# GEOTECHNICAL INVESTIGATION
# PROPOSED FIREHALL REPLACEMENT & NEW OFFICE BUILDING
# GRASSLANDS NATIONAL PARK
# VAL MARIE, SASKATCHEWAN

Submitted to:

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# 1.0 INTRODUCTION

## 1.1 General

This report presents the results of a geotechnical investigation completed by Amec Foster Wheeler Environment and Infrastructure, a division of Amec Foster Wheeler Americas Limited (Amec Foster Wheeler) for a new firehall and office buildings to be constructed at the Grasslands National Park near the Town of Val Marie, Saskatchewan.

## 1.2 Terms of Reference

The scope of this geotechnical investigation consisted of:

- Assessment of subsurface conditions at eleven (11) test hole locations; and
- Provision of geotechnical recommendations for:
    - Site preparation; and
    - Design and construction of the foundation, grade supported concrete slabs and pavement structures.

## 1.3 Project Description

It is understood that the proposed firehall and office buildings, Phases 1 and 2, respectively will be constructed on a 32 hectare property situated within NE 30-03-13 W3M near the Town of Val Marie, Saskatchewan. The geographic location for the project and the layout of the site are shown on Figures 1 and 2, respectively, in Appendix A.

A review of the scope of work outlined in a Public Works and Government Services Canada (PWGSC) document titled “Geotechnical Services, Grasslands National Park Firehall Replacement” dated October 07, 2015, and the proposed schematic site plan and office building layout, revealed the following project details:

- The proposed firehall and office buildings will be steel framed structures.
- The office building will have dimensions of approximately 30.3 m by 25.9 m.

Other design details about the proposed development, such as expected foundation loads, were not known to Amec Foster Wheeler. Photographs taken during Amec Foster Wheeler’s field investigation are included in Appendix B.

# 2.0 EXTENT OF GEOTECHNICAL INVESTIGATION

## 2.1 Scope of Field Investigation

Amec Foster Wheeler's field investigation was conducted on 16 and 17 November 2015 and consisted of eleven (11) test holes, numbered as TH15-01 to TH15-11, that were drilled at locations shown on Figure 2 in Appendix A. The test holes were drilled using a truck mounted rig that was equipped with 150 mm diameter solid stem augers.

Amec Foster Wheeler field personnel visually classified soil samples during test drilling according to the Modified Unified Soil Classification System. Groundwater seepage and sloughing conditions encountered at the time of the investigation were also recorded.

Disturbed soil samples were taken at regular depth intervals from the augers or the split spoon sampler. Standard Penetration Tests (SPTs) were also performed.

The test holes were left open for approximately five minutes after the completion of drilling in order to monitor short term groundwater levels and sloughing conditions. The test holes were then backfilled with a combination of compacted soil cuttings and a bentonite plug. Excess auger cuttings were spread on the ground surface near the test holes. Specific information pertaining to backfill details is shown on the test hole logs in Appendix A.

All soil samples obtained during the field investigation were labelled, sealed in plastic bags to minimize moisture loss, and transported to Amec Foster Wheeler's Regina office for further visual examination and laboratory testing.

Test hole elevations were measured relative to a selected benchmark. The benchmark chosen was the surface of a concrete slab pad located as shown on Figure 2 in Appendix A. The benchmark was arbitrarily assigned an assumed relative elevation of 100.0 m for purposes of this investigation.

The ground surface was observed to be slightly undulating to relatively flat. The maximum surface relief between test hole locations was measured to be approximately 1.1 m. Test hole depths, locations and relative elevations are summarized in Table 1.

**TABLE 1**
**TEST HOLE LOCATIONS AND DRILLED DEPTHS**

| Test Hole No. | Drilled Depth Below the Existing Ground Surface (m) | Universal Transverse Mercator (UTM) Grid Coordinates (m) | | Relative Ground Surface Elevation (m) |
|---|---|---|---|---|
| | | Easting | Northing | |
| TH15-01 | 11.1 | 0301009 | 5457911 | 100.8 |
| TH15-02 | 11.1 | 0301032 | 5457922 | 100.2 |
| TH15-03 | 11.1 | 0301049 | 5457924 | 100.1 |
| TH15-04 | 11.1 | 0301032 | 5457935 | 100.7 |
| TH15-05 | 10.7 | 0301048 | 5457911 | 99.8 |
| TH15-06 | 11.1 | 0301059 | 5457930 | 100.1 |
| TH15-07 | 11.1 | 0301066 | 5457914 | 99.7 |
| TH15-08 | 3.0 | 0301056 | 5457934 | 100.4 |
| TH15-09 | 3.0 | 0301031 | 5457930 | 100.5 |
| TH15-10 | 3.0 | 0301053 | 5457916 | 99.8 |
| TH15-11 | 3.0 | 0301014 | 5457916 | 99.7 |

### 2.2 Laboratory Testing

Visual classification and moisture content tests were performed on all soil samples. Other laboratory tests consisted of two (2) hydrometer grain size distribution tests.

Laboratory test results and other relevant subsurface information are summarized on the test hole logs, Figures 3 to 13, included in Appendix A. Hydrometer grain size distribution graphs are included in Appendix C.

## 3.0 SUBSURFACE CONDITIONS

### 3.1 Soil Profile and Properties

#### 3.1.1 Soil Profile

The following information pertains to subsurface soils and conditions encountered at eleven (11) test holes drilled at the site by Amec Foster Wheeler. Test hole logs describing the soils encountered are shown on Figures 3 to 13 in Appendix A. It should be noted that subsurface soils and conditions at other locations might differ from those encountered at the selected test hole locations.

A layer of sand topsoil, 150 mm to 225 mm in thickness, was encountered at the surface of test holes TH15-01, TH15-07 and TH15-11. A layer of sand and gravel fill, 150 mm to 200 mm in thickness, was encountered at the surface of the remaining test holes. Possible sand fill was also present below the sand and gravel fill in test holes TH15-06, TH15-07 and TH15-08 to a maximum depth of 1.3 m.

Naturally deposited sand was encountered below the topsoil and fill which extended to an average depth of 9.5 m in most test holes and, in turn, was underlain by glacial clay till to the full depth of exploration. Silt layers, 1.0 m to 1.2 m in thickness, were also encountered at various depths in test holes TH15-02, TH15-03 and TH15-07.

Amec Foster Wheeler's test hole logs, shown as Figures 3 to 13 in Appendix A, provide more detailed information regarding depths and thicknesses of strata encountered.

### 3.1.2 General Soil Properties

**Sand Topsoil**

The sand topsoil was typically silty, damp, poorly graded, fine to medium grained and loose.

**Sand and Gravel Fill**

The sand and gravel fill was damp, poorly graded, fine to medium grained and loose.

**Sand**

The sand typically contained trace clay. It also contained cobbles and gravel below a depth of 8.5 m at most test hole locations.

The sand was poorly graded, fine to medium grained, typically loose becoming medium dense below 7.5 m. The sand was, for the most part, moist to very moist near surface becoming wet below a depth of approximately 0.9 to 1.5 m. Exceptions were as follow:

- It was wet near surface at test hole TH 15-01; and
- It was damp near surface becoming wet with increasing depth at test hole TH 15-09.

The grain size distribution graphs included in Appendix C indicate that the sand had the following soil compositions, by dry soil weight:

| | |
|---|---|
| Sand - | 68% to 73% (Average 70%) |
| Silt - | 17% to 21% (Average 19%) |
| Clay - | 10% to 11% (Average 11%) |

**Silt**

The silt contained sand and trace clay. It was typically wet, firm, and low plastic.

**Clay (Till)**

The clay till was silty and contained some sand and gravel. It was moist, stiff to hard, medium plastic and grey.

### 3.2 Seepage and Sloughing Conditions

Significant seepage and sloughing conditions were detected during drilling. Seepage and sloughing conditions encountered during test drilling are shown on the test hole logs provided in Appendix A. Water level measurements are also summarized in Table 1. More severe sloughing conditions would be expected for large diameter drill holes and excavations at this site.

**TABLE 2**
**RECORDED GROUNDWATER AND SLOUGH DEPTHS**

| Test Hole | Relative Surface Elevation (m) | Drilling Depth (m) | Depth to Top of Water Immediately after Drilling | Depth to Top of Slough (m) Immediately after Drilling |
|---|---|---|---|---|
| TH15-01 | 100.84 | 11.1 | 0.9 | 2.7 |
| TH15-02 | 100.18 | 11.1 | - | 4.3 |
| TH15-03 | 100.13 | 11.1 | - | 4.6 |
| TH15-04 | 100.69 | 11.1 | - | 5.5 |
| TH15-05 | 99.85 | 10.7 | - | 3.0 |
| TH15-06 | 100.10 | 11.1 | 4.0 | 6.7 |
| TH15-07 | 99.67 | 11.1 | - | - |
| TH15-08 | 100.43 | 3.0 | - | - |
| TH15-09 | 100.46 | 3.0 | - | - |
| TH15-10 | 99.80 | 3.0 | - | - |
| TH15-11 | 99.69 | 3.0 | - | - |

# 4.0 SITE PREPARATION RECOMMENDATIONS

## 4.1 Stripping

Topsoil, vegetation and other deleterious materials positioned below the proposed building footprints and grade supported structures should be removed in accordance with the following:

- The near surface subgrade soil at this site consists of sand. This type of soil will be significantly sensitive to construction disturbance and, as such, precautionary steps should be exercised to minimize disturbance which could negatively affect the performance of grade supported structures. Use of light, track mounted equipment and/or trackhoes and preventing concentrated heavy truck traffic from travelling across the stripping areas are examples.

- Excavate and remove all deleterious materials including topsoil and vegetation, if present. The topsoil thickness encountered by Amec Foster Wheeler at the test hole locations varied from approximately 150 mm to 225 mm, but it is considered possible that the topsoil thickness at other locations may be different, potentially exceeding 225 mm. A minimum stripping depth of 25 mm to 50 mm below the underside of the topsoil is recommended.

- Soil containing major roots and/or other deleterious materials that may be present below the initial stripping depth should be completely removed and wasted.

- The stripping width should be extended to at least 1.5 m beyond the proposed building footprints.

- Soils excavated which contain topsoil, vegetation, debris or other deleterious materials should not be re-used as fill.

- All stripping work should be monitored and approved by Amec Foster Wheeler's representative on a full time basis during construction. Approval for increased stripping depths should only be provided by qualified personnel.

## 4.2 Grading and Drainage Guidelines

Typical features that should be included in the grading and drainage design plans are:

- Raising site grades to above adjacent lands using competent fills;
- Cut depths should be minimized to the extent practicable to avoid the shallow water table;
- Removal of weak soils and deleterious materials such as poorly consolidated fills, etc.;
- Providing adequate finished cross slopes, crowns, ditches and swales; and
- Installing culverts at strategic drainage locations, if appropriate and acceptable.

## 4.3 Fill Selection

The existing sand is not considered suitable as borrow material. It was poorly graded, fine to medium grained and moist to very moist near ground surface at many locations. It will likely be difficult to adequately compact and it may also become unstable due to construction activities. As such, Amec Foster Wheeler recommends that a well graded gravel base, subbase or pitrun gravel material be used instead.

“Fillcrete” can be particularly advantageous for cold weather applications and similar situations when quick completion of construction is necessary, if volumes are relatively low.

Gravel fills should be free of frozen soil, snow and ice, organic materials, contamination and deleterious construction materials. All fill should also be placed and compacted in a manner that is consistent with performance requirements and project specifications.

Table 3 summarizes Saskatchewan Ministry of Highways and Infrastructure (SMHI) standard gradation specifications for Type 32 and 33 base course and Type 8 subbase course aggregates which are generally available throughout Saskatchewan. Amec Foster Wheeler should be given the opportunity to assess and approve all imported aggregates.

**TABLE 3**
**GRADATION REQUIREMENTS FOR BASE AND SUBBASE AGGREGATES**

| Metric Sieve Designation | Percent by Dry Weight Passing Sieve Size | | |
|---|---|---|---|
| | SMHI Base Course Type | | SMHI Subbase Type 8 |
| | 32 | 33 | |
| 50 mm | - | - | 100 |
| 25 mm | 100 | 100 | - |
| 18 mm | 87 - 100 | 100 | - |
| 12.5 mm | 72 - 93 | 75 - 100 | - |
| 5 mm | 45 - 77 | 50 - 75 | - |
| 2 mm | 29 - 56 | 32 - 52 | 0 - 90 |
| 900 µm | 18 - 39 | 20 - 35 | - |
| 400 µm | 13 - 26 | 15 - 25 | 0 - 60 |
| 160 µm | 7 - 16 | 8 - 15 | 0 - 25 |
| 71 µm | 6 - 11 | 6-11 | 0 - 15 |

**Notes to Table 3:**

1. Granular base course should be composed of sound, durable particles of crushed rock, stone, gravel, sand and fine soil. It should not contain thin elongated particles, sods, topsoil, roots or plants.

2. Type 32 and 33 base aggregates have the following additional SMHI material compliance requirements:

   - Material passing the 0.4 mm sieve: 0 ≤ Plasticity Index ≤ 6
   - 50% of the total aggregate should be fractured (1 face)
   - Contain less than 5% lightweight particles
   - Can have up to 3% total oversize provided the maximum aggregate size does not exceed 22.4 mm

3. Type 8 subbase should have the following additional SMHI material compliance requirements:

   - Material passing the 0.4 mm sieve: 0 ≤ Plasticity Index ≤ 6
   - 50% of the total aggregate should be fractured (1 face)
   - Can have up to 3% total oversize provided the maximum aggregate size does not exceed 63 mm

**TABLE 4**
**GRADATION REQUIREMENTS FOR PITRUN AGGREGATE**

| Sieve Size (mm) | Percent (%) By Dry Weight Passing Sieve |
|---|---|
| 75 | 100 |
| 4.75 | 40 – 80 |
| 75 µm | 8 – 18 |

### 4.4 Proof Rolling

After stripping and initial subgrade excavation works are completed, it is recommended that heavy proof rolling be applied in all new slab-on-grade and pavement areas as a precautionary measure to help delineate existing weak fills that may be present. Weak subgrades that are identified as a result of the proof rolling should be evaluated by the geotechnical consultant and a remedial plan developed. For this specific site and given the shallow groundwater table, a traditional method of additional excavation and fill replacement is not advised. Rather, a geotextile approach should be considered, provided the deflection during proof rolling is not unacceptable. Unacceptable deflection should be defined as 20 mm or greater. In the event that unacceptable deflection occurs over a relatively large area, a revised approach to the slab and foundation design would likely be necessary, potentially consisting of a structurally supported floor slab system and with corresponding increased pile numbers and pile loads. Where deflections during proof rolling range from 10 mm to 20 mm, application of a geogrid reinforcement material is expected to be appropriate and should be implemented in accordance with recommendations from the geotechnical consultant at the time of construction. If the deflection during proof rolling is less than 10 mm, then no additional measures are considered necessary.

### 4.5 Fill Placement and Compaction

Fill placement should consist of spreading fill materials in controlled, uniform horizontal lifts not exceeding the lesser of the values outlined below, or the ability of the compaction equipment to attain minimum specified density requirements.

It is recommended that all granular fills placed below a depth of 0.3 m from the final subgrade surface be compacted to a minimum *average* density of 98 % of Standard Proctor Maximum Dry Density (SPMDD), with a minimum *individual* density test of not less than 95 % of SPMDD. All granular fills placed above this depth should be compacted to not less than 100 % of SPMDD.

It is emphasized that placement of fills in equal lifts of 150 mm or less and obtaining uniform densities, both vertically and horizontally across the site, would be critical to reducing differential settlements.

Qualified geotechnical personnel should monitor the quality and placement of fill soils and the compaction of the fill should be monitored by field density testing at regular frequencies. The density of each compacted lift should be tested prior to placing the next lift to confirm that adequate compaction has been achieved.

If the material fails to meet the required density, then the material must be reworked or replaced and construction methods altered as necessary to obtain the required density.

## 4.6 Potential Magnitude of Fill Settlements

The quality of the fill material used and the fill compaction standards considered necessary from an engineering perspective is dependent on allowable settlements and the level of risk that the Owner is prepared to accept. The extent of fill settlement will be dependent on the applied loads, the type and quality of fill selected, and the density to which the fill is compacted.

Past experience has shown that the following approximate order of settlements may be expected for engineered fills under nominal loads, assuming that they have been compacted in compliance with Table 5:

- Native sand and pitrun gravel — 1.0 to 2.0 % X $H_f$
- Base and subbase aggregates — 0.5 to 1.0 % X $H_f$
- 'Fillcrete' — < 0.5 % X $H_f$

Where: $H_f$ = thickness for each fill type, accumulative calculations required if more than one fill type

## 4.7 Subgrade Preparation

As previously noted in Subsection 4.1, special precautions will be needed to reduce excessive rutting and subgrade failures during construction because the sand subgrade is expected to be very sensitive to disturbance from construction traffic and will especially become weak if it becomes saturated, such as could occur due to flooding, after significant rainfall or if excessive water is applied during compaction. It is to be noted that this may become particularly problematic due to the existing shallow groundwater table at this site.

Precautionary measures to prevent unacceptable subgrade disturbance would typically include the use of light tracked equipment, restricting traffic over weak subgrades, excavators operating from the edge of excavations, using excavators equipped with a smooth bladed bucket, etc.

The exposed subgrade should be kept from freezing and inundation throughout the duration of construction.

The subgrade should be prepared in accordance with the following minimum recommendations:

- Stripping should be performed in accordance with Subsection 4.1 of this report.
- Further excavate the subgrade to the required design elevations if required. Deleterious materials that are exposed at the bottom of the design elevation should be over-excavated and wasted.
- Excavation work should be undertaken using a backhoe or trackhoe equipped with a finishing bucket (i.e. without ripper teeth). Once the design subgrade elevation has been reached, the subgrade should be evaluated by an Amec Foster Wheeler field representative to confirm that the soils are consistent with those observed at the test hole locations.
- It is considered possible that unconsolidated, weak soils and/or deleterious materials may be exposed below the initial excavation depth. Weak areas that are exposed during construction should be assessed by the geotechnical consultant and dealt with on an as-required basis.
- Where stable subgrade conditions are present, the subgrade should be scarified to a minimum depth of 150 mm and then the subgrade should be uniformly compacted such that the average of all density tests is not less than 98 percent of SPMDD and locations where individual test densities are less than 95 percent of SPMDD should not be accepted.
- Replacement fills used to raise grade elevations should be **uniformly** in compliance with Subsection 4.5 of this report.
- Compaction should be undertaken using a smooth drum roller. Fill placement should be observed in the field by an Amec Foster Wheeler representative on a full time basis in order to identify areas that may require special attention and remedial construction measures. This procedure is considered particularly important for this project because it should help detect weak spots and, through remediation on an as-required basis, it should help reduce long term settlements.
- Notwithstanding the above, the upper 150 mm (minimum) of soil immediately underlying all concrete slabs should conform to a Saskatchewan Ministry of Highways and Infrastructure (SMHI) Type 32 or 33 gravel base course. The gravel base course should be uniformly compacted to a minimum density of 100% of SPMDD.

- The finished surface of the subgrade should be crowned near the centre of concrete slab and pavement areas and sloped to promote subdrainage away from these structures. A minimum cross slope gradient of two (2) to three (3) percent is recommended. If a centralized crown with cross slope of 2 to 3 percent is impractical, an alternative approach should be developed to promote subdrainage.

- The finished surface below slabs-on-grade should be protected against disturbance, excessive wetting or drying, and freezing. It is imperative that the schedule for casting of concrete slabs allow for concrete placement shortly after the base has been constructed and approved.

- Stringent quality controls (i.e. monitoring, materials selection and testing, approval, etc.) should be provided by a qualified and knowledgeable materials testing agency, working directly for *Public Works and Government Services Canada* and under the close supervision of Amec Foster Wheeler, on a full time basis during construction.

## 4.8 Excavations and Backfilling

### 4.8.1 General Overview

Details regarding excavations were not available at the time of writing.

As a minimum, all excavations should comply with the requirements of Saskatchewan Occupational Health and Safety guidelines. The excavation work should be undertaken by experienced contractors and should also be closely supervised by knowledgeable safety personnel.

The water table at this site was found to be relatively shallow at the time of Amec Foster Wheeler's field investigation, varying from as shallow as 0.9 m to as deep as 1.5 m. It is also expected to be highly reactive to time of year and precipitation events, particularly rising during periods of significant rainfall and snow melt, and it may be shallower than shown on the test hole logs at the time of construction. To the extent that is possible and practicable, excavation depths should be minimized. The actual water table depth at the time of construction should be confirmed by means of further exploration and monitoring well installations. Where excavations to below the confirmed water table depth (i.e. at the time of construction) are essential, special measures to deal with the water table and excavations will be required. Given the porosity of the sand, installation of interior sump pumps to dewater excavations may have limited success because of the expected high rate of water ingress and large volumes of water to be removed. Thus, a series of de-watering wells placed around the perimeter of excavations may become necessary.

Excavations, which experience unusual difficulties, should be brought to the immediate attention of Amec Foster Wheeler so that engineered solutions to the problem can be appropriately determined.

**Notes:**

1. In order to reduce the impact of long term trench subsidence on new, grade supported structures such as concrete slabs-on-grade, sidewalks and pavements, the ideal approach to construction would be to have service trenches excavated and backfilled as the first phase of construction prior to other works. In this manner, it will take advantage of additional consolidation that could occur as a result of construction traffic and subgrade preparation compaction. It will also reduce the effects of disturbance to areas already previously prepared.

2. Placement of horizontal insulation in accordance with manufacturer recommendations (e.g. Dow Chemical) could be considered above and adjacent to underground service trenches, where reduced excavation depths are deemed necessary to avoid the shallow water table and to help prevent freezing of sewer and water pipes.

### 4.8.2 Excavation Slopes and Shoring Requirements

The naturally deposited sand *above the water table* should be treated as a Type IV soil in accordance with Saskatchewan Occupational Health and Safety regulations. Short term (e.g. 30 days or less), open excavations in this type of soil should have excavation slopes no steeper than 3H:1V or 19 degrees from the horizontal.

Open excavations that extend to *below the water table* could be subject to collapse as a result of soil piping failures and unstable slopes. Consequently, under any circumstances, excavations to below the water table depth should not be undertaken unless the excavations are adequately shored and dewatered. Regardless of the water table depth, trench boxes or temporary shoring be used wherever workers operate inside trenches that are deeper than 1.2 m. All shoring systems should be engineered and approved by a qualified Professional Engineer.

If open shallow cut excavations above the water table are allowed to remain open for prolonged periods of time of if cut slopes extend through weak soils, side slopes flatter than 3H:1V may be required. Exposed slopes may experience significant erosion unless adequately protected. All open cuts should be examined and assessed by qualified geotechnical personnel and they should be protected against destabilization due to erosion, etc.

Stockpiles of construction materials or other surcharge loads (e.g. equipment, wheel loads, etc.) should not encroach closer than the horizontal equivalent of the excavation depth from the top edge of any excavation.

### 4.8.3 Backfill Compaction

Fill used to backfill open excavations should be compacted to a standard that is in keeping with the performance requirements for that area. If the area is not to be used for an end use (such as staging areas, etc.), a minimum compaction standard and common fill materials could be considered.

Where fill subsidence is to be minimized, an increased compaction density and select fill materials should be specified. The amount of future subsidence that will occur in trench and excavation areas cannot be accurately predicted because of the many variables involved (e.g. time, surface traffic conditions, type of material used, compactive effort, quality controls, width and depth of excavation, etc.). Suggested guidelines for fill settlement expectations are provided in Subsection 4.5 of this report.

In areas where fill subsidence is to be controlled, the fill should be placed in maximum 150 mm thick lifts and uniformly compacted as previously outlined in this report.

## 4.9 Cold Weather Considerations

Special considerations during cold weather conditions are as follows:

(a) Amec Foster Wheeler should be consulted before construction proceeds.

(b) Fill placement and compaction during extreme cold weather conditions incorporates a very high risk of unacceptable fill performance, particularly with regard to consolidation and differential settlements. As such, subgrade work under cold weather conditions is generally not recommended except under severely restricted criteria and construction constraints. Even gravels, which give an appearance of being not affected by freezing conditions, can contain ice crystals which limit the achievable degree of compaction. A high degree of fill density and reliability can only be achieved when the fill soils are unfrozen and remain unfrozen during the entire compaction process. This may require that the compaction area is hoarded and heated and/or that the fills are preheated.

(c) When construction must proceed in freezing conditions, past experience has occasionally shown that acceptable compaction and fill density can be achieved during cold weather conditions, provided the fill placed is kept unfrozen and temperatures during fill placement are not colder than -5°C to -10°C. This type of specialized approach requires the full co-operation of the contractor to help expedite completion of the work to the maximum extent possible. Shift work may be required to ensure that the fill is placed and compacted on a continuous basis, not allowing the underlying soil the opportunity to freeze before the next lift of fill is placed and compacted. Experience has also shown that fill with a moisture content near its OMC is much more favourable for expeditious compaction than fill that is wet of its OMC.

(d) In most cases when extreme cold temperatures prevail or may be forecasted, heating of the fill soils, and hoarding and heating of the fill placement area will be necessary to achieve the required degree of compaction. It should also be noted that unless the fill placement area is hoarded and heated, the addition of water to the fill to promote its compaction (in the case of dry soils) would not be possible at freezing temperatures. If mass heating of fill stockpiles is to be considered, the application of heat will need to be controlled so the fill does not become overly dry, which would then necessitate the undesirable addition of water to facilitate its compaction.

(e) Non-compliance to the above guidelines may result in significant fill settlement.

(f) Amec Foster Wheeler suggests that use of ‘fillcrete’ be considered for some applications as an alternative to compacted fills if and when cold weather conditions become a concern.

## 5.0 EXPECTED FROST PENETRATION DEPTH

The theoretical maximum design freezing depth at this site was calculated to be in the approximate order of 2.7 m. It should be noted that the above calculated freezing depth has been based on emperical equations, soil thermal data and graphs presented in Subsection 13.4 of the Canadian Foundation Engineering Manual (4th Edition, 2006) and an assumed design freezing index of 2,035 °C – days for this area (McCormick, Environment Canada, et al).

Frost penetration depths near unheated steel and/or concrete structures, which can act as thermal conductors, could be greater than the above calculated value and should be individually assessed, as required, based on the exposure conditions and other pertinent parameters.

The most common method to reduce freezing depth is to use engineered horizontal insulation, such as Dow Chemical's HI™ Styrofoam. Insulation thickness and horizontal dimension requirements will be dependent on several factors such as thermal properties of the manufacturer's insulation, design freezing index (see above), thermal transmissivity of a structure, average ambient air temperatures within a structure, soil types and densities, etc. As a minimum, design and construction details should comply with manufacturer specifications.

## 6.0 FOUNDATION RECOMMENDATIONS

### 6.1 Foundation Selection Considerations

Amec Foster Wheeler is unaware of the expected maximum unfactored foundation loads for the proposed buildings. In consideration of the soil conditions encountered and based on Amec Foster Wheeler's previous experience, a piled foundation is expected to be the most economical and feasible foundation system and is recommended.

Continuous flight auger piles (CFAs) and helical screw piles (HSPs) are expected to be best suited for the subsurface conditions encountered and for support of the relatively light to moderate loads expected. Driven steel pipe piles are also considered to be technically well suited, but are not expected to be as economical as CFAs and HSPs.

Geotechnical recommendations for the design and construction of CFAs, driven steel pipe piles and HSPs are included in the following subsections this report. Amec Foster Wheeler should be consulted if recommendations for design and construction of other foundation systems are required.

### 6.2 Design Approach

Geotechnical recommendations provided below are based on Limit States Design (LSD) which is a design method used in structural engineering. A limit state is a condition of a structure beyond which it no longer fulfills the relevant design criteria. The condition may refer to a degree of loading or other actions on the structure, while the criteria refer to structural integrity, fitness for use, durability or other design requirements. A structure designed by LSD is proportioned to sustain all actions likely to occur during its design life, and to remain fit for use, with an appropriate level of reliability for each limit state. LSD requires the structure to satisfy two principal criteria: the <u>ultimate limit</u> state (ULS) <u>and</u> the serviceability limit state (SLS).

Ultimate limit state (ULS) with respect to soils and foundations is reached when the ultimate load carrying capacity of the soil is exceeded (due to compression, uplift, sliding or overturning), or when soil deformation causes an ULS in the structure without soil failure, or when overall stability is lost. A structure is deemed to satisfy the ULS criteria if all *factored loads* ($\Sigma\alpha\cdot P$) are less than the *factored resistances* ($\Sigma Ø\cdot R$).

To satisfy the serviceability limit state (SLS) criteria, a structure must remain functional for its intended use. A structure is deemed to satisfy the SLS when the elements do not deflect by more than limits provided in the National Building Code of Canada or when other restrictions, such as vibrations, need to be considered.

With respect to foundations used for this project, a SLS is assumed to be present when one or both of the following foundation movements occur:

- Vertical = 25 mm
- Horizontal = 8 mm

If the above SLS criteria are incorrect, Amec Foster Wheeler should be advised so that adjustments can be made to the recommendations contained herein.

### 6.3 Frost Action and Associated Forces (Piled Foundations)

Frost action and associated forces that can be applied to a structure by the soil are a function of many variables and generally cannot be reliably predicted without comprehensive analyses, research and an in-depth understanding of the site conditions and structure properties, which include factors other than just soil properties.

Piles exposed to frozen soil should be designed to accommodate potential uplift forces. Upward acting, frost adfreezing stresses acting on vertical surfaces in contact with the soil (such as pile perimeters) have been measured through previous research and testing. An adfreeze uplift stress of 65 kPa may be assumed acting along the pile shaft perimeter within the depth of freezing. The maximum design freezing depth may be taken as 1.4 m for perimeter piles supporting heated structures, and 2.7 m for piles supporting unheated structures.

### 6.4 General Recommendations

The following comments and recommendations apply to the design and construction of CFAs, driven steel pipe piles and HSPs at this site:

- The recommended resistance values for piles assume that existing or new fills will not exceed 1.0 m in depth. Amec Foster Wheeler should be consulted if this not the case so that allowances for negative skin friction acting on the piles may be provided to the designer.

- Adequate drainage should be provided to direct water away from foundations.

- Piles in groups should be installed with center-to-center spacing's equal to the greater of:
  - one (1) meter;
  - three (3) times the greatest adjacent pile diameter for CFAs and driven steel pipe piles; and
  - two (2) times the greatest adjacent helix diameter for HSPs.

- Geotechnical resistance factors, Φ, of 0.4 and 0.3 should be applied to the ultimate skin friction values for compression and uplift resistance, respectively, to obtain the factored resistance at the ultimate limit state.

- With regard to the serviceability limit state (SLS), pile settlements will be a function of structural loads, pile length and diameter, number of piles and spacings between piles and can be evaluated in detail once the foundation design details are known. Settlement for a small group of piles (e.g. 4 piles per group) at spacings of not less than the recommended values noted above would be expected to be less than 25 mm, when operating at the factored resistance. As such, no reduction in the factored resistance values for SLS is required for this type of design. Amec Foster Wheeler should be advised if more than 4 piles will be installed.

- Pile installations should be monitored by the geotechnical consultant on a full time basis during construction, acting independently of the contractor. In the case of driven steel pipe piles, the geotechnical consultant should determine pile capacity, based on the driving energies applied and the performance of the pile during driving.

### 6.5 Continuous Flight Augured Piles (CFAs)

CFAs may be designed on the basis of skin friction only in accordance with Limits States design approach and the recommended unfactored ultimate skin friction values listed in Table 5.

**TABLE 5**
**UNFACTORED ULTIMATE SKIN FRICTION VALUES**
**FOR STATICALLY LOADED, CFAs**

| Depth (m) Below the Existing Ground Surface | Unfactored Ultimate Skin Friction (kPa) |
|---|---|
| If total fill (existing and/or new) thickness ≤ 1 m, 0.0 to 2.0 | 0 |
| If total fill (existing and/or new) thickness > 1 m, 0.0 to underside of fill | -30 |
| 2.0 or from underside of fill, whichever is deeper, to 8.0 | 2.0 X Z |
| 8.0 to 10.0 | 3.0 X Z |
| > 10.0 | 90 |

**Where:** Z = depth (m, below the final ground surface)

**Notes to Table 5:**

- CFAs should be adequately reinforced over their full length or to such a length deemed adequate by the structural engineer, ensuring that applied pile loads can be adequately transferred from the pile to the soil in both the vertical and lateral directions.

- Approved plastic spacers should be placed at minimum length intervals of 3 m along the vertical reinforcing steel for CFAs to ensure that the steel does not come in contact with the adjacent soil during placement.

- CFAs should be extended to a minimum embedment lengths of 7.5 m and 9.5 below the final ground surface in heated and unheated areas, respectively.

## 6.6 Driven Open End Steel Pipe Piles

Driven open end steel pipe piles may be designed on the basis of skin friction resistance only in accordance with the Limit States design approach and with the recommended unfactored ultimate skin friction values listed in Table 6, applied to the exterior portion of the pipe.

**TABLE 6**
**UNFACTORED ULTIMATE SKIN FRICTION VALUES**
**FOR STATICALLY LOADED, DRIVEN STEEL PIPE PILES**

| Depth Below the Existing Ground Surface (m) | Unfactored Ultimate Skin Friction, $f_{ult}$ (kPa) |
|---|---|
| If total fill (existing and/or new) thickness ≤ 1 m, 0.0 to 2.0 | 0 |
| If total fill (existing and/or new) thickness > 1 m, 0.0 to underside of fill | -20 |
| 2.0 or from underside of fill, whichever is deeper, to 8.0 | 1.4 X Z |
| 8.0 to 10.0 | 2.0 X Z |
| > 10.0 | 60 |

**Notes to Table 6:**

1. The values provided in Table 6 should be applied to the exterior perimeter of steel pipe piles (π X pipe diameter)

2. The ultimate unfactored uplift resistance of steel pipe piles may be determined by assuming skin friction resistance below the maximum depth of frost penetration only.

3. Driven pipe piles should be extended to a minimum embedment lengths of 7.5 m and 9.5 below the final ground surface in heated and unheated areas, respectively.

4. Steel pipe piles that encounter cobbles or boulders may become damaged, either at the pile tip or pile head, and unsuitable for support. If damage occurs, the damaged pile should be replaced by one or more other piles, as considered necessary by the structural engineer.

5. Driven steel pipe piles must be designed to withstand maximum design loads and driving forces during installation. Driving energies should not exceed 600 Joules per $cm^2$ of cross sectional area to prevent pile damage.

6. The hammer used should have a mass that is a minimum of three (3) times the mass of the pile.

7. Piles should be driven continuously to the required design lengths, once driving is initiated.

8. Because the subsurface soils could vary with depth and location, pile driving conditions and pile capacities can also vary. Estimates of achieved pile capacities and trends of pile capacities across the site should be determined by Amec Foster Wheeler on an on-going basis during pile installation, acting independently of the Contractor. Adjustments to pile installation procedures may be required.

9. The sequence of pile installations should be reviewed and carefully selected in order to prevent a build-up of excessive pore pressures in the soil and, correspondingly, to minimize heave and/or lateral displacement of adjacent piles during driving. Amec Foster Wheeler generally recommends that alternating the sequence of piles be used for this purpose, but it is also acknowledged that space and access restrictions may make this impractical. Surveyed elevations and locations of adjacent driven piles should be obtained by the contractor and reported to the geotechnical consultant on a regular basis so that the extent of pile movements can be properly evaluated. Piles heaved in excess of 6mm must be re-driven. If excessive movements occur, a number of remedial measures may be required, possibly including:

   a. Reduced driving energies;
   b. Pre-boring;
   c. Delayed driving; and/or
   d. Changes to the sequence of pile driving.

10. Amec Foster Wheeler's recommendations have been prepared based on the following assumptions:

   a. Only <u>new</u> steel will be used;
   b. The steel will have a minimum wall thickness of 10 mm; and
   c. The steel will have a minimum yield stress of 310 MPa.

11. Practical refusal for driven steel piles should be defined as a maximum pile penetration of 25 mm for 10 blows for a hammer operating at a maximum allowable driving energy of 600 Joules per $cm^2$ of steel cross sectional area. Piles reaching this termination criteria may be assumed to have a maximum unfactored ultimate compressive capacity determined by multiplying the steel cross sectional area times 70% of the maximum ultimate fiber stress in the steel, which is assumed to be fy (where fy is the yield strength of steel), provided a WEAP analysis is conducted beforehand and the delivered energies are confirmed in the field by Amec Foster Wheeler during construction.

12. Although not expected for this project, if refusal conditions are encountered at embedment lengths that are less than the recommended minimum pile length stated in Note (3) above, Amec Foster Wheeler must be consulted before further driving proceeds.

13. Pre-boring of driven steel pipe piles would be required if the ground is frozen at the time of construction. The pre-bore diameter and depth should not exceed B – 25 mm (where, B= pile diameter, mm) and 1.5 m, respectively.

14. Selective re-tapping of piles, after a minimum lapse period of 1 to 2 days, may be requested by the geotechnical consultant to allow re-assessment of pile capacities.

## 6.7 Helical Screw Piles

### 6.7.1 Unfactored Ultimate Resistance

Single helixes and the uppermost helix of multiple helix piles should be extended to a minimum depth 4.0 m below the final ground surface. HSPs extended to this depth may be designed on the basis of end bearing only in accordance with Limits States design approach and the recommended methodology presented below.

HSPs are also well suited to resisting tensile loads. The uplift resistance of a screw pile can be considered as the “pullout” resistance of a cylindrical mass of soil projected above the circumference of the helix. The resistance will include the shear forces in the soil as the pile is lifted and the combined effective weight of the pile and soil above the helix. For sustained load conditions, the approximate unfactored ultimate uplift resistance of a screw pile may be determined by the recommended methodology presented below.

**For a Single Pile Helix:**

**Ultimate Compressive Load Capacity**

$$Q_c = \gamma' \cdot [A_b \cdot H \cdot N_q + \tfrac{1}{2}\pi \cdot d \cdot H_{eff}^2 K_s \tan Ø]$$

**Where:**

| | | |
|---|---|---|
| $Q_c$ | = | ultimate compressive load capacity (kN) |
| $\gamma'$ | = | average effective unit weight of soil<br>assume unit weight = 9 kN/m$^3$ (submerged) |
| $A_b$ | = | total area of helix (m$^2$) at the bottom = $\pi D^2/4$ |
| d | = | diameter of the shaft (m) |
| D | = | diameter of helix (m) |
| H | = | depth to helix (m) |
| $H_{eff}$ | = | effective depth to helix = H – D (m) |
| $N_q$ | = | dimensionless bearing capacity factor at the depth of the helix<br>assume $N_q$ = 15 from 4.0 m to 8.0 m<br>= 23 below 8.0 m |
| Ø | = | internal angle of friction<br>assume Ø = 28º from 4.0 m to 8.0 m<br>= 32º below 8.0 m |
| $K_s$ | = | lateral earth pressure coefficient in compression<br>assume average $K_s$ = 0.5 |

**Uplift Load Capacity:**

- If $H/D \le (H/D)_{cr}$, then

$$Q_t = F_q \cdot A_e \cdot H \cdot \gamma'$$

- If $H/D \ge (H/D)_{cr}$, then

$$Q_t = \gamma' \cdot [A_e \cdot H \cdot Fq' \cdot + \tfrac{1}{2}\pi \cdot d \cdot H_{eff}^2 K_u \tan Ø]$$

**Where:**

| | | |
|---|---|---|
| $Q_t$ | = | Ultimate tensile capacity of pile (kN) |
| $F_q$ | = | breakout factors for shallow condition, $H/D \le (H/D)_{cr}$ (see Figure 14) |
| $F_q'$ | = | breakout factors for deep condition, $H/D \ge (H/D)_{cr}$ (see Figure 15) |
| $(H/D)_{cr}$ | = | critical H/D ratio, assume $(H/D)_{cr}$ = 5.0 |
| $K_u$ | = | coefficient of lateral earth pressure in uplift<br>assume $K_u$ = 0.80 from 4.0 m to 8.0 m<br>= 1 below 8.0 m |

Other variables are previously defined.

**For a Multiple Helix Pile:**

**Ultimate Compressive Load Capacity**

$$Q_c = \gamma' \cdot [A_b \cdot H \cdot N_q + ½\, \pi \cdot \{D_A \cdot (H_3^2 - H^2) + d \cdot H_{eff}^2\} \cdot K_s \cdot \tan Ø]$$

**Where:**

| | | |
|---|---|---|
| $A_b$ | = | total area of helix ($m^2$) at the bottom helix = $\pi D^2/4$ |
| d | = | diameter of the shaft (m) |
| D | = | diameter of top helix helix (m) |
| H | = | actual depth to top helix (m) |
| $H_{eff}$ | = | effective depth to helix = H – D (m) |
| $H_3$ | = | depth to bottom helix (m, for multi-helix) |
| $D_A$ | = | average diameter of helix (m) |

Other variables are same as provided for a single helix.

**Uplift Load Capacity**

- If $H/D \leq (H/D)_{cr}$, then

$$Q_t = \gamma' \cdot [A_e \cdot H \cdot F_q + ½\, \pi \cdot D_A \cdot (H_3^2 - H^2) \cdot K_u \cdot \tan Ø]$$

- If $H/D \geq (H/D)_{cr}$, then

$$Q_t = \gamma' \cdot [A_e \cdot H \cdot F_q' + ½\, \pi \cdot \{D_A \cdot (H_3^2 - H^2) + d \cdot H_{eff}^2\} \cdot K_u \cdot \tan Ø]$$

**Where:**

| | | |
|---|---|---|
| $F_q$ | = | breakout factors for shallow condition, $H/D \leq (H/D)_{cr}$ (see Figure 14) |
| $F_q'$ | = | breakout factors for deep condition, $H/D \geq (H/D)_{cr}$ (see Figure 15) |

Other variables are previously defined.

### 6.7.2 Use of Torque Measurements

Traditionally, torque measurements have been used in predicting the vertical capacities of helical piles. However, various researchers have indicated that torque correlations with vertical capacities can be unreliable, with deviations as much as 300 percent between the predicted and actual capacities from load tests. Hence, the use of torque measurements alone as a design tool in the absence of a pile load test is not recommended.

## 6.8 Lateral Load Resistance

The following elastic coefficients of horizontal subgrade reaction may be assumed for calculation of lateral resistance.

**TABLE 7**
**ESTIMATED ELASTIC COEFFICIENTS OF HORIZONTAL SUBGRADE REACTION FOR PILES**

| Depth (m) Below Existing Ground Surface | Estimated Elastic Coefficient of Horizontal Subgrade Reaction ($kN/m^3$) |
|---|---|
| 0.0 to 1.0 | 0.0 |
| 1.0 to 8.0 | 120 (Z/B) |
| 8.0 to 10.0 | 180 (Z/B) |
| > 10 | 3,600/B |

**Notes to Table 7:**

1. B is the pile shaft diameter (m).

2. The ‘K’ values presented in Table 7 assume undisturbed soil conditions adjacent to the pile axis and that the soil is behaving elastically only. In the case of HSPs, soil disturbance will occur as the helical screw(s) travel(s) through the soil. The magnitude of disturbance is expected to be primarily a function of soil type, obstructions encountered below the surface (e.g. cobbles), number of helixes, type of equipment and installation methodology (e.g. rate of penetration, rate of rotation, etc) and, therefore, is indeterminate at this time. Past research and testing has been undertaken in an attempt to quantify the difference between the *undisturbed* and *disturbed* ‘K’ values for helical screw piles. With an experienced contractor, good equipment and well controlled conditions, a reduction factor of 0.3 X K has been typically assumed to obtain $K_{disturbed}$ for screw pile applications. A lateral pile load test would be preferred if a more accurate prediction of $K_{disturbed}$ is required

3. The values presented in Table 7 should be considered as estimates only. To limit lateral earth pressures to within the allowable elastic stress range of the soil, the deflection at the pile top should be less than 8 mm at service load conditions.

4. The lateral deflection of piles may be estimated under service load conditions using the values presented in Table 7.

5. Lateral analysis of piles using LPILE (i.e. method of p-y curves) can be provided by Amec Foster Wheeler, on request, when the pile dimensions and properties (including stiffness), loads and fixity conditions are made available.

Laterally loaded piles structurally contained in a group by a pile cap or grade beam may be considered to act individually when the center-to-center spacing is greater than 3 diameters perpendicular to loading (side-by-side) and greater than 6 diameters in the direction parallel to loading (in-line).

For pile layouts not conforming to these criteria, the effect of pile interaction should be considered in the design by applying the group reduction factors given in Table 8. The group reduction factor is the ratio of lateral capacity of the pile group to the sum of lateral capacities of individual piles. The lateral capacity of a pile group can then be estimated by multiplying the sum of lateral capacity of individual piles in the group with the group reduction factor. The reduction factors should be applied equally to all piles within the group regardless of an individual pile's relative location within the group.

The group reduction factor is defined as:

$$\eta = R_g/[n \cdot R_s]$$

**Where:** $\eta$ = group reduction factor for lateral load
$R_g$ = lateral load capacity of a pile group
$R_s$ = lateral load capacity of a single pile in a pile group
n = number of piles in a pile group

**TABLE 8**
**GROUP REDUCTION FACTOR, η, FOR LATERALLY LOADED PILE GROUPS**

| Pile Group | Load Direction | η value based on Pile Spacing (S) | | | |
|---|---|---|---|---|---|
| | | S = 3B | S = 4B | S = 5B | S = 6B |
| 2 x 1 | in line | 0.85 | 0.90 | 0.95 | 1.00 |
| | side by side | 0.95 | 1.00 | 1.00 | 1.00 |
| 2 x 2 | - | 0.75 | 0.90 | 0.95 | 1.00 |
| 2 x 3 | parallel to long side | 0.70 | 0.85 | 0.90 | 0.95 |
| | parallel to short side | 0.70 | 0.90 | 0.95 | 1.00 |
| 3 x 3 | - | 0.65 | 0.85 | 0.90 | 0.95 |
| 3 x 4 | parallel to long side | 0.60 | 0.85 | 0.90 | 0.95 |
| | parallel to short side | 0.65 | 0.85 | 0.90 | 0.95 |
| 4 x 4 | - | 0.60 | 0.85 | 0.90 | 0.95 |
| 5 x 5 | - | 0.65 | 0.80 | 0.90 | 0.95 |

**Note to Table 8:** S = pile spacing (center to center, m), B = pile diameter (m)

Piles resist lateral loads and moments by deflection until the necessary reaction in the surrounding soil is mobilized. The behavior of a pile under such loading conditions depends on the stiffness of the pile and the soil strength.

The horizontal load capacity of piles is limited in three different ways:

a. Soil capacity
b. Excessive bending stresses in the pile material
c. Pile deflection exceeds the maximum allowed.

All three methods of failure should be considered in the design.

## 7.0 CONCRETE SLABS

### 7.1 Settlement Considerations

Layers of sand and gravel fill (up to 1.3 m deep) were encountered at some test hole locations. Existing fill thickness and composition variabilities are shown on the logs included in Appendix A, but may be different at other unexplored locations on the site.

A key geotechnical issue at this site relates to the fill (existing and new) and its potential for future fill settlement. The potential magnitude of future total and differential subsidence cannot be reliably estimated, but assuming that existing fills were well compacted and that a combination of new and existing fill thicknesses will not exceed 1.5 m, it is considered reasonable to expect that consolidation settlements due to self weight of the fill will not exceed 20 mm. If the existing fill is not well consolidated and/or if the new fill is not adequately compa1cted, long term settlements could exceed 20 mm.

Allowances for future fill settlement needs to be accommodated in the design, and should be expected during in-service.

### 7.2 Interior Grade Supported Slabs (Heated Enclosures)

Although heave due to frost action would not normally be a concern for interior, grade supported concrete slabs located within a heated building, past experience has shown that, under some circumstances, frost can penetrate beneath interior slabs that are located near exterior doorways, especially wide doorways (e.g. the firehall), and along the edges of small buildings unless measures are undertaken to prevent this (e.g. with in-floor heating and/or exterior horizontal insulation).

A thickened slab with additional reinforcing steel would help distribute localized forces, either from low to moderately concentrated vertical loads applied to the slab surface or from differential pressures applied to the slab by the soil.

The following minimum guidelines are recommended for lightly loaded, grade supported concrete floor slabs constructed within heated building areas:

- The subgrade should be prepared in accordance with the minimum requirements outlined in Section 4.0.

- Grade supported slabs should not be used as a foundation. A piled or pier and footing foundation should be used to support concentrated loads. As a general guideline, floor slab bearing pressures should not exceed 15 kPa.

- The perimeter elevation of the finished subgrade beneath the slab granular soil should lie above exterior grades and the subgrade should be adequately sloped from the center to the slab perimeter.

- The slabs should be underlain by a layer (150 mm minimum thickness) of compacted gravel base course. The base course should conform to Saskatchewan Ministry of Highways and Infrastructure (SMHI) Type 32 or 33 Base Specifications.

- Undertake design and construction precautions to reduce the probability of water ingress beneath the slabs.

- Provide extra concrete thickness and steel reinforcing where applicable.

- Provide joints in the concrete surface at spacings not exceeding 4 m in order to reduce the potential for uncontrolled cracking.

  **Note:** Special attention to design details and precautions during construction with inter-panel shear joint details, size and spacing of slab panels, providing adequate curing time allowances between slab panel pours, spacing of control joints, and size and spacing of reinforcement are some of the critical details that need to be addressed.

- Provide caulking along the joint interface of exterior concrete slabs that abut into the perimeter concrete grade beams, after adequate concrete curing (i.e. shrinkage) has taken place.

- Allow the slabs to float independently of other structural elements. For example, steel dowelling between slabs and grade beams and pile caps should not be installed and slip materials/bond breakers should be applied between slabs and grade beams, pile caps and other elements which protrude through the slabs such as pipes, steel columns, electrical conduits, etc.

- The schedule for casting of the concrete slab should allow for placement shortly after the subgrade and base course has been placed, respectively.

- Stringent quality assurance and controls (i.e. monitoring, materials selection and testing, approval, etc.) should be provided by a qualified and knowledgeable materials testing agency, working directly for the Owner, and under the close supervision of a qualified geotechnical engineer, on a full time basis during construction.

### 7.3 Slabs Exposed to Freezing Temperatures (Exterior Grade Supported Slabs)

In addition to the minimum requirements for interior concrete floor slabs, the following key considerations should be incorporated into the design and construction of grade supported (exterior) concrete slabs:

- Slab insulation (e.g. with Dow Chemical's Styrofoam*HI 60 or HI-100) or constructing the slab as structurally supported over a void space (150 mm minimum) are the most reliable methods for preventing slab heave due to frost action. Constructing all concrete slabs in unheated areas with underlying insulation or as structurally supported may not be practical or affordable. Placement of heat coils in combination with insulation could also be considered.

- Providing drainage is a sound measure to help reduce the threat of thick frost lens development.

- Based on an assumed design freezing index of 2,035 °C – days for this area, Amec Foster Wheeler expects that 75mm of Styrofoam*HI insulation would be required to prevent significant frost penetration into the subgrade in the absence of heat being applied within or below a concrete slab. If concrete slabs are to be insulated against frost action, the insulation details, particularly the thickness and lateral extension, should conform to the manufacturer’s requirements and specifications (e.g. Dow Chemical).

- Engineered rigid insulation can also be used to reduce the depth of soil cover needed for frost protection of other elements such as water and sewer lines.

- If some freezing of the subgrade and subsequent frost heave of exposed concrete slabs is acceptable, a reduced insulation thickness may be considered. Amec Foster Wheeler cannot reliably predict how much frost heave will or will not occur in the absence of insulation or where a lesser amount of insulation is present, unless a more detailed thermal analysis is completed.

- Dow Chemical also advises that approximately 0.5 m of well graded, compacted granular material should be placed between the insulation and the underside of the slab in order to reduce the potential for "surface icing". If "surface icing" is not a concern, Dow Chemical indicates that the rigid insulation could be placed directly below the slab. For previous projects, it has been Amec Foster Wheeler's experience that contractors prefer to place the insulation between the underside of the concrete slab and the underlying gravel base.

## 7.4 Grade Supported Slab Thickness

The slab should be designed with adequate thickness and with adequate reinforcement to accommodate the loads applied and to assist in uniform distribution of displacements either from loading or from subgrade movement.

A soaked CBR value of 6.0 and an elastic coefficient of vertical subgrade reaction value, as noted below, may be assumed for the design of lightly loaded, grade supported slabs:

$$\mathbf{K_v = 5{,}900/B}$$

**Where**,

$K_v$ is an ultimate value in units of kPa/m

B = 'effective' width of loaded area (m)

**Notes to the above:**

1. $K_v$ should only be used for purposes of assessing and determining slab thickness, strength and reinforcing requirements. It should not be used to predict settlements. Amec Foster Wheeler should be consulted if prediction of settlements is required, once the slab loading details are known.

2. The ‘effective’ width, B, of the loaded area should take the structural stiffness of the concrete slab into consideration (i.e. the ability of the slab to uniformly distribute bearing stresses into the underlying soil as a result of external loads applied to the slab). The stiffer the slab, the greater the ability to more evenly distribute the applied load to the soil and, thus, the greater the ‘effective’ width. Under any circumstances, the effective width, B, should not be taken as the total slab width, W. For preliminary purposes, it is suggested that the effective width of reinforced concrete slabs with a thickness of 150 mm to 200 mm be assumed to be in the order of 1 m ±. It is emphasized that detailed structural analyses would be required, however, to verify the ‘effective’ width.

3. As a general rule, concrete slabs-on-grade should not be used to support concentrated loads and concentrated loads should be supported by a footing or piled foundation only.

### 7.5 Structurally Supported Concrete Slabs

Structurally supported concrete slabs should be designed and constructed so that they have a minimum void space of 150 mm beneath the slab. The void form material should preferably be bio-degradable and should collapse easily when subject to loads exceeding those due to the dead weight of the concrete slab and live loads due to construction equipment and personnel.

## 8.0 PILE CAPS AND GRADE BEAMS

Pile caps and grade beams should be constructed with adequate reinforcement. A void space (minimum of 150 mm) should be constructed below all pile caps and grade beams. The void material should be a low compressive strength, biodegradable material.

## 9.0 ASPHALT PAVEMENT STRUCTURES

### 9.1 Performance Considerations

Subgrade preparation below pavements should comply with the requirements outlined in Section 4.0 of this report. The long term performance of pavements at this site will likely be impacted by frost action and possibly by settlement in fill areas. Raising grades at the site to the extent practicable above surrounding grades, providing good surface drainage away from the structures, installing ditches and culverts, and taking other positive measures to prevent ingress of water into the subgrade should be implemented and would be useful in improving the performance of the pavement structure (and concrete slabs). As noted previously, special precautions will be required to prevent unacceptable disturbance to the subgrade during construction.

## 9.2 Recommended Asphalt Surfaced Structures

Recommended asphalt surfaced pavements for heavy, medium, and light duty pavement areas constructed above prepared subgrades, as recommended in Section 4.0, are summarized in Table 9.

The recommended pavement structures shown in Table 9 are based on the above design assumptions and an assumed soaked subgrade CBR value of 6.0. The material requirements in the notes following Table 9 should be considered as a minimum.

**TABLE 9**
**RECOMMENDED PAVEMENT STRUCTURES**

| **Material** | **Pavement Structure Component Thickness (mm)** | | |
|---|---|---|---|
| | **Light Duty Max. Wheel Load = 18 kN** | **Medium Duty Max. Wheel Load = 28 kN** | **Heavy Duty Max. Wheel Load = 40 kN** |
| Asphaltic Concrete | 65 | 80 | 100 |
| Granular Base | 130 | 140 | 160 |
| Granular Subbase | 175 | 230 | 270 |
| **TOTAL** | **370** | **450** | **530** |

**Notes to Table 9:**

1. Asphaltic concrete should be composed of a dense graded granular mix with a minimum 50 blow Marshall Stability of 7000 Newtons and 3 to 5 percent air voids. An increased Marshall Stability of 9000 Newtons should be considered for the heavy duty areas.

2. Granular base refers to 18 mm maximum, Saskatchewan Ministry of Highways and Infrastructure (SMHI) Type 33, (see Table 3, or equivalent), crushed gravel with a minimum CBR of 60 and compacted to 100 percent of SPMDD.

3. Granular sub base refers to 50 mm maximum, SMHI Type 8 (see Table 3 or equivalent), gravel with a minimum CBR of 20 and compacted to 100 percent of SPMDD.

4. A periodic pavement inspection and maintenance program should be undertaken to help extend the life span of the pavement structure and also to help improve the overall performance and appearance of the asphalt surface. As part of the maintenance program, all cracks in the asphalt should be filled with an approved sealant. Other more significant pavement distress areas should be brought to the attention of the consultant so that appropriate remedial repair options can be determined.

5. Elevations at the finished pavement surface should be adjusted to allow good surface drainage. A minimum cross slope gradient of two (2) percent is recommended.

## 10.0 SEISMIC CLASSIFICATION

The 2010 National Building Code of Canada (NBCC) defines seismic classification in accordance with Table 10 below.

It should be noted that drill depths for this project were limited to less than 30 m and, therefore, an assumption is required with regard to the soil conditions below the maximum drilled depths. Based on this consideration and in accordance with Table 10, a Site Seismic Classification of ‘D’ is considered appropriate for design.

**TABLE 10**
**SITE CLASSIFICATION FOR SEISMIC SITE RESPONSE**

| Site Class | Ground Profile Name | Average Properties in Top 30 m | | |
|---|---|---|---|---|
| | | **Average Shear Wave Velocity, $V_s$ (m/s)** | **Average Standard Penetration Resistance, $N_{60}$** | **Soil Undrained Shear Strength, $S_u$ (kPa)** |
| A | Hard Rock | $V_s > 1500$ | N/A | N/A |
| B | Rock | $760 < V_s \le 1500$ | N/A | N/A |
| C | Very Dense Soil and Soft Rock | $360 < V_s < 760$ | $N_{60} > 50$ | $S_u > 100$ |
| D | Stiff Soil | $180 < V_s < 360$ | $15 \le N_{60} \le 50$ | $50 < S_u \le 100$ |
| E | Soft Soil | $V_s < 180$ | $N_{60} < 15$ | $S_u < 50$ |
| | | Any profile with more than 3 m of soil with the following characteristics<br>Plasticity index: PI > 20<br>Moisture content: w ≥ 40%, and<br>Undrained Shear Strength; $S_u$ < 25 kPa | | |
| F | Other Soils(1) | Site-specific evaluation required | | |

(1) **Other Soils include:**

(a) Liquefiable *soils*, quick and highly sensitive clays, collapsible weakly cemented *soils*, and other *soils* susceptible to failure or collapse under seismic loading;
(b) Peat and / or highly organic clays greater than 3 m in thickness;
(c) Highly plastic clays (PI >75) more than 8 m thick; and
(d) Soft to medium stiff clays more than 30 m thick

## 11.0 CONCRETE TYPE

Based on Amec Foster Wheeler's experience in the area and elsewhere in Saskatchewan, all foundation concrete should be designed in accordance with CSA Standard CAN3-A23.1-09, assuming an S-2 (severe) rating and the following:

- Use of High Sulphate Resistant (CSA Type HS) Portland cement;
- A maximum water to cement ratio of 0.45 by mass;
- A minimum compressive strength of 32 MPa at 56 days; and
- Adequate air-entrainment and curing as per CSA Table 2.

## 12.0 QUALITY CONTROL

The geotechnical recommendations presented within this report are based on the assumption that an adequate level of quality assurance and quality control will be provided during construction and that qualified contractors experienced in foundations and earthworks will carry out the construction. An adequate level of quality assurance is considered to be full-time monitoring and approval by qualified representatives of the geotechnical engineer during the installation of foundations and excavations.

An adequate level of quality control is considered to be full time testing, with a qualified engineer's supervision and review. Amec Foster Wheeler further requests the opportunity to review drawings and specifications related to any foundation, earthworks or other designs, based on the recommendations provided in this report, to confirm that Amec Foster Wheeler's geotechnical recommendations have been correctly interpreted.

## 13.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering principles and practice. The findings and recommendations were based on the results of field and laboratory investigations, combined with an interpolation of soil and groundwater conditions found at and within the depth of test holes drilled. If conditions encountered during construction appear to be different from those shown by the test holes drilled at the site or if the assumptions stated herein are not in keeping with the design, this office should be notified in order that the recommendations can be reviewed and modified, if found to be necessary.

The construction of foundations should be monitored by Amec Foster Wheeler. Similarly, subgrade preparation and construction of any structural fill should be monitored by both on-site visual inspection and compaction tests. Recommendations presented herein shall be considered invalid if an adequate level of inspection is not provided during construction or if relevant building code requirements are not met.

Soil conditions, by their nature, can be highly variable across a construction site. A contingency should always be included in any construction budget to allow for the possibility of variation in soil conditions, which may result in modification of the design and construction procedures.This report has been prepared for the exclusive use of *Public Works and Government Services Canada* and its agents for specific application to the proposed firehall. Amec Foster Wheeler accepts no liability for the information contained herein except for the specific purpose that it was intended by Amec Foster Wheeler.

Respectfully submitted,

**Amec Foster Wheeler Environmental & Infrastructure,**
**A Division of Amec Foster Wheeler Americas Limited**

PROFESSIONAL ENGINEER
R.M. SAADELDIN
MEMBER 22766
16 01 12
YR. MO. DAY
SASKATCHEWAN

Association of Professional Engineers & Geoscientists of Saskatchewan

CERTIFICATE OF AUTHORIZATION

Amec Foster Wheeler Environment & Infrastructure, a Division of Amec Foster Wheeler Americas ~~Limited~~

Number C0545

Permission to Consult held by:

| Discipline | Sk. Reg. No. |
|---|---|
| GEOTECH | 4624 |

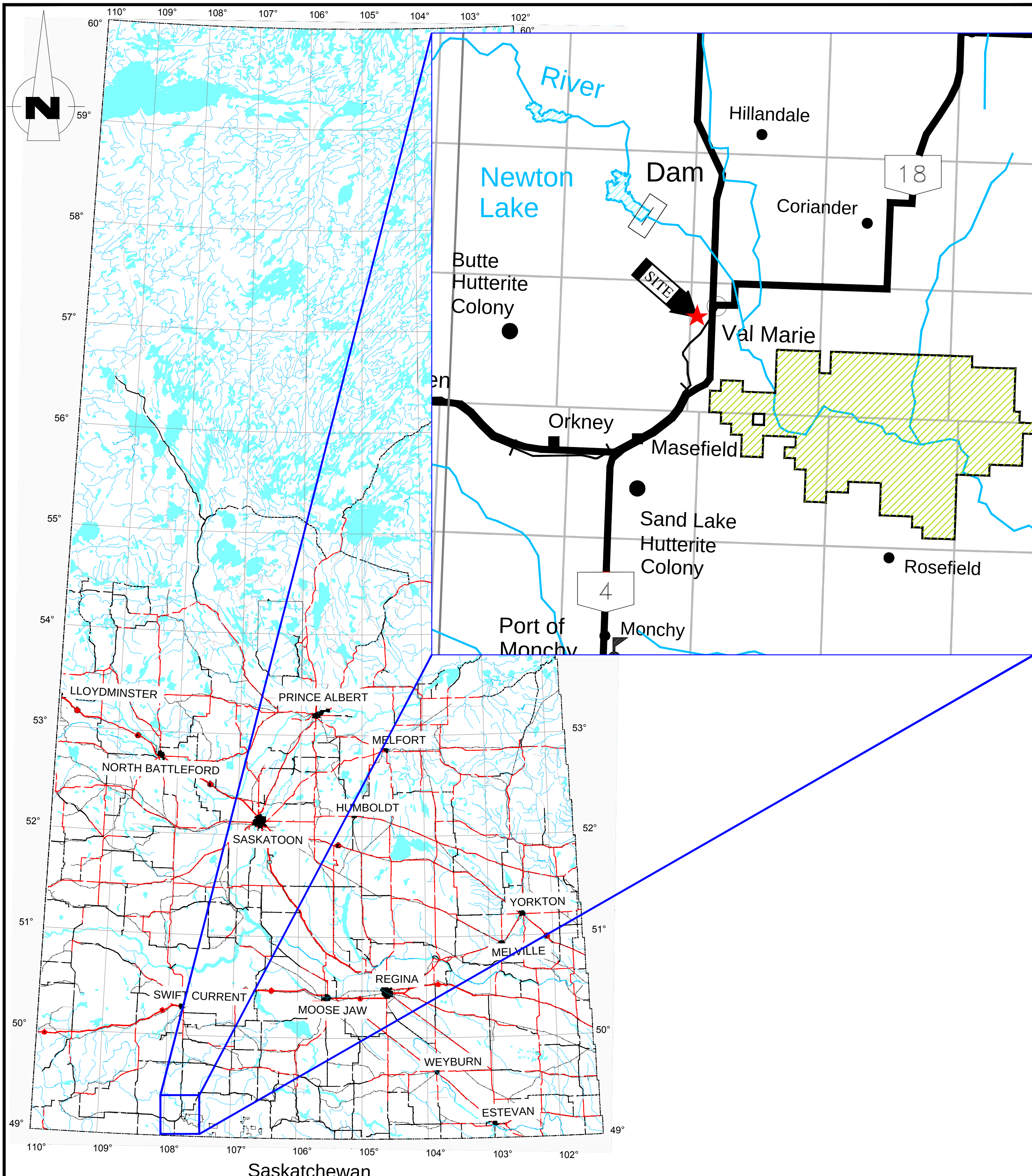
Ramy Saadeldin, M.Sc., P.Eng.
Geotechnical Engineer

Reviewed by,
Gene Froc, P.Eng.
Senior Associate Geotechnical Engineer

amec foster wheeler

# APPENDIX A

## Key Plan, Test Hole Location Plan and Test Hole Logs




| | | | | |
|---|---|---|---|---|
| Amec Foster Wheeler<br>Environment & Infrastructure | amec foster wheeler | CLIENT LOGO | CLIENT<br>PUBLIC WORKS AND GOVERNMENT SERVICES CANADA | |
| PROJECT<br>GEOTECHNICAL INVESTIGATION<br>GRASSLANDS FIREHALL REPLACEMENT<br>VAL MARIE, SASKATCHEWAN | | DWN BY: C.Y.W. | DATUM NAD 83 | DATE NOVEMBER 2015 |
| | | CHK'D BY: C.K. | REV. NO.: - | PROJECT NO.: JX61486 |
| TITLE<br>KEY PLAN | | PROJECTION: UTM ZONE 13 | SCALE: N.T.S. | FIGURE NO.: FIGURE 1 |


LEGEND:

- TEST HOLE LOCATION
- UNDERGROUND GAS LINE
- UNDERGROUND POWER LINE
- UNDERGROUND TELEPHONE LINE
- UNDERGROUND PIPE LINE

| | |
|---|---|
| CLIENT LOGO | |
| CLIENT | PUBLIC WORKS AND GOVERNMENT SERVICES CANADA |
| | amec foster wheeler — Environment & Infrastructure, 608 McLeod Street, Regina, SASKATCHEWAN S4N 4Y1, Tel 306-721-7100 www.amecfw.com |
| DWN BY: | C.Y.W. |
| CHK'D BY: | C.K. |
| DATUM: | NAD 83 |
| PROJECTION: | UTM ZONE 13 |
| SCALE: | 1:500 |
| PROJECT | GEOTECHNICAL INVESTIGATION GRASSLANDS FIREHALL REPLACEMENT VAL MARIE, SASKATCHEWAN |
| TITLE | TEST HOLE LOCATION PLAN |
| REV.No.: | - |
| DATE: | NOVEMBER 2015 |
| PROJECT No. | JX61486 |
| FIGURE No. | FIGURE 2 |

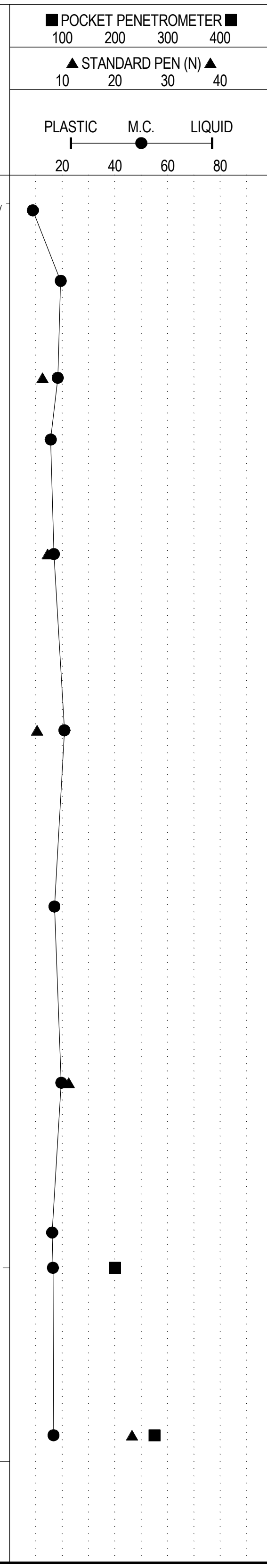
P:\Projects\REGINA\2015 Project File\jx61486\Drawings\CAD\JX61486.dwg – FIG2 – Nov. 19, 2015 9:01am– caroline.wu

| | | |
|---|---|---|
| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-01** |
| PWGSC | TEST HOLE LOCATION: N:5457911, E:0301009 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 100.8 m |

SAMPLE TYPE: THIN WALL TUBE, DISTURBED, SPT, A-CASING, HOLLOW STEM, CORE

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

| Depth (m) | SAMPLE TYPE | SPT (N) | USCS | SOIL SYMBOL | SOIL DESCRIPTION | COMMENTS | BACKFILL DETAILS | ELEVATION (m) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | OL | | ORGANIC SAND TOPSOIL (225mm) - SILTY, POORLY GRADED, FINE TO COARSE GRAINED, DAMP, BROWN | | | |
| | | | | | SAND - SOME SILT, TRACE CLAY, MOIST TO VERY MOIST, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SLOUGHED DURING DRILLING<br>- WET BELOW 0.9 m | | | |
| 1.0 | | | | | | - HYDROMETER ANALYSIS: 0% GRAVEL, 68% SAND 21% SILT, 11% CLAY<br>- GROUNDWATER LEVEL AT 0.9 m BELOW GROUND LEVEL IMMEDIATELY AFTER COMPLETION OF DRILLING | | 100.0 |
| | | 6 | | | - TRACE SEEPAGE AT 1.5m<br>- FINE TO COARSE GRAINED BELOW 1.5m | | | |
| 2.0 | | | | | | | | 99.0 |
| 3.0 | | | | | | - TOP OF SLOUGH MEASURED AT 2.7 m BELOW GROUND LEVEL IMMEDIATELY AFTER COMPLETION OF DRILLING | | 98.0 |
| | | 7 | | | - SIGNIFICANT SEEPAGE AND SLOUGHING BELOW 3.0m | | | |
| 4.0 | | | | | | | | 97.0 |
| | | 5 | SP | | | | | 96.0 |
| 5.0 | | | | | | | | |
| 6.0 | | | | | | | | 95.0 |
| 7.0 | | | | | | | | 94.0 |
| | | 11 | | | - MEDIUM DENSE BELOW 7.6m | | | 93.0 |
| 8.0 | | | | | | | | |
| 9.0 | | | | | | | | 92.0 |
| | | | | | - COBBLE/BOULDER AT 9.1m<br>- COBBLY AND GRAVELLY BETWEEN 9.1m AND 9.4m | | | |
| | | | | | CLAY (TILL) - SILTY, SOME SAND AND GRAVEL, MOIST, VERY STIFF, MEDIUM PLASTIC, GREY | | | 91.0 |
| 10.0 | | | CI | | | | | |
| | | 23 | | | | | | 90.0 |
| 11.0 | | | | | - END OF TEST HOLE AT 11.1m | | | |
| 12 | | | | | | | | 89.0 |



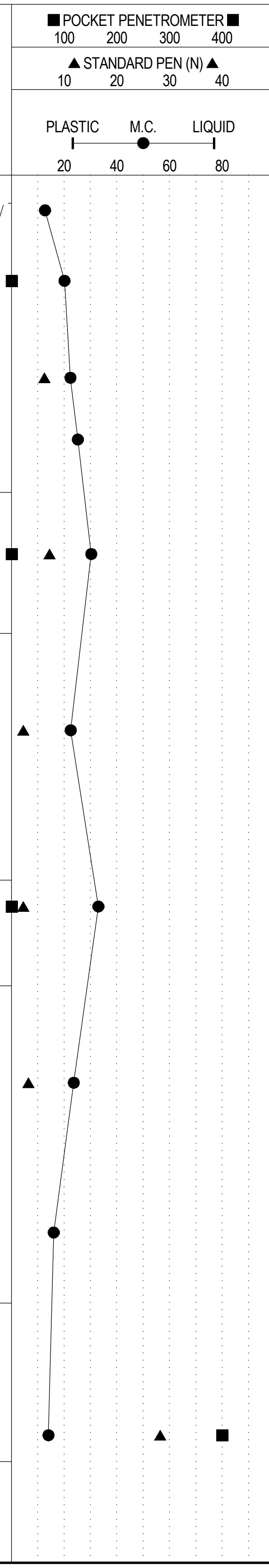

| | | |
|---|---|---|
| Amec Foster Wheeler Environment & Infrastructure<br>Regina, Saskatchewan | LOGGED BY: C.K. | COMPLETION DEPTH: 11.1 m |
| | REVIEWED BY: R.S./G.F. | COMPLETION DATE: 16 November 2015 |
| | Fig. No: 3 | Page 1 of 1 |

| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-02** |
|---|---|---|
| PWGSC | TEST HOLE LOCATION: N:5457922, E:0301032 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 100.2 m |

SAMPLE TYPE: THIN WALL TUBE, DISTURBED, SPT, A-CASING, HOLLOW STEM, CORE

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

| Depth (m) | SPT (N) | USCS | SOIL DESCRIPTION | COMMENTS |
|---|---|---|---|---|
| 0 | | SC | SAND AND GRAVEL (FILL, 200mm) - POORLY GRADED, FINE TO MEDIUM GRAINED, DAMP, LOOSE, BROWN | |
| | 6 | SP | SAND - SOME SILT AND CLAY, MOIST TO VERY MOIST, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SLOUGHED DURING DRILLING<br>- WET BELOW 0.9 m | |
| | 7 | ML | SILT - SOME CLAY, TRACE SAND, WET, FIRM, LOW PLASTIC, BROWN | |
| | 2 | SP | SAND - SOME SILT, TRACE CLAY, WET, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SIGNIFICANT SEEPAGE AND SLOUGHING THROUGHOUT<br>- TRACE CLAY BELOW 1.5m | - TOP OF SLOUGH MEASURED AT 4.3 m BELOW GROUND LEVEL IMMEDIATELY AFTER COMPLETION OF DRILLING |
| | 2 | ML | SILT - SOME SAND, WET, SOFT, LOW PLASTIC, BROWN | |
| | 3 | SP | SAND - TRACE CLAY AND SILT, WET, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SIGNIFICANT SEEPAGE AND SLOUGHING THROUGHOUT<br>- COBBLY AND GRAVELLY BETWEEN 8.5m AND 9.8m | |
| | 28 | CI | CLAY (TILL) - SILTY, SOME SAND AND GRAVEL, WET, VERY STIFF TO HARD, MEDIUM PLASTIC, GREY | |
| 12 | | | - END OF TEST HOLE AT 11.1m<br>NOTE: NO ACCUMULATION OF GROUNDWATER IMMEDIATELY AFTER COMPLETION OF DRILLING | |




BACKFILL DETAILS / ELEVATION (m): 100.0, 99.0, 98.0, 97.0, 96.0, 95.0, 94.0, 93.0, 92.0, 91.0, 90.0, 89.0

Depth (m): 0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12

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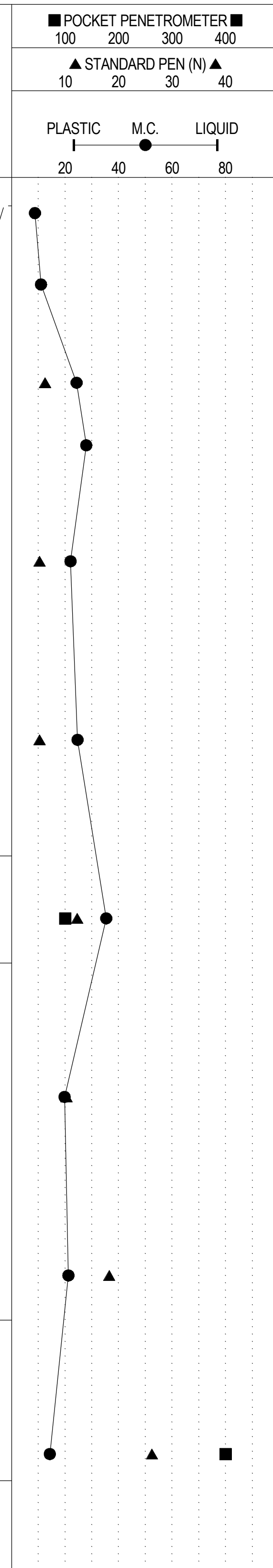
| Amec Foster Wheeler Environment & Infrastructure Regina, Saskatchewan | LOGGED BY: C.K. | COMPLETION DEPTH: 11.1 m |
|---|---|---|
| | REVIEWED BY: R.S./G.F. | COMPLETION DATE: 17 November 2015 |
| | Fig. No: 4 | Page 1 of 1 |

| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-03** |
|---|---|---|
| PWGSC | TEST HOLE LOCATION: N:5457924, E:0301049 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 100.1 m |

SAMPLE TYPE: THIN WALL TUBE, DISTURBED, SPT, A-CASING, HOLLOW STEM, CORE

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

| Depth (m) | SPT (N) | USCS | SOIL DESCRIPTION | COMMENTS | ELEVATION (m) |
|---|---|---|---|---|---|
| 0 | | SC | SAND AND GRAVEL (FILL, 175mm) - POORLY GRADED, FINE TO MEDIUM GRAINED, DAMP, LOOSE, BROWN | | 100.0 |
| | | | SAND - SOME SILT, TRACE CLAY, MOIST, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN | | |
| 1.0 | | | | - HYDROMETER ANALYSIS: 0% GRAVEL, 73% SAND 17% SILT, 10% CLAY | 99.0 |
| | 6 | | - WET WITH SIGNIFICANT SEEPAGE AND SLOUGHING BELOW 1.5m | | |
| 2.0 | | | | | 98.0 |
| 3.0 | 5 | SP | | | 97.0 |
| 4.0 | | | | | 96.0 |
| | 5 | | | - TOP OF SLOUGH MEASURED AT 4.6 m BELOW GROUND LEVEL IMMEDIATELY AFTER COMPLETION OF DRILLING | |
| 5.0 | | | | | 95.0 |
| 6.0 | 12 | ML | SILT - SOME CLAY AND SAND, WET, STIFF, LOW PLASTIC, BROWN | | 94.0 |
| 7.0 | | | SAND - SOME SILT, TRACE CLAY, WET, POORLY GRADED, FINE TO COARSE GRAINED, MEDIUM DENSE, BROWN<br>- SIGNIFICANT SEEPAGE AND SLOUGHING THROUGHOUT | | 93.0 |
| 8.0 | 10 | SP | | | 92.0 |
| 9.0 | 18 | | - GRAVELLY BETWEEN 9.1m AND 9.8m | | 91.0 |
| 10.0 | | CI | CLAY (TILL) - SILTY, SOME SAND AND GRAVEL, MOIST, VERY STIFF TO HARD, MEDIUM PLASTIC, GREY | | 90.0 |
| 11.0 | 26 | | | | 89.0 |
| | | | - END OF TEST HOLE AT 11.1m<br>NOTE: NO ACCUMULATION OF GROUNDWATER IMMEDIATELY AFTER COMPLETION OF DRILLING | | |
| 12 | | | | | |



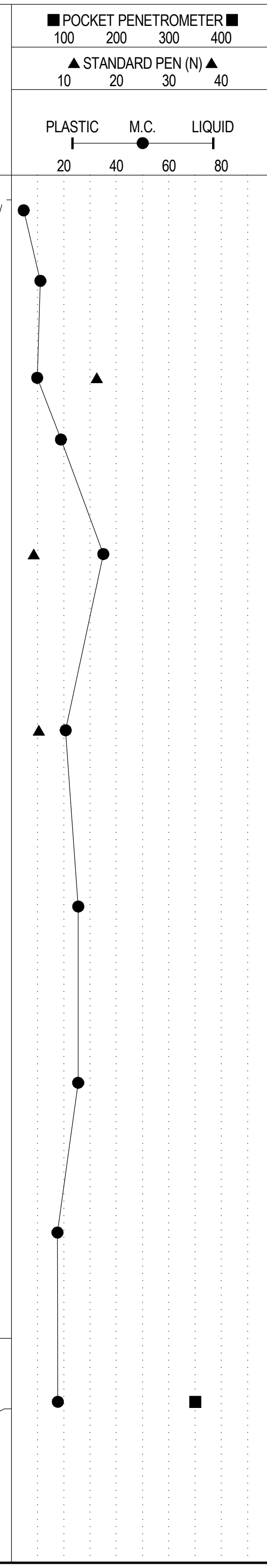

**Amec Foster Wheeler Environment & Infrastructure**
**Regina, Saskatchewan**

| LOGGED BY: C.K. | COMPLETION DEPTH: 11.1 m |
|---|---|
| REVIEWED BY: R.S./G.F. | COMPLETION DATE: 17 November 2015 |
| Fig. No: 5 | Page 1 of 1 |

| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-04** |
|---|---|---|
| PWGSC | TEST HOLE LOCATION: N:5457935, E:0301032 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 100.7 m |

| SAMPLE TYPE | THIN WALL TUBE | DISTURBED | SPT | A-CASING | HOLLOW STEM | CORE |
|---|---|---|---|---|---|---|
| BACKFILL TYPE | BENTONITE | PEA GRAVEL | SLOUGH | GROUT | DRILL CUTTINGS | SAND |

| Depth (m) | SAMPLE TYPE | SPT (N) | USCS | SOIL DESCRIPTION | COMMENTS | BACKFILL DETAILS | ELEVATION (m) |
|---|---|---|---|---|---|---|---|
| 0 | | | SC | SAND AND GRAVEL (FILL, 175mm) - POORLY GRADED, FINE TO MEDIUM GRAINED, DAMP, LOOSE, BROWN | | BENTONITE | |
| | | | SP | SAND - SOME SILT, TRACE CLAY, MOIST, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SLOUGHED DURING DRILLING | | | 100.0 |
| 1.0 | | | | | | | |
| | | 16 | | - MEDIUM DENSE AT 1.5m<br>- WET WITH SIGNIFICANT SEEPAGE AND SLOUGHING BELOW 1.5m | | DRILL CUTTINGS | 99.0 |
| 2.0 | | | | | | | 98.0 |
| 3.0 | | 4 | | | | | 97.0 |
| 4.0 | | 5 | | | | | 96.0 |
| 5.0 | | | | | - TOP OF SLOUGH MEASURED AT 5.5 m BELOW GROUND LEVEL IMMEDIATELY AFTER COMPLETION OF DRILLING | SLOUGH | 95.0 |
| 6.0 | | | | | | | 94.0 |
| 7.0 | | | | | | | 93.0 |
| 8.0 | | | | - GRAVELLY BELOW 8.5m | | | 92.0 |
| 9.0 | | | | - BLACK/ORANGE STAINING BELOW 9.1m | | | 91.0 |
| 10.0 | | | CI | CLAY (TILL) - SILTY, SOME SAND AND GRAVEL, MOIST, VERY STIFF, MEDIUM PLASTIC, GREY<br>- COBBLE/BOULDER AT 10.1m | | | 90.0 |
| | | | | - END OF TEST HOLE AT 10.7m<br>NOTE: NO ACCUMULATION OF GROUNDWATER IMMEDIATELY AFTER COMPLETION OF DRILLING | | | |
| 11.0 | | | | | | | 89.0 |
| 12 | | | | | | | |




Amec Foster Wheeler Environment & Infrastructure
Regina, Saskatchewan

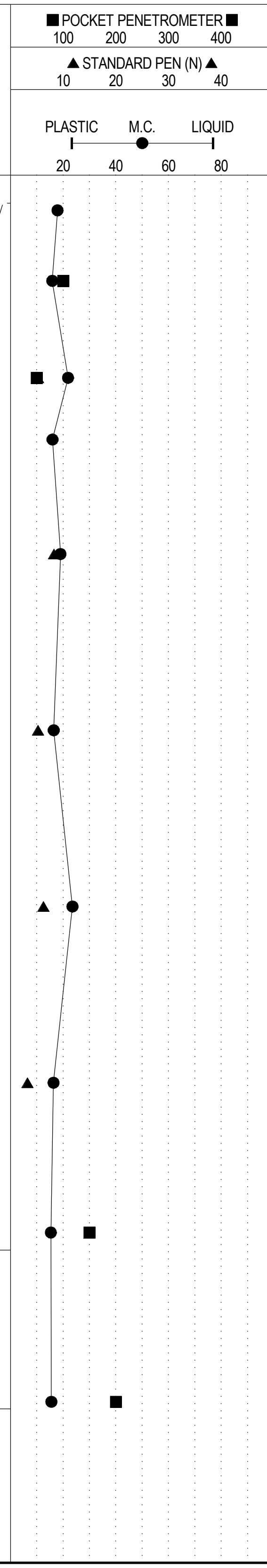
| LOGGED BY: C.K. | COMPLETION DEPTH: 10.7 m |
|---|---|
| REVIEWED BY: R.S./G.F. | COMPLETION DATE: 17 November 2015 |
| Fig. No: 6 | Page 1 of 1 |

| | | |
|---|---|---|
| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-05** |
| PWGSC | TEST HOLE LOCATION: N:5457911, E:0301048 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 99.8 m |

SAMPLE TYPE: THIN WALL TUBE, DISTURBED, SPT, A-CASING, HOLLOW STEM, CORE

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

| Depth (m) | SAMPLE TYPE | SPT (N) | USCS | SOIL SYMBOL | SOIL DESCRIPTION | COMMENTS | BACKFILL DETAILS | ELEVATION (m) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | SC | | SAND AND GRAVEL (FILL, 200mm) - POORLY GRADED, FINE TO MEDIUM GRAINED, DAMP, LOOSE, BROWN | | | |
| | | | | | SAND - SOME SILT AND CLAY, WET, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SLOUGHED DURING DRILLING | | | 99.0 |
| 1.0 | | 5 | | | | | | 98.0 |
| 2.0 | | | | | - SIGNIFICANT SEEPAGE AND SLOUGHING BELOW 2.3m | | | 97.0 |
| 3.0 | | 8 | | | | - TOP OF SLOUGH MEASURED AT 3.0 m BELOW GROUND LEVEL IMMEDIATELY AFTER COMPLETION OF DRILLING | | 96.0 |
| 4.0 | | 5 | SP | | | | | 95.0 |
| 5.0 | | | | | | | | 94.0 |
| 6.0 | | 6 | | | - BLACK STAINING BELOW 6.1m | | | 93.0 |
| 7.0 | | | | | - CLAYEY BETWEEN 7.0m AND 7.6m | | | |
| | | 3 | | | - FINE TO COARSE GRAINED, GREY BELOW 7.6m | | | 92.0 |
| 8.0 | | | | | | | | 91.0 |
| 9.0 | | | | | | | | |
| | | | CI | | CLAY (TILL) - SILTY, SOME SAND AND GRAVEL, MOIST, VERY STIFF, MEDIUM PLASTIC, GREY | | | 90.0 |
| 10.0 | | | | | | | | 89.0 |
| | | | | | - END OF TEST HOLE AT 10.7m<br>NOTE: NO ACCUMULATION OF GROUNDWATER IMMEDIATELY AFTER COMPLETION OF DRILLING | | | |
| 11.0 | | | | | | | | 88.0 |
| 12 | | | | | | | | |



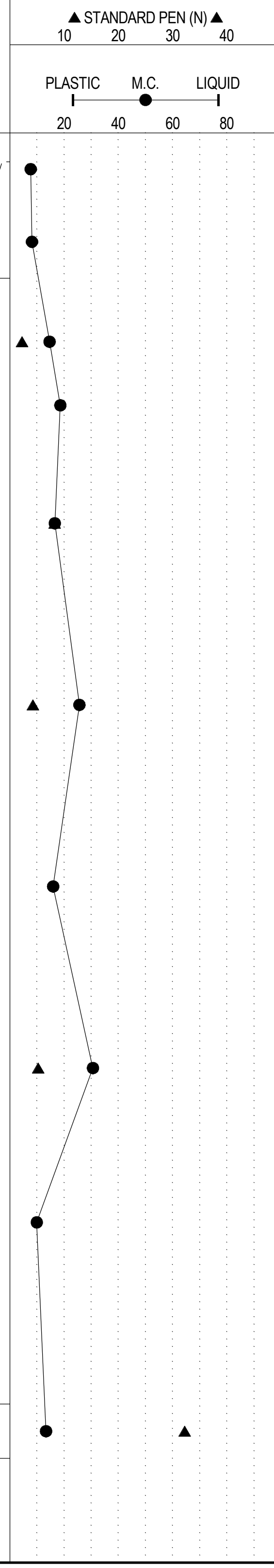

| | | |
|---|---|---|
| **Amec Foster Wheeler Environment & Infrastructure**<br>**Regina, Saskatchewan** | LOGGED BY: C.K. | COMPLETION DEPTH: 10.7 m |
| | REVIEWED BY: R.S./G.F. | COMPLETION DATE: 16 November 2015 |
| | Fig. No: 7 | Page 1 of 1 |

| | | |
|---|---|---|
| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-06** |
| PWGSC | TEST HOLE LOCATION: N:5457930, E:0301059 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 100.1 m |

SAMPLE TYPE: THIN WALL TUBE, DISTURBED, SPT, A-CASING, HOLLOW STEM, CORE

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

| Depth (m) | SAMPLE TYPE | SPT (N) | USCS | SOIL DESCRIPTION | COMMENTS | ELEVATION (m) |
|---|---|---|---|---|---|---|
| 0 | | | SC | SAND AND GRAVEL (FILL, 175mm) - POORLY GRADED, FINE TO MEDIUM GRAINED, DAMP, LOOSE, BROWN | | 100.0 |
| | | | SC | SAND (FILL?, 900 mm) - TRACE CLAY, MOIST, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, DARK BROWN | | |
| 1.0 | | | | | | 99.0 |
| | | | | SAND - SOME SILT, TRACE CLAY, WET, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SLOUGHED DURING DRILLING | | |
| | | 2 | | | | |
| 2.0 | | | | | | 98.0 |
| 3.0 | | 8 | | - SIGNIFICANT SEEPAGE AND SLOUGHING BELOW 3.0m<br>- FINE TO COARSE GRAINED BELOW 3.0m | | 97.0 |
| 4.0 | | | | | - GROUNDWATER LEVEL AT 4.0 m BELOW GROUND LEVEL IMMEDIATELY AFTER COMPLETION OF DRILLING | 96.0 |
| | | 4 | | | | |
| 5.0 | | | | | | 95.0 |
| 6.0 | | | SP | | | 94.0 |
| 7.0 | | | | | - TOP OF SLOUGH MEASURED AT 6.7 m BELOW GROUND LEVEL IMMEDIATELY AFTER COMPLETION OF DRILLING | 93.0 |
| | | 5 | | | | |
| 8.0 | | | | | | 92.0 |
| 9.0 | | | | - GRAVELLY BELOW 9.1m | | 91.0 |
| | | | | CLAY (TILL) - SILTY, SOME SAND AND GRAVEL, MOIST, VERY STIFF TO HARD, MEDIUM PLASTIC, GREY | | |
| 10.0 | | | CI | | | 90.0 |
| 11.0 | | 32 | | | | 89.0 |
| | | | | - END OF TEST HOLE AT 11.1m | | |
| 12 | | | | | | |

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|---|---|---|
| Amec Foster Wheeler Environment & Infrastructure<br>Regina, Saskatchewan | LOGGED BY: C.K. | COMPLETION DEPTH: 11.1 m |
| | REVIEWED BY: R.S./G.F. | COMPLETION DATE: 16 November 2015 |
| | Fig. No: 8 | Page 1 of 1 |

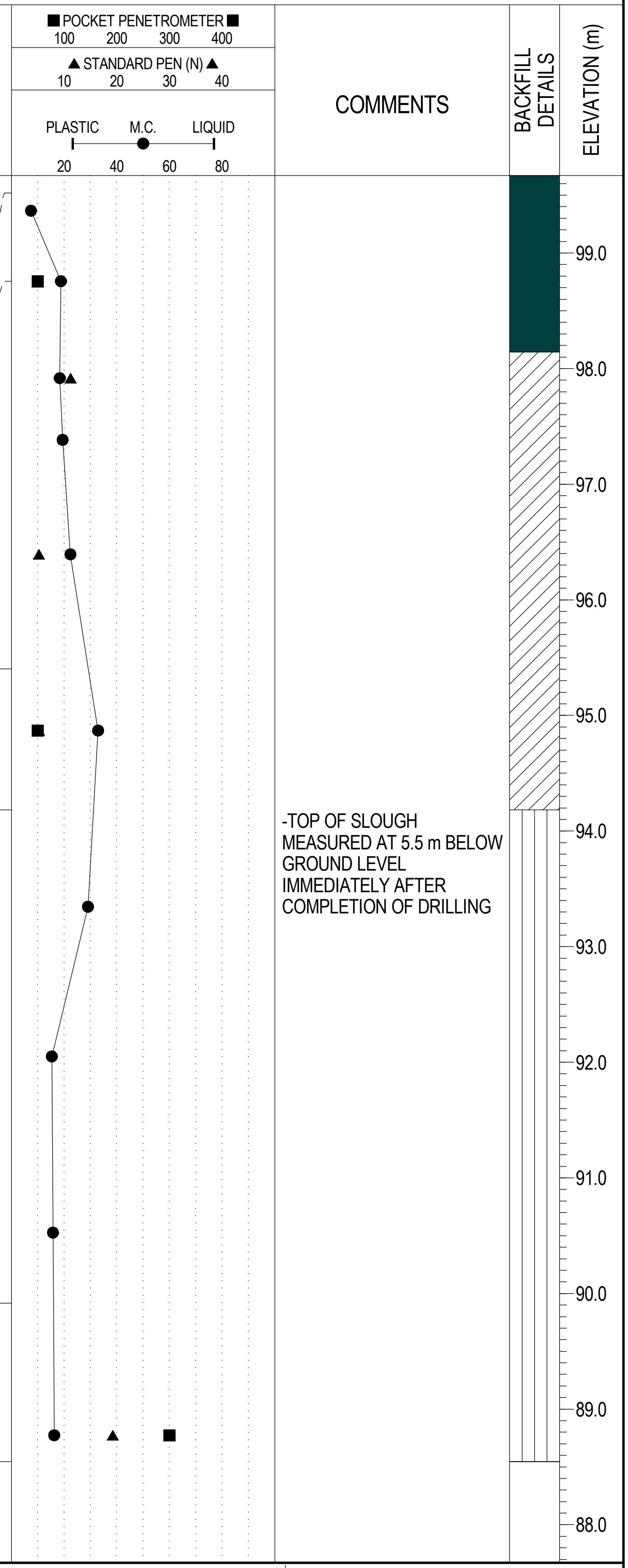
JX61486.GPJ 16/01/11 05:46 PM (ORIGINAL BOREHOLE LOG)

| | | |
|---|---|---|
| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-07** |
| PWGSC | TEST HOLE LOCATION: N:5457914, E:0301066 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 99.7 m |

SAMPLE TYPE: THIN WALL TUBE, DISTURBED, SPT, A-CASING, HOLLOW STEM, CORE

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

| Depth (m) | SAMPLE TYPE | SPT (N) | USCS | SOIL DESCRIPTION | COMMENTS |
|---|---|---|---|---|---|
| 0 | | | OL | ORGANIC SAND TOPSOIL (150mm) - SILTY, POORLY GRADED, FINE TO COARSE GRAINED, DAMP, BROWN | |
| | Disturbed | | SC | SAND (FILL?, 800 mm) - TRACE CLAY, MOIST, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, DARK BROWN | |
| 1.0 | Disturbed | | | | |
| | | | | SAND - SOME SILT, TRACE CLAY, WET, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN | |
| | SPT | 11 | | | |
| 2.0 | | | | - SIGNIFICANT SEEPAGE AND SLOUGHING BELOW 1.8m | |
| | Disturbed | | SP | | |
| 3.0 | SPT | 5 | | | |
| 4.0 | | | | | |
| | | | | SILT - SOME SAND, TRACE CLAY, WET, FIRM, LOW PLASTIC, BROWN | |
| 5.0 | SPT | 5 | ML | | |
| | | | | SAND - TRACE CLAY AND SILT, WET, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SIGNIFICANT SEEPAGE AND SLOUGHING THROUGHOUT<br>- VERY LOOSE AT 6.1m | -TOP OF SLOUGH MEASURED AT 5.5 m BELOW GROUND LEVEL IMMEDIATELY AFTER COMPLETION OF DRILLING |
| 6.0 | SPT | | | | |
| 7.0 | | | | | |
| | Disturbed | | SP | - COBBLY AND GRAVELLY BETWEEN 7.6 AND 9.8m | |
| 8.0 | | | | | |
| 9.0 | Disturbed | | | | |
| 10.0 | | | | CLAY (TILL) - SILTY, SOME SAND AND GRAVEL, MOIST, VERY STIFF, MEDIUM PLASTIC, GREY | |
| | | | CI | | |
| 11.0 | SPT | 19 | | | |
| | | | | - END OF TEST HOLE AT 11.1m<br>NOTE: NO ACCUMULATION OF GROUNDWATER IMMEDIATELY AFTER COMPLETION OF DRILLING | |
| 12 | | | | | |




JX61486.GPJ 16/01/11 05:46 PM (ORIGINAL BOREHOLE LOG)

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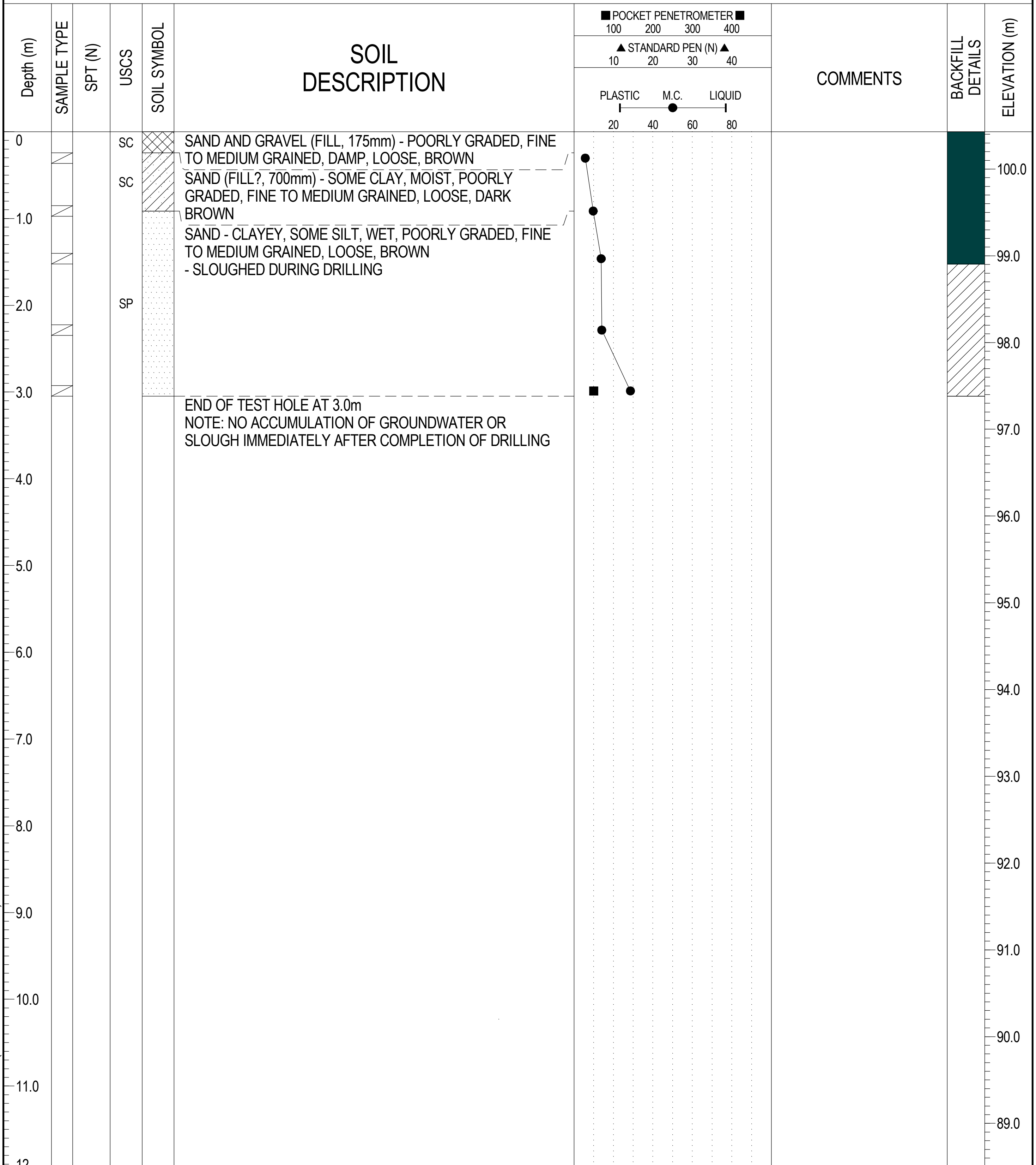
| **Amec Foster Wheeler Environment & Infrastructure**<br>**Regina, Saskatchewan** | LOGGED BY: C.K. | COMPLETION DEPTH: 11.1 m |
| | REVIEWED BY: R.S./G.F. | COMPLETION DATE: 17 November 2015 |
| | Fig. No: 9 | Page 1 of 1 |

| | | |
|---|---|---|
| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-08** |
| PWGSC | TEST HOLE LOCATION: N:5457934, E:0301056 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 100.4 m |

SAMPLE TYPE: THIN WALL TUBE, DISTURBED, SPT, A-CASING, HOLLOW STEM, CORE

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

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| Amec Foster Wheeler Environment & Infrastructure<br>Regina, Saskatchewan | LOGGED BY: C.K. | COMPLETION DEPTH: 3.0 m |
| | REVIEWED BY: R.S./G.F. | COMPLETION DATE: 16 November 2015 |
| | Fig. No: 10 | Page 1 of 1 |

| | | |
|---|---|---|
| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-09** |
| PWGSC | TEST HOLE LOCATION: N:5457930, E:0301031 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 100.5 m |

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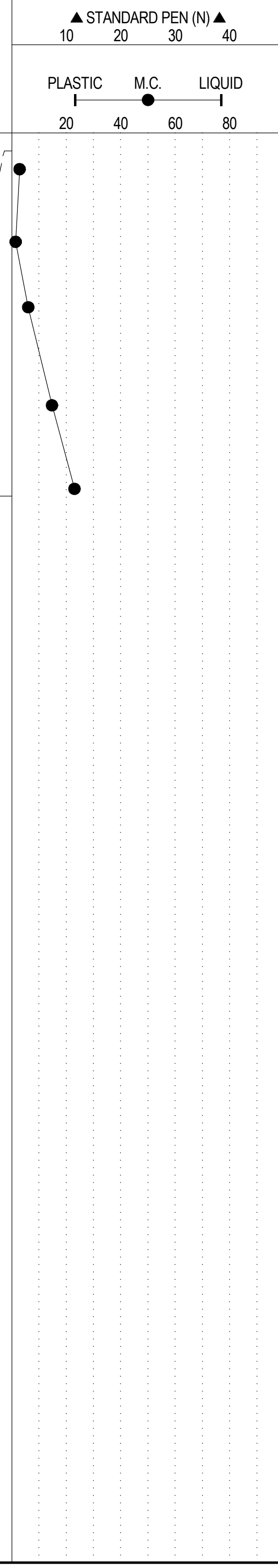
|---|---|---|---|---|---|---|
| SAMPLE TYPE | THIN WALL TUBE | DISTURBED | SPT | A-CASING | HOLLOW STEM | CORE |
| BACKFILL TYPE | BENTONITE | PEA GRAVEL | SLOUGH | GROUT | DRILL CUTTINGS | SAND |

| Depth (m) | SAMPLE TYPE | SPT (N) | USCS | SOIL SYMBOL | SOIL DESCRIPTION | COMMENTS | BACKFILL DETAILS | ELEVATION (m) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | SC | | SAND AND GRAVEL (FILL, 150mm) - POORLY GRADED, FINE TO MEDIUM GRAINED, DAMP, LOOSE, BROWN | | | |
| | | | | | SAND - SOME SILT, TRACE CLAY, DAMP, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SLOUGHED DURING DRILLING | | | 100.0 |
| 1.0 | | | SP | | - CLAYEY AND WET BELOW 1.5m | | | 99.0 |
| 2.0 | | | | | | | | 98.0 |
| 3.0 | | | | | END OF TEST HOLE AT 3.0m<br>NOTE: NO ACCUMULATION OF GROUNDWATER OR SLOUGH IMMEDIATELY AFTER COMPLETION OF DRILLING | | | 97.0 |
| 4.0 | | | | | | | | 96.0 |
| 5.0 | | | | | | | | 95.0 |
| 6.0 | | | | | | | | 94.0 |
| 7.0 | | | | | | | | 93.0 |
| 8.0 | | | | | | | | 92.0 |
| 9.0 | | | | | | | | 91.0 |
| 10.0 | | | | | | | | 90.0 |
| 11.0 | | | | | | | | 89.0 |
| 12 | | | | | | | | |

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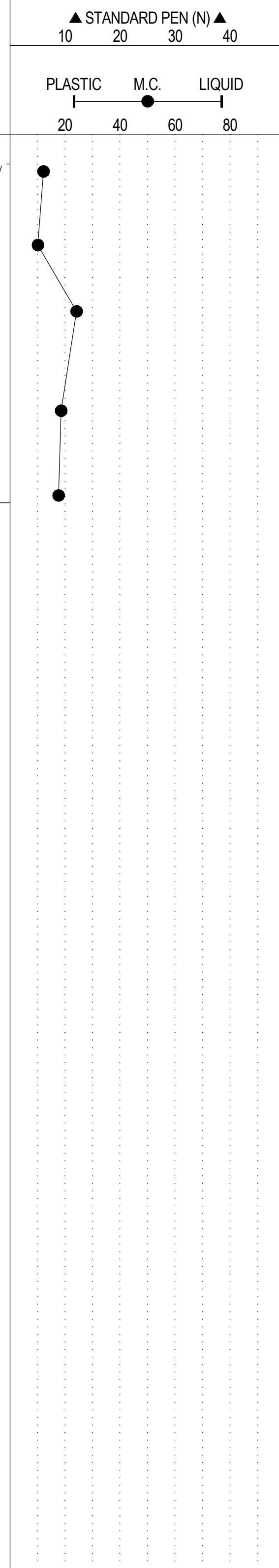
| **Amec Foster Wheeler Environment & Infrastructure**<br>**Regina, Saskatchewan** | LOGGED BY: C.K. | COMPLETION DEPTH: 3.0 m |
| | REVIEWED BY: R.S./G.F. | COMPLETION DATE: 16 November 2015 |
| | Fig. No: 11 | Page 1 of 1 |

| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-10** |
|---|---|---|
| PWGSC | TEST HOLE LOCATION: N:5457916, E:0301053 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 99.8 m |

SAMPLE TYPE: THIN WALL TUBE, DISTURBED, SPT, A-CASING, HOLLOW STEM, CORE

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

| Depth (m) | SAMPLE TYPE | SPT (N) | USCS | SOIL SYMBOL | SOIL DESCRIPTION | COMMENTS | BACKFILL DETAILS | ELEVATION (m) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | SC | | SAND AND GRAVEL (FILL, 200mm) - POORLY GRADED, FINE TO MEDIUM GRAINED, DAMP, LOOSE, BROWN | | | |
| | | | SP | | SAND - SOME CLAY AND SILT, MOIST TO VERY MOIST, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SLOUGHED DURING DRILLING<br>- CLAYEY AND WET BELOW 1m | | | 99.0 |
| 1.0 | | | | | | | | |
| 2.0 | | | | | - TRACE SEEPAGE AT 2.3m | | | 98.0 |
| 3.0 | | | | | END OF TEST HOLE AT 3.0m<br>NOTE: NO ACCUMULATION OF GROUNDWATER OR SLOUGH IMMEDIATELY AFTER COMPLETION OF DRILLING | | | 97.0 |
| 4.0 | | | | | | | | 96.0 |
| 5.0 | | | | | | | | 95.0 |
| 6.0 | | | | | | | | 94.0 |
| 7.0 | | | | | | | | 93.0 |
| 8.0 | | | | | | | | 92.0 |
| 9.0 | | | | | | | | 91.0 |
| 10.0 | | | | | | | | 90.0 |
| 11.0 | | | | | | | | 89.0 |
| 12 | | | | | | | | 88.0 |




Amec Foster Wheeler Environment & Infrastructure
Regina, Saskatchewan

| LOGGED BY: C.K. | COMPLETION DEPTH: 3.0 m |
|---|---|
| REVIEWED BY: R.S./G.F. | COMPLETION DATE: 16 November 2015 |
| Fig. No: 12 | Page 1 of 1 |

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|---|---|---|
| PROPOSED FIREHALL REPLACEMENT | DRILLING METHOD: 150mm Solid Stem Auger | TEST HOLE NO: **15-11** |
| PWGSC | TEST HOLE LOCATION: N:5457916, E:0301014 | PROJECT NO: JX61486 |
| VAL MARIE, SASKATCHEWAN | | ELEVATION: 99.7 m |

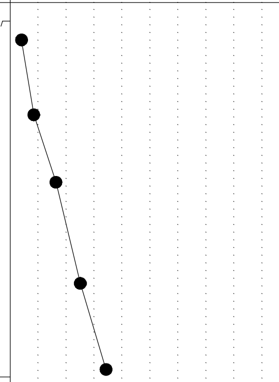
SAMPLE TYPE: THIN WALL TUBE, DISTURBED, SPT, A-CASING, HOLLOW STEM, CORE

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

| Depth (m) | USCS | SOIL DESCRIPTION | COMMENTS | BACKFILL DETAILS |
|---|---|---|---|---|
| 0 | OL | ORGANIC SAND TOPSOIL (150mm) - SILTY, POORLY GRADED, FINE TO COARSE GRAINED, DAMP, BROWN | | BENTONITE (0 to 1.5m) |
| 0.15–3.0 | SP | SAND - SOME SILT, TRACE CLAY, MOIST, POORLY GRADED, FINE TO MEDIUM GRAINED, LOOSE, BROWN<br>- SLOUGHED DURING DRILLING<br>- CLAYEY AND WET BELOW 1.0m<br>- DARK BROWN TO BLACK STAINING AT 1.5m | | DRILL CUTTINGS (1.5 to 3.0m) |
| 3.0 | | END OF TEST HOLE AT 3.0m<br>NOTE: NO ACCUMULATION OF GROUNDWATER OR SLOUGH IMMEDIATELY AFTER COMPLETION OF DRILLING | | |

▲ STANDARD PEN (N) ▲ 10 20 30 40

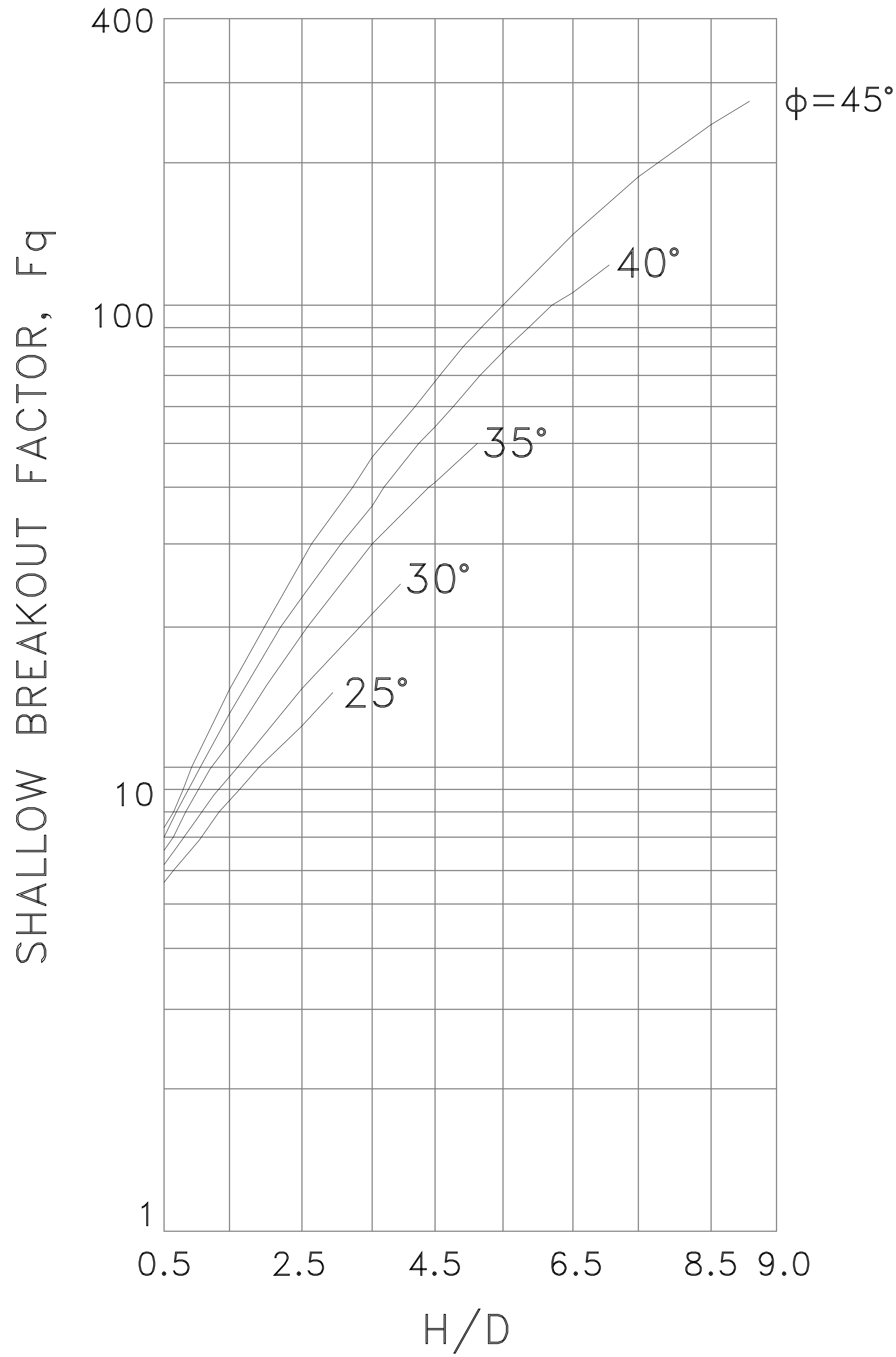
PLASTIC M.C. LIQUID 20 40 60 80



Depth (m): 0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12

ELEVATION (m): 99.0, 98.0, 97.0, 96.0, 95.0, 94.0, 93.0, 92.0, 91.0, 90.0, 89.0, 88.0

| | | |
|---|---|---|
| **Amec Foster Wheeler Environment & Infrastructure**<br>**Regina, Saskatchewan** | LOGGED BY: C.K. | COMPLETION DEPTH: 3.0 m |
| | REVIEWED BY: R.S./G.F. | COMPLETION DATE: 16 November 2015 |
| | Fig. No: 13 | Page 1 of 1 |

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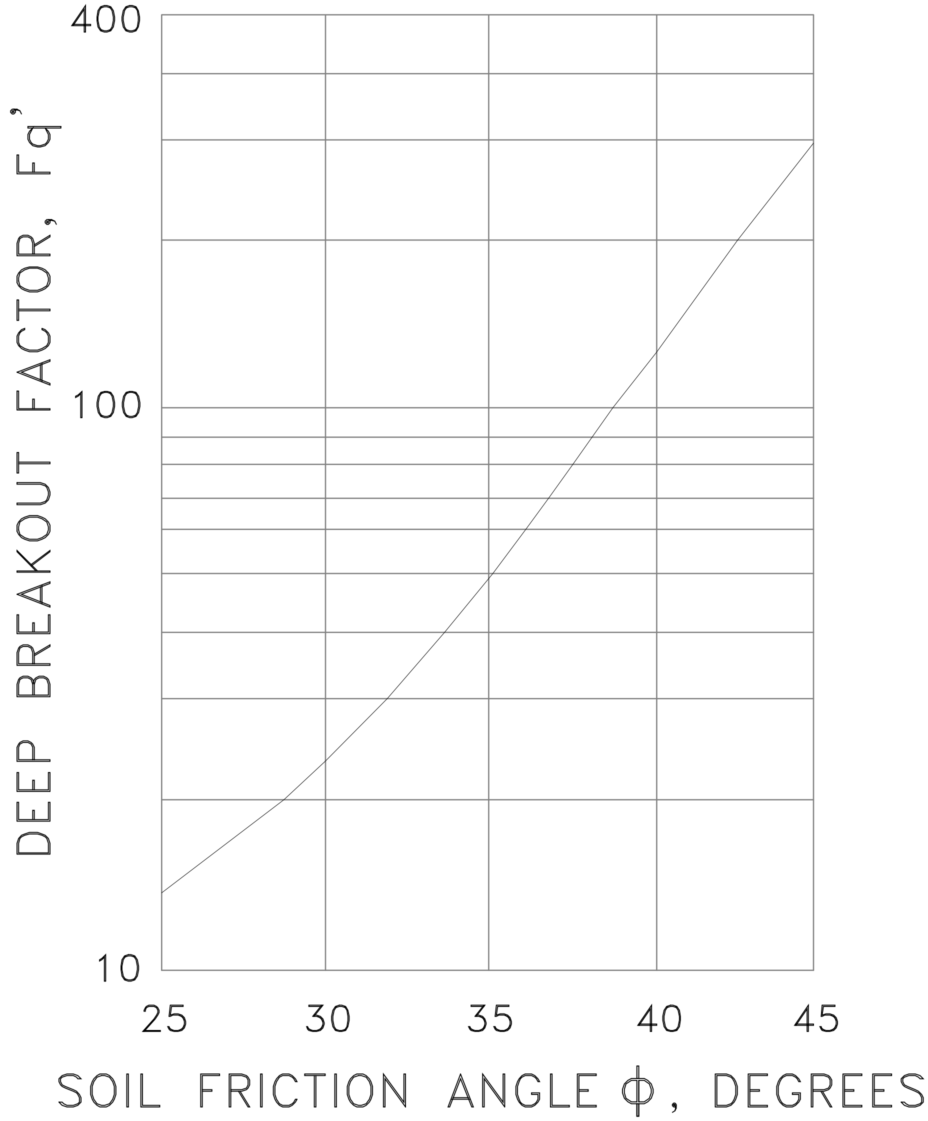
|---|---|---|---|
| CLIENT: [illegible] | DWN BY: C.Y.W. | PROJECT: GEOTECHNICAL INVESTIGATION<br>GRASSLANDS FIREHALL REPLACEMENT<br>VAL MARIE, SASKATCHEWAN | DATE: NOVEMBER 2015 |
| | CHK'D BY: RS | | PROJECT No.: JX61486 |
| amec foster wheeler<br>Environment & Infrastructure<br>608 McLeod Street<br>Regina, SASKATCHEWAN S4N 4Y1<br>Tel 306-721-7100 www.amecfw.com<br>608 McLEOD STREET, REGINA, SK S4N 4Y1 | DATUM: - | | REV. No.: - |
| | PROJECTION: - | TITLE: VARIATION OF BREAKOUT FACTOR,<br>SHALLOW CONDITION (AFTER DAS, 1990) | FIGURE No.: FIGURE 14 |
| | SCALE: NTS | | |

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| CLIENT: Public Works and Government Services Canada | DWN BY: C.Y.W.<br>CHK'D BY: RS<br>DATUM: - | PROJECT: GEOTECHNICAL INVESTIGATION GRASSLANDS FIREHALL REPLACEMENT VAL MARIE, SASKATCHEWAN | DATE: NOVEMBER 2015<br>PROJECT No.: JX61486<br>REV. No.: - |
| amec foster wheeler Environment & Infrastructure<br>608 McLeod Street<br>Regina, SASKATCHEWAN S4N 4Y1<br>Tel 306-721-7100 www.amecfw.com | PROJECTION: -<br>SCALE: NTS | TITLE: VARIATION OF BREAKOUT FACTOR, DEEP CONDITION (AFTER DAS, 1990) | FIGURE No.: FIGURE 15 |

amec foster wheeler

# APPENDIX B

## Photographs

**PHOTOGRAPH 1: TESTHOLE 15-01 AND 15-011 FACING WEST**

**PHOTOGRAPH 2: TEST HOLE 15-02, 15-04 AND 15-09 FACING SOUTH**

| amec foster wheeler | SITE PHOTOGRAPHS<br>GEOTECHNICAL INVESTIGATION<br>GRASSLANDS NATIONAL PARK<br>FIREHALL REPLACEMENT<br>NE 30-03-13 W3M<br>VAL MARIE, SASKATCHEWAN |
|---|---|
| **CLIENT: PUBLIC WORKS AND GOVERNMENT SERVICES CANADA** | |

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| **MADE BY:** C.K. | **SCALE:** N/A | **DATE:** NOV 15 | **PROJECT NO.** JX61486 | **PAGE** 1 |

**PHOTOGRAPH 3: FACING EAST TOWARDS TESTHOLES 15-03, 15-05, 15-06, 15-08 AND 15-10**

**PHOTOGRAPH 4: FACING SOUTH TOWARDS TEST HOLES 15-03, 15-05, 15-06, 15-08 AND 15-10**

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|---|---|
| amec foster wheeler | **SITE PHOTOGRAPHS**<br>**GEOTECHNICAL INVESTIGATION**<br>**GRASSLANDS NATIONAL PARK**<br>**FIREHALL REPLACEMENT**<br>**NE 30-03-13 W3M**<br>**VAL MARIE, SASKATCHEWAN** |
| **CLIENT:** □□B□□ □ □□□S □□□ □□□□□□M□□T S□□□□□□S □□□□□□ | |

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**PHOTOGRAPH 5: TESTHOLE 15-07 FACING SOUTHEAST**

**PHOTOGRAPH 6: TESTHOLE 15-06 AND 15-08 FACING NORTHWEST**

amec foster wheeler

**CLIENT:** [illegible]

**SITE PHOTOGRAPHS**
**GEOTECHNICAL INVESTIGATION**
**GRASSLANDS NATIONAL PARK**
**FIREHALL REPLACEMENT**
**NE 30-03-13 W3M**
**VAL MARIE, SASKATCHEWAN**

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| **MADE BY:** C.K. | **SCALE:** N/A | **DATE:** NOV 15 | **PROJECT NO.** JX61486 | **PAGE** 3 |

**PHOTOGRAPH 7: SITE FACING SOUTHWEST FROM NORTHEAST CORNER**

**PHOTOGRAPH 8: SITE FACING NORTH FROM SOUTHWEST CORNER**

amec foster wheeler

**CLIENT:** □□**B**□□ □ □□□**S** □□□ □□□□□□**M**□□**T** **S**□□□□□□**S** □□□□□□

**SITE PHOTOGRAPHS**
**GEOTECHNICAL INVESTIGATION**
**GRASSLANDS NATIONAL PARK**
**FIREHALL REPLACEMENT**
**NE 30-03-13 W3M**
**VAL MARIE, SASKATCHEWAN**

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| **MADE BY:** C.K. | **SCALE:** N/A | **DATE:** NOV 15 | **PROJECT NO.** JX61486 | **PAGE** 4 |

# APPENDIX C

## Hydrometer Grain Size Distribution Graphs

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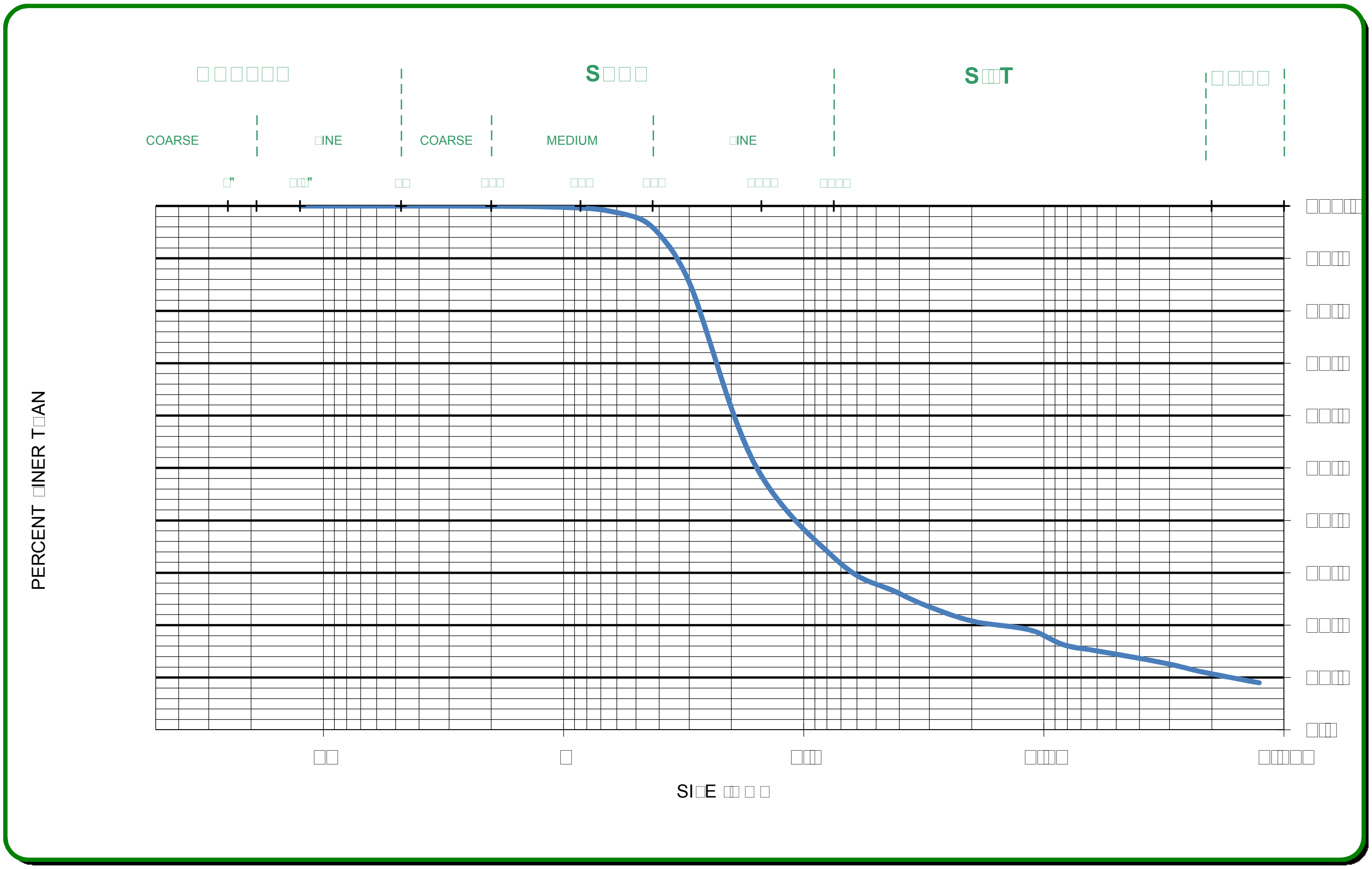


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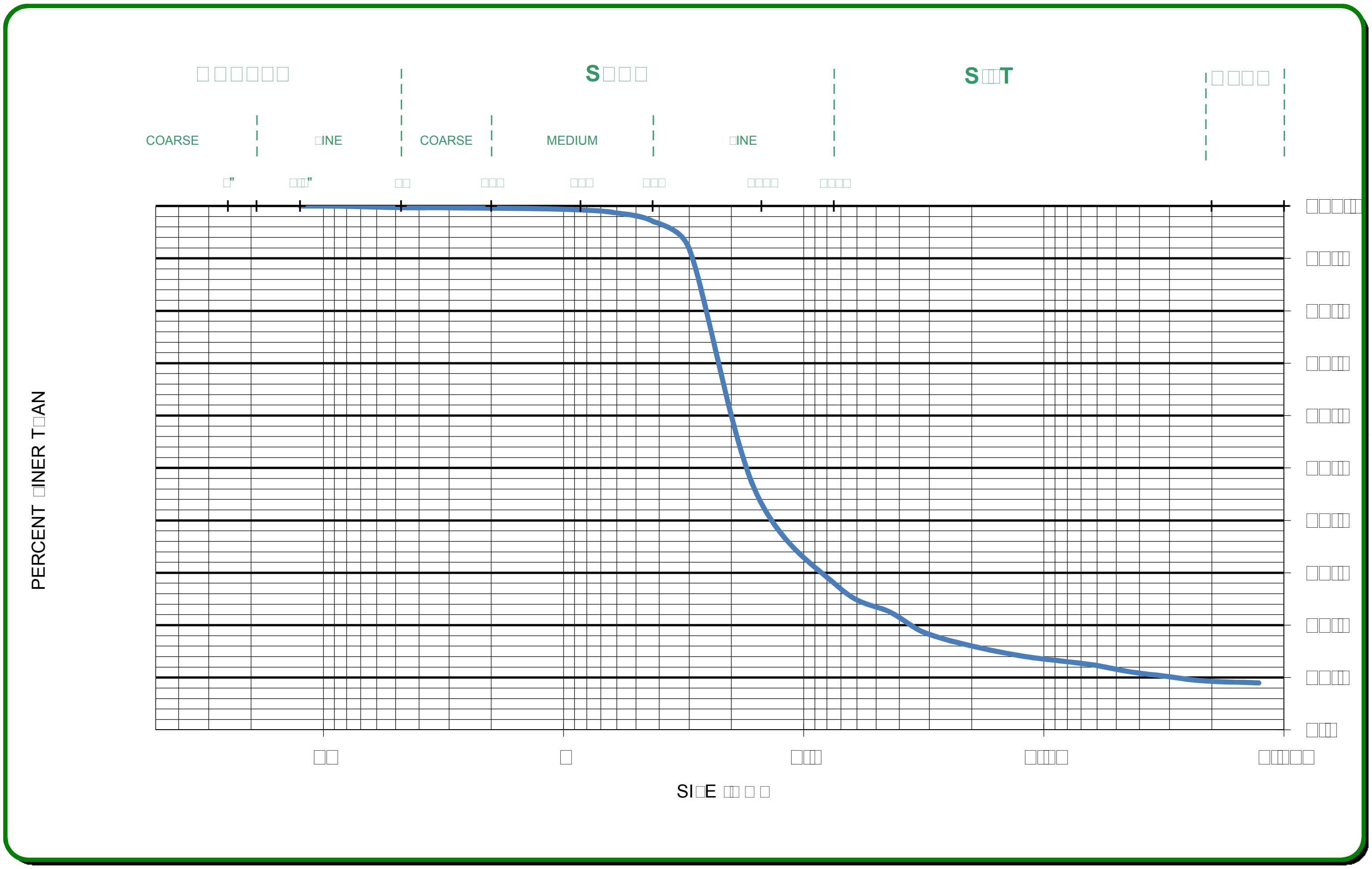


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