SEWPCC Upgrading/Expansion Project: Geotechnical Investigation Report

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SEWPCC Upgrading/Expansion/Civil/Geotech Geotechnical Investigation Report

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1.0 INTRODUCTION

CH2MHill in association with KGS Group and SNC-Lavalin was retained by City of Winnipeg to upgrade the South End Water Pollution Control Centre (SEWPCC) Upgrading/Expansion Project. A geotechnical site investigation program was defined in the SEWPCC Technical Memorandum 7A (TM7A) Project Definition. The purpose of the geotechnical site investigation was to determine subsurface soil, bedrock and groundwater conditions at the site in order to provide geotechnical recommendations for the foundations design of the proposed new expansion structures and related works.

Based upon the TM7A, KGS Group has completed the geotechnical site investigation for the SEWPCC Upgrading/Expansion Project. The main components of the geotechnical investigation consisted of:

1. Review of all pertinent background information including previous reports/studies, Manitoba Water Stewardship's GWDrill database, aerial photos and site photos with respect to the SEWPCC Upgrading/Expansion Project.
2. A geotechnical field investigation consisting of pushing five (5) Cone Penetration Testing with pore pressure response (CPTU) holes, drilling ten (10) test holes with two (2) test holes completed to power auger refusal in till and three (3) of them extended approximately two (2) to three (3) meters into bedrock underneath the till.
3. A groundwater level monitoring program was established for monitoring the groundwater conditions within the overburden soils, till, sand and gravel layers and bedrock with a total installation of eight (8) Casagrande Standpipes (5 in the glacial till/sand and gravel layers, and 3 in the bedrock) and six (6) pneumatic piezometers within the overburden soils.
4. A diagnostic laboratory testing program on selected soil samples.
5. A comprehensive review and analysis based upon all the findings obtained from the field investigation and groundwater monitoring for the foundation assessment.
6. A detailed report outlining the field and laboratory results, alternate foundation options, and geotechnical recommendations for the proposed new expansion structures and their related works of the SEWPCC Upgrading/Expansion Project.

1.1 FINAL REPORT REV.1 – ADDITIONAL INFORMATION

PDA Test Results

In addition to the TM7A, a dynamic load testing on seven (7) pre-cast pre-stressed concrete test piles using the Pile Driving Analyzer (PDA) system was conducted on January 30, 2014. A letter report including the results of the PDA pile load testing was submitted on February 24, 2014 and also included in Appendix B.

The results of the PDA tests on the 406 mm hexagonal pre-cast pre-stressed concrete piles showed the piles can be driven to achieve a total mobilized resistance ranging from 2,100 kN to 2,650 kN. It is therefore, the design of the piles should be based on unfactored unit resistance for pre-cast pre-stressed concrete piles of 2,100 kN with an applicable geotechnical resistance factor, Φ, of 0.5. Based on the PDA pile load testing results, the Driven Pre-Stressed Pre-Cast Concrete Pile Capacity Table in Section 5.2 has been adjusted accordingly.

KGS Group recommends full time on-site pile inspection and PDA tests on 5% to 10% representative production piles should be performed during pile driving operation installation as part of the quality control and quality assurance program that was addressed in Section 5.8.

Phase I Vibration Monitoring Results

During the installation of the seven (7) pre-cast pre-stressed concrete test piles, KGS Group conducted the phase I vibration monitoring program on January 29, 2014. The purpose of this vibration monitoring program is to provide data on vibration attenuation for use in the future during construction and pile installation. The results of the phase 1 vibration monitoring program are included in Appendix C.

Based on the vibration monitoring results, it is unlikely that vibration-induced structural or aesthetic damage would occur to adjacent structures during pile installation. However, KGS Group recommends Phase II vibration monitoring program should be conducted throughout the pile driving operation as part of the quality control and quality assurance program that was addressed in Section 5.8.

1.2 FINAL REPORT REV.2

Table on Pages 13 and 22 has been changed as below:

DRIVEN PRE-STRESSED PRE-CAST CONCRETE PILE CAPACITY (AFTER PDA PILE LOADING TESTING)

Pile Diameter	Factored Serviceability Limit State (SLS) Pile Loading Capacity*	Factored Ultimate Limit State (ULS) Pile Loading Capacity*	Final Set (Blows per 25 mm)**
300 mm	555 kN	650 kN	5
350 mm	780 kN	900 kN	8
400 mm	1050 kN	1200 kN	12

* A Geotechnical Resistance Factor (Φ) of 0.5 is applied.
** If higher energies or other types of hammers are used, they should be evaluated to ensure that piles are not overstressed and suitable refusal criteria to be determined.

Note that the ultimate pile capacity (ULS) value for the 400 mm diameter precast concrete pile was conservatively estimated to be 2,400 kN which was based the PDA test results, the CPT test results and KGS experience. A geotechnical factor of 0.5 was applied to achieve a factored ULS of 1,200 kN.

1.3 FINAL REPORT REV.3

The final report is reissued with the following documents in the Appendices were signed and sealed.

- Appendix B: Pile Load Capacity Verification - PDA Test Results
- Appendix C: Vibration Monitoring for the SEWPCC Test Pile Installation – Phase 1 Vibration Monitoring Program.

2.0 BACKGROUND

2.1 GENERAL

The SEWPCC Upgrading/Expansion project will meet the growing needs of the City of Winnipeg and address increased environmental performance standards requirements. This is the first major project of the Winnipeg Sewage Treatment upgrading program and is part of an overall plan to deliver quality performance and value in the provision of wastewater infrastructure to Winnipeg.

Figures 01 and 02 illustrated the general site plan and layout plan (existing structures, proposed new and proposed future structures) of the SEWPCC Upgrading/Expansion project. Major proposed new structures include:

1. Grit and Screenings Handling and Truck Loading areas,
2. Grit Expansion and Gallery areas,
3. High Rate Clarification of Wet Weather Flow,
4. Two (2) 45.7 m diameter Secondary Clarifiers Units,
5. UV Expansion Building,
6. Three (3) Bioreactors Units and associated Structures,
7. Odour Treatment Building, and
8. By-pass Pipes.

At this stage, the foundation assessment for the proposed future structures (see Figure 02) is out of the scope of work of the TM7A Project Definition and therefore is not included in this report. No major changes in final site grading are anticipated for this expansion.

2.2 GEOTECHNICAL REVIEW

Geotechnical review as conducted for the SEWPCC Upgrading/Expansion Project included:

1. Geohydrology of the metropolitan Winnipeg Area as Related to Groundwater Supply and Construction, by Frank Render, Canadian Geotechnical Journal, Volume 7, 1970.
2. Report on Subsoil Investigation Proposed South End Pollution Control Centre, Winnipeg, Manitoba, by Ripley, Klohn & Leonoff International Ltd. March 8, 1971.
3. Report on Installation of Test Caissons at South End Pollution Control Centre, Winnipeg, Manitoba, by Ripley, Klohn & Leonoff International Ltd. March 24, 1971.

4. Test Holes Drilled at Outfall Stage Associated with South End Pollution Control Centre, Winnipeg, Manitoba, by Ripley, Klohn & Leonoff International Ltd. April 14, 1971.
5. Report on Solution to Problems in Connection with Control of Groundwater & Excavation at the South End Water Pollution Control Centre, Winnipeg, Manitoba, by Ripley, Klohn & Leonoff International Ltd. September 28, 1971.
6. Report on Excavation & Groundwater Control for Pump Well Excavation of the South End Water Pollution Control Centre, Winnipeg, Manitoba, by Ripley, Klohn & Leonoff International Ltd. November 1, 1971.
7. Groundwater Resources in South St. Vital and Northern R.M. of Ritchot, Province of Manitoba Department of Mines, Resources, and Environmental Management, Water Resources Division, 1975.
8. Geological Engineering Report for Urban Development of Winnipeg, Department of Geological Engineering, The University of Manitoba, February 1983.
9. Geotechnical Engineering Report South End Water Pollution Control Centre, Dyregrov and Burgess, April 15, 1988 (Soil logs 1 to 12 only).
10. Geotechnical Report Proposed Disinfection Building South End Water Pollution Control Centre, City of Winnipeg, Dyregrov Consultants, February 1998 (Soil logs 1 to 3 only).
11. Geotechnical Report South End Water Pollution Control Centre, Proposed Expansion, Dyregrov Consultants, February 2008.

The above existing geotechnical information is not included in this document, but is available upon request. However, all the test hole locations within the property of SEWPCC are shown on Figures 01 and 02.

3.0 FIELD INVESTIGATION PROGRAM

3.1 TEST HOLE DRILLING AND SAMPLING PROGRAM

A drilling and sampling program consisting of conventional drilling and Cone Penetration testing with pore pressure response (CPTU) was completed between November 18 and 27, 2013 for the SEWPCC Upgrading/Expansion project. Drilling services were provided by Paddock Drilling Ltd. of Brandon, Manitoba, with continuous KGS Group supervision. Locations of the test holes and the CPTU holes are shown on Figures 01 and 02. Prior to the drilling operation, a Job Safety Analysis (JSA) was prepared and submitted to the City of Winnipeg for review and approval. On-site utilities clearance was conducted.

A total of ten (10) test holes and five (5) CPTU holes were conducted. All CPTU holes were pushed and tested to refusal between depths of 15.9 m± (El. 216.0 m, CPT13-07) and 18.3 m± (El 214.4 m, CPT13-05). Five (5) test holes were advanced to power auger refusal in till to depths between 17.4 m± (El. 214.5 m, TH13-13) and 23.5 m± (El. 208.5 m, TH13-14) and three (3) of the five test holes (TH13-02, TH13-03 and TH 13-13) were extended 2.0 m± to 2.7 m± into bedrock underneath the till between depths of 22.0 m± (El. 209.9 m, TH13-13) and 22.9 m± (El. 209.9 m, TH13-03). Five (5) 12.2 m deep test holes (TH13-08 through TH13-12) were drilled along the proposed by-pass pipe vicinity areas. The CPTU holes and the test holes were advanced using a truck mounted B-59 drill rig and Acker SS drill rig with 125 mm diameter solid stem continuous flight augers. The bedrock was cored with an HQ (63 mm diameter) sized core barrel.

Representative soil samples were obtained in all test holes at 1.5 m (5 ft) intervals, or at any change in soil strata. Soil samples were collected directly off the auger flights and visually classified in the field following the Unified Soil Classification System (USCS). Clay samples were tested with a field Torvane to evaluate consistency and estimate the undrained shear strength. Standard Penetration Tests (SPT's) were performed in the till to determine its relative in-situ density. Upon completion of the drilling, each test hole was examined for indications of sloughing and seepage.

All power auger refusal test holes were backfilled with bentonite grout in the overburden soil layer. The bedrock-cored test holes were backfilled with grout seal through the bedrock, till and up into the overburden. The shallow test hole were backfilled with bentonite chips at the top and bottom of the hole, and auger cuttings in the middle.

Detailed summary soil logs incorporating all field observations plus instrumentation installation details and the CPTU logs are provided in Appendix A.

3.2 INSTRUMENTATION

A total installation of eight (8) Casagrande Standpipe piezometers (5 in the glacial till/sand and gravel layers, and 3 in the bedrock) were installed to obtain direct groundwater measurements. In addition, six (6) pneumatic piezometers were installed within the overburden clays. These piezometers were used for groundwater monitoring within the overburden soils, till, sand and gravel layers, and bedrock of the SEWPCC Upgrading/Expansion project site prior to and during the construction period. Details of the piezometer installations are provided on the test hole logs in Appendix A.

3.3 LABORATORY TESTING

A diagnostic laboratory testing program was performed on representative soil samples to determine the engineering properties of the subsurface soils relative to the assessment. Diagnostic testing included forty one (41) moisture content, five (5) Atterberg Limits, and six (6) grain size analyses. The results of the testing are shown on the test hole logs and included in Appendix A.

4.0 SITE STRATIGRAPHY AND GROUNDWATER CONDITIONS

4.1 SITE STRATIGRAPHY

In general, the stratigraphy at the site consisted of various thicknesses of fill and topsoil, underlain by lacustrine clay, glacial till, layers of sand and gravel, and limestone bedrock.

4.1.1 Topsoil and Fills

Topsoil and fills were encountered up to a depth of 1.5 m± (TH13-02). The topsoil consisted mainly of black organic clays. The fills were silty clays which were brown in colour, moist, stiff in consistency, intermediate to high plasticity, with a trace of coarse grained sand and gravel. The depth of fill ranged from 0.4 m± (TH13-09) to 1.5 m± (TH13-02).

4.1.2 Silty Clay

Silty clay was encountered underneath the fill materials to Elevations between 216.2 m± (TH13-13) and 219.1 m± (TH13-03) or to depths of 13.7 m (TH13-03) and 15.7 m (TH13-13) below ground surface. The silty clay was of high plasticity, was brown to grey in colour, moist, stiff to firm in relative consistency to depths of 6 m± to 8 m±, then becoming softer with depth, and contained trace amounts of silt nodules and till inclusions. The undrained shear strength of the silty clay, determined from the field Torvane on disturbed auger cutting samples, ranged from 90 kPa near top of the layer to 20 kPa near the till contact.

The Moisture content ranged from 41.5% to 63.1%, with an average of 51.0%. Atterberg Limit testing of five (5) samples indicated a liquid limit of 81% to 99% and a plasticity index of 54% to 66% with the materials being classified as CH, Fat Clay. Note that, in general, the Winnipeg lacustrine clays are considered highly expansive in nature, which means there is a significant potential to swell or shrink under changing groundwater conditions. The clay is also soft and compressible below the 8 m depth.

In the upper zone of the silty clay soil profile, in eight (8) of the ten (10) drilled test holes, contained a silt layer of variable thicknesses up to 0.5 m± (TH13-11), beginning at depths between 0.4 m± and 2.4 m± below grade. Various thicknesses of silt layers were also identified from the CPTU results up to a depth of 2.5 m± (CPTU13-04, CPTU13-05 and CPTU13-06) below surface.

4.1.3 Silt Till (Glacial Till)

Till was encountered below the silty clay at Elevations between 216.2 m± (TH13-13) and 219.1 m± (TH13-03). The till was light grey in colour, moist to wet, and compact to very dense in relative density based upon Standard Penetration Tests (SPT) in the till. The till matrix was dominated by silt with some fine to coarse grained gravel, some coarse to fine grained sand, a trace of clay and occasional cobbles and boulders. Power auger refusal was encountered in the dense till between Elevations of 213.5 m± and 214.5 m±. The Moisture content ranged from 7.7% to 20.3%, with an average of 14.6%. Grain size analyses of select samples consisted of 0.4% to 14.1% gravel sized particles, 24.6% to 45.8% sand sized particles and 42.3% to 72.5% silt and clay sized particles. Uncorrected SPT blow counts (N) ranged from 11 to 23 at the clay-till interface and increased with depth to N values ranged from 34 to 38. The results of the SPT testing are included in the soil logs in Appendix A.

4.1.4 Sand and Gravel Layers

Layers of sand and gravel were encountered underlying the glacier till between Elevations 212.0 m± (TH13-14) and 214.5 m± (TH13-03 and TH13-13). The sand and gravel were brown in colour, fine to coarse grained, moist to wet, and compact. Power auger refusal was encountered at the interface of the till with the sand and gravel layers in TH13-02, TH13-03 and TH13-13 (between El. 213.5 m± and 214.5 m±) and in the sand and gravel layers in TH13-14 and TH13-15 (between El. 208.4 m± and 209.4 m±).

4.1.5 Limestone Bedrock

The sand and gravel layers was underlain by limestone bedrock with the top of the bedrock surface ranging from 19.5 m± (TH13-02) to 20.9 m± (TH13-032) below ground or at approximately Elevations 212.0 m± (TH13-02, TH13-03 and TH13-13). In general, the bedrock encountered in this upper zone was highly fractured. Tightly spaced horizontal and vertical fractures were observed throughout the bedrock deposit, which is typical of bedrock conditions in Winnipeg and the surrounding area. Localized clay infilling was observed at some joint locations. Rock Quality Designation (RQD) values generally ranged from 0% to 60% resulting in a description of the bedrock quality as Very Poor to fair as summarized in the table below. Detailed Geological Fracture Logs were included in the test hole logs and the rock core photos were included in Appendix A.

TH 13-02		TH 13-03		TH 13-13	
Depth	**RQD**	**Depth**	**RQD**	**Depth**	**RQD**
19.1 – 19.8 m (62.5' – 65.0')	35 Poor	18.6 – 19.8 m (61.0' – 65.0')	0 Very Poor	17.4 – 18.3 m (57.3' – 60.0')	0 Very Poor
19.8 – 21.3 m (65.0' – 70.0')	37 Poor	19.8 – 21.3 m (65.0' – 70.0')	13 Very Poor	18.3 – 19.8 m (60.0' – 65.0')	0 Very Poor
21.3 – 22.3 m (70.0' – 73.0')	0 Very Poor	21.3 – 22.9 m (70.0 – 75.0')	60 Fair	19.8 – 21.3 m (65.0' – 70.0')	45 Poor
				21.3 – 21.9 m (70.0' – 72.0')	37-38 Poor

Based on our previous experience in having completed numerous coring investigations as well as design and construction inspection of deep foundations in Winnipeg, the limestone bedrock conditions can be highly variable over a given project site. The upper bedrock surface can be karstic and solutioned with crevasses and depressions in the bedrock surface, fractures which are infilled with shattered rock, rubble, and soil which can occur locally and unpredictably within the deposit. Zones of highly fractured and soft rock as well as voids and solution deeper cavities within the bedrock are also not uncommon.

4.2 GROUNDWATER CONDITIONS

Groundwater readings were taken on December 5, 2013, approximately two (2) weeks after completion of the geotechnical field investigation, and again on January 13, 2014. The piezometric monitoring results are summarized in Table 1. Groundwater levels in the silty clays ranged from Elevations of 225.0 m± (TH13-13) to 227.3 m± (TH13-13). Measured piezometric levels in the till and the sand and gravel layers were at Elevations of 223.3 m± (TH13-14) to 225.0 m± (TH13-02). Piezometric levels in the limestone bedrock ranged from Elevations of 223.9 m± (TH13-13) to 224.7 m± (TH13-03). Groundwater elevations vary seasonally and annually such that actual levels at the site may differ from those identified in this report.

Review of bedrock hydrographs from the Provincial Groundwater Monitoring Wells in Winnipeg areas show that there has been a trend toward higher groundwater levels since 1970. Typical potentiometric groundwater surface of the aquifer in the SEWPCC site has varied between approximately El. 222.5 m± to El. 225.5 m±, or 8 m± to 10 m± below ground surface, depending on the seasons, consistent with our groundwater level monitoring results. However, based on available long-term Provincial monitoring data, seasonal peaks in groundwater piezometric pressures in the region may be as high as El. 226.5 m± to El. 227.5 m±, such as during spring flood conditions.

5.0 FOUNDATION CONSIDERATIONS

Geotechnical site investigations have been conducted previously in 1970, 1971, 1988 and 2007 for the initial design and construction, as well as for the 1988 and 2007 expansions, with over 50 test holes and two test caissons drilled within the property of the SEWPCC site. Most of the test holes were drilled to till with a high percentage of them terminated at power auger refusal in till. Some were drilled into the bedrock in the vicinity of the existing wet well and pump house locations.

The 2008 Dyregrov Geotechnical Report for the SEWPCC expansion stated that “the geotechnical conditions are best suited to use of hexagonal, pre-stressed, precast concrete piles that are driven to practical refusal in the underlying glacial till. These have been the type of pile which has been used to support the majority of the structures for the existing plant. The variable condition of the glacial till deposit and potential problems related to water seepage and bell instability are factors that render the site unsuitable for widespread use of high capacity cast-in-place concrete caissons and this type of foundation is not recommended.”

It is our understanding that all the proposed heavily loaded new structures for the SEWPCC Upgrading/Expansion project would be supported by driven end bearing piles to practical refusal in the underlying glacial till. The driven piles could be either precast concrete piles or steel H piles. At this stage, for the proposed heavy loading structures, other foundation types such as end bearing cast-in-place caissons and rock socketed caissons are not considered due to the poor upper bedrock conditions and the previously well documented possible groundwater blowout conditions during construction.

5.1 LIMITED STATES DESIGN

Effective October 1, 2012, the City of Winnipeg requires that all foundation design be done in accordance with Limit States Design (LSD) as prescribed in the Manitoba Building Code (MBC) 2011 Edition. The foundation considerations as described in this report follow the LSD guidelines.

Limit States Design requires consideration of two main loading states which are the Ultimate Limit States and the Serviceability Limit States. The Ultimate Limit States (ULS) are primarily concerned with collapse mechanisms of the structure and safety, while the Serviceability Limit States (SLS) present conditions or mechanisms that restrict or constrain the intended use, function or occupancy of the structure under expected service or working loads. Settlements are typically the constraint. For pile foundation design, each loading state prescribes Geotechnical Resistance Factors (Φ) that are based upon the method used to evaluate pile capacity during construction to obtain the Factored Serviceability Limit State (SLS) and Factored Ultimate Limit State (ULS) pile capacity values. A Geotechnical Resistance Factor (Φ) of 0.5 is applied after the PDA pile loading testing.

5.2 DRIVEN PRESTRESSED PRECAST CONCRETE PILES

Hexagonal, pre-stressed, pre-cast concrete end bearing piles are used extensively in Winnipeg and may be assigned with the following factored Ultimate Limit state (ULS) and Serviceability Limit State (SLS) pile loading capacities when driven to practical refusal on the underlying till or bedrock with diesel hammers having a rated energy per blow of not less than 40,000 Joules to final set as follows:

DRIVEN PRE-STRESSED PRE-CAST CONCRETE PILE CAPACITY
(AFTER PDA PILE LOADING TESTING)

Pile Diameter	Factored Serviceability Limit State (SLS) Pile Loading Capacity*	Factored Ultimate Limit State (ULS) Pile Loading Capacity*	Final Set (Blows per 25 mm)**
300 mm	555 kN	650 kN	5
350 mm	780 kN	900 kN	8
400 mm	1050 kN	1200 kN	12

* A Geotechnical Resistance Factor (Φ) of 0.5 is applied.
** If higher energies or other types of hammers are used, they should be evaluated to ensure that piles are not overstressed and suitable refusal criteria to be determined.

Piles can typically be cast in lengths ranging from 10 to 18 m. Pre-boring of a slightly oversized pilot hole typically 50 mm greater than the pile size to approximately 3.0 to 4.0 m below grade at all driven pile locations is considered standard construction practice in Winnipeg to allow for setting up of the piles, and to reduce ground vibration and potential ground heave in large pile groups. If significant squeezing or sloughing of the bore hole occurs during pre-boring then the pre-boring depth may be altered accordingly. To minimize potential rebound or pile heave during driving, the spacing between adjacent piles should be a minimum of three (3) pile diameters from centre to centre. Careful attention will be required during driving, especially as the pile tip approaches bedrock/refusal, to avoid breaking the pile.

It should be assumed by the designer that the tensile strength of these piles is minimal and they have little capacity to the resist bending. The age of the precast pile concrete should be specified to be at least seven days old prior to driving.

5.3 DRIVEN STEEL PILES

Driven steel piles may be used where high load carrying capacity is required or in areas close to the existing building to minimize possible damages by ground vibration causing by driven precast piles, but they are not generally used locally for light and medium loads. Steel H piles driven to practical refusal on the underlying till or bedrock may be assigned a factored ULS capacity of 100 MPa and a factored SLS capacity of 80 MPa, multiplied by the cross sectional area of the steel. Driving shoes should be used for all driven steel piles. It is cautioned that steel H piles typically drive through the till into the bedrock and it can be difficult to determine when adequate resistance (usually skin friction and end bearing) has been achieved. Dynamic pile analysis and PDA testing is required to optimize the actual design of this type of pile.

Full time inspection by experienced geotechnical personal during driving of either precast concrete or steel piles is recommended. A minimum 200 mm void form should be used below all grade beams and pile caps to protect against potential uplift from swelling clay and potential frost heave below perimeter grade beams.

5.4 ADDITIONAL RECOMMENDATIONS FOR DRIVEN PILES

A geotechnical resistance Factor (Φ) of 0.4 was applied to the above noted factored ULS and SLS values based upon the laboratory and in-situ test results. However, analyses with the dynamic and static pile loading testing results can increase the geotechnical resistance factor (Φ) from 0.4 to 0.5 or 0.6 respectively. As the results, the factored ULS and SLS pile capacity values can be increased by 25% (with $\Phi = 0.5$) when PDA testing is completed or by 50% (with $\Phi = 0.6$) if static pile load tests are performed when the tests show positive results. This could reduce the foundation cost by reducing the number of structure piles.

As per the '2008 design' (IFAS BNR Option), over 2,300 structure piles were required for the SEWPCC Upgrading/Expansion project. The cost of the structure piles is estimated to be about $4,000 per pile in 2013 (supply and install). If the dynamic and static pile loading testing results show positive results and the pile capacity values could be increased by 25%, this may result in a reduction of 15% to 20% of the required piles with a saving of 1.5 to 1.6 million dollars of the piling cost for the foundations.

KGS Group therefore recommends conducting Pile Driving Analyzer (PDA) testing and/or static pile load tests to confirm the loading capacity of the driven piles and to allow for the use of higher resistance factors in design. Preferably, these tests should be conducted at the preliminary stage of the foundation design, right after the geotechnical field investigation program. Results of the tests will be used to confirm and to finalize the foundation design for the proposed new structures. If either PDA testing or static load testing is undertaken, they should be completed under the supervision of an experienced geotechnical engineer and KGS Group should review the results of any testing and pile capacities.

KGS Group recommends conducting the pile load testing in two (2) stages. Stage I pile loading test will be to conduct PDA testing followed by CAPWAP analysis for six (6) piles driven on site prior to the preliminary foundation design to confirm the ULS values and to allow for a geotechnical resistance factor of $\Phi = 0.5$ to be applied to the foundation design. If the PDA testing results suggest positive results for the ultimate pile capacity, KGS Group would recommend the Stage II pile loading test with one (1) to three (3) static pile load tests completed to allow a higher geotechnical resistance factor of $\Phi = 0.6$ to be applied to the foundation design.

Seven (7) Pile Driving Analyzer (PDA) tests had been conducted at the locations of the new proposed structures including the clarifier (2 PDA tests), the bioreactors (2 PDA tests), and high rate clarification (3 PDA tests) on January 30, 2014. The preliminary results indicated that the total pile capacity of the 400 mm diameter precast concrete piles ranged from 2,100 kN to 2,650 kN. Driving stresses were well within acceptable limits. The final PDA test results are included in Appendix B.

Downdrag is not a design issue unless fill is being placed but we understand no major changes to site grading are anticipated at this time.

5.5 CAST-IN-PLACE CONCRETE CAISSONS

As mentioned before, at this stage, other foundation types for heavy loaded structures such as end bearing cast-in-place caissons and rock socketed caissons are not recommended due to the poor upper bedrock conditions and the possible groundwater blowout conditions during construction.

5.6 CAST-IN-PLACE CONCRETE FRICTION PILES

Lightly loaded structures can be supported on cast-in-place concrete friction piles which can be designed on the basis of skin friction values with a factored ULS capacity of 20 kPa and a factored SLS capacity of 16 kPa. The top three (3) meters of shaft support should not be accounted for due to potential soil shrinkage around the pile. A minimum pile diameter of 600 mm should be specified. Temporary casings should be used if caving and seepage conditions occur during pile boring and installation. A mixture of skin friction piles and end bearing piles is not recommended, nor groups of skin friction piles.

Foundations which might be subject to freezing conditions should be protected from frost heave effects. The use of flat lying rigid insulation, such as Styrofoam HI, is recommended to prevent frost penetration into the soil around the piles. Alternatively, the pile lengths should be a minimum of eight (8) meters and should contain full length reinforcement regardless of design loads.

5.7 RECOMMENDED FOUNDATION TYPE

Detailed loading requirements of the proposed new structures as mentioned in Section 2.1 were not provided to KGS Group prior to the preparation of this report. However, each of the above foundation types will be suitable to support the proposed new structures with the optimum being a function of the required foundation capacity. Potential settlements with all of the pile types considered in this report are anticipated to be within generally acceptable limits for structures.

Where pre-stressed, precast concrete piles form the foundations, it will be preferable to resist lateral loads with battered piles. In addition, it is recommended that all concrete piles utilize CSA Type HS sulphate resistant cement. Verical steel piles can be designed to resist lateral loads but local practice is generally to batter these piles as well.

5.8 QUALITY CONTROL AND QUALITY ASSURANCE PROGRAM

KGS Group recommends the following quality control and quality assurance (QC/QA) programs for the SEWPCC Upgrading/Expansion project. These programs should be implemented during pile driving for the foundation construction. The QC/QA programs will consist of:

1. On-site pile inspection during the pile driving operation as recommended in Section 5.3. This QC/QA program will provide the pile driving records of all the piles and produce progress reports for the pile driving operation during construction.
2. In additional to the PDA pile load testing mentioned in Section 5.4, PDA testing should be conducted for the pile installation during foundation construction on a minimum 3% of the driven piles to confirm the loading capacity. The PDA testing will also measure/confirm the rated driving energy of the pile hammers, detect any possible broken pile conditions, and allow for establishment of appropriate refusal criteria.
3. Vibration monitoring for the existing structures during pile driving is recommended. The vibration monitoring will consist of two (2) phases. Phase I is to develop the tolerance criteria and attenuation curves that will be used to identify any areas of concern during the pile driving operation. Phase II is an ongoing vibration monitoring program throughout construction. KGS Group maintains all equipment and expertise in house. Phase I vibration monitoring can be conducted during the Pile Driving Analysis (PDA) Testing as mentioned in Section 5.4.

5.9 EXCAVATIONS AND TEMPORARY SHORING

Deep excavations will be required for the majority of the proposed new major structures of the SEWPCC Upgrading/Expansion project. Where structures are located in the open areas, it may be possible to used sloped excavations. Structures adjacent to the existing buildings will require a temporary shoring system. Because excavations and temporary shoring will impact on the construction activities and schedules, KGS Group recommends that the successful contractor be required to submit an excavation and temporary shoring plan which should be prepared by a Manitoba registered Professional Engineer who is skilled in these designs. Design and approval should be followed by regular onsite inspections for stability.

It is our understanding that some of the proposed new major structures of the SEWPCC Upgrading/Expansion project require excavation and construction of project components at invert elevations to approximately El. 225.0 m. Given that the documented typical groundwater piezometric pressures in the bedrock are reported to range from El. 222.5 m± to El. 225.5 m±, the groundwater monitoring results of piezometer levels ranged from El. 223.3 m± to 224.7 m± in the till and the sand and gravel layers, and given a proposed deep construction invert elevation of El. 225.0 m±, there may not be any specific groundwater depressurization requirements associated with the project. However, the excavation and temporary shoring plan should recognize the potential for possible bottom heave of the deeper excavations due to the hydrostatic groundwater pressures within the underlying glacier till, sand and gravel layers and bedrock. The established groundwater monitoring program will be continued to provide groundwater readings for the design of excavation and temporary shoring during construction.

The design of the excavation and temporary shoring should review the soil stratigraphy and piezometric conditions which might prevail at the time of construction. The presence of the silt deposit within the upper portion of the overburden should be considered as sloughing and seepage of exposed excavation faces should be expected during periods of heavy rainfall. Particular attention should be paid to the temporary shoring system adjacent to the existing major structures and facilities. For the preliminary design purposes, the temporary shoring system can be designed on the basis of the active and passive lateral earth pressure coefficients of K_a = 0.6 and K_p = 2.5 respectively. Ground movement behind the temporary shoring system will occur and largely be unavoidable. The amount that will occur cannot be predicted with much accuracy mainly because the ground movement is a function of excavation procedures and workmanship.

5.10 LATERAL EARTH PRESSURE FOR FINAL BACKFILL

Backfill around the proposed new structure walls and any retaining walls should be a clean granular pitrun material with less than 5% fines (passing the #200 sieve). The granular backfill should be compacted uniformly in maximum 150 mm lifts to a density of at least 98% Standard Proctor Maximum Dry Density (SPMDD). The top meter of the backfill should consist of well compacted high plasticity clay to reduce surface runoff infiltration. In addition, the base of the walls should be provided with a filter protected drainage system to prevent hydrostatic pressures build up against walls. Where drainage is not provided, the hydrostatic pressures against wall should be assumed with a groundwater level to be at the surface.

For design purpose, the following lateral earth pressure coefficients are recommended for earth resistance pressures of the retaining structure design.

LATERAL EARTH PRESSURE COEFFICIENTS **Well Graded Compacted Granular ($\Phi = 35^o$)**	
Active Earth Pressure Coefficient	0.27
Passive Earth Pressure Coefficient	3.69
At ‘Rest’ Earth Pressure Coefficient	0.42

Surface live loads should be included if a significant loading is applied within a distance equal to the height of the wall. The lateral earth pressure due to the surface live load should be equal to 50 percent of the vertical pressure due to the surface live load.

5.11 FLOOR SLAB

The proposed new structures may contain floors which may consist of either a slab-on-grade or structural slab construction. The following design is recommended for a slab-on-grade floor:

- Sub-excavate (if required) to the subgrade design elevation and perform proof roll compaction to expose any soft spots. If any soft spots are encountered the in-situ soil should be sub-excavated a minimum 600 mm depth and replaced with compacted granular subbase.

- A minimum 150 mm thick layer of granular base and 300 mm thick layer of subbase should be placed immediately below the slab. All granular should be placed in a maximum 150 mm thick lifts and compacted to 98% Standard Proctor Maximum Dry Density (SPMDD). Granular base and subbase materials should be in accordance with standard City of Winnipeg specifications.
- Depending on the elevations of the foundations, provisions for groundwater control in the vicinity of the foundations may need to be included. The system should include a perimeter and under-floor weeping tile system around the perimeter of the foundations and under the foundations floor leading to a facility sump pit.
- Some movements, potential cracking, and/or differential settlement of the concrete slab is likely to occur with grade supported slabs due to the expansive (swelling and shrinking) nature of the underlying clay.

For structurally supported floor slabs, the slabs should be separated from the underlying subgrade soils by a minimum 200 mm void space (void form) to minimize potential heave due to possible swelling of the underlying clay soils.

5.12 PAVEMENT CONSIDERATIONS

The following is recommended for the construction of pavement at the site:

- Sub-excavate the surfacial soils to the subgrade design elevation and perform proof-roll compaction of the granular fill or silty clay subgrade. Areas that exhibit unsuitable deflection (organic matter and concrete waste) or if unsuitable soils such as silt and soft clays are encountered; they should be sub-excavated an additional 600 mm and replaced with compacted granular subbase.
- For lightly loaded areas a minimum thickness of 300 mm of granular subbase and 150 mm of granular base is recommended with a minimum of 75 mm asphalt pavement.
- For heavily loaded areas a minimum thickness of 450 mm granular subbase and 150 mm granular base is recommended with a minimum of 100 mm asphalt pavement. Granular base and subbase should be placed in maximum 150 mm thick lifts and compacted to 98% SPMDD.
- A light weight non-woven geotextile should be placed as separator on the top of the sub-grade soil prior to placing sub-base and base courses.
- The final ground elevation around the perimeter of the building should be sloped away at a minimum 2% grade, to protect against surface water ponding.

6.0 CONCLUSIONS

1. In general, the stratigraphy at the site consisted of various thicknesses of fill and topsoil overlaying lacustrine clay, glacial till, sand and gravel layers and limestone bedrock.
2. Groundwater levels in the silty clays ranged at Elevations of 225.0 m± (TH13-13) to 227.1 m± (TH13-15). Measured piezometric levels in the till and the sand and gravel layers were at Elevations of 223.3 m± (TH13-14) to 224.7 m± (TH13-02). Piezometric levels in the limestone bedrock ranged from Elevations of 223.9 m± (TH13-13) to 224.6 m± (TH13-03). The established groundwater monitoring program will be continued to provide groundwater readings for the design of excavation and temporary shoring during construction.
3. It is our understanding that some of the proposed new major structures of the SEWPCC Upgrading/Expansion project require excavation and construction of project components at invert elevations to approximately El. 225.0 m. Given that the documented typical groundwater piezometric pressures in the bedrock are reported to range from El. 222.5 m± to El. 225.5 m±, given that the groundwater monitoring results of piezometer levels ranged from El. 223.3 m± to 224.7 m± in the till and the sand and gravel layers, and given a proposed deep construction invert elevation of El. 225.0 m±, there may not be any specific groundwater depressurization requirements associated with the project.
4. All the proposed heavy loading new structures for the SEWPCC Upgrading/Expansion project could be supported by driven end bearing piles to practical refusal in the underlying glacial till or bedrock. The driven piles could be either precast concrete piles or steel H piles. At this stage, for the proposed heavy loading structures, other foundation types such as end bearing cast-in-place caissons and rock socketed caissons are not considered due to the poor upper portion/zone bedrock conditions and the possible groundwater blowout conditions during construction.

7.0 RECOMMENDATIONS

1. Depending on the elevations of the foundations and the season during construction, utilizing a de-watering system to control the possible high groundwater conditions may be required during the excavation for the major structures.
2. Should temporary shoring or bracing of excavations be necessary, then the in-situ silty clay may be assigned active and passive lateral earth pressure coefficients of K_a = 0.6 and K_p = 2.5. The excavation and temporary shoring plan should assess the potential for base heave of the temporary excavation.
3. The proposed major new structures for the SEWPCC Upgrading/Expansion project should be supported by foundations end bearing on the underlying till, sand and gravel layers or limestone bedrock. Suitable foundation types for consideration include driven precast concrete piles and driven steel piles. Lightly loaded structures could be supported on cast-in-place concrete friction piles. The optimum foundation type is a function of the required load carrying capacity.
4. Based on the PDA test results, the pre-stressed pre-cast concrete piles may be assigned load capacities as listed below:

**DRIVEN PRE-STRESSED PRE-CAST CONCRETE PILE CAPACITY
(AFTER PDA PILE LOADING TESTING)**

Pile Diameter	Factored Serviceability Limit State (SLS) Pile Loading Capacity*	Factored Ultimate Limit State (ULS) Pile Loading Capacity*	Final Set (Blows per 25 mm)**
300 mm	555 kN	650 kN	5
350 mm	780 kN	900 kN	8
400 mm	1050 kN	1200 kN	12

* A Geotechnical Resistance Factor (Φ) of 0.5 is applied.
** If higher energies or other types of hammers are used, they should be evaluated to ensure that piles are not overstressed and suitable refusal criteria to be determined.

5. Driven steel piles may be assigned a factored ULS capacity of 100 MPa and a factored SLS capacity of 80 MPa, multiplied by the cross sectional area of the steel, when driven to practical refusal on the underlying till or bedrock. Driving shoes should be used for all driven steel piles.

6. Lightly loaded structures can be supported on cast-in-place concrete friction piles which can be designed on the basis of skin friction values with a factored ULS capacity of 20 kPa and a factored SLS capacity of 16 kPa. The top three (3) meters of shaft support should not be accounted for due to potential soil shrinkage around the pile. A minimum pile diameter of 600 mm should be specified. Temporary casings should be used if caving and seepage conditions occur during pile boring and installation. However, a mixture of skin friction piles and end bearing piles, and groups of skin friction piles are not recommended.
7. End bearing cast-in-place caissons and rock socketed caissons are not recommended due to the poor upper portion/zone bedrock conditions and the potential groundwater blowout conditions during construction.
8. Lateral loads of either precast concrete or steel piles should be resisted with battered piles.
9. Two stages of pile load testing is recommended to allow for increased geotechnical resistance factors to be applied to pile design. Stage I pile loading test will conduct PDA testing followed by CAPWAP analysis for six (6) piles driven on site prior to the preliminary foundation design to confirm the ULS values and to allow for a geotechnical resistance factor of $\Phi = 0.5$ to be applied to the final foundation design. If the PDA testing results suggest positive results of the ultimate pile capacity, Stage II pile loading test should be conducted with three (3) static pile load tests to allow a higher geotechnical resistance factor of $\Phi = 0.6$ to be applied to the foundation design.
10. Quality control and quality assurance (QC/QA) programs as mentioned in Section 5.8 are recommended. In addition full time inspection by experienced geotechnical personnel should be performed throughout construction of foundations.
11. Provisions for groundwater control in the vicinity of the foundations may need to be included.
12. All concrete in contact with soil should utilize sulphate resistance cement (CSA Type 50).
13. Pavement design for lightly loaded traffic areas a minimum thickness of 300 mm of granular subbase and 150 mm of granular A-base includes with minimum 75 mm asphalt. For heavily loaded traffic areas a minimum thickness of 450 mm of granular subbase and 150 mm of granular A-base includes with minimum 100 mm asphalt. If unsuitable subgrade materials such as silt or soft slay is encountered they should be excavation

with additional 600 mm and replaced with compacted granular fill. Alternatively, the use of a geotextile fabric below the granular subbase as separator may be considered.

14. All temporary excavations and shoring should be designed by the contractor's professional engineer to meet all Manitoba Workplace Health and Safety requirements for safety.

8.0 STATEMENT OF LIMITATIONS

8.1 THIRD PARTY USE OF REPORT

This report has been prepared for the SEWPCC Upgrading/Expansion project to whom this report has been addressed and any use a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. KGS Group accepts no responsibility for damages, if any, suffered by a third party as a result of decisions made or actions undertaken based on this report.

8.2 GEOTECHNICAL INVESTIGATION STATEMENT OF LIMITATIONS

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

9.0 REFERENCE

1. Geohydrology of the metropolitan Winnipeg Area as Related to Groundwater Supply and Construction, by Frank Render, Canadian Geotechnical Journal, Volume 7, 1970.
2. Report on Subsoil Investigation Proposed South End Pollution Control Centre, Winnipeg, Manitoba, by Ripley, Klohn & Leonoff International Ltd. March 8, 1971.
3. Report on Installation of Test Caissons at South End Pollution Control Centre, Winnipeg, Manitoba, by Ripley, Klohn & Leonoff International Ltd. March 24, 1971.
4. Test Holes Drilled at Outfall Stage Associated with South End Pollution Control Centre, Winnipeg, Manitoba, by Ripley, Klohn & Leonoff International Ltd. April 14, 1971.
5. Report on Solution to Problems in Connection with Control of Groundwater & Excavation at the South End Water Pollution Control Centre, Winnipeg, Manitoba, by Ripley, Klohn & Leonoff International Ltd. September 28, 1971.
6. Report on Excavation & Groundwater Control for Pump Well Excavation of the South End Water Pollution Control Centre, Winnipeg, Manitoba, by Ripley, Klohn & Leonoff International Ltd. November 1, 1971.
7. Groundwater Resources in South St. Vital and Northern R.M. of Ritchot, Province of Manitoba Department of Mines, Resources, and Environmental Management, Water Resources Division, 1975.
8. Geological Engineering Report for Urban Development of Winnipeg, Department of Geological Engineering, The University of Manitoba, February 1983.
9. Geotechnical Engineering Report South End Water Pollution Control Centre, Dyregrov and Burgess, April 15, 1988 (Soil logs 1 to 12 only).
10. Geotechnical Report Proposed Disinfection Building South End Water Pollution Control Centre, City of Winnipeg, Dyregrov Consultants, February 1998 (Soil logs 1 to 3 only).
11. Geotechnical Report South End Water Pollution Control Centre, Proposed Expansion, Dyregrov Consultants, February 2008.
12. Canadian Foundation Engineering Manual, 4th Edition, Canadian Geotechnical Society 2006.

TABLES

TABLE 1
PIEZOMETRIC MONITORING RESULTS

Test Hole:		**TH13-02**		**TH13-03**		**TH13-13**				**TH13-14**			**TH13-15**		
Ground Elevation (m):		**232.46**	**232.46**	**232.84**	**232.84**	**231.85**	**231.85**	**231.85**	**231.85**	**231.85**	**231.85**	**231.85**	**232.546**	**232.546**	**232.546**
Piezometer No.:		**STP**	**STP**	**STP**	**STP**	**STP**	**STP**	**PN35525**	**PN35528**	**STP**	**PN35529**	**PN35527**	**STP**	**PN35526**	**PN35530**
Top of Pipe Elevation (m):		**233.43**	**233.43**	**233.81**	**233.81**	**232.76**	**232.76**			**232.70**			**233.356**		
Tip Elevation (m):		**215.70**	**210.82**	**212.42**	**209.98**	**214.93**	**209.90**	**218.13**	**224.23**	**210.36**	**223.93**	**219.66**	**212.736**	**219.746**	**224.626**
Monitoring Zone:		**Till**	**Bedrock**	**Sand & Gravel**	**Bedrock**	**Till**	**Bedrock**	**Silty Clay**	**Silty Clay**	**Sand & Gravel**	**Silty Clay**	**Silty Clay**	**Sand & Gravel**	**Silty Clay**	**Silty Clay**
Date	**River Level (m)**	**Piezometric Elevation (m)**													
5-Dec-13	-	224.66	224.22	224.61	224.57	223.96	223.93	225.59	227.05	223.27	225.40	225.21	224.12	224.95	226.74
13-Jan-14	-	225.02	224.34	224.71	224.70	224.13	224.14	225.87	227.26	224.37	226.04	225.49	224.46	225.80	227.16
5-Mar-14	-	225.03	224.25	224.61	224.60	224.05	224.05	225.52	226.98	224.25	226.04	225.35	224.34	226.08	227.44

*Table 1_Final Report Rev 1

FIGURES

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SEWPCC UPGRADING/EXPANSION PROJECT
SITE PLAN WITH TEST HOLE LOCATIONS

FEBRUARY 2014
FIGURE 01 REV 0

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

LEGEND:

2013 Drilling

Hole Type

- 🟥 Test Holes
- 🟪 Test Holes into Bedrock with Standpipes
- ■ CPT
- 🟦 Test Holes to Refusal with Standpipes
- ◈ Historical Drilling
- 🟩 PDA Pile Locations

NOTES:

1. Imagery from Chartis, August 2013.
2. Issued with Final Report, February 7, 2014. by TNN.

20 0 20 40 60 80
Metres
SCALE: 1:2,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

Winnipeg

KGS GROUP
CONSULTING ENGINEERS

FileName: P:\Projects\2013\13-0338-002\Dwg\GIS\MXDs\Rev0\Geotech_Investigations_Report\13-0338-002-F01_Rev0.mxd
11"x17" PLOT SCALE 1:1

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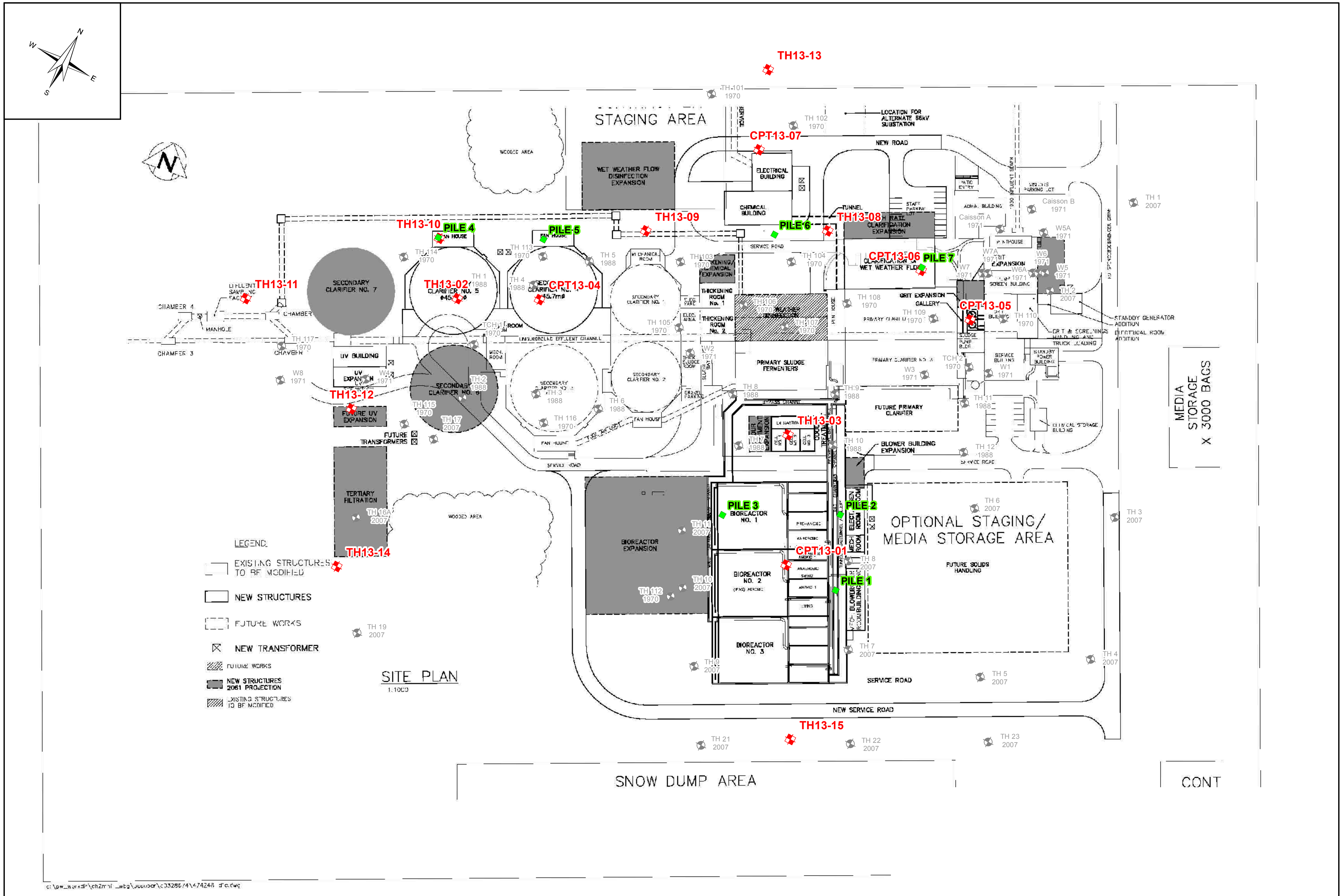


LEGEND:

🟥	2013 Test Hole Locations
⊕	Historical Drilling
🟩	PDA Pile Locations

NOTES:

1. Issued with Draft Report, February 7, 2014, by TNN.

20 0 20 40 60 80
Metres

SCALE: 1:2,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

SEWPCC UPGRADING/EXPANSION PROJECT
PRELIMINARY LAYOUT PLAN
WITH TEST HOLE LOCATIONS

FEBRUARY 2014
FIGURE 01 REV 0

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

Winnipeg

KGS GROUP
CONSULTING ENGINEERS

FileName: P:\Projects\2013\13-0338-002\Dwg\GIS\MXDs\Rev0\Geotech_Investigations_Report\13-0338-002-F02_Rev0.mxd
11"x17" PLOT SCALE 1:1

APPENDICES

APPENDIX A

SOIL LOGS, CPTU RESULTS, CORE PHOTOS AND LAB TESTING RESULTS

KGS GROUP

SUMMARY LOG

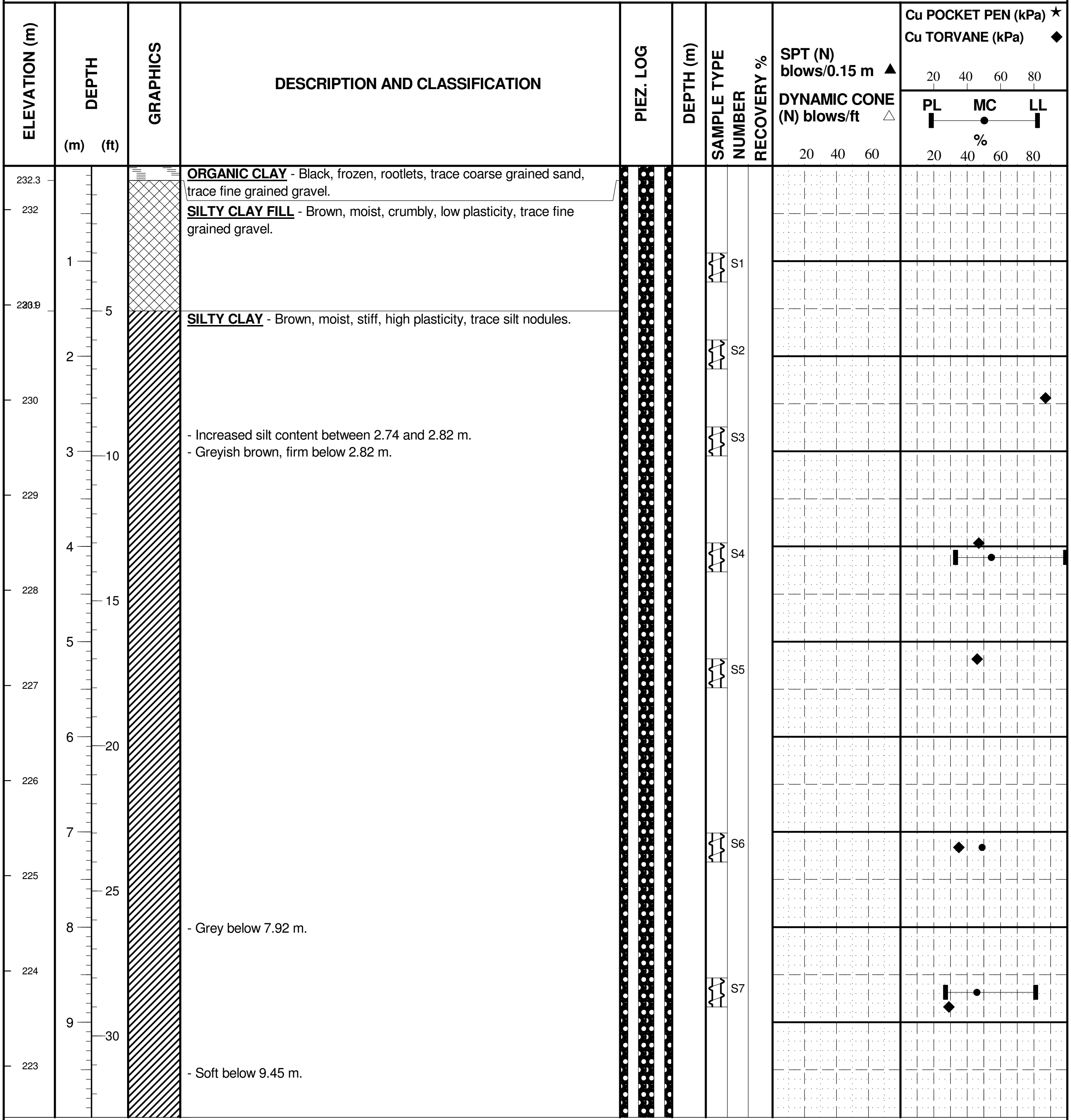
REFERENCE NO.

HOLE NO.
TH13-02

SHEET 1 of 3

CLIENT CH2M HILL/CITY OF WINNIPEG
PROJECT SEWPCC UPGRADING/EXPANSION PROJECT
SITE South End Water Pollution Control Centre
LOCATION
DRILLING METHOD 125 mm ø Solid Stem Auger and HQ Core Barrel, B-59 Drill Rig

JOB NO. 13-0338-002
GROUND ELEV. 232.46 m
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED 11/21/2013
UTM (m) N 5,517,463
E 636,767



ORGANIC CLAY - Black, frozen, rootlets, trace coarse grained sand, trace fine grained gravel.

SILTY CLAY FILL - Brown, moist, crumbly, low plasticity, trace fine grained gravel.

SILTY CLAY - Brown, moist, stiff, high plasticity, trace silt nodules.

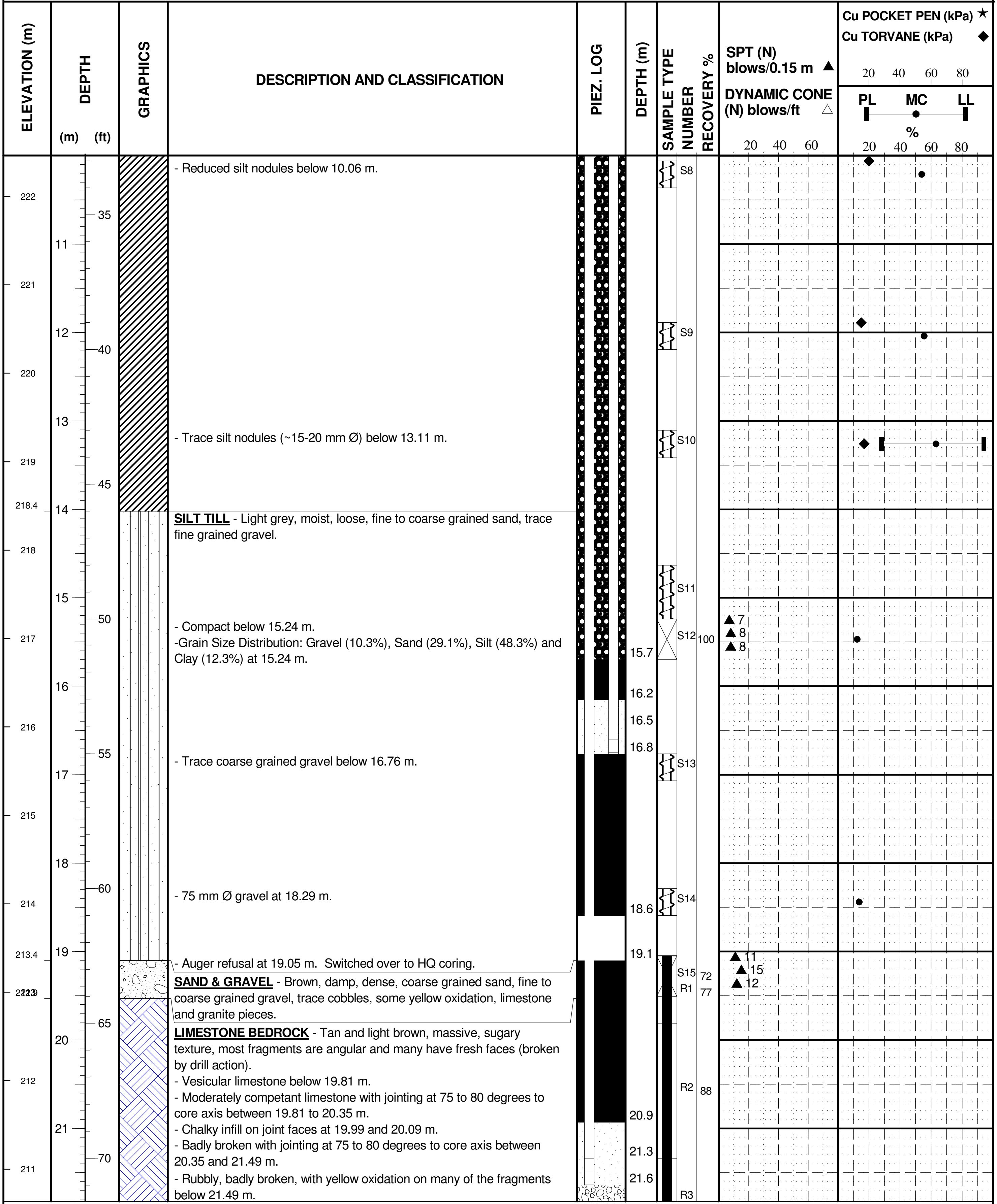
- Increased silt content between 2.74 and 2.82 m.
- Greyish brown, firm below 2.82 m.

- Grey below 7.92 m.

- Soft below 9.45 m.

SAMPLE TYPE ⌇ Auger Grab ☒ Split Spoon ▮ Core Barrel

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14



- Reduced silt nodules below 10.06 m.

- Trace silt nodules (~15-20 mm Ø) below 13.11 m.

SILT TILL - Light grey, moist, loose, fine to coarse grained sand, trace fine grained gravel.

- Compact below 15.24 m.

-Grain Size Distribution: Gravel (10.3%), Sand (29.1%), Silt (48.3%) and Clay (12.3%) at 15.24 m.

- Trace coarse grained gravel below 16.76 m.

- 75 mm Ø gravel at 18.29 m.

- Auger refusal at 19.05 m. Switched over to HQ coring.

SAND & GRAVEL - Brown, damp, dense, coarse grained sand, fine to coarse grained gravel, trace cobbles, some yellow oxidation, limestone and granite pieces.

LIMESTONE BEDROCK - Tan and light brown, massive, sugary texture, most fragments are angular and many have fresh faces (broken by drill action).

- Vesicular limestone below 19.81 m.

- Moderately competant limestone with jointing at 75 to 80 degrees to core axis between 19.81 to 20.35 m.

- Chalky infill on joint faces at 19.99 and 20.09 m.

- Badly broken with jointing at 75 to 80 degrees to core axis between 20.35 and 21.49 m.

- Rubbly, badly broken, with yellow oxidation on many of the fragments below 21.49 m.

SAMPLE TYPE ⌇ Auger Grab ☒ Split Spoon ■ Core Barrel

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.15 m ▲ DYNAMIC CONE (N) blows/ft △ 20 40 60	Cu POCKET PEN (kPa) ★ Cu TORVANE (kPa) ◆ 20 40 60 80 PL MC LL % 20 40 60 80
210.2	22		▨			22.3	▮		33		
210		75		**END OF TEST HOLE AT 22.25 m.**							
209	23										
208	24	80									
207	25										
206	26	85									
205	27	90									
204	28										
203	29	95									
202	30	100									
201	31										
200	32	105									
199	33	110									

Notes:
1. Water level noted at 17.53 m below grade after drilling to 19.05 m.
2. Installed Casagrande standpipe in the bedrock at a depth of 21.64 m and a Casagrande standpipe in the till at a depth of 16.76 m. Both standpipes have a stick-up of 0.97 m.
3. Backfilled test hole with sloughed rock from 22.25 to 21.64 m, silica sand from 21.64 to 20.93 m, bentonite chips from 20.93 to 19.10 m, slough from 19.10 to 18.59 m, bentonite chips from 18.59 to 16.76 m, silica sand from 16.76 to 16.15 m, bentonite chips from 16.15 to 15.70 m and a bentonite grout mixture from 15.70 m to grade.

SAMPLE TYPE: Auger Grab ☒ Split Spoon ▮ Core Barrel

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

KGS GROUP

SUMMARY LOG

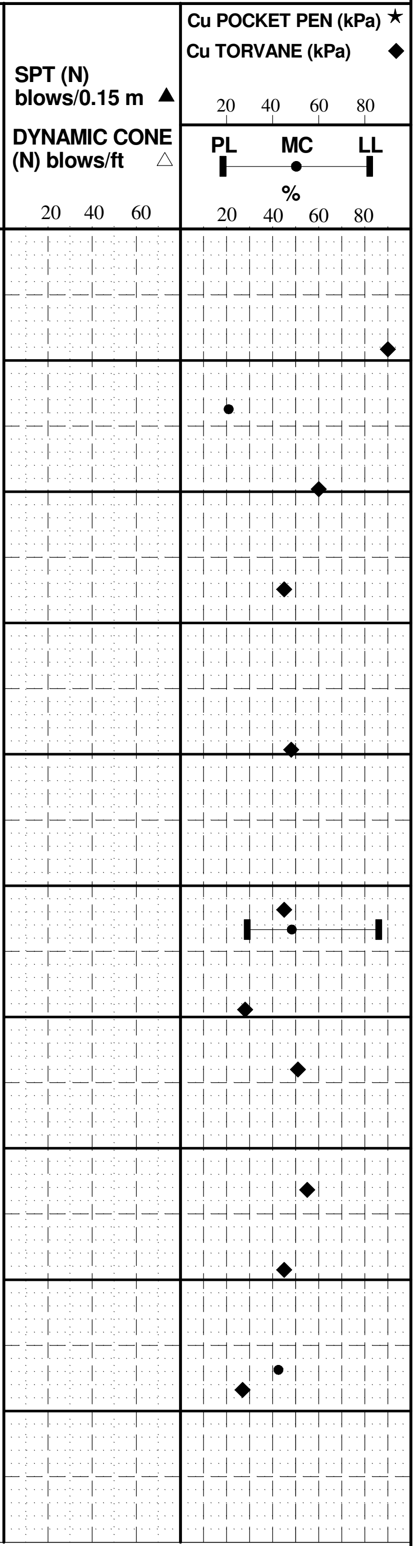
REFERENCE NO.

HOLE NO.
TH13-03

SHEET 1 of 3

CLIENT	CH2M HILL/CITY OF WINNIPEG	JOB NO.	13-0338-002
PROJECT	SEWPCC UPGRADING/EXPANSION PROJECT	GROUND ELEV.	232.84 m
		TOP OF PVC ELEV.	
SITE	South End Water Pollution Control Centre	WATER ELEV.	
LOCATION		DATE DRILLED	11/21/2013
DRILLING METHOD	125 mm ø Solid Stem Auger and HQ Core Barrel, B-59 Drill Rig	UTM (m)	N 5,517,482 E 636,943

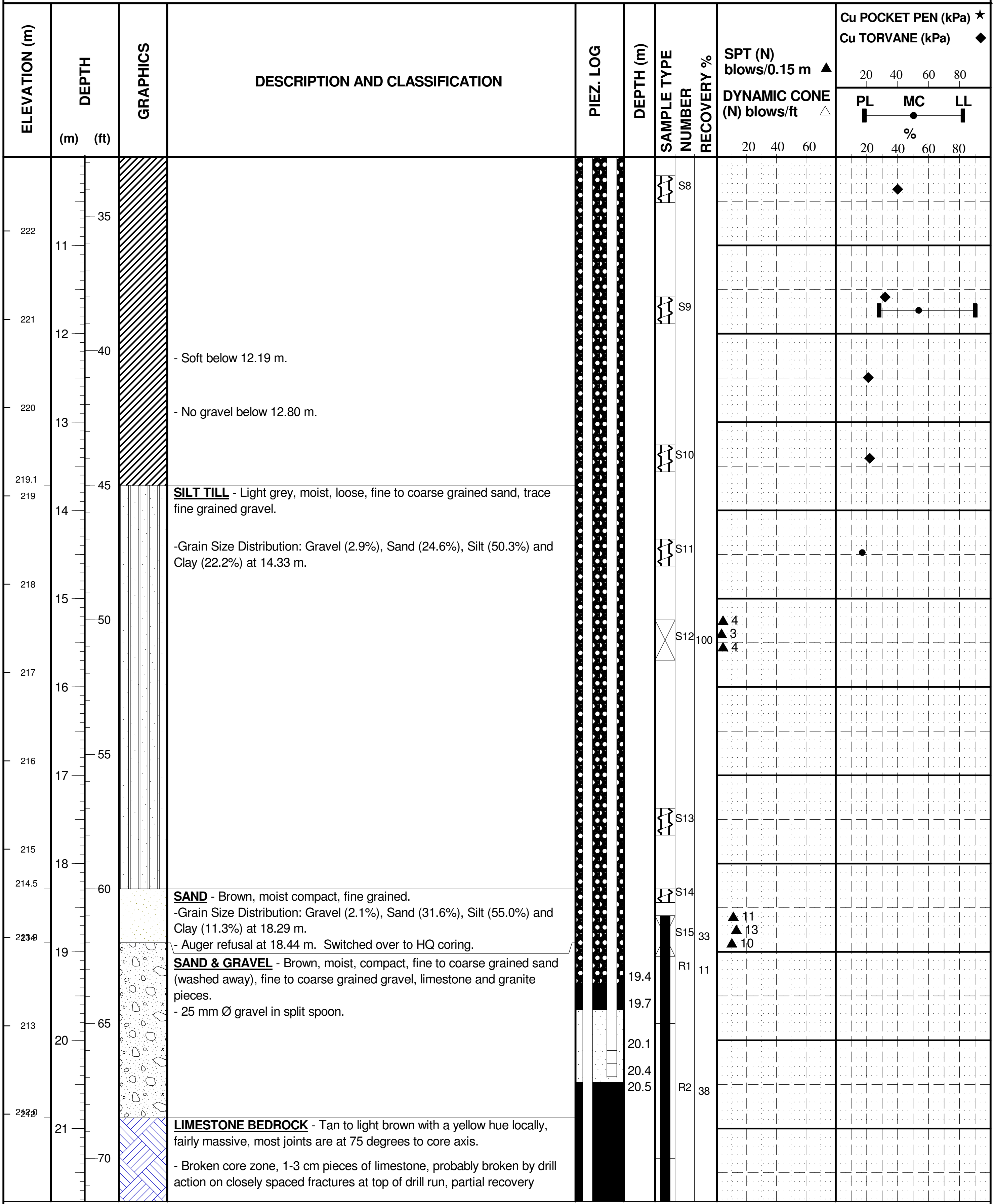
ELEVATION (m)	DEPTH (m)	DEPTH (ft)	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER
			ORGANIC CLAY - Black, frozen, crumbly, rootlets, trace coarse grained sand, trace fine grained gravel.		
232.7			**SILTY CLAY** - Brown, moist, stiff, intermediate to high plasticity, trace silt nodules.		
232	1			Auger Grab	S1
231.6			**SILT** - Tan, moist, firm, low plasticity.	Auger Grab	S2
231.3		5	**SILTY CLAY** - Brown, moist, stiff, high plasticity, trace silt nodules, trace gypsum nodules.		
231	2				
			- Firm below 2.44 m.	Auger Grab	S3
230	3	10			
229	4			Auger Grab	S4
		15			
228	5			Auger Grab	S5
227	6	20			
			- Stiff below 6.40 m.		
226	7			Auger Grab	S6
		25	- Firm below 7.62 m.		
225	8				
			- Grey, no gypsum below 8.38 m.	Auger Grab	S7
224	9	30			
			- Trace fine grained gravel below 9.45 m.		
223					



SAMPLE TYPE: Auger Grab, Split Spoon, Core Barrel

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

GEOTECHNICAL-SOIL LOG P:\PROJECTS\2013\13-0338-002\DESIGN\GEO\LOGS\SEWPCC.GPJ



- Soft below 12.19 m.

- No gravel below 12.80 m.

SILT TILL - Light grey, moist, loose, fine to coarse grained sand, trace fine grained gravel.

-Grain Size Distribution: Gravel (2.9%), Sand (24.6%), Silt (50.3%) and Clay (22.2%) at 14.33 m.

SAND - Brown, moist compact, fine grained.
-Grain Size Distribution: Gravel (2.1%), Sand (31.6%), Silt (55.0%) and Clay (11.3%) at 18.29 m.
- Auger refusal at 18.44 m. Switched over to HQ coring.

SAND & GRAVEL - Brown, moist, compact, fine to coarse grained sand (washed away), fine to coarse grained gravel, limestone and granite pieces.
- 25 mm Ø gravel in split spoon.

LIMESTONE BEDROCK - Tan to light brown with a yellow hue locally, fairly massive, most joints are at 75 degrees to core axis.

- Broken core zone, 1-3 cm pieces of limestone, probably broken by drill action on closely spaced fractures at top of drill run, partial recovery

SAMPLE TYPE			
	Auger Grab	Split Spoon	Core Barrel

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO.
TH13-03

SHEET 3 of 3

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.15 m ▲ / DYNAMIC CONE (N) blows/ft △ (20 40 60)	Cu POCKET PEN (kPa) ★ / Cu TORVANE (kPa) ◆ (20 40 60 80) / PL MC LL % (20 40 60 80)
	22		▨	between 21.34 and 21.51 m. - Open joint, irregularily shaped joint face, probable location of core barrel drop and lost water return, joint appears open but not altered or oxidized at 22.12 m.		22.1 22.6 22.9	■	R3	94		
210.0 210	23	75		**END OF TEST HOLE AT 22.86 m.**							
209	24										
208	25	80									
207	26	85									
206	27										
205	28	90									
204	29	95									
203	30										
202	31	100									
201	32	105									
200	33	110									

Notes:
1. Installed Casagrande standpipe in the bedrock at a depth of 22.86 m and a Casagrande standpipe in the sand & gravel at a depth of 20.42 m. Both standpipes have a stick-up of 0.97 m.
2. Backfilled test hole with silica sand from 22.86 to 22.10 m, bentonite chips from 22.10 to 20.47 m, silica sand from 20.47 to 19.66 m, bentonite chips from 19.66 to 19.35 m and a bentonite grout mixture from 19.35 m to grade.
3. The driller noted that the core barrel dropped and he lost circulation around 22.25 m.

SAMPLE TYPE ⌇ Auger Grab ☒ Split Spoon ■ Core Barrel

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

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KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO.
TH13-08

SHEET 1 of 2

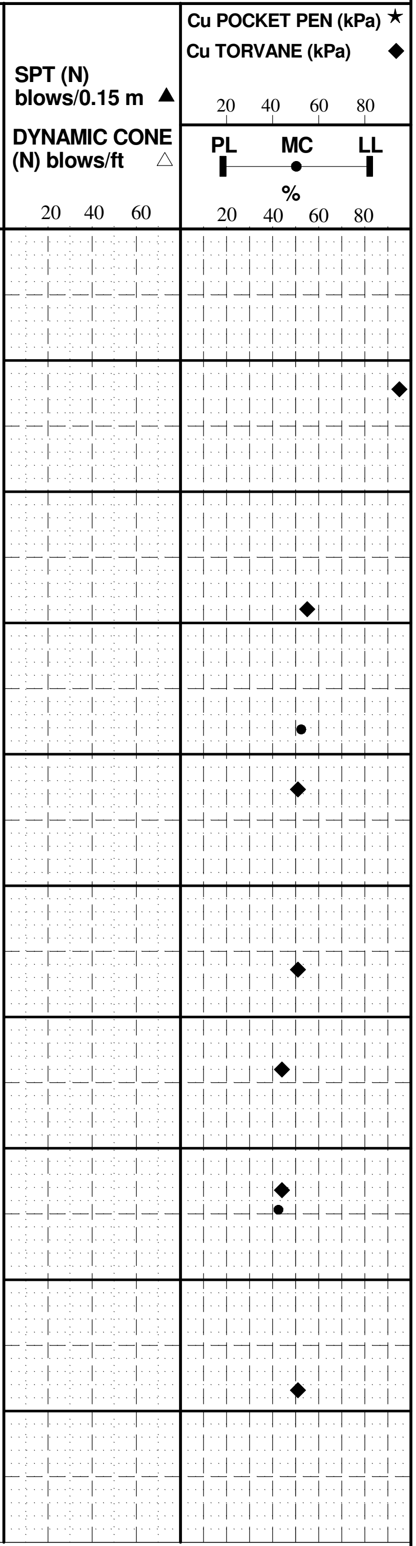
CLIENT CH2M HILL/CITY OF WINNIPEG
PROJECT SEWPCC UPGRADING/EXPANSION PROJECT
SITE South End Water Pollution Control Centre
LOCATION
DRILLING METHOD 125 mm ø Solid Stem Auger, ACKER SS Drill Rig

JOB NO. **13-0338-002**
GROUND ELEV. **232.56 m**
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED **11/26/2013**
UTM (m) N **5,517,580**
E **636,912**

ELEVATION (m)	DEPTH (m) (ft)	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY %
		SILTY CLAY FILL - Brown, frozen to 0.15 m then moist, stiff, intermediate to high plasticity, trace coarse grained sand, trace fine grained gravel.			
231.8		**SILTY CLAY** - Brown, moist, stiff, high plasticity, trace silt nodules.		S1	
230.3		**SILT** - Tan, moist, firm, low plasticity.		S2	
230.1		**SILTY CLAY** - Brown, moist, stiff, high plasticity, trace silt nodules.			
				S3	
				S4	
		- Firm below 6.10 m.			
				S5	
		- Grey between 8.23 and 8.84 m.			
				S6	
		- Grey, trace medium grained sand below 9.75 m.			

SPT (N) blows/0.15 m ▲
DYNAMIC CONE (N) blows/ft △

Cu POCKET PEN (kPa) ★
Cu TORVANE (kPa) ◆
PL MC LL
%



SAMPLE TYPE Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

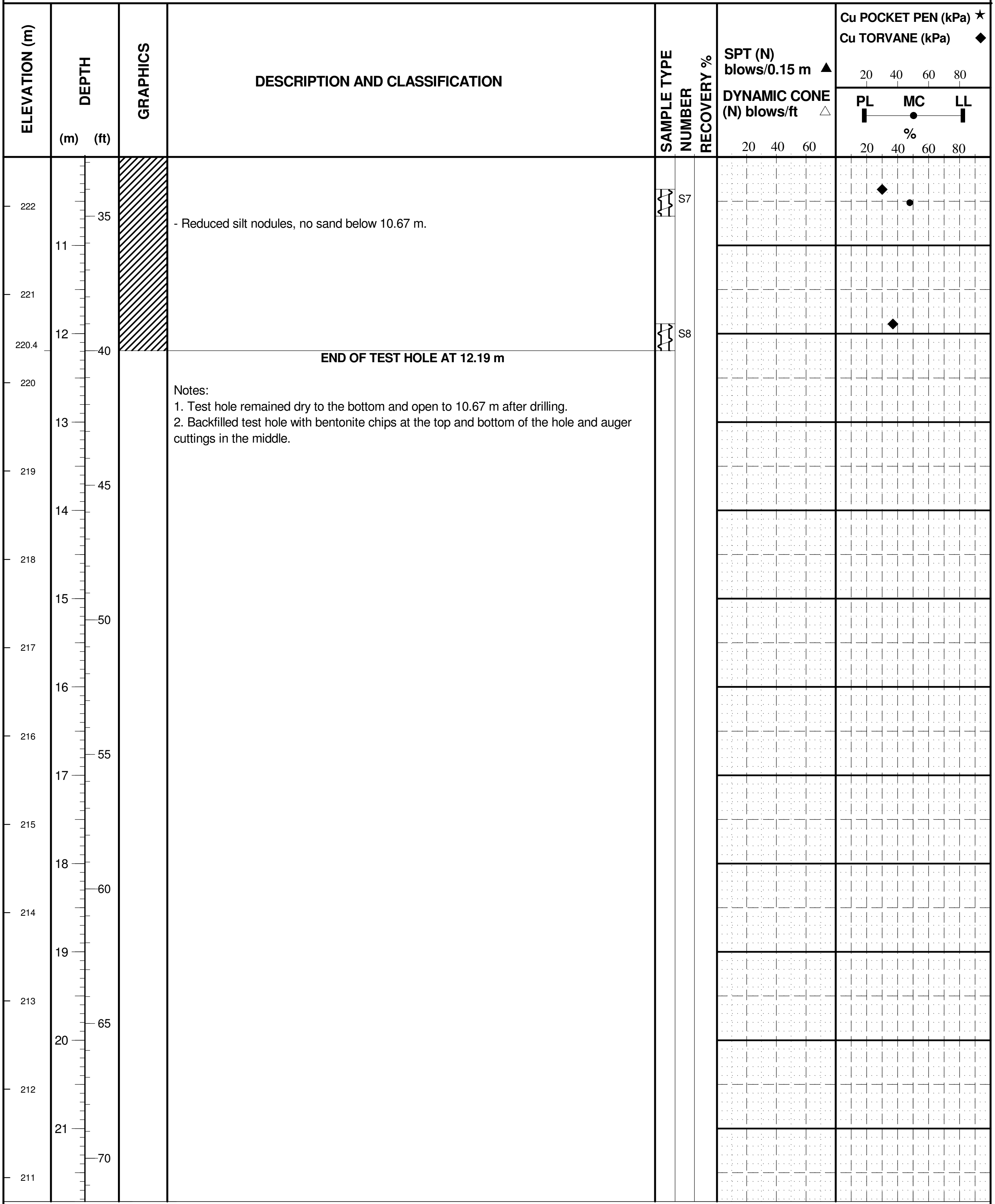
KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH13-08**

SHEET 2 of 2



- Reduced silt nodules, no sand below 10.67 m.

END OF TEST HOLE AT 12.19 m

Notes:
1. Test hole remained dry to the bottom and open to 10.67 m after drilling.
2. Backfilled test hole with bentonite chips at the top and bottom of the hole and auger cuttings in the middle.

SAMPLE TYPE {[Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

GEOTECHNICAL-SOIL LOG P:\PROJECTS\2013\13-0338-002\DESIGN\GEO\LOGS\SEWPCC.GPJ

KGS GROUP

SUMMARY LOG

REFERENCE NO.

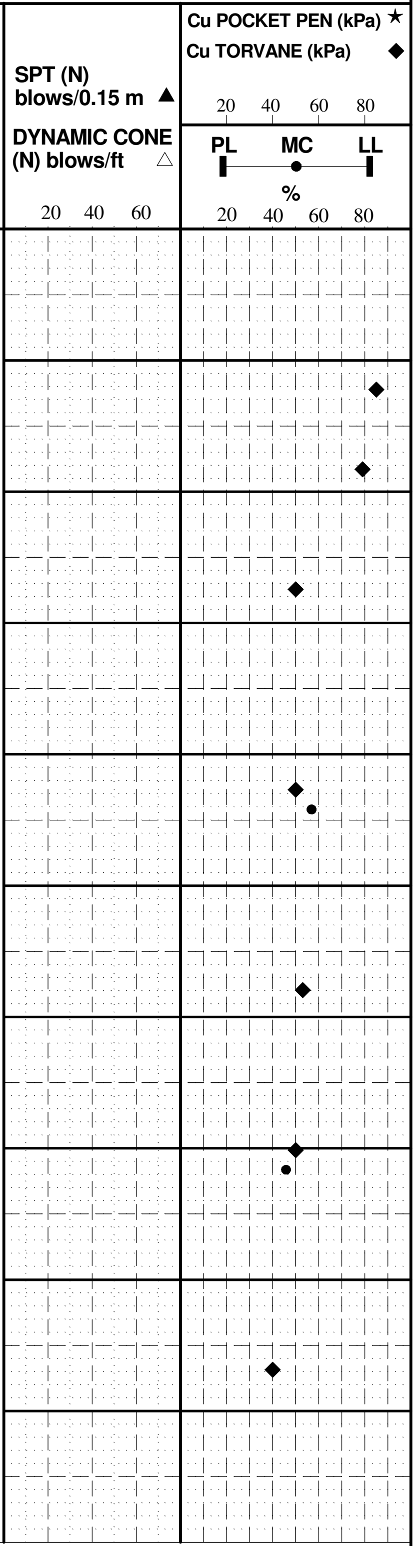
HOLE NO.
TH13-09

SHEET 1 of 2

CLIENT CH2M HILL/CITY OF WINNIPEG
PROJECT SEWPCC UPGRADING/EXPANSION PROJECT
SITE South End Water Pollution Control Centre
LOCATION
DRILLING METHOD 125 mm ø Solid Stem Auger, ACKER SS Drill Rig

JOB NO. 13-0338-002
GROUND ELEV. 232.10 m
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED 11/26/2013
UTM (m) N 5,517,537
E 636,833

ELEVATION (m)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER
232		**SILTY CLAY FILL** - Black, frozen to 0.15 m then moist, stiff, intermediate to high plasticity, trace coarse grained sand, trace fine grained gravel.	Auger Grab	S1
231.7		**SILT** - Tan, moist, firm, low plasticity.		
231.5		**SILTY CLAY** - Brown, moist, stiff, high plasticity, trace silt nodules.		
231	1		Auger Grab	S2
230	2		Auger Grab	S3
229	3			
228	4		Auger Grab	S4
227	5	- Reduced silt nodules below 4.88 m.		
226	6		Auger Grab	S5
225	7		Auger Grab	S6
224	8	- Grey, firm below 7.47 m.		
223	9		Auger Grab	S7



SAMPLE TYPE Auger Grab

CONTRACTOR
Paddock Drilling Ltd.

INSPECTOR
C. FRIESEN

APPROVED
T. NG

DATE
2/6/14

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KGS GROUP

SUMMARY LOG

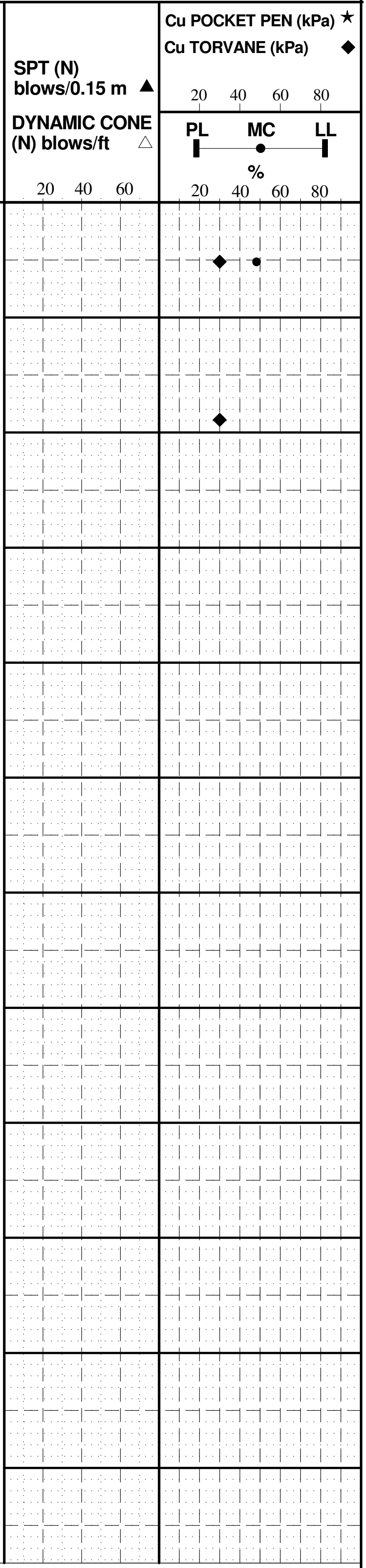
REFERENCE NO.

HOLE NO. **TH13-09**

SHEET 2 of 2

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
222			▨				
		35	▨		{}	S8	
221	11		▨				
	12		▨		{}	S9	
219.9		40		**END OF TEST HOLE AT 12.19 m**			
219	13						
		45					
218	14						
217	15	50					
216	16						
		55					
215	17						
214	18	60					
213	19						
		65					
212	20						
211	21	70					

Notes:
1. Test hole remained dry to the bottom and open to 9.75 m after drilling.
2. Backfilled test hole with bentonite chips at the top and bottom of the hole and auger cuttings in the middle.



SAMPLE TYPE {} Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

KGS GROUP

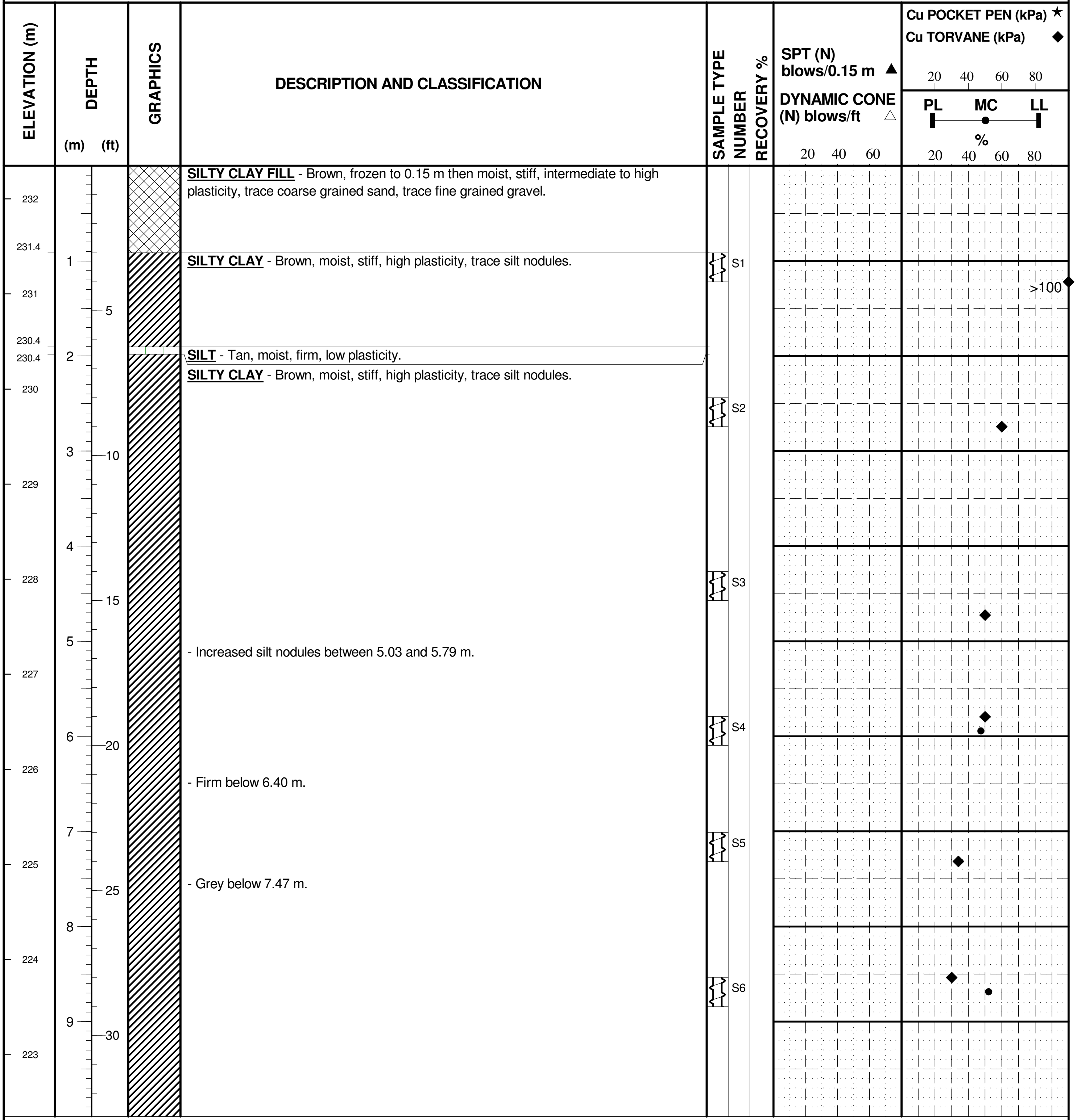
SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH13-10**

SHEET 1 of 2

CLIENT	CH2M HILL/CITY OF WINNIPEG	JOB NO.	13-0338-002
PROJECT	SEWPCC UPGRADING/EXPANSION PROJECT	GROUND ELEV.	232.35 m
		TOP OF PVC ELEV.	
SITE	South End Water Pollution Control Centre	WATER ELEV.	
LOCATION		DATE DRILLED	11/25/2013
DRILLING METHOD	125 mm ø Solid Stem Auger, ACKER SS Drill Rig	UTM (m)	N 5,517,485 E 636,745



Description and Classification
SILTY CLAY FILL - Brown, frozen to 0.15 m then moist, stiff, intermediate to high plasticity, trace coarse grained sand, trace fine grained gravel.
SILTY CLAY - Brown, moist, stiff, high plasticity, trace silt nodules.
SILT - Tan, moist, firm, low plasticity.
SILTY CLAY - Brown, moist, stiff, high plasticity, trace silt nodules.
- Increased silt nodules between 5.03 and 5.79 m.
- Firm below 6.40 m.
- Grey below 7.47 m.

SAMPLE TYPE ⌇ Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

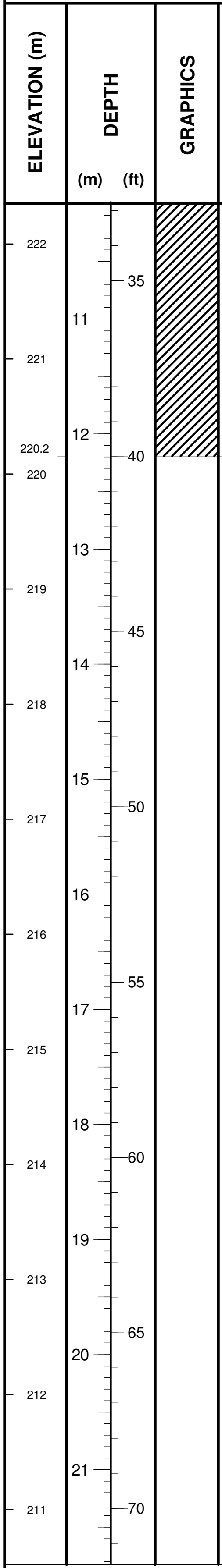
KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO.
TH13-10

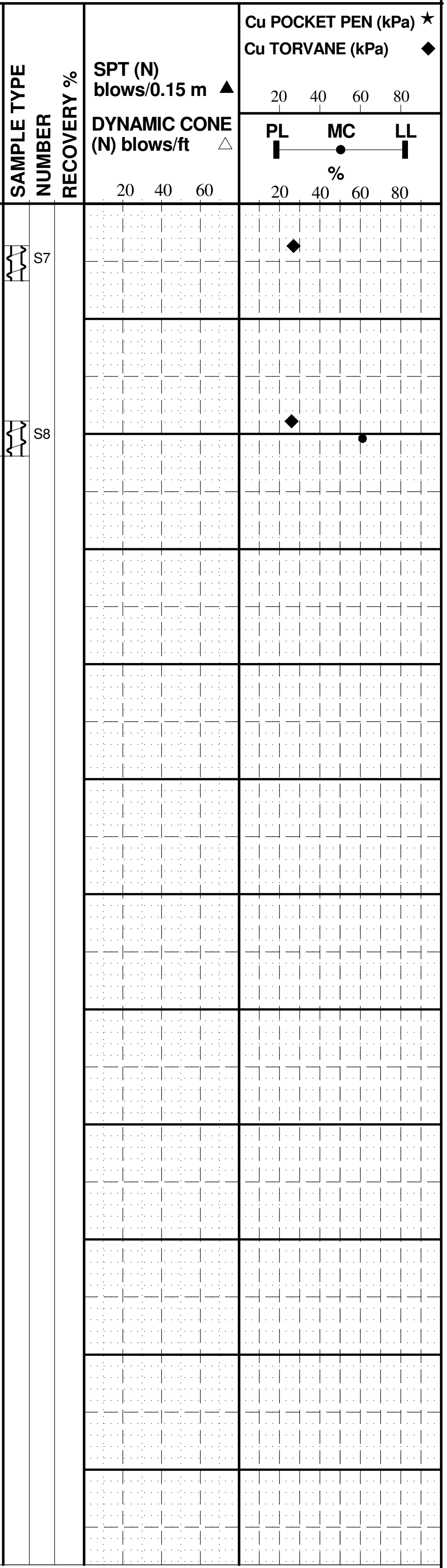
SHEET 2 of 2



DESCRIPTION AND CLASSIFICATION

END OF TEST HOLE AT 12.19 m

Notes:
1. Test hole remained dry to the bottom and open to 12.19 m after drilling.
2. Backfilled test hole with bentonite chips at the top and bottom of the hole and auger cuttings in the middle.



SAMPLE TYPE ⌇ Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

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KGS GROUP

SUMMARY LOG

REFERENCE NO.

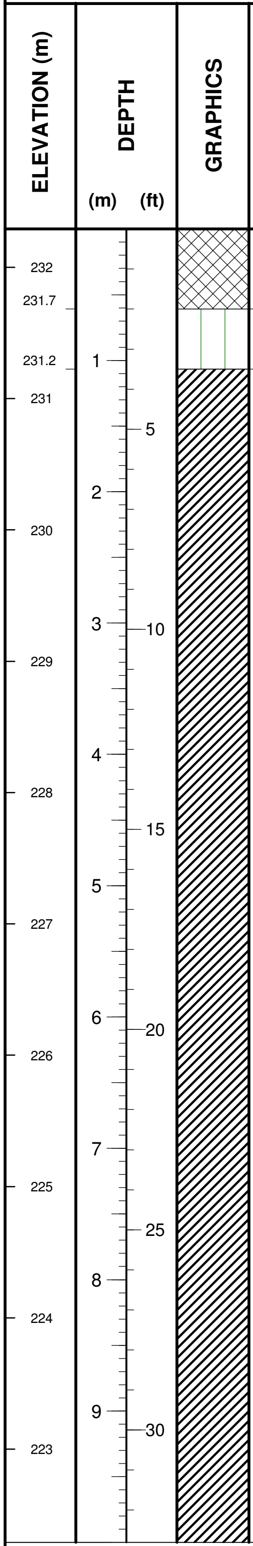
HOLE NO.
TH13-11

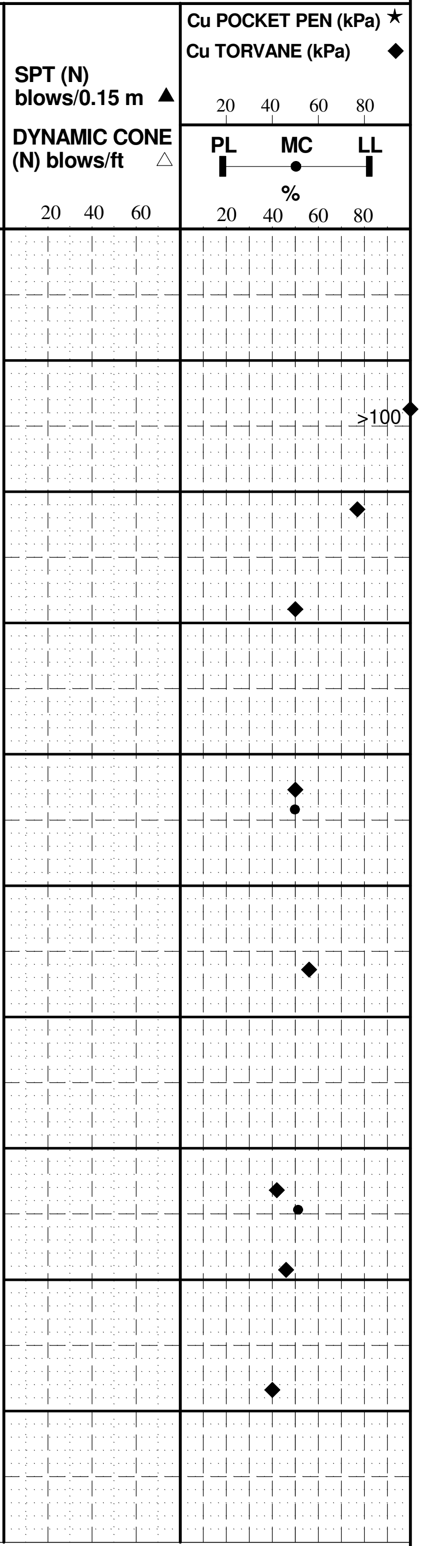
SHEET 1 of 2

CLIENT CH2M HILL/CITY OF WINNIPEG
PROJECT SEWPCC UPGRADING/EXPANSION PROJECT
SITE South End Water Pollution Control Centre
LOCATION
DRILLING METHOD 125 mm ø Solid Stem Auger, ACKER SS Drill Rig

JOB NO. **13-0338-002**
GROUND ELEV. **232.29 m**
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED **11/25/2013**
UTM (m) N **5,517,413**
E **636,675**

ELEVATION (m)	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE NUMBER
	SILTY CLAY FILL - Brown, frozen to 0.15 m then moist, stiff, intermediate to high plasticity, trace coarse grained sand, trace fine grained gravel.	
231.7	**SILT** - Tan, damp, firm, low plasticity.	S1
231.2	**SILTY CLAY** - Brown, moist, stiff, high plasticity, trace silt nodules.	S2 S3 S4 S5
	- Firm below 6.40 m.	S6
	- Grey below 8.00 m.	S7





SAMPLE TYPE Auger Grab

CONTRACTOR
Paddock Drilling Ltd.

INSPECTOR
C. FRIESEN

APPROVED
T. NG

DATE
2/6/14

KGS GROUP

SUMMARY LOG

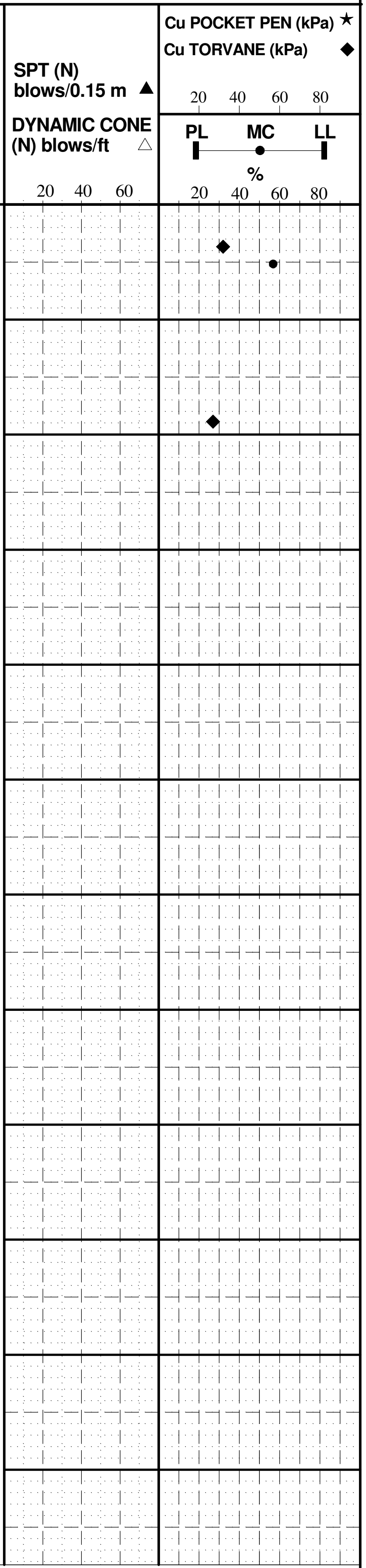
REFERENCE NO.

HOLE NO.
TH13-11

SHEET 2 of 2

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
222		35	▨				
221	11		▨		{}	S8	
	12		▨		{}	S9	
220.1		40		**END OF TEST HOLE AT 12.19 m**			
220	13						
219	14	45					
218	15						
217	16	50					
216	17	55					
215	18						
214	19	60					
213	20	65					
212	21						
211		70					

Notes:
1. Test hole remained dry to the bottom and open to 10.06 m after drilling.
2. Backfilled test hole with bentonite chips at the top and bottom of the hole and auger cuttings in the middle.



SAMPLE TYPE {} Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

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KGS GROUP

SUMMARY LOG

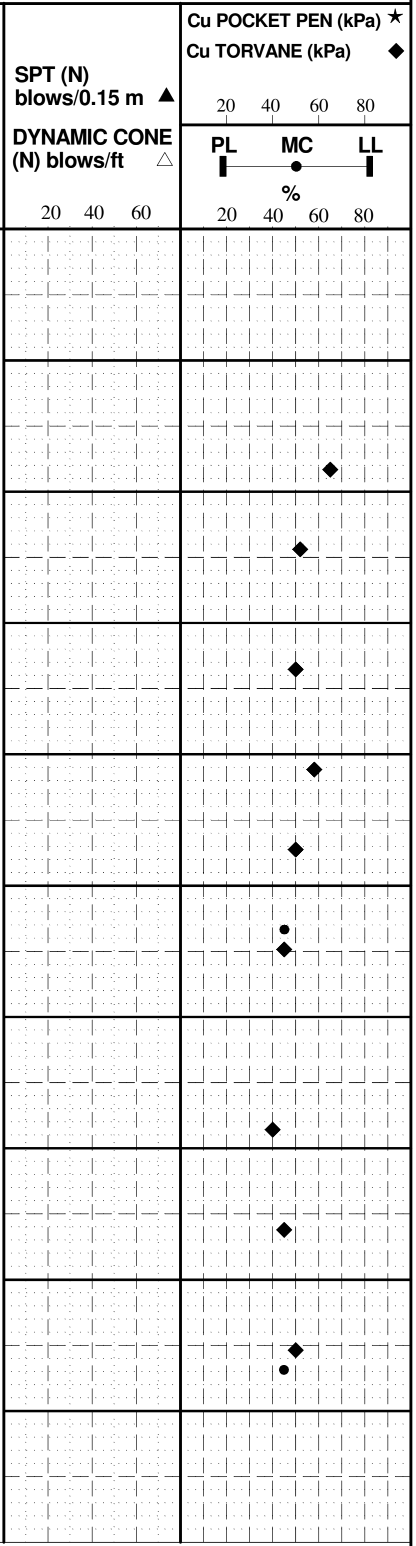
REFERENCE NO.

HOLE NO. **TH13-12**

SHEET 1 of 2

CLIENT	CH2M HILL/CITY OF WINNIPEG	JOB NO.	13-0338-002
PROJECT	SEWPCC UPGRADING/EXPANSION PROJECT	GROUND ELEV.	232.33 m
		TOP OF PVC ELEV.	
SITE	South End Water Pollution Control Centre	WATER ELEV.	
LOCATION		DATE DRILLED	11/25/2013
		UTM (m)	N 5,517,390
DRILLING METHOD	125 mm ø Solid Stem Auger, ACKER SS Drill Rig		E 636,747

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	SAMPLE TYPE	NUMBER	RECOVERY %
232.33	0	0	▩	**SILTY CLAY FILL** - Brown, frozen to 0.15 m then moist, stiff, intermediate to high plasticity, trace coarse grained sand, trace fine gravel, trace silt nodules.			
					⦃	S1	
231.4			▨	**SILTY CLAY** - Brown, moist, stiff, high plasticity, trace silt nodules.			
					⦃	S2	
					⦃	S3	
				- Gypsum nodules (10 mm Ø) between 4.27 and 4.42 m.			
				- Firm below 4.88 m.			
					⦃	S4	
					⦃	S5	
				- Grey below 7.92 m.			
				- Trace fine grained gravel below 8.23 m.			
					⦃	S6	



SAMPLE TYPE ⦃ Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

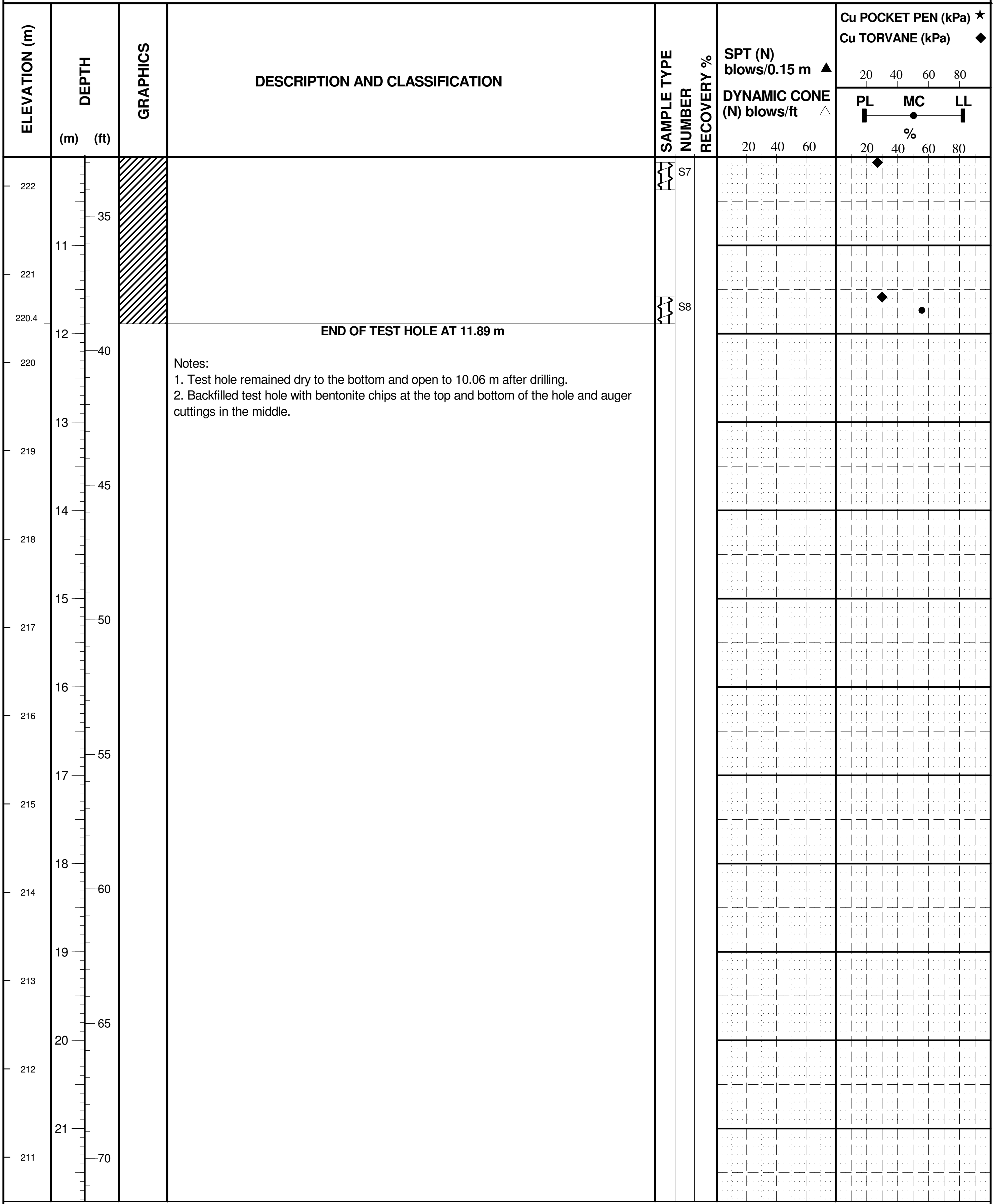
KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO.
TH13-12

SHEET 2 of 2



END OF TEST HOLE AT 11.89 m

Notes:
1. Test hole remained dry to the bottom and open to 10.06 m after drilling.
2. Backfilled test hole with bentonite chips at the top and bottom of the hole and auger cuttings in the middle.

SAMPLE TYPE {{ Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

KGS GROUP

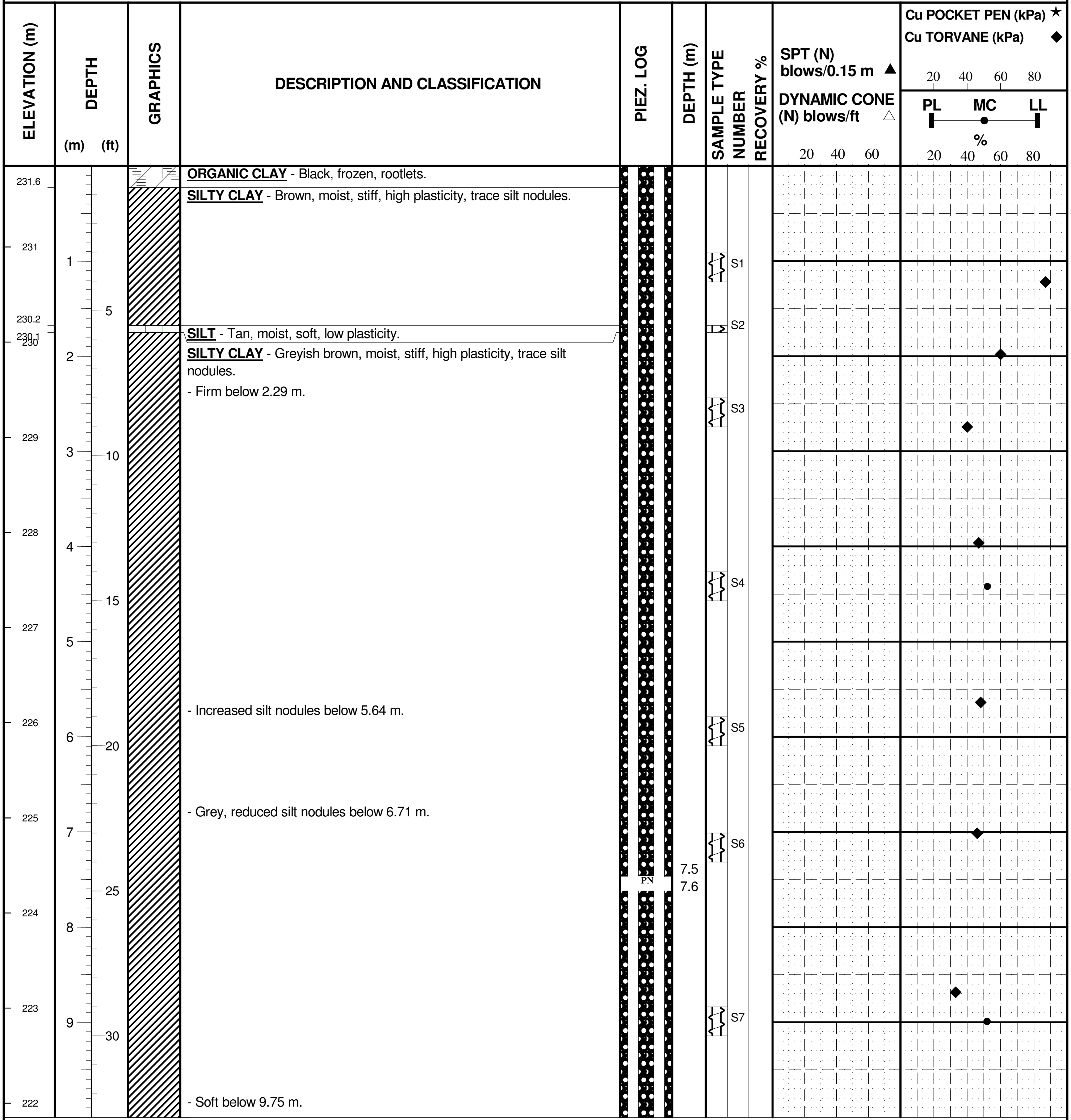
SUMMARY LOG

REFERENCE NO.

HOLE NO.
TH13-13

CLIENT CH2M HILL/CITY OF WINNIPEG
PROJECT SEWPCC UPGRADING/EXPANSION PROJECT
SITE South End Water Pollution Control Centre
LOCATION
DRILLING METHOD 125 mm ø Solid Stem Auger and HQ Core Barrel, B-59 Drill Rig

JOB NO. 13-0338-002
GROUND ELEV. 231.85 m
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED 11/20/2013
UTM (m) N 5,517,636
E 636,848



ORGANIC CLAY - Black, frozen, rootlets.

SILTY CLAY - Brown, moist, stiff, high plasticity, trace silt nodules.

SILT - Tan, moist, soft, low plasticity.

SILTY CLAY - Greyish brown, moist, stiff, high plasticity, trace silt nodules.
- Firm below 2.29 m.
- Increased silt nodules below 5.64 m.
- Grey, reduced silt nodules below 6.71 m.
- Soft below 9.75 m.

SAMPLE TYPE ⌇ Auger Grab ☒ Split Spoon ▮ Core Barrel

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

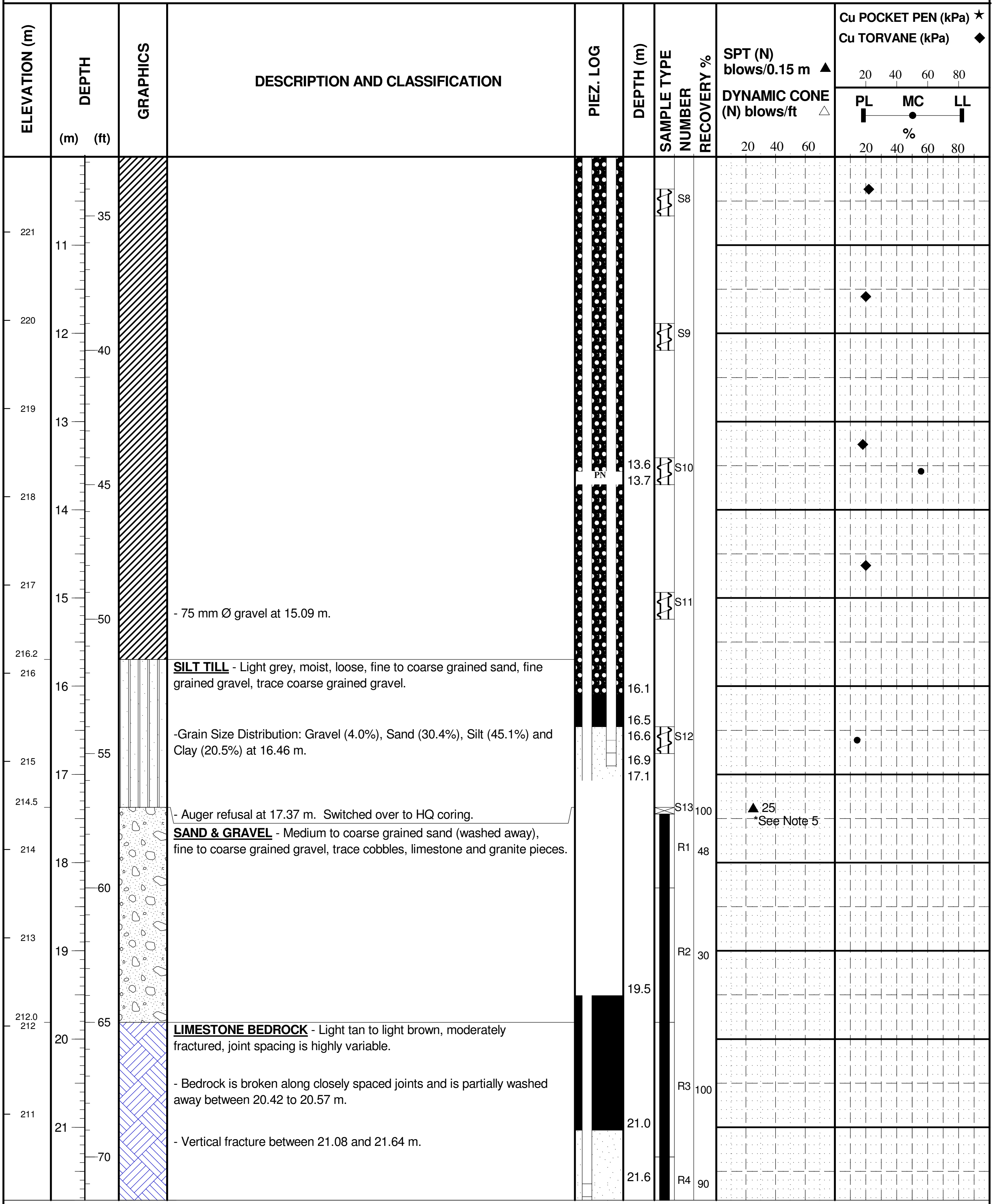
KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO.
TH13-13

SHEET 2 of 3



- 75 mm Ø gravel at 15.09 m.

SILT TILL - Light grey, moist, loose, fine to coarse grained sand, fine grained gravel, trace coarse grained gravel.

-Grain Size Distribution: Gravel (4.0%), Sand (30.4%), Silt (45.1%) and Clay (20.5%) at 16.46 m.

- Auger refusal at 17.37 m. Switched over to HQ coring.

SAND & GRAVEL - Medium to coarse grained sand (washed away), fine to coarse grained gravel, trace cobbles, limestone and granite pieces.

LIMESTONE BEDROCK - Light tan to light brown, moderately fractured, joint spacing is highly variable.

- Bedrock is broken along closely spaced joints and is partially washed away between 20.42 to 20.57 m.

- Vertical fracture between 21.08 and 21.64 m.

SAMPLE TYPE ⌇ Auger Grab ☒ Split Spoon ■ Core Barrel

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.15 m ▲ DYNAMIC CONE (N) blows/ft △ 20 40 60	Cu POCKET PEN (kPa) ★ Cu TORVANE (kPa) ◆ 20 40 60 80 PL MC LL % 20 40 60 80
209.9	22			**END OF TEST HOLE AT 21.95 m.**		21.9	■				
209	23	75									
208	24	80									
207	25										
206	26	85									
205	27	90									
204	28										
203	29	95									
202	30	100									
201	31										
200	32	105									
199	33	110									

Notes:
1. Water level noted at 9.14 m below grade after drilling to 17.37 m.
2. Installed Casagrande standpipe in the bedrock at a depth of 21.95 m and a Casagrande standpipe in the till at a depth of 16.92 m. Both standpipes have a stick-up of 0.91 m.
3. Installed pneumatic piezometer (35528) at a depth of 7.62 m and pneumatic piezometer (35525) at a depth of 13.72 m.
4. Backfilled test hole with silica sand from 21.95 to 21.03 m, bentonite chips from 21.03 to 19.51 m, slough from 19.51 to 17.07 m, silica sand from 17.07 to 16.46 m, bentonite chips from 16.46 to 16.10 m and a bentonite grout mixture from 16.10 m to grade.
5. SPT bouncing on possible boulder/cobble 75 mm into first set.

SAMPLE TYPE ⌇ Auger Grab ☒ Split Spoon ■ Core Barrel

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

KGS GROUP

SUMMARY LOG

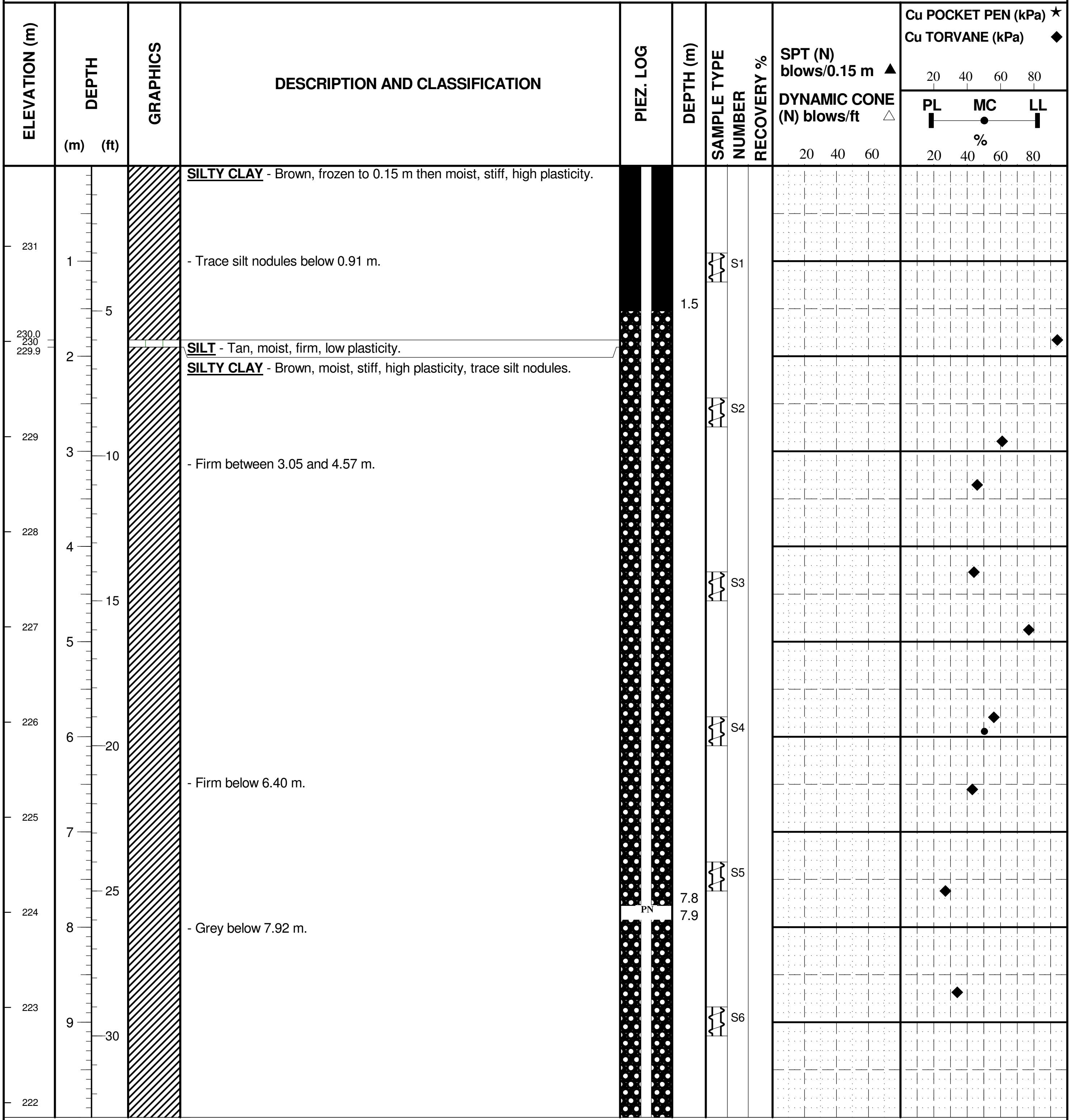
REFERENCE NO.

HOLE NO.
TH13-14

SHEET 1 of 3

CLIENT CH2M HILL/CITY OF WINNIPEG
PROJECT SEWPCC UPGRADING/EXPANSION PROJECT
SITE South End Water Pollution Control Centre
LOCATION
DRILLING METHOD 125 mm ø Solid Stem Auger, ACKER SS Drill Rig

JOB NO. 13-0338-002
GROUND ELEV. 231.85 m
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED 11/27/2013
UTM (m) N 5,517,318
E 636,778



SILTY CLAY - Brown, frozen to 0.15 m then moist, stiff, high plasticity.

- Trace silt nodules below 0.91 m.

SILT - Tan, moist, firm, low plasticity.

SILTY CLAY - Brown, moist, stiff, high plasticity, trace silt nodules.

- Firm between 3.05 and 4.57 m.

- Firm below 6.40 m.

- Grey below 7.92 m.

SAMPLE TYPE {} Auger Grab ☒ Split Spoon

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

GEOTECHNICAL-SOIL LOG P:\PROJECTS\2013\13-0338-002\DESIGN\GEO\LOGS\SEWPCC.GPJ

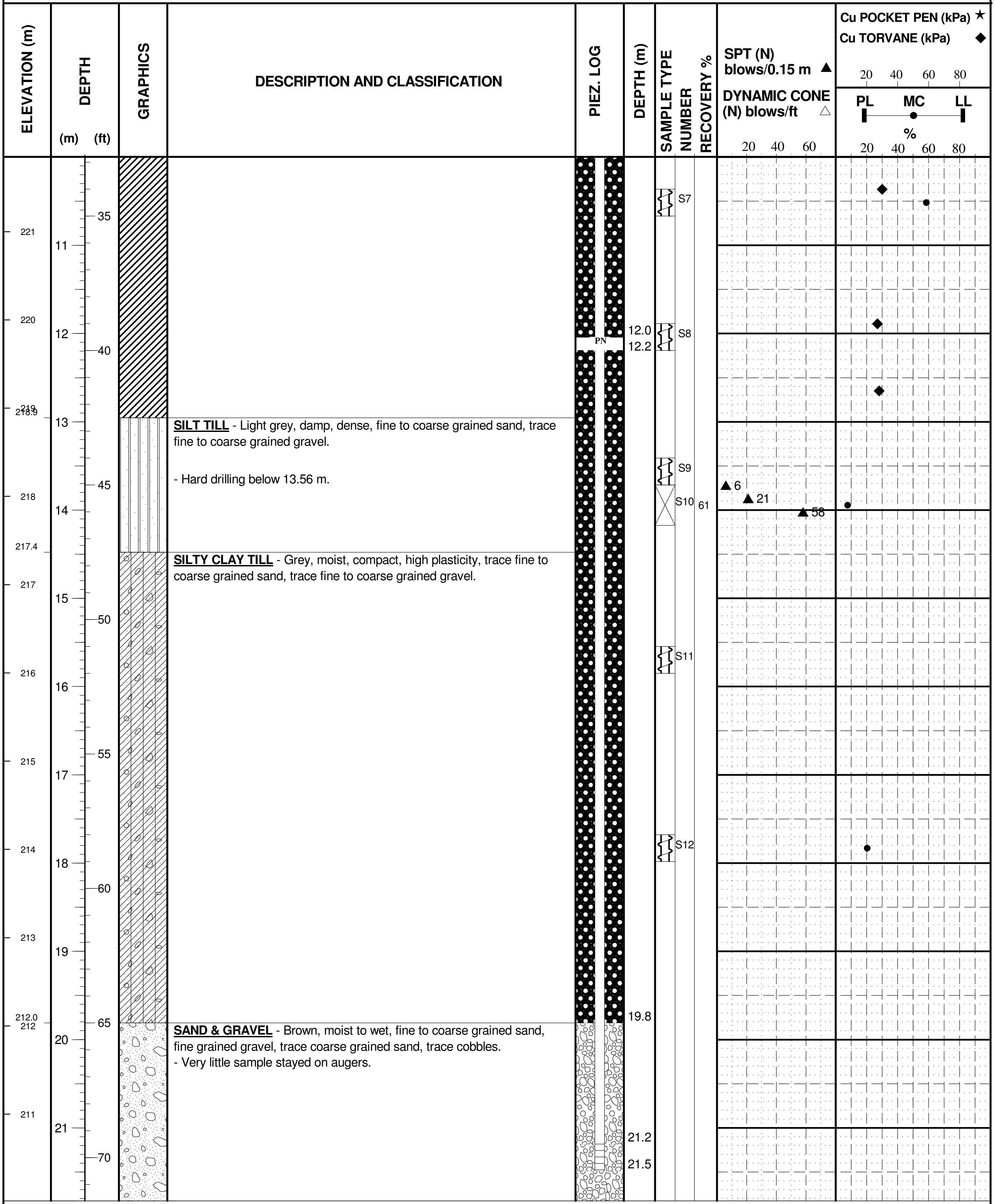
KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO.
TH13-14

SHEET 2 of 3



SILT TILL - Light grey, damp, dense, fine to coarse grained sand, trace fine to coarse grained gravel.

- Hard drilling below 13.56 m.

SILTY CLAY TILL - Grey, moist, compact, high plasticity, trace fine to coarse grained sand, trace fine to coarse grained gravel.

SAND & GRAVEL - Brown, moist to wet, fine to coarse grained sand, fine grained gravel, trace coarse grained sand, trace cobbles.
- Very little sample stayed on augers.

SAMPLE TYPE ⌇ Auger Grab ☒ Split Spoon

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

GEOTECHNICAL-SOIL LOG P:\PROJECTS\2013\13-0338-002\DESIGN\GEO\LOGS\SEWPCC.GPJ

KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO.
TH13-14

SHEET 3 of 3

ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.15 m ▲ DYNAMIC CONE (N) blows/ft △ 20 40 60	Cu POCKET PEN (kPa) ★ Cu TORVANE (kPa) ◆ 20 40 60 80 PL MC LL % 20 40 60 80
	22										
209	23	75		-Grain Size Distribution: Gravel (14.1%), Sand (43.6%), Silt (24.6%) and Clay (17.7%) at 22.86 m.		23.5	Auger Grab	S13			
208.4				**AUGER REFUSAL AT 23.47 m.**							
208	24			Notes: 1. Water level noted at 5.49 m below grade after drilling. 2. Test hole sloughed in to 21.49 m. 3. Installed Casagrande standpipe at a depth of 21.49 m with a stick-up of 0.85 m. 4. Installed pneumatic piezometer (35529) at a depth of 7.92 m and pneumatic piezometer (35527) at a depth of 12.19 m. 5. Backfilled test hole with a bentonite grout mixture from 19.81 to 1.52 m and bentonite chips from 1.52 m to grade.							
		80									
207	25										
206	26	85									
205	27										
		90									
204	28										
203	29	95									
202	30										
		100									
201	31										
200	32	105									
199	33										
		110									

SAMPLE TYPE ⟅ Auger Grab ☒ Split Spoon

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

KGS GROUP

SUMMARY LOG

REFERENCE NO.

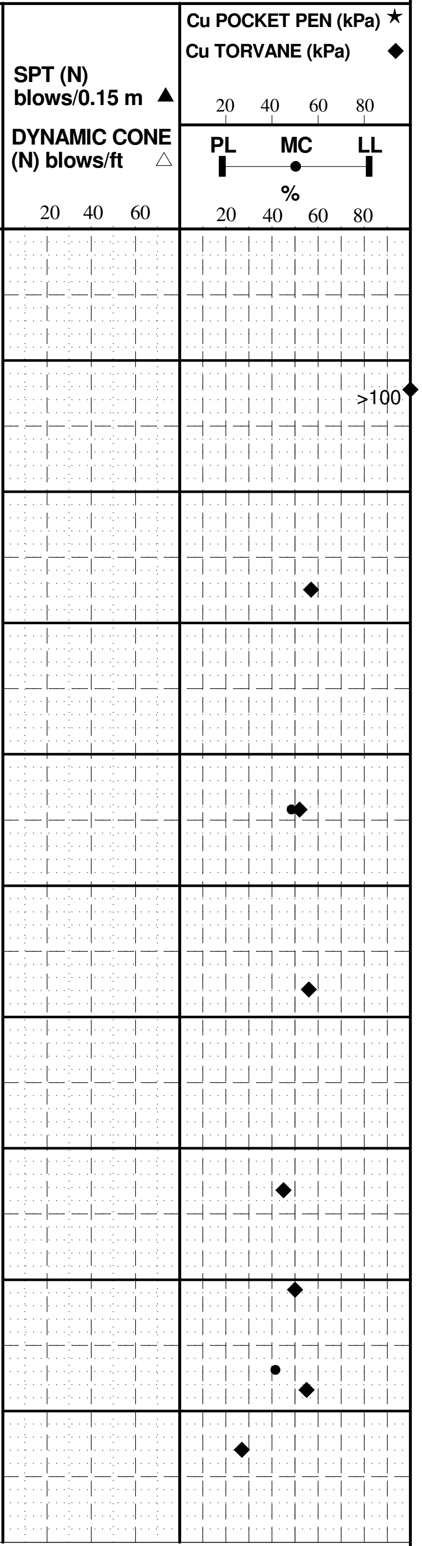
HOLE NO. **TH13-15**

SHEET 1 of 3

CLIENT CH2M HILL/CITY OF WINNIPEG
PROJECT SEWPCC UPGRADING/EXPANSION PROJECT
SITE South End Water Pollution Control Centre
LOCATION
DRILLING METHOD 125 mm ø Solid Stem Auger, ACKER SS Drill Rig

JOB NO. **13-0338-002**
GROUND ELEV. **232.55 m**
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED **11/27/2013**
UTM (m) N **5,517,350** E **637,016**

ELEVATION (m)	DEPTH (m)	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER
		SAND & GRAVEL - Brown, frozen to 0.3 m then damp, compact, well graded, fine to coarse grained sand, fine grained gravel, trace coarse grained gravel.			Auger Grab	S1
232.1		**SILTY CLAY** - Black, moist, stiff, high plasticity.				
		- Brown below 1.07 m.			Auger Grab	S2
230.7		**SILT** - Tan, moist, firm, low plasticity.				
230.6		**SILTY CLAY** - Brown, moist, stiff, high plasticity, trace silt nodules.				
					Auger Grab	S3
					Auger Grab	S4
					Auger Grab	S5
		- Firm below 7.01 m.			Auger Grab	S6
			PN	7.8 7.9		
		- Stiff between 7.77 and 8.99 m.				
		- Grey between 8.08 and 8.53 m.			Auger Grab	S7
		- Grey below 9.14 m.				



SAMPLE TYPE Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

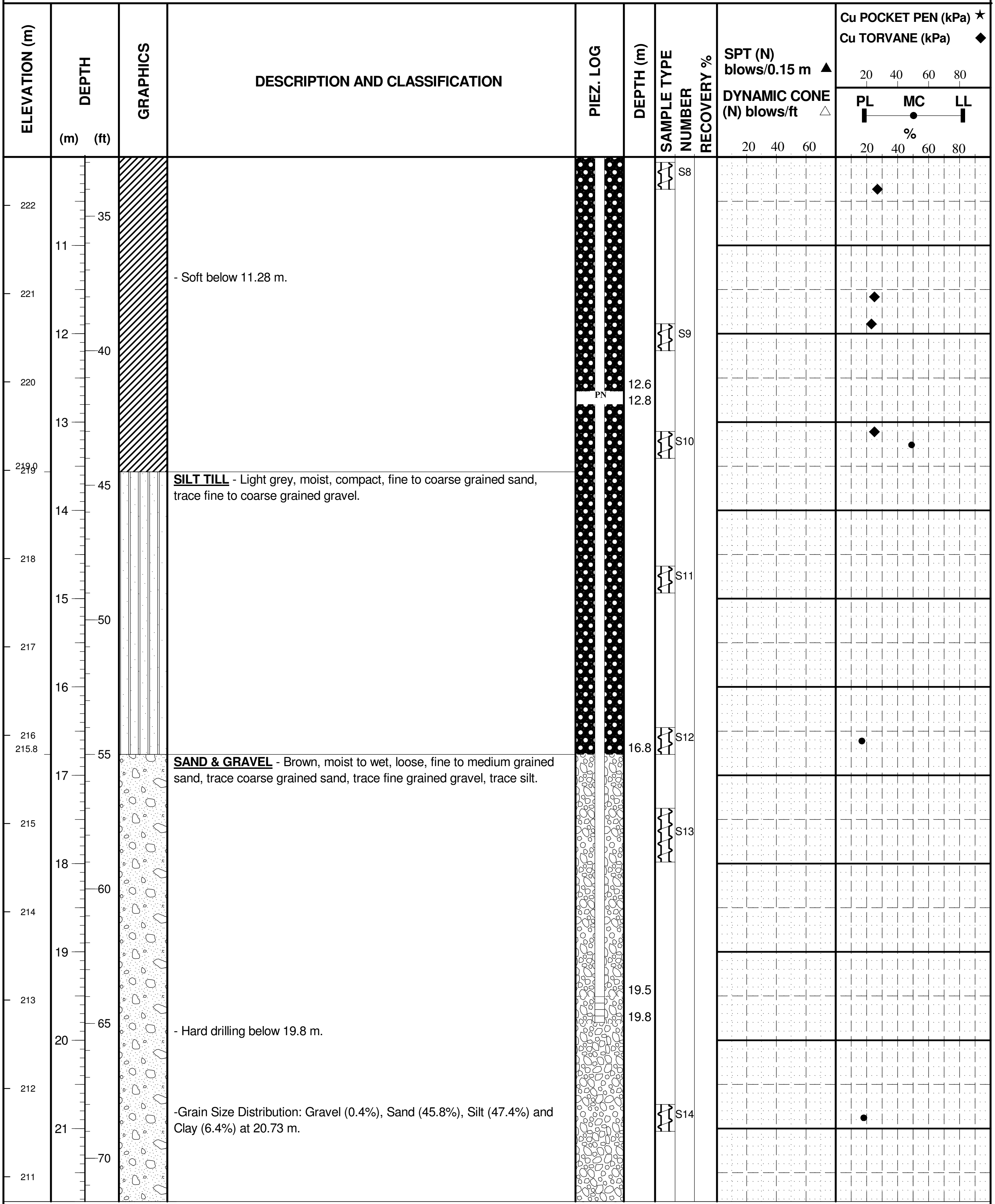
KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO.
TH13-15

SHEET 2 of 3



- Soft below 11.28 m.

SILT TILL - Light grey, moist, compact, fine to coarse grained sand, trace fine to coarse grained gravel.

SAND & GRAVEL - Brown, moist to wet, loose, fine to medium grained sand, trace coarse grained sand, trace fine grained gravel, trace silt.

- Hard drilling below 19.8 m.

-Grain Size Distribution: Gravel (0.4%), Sand (45.8%), Silt (47.4%) and Clay (6.4%) at 20.73 m.

SAMPLE TYPE ⦃ Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

KGS GROUP

SUMMARY LOG

REFERENCE NO.

HOLE NO.
TH13-15

SHEET 3 of 3

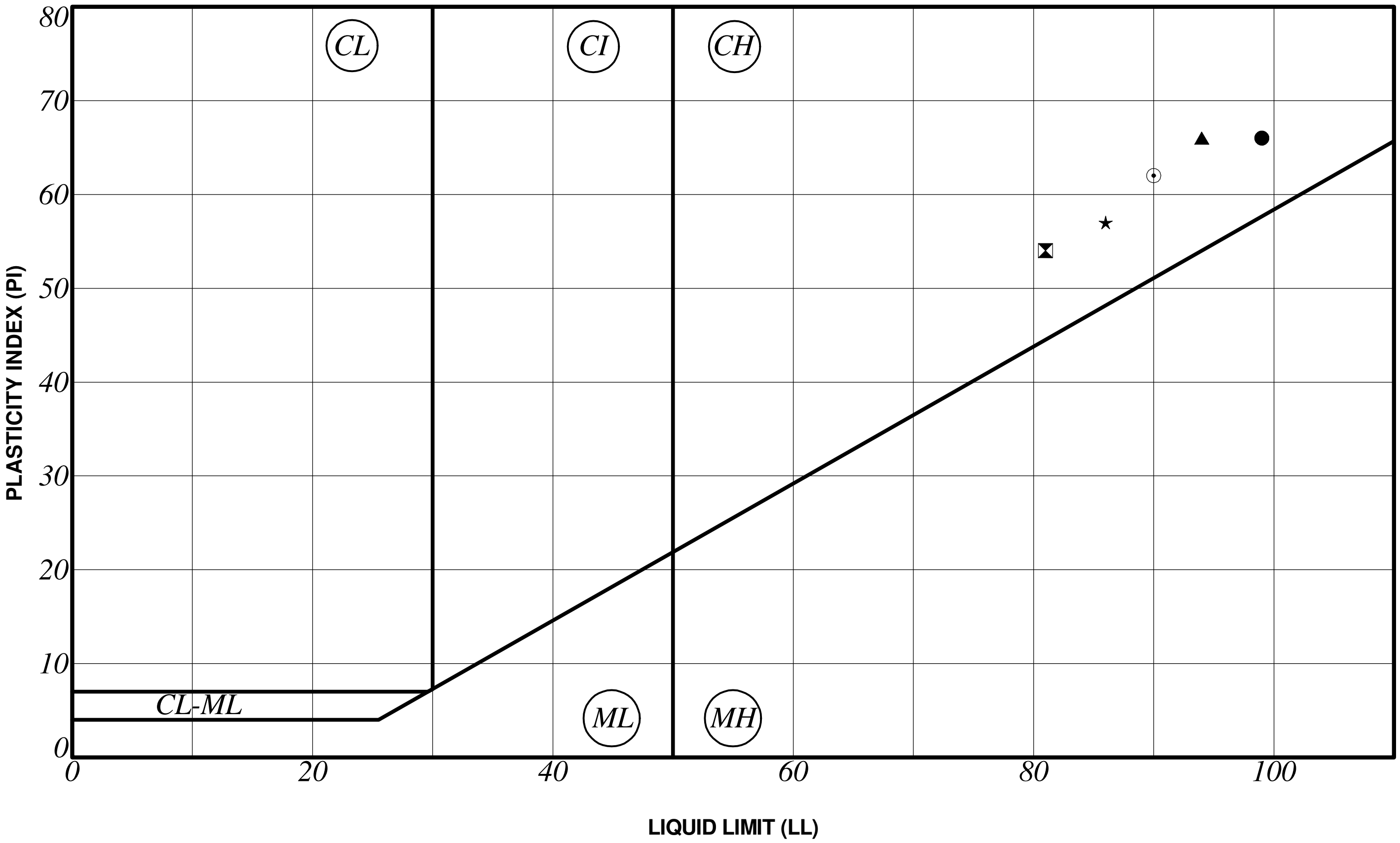
ELEVATION (m)	DEPTH (m)	DEPTH (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.15 m ▲ DYNAMIC CONE (N) blows/ft △ 20 40 60	Cu POCKET PEN (kPa) ★ Cu TORVANE (kPa) ◆ 20 40 60 80 PL MC LL % 20 40 60 80
	22										
210											
	23	75									
209.4						23.2					
				AUGER REFUSAL AT 23.16 m.							
209											
	24										
		80									
208											
	25										
207											
	26	85									
206											
	27										
		90									
205											
	28										
204											
	29	95									
203											
	30										
		100									
202											
	31										
201											
	32	105									
200											
	33										
199		110									

Notes:
1. Water level noted at 9.14 m below grade after drilling.
2. Test hole sloughed in to 16.76 m.
3. Installed Casagrande standpipe at a depth of 19.81 m with a stick-up of 0.81 m.
4. Installed pneumatic piezometer (35530) at a depth of 7.92 m and pneumatic piezometer (35526) at a depth of 12.80 m.
5. Backfilled test hole with a bentonite grout mixture from 16.76 m to grade.

SAMPLE TYPE ⧉ Auger Grab

CONTRACTOR	INSPECTOR	APPROVED	DATE
Paddock Drilling Ltd.	C. FRIESEN	T. NG	2/6/14

GEOTECHNICAL-SOIL LOG P:\PROJECTS\2013\13-0338-002\DESIGN\GEO\LOGS\SEWPCC.GPJ



SYMBOL	HOLE	DEPTH (m)	SAMPLE #	LL	PL	PI	% SAND	% SILT	% CLAY	% MC	CLASSIFICATION
●	TH13-02	4.0	S4	99	33	66				54.5	CH
⧗	TH13-02	8.5	S7	81	27	54				45.8	CH
▲	TH13-02	13.1	S10	94	28	66				63.1	CH
★	TH13-03	5.2	S5	86	29	57				48.3	CH
☉	TH13-03	11.6	S9	90	28	62				53.6	CH

Notes:
ML - Low Plasticity Silt
MH - High Plasticity Silt
CL-ML - Silty Clay
CL - Low Plasticity Clay
CI - Intermediate Plasticity Clay
CH - High Plasticity Clay
LL - Liquid Limit
PL - Plastic Limit
PI - Plasticity Index
MC - Moisture Content
NP - Non-Plastic

KGS GROUP

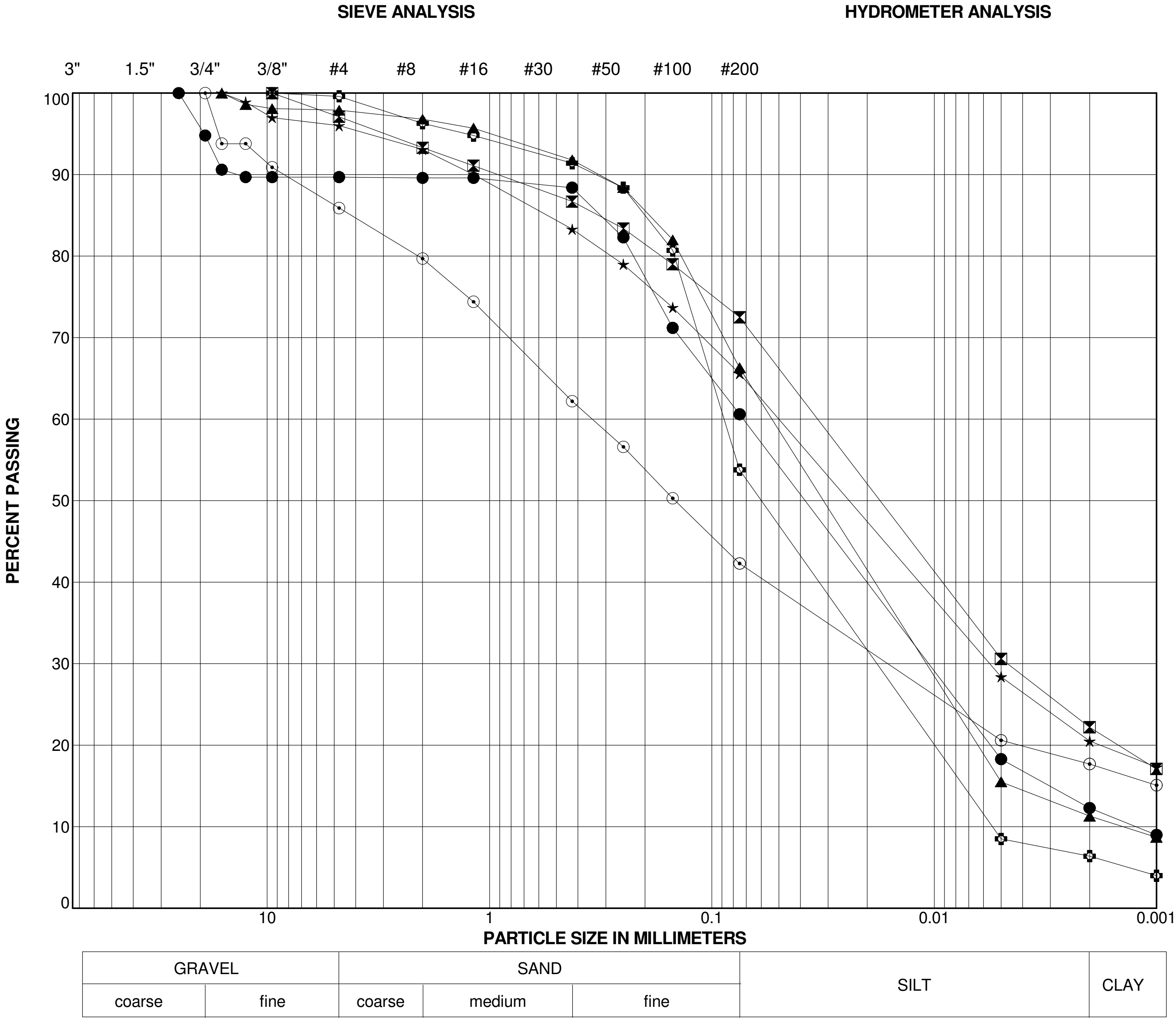
CH2M HILL/CITY OF WINNIPEG

SEWPCC UPGRADING/EXPANSION PROJECT

A-LINE PLOT

February 2014 | **Figure 1** | **Page 1 of 1**

A-LINE PLOT P:\PROJECTS\2013\13-0338-002\DESIGN\GEO\LOGS\SEWPCC.GPJ



SYMBOL	HOLE	DEPTH (m)	SAMPLE #	% GRAVEL	% SAND	% SILT	% CLAY	% SILT & CLAY	Cu	Cc	CLASSIFICATION
●	*TH13-02*	*15.2*	*S12*	*10.3*	*29.1*	*48.3*	*12.3*	*60.6*	*58.5*	*1.3*	
⧗	*TH13-03*	*14.3*	*S11*	*2.9*	*24.6*	*50.3*	*22.2*	*72.5*			
▲	*TH13-03*	*18.3*	*S14*	*2.1*	*31.6*	*55.0*	*11.3*	*66.3*	*37.9*	*1.5*	
★	*TH13-13*	*16.5*	*S12*	*4.0*	*30.4*	*45.1*	*20.5*	*65.6*			
⊙	*TH13-14*	*22.9*	*S13*	*14.1*	*43.6*	*24.6*	*17.7*	*42.3*			
✤	*TH13-15*	*20.7*	*S14*	*0.4*	*45.8*	*47.4*	*6.4*	*53.8*	*16.1*	*0.7*	

KGS GROUP

CH2M HILL/CITY OF WINNIPEG

SEWPCC UPGRADING/EXPANSION PROJECT

GRAIN SIZE ANALYSES

February 2014 | **Figure 2** | **Page 1 of 1**

SIEVE ANALYSIS P:\PROJECTS\2013\13-0338-002\DESIGN\GEO\LOGS\SEWPCC.GPJ

TH13-13

57’3” to 72’0”

TH13-02

62’6” to 73’0”

TH13-03

61’0” to 75’0”

Stantec Consulting Ltd.
199 Henlow Bay
Winnipeg MB R3Y 1G4
Tel: (204) 488-6999
Fax: (204) 488-6947

January 14, 2013

KGS Group Inc.
3rd Floor-865 Waverley Street
Winnipeg, MB
R3T 5P4

Attention: Caleb Friesen

Caleb,

Re: South End Water Pollution Control Centre - Soils Test Report

Soils sample were submitted to our laboratory on January 6, 2014. The following tests were conducted on selected soil samples:

- Water content (ASTM D2216)
- Particle size analysis (ASTM D422)
- Liquid limit (multi-point), plastic limit, and plasticity index (ASTM D4318)

The test results are summarized in the attached tables and particle size analysis reports.

We appreciate the opportunity to assist you in this project. Please call if you have any questions regarding this report.

German Leal.

German Leal, B.Sc., P.Eng.
Project Manager, Geotechnical Engineering

TABLE 1 - PARTICLE SIZE AND ATTERBERG LIMITS TEST DATA

Testhole ID	Sample No.	Gravel (%) 75 to 4.75 mm	Sand (%)			Silt (%) <0.075 to 0.002 mm	Clay (%) <0.002 mm	Liquid Limit	Plastic Limit	Plasticity Index
			Coarse <4.75 to 2.0 mm	Medium <2.0 to 0.425 mm	Fine <0.425 to 0.075 mm					
TH13-02	S4	-	-	-	-	-	-	99	33	66
TH13-02	S7	-	-	-	-	-	-	81	27	54
TH13-02	S10	-	-	-	-	-	-	94	28	66
TH13-02	S12	10.3	0.1	1.2	27.8	48.3	12.3	-	-	-
TH13-03	S5	-	-	-	-	-	-	86	29	57
TH13-03	S9	-	-	-	-	-	-	90	28	62
TH13-03	S11	2.9	3.8	6.6	14.2	50.3	22.2	-	-	-
TH13-03	S14	2.1	1.1	5.0	25.5	55.0	11.3	-	-	-
TH13-13	S12	4.0	2.9	9.8	17.7	45.1	20.5	-	-	-
TH13-14	S13	14.1	6.2	17.5	19.9	24.6	17.7	-	-	-
TH13-15	S14	0.4	3.3	4.9	37.6	47.4	6.4	-	-	-

Notes:

1. A high speed stirring device was used for 1 minute to disperse the test samples for particle size analysis
2. Atterberg limits conducted in accordance with ASTM D4318 Method A (multi-point liquid limit)
3. The soil samples were air-dried during sample preparation for Atterberg limits and particle size analysis

TABLE 2 - WATER CONTENT TEST DATA

Testhole ID	Sample No.	Moisture Content (%)	Testhole ID	Sample No.	Moisture Content (%)
TH13-02	S4	54.5	TH13-11	S4	49.8
TH13-02	S6	48.9	TH13-11	S6	51.2
TH13-02	S7	45.8	TH13-11	S8	56.8
TH13-02	S8	53.9	TH13-12	S4	45.1
TH13-02	S9	55.5	TH13-12	S6	44.9
TH13-02	S10	63.1	TH13-12	S8	55.6
TH13-02	S12	12.4	TH13-13	S4	52.1
TH13-02	S14	13.7	TH13-13	S7	51.9
TH13-03	S2	20.9	TH13-13	S10	55.7
TH13-03	S5	48.3	TH13-13	S12	14.4
TH13-03	S7	42.5	TH13-14	S4	50.3
TH13-03	S9	53.6	TH13-14	S7	58.5
TH13-03	S11	17.1	TH13-14	S10	7.7
TH13-08	S3	52.4	TH13-14	S12	20.3
TH13-08	S5	42.5	TH13-15	S4	48.5
TH13-08	S7	47.8	TH13-15	S7	41.5
TH13-09	S4	56.9	TH13-15	S10	49.0
TH13-09	S6	45.8	TH13-15	S12	16.9
TH13-09	S8	48.2	TH13-15	S14	18.1
TH13-10	S4	47.5			
TH13-10	S6	52.1			
TH13-10	S8	61.1			

PARTICLE SIZE ANALYSIS
ASTM D422

KGS Group Inc.
3rd Floor - 865 Waverley Street
Winnipeg, Manitoba
R3T 5P4

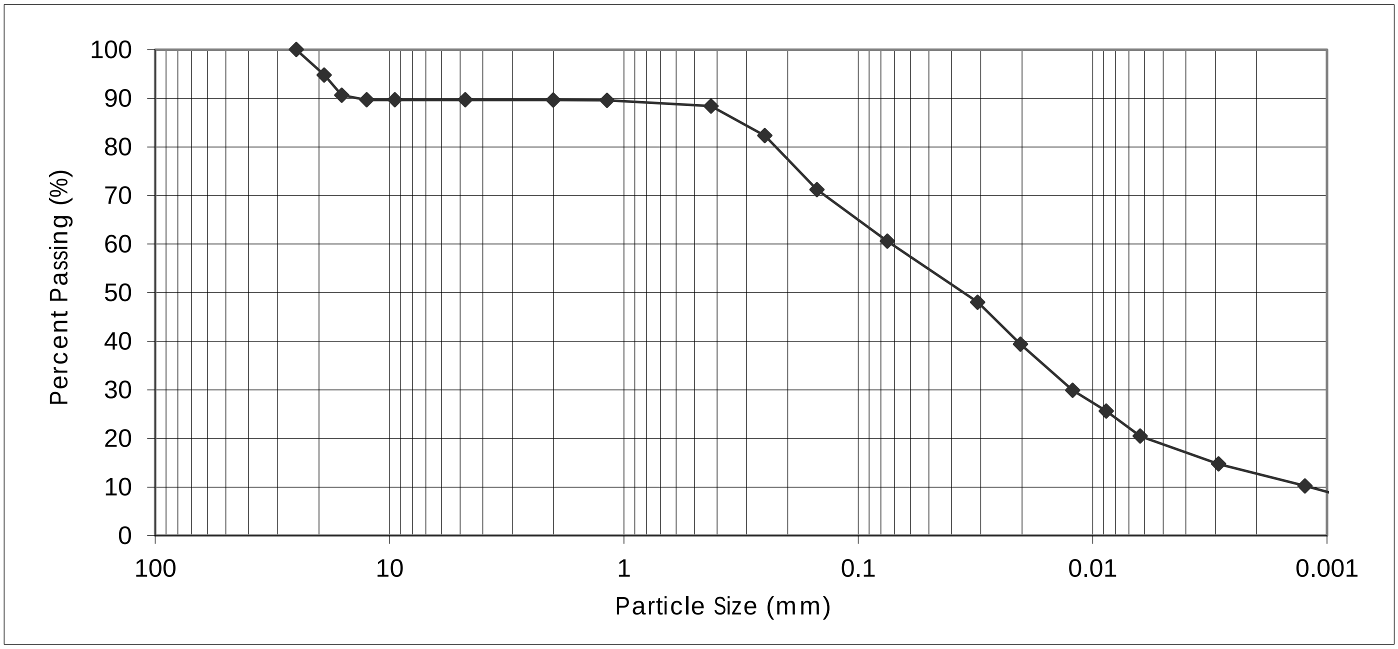
PROJECT: South End Water Pollution Control Centre (13-0338-002)

Attention: Caleb Friesen

PROJECT NO.: 123301317

SAMPLED BY: Client
SAMPLE ID: TH13-02, S12

DATE RECEIVED: January 6, 2014
TESTED BY: Nestor Abarca



PARTICLE SIZE	PERCENT PASSING
37.50 mm	100.0
25.00 mm	100.0
19.00 mm	94.8
16.00 mm	90.6
12.50 mm	89.7
9.50 mm	89.7
4.75 mm	89.7
2.00 mm	89.6

PARTICLE SIZE	PERCENT PASSING
1.18 mm	89.6
0.425 mm	88.4
0.250 mm	82.3
0.150 mm	71.2
0.075 mm	60.6
0.005 mm	18.3
0.002 mm	12.3
0.001 mm	9.0

Gravel, % 75 to 4.75 mm	Sand, %			Silt, % <0.075 to 0.002 mm	Clay, % <0.002 mm	Colloids, % < 0.001 mm
	Coarse <4.75 to 2.0 mm	Medium <2.0 to 0.425 mm	Fine <0.425 to 0.075 mm			
10.3	0.1	1.2	27.8	48.3	12.3	9.0

January 14, 2014

REVIEWED BY: German E. Leal, B.Sc., P. Eng.

PARTICLE SIZE ANALYSIS
ASTM D422

KGS Group Inc.
3rd Floor - 865 Waverley Street
Winnipeg, Manitoba
R3T 5P4

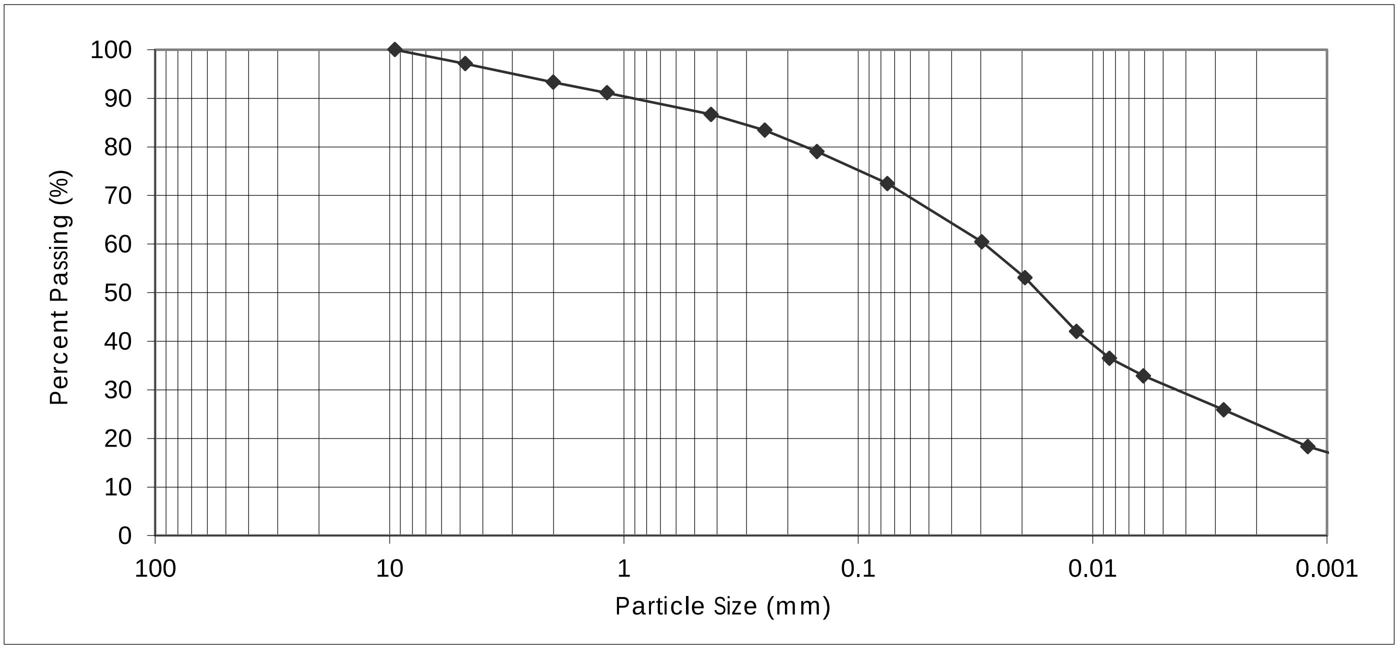
PROJECT: South End Water Pollution Control Centre (13-0338-002)

Attention: Caleb Friesen

PROJECT NO.: 123301317

SAMPLED BY: Client
SAMPLE ID: TH13-03, S11

DATE RECEIVED: January 6, 2014
TESTED BY: Nestor Abarca



PARTICLE SIZE	PERCENT PASSING
37.50 mm	100.0
25.00 mm	100.0
19.00 mm	100.0
16.00 mm	100.0
12.50 mm	100.0
9.50 mm	100.0
4.75 mm	97.1
2.00 mm	93.3

PARTICLE SIZE	PERCENT PASSING
1.18 mm	91.1
0.425 mm	86.7
0.250 mm	83.4
0.150 mm	79.0
0.075 mm	72.5
0.005 mm	30.6
0.002 mm	22.2
0.001 mm	17.1

Gravel, % 75 to 4.75 mm	Sand, %			Silt, % <0.075 to 0.002 mm	Clay, % <0.002 mm	Colloids, % < 0.001 mm
	Coarse <4.75 to 2.0 mm	Medium <2.0 to 0.425 mm	Fine <0.425 to 0.075 mm			
2.9	3.8	6.6	14.2	50.3	22.2	17.1

January 14, 2014

REVIEWED BY: German E. Leal, B.Sc., P. Eng.

PARTICLE SIZE ANALYSIS
ASTM D422

KGS Group Inc.
3rd Floor - 865 Waverley Street
Winnipeg, Manitoba
R3T 5P4

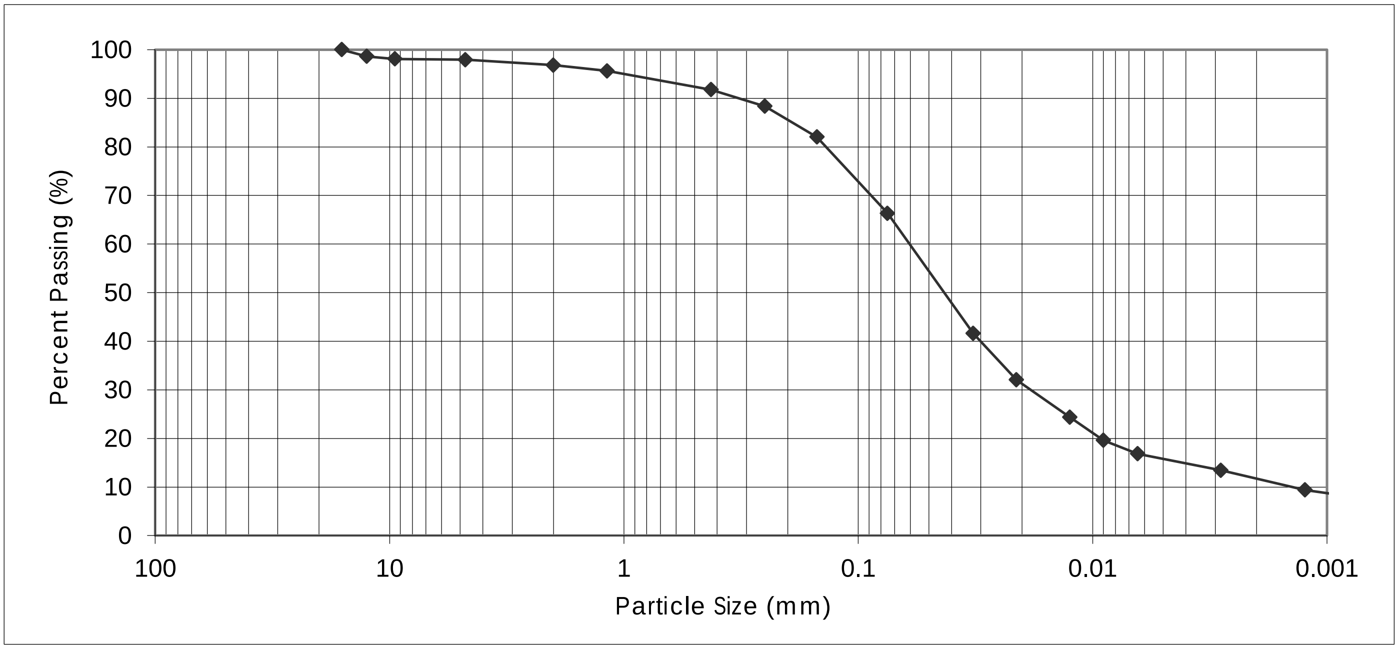
PROJECT: South End Water Pollution Control Centre (13-0338-002)

Attention: Caleb Friesen

PROJECT NO.: 123301317

SAMPLED BY: Client
SAMPLE ID: TH13-03, S14

DATE RECEIVED: January 6, 2014
TESTED BY: Nestor Abarca



PARTICLE SIZE	PERCENT PASSING
37.50 mm	100.0
25.00 mm	100.0
19.00 mm	100.0
16.00 mm	100.0
12.50 mm	98.6
9.50 mm	98.1
4.75 mm	97.9
2.00 mm	96.8

PARTICLE SIZE	PERCENT PASSING
1.18 mm	95.7
0.425 mm	91.8
0.250 mm	88.4
0.150 mm	82.0
0.075 mm	66.3
0.005 mm	15.5
0.002 mm	11.3
0.001 mm	8.7

Gravel, % 75 to 4.75 mm	Sand, %			Silt, % <0.075 to 0.002 mm	Clay, % <0.002 mm	Colloids, % < 0.001 mm
	Coarse <4.75 to 2.0 mm	Medium <2.0 to 0.425 mm	Fine <0.425 to 0.075 mm			
2.1	1.1	5.0	25.5	55.0	11.3	8.7

January 14, 2014

REVIEWED BY: German E. Leal, B.Sc., P. Eng.

PARTICLE SIZE ANALYSIS
ASTM D422

KGS Group Inc.
3rd Floor - 865 Waverley Street
Winnipeg, Manitoba
R3T 5P4

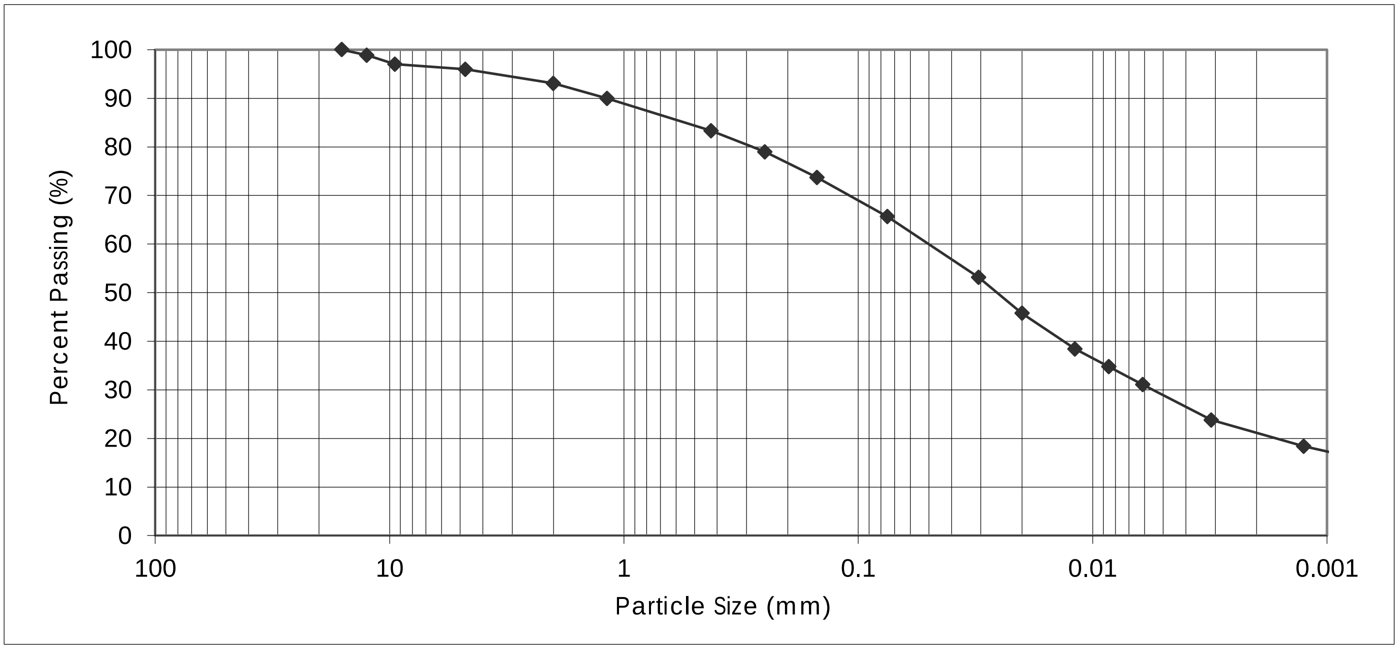
PROJECT: South End Water Pollution Control Centre (13-0338-002)

Attention: Caleb Friesen

PROJECT NO.: 123301317

SAMPLED BY: Client
SAMPLE ID: TH13-13, S12

DATE RECEIVED: January 6, 2014
TESTED BY: Nestor Abarca



PARTICLE SIZE	PERCENT PASSING
37.50 mm	100.0
25.00 mm	100.0
19.00 mm	100.0
16.00 mm	100.0
12.50 mm	98.9
9.50 mm	97.0
4.75 mm	96.0
2.00 mm	93.1

PARTICLE SIZE	PERCENT PASSING
1.18 mm	90.0
0.425 mm	83.3
0.250 mm	79.0
0.150 mm	73.7
0.075 mm	65.6
0.005 mm	28.4
0.002 mm	20.5
0.001 mm	17.3

Gravel, % 75 to 4.75 mm	Sand, %			Silt, % <0.075 to 0.002 mm	Clay, % <0.002 mm	Colloids, % < 0.001 mm
	Coarse <4.75 to 2.0 mm	Medium <2.0 to 0.425 mm	Fine <0.425 to 0.075 mm			
4.0	2.9	9.8	17.7	45.1	20.5	17.3

January 14, 2014

REVIEWED BY: German E. Leal, B.Sc., P. Eng.

PARTICLE SIZE ANALYSIS
ASTM D422

KGS Group Inc.
3rd Floor - 865 Waverley Street
Winnipeg, Manitoba
R3T 5P4

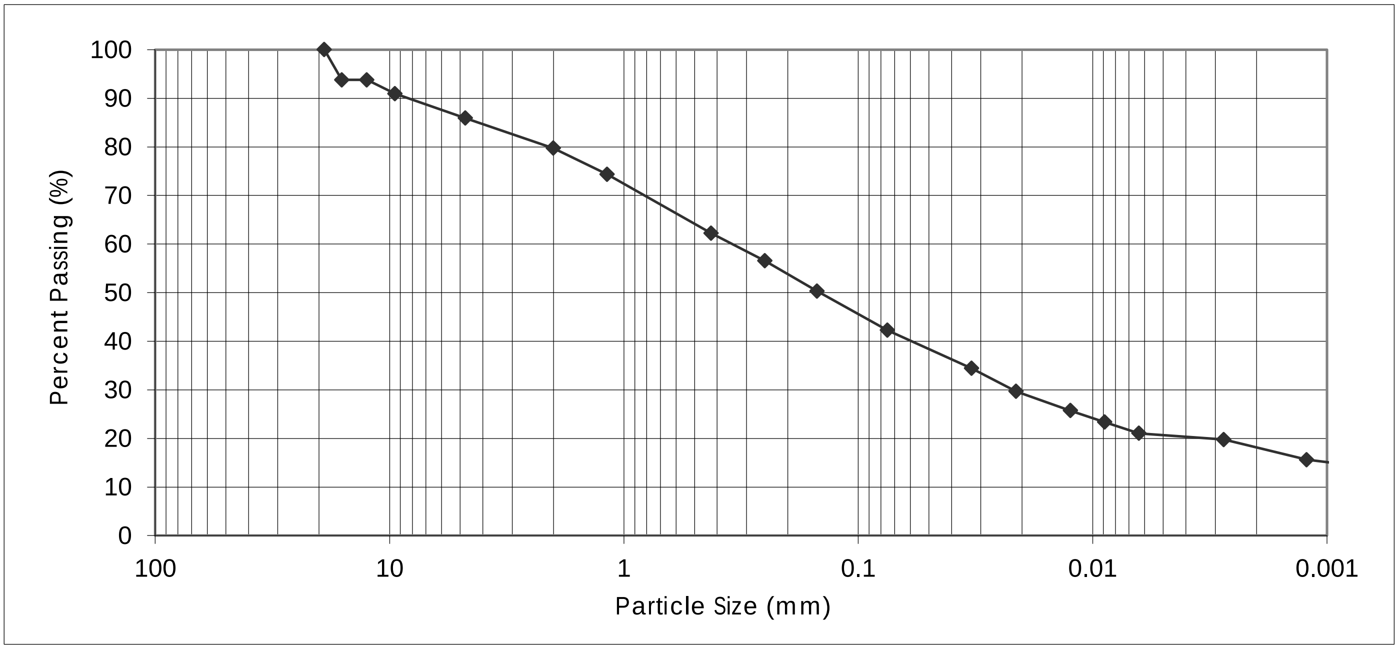
PROJECT: South End Water Pollution
Control Centre (13-0338-002)

Attention: Caleb Friesen

PROJECT NO.: 123301317

SAMPLED BY: Client
SAMPLE ID: TH13-14, S13

DATE RECEIVED: January 6, 2014
TESTED BY: Nestor Abarca



PARTICLE SIZE	PERCENT PASSING
37.50 mm	100.0
25.00 mm	100.0
19.00 mm	100.0
16.00 mm	93.8
12.50 mm	93.8
9.50 mm	90.9
4.75 mm	85.9
2.00 mm	79.7

PARTICLE SIZE	PERCENT PASSING
1.18 mm	74.4
0.425 mm	62.2
0.250 mm	56.6
0.150 mm	50.3
0.075 mm	42.3
0.005 mm	20.6
0.002 mm	17.7
0.001 mm	15.1

Gravel, % 75 to 4.75 mm	Sand, %			Silt, % <0.075 to 0.002 mm	Clay, % <0.002 mm	Colloids, % < 0.001 mm
	Coarse <4.75 to 2.0 mm	Medium <2.0 to 0.425 mm	Fine <0.425 to 0.075 mm			
14.1	6.2	17.5	19.9	24.6	17.7	15.1

January 14, 2014

REVIEWED BY: German E. Leal, B.Sc., P. Eng.

PARTICLE SIZE ANALYSIS
ASTM D422

KGS Group Inc.
3rd Floor - 865 Waverley Street
Winnipeg, Manitoba
R3T 5P4

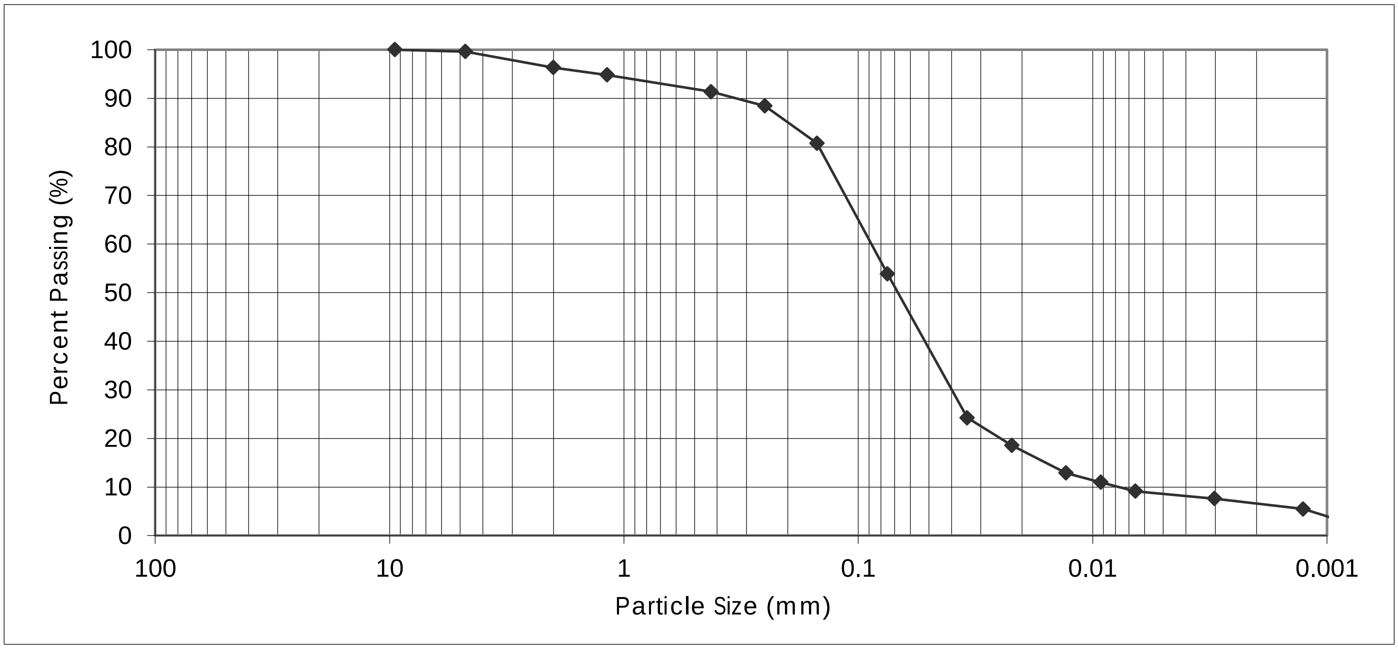
PROJECT: South End Water Pollution Control Centre (13-0338-002)

Attention: Caleb Friesen

PROJECT NO.: 123301317

SAMPLED BY: Client
SAMPLE ID: TH13-15, S14

DATE RECEIVED: January 6, 2014
TESTED BY: Nestor Abarca



PARTICLE SIZE	PERCENT PASSING
37.50 mm	100.0
25.00 mm	100.0
19.00 mm	100.0
16.00 mm	100.0
12.50 mm	100.0
9.50 mm	100.0
4.75 mm	99.6
2.00 mm	96.3

PARTICLE SIZE	PERCENT PASSING
1.18 mm	94.8
0.425 mm	91.4
0.250 mm	88.4
0.150 mm	80.7
0.075 mm	53.8
0.005 mm	8.5
0.002 mm	6.4
0.001 mm	4.0

Gravel, % 75 to 4.75 mm	Sand, %			Silt, % <0.075 to 0.002 mm	Clay, % <0.002 mm	Colloids, % < 0.001 mm
	Coarse <4.75 to 2.0 mm	Medium <2.0 to 0.425 mm	Fine <0.425 to 0.075 mm			
0.4	3.3	4.9	37.6	47.4	6.4	4.0

January 14, 2014

REVIEWED BY: German E. Leal, B.Sc., P. Eng.

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS
ASTM 4318

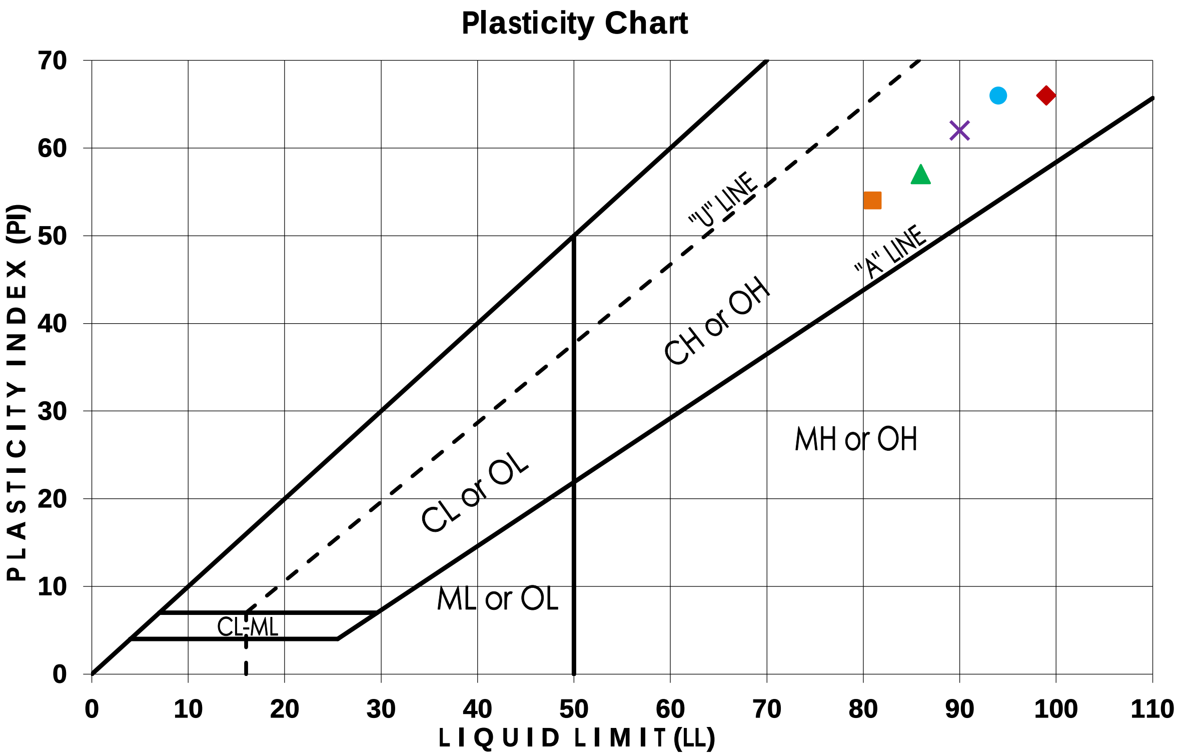
KGS Group Inc.
3rd Floor - 865 Waverley Street
Winnipeg, Manitoba
R3T 5P4

PROJECT: South End Water Pollution
Control Center (13-0338-002)

Attention: Caleb Friesen

PROJECT NO.: 123301317

Symbol	Testhole No.	Depth (m)	Liquid Limit	Plastic Limit	Plasticity Index	USCS
◆	TH13-02	S4	99	33	66	CH
■	TH13-02	S7	81	27	54	CH
●	TH13-02	S10	94	28	66	CH
▲	TH13-03	S5	86	29	57	CH
×	TH13-03	S9	90	28	62	CH



January 14, 2014

Reviewed by: German E. Leal, B.Sc., P. Eng.

APPENDIX B

PILE LOAD CAPACITY VERIFICATION – PDA TEST RESULTS

Kontzamanis Graumann Smith MacMillan Inc.

KGS GROUP
CONSULTING ENGINEERS

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October 24, 2014

File No. 13-0338-002

CH2M Hill
1301 Kenaston Boulevard
Winnipeg, Manitoba
R3P 2P2

ATTENTION: Mr. Barry Williamson
Senior Project Manager

RE: SEWPCC Upgrading/Expansion Project
682-2012 Civil/Geotechnical Work
South-East Water Pollution Control Centre, Winnipeg, MB
Pile Load Capacity Verification – PDA Test Results

Dear Mr. Williamson:

Subterranean (Manitoba) Limited retained the services of AATech Scientific Inc (ASI). to complete dynamic load testing on a number of test piles using the Pile Driving Analyzer (PDA) system. The pile load tests were performed to verify that the piles have the required factored serviceability limit state (SLS) capacity of approximately 800 kN or an unfactored SLS of 2000 kN as provided in the KGS Group final report "SEWPCC Upgrading/Expansion/Civil/Geotech Geotechnical Investigation" dated February 2014. The hammer energy and the driving stresses on the piles were also monitored during the load tests to confirm that stresses on the piles are within acceptable limits.

This letter report contains KGS Group's review of the PDA reports prepared by ASI and provides recommendations for pile design.

1.0 DESCRIPTION OF TEST PILES

Dynamic pile load tests were performed on seven piles (Pile 1 through Pile 7) on January 30, 2014 located as shown on the attached plan. All the piles were each 406 mm hexagonal pre-cast pre-stressed concrete piles. The piles were tested at restrike, 24 hours after the end of driving, with a Junttan HHK5A hydraulic hammer with a rated energy of 59 kJ. During the testing of Pile 3 a tensile reflection was observed, which usually indicates pile damage or a loose splice, resulting in a low penetration resistance. Hence, the PDA data for the Pile 3 was discarded and not included in this review.

2.0 DYNAMIC LOAD TESTS

The report prepared by AATech Scientific Inc. containing details of the dynamic load testing programs, analyses and interpretation of test results are provided in Appendix A. The driving log records for the test piles are included in Appendix B.

CAPWAP analyses were performed for representative hammer blow records from the test data obtained during the restrike of the tested piles. Results obtained from the CAPWAP analyses for the piles were used to verify the applicable CASE Method estimate and to determine soil parameters and resistance distribution for evaluating the test results. Results of the CAPWAP analyses, complete output data and values of selected PDA data (transferred energy, hammer drop height, driving stresses, penetration resistance etc.) for selected hammer blow records are all presented in Appendix A. Estimated pile load capacities obtained from analyses are summarized on Table 1.

Driving stresses were below 20 MPa throughout the testing, which is within the acceptable limits for 35 MPa pre-cast concrete piles.

TABLE 1
ESTIMATED CAPACITIES FOR 406 MM PRE-CAST PRE-STRESSED CONCRETE PILES

Pile ID	Embedment (m)	Testing Condition	Time After Driving (Days)	CASE Method Estimated Capacity (kN)	Computed Resistance		
					Estimated Capacity (kN)	Shaft Resistance (kN)	Toe Resistance (kN)
1	18.30	Restrike	1	1,968	2,093	1,078	1,016
2	21.00	Restrike	1	2,404	2,411	1,095	1,316
3	21.60 (17.40)	Restrike	1	511			
4	16.80	Restrike	1	2,120	2,229	1,097	1,132
5	18.00	Restrike	1	2,211	2,245	1,001	1,244
6	17.10	Restrike	1	2,163	2,260	1,080	1,180
7	2.00	Restrike	1	2,625	2,643	1,150	1,493

3.0 PILE DRIVING ANALYSIS

ASI performed preliminary pile driving analysis using Wave Equation Analysis (WEAP) approach to estimate the termination blow count that would be needed to achieve the required ultimate load capacity for the piles. The details and results of the WEAP analysis are provided in Appendix A. The analysis indicated that an end-of-drive (EOD) resistance of 2,000 kN can be achieved at about 20 blows per 25 mm or practical refusal.

Based on the analysis and agreement between the results from the CASE Method and WEAP approach, a resistance of 2,000 kN can be used for the capacity of the pre-cast concrete piles and the geotechnical resistance factor, Φ, can be increased from 0.4 to 0.5.

4.0 CONCLUSIONS

The results of the PDA tests on the 406 mm hexagonal pre-cast pre-stressed concrete piles showed that the piles can be driven to achieve a total mobilized resistance ranging from 2,100 kN to 2,650 kN. For the driving energy applied to the piles.

The test results confirmed that resistances were derived from both the toe and shaft of the pile.

The driving stresses as measured during the testing program were well within the acceptable limits.

5.0 RECOMMENDATIONS

The design of the piles should be based on unfactored unit resistance for pre-cast pre-stressed concrete piles of 2,100 kN with an applicable geotechnical resistance factor, Φ, of 0.5.

PDA tests should be performed on 5 to 10% representative production piles to verify the integrity and load capacities of the piles as part of the quality assurance and quality control program.

6.0 STATEMENT OF LIMITATIONS AND CONDITIONS

6.1 THIRD PARTY USE OF REPORT

This report has been prepared for Ch_2MHill and City of Winnipeg to whom this report has been addressed and any use a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. KGS Group accepts no responsibility for damages, if any, suffered by a third party as a result of decisions made or actions undertaken based on this report.

6.2 GEOTECHNICAL ENGINEERING STATEMENT OF LIMITATIONS

The conclusions and recommendations contained in this report were prepared in accordance with generally accepted professional engineering principles and practice. The conclusions and recommendations are based on the from the PDA tests and analyses that was made available to KGS Group by Subterranean (Manitoba) Ltd, combined with information on soil and groundwater conditions described in existing soils report and those encountered at and within the depth of the test holes drilled by KGS at this site. If conditions encountered during construction appear to be different from those shown on the existing soil report or test holes drilled by KGS 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 necessary.

7.0 CLOSURE

We trust that this report letter is sufficient for your present needs. Please do not hesitate to contact the undersigned at your convenience if you have any question.

Prepared By:

David Anderson, M.Sc., P.Eng.
Geotechnical Engineer

PROVINCE OF MANITOBA
Oct. 24/14
D.E.S.
ANDERSON
Member
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REGISTERED PROFESSIONAL ENGINEER

Approved By:

Rob Kenyon, Ph. D., P.Eng.
Manager, Geotechnical Engineering Services

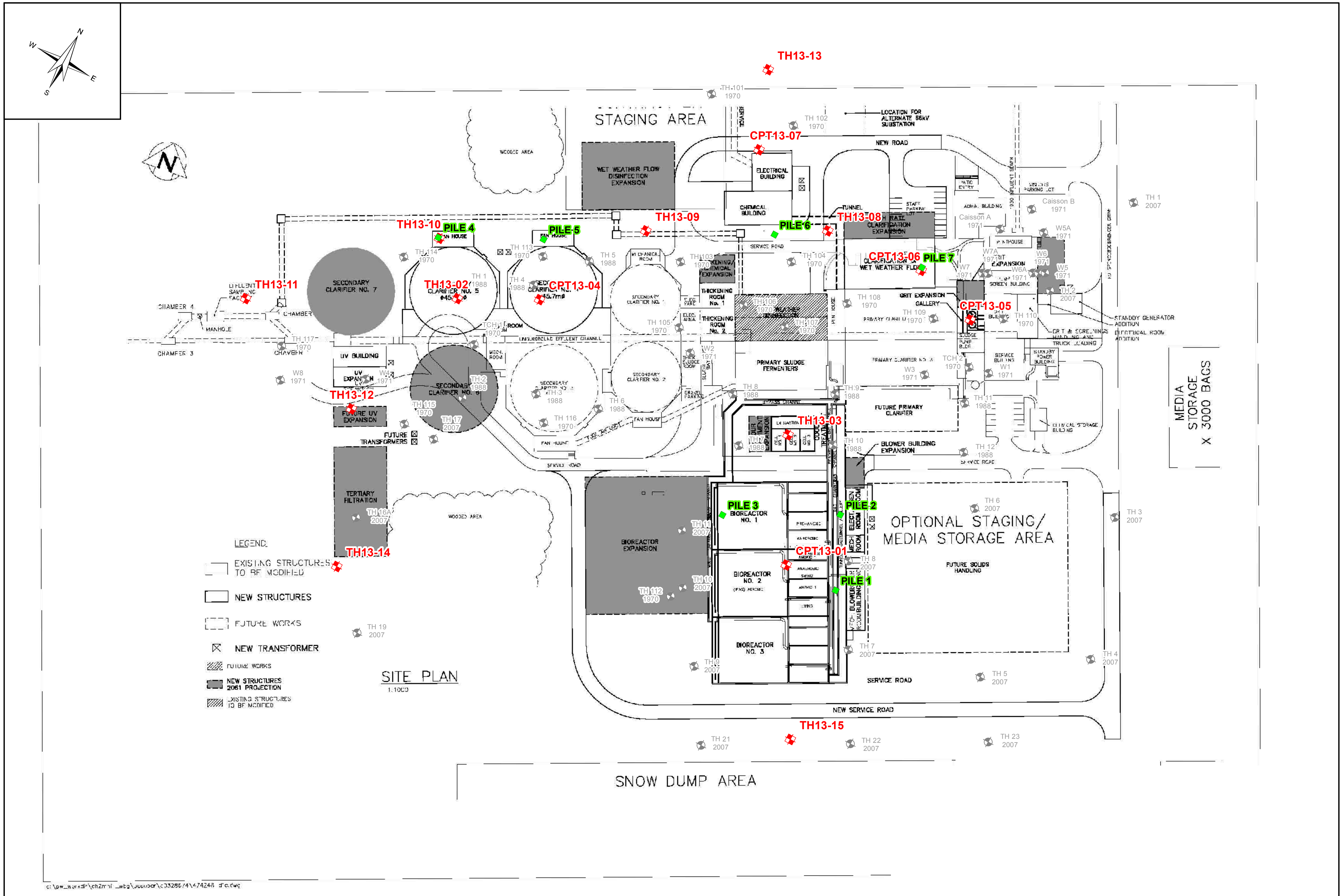
DEA/mlb
Enclosure

Cc. Roy Houston – KGS Group
Tony Ng – KGS Group

SITE PLAN FIGURES

Portions of data presented are owned by the Province of Manitoba and are produced under the licence agreement with the Province of Manitoba 2014 Queen's Printer.

FileName: P:\Projects\2013\13-0338-002\Dwg\GIS\MXDs\Rev0\Geotech_Investigations_Report\13-0338-002-F02_Rev0.mxd
11"x17" PLOT SCALE 1:1



LEGEND:

🟥	2013 Test Hole Locations
⊕	Historical Drilling
🟩	PDA Pile Locations

NOTES:

1. Issued with Draft Report, February 7, 2014, by TNN.

20 0 20 40 60 80
Metres

SCALE: 1:2,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

SEWPCC UPGRADING/EXPANSION PROJECT
PRELIMINARY LAYOUT PLAN
WITH TEST PILE (PDA) LOCATIONS
FEBRUARY 2014
FIGURE 01 REV 0

Winnipeg

KGS GROUP
CONSULTING ENGINEERS

APPENDICES

APPENDIX A

PDA TEST RESULTS

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South-East Water Pollution Control Center
Winnipeg, Manitoba

Dynamic testing of piles

Report 1

Project No. 9821401

Prepared for

Subterranean (Manitoba) Ltd
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West St Paul, MB R2P 2W5

February 4, 2014

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South-East Water Pollution Control Center
Winnipeg, Manitoba

Dynamic testing of piles

Report 1

Project No. 9821401

Prepared for

Subterranean (Manitoba) Ltd
6 St Paul Blvd
West St Paul, MB R2P 2W5

PROVINCE OF MANITOBA
F. AGHARAZI
Member
22386
REGISTERED PROFESSIONAL ENGINEER

February 4, 2014

Prepared by:

Ion Bejancu, B.A.Sc.

Fred Agharazi, M. Eng, P. Eng.

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Topic **Page No.**

Appendices

South-East Water Pollution Control Center
Winnipeg, Manitoba

Report 1

INTRODUCTION

AATech Scientific Inc. (ASI) was retained by Subterranean (Manitoba) Ltd. to perform dynamic PDA testing on driven piles at South East Water Pollution Control Center construction site in Winnipeg, Manitoba. This report presents the factual results of the PDA testing performed during one site visit, on January 30, 2014. Seven piles in total were tested at restrike, twenty-four hours after the end of driving during this visit. The tested piles are precast hexagonal 406 mm width concrete piles. A Junttan HHK5A hydraulic hammer, rated energy of 59 kJ, was used to drive and test the piles at this site. As reported to us on site, the hammer was operated at variable energy setting during PDA testing. The required capacity, as reported to us on site, is 2,000 kN.

The PDA testing and the interpretation provided in this report are in accordance with ASTM Standard D4945-00.

TEST RESULTS

A total of seven piles were tested during this site visit. It is our understanding that the required capacity for the piles at this location is 2,000 kN.

A total of six CAPWAP analyses were performed on a representative hammer blow record from the PDA data. CAPWAP analyses are performed mainly to verify the applicable CASE Method estimates, and to determine soil parameters and resistance distribution for evaluating the test results. The mobilized static resistance computed by CAPWAP showed an agreement with CASE Method Estimate (CMES) RMX with j-factor (CASE damping factor) of 0.8 (RX8). Results of the CAPWAP analyses are summarized in Table 1, and the complete outputs are enclosed in Appendix 1 at the end of this report. Values of RX8 as well as other PDA data (transferred energy, driving stresses, penetration resistance…) for selected hammer blow records are also presented in Table 1.

All tested piles showed a penetration resistance in excess of 20 blows per 25 mm (refusal), with the exception of Pile 3, which showed a tensile reflection at approximate depth of 17.5 m (about 4 m above the pile toe). A tensile reflection is usually an indication of pile damage or a loose splice. This pile showed a low penetration resistance (about 3 blows per 25 mm and a low capacity of about 500 kN, as indicated by PDA data.

Based on the test results the tested piles, except Pile 3, showed a total mobilized resistance ranging from 2,100 kN through 2,650 kN, which is in excess of the required capacity of 2,000 kN. It should be noted that the pile resistance measured at or beyond practical refusal (20 blows per 25 mm) is in fact the resistance mobilized by the hammer impact and may not necessarily represent the full capacity of the pile.

Driving stresses were below 20 MPa throughout the testing, which is within the acceptable limits for 35 MPa precast concrete piles.

CONCLUSION AND RECOMMENDATIONS

Driving stresses were maintained within acceptable limits throughout the testing.

All tested piles, except Pile 3, showed a mobilized resistance in excess of the required capacity. Additional resistance may be expected with time.

Pile 3 showed a tensile reflection (damage indication) at approximate depth of 17.5 m (about 4 m above the pile toe).

These test results are representative of site conditions at the time of testing (water level, existing ground level around the location of the test piles), and apply only to production piles in the same site location, and showing similar behavior to that of the tested piles. Any changes in site conditions and/or pile behavior during driving should be reported to the engineer for further evaluation.

Table 1: PDA Data and CAPWAP Summary Table

Pile	Pile type	Pile Inclination	Hammer	Date	Date	Test	Blow	Embed.	EMX	EMX Ratio	FMX	CSX	CSB	TSX	PRES	Case Method Est.	Computed Resistance (kN)			Smith damping (s/m)		Quake (mm)	
No.	& size (mm)	(Vertical/Battered)	Type	Driven	Tested	(E / R)	No.	(m)	(kN-m)	(%)	(kN)	(Mpa)	(Mpa)	(Mpa)	(Bl./25mm)	RX8 (kN)	Total	Shaft	Toe	Shaft	Toe	Shaft	Toe
1	Hex 406	V	HHK 5A	Jan 29, 2014	Jan 30, 2014	ER	25	18.30	19.3	33	2,697	18.9	15.6	3.9	20	1,968	**2,093**	1,078	1,016	0.3	0.2	4.0	8.6
2	Hex 406	V	HHK 5A	Jan 29, 2014	Jan 30, 2014	R	4	21.00	14.3	24	2,141	15.0	19.2	2.4	20	2,404	**2,411**	1,095	1,316	0.4	0.2	4.0	5.0
3	Hex 406	V	HHK 5A	Jan 29, 2014	Jan 30, 2014	ER	21	21.60 (17.40)	12.2	21	1,931	13.5	10.0	4.0	3	511*							
4	Hex 406	V	HHK 5A	Jan 29, 2014	Jan 30, 2014	R	3	16.80	13.3	23	2,020	14.1	16.3	2.8	25	2,120	**2,229**	1,097	1,132	0.3	0.3	4.0	6.2
5	Hex 406	V	HHK 5A	Jan 29, 2014	Jan 30, 2014	R	3	18.00	12.5	21	1,924	13.5	19.3	3.0	25	2,211	**2,245**	1,001	1,244	0.4	0.4	4.9	5.1
6	Hex 406	V	HHK 5A	Jan 29, 2014	Jan 30, 2014	R	4	17.10	10.4	18	1,815	12.7	16.4	2.6	25	2,163	**2,260**	1,080	1,180	0.3	0.2	4.0	4.9
7	Hex 406	V	HHK 5A	Jan 29, 2014	Jan 30, 2014	ER	23	21.00	10.7	18	2,058	14.4	18.6	3.6	60	2,625	**2,643**	1,150	1,493	0.4	0.4	3.3	2.9

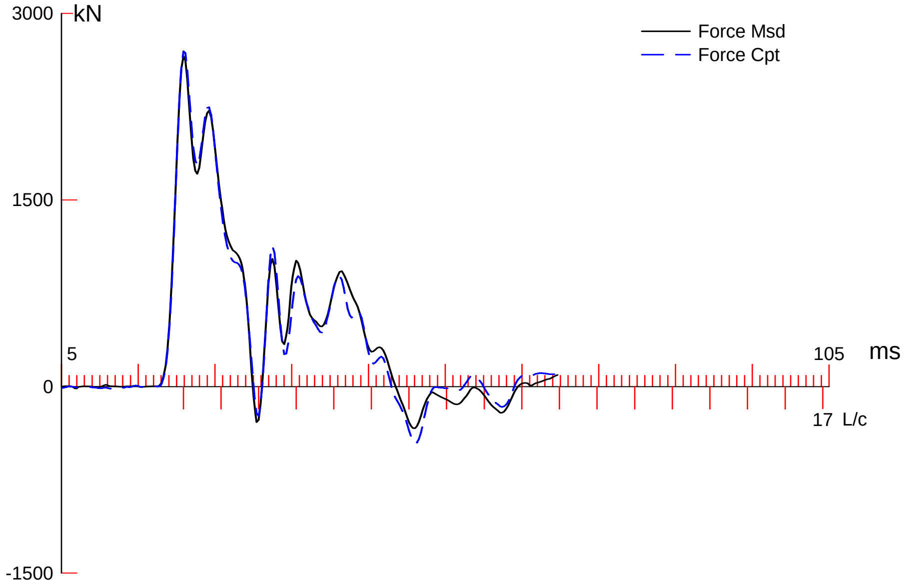
Embed.	Length below adjacent grade at the time of testing	E	End of driving	CSX	Maximum compressive stress measured in the pile	*	Pile showing significant dalmage
EMX	Maximum energy transferred to the pile head	R	Restrike	CSB	Computed compressive stress near the pile toe		
EMX Ratio	Ratio of transferred energy to rated energy of hammer	ER	End of Restrike	TSX	Tensile stress		
FMX	Maximum force measured	PRES	Penetration resistance (Blows per 25 mm)	RX8	RMX / RSP CASE Method with a J-Factor of #		

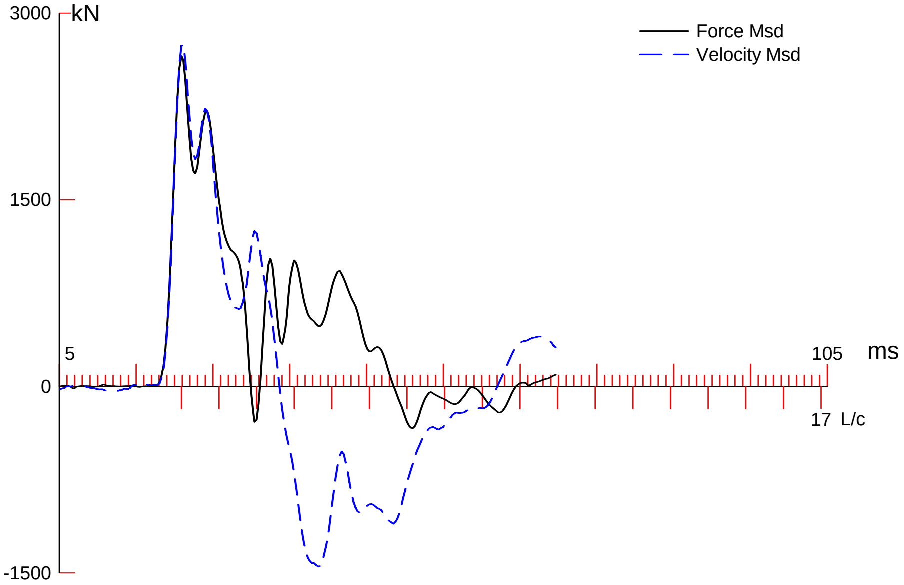
Appendix 1

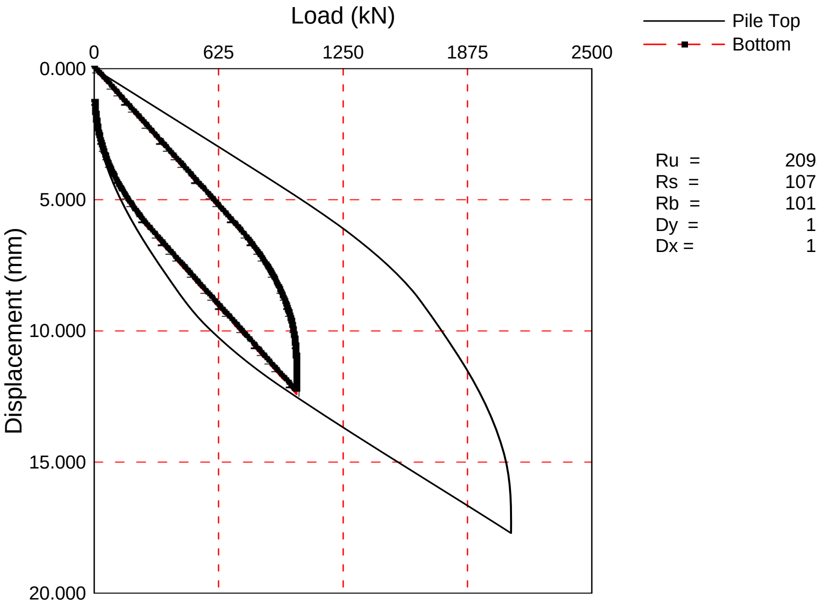
CAPWAP Analysis Results

SEWPCC; Pile: 1; ER; Blow: 25 (Test: 18-Jan-2014 19:38:)
AATech Scientific Inc

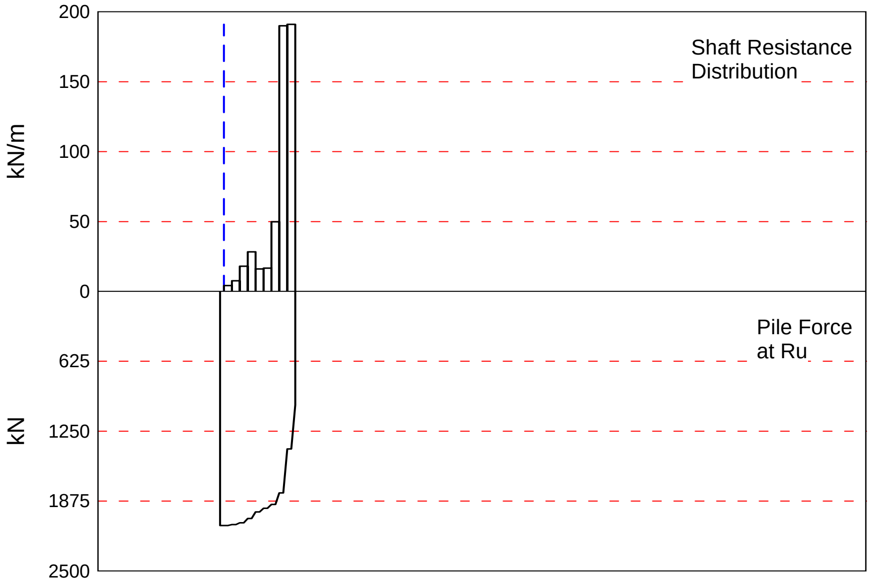
03-Feb-2014
CAPWAP(R) 2006-2





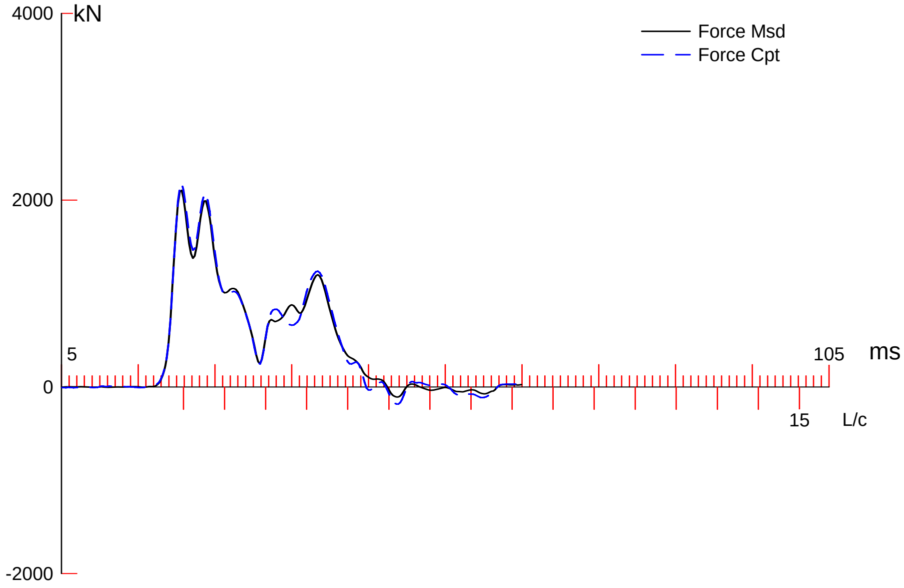


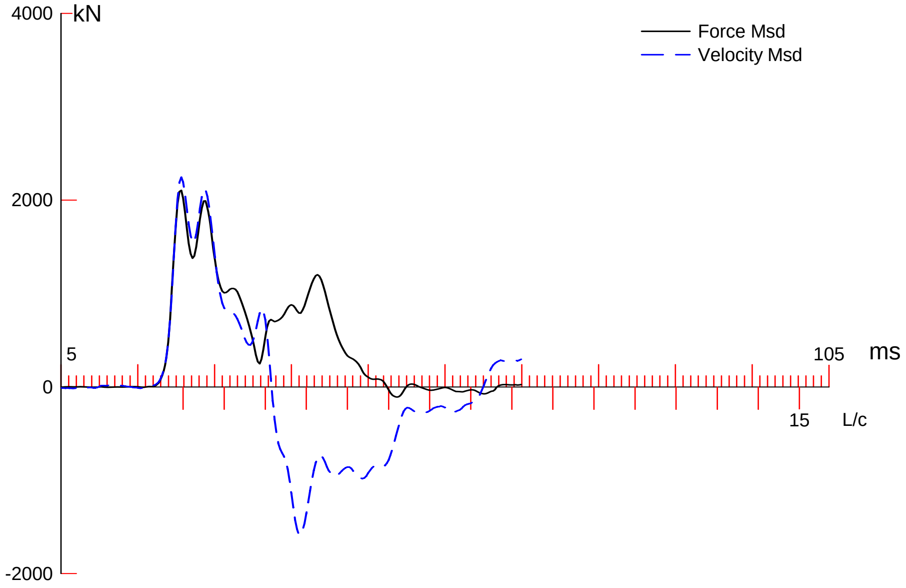
Ru = 2093.5 kN
Rs = 1077.5 kN
Rb = 1016.0 kN
Dy = 16.5 mm
Dx = 17.7 mm

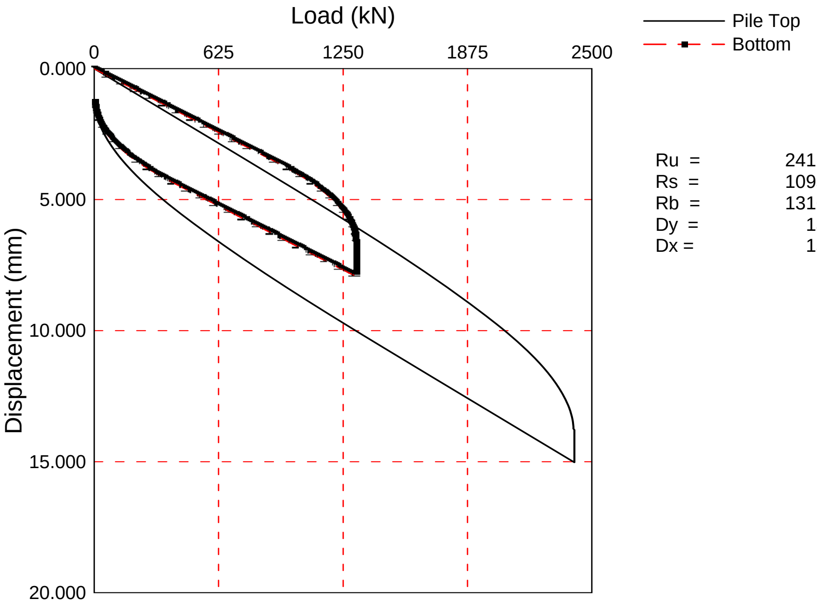


```
                                   CAPWAP SUMMARY RESULTS
Total CAPWAP Capacity:   2093.5; along Shaft   1077.5; at Toe   1016.0  kN
    Soil     Dist.     Depth        Ru      Force        Sum       Unit       Unit      Smith
   Sgmnt     Below     Below              in Pile         of    Resist.    Resist.    Damping
     No.     Gages     Grade                              Ru    (Depth)     (Area)     Factor
                 m         m        kN         kN         kN       kN/m        kPa        s/m

                                            2093.5
       1       3.1       1.8       8.8     2084.7        8.8       4.90       3.49      0.300
       2       5.2       3.9      15.9     2068.8       24.7       7.71       5.48      0.300
       3       7.2       5.9      37.3     2031.5       62.0      18.08      12.85      0.300
       4       9.3       8.0      58.6     1972.9      120.6      28.40      20.20      0.300
       5      11.3      10.0      33.2     1939.7      153.8      16.09      11.44      0.300
       6      13.4      12.1      34.6     1905.1      188.4      16.77      11.92      0.300
       7      15.5      14.2     102.9     1802.2      291.3      49.88      35.46      0.300
       8      17.5      16.2     392.1     1410.1      683.4     190.05     135.13      0.300
       9      19.6      18.3     394.1     1016.0     1077.5     191.02     135.82      0.300

  Avg. Shaft                     119.7                             58.88      41.86      0.300

     Toe                        1016.0                                      7117.24      0.200

Soil Model Parameters/Extensions                            Shaft      Toe

Quake                       (mm)                            4.000    8.600
Case Damping Factor                                         0.205    0.129
Unloading Quake             (% of loading quake)               90       82
Reloading Level             (% of Ru)                         100      100
Unloading Level             (% of Ru)                          30
Soil Plug Weight            (kN)                                      0.20
Soil Support Dashpot                                        0.800    0.000
Soil Support Weight         (kN)                            14.51     0.00

CAPWAP match quality        =    4.76        (Wave Up Match) ; RSA = 0
Observed: final set         =   1.250 mm;     blow count      =     800 b/m
Computed: final set         =   1.511 mm;     blow count      =     662 b/m

max. Top Comp. Stress       =    18.9 MPa      (T=  21.4 ms, max= 1.001 x Top)
max. Comp. Stress           =    18.9 MPa      (Z=   2.1 m, T=  21.7 ms)
max. Tens. Stress           =   -4.11 MPa      (Z=   5.2 m, T=  50.5 ms)
max. Energy (EMX)           =   19.66 kJ;      max. Measured Top Displ. (DMX)=12.26 mm
```

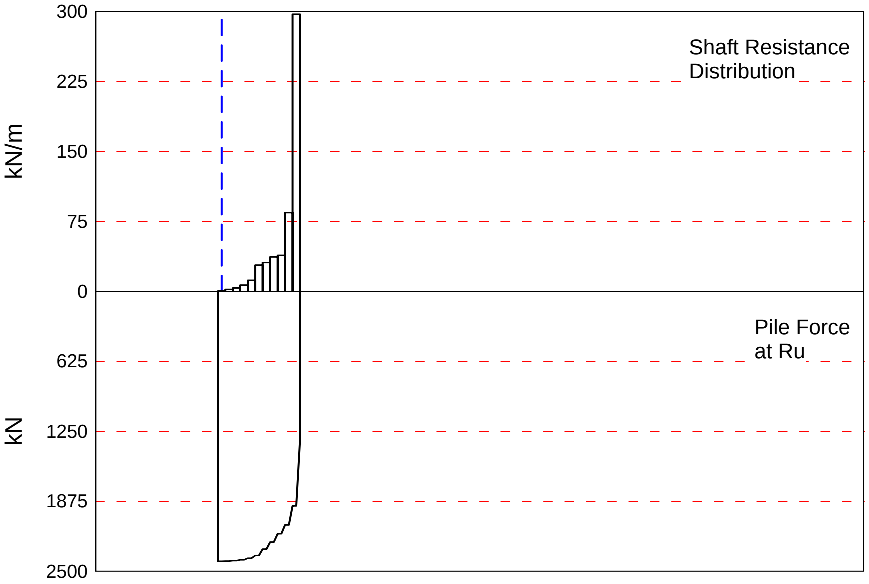
```
                                  EXTREMA TABLE
    Pile     Dist.      max.      min.      max.      max.      max.      max.      max.
   Sgmnt     Below     Force     Force     Comp.     Tens.   Trnsfd.    Veloc.    Displ.
     No.     Gages                        Stress    Stress    Energy
                 m        kN        kN       MPa       MPa        kJ       m/s        mm

       1       1.0    2700.0    -510.3      18.9     -3.57     19.66       1.7    12.173
       2       2.1    2702.1    -540.6      18.9     -3.79     19.63       1.7    12.096
       3       3.1    2701.8    -561.8      18.9     -3.94     19.59       1.7    11.981
       4       4.1    2693.6    -579.5      18.9     -4.06     19.39       1.7    11.822
       5       5.2    2694.5    -586.9      18.9     -4.11     19.27       1.7    11.622
       6       6.2    2684.9    -559.2      18.8     -3.92     18.88       1.7    11.386
       7       7.2    2692.6    -515.1      18.9     -3.61     18.66       1.7    11.119
       8       8.3    2670.9    -440.2      18.7     -3.08     18.04       1.7    10.904
       9       9.3    2680.8    -369.6      18.8     -2.59     17.97       1.7    10.752
      10      10.3    2633.5    -313.1      18.4     -2.19     17.19       1.7    10.600
      11      11.3    2647.8    -320.4      18.5     -2.24     17.10       1.8    10.430
      12      12.4    2632.2    -325.8      18.4     -2.28     16.59       1.9    10.238
      13      13.4    2653.6    -346.5      18.6     -2.43     16.45       1.8    10.028
      14      14.4    2609.3    -345.3      18.3     -2.42     15.89       1.7     9.802
      15      15.5    2560.6    -346.2      17.9     -2.43     15.71       1.7     9.564
      16      16.5    2288.8    -289.5      16.0     -2.03     14.45       1.9     9.331
      17      17.5    2164.1    -269.0      15.2     -1.88     14.27       2.1     9.090
      18      18.6    1616.7    -142.7      11.3     -1.00     10.43       2.2     8.887
      19      19.6    1580.4    -187.5      11.1     -1.31      7.01       2.2     8.670

Absolute       2.1                          18.9                          (T =     21.7 ms)
               5.2                                   -4.11                (T =     50.5 ms)


                                   CASE METHOD
J =            0.0     0.1     0.2     0.3     0.4     0.5     0.6     0.7     0.8     0.9
RP          2007.4  1659.5  1311.5   963.5   615.5   267.5     0.0     0.0     0.0     0.0
RX          2513.7  2342.6  2188.6  2092.2  2027.8  1983.4  1955.6  1945.3  1937.9  1930.7
RU          2007.4  1659.5  1311.5   963.5   615.5   267.5     0.0     0.0     0.0     0.0

RAU =   1916.3 (kN);  RA2 =   2083.7 (kN)

Current CAPWAP Ru = 2093.5 (kN); Corresponding J(RP)= 0.00; J(RX) = 0.30

      VMX      TVP    VT1*Z      FT1      FMX      DMX      DFN      SET      EMX      QUS
      m/s       ms       kN       kN       kN       mm       mm       mm       kJ       kN
     1.77    21.15   2790.2   2697.1   2697.1   12.263    1.249    1.250     19.7   2915.9


                             PILE PROFILE AND PILE MODEL
             Depth             Area        E-Modulus     Spec. Weight            Perim.
                 m              cm²              MPa            kN/m³                 m

              0.00          1427.52          50000.0           24.000             1.406
             19.60          1427.52          50000.0           24.000             1.406

Toe Area                      0.143       m²

Top Segment Length      1.03 m, Top Impedance  1579.11 kN/m/s

Pile Damping    2.0 %, Time Incr  0.258 ms, Wave Speed   4000.0 m/s, 2L/c   9.8 ms
```







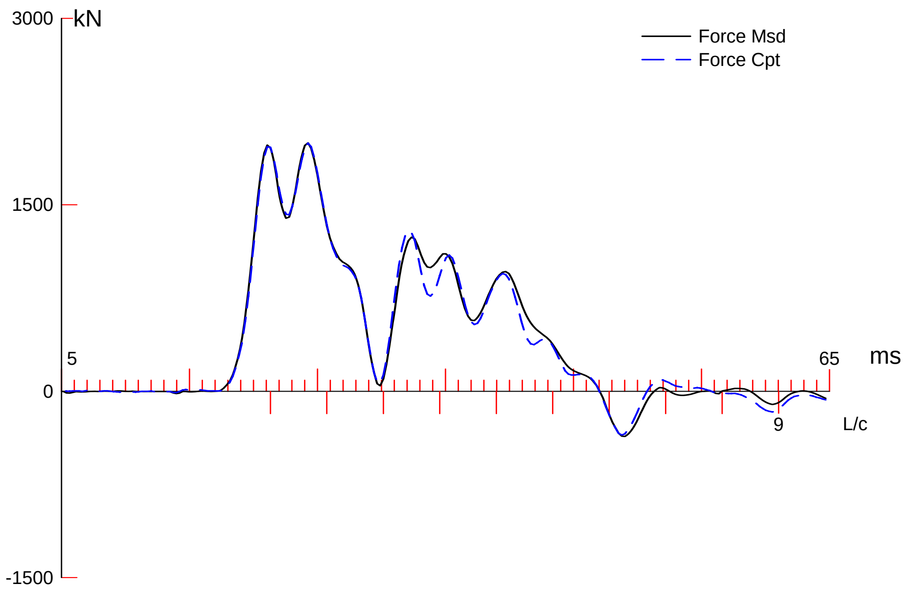
Ru = 2411.0 kN
Rs = 1095.0 kN
Rb = 1316.0 kN
Dy = 13.8 mm
Dx = 15.0 mm

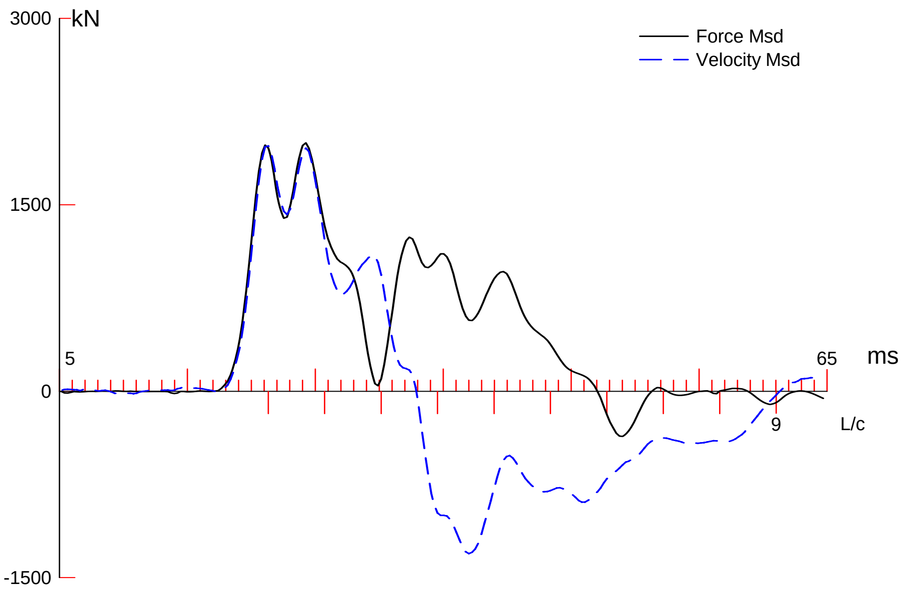


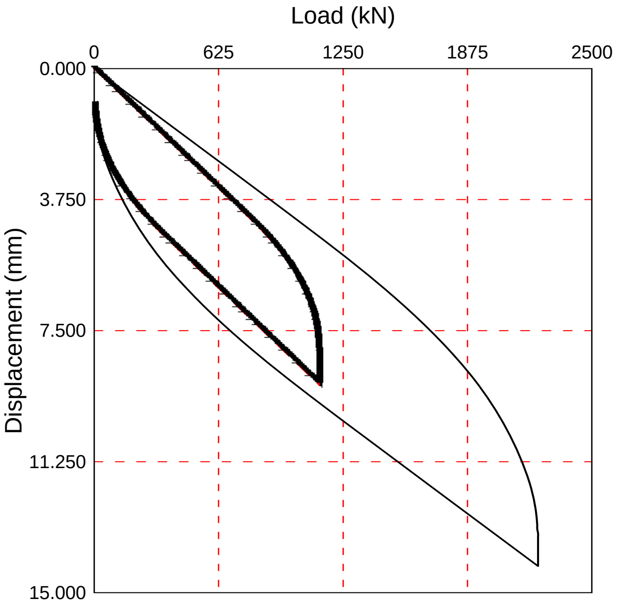
```
                              CAPWAP SUMMARY RESULTS
Total CAPWAP Capacity:   2411.0; along Shaft   1095.0; at Toe   1316.0  kN
   Soil     Dist.    Depth       Ru      Force        Sum      Unit      Unit     Smith
  Sgmnt     Below    Below             in Pile         of   Resist.   Resist.   Damping
    No.     Gages    Grade                             Ru   (Depth)    (Area)    Factor
                m        m       kN         kN         kN      kN/m       kPa       s/m

                                         2411.0
      1       2.0      0.8      1.1     2409.9        1.1      1.34      0.96     0.400
      2       4.0      2.8      4.4     2405.5        5.5      2.18      1.55     0.400
      3       6.1      4.9      7.7     2397.8       13.2      3.82      2.71     0.400
      4       8.1      6.9     13.8     2384.0       27.0      6.84      4.86     0.400
      5      10.1      8.9     24.2     2359.8       51.2     11.99      8.53     0.400
      6      12.1     10.9     57.3     2302.5      108.5     28.39     20.19     0.400
      7      14.1     12.9     62.8     2239.7      171.3     31.12     22.12     0.400
      8      16.1     14.9     74.9     2164.8      246.2     37.11     26.39     0.400
      9      18.2     17.0     78.2     2086.6      324.4     38.75     27.55     0.400
     10      20.2     19.0    170.7     1915.9      495.1     84.58     60.14     0.400
     11      22.2     21.0    599.9     1316.0     1095.0    297.25    211.35     0.400

  Avg. Shaft                   99.5                           52.14     37.07     0.400

       Toe                   1316.0                                   9218.79     0.200

Soil Model Parameters/Extensions                         Shaft      Toe

Quake                      (mm)                          4.001    5.129
Case Damping Factor                                      0.277    0.167
Unloading Quake            (% of loading quake)            110       30
Reloading Level            (% of Ru)                       100      100
Unloading Level            (% of Ru)                        25
Resistance Gap (included in Toe Quake) (mm)                       0.229
Soil Plug Weight           (kN)                                    0.02

CAPWAP match quality       =    4.35         (Wave Up Match) ; RSA = 0
Observed: final set        =   1.250 mm;     blow count      =     800 b/m
Computed: final set        =   0.658 mm;     blow count      =    1521 b/m

max. Top Comp. Stress      =    15.2 MPa      (T=  21.2 ms, max= 1.099 x Top)
max. Comp. Stress          =    16.7 MPa      (Z=  20.2 m, T=  28.9 ms)
max. Tens. Stress          =   -2.45 MPa      (Z=  10.1 m, T=  47.2 ms)
max. Energy (EMX)          =   14.38 kJ;      max. Measured Top Displ. (DMX)=10.24 mm
```

```
------------------------------------------------------------------------------------------
                                      EXTREMA TABLE
------------------------------------------------------------------------------------------
      Pile     Dist.      max.      min.      max.      max.      max.      max.      max.
     Sgmnt     Below     Force     Force     Comp.     Tens.   Trnsfd.    Veloc.    Displ.
       No.     Gages                        Stress    Stress    Energy
                   m        kN        kN       MPa       MPa        kJ       m/s        mm
------------------------------------------------------------------------------------------
         1       1.0    2173.0    -257.0      15.2     -1.80     14.38       1.4     9.864
         2       2.0    2173.9    -312.1      15.2     -2.19     14.27       1.4     9.682
         3       3.0    2173.1    -340.8      15.2     -2.39     14.14       1.4     9.493
         4       4.0    2174.0    -348.7      15.2     -2.44     14.05       1.4     9.329
         5       5.0    2170.7    -334.2      15.2     -2.34     13.93       1.4     9.178
         6       6.1    2174.0    -322.3      15.2     -2.26     13.85       1.4     9.020
         7       7.1    2171.6    -318.0      15.2     -2.23     13.67       1.4     8.842
         8       8.1    2177.6    -334.0      15.3     -2.34     13.54       1.4     8.646
         9       9.1    2172.5    -345.7      15.2     -2.42     13.25       1.4     8.430
        10      10.1    2182.6    -349.6      15.3     -2.45     13.08       1.3     8.196
        11      11.1    2173.6    -313.0      15.2     -2.19     12.69       1.3     7.974
        12      12.1    2187.3    -272.4      15.3     -1.91     12.55       1.3     7.763
        13      13.1    2144.3    -251.3      15.0     -1.76     11.91       1.4     7.547
        14      14.1    2155.6    -247.1      15.1     -1.73     11.73       1.5     7.314
        15      15.1    2114.9    -233.1      14.8     -1.63     11.03       1.5     7.071
        16      16.1    2121.1    -239.9      14.9     -1.68     10.82       1.4     6.816
        17      17.2    2171.2    -228.7      15.2     -1.60     10.07       1.3     6.568
        18      18.2    2377.6    -253.1      16.7     -1.77      9.87       1.3     6.322
        19      19.2    2380.3    -244.5      16.7     -1.71      9.18       1.4     6.080
        20      20.2    2388.0    -256.3      16.7     -1.80      8.96       1.6     5.822
        21      21.2    2117.5    -203.9      14.8     -1.43      7.75       1.6     5.563
        22      22.2    2146.3    -218.6      15.0     -1.53      4.78       1.6     5.274

Absolute      20.2                          16.7                        (T =      28.9 ms)
              10.1                                   -2.45              (T =      47.2 ms)


                                       CASE METHOD
------------------------------------------------------------------------------------------
J =          0.0     0.1     0.2     0.3     0.4     0.5     0.6     0.7     0.8     0.9
RP        2015.9  1795.7  1575.6  1355.4  1135.2   915.1   694.9   474.7   254.6    34.4
RX        2915.6  2810.6  2705.6  2616.7  2540.8  2477.9  2436.0  2400.2  2380.0  2375.3
RU           0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0

RAU =   2296.5 (kN);  RA2 =   2296.5 (kN)

Current CAPWAP Ru = 2411.0 (kN); Corresponding J(RP)= 0.00; J(RX) = 0.67

      VMX      TVP    VT1*Z      FT1      FMX      DMX      DFN      SET      EMX      QUS
      m/s       ms       kN       kN       kN       mm       mm       mm       kJ       kN
     1.42    21.15   2193.8   2023.8   2105.0   10.236   -0.065    1.250     14.4   2513.4


                               PILE PROFILE AND PILE MODEL
------------------------------------------------------------------------------------------
             Depth              Area         E-Modulus      Spec. Weight            Perim.
                 m               cm²               MPa             kN/m³                 m
------------------------------------------------------------------------------------------
              0.00           1427.52           50000.0            24.000             1.406
             22.20           1427.52           50000.0            24.000             1.406

Toe Area                       0.143       m²

Top Segment Length      1.01 m, Top Impedance  1579.11 kN/m/s
```

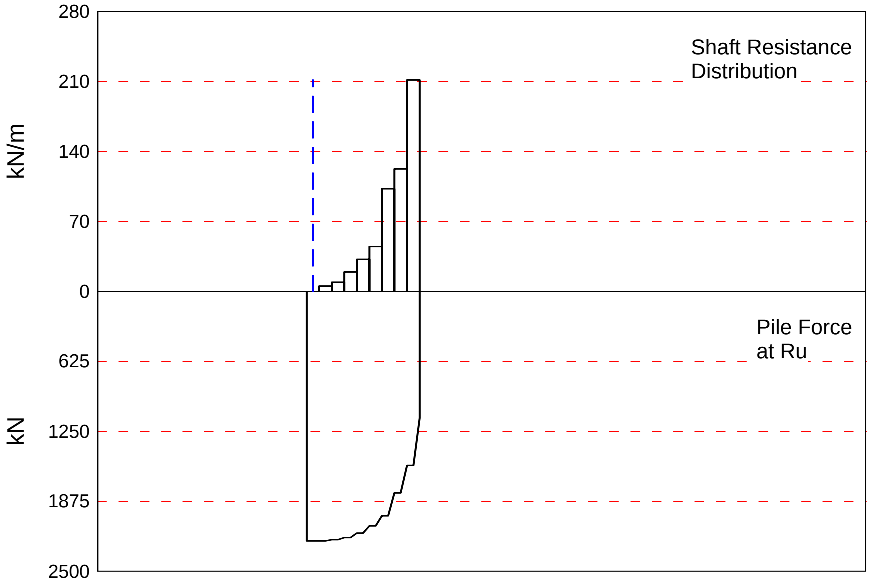
```
Pile Damping    2.0 %, Time Incr  0.243 ms, Wave Speed   4150.0 m/s, 2L/c  10.7 ms
```





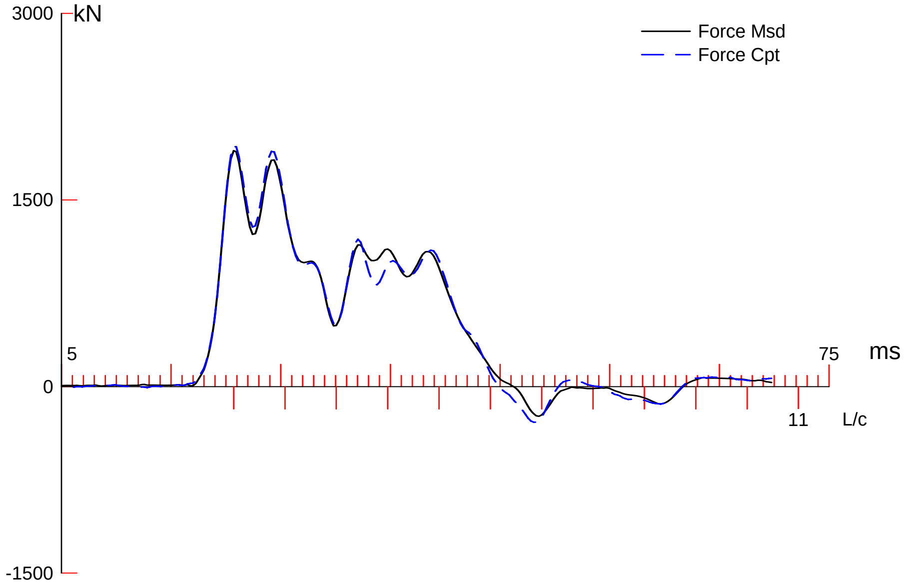


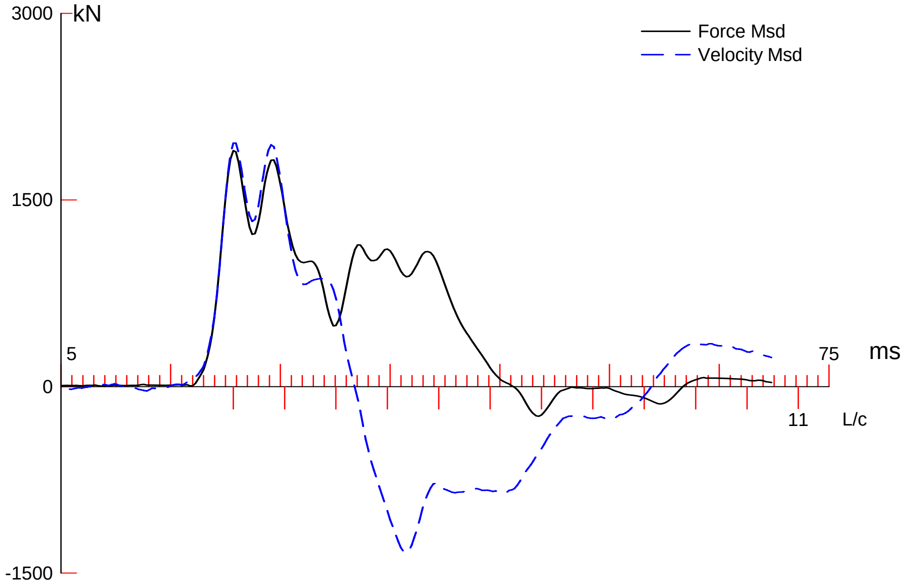
Ru = 2229.0 kN
Rs = 1097.3 kN
Rb = 1131.7 kN
Dy = 13.3 mm
Dx = 14.2 mm

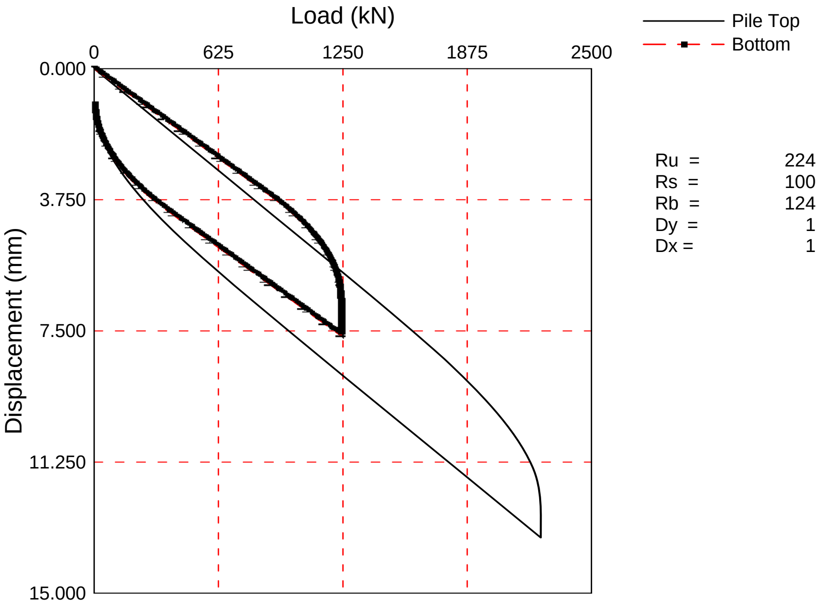


```
                                  CAPWAP SUMMARY RESULTS
Total CAPWAP Capacity:   2229.0; along Shaft   1097.3; at Toe   1131.7  kN
    Soil    Dist.     Depth       Ru      Force       Sum       Unit       Unit     Smith
   Sgmnt    Below     Below             in Pile        of    Resist.    Resist.   Damping
     No.    Gages     Grade                            Ru    (Depth)     (Area)    Factor
                m         m       kN         kN        kN       kN/m        kPa       s/m

                                          2229.0
       1      4.0       2.8     10.9     2218.1      10.9       3.89       2.77     0.250
       2      6.0       4.8     18.8     2199.3      29.7       9.40       6.68     0.250
       3      8.0       6.8     39.2     2160.1      68.9      19.60      13.94     0.250
       4     10.0       8.8     64.3     2095.8     133.2      32.15      22.86     0.250
       5     12.0      10.8     90.0     2005.8     223.2      45.00      32.00     0.250
       6     14.0      12.8    205.5     1800.3     428.7     102.75      73.06     0.250
       7     16.0      14.8    245.4     1554.9     674.1     122.70      87.24     0.250
       8     18.0      16.8    423.2     1131.7    1097.3     211.60     150.45     0.250

  Avg. Shaft                   137.2                            65.32      46.44     0.250

       Toe                    1131.7                                     7927.73     0.300

Soil Model Parameters/Extensions                         Shaft      Toe

Quake                      (mm)                          4.002    6.200
Case Damping Factor                                      0.174    0.215
Damping Type                                                      Smith
Unloading Quake            (% of loading quake)             50      110
Reloading Level            (% of Ru)                       100      100
Unloading Level            (% of Ru)                        28
Soil Plug Weight           (kN)                                    0.80
Soil Support Dashpot                                     1.000    0.000
Soil Support Weight        (kN)                          14.06     0.00

CAPWAP match quality       =    4.53        (Wave Up Match) ; RSA = 0
Observed: final set        =   1.000 mm;    blow count      =    1000 b/m
Computed: final set        =   0.100 mm;    blow count      =    9999 b/m

max. Top Comp. Stress      =    14.1 MPa      (T=  24.8 ms, max= 1.078 x Top)
max. Comp. Stress          =    15.2 MPa      (Z=  14.0 m, T=  28.4 ms)
max. Tens. Stress          =   -2.48 MPa      (Z=   1.0 m, T=  49.0 ms)
max. Energy (EMX)          =   13.03 kJ;      max. Measured Top Displ. (DMX)= 9.89 mm
```

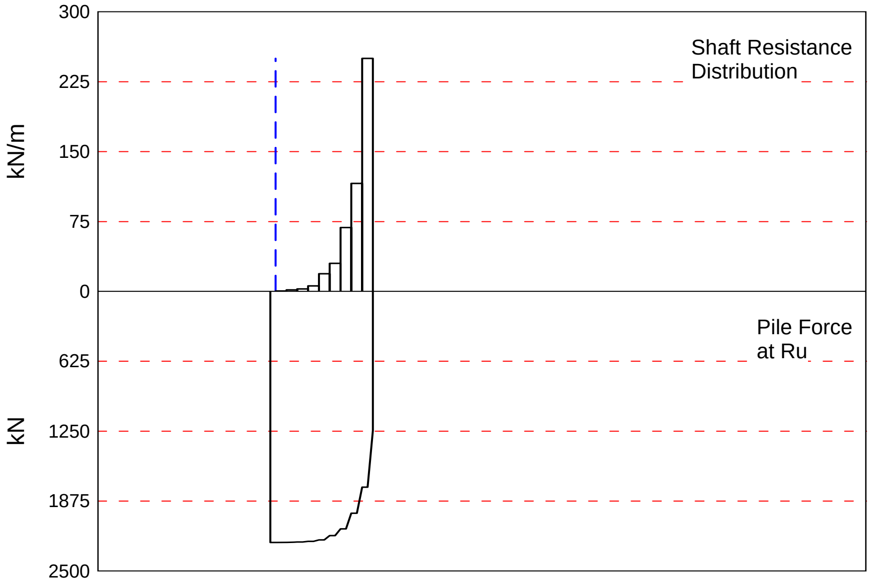
```
                                  EXTREMA TABLE
     Pile     Dist.      max.      min.      max.      max.      max.      max.      max.
    Sgmnt     Below     Force     Force     Comp.     Tens.   Trnsfd.    Veloc.    Displ.
      No.     Gages                        Stress    Stress    Energy
                  m        kN        kN       MPa       MPa        kJ       m/s        mm

        1       1.0    2007.8    -354.4      14.1     -2.48     13.03       1.3     9.657
        2       2.0    2023.2    -328.7      14.2     -2.30     12.92       1.2     9.481
        3       3.0    2044.8    -297.6      14.3     -2.08     12.79       1.2     9.276
        4       4.0    2067.3    -268.9      14.5     -1.88     12.63       1.2     9.055
        5       5.0    2066.9    -250.9      14.5     -1.76     12.41       1.2     8.859
        6       6.0    2059.2    -259.5      14.4     -1.82     12.33       1.2     8.697
        7       7.0    1986.6    -266.3      13.9     -1.87     12.09       1.2     8.539
        8       8.0    1988.5    -272.2      13.9     -1.91     11.99       1.3     8.370
        9       9.0    1979.6    -269.8      13.9     -1.89     11.57       1.4     8.193
       10      10.0    1999.8    -270.9      14.0     -1.90     11.44       1.5     7.994
       11      11.0    1962.5    -247.2      13.7     -1.73     10.81       1.6     7.780
       12      12.0    1951.5    -252.2      13.7     -1.77     10.62       1.5     7.543
       13      13.0    1950.7    -233.8      13.7     -1.64      9.80       1.4     7.302
       14      14.0    2165.2    -239.4      15.2     -1.68      9.60       1.3     7.052
       15      15.0    2048.7    -174.6      14.4     -1.22      8.16       1.5     6.834
       16      16.0    2059.3    -177.2      14.4     -1.24      8.00       1.6     6.612
       17      17.0    1704.1    -111.7      11.9     -0.78      6.47       1.6     6.409
       18      18.0    1678.3    -112.4      11.8     -0.79      4.41       1.6     6.185

Absolute       14.0                          15.2                        (T =     28.4 ms)
                1.0                                   -2.48              (T =     49.0 ms)


                                   CASE METHOD
J =          0.0     0.1     0.2     0.3     0.4     0.5     0.6     0.7     0.8     0.9
RP        1470.8  1216.9   963.1   709.3   455.4   201.6     0.0     0.0     0.0     0.0
RX        2575.8  2435.6  2300.5  2199.0  2149.8  2130.9  2121.9  2113.8  2113.8  2113.8
RU        1470.8  1216.9   963.1   709.3   455.4   201.6     0.0     0.0     0.0     0.0

RAU =   2113.8 (kN);  RA2 =   2175.9 (kN)

Current CAPWAP Ru = 2229.0 (kN); Corresponding J(RP)= 0.00; J(RX) = 0.27

      VMX      TVP    VT1*Z       FT1       FMX       DMX       DFN       SET       EMX       QUS
      m/s       ms       kN        kN        kN        mm        mm        mm        kJ        kN
     1.27    21.57   2007.7    2001.5    2019.6     9.887     0.930     1.000      13.2    2424.0


                            PILE PROFILE AND PILE MODEL
             Depth               Area            E-Modulus     Spec. Weight              Perim.
                 m                cm²                  MPa            kN/m³                   m

              0.00            1427.52              50000.0           24.000               1.406
             18.00            1427.52              50000.0           24.000               1.406

Toe Area                        0.143        m²

Top Segment Length      1.00 m, Top Impedance  1579.11 kN/m/s

Pile Damping    2.0 %, Time Incr  0.245 ms, Wave Speed   4080.0 m/s, 2L/c   8.8 ms
```







Ru = 2244.9 kN
Rs = 1000.4 kN
Rb = 1244.5 kN
Dy = 12.4 mm
Dx = 13.4 mm

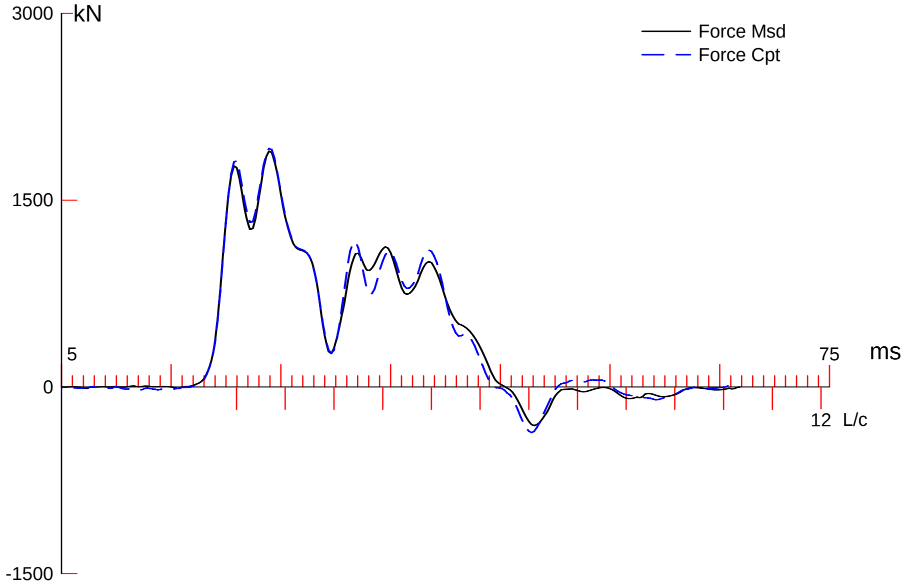


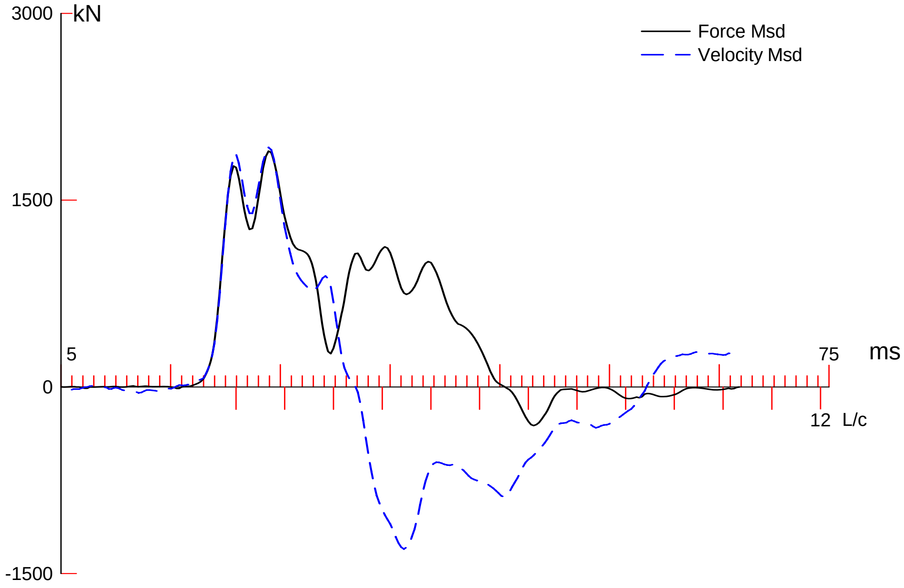
```
SEWPCC; Pile: 5                                                  Test: 18-Jan-2014 17:18:
Restrike; Blow: 3                                                       CAPWAP(R)  2006-2
AATech Scientific Inc                                                              OP: DF
-----------------------------------------------------------------------------------------
                                  CAPWAP SUMMARY RESULTS
Total CAPWAP Capacity:   2244.9; along Shaft   1000.4; at Toe   1244.5  kN
-----------------------------------------------------------------------------------------
   Soil     Dist.    Depth        Ru      Force       Sum       Unit      Unit      Smith
  Sgmnt     Below    Below              in Pile        of    Resist.   Resist.    Damping
    No.     Gages    Grade                             Ru    (Depth)    (Area)     Factor
                m        m        kN         kN        kN       kN/m       kPa        s/m
-----------------------------------------------------------------------------------------
                                          2244.9
      1       3.0      1.8       1.1     2243.8       1.1       0.60      0.43      0.380
      2       5.1      3.9       3.3     2240.5       4.4       1.63      1.16      0.380
      3       7.1      5.9       5.5     2235.0       9.9       2.72      1.93      0.380
      4       9.1      7.9      12.2     2222.8      22.1       6.04      4.29      0.380
      5      11.1      9.9      38.5     2184.3      60.6      19.05     13.54      0.380
      6      13.1     11.9      61.1     2123.2     121.7      30.23     21.50      0.380
      7      15.2     14.0     138.8     1984.4     260.5      68.68     48.83      0.380
      8      17.2     16.0     234.4     1750.0     494.9     115.98     82.46      0.380
      9      19.2     18.0     505.5     1244.5    1000.4     250.12    177.84      0.380

  Avg. Shaft                   111.2                            55.58     39.52      0.380

       Toe                    1244.5                                    8717.92      0.350
Soil Model Parameters/Extensions                         Shaft      Toe
-----------------------------------------------------------------------------------------
Quake                      (mm)                          4.900    5.100
Case Damping Factor                                      0.241    0.276
Unloading Quake            (% of loading quake)             33      110
Reloading Level            (% of Ru)                       100      100
Unloading Level            (% of Ru)                        41
Soil Plug Weight           (kN)                                    0.30
-----------------------------------------------------------------------------------------
CAPWAP match quality       =    3.53        (Wave Up Match) ; RSA = 0
Observed: final set        =   1.000 mm;    blow count      =    1000 b/m
Computed: final set        =   1.083 mm;    blow count      =     923 b/m
max. Top Comp. Stress      =    13.5 MPa      (T=  21.2 ms, max= 1.217 x Top)
max. Comp. Stress          =    16.5 MPa      (Z=  15.2 m, T=  28.6 ms)
max. Tens. Stress          =   -2.10 MPa      (Z=   4.0 m, T=  47.6 ms)
max. Energy (EMX)          =   12.92 kJ;      max. Measured Top Displ. (DMX)= 9.73 mm
```

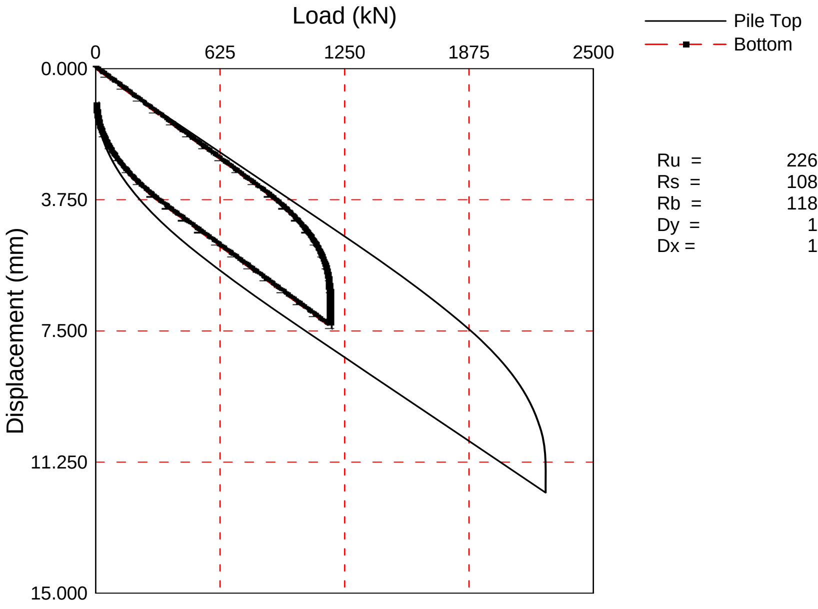
```
                                      EXTREMA TABLE
      Pile     Dist.      max.      min.      max.      max.      max.      max.      max.
     Sgmnt     Below     Force     Force     Comp.     Tens.   Trnsfd.    Veloc.    Displ.
       No.     Gages                        Stress    Stress    Energy
                   m        kN        kN       MPa       MPa        kJ       m/s        mm

         1       1.0    1932.9    -298.9      13.5     -2.09     12.92       1.2     9.409
         2       2.0    1932.9    -296.9      13.5     -2.08     12.82       1.2     9.237
         3       3.0    1931.9    -295.8      13.5     -2.07     12.72       1.2     9.057
         4       4.0    1927.0    -299.4      13.5     -2.10     12.59       1.2     8.865
         5       5.1    1937.9    -298.9      13.6     -2.09     12.45       1.2     8.658
         6       6.1    1940.4    -296.5      13.6     -2.08     12.27       1.2     8.439
         7       7.1    1939.5    -288.2      13.6     -2.02     12.13       1.2     8.225
         8       8.1    1928.3    -284.5      13.5     -1.99     11.94       1.2     8.016
         9       9.1    1938.6    -288.8      13.6     -2.02     11.79       1.2     7.798
        10      10.1    1940.0    -286.9      13.6     -2.01     11.53       1.3     7.566
        11      11.1    1953.5    -289.7      13.7     -2.03     11.33       1.3     7.315
        12      12.1    1932.3    -271.3      13.5     -1.90     10.83       1.2     7.054
        13      13.1    2094.0    -265.7      14.7     -1.86     10.59       1.2     6.775
        14      14.1    2216.7    -247.5      15.5     -1.73      9.96       1.2     6.501
        15      15.2    2352.7    -270.5      16.5     -1.89      9.73       1.2     6.230
        16      16.2    2247.4    -223.3      15.7     -1.56      8.76       1.3     5.979
        17      17.2    2235.0    -244.4      15.7     -1.71      8.55       1.3     5.714
        18      18.2    1922.3    -147.3      13.5     -1.03      7.23       1.3     5.467
        19      19.2    1988.4    -173.5      13.9     -1.22      5.15       1.3     5.202

Absolute      15.2                          16.5                       (T =     28.6 ms)
               4.0                                     -2.10             (T =     47.6 ms)


                                       CASE METHOD
J =            0.0     0.1     0.2     0.3     0.4     0.5     0.6     0.7     0.8     0.9
RP          1866.7  1662.3  1457.9  1253.6  1049.2   844.8   640.5   436.1   231.8    27.4
RX          2781.7  2681.3  2581.0  2482.9  2403.8  2336.7  2276.9  2241.3  2211.4  2200.6
RU          1866.7  1662.3  1457.9  1253.6  1049.2   844.8   640.5   436.1   231.8    27.4

RAU =   2154.2 (kN);  RA2 =   2223.1 (kN)

Current CAPWAP Ru = 2244.9 (kN); Corresponding J(RP)= 0.00; J(RX) = 0.69

      VMX      TVP    VT1*Z      FT1      FMX      DMX      DFN      SET      EMX      QUS
      m/s       ms       kN       kN       kN       mm       mm       mm       kJ       kN
     1.26    20.95   1986.7   1923.6   1923.6    9.733    0.998    1.000     13.1   2433.7


                               PILE PROFILE AND PILE MODEL
             Depth              Area         E-Modulus      Spec. Weight            Perim.
                 m               cm²               MPa             kN/m³                 m

              0.00           1427.52           50000.0            24.000             1.406
             19.20           1427.52           50000.0            24.000             1.406

Toe Area                       0.143       m²

Top Segment Length      1.01 m, Top Impedance  1579.11 kN/m/s

Pile Damping    2.0 %, Time Incr  0.246 ms, Wave Speed   4100.0 m/s, 2L/c   9.4 ms
```

SEWPCC; Pile: 6; R; Blow: 4 (Test: 18-Jan-2014 16:50:)
AATech Scientific Inc

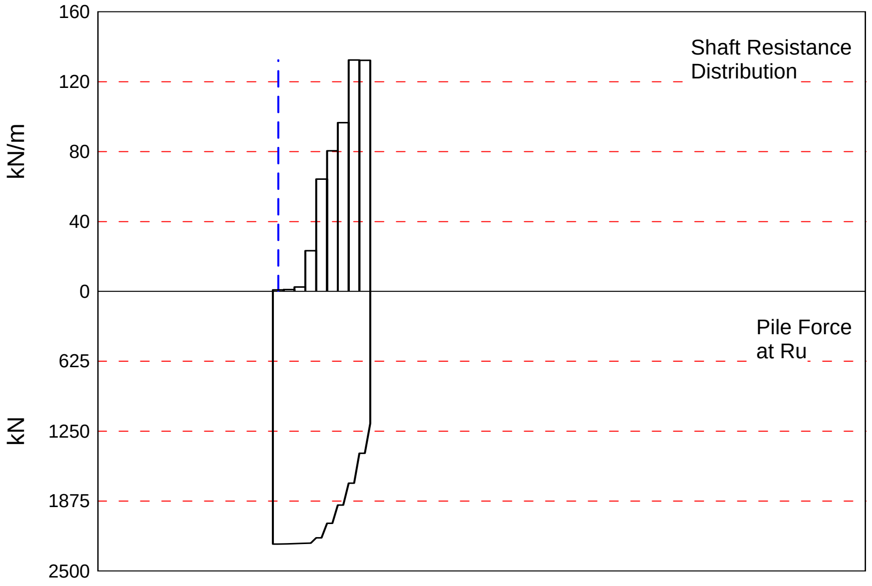
03-Feb-2014
CAPWAP(R) 2006-2





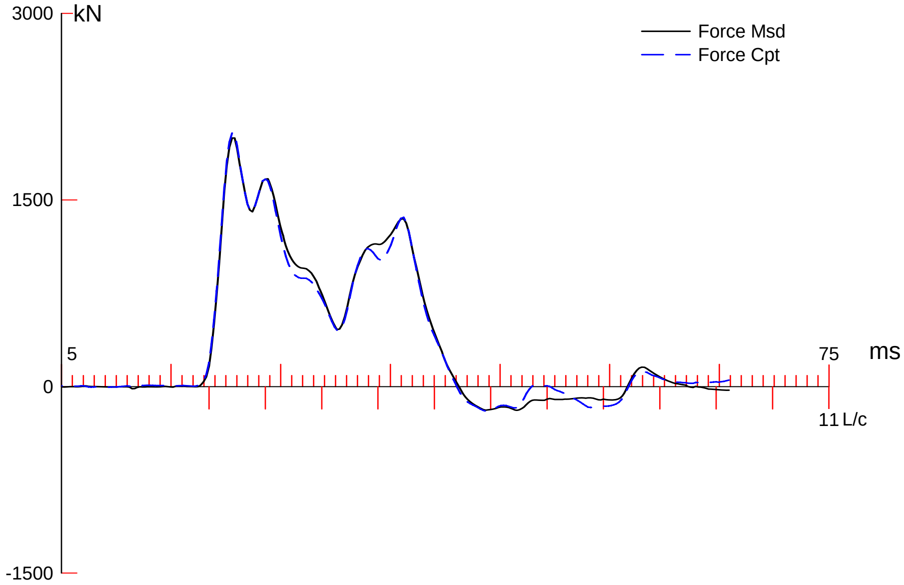


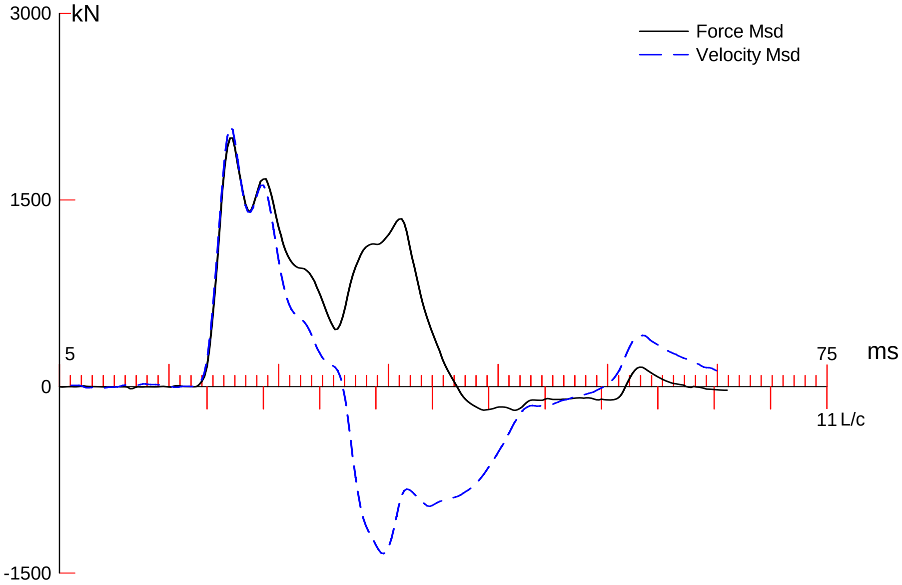
Ru = 2260.1 kN
Rs = 1080.1 kN
Rb = 1180.0 kN
Dy = 11.1 mm
Dx = 12.1 mm

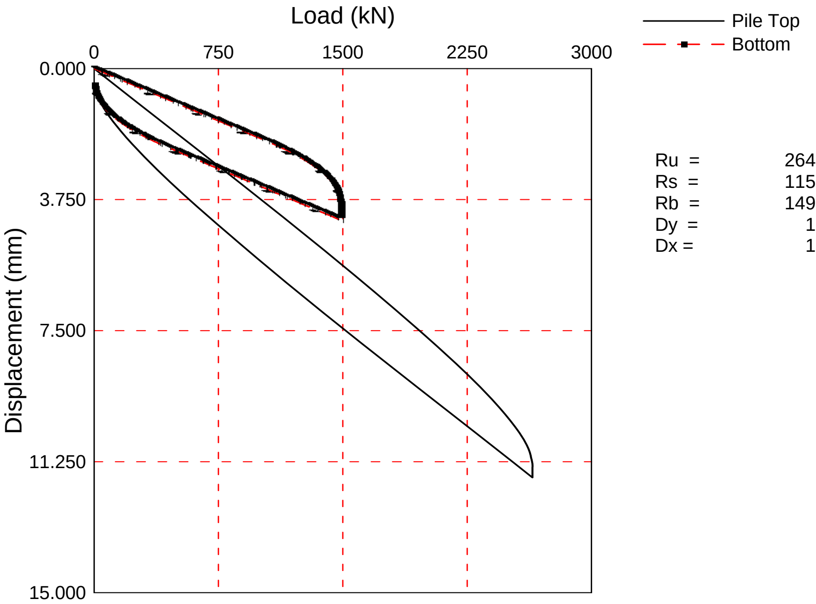


```
                                  CAPWAP SUMMARY RESULTS
Total CAPWAP Capacity:   2260.1; along Shaft   1080.1; at Toe   1180.0  kN
   Soil   Dist.    Depth      Ru     Force       Sum      Unit     Unit     Smith    Quake
  Sgmnt   Below    Below            in Pile       of   Resist.  Resist.   Damping
    No.   Gages    Grade                          Ru   (Depth)   (Area)    Factor
              m        m      kN        kN        kN      kN/m      kPa       s/m       mm

                                    2260.1
      1     2.0      0.9     1.8    2258.3       1.8      1.95     1.39     0.270    4.200
      2     4.0      2.9     2.3    2256.0       4.1      1.14     0.81     0.270    4.201
      3     6.1      5.0     5.3    2250.7       9.4      2.62     1.86     0.270    4.201
      4     8.1      7.0    47.2    2203.5      56.6     23.34    16.60     0.270    4.201
      5    10.1      9.0   130.1    2073.4     186.7     64.34    45.74     0.270    4.201
      6    12.1     11.0   162.7    1910.7     349.4     80.46    57.21     0.270    4.201
      7    14.2     13.1   195.3    1715.4     544.7     96.58    68.67     0.270    4.201
      8    16.2     15.1   267.9    1447.5     812.6    132.48    94.19     0.270    4.034
      9    18.2     17.1   267.5    1180.0    1080.1    132.28    94.05     0.270    3.704

  Avg. Shaft               120.0                        63.16    44.91     0.270    4.037

       Toe                1180.0                              8266.08     0.210    4.907

Soil Model Parameters/Extensions                         Shaft      Toe

Case Damping Factor                                      0.176    0.150
Unloading Quake            (% of loading quake)             44      109
Reloading Level            (% of Ru)                       100      100
Unloading Level            (% of Ru)                        27
Resistance Gap (included in Toe Quake) (mm)                       0.007
Soil Plug Weight           (kN)                                    0.25

CAPWAP match quality       =    4.87         (Wave Up Match) ; RSA = 0
Observed: final set        =   1.000 mm;     blow count      =    1000 b/m
Computed: final set        =   0.197 mm;     blow count      =    5069 b/m

max. Top Comp. Stress      =    13.5 MPa      (T=  24.4 ms, max= 1.057 x Top)
max. Comp. Stress          =    14.3 MPa      (Z=  14.2 m, T=  28.1 ms)
max. Tens. Stress          =   -2.59 MPa      (Z=   4.0 m, T=  46.9 ms)
max. Energy (EMX)          =   11.39 kJ;      max. Measured Top Displ. (DMX)= 8.51 mm
```

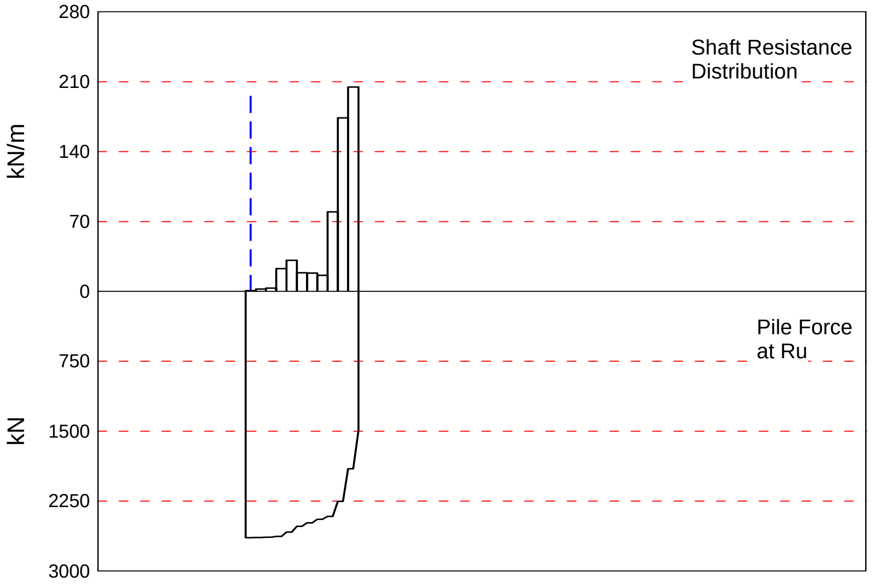
```
                                EXTREMA TABLE
     Pile     Dist.      max.      min.      max.      max.      max.      max.      max.
    Sgmnt     Below     Force     Force     Comp.     Tens.   Trnsfd.    Veloc.    Displ.
      No.     Gages                        Stress    Stress    Energy
                  m        kN        kN       MPa       MPa        kJ       m/s        mm

        1       1.0    1926.7    -356.3      13.5     -2.50     11.39       1.1     8.328
        2       2.0    1939.6    -353.1      13.6     -2.47     11.33       1.1     8.193
        3       3.0    1957.3    -352.2      13.7     -2.47     11.23       1.1     8.042
        4       4.0    1981.0    -369.8      13.9     -2.59     11.14       1.1     7.873
        5       5.1    2004.8    -369.1      14.0     -2.59     11.00       1.1     7.688
        6       6.1    2028.6    -351.7      14.2     -2.46     10.89       1.1     7.504
        7       7.1    2027.1    -316.1      14.2     -2.21     10.76       1.1     7.339
        8       8.1    1996.0    -303.4      14.0     -2.13     10.66       1.1     7.164
        9       9.1    1841.8    -293.5      12.9     -2.06     10.24       1.2     6.980
       10      10.1    1852.0    -294.6      13.0     -2.06     10.11       1.3     6.779
       11      11.1    1793.0    -259.6      12.6     -1.82      9.18       1.3     6.576
       12      12.1    1813.2    -266.8      12.7     -1.87      9.01       1.3     6.349
       13      13.1    1858.2    -225.3      13.0     -1.58      7.96       1.2     6.134
       14      14.2    2036.5    -225.6      14.3     -1.58      7.79       1.1     5.902
       15      15.2    1890.4    -167.4      13.2     -1.17      6.69       1.2     5.694
       16      16.2    1883.0    -166.7      13.2     -1.17      6.53       1.3     5.475
       17      17.2    1504.9     -86.4      10.5     -0.60      5.22       1.4     5.283
       18      18.2    1549.9    -109.4      10.9     -0.77      4.21       1.4     5.074

Absolute      14.2                           14.3                          (T =     28.1 ms)
               4.0                                     -2.59               (T =     46.9 ms)


                                  CASE METHOD
J =          0.0     0.1     0.2     0.3     0.4     0.5     0.6     0.7     0.8     0.9
RP        1848.9  1661.6  1474.3  1287.0  1099.7   912.4   725.1   537.8   350.5   163.2
RX        2441.7  2299.1  2156.9  2060.3  2060.3  2060.3  2060.3  2060.3  2060.3  2060.3
RU        1848.9  1661.6  1474.3  1287.0  1099.7   912.4   725.1   537.8   350.5   163.2

RAU =   2025.8 (kN);  RA2 =   2011.4 (kN)

Current CAPWAP Ru = 2260.1 (kN); Corresponding J(RP)= 0.00; J(RX) = 0.13

      VMX      TVP    VT1*Z      FT1      FMX      DMX      DFN      SET      EMX      QUS
      m/s       ms       kN       kN       kN       mm       mm       mm       kJ       kN
     1.18    21.21   1908.7   1813.3   1919.3    8.514    0.998    1.000     11.5   2421.7


                          PILE PROFILE AND PILE MODEL
             Depth                Area           E-Modulus     Spec. Weight              Perim.
                 m                 cm²                 MPa            kN/m³                   m

              0.00             1427.52             55000.0           24.000               1.406
             18.20             1427.52             55000.0           24.000               1.406

Toe Area                         0.143        m²

Top Segment Length      1.01 m, Top Impedance  1656.18 kN/m/s

Pile Damping    2.0 %, Time Incr  0.247 ms, Wave Speed   4100.0 m/s, 2L/c   8.9 ms
```







Ru = 2643.4 kN
Rs = 1150.4 kN
Rb = 1493.0 kN
Dy = 11.3 mm
Dx = 11.7 mm



```
                                 CAPWAP SUMMARY RESULTS
Total CAPWAP Capacity:   2643.4; along Shaft   1150.4; at Toe   1493.0  kN
   Soil   Dist.    Depth      Ru     Force      Sum      Unit     Unit     Smith    Quake
  Sgmnt   Below    Below           in Pile       of   Resist.  Resist.   Damping
    No.   Gages    Grade                         Ru   (Depth)   (Area)    Factor
              m        m      kN        kN       kN      kN/m      kPa       s/m       mm

                                    2643.4
      1     2.0      0.9     1.6    2641.8      1.6      1.76     1.25     0.400    3.500
      2     4.0      2.9     4.9    2636.9      6.5      2.44     1.73     0.400    3.501
      3     6.0      4.9     6.7    2630.2     13.2      3.33     2.37     0.400    3.501
      4     8.0      6.9    46.1    2584.1     59.3     22.95    16.31     0.400    3.501
      5    10.0      8.9    62.9    2521.2    122.2     31.31    22.26     0.400    3.501
      6    12.1     11.0    37.7    2483.5    159.9     18.76    13.34     0.400    3.501
      7    14.1     13.0    36.9    2446.6    196.8     18.37    13.06     0.400    3.501
      8    16.1     15.0    32.7    2413.9    229.5     16.28    11.57     0.400    3.501
      9    18.1     17.0   160.2    2253.7    389.7     79.74    56.70     0.400    3.501
     10    20.1     19.0   349.3    1904.4    739.0    173.86   123.62     0.400    3.501
     11    22.1     21.0   411.4    1493.0   1150.4    204.77   145.60     0.400    3.033

  Avg. Shaft               104.6                        54.78    38.95     0.400    3.334

       Toe                1493.0                              10458.70     0.390    2.981

Soil Model Parameters/Extensions                         Shaft      Toe

Case Damping Factor                                      0.286    0.362
Unloading Quake            (% of loading quake)             40       98
Reloading Level            (% of Ru)                       100      100
Unloading Level            (% of Ru)                        30
Resistance Gap (included in Toe Quake) (mm)                       0.001
Soil Plug Weight           (kN)                                    0.39

CAPWAP match quality       =    3.60         (Wave Up Match) ; RSA = 0
Observed: final set        =   0.417 mm;     blow count      =    2400 b/m
Computed: final set        =   0.100 mm;     blow count      =    9999 b/m

max. Top Comp. Stress      =    14.3 MPa      (T=  21.0 ms, max= 1.169 x Top)
max. Comp. Stress          =    16.7 MPa      (Z=  18.1 m, T=  28.5 ms)
max. Tens. Stress          =   -2.80 MPa      (Z=   8.0 m, T=  45.1 ms)
max. Energy (EMX)          =   10.77 kJ;      max. Measured Top Displ. (DMX)= 7.53 mm
```

```
                                  EXTREMA TABLE
     Pile     Dist.      max.      min.      max.      max.      max.      max.      max.
    Sgmnt     Below     Force     Force     Comp.     Tens.   Trnsfd.    Veloc.    Displ.
      No.     Gages                        Stress    Stress    Energy
                  m        kN        kN       MPa       MPa        kJ       m/s        mm

        1       1.0    2038.9    -226.0      14.3     -1.58     10.77       1.3     7.532
        2       2.0    2038.7    -244.9      14.3     -1.72     10.74       1.3     7.441
        3       3.0    2040.6    -274.3      14.3     -1.92     10.69       1.3     7.343
        4       4.0    2041.4    -312.9      14.3     -2.19     10.65       1.3     7.228
        5       5.0    2040.4    -346.1      14.3     -2.42     10.55       1.3     7.100
        6       6.0    2051.2    -376.2      14.4     -2.64     10.48       1.3     6.950
        7       7.0    2057.3    -391.5      14.4     -2.74     10.33       1.2     6.784
        8       8.0    2070.7    -400.3      14.5     -2.80     10.21       1.2     6.595
        9       9.0    2039.5    -386.7      14.3     -2.71      9.76       1.2     6.398
       10      10.0    2051.7    -382.0      14.4     -2.68      9.61       1.2     6.190
       11      11.0    2000.6    -340.8      14.0     -2.39      9.07       1.2     5.985
       12      12.1    2008.0    -318.7      14.1     -2.23      8.92       1.2     5.771
       13      13.1    1982.8    -274.8      13.9     -1.92      8.53       1.2     5.549
       14      14.1    1997.5    -301.1      14.0     -2.11      8.33       1.2     5.303
       15      15.1    1983.9    -312.4      13.9     -2.19      7.90       1.2     5.042
       16      16.1    2066.9    -344.8      14.5     -2.42      7.63       1.2     4.756
       17      17.1    2232.2    -363.1      15.6     -2.54      7.18       1.2     4.459
       18      18.1    2384.0    -398.3      16.7     -2.79      6.86       1.2     4.153
       19      19.1    2303.5    -370.9      16.1     -2.60      6.03       1.2     3.871
       20      20.1    2336.1    -398.6      16.4     -2.79      5.74       1.2     3.581
       21      21.1    1983.4    -281.8      13.9     -1.97      4.57       1.2     3.325
       22      22.1    2013.3    -287.3      14.1     -2.01      3.64       1.1     3.052

Absolute       18.1                          16.7                         (T =     28.5 ms)
                8.0                                    -2.80              (T =     45.1 ms)


                                   CASE METHOD
J =          0.0     0.1     0.2     0.3     0.4     0.5     0.6     0.7     0.8     0.9
RP        2737.2  2593.2  2449.2  2305.2  2161.3  2017.3  1873.3  1729.3  1585.3  1441.4
RX        2929.7  2889.3  2848.9  2808.5  2768.1  2727.7  2687.3  2646.9  2606.5  2566.7
RU        2761.3  2619.7  2478.1  2336.6  2195.0  2053.4  1911.9  1770.3  1628.7  1487.1

RAU =   2155.2 (kN);  RA2 =   2542.7 (kN)

Current CAPWAP Ru = 2643.4 (kN); Corresponding J(RP)= 0.07; J(RX) = 0.71

      VMX      TVP    VT1*Z      FT1      FMX      DMX      DFN      SET      EMX      QUS
      m/s       ms       kN       kN       kN       mm       mm       mm       kJ       kN
     1.32    18.69   2118.6   2058.4   2058.4    7.532    0.416    0.417     10.8   2712.5


                            PILE PROFILE AND PILE MODEL
                 Depth             Area        E-Modulus     Spec. Weight         Perim.
                     m              cm²              MPa            kN/m³              m

                  0.00          1427.52          52000.0           24.000          1.406
                 22.10          1427.52          52000.0           24.000          1.406

Toe Area                          0.143        m²

Top Segment Length      1.00 m, Top Impedance  1610.38 kN/m/s
```

```
Pile Damping    2.0 %, Time Incr  0.234 ms, Wave Speed   4300.0 m/s, 2L/c  10.3 ms
```

APPENDIX B

DRIVING LOG RECORD FOR TEST PILES

South End Water Pollution Control Centre

CONTRACTOR - Subterranean

DRIVEN PILE INSPECTION REPORT

Project No.: 13-0338-002
Project: SEWPCC PDA Testing
Structure:

Inspector: C. Friesen
Driving Date: Jan. 29, 2014
Rig / Hammer: Manitowoc Crane w Juntton Hammer
Pile Cushion: oak cushion Block
Pile Type: Pre-cast Hexagonal

Pile Size (mm)	300	350	400	(blows per 25 mm penetration, maximum.
Set Criteria:	5	8	12	3 consecutive sets)
Redrive	7	12	18	

	Pile No.	Date Cast	Cast #	Pile Length	Pile Size	Pile Batter	Pile Plumbness (verticality)	Final 3 Sets				Re-Strike	Prebore Depth	Pile Stick-up	Depth Driven	Existing Grade Elevation	Cut-off Elevation	Tip Elevation	Remarks
								Specified: Blows/set	Penetration Set 1	Penetration Set 2	Penetration Set 3	Penetration							
5517426 144 637000	1	24-1-14	L2-6	24m	400	vertical	3 mm on 1.2m South	12	13	13	13		9.1m	~5.4m	9.5m	~232.7m±	—	~214.1m±	
5517460 144 636984	2	24-1-14	L2-1	24m	400		8mm on 1.2m NE	12	19	16	16		9.1m	3.0m	11.9m	~232.7m±	—	~211.7m±	
5517432 144636933 ✱	3	24-1-14	L2-5	24m	400		15mm on 1.2m NE	12	~~18~~	~~16~~	~~16~~		9.1m	~~2.45m~~	~~12.45m~~	~232.7m±	—	~209.0m±	Performed PDA and Pile dropped by ~50-25mm per set. Drove pile to the ground level. New stick-up is ~0.3m. Redrove and didn't refus
5517486 144 636744	4	24-1-14	L2-1	24m	400		15mm on 1.2m North	12	16	18	17		9.1m	~7.3m	7.6m	~232.5m±	—	~215.8m±	
5517510 144 636790	5	24-1-14	L2-3	24m	400		2mm on 1.2m SE	12	19	19	18		9.1m	~6.0m	8.9m	~233.1m±	—	~215.1m±	
5517567 144 636889	6	24-1-14	L2-2	24m	400		2mm on 1.2m SE	12	19	18	17		9.1m	~6.9m	8.0m	~232.7m±	—	~215.6m±	
5517587 144 636959	7	24-1-14	L2-4	24m	400		15mm on 1.2m South	12	19	18	17		9.1m	3.4m	11.5m	~233.7m±	—	~213.1m±	

Dropped hammer from 1' for all piles.
PDA tested all piles on Jan. 30/14

APPENDIX C

VIBRATION MONITORING FOR THE SEWPCC TEST PILE INSTALLATION – PHASE 1 VIBRATION MONITORING PROGRAM

KGS
GROUP
CONSULTING
ENGINEERS

MEMORANDUM

TO: Tony Ng, P.Eng. & Roy Houston, P.Eng.

FROM: David Suderman, EIT & Ken Dyck, EIT

DATE: October 24, 2014

PROJECT NO: 13-0338-002

RE: Vibration Monitoring for the SEWPCC Test Pile Installation

1.0 GENERAL

KGS Group retained Subterranean to provide test piling services at the South End Water Pollution Control Centre (SEWPCC) in Winnipeg. During the driving of the 7 pre-cast concrete test piles, KGS Group conducted the Phase 1 vibration monitoring program. The purpose of this vibration program is to provide data on vibration attenuation for use in the future planned expansion at the SEWPCC. This memorandum outlines the monitoring program and summarizes the data collected.

Three portable seismograph units were installed at varied distances away from each test pile during driving. The monitoring units installed were the "Minimate Plus" and the "Blastmate II" models produced by Instantel, which have a range of measurement of peak particle velocities (PPVs) up to 254 mm/s. External transducers (geophones) are attached to each unit to measure vibrations across a broad range of frequencies (2 to 400 Hz) in three axes (transverse, vertical, and longitudinal).

2.0 BACKGROUND INFORMATION

Peak particle velocity (PPV) is the calculated vector sum of the vibrations occurring along the three axes simultaneously, and are the best measure of the magnitude of soil movement.

The generally accepted tolerance level of ground vibrations to avoid damage to adjacent structures is 25 mm/s PPV. A more stringent limit of 12 mm/s is often set when construction activity is occurring adjacent to historical structures, and is the standard used by Parks Canada for application around sensitive structures. Cosmetic damage including cracking of plaster may occur at approximately 12.7 mm/s, while drywall is less sensitive and can withstand a PPV of 19 mm/s or greater without any negative effects.

For this construction site, PPV below 12 mm/s are not of concern. Construction practices and methodologies should be reviewed in the event that PPV within the range of 12-25 mm/s are recorded, and to allow changes to be made to avoid PPV exceeding 25 mm/s.

3.0 SITE: SOUTH END WATER POLLUTION CONTROL CENTRE (SEWPCC)

Vibration Monitoring was conducted on January 29, 2014 by Mr. Ken Dyck and Mr. David Suderman of KGS Group. 24m long precast concrete test piles were being driven in order to conduct a pile dynamic analysis (PDA) used for piling design at the site. The piles were driven using a crane-mounted hydraulic hammer.

Three monitors were set up at varying distances from the source during each pile operation in order to measure attenuation with distance. Each monitor was set to record maximum PPVs at 5 second intervals. The external transducers were installed along the ground surface underneath a 20kg sandbag to maintain firm contact. Photos 1 and 2 in the attached appendix show the typical setup and positioning of the vibration monitors.

Distance measurements were made with a standard tape measure and then confirmed by a KGS survey at the pile locations.

Results

Table 1 summarizes the maximum PPV observed during the monitoring program for each pile and monitor location. Monitor and test pile locations are displayed in Figure 1.

TABLE 1
SEWPCC SITE VIBRATION RESULTS

Test Pile Number	Monitor	Distance From Pile [m]	Maximum PPV Experienced [mm/s]
1	P1M1	36.3	1.45
	P1M2	41.2	1.28
	P1M3	60.0	0.40
2	P2M2	7.3	6.75
	P2M1	29.6	1.62
	P2M3	58.7	1.46
3	P3M3	7.2	9.24
	P3M1	30.0	1.80
	P3M2	66.6	1.18
4	P4M3	8.5	4.67
	P4M2	24.5	1.39
	P4M1	50.3	0.75
5	P5M1	3.0	8.10
	P5M2	28.5	2.73
	P5M3	44.3	1.24
6	P6M2	5.1	7.76
	P6M3	30.0	1.75
	P6M1	N/A*	N/A*
7	P7M1	4.0	4.47
	P7M3	21.2	3.81
	P7M2	73.5	0.97

* Note – Monitor 1 was not in use for the driving of pile #6.

As exhibited in the table above, the ground surface vibration magnitudes decrease with distance for each pile operation. The vibrations observed follow an exponential/logarithmic scale, as shown in Figure 2. This implies that the vibration increases at an increasing rate as the monitors were moved closer to the vibration source.

4.0 IMPLICATIONS OF MONITORING & RECOMMENDATIONS

This report is intended to provide a guide to probable "order-of-magnitude" ground vibration that may occur at the SEWPCC site during pile installation. Based on these results, it is unlikely that vibration-induced structural or aesthetic damage will occur to adjacent structures during pile installation. KGS Group recommends that existing structures should be monitored throughout piling operations to ensure that vibrations remain below established tolerance levels. Specifically, structures founded on deep foundations adjacent to new piling should be monitored for vibration that can be transmitted through the underlying till.

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Photo 1 – The Setup of Monitor M2

Photo 2 – Pile #1, Showing Monitor M1

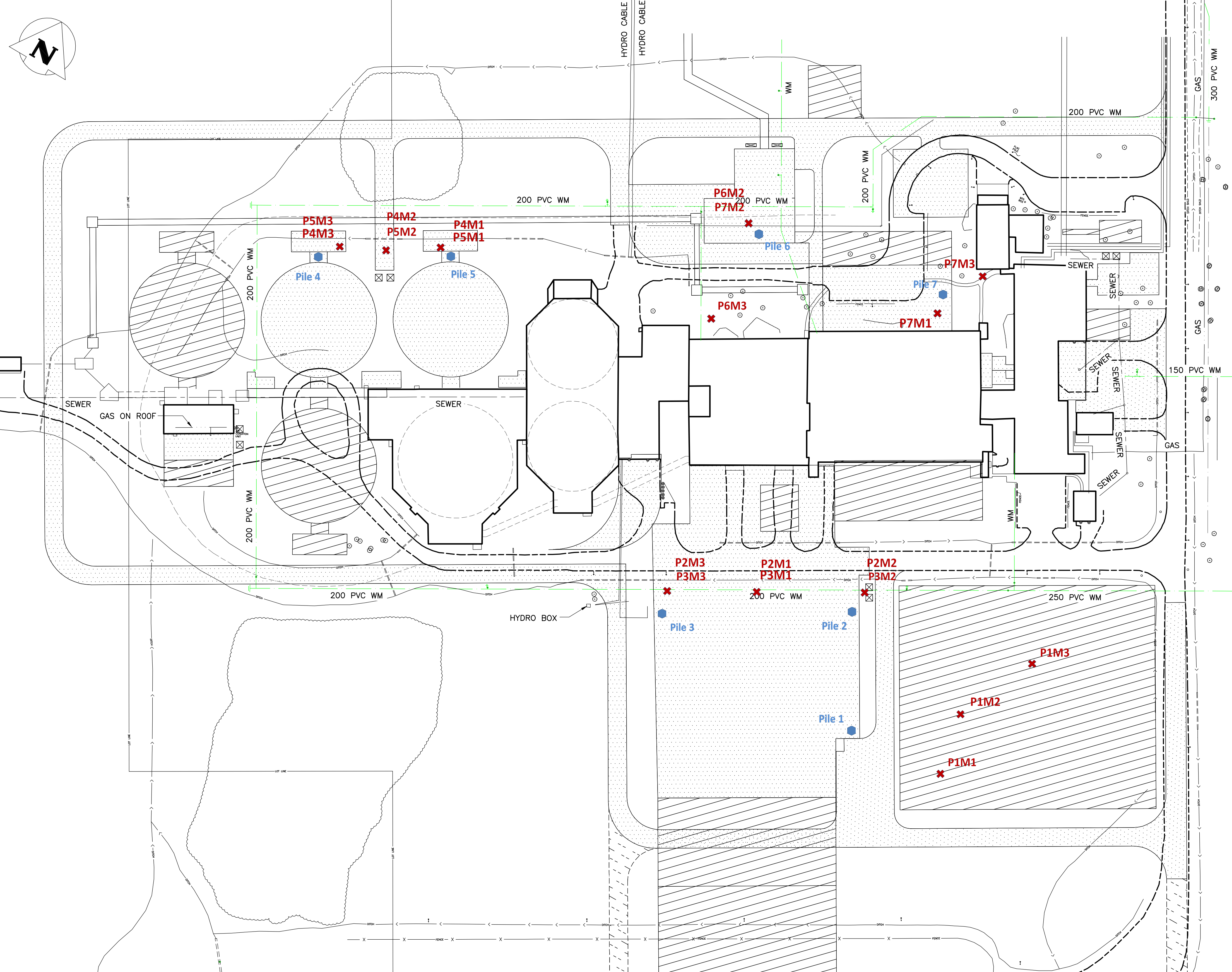

Figure 1: Test Pile & Monitor Locations

Figure 2: Variation in Ground Vibration with Distance