Geotechnical, Environmental and Materials Engineering

GEOTECHNICAL INVESTIGATION REPORT

PROPOSED HAPPY VALLEY FIRE HALL AND WATER POND

WITHIN SE 14-078-08 W6M
WEST OF RYCROFT, SADDLE HILLS COUNTY, ALBERTA

PREPARED FOR

SADDLE HILLS COUNTY
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1.0 INTRODUCTION

Saddle Hills County (the Client) is proposing to construct a new firehall and a pond, located in Happy Valley, Saddle Hills County west of Rycroft, Alberta. The Client has requested ParklandGEO Ltd. (ParklandGEO) to perform a geotechnical investigation at the proposed site. This report summarizes results of the field and laboratory testing programs and presents geotechnical recommendations for general site preparation and foundations.

1.1 SCOPE OF WORK

The scope of work for this project was outlined in ParklandGEO's proposal dated May 15, 2025 (E2025-0664). Authorization to proceed with this investigation was given by Mr. Brice Daly of the Saddle Hills County on May 16, 2025 by signing a Professional Services Agreement.

1.2 PREVIOUS INVESTIGATIONS

ParklandGEO is not aware of any previous geotechnical investigation having been conducted on this site.

2.0 PROJECT INFORMATION

2.1 SITE DESCRIPTION

The site is located within LSD SE 14-078-08 W6M, around 25 km west of Rycroft, Alberta as shown on the Location Plan, Figure 1. Access to the site was via Range Road 81 south of Spirit River highway. The proposed site is surrounded by farmland and bordered by Range Road 81 to the east. At the time of investigation, the site was located within a relatively flat agricultural field that appeared to be under early-stage crop cultivation. The area was open and undeveloped, with no visible above-ground impediments. Borehole elevation ranged from 745.5 to 746.7 m.

2.2 PROJECT DESCRIPTION

Based on the preliminary plan provided by the Client, the proposed development will include a 520 m² firehall building and a 77 m x 45 m pond. It is understood that the base of the pond will be around 6 to 9 meters (around elevations 737 to 740 m) below ground surface and located on the south side of the proposed fire hall. The building is expected to impose light to moderate foundation loads. Traffic loads throughout the site are also expected to range from light to heavy. A gravel-surfaced yard is planned around the firehall. Final site grading details were not available, however, minor site grading (less than 1 m) is expected based in borehole elevations.

3.0 FIELD AND LABORATORY PROGRAMS

On May 28, 2025, six boreholes were drilled at the site. Boreholes were drilled to depths ranging from 3.5 to 8.0 meters below grade level (mbgl). The following sampling and testing procedures were followed during the field program:

1. Prior to mobilizing the drilling rig, ParklandGEO personnel completed an Alberta One Call to verify the drill site was clear of underground utilities.

2. The boreholes were drilled using a track-mounted, with 150 mm diameter solid stem augers drilling rig. The drill rig was owned and operated by Frontier Enviro-Drilling Ltd. of Clairmont, Alberta.

3. Drilling operations were monitored by members of ParklandGEO's geotechnical staff. The soil encountered was visually examined during drilling and logged according to the Modified Unified Soil Classification System.

4. Soil samples were collected from auger cuttings at 1.0 m intervals in order to determine the soil/moisture profile and from other selected depths for other testing. Disturbed soil samples were also obtained from Standard Penetrations Tests (SPTs), which were performed at selected depth intervals.

5. At the completion of drilling, 25 mm hand-slotted PVC standpipes were installed in Boreholes 25-01 and 25-03 and backfilled with sand, auger cuttings and bentonite. Groundwater levels and depths where seepage zones were encountered were noted during drilling. Groundwater measurements were recorded on June 11, 2025, about 13 days after drilling.

6. Soil samples were returned to ParklandGEO's Grande Prairie lab for further testing. The results of all laboratory testing are shown on the borehole logs (Appendix A) and individual test results presented in Appendix B. The laboratory program consisted of moisture contents, soil sulphates, Atterberg Limits, and hydrometer particle size analysis.

7. The borehole locations and elevations were surveyed by ParklandGEO personnel by using a rod-mounted Trimble Geo 7X. The elevation survey was performed referencing to a geodetic datum (accurate to within 50 cm). UTM coordinates are provided in the borehole logs in Appendix A.

4.0 SUBSURFACE CONDITIONS

The general soil profile at the borehole locations consisted of topsoil overlying clay. Detailed descriptions of the soil profiles at the borehole locations are provided on the borehole logs in Appendix A. Definitions of the terminology and symbols used on the borehole logs are provided on the explanation sheets, also in Appendix A. The following is a brief description of the main soil types found at the site.

4.1 TOPSOIL

A 130 to 170 mm thick layer of topsoil was encountered at the surface in all boreholes. The thickness of this layer may vary across the site between borehole locations. The topsoil contained rootlets inclusions and was dry. The topsoil is considered as a weak material and is compressible under loads.

4.2 CLAY

Clay was encountered below the clay in all boreholes beneath topsoil and extended beyond the maximum depth of drilling (i.e., 8 mbgl in Boreholes 25-01 to 25-04, 5 mbgl in Borehole 25-05, and 3.5 m in Boreholes 25-04 and 25-06). The clay contained little to some silt, trace to little sand, trace gravel, was high plastic and dry to damp. Trace cobbles were noted below 1.5 to 3.0 depth in most boreholes drilled onsite. The SPT “N” values ranged from 14 to 22 blows, which indicates stiff to very stiff consistency. The moisture content ranged between 14 and 22 percent.

4.3 SUMMARY OF LABORATORY SOIL TEST RESULTS

The following tables summarize the routine laboratory soil classification and soil chemistry test results.

TABLE 1: SOIL CLASSIFICATION TEST RESULTS

Borehole No.	Soil	Depth (mbgl)	Grain Size Distribution (%)				Plasticity (%)			Soil Type*
			Clay	Silt	Sand	Gravel	PL	LL	PI	
25-02	Clay	0.7	54	36	10	0	15	58	43	CH
25-03	Clay	2.3	51	35	14	0	14	54	40	CH
25-01	Clay	6.8	45	33	13	9	14	54	40	CH

* Modified Unified Soil Classification System

TABLE 2: SOIL CHEMISTRY

Borehole No.	Depth (m)	Soluble Sulphate (%)
25-01	0.5	<0.01
25-02	1.0	0.02
25-06	1.0	0.02

The concentrations of sulphates are expressed as a percentage of the dry mass of soil. The concentration of water-soluble sulphate of the native soil indicated a negligible potential for sulphate attack on buried concrete in direct contact with soil as per CAN/CSA-A23.1-19 standards. Concrete recommendations are provided in Section 6.12.

4.4 FROST SUSCEPTIBILITY AND DEPTH

The predominant subgrade soils within the expected depth of frost at this site consisted of clay. These soils are considered low to moderately frost susceptible. Based on the 1991 to 2020 (30-year) freezing index of 1564^{O}C-Days in the Grande Prairie area, the design frost depth for fully exposed subgrades is 2.7 m in the clay profile. Total frost heaves in the order of 50 to 100 mm can be observed in these soils. Frost penetration depths will be impacted by actual winter temperatures and different surface cover conditions (i.e. snow cover, vegetation, pavements, etc.).

5.0 GROUNDWATER CONDITIONS

Trace groundwater seepage was observed in Boreholes 25-01 and 25-03 between depths 6 to 7.5 mbgl (elevations 738 to 739.8 m) during drilling. The groundwater level in Boreholes 25-01 and 25-03 were measured at 7.4 and 6.5 mbgl respectively (elevations 738.1 and 739.4 m) on June 11, 2025, 13 days after the drilling. The upper soil consists of clay which are considered relatively low permeable soils and resist groundwater infiltration and lateral movement. Groundwater elevations are expected to fluctuate on a seasonal basis and will be highest after periods of prolonged heavy precipitation or snow melt. Seasonally perched groundwater may be encountered within cracks and fissures or sand or gravel lenses within the clay soils above the water table. Perched conditions will dissipate as groundwater infiltrates down to the static groundwater table during drier periods.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 GENERAL EVALUATION

The site development will consist of a 520 m^2 building with gravel access roads and parking areas, a large gravel yard, and a water pond. Foundation loads for the proposed building are expected to be light to moderate. The proposed access ways and parking areas are expected to see light to heavy duty service conditions.

Overall, the site soil conditions were considered suitable for the proposed development although the presence of high plastic clay soils will have to be taken into consideration. The observed groundwater conditions suggest a relatively deep groundwater table. The most significant geotechnical issues at this site include:

1. High plastic clay was encountered at this site. Problems related to swelling clays and some recommendations to mitigate these problems are presented in Section 6.2

2. Based on the relatively level building site, only minor grading is expected (i.e. less than 1 m). The subgrade is expected to be high plastic clay which is not considered to be ideal for use as engineered fill and will provide a relatively low level of subgrade support for pavements and slabs. High plastic clay will require careful moisture control if used for engineered fill. Like all fine grained soils, clay subgrades may be subject to softening or rutting if the exposed subgrade is allowed to get wet during or after construction. Therefore, weather will play a significant role during site development.

3. The foundation conditions are considered to be suitable for the several pile options such as, bored cast-in-place (CIP) concrete piles or screw piles. Strip or spread footings are not well suited for this site due to the presence of high plastic clay at typical footing depths. Foundation recommendations are given for bored CIP concrete piles, screw piles and shallow foundations. Additional recommendations for other foundation options or pile types can be provided upon request.

4. High plastic clay soils have significant potential for swelling/shrinking. Swelling pressures at this site are expected to be comparable if not higher than foundation loads if the soils are not prepared properly. Detailed testing of fill materials, settlement analysis and review of proposed bearing pressures versus swelling potential will need to be undertaken as part of the design if footings are proposed at this site.

5. Groundwater is expected to intersect the pond base if excavated to a depth of 6 to 9 m. To mitigate this, the pond base should be raised above the groundwater table and lined with compacted medium plastic clay. The existing high-plasticity clay is prone to shrinkage and cracking and is therefore only marginally suitable. Alternatively, a synthetic liner may be used, provided that seepage and buoyant uplift control measures are implemented.

6.2 SWELLING CLAY

High plastic, swelling clay was present at the surface at this site. High plastic soils exhibit volume changes such as swelling and shrinkage with changes in soil moisture content. Swelling potential decreases at higher soil moisture contents in the order of 35 percent or greater. The high plastic soils observed at this site have soil moisture levels ranging between 14 to 28 percent with an average of 22 percent. The typical problem with swelling soils is that the soils are exposed to wet and dry conditions during construction. When a slab or pavement is placed over the soil the evapo-transpiration conditions change and the soil gains moisture. Since structural features are placed after shrinkage, the effects of swelling are magnified when the soil re-establishes a new soil moisture equilibrium. Swelling pressures in the order of 200 kPa are considered possible in the high plastic clay at this site, which is well in excess of typical slab and footing foundation loads. The swelling problems are magnified by the variation of plasticity in the subgrade, which might lead to non-uniform swelling and harmful differential heave.

If swelling is a concern, the ideal option is to provide a structurally supported floor slab underlain by a crushable void form or crawl space. This option is relatively expensive, but it will provide the most predictable level of slab performance and may be justified for slabs with strict vertical tolerances. If the owner is willing to accept some risk, then it may be acceptable to construct a grade supported slab and try to minimize the potential for differential slab movement. If subgrade conditions are uniform heave will still occur but the potential for differential heave may be reduced. The following construction practices can be used to try and reduce possible problems with heaving/shrinking:

1. Higher plastic clays could be removed and replaced or mixed with a suitable low to medium plastic fill material. Given the deep deposits of high plastic clay at this site, removal and replacement of swelling clay is not considered practical.

2. Swelling pressures and heave potential are reduced when soil moisture contents approach 35 percent. Soils drier than this will be subject to higher swelling. It is crucial not to allow exposed high plastic clay soils to dry during construction through the use of protective soil layers and mud slabs.

3. Interior non-load bearing walls need to be designed to accommodate potential vertical movement of the slab.

4. The design of water lines and heating ducts beneath slab on grade floors can have a significant impact on subgrade soils and require very careful design and construction measures.

5. Exterior drainage around the building perimeter is important to minimize the potential for infiltration into subgrade soils. Roof and other drains should discharge into storm sewers. If this is not possible, roof drains should discharge well clear of the building. The use of paving stones adjacent to buildings is also not recommended unless special design considerations are used to promote the drainage of water away from buildings. Pavement areas around the building should be kept high. The surface of the top of the subgrade should mirror the surface grades and shed infiltration water away. The placement of snow piles should be located well away from buildings.

6. Landscaping should be designed to minimize the need for watering adjacent to the proposed building. Planting trees and larger shrubs within 3 to 5 m of the building should be avoided, because root systems can take moisture from subgrade soils and lead to possible subgrade shrinkage and settlement.

These steps can be taken to reduce and possibly eliminate the detrimental effects of swelling clays on foundations and slab work. However, due to the nature of these soils there are no measures that can totally eliminate the risk of swelling other than construction of a structurally supported floor slab.

6.3 SITE PREPARATION

The present site is relatively level, so it is anticipated that only minor cuts and fills will be required at the site. Surficial topsoil, organics, non-engineered fill, or unsuitable soils should be stripped from the development area. The exposed subgrade should be scarified to a minimum depth of 150 to 200 mm and recompacted uniformly to a minimum of 98 percent of Standard Proctor Maximum Dry Density (SPMDD). Site preparation measures should be monitored by qualified and experienced geotechnical personnel to identify potential soft areas. Soft areas and/or buried organics should be sub-cut and replaced with a suitable fill material to a depth sufficient to support construction traffic. If excessively soft subgrade conditions are encountered across this site, preparation procedures should be reviewed. Methods to avoid subgrade failure of soft subgrades may include: limiting construction traffic, modification of site preparation procedures (scarification, recompaction, etc.) and sub-cut and replacement with a suitable engineered fill material.

If required, fill to bring the site up to grade should preferably be low to medium plastic clay or well graded select granular material such as sand or gravel. The native surficial clay is high plastic which is not ideally suited for use as engineered fill. It should be understood that moisture control will be very important if high plastic clay is considered for fill. The lower moisture content levels required to achieve good compaction will lead to larger swelling potential, which can be magnified if the moisture content is not uniform.

New fills up to 0.6 m thick in proposed building areas should be placed to a uniform density of 98 percent of SPMDD. Deeper new fills, including trench backfills more than 0.6 m thick, within building areas should be placed uniformly in thin lifts and compacted to 100 percent of SPMDD and either be at or slightly above Optimum Moisture Content (OMC). Exterior backfill outside of building footprints should be placed in thin lifts compacted to at least 95 percent of SPMDD. Compliance with this recommendation for exterior areas is important because poorly compacted backfill adjacent to foundation structures will settle, which may lead to ponding of surface water against foundation walls or grade beams.

The lift thicknesses should be governed by the ability of the selected compaction equipment to uniformly achieve the recommended density. However, it is generally recommended to use lifts with a maximum compacted thickness of 200 mm for granular fill and 150 mm for clay fill. Uniformity is of most importance. Granular fill is best compacted with large smooth drum vibratory rollers while clay fill is best compacted with large vibratory "padfoot" or "sheepsfoot" rollers. In areas which require higher compaction, it is recommended that granular fill be placed at moisture contents 0 to 2 percent below the OMC and that clay fill be placed at moisture contents about 0 to 2 percent above the OMC. This will help reduce compactive effort and potential risk of subgrade disturbance needed to achieve maximum density.

Fill placement and compaction during the winter months is challenging due to the difficulty in moisture conditioning fill soils and obtaining high compaction levels. Materials and methodology should be reviewed prior to construction if cold weather compaction of clay fills is proposed. High compaction levels can only be achieved using fill soils that are unfrozen provided the compaction area is heated and hoarded to prevent freezing during placement and compaction.

Surface water should be drained away from the site as quickly as possible, both during and after construction. Site drainage should be directed away from the foundation walls. It is recommended to provide a 5 percent back slope from buildings for a distance of at least 3 m. Roof and other drains should discharge well clear of buildings. The slope of exterior backfill should be checked periodically to verify water is shed away from buildings. If the backfill settles causing water to pond against foundation walls, the surface should be re-graded. Water should not be allowed to pond adjacent to buildings, equipment sites or pavement areas. A minimum grade of 2 percent is recommended to promote surface runoff and minimize potential saturation and degradation of the parking area subgrade.

6.4 ALBERTA BUILDING CODE

In accordance with the National Building Code – 2023 Alberta Edition (NBC-AE), the use of Limit States Design (LSD) is required for the design of buildings and their structural components including foundations. The limit states of LSD design are classified into two groups; the Ultimate Limit States (ULS) and the Serviceability Limit States (SLS). The ULS design requirements in the ABC reference the Structural Commentaries in the User Guide of the National Building Code of Canada (NBCC).

6.4.1 Ultimate Limit States (ULS)

The ULS case is primarily concerned with safety and the levels of load and resistance at the point of collapse or structural failure. The geotechnical value for this case is the ultimate resistance. For foundation design this ultimate resistance value is reduced using a Geotechnical Resistance Factor (GRF) which is based on the reliability index of the geotechnical data used to determine the ultimate resistance for the foundation loading case. The following GRF values should be used for foundation design for shallow and deep foundations:

TABLE 3: LSD GEOTECHNICAL RESISTANCE FACTORS

Geotechnical Case	Resistance Factors
DEEP FOUNDATIONS (PILES)	
Vertical resistance by semi-empirical analysis and in-situ test data	0.4
Vertical resistance from analysis of dynamic monitoring results	0.5
Vertical resistance from analysis of static load test results	0.6
SHALLOW FOUNDATIONS (FOOTINGS AND MATS)	
Vertical resistance by semi-empirical analysis and in-situ test data	0.5

* NBCC - Users Guide - Structural Commentaries (Part 4 of Division B) - Commentary K –Foundations.

6.4.2 Serviceability Limit States (SLS)

The SLS case occurs when the foundation loads cause movements or vibrations that are greater than are tolerable for the intended use of the structure. The SLS case is addressed by determining the maximum available resistance to keep the foundation deformation within tolerable limits under service loads (i.e. settlement, lateral deflection, etc.). Typically, the foundation loads, configurations and serviceability tolerances have to be known to properly determine geotechnical SLS resistance values. In some foundation cases, such as small footings, basic assumptions can be made to provide SLS resistance values under specific stated conditions.

For axial loading conditions the SLS resistance is addressed by determining the limiting load to keep foundation settlements within tolerable limits. Tolerable total and differential settlements should be verified by the structural engineer, but for normal buildings the tolerable limit of total settlement for foundations is typically about 25 mm.

For the expected piles less than 25 mm of settlement is expected to be required to mobilize the ultimate resistance. Therefore, the serviceability limit states are not expected to govern pile design unless very strict settlement tolerances are required (i.e. less than 10 mm of settlement).

6.4.3 Seismic Classification

The National Building Code of Canada (2019) requires buildings to be designed to resist a minimum earthquake force. The formula for obtaining minimum earthquake force is dependent of several factors including Foundation Factors (F_a and F_v) which should be determined using a Site Class of D for this site (Table 4.1.8.4.A). The subgrade soil is stiff to very stiff clay extending to below the maximum depth drilling of 8.0 m.

6.5 BORED CIP CONCRETE PILES – ULS DESIGN

6.5.1 Axial Resistance

It is assumed that bored CIP concrete piles may be considered for this site. The soil conditions for this site are suitable for bored CIP concrete piles. It should be noted that cobbles were present below 1.5 and 3.0 m in most boreholes, so difficulties during installation may be encountered. Casing should be available onsite due to potential water seepage below about 6.0 mbgl. Either straight-shaft friction piles or belled end-bearing piles could be considered. Bored CIP concrete piles may be designed based on the ultimate skin friction or end-bearing resistance provided in the following table.

TABLE 4: BORED CIP PILES – PILE RESISTANCE

Soil Type	Depth (mbgl)	Elevation (m)	Ultimate (kPa)		Factored ULS (φ = 0.4)	
			Skin Friction	End Bearing	Skin Friction	End Bearing
Frost	1.5*	-	-	-	-	-
Upper Clay	1.5 to 4.5	Above 741.5	45	-	18	-
Lower Clay	> 4.5	Below 741.5	50	1000**	20	400**

* 2.7 m for unheated structures
** End bearing values for piles below minimum embedment depth

The factored values in this table are calculated by multiplying the ultimate values by a geotechnical resistance factor (GRF) of 0.4. The GRF for resistance to axial compression may be increased if the pile capacities are verified by a dynamic monitoring method or by static load test.

The upper 1.5 m of pile shaft, or the length of pile shaft in new fill, whichever is greater, should be assumed to carry no load. This depth should be increased to 2.7 m for structures that are not continuously heated to at least 10°C when the foundation will not freeze. Due to the significant potential for disturbance at the tip of the pile, end bearing should not be considered for straight shaft piles without further detailed review. For end-bearing piles, the shaft should be assumed to carry no load. For belled piles, care must be taken to provide a bearing surface at the base of the pile free from all loose and disturbed soil. It should be noted that pile bells must be formed within the self-supporting clay soils.

6.5.2 Other Design and Construction Recommendations for CIP Piles

1. For bored CIP concrete friction piles, the end bearing contribution to pile capacity should not be used in the design without further review.

2. Steel casing should be available on-site during construction and should be used to prevent sloughing and groundwater seepage into the drill-hole, if required.

3. To resist uplift forces created by frost action, the minimum depth of straight shaft piles for heated structures should be 6.0 m below final grade. If this embedment requirement cannot be met, the piles should be belled to provide the necessary protection against frost uplift. Belled piles should have a minimum depth of 4.5 m.

4. A minimum shaft diameter of 400 mm is recommended for CIP concrete piles. The minimum centre to centre spacing of straight shaft piles should be 3 pile diameters unless group reduction factors are applied. The minimum spacing of belled piles should be sufficient to provide at least 0.5 m of separation between the edges of adjacent bells founded at similar elevations.

5. If belled CIP concrete piles are used:

   - a bell to shaft ratio up to 3:1 would be acceptable, as long as the angle on the roof of the bell is 45^{O} or steeper, to ensure that the load being transferred from the shaft to the bell is properly spread over the entire area of the bell.

   - the minimum distance from the underside of any sand layer to the roof of the bell should be 1.5 m.

   - to avoid potential settlement in loose soils remaining after shaft is drilled, the base of the bell should be sub-cut by the belling tool at least 50 mm below the final depth achieved by the auger used to drill the shaft.

   - bells should not be placed within sand lenses or sand/gravel layer (if encountered).

6. Steel reinforcement should extend the full length of the pile for belled piles and at least 6 m for friction piles.

7. All pile installations should be inspected by a qualified geotechnical engineer or technician to verify that design criteria are met or exceeded.

6.6 STEEL SCREW PILES – ULS DESIGN

6.6.1 ULS Axial Compression

Steel screw piles are considered to be an acceptable deep foundation option for this site. The design and installation of screw piles is usually undertaken by specialty contractors based on service loads supplied by the structural engineer. The ultimate load carrying capacity of a single helix screw pile in vertical compression may be calculated based on a rational method using the bearing capacity developed by the bottom helix. It is assumed that the maximum screw pile depth would be 9 to 10 m, which is the practical limit for most equipment before costly splicing is required. The recommended formula for the ultimate bearing resistance of screw piles in cohesive soils for the cylindrical shear model are given below:

$$Q_U = (N_C\ C_U\ A_H) + (SF\ \pi\ C_U\ D_H\ (H_3\text{-}H_1)) \qquad \text{(Cohesive Soil H/D} \geq 4\text{)}$$

Where:
Q_U = ultimate axial pile resistance (kN)
N_C = bearing capacity factor for cohesive soils (for H/D > 4 use N_C = 9)
C_U = undrained shear strength of soil at the depth of the helix plates (kPa)
A_H = gross area of the bottom helix (m^2)
SF = spacing factor (use 1 for $(H_3\text{-}H_1)/D_H < 2.5$)
D_H = average diameter of helix plates (m)
H_3 = depth to bottom helix (m)
H_1 = depth to top helix (m)

The “factored” geotechnical resistance for ULS analysis for axial compression should be calculated by multiplying the ultimate values above by a geotechnical resistance factor (GRF) of 0.4 in accordance with the building code. The GRF for resistance to axial compression may be increased to 0.6 if a static load test program is performed.

Note, the formula above is dependent on meeting the stated H/D conditions. If the given H/D requirements are not met, the capacity calculations should be reviewed.

6.6.2 Design Parameters for Steel Screw Piles

Design parameters for the steel screw pile design formulas provided above are given in the following table.

TABLE 5: STEEL SCREW PILE DESIGN PARAMETERS

Design Parameter	Upper Clay	Lower Clay
Elevation (m)	Above 741.5	Below 741.5
Effective Unit Weight of Soil, γ’ (kN/m^3)	20	20
Undrained Shear Strength, C_U (kPa)	95	120

The effective unit weight used in the relationship will depend on the position of the helix relative to the groundwater table. The unit weights presented above are for soils above the groundwater table. Below the groundwater table use $\gamma` = \gamma`_{SOIL} - \gamma`_{WATER}$.

6.6.3 Additional Recommendations for Screw Piles

1. The cylindrical shear model is valid for inter-helix spacings of less than 3 helix diameters in silt deposits and 2 helix diameters in sand. Multiple helixes can be provided to meet this criterion, but if this spacing criterion is not met, the method of determining pile capacity must be reviewed, since individual helix resistance may govern.

2. Screw piles should have a minimum embedment depth from ground surface to achieve sufficient frost jacking resistance. The minimum screw pile embedment depth from ground surface to the top of the top helix should be taken as the greater of five times the helix diameter or the frost depth plus one helix diameter.

3. The maximum torque allowed during installation or "torque rating" of the proposed pile section must be considered during screw pile selection. The torque rating is an allowable value based on the ultimate torque strength or point of torsional fracture for the cross-sectional area of the steel in the pile shaft. The estimated torque required to install the screw pile to the design resistance should not exceed the torque rating for the pile. For preliminary purposes, the minimum required torque may be taken as:

   $$T = Q / K_T \qquad \text{(Hoyt \& Clemence)}$$

   Where: T = Minimum Torque Requirement (kN-m)
   Q = Design resistance of the pile (kN)
   K_T = empirical torque correlation factor which depends on shaft diameter

   The torque rating should be provided by the pile manufacturer.

4. The actual torque required to install the screw pile should be measured for field verification of pile capacity. During installation, the torque applied to the pile should not exceed the torque rating for the pile shaft. Using excessive torque on the pile may cause damage to the pile shaft. For difficult soils, screw piles with several smaller helixes may penetrate the subgrade easier than screw piles with larger helixes. If the screw piles cannot be installed to the design depth without exceeding the torque rating of the pile shaft, the shaft thickness will need to be increased.

5. The minimum allowable centre to centre pile spacing should be taken as at least five helix diameters or 3.0 m, whichever is greater. If groups of piles are installed at a pile spacing less than the minimum, a group reduction factor must be applied to the capacity of each pile.

6. Screw piles installed through new fill should have a negative skin friction (down-drag) load applied to the pile shaft within the entire depth of fill.

7. Corrosion of the pile shaft in a partially saturated medium must be considered in selecting pipe shaft wall thickness. It is suggested to fill the pile shafts with concrete after installation to add strength to the pile shaft and reduce the corrosion potential inside the shaft pipe. Additional protection measures should be based on the manufacturer's recommendations. This may be zinc metal coating or cathodic protection or increase of steel thicknesses.

8. The soil strength is considered to be sufficient to provide lateral confinement to the pile shaft. Therefore, buckling of the shaft is not expected to be a concern.

6.7 FOOTINGS

The site soil conditions are not well suited for footings installed within the high plastic clay deposits. If footings are considered, it is very important that the bearing surface is not allowed to dry out during construction. Due to the potential for subgrade swelling/shrinkage with moisture fluctuations during construction, protection of the bearing surface immediately after excavation with the placement of a mud slab (i.e. a thin layer of lean mix concrete) should be considered. Footings founded on the undisturbed native clay material between 1.5 to 3.5 m below the existing grade may be designed based on the bearing capacity values in the following table:

TABLE 6: BEARING RESISTANCE FOR FOOTINGS IN CLAY

Footing Type	Depth (m)	ULS (kPa)		SLS (kPa)
		Ultimate	Factored	
Strip	1.5 - 3.5	490	245	160
Spread	1.5 - 3.5	590	295	190

The factored Ultimate Limit State (ULS) resistance given above has been calculated by multiplying the ultimate bearing capacity values by a geotechnical resistance factor of 0.5. The serviceability bearing resistance values given above are based on limiting the settlement to less than 25 mm and is applicable to footings to a maximum dimension of 1.2 m wide or 1.5 x 1.5 m^2. If very strict settlement tolerances are required or if larger footings are proposed, the footing sizes and settlement must be reviewed. For larger footings the settlement potential will govern the allowable SLS bearing capacity and lower bearing resistance will be available. While adhering to the minimum footing width set by the NBC, footings should not be oversized to reduce bearing pressure because it may result in extra heave in the case of swelling. If weak soils are encountered during the excavations for the footings an alternative configuration is to replace the weak soils with lean mix concrete or a mat of compacted or free draining gravel.

The bearing resistance provided assumes an undisturbed base free of loosened soil. The following additional recommendations are submitted for footing design and construction:

1. Footings should bear on native clay free from loosened material. Excavation of the footing trenches should be undertaken in a manner to minimize disturbance to the bearing surface.

2. It is crucial not to allow bearing surfaces to dry out during or after construction. In general, the use of protective mud slabs to protect the bearing surface would be prudent and are strongly recommended. The mud slab should be placed immediately after final excavation.

3. For protection against frost action, exterior footings in continuously heated structures should be provided with a minimum depth of ground cover of 1.5 m, with no footing insulation required.

4. Isolated footings and exterior footings in unheated structures will require 2.7 m of ground cover. Styrofoam insulation may be used to prevent frost penetration where adequate depths of ground cover cannot be economically provided.

5. Foundation soils must not be allowed to freeze at anytime during or after construction.

6. Preparation of the bearing surfaces should be monitored by a qualified geotechnical engineer prior to placement of footings to verify that design criteria are met.

6.8 GRADE SUPPORTED SLABS

The site soil conditions are not well suited for slab on grade floor construction due to the presence of high plastic clay subgrade. Ideally, the preferred design option is to provide structurally support slabs. However, it is acknowledged that many local builders and owners choose to use slab on grade floors due to budgetary constraints, and have experienced performance ranging from very poor to good. The swelling potential for the clay on this site is estimated between 50 and 150 mm, depending on what precautions are taken to minimize drying of the subgrade during construction. Possible problems with non-uniform heaving and shrinking can be minimized (but not eliminated) using the construction practices as discussed in Section 6.2.

If proposed, grade supported floor slabs in continuously heated buildings should be designed based on a modulus of subgrade reaction (Ks) of 35,000 kN/m^3 in the native clay for slabs placed on a mud slab or at least 150 mm of compacted gravel base. The following recommendations should be followed:

1. Lightly loaded grade supported concrete slabs (less than 10 kPa) should be underlain with 150 mm of well graded, free draining, crushed gravel compacted to 100 percent of SPMDD;

2. For heavily loaded floor slabs (greater than 10 kPa), the gravel layer should be increased to a minimum of 250 mm of free draining crushed gravel compacted uniformly to 100 percent of SPMDD. The gravel used should be well graded, free draining, granular base with a maximum aggregate size of 50 mm and less than 10 percent passing the 0.080 mm sieve. Alternatively, the slab thickness and reinforcement can be upgraded to provide an equivalent level of support (nominally an extra 25 mm of concrete);

3. It is crucial not to allow the native clay subgrade to dry out during or after construction. Prior to placement of the gravel base for the slab, consideration should be given to controlled wetting of the subgrade underneath the slab to provide moisture for swelling prior to placement of slabs to reduce potential for swelling after the slab is placed.

4. Slabs should be provided with construction joints or sawcuts in accordance with local practice. The concrete slab should be reinforced with steel bars or equivalent wire mesh and dimensioned in accordance with the structural engineer's requirements. The reinforcing bars can be carried through the construction joints. As a minimum, it is recommended that floor slabs should be reinforced with 10 mm bars at 600 mm spacings in both directions, or by an equivalent wire mesh;

5. Slabs should be constructed independently of all walls, columns and grade beams. Slab on grade floors should be tied into the grade beam with dowels at doorways. Alternatively, the slab may tied to grade beams if a construction joint is placed parallel to the wall at a distance of about 2.0 m;

6. Non-load bearing partitions should be designed to accommodate vertical movements of at least 50 mm. Service connections should be flexible enough to allow for small differential movements;

7. If possible, water lines should not be placed beneath slab-on-grade floors. If required, below slab water lines should be placed in trenches which are lined with a geo-membrane and graded to collection areas, so that water from leaks or a pipe break can be contained. Allowing water to be trapped below the slab at this site significantly increases the potential of differential subgrade heave;

8. Heating ducts beneath the floor should not be used unless the subgrade is protected from drying out with the use of rigid insulation; and

9. The drainage, grading and landscaping recommendations given in Section 6.3 should be followed.

To emphasize the comments above, steps can be taken to reduce the detrimental effects of swelling clay on foundations and slab work. Due the nature of these soils however, there is no procedure that can be followed that can totally eliminate the risk of vertical movement other than the provision of a properly protected structural slab.

6.9 GRAVEL PAVEMENT

Gravel surfaced access ways and parking areas have been proposed for this site. These site surfaces access ways will be designated around the site. The proposed pavement sections are based on the assumption that the subgrade will be constructed on a stable, suitably prepared subgrade with a California Bearing Ratio of 3.0. This is indicative of a low level of subgrade support as expected during spring thaw when the subgrade soils will exist in a weakened condition. It is expected that traffic around the site can be controlled so that some of the areas of the site will receive minimum traffic. Therefore, a light and heavy duty traffic gravel pavement structure is proposed. The heavy-duty condition assumes about 25 heavy truck loadings per day over a twenty-year design period (3.7×10^5 ESALs).

6.9.1 Proposed Granular Pavement Design

The proposed pavement sections are considered to be the minimum assuming a stable prepared subgrade. If soft subgrade conditions are encountered, it is assumed that the subgrade will be improved with select clay fill or coarse gravel to support construction traffic and paving activities.

TABLE 7: GRANULAR PAVEMENT STRUCTURES

Pavement Material	Thickness (mm)	
	Light Traffic	Heavy Traffic
Granular Base Course (20 mm)	100	200
Granular Sub-base Course (80 mm)	250	400
Non-Woven Geotextile	Yes	Yes
Geo Grid	Recommended	Recommended
150 mm Subgrade Preparation	Yes	Yes

Additional localized thickening may be required depending on conditions at the time of construction. Thickness of the subbase layer given above is considered to be the minimum requirement assuming no subgrade improvement is required. If required, subgrade improvement gravel and the subbase layer are typically placed together effectively increasing the thickness of the sub-base layer.

No special pre-design considerations are given to thickening the pavement section over backfilled trenches. Thickening the pavement section will not significantly reduce the problems of long term fill settlement. The settlement of trenches is caused mainly by the long term self-weight of the fill, not the short term live loads from traffic. The road section or the thickness of granular subbase placed over trenches in the road bed should be decided based on the level of support expected from the subgrade based on field observations. The final asphalt surface should be delayed as long as possible after placement of the fill to allow time for fill settlement.

Aggregate materials for granular base course should be composed of sound, hard, durable particles free from organics and other foreign material. It is recommended to use aggregates conforming to the following Alberta Transportation specifications.

TABLE 8: AGGREGATE SPECIFICATIONS

	Alberta Transportation
Granular Base Course	Designation 2, Class 20 or 25
Granular Sub-base Course	Designation 6, Class 80

Aggregate material shall be sound, hard, durable particles free from elongated particles, organics or other foreign matter. Lightweight particles shall not exceed 2 percent when tested in heavy liquid with relative density of 2.0. Based on availability of local materials at the time of tendering or construction, alternate materials could be considered upon review by the geotechnical engineer.

6.9.2 Site Grading and Drainage

All pavement area surface and subgrade should be sloped and graded to effectively remove all surface and subsurface water as rapidly as possible. It is recommended to provide adequate surface drainage with cross slope crowns of at least 2 percent on regularly maintained gravel surfaces. Yard areas should be sloped and graded to effectively remove all surface water as rapidly as possible. To minimize the occurrence of surface water ponding in the yard, surface grades of at least 2 percent are recommended. Allowing water to pond on the surface will lead to infiltration of the water into the subgrade which could result in weakening of the subgrade soils. For large yard areas, additional grade may be needed to shed water from the surface.

6.9.3 Filer Fabric

Filter fabric is commonly placed as a separation barrier between fine grained subgrade soils and subsurface gravel features such as base and subbase gravel layers in roads. Proposed geotextiles should be reviewed based on their proposed end use. In conventional road applications woven filter cloth is recommended unless the fabric is used in conjunction with a geo-grid. As a general rule, if a road subgrade is too soft to undertake a conventional subgrade preparation, the use of filter fabric should be considered. Woven fabrics have lower strain characteristics under loads than non-woven filter fabrics (i.e., <25% versus about 100% elongation at failure) resulting in more mobilized strength as subgrades deflect under loads. Non-woven products have better permeability characteristics than woven products (i.e., by 25 times or greater) resulting in better drainage and filtration performance. Also, since non-woven fabric will stretch more before tearing it is better suited for placement on uneven surfaces. If a filter fabric is not placed, more gravel may be needed to achieve the same performance.

For woven filter fabric, it is recommended to use a medium weight material made with high density polypropylene fibres meeting the following Alberta Transportation (AT) specification:

TABLE 9: RECOMMENDED WOVEN GEOTEXTILE SPECIFICATION

Characteristic	Specification	Test Method
Minimum Grab Tensile Strength	1100 N	ASTM D 4632
Minimum Elongation at Break	<50 percent	ASTM D 4632
Minimum Static (CBR) Puncture	3300 N	ASTM D 6241
Minimum Tear Strength	400 N	ASTM D 4533
Maximum Equivalent Opening Size	600 microns	ASTM D 4751

The recommended non-woven geo-textile should be made from pre-stressed, high modulus polypropylene strands or yarn woven into a dimensionally stable fabric with the following medium weight AT specification:

TABLE 10: RECOMMENDED NON-WOVEN GEOTEXTILE SPECIFICATION

Characteristic	Specification	Test Method
Minimum Grab Tensile Strength	650 N	ASTM D 4632
Maximum Elongation at Break	50 percent	ASTM D 4632
Minimum Static (CBR) Puncture	1100 N	ASTM D 6241
Minimum Tear Strength	250 N	ASTM D 4533
Maximum Apparent Opening Size	0.2 mm	ASTM D 4751
Minimum Flow Rate	102 L/sec/m^2	ASTM D 4491

A slightly less robust geotextile could be given consideration based on initial field performance.

6.9.4 Geo-Grid

Geo-grids may be considered for yard rehabilitation in areas of soft subgrade. In road construction, the use of geo-grids may be considered for either subgrade improvement (in conjunction with a filter cloth) or within the pavement section as base reinforcement. Typical geo-grids should be integrally formed by punching and stretching high density polyethylene. The feasibility of using any geo-synthetics will be dependent on cost and anticipated performance based on field observations and a review of the proposed surface development.

A medium strength geo-grid is proposed for typical pavement reinforcement applications. The following table provides a traditional product specification for medium strength bi-axial grid.

TABLE 11: BI-AXIAL GEO-GRID SPECIFICATIONS

Test Parameter	Medium Strength	Test Method
Minimum Ultimate Tensile Strength	12.0 (17.5) kN-m	ASTM D 6637
Minimum Junction Efficiency	90%	GRI GG2
Min. Tensile Strength @ 2% Strain	4.0 (6.5) kN-m	ASTM D 6637
Min. Tensile Strength @ 5% Strain	8.5 (13.0) kN-m	Direct Measure
Aperture Size	25 – 40 mm	ASTM D 6637
Minimum Flexural Rigidity	250,000 mg-cm	ASTM D 1388

6.10 EXCAVATIONS

Excavations will be required for foundations and underground utility installations. All excavation work must comply with the requirements of the Alberta Occupational Health and Safety Act (OHS Act, 2021), OHS Regulation (2021) and OHS Code (2024). The OHS Code contains the technical requirements that support the Act and Regulation. Excavation side slopes are not expected to be able to stand near vertical for extended periods of time. For all excavations deeper than 1.5 m in the native clay, side slopes should be cut back to 1H:1V. If space does not permit the slopes to be cut back, some form of temporary shoring must be installed to protect workers in the trench. For excavations into the groundwater table, flatter side-slopes may be required. All temporary surcharge loads should be kept back from the excavated faces a distance of at least one-half the depth of the excavation. All vehicles delivering materials to the site should be kept back from excavated faces at least 1.0 m or one times the excavation depth, whichever is greater.

6.11 WATER POND

It is understood that the proposed development will include a water pond located on the south side of the proposed fire hall. The groundwater was measured between 6.5 and 7.4 mbgl (elevations 738.1 and 739.4 m). Based on conceptual design depths and current ground elevations, the base elevation of the water pond is expected to range between 737 and 740 m (approximately 6 to 9 mbgl). At this depth, the pond base will be situated within the very stiff clay, below the anticipated groundwater table. The pond's design water level is expected to be well above the water table. Therefore, seepage losses are expected. At this depth, natural clay liner is not suitable.

Subsurface conditions at the proposed pond location consist of high-plasticity lacustrine clay with very low permeability when properly scarified and compacted. Visual classification and laboratory testing indicate that the soils at the pond base are consistent with the materials encountered throughout the site. Two potential liner systems may be considered for the pond.

6.11.1 High Density Polyethylene (HDPE) Liner

In review of this assessment, it is recommended to use a geosynthetic liner. Due to shallow groundwater at this site, one of the main design issues for HDPE liner will be buoyancy acting on the underside of the liner when the pond is emptied and the groundwater levels inside the pond are lower than the steady state groundwater table outside of the pond. If the weight of water and ballast inside the pond are less than the pressure head from the groundwater table, water will migrate into the space between the cut face of the soil basin and the liner and potentially float the liner if the pressure imbalance is high enough. The pond would be most susceptible to this issue when the pond is discharged and water levels are low.

The typical way of handling this situation is to provide an under-drain layer below the base of the pond and drain it off to a manhole well beside the pond. Water in the manhole is typically re-circulated back into the pond via a pump when levels are close to or above the pond surface. Direct discharge from the manhole is generally not desirable since it may require permitting, whereas re-circulating the water into the pond would be covered by the storm pond's permit. The use of the under-drain also has the secondary benefit of allowing the water to be tested and compared to the water in the pond as a means of checking for leaks. A suggested design for the sub-drain system is provided in Figure 4. The installation of a HDPE liner will require de-watering measures to be in place and used during construction.

6.11.2 Compacted Clay Liner

If the client is willing to accept some risk, the native clay can be used if the pond geometry is revised such that the bottom of the pond is located above the local groundwater seepage elevation (6.1 mbgl). This would typically involve reducing the pond depth and increasing the pond perimeter to maintain required storage volume. The native clay was found to be high plastic and is considered marginally suitable due to plasticity. It should be noted that this may result in design and maintenance implications for slopes and exposed surfaces (i.e. desiccation). The clay liner system should be constructed by scarifying the pond base and side slopes to a depth of at least 300 mm, moisture conditioning to 0 to 2 percent above OMC, and compacting to a minimum of 98 percent of SPMDD.

Long-term, high plastic clays are subject to shrinkage and cracking due to soil moisture losses caused by seasonal drying and freezing. This can significantly affect the performance of clay in exposed slopes between the high and low water levels in the reservoirs, since shrinkage cracks will result in higher seepage potential than in similar undisturbed clay deposits. If this soil is not protected the depth of future desiccation would be around 1 to 2 m depth found in the existing native deposits. Even though the permeability of the desiccated clay zone may slightly higher than the intact clay, it will still be relatively low; and likely still meet the requirements for a natural clay barrier

The weathering/desiccation in the upper interior slope faces can be minimize by installing geotextile and riprap armouring. This protective measure will also help prevent erosion caused by wave action along the shoreline. Side slopes should be vegetated to limit surface evaporation and reduce shrinkage cracking. The outside slopes should also be well vegetated as soon as possible after construction to protect against slumping and erosion. Regular monitoring and proactive maintenance will be required to seal any desiccation cracks or localized seepage zones that develop over time.

6.11.3 Pond Construction

As discussed, the native clay is considered marginally suitable for use as a pond liner. For a clay liner, an interior slope of at least 4H:1V (or flatter) is recommended, whereas for a synthetic liner such as HDPE, a slope of 3H:1V (or flatter) is acceptable.

The subgrade below the drainage layer should be subcut to 300mm, scarified and recompacted in thin lifts. The clay subgrade and at least 1.5 m of the interior berm slope should be compacted to 97 percent of SPMDD at 0 to 2 percent above OMC. The exterior berm slope should be compacted to at least 95 percent of SPMDD. The slope face should be constructed by keying fill into the slope in horizontal layers and cutting the slope face to shape. The pond shoreline should be protected against erosion from wave action because shoreline erosion may destabilize the pond slopes. Side slopes should be vegetated as soon as possible after construction. The outside slopes should also be well vegetated to protect against slumping and erosion.

Groundwater ingress into the pond excavation should be expected during construction. Temporary dewatering may be required during excavation to allow for proper subgrade preparation.

6.12 CONCRETE

The water-soluble sulphate concentrations in this area typically indicate negligible potential for sulphate attack. Therefore, either general use (GU) cement or moderate sulphate resistant MS hydraulic cement can be used for concrete placed in contact with native soil. If using GU cement, the minimum 56 day compressive strength is 25 MPa with a maximum water cement ratio of 0.5.

6.13 INSPECTION

It is recommended that on-site inspection and testing be performed to verify that actual site conditions are consistent with assumed conditions which meet or exceed design criteria. Based on the Alberta Building Code, adequate levels of inspection include: testing of engineered fill, review of all completed bearing surfaces for footings and full time inspection during construction of deep foundations.

7.0 LIMITATIONS

Geological conditions are variable. The recommendations presented in this report, and any subsequent correspondence, are based on an evaluation of information derived from the boreholes, and additional sources of information referenced in this report. If conditions are noted during construction which are believed to be at variance with the conditions described in this report, this office should be contacted immediately.

This report has been prepared for the exclusive use of the Client and their approved agents for the scope of work and property outlined in this report. Use of the report is subject to acceptance of the General Terms, Conditions and Limitations provided in this report. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. ParklandGEO Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We trust that this report meets with your current requirements. If there are any questions, please do not hesitate to contact our office.

Respectfully submitted,
PARKLANDGEO LTD.

PROFESSIONAL ENGINEER ALBERTA
MARK MEDALLA
ID 93308
2025-07-25

Vishalkumar Shiroya, M.Eng., E.I.T
Geotechnical Engineer in Training

Mark Medalla, P.Eng.
Geotechnical Engineer

APEGA PERMIT TO PRACTICE P007312

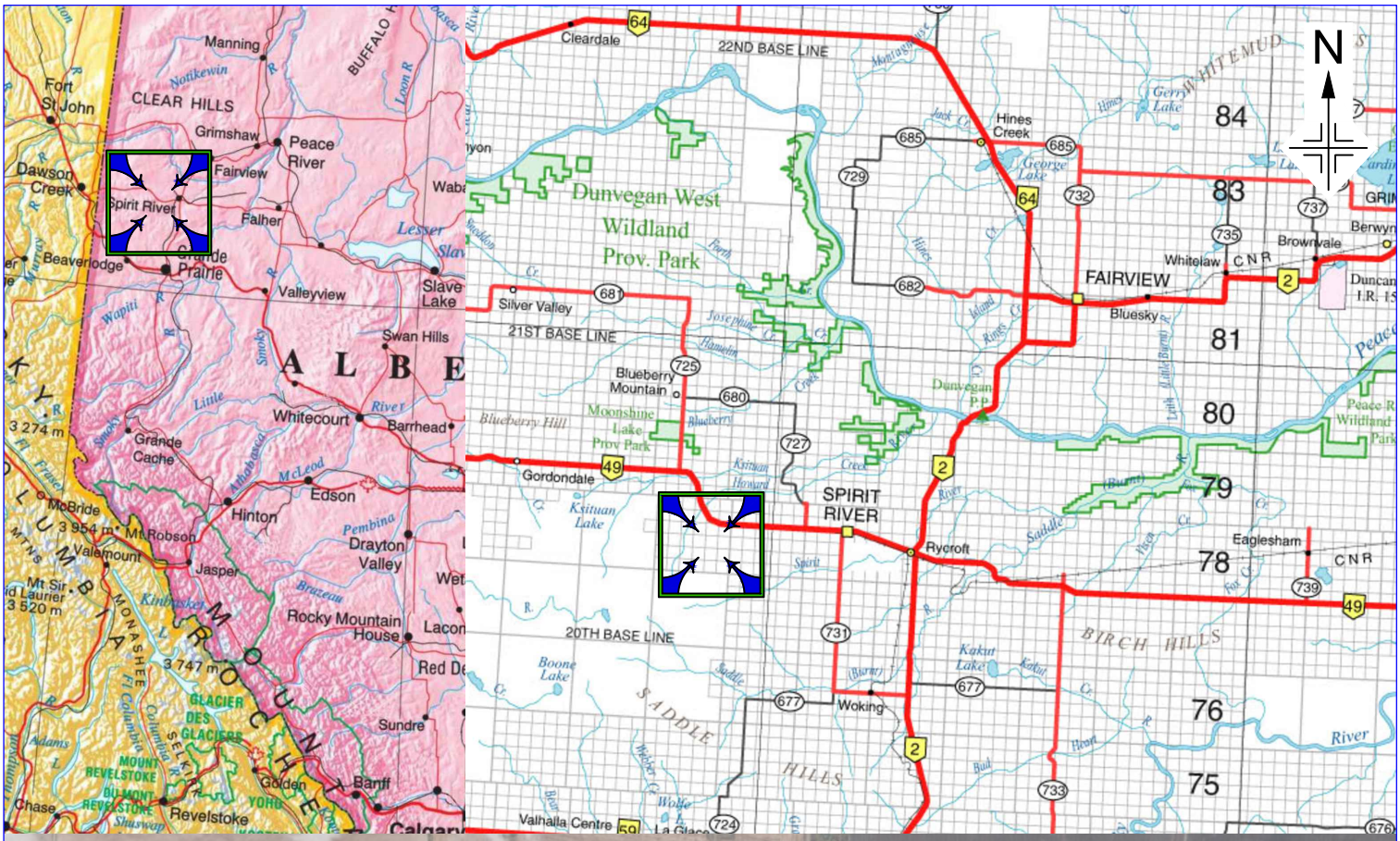
PERMIT TO PRACTICE
PARKLANDGEO LTD.
RM SIGNATURE: _____
RM APEGA ID #: 126404
DATE: 2025-Jul-25
PERMIT NUMBER: P007312
The Association of Professional Engineers and Geoscientists of Alberta (APEGA)

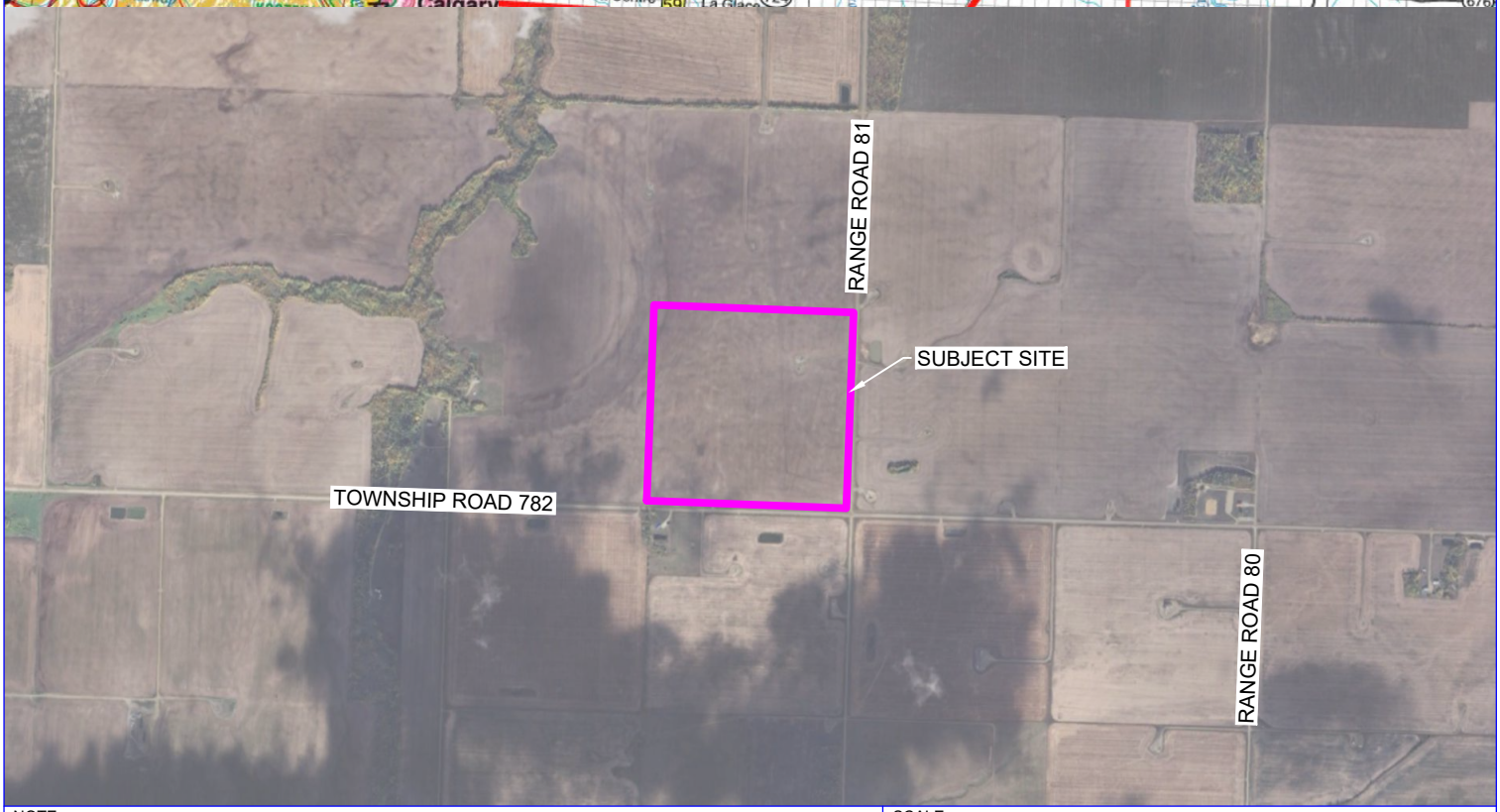
Connan Squair, P.Eng.
Geotechnical Engineer
Responsible Member

Reviewed by:
Russ Washburn
Geotechnical Engineer

FIGURES

Figure 1 – Location Plan
Figure 2 – Area Plan
Figure 3 – Site Plan
Figure 4 – Typical Pond Sub-Drain System
Figure 5 – SPT vs Elevation





NOTE:

1. AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH, DATED SEPTEMBER 29, 2025.

SCALE: NTS

Parkland GEO

CLIENT: SADDLE HILLS COUNTY

LOCATION PLAN

PROPOSED FIRE HALL AND WATER POND
SE 14-078-08 W6M, WEST OF RYCROFT, ALBERTA

DRAWN:	CHK'D.:	DATE:
JW	NM	JULY 2025
PROJECT NO.	REV. NO.	FIGURE NO.
25-0555GP	0	1

NOTE:

1. AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH, DATED SEPTEMBER 29, 2025.

SCALE: 0 75 150 300

SCALE (metres): 1:7500

Parkland GEO

CLIENT:

SADDLE HILLS COUNTY

AREA PLAN

PROPOSED FIRE HALL AND WATER POND
SE 14-078-08 W6M, WEST OF RYCROFT, ALBERTA

DRAWN:	CHK'D.:	DATE:
JW	NM	JULY 2025
PROJECT NO.	REV. NO.	FIGURE NO.
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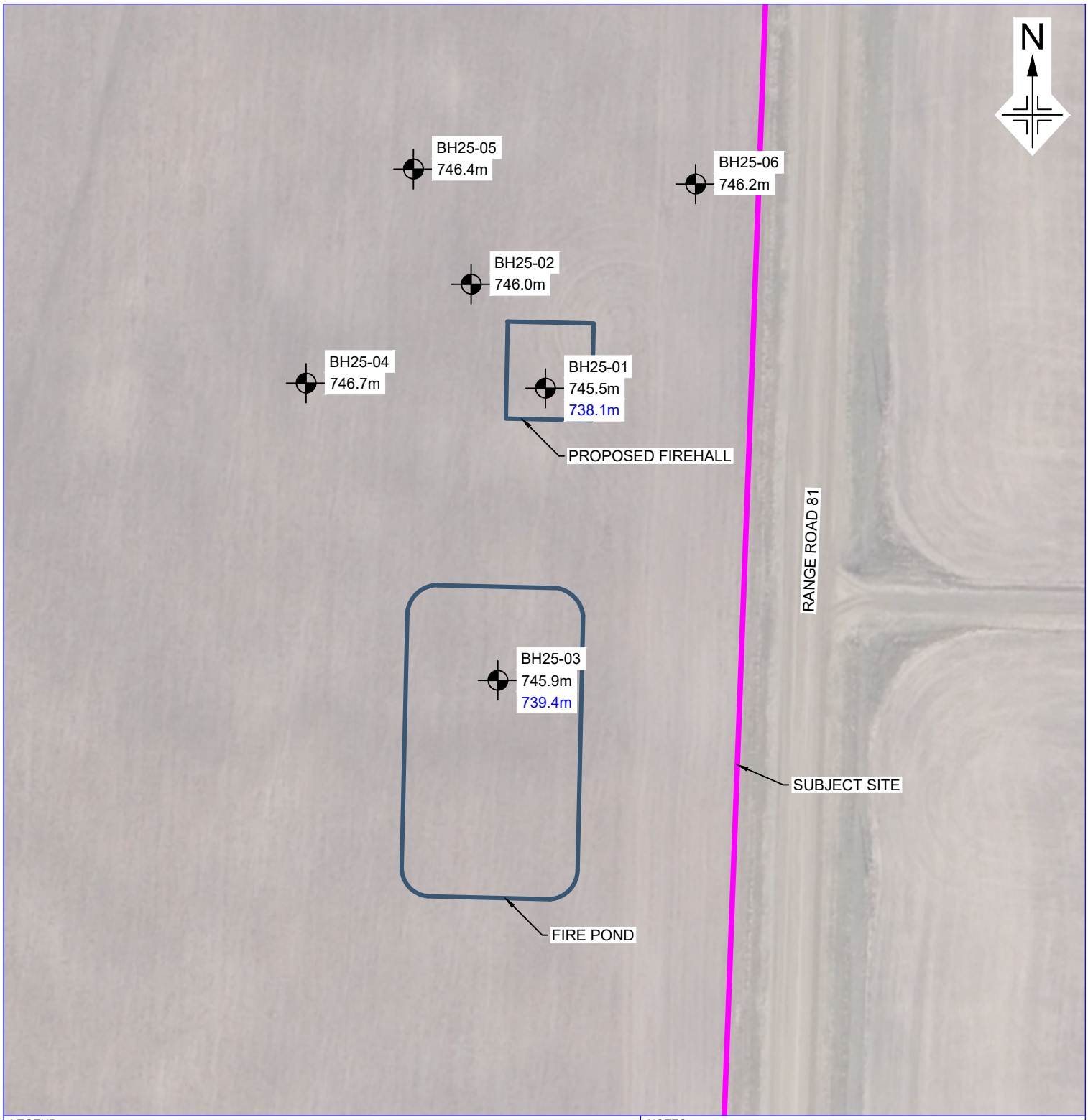

LEGEND:

PARKLANDGEO BOREHOLE

745.5m SURFACE ELEVATION

738.1m GROUNDWATER ELEVATION (JUNE 11, 2025)

NOTES:

1. SITE PLAN PROVIDED BY CLIENT (PROPOSED SITE FOR NEW FIREHALL V2, DATED MAY 16, 2025).
2. AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH, DATED SEPTEMBER 29, 2025.
3. ALL BOREHOLE LOCATIONS ARE APPROXIMATE.
4. NOT TO BE USED FOR DESIGN OR CONSTRUCTION PURPOSES.

SCALE: 0 12.5 25 50

SCALE (metres): 1:1250

ParklandGEO

CLIENT:

SADDLE HILLS COUNTY

SITE PLAN

PROPOSED FIRE HALL AND WATER POND
SE 14-078-08 W6M, WEST OF RYCROFT, ALBERTA

DRAWN:	CHK'D.:	DATE:
JW	NM	JULY 2025
PROJECT NO.	REV. NO.	FIGURE NO.
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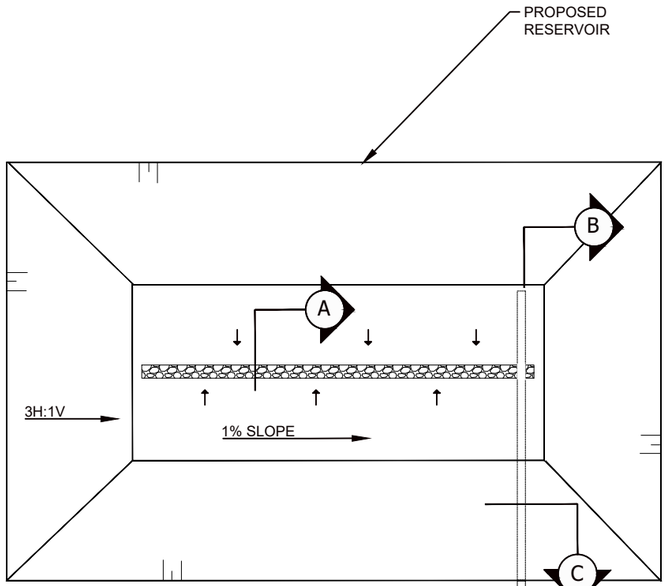
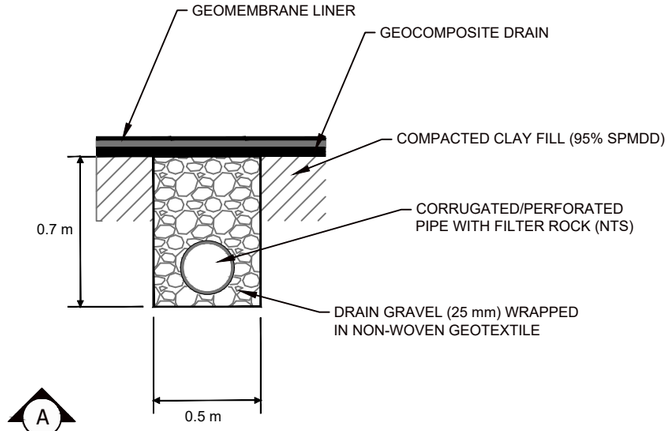
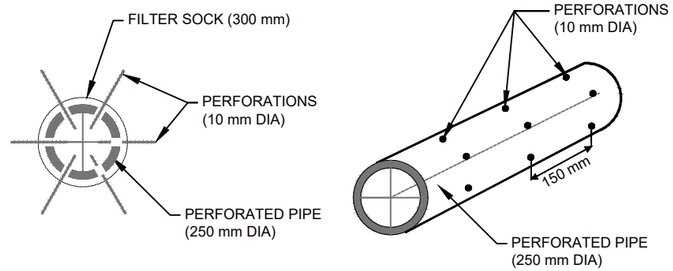


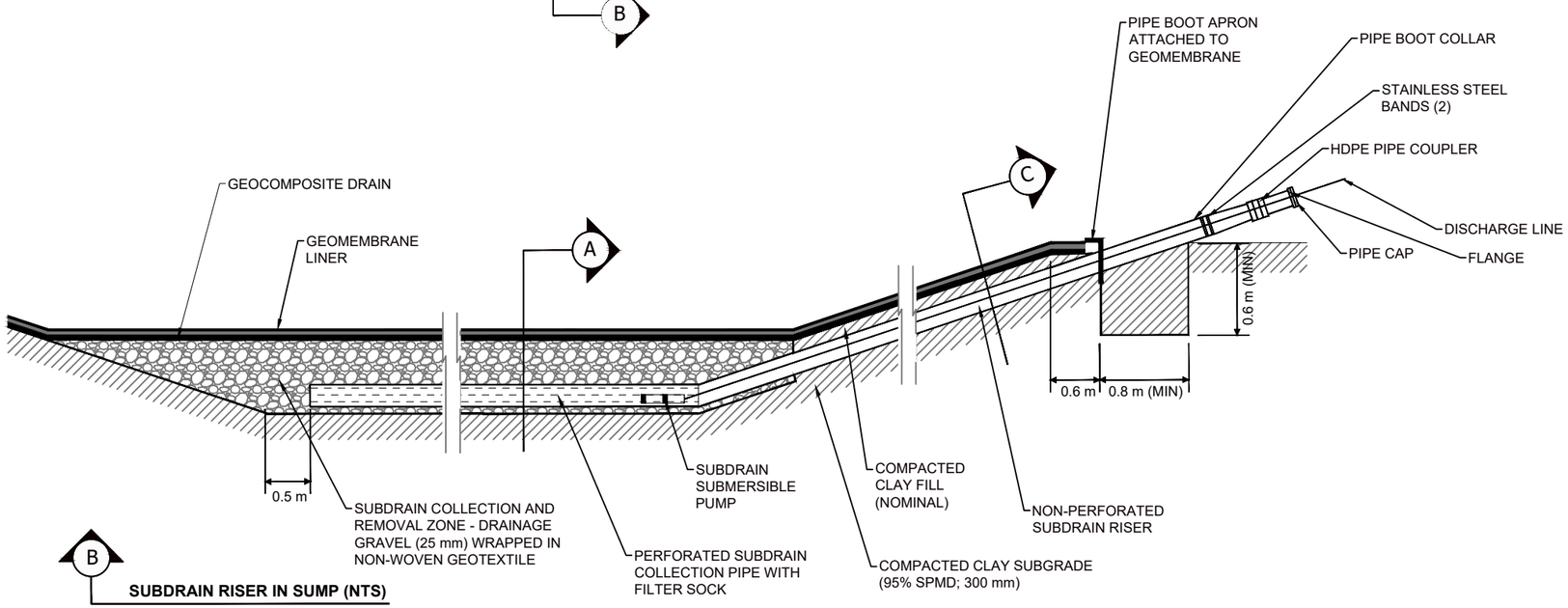
CONTAINMENT POND PLAN (NTS)



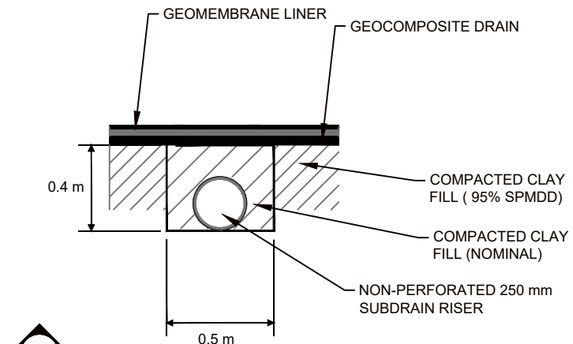
SUBDRAIN PIPE COLLECTION TRENCH (NTS)



SUBDRAIN COLLECTION PIPE PERFORATION DETAIL (NTS)



SUBDRAIN RISER IN SUMP (NTS)



SUBDRAIN RISER PENETRATION BOOT (NTS)

NOT FOR CONSTRUCTION

NOTES:

1. GEOCOMPOSITE DRAIN WITH DOUBLE SIDED GEOTEXTILE COVER
2. GEOMEMBRANE (NOMINAL 60 mil LLDPE OR HDPE TEXTURED LINER)
3. 25 mm DISCHARGE LINE DIRECTED INTO RESERVOIR, INCLUDING ELECTRICAL, SAFETY CABLE AND SCADA SENSORS

REV #	DATE	DETAILS

DRAWN:	CHK'D.:	REV #:	DATE:
JW	NM	0	JULY 2025

Parkland GEO

CLIENT:

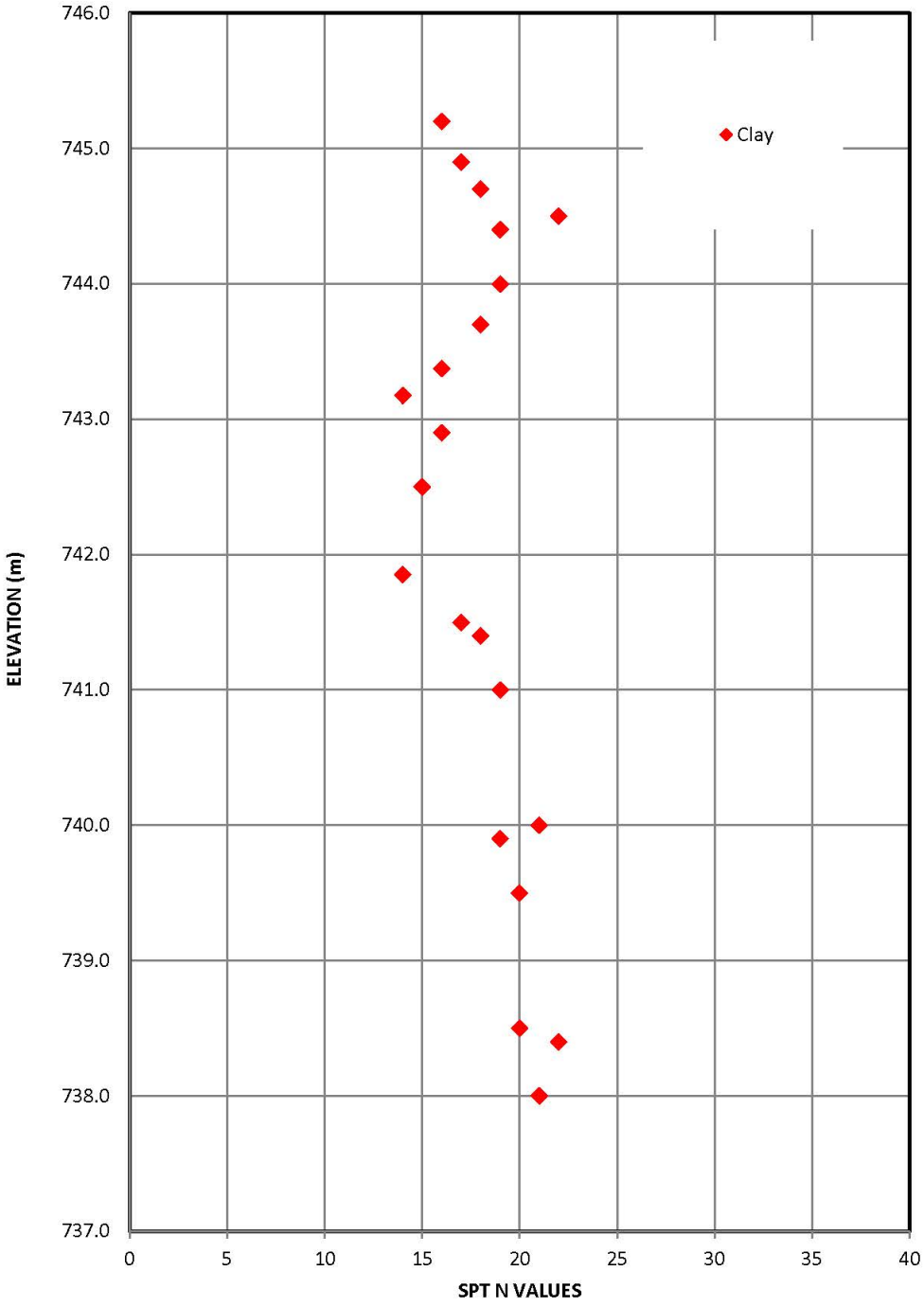
SADDLE HILLS COUNTY

TYPICAL POND SUB-DRAIN SYSTEM

PROPOSED FIRE HALL AND WATER POND
SE 14-078-08 W6M, WEST OF RYCROFT, ALBERTA

SCALE:	JOB NO.	DRAWING NO.
NTS	25-0555GP	FIGURE 4

Saddle Hills Fire Hall



Parkland GEO	CLIENT: SADDLE HILLS COUNTY	**SPT VS ELEVATION**
		PROPOSED FIRE HALL AND WATER POND SE 14-078-08 W6M, WEST OF RYCROFT, ALBERTA
		DRAWN: JW / CHK'D.: NM / DATE: JULY 2025
		PROJECT NO.: 25-0555GP / REV. NO.: 0 / FIGURE NO.: 5

Photograph 1: Looking East, site entrance from RR81 (May 28, 2025).

Photograph 2: Borehole 25-03 on completion (May 28, 2025).

APPENDIX A

Borehole Logs (6)
Explanation of Terms and Symbols

BOREHOLE NO.: 25-01

CLIENT: Saddle Hills County
SITE: Happy Valley Firehall, West of Rycroft

PROJECT NO.: 25-0555GP
BH LOCATION:

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl; 25 50 75)	Type	Sample No	SPT (N)	Comments	Well Completion Details	Elevation (m)
	GROUND SURFACE								745.5
0	***Topsoil (150 mm)*** Rootlets inclusions, dry		20						
	Clay Some silt, little to some sand, trace gravel, trace sand pockets inclusions, grey, medium to high plastic, very stiff, dry		18	G	1G1				
1									
	- At 1.5 m, brown, medium plastic, coal inclusions, sand pockets, damp to moist		19		1D1	19			
2				G	1G2				
			21						
3					1D2	15			
			22	G	1G3				
4	- At 4.5 m, increase in gravel								
			21		1D3	19			
5				G	1G4				
			22				Grain Size Analysis (at 6.8 m) Clay: 45% Silt: 33% Sand: 13% Gravel: 9% Atterberg Limits PL: 14% LL: 54% PI: 40%		
6	- At 6.0 m, sand lenses, dark grey pockets inclusions, trace gypsum				1D4	20			
			21	G	1G5				
7									
	- At 7.5 m, trace cobbles, trace seepage				1D5	21			
8	END OF BOREHOLE								737.5
	Borehole dry upon completion. Groundwater at 7.4 m on June 11, 2025.								
9									
10									
11									

Well Completion Details: Bentonite; Solid PVC Pipe; Auger Cuttings; Slotted PVC Pipe; Sand; June 11, 2025

LOGGED BY: MP
CONTRACTOR: Frontier Enviro-Drilling Ltd.
RIG/METHOD: Track Mounted Solid Stem Augers
NOTES:

DATE: May 28, 2025
GROUND ELEVATION (m): 745.5
NORTHING (m): 6180874
EASTING (m): 0367386

Parkland GEO

CLIENT: Saddle Hills County
SITE: Happy Valley Firehall, West of Rycroft

BOREHOLE NO.: 25-02
PROJECT NO.: 25-0555GP
BH LOCATION:

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl; 25 50 75)	Type	Sample No	SPT (N)	Comments	Well Completion Details	Elevation (m)
	GROUND SURFACE								746.0
0	***Topsoil (150 mm)*** Rootlets inclusions, dry		16, 16				Grain Size Analysis (at 0.7 m) Clay: 54% Silt: 36% Sand: 10% Atterberg Limits PL: 15% LL: 58% PI: 43%	Auger Cuttings	
1	***Clay*** Some silt, trace to little sand, trace gravel, grey to brown, medium to high plastic, very stiff, dry to damp		15	G	2G1				
	- At 1.5 m, sand pockets, trace cobbles, damp				2D1	22			
2			21	G	2G2				
3			19		2D2	15			
4			21	G	2G3				
					2D3	17			
5			21	G	2G4				
6	- At 6.1 m, trace oxides, trace gravel		21		2D4	21			
7			21	G	2G5				
	- At 7.6 m, trace cobbles, dark grey pockets				2D5	20			738.0
8	END OF BOREHOLE								
	Borehole dry upon completion.								
9									
10									
11									

LOGGED BY: MP
CONTRACTOR: Frontier Enviro-Drilling Ltd.
RIG/METHOD: Track Mounted Solid Stem Augers
NOTES:

DATE: May 28, 2025
GROUND ELEVATION (m): 746.0
NORTHING (m): 6180898
EASTING (m): 0367369

BOREHOLE NO.: 25-03

CLIENT: Saddle Hills County
SITE: Happy Valley Firehall, West of Rycroft

PROJECT NO.: 25-0555GP
BH LOCATION:

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl; 25 50 75)	Type	Sample No	SPT (N)	Comments	Well Completion Details	Elevation (m)
	GROUND SURFACE								745.9
0	***Topsoil (130 mm)*** Rootlets inclusions, dry		16						
	Clay Little to some silt, some sand, trace gravel, grey, medium to high plastic, very stiff, dry to damp - At 1.2 m, damp to moist		18	G	3G1				
1							Grain Size Analysis (at 2.3 m) Clay: 51% Silt: 35% Sand: 14% Atterberg Limits PL: 14% LL: 54% PI: 40%		
			19		3D1	19			
2			19	G	3G2				
	- At 3.0 m, coal inclusions, trace oxides		19		3D2	16			
3									
			21	G	3G3				
4	- At 4.6 m, trace cobbles, sand pockets				3D3	18			
			21	G	3G4				
5									
	- At 6.1 m, grey pockets inclusions, trace oxides, trace groundwater seepage, trace gypsum		21		3D4	19			
6				G	3G5				
			21						
7	- At 7.6 m, moist to wet				3D5	22			
									737.9
8	END OF BOREHOLE Borehole dry upon completion. Groundwater at 6.5 m on June 11, 2025.								
9									
10									
11									

Well Completion Details: Bentonite; Solid PVC Pipe; Auger Cuttings; Slotted PVC Pipe; Sand; ▼ June 11, 2025

LOGGED BY: MP
CONTRACTOR: Frontier Enviro-Drilling Ltd.
RIG/METHOD: Track Mounted Solid Stem Augers
NOTES:

DATE: May 28, 2025
GROUND ELEVATION (m): 745.9
NORTHING (m): 6180807
EASTING (m): 0367375

BOREHOLE NO.: 25-04

Parkland GEO

CLIENT: Saddle Hills County
SITE: Happy Valley Firehall, West of Rycroft

PROJECT NO.: 25-0555GP
BH LOCATION:

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl; 25 50 75)	Type	Sample No	SPT (N)	Comments	Well Completion Details	Elevation (m)
0	GROUND SURFACE								746.7
0	***Topsoil (150 mm)*** Rootlets inclusions, dry		15						
1	***Clay*** Little to some silt, some sand, trace gravel, trace oxides inclusions, grey to brown, medium to high plastic, very stiff, dry to damp - At 1.1 m, damp		14	G	4G1				
2	- At 1.5 m, sand pockets		20		4D1	16			
2				G	4G2				
3	- At 3.0 m, trace rootlets		20		4D2	18		Auger Cuttings	743.2
4	END OF BOREHOLE Borehole dry upon completion.								

LOGGED BY: MP
CONTRACTOR: Frontier Enviro-Drilling Ltd.
RIG/METHOD: Track Mounted Solid Stem Augers
NOTES:

DATE: May 28, 2025
GROUND ELEVATION (m): 746.7
NORTHING (m): 6180875
EASTING (m): 0367331

BOREHOLE NO.: 25-05

CLIENT: Saddle Hills County
SITE: Happy Valley Firehall, West of Rycroft

PROJECT NO.: 25-0555GP
BH LOCATION:

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl; 25 50 75)	Type	Sample No	SPT (N)	Comments	Well Completion Details	Elevation (m)
	GROUND SURFACE								746.4
0	***Topsoil (170 mm)*** Rootlets inclusions, dry		17						746.2
	Clay Some silt, some sand, trace gravel, trace oxides inclusions, brown, medium to high plastic, stiff to very stiff, dry to damp - From 0.2 to 1.2 m, sand lenses - At 1.2 m, moist		16	G	5G1				
1									
			22		5D1	17			
2				G	5G2				
			21						
3					5D2	16			
	- At 3.8 m, increase in gravel, trace oxides		23	G	5G3				
4	- From 4.5 to 5.0 m, trace gypsum								
					5D3	14			
5	END OF BOREHOLE Borehole dry upon completion.							Auger Cuttings	741.4
6									
7									
8									
9									
10									
11									

LOGGED BY: MP
CONTRACTOR: Frontier Enviro-Drilling Ltd.
RIG/METHOD: Track Mounted Solid Stem Augers
NOTES:

DATE: May 28, 2025
GROUND ELEVATION (m): 746.4
NORTHING (m): 6180925
EASTING (m): 0367356

Parkland GEO

CLIENT: Saddle Hills County
SITE: Happy Valley Firehall, West of Rycroft

BOREHOLE NO.: 25-06
PROJECT NO.: 25-0555GP
BH LOCATION:

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl; 25 50 75)	Type	Sample No	SPT (N)	Comments	Well Completion Details	Elevation (m)
	GROUND SURFACE								746.2
0	***Topsoil (160 mm)*** Rootlets inclusions, dry		16						
	Clay Some silt, some sand, trace gravel, grey, medium to high plastic, stiff to very stiff, dry to damp - At 1.3 m, damp		15	G	6G1				
1									
			20		6D1	18			
2				G	6G2				
			20						
3	- At 3.0 m, trace oxides, sand pockets, trace cobbles				6D2	14		Auger Cuttings	742.7
	END OF BOREHOLE								
4	Borehole dry upon completion.								
5									
6									
7									
8									
9									
10									
11									

LOGGED BY: MP
CONTRACTOR: Frontier Enviro-Drilling Ltd.
RIG/METHOD: Track Mounted Solid Stem Augers
NOTES:

DATE: May 28, 2025
GROUND ELEVATION (m): 746.2
NORTHING (m): 6180921
EASTING (m): 0367421

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EXPLANATION OF TERMS AND SYMBOLS

The terms and symbols used on the borehole logs to summarize the results of the field investigation and subsequent laboratory testing are described on the following pages.

The borehole logs are a graphical representation summarizing the soil profile as determined during site specific field investigation. The materials, boundaries, and conditions have been established only at the borehole location at the time of drilling. The soil conditions shown on the borehole logs are not necessarily representative of the subsurface conditions elsewhere across the site. The transitions in soil profile can have gradual rather than distinct boundaries.

1. **PRINCIPAL SOIL TYPE** – The major soil type by weight of material or by behaviour.

Material	Grain Size
Boulders	Larger than 300 mm
Cobbles	75 mm to 300 mm
Coarse Gravel	19 mm to 75 mm
Fine Gravel	5 mm to 19 mm
Coarse Sand	2 mm to 5 mm
Medium Sand	0.425 mm to 2 mm
Fine Sand	0.075 mm to 0.425 mm
Silt	0.020 to 0.075 mm
Clay	Smaller than 0.020 mm

2. **DESCRIPTION OF MINOR SOIL TYPE** – Minor soil types are identified by weight of minor component.

Descriptor	Percent
and	35 to 50
some	20 to 35
little	10 to 20
trace	1 to 10

3. **CONSISTENCY OF FINE GRAINED SOILS** – The following terms are used relative to undrained shear strength and Standard Penetration Test (SPT), N value, for blows per 300 mm penetration (ASTM D1586).

Description	Undrained Shear Strength, C_u (kPa)	SPT N Value
Very Soft	Less than 12	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 150	15 to 30
Hard	Over 150	Over 30

4. **RELATIVE DENSITY OF COARSE GRAINED SOIL** – The following terms are used relative to Standard Penetration Test (SPT), N value, for blows per 300 mm penetration (ASTM D1586).

Description	SPT N Value
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Over 50

5. **TYPICAL SEDIMENTARY BEDROCK TYPES AND CLASSIFICATION** – The following terms are based on visual inspection and field/laboratory identification tests.

Characteristic	Sandstone	Mudrocks			
		Siltstone	Mudstone	Clayshale	Claystone
Composition	>50% Sand $CaCO_3$ or silica binder. Use weak acid to test for $CaCO_3$.	>50% Silt	33% to 66% Silt & 33% to 66% Clay	>50% Clay & <33% Silt	
Bedding	Banding possible Non- Fissile Wackes – dirty sandstone matrix (>15% clay)	Non-Fissile & Non-laminated	Non-Fissile & Non-laminated	Fissile	Non-Fissile

Definitions

Fissile Breaks apart on bedding planes, not fractures.
Shale Only used to describe a fissile clay mudrock.
Slate Hard mudstone exposed to high pressure and temperature.
Limestone Sedimentary rock (i.e. particles) formed from calcium carbonate minerals from skeletal fragments of marine organisms such as coral. Particles generally too small to see with eye.

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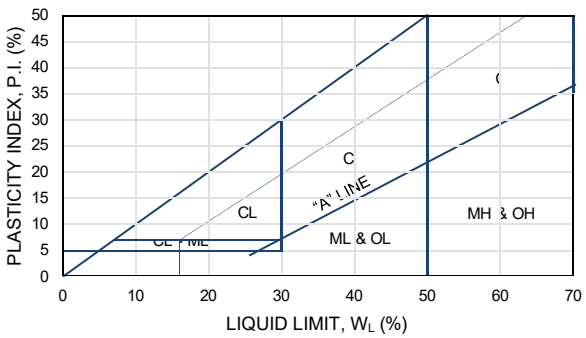
EXPLANATION OF TERMS AND SYMBOLS

MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISION			GROUP SYMBOL	GRAPH SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN NO. 200 SIEVE)	GRAVELS MORE THAN HALF COARSE GRAINS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	GW		WELL GRADED GRAVELS, GRAVEL-SAND MIXTURE, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} \geq 4$ AND $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP		POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY GRAVELS (WITH SOME FINES)	GM		SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4
			GC		CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE AND P.I. GREATER THAN 7
	SANDS MORE THAN HALF FINE GRAINS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)	SW		WELL GRADED SANDS, GRAVELLY SANDS WITH LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} \geq 6$ AND $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			SP		POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY SANDS (WITH SOME FINES)	SM		SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4
			SC		CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE AND P.I. GREATER THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT PASSES NO. 200 SIEVE)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML		INORGANIC SILTS & VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH 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 SOILS		
		$30\% < W_L < 50\%$	CI		INORGANIC CLAYS OF MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, 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 AND MEDIUM PLASTICITY		
		$W_L > 50\%$	OH		ORGANIC CLAYS OF HIGH PLASTICITY, ORGANIC SILTS		
HIGHLY ORGANIC SOILS			Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE	



NOTES ON SOIL CLASSIFICATION AND DESCRIPTION:

1. Soil are classified and described according to their engineering properties and behaviour.
2. Boundary classification for soil with characteristics of two groups are given combined group symbols (e.g. GW-GC is a well graded gravel sand mixture with clay binder between 5 and 12%).
3. Soil classification is in accordance with the Unified Soil Classification System (ASTM D2487) with the exception that an inorganic clay of medium plasticity (CI) is recognized.
4. The use of modifying adjectives may be employed to define the estimated percentage range of minor components.

6. **TERMS DESCRIBING ROCK MASS** – The following terms related to weathering are based on field measurements.

Joint Classification	Bedding, Laminations, Bands	Spacing (mm)
---	Thinly Laminated	< 6
Extremely Close	Laminated	< 20
Very Close	Very Thin	20 – 60
Close	Thin	60 – 200
Moderate	Medium	200 – 600
Wide	Thick	600 – 2000
Very Wide	Very Thick	2000 – 6000
Extremely Wide	---	> 6000

7. **ROCK QUALITY DESIGNATION** – The following terms are used relative to RQD of Rock Cores, ASTM D6032, which uses modified core recovery percentage in which pieces of core over 100 mm long are counted as recovery.

Rock Mass Quality	RQD
Very Poor	0 – 25%
Poor	25 – 50%
Fair	50 – 75%
Good	75 – 90%
Excellent	90 – 100%

8. **ROCK STRENGTH** – The following terms related to rock strength are based on field identification tests (i.e. hand, geological hammer, and/or pocket knife).

Strength Classification	Grade	Estimated UCS (MPa)	Field Identification
Extremely Weak	R0	0.25 – 1.0	Indented by thumbnail.
Very Weak	R1	1 – 5	Crumbles under firm blows with point of geological hammer and can be peeled by a pocket knife.
Weak	R2	5 – 25	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.
Medium Strong	R3	25 – 50	Cannot be scraped or peeled with a pocket knife, specimen can be fracture with single firm blow of geological hammer.
Strong	R4	50 – 100	Specimen required more than one blow of geological hammer to fracture it.
Very Strong	R5	100 – 250	Specimen required many blows of geological hammer to fracture it.
Extremely Strong	R6	> 250	Specimen can only be chipped with geological hammer.

9. **WEATHERING** – The following terms related to weathering are based on visual inspection and are descriptive only.

Term	Grade	Abbreviation	Description
Fresh	I	F	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.
Slightly Weathered	II	SW	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.
Moderately Weathered	III	MW	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
Highly Weathered	IV	HW	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely Weathered	V	CW	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
Residual Soil	VI	RS	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

Note: Sections 6, 8 and 9 reference the International Society for Rock Mechanics and Rock Engineering (ISRM) *Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses* which provides explanation of recommended procedures to follow in the various aspects of rock characterization.

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EXPLANATION OF TERMS AND SYMBOLS

GENERAL SYMBOLS

FS	factor of safety
V	volume
W	weight
π	Pi = 3.1416
g	acceleration due to gravity = 9.81 m/s^2
t	time
Δ	change in, (e.g. Δt is change in time)

SOIL INDEX PROPERTIES

w, mc	soil water (moisture) content
ρ	bulk density
ρ_d	dry density
ρ_w	density of water
ρ_s	density of solid particles
γ	bulk unit weight = ρg (i.e. mass density x gravity)
γ_d	dry unit weight
γ_w	unit weight of water = 9.81 kN/m^3
γ_s	unit weight of solid particles
γ'	unit weight of submerged soil
D_R	relative density of solid particles ($D_R = \rho_s / \rho_w$) (formerly specific gravity G_S)
e	void ratio
n	porosity
S	degree of saturation
LL, w_L	liquid limit
PL, w_P	plastic limit
PI, I_P	plasticity index = (LL - PL)
w_S	shrinkage limit
I_L	liquidity index = (w - PL) / PI
I_C	consistency index = (LL - w) / PI
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = (e_{max} - e) / (e_{max} - e_{min}) (formerly relative density)

ONE DIMENSIONAL CONSOLIDATION PROPERTIES

c_c	compression index in normally consolidated range
c_r	recompression index in over-consolidated range
c_s	swelling index
c_a	coefficient of secondary consolidation
c_v	coefficient of consolidation
m_v	coefficient of volume change
T_V	time factor (vertical direction)
U	degree of consolidation
σ_p	pre-consolidation stress
OCR	over-consolidation ratio = σ_p / σ'_{vo}

HYDRAULIC PROPERTIES

h	hydraulic head
i	hydraulic gradient
i_v	vertical hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
K	permeability (m^2)
q	volumetric rate of flow
i	velocity of flow
j	seepage force per unit volume

SHEAR STRENGTH PROPERTIES

τ	shear strength = c' + σ' tan φ'
τ	shear strength = q_U / 2
q_U	compressive strength = (σ_1 - σ_3) = 2 c_U
τ_p	peak shear strength
τ_r	residual shear strength
φ'	effective angle of internal friction (phi)
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_U, s_U	undrained shear strength (for φ = 0 case)
p	mean total stress = (σ_1 + σ_3)/2
p'	mean effective stress = (σ'_1 + σ'_3)/2
S_t	sensitivity

STRESS AND STRAIN PROPERTIES

γ	shear strain
ε	linear strain
ε_V	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress = ($\sigma' = \sigma - u$)
u	pore water pressure
σ'_{vo}	initial effective overburden stress
σ_1	major principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress
σ_{oct}	octahedral (mean) stress = (σ_1 + σ_2 + σ_3)/3
τ	shear stress
E	Young's modulus (modulus of elasticity)
G	shear modulus of deformation
G_{max}	small strain shear modulus = ρV_S
V_S	shear wave velocity
K	bulk modulus of compressibility

APPENDIX B

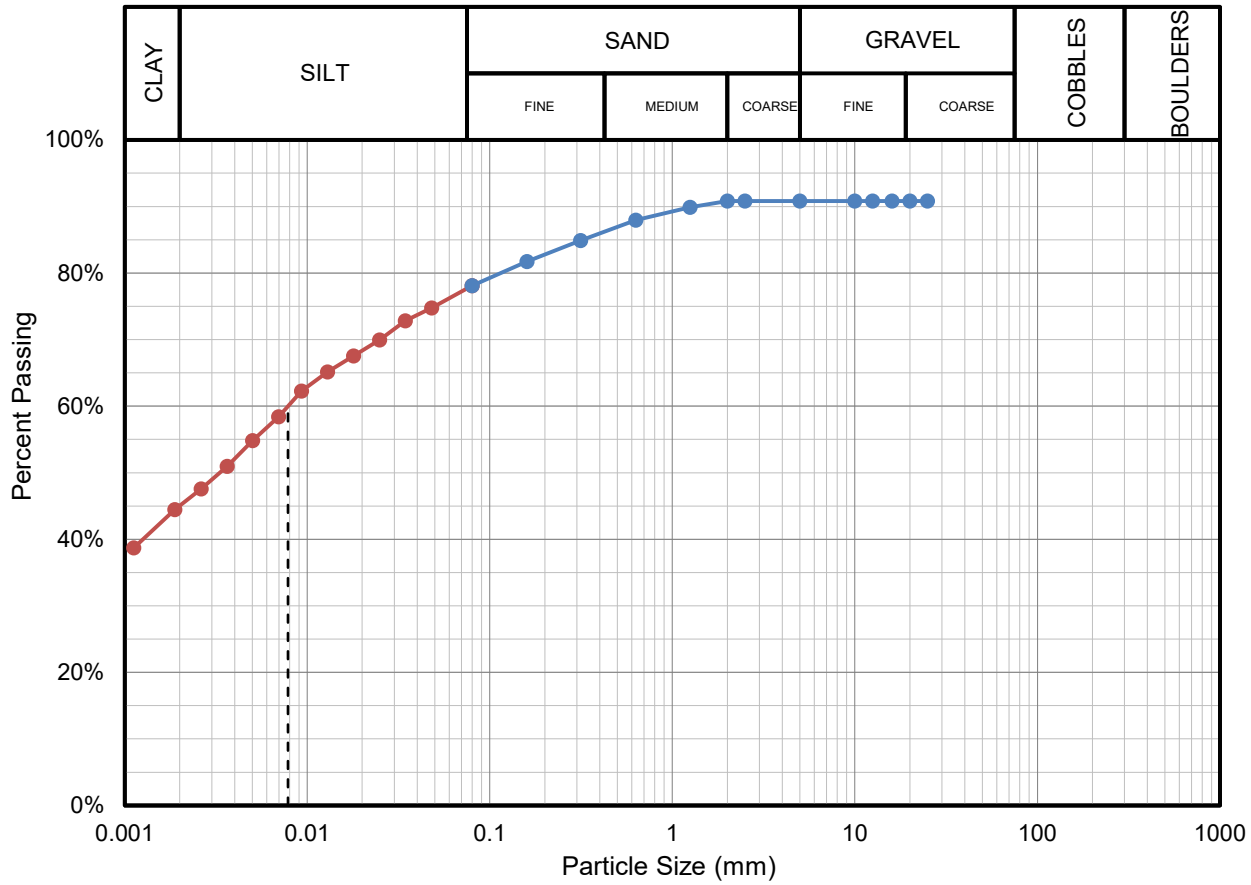
Soil Test Results

PARTICLE-SIZE ANALYSIS, LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY

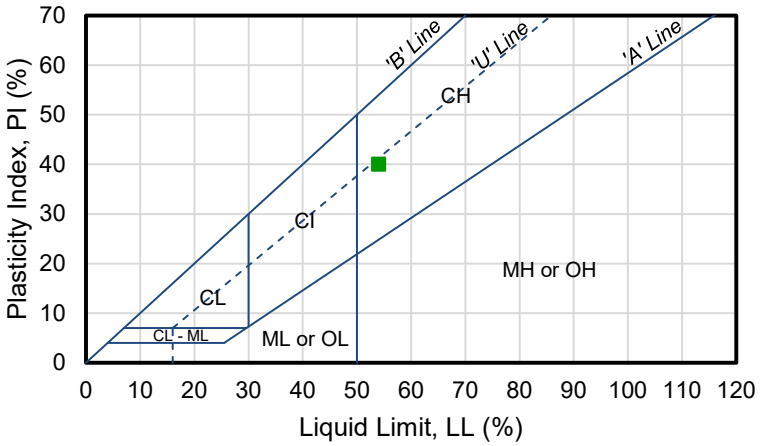
ASTM D422 & ASTM D4318

PROJECT: New Saddle Hills Firehall
PROJECT#: 25-0555GP
CLIENT: Saddle Hills County
SOIL DESCRIPTION: clay, some silt, little sand, trace gravel

SAMPLE DATE: May 28, 2025
TEST DATE: June 9, 2025
SAMPLE ID: 1G5
DEPTH: 6.8 m



PARTICLE-SIZE ANALYSIS	Gravel	9.2%
	Sand	13.1%
	Silt	32.6%
	Clay	45.0%
	D_{10}	---
	D_{30}	---
	D_{60}	0.0078 mm
	C_U	---
	C_C	---
LIMITS	PL	14
	LL	54
	PI	40



Modified Unified Soil Classification	Group Symbol
Fat clay with sand	CH

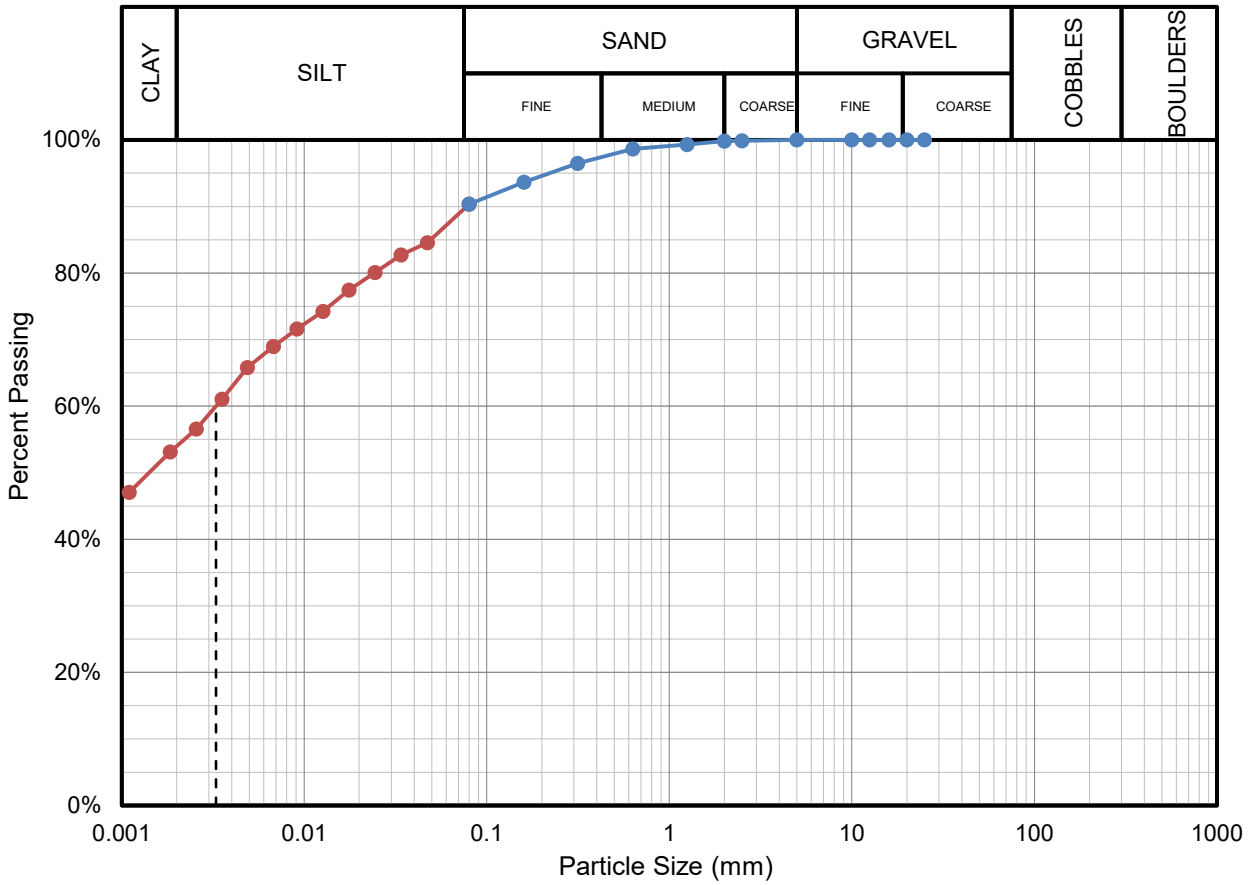
TECH: JL
CHECKED: CS

PARTICLE-SIZE ANALYSIS, LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY

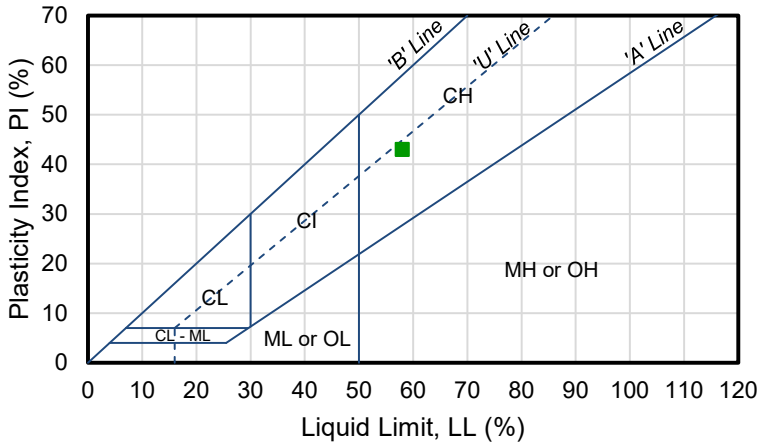
ASTM D422 & ASTM D4318

PROJECT: New Saddle Hills Firehall
PROJECT#: 25-0555GP
CLIENT: Saddle Hills County
SOIL DESCRIPTION: clay, and silt, little sand

SAMPLE DATE: May 28, 2025
TEST DATE: June 9, 2025
SAMPLE ID: 2G1
DEPTH: 0.7 m



PARTICLE-SIZE ANALYSIS	Gravel	0.0%
	Sand	10.4%
	Silt	35.7%
	Clay	54.0%
	D_{10}	---
	D_{30}	---
	D_{60}	0.0033 mm
	C_U	---
	C_C	---
LIMITS	PL	15
	LL	58
	PI	43



Modified Unified Soil Classification	Group Symbol
Fat clay	CH

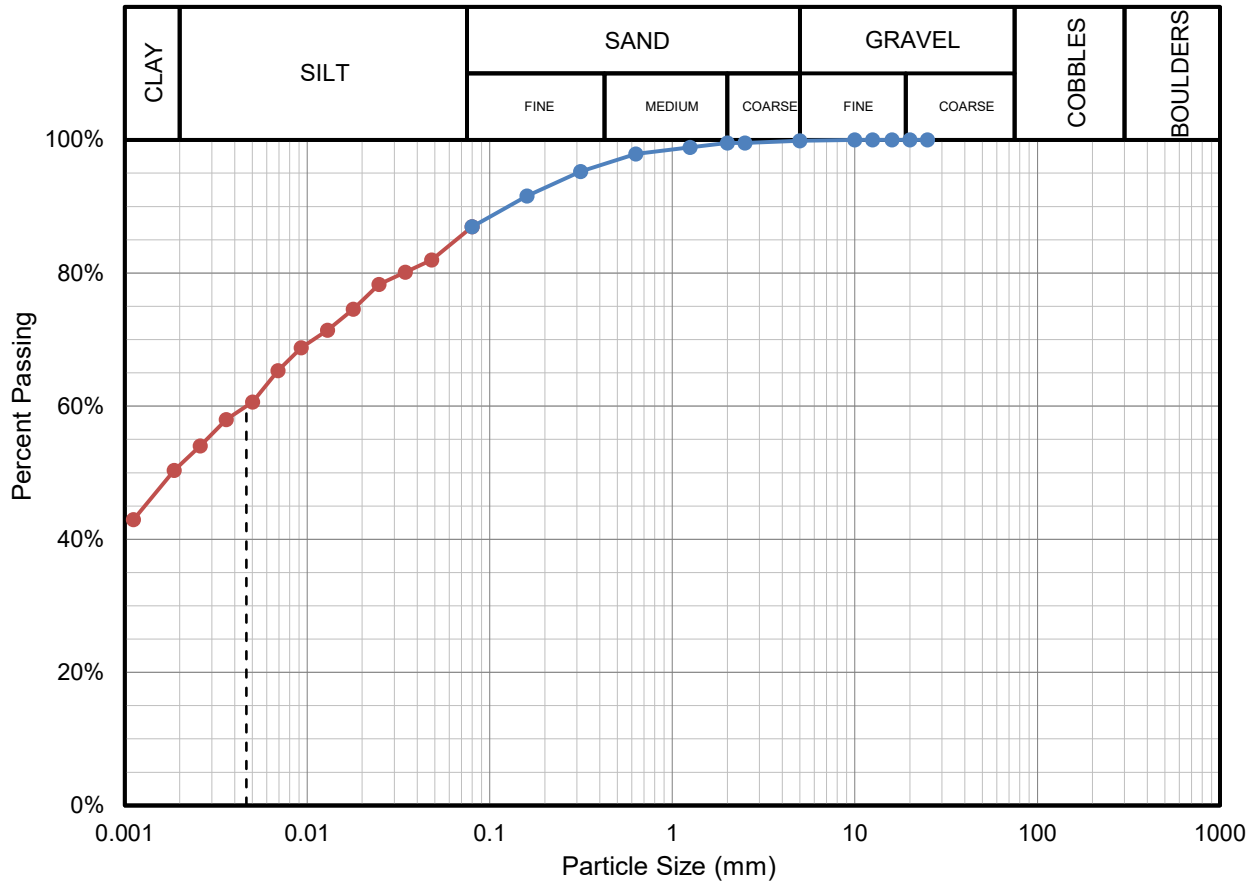
TECH: JL
CHECKED: CS

PARTICLE-SIZE ANALYSIS, LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY

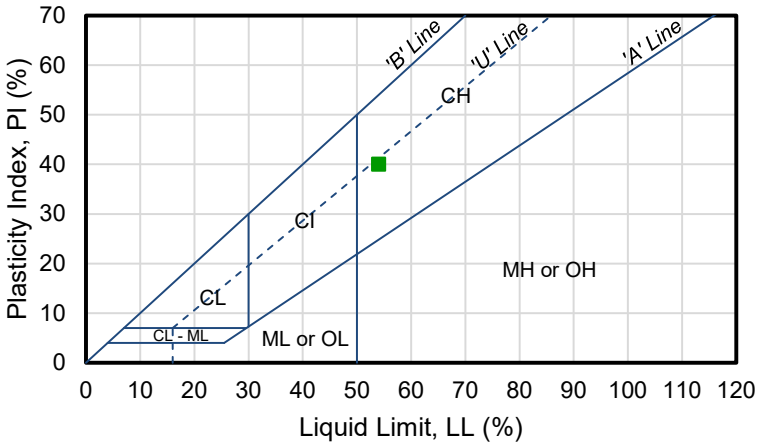
ASTM D422 & ASTM D4318

PROJECT: New Saddle Hills Firehall
PROJECT#: 25-0555GP
CLIENT: Saddle Hills County
SOIL DESCRIPTION: clay, and silt, little sand

SAMPLE DATE: May 28, 2025
TEST DATE: June 9, 2025
SAMPLE ID: 3G2
DEPTH: 2.3 m



PARTICLE-SIZE ANALYSIS	Gravel	0.1%
	Sand	13.6%
	Silt	35.2%
	Clay	51.1%
	D_{10}	---
	D_{30}	---
	D_{60}	0.0046 mm
	C_U	---
	C_C	---
LIMITS	PL	14
	LL	54
	PI	40



Modified Unified Soil Classification	Group Symbol
Fat clay	CH

TECH: JL
CHECKED: CS

AGAT Laboratories

6310 ROPER ROAD
EDMONTON, ALBERTA
CANADA T6B 3P9
TEL (780)395-2525
FAX (780)462-2490
http://www.agatlabs.com

Quality Assurance

CLIENT NAME: PARKLANDGEO LTD

AGAT WORK ORDER: 25G307212

PROJECT: 25-0555GP

ATTENTION TO: Connan Squair

SAMPLING SITE:

SAMPLED BY:

Soil Analysis

RPT Date: Jun 12, 2025			DUPLICATE				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Soil Analysis - Sulfate %															
Electrical Conductivity (Sat. Paste)	163	6800440	5.25	5.78	9.6%	< 0.05	106%	80%	120%	NA			NA		
Saturation Percentage	163	6800440	66	61	7.9%	< 1	89%	80%	120%	NA			NA		
Sulfate, Soluble	163	6800440	3200	2600	20.7%	< 2	88%	70%	130%	89%	80%	120%	88%	70%	130%

Comments: If the RPD value is NA, the results of the duplicates are under 5X the RDL and will not be calculated.

Certified By: gmills

LIMITATIONS

General Terms and Conditions

Professional services are subject to the following general terms, conditions, and limitations. If there is a conflict between the general terms, conditions, and limitations stated herein and those in the fully executed agreement signed between the CLIENT and ParklandGEO Ltd. (the "CONSULTANT") those of the fully executed agreement have precedence.

1. STANDARD OF CARE

1.1. In the performance of professional services, the CONSULTANT will use that degree of care and skill ordinarily exercised under similar circumstances by reputable members of its profession practicing in the same or similar localities. No other warranty, representation, or guarantee, either expressed or implied, is made or intended by this Agreement or by furnishing oral or written reports of the findings made, including but not limited as to CONSULTANT's professional services.

2. SITE INFORMATION

2.1. The CLIENT agrees to fully cooperate with the CONSULTANT and provide all information with respect to the past, present and proposed conditions and use of the project site (the "**Site**") whether specifically requested or not. The CLIENT acknowledges that in order for the CONSULTANT to properly provide the Services for which it is engaged and advise and assist the CLIENT, the CONSULTANT is relying upon full disclosure by the CLIENT of all matters and information pertinent to the Site, and upon the accuracy of such matters and information. The CONSULTANT shall not be responsible to the CLIENT or others for any consequences of any error or omissions contained in such CLIENT-supplied information, or as a result of the CLIENT's non-disclosure of information.

2.2. The CLIENT recognizes that subsurface conditions will vary from those encountered at the location where borings, surveys, or explorations are made and that the data, interpretations, and recommendation of the CONSULTANT are based solely on the information available to it. Classification and identification of soils, rocks, geological units, contaminated materials, and contaminant quantities will be based on commonly accepted practices in geotechnical and environmental consulting practice in this area.

2.3. Where specifically stated in the scope of work/Services, the CONSULTANT will perform a review of the historical information obtained or provided by the Client to assist in the investigation of the Site unless and except to the extent that such a review is limited or excluded from the scope of work.

3. WORK PRODUCT AND COMPLETE REPORT

3.1. Test results and reports, including the final report (the "**Report**"), provided by the CONSULTANT are of a summary nature and are not intended to stand alone without reference to the instructions given to the CONSULTANT by the CLIENT, communications between the CONSULTANT and the CLIENT, and to any other reports, writings or documents prepared by the CONSULTANT for the CLIENT relative to the specific Site, all of which shall constitute the "**Work Product**". In order to properly understand the suggestions, recommendations and opinions expressed by the CONSULTANT, reference must be made to the whole of the Work Product and Report. The CONSULTANT shall not be responsible for use of any part or portions of the of the Work Product or Report without reference to the whole of the Work Product and Report. The CLIENT agrees that any and all reports prepared by the CONSULTANT shall contain the following statement, and if such statement does not appear in the Work Product or Report the CLIENT agrees it is deemed to be included therein by way of this provision:

> *"This report has been prepared for the exclusive use of the Client and their approved agents for the scope of work and property outlined in this report. Use of the report is subject to acceptance of the General Terms, Conditions, and Limitations provided in this report. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. ParklandGEO Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report."*

3.2. The CLIENT agrees that in the event that any such report is released to a third party, such disclaimer shall not be removed, deleted, excluded or altered in any manner. The CLIENT further agrees that all Reports, including but not limited to any interim, preliminary or other reports, writings or documents prepared by the CONSULTANT for the CLIENT relative to the services and/or Site, shall be used solely for the purposes of the CLIENT and shall not be released, disclosed or used by others without the prior written permission of the CONSULTANT. No third parties are entitled to rely upon the Report or any portion thereof, and the Report shall not be considered accurate, complete or appropriate or suitable for such third parties.

4. INTERPRETATION OF THE REPORT AND/OR RESULTS

4.1. The CLIENT recognizes that subsurface conditions will vary from those encountered at the location where borings, surveys, or explorations are made and that the data, interpretations, and recommendations of the CONSULTANT are based solely on the information available to it. Classification and identification of soils, rocks, geological units, contaminated materials, and contaminant quantities will be based on commonly accepted practices in geotechnical and environmental consulting practice in this area. The CONSULTANT will not be responsible for the interpretation by others of the information developed by the CONSULTANT.

5. LIMITATIONS ON SCOPE OF INVESTIGATION AND WARRANTY

5.1. There is no warranty, expressed or implied, by the CONSULTANT that:

1. the investigation and services provided by the CONSULTANT shall uncover all potential geo-hazards, contaminants or environmental liabilities on the Site; or
2. the Site will be entirely free of all geo-hazards or contaminants as a result of any investigation and Services provided by the CONSULTANT, or cleanup work undertaken on the Site. The CLIENT acknowledges and agrees that it is not possible, even with exhaustive sampling, testing and analysis, to document all potential geo-hazards or contaminants on the Site.

5.2. The CLIENT acknowledges that:

1. the investigation findings, including but not limited to those in the Work Product and/or Report, are based solely on the information generated as a result of the specific scope of the investigation authorized by the CLIENT;
2. unless specifically stated in the agreed scope of work, the investigation will not, nor is it intended to assess or detect potential contaminants or environmental liabilities on the Site;
3. any assessment regarding geological conditions on the Site is based on the interpretation of conditions determined at specific sampling locations and depths, and that conditions may vary between sampling locations, hence there can be no assurance that undetected geological conditions, including soils or groundwater are not located on the Site;
4. any assessment is also dependent on and limited by the accuracy of the analytical data generated by the sample analyses;
5. any assessment is also limited by the scientific possibility of determining the presence of unsuitable geological conditions for which scientific analyses have been conducted;

6. the laboratory testing program and analytical parameters selected are limited to those outlined in the CLIENT's authorized scope of investigation; and
7. there are risks associated with the discovery of hazardous materials in and upon the lands and premises which may inadvertently be discovered as part of this investigation. The CLIENT acknowledges that it may have a responsibility in law to inform the owner of any affected property of the existence or suspected existence of hazardous materials. The CLIENT further acknowledges that any such discovery may result in the fair market value of the lands and premises and of any other lands and premises adjacent thereto to be adversely affected in a material respect.

6. COST ESTIMATES

6.1. Where provided, estimates of remediation or construction costs can only be based on the specific information generated and the technical limitations of the investigation authorized by the CLIENT at the date of the report. Accordingly, estimated costs for construction are based on the known Site conditions, which can vary as new information is discovered during construction or otherwise. The CLIENT acknowledges construction activities are an iterative exercise and cost estimates in relation to same may not be or remain accurate. The CONSULTANT shall not be liable for the accuracy of any estimates of remediation or construction costs provided.

7. LIMITATIONS OF LIABILITY AND INDEMNIFICATION

7.1. **LIMITATION OF LIABILITY** – The CLIENT hereby agrees that to the fullest extent permitted by the law the CONSULTANT's total liability to the CLIENT, and the CLIENT's maximum recourse against the CONSULTANT, for any and all manner of actions, causes of action, claims, injuries, demands, damages (including without limitation punitive, exemplary, indirect or consequential damages), losses, costs or expenses, of any and every kind and nature whatsoever, at law or in equity, in tort, contract, or other legal theory, whatsoever, in any way arising out of or in any way related or connected to this Agreement, the CONSULTANT's services, any communications, representations, warranties or otherwise in respect of same, the project, and/or the Site, including but not limited to the CONSULTANT's negligence, errors, omissions, strict liability, breach of contract, or breach of warranty, shall be absolutely limited and shall not exceed $50,000 (Canadian Dollars) or the total amount of the professional liability insurance amount, whichever is greater.

7.2. **NO SPECIAL OR CONSEQUENTIAL DAMAGES** – The CLIENT and CONSULTANT agree that to the fullest extent permitted by law the CONSULTANT shall not be liable to the CLIENT for any special, indirect, incidental, exemplary, punitive or consequential damages whatsoever, whether or not the CONSULTANT had or should have had knowledge of such damages, and on any theory of liability, whether in contract, strict liability, tort, negligence, or otherwise (including, but not limited to loss of use, loss of opportunity, loss of profits, or business interruption) arising out of or in any way related to this Agreement and/or the Services or any other services provided in connection with this Agreement, whether caused by the CONSULTANT's negligence, errors, omissions, strict liability, breach of contract, breach of warranty or otherwise.

7.3. **INDEMNIFICATION** – The CLIENT agrees to defend, indemnify and hold harmless the CONSULTANT, its directors, officers, employees, agents, subconsultants and subcontractors, from and against any and all manner of actions, injuries, claims, losses, expenses, damages (including without limitation punitive, exemplary, indirect or consequential damages), and costs, including legal fees on a solicitor-client full indemnity basis, and other liabilities, of any and every kind and nature whatsoever, at law or in equity, in tort, contract, or other legal theory, arising out of, or in any way related or connected to any one or more of the following: this Agreement; the Services; the CONSULTANT's reports or recommendations concerning this Agreement; the CONSULTANT's services; the CONSULTANT's presence on project property; the CONSULTANT's communications, representations, warranties or otherwise in respect of same; the presence, release, or threatened release of hazardous substances or pollutants on or from the Site. Notwithstanding the foregoing, the CLIENT shall not indemnify the CONSULTANT against liability for damages to the extent caused solely by the gross negligence or intentional misconduct of the CONSULTANT.

7.4. The CLIENT agrees that Section 13 of this Agreement, and all limitations, exclusions and indemnifications in respect of the CONSULTANT's liability, Services, or Work Product under this Agreement, including but not limited to waivers of damages and indemnifications by the CLIENT to the CONSULTANT, shall extend to and be for the benefit of those individuals and entities the CONSULTANT retains in connection with performance of the services under this Agreement, including but not limited to CONSULTANT's officers, directors, partners and employees, as well as CONSULTANT's subconsultants and subcontractors. The CLIENT further agrees that it shall make no claims or take any proceedings whatsoever directly against the CONSULTANT's officers, directors, partners and employees, whether the claims arise in contract, tort or otherwise, and whether covered by insurance. The CLIENT further agrees that Section 13 in its entirety shall survive termination of this Agreement for any reason.

8. DOCUMENTS, PATENTS, TRADEMARKS, AND INTELLECTUAL PROPERTY

8.1. The Report, documents and instruments in respect of the CONSULTANT's services, including but not limited to drawings, plans, digital and physical models, designs, specifications, studies, reports (including any drafts of reports), photographs, computer software, concepts, products, or processes if proprietary to the CONSULTANT, surveys, calculations and other data required by the scope of its services, and which were prepared by or on behalf of the CONSULTANT, including those which are deliverables in respect of its services hereunder (collectively, the "**Instruments**"), are the property of the CONSULTANT and the CONSULTANT shall retain the originals thereof, whether the work is completed or not. The CONSULTANT reserves the copyrights in connection with same.

8.2. The CLIENT may not use the Report and such Instruments without having paid all amounts owing to the CONSULTANT. Provided all amounts owing to the CONSULTANT are paid in full: (i) the CLIENT is entitled to keep a copy of the Report and those Instruments as agreed to by the CONSULTANT in writing, and (ii) the CLIENT shall have a non-exclusive license to use the Report and those Instruments as agreed to by the CONSULTANT in writing which relate to or result from the services provided by the CONSULTANT under this Agreement, but solely for purposes for which they were provided by the CONSULTANT under this Agreement. The CONSULTANT is entitled to injunctive relief in respect of the Report and Instruments if it is not paid. The CONSULTANT retains ownership of all patents, trademarks, copyrights, industrial or other intellectual property rights resulting from its services, from the Report and Instruments which are developed or first reduced to practice by the CONSULTANT in the provision of its services. The CLIENT shall not use, infringe or appropriate such proprietary rights without the prior written consent and compensation of the CONSULTANT.

8.3. Use of the Report and Instruments by the CLIENT for purposes other than the purpose for which provided by the CONSULTANT, including but not limited to in connection with the Site project, without notifying the CONSULTANT and without its prior written consent is done so at the sole risk of the CLIENT. In addition to and without limiting the indemnification obligations provided in this Agreement, the CLIENT shall indemnify the CONSULTANT against any and all manner of actions, injuries, claims, losses, expenses, damages (including without limitation punitive, exemplary, indirect or consequential damages), and costs (including legal fees on a solicitor-client full indemnity basis), arising out of, or in any way related or connected to (i) the CLIENT's improper use of the Report and Instruments, and each of them, and (ii) alterations to the Report and Instruments, and each of them, by, on behalf of, and/or at the direction of the CLIENT, without notifying the CONSULTANT and without the CONSULTANT's prior written consent.

8.4. The CONSULTANT shall not be responsible for the consequences of any such improper use and alterations as described herein. In addition to any rights or remedies under this Agreement or at law, the CONSULTANT shall be entitled to compensation for such improper use and alterations and to prevent such improper use and alterations.