL MIXTURE WITH CLAY BINDER BETWEEN 5% AND 12%

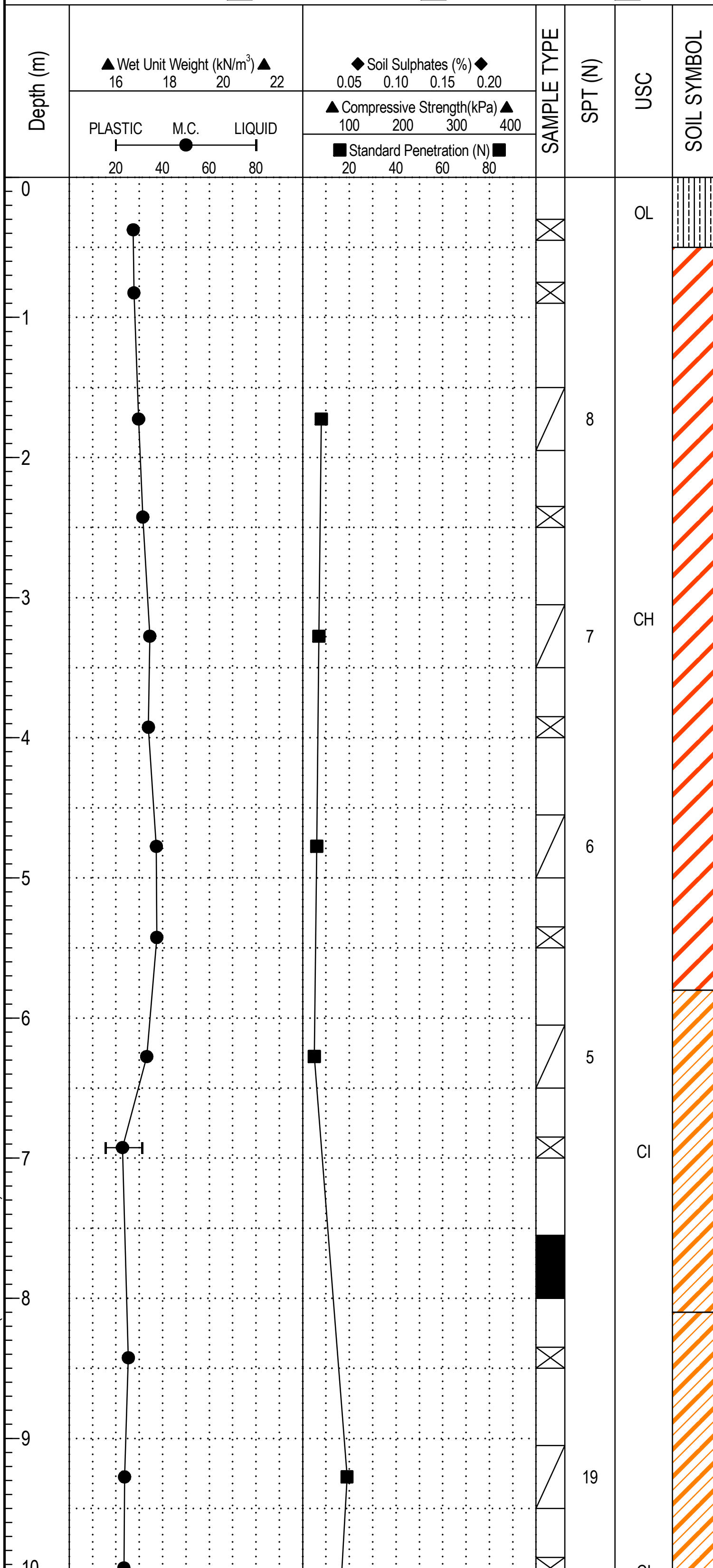
MODIFIED UNIFIED SOIL CLASSIFICATION SYSTEM

February 2021

Queen Alexandra Housing	10828 - 71 Avenue	BOREHOLE NO: **TH21-01**
Integrated Infrastructure Services	UTM ZONE: - N5930660.603 E32463.782	PROJECT NO: 60652243
START DATE: 2021/02/19	METHOD: Solid Stem Augers	ELEVATION: 668.975 m

SAMPLE TYPE: Shelby Tube, Drive Sample, Auger Sample, No Recovery, A Casing, Cored Sample

BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND



SOIL DESCRIPTION

ORGANIC CLAY - silty, trace fine grained sand, trace gravel, trace rootlets, low to medium plasticity, firm, some silt laminations, black, moist

CLAY (LACUSTRINE) - silty, trace fine grained sand, high plasticity, firm, some fine grained sand and silt laminations, oxidized, brown, moist to wet

- stiff

- firm

CLAY TILL - sandy, silty, medium to low plasticity, firm, oxidized, brown, wet

- increasing clay content, some clay layering

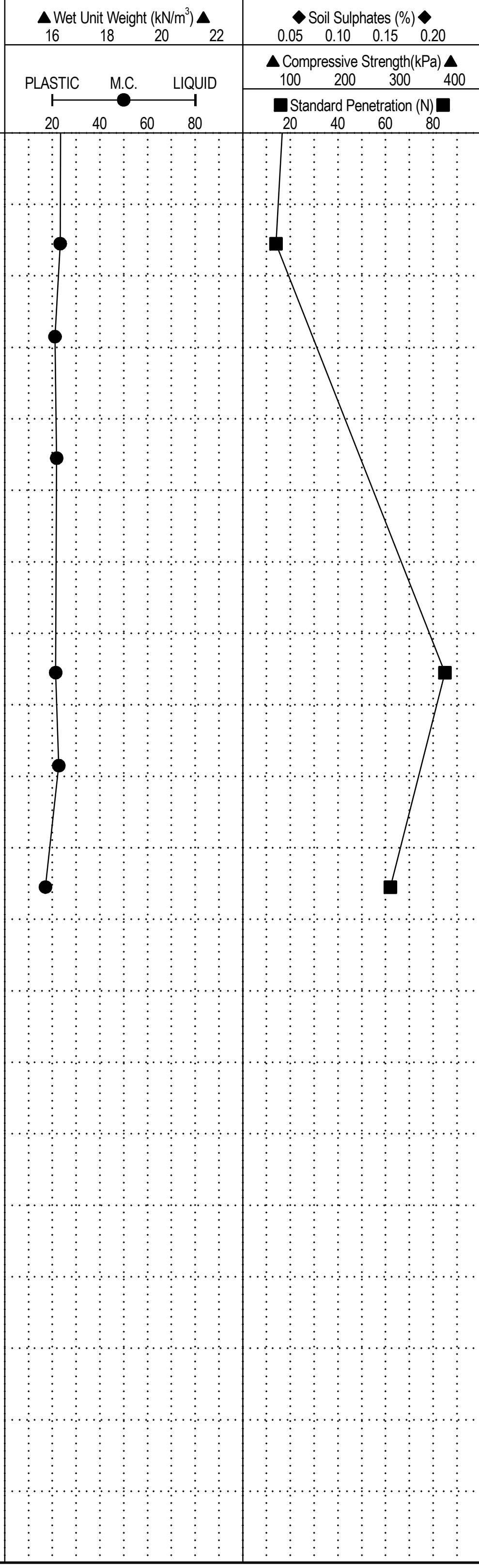
CLAY TILL - some to trace silt, trace sand, trace gravel, medium plasticity, very stiff, trace fine grained sand and silt laminations, oxidized, brown and grey, moist

ELEVATION (m): 668, 667, 666, 665, 664, 663, 662, 661, 660

Queen Alexandra Housing	10828 - 71 Avenue	BOREHOLE NO: **TH21-01**
Integrated Infrastructure Services	UTM ZONE: - N5930660.603 E32463.782	PROJECT NO: 60652243
START DATE: 2021/02/19	METHOD: Solid Stem Augers	ELEVATION: 668.975 m

SAMPLE TYPE: Shelby Tube; Drive Sample; Auger Sample; No Recovery; A Casing; Cored Sample

BACKFILL TYPE: BENTONITE; PEA GRAVEL; SLOUGH; GROUT; DRILL CUTTINGS; SAND



Depth (m)	SPT (N)	USC	SOIL DESCRIPTION	ELEVATION (m)
10 – 11.8	14		- some clay shale layering	658
11.8 – 15.5	85, 62	CS	CLAY SHALE - silty, trace fine grained sand, medium plasticity, some fine grained sand and silt laminations, very hard, poorly lithified, grey, damp - trace grey sandstone layering	657, 656, 655, 654
15.5			END OF TESTHOLE AT 15.5 mBGS - testhole open upon drilling completion - trace groundwater at bottom of testhole upon drilling completion - testhole backfilled with bentonite chips to ground surface - no monitoring well installed - testhole completed to scheduled depth	653

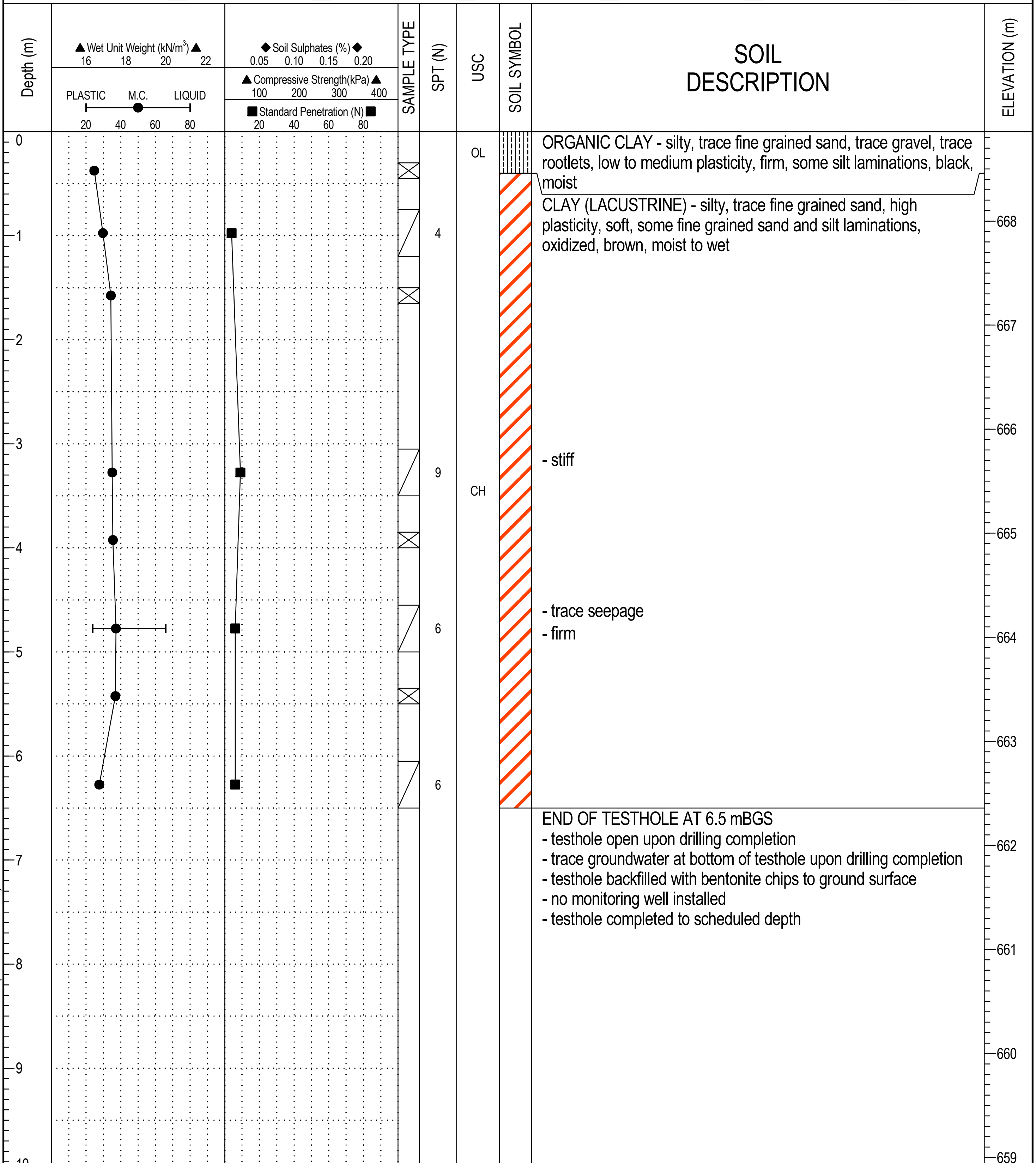
Depth scale: 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

Elevation scale: 658, 657, 656, 655, 654, 653, 652, 651, 650

Queen Alexandra Housing	10824 - 71 Avenue	BOREHOLE NO: **TH21-02**
Integrated Infrastructure Services	UTM ZONE: - N5930650.623 E32471.517	PROJECT NO: 60652243
START DATE: 2021/02/19	METHOD: Solid Stem Augers	ELEVATION: 668.859 m

SAMPLE TYPE: Shelby Tube, Drive Sample, Auger Sample, No Recovery, A Casing, Cored Sample

BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND



SOIL DESCRIPTION

ORGANIC CLAY - silty, trace fine grained sand, trace gravel, trace rootlets, low to medium plasticity, firm, some silt laminations, black, moist

CLAY (LACUSTRINE) - silty, trace fine grained sand, high plasticity, soft, some fine grained sand and silt laminations, oxidized, brown, moist to wet

- stiff

- trace seepage
- firm

END OF TESTHOLE AT 6.5 mBGS
- testhole open upon drilling completion
- trace groundwater at bottom of testhole upon drilling completion
- testhole backfilled with bentonite chips to ground surface
- no monitoring well installed
- testhole completed to scheduled depth

STANDPIP 60652243 - QUEEN ALEX HOUSING (COE TEMPLATE).GPJ EDMONTON.GDT 4/16/21

THE CITY OF Edmonton

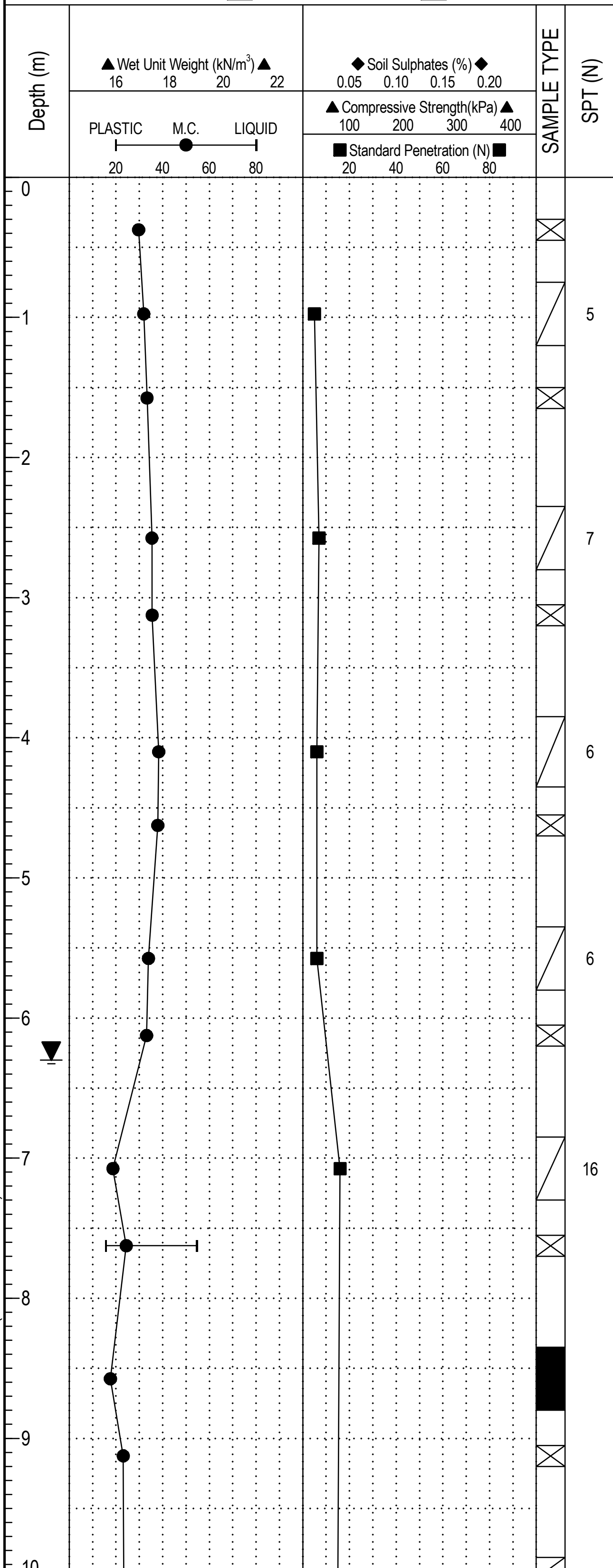
Integrated Infrastructure Services
Engineeering Services Section

LOGGED BY: PE	COMPLETION DEPTH: 6.50 m
REVIEWED BY: FA	COMPLETION DATE: 2021/02/19
Fig. No: 2	Page 1 of 1

Queen Alexandra Housing	10820 - 71 Avenue	BOREHOLE NO: **TH21-03**
Integrated Infrastructure Services	UTM ZONE: - N5930641.004 E32479.672	PROJECT NO: 60652243
START DATE: 2021/02/19	METHOD: Solid Stem Augers	ELEVATION: 669.002 m

SAMPLE TYPE: Shelby Tube, Drive Sample, Auger Sample, No Recovery, A Casing, Cored Sample

BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND



SOIL SYMBOL	SOIL DESCRIPTION	ELEVATION (m)
	ORGANIC CLAY - silty, trace fine grained sand, trace gravel, trace rootlets, low to medium plasticity, firm, some silt laminations, black, moist	
	CLAY (LACUSTRINE) - silty, trace fine grained sand, high plasticity, firm, some fine grained sand and silt laminations, oxidized, brown, moist to wet	668
		667
		666
		665
	- trace seepage	
		664
	SILT - some clay, trace fine grained sand, low plasticity to non plastic, firm, oxidized, brown, wet	663
	SAND - fine grained, silty, trace clay, compact, oxidized, brown, saturated	662
	CLAY TILL - silty, some sand, high plasticity, very stiff, trace fine grained sand and silt laminations, oxidized, brown, moist	
		661
	- 450 mm sand layer, wet	
		660
	SAND - some silt, trace clay, compact, fine grained, oxidized,	

STANDPIP 60652243 - QUEEN ALEX HOUSING (COE TEMPLATE).GPJ EDMONTON.GDT 4/16/21

THE CITY OF Edmonton

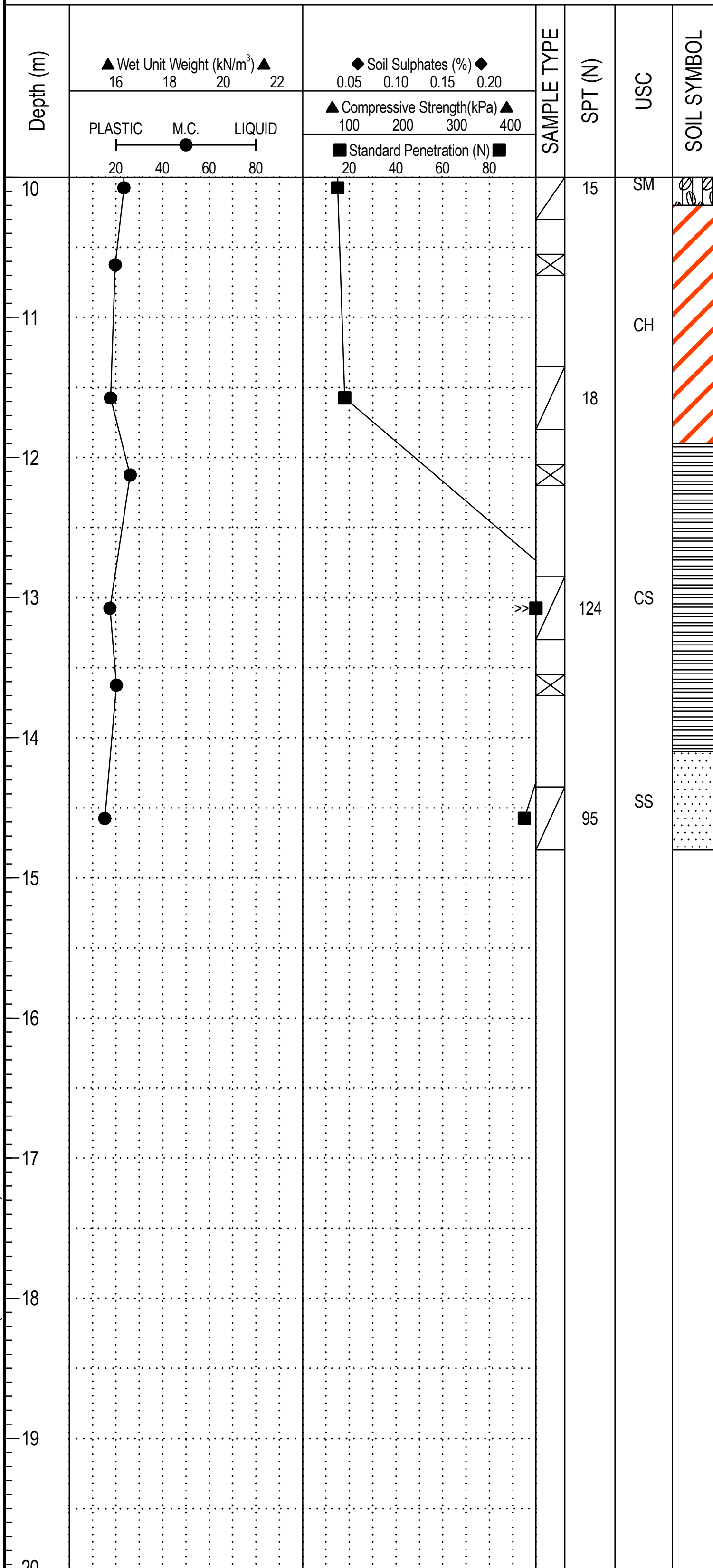
Integrated Infrastructure Services
Engineeering Services Section

LOGGED BY: PE	COMPLETION DEPTH: 14.80 m
REVIEWED BY: FA	COMPLETION DATE: 2021/02/19
Fig. No: 2	

Queen Alexandra Housing	10820 - 71 Avenue	BOREHOLE NO: **TH21-03**
Integrated Infrastructure Services	UTM ZONE: - N5930641.004 E32479.672	PROJECT NO: 60652243
START DATE: 2021/02/19	METHOD: Solid Stem Augers	ELEVATION: 669.002 m

SAMPLE TYPE: Shelby Tube, Drive Sample, Auger Sample, No Recovery, A Casing, Cored Sample

BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND



SOIL DESCRIPTION

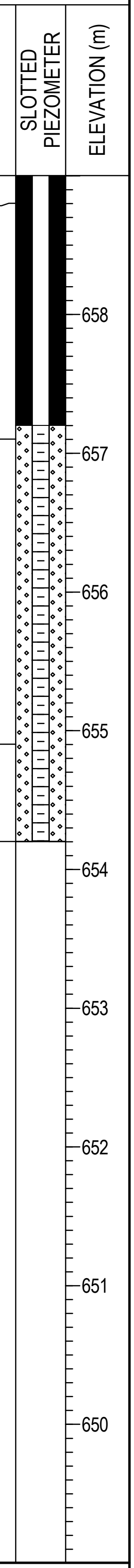
brown, saturated

CLAY TILL - silty, some sand, high plasticity, very stiff, trace fine grained sand and silt laminations, oxidized, dark brown, moist

CLAY SHALE - silty, trace fine grained sand, medium plasticity, some silt and fine grained sand laminations, very hard, poorly lithified, grey, damp

SANDSTONE - fine grained, silty, trace silt laminations, very dense, poorly lithified, grey, damp

END OF TESTHOLE AT 14.8 mBGS
- testhole open upon drilling completion
- groundwater encountered at 8.1 mBGS upon drilling completion
- 25 mm diameter standpipe piezometer installed to 14.8 mBGS
- testhole completed to scheduled depth
- groundwater measured at 6.3 mBGS on 03/03/2021



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Appendix C

Laboratory Results

WATER CONTENT (ASTM D2216)

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

CLIENT:	City of Edmonton
PROJECT:	Queen Alexandra Housing
JOB No.:	60652243
DATE :	February 24, 2021 TECHNICAN : CK/GU

HOLE No.	21-01							
DEPTH								
SAMPLE No.	1	2	3	4	5	6	7	8
TARE No.								
WT. SAMPLE WET + TARE	481.7	556.1	533.4	586.6	547.5	521.3	640.0	588.5
WT. SAMPLE DRY + TARE	380.9	438.2	414.3	449.4	410.4	392.8	469.7	431.6
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**27.4%**	**27.7%**	**29.7%**	**31.5%**	**34.5%**	**33.9%**	**37.3%**	**37.5%**
HOLE No.	21-01							
DEPTH								
SAMPLE No.	9	10	12	13	14	15	16	18
TARE No.								
WT. SAMPLE WET + TARE	716.1	769.3	687.0	611.3	792.2	590.6	618.7	559.2
WT. SAMPLE DRY + TARE	541.1	629.1	550.8	496.9	644.5	481.3	513.1	461.5
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**33.2%**	**22.8%**	**25.3%**	**23.7%**	**23.4%**	**23.3%**	**21.1%**	**21.8%**
HOLE No.	21-01			21-02				
DEPTH								
SAMPLE No.	19	20	21	1	2	3	4	5
TARE No.								
WT. SAMPLE WET + TARE	674.6	560.8	682.8	594.9	317.5	659.4	600.0	556.4
WT. SAMPLE DRY + TARE	558.1	459.6	585.1	479.8	247.9	494.3	447.6	414.1
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**21.4%**	**22.7%**	**17.1%**	**24.7%**	**29.7%**	**34.3%**	**35.1%**	**35.5%**
HOLE No.	21-02			21-03				
DEPTH								
SAMPLE No.	6	7	8	1	2	3	4	5
TARE No.								
WT. SAMPLE WET + TARE	562.3	580.9	788.5	523.0	332.9	633.5	650.9	552.9
WT. SAMPLE DRY + TARE	413.5	428.0	620.7	406.2	255.5	478.6	484.2	411.4
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**37.2%**	**36.9%**	**27.6%**	**29.7%**	**31.9%**	**33.3%**	**35.4%**	**35.5%**

WATER CONTENT (ASTM D2216)

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

CLIENT:	City of Edmonton		
PROJECT:	Queen Alexandra Housing		
JOB No.:	60652243		
DATE :	February 24, 2021	TECHNICAN :	CK/GU

HOLE No.	21-03							
DEPTH								
SAMPLE No.	6	7	8	9	10	11	13	14
TARE No.								
WT. SAMPLE WET + TARE	658.2	612.1	773.3	802.3	764.1	608.8	700.7	747.8
WT. SAMPLE DRY + TARE	479.5	447.5	580.8	606.1	646.0	492.0	571.8	608.7
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**38.3%**	**37.9%**	**33.9%**	**33.1%**	**18.7%**	**24.4%**	**23.1%**	**23.4%**
HOLE No.	21-03							
DEPTH								
SAMPLE No.	15	16	17	18	19	20		
TARE No.								
WT. SAMPLE WET + TARE	757.0	751.5	584.2	604.1	610.0	584.0		
WT. SAMPLE DRY + TARE	634.4	640.5	465.9	516.7	509.9	508.7		
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2		
WATER CONTENT W%	**19.7%**	**17.7%**	**26.1%**	**17.4%**	**20.2%**	**15.2%**		
HOLE No.	21-03							
DEPTH	TUBE							
SAMPLE No.	12							
TARE No.								
WT. SAMPLE WET + TARE	953.9							
WT. SAMPLE DRY + TARE	835.6							
WT. TARE	163.8							
WATER CONTENT W%	**17.6%**							
HOLE No.								
DEPTH								
SAMPLE No.								
TARE No.								
WT. SAMPLE WET + TARE								
WT. SAMPLE DRY + TARE								
WT. TARE								
WATER CONTENT W%								

ATTERBERG LIMITS (ASTM D4318)

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

CLIENT :	City of Edmonton		
PROJECT :	Queen Alex		
JOB No. :	60652243		
LOCATION :		SAMPLE:	10
TESTHOLE:	21-01	DEPTH :	
DATE :	February 26, 2021	TECHNICIAN :	GU

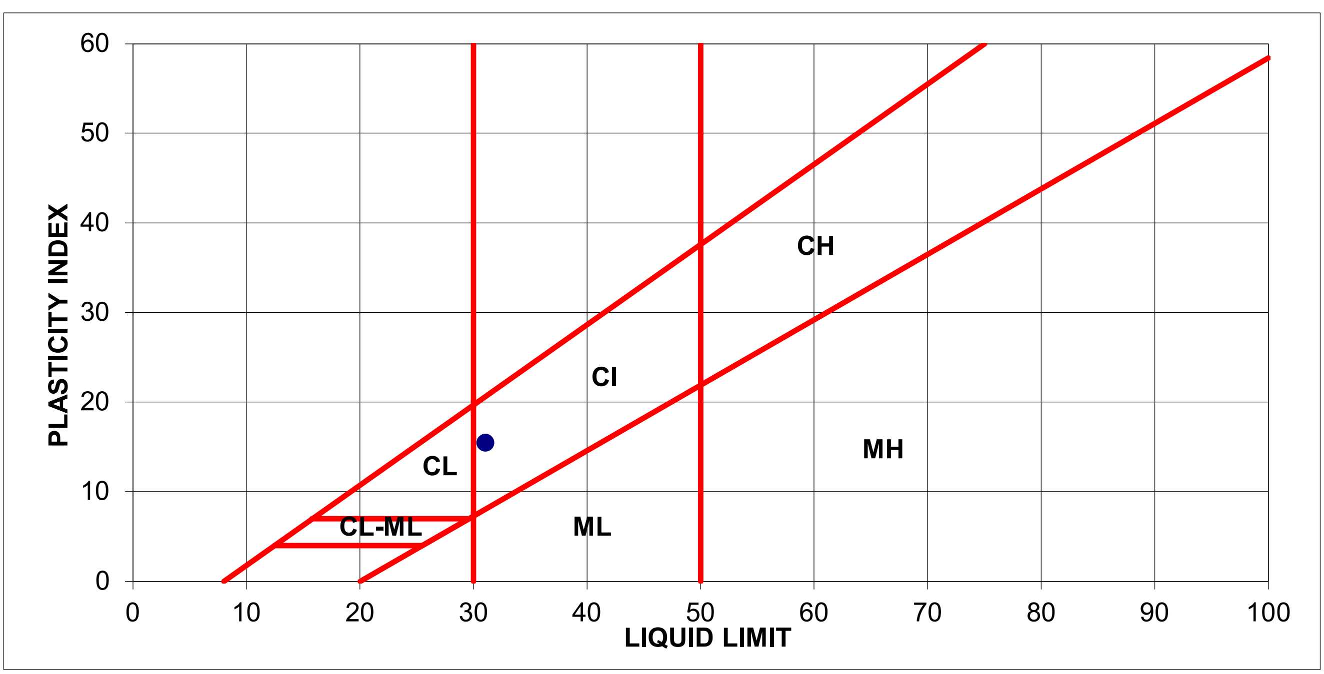
LIQUID LIMIT						
Trial No.	1					
Number of Blows	30					
Container Number						
Wt. Sample (wet+tare)(g)	58.32					
Wt. Sample (dry+tare)(g)	48.51					
Wt. Tare (g)	16.22					
Wt. Dry Soil (g)	32.3					
Wt. Water (g)	9.8					
Water Content (%)	30.4%					

AVERAGE VALUES	
Liquid Limit	31.1
Plastic Limit	15.6
Plasticity Index	15.5

SAMPLE DESCRIPTION

Classification: **CI-CL**

PLASTIC LIMIT			
Trial No.	1		
Container Number			
Wt. Sample (wet+tare)(g)	30.51		
Wt. Sample (dry+tare)(g)	28.03		
Wt. Tare (g)	12.10		
Wt. Dry Soil (g)	15.9		
Wt. Water (g)	2.5		
Water Content (%)	15.6%		



FORM: COE Queen Alex TH21-01 #10 Atterberg .xls
DATE: 3/1/2021

ATTERBERG LIMITS (ASTM D4318)

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

CLIENT :	City of Edmonton		
PROJECT :	Queen Alexandra Housing		
JOB No. :	60652243		
LOCATION :		SAMPLE:	6
TESTHOLE:	21-02	DEPTH :	
DATE :	March 18, 2021	TECHNICIAN :	GU

LIQUID LIMIT

Trial No.	1					
Number of Blows	28					
Container Number						
Wt. Sample (wet+tare)(g)	52.57					
Wt. Sample (dry+tare)(g)	38.16					
Wt. Tare (g)	15.95					
Wt. Dry Soil (g)	22.2					
Wt. Water (g)	14.4					
Water Content (%)	64.9%					

AVERAGE VALUES

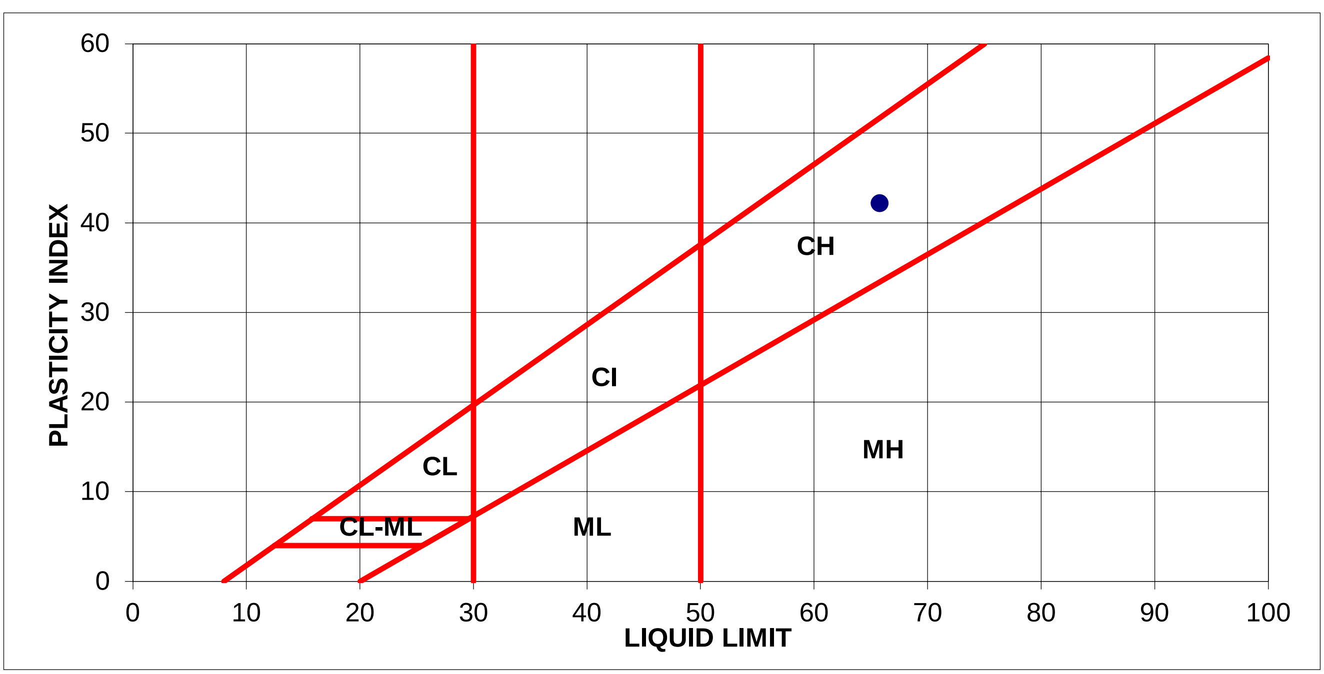
Liquid Limit	65.8
Plastic Limit	23.6
Plasticity Index	42.2

SAMPLE DESCRIPTION

Classification: **CH**

PLASTIC LIMIT

Trial No.	1		
Container Number			
Wt. Sample (wet+tare)(g)	29.55		
Wt. Sample (dry+tare)(g)	26.20		
Wt. Tare (g)	11.99		
Wt. Dry Soil (g)	14.2		
Wt. Water (g)	3.4		
Water Content (%)	23.6%		



FORM: COE Queen Alex TH21-02 #6 Atterberg .xls
DATE: 4/16/2021

ATTERBERG LIMITS (ASTM D4318)

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

CLIENT :	City of Edmonton		
PROJECT :	Queen Alexandra Housing		
JOB No. :	60652243		
LOCATION :		SAMPLE:	11
TESTHOLE:	21-03	DEPTH :	
DATE :	February 26, 2021	TECHNICIAN :	CK

LIQUID LIMIT

Trial No.	1					
Number of Blows	27					
Container Number						
Wt. Sample (wet+tare)(g)	43.52					
Wt. Sample (dry+tare)(g)	32.37					
Wt. Tare (g)	11.80					
Wt. Dry Soil (g)	20.6					
Wt. Water (g)	11.2					
Water Content (%)	54.2%					

AVERAGE VALUES

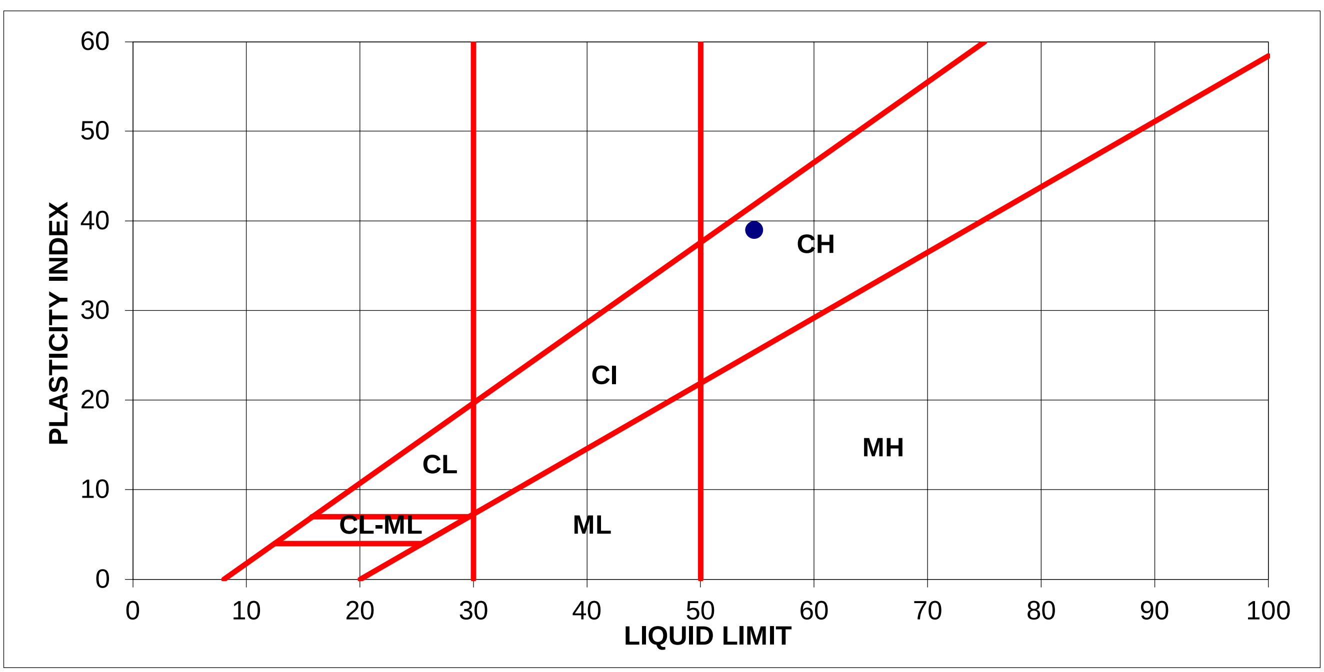
Liquid Limit	54.7
Plastic Limit	15.7
Plasticity Index	39.0

SAMPLE DESCRIPTION

Classification: **CH**

PLASTIC LIMIT

Trial No.	1		
Container Number			
Wt. Sample (wet+tare)(g)	32.59		
Wt. Sample (dry+tare)(g)	30.12		
Wt. Tare (g)	14.40		
Wt. Dry Soil (g)	15.7		
Wt. Water (g)	2.5		
Water Content (%)	15.7%		



FORM: COE Queen Alex TH21-03 #11 Atterberg .xls
DATE: 4/16/2021

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

GRAIN SIZE ANALYSIS (ASTM D422)

CLIENT :	City of Edmonton		
PROJECT :	Queen Alexandra Housing		
JOB No. :	60652243		
LOCATION :		SAMPLE:	10
TESTHOLE:	21-01	DEPTH :	
DATE :	February 27, 2021	TECHNICIAN :	GU

TOTAL DRY WEIGHT OF SAMPLE		SIEVE NO. (µm)	SIZE OF OPENING APPROX. INCHES	SIZE OF OPENING mm	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT FINER THAN	REMARKS
Before Washing		150,000	6	150.0		0%	100%	
Wet + Tare		75,000	3	75.0		0%	100%	
Dry+Tare	709.3	50,000	2	50.0		0%	100%	
Tare	100.0	40,000	1 1/2	40.0		0%	100%	
Wt. Dry	609.3	25,000	1	25.0		0%	100%	
Moisture Content		20,000	3/4	20.0		0%	100%	
Wet + Tare		16,000	5/8	16.0		0%	100%	
Dry+Tare		12,500	1/2	12.5		0%	100%	
Tare		10,000	3/8	10.0		0%	100%	
MC (%)		5,000	0.185	5.0	3.3	1%	99.5%	
	Passing							
After Washing		2,000	0.0937	2.0	10.0	2%	98.4%	
Wt. Dry+Tare		1,250	0.0469	1.25	16.0	3%	97.4%	
Tare		630	0.0234	0.63	28.0	5%	95.4%	
Wt. Dry		315	0.0116	0.315	59.1	10%	90.3%	
Tare No.		160	0.0059	0.160	110.7	18%	81.8%	
		75	0.00295	0.075	161.0	26%	73.6%	
		PAN						

HYDROMETER DATA		READING	TIME (min)	DIAMETER (mm)	TEMP. (°C)	CORR. READING	PERCENT FINER THAN	REMARKS
Wt Dry+Tare	709.3	41	0.5	0.059	20	37	71.1%	
Wt Tare	100.0	39	1	0.042	20	35	67.2%	
Wt Dry	609.3	37	2	0.030	20	33	63.3%	
Sample Size :	50	35	5	0.020	20	31	59.4%	
Wt Retained 2 mm:	10.0	31	15	0.012	20	27	51.6%	
% Passing 2 mm:	98.4%	28	30	0.008	20	24	45.8%	
Specific Gravity :	2.70	24	60	0.006	20	20	38.0%	
Hydrometer No.:	43-9856	22	120	0.004	20	18	34.1%	
Solution (g/L) :	40	20	240	0.003	20	16	30.2%	
		17	1440	0.001	20	13	24.3%	
		16	2880	0.001	20	12	22.4%	

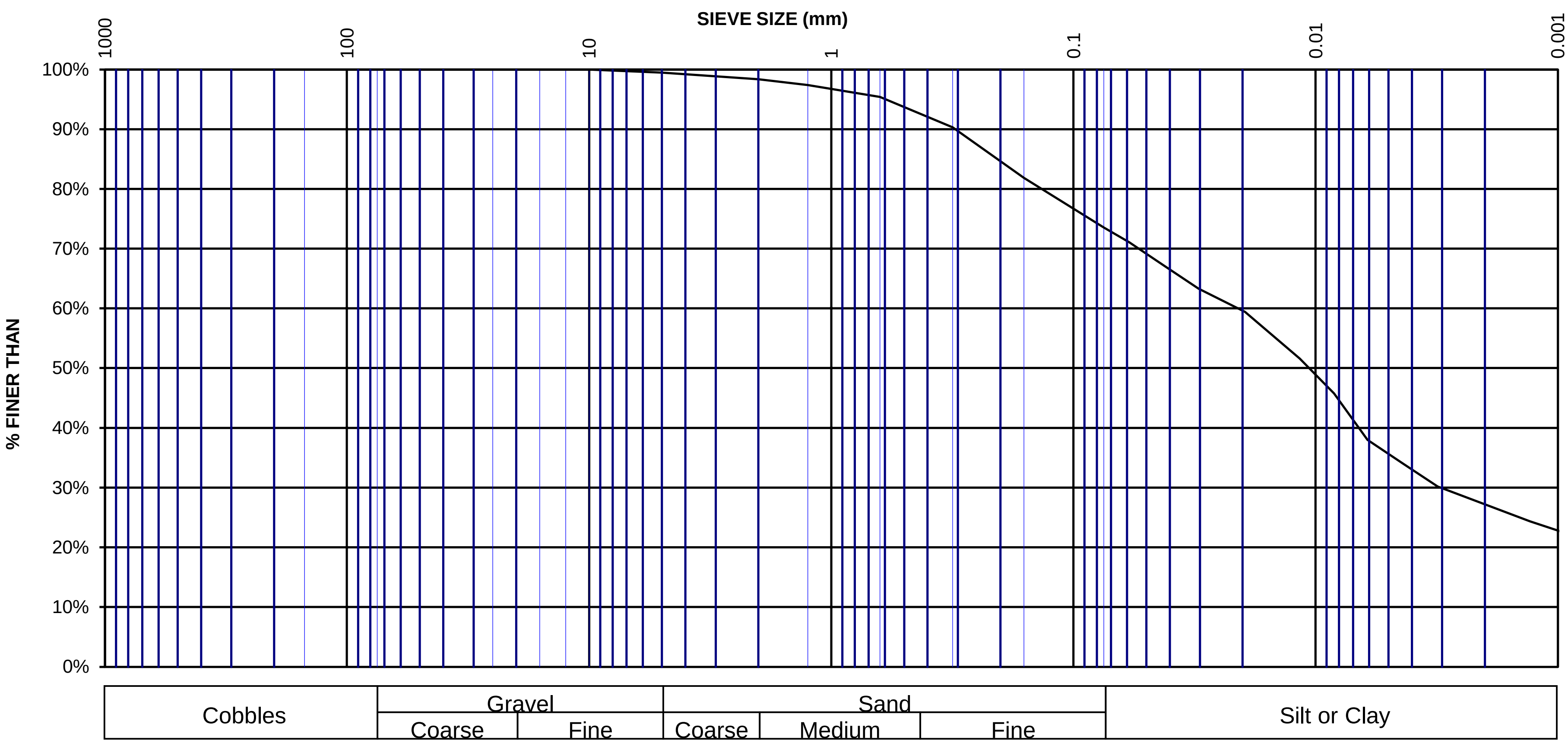
FORM: COE Queen Alex TH21-01 #10 Hydro.xls
DATE: 4/16/2021

AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

GRAIN SIZE ANALYSIS (ASTM D422)

CLIENT :	City of Edmonton		
PROJECT :	Queen Alexandra Housing		
JOB No. :	60652243		
LOCATION :		SAMPLE:	10
TESTHOLE:	21-01	DEPTH :	
DATE :	February 27, 2021	TECHNICIAN :	GU

Gravel = 0.5% **Sand = 25.9%** **Silt = 46.3%** **Clay = 27.3%**



FORM: COE Queen Alex TH21-01 #10 Hydro.xls
DATE: 4/16/2021

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

GRAIN SIZE ANALYSIS (ASTM D422)

CLIENT :	City of Edmonton		
PROJECT :	Queen Alexandra Housing		
JOB No. :	60652243		
LOCATION :		SAMPLE:	11
TESTHOLE:	21-03	DEPTH :	
DATE :	February 27, 2021	TECHNICIAN :	GU

TOTAL DRY WEIGHT OF SAMPLE		SIEVE NO. (µm)	SIZE OF OPENING APPROX. INCHES	SIZE OF OPENING mm	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT FINER THAN	REMARKS
Before Washing		150,000	6	150.0		0%	100%	
Wet + Tare		75,000	3	75.0		0%	100%	
Dry+Tare	571.7	50,000	2	50.0		0%	100%	
Tare	100.0	40,000	1 1/2	40.0		0%	100%	
Wt. Dry	471.7	25,000	1	25.0		0%	100%	
Moisture Content		20,000	3/4	20.0		0%	100%	
Wet + Tare		16,000	5/8	16.0		0%	100%	
Dry+Tare		12,500	1/2	12.5		0%	100%	
Tare		10,000	3/8	10.0		0%	100%	
MC (%)		5,000	0.185	5.0		0%	100%	
	Passing							
After Washing		2,000	0.0937	2.0		0%	100%	
Wt. Dry+Tare		1,250	0.0469	1.25	0.9	0%	99.8%	
Tare		630	0.0234	0.63	3.8	1%	99.2%	
Wt. Dry		315	0.0116	0.315	14.2	3%	97.0%	
Tare No.		160	0.0059	0.160	37.7	8%	92.0%	
		75	0.00295	0.075	60.4	13%	87.2%	
		PAN						

HYDROMETER DATA		READING	TIME (min)	DIAMETER (mm)	TEMP. (°C)	CORR. READING	PERCENT FINER THAN	REMARKS
Wt Dry+Tare	571.7	48	0.5	0.055	20	43	85.1%	
Wt Tare	100.0	46	1	0.040	20	42	82.2%	
Wt Dry	471.7	45	2	0.028	20	41	80.2%	
Sample Size :	50	44	5	0.018	20	40	78.2%	
Wt Retained 2 mm:	0.0	42	15	0.011	20	38	74.3%	
% Passing 2 mm:	100.0%	40	30	0.008	20	36	70.3%	
Specific Gravity :	2.70	38	60	0.006	20	34	66.3%	
Hydrometer No.:	43-9856	36	120	0.004	20	32	62.4%	
Solution (g/L) :	40	35	240	0.003	20	30	59.4%	
		30	1440	0.001	20	26	50.5%	
		28	2880	0.001	20	24	46.5%	

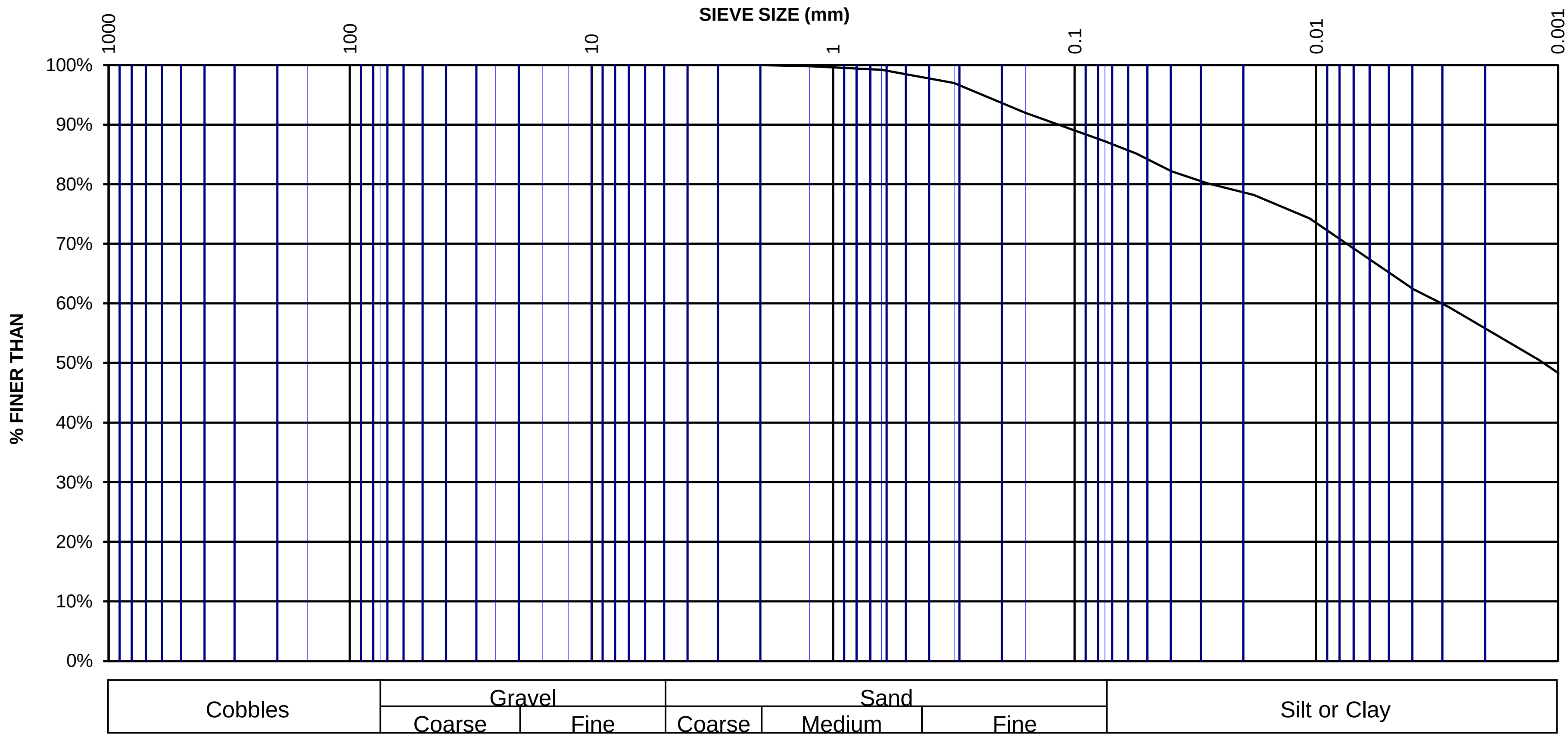
FORM: COE Queen Alex TH21-03 #11 Hydro.xls
DATE: 4/16/2021

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

GRAIN SIZE ANALYSIS (ASTM D422)

CLIENT :	City of Edmonton		
PROJECT :	Queen Alexandra Housing		
JOB No. :	60652243		
LOCATION :		SAMPLE:	11
TESTHOLE:	21-03	DEPTH :	
DATE :	February 27, 2021	TECHNICIAN :	GU

Gravel = 0.0% **Sand = 12.8%** **Silt = 32.3%** **Clay = 54.9%**



FORM: COE Queen Alex TH21-03 #11 Hydro.xls
DATE: 4/16/2021

UNCONFINED COMPRESSION TEST (ASTM-D2166)

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

CLIENT :	City of Edmonton		
PROJECT :	Queen Alexandra Housing		
JOB No. :	6052243		
LOCATION :		SAMPLE:	11
TESTHOLE:	21-01	DEPTH :	7.55m
DATE :	February 27, 2021	TECHNICIAN :	CK

DENSITY DETERMINATION		WATER CONTENT		SAMPLE DESCRIPTION
Wt. Sample (g)	1174.5	Tare Number		SILTY CLAY TILL - some sand, trace oxidized inclusions, coal, moist to wet, soft, olive grey
Initial Length (mm)	146.5	Wt. Sample (wet+tare) (g)	1327.5	
Initial Diameter (mm)	72.2	Wt. Sample (dry+tare)(g)	1060.6	
Wet Unit Weight (kN/m³)	19.2	Wt. Tare (g)	154.1	
Dry Unit Weight (kN/m³)	14.8	Water Content (%)	29.4%	

LOAD DATA		FAILURE DATA		FAILURE MODE
Ring #	3491	Load (N)	462	45^0 cracking at top 2/3 of sample with some vertical cracking.
Gears Used	Humbolt	% Strain :	9.1%	
Loading Rate	.055"/min	Corrected Q_U (kPa)	103	

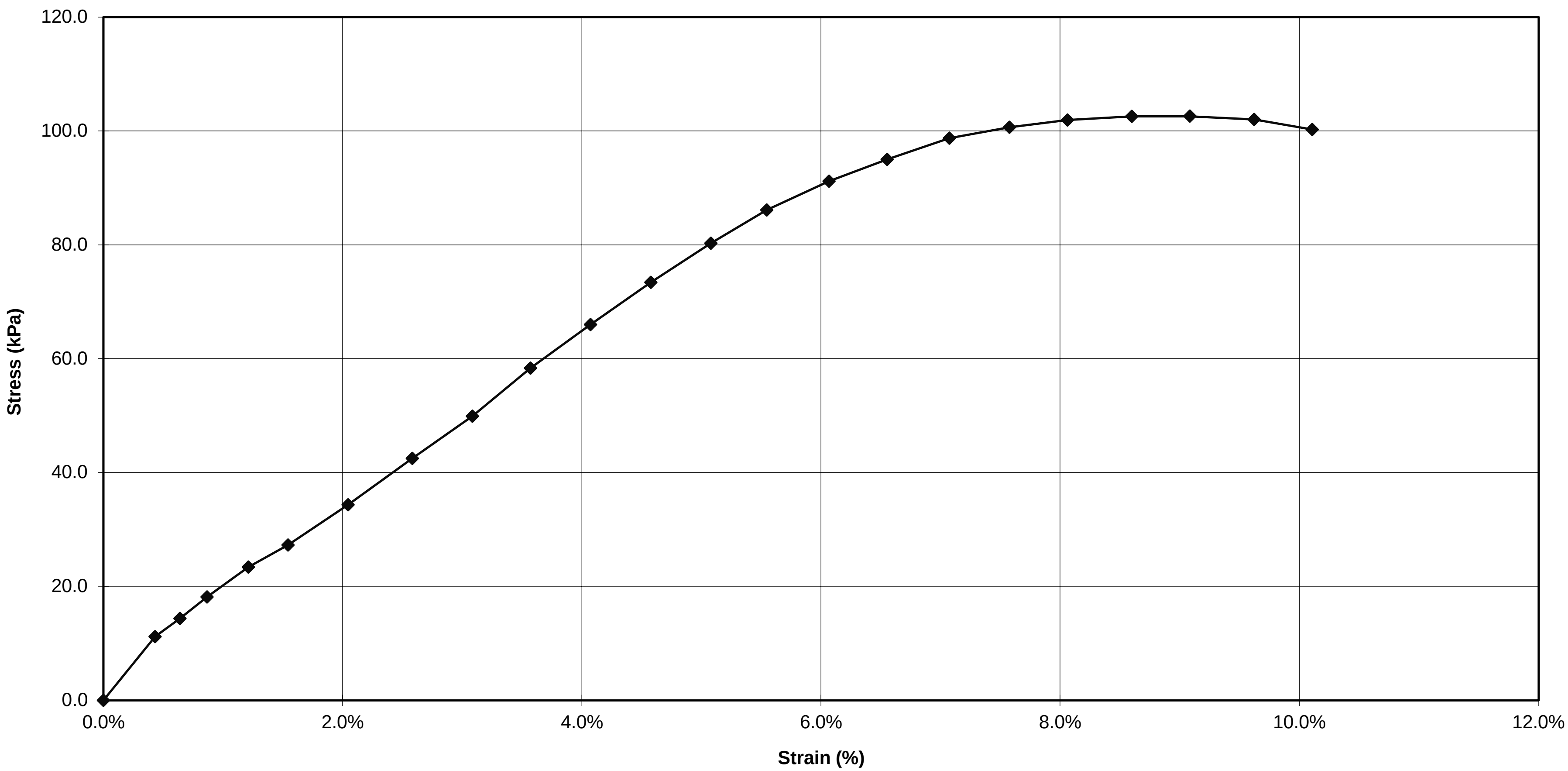
Time (min)	Load Dial (0.0001")	Load (N)	Strain Dial (0.001")	Strain (%)	Area (mm²)	Q_U (kPa)	Comments
0	0	0	1000	0.0%	4094	0.0	
0.25	16	46	975	0.4%	4112	11.2	
0.5	21	59	963	0.6%	4121	14.4	
0.75	27	75	950	0.9%	4130	18.1	
1	35	97	930	1.2%	4144	23.4	
1.5	41	113	911	1.5%	4158	27.3	
2	52	143	882	2.0%	4180	34.3	
2.5	65	178	851	2.6%	4203	42.5	
3	77.5	211	822	3.1%	4225	49.9	
3.5	91	248	794	3.6%	4246	58.3	
4	104	282	765	4.1%	4268	66.0	
4.5	117	315	736	4.6%	4291	73.4	
5	129	346	707	5.1%	4313	80.3	
5.5	139	373	680	5.5%	4335	86.1	
6	148	397	650	6.1%	4359	91.2	
6.5	155	416	622	6.6%	4381	95.0	
7	162	435	592	7.1%	4406	98.7	
7.5	166	446	563	7.6%	4430	100.6	
8	169	454	535	8.1%	4453	101.9	
8.5	171	459	504	8.6%	4479	102.5	
9	172	462	476	9.1%	4503	102.6	
9.5	172	462	445	9.6%	4530	102.0	
10	170	457	417	10.1%	4555	100.3	

FORM : COE Queen Alex TH21-01 #11 UCT .xls
DATE: 4/16/2021

UNCONFINED COMPRESSION TEST (ASTM D2166)

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

CLIENT :	City of Edmonton		
PROJECT :	Queen Alexandra Housing		
JOB No. :	6052243		
LOCATION :		SAMPLE:	11
TESTHOLE:	21-01	DEPTH :	7.55m
DATE :	27-Feb-21	TECH. :	CK



UNCONFINED COMPRESSION TEST (ASTM-D2166)

AECOM
AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

CLIENT :	City of Edmonton		
PROJECT :	Queen Alexandra Housing		
JOB No. :	6052243		
LOCATION :		SAMPLE:	17
TESTHOLE:	21-01	DEPTH :	12.05m
DATE :	February 27, 2021	TECHNICIAN :	CK

DENSITY DETERMINATION		WATER CONTENT		SAMPLE DESCRIPTION
Wt. Sample (g)	1273.2	Tare Number		CLAYSTONE/CLAY TILL - silty, some sand, trace oxidized inclusions, coal, alkalines, damp, medium stiff, light to dark grey
Initial Length (mm)	147.8	Wt. Sample (wet+tare) (g)	1472.4	
Initial Diameter (mm)	72.5	Wt. Sample (dry+tare)(g)	1267.4	
Wet Unit Weight (kN/m³)	20.5	Wt. Tare (g)	200	
Dry Unit Weight (kN/m³)	17.2	Water Content (%)	19.2%	

LOAD DATA		FAILURE DATA		FAILURE MODE
Ring #	3491	Load (N)	863	45^0 cracking from top to bottom corners.
Gears Used	Humbolt	% Strain :	2.4%	
Loading Rate	.055"/min	Corrected Q_U (kPa)	204	

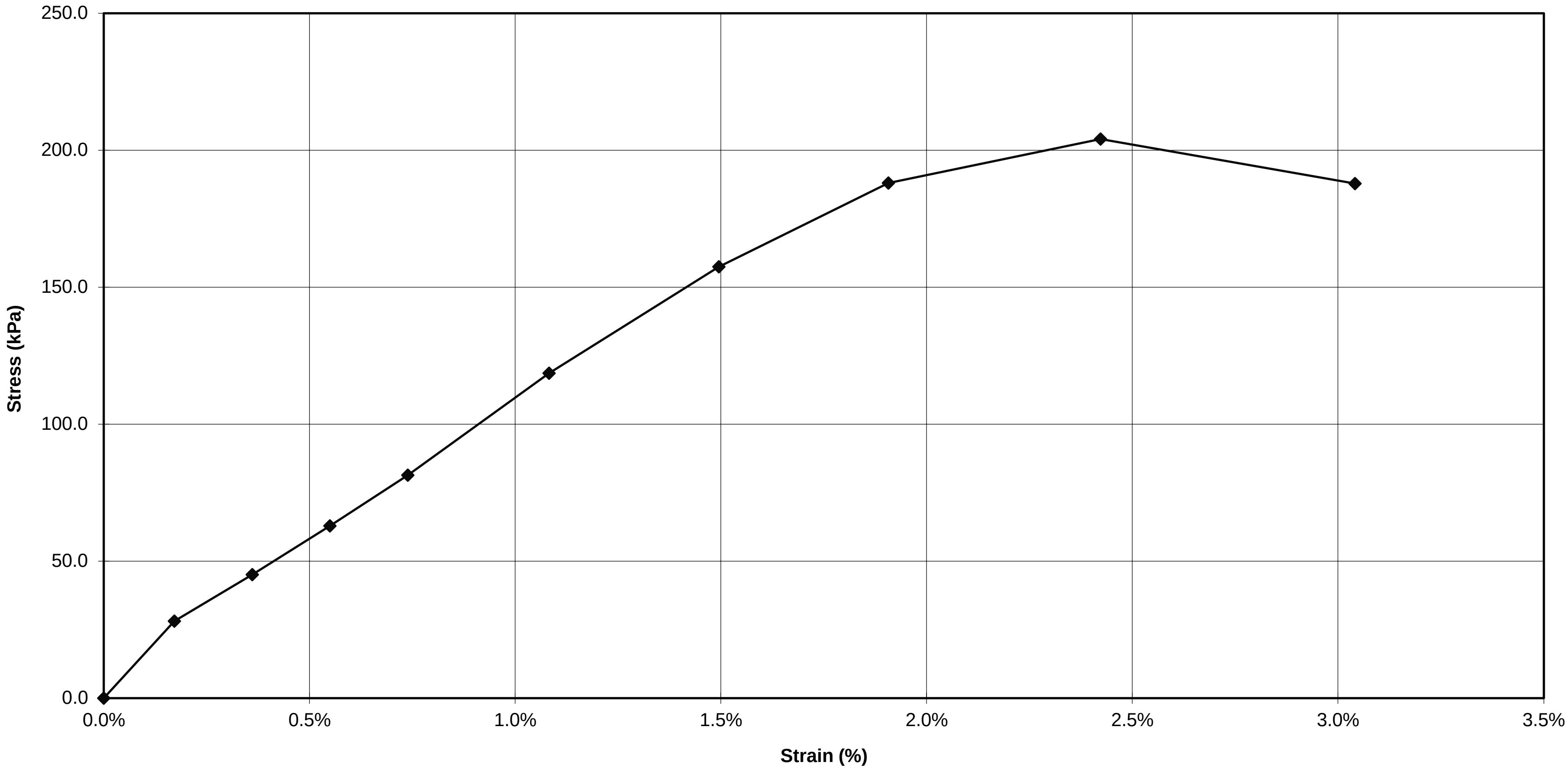
Time (min)	Load Dial (0.0001")	Load (N)	Strain Dial (0.001")	Strain (%)	Area (mm²)	Q_U (kPa)	Comments
0	0	0	1000	0.0%	4128	0.0	
0.25	42	116	990	0.2%	4135	28.1	
0.5	68	187	979	0.4%	4143	45.0	
0.75	96	261	968	0.5%	4151	62.8	
1	126	338	957	0.7%	4159	81.3	
1.5	184	495	937	1.1%	4173	118.6	
2	245	660	913	1.5%	4191	157.4	
2.5	293	791	889	1.9%	4209	188.0	
3	320	863	859	2.4%	4231	204.1	
3.5	296	800	823	3.0%	4258	187.8	

FORM : COE Queen Alex TH21-01 #17 UCT .xls
DATE: 4/16/2021

UNCONFINED COMPRESSION TEST (ASTM D2166)

AECOM Canada Ltd.
Materials Testing Lab
Bay#14-1511 Highfield Cres.SE
Calgary, Alberta T2G 5M4

CLIENT :	City of Edmonton		
PROJECT :	Queen Alexandra Housing		
JOB No. :	Queen Alexandra Housing		
LOCATION :		SAMPLE:	17
TESTHOLE:	21-01	DEPTH :	12.05m
DATE :	27-Feb-21	TECH. :	CK



FORM : COE Queen Alex TH21-01 #17 UCT .xls
DATE: 4/16/2021

AECOM Canada Ltd.
ATTN: Chris Keeley
Suite 300, 48 Quarry Park Blvd SE
Calgary AB T2C 5P2

Date Received: 25-FEB-21
Report Date: 08-MAR-21 11:46 (MT)
Version: FINAL

Client Phone: 403-254-3301

Certificate of Analysis

Lab Work Order #: L2561454
Project P.O. #: NOT SUBMITTED
Job Reference: CITY OF EDMONTON - QUEEN ALEX - 60652243
C of C Numbers:
Legal Site Desc:

Comments: Total Sulphate Ion Content result is <0.2% for sample L2561454-2. Water Soluble Sulphate Ion Content test is not required unless Total Sulphate Ion Content result is greater than 0.2%. Water Soluble Sulphate Ion Content analysis has been removed for this sample.

Dhaliwal

Inayat Dhaliwal
Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2561454-1 COE - QUEEN ALEX - TH21-01 #4							
Sampled By: CLIENT on 19-FEB-21							
Matrix: Soil							
Miscellaneous Parameters							
% Saturation	124		1.0	%	06-MAR-21	06-MAR-21	R5397522
Chloride (Cl)	<20		20	mg/L		06-MAR-21	R5397529
Resistivity	400		1.0	ohm cm		06-MAR-21	R5397509
Sulfur (as SO4)	2770		6.0	mg/L		06-MAR-21	R5397526
Total Sulphate Ion Content	0.250		0.050	%	01-MAR-21	01-MAR-21	R5395612
Water Soluble Sulphate Ion Content	0.319		0.050	%	03-MAR-21	04-MAR-21	R5397192
pH in Saturated Paste	7.80		0.10	pH		06-MAR-21	R5397508
Salinity in mg/kg							
Chloride (Cl)	<25		25	mg/kg		06-MAR-21	
Sulfur (as SO4)	3420		7.4	mg/kg		06-MAR-21	
L2561454-2 COE - QUEEN ALEX - TH21-03 #13							
Sampled By: CLIENT on 19-FEB-21							
Matrix: Soil							
Miscellaneous Parameters							
% Saturation	135		1.0	%	06-MAR-21	06-MAR-21	R5397522
Chloride (Cl)	<20		20	mg/L		06-MAR-21	R5397529
Resistivity	1100		1.0	ohm cm		06-MAR-21	R5397509
Sulfur (as SO4)	754		6.0	mg/L		06-MAR-21	R5397526
Total Sulphate Ion Content	0.086		0.050	%	01-MAR-21	01-MAR-21	R5395612
pH in Saturated Paste	8.10		0.10	pH		06-MAR-21	R5397508
Salinity in mg/kg							
Chloride (Cl)	<27		27	mg/kg		06-MAR-21	
Sulfur (as SO4)	1020		8.1	mg/kg		06-MAR-21	

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
CL-PASTE-COL-CL	Soil	Chloride in Soil (Paste) by Colorimetry	CSSS, APHA 4500-Cl E

A soil extract produced by the saturated paste extraction procedure is analyzed for Chloride by Colourimetry.

ALS Test Code	Matrix	Test Description	Method Reference**
PH-PASTE-CL	Soil	pH in Saturated Paste	CSSS Ch. 15

A soil extract produced by the saturated paste extraction procedure is analyzed by pH meter.

ALS Test Code	Matrix	Test Description	Method Reference**
RESISTIVITY-PASTE-CL	Soil	PASTE RESISTIVITY	ASTM G57-95A

This analysis is carried out using procedures adapted from ASTM G57-95a (2001) "Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method". In summary, 200 to 500 grams of sample is mixed with deionized water as required to create a saturated paste. The sample is then placed directly into a four electrode resistivity soil box and measured for resistivity using a resistivity meter.

ALS Test Code	Matrix	Test Description	Method Reference**
SAL-MG/KG-CALC-CL	Soil	Salinity in mg/kg	Manual Calculation
SAT-PCNT-N-CL	Soil	% Saturation	CSSS Ch. 15

Saturation Percentage (SP) is the total volume of water present in a saturated paste (in mL) divided by the dry weight of the sample (in grams), expressed as a percentage, as described in "Soil Sampling and Methods of Analysis" by M. Carter.

ALS Test Code	Matrix	Test Description	Method Reference**
SO4-PASTE-ICP-CL	Soil	Sulphate (SO4)	CSSS CH15/EPA 6010D

A soil extract produced by the saturated extraction procedure is analyzed for sulfate by ICPOES.

ALS Test Code	Matrix	Test Description	Method Reference**
SO4-S-CSA-A23-ED	Soil	Water Soluble Sulphate Ion Content	CSA INTERNATIONAL A23.2-3B

Soluble sulphate ion content is determined by agitating the soil with water at a specific ratio determined by a preceding total sulphate ion content test, for 6 hours. Analysis by ion chromatography follows.
NOTE: the CSA-A23 method states that for a total sulphate ion content greater than 0.2%, soluble sulphate ion content shall be determined on the basis of a water extraction. This water extraction requires the total sulphate ion content result to calculate the correct ratio for the water extraction.

ALS Test Code	Matrix	Test Description	Method Reference**
SO4-T-CSA-A23-ED	Soil	Total Sulphate Ion Content	CSA INTERNATIONAL A23.2-3B

Total sulphate content is determined by mixing soil with water then hydrochloric acid, and digesting just below boiling point, for 15 minutes. Analysis by ion chromatography follows.
NOTE: the CSA-A23 method states that for a total sulphate ion content greater than 0.2%, soluble sulphate ion content shall be determined on the basis of a water extraction. This water extraction requires the total sulphate ion content result to calculate the correct ratio for the water extraction.

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
ED	ALS ENVIRONMENTAL - EDMONTON, ALBERTA, CANADA
CL	ALS ENVIRONMENTAL - CALGARY, ALBERTA, CANADA

Chain of Custody Numbers:

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
mg/kg - milligrams per kilogram based on dry weight of sample
mg/kg wwt - milligrams per kilogram based on wet weight of sample
mg/kg lwt - milligrams per kilogram based on lipid-adjusted weight
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Quality Control Report

Client: AECOM Canada Ltd.
Suite 300, 48 Quarry Park Blvd SE
Calgary AB T2C 5P2

Contact: Chris Keeley

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
CL-PASTE-COL-CL	Soil							
Batch R5397529								
WG3497948-4 IRM		**SAL-STD10**						
Chloride (Cl)			90.7		%		70-130	06-MAR-21
WG3497948-3 LCS								
Chloride (Cl)			104.6		%		70-130	06-MAR-21
WG3497948-1 MB								
Chloride (Cl)			<20		mg/L		20	06-MAR-21
PH-PASTE-CL	Soil							
Batch R5397508								
WG3497948-4 IRM		**SAL-STD10**						
pH in Saturated Paste			7.42		pH		6.94-7.54	06-MAR-21
WG3497948-3 LCS								
pH in Saturated Paste			7.00		pH		6.7-7.3	06-MAR-21
RESISTIVITY-PASTE-CL	Soil							
Batch R5397509								
WG3498088-2 IRM		**SAL-STD10**						
Resistivity			80.2		%		70-130	06-MAR-21
WG3498088-1 LCS								
Resistivity			98.2		%		70-130	06-MAR-21
SAT-PCNT-N-CL	Soil							
Batch R5397522								
WG3497947-3 IRM		**SAL-STD10**						
% Saturation			99.9		%		70-130	06-MAR-21
WG3497947-1 MB								
% Saturation			<1.0		%		1	06-MAR-21
SO4-PASTE-ICP-CL	Soil							
Batch R5397526								
WG3497948-4 IRM		**SAL-STD10**						
Sulfur (as SO4)			108.5		%		70-130	06-MAR-21
WG3497948-3 LCS								
Sulfur (as SO4)			85.6		%		80-120	06-MAR-21
WG3497948-1 MB								
Sulfur (as SO4)			<6.0		mg/L		6	06-MAR-21
SO4-S-CSA-A23-ED	Soil							
Batch R5397192								
WG3496334-3 IRM		**ALS SAL 2019**						
Water Soluble Sulphate Ion Content			119.2		%		70-130	04-MAR-21
WG3496334-2 LCS								

Quality Control Report

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
SO4-S-CSA-A23-ED	Soil							
Batch R5397192								
WG3496334-2 LCS								
Water Soluble Sulphate Ion Content			102.0		%		70-130	04-MAR-21
WG3496334-1 MB								
Water Soluble Sulphate Ion Content			<0.050		%		0.05	04-MAR-21
SO4-T-CSA-A23-ED	Soil							
Batch R5395612								
WG3494771-3 CRM		**ED-634A_CEMENT**						
Total Sulphate Ion Content			107.9		%		80-120	01-MAR-21
WG3494771-4 DUP		**L2561454-2**						
Total Sulphate Ion Content		0.086	0.058	J	%	0.028	0.1	01-MAR-21
WG3494771-2 LCS								
Total Sulphate Ion Content			99.2		%		70-130	01-MAR-21
WG3494771-1 MB								
Total Sulphate Ion Content			<0.050		%		0.05	01-MAR-21

Quality Control Report

Legend:

Limit	ALS Control Limit (Data Quality Objectives)
DUP	Duplicate
RPD	Relative Percent Difference
N/A	Not Available
LCS	Laboratory Control Sample
SRM	Standard Reference Material
MS	Matrix Spike
MSD	Matrix Spike Duplicate
ADE	Average Desorption Efficiency
MB	Method Blank
IRM	Internal Reference Material
CRM	Certified Reference Material
CCV	Continuing Calibration Verification
CVS	Calibration Verification Standard
LCSD	Laboratory Control Sample Duplicate

Sample Parameter Qualifier Definitions:

Qualifier	Description
J	Duplicate results and limits are expressed in terms of absolute difference.

Hold Time Exceedances:

All test results reported with this submission were conducted within ALS recommended hold times.

ALS recommended hold times may vary by province. They are assigned to meet known provincial and/or federal government requirements. In the absence of regulatory hold times, ALS establishes recommendations based on guidelines published by the US EPA, APHA Standard Methods, or Environment Canada (where available). For more information, please contact ALS.

The ALS Quality Control Report is provided to ALS clients upon request. ALS includes comprehensive QC checks with every analysis to ensure our high standards of quality are met. Each QC result has a known or expected target value, which is compared against pre-determined data quality objectives to provide confidence in the accuracy of associated test results.

Please note that this report may contain QC results from anonymous Sample Duplicates and Matrix Spikes that do not originate from this Work Order.

Chain of Custody (COC) / Analytical Request Form

ALS Environmental
www.alsglobal.com

Canada Toll Free: 1 800 668 9878

L2561454-COFC

COC Number: 14 -

Page 1 of 1

Report To		Report Format / Distribution		Select Service Level Below (Rush Turnaround Time (TAT) is not available for all tests)
Company:	AECOM Canada Ltd. (acct# 10482)	Select Report Format:	☑ PDF ☐ EXCEL ☐ EDD (DIGITAL)	R ☑ Regular (Standard TAT if received by 3 pm - business days)
Contact:	Chris Keeley	Quality Control (QC) Report with Report	☑ Yes ☐ No	P ☐ Priority (2-4 bus. days if received by 3pm) 50% surcharge - contact ALS to confirm TAT
Address:	48 Quarry Park Blvd. SE, Suite 300	☐ Criteria on Report - provide details below if box checked		E ☐ Emergency (1-2 bus. days if received by 3pm) 100% surcharge - contact ALS to confirm TAT
	Calgary, AB T2C 5P2	Select Distribution:	☑ EMAIL ☐ MAIL ☐ FAX	E2 ☐ Same day or weekend emergency - contact ALS to confirm TAT and surcharge
Phone:	403.254.3301	Email 1 or Fax	Chris.Keeley@aecom.com	Specify Date Required for E2,E or P:
		Email 2	alex.tam2@aecom.com	Analysis Request

Invoice To		Invoice Distribution	
Same as Report To	☐ Yes ☑ No		
Copy of Invoice with Report	☐ Yes ☑ No	Select Invoice Distribution:	☑ EMAIL ☐ MAIL ☐ FAX
Company:	AECOM Canada Ltd.	Email 1 or Fax	Kristen.Tackney@aecom.com
Contact:	Kristen.Tackney@aecom.com	Email 2	

Project Information		Oil and Gas Required Fields (client use)	
ALS Quote #:		Approver ID:	Cost Center:
Job #:	City of Edmonton - Queen Alex - 60652243	GL Account:	Routing Code:
PO / AFE:		Activity Code:	
LSD:		Location:	
ALS Lab Work Order # (lab use only)	459	ALS Contact: Nelson Kwan	Sampler: N/A

Indicate Filtered (F), Preserved (P) or Filtered and Preserved (F/P) below

ALS Sample # (lab use only)	Sample Identification and/or Coordinates (This description will appear on the report)	Date (dd-mmm-yy)	Time (hh:mm)	Sample Type	CL-PASTE-COL-CL	PH-PASTE-CL	RESISTIVITY-PASTE-CL	SAL-MG/KG-CALC-CL	SAT-PCNT-CL	SO4-PASTE-ICP-CL	SO4-S-CSA-A23-ED	SO4-T-CSA-A23-ED	Number of Containers
	COE - Queen Alex - TH21-01 #4	19-Feb-20	-	Soil	R	R	R	R	R	R	R	R	1
	COE - Queen Alex - TH21-03 #13	19-Feb-20	-	Soil	R	R	R	R	R	R	R	R	1

Drinking Water (DW) Samples[1] (client use)

Are samples taken from a Regulated DW System?
☐ Yes ☐ No

Are samples for human drinking water use?
☐ Yes ☐ No

Special Instructions / Specify Criteria to add on report (client Use)

Please report sulphate results in % for SO4-T/S CSA method and results in mg/L for SO4 via paste, salinity package. Please report resistivity in ohm-cm. Please report chlorides in mg/kg & mg/L

SAMPLE CONDITION AS RECEIVED (lab use only)

Frozen ☐ SIF Observations Yes ☐ No ☐
Ice packs Yes ☐ No ☐ Custody seal intact Yes ☐ No ☐
Cooling Initiated ☐

INITIAL COOLER TEMPERATURES °C	FINAL COOLER TEMPERATURES °C
10	

SHIPMENT RELEASE (client use)			INITIAL SHIPMENT RECEPTION (lab use only)			FINAL SHIPMENT RECEPTION (lab use only)		
Released by: [signature]	Date: Feb.25/21	Time: 11:39	Received by: [signature]	Date: 2/25	Time: 2:50	Received by:	Date:	Time:

REFER TO BACK PAGE FOR ALS LOCATIONS AND SAMPLING INFORMATION WHITE - LABORATORY COPY YELLOW - CLIENT COPY

NA-FM-0326e v09 Front/04 January 2014

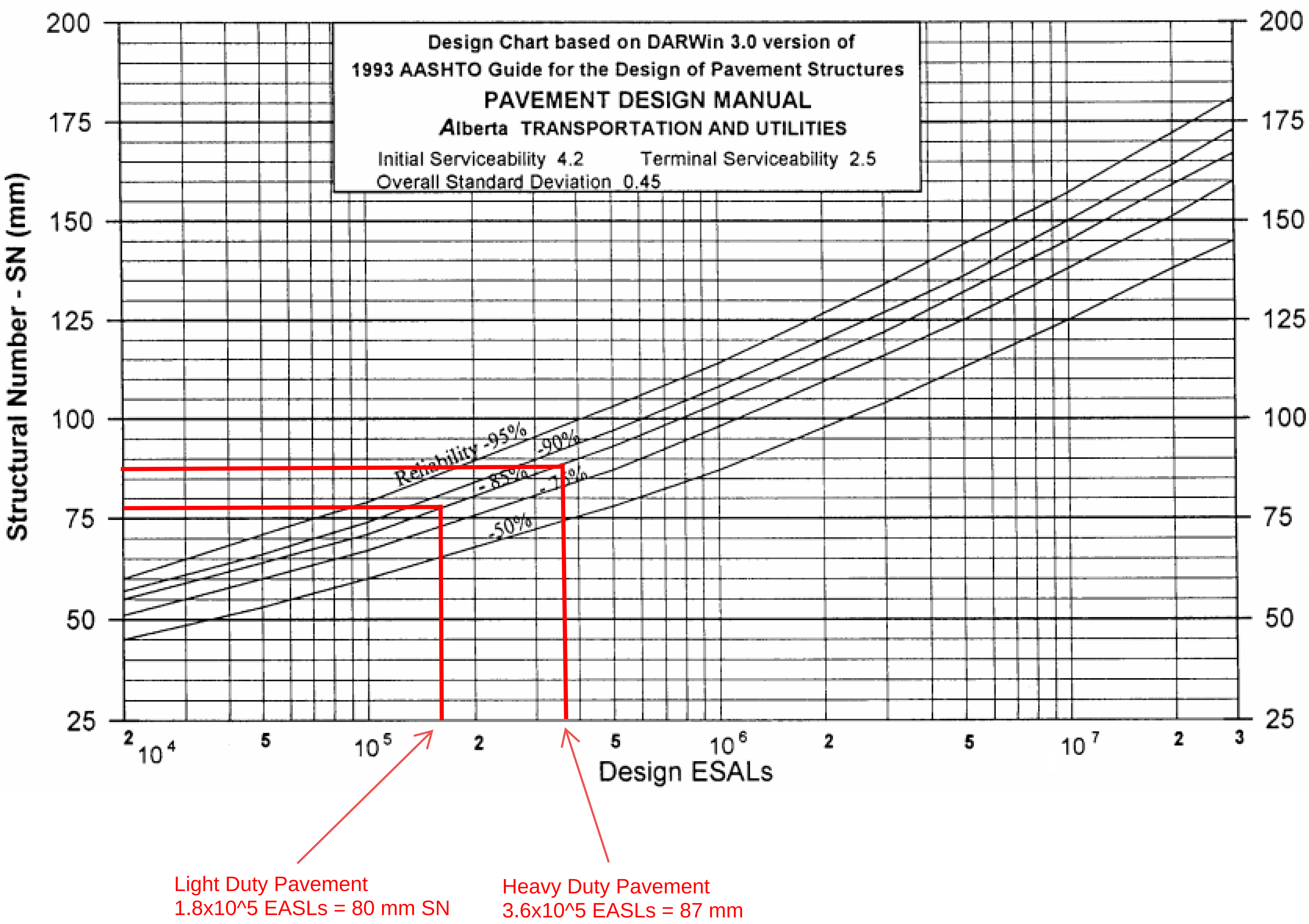
Failure to complete all portions of this form may delay analysis. Please fill in this form LEGIBLY. By the use of this form the user acknowledges and agrees with the Terms and Conditions as specified on the back page of the white - report copy.

1. If any water samples are taken from a **Regulated Drinking Water (DW) System**, please submit using an **Authorized DW COC form**.

Appendix D

Alberta Transportation Structural Number and ESALs Figure

Structural Number for Effective Roadbed Resilient Modulus of 30 MPa



Brian Nguyen, P.Eng.
Geotechnical Engineer
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E: Brian.Nguyen@aecom.com

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