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April 20, 2012
Our File: H0805-168

Attention: **Mr. Fredd Eitzen, Project Manager**

Dear Sir:

Re: Geotechnical Site Investigation
Proposed Commercial Development
Lot 3, Block 2, Plan 9826014
525 St. Albert Road
St. Albert, Alberta

1.0 INTRODUCTION

As requested, a geotechnical site investigation was carried out by Hagstrom Geotechnical Services Ltd (HGSL) at the above referenced project. The scope of work for the investigation was to provide an assessment of the soil and groundwater conditions, provide recommendations for foundation systems and preparation of this report. Field drilling was carried out in mid 2008 for Nova Builders and a data report was submitted in June, 2008. The property was transferred to the current owner in February, 2012 and this report presents an evaluation of all results and recommendations for new foundations. Recommendations for asphalt pavement structures are also provided.

2.0 PROJECT DESCRIPTION AND PROPOSED DEVELOPMENT

The proposed commercial development site is located within the north limits of St. Albert, Alberta between Inglewood Drive and St. Albert Road. The legal description is Lot 3, Block 2, Plan 9826014 and covers about 1.91 hectares of vacant land. The site is roughly rectangular in shape and slopes from north to south with an elevation difference of about 7.5 meters. There is a narrow earth bench of about 4 to 5 metres in width located in the south and south east limits of the site.

It is understood that the development will consist of construction of three new buildings located at approximate locations shown on Plate 13, Appendix A. It is understood that Building 1 will cover about 25470 square feet and will comprise a single storey restaurant, a two storey CRU and a single storey single storey multi- tenant building. Building 2 will cover about 12750 square feet for a single storey Visions building. Building 3 will cover about 24800 square feet for a two storey Goodlife building. It is understood that the buildings will be continuously heated and will not have basements. Surface parking will be provided around the perimeter of the buildings with capacity for about 245 vehicle stalls. The final elevations of the proposed buildings are assumed to be near existing ground level.

3.0 INVESTIGATION PROCEDURE

Ten deep boreholes were drilled on the site on June 13, 2008. The boreholes were drilled to depths ranging from 9.0 to 12.0 metres deep using a wheel mounted drill rig equipped with a continuous flight, 150-millimeter diameter, solid-stem augers. Supervision of drilling, soil sampling, and logging of the various soil strata was performed by Mr. Merle Hagstrom, P. Eng of HGSL. The soils were described in accordance with the Modified Unified Soil Classification System described on Plates 1 and 2, Appendix A. The soil and groundwater conditions encountered during field drilling were recorded and are presented on the borehole logs shown on Plates 3 to 12, Appendix A. The borehole locations are presented on Plate 13, Appendix A.

Borehole elevations were determined at drilling completion and were referenced to local datum. An elevation of 200.00 meters was assigned to a steel bolt located along St. Albert Road (refer to Plate 13, Appendix A).

Soil sampling for laboratory analysis generally consisted of disturbed auger soil samples at 0.75 meter intervals obtained from all ten boreholes. Standard penetration tests were conducted on five boreholes at selected depths. In addition, pocket penetrometer (PP) readings were taken on intact cohesive soil samples at approximately 0.75 m intervals from all boreholes to obtain an indication of the unconfined compressive strength (Q_u) of the soil.

Groundwater conditions were monitored during drilling, at drilling completion, several hours later and 26 days after drilling completion. All of the water level results are presented on the boreholes logs in Appendix A.

In addition to the routine moisture content analysis, the laboratory analyses consisted of sixteen soluble sulphates analyses and five Atterberg limit tests. The laboratory analyses results are presented on the borehole logs in Appendix A.

4.0 SUBSOIL AND GROUNDWATER CONDITIONS

The soil profile at the borehole locations are variable and generally consist of a thick layer of clay fill followed by variable thick layers of silt and clay that extended below the termination depth of the boreholes. Clay till was encountered below the silt and clay in the north five boreholes and weathered clay shale bedrock was encountered below the clay till in two boreholes. The clay till depth ranged from 6.6 to 9.4 metres. Detailed borehole logs are presented in Appendix A and a description of various soil units and their properties are presented in the paragraphs below.

Clay fill was encountered at the ground surface in all ten boreholes and extended to depths ranging from 0.3 to 2.9 metres. The least thickness of clay fill was found at the north end of the site and the largest thickness was found at the south end of the site. The clay fill was generally described as silty, damp, loose, poorly compacted and dark brown. Topsoil lenses/layers, wood fragments, and concrete chunks were found within the clay fill. “N” values in the clay fill ranged from 4 and 5 blows per 300 mm of penetration. These values are considered to be low of which the clay fill appears to be random fill that was placed under uncontrolled conditions. The thickness of fill may vary between borehole locations.

Clay and silt of variable thickness and at variable depths was encountered in all boreholes. The clay was described as silty, damp to moist, stiff to very stiff, low to medium plasticity, and light olive brown to dark olive brown in colour. The clay was also noted to contain occasional coal chips, rust stained fissures and occasional silt lenses. Moisture contents in the clay ranged from 11 to 34 percent and pocket penetrometer readings ranged from 120 to 410 kPa. “N’ values in the clay ranged from 4 to 18 blows per 300 mm of penetration. The silt was described as clayey, with some sand, damp to moist, stiff consistency and dark olive brown to light brown in colour. Moisture contents in the silt ranged from 11 to 20 percent and ‘N” values on the silt ranged from 9 to 14 blows per 300 mm of penetration. The silt is considered to be highly frost susceptible. The clay and silt layers exhibit moderate compressibility under light to moderate loads.

The clay exhibits a moderate potential for swelling and thus will lift light loads such as floor slabs when exposed to excess water.

Clay till was encountered in Boreholes 08-1, 08-2, 08-3, 08-4, and 08-6 between depths of 6.6 and 9.4 metres. In addition, weathered clay shale bedrock was encountered below the clay till in Boreholes 08-1 and 08-3. The clay till was described as silty, moist, very stiff to hard, medium plasticity, with occasional coal chips and clay shale inclusions and dark brown to dark olive brown in colour. Moisture contents in the clay till ranged from 13 to 14 percent and pocket penetrometer readings ranged from 290 to 410 kPa. The clay shale bedrock was described as silty, highly weathered bedrock, hard consistency soil and dark grey in colour. The clay till and weathered bedrock are competent soils and exhibit low compressibility under moderate to heavy loads.

Groundwater conditions were monitored during drilling, at drilling completion, several hours later and 26 days after drilling completion. Groundwater seepage was encountered in one borehole during drilling and there was no evidence of groundwater seepage during drilling in the remaining nine boreholes. All water table results in the boreholes are summarized in Table 1, below and the individual results are presented on the borehole logs in Appendix A. For design purposes, the water table in the area of proposed Buildings 1, 2 and 3 should be assumed at depths of 4.0 m, 9.0 m and 8.0 m, respectively.

TABLE 1
SUMMARY OF GROUNDWATER OBSERVATIONS

Borehole Number	Depth to Groundwater Seepage (m)	Depth of Water (m)		
		At Drilling Completion	1 to 5 Hours Later	26 Days Later
08-1	Nil	9.4 (dry)	8.0	4.0 (dry)
08-2	Nil	8.9 (dry)	5.2	4.0
08-3	Nil	9.4 (dry)	5.5	6.4
08-4	Nil	10.3 (dry)	8.2	6.7 (dry)
08-5	Nil	11.9 (dry)	11.4	11.6 (dry)
08-6	Nil	11.8 (dry)	11.4	11.3 (dry)
08-7	Nil	9.4 (dry)	8.1 (dry)	11.9 (dry)
08-8	10.4	11.8 (dry)	10.2	10.1
08-9	Nil	9.4 (dry)	8.9 (dry)	8.8 (dry)
08-10	Nil	11.8 (dry)	11.1 (dry)	9.8 (dry)

4.1 FROST PENETRATION

The expected maximum depth of frost penetration for various soil types is given in Table 2, below. The penetration is based on a freezing index for a 25-year return period of 2200 degrees-days Celsius. The depth of frost penetration assumes a uniform soil type without topsoil or snow cover.

Buried water lines across the site should have a minimum cover of soil that suits the soil conditions presented below. Water pipes buried with less than the recommended soil cover should be protected with rigid insulation to avoid frost effects that may cause damage to or breakage of the pipes. Rigid insulation placed under areas subject to vehicular wheel loadings should be provided with a cover of compacted granular base with a minimum soil cover of 600 mm.

TABLE 2
ESTIMATED DEPTH OF FROST PENETRATION

Soil Type		Depth of Frost Penetration (m)
In-situ	Clay and Clay Till	2.5
	Silt and Sand	2.9
	Weathered Bedrock	3.0
	Gravel	3.5
Compacted Backfill (95 % SPMDD*)	Clay and Clay Till	2.3
	Silt and Sand	2.7
	Weathered Bedrock	2.9
	Gravel	3.3

*SPMDD- Standard Proctor Maximum Dry Density

5.0 GEOTECHNICAL EVALUATION

All foundation design recommendations presented in this report are based on the assumption that an adequate level of construction monitoring during foundation excavation will be provided, and that all construction will be carried out by a suitably qualified, experienced contactor. An adequate level of construction monitoring is considered to be: (a), review of all design drawings, and full-time monitoring during construction of foundations, and (b), full-time monitoring and compaction testing for earthworks by suitably qualified geotechnical personnel.

One of the purposes of providing an adequate level of monitoring is to check the recommendations based on data obtained at discrete borehole locations are relevant to other areas of the site. It is recognized that ten deep boreholes were drilled across the entire large site.

The foundation recommendations presented in this report were prepared in accordance with Limit Sate design (LSD) methodology.

The Alberta Building Code 2006 (ABC 2006) and National Building Code of Canada 2005 (NBCC 2005) stipulates geotechnical aspects of foundation design should be conducted using LSD methodology rather than the past approach of using allowable design methods. Under LSD methodology, foundations should be designed on the basis of factored Ultimate Limit States (ULS). To determine the applicable working capacity, Serviceability Limit States (SLS) should also be considered. The lower of the factored ULS resistance and the unfactored SLS resistance should be used as the working capacity for foundation design purposes.

Under LSD, the geotechnical resistance factors required to calculate the factored foundation resistance from semi-empirical analysis are 0.5 and 0.4 for axial loads on piles and shallow foundations, respectively.

It is understood that the site will be developed into three buildings that vary in size from 12750 to 25472 square feet. The final elevation of each building is assumed to be near existing grade. The soil conditions at the site are considered to be variable although favourable below the existing poorly compacted random fill. Competent clay till was encountered at deep depths in the area of proposed Building 1. Shallow foundations such as strip and spread footings can be considered for all buildings located on the site. Deep foundations such as cast-in-place straight shaft concrete piles can be considered for all buildings and end bearing belled piles can be considered for Building 1. For a hot mix pavement structure, the subgrade conditions below the existing fill are considered to be favourable.

5.1 SHALLOW FOUNDATIONS

The term "shallow foundations' refer to concrete strip and spread footings at a shallow depth below finished exterior grade. In this case, ultimate and factored base resistance parameters for strip and spread footings supporting axial compressive loads are presented on Table 3, below.

TABLE 3
SHALLOW FOUNDATION DESIGN PARAMETERS - ULS

Footing Type	Ultimate Base Resistance (kPa)	Factored Base Resistance (kPa)
Strip Footing	130	65
Spread Footing	170	85

*Denotes geotechnical resistance factor of 0.5 applied to obtain factored base resistance values

The footings must be founded within the native, inorganic soils which include clay or silt. They should be constructed at a minimum depth of 1.5 m below exterior grade for a heated structure. The interior footings within a heated structure must have at least 0.8 metres of soil cover. All footings for unheated structures should be founded in native, inorganic native clay or silt below the frost penetration of 2.9 meters. Footings must not be placed on any uncontrolled fill, topsoil, organic soils or loose, disturbed or frozen soils. Footing excavations must be protected from frost, desiccation, or the ingress of water. Bearing soils, which become frozen, dried or softened, should be removed and replaced with concrete or the footings should be extended to reach soil in an unaffected condition. It is essential that the foundation soils not be allowed to freeze at any time before or after concrete for the footings have been placed.

The footings for any structure within the depth of frost penetration will be subjected to uplift loads resulting in potential movement of the structure. If potential movement of the structure cannot be tolerated, a frost protected footing must be adopted. High strength extruded polystyrene insulation, with a minimum thickness of 100 mm should be provided. The insulation should be continuous on top of the entire foundation (including the vertical face of the foundation), extending a minimum of 2.5 meters horizontally out from the foundation. The insulation should have a minimum soil cover of 300 mm.

An estimate of foundations settlement for new footings should be carried out by HGSL after the footing sizes and footing elevations have been determined.

5.2 CAST-IN-PLACE CONCRETE FRICTION PILES

Cast-in-place concrete friction piles are considered feasible for all three proposed buildings.

Based on the site investigation results, there is a potential for groundwater seepage and necking/sloughing soil conditions at depths below 5 to 6 meters below existing grade. If sloughing or seepage does occur during pile construction, temporary steel casing should be made available on-site and used during pile construction to seal off zones where soil sloughs into the pile drill holes. If concrete delivery for construction of concrete straight shaft piles is timed so concrete can be poured within a few minutes from completion of drilling, it will help to reduce the chance of soil sloughing in the case of concrete straight shaft piles. The piling contractor should make his own estimate of steel casing requirements, considering such factors as construction procedures and pile diameters.

Ultimate and factored shaft resistance parameters for cast-in-place concrete friction piles supporting axial loads for proposed Building 1 are presented in Table 4, below.

TABLE 4
CAST-IN-PLACE CONCRETE FRICTION PILE DESIGN PARAMETERS - ULS

Depth Interval (m)	Ultimate Shaft Resistance (kPa)	Factored Shaft Resistance (kPa)
0 to 2	0	0
2 to 9	50	20
Below 9	80	32

* Denotes geotechnical resistance factor of 0.4 applied to obtain factored shaft resistance values

The shaft friction should be neglected within the top 2 meters of pile length, due to soil desiccation and low adhesion effects. Any portion of the pile shaft in contact with fill soils should also be neglected.

The site does not have competent clay till and/or clay shale bedrock to depths of about 12 metres in the area of proposed Buildings 2 and 3. Ultimate and factored shaft resistance parameters for cast-in-place concrete friction piles supporting axial loads for Buildings 2 and 3 are presented in Table 5, below.

TABLE 5
CAST-IN-PLACE CONCRETE FRICTION PILE DESIGN PARAMETERS - ULS

Depth Interval (m)	Ultimate Shaft Resistance (kPa)	Factored Shaft Resistance (kPa)
0 to 2	0	0
Below 2	50	20

* Denotes geotechnical resistance factor of 0.4 applied to obtain factored shaft resistance values

The shaft friction should be neglected within the top 2 meters of pile length, due to soil desiccation and low adhesion effects. Any portion of the pile shaft in contact with fill soils should also be neglected.

Friction concrete piles should have a minimum shaft diameter of 400 mm. A minimum depth of 6.0 metres should be provided for piles for a heated building and a minimum depth of 7.5 metres for an unheated building. All friction piles subjected to uplift loads including frost action should be reinforced over the entire pile length. No allowance should be made for end bearing of the straight shaft pile base unless specific cleaning tools and care are implemented by the piling contractor.

5.3 CAST-IN-PLACE CONCRETE END BEARING PILES

Cast-in-place concrete piles using end bearing resistance are considered feasible for proposed Building 1. Based on the site investigation results, there is a potential for groundwater seepage and necking/sloughing soil conditions at depths below 5 to 6 meters below existing grade. If sloughing or seepage does occur during pile construction, temporary steel casing should be made available on-site and used during pile construction to seal off zones where soil sloughs into the pile drill holes. If concrete delivery for construction of concrete straight shaft piles is timed so concrete can be poured within a few minutes from completion of drilling, it will help to reduce the chance of soil sloughing in the case of concrete straight shaft/belled piles. The piling contractor should make his own estimate of steel casing requirements, considering such factors as construction procedures and bore/bell pile diameters.

Belled end-bearing piles based on a minimum depth of 10.0 metres below existing grade in competent clay till may be considered using the ultimate and factored base resistance parameters presented in Table 5, below. All concrete bells must be constructed at least 1.2 metres into the clay till/bedrock.

TABLE 6
CAST-IN-PLACE CONCRETE END BEARING PILE DESIGN PARAMETREES - ULS

Approximate Belling Depth (m)	Soil Type	Ultimate Base Resistance (kPa)	Factored Base Resistance (kPa)
10.0	Clay till/bedrock	900	360

* Denotes geotechnical resistance factor of 0.4 applied to obtain factored base resistance values

Bell diameters for end bearing piles should have a minimum of 2 and a maximum of 3 times the shaft diameter. The ratio of pile depth to bell diameter should be a minimum of 2.5. The belled piles may include the effects of shaft friction provided in Table 4, above, but should not include the shaft friction for two shaft diameters above the top of the bell.

To allow adequate support for the roof of the bell, the minimum distance from the underside of a soil sloughing layer to the top of the roof of a bell should be 1.0 meter. This may require altering the pile type or field alteration of bell elevation (by increasing the length of the piles) to verify the bells are formed in acceptable soil bearing strata.

The base of all end-bearing piles must be thoroughly cleaned all loosened soil by mechanical methods. Following drilling and cleaning, all pile drill holes must be inspected to confirm an adequate bearing surface was prepared at an appropriate depth.

An estimate of concrete pile settlements for various size of piles should be carried out after the pile sizes and depths have been determined.

5.4 SUBGRADE PREPARATION FOR BUILDING AREAS

The site contains a significant amount of poorly compacted clay fill that is contaminated with topsoil, wood fragments and concrete chunks that ranges in thickness from 30 to 290 centimeters. Based on the current ten boreholes drilled across the site, the fill thickness appears to increase in thickness from north to south direction.

To achieve a reasonable level of performance from a building grade-supported floor slab, it is essential to have a relatively uniform subgrade. Cracking, differential movements and poor performance of floor slabs are typically related to variations of the slab support. The following site recommendations should be followed to achieve uniformity across the footprint of each proposed building.

All existing fill and organic soils should be removed from the site within the proposed building footprint. At completion of this, the exposed subgrade within the building footprint should be scarified to a minimum depth of 15 centimeters and moisture conditioned to within 2% of the optimum moisture content (OMC), and re-compacted to at least 98% of standard Proctor maximum dry density (SPMDD).

The prepared subgrade should be proof-rolled with a tandem axle truck to identify any soft and weak areas. A geotextile should then be placed on the subcut base of the excavation if wet and soft soil conditions are encountered. Fill used to backfill subcut areas should be moisture conditioned to within 2% of OMC and compacted to not less than 98% SPMDD in lifts no greater than 150 mm in compacted thickness. The fill should be sand, silt or low plastic clay. The soil excavated from any subcut may be suitable for use as backfill, provided they are suitably moisture conditioned and free of organic soils or other deleterious materials. The earth moving contractor should be aware that subgrade moisture conditioning may be required to obtain the required level of compaction.

The entire construction site should be graded during construction to allow for positive drainage of about 1.5 to 2% grade.

Differential settlements, rather than total settlements, are usually the governing factor in structural and architectural design. To reduce abrupt differential movement resulting from varying fill depths beneath a grade supported floor, the amount of new fill should be feathered from the deepest point by stepping the fill into the side of the excavation at a slope of 2H:1V or flatter.

Full time construction monitoring and compaction testing should be provided during fill placement to confirm subgrade conditions are properly prepared. Qualified geotechnical personnel should complete this monitoring and testing.

5.5 CONCRETE FLOOR SLABS

The site has a significant thickness of uncontrolled clay fill that must be removed within the proposed building footprint for each building. Moisture content in soils below a concrete floor slab will generally increase with time due to new building sealing the surface and preventing evaporation of moisture. The use of medium and high plastic soil fill as engineered fill within the new building is not recommended due to the potential for swelling of the clay associated with increasing moisture content. The moisture from the water lines or sewer lines placed under the floor slab that may leak, or additional moisture from improper site grading, could compound the problem and swelling or settlement of 20 to 40 mm could be possible in localized areas.

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

Loads on slab-on-grade concrete floors are assumed to be less than 15 kPa. For design purposes, the existing native clay subgrade and properly compacted engineered fill can be assumed to have a subgrade modulus of 15 MPa/m. The concrete floor slab should be reinforced with steel. The reinforcing steel can be carried through the construction joints. Proper saw cut construction joints are required to prevent shrinkage cracks.

Non-load bearing partitions resting on slab-on-grade floor should be designed such that floor movements can be accommodated. An allowance of 25 mm is recommended. Partitions should not be tied into the exterior wall or load bearing columns.

Mechanical equipment placed on the slab-on-grade floor should be designed to permit some re-levelling should the equipment be susceptible to small changes in level. Piping and electrical conduit connections should be laid out to permit some flexibility, as vertical movement of such equipment such as water meters, furnaces, and electric equipment may cause distress in the piping. This provision is particularly important where there are short pipe runs between mechanical equipment and points where piping passes through walls and concrete slabs.

Slab-on-grade floors should be tied into the grade beam with steel dowels at doorways. At all other points, it would be preferable to allow the concrete slab to float. Alternatively, the concrete slab can be tied into the grade beam at all points provided that a construction joint or saw cut is placed parallel to the concrete grade beam at a distance of 2.0 meters from the grade beam. A void form underlying the perimeter of the floor slab extending out a distance of 500 to 600 mm from the inside face of the grade beam, is recommended to reduce uplift stresses and to reduce the risk of damage to drywall along perimeter walls.

Any new backfill including engineered fill required to raise the subgrade elevation should be completed using crushed or pit run gravel or low plastic clay compacted to at least 98% of SPMDD, at a moisture content within 2% of OMC. Imported fill should be placed in lifts not exceeding 150 millimetres in compacted thickness.

A minimum of 150 millimetres of clean, well-graded crushed gravel is recommended directly beneath the floor slab. Where the floor slab is required to support heavy equipment, the granular base should be increased to 250 millimetres in thickness. Coarse material greater than 50 millimetres in diameter should be avoided directly beneath the floor slab to prevent stress concentrations in the slab. The granular coarse material should be compacted to a uniform dry density of about 100% of SPMDD. A recommended typical gradation for stable granular material for use under the floor slab is provided in Table 7, below.

TABLE 7
GRADATION REQUIREMENTS FOR GRANULAR BACKFILL

Sieve	% Passing
20,000 um	100
10,000 um	35-75
5,000 um	20-55
1,250 um	10-30
315 um	5-20
80 um	2-12

The percent fracture by weight (2 faces) should be at least 40 percent. Other appropriate materials, which fall outside the above recommended gradation limits may be suitable but should be evaluated by a geotechnical engineer prior to use.

5.6 LATERAL EARTH PRESSURES

It is understood that the site may have several concrete retaining walls. Assuming that the concrete walls will be designed as an unyielding wall, lateral earth pressures acting against the foundation wall of the concrete walls should be computed using the following equation:

$$P = K_o\, Q + K_o\, \gamma H$$

Where:
- P = Lateral earth pressure at depth H below ground level (kPa)
- Q = Any surcharge loading at the ground surface (kPa)
- K_o = Coefficient of lateral earth pressure at rest
- γ = Total unit weight of backfill compacted to 95% SPMDD (kN/m^3)
- H = Depth below ground level (m)

Recommended design values for these parameters depend on the type of backfill used as given in Table 8, below.

TABLE 8
LATERAL EARTH PRESSURE PARAMETERS

Type of Backfill	Total Unit Weight (kN/m^3)	Coeff. of Lateral Earth Pressure at Rest, K_o
Free draining granular fill	21	0.4
Clay, Silt or Sand Fill	20	0.6

The preceding relationship makes no allowance for additional horizontal forces due to frost or hydrostatic pressures to build up behind the wall on the assumption that frost protection and a weeping drain system will be utilized. If no frost protection is provided, the lateral earth pressures should be increased by a factor of 2. If no weeping tiles are provided, then hydrostatic

pressures should be added to the above expression.
The above expression also assumes nominal compaction (maximum 95% SPMDD) adjacent to the wall. If a high degree of compaction is proposed (above 95% SPMDD), the design lateral earth pressure will adopt a combined triangular-trapezoidal distribution. If this is the case, Hagstrom Geotechnical Services Ltd. can provide further details upon request.

5.7 SULPHATES IN CONCRETE

Chemical testing for water-soluble sulphates concentrations on sixteen selected samples of native soil revealed sulphate concentrations of 0.02 to 0.18 percent water-soluble sulphate by dry weight of soil. The results indicate a negligible to moderate potential for sulphate attack on concrete in contact with native soils at this site. Therefore, all concrete in contact with the native soils should be made from CSA Type HS sulphate resistant cement possessing a minimum 56 days compressive strength of 30 MPa. The maximum water cement ratio should be 0.50. An air entrainment agent of 4 to 7 percent is recommended for improved workability and durability. Increased compressive strength and/or air content may be required due to structural requirements or other exposure conditions.

If concrete is placed in contact with imported fill, that fill should be tested for water soluble sulphate ion content, and the above recommendations be re-evaluated.

5.8 EXCAVATIONS AND TRENCH BACKFILL

Excavations should be carried out in accordance with Occupational Health and Safety regulations. The density and consistencies of the native soils encountered at the site are such that conventional hydraulic excavators can be used to remove these soils.

For this project, the depth of excavations are anticipated to be relatively shallow to moderate and will be carried out for removal of the existing fill, and for such components as grade beams, pile caps, service trenches and underground services.

For temporary excavations in clay and silt, the side slopes shored and braced or the slopes cut 1.0 horizontal to 1.0 vertical (1.0H:1.0V) or flatter. Flatter sides slopes may be required at depths below 2.0 meters from existing grade. Where excavations are open for longer than one month or if significant groundwater seepage is encountered, shallower side slopes of 2H:1V may be required.

Temporary surcharge loads, such as construction materials and equipment, should not be allowed within 3.0 metres of an unsupported excavated face or the depth of excavation, whichever is greater. A further setback may be required for deeper excavations. Vehicles delivering construction materials should be kept form excavated faces by at least 1.5 metres.

Trenches outside each building footprint should generally be backfilled with soil similar to the adjacent native soils and the backfill should be compacted to moisture contents within 2% of OMC. A minimum compaction of 98% of SPMDD is recommended for all trenches of which includes the top 100 cm. The compacted thickness of each lift of backfill should not exceed 150 mm. The upper 1.5 meters of service trenches below paved areas should be cut to a maximum slope of 2H:1V, to avoid an abrupt transition between backfill and insitu soil. Within each building footprint, all trench backfill should be compacted to at least 98% SPMDD at a moisture content within 2% of OMC. The compacted thickness of each lift of backfill should not exceed 150 mm.

The ultimate performance of the trench backfill is directly related to the uniformity of the backfill compaction. In order to achieve this uniformity, the lift thickness and compaction criteria must be strictly enforced during construction.

5.9 SIESMIC RESPONSE

Site classification for soil seismic response for this site is Class "D" which is according to the requirements of the National Building Code of Canada (Table 4.1.8.4.A).

6.0 FLEXIBLE PAVEMENT DESIGN

6.1 SUBGRADE PREPARATION

The existing clay fill is poorly compacted and is contaminated with organic soils and debris and thus should be removed and should not be utilized as subgrade support for the new pavement structures. New fill required to raise the site should consist of sand or gravel and be placed in thin lifts of 15 to 20 cm thick and compacted to 98% of SPMDD. The final subgrade surface should be compacted to at least 98% of SPMDD at a moisture content at or slightly in excess of the optimum moisture content to a depth of at least 200 millimetres in automobile parking areas and at least 300 millimetres in truck traffic areas (subgrade preparation). Prior to placement of the pavement structure, the entire surface should be proof rolled with a fully loaded tandem axle or single axle dump truck to detect any soft area. Soil drying or stabilization of the soil subgrade with Portland cement may be required in areas where the clay/silt subgrade is too wet to work and achieve compaction.

6.2 Flexible Pavement Sections

The pavement sections given in Tables 9 and 10 are the minimum requirements to accommodate the assumed traffic loading conditions and frequencies for the project site. These sections are designed based on an assumed California Bearing Ratio (CBR) value of 3.0, a design period of 20 years, and a maximum axle load of 80 kN (18 kips).

For areas subjected to automobile and light truck traffic only, the pavement structure is assumed to be used only by cars and light trucks (i.e. vans and ½-ton pickups), one of the alternate pavement sections given in Table 9 may be used.

TABLE 9
FLEXIBLE PAVEMENT SECTION THICKNESS
(Light Traffic Areas)

Layer	**Minimum Thickness (mm)**		
	Option 1	**Option 2**	**Option 3**
Asphaltic concrete (50 blows)	185 mm full depth	50 mm at surface	75 mm at surface
Granular base	--	200 mm, plant mix cement stabilized	250 mm of 20 mm granular base

In areas subjected to heavy traffic, (entrance, accessways, and heavy truck parking), one of the alternate pavement sections given in Table 10 may be used.

TABLE 10
FLEXIBLE PAVEMENT SECTION THICKNESS
(Heavy Traffic Areas)

Layer	**Minimum Thickness (mm)**		
	Option 1	**Option 2**	**Option 3**
Asphaltic concrete (50 blows)	240 mm full depth	125 mm at surface	100 mm at surface
Granular base	--	180 mm, plant mix cement stabilized	320 mm of 20 mm granular base

The granular base material should be hard, clean, well graded, crushed aggregate, free of organics, coal clay lumps, and other deleterious material. The minimum sand equivalent should be 35 and the maximum percent passing the 80-micrometer screen should be 7 percent. Each mat of hot-mix asphalt should be compacted to at least 98 percent of maximum Marshall density.

It is recommended that concrete pads for garbage bins be constructed for use by garbage trucks. During the garbage bin pickup and removal, the loading intensity of front axle of the garbage truck may exceed the capacity of the recommended asphalt pavement structure. Accordingly, it is recommended that for all such bins, a 200-millimeter thick concrete pad with 10 mm steel bars at 30 cm centre-to-centre be constructed on 200 millimeters of compacted granular fill.

6.3 SITE GRADING

The finished grade around each building should be such that the surface water drains away from the building. The upper 0.5-meter of backfill around each building should consist of compacted clay to act as a seal against the ingress of runoff water. The clay should extend for a distance of 3 meters around the building and should be graded at a slope of 2% away from the building. General landscaped areas should have grades no less than 2% to minimize water ponding.

7.0 CLOSURE

This report is based on the findings at ten deep borehole locations. Should different subsoil or groundwater conditions be encountered during construction, Hagstrom Geotechnical Services Ltd. must be notified immediately and the recommendations provided herein will be reviewed and revised as required.

Relative to the existing clay fill located on the site within each building and adjacent parking areas, if the cost of fill removal is prohibitive, consideration may be given to partial fill removal. In this case, it is recommended that about ten to twelve test pits be excavated across the site and supervised by HGSL to determine the variation in depths and fill properties. This work should be carried out with a rubber tire or tracked backhoe.

Respectfully Submitted,
Hagstrom Geotechnical Services Ltd.

Merle Hagstrom

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APPENDIX A

Explanation of Terms and Symbols Used on Borehole Logs
Borehole Logs
Site Plan Showing Borehole Locations