Geotechnical Report
Saskatchewan Multi-Use Facility Feasibility Study

Regina, Saskatchewan

Crown Investments Corporation of Saskatchewan

File R4397 09 December 2009

Clifton Associates Ltd.
engineering science technology

09 December 2009
File R4397

Crown Investments Corporation of Saskatchewan
Policy Analysis & Development
400, 2400 College Avenue
Regina, Saskatchewan
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Attention: Mr. Bob Trafananko
Project Manager

Dear Sir:

Subject: Geotechnical Report
Saskatchewan Multi-Use Facility Feasibility Study
Regina, Saskatchewan

We are pleased to present to you our geotechnical report regarding the above subject.

We thank you for the opportunity to work with you on this project. If you have any questions regarding this report, please contact me.

Yours truly,

Clifton Associates Ltd.

Richard T. Yoshida, P.Eng.
RTY/dkb

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1.0 Introduction

This report presents preliminary results of a geotechnical investigation conducted for the Saskatchewan Multi-Use Facility Feasibility Study located in the existing Canadian Pacific Railway Intermodal Facility and Yard in Regina, Saskatchewan. The site is at approximately Mile 93.8 of the Indian Head Subdivision. Authorization to proceed was received by letter dated 18 September 2009.

The objectives of this investigation were:

- To define the subsurface soil strata including material properties and existing conditions.
- To provide recommendations for suitable methods of foundation support for proposed structure.
- To provide commentary on pertinent geotechnical issues identified during the subsurface investigation, including floors, excavation, and shoring.

2.0 Description of the Site and Proposed Structures

The site is presently Canadian Pacific Railway’s intermodal facility. The site is relatively level, with gravel and asphaltic concrete surface structures. At present, the facility is at the conceptual stage, and no details are presently known. However, it is likely that the main level of the facility will be several metres below existing ground surface.

General and preliminary foundation recommendations contained herein are provided. This information and recommendations can be revised once additional details are known, and once a more detailed investigation is undertaken.

3.0 Field and Laboratory Investigation

The geotechnical investigation was conducted on 02 and 05 October 2009. The locations of the bore holes are shown in Drawing No. R4397-1. Two bore holes were drilled using a truck mounted Brat 22 drill rig and 125 mm solid stem auger to a depth of 22.9 m and 33.0 m below ground surface.

Representative disturbed and undisturbed samples were recovered for laboratory analysis. The natural water content of each sample was determined. Other testing included determination of Atterberg limits, unit weight, and confined compressive strength of selected representative samples. The undrained shear strength of undisturbed samples was estimated using laboratory vane shear and the pocket penetrometer apparatus.

One standpipe piezometer was installed. The piezometer was constructed using 50 mm diameter Schedule 40 PVC pipe and screen. The screen was covered with frac sand and the remainder of the annulus was filled with bentonite chips to eliminate surface water infiltration. A flush mount protector was installed.

Observations made during the field investigation, visual descriptions and the results of the laboratory tests are recorded in the Bore Hole Logs and the Summary of Sampling and Laboratory Test Data which are appended to this report. An explanation of the symbols and terms used in the bore hole logs is included in the Symbols and Terms section.

4.0 Analysis

4.1 Stratigraphy

Stratigraphy in the area consisted of fill overlying high plasticity clay, silty clay, glacial till, and various stratified silty, sandy and clay strata.

Fill extended to a depth of 0.8 m to 2.8 m. Fill consisted of sandy, gravelly clay with some construction rubble.

High plasticity clay with some silt extended to a depth of 9.0 m to 10.7 m. It was moist and very stiff in consistency, with an estimated undrained shear strength of 130 kPa. In situ water content of the clay stratum was close to or at the plastic limit, which was about 30 percent to 34 percent. The dry density of clay was about 1,350 kg/m^3. The wet density was 1,800 kg/m^3.

Silty clay was encountered next. It extended to a depth of 12.5 m to 13.7 m. Silty clay was moist and firm in consistency, with an estimated undrained shear strength of 35 kPa.

Oxidized till was encountered next. In Bore Hole 101, till extended to a depth of 17.3 m. Till extended to the depth of exploration in Bore Hole 102, which was 22.9 m. Till had a silty, sandy clay matrix with a trace of gravel. It was moist and hard to very hard in consistency, with an estimated undrained shear strength of 360 kPa or higher based on test results.

Although not encountered during the investigation, cobbles and boulders are commonly encountered in glacial till. The dry density of till was about 1,885 kg/m^3. The wet density was 2,210 kg/m^3.

A saturated gravel stratum was encountered in Bore Hole 102 at a depth of about 14.1 m.

Stratified sand, silt and clay was encountered below a depth of 17.3 m.

4.2 Groundwater Regime

Some seepage was encountered within the till stratum in sand and sandy or gravelly lenses. A static groundwater level is generally not encountered within the upper few metres. However, water does infiltrate through fissures and fractures in the clay stratum and can accumulate in or around buried structures. Groundwater levels are expected to fluctuate with the level of development in the area, as well as seasonal changes in precipitation, infiltration and evaporation.

5.0 Discussion

The principal geotechnical issues associated with this project are:

- foundations to support vertical and lateral loads,
- accommodation of heave of high plasticity clay,
- shoring and potential loss of ground around a deep excavation, and
- accommodation of groundwater.

Deep foundations will extend into glacial till, and will develop their capacity on the basis of skin friction or end bearing.

High plasticity clay encountered in the upper 10 m will heave over time as its soil moisture increases, and will shrink when dried. As much as 150 mm or more heave may occur in structures founded near surface and even greater amounts in excavations where overburden pressures are reduced.

Deep excavations will require shoring to prevent loss of ground, particularly due to the proximity of surface and subsurface infrastructure such as the Canadian Pacific Railway main line adjacent commercial space, and streets. Excavations may be supported by various means, including tied back, tangent pile, or diaphragm walls. A tied back wall will require anchors that may extend off the property.

Deep basements may encounter groundwater, which will have to be controlled using a collection system. The potential presence of groundwater producing uplift pressures will have to be assessed.

5.1 Seismic Site Response

The site classification for seismic site response as described in NBCC 2005 (Table 4.1.8.4A) has been defined on the basis of the estimated undrained shear strength and standard penetration test, ‘N’ values since the field investigation did not determine shear wave velocity. The undrained shear strength of high plasticity clay and glacial till encountered was greater than 100 kPa and 200 kPa, respectively. On this basis, design can assume Site Class C conditions for seismic response.

NBCC 2005 prefers to classify sites on the basis of shear wave velocity. Additional research or field work may be required to confirm that the average shear wave velocity within the upper 30 m is 360 m/s or higher, which corresponds to Site Class C conditions.

5.2 Depth of Frost

The depth of freezing in this area is estimated to be 1.8 m (6 ft) and will vary depending on air temperature, ground cover, the type of any fill material utilized during development, and other factors.

5.3 Foundation Alternatives

Selection of foundation type will depend on the magnitude and nature of the applied load. Augered cast-in-place concrete piles are recommended for structures sensitive to vertical movement. Augered piles can be straight shafts developing their capacity on the basis of skin friction or expanded base or belled piles developing capacity on the bases of end bearing belled piles (under reamed piers) where higher capacity is required. One or more pile load tests are recommended to confirm design parameters for piles. Refinement of design parameters and utilization of a larger resistance factor as defined by NBCC 2005 will provide an economical foundation.

For a pile foundation, a void of 150 mm should be created under grade beams and pile caps to eliminate the potential for damage associated with heave due to an increase in soil moisture.

Augered cast-in-place concrete piles may be designed to develop their capacity on the basis of skin friction or end bearing, but not both. Some sloughing and seepage associated with sand or gravel lenses may occur within the upper 15 m. While sand lenses may be of limited extent, sleeving of excavations may be required where they are present. Sand strata were encountered below a depth of about 17 m.

Unfactored skin friction values for pile foundations are summarized in Table 5.1. An unfactored end bearing value of 2,150 kPa may be used for well constructed, machine cleaned bells in the hard till below a depth of about 16 m.

Resistance factors for shallow and deep foundations from NBCC 2005 have been summarized in Table 5.2.

Table 5.1
Summary of Unfactored Skin Friction Values

Soil	Depth (m)		Unfactored Skin Friction (kPa)
	from	to	
clay	0	1.8	0
clay	1.8	10.0	75
till	10.0	17.5	125

Table 5.2
Resistance Factors for Shallow and Deep Foundations

Case	Resistance Factor
• Shallow Foundations: vertical resistance by semi-empirical analysis using laboratory and in situ test data	0.5
• Deep Foundation: bearing resistance to axial load based on semi-empirical analysis using laboratory and in situ test data	0.4
• Deep Foundation: analysis using dynamic monitoring results	0.5
• Deep Foundation: analysis using static loading test results	0.6
• Uplift Analysis: by semi-empirical analysis	0.3
• Uplift Analysis: using load test results	0.4
• Horizontal Load Resistance	0.5

The structure can be supported on augered cast-in-place concrete piles designed on the basis of skin friction or end bearing, but not both. Piles developing their capacity on the basis of end bearing will undergo larger settlement than piles developing their capacity on the basis of skin friction.

The desirable centre to centre spacing for piles developing their capacity on the basis of skin friction is 2.5 times the pile diameter. End bearing piles should be designed so that bells do not overlap.

The skin friction contribution of the upper 1.8 m of pile below finished grade should be ignored in the determination of pile capacity. Unfactored skin friction values for augered cast-in-place concrete piles are shown in Table 5.1.

On the basis of observations during the field investigation with bore holes drilled with 125 mm diameter auger, excavation conditions for piles are expected to be good; sleeving may be required for piles due to the presence of occasional sand or gravel lenses. However, it will be prudent to pour concrete as soon as possible after excavation for the piles.

Grade beams must be constructed with a minimum 150 mm void space so that heaving soil does not exert an upward force on piles. This can result in separation of the grade beam from the pile and distortion of the structure.

Concrete should be placed within 2 hrs of excavation to minimize softening of the clay, which can reduce pile capacity and squeezing of soil, which can result in ‘necking’ of the shaft. The aspect ratio of a pile, defined as the ratio between length and diameter, should not exceed 30. This should ensure that good contact is maintained between the concrete and soil and that no voids are created.

The use of water to facilitate excavation of piles should be avoided, since this will result in softening of the soil in contact with the concrete, reducing pile capacity. Inspection during construction is recommended to ensure compliance with specifications.

Settlement of an augered cast-in-place concrete pile developing its capacity on the basis of skin friction will be less than about 5 mm. Settlement of an augered cast-in-place concrete pile developing its capacity on the basis of end bearing will be less than about 15 mm.

Uplift capacity of an augered cast-in-place concrete pile can be calculated on the basis of values provided in Table 5.1 and resistance factors provided in Table 5.2. The end bearing component should not be included in the calculation of uplift capacity. The uplift capacity of a pile group will be the lesser of the sum of the uplift resistance of the piles in the group, or the sum of the resistance mobilized on the surface perimeter of the group, plus the effective weight of the soil and piles enclosed within this perimeter.

5.4 Lateral Loads

The deflection and moment in a pile subjected to a lateral load or moment can be determined using software such as LPILE. This program computes deflection, shear, bending moment and soil response with respect to depth in a nonlinear soil. Soil behaviour is modeled with *p-y* curves that are generated by the software following published recommendations for various types of soils. These relationships consider the relationship between undrained shear strength and soil modulus, as well as strain at 50 percent of the maximum stress.

The relationship between the bending stiffness, EI and bending moment can also be determined. An interaction diagram showing the relationship between unfactored axial load and unfactored bending moment can be created. In this software, the ultimate bending moment for the reinforced concrete section is taken at a maximum strain of concrete of 0.003 based on ACI code. The EI vs. Moment diagrams illustrate the highest values for EI for an uncracked concrete section, the reduction in stiffness as cracking occurs and values for large strain.

5.5 Modulus of Subgrade Reaction

The modulus of subgrade reaction is a conceptual relationship between a pressure applied to the soil and the resulting deflection. The value for the modulus of subgrade reaction is dependent on the size of the foundation and the soil strain.

For clay, the subgrade modulus may be estimated as:

$$k_s = (25/\Delta H) \times 40{,}000 \ (\text{kN/m}^3)$$

where ΔH is the allowable settlement (mm).

When used in analyses, the value for subgrade modulus should be varied over a wide range to assess the sensitivity of performance to the assumed value.

5.6 Coefficient of Earth Pressure

Active and passive earth pressure can be calculated using earth pressure coefficients. For clay, estimated values for active, at rest and passive earth pressure coefficients are as follows:

K_a = 0.5, K_o = 0.67, and K_p = 2, assuming an angle of internal friction, ϕ' = 19º. The in situ total unit weight of clay was about 17.7 kN/m^3.

5.7 Floors

Floor slab design requires consideration of final grades. Construction of grade supported floor slabs should avoid fill material of unknown composition and condition. If significant fill is contemplated, adequate compaction control and material selection criteria will be crucial to ensure suitable performance. Granular fill is preferred considering strength and the potential for vertical movement. It is important to provide a uniform, well constructed granular structure to support the concrete slab so that differential vertical movement is minimized.

Floors can be grade supported, although some vertical movement of floors can be anticipated due to moisture changes in soil, which can cause some cracking and loss of utility. Heave will also occur in deep excavations due to rebound. If this level of performance is not acceptable, a structurally supported floor can be considered.

The subgrade at this site consisted of high plasticity clay. Where encountered, high plasticity clay can be subcut and replaced with granular fill to improve the performance of a grade supported slab. An engineered fill consisting of pit run gravel will reduce the potential for vertical movement associated with heave. High plasticity clay possesses a high potential for heave, estimated to be 100 mm to 150 mm.

Any organic or soft material should be removed and the subgrade should be proof rolled to determine the location of any soft areas. These areas should be excavated and filled with a well graded, compacted granular fill. The subgrade should be compacted to at least 95 percent of the maximum dry density as determined in accordance with the standard Proctor test.

5.8 Excavations

Some seepage will occur from fractures within the surficial clay stratum. Significant dewatering should not be required for excavations. This may change as a result of precipitation and infiltration, as water will move through fissures and fractures.

Silty clay encountered at a depth of about 10 m is generally firm in consistency with a relatively low undrained shear strength. This soil stratum will be untrafficable under wheeled vehicles. A stable working surface can be created by placing a minimum 900 mm of well graded pit run gravel over geotextiles or geogrids, or both, to provide separation and reinforcement. The design of the surface will be dependent on the imposed loads.

In general, excavations should be no steeper than about 1 horizontal to 1 vertical (1:1). Although excavations through these materials may stand in the short term at near vertical angles, oversteepened slopes will slough and collapse if they are left open for long periods of time or if water is allowed to infiltrate. Failure may be sudden and may endanger personnel and equipment working in the vicinity.

The performance of temporary shoring will be dependent upon the quality of workmanship. To limit any loss of ground associated with deformation of the shoring system, alternatives include a rigid tangent pile wall or anchored wall. A tangent pile wall designed to be rigid will generally limit loss of ground when compared to a supported shoring system. The use of tie-back anchors will be limited by property restrictions. It may also be possible to incorporate the tangent pile wall into the basement wall system.

A diaphragm wall can be considered. This type of wall may be constructed by excavating a trench filled with a bentonite slurry, which is later replaced by reinforcing steel and concrete. Alternately, concrete can be used instead of the slurry to construct the wall in one step. The wall can be used as a structural element for the building.

Care of water during construction of the wall and maintenance is very important for good performance. The potential for piping must be considered during design, construction and operation. Local and global stability of any shoring system, particularly anchored walls, should be analyzed.

A total unit weight, γ_t = 17.7 kN/m^3 may be used in the determination of the pressure distribution on shoring. Hydrostatic pressure should be included in the assessment of any shoring design. Design of shoring must also consider any surcharges associated with existing structures or conditions, as well as construction conditions.

An angle of internal friction of 19° can be assumed for high plasticity clay. For a rigid wall, an at rest earth pressure coefficient K_o = 0.67 with a triangular pressure distribution may be used, since rotation of the wall is not desirable. Where some rotation of the wall is allowed, an active earth pressure coefficient, K_a = 0.5 may be used in the determination of the earth pressure distribution.

For an anchored or braced wall, the pressure distribution can be assumed to be a trapezoid with a maximum earth pressure equal to 0.25 γ_t H, where H = height of wall (m). The pressure distribution will be zero at the base of the wall, and will increase to the maximum pressure at a height of 0.2 H from the base of the wall.

Piles for retaining structures will extend into hard glacial till. A tangent pile wall can be constructed by drilling through surficial clay into hard glacial till. Sleeving may be necessary if seepage or sloughing is encountered below the high plasticity clay stratum.

The capacity of anchors can be estimated using the following equation:

$$R = 2\, \sigma_z'\, A_s\, L_s$$

Where σ_z' = effective vertical stress at the midpoint of the load carrying portion of the anchor, A_s = effective unit surface area of the anchor, and L_s = effective embedment length of the anchor. All anchors must be verified by test or proof loading during construction. Loading must account for the possibility of creep.

Care of water during construction and maintenance of temporary shoring will be important to minimize the potential for loss of ground associated with excessive deflection of the wall, or piping, which is the movement of fine soil particles with water.

5.9 General Site Development

Surface material on the site includes asphaltic concrete, clay fill, gravel, slag, and some construction debris. The extent and composition of fill material present should be assessed prior to construction of structures or other surfaces that may be sensitive to settlement. Surfaces may be proof rolled with a heavily loaded tandem truck to locate any soft or compressible areas. These can be excavated and filled with an engineered material. Organic topsoil should be removed prior to placement of any fill to minimize the potential for settlement.

Environmental issues have been discussed under separate cover. Hydrocarbon impacted soil may be encountered in excavations. Special safety precautions and methods of disposal may be required.

Engineered fills supporting important structures should consist of silty, sandy clay or pit run sand or gravel. Cut or fill slopes in clay encountered at this site will be stable at slopes of

2.5:1, but may be subject to increased rates of erosion. Flatter slopes are preferred for landscaping purposes.

The surface of the subgrade for any roads or parking areas should have enough cross-slope to ensure positive surface drainage prior to surfacing, nominally 2 percent.

5.10 Compaction Specifications

Compaction specifications must consider the desired properties of the fill. Specifications will typically require compaction to a percentage of the maximum dry density determined in accordance with the standard Proctor test and may include a range of water contents that are desirable. Depending on the desired properties for the compacted soil, the water content is often provided as a guide to the contractor, since the compactive effort will usually be minimized if the soil is compacted close to the optimum water content determined in accordance with the standard Proctor test. If the soil is wet of optimum, it will be possible to attain a specified density if greater compactive effort or more work is applied to the soil.

The compaction water content will have an impact on the properties of the compacted soil. Soil strength and compressibility is better if the water content is lower than optimum. Soil compacted wet of optimum to the necessary density may be more compressible under low pressure and may have reduced strength. The swelling potential and permeability of a soil will generally be reduced; however, if the soil is compacted wet of optimum.

The following recommendations are provided for compaction.

The excavated subgrade should be uniformly compacted to 95 percent of its maximum dry density determined in accordance with ASTM D698-00a, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lbf/ft^3 (600 kN-m/m^3)]. The water content of the subgrade should be close to optimum water content.

Fill material that will be under a floor slab or paved areas should be compacted to a minimum 98 percent of maximum dry density determined in accordance with the standard Proctor test in lifts no thicker than 150 mm in compacted thickness. The water content should be controlled to be at optimum or optimum plus 2 percent to reduce the amount of potential heave. Fill under landscaped areas should be compacted to a minimum 95 percent of maximum dry density determined in accordance with the standard Proctor test.

Backfill of trenches in areas that already have been compacted should be with new subbase material as specified previously and compacted to a minimum 98 percent of maximum dry density determined in accordance with the standard Proctor test.

Base course or granular material that will be under the floor slab or paved areas should be compacted to a minimum 98 percent of maximum dry density determined in accordance with the standard Proctor test.

Backfill and compact simultaneously each side of walls in layers of 300 mm to ensure that excessive pressure is not applied to one side of the wall.

For estimates of earthwork volumes, a shrinkage factor of 20 percent may be used for silky sand. This should be reduced to about 10 percent for hard clay.

5.11 Potential for Sulphate Attack

In situ soil contains greater than 0.1 percent sulphates by dry weight of soil. On this basis, the potential for sulphate attack will be severe. Sulphate Resistant (Type HS) cement must be specified for all concrete in contact with the native soil. Recommendations regarding sulphate resistant cement may be found in CSA A23.1.

6.0 General Foundation Recommendations

Augered cast-in-place piles developing their load carrying capacity on the basis of skin friction can be considered for structures at this site. If additional capacity is required, piles may be designed on the basis of end bearing. Our specific design criteria and recommendations for augered cast-in-place piles are as follows:

1. Foundation loads may be supported on piles designed as straight shafts, developing load carrying capacity on the basis of skin friction only. Unfactored skin friction values are summarized in Table 5.1. The skin friction contribution of the upper 1.8 m (minimum) below finished grade and any fill material should be ignored in the calculation of pile capacity.

2. On the basis of observations during the field investigation, excavation conditions for piles are expected to be good. Generally, sleeving may be required due to the occasional presence of wet sand lenses. Concrete should be placed within 2 hrs of

excavation to minimize softening of the very stiff clay and clay till, which can reduce pile capacity and squeezing of soil, which can result in necking of the pile shaft.

3. The aspect ratio of a pile, defined as the ratio between length and diameter, should not exceed 30. This should ensure that good contact is maintained between the concrete and soil and that no voids are created.

4. The use of water to facilitate excavation of piles should be avoided, since this will result in softening of the soil in contact with the concrete, reducing pile capacity. Inspection during construction is suggested to ensure compliance with specifications.

5. Should seepage or sloughing occur, the hole should be pumped dry before concreting. The concrete should be placed immediately after drilling while the hole is free of water.

6. Pile shafts must be adequately reinforced to withstand the imposed stresses. Pile reinforcement should extend at least 5 m below finished grade and not less than two thirds the pile length.

7. A void of at least 150 mm beneath grade beams, foundation concrete or other structural portions of the structure constructed atop the piles is recommended to prevent uplifting by soil heaving.

8. If additional pile capacity is required, the piles may be belled out or expanded at the base. For a well constructed, machine cleaned pile, an unfactored end bearing value of 2,150 kPa may be used for foundations below a depth of about 16 m. This value and geotechnical resistance factors used for design may be revised on the basis of a pile load test.

9. Inspection by qualified geotechnical personnel is necessary to ensure that end bearing piles constructed in this manner will be capable of developing these capacities. Skin friction along the shafts of belled piles may not be used in the design of end bearing piles.

10. If hand cleaned bells are employed, belled piles should have a minimum shaft diameter of 700 mm to allow the base to be properly cleaned and inspected prior to placing concrete and steel reinforcement. Such piles should be inspected by a qualified Geotechnical Engineer to confirm the soil strength parameters and approve the piling construction methods.

11. Bells should be constructed with a sideslope not less than 45° and preferably 60° from the horizontal. The base of the bells should be excavated vertically a minimum of 200 mm to allow adequate load transfer to the soil. The base of the bell must be excavated into undisturbed foundation soils of adequate capacity as described in previous sections to carry the design loads.

7.0 Floor Considerations

7.1 Grade Supported Floors

There is a potential of heave associated with an increase in soil moisture with time, and from rebound associated with unloading in deep excavations. Our recommendations for a grade supported floor slab are as follows:

1. The subgrade soil below the proposed floor slab should be excavated to undisturbed soil. Construction on fill material of unknown quality and composition can result in uneven settlement or heave. All topsoil must be removed from the site during subgrade preparation for the grade supported floor slab. Care must be exercised to remove all loose soil and debris. Soft, wet areas, which do not have sufficient trafficability for construction purposes, may be further excavated and replaced with a pit run sand or gravel which complies with the attached specifications.

2. The excavated subgrade should be uniformly compacted to 95 percent of its maximum dry density determined in accordance with ASTM D698-00a, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lbf/ft^3 (600 kN-m/m^3)]. The water content of the subgrade should be at or above the optimum water content.

3. Place a crushed base course which complies with the specifications given in the Recommended Specifications for Granular Materials contained in Appendix A of this report for Type 32 or 33 base course.

4. Compact the base course to a minimum average 98 percent of its maximum dry density for four (4) consecutive tests, with no single test less than 96 percent, as determined in accordance with ASTM D698-00a, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lbf/ft^3 (600 kN-m/m^3)]. Water may be used as an aid to compaction and vibratory compaction equipment is recommended.

5. A layer of polyethylene sheeting 150 µm (minimum) thick should be placed between the granular base and the concrete slab to deter the migration of moisture through the floor.

6. The floor must be structurally isolated from other building elements, service lines and appurtenant structures to prevent stresses caused by floor movement from being transmitted to these elements.

7. Positive site drainage around the building and control of roof drainage away from the building reduces the risk of volume change in grade supported floors.

7.2 Structurally Supported Floor

The following procedures are recommended for the construction of a structurally supported floor:

1. The subgrade should be positively graded to a sump to remove water, which may inadvertently pond beneath the floor.

2. Migration of moisture from the soil should be prevented by installing 150 µm (minimum) thick polyethylene vapour barrier covered with 50 mm of sand.

3. Floors designed, as a structurally supported system with a crawl space between the floor and the subgrade should have some provision to ventilate the crawl space, particularly during the summer months.

4. As an alternative to a crawl space, the floor may be cast upon waxed cardboard carton ‘void form’ that is designed to degrade following the placement of the concrete. The cardboard cartons must have a strength sufficient to support the fresh concrete until it has sufficient strength to be self-supporting. Great care is required during construction of such floor systems to ensure that the collapse of the cartons does not take place, resulting in a grade supported slab. Careful inspection of these floors during construction is required to ensure that the void does not collapse during the placement of the floor. Further, care must be taken during selection of ‘void form’ used. Materials, which depend upon biologic degradation, should be avoided.

8.0 Excavation Considerations

The stability of cut slopes and the stability of any adjacent structure must be considered for any excavations on the site. The anticipated sideslopes for the excavation will depend on the soil texture, water content and length of time that the excavation is left open.

Only minor seepage is expected for excavations that extend into the bedrock clay shale through surficial sand or till. Where encountered, lenses of sand may cause some seepage and sloughing during excavation.

Excavations should be performed in compliance with provincial safety regulations. If construction personnel will be in the excavation, then sideslopes for the excavation should be not steeper than 1:1 for safety as stated in provincial safety guidelines. Sideslopes may have to be adjusted in the field as excavation progresses, depending upon conditions encountered. Continuous inspection is recommended since slope failure could be sudden. Failure may endanger personnel and equipment working in the vicinity.

Soil strata can be jointed and fissured, with jointing occurring randomly. The condition of the soil around the excavation should be carefully observed to ensure that slope failures or sloughing does not occur along any fractures, fissures or joints in the clay. All loose material on the sides of the excavation should be trimmed. The excavation should be left open the minimum amount of time required for construction. Some loss of strength in the soil can be expected with the passing of time, resulting in sloughing and slope failure.

As described in Occupational Health and Safety Regulations, a competent worker should be stationed on the surface to alert any worker in the excavation about the development of any potentially unsafe conditions. Machinery and heavy equipment should not be allowed closer to the excavation than one half of the depth of the excavation, unless precautions are implemented to ensure that workers in the excavation are safe. Spoil material should not be piled closer than 600 mm from the edge of the excavation and with sideslopes no steeper than 1:1.

Infiltration of water into the soil around the excavation can result in loss of strength and collapse of the excavation walls. It is recommended that workers not be in the excavation during rainfall and that excavation walls be carefully inspected for cracking and potential failures after rainfall before work continues in the excavation.

9.0 Underground Walls

It is recommended that the underground walls should be designed to withstand the lateral earth pressure (p) at any depth (H) as estimated by the following expression:

$$p = K (\gamma H + q)$$

where: γ = unit weight of the wall backfill, estimated at 18 kN/m^3 (127 lb/ft^3)

q = the vertical pressure of any surcharge acting at ground surface near the wall

K = 0.4 for a good granular backfill and 0.67 for a backfill composed of native material

This expression assumes that the wall will be backfilled with a free draining granular backfill and will not be subject to buildup of water pressure behind the wall. If effective wall drainage cannot be guaranteed, full hydrostatic pressure, which may act on the wall, must be considered in the design.

Free draining backfill materials should be placed adjacent to the exterior underground walls. Free draining means that the granular material should be well graded and have less than 3 percent passing the 75 µm sieve. The upper 0.5 m of backfill should consist of local compacted soil or the surface must be covered with some other suitable impermeable material. The ground surface should be contoured away from the building to further discourage the entry of surface runoff into the backfill. Regardless of the type of backfill used behind the wall, it is recommended that the wall be effectively damp-proofed to prevent migration of moisture through the concrete. Damp-proofing also aids in reducing the rate of deterioration of the concrete due to chemical attack and weathering.

10.0 Closure

This report was prepared by Clifton Associates Ltd. for the use of Crown Investments Corporation of Saskatchewan and their agents for specific application to the Saskatchewan Multi-Use Facility Feasibility Study in Regina, Saskatchewan. The material in it reflects Clifton Associates Ltd. best judgment available to it at the time of preparation. 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. Clifton Associates 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.

This report has been prepared with generally accepted engineering practices common to the local area. No other warranty, expressed or implied, is made.

Our conclusions and recommendations are preliminary and based upon the information obtained from the referenced subsurface exploration. The borings and associated laboratory testing indicate subsurface and groundwater conditions only at the specific locations and times investigated, only to the depth penetrated and only for the soil properties tested. The subsurface and groundwater conditions may vary between the bore holes and with time. The subsurface interpretation provided is a professional opinion of conditions and not a certification of the site conditions. The nature and extent of subsurface variation may not become evident until construction or further investigation. If variations or other latent conditions do become evident, Clifton Associates Ltd. should be notified immediately so that we may re-evaluate our conclusions and recommendations. Although subsurface conditions have been explored, we have not conducted analytical laboratory testing on samples obtained nor evaluated the site with respect to the potential presence of contaminated soil or groundwater.

The enclosed report contains the results of our investigation as well as certain recommendations arising out of such investigations. Our recommendations do not constitute a design, in whole or in part, of any elements of the proposed work. Incorporation of any or all of our recommendations into the design of any such element does not constitute us as designers or co-designers of such elements, nor does it mean that the design is appropriate in geotechnical terms. The designers of such elements must consider the appropriateness of our recommendations in light of all design criteria known to them, many of which may not be known to us. Our mandate has been to investigate and recommend which we have completed by means of this report. We have had no mandate to design, or review the design, of any elements of the proposed work and accept no responsibility for such design or design review.

Clifton Associates Ltd.

Richard T. Yoshida, P.Eng.

Association of Professional
Engineers and Geoscientists of Saskatchewan
Certificate of Authorization No. 238

Clifton Associates Ltd. engineering science technology	# Symbols and Terms

Soil Descriptive Terms

A soil description for geotechnical applications includes a description of the following properties:

- texture
- color, oxidation
- consistency and condition
- primary and secondary structure

Texture

The soil texture refers to the size, size distribution and shape of the individual soil particles which comprise the soil. The Unified Soil Classification System (ASTM D2487-00) is a quantitative method of describing the soil texture. The basis of this system is presented overleaf. The following terms are commonly used to describe the soil texture.

Particle Size
(ASTM D2487-00)

Boulder	300 mm plus
Cobble	75 - 300 mm
Gravel	4.75 - 75 mm
Coarse	19 - 75 mm
Fine	4.75 - 19 mm
Sand	0.075 - 4.75 mm
Coarse	2 - 4.75 mm
Medium	0.425 - 2 mm
Fine	0.075 - 0.425 mm
Silt and Clay	Smaller than 0.075 mm

Relative Proportions
(CFEM, 3rd Ed., 1992)

Trace	1 - 10 %
Some	10 - 20 %
Gravelly, sandy, silty, clayey, etc.	20 - 35 %
And	>35 %
Gravel, Sand, Silt, Clay	>35 % and main fraction

Gradation

Well Graded	Having a wide range of grain sizes and substantial amount of all intermediate sizes.
Uniform or Poorly Graded	Possessing particles of predominantly one size.
Gap Graded	Possessing particles of two distinct sizes.

Particle Shape

Angular	Sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Similar to 'angular' but have rounded edges.
Subrounded	Well-rounded corners and edges, nearly plane sides.
Rounded	No edges and smoothly curved sides.

Also may be flat, elongated or both.

The term “TILL” may be used as a textural term to describe a soil which has been deposited by glaciers and contains an unsorted, wide range of particle sizes.

Color And Oxidation

The soil color at its natural moisture content is described by common colors and, quantitatively, in terms of the Munsell color notation; (eg. 5Y 3/1). The notation combines three variables, hue, value and chroma to describe the soil color. The hue indicates its relation to red, yellow, green, blue and purple. The value indicates its lightness. The chroma indicates its strength of departure from a neutral of the same lightness.

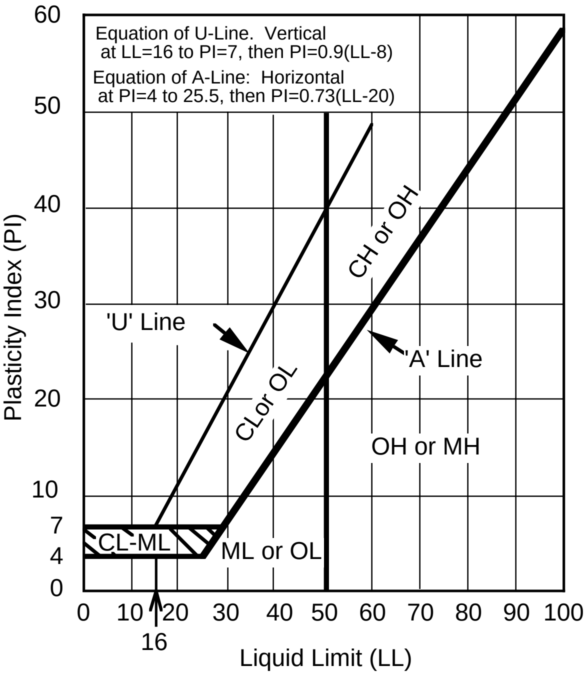
Departure of the soil color from a neutral color indicates the soil has been oxidized. Oxidation of a soil occurs in a oxygen rich environment where most commonly metallic iron, oxidizes and turns a neutral colored soil 'rusty' or reddish brown. Oxidized manganese gives a purplish tinge to the soil. Oxidation may occur throughout the entire soil mass or on fracture/joint/fissure surfaces.

Classification of Soils for Engineering Purposes

ASTM Designation D 2487-00 (Unified Soil Classification System)

Major divisions			Group Symbols	Typical names	Classification criteria			
Coarse-grained soils More than 50% retained on No. 200 sieve* (>0.075 mm)	Gravels More than 50% of coarse fraction retained on No. 4 sieve(≥4.75 mm)	Clean gravels <5% fines	GW	Well-graded gravel	If ≥ 15% sand add "with sand" to group name	Classification on basis of percentage of fines Less than 5% pass No. 200 sieve......GW, GP, SW, SP More than 12% pass No. 200 sieve.....GM, GC, SM, SC 5 to 12% pass No. 200 sieve.......Borderline classifications requiring use of dual symbols	$C_u = \frac{D_{60}}{D_{10}} \geq 4$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravel			Not meeting either C_u or C_c criteria for GW	
		Gravels with fines >12% fines	GM	Silty gravel			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols If fines are organic add "with orgnic fines" to group name
			GC	Clayey gravel			Atterberg limits on or above "A" line and PI > 7	
	Sands 50% or more of coarse fraction passes No. 4 sieve(<4.75 mm)	Clean sands <5% fines	SW	Well-graded sand	If ≥ 15% gravel add "with gravel to group name		$C_u = \frac{D_{60}}{D_{10}} \geq 6$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sand			Not meeting either C_u or C_c criteria for SW	
		Sands with fines >12% fines	SM	Silty sand			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols If fines are organic add "with orgnic fines" to group name
			SC	Clayey sand			Atterberg limits on or above "A" line and PI > 7	
Fine-grained soils 50% or more passes No. 200 sieve* (≤0.075 mm)	Silts and Clays Liquid limit <50%	Inorganic	ML	Silt	If 15 to 29% coarse-grained, add "with sand" or "with gravel" as appropriate If > 30% coarse-grained , add "sandy" or "gravelly" as appropriate Class as organic when oven dried liquid limit is < 75% of undried liquid limit	Plasticity Chart		
			CL	Lean Clay -low plasticity				
		Organic	OL	Organic clay or silt (Clay plots above 'A' Line)				
	Silts and Clays Liquid limit ≥50%	Inorganic	MH	Elastic silt				
			CH	Fat Clay -high plasticity				
		Organic	OH	Organic clay or silt (Clay plots above 'A' Line)				
	Highly organic soils		PT	Peat, muck and other highly organic soils				

Plasticity Chart



*Based on the material passing the 3 in.(75 mm) sieve, if field samples contain cobbles or boulders, add "with cobbles or boulders" to group name

Consistency And Condition

The consistency of a cohesive soil is a qualitative description of its resistance to deformation and can be correlated with the undrained shear strength of the soil. The condition of a coarse grained soil qualitatively describes the soil compactness and can be correlated with the standard penetration resistance (ASTM D1586-99).

Consistency Of Cohesive Soil (CFEM, 3rd Edit., 1992)

Consistency	Undrained Shear Strength (kPa) (CFEM, 3rd Edt., 1992)	Field Identification (ASTM D 2488-00)
Very Soft	<12	Thumb will penetrate soil more than 25 mm.
Soft	12-25	Thumb will penetrate soil about 25 mm.
Firm	25-50	Thumb will indent soil about 6 mm.
Stiff	50-100	Thumb will indent, but penetrate only with great effort (CFEM).
Very Stiff	100-200	Readily indented by thumbnail (CFEM).
Hard	>200	Thumb will not indent soil but readily indented with thumbnail.
Very Hard	N/A	Thumbnail will not indent soil.

Condition Of Coarse Grained Soil (CFEM, 3rd Edt., 1992)

Compactness Condition	SPT N - Index (Blows/300mm)
Very Loose	0 - 4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very Dense	over 50

Moisture Conditions (ASTM D2488-00)

Description	Criteria
Dry	Absence of moisture, dusty, dry to touch
Moist	Damp but no visible water
Wet	Visible, free water, usually soil is below water table

Structure

The soil structure is the manner in which the individual soil particles are assembled to form the soil mass. The primary soil structure is the arrangement of soil particles as originally deposited. The secondary soil structure refers to any rearrangement of the soil such as deformation and cracking which has taken place since deposition.

Primary Soil Structure (Depositional)

A. Geometry

Stratum - A single sedimentary 'layer', greater than 10 mm in thickness, visibly separable from other strata by a discrete change in lithology and/or sharp physical break.

Homogeneous - Same color and appearance throughout.

Stratified - Consisting of a sequence of layers which are generally of contrasting texture or color.

Laminated - Stratified with layer thicknesses between 2 mm and 10 mm.

Thinly laminated - Stratified with layer thickness less than 2 mm.

Bedded - Stratified with layer thicknesses greater than 10 mm.

Very Thinly Bedded (Flaggy) - Stratified with layer thicknesses between 10 and 50 mm.

Thinly Bedded (Slabby) - Stratified with layer thicknesses between 50 and 600 mm.

Thickly Bedded (Blocky) - Stratified with layer thicknesses between 600 and 1200 mm.

Thick-Bedded (Massive) - Stratified with layer thicknesses greater than 1200 mm.

Lensed - Inclusions of small pockets of different soils, such as small lenses of sand material throughout a mass of clay.

B. Bedding Structures

Cross-bedding - Internal 'bedding' inclined to the general bedding plane.

Ripple-bedding - Internal 'wavy bedding'.

Graded-bedding - Internal gradation of grain size from coarse at base to finer at top of bed.

Horizontal bedded - Internal bedding is parallel and flat lying

Secondary Soil Structure (Post-Depositional)

A. Accretionary Structures

Includes nodules, concretions, crystal aggregates, veinlets, color banding and

Cementation - Chemically precipitated material, commonly calcite ($CaCO_3$), binds the grains of soil, usually sandstone. Described as weak, moderate, strong (ASTM D2488-00).

Salt Crystals - Groundwater flowing through the soil/rock often precipitates visible amounts of salts. Calcite ($CaCO_3$), glauber salts ($Na_2Ca(SO_4)_2$), and gypsum ($CaSO_4*2H_2O$) are common.

B. Fracture Structures

Fracture - A break or discontinuity in the soil or rock mass caused by stress exceeding the materials strength.

Joint - A fracture along which no displacement has occurred.

Fissure - A gapped fracture, which may open and close seasonally. Usually an extensive network of closely spaced fractures, giving the soil a 'nuggetty' structure.

Slickensides - Fractures in a clay that are slick and glossy in appearance, caused by shear movements.

Brecciated - Contains randomly oriented angular fragments in a finer mass, usually associated with shear displacements in soils.

Fault - A fracture or fracture zone along which there has been displacement.

Blocky - A cohesive soil that can be broken down into small angular lumps which resist further breakdown.

Symbols Used on Bore Hole Logs

Lithology Type

- ░ CLAY
- ░ SILT
- ░ SAND
- ░ GRAVEL
- ▓ COBBLES
- ░ TILL-oxidized
- ░ TILL-unoxidized
- ░ PEAT
- ░ TOPSOIL or ORGANIC SOIL
- ░ COAL
- ░ FILL (Undifferentiated)
- ░ CONCRETE
- ■ ASPHALT
- ▤ CLAY SHALE
- ░ SANDSTONE
- ▨ MUDSTONE
- ░ BEDROCK (Undifferentiated)

Borehole Completion and Backfill Materials

- ■ Bentonite
- ░ Concrete
- ▦ Cover
- ░ Cuttings
- ░ Grout
- ░ Sand
- ▧ Slough
- ▯ Solid Pipe
- ▯ Slotted Pipe

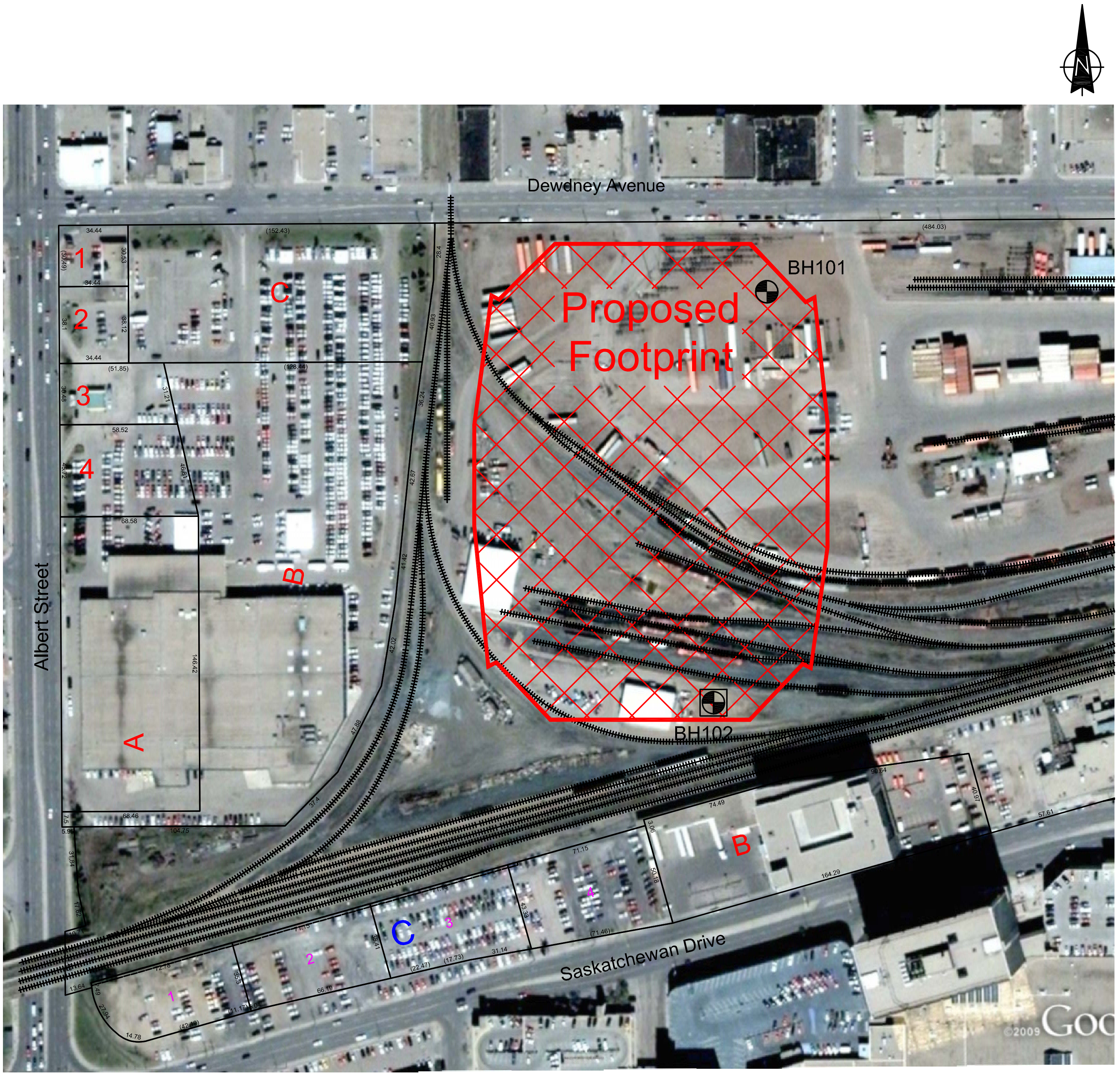
Soil Sample Type

- ■ Thin Walled Tube
- ▩ Driven Spoon
- ▤ Disturbed
- ▥ Core (any type)
- ▨ No Recovery

Groundwater Symbols

- ▼ Piezometric elevation as determined by a piezometer installation
- ▽ Water levels measured in borings at the time and under the conditions noted

Clifton Associates Ltd. engineering science technology	**Drawings**



LEGEND:

BORE HOLE	◒
PIEZOMETER	▣

NOTES:

1. IMAGE SOURCE: GOOGLE.

DRAWING REVISIONS

NO.	DD/MM/YY	DESCRIPTION	BY

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CLIENT
CROWN INVESTMENTS
CORPORATION OF SASKATCHEWAN

PROJECT TITLE
SASKATCHEWAN MULTI–USE
FACILITY FEASIBILITY STUDY

DRAWING TITLE
BORE HOLE
LOCATION PLAN

PROJECT NO. R4397		FILE NO. R4397	
DATE 22/10/09	SCALE NTS	DWG. NO. R4397–1	
DRAWN KML	CHECKED RTY		

J:\Sdsk\proj\R\R4300s\R4397\R4397.dwg

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Bore Hole Logs and Laboratory Test Data

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BORE HOLE LOG

Bore Hole: **101**
Page: 1 of 4

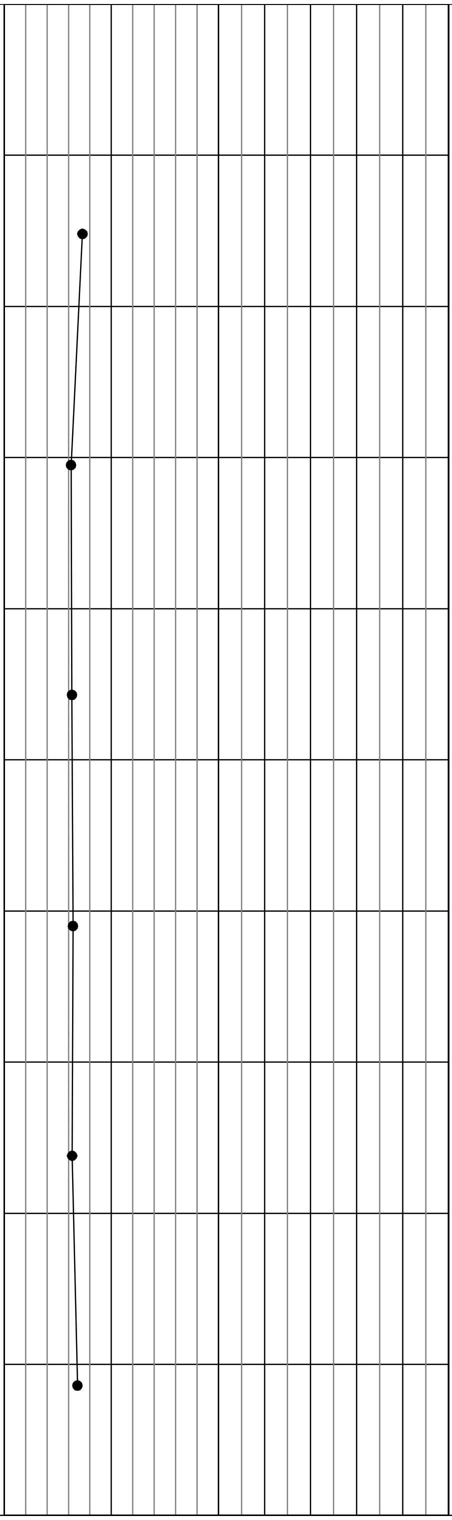
Client:	Crown Investments Corporation	Northing:	-	Date Drilled:	02 Oct 09
Project:	Saskatchewan Multi-Use Facility	Easting:	-	Drill:	Brat 22
Location:	Regina, SK	Ground Elev.:	-	Drilling Method:	Solid Stem Auger
Project No.:	R4397	Top Casing Elev.:	-	Logged by:	ADL

Depth (m)	Symbol	Soil Description	Sample Type	Sample No.	SPT 'N'	USC	% Sulphate
0		Fill: Sandy, gravely clay to 1.0m, then predominantly clay. Black. Moist. Some construction rubble.					
1			▤	AL1			
2		Clay: Some silt. Oxidized, calcareous. Very dark grayish brown (2.5Y 3/2). Homogeneous structure. Moist, very stiff. Salts.					
3			▤	AL2			
4			▤	AL3			
5							
6			▤	AL4			
7			▤	AL5			
8							
9			▤	AL6			
10							

Moisture Content
percent
Plastic Limit ▲ Natural Moisture ● Liquid Limit ◆
0 50 100

▲ Dry Density - kg/m3
1800 2200
Shear Strength - kPa
Unconf. ■ Pocket Pen. ● Lab Vane ◆
100 200 300 400

Piezometer Construction Detail



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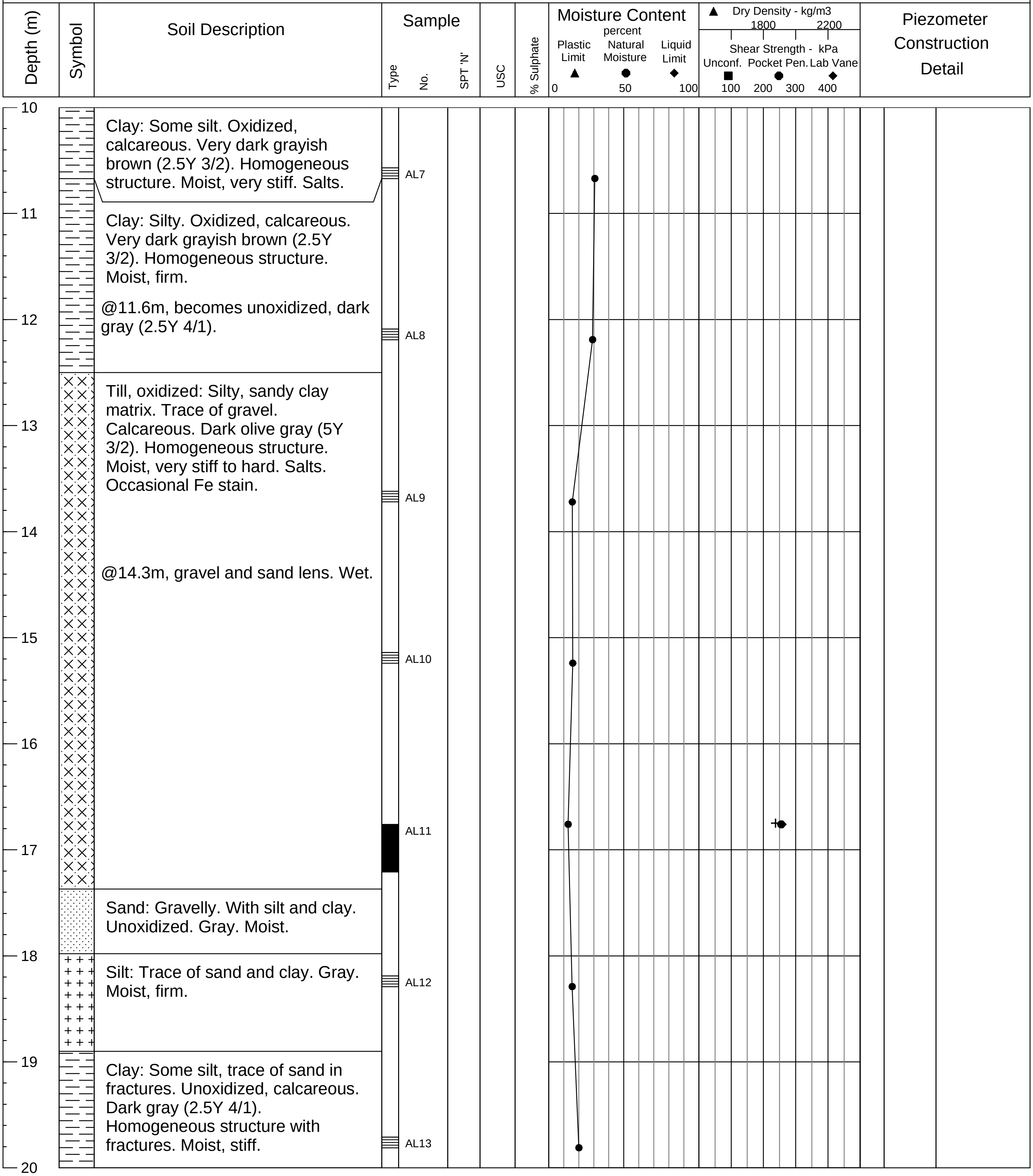
BORE HOLE LOG

Bore Hole: 101
Page: 2 of 4

Client: Crown Investments Corporation
Project: Saskatchewan Multi-Use Facility
Location: Regina, SK
Project No.: R4397
Northing: -
Easting: -
Ground Elev.: -
Top Casing Elev.: -
Date Drilled: 02 Oct 09
Drill: Brat 22
Drilling Method: Solid Stem Auger
Logged by: ADL

Soil Description	Sample No.
Clay: Some silt. Oxidized, calcareous. Very dark grayish brown (2.5Y 3/2). Homogeneous structure. Moist, very stiff. Salts.	AL7
Clay: Silty. Oxidized, calcareous. Very dark grayish brown (2.5Y 3/2). Homogeneous structure. Moist, firm. @11.6m, becomes unoxidized, dark gray (2.5Y 4/1).	AL8
Till, oxidized: Silty, sandy clay matrix. Trace of gravel. Calcareous. Dark olive gray (5Y 3/2). Homogeneous structure. Moist, very stiff to hard. Salts. Occasional Fe stain. @14.3m, gravel and sand lens. Wet.	AL9 AL10 AL11
Sand: Gravelly. With silt and clay. Unoxidized. Gray. Moist.	
Silt: Trace of sand and clay. Gray. Moist, firm.	AL12
Clay: Some silt, trace of sand in fractures. Unoxidized, calcareous. Dark gray (2.5Y 4/1). Homogeneous structure with fractures. Moist, stiff.	AL13



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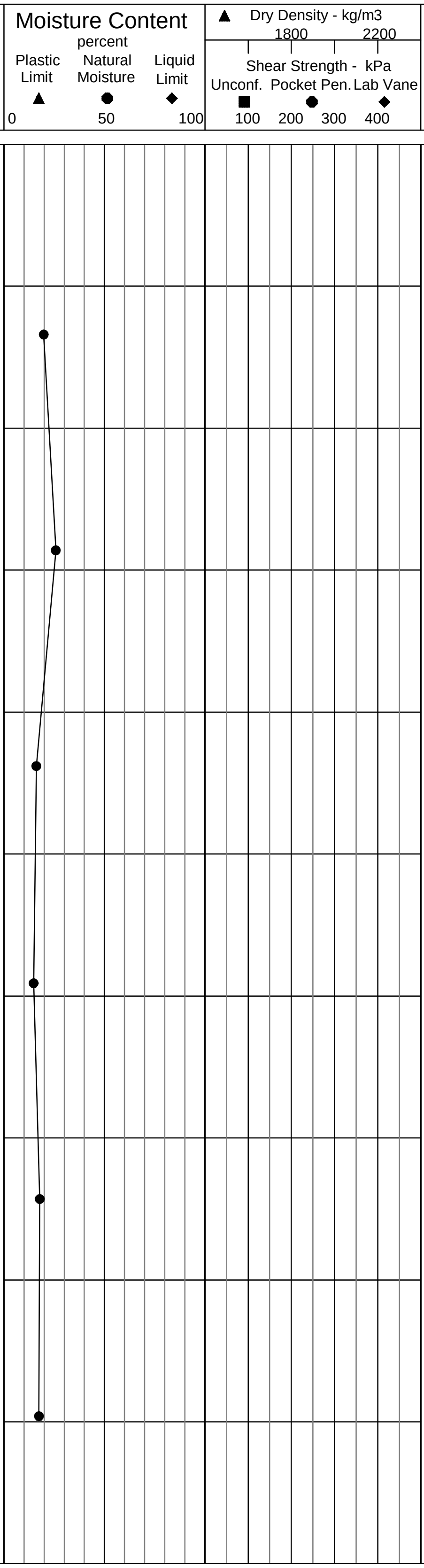
BORE HOLE LOG

Bore Hole: **101**
Page: 3 of 4

Client:	Crown Investments Corporation	Northing:	-	Date Drilled:	02 Oct 09
Project:	Saskatchewan Multi-Use Facility	Easting:	-	Drill:	Brat 22
Location:	Regina, SK	Ground Elev.:	-	Drilling Method:	Solid Stem Auger
Project No.:	R4397	Top Casing Elev.:	-	Logged by:	ADL

Depth (m)	Symbol	Soil Description	Sample Type	Sample No.	SPT 'N'	USC	% Sulphate	Piezometer Construction Detail
20		Clay: Some silt, trace of sand in fractures. Unoxidized, calcareous. Dark gray (2.5Y 4/1). Homogeneous structure with fractures. Moist, stiff.						
21			☰	AL14				
22		Clay: Some silt and sand. Unoxidized, calcareous. Light gray (5Y 7/1). Homogeneous structure. Moist, stiff.	☰	AL15				
23		Till, unoxidized: Silty, sandy clay matrix. Trace of gravel. Calcareous. Gray. Homogeneous structure. Moist, stiff.						
24		Sand: Coarse grained. Some gravel. Wet.	☰	AL16				
25			☰	AL17				
26								
27			☰	AL18				
28								
29			☰	AL19				
30								



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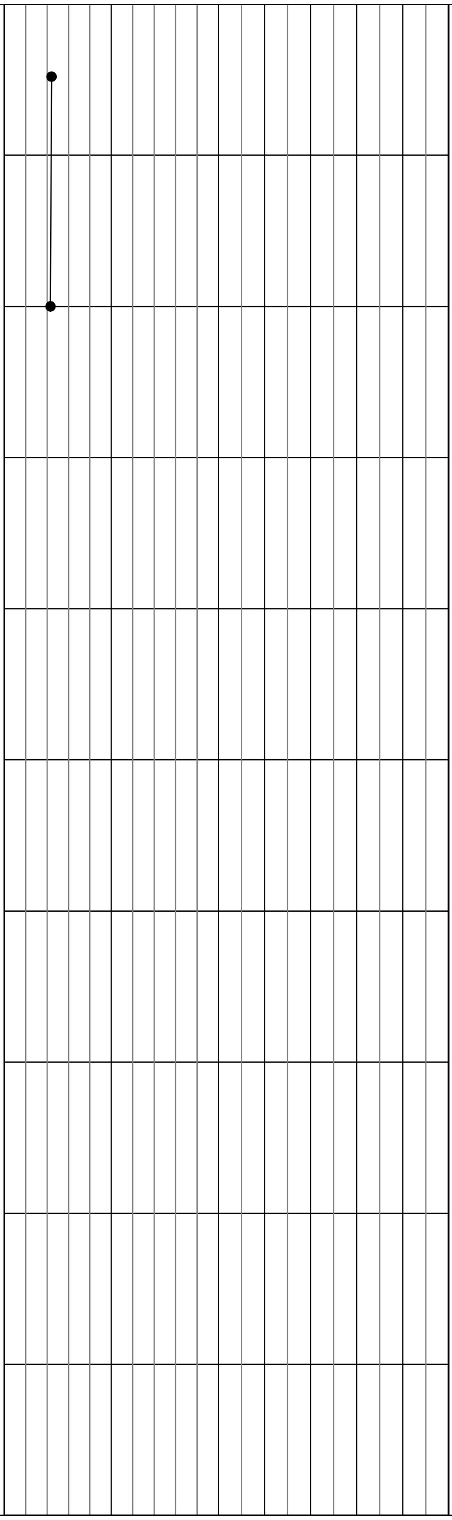
BORE HOLE LOG

Bore Hole: **101**
Page: 4 of 4

Client:	Crown Investments Corporation	Northing:	-	Date Drilled:	02 Oct 09
Project:	Saskatchewan Multi-Use Facility	Easting:	-	Drill:	Brat 22
Location:	Regina, SK	Ground Elev.:	-	Drilling Method:	Solid Stem Auger
Project No.:	R4397	Top Casing Elev.:	-	Logged by:	ADL

Depth (m)	Symbol	Soil Description	Sample Type	Sample No.	SPT 'N'	USC	% Sulphate	Moisture Content percent (Plastic Limit ▲, Natural Moisture ●, Liquid Limit ◆; 0 50 100)	▲ Dry Density - kg/m3 (1800 2200); Shear Strength - kPa (Unconf. ■, Pocket Pen. ●, Lab Vane ◆; 100 200 300 400)	Piezometer Construction Detail
30		Sand: Coarse grained. Some gravel. Wet.								
		Sand: Fine grained, some clay and silt. Trace of gravel. Calcareous. Olive gray (5Y 4/2). Homogeneous structure. Wet.	☰	AL20				●		
31										
32		Notes: 125mm diam continous flight auger used. Sloughing to 13.1m. Water to 6.4m at completion of drilling. Backfilled with cuttings and bentonite chips.	☰	AL21				●		
33										
34										
35										
36										
37										
38										
39										
40										



SUMMARY OF SAMPLING AND LABORATORY TEST DATA

SAMPLE					CONSISTENCY				GRADATION					SHEAR STRENGTH			
DEPTH	NUMBER	TYPE	RECOVERY	WATER CONTENT	PLASTIC LIMIT	LIQUID LIMIT	PLASTICITY INDEX	USC	GRAVEL	SAND	SILT	CLAY	SULPHATE CONTENT	COMPRESSION TEST	LAB VANE	POCKET PEN	DRY DENSITY
meters			mm	%	%	%	%		%	%	%	%	%	kPa	kPa	kPa	kg/m^3
1.52	AL1	BAG		36.5													
3.05	AL2	BAG		31.2													
4.57	AL3	BAG		31.6													
6.10	AL4	BAG		32.1													
7.62	AL5	BAG		31.7													
9.14	AL6	BAG		34.2													
10.67	AL7	BAG		30.7													
12.19	AL8	BAG		29.2													
13.72	AL9	BAG		15.7													
15.24	AL10	BAG		16.0													
16.76	AL11	SY	445	12.9											260+	255	
18.29	AL12	BAG		15.6													
19.81	AL13	BAG		20.1													
21.34	AL14	BAG		19.8													
22.86	AL15	BAG		25.8													
24.38	AL16	BAG		16.1													
25.91	AL17	BAG		14.8													

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PROJECT Saskatchewan Multi-Use Facility
LOCATION Regina, Saskatchewan
PROJECT NO. R4397

BORE HOLE NO.

101

SUMMARY OF SAMPLING AND LABORATORY TEST DATA

SAMPLE					CONSISTENCY				GRADATION					SHEAR STRENGTH			
DEPTH	NUMBER	TYPE	RECOVERY	WATER CONTENT	PLASTIC LIMIT	LIQUID LIMIT	PLASTICITY INDEX	USC	GRAVEL	SAND	SILT	CLAY	SULPHATE CONTENT	COMPRESSION TEST	LAB VANE	POCKET PEN	DRY DENSITY
meters			mm	%	%	%	%		%	%	%	%	%	kPa	kPa	kPa	kg/m^3
27.43	AL18	BAG		17.8													
28.96	AL19	BAG		17.4													
30.48	AL20	BAG		22.1													
32.00	AL21	BAG		21.6													

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PROJECT Saskatchewan Multi-Use Facility
LOCATION Regina, Saskatchewan
PROJECT NO. R4397

BORE HOLE NO.

101

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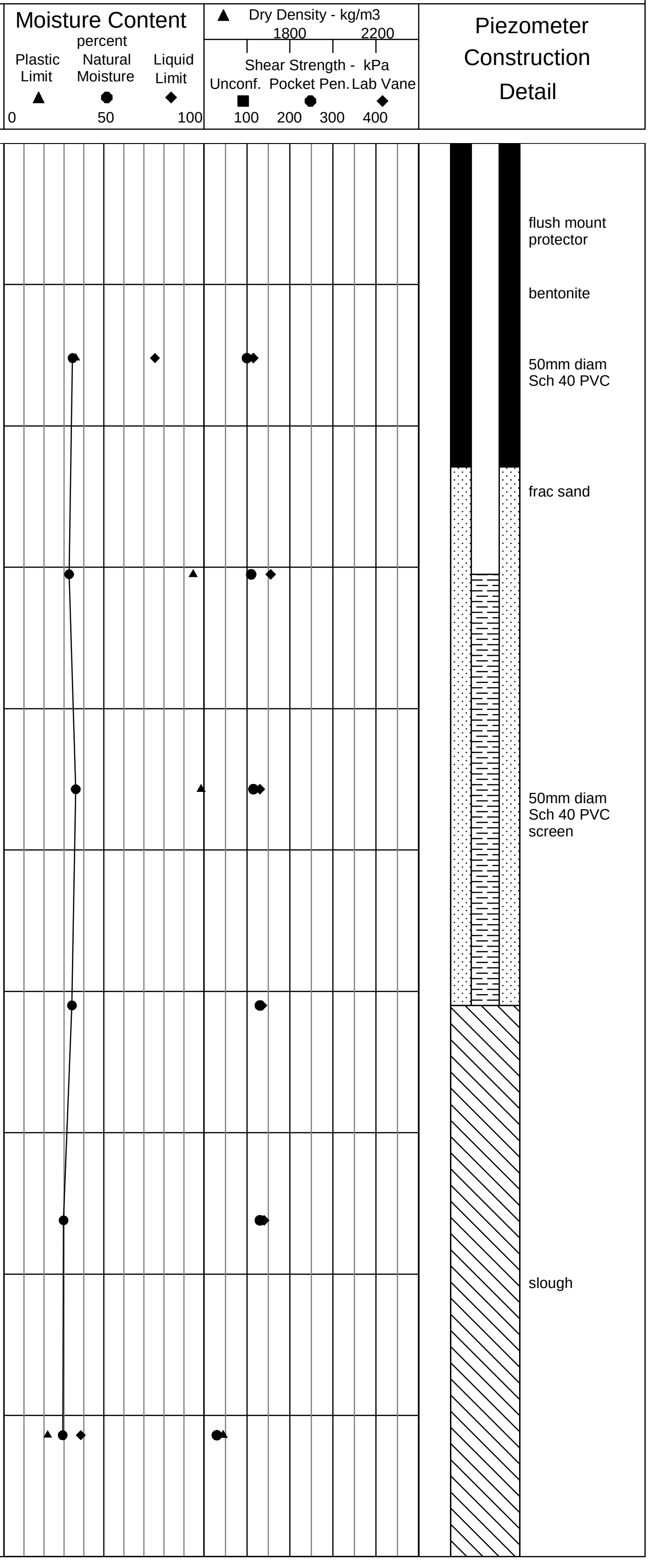
BORE HOLE LOG

Bore Hole: **102**
Page: 1 of 3

Client:	Crown Investments Corporation	Northing:	-	Date Drilled:	02 Oct 09
Project:	Saskatchewan Multi-Use Facility	Easting:	-	Drill:	Brat 22
Location:	Regina, SK	Ground Elev.:	-	Drilling Method:	Solid Stem Auger
Project No.:	R4397	Top Casing Elev.:	-	Logged by:	ADL

Depth (m)	Symbol	Soil Description	Sample Type	No.	SPT 'N'	USC	% Sulphate
0		Fill: Sandy gravel, with clay. Black.					
1		Clay: Some silt. Oxidized, calcareous. Very dark grayish brown (2.5Y 3/2). Homogeneous structure. Moist, very stiff. Salts.	■	AL22		CH	
2							
3			■	AL23			
4			■	AL24			
5							
6			■	AL25			
7			■	AL26			
8							
9		Clay: Silty. Oxidized, calcareous. Dark grayish brown (2.5Y 4/2). Homogeneous structure. Moist, firm.	■	AL27		CL	
10							



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BORE HOLE LOG

Bore Hole: 102
Page: 2 of 3

Client:	Crown Investments Corporation	Northing:	-	Date Drilled:	02 Oct 09
Project:	Saskatchewan Multi-Use Facility	Easting:	-	Drill:	Brat 22
Location:	Regina, SK	Ground Elev.:	-	Drilling Method:	Solid Stem Auger
Project No.:	R4397	Top Casing Elev.:	-	Logged by:	ADL

Depth (m) | Symbol | Soil Description

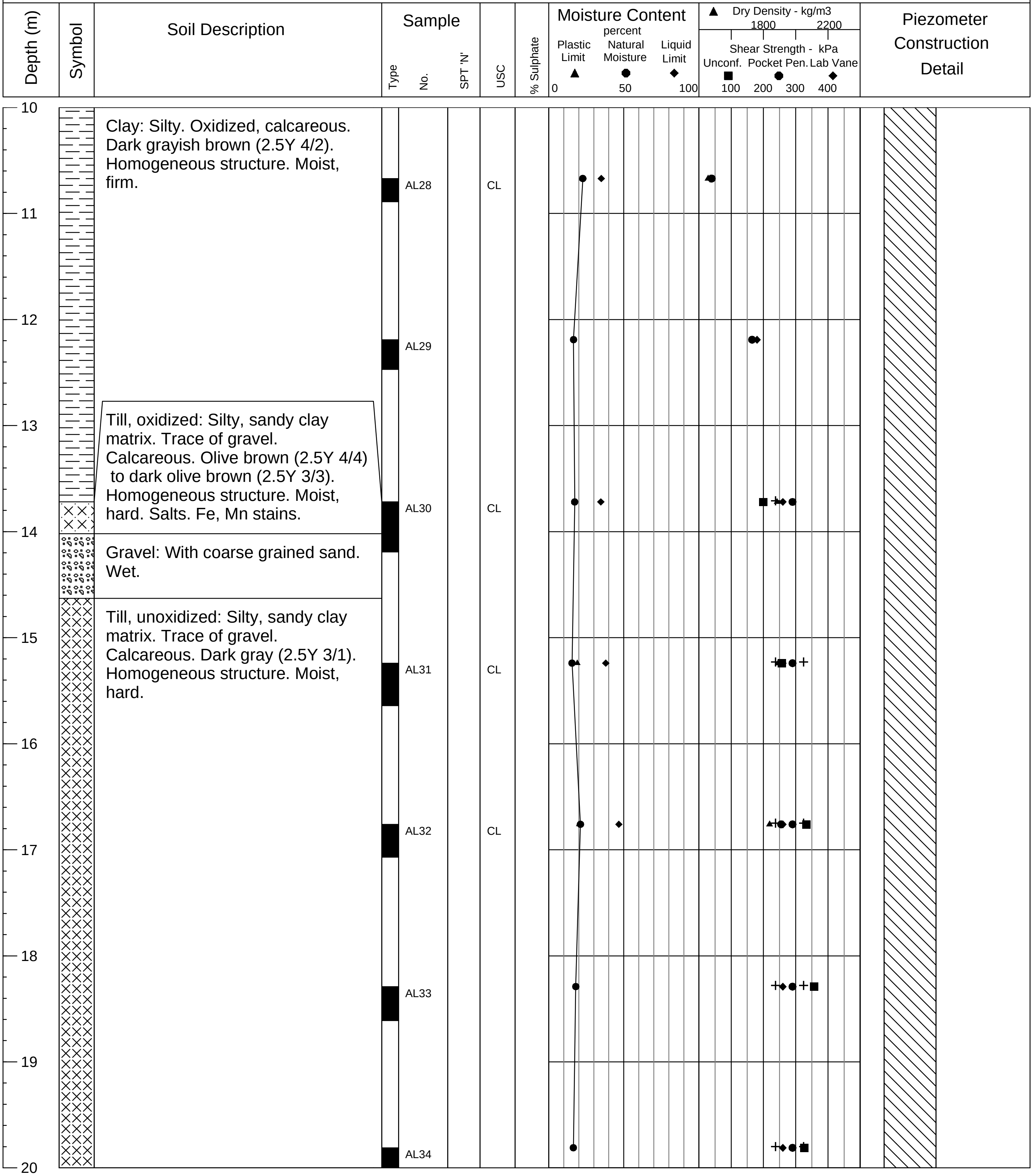
Clay: Silty. Oxidized, calcareous. Dark grayish brown (2.5Y 4/2). Homogeneous structure. Moist, firm.

Till, oxidized: Silty, sandy clay matrix. Trace of gravel. Calcareous. Olive brown (2.5Y 4/4) to dark olive brown (2.5Y 3/3). Homogeneous structure. Moist, hard. Salts. Fe, Mn stains.

Gravel: With coarse grained sand. Wet.

Till, unoxidized: Silty, sandy clay matrix. Trace of gravel. Calcareous. Dark gray (2.5Y 3/1). Homogeneous structure. Moist, hard.

Sample Type	Sample No.	SPT 'N'	USC	% Sulphate
■	AL28		CL	
■	AL29			
■	AL30		CL	
■	AL31		CL	
■	AL32		CL	
■	AL33			
■	AL34			



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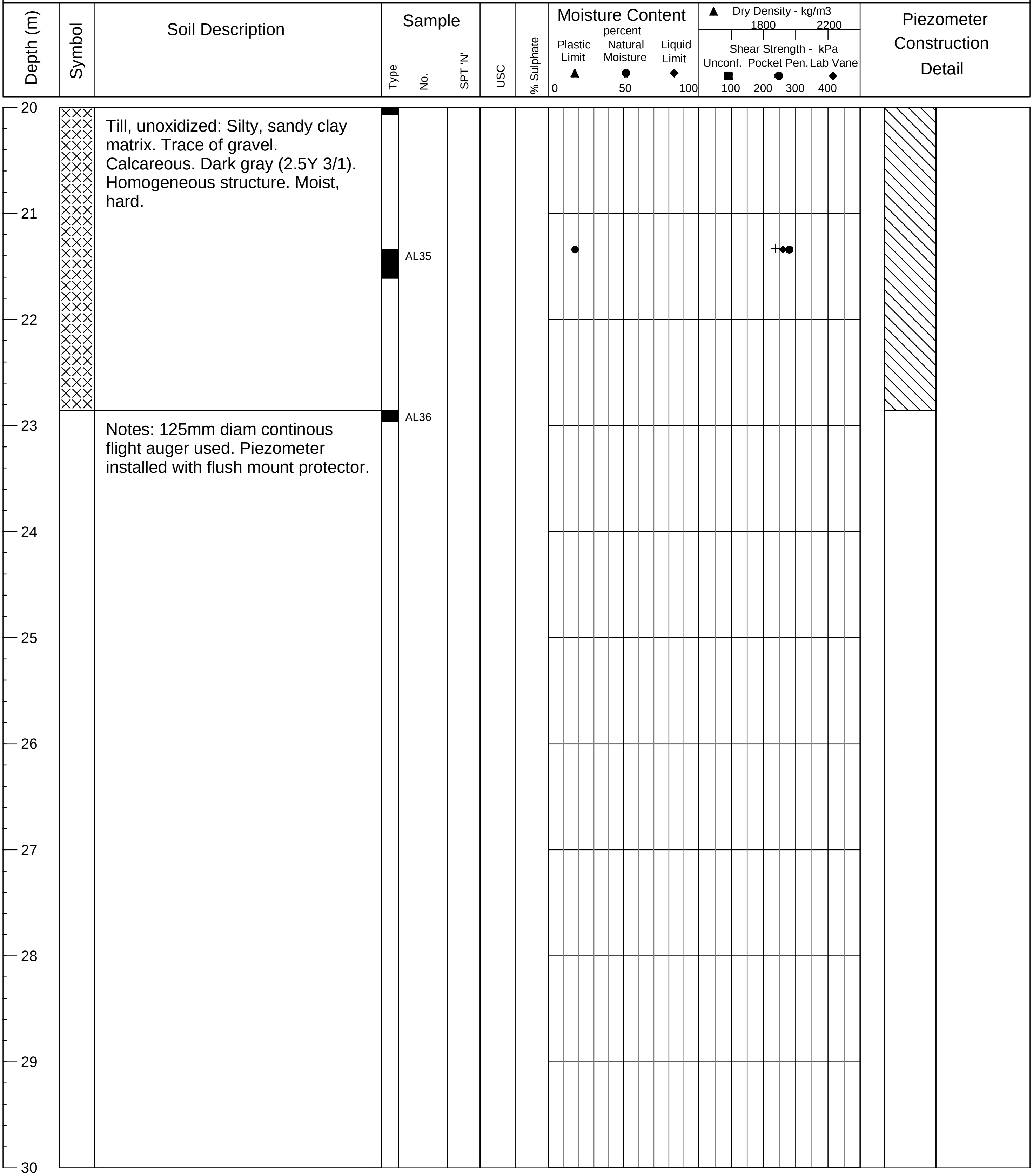
BORE HOLE LOG

Bore Hole: 102
Page: 3 of 3

Client:	Crown Investments Corporation	Northing:	-	Date Drilled:	02 Oct 09
Project:	Saskatchewan Multi-Use Facility	Easting:	-	Drill:	Brat 22
Location:	Regina, SK	Ground Elev.:	-	Drilling Method:	Solid Stem Auger
Project No.:	R4397	Top Casing Elev.:	-	Logged by:	ADL



Till, unoxidized: Silty, sandy clay matrix. Trace of gravel. Calcareous. Dark gray (2.5Y 3/1). Homogeneous structure. Moist, hard.

Notes: 125mm diam continous flight auger used. Piezometer installed with flush mount protector.

SUMMARY OF SAMPLING AND LABORATORY TEST DATA

SAMPLE					CONSISTENCY				GRADATION					SHEAR STRENGTH			
DEPTH	NUMBER	TYPE	RECOVERY	WATER CONTENT	PLASTIC LIMIT	LIQUID LIMIT	PLASTICITY INDEX	USC	GRAVEL	SAND	SILT	CLAY	SULPHATE CONTENT	COMPRESSION TEST	LAB VANE	POCKET PEN	DRY DENSITY
meters			mm	%	%	%	%		%	%	%	%	%	kPa	kPa	kPa	kg/m^3
1.52	AL22	SY	180	34.3	35.9	75.5	39.6	CH	0.0	4.7	95.3				115	100	
3.05	AL23	SY	280	32.6											155	110	1350
4.57	AL24	SY	340	35.9											130	115	1387
6.10	AL25	SY	445	34.0											135	130	
7.62	AL26	SY	360	29.8											140	130	
9.14	AL27	SY	530	29.4	21.9	38.4	16.4	CL	0.0	3.3	96.7				30	30	1490
10.67	AL28	SY	220	22.7	22.2	35.0	12.8	CL	0.0	7.7	92.3				35	40	1457
12.19	AL29	SY	280	16.5											180	165	
13.72	AL30	SY	470	17.3	16.9	34.7	17.8	CL	1.1	34.0	64.9			199@7%	260+	290	1885
15.24	AL31	SY	400	15.5	19.0	38.0	19.0	CL	0.6	38.2	61.2			257@4%	260+	290+	1891
16.76	AL32	SY	310	21.1	20.2	46.7	26.5	CL	1.8	25.9	72.3			333@14%	260+	290+	1838
18.29	AL33	SY	320	18.0										357@11%	260+	290+	
19.81	AL34	SY	260	16.4										327@12%	260+	290+	
21.34	AL35	SY	270	17.5											260+	280	
22.86	AL36	SY	NR														

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PROJECT	Saskatchewan Multi-Use Facility
LOCATION	Regina, Saskatchewan
PROJECT NO.	R4397

BORE HOLE NO.

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Clifton Associates Ltd. engineering science technology	**Appendix A**

Recommended Specifications For Granular Materials

1. Granular materials shall be composed of fragments of durable rock free from undesirable quantities of soft or flaky particles, topsoil, organic matter, clay or silt lumps, lumps of frozen granular soil, ice, snow or construction rubble.

2. The Pit Run Fill shall have a plasticity index less than 10 percent. The Crushed Base Course shall have a plasticity index less than 6 percent.

3. For Pit Run Sand, $\frac{D_{60}}{D_{10}} > 6$, and $1 < \frac{(D_{30})^2}{D_{10} x D_{60}} < 3$. For Pit Run Gravel, $\frac{D_{60}}{D_{10}} > 4$, and $1 < \frac{(D_{30})^2}{D_{10} x D_{60}} < 3$.

4. Granular materials shall be excavated, loaded, hauled, placed and levelled in such a manner to prevent contamination with undesirable materials described in Point 1 above and to prevent excessive segregation of coarse and fine particles.

5. Granular material shall conform to the following gradation specifications:

	Percent by Weight Passing U.S. Standard Sieve Series						
			Crushed Base Course				
Sieve	Pit Run Gravel Fill	Pit Run Sand Fill	32	33	34	35	36
50.0 mm	100						
25.0 mm	85 - 100		100				
18.0 mm	80 - 100		87 - 100	100	100	100	100
12.5 mm	70 - 100	100	79 - 93	81 - 100	91 - 100	81 - 100	91 - 100
5.0 mm	50 - 85	75 - 100	47 - 77	50 - 80	70 - 85	50 - 85	70 - 85
2.0 mm	35 - 75	50 - 90	29 - 56	32 - 52	45 - 65	32 - 65	45 - 70
900 µm	25 - 50	30 - 75	18 - 39	20 - 35	28 - 43	20 - 43	28 - 51
400 µm	15 - 35	15 - 50	13 - 26	15 - 25	20 - 30	15 - 30	20 - 35
160 µm	8 - 22	5 - 30	7 - 16	8 - 15	11 - 18	8 - 18	11 - 21
71 µm	0 - 13	0 - 15	6 - 11	7 - 10	8 - 12	7 - 12	8 - 13

Environmental Remediation

The following information provides a brief overview of the work completed to date to assess the level of contamination at Canadian Pacific Railway's (CP) downtown rail yards. This assessment forms part of the due diligence for the feasibility study and will help inform the feasibility results regarding the development of a multi-purpose entertainment facility to be located at CP's existing downtown location.

The results of this work are confidential and will not be released. As well, there is no assurance the land will be sold as there have been no decisions yet regarding the facility.

CIC engaged Clifton Associates Ltd. (Clifton) to undertake a Phase II environmental site assessment of the CP inter-modal facility. Subject to a non-disclosure agreement, CP provided Clifton with access to previous environmental monitoring information (Phase I environmental assessment and other data). CP also provided Clifton with access to the site for its assessment procedures including the drilling of test holes.

Clifton used CP's previously completed Phase I environmental assessment to identify procedures and requirements for the Phase II assessment. A Phase I assessment undertakes an initial review of site factors (e.g., land use history, surface and sub-soil types or location of water table) to provide an estimation of such things as the possibility of contamination, the type of contaminants and the potential extent of contamination. A Phase II environmental assessment seeks to identify and determine the extent of contaminants at a particular site and develop cost estimates for various levels of clean up which is also referred to as remediation.

Based on the Phase I assessment, Clifton identified test hole locations at the CP site. From various depths within the test holes, Clifton obtained soil and water samples for analysis. The results of this analysis help to quantify the nature and extent of contamination. Clifton determined that there is some contamination of a nature and level expected on an industrial site such as a rail yard including fuels such as gasoline and diesel, creosote leaching from treated rail ties and buried material such as pipe and concrete from previous structures.

Clifton's Phase II assessment also provided an estimate of cost for site remediation. Much of the contaminated material identified will require excavation and removal. A significant amount of the contaminated soil is situated within the proposed facility's footprint. Site preparation for the construction of the facility would already require the excavation of soil therefore the incremental cost of remediation would be for hauling contaminated material to off site locations. Remediation could include either hauling contaminated material to a site capable of cleaning the material (e.g. soil for reuse) or hauling contaminated material to a land fill site licensed to accept and store such material.