CP-008872 Garneau Housing Development Preliminary Geotechnical Investigation – Final

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 Garneau Housing project. It is understood that the CoE intends to develop the two lots at 11053 and 11049 – 83 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 23 metres (m). 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 6.25 and 14.94 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.
    - Conclusion on the results of the geotechnical 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 conducting 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 March 23, 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 Garneau Housing project site. The three testholes, TH21-01, TH21-02, and TH21-03, were drilled to depths of 14.94 mBGS, 6.25 mBGS and 14.94 mBGS respectively, on March 25, 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-mm diameter polyvinyl chloride (PVC) monitoring well was installed in testhole TH21-01 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 25, 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	11049 - 83 Ave NW	14.94	5931893.3	32002.1	670.8	Y
TH21-02	11053 - 83 Ave NW	6.25	5931878.8	31996.9	670.8	N
TH21-03	11053 - 83 Ave NW	14.94	5931861.5	31992.2	670.8	N

1 Coordinates and elevations 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, 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	49	Testhole Logs and Appendix C
Atterberg limits determination on selected soil samples	4	Testhole Logs and Appendix C
Grain Size Analysis on selected samples	3	Testhole Logs and Appendix C
Soil Chemical Testing	2	Testhole Logs and Appendix C

3. Subsurface Conditions

3.1 Topsoil

Topsoil was encountered at the ground surface in testhole TH21-01 and was 760 mm thick. The topsoil was silty, contained some clay, and contained trace sand. The topsoil also contained some rootlets and was moist, and dark brown in colour.

3.2 Clay Fill

Clay fill was encountered at the ground surface in testholes TH21-02 and TH21-03, and varied in thickness of between 0.61 m and 1.52 m. The clay fill was silty and sandy to containing some sand, and occasionally contained some gravel. The clay fill was also noted to be of high plasticity, and contained some wood debris, trace coal, trace brick debris, trace silt layers and trace rootlets. The clay fill was damp to wet, and brown to dark brown in colour.

One SPT was completed in the clay fill and was 6 blows per 300 mm of penetration, indicating the clay fill was firm. Moisture contents were determined on three clay fill samples and the results varied from 15.2% to 30.9%.

3.3 Clay

Clay was encountered in all testholes and the thickness varied between 1.53 m and 1.68 m. The clay was silty and contained some sand. Occasionally, the clay contained some to trace silt pockets and trace coal. The clay was moist and brown in colour. The clay was noted to be of high plasticity.

SPT N-values for the clay ranged from 10 to 15 blows per 300 mm of penetration, indicating the clay was stiff to very stiff. Moisture contents were determined on six samples and varied from 23.3% to 30.0%. Two Atterberg Limits tests were completed on the clay and the results are summarized in **Table 3-1** below.

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

Testhole	Sample Number	Depth (mBGS)	MUSCS	Moisture (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
TH21-01	3	1.52 – 1.83	CH	25.2	71.4	21.1	50.3
TH21-03	3	1.52 – 1.83	CH	27.4	67.1	21.9	45.2

3.4 Clay Till

Clay till was encountered in all testholes below the clay layer and varied in thickness between 2.74 m and 3.04 m. The clay till was silty and sandy to containing some sand. Occasionally, the clay till contained trace gravel, some sand and silt laminations, trace to some oxidation and trace coal. The clay till was moist to wet, and brown in colour. The clay till was noted to of high plasticity.

SPT N-values for the clay till ranged from 3 to 24 blows per 300 mm of penetration, indicating the clay till was soft to very stiff. Moisture contents were determined on 12 clay till samples and the results varied from 19.7% to 36.7%.

3.5 Sand and Silt

Sand and silt was encountered in testhole TH21-01 below the silt layer. The thickness of the sand and silt was 3.96 m. The sand and silt contained trace clay and was fine grained. The sand and silt was damp and brown in colour. Moisture contents were determined on six samples and the results varied from 10.3% to 21.6%. One grain size analysis was completed on the sand and silt and the results are summarized in **Table 3-2** below.

Table 3-2: Summary Grain Size Analyses Test Results for Sand and Silt

Testhole	Sample Number	Depth (mBGS)	MUSCS	Moisture (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
TH21-01	13	9.14 – 9.45	SM-ML	12.6	0.0	46.2	46.1	7.7

3.6 Silt

Silt was encountered in all testholes below the clay till layer and varied in thickness between 0.46 m and 5.03 m. Testhole TH2-02 was terminated in the silt. The silt was sandy and contained some clay. The silt was damp and light brown to brown in colour. The silt was noted to be of low plasticity.

SPT N-values for the silt ranged from 15 to 32 blows per 300 mm of penetration, indicating the silt was compact to dense. Moisture contents were determined on 11 silt samples and the results varied from 7.5% to 25.9%. Two Atterberg Limits tests and two grain size analyses were completed on the silt and the results are summarized in **Table 3-3** below.

Table 3-3: Summary of Atterberg Limits Test and Grain Size Analyses Test Results for Silt

Testhole	Sample Number	Depth (mBGS)	MUSCS	Moisture (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
TH21-01	9	6.10 – 6.40	ML	11.4	0.0	23.8	59.9	16.3	19.9	18.6	1.3
TH21-03	8	5.33 – 5.79	CL-ML	17.1	-	-	-	-	25.1	20.6	4.5
TH21-03	12	8.38 – 8.84	ML	17.8	0.5	21.3	65.9	12.3	-	-	-

3.7 Sandstone

Weathered sandstone was encountered in testholes TH21-01 and TH21-03. Both testholes TH21-01 and TH21-03 were terminated in the sandstone layer. The sandstone was silty and poorly lithified. The sandstone was damp and grey in colour.

SPT N-values for the sandstone ranged from 71 to 81 blows per 300 mm of penetration, indicating the sandstone was very dense. Moisture contents were determined on 11 sandstone samples and the results varied from 13.7% to 21.0%.

3.8 Groundwater

Groundwater levels were measured upon completion of drilling on March 25, 2020 and 19 days after on April 13, 2021. The results of the groundwater measurements are summarized in **Table 3-4**.

Table 3-4: Summary of Groundwater Measurements

Testhole	Testhole Elevation (mASL)	Depth of Standpipe (mBGS)	Upon Completion of Drilling March 25, 2021 (mBGS)	Groundwater Depth During Monitoring Event on April 13, 2021 (mBGS)	Groundwater Elevation During Monitoring Event on April 13, 2021 (mASL)
TH21-01	670.826	14.94	Trace groundwater at bottom of testhole	10.84	660.0
TH21-02	670.819	-	Trace groundwater at bottom of testhole	-	-
TH21-03	670.762	-	Trace groundwater at bottom of testhole	-	-

- No monitoring wells installed in testholes TH21-02 and TH21-03.

Measured groundwater depths are also shown on the testhole logs in **Appendix B**. It should be noted that the groundwater levels in **Table 3-4** 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 fill (CH), clay (CH) and clay till (CH). 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 3-5**.

Table 3-5: Frost Susceptibility

Soil Unit	MUSC	Finer than 0.02 mm (%)	Plasticity Index (%)	Frost Group
Clay Fill, Clay, Clay Till	CH	-	-	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 (4^{th} 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 Garneau Housing site is summarized in **Table 3-6**.

Table 3-6: Frost Penetration Depth

Soil Unit	Frost Penetration Depth (m)
Clay	2.5[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 3-7** below in accordance with the Handbook of Corrosion Engineering (Roberge, P. R., 2000) and the Canadian Standards Association Guidelines (CSA, 2018).

Table 3-7: 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	7	4.57 – 4.88	1640	23	<0.050	7.66	Highly Corrosive	Low
TH21-03	10	6.86 – 7.32	1550	59	<0.050	7.56	Highly Corrosive	Low

Based on the above test results, the degree of corrosivity is expected to be highly corrosive at this site. The potential for sulphate attack in concrete is expected to be low 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 TH21-01 and TH21-03 was used for the seismic site classification. The soil stratigraphy at this site consisted of clay, clay till, silt and sandstone. Testholes TH21-01 and TH21-03 were 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 testholes TH21-01 and TH21-03 continues to a depth of 30 mBGS
- The average SPT of 76 blows per 300 mm of penetration in Testholes TH21-01 and TH21-03 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.

3.13 Liquefaction Potential

Soil liquefaction is a process where soils may suddenly, and drastically lose their strength in response to seismic activity or earthquake loadings. Soils that are most susceptible to liquefaction include:

- Loose and cohesionless soils (sands, gravels, and silts)
- Saturated soils
- Unconsolidated soils
- Soils containing a high fines content (Poorly drained soils)

During the investigation, sand and silt was encountered near the surface in TH21-01 at 7.6 mBGS and silt was encountered in all testholes at variable depths. Liquefaction assessments are generally recommended if loose or saturated sands are encountered, or sand containing a large percentage of fines is present. The fines content within one sand sample in TH21-01 at the proposed housing site was 53.8%, which is considered a high fines content.

The liquefaction potential of the sands was assessed following the procedure outlined by the CFEM (2006), Screening Guide for Rapid Assessment of Liquefaction Hazard at Highway Bridge Sites (Technical Report MCEER-98-0005, dated June 16, 1998), and Youd et al. (2001). Three key parameters are required for assessing a site for liquefaction: the $(N_1)_{60}$ value, the peak horizontal acceleration, and the design earthquake magnitude. The peak horizontal ground acceleration for the site was obtained using the NBCC (2015) seismic hazard value interpolator from the Natural Resources Canada website. The peak horizontal ground acceleration was found to be 0.1g, where g is the acceleration of gravity. The largest magnitude earthquake that was recorded near Edmonton, AB between the years 1627 and 2015 had a magnitude of 6. An earthquake magnitude of 6 was therefore used as the design earthquake. The magnitude of earthquake was obtained from the Earthquake Canada Website which includes maps showing the historic locations and magnitudes of earthquakes in Canada.

Based on our assessment, the factor of safety against liquefaction is greater than 1.5, which indicates a low hazard risk with respect to liquefaction.

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 23 metres. Shallow foundations founded within 5.3 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. Shallow foundations are suitable if founded below 5.3 mBGS. Based on the soil conditions encountered during this geotechnical investigation, the primary geotechnical risks with the proposed site include:

- The near surface clay fill, clay and clay till encountered at this site was soft to stiff, with the moisture content of this clay varying from 15.2% to 36.7%. This moisture content is considered high relative to the native clay and 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 sandy and silty soils was noted in TH21-01 and TH21-03. Groundwater depth was measured at 10.84 mBGS in TH21-01 within the sand layer. Wet or saturated sandy and silty 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 TH21-01, TH21-02 and 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, which may be problematic for pavement structures.
- 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.
- Sand and silt soils have the potential for liquefaction under seismic loading. (Edmonton, AB is not known to have a high frequency of seismic activity, but the risk of liquefaction should not be completely ignored.)

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 **Section 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 a geotechnical representative 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 665.5 m or below (5.3 mBGS). The scarified 150 mm layer below the foundation base should be moisture conditioned to between 0 and 2% above the Optimum Moisture Content (OMC) and recompacted to 98% 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%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% 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 required side slopes 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 stability 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

Groundwater was measured during the geotechnical investigation to be at 10.83 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 topsoil, clay fill and high plasticity clay and clay till. The topsoil should be excavated and stockpiled separately from the underlying clay and clay till and can be used for future landscaping purposes. The surficial clay fill 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 2.5 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 plasticity clay or clay till, as fine-grained soils offer better frost protection than granular soil.

5. Preliminary Foundation Recommendations

Strip footings and raft foundations are feasible for lightly and moderately loaded structures respectively, which can be expected for low rise housing buildings. 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, which would require relatively deep excavations, dewatering and concrete form works, pile foundations are likely to be more cost effective than footings or rafts.

The soil conditions commencing at depths of 2.5 to 6 mBGS are generally favorable for pile foundations if heavily loaded structures such as a mid to high rise building will be selected to construct the housing building. However, silts and sand encountered below the groundwater level may present some concerns if not properly addressed. 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 due to the sandy conditions noted within the subsurface. Based on the testholes from this investigation, belled concrete piles are also not considered to be suitable at the site, due to the presence of very dense sandstone at approximate depths of 10.3 mBGS and 11.6 mBGS and forming the bell in the sandstone would be difficult to construct. Therefore, straight shaft cast-in-place piles are considered more practical to be used for this site. Construction of straight shaft concrete piles will need to incorporate contingencies for proper installation including temporary steel casings, groundwater handling, and concrete by tremie methods.

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 Subgrade Preparation for Shallow Foundations

The presence of high plasticity clay within the subsurface complicates the subgrade requirements at this site for shallow 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 loading. 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 mBGS, a significant amount of high plasticity clay will be required to be removed. 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.

5.2 Strip Footings

Strip footings can be used for lightly loaded structures for low rise housing building if founded on competent soils (compact silt) and where some settlement and/or differential settlement can be tolerated. The elevation of competent soil is expected to be at 665.5 m and below. Strip footings founded at a higher elevation may be possible if some ground improvements or soil replacement is completed at this site prior to placement of the strip footing. In all instances, strip footings should be founded at least below the seasonal frost depth. The minimum footing widths should be 600 mm for strip footings. Footings supporting heated structures should have a minimum soil cover of 1.5 m below the finished ground level to provide adequate protection against frost. For unheated structures, exterior and interior footings should be founded at a minimum depth of 2.5 m below the floor slab level or frost mitigation measures installed (such as insulation) to minimize potential of frost effects on footings.

The estimated ultimate bearing capacities for typical strip is provided in **Table 5-1** for footings founded on compact native silt compacted to 100% SPMDD within ± 2% of the OMC. For working stress design, a factor of safety of 3 should be applied to the ultimate bearing capacity. For ULS design, a resistance factor of 0.5 should be used on the ultimate bearing capacities to obtain the factored bearing capacity.

Table 5-1: Ultimate Bearing Capacities for 0.6 m Wide Strip Footing

Foundation Elevation (mASL)	Ultimate Bearing Capacity (kPa)	Factored Ultimate Bearing Capacity (kPa)[1]
665.5 – 655.9	400	200

[1]Assumed Friction Angle ($\emptyset = 27°$), groundwater assumed to be below footing base due to subsurface drainage. The above ultimate bearing capacity would be reduced by 50% if groundwater is present at the footing base.

The estimated total settlement for the foundations discussed in **Table 5-1** is expected to be less than 25 mm if the applied load does not exceed 125 kPa. More detailed settlement estimates should be established from a detailed investigation once the building size, foundation elevation, and building type are determined.

5.3 Raft Foundations

5.3.1 General

Raft foundation are feasible for moderately loaded structures such as a mid-rise housing building. If raft foundations are selected to construct the housing building, it is recommended that raft foundations be founded at an elevation of at least 665.5 m (5.3 mBGS) or below. Raft foundations may be designed using a factored ultimate bearing capacity and subgrade reaction modulus values summarized in **Table 5-2.**

Table 5-2: Bearing Capacity and Subgrade Reaction for Raft Foundations

Raft Foundation Base Elevation (m)	Ultimate Bearing Capacity (kPa)	Factored Ultimate Bearing Capacity (kPa)[1]	Subgrade Reaction Modulus (kN/m^3)
665.5 – 655.9	600	300	15,000

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

For serviceability limits states design, the total settlement is expected to be less than 25 mm if the applied load does not exceed 150 kPa and subgrade preparation recommendations provided in this report are followed, with a minimum scarification depth of 150 mm. The total settlement of a raft foundation, if selected, should be determined during the detailed design phase when the building type, building size and foundation elevation are known. 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. A granular layer of 150 mm thick should be placed if silt is encountered below the raft base to obtain a stable base during construction.

5.3.2 Subgrade Protection

The base of the 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, if encountered, 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.

5.3.3 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 27° for silt)

5.3.4 Buoyant Uplift

Raft foundations may be prone uplift forces. Based on groundwater observations completed on April 13, 2021, the depth of the groundwater table was 10.8 mBGS (Elev. 660.0 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 m above observed ground water levels of 6.8 mBGS (Elev. 664.0 m) be used. Further groundwater monitoring is required to confirm the depth of the groundwater on site during the detailed design phase.

The magnitude of hydrostatic uplift forces applied to below grade structures should be calculated, assuming that the groundwater table is at 6.8 mBGS (Elev. 664.0 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 walls (m)
W = Width of projected base slab beyond 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)

5.4 Cast-in-Place (CIP) Concrete Piles

5.4.1 CIP Concrete Pile Design Parameters

Straight shaft drilled CIP concrete piles designed based only shaft friction or on a combination of shaft friction plus end bearing resistance is another foundation alternative considered suitable for the proposed housing building if a high-rise structure will be constructed. The use of casing may be required for cast-in-place concrete piles due to presence of water bearing sand and silt overlying the sandstone.

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 5-3** and the above equation.

Table 5-3: Ultimate Design Parameters for CIP Concrete Piles

Elevation (m)	Soil Type	Ultimate Skin Friction (kPa)[1]
670.8 – 668.3	Clay (within Frost Depth)	-
668.3 – 665.0	Clay, Clay Till	30
665.0 – 659.1	Silt/Sand	50
659.1 – 655.9	Sandstone	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 5-3** are considered applicable for downward (compressive) static loads. All piles should have a minimum diameter of 400 mm.

End bearing piles may be founded a minimum 1.5 m within the sandstone (below a depth of 11.6 m). The ultimate bearing pressure at this depth can be taken as 1500 kPa. For Ultimate Limit States (ULS) design, a resistance factor of 0.4 should be applied to the ultimate bearing pressure to obtain the factored end bearing pressure. The design may consider end bearing in addition to shaft friction as provided above in order to determine the total pile capacity.

5.4.2 CIP Concrete Pile Design and Construction Recommendations

The subsurface stratigraphy at the site generally consists of clay overlying clay till, overlying sand and silt, underlain by sandstone at an approximate elevation of 659 m. The groundwater was recorded at 660.0 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 some 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 sandstone at elevation 659 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 2.5 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 6 mBGS to adequately resist uplift pressures due to frost.
- Shaft resistance of CIP concrete piles should be designed using the parameters provided in **Table 5-3**.
- A minimum pile spacing of 3 times the shaft diameter is recommended for straight shaft piles.
- 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.
- 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.
- The presence of cobbles and boulders and boulder could impede the installation of drilled CIP piles. Cobbles and boulders were not encountered during this geotechnical investigation. However, the native clay till in Edmonton, AB is noted to occasionally contain cobbles and boulders. Hard drilling may be expected if cobbles and boulders are encountered and may require rock coring or chiselling with alternative heavy construction equipment.

5.4.3 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 in 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.

5.4.4 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 and clay till) 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 5-4**.

Table 5-4: Undrained Shear Strength of Soil Units

Soil Type	Elevation (m)	Undrained Shear Strength, S_u (kPa)
Clay/Clay Till	670.8 – 668.3	15 to 40
Silt/Silt and Sand	668.3 – 665.0	-

For cohesionless soils (sand, silt, and sand and gravel), k_s can be estimated using the following equation:

$k_s = n_h\ z/d$ (MN/m^3)
where:
z = Pile embedment depth (m)
d = Pile diameter (m)

The values for the factor n_h for cohesionless soils are summarized in the table below.

Table 5-5: Values of n_h for Cohesionless Soils

Soil Condition	n_h (MN/M[1])	
	Above Groundwater Table	Below Groundwater Table
Loose	2.5	1.5
Compact	7.0	4.5
Dense	18.0	11.0

[1] Values excerpted from Evaluation of Coefficient of Subgrade Reaction (Terzaghi, 1955).

Calculations for the coefficient of horizontal subgrade reaction along the length of the pile, used in determining lateral pile deformations will likely only include the cohesionless soil parameters described above.

5.4.5 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 5-3** 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).

5.4.6 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 6 m 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.

5.5 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 near surface 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 98% 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 to 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.

5.6 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 5-6**;
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 5-6**.

Table 5-6: 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	0.441
Clay/Clay Till (Low to Medium Plasticity)	18	25	0.577
Clay/Clay Till (High Plasticity)	18	21	0.642
Silt	19	30	0.500
Sand and Silt	19	30	0.500

5.7 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 6.8 mBGS (Elev. 664.0 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.

5.8 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 (**Table 3-6**) at this site. It is highly recommended additional sulphate testing be completed on imported fill used for construction at this site. While the potential for sulphate attack at this site was rated as low, all concrete in contact with soil at this site should be designed for an exposure class of S-3, as presented in **Table 5-7** to account for potential soil variability.

Table 5-7: 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 1550 ohm-cm and 1640 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.

5.9 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 Garneau Housing buildings and other 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 5-8**.

Table 5-8: 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.

5.10 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 Garneau Housing project may include 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, uncontrolled fill, 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 topsoil, clay fill and high plasticity clay. Compaction records of the clay fill were not made available to AECOM, therefore is considered to be uncontrolled. Additionally, the presence of organics, brick, wood debris and potential hydrocarbon staining within the clay fill indicate that the clay fill is not a suitable bearing stratum. The high plasticity clay is poor for use as a pavement structures bearing stratum, as these soils have the potential to swell and are typically more frost susceptible.

Following the stripping of the surficial topsoil and clay fill 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 0 to 3 percent of OMC. A layer of non-woven geotextile fabric is recommended between granular fill and the existing clay soil to prevent migration of fines from traffic that may cause pumping of the clay subgrade. Placement of the 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 (AASHTO). 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). Site specific data like EASLs will need to be re-evaluated by AECOM when this information is made available to confirm the optimum pavement structure.

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	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	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, resilient subgrade modulus of 30 MPa was selected for the design of the light and heavy-duty pavement structure.

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.

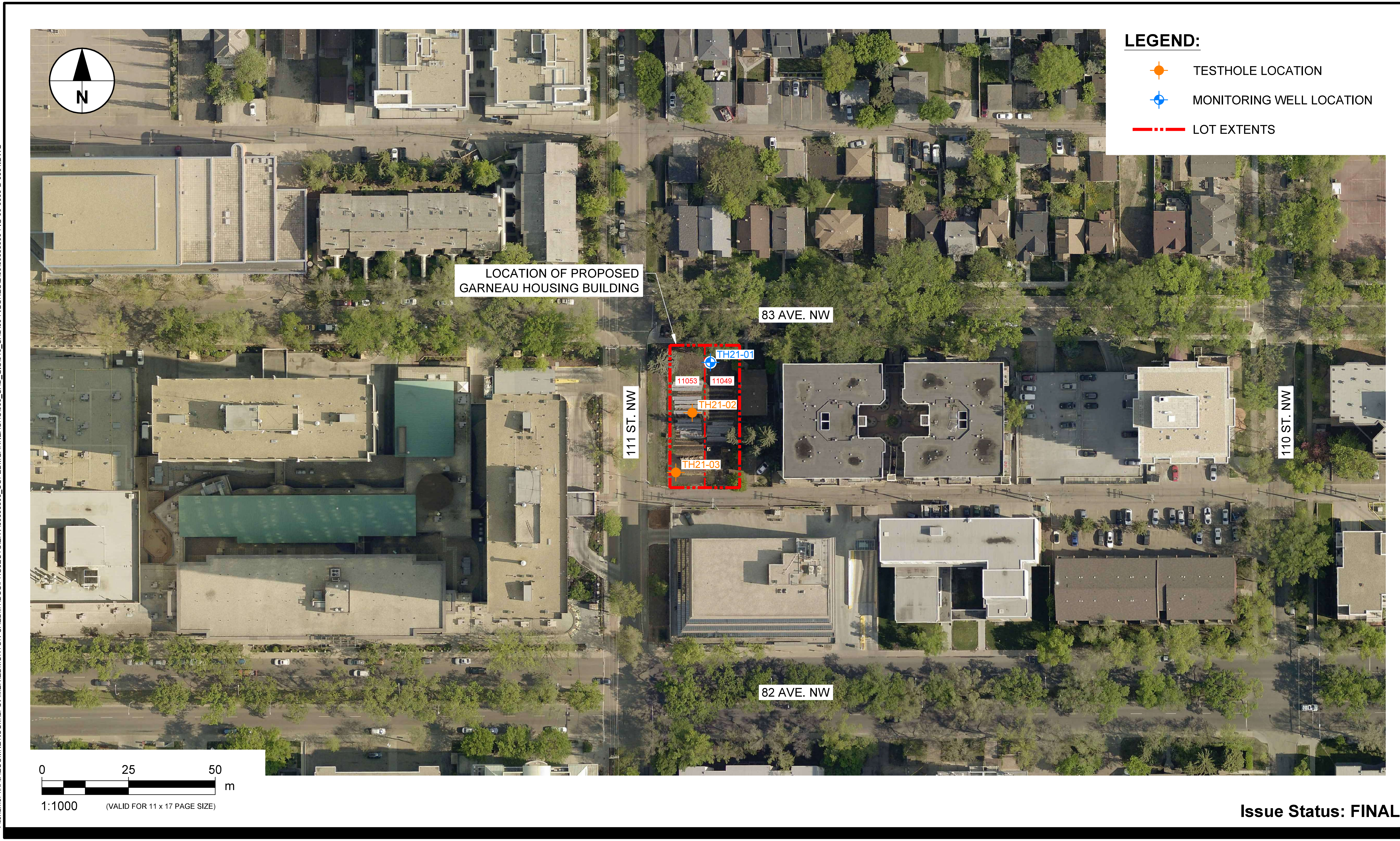
A non-woven filter fabric should be provided between the base of the granular fill and subgrade to prevent migration of fine materials into the granular fill.

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, strip footings, 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.

Appendix A

Testhole Location Plan



Garneau Housing
Preliminary Geotechnical Investigation
City of Edmonton
Project No.: 60655308 Date: 2021-05-05

TESTHOLE LOCATION PLAN

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Figure 1

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

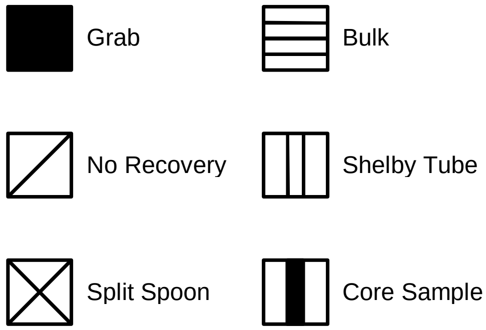
N	Consistency	C_u (kPa) approx.
0 - 1	Very Soft	<10
1 - 4	Soft	10 - 25
4 - 8	Firm	25 - 50
8 - 15	Stiff	50 - 100
15 - 30	Very Stiff	100 - 200
30 - 60	Hard	200 - 300
>60	Very Hard	>300

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 (▼).

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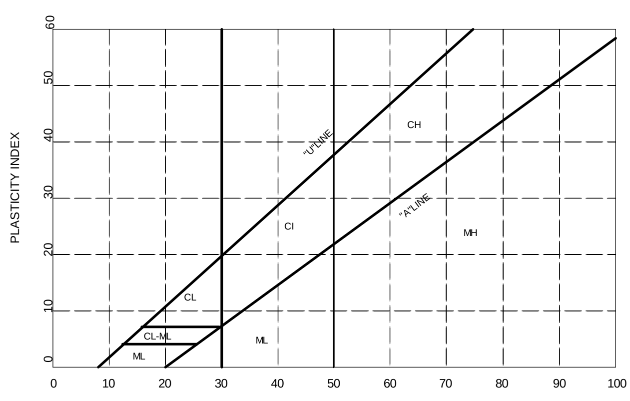
MAJOR DIVISION			LOG SYMBOLS	UCS	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS	GRAVELS (MORE THAN HALF COARSE GRAINS LARGER THAN 4.75 mm)	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4 \quad C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
				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 COBBLES 75 mm TO 300 mm BOULDERS >300 mm	ANGULAR 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%

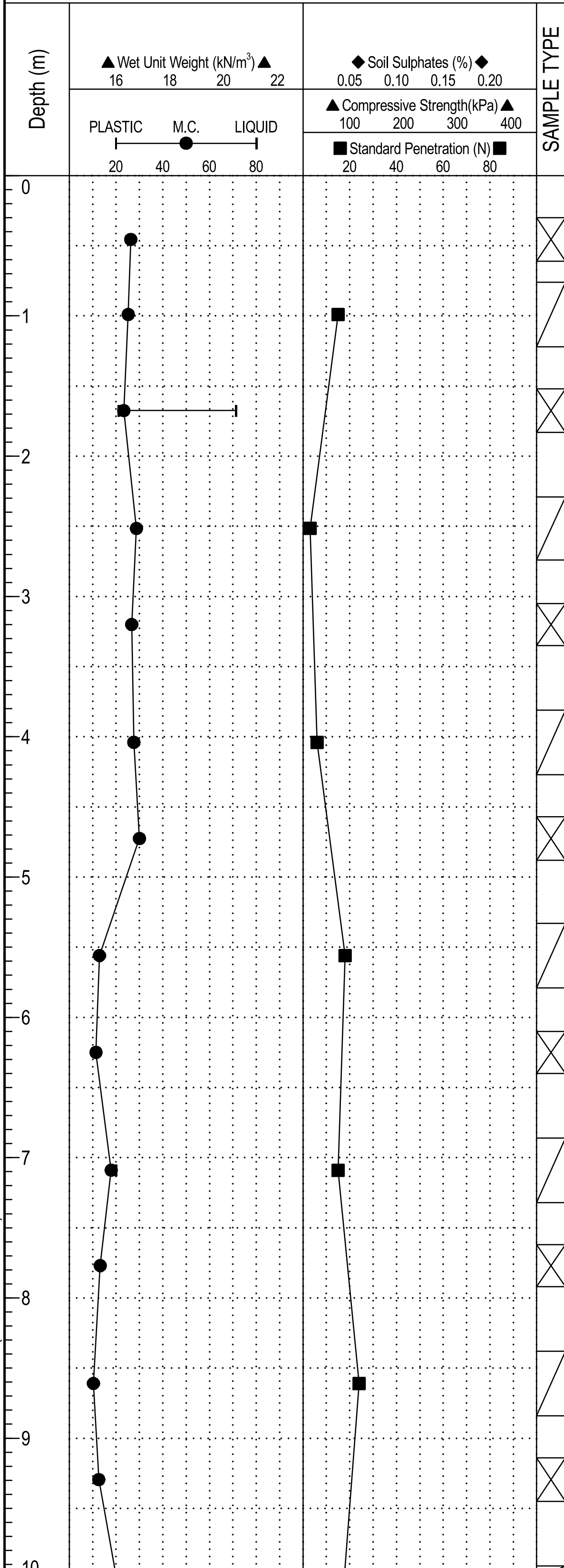
MODIFIED UNIFIED SOIL CLASSIFICATION SYSTEM

February 2021

Garneau Housing	11049 - 83 Ave NW	BOREHOLE NO: **TH21-01**
Integrated Infrastructure Services	3TM ZONE: N5931893.3 E32002.1	PROJECT NO: 60655308
START DATE: 2021/03/25	METHOD: Solid Stem Augers	ELEVATION: 670.8 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



SPT (N)	USC	SOIL DESCRIPTION
	OR	TOPSOIL (760 mm) - silty, some clay, trace sand, some rootlets, dark brown, moist
15	CH	CLAY - silty, some sand, high plasticity, stiff, frozen, some silt pockets, trace coal, brown, moist - numerous silt pockets
3, 6	CH	CLAY TILL - silty, sandy to some sand, trace gravel, high plasticity, soft, some sand and silt laminations, trace oxidation, brown, moist - silt content increasing - firm - wet
18, 15	ML	SILT - sandy, some clay, low to non plastic, compact, light brown, damp - clay content decreased to trace clay, sand content increased to sandy
24	SM	SAND AND SILT- trace clay, fine grained, compact, brown, damp

ELEVATION (m): 670, 669, 668, 667, 666, 665, 664, 663, 662, 661

THE CITY OF Edmonton

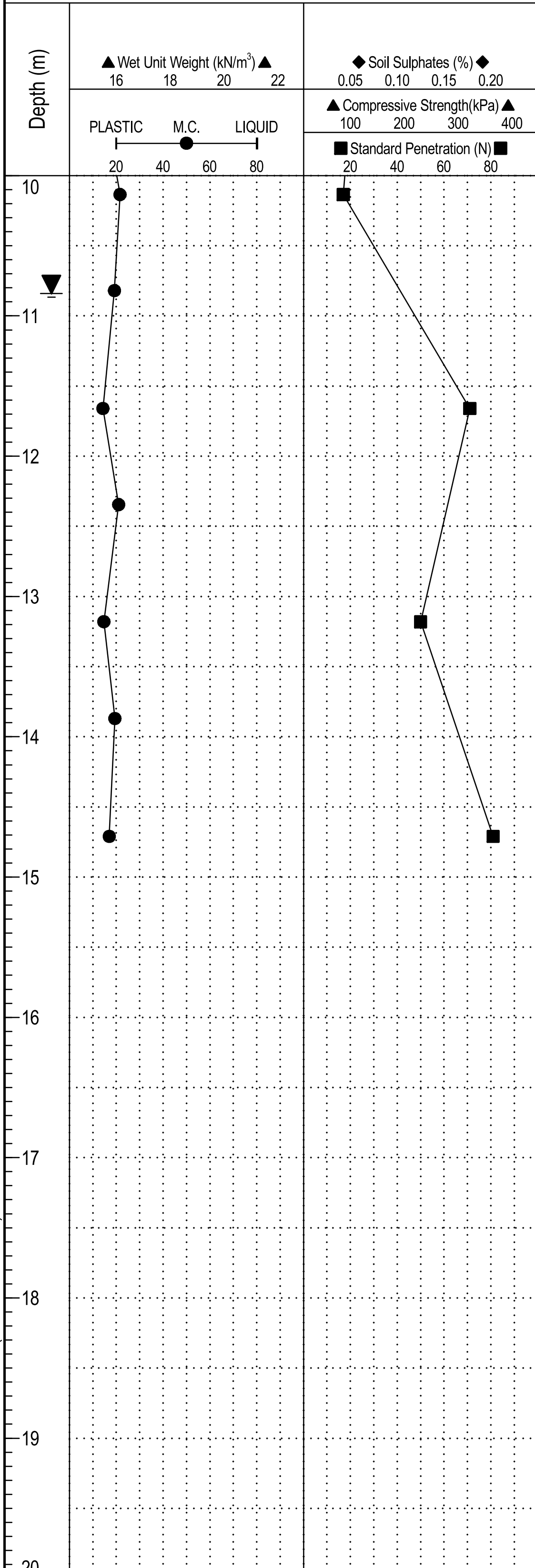
Integrated Infrastructure Services
Engineeering Services Section

LOGGED BY: AT	COMPLETION DEPTH: 14.94 m
REVIEWED BY: FA	COMPLETION DATE: 2021/03/25
Fig. No: 2	

Garneau Housing	11049 - 83 Ave NW	BOREHOLE NO: **TH21-01**
Integrated Infrastructure Services	3TM ZONE: N5931893.3 E32002.1	PROJECT NO: 60655308
START DATE: 2021/03/25	METHOD: Solid Stem Augers	ELEVATION: 670.8 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



SAMPLE TYPE	SPT (N)	USC	SOIL DESCRIPTION	ELEVATION (m)
Drive Sample	17		- moist	
Auger Sample			- trace clay, damp	660
Drive Sample	71		SANDSTONE - silty, very dense, poorly lithified, weathered, grey, damp, trace groundwater	659
Auger Sample			- moist	
				658
Drive Sample	50/125	BR	- damp	
Auger Sample				657
Drive Sample	81			656
			END OF TESTHOLE at 14.94 mBGS - testhole open upon drilling completion - trace groundwater encountered at bottom of testhole upon drilling completion - testhole drilled to scheduled depth - 25 mm diameter monitoring well installed to 14.94 mBGS	655
				654
				653
				652
				651

SLOTTED PIEZOMETER

THE CITY OF Edmonton

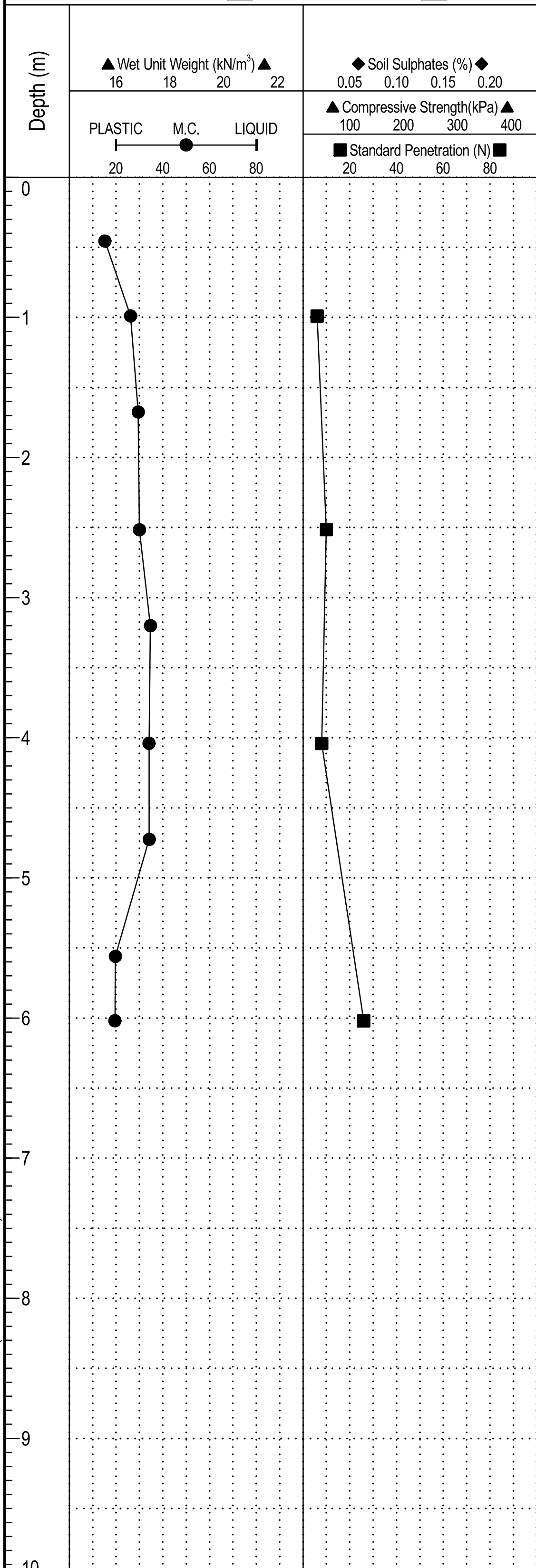
Integrated Infrastructure Services
Engineeering Services Section

LOGGED BY: AT	COMPLETION DEPTH: 14.94 m
REVIEWED BY: FA	COMPLETION DATE: 2021/03/25
Fig. No: 2	Page 2 of 2

Garneau Housing	11053 - 83 Ave NW	BOREHOLE NO: **TH21-02**
Integrated Infrastructure Services	3TM ZONE: N5931878.8 E31996.9	PROJECT NO: 60655308
START DATE: 2021/03/25	METHOD: Solid Stem Augers	ELEVATION: 670.8 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)
0 – 1.5	6	CH	CLAY FILL - silty, sandy, some gravel, high plasticity, firm, trace coal, trace brick, trace silt layers, trace rootlets, some black pockets, brown, damp - moist	670
1.5 – 3.05	10	CH	CLAY - silty, some sand, high plasticity, stiff, trace silt pockets, brown, moist - wet	669, 668
3.05 – 5.8	8	CH	CLAY TILL - silty, some sand, high plasticity, stiff, some oxidation, trace coal, brown, wet - damp	667, 666
5.8 – 6.25	26	ML	SILT - sandy, some clay, trace sand, compact, light brown, damp	665

END OF TESTHOLE at 6.25 mBGS
- testhole open and dry upon drilling completion
- testhole drilled to scheduled depth
- testhole backfilled with bentonite chips to ground surface

ELEVATION (m): 670, 669, 668, 667, 666, 665, 664, 663, 662, 661

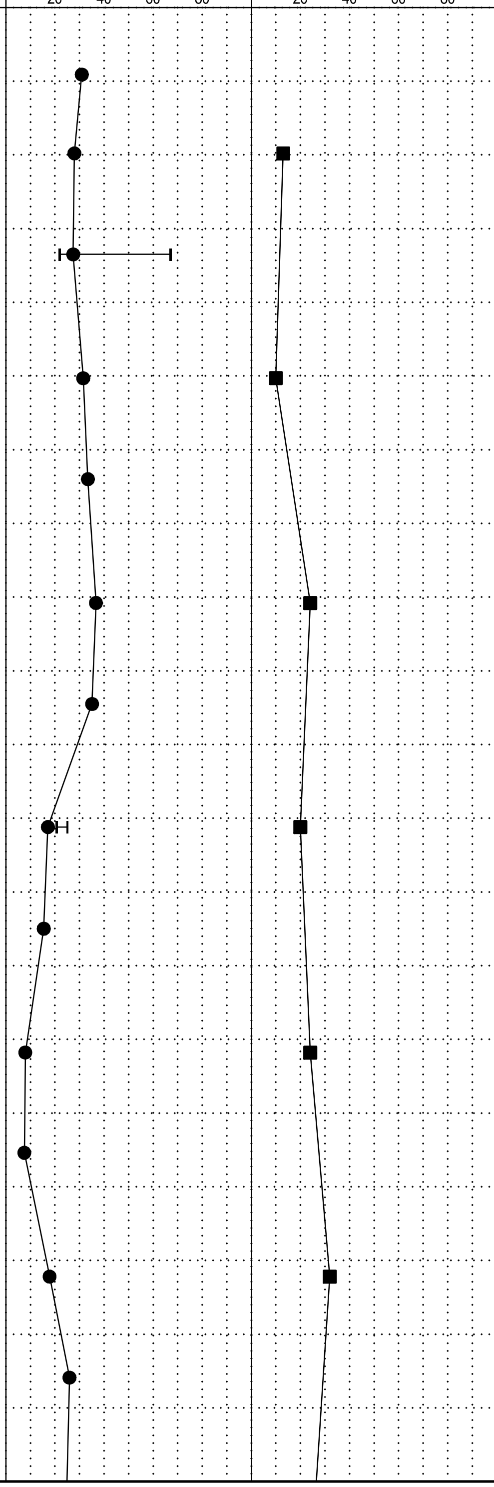
THE CITY OF Edmonton — Integrated Infrastructure Services, Engineeering Services Section	LOGGED BY: AT	COMPLETION DEPTH: 6.25 m
	REVIEWED BY: FA	COMPLETION DATE: 2021/03/25
	Fig. No: 2	Page 1 of 1

Garneau Housing	11053 - 83 Ave NW	BOREHOLE NO: **TH21-03**
Integrated Infrastructure Services	3TM ZONE: N5931861.5 E31992.2	PROJECT NO: 60655308
START DATE: 2021/03/25	METHOD: Solid Stem Augers	ELEVATION: 670.8 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) | ▲ Wet Unit Weight (kN/m^3) ▲ 16 18 20 22 | PLASTIC M.C. LIQUID 20 40 60 80 | ◆ Soil Sulphates (%) ◆ 0.05 0.10 0.15 0.20 | ▲ Compressive Strength(kPa) ▲ 100 200 300 400 | ■ Standard Penetration (N) ■ 20 40 60 80 | SAMPLE TYPE | SPT (N) | USC | SOIL SYMBOL | SOIL DESCRIPTION | ELEVATION (m)



Depth (m)	SPT (N)	USC	SOIL DESCRIPTION	ELEVATION (m)
0		CH	CLAY FILL - silty, some sand, high plasticity, stiff, some wood debris, dark brown, wet	
1	13	CH	CLAY - silty, some sand, high plasticity, frozen, some silt pockets, some black pockets, brown, moist	670
			- no black pockets	
2				669
	10	CH	CLAY TILL - silty, some sand, high plasticity, stiff, trace coal, some silt pockets, brown, wet	
3				668
4	24			667
5				666
	20		SILT - sandy, some clay, low to non plastic, compact, brown, damp	
6				665
7	24		- trace to no clay, humid	664
8		ML		663
	32		- dense, damp	662
9			- clay content increasing to some clay, moist	
10				661

STANDPIP 60655308 - GARNEAU HOUSING (COE TEMPLATE).GPJ EDMONTON.GDT 5/2/21

THE CITY OF Edmonton

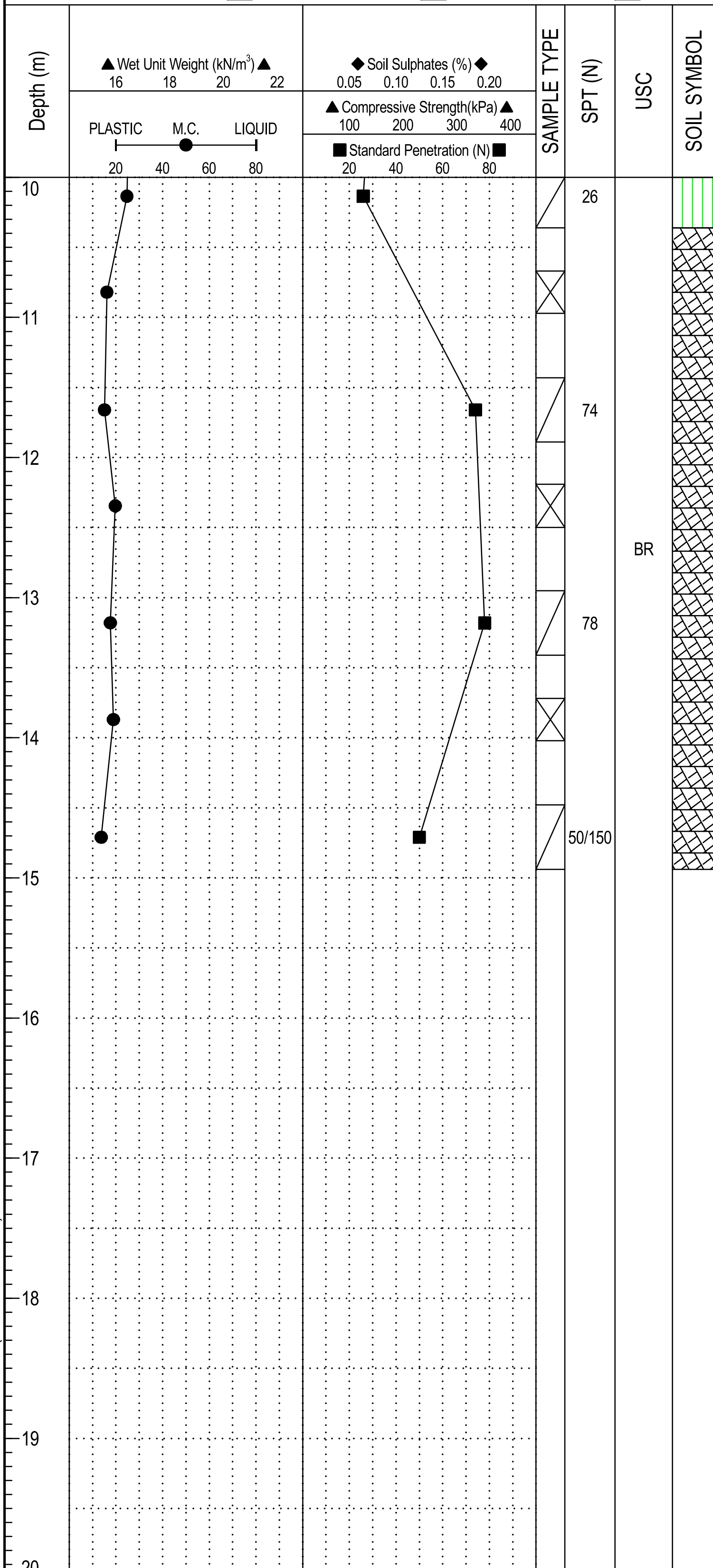
Integrated Infrastructure Services
Engineeering Services Section

LOGGED BY: AT	COMPLETION DEPTH: 14.94 m
REVIEWED BY: FA	COMPLETION DATE: 2021/03/25
Fig. No: 2	Page 1 of 2

Garneau Housing	11053 - 83 Ave NW	BOREHOLE NO: **TH21-03**
Integrated Infrastructure Services	3TM ZONE: N5931861.5 E31992.2	PROJECT NO: 60655308
START DATE: 2021/03/25	METHOD: Solid Stem Augers	ELEVATION: 670.8 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

- compact

SANDSTONE - silty, very dense, poorly lithified, weathered, grey, damp

- trace groundwater encountered

END OF TESTHOLE at 14.94 mBGS
- testhole open upon drilling completion
- trace groundwater encountered at bottom of testhole upon drilling completion
- testhole drilled to scheduled depth
- testhole backfilled with bentonite chips to ground surface

ELEVATION (m): 660, 659, 658, 657, 656, 655, 654, 653, 652, 651

STANDPIP 60655308 - GARNEAU HOUSING (COE TEMPLATE).GPJ EDMONTON.GDT 5/2/21

THE CITY OF Edmonton

Integrated Infrastructure Services
Engineeering Services Section

LOGGED BY: AT	COMPLETION DEPTH: 14.94 m
REVIEWED BY: FA	COMPLETION DATE: 2021/03/25
Fig. No: 2	

Appendix C

Laboratory Results

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

WATER CONTENT (ASTM D2216)

CLIENT:	City of Edmonton
PROJECT:	Garneau Housing
JOB No.:	60655308
DATE :	March 29, 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	518.0	577.8	566.3	240.0	613.9	378.2	605.3	593.8
WT. SAMPLE DRY + TARE	412.8	464.0	461.7	189.4	487.4	299.2	468.7	527.6
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**26.3%**	**25.2%**	**23.3%**	**28.7%**	**26.7%**	**27.6%**	**30.0%**	**12.9%**
HOLE No.	21-01							
DEPTH								
SAMPLE No.	9	10	11	12	13	14	15	16
TARE No.								
WT. SAMPLE WET + TARE	505.0	523.8	511.5	566.6	448.0	652.8	788.1	546.0
WT. SAMPLE DRY + TARE	454.5	446.4	453.3	514.9	399.3	539.1	663.3	479.3
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**11.4%**	**17.9%**	**13.2%**	**10.3%**	**12.6%**	**21.6%**	**19.2%**	**14.3%**
HOLE No.	21-01				21-02			
DEPTH								
SAMPLE No.	17	18	19	20	1	2	3	4
TARE No.								
WT. SAMPLE WET + TARE	713.8	774.2	880.8	724.2	614.7	339.1	654.4	621.7
WT. SAMPLE DRY + TARE	592.1	676.5	740.0	620.9	535.2	271.5	508.4	481.4
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**21.0%**	**14.7%**	**19.4%**	**17.0%**	**15.2%**	**26.2%**	**29.5%**	**30.0%**
HOLE No.	21-02					21-03		
DEPTH								
SAMPLE No.	5	6	7	8	9	1	2	3
TARE No.								
WT. SAMPLE WET + TARE	608.5	625.6	600.5	549.4	352.6	465.1	430.7	674.0
WT. SAMPLE DRY + TARE	455.1	469.9	450.7	461.1	297.3	358.4	339.6	531.8
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**34.7%**	**34.1%**	**34.2%**	**19.7%**	**19.5%**	**30.9%**	**27.9%**	**27.4%**

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

WATER CONTENT (ASTM D2216)

CLIENT:	City of Edmonton
PROJECT:	Garneau Housing
JOB No.:	60655308
DATE :	March 29, 2021 TECHNICAN : CK/GU

HOLE No.	21-03							
DEPTH								
SAMPLE No.	4	5	6	7	8	9	10	11
TARE No.								
WT. SAMPLE WET + TARE	525.8	671.7	553.3	598.6	488.0	490.4	451.2	491.2
WT. SAMPLE DRY + TARE	402.9	506.7	408.3	446.4	418.5	426.7	419.2	458.0
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**31.5%**	**33.4%**	**36.7%**	**35.1%**	**17.1%**	**15.4%**	**7.9%**	**7.5%**
HOLE No.	21-03							
DEPTH								
SAMPLE No.	12	13	14	15	16	17	18	19
TARE No.								
WT. SAMPLE WET + TARE	691.0	800.7	684.7	828.8	745.2	779.1	752.5	754.4
WT. SAMPLE DRY + TARE	588.5	638.6	551.5	715.7	649.3	653.2	641.6	636.6
WT. TARE	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
WATER CONTENT W%	**17.8%**	**25.9%**	**24.7%**	**16.1%**	**15.1%**	**19.7%**	**17.6%**	**18.9%**
HOLE No.	21-03							
DEPTH								
SAMPLE No.	20							
TARE No.								
WT. SAMPLE WET + TARE	723.9							
WT. SAMPLE DRY + TARE	638.4							
WT. TARE	13.2							
WATER CONTENT W%	**13.7%**							
HOLE No.								
DEPTH								
SAMPLE No.								
TARE No.								
WT. SAMPLE WET + TARE								
WT. SAMPLE DRY + TARE								
WT. TARE								
WATER CONTENT W%								

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Materials Testing Lab
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Calgary, Alberta T2G 5M4

ATTERBERG LIMITS (ASTM D4318)

CLIENT :	City of Edmonton		
PROJECT :	Garneau Housing		
JOB No. :	60655308		
LOCATION :		SAMPLE:	3
TESTHOLE:	21-01	DEPTH :	
DATE :	April 26, 2021	TECHNICIAN :	GU

LIQUID LIMIT

Trial No.	1					
Number of Blows	23					
Container Number						
Wt. Sample (wet+tare)(g)	50.52					
Wt. Sample (dry+tare)(g)	35.38					
Wt. Tare (g)	14.39					
Wt. Dry Soil (g)	21.0					
Wt. Water (g)	15.1					
Water Content (%)	72.1%					

AVERAGE VALUES

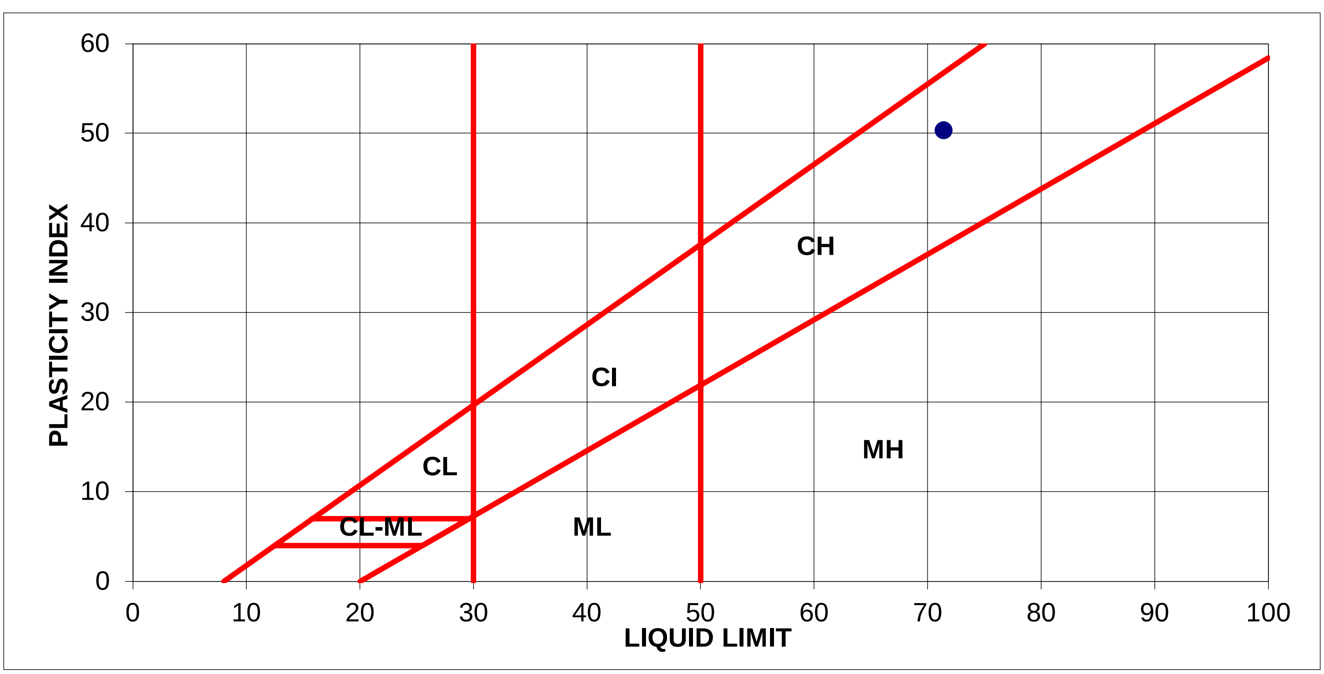
Liquid Limit	71.4
Plastic Limit	21.1
Plasticity Index	50.3

SAMPLE DESCRIPTION

Classification: **CH**

PLASTIC LIMIT

Trial No.	1		
Container Number			
Wt. Sample (wet+tare)(g)	23.88		
Wt. Sample (dry+tare)(g)	21.78		
Wt. Tare (g)	11.82		
Wt. Dry Soil (g)	10.0		
Wt. Water (g)	2.1		
Water Content (%)	21.1%		



FORM: COE Garneau TH21-01 #3 Atterberg .xls
DATE: 4/27/2021

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 :	Garneau Housing		
JOB No. :	60655308		
LOCATION :		SAMPLE:	9
TESTHOLE:	21-01	DEPTH :	
DATE :	March 31, 2021	TECHNICIAN :	GU

LIQUID LIMIT

Trial No.	1					
Number of Blows	30					
Container Number						
Wt. Sample (wet+tare)(g)	62.40					
Wt. Sample (dry+tare)(g)	54.88					
Wt. Tare (g)	16.22					
Wt. Dry Soil (g)	38.7					
Wt. Water (g)	7.5					
Water Content (%)	19.5%					

AVERAGE VALUES

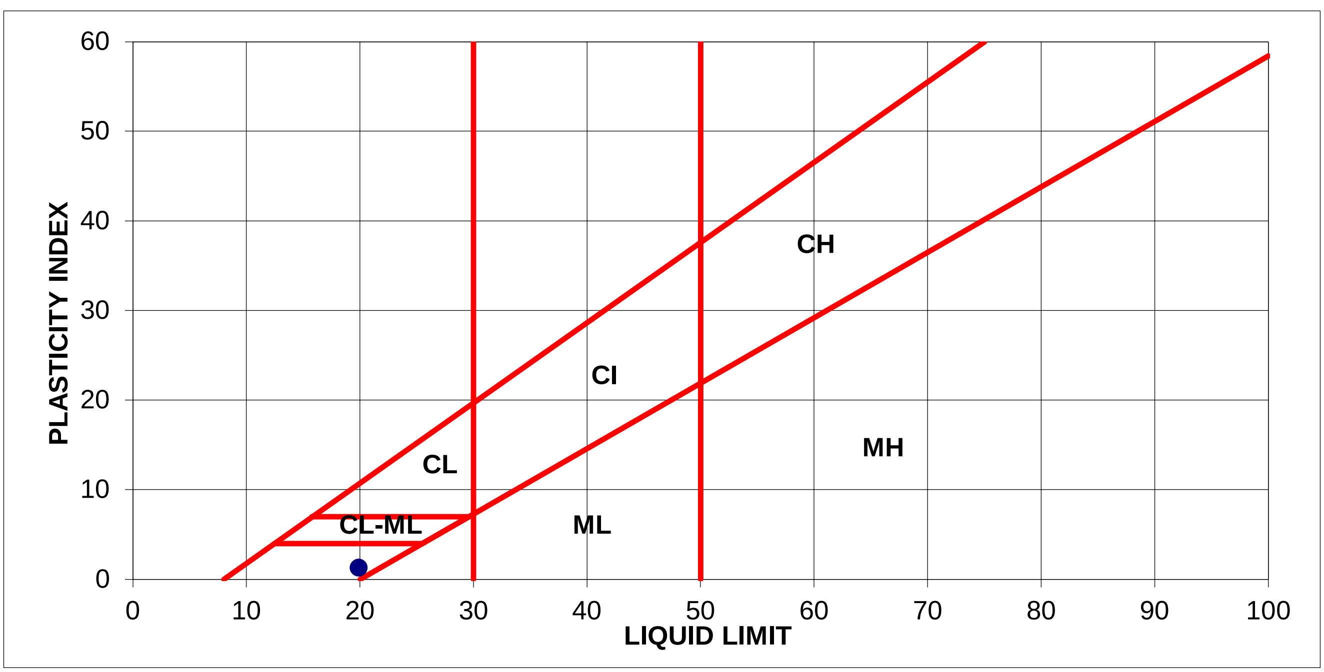
Liquid Limit	19.9
Plastic Limit	18.6
Plasticity Index	1.3

SAMPLE DESCRIPTION

Classification: **ML**

PLASTIC LIMIT

Trial No.	1		
Container Number			
Wt. Sample (wet+tare)(g)	31.36		
Wt. Sample (dry+tare)(g)	28.31		
Wt. Tare (g)	11.90		
Wt. Dry Soil (g)	16.4		
Wt. Water (g)	3.1		
Water Content (%)	18.6%		



FORM: COE Garneau TH21-01 #9 Atterberg .xls
DATE: 4/19/2021

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

ATTERBERG LIMITS (ASTM D4318)

CLIENT :	City of Edmonton		
PROJECT :	Garneau Housing		
JOB No. :	60655308		
LOCATION :		SAMPLE:	3
TESTHOLE:	21-03	DEPTH :	
DATE :	March 31, 2021	TECHNICIAN :	CK

LIQUID LIMIT

Trial No.	1					
Number of Blows	30					
Container Number						
Wt. Sample (wet+tare)(g)	41.58					
Wt. Sample (dry+tare)(g)	29.83					
Wt. Tare (g)	11.93					
Wt. Dry Soil (g)	17.9					
Wt. Water (g)	11.8					
Water Content (%)	65.6%					

AVERAGE VALUES

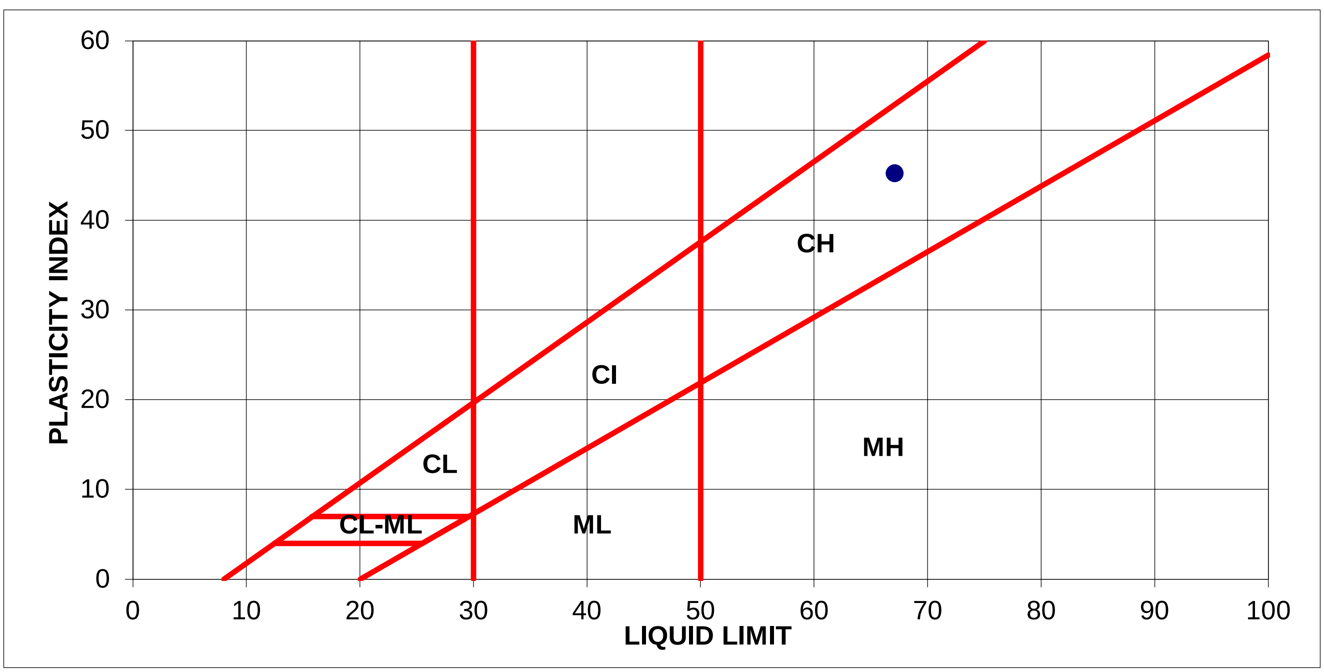
Liquid Limit	67.1
Plastic Limit	21.9
Plasticity Index	45.2

SAMPLE DESCRIPTION

Classification: **CH**

PLASTIC LIMIT

Trial No.	1		
Container Number			
Wt. Sample (wet+tare)(g)	29.32		
Wt. Sample (dry+tare)(g)	26.97		
Wt. Tare (g)	16.22		
Wt. Dry Soil (g)	10.8		
Wt. Water (g)	2.4		
Water Content (%)	21.9%		



FORM: COE Garneau TH21-03 #3 Atterberg .xls
DATE: 4/19/2021

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 :	Garneau Housing		
JOB No. :	60655308		
LOCATION :		SAMPLE:	8
TESTHOLE:	21-03	DEPTH :	
DATE :	March 31, 2021	TECHNICIAN :	GU

LIQUID LIMIT

Trial No.	1					
Number of Blows	18					
Container Number						
Wt. Sample (wet+tare)(g)	63.45					
Wt. Sample (dry+tare)(g)	53.63					
Wt. Tare (g)	15.97					
Wt. Dry Soil (g)	37.7					
Wt. Water (g)	9.8					
Water Content (%)	26.1%					

AVERAGE VALUES

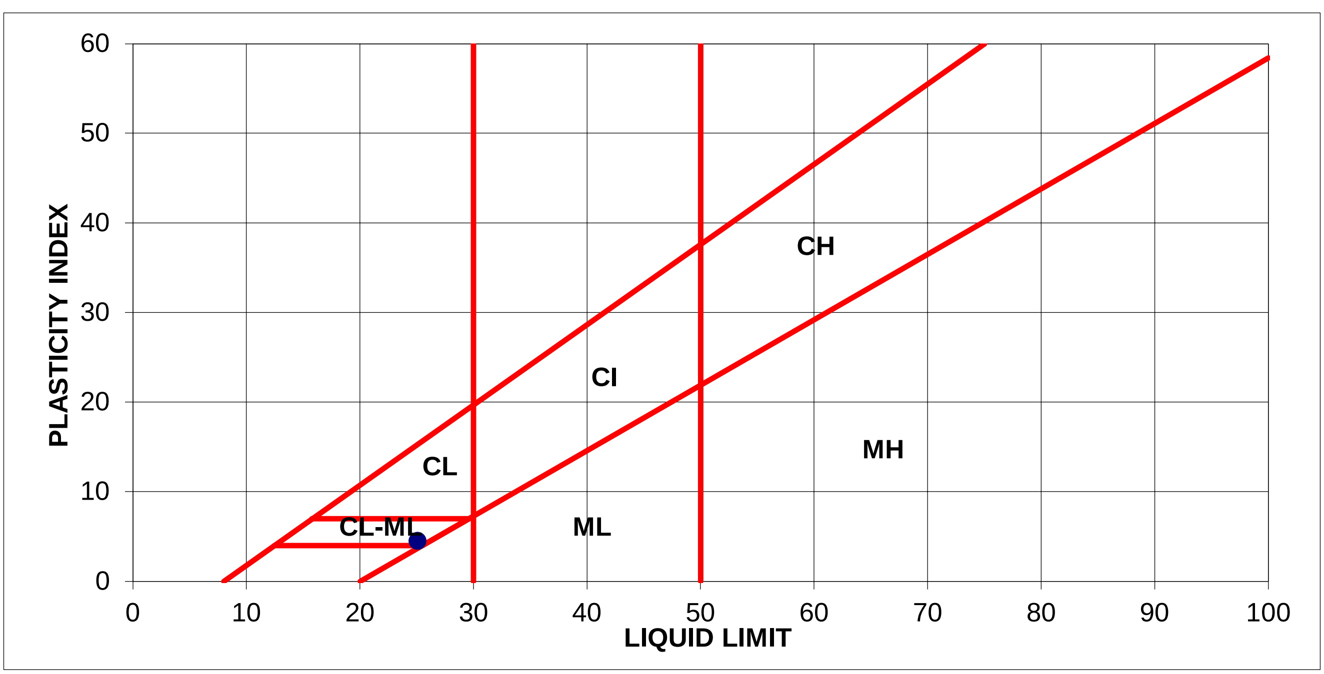
Liquid Limit	25.1
Plastic Limit	20.6
Plasticity Index	4.5

SAMPLE DESCRIPTION

Classification: **CL-ML**

PLASTIC LIMIT

Trial No.	1		
Container Number			
Wt. Sample (wet+tare)(g)	34.73		
Wt. Sample (dry+tare)(g)	30.85		
Wt. Tare (g)	11.97		
Wt. Dry Soil (g)	18.9		
Wt. Water (g)	3.9		
Water Content (%)	20.6%		



FORM: COE Garneau TH21-03 #8 Atterberg .xls
DATE: 4/19/2021

GRAIN SIZE ANALYSIS (ASTM D422)

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

CLIENT :	City of Edmonton		
PROJECT :	Garneau Housing		
JOB No. :	60655308		
LOCATION :		SAMPLE:	9
TESTHOLE:	21-01	DEPTH :	
DATE :	March 30, 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	538.5	50,000	2	50.0		0%	100%	
Tare	100.0	40,000	1 1/2	40.0		0%	100%	
Wt. Dry	438.5	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	1.8	0%	99.6%	
Wt. Dry		315	0.0116	0.315	3.5	1%	99.2%	
Tare No.		160	0.0059	0.160	7.9	2%	98.2%	
		75	0.00295	0.075	104.4	24%	76.2%	
		PAN						

HYDROMETER DATA		READING	TIME (min)	DIAMETER (mm)	TEMP. (°C)	CORR. READING	PERCENT FINER THAN	REMARKS
Wt Dry+Tare	538.5	39	0.5	0.059	21	35	68.8%	
Wt Tare	100.0	32	1	0.044	21	28	54.9%	
Wt Dry	438.5	27	2	0.032	21	23	45.0%	
Sample Size :	50	21	5	0.021	21	17	33.2%	
Wt Retained 2 mm:	0.0	17	15	0.013	21	13	25.2%	
% Passing 2 mm:	100.0%	16	30	0.009	21	12	23.3%	
Specific Gravity :	2.70	15	60	0.006	21	11	21.3%	
Hydrometer No.:	43-9856	14	120	0.005	21	10	19.3%	
Solution (g/L) :	40	13	240	0.003	21	9	17.3%	
		12	1440	0.001	21	8	15.3%	
		12	2880	0.001	21	7	14.4%	

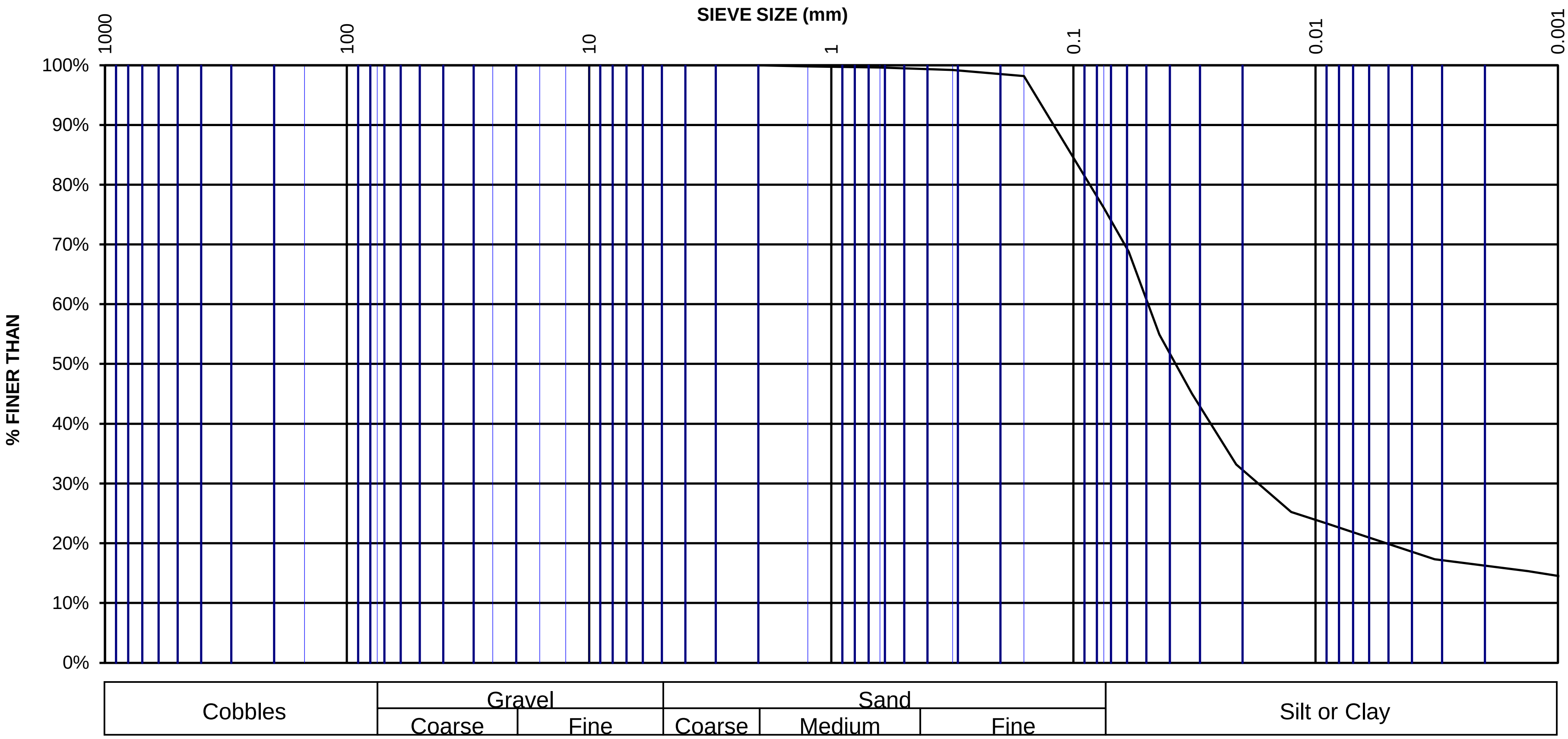
FORM: COE Garneau TH21-01 #9 Hydro .xls
DATE: 4/19/2021

GRAIN SIZE ANALYSIS (ASTM D422)

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

CLIENT :	City of Edmonton		
PROJECT :	Garneau Housing		
JOB No. :	60655308		
LOCATION :		SAMPLE:	9
TESTHOLE:	21-01	DEPTH :	
DATE :	March 30, 2021	TECHNICIAN :	GU

Gravel = 0.0% **Sand = 23.8%** **Silt = 59.9%** **Clay = 16.3%**



FORM: COE Garneau TH21-01 #9 Hydro .xls
DATE: 4/19/2021

GRAIN SIZE ANALYSIS (ASTM D422)

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

CLIENT :	City of Edmonton		
PROJECT :	Garneau Housing		
JOB No. :	60655308		
LOCATION :		SAMPLE:	13
TESTHOLE:	21-01	DEPTH :	30-31'
DATE :	April 14, 2021	TECHNICIAN :	CK

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	460.7	50,000	2	50.0		0%	100%	
Tare	100.0	40,000	1 1/2	40.0		0%	100%	
Wt. Dry	360.7	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%	100%	
Tare		630	0.0234	0.63	0.4	0%	99.9%	
Wt. Dry		315	0.0116	0.315	0.7	0%	99.8%	
Tare No.		160	0.0059	0.160	6.5	2%	98.2%	
		75	0.00295	0.075	166.6	46%	53.8%	
		PAN						

HYDROMETER DATA		READING	TIME (min)	DIAMETER (mm)	TEMP. (°C)	CORR. READING	PERCENT FINER THAN	REMARKS
Wt Dry+Tare	460.7	44	0.5	0.055	23	40	39.8%	
Wt Tare	100.0	35	1	0.042	23	31	30.9%	
Wt Dry	360.7	25	2	0.032	23	21	21.0%	
Sample Size :	100	19	5	0.021	23	15	15.1%	
Wt Retained 2 mm:	0.0	17	15	0.012	23	13	13.1%	
% Passing 2 mm:	100.0%	15	30	0.009	23	11	11.1%	
Specific Gravity :	2.70	14	60	0.006	23	10	10.1%	
Hydrometer No.:	43-9856	14	120	0.004	23	10	9.7%	
Solution (g/L) :	40	13	240	0.003	23	9	8.7%	
		11	1440	0.001	21	7	6.7%	
		11	2880	0.001	21	6	6.2%	

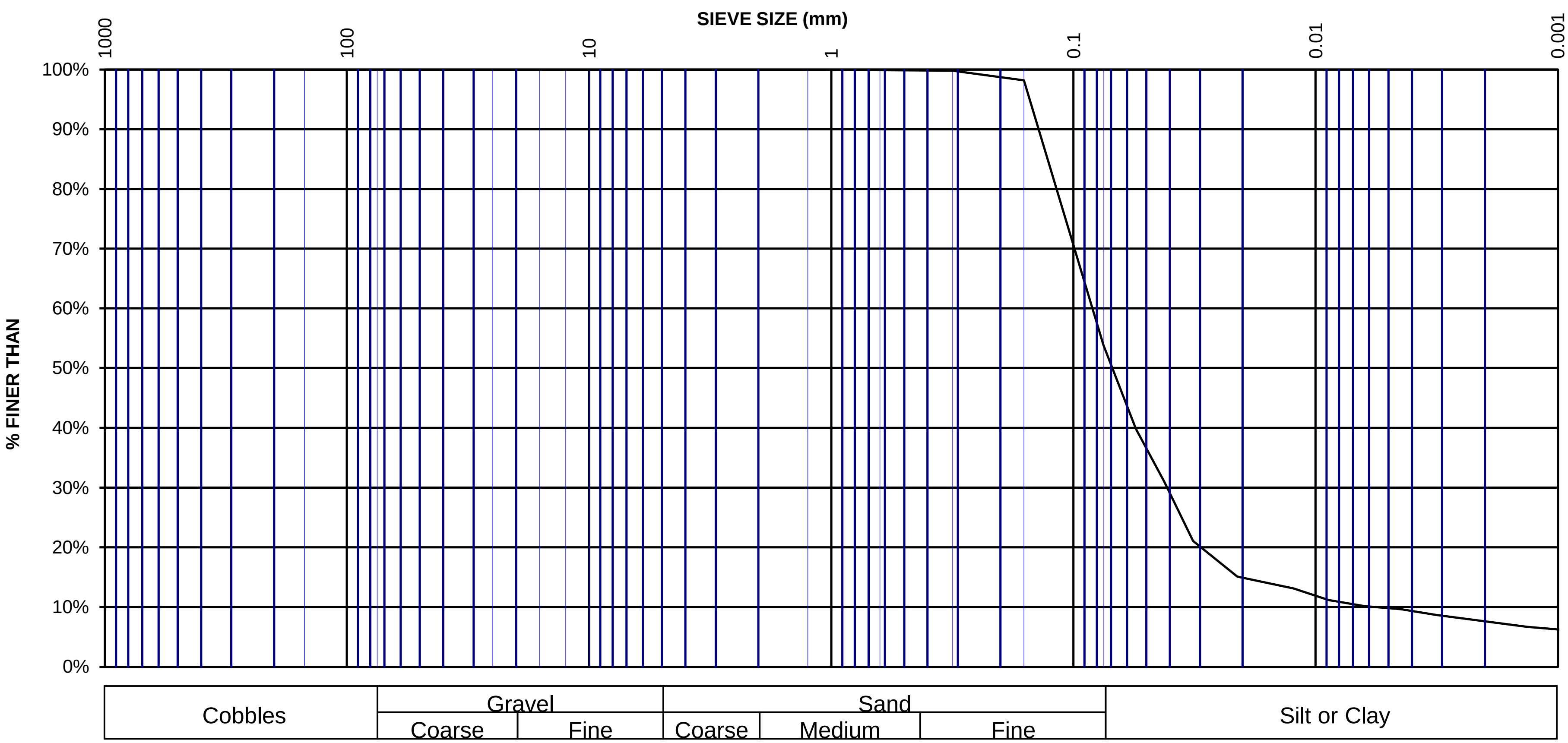
FORM: COE Garneau TH21-01 #13 Hydro .xls
DATE: 4/19/2021

GRAIN SIZE ANALYSIS (ASTM D422)

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

CLIENT :	City of Edmonton		
PROJECT :	Garneau Housing		
JOB No. :	60655308		
LOCATION :		SAMPLE:	13
TESTHOLE:	21-01	DEPTH :	30-31'
DATE :	April 14, 2021	TECHNICIAN :	CK

Gravel = 0.0% **Sand = 46.2%** **Silt = 46.1%** **Clay = 7.7%**



FORM: COE Garneau TH21-01 #13 Hydro .xls
DATE: 4/19/2021

GRAIN SIZE ANALYSIS (ASTM D422)

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

CLIENT :	City of Edmonton		
PROJECT :	Garneau Housing		
JOB No. :	60655308		
LOCATION :		SAMPLE:	12
TESTHOLE:	21-03	DEPTH :	
DATE :	March 30, 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	673.3	50,000	2	50.0		0%	100%	
Tare	100.0	40,000	1 1/2	40.0		0%	100%	
Wt. Dry	573.3	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	2.6	0%	99.5%	
	Passing							
After Washing		2,000	0.0937	2.0	2.8	0%	99.5%	
Wt. Dry+Tare		1,250	0.0469	1.25	2.8	0%	99.5%	
Tare		630	0.0234	0.63	5.1	1%	99.1%	
Wt. Dry		315	0.0116	0.315	8.5	1%	98.5%	
Tare No.		160	0.0059	0.160	30.2	5%	94.7%	
		75	0.00295	0.075	124.9	22%	78.2%	
		PAN						

HYDROMETER DATA		READING	TIME (min)	DIAMETER (mm)	TEMP. (°C)	CORR. READING	PERCENT FINER THAN	REMARKS
Wt Dry+Tare	673.3	41	0.5	0.058	21	37	72.4%	
Wt Tare	100.0	37	1	0.042	21	33	64.5%	
Wt Dry	573.3	31	2	0.031	21	27	52.7%	
Sample Size :	50	24	5	0.021	21	20	38.9%	
Wt Retained 2 mm:	2.8	17	15	0.013	21	13	25.1%	
% Passing 2 mm:	99.5%	15	30	0.009	21	11	21.2%	
Specific Gravity :	2.70	13	60	0.006	21	9	17.2%	
Hydrometer No.:	43-9856	12	120	0.005	21	8	15.3%	
Solution (g/L) :	40	11	240	0.003	21	7	13.3%	
		10	1440	0.001	21	6	11.3%	
		10	2880	0.001	21	5	10.3%	

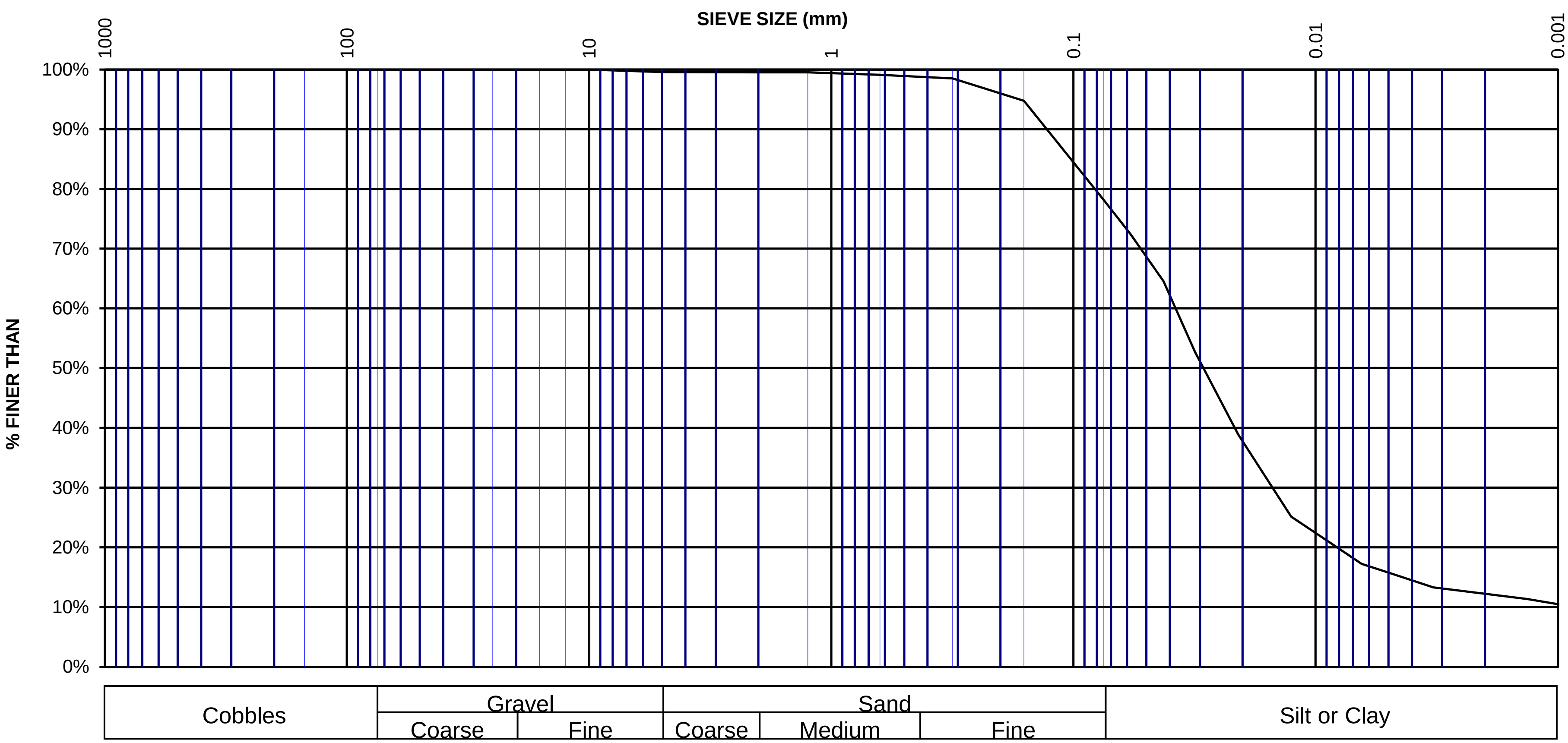
FORM: COE Garneau TH21-03 #12 Hydro .xls
DATE: 4/19/2021

GRAIN SIZE ANALYSIS (ASTM D422)

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

CLIENT :	City of Edmonton		
PROJECT :	Garneau Housing		
JOB No. :	60655308		
LOCATION :		SAMPLE:	12
TESTHOLE:	21-03	DEPTH :	
DATE :	March 30, 2021	TECHNICIAN :	GU

Gravel = 0.5% **Sand = 21.3%** **Silt = 65.9%** **Clay = 12.3%**



FORM: COE Garneau TH21-03 #12 Hydro .xls
DATE: 4/19/2021

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

Date Received: 29-MAR-21
Report Date: 07-APR-21 17:03 (MT)
Version: FINAL

Client Phone: 403-254-3301

Certificate of Analysis

Lab Work Order #: L2571396

Project P.O. #:	NOT SUBMITTED
Job Reference:	CITY OF EDMONTON - GARNEAU - 60655308 LAB TESTING
C of C Numbers:	
Legal Site Desc:	

Comments: Total Sulphate Ion Content results are <0.2% for all samples. 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 analyses have been removed.

Dhaliwal

Inayat Dhaliwal
Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2571396-1 CITY OF EDMONTON - GARNEAU - TH21-01 #7							
Sampled By: N/A on 29-MAR-21							
Matrix: SOIL							
Miscellaneous Parameters							
% Saturation	65.5		1.0	%	01-APR-21	01-APR-21	R5418837
Chloride (Cl)	23		20	mg/L		05-APR-21	R5419380
Resistivity	1640		1.0	ohm cm		01-APR-21	R5418438
Sulfur (as SO4)	170		6.0	mg/L		04-APR-21	R5420347
Total Sulphate Ion Content	<0.050		0.050	%	01-APR-21	01-APR-21	R5418713
pH in Saturated Paste	7.66		0.10	pH		01-APR-21	R5418571
Salinity in mg/kg							
Chloride (Cl)	15		13	mg/kg		07-APR-21	
Sulfur (as SO4)	111		3.9	mg/kg		07-APR-21	
L2571396-2 CITY OF EDMONTON - GARNEAU - TH21-03 #10							
Sampled By: N/A on 29-MAR-21							
Matrix: SOIL							
Miscellaneous Parameters							
% Saturation	36.1		1.0	%	01-APR-21	01-APR-21	R5418837
Chloride (Cl)	59		20	mg/L		05-APR-21	R5419380
Resistivity	1550		1.0	ohm cm		01-APR-21	R5418438
Sulfur (as SO4)	1530		6.0	mg/L		04-APR-21	R5420347
Total Sulphate Ion Content	<0.050		0.050	%	01-APR-21	01-APR-21	R5418713
pH in Saturated Paste	7.56		0.10	pH		01-APR-21	R5418571
Salinity in mg/kg							
Chloride (Cl)	21.4		7.2	mg/kg		07-APR-21	
Sulfur (as SO4)	551		2.2	mg/kg		07-APR-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.

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.

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.

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.

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.

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.

ALS Environmental

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
PH-PASTE-CL	**Soil**							
Batch R5418571								
WG3512199-9 IRM		**SAL-STD10**						
pH in Saturated Paste			7.29		pH		6.94-7.54	01-APR-21
WG3512199-8 LCS								
pH in Saturated Paste			7.02		pH		6.7-7.3	01-APR-21
RESISTIVITY-PASTE-CL	**Soil**							
Batch R5418438								
WG3511814-2 IRM		**SAL-STD10**						
Resistivity			106.0		%		70-130	01-APR-21
WG3511814-1 LCS								
Resistivity			96.4		%		70-130	01-APR-21
SAT-PCNT-N-CL	**Soil**							
Batch R5418837								
WG3512087-3 IRM		**SAL-STD10**						
% Saturation			93.3		%		70-130	01-APR-21
WG3512087-1 MB								
% Saturation			<1.0		%		1	01-APR-21
SO4-T-CSA-A23-ED	**Soil**							
Batch R5418713								
WG3512100-3 CRM		**ED-634A_CEMENT**						
Total Sulphate Ion Content			84.0		%		80-120	01-APR-21
WG3512100-2 LCS								
Total Sulphate Ion Content			99.7		%		70-130	01-APR-21
WG3512100-1 MB								
Total Sulphate Ion Content			<0.050		%		0.05	01-APR-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

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

L2571396-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 Calgary, AB T2C 5P2	☐ 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
		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	

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

Project Information		Oil and Gas Required Fields (client use)	
ALS Quote #:		Approver ID:	Cost Center:
Job #:	City of Edmonton - Garneau - 60655308.Lab Testing	GL Account:	Routing Code:
PO / AFE:		Activity Code:	
LSD:		Location:	
ALS Lab Work Order # (lab use only)		ALS Contact: Nelson Kwan	Sampler: N/A

Analysis Request — 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
	City of Edmonton - Garneau - TH21-01 #7	29-Mar-21	-	Soil	R	R	R	R	R	R	R	R	1
	City of Edmonton - Garneau - TH21-03 #10	29-Mar-21	-	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
16	

SHIPMENT RELEASE (client use)			INITIAL SHIPMENT RECEPTION (lab use only)			FINAL SHIPMENT RECEPTION (lab use only)		
Released by: Chris Keeley	Date: Mar. 29/21	Time: 14:00	Received by:	Date: 3/29	Time: 1530	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**.

Brian Nguyen, P.Eng.
Geotechnical Engineer
T: 780-486-7676
E: Brian.Nguyen@aecom.com

AECOM Canada Ltd.
101-18817 Stony Plain Road NW
Edmonton, AB T5S 0C2
Canada

T: 780.486.7000
F: 780.486.7070
aecom.com