APPENDIX 9:

SOUTH WINNIPEG RECREATION CAMPUS

SWRC GEOTECHNICAL REPORT

JAN-2023

CITY OF WINNIPEG

South Winnipeg Recreation Campus – Geotechnical Engineering Report

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Revision:
Final

KGS Group Project:
23-0107-005

Date:
July 21, 2023

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STATEMENT OF LIMITATIONS AND CONDITIONS

Limitations

This report has been prepared for the City of Winnipeg in accordance with the agreement between KGS Group and the City of Winnipeg (the “Agreement”). This report represents KGS Group’s professional judgment and exercising due care consistent with the preparation of similar reports. The information, data, recommendations and conclusions in this report are subject to the constraints and limitations in the Agreement and the qualifications in this report. This report must be read as a whole, and sections or parts should not be read out of context.

This report is based on information made available to KGS Group by the City of Winnipeg. Unless stated otherwise, KGS Group has not verified the accuracy, completeness or validity of such information, makes no representation regarding its accuracy and hereby disclaims any liability in connection therewith. KGS Group shall not be responsible for conditions/issues it was not authorized or able to investigate or which were beyond the scope of its work. The information and conclusions provided in this report apply only as they existed at the time of KGS Group’s work.

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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 field and laboratory investigations, combined with an interpolation of soil and groundwater conditions found at and within the depth of the test holes drilled by KGS Group at the site at the time of drilling. 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, KGS Group should be notified in order that the recommendations can be reviewed and modified if necessary.

1.0 INTRODUCTION

The firm of Kontzamanis Graumann Smith MacMillan Inc. (KGS Group) was retained by the City of Winnipeg to provide geotechnical services for the proposed South Winnipeg Recreation Campus (SWRC) to be located within the community of Waverley West. The project site is located directly west of the existing Pembina Trails Collegiate campus, south of Cadboro Road, however, it is understood that Cadboro Road will be replaced by the future extension of Bison Drive. The project site is approximately 9.2 hectares (22.8 acres) in size. This report summarizes the results of the field and laboratory investigations and preliminary geotechnical design recommendations as part of the Phase 1 Geotechnical Investigation.

KGS Group's scope of services for this project were outlined in our proposal No. 22-000-1929 "South Winnipeg Recreation Campus – Winnipeg, Manitoba, Proposal for Geotechnical Services" dated January 4, 2023.

The main components of the geotechnical scope of services included:

- A subsurface investigation consisting of four (4) deep test holes advanced to depths ranging from approximately 16.6 to 17.2 below ground surface (BGS), and two (2) shallow test holes drilled to a depth of approximately 6.1 m (20 ft);
- A laboratory testing program that included moisture contents, flexible wall hydraulic conductivity test, grain size analysis, and Atterberg limits;
- A comprehensive review and analysis of the findings obtained from the site investigation and laboratory testing; and
- A geotechnical report outlining the field and laboratory test results, foundation recommendations for the proposed recreation center, as well as other geotechnical design recommendations.

2.0 PROJECT UNDERSTANDING

The project site is located within a currently undeveloped portion of the community of Waverley West in Winnipeg, Manitoba, and directly west of the existing Pembina Trails Collegiate campus. The project site is located south of Cadboro Road; however, it is understood that Cadboro Road will be replaced by the future extension of Bison Drive. The project site is approximately 9.2 hectares (22.8 acres) in size.

It is understood that the proposed recreational campus will include an aquatic centre, multiple gymnasiums, a variety of programming spaces, multi-purpose rooms, indoor walking track, fitness space, a community library, and outdoor spray pad, adjacent park space, athletic fields, with potential future development of a twin arena. It is also understood that this site will also include a new fire hall.

Phase 1 of the project will focus on development of three (3) gymnasiums with mezzanine walking/running track and fitness areas, several multi-purpose rooms, change rooms, offices, washrooms, kitchen, lobby, potential tenant leasing spaces, parking areas, outdoor spray pad, park amenities and pathways, public art components, and attached daycare facility. The phase one building will likely be two stories in height, will have limited basement/crawl spaces and is anticipated to have column load in the range of 1,800 to 2,300 kN.

As per the Request for Proposal (RFP), the geotechnical investigation for this project will be split into two parts, a general site investigation and a detailed site investigation, as described below:

Phase 1 General Investigation – A general site investigation will be completed first, and at this stage of the project the location of the proposed recreational facility on the property has yet to be determined. The purpose of the geotechnical investigation is to provide preliminary geotechnical design recommendations as requested in the RFP for Phase 1 General Investigation.

Phase 2 Detailed Investigation – A detailed geotechnical investigation will be completed following development of the project site plan and will be focused within the footprint of the proposed building. The purposed detailed geotechnical investigation will be a supplement investigation to confirm or enhance the preliminary design recommendations of the Phase 1 General Investigation for Phase 2 detailed design.

This report summarizes the work completed as part of the Phase 1 General Investigation.

3.0 INVESTIGATION PROGRAM

A geotechnical investigation program was conducted by KGS Group as part of the Phase 1 General Site Investigation to determine the subsurface soil profile including soil type, moisture, color, and density/consistency; in-situ strength properties of soil; and groundwater conditions.

3.1 Regional Geology

The geology in Winnipeg in general terms, consists of glaciolacustrine clay deposits over till, consistent with the findings of the materials encountered at this project site. Additional information on the reginal geology can be found in the Geological Engineering Report for Urban Development of Winnipeg by the University of Manitoba.

3.2 Test Hole Drilling and Sampling

KGS Group obtained clearance from public utility companies for underground buried utilities at the site prior to conducting the geotechnical drilling investigation. A drilling and sampling program was completed between April 11 and 14, 2023 under continuous supervision of KGS Group, with drilling services provided by Paddock Drilling Ltd. of Winnipeg, Manitoba. Drilling was performed using a Mobile B57 track mounted drill rig equipped with 125 mm diameter solid stem auger, 250 mm diameter hollow stem auges, and an automatic Standard Penetration Test (SPT) hammer.

As part of the Phase 1 General Site Investigation a total of 6 test holes were advanced during the drilling and sampling program including, four (4) deep test holes advanced to depths ranging 16.6 to 17.2 m (54.5 to 56 ft) BGS near the corners of the project site, and two (2) shallow test holes were drilled to a depth of 6.1 m (20 ft) BGS near the center of the project site. Four (4) deep standpipe piezometers were installed within test holes TH23-01, TH23-03, TH23-04 and TH23-06, with the screened portion of the piezometer within the underlying silt till materials. Two secondary test holes were drilled directly adjacent to test holes TH23-05 and TH23-06 to facilitate installation of shallow standpipe piezometers screened within the upper silty clay materials, these test holes are denoted as TH23-05-PZ and TH23-06-PZ.

Representative soil samples were obtained from each test hole at 0.75 to 1.5 m (2.5 to 5 ft) intervals or at any change in soil strata. Soil samples were collected directly off the auger, or split spoon samplers, and visually classified in the field in general accordance with the modified Unified Soil Classification System (USCS). The SPT's were advanced at 1.5 m (5 ft) intervals in granular material, while clay samples were tested with a pocket Torvane to estimate the undrained shear strengths. Representative undisturbed Shelby tube samples were collected from the clay layer. Select samples were subsequently submitted for testing at a Canadian Council of Independent Laboratories (CCiL) certified soil testing laboratory in Winnipeg, Manitoba. Tests were completed in general accordance with American Society for Testing and Materials (ASTM) standards.

Test hole UTM coordinates and elevations were collected with GPS survey equipment in reference to the CGVD28 datum, and are shown in Figure 1 and listed in Table 1 below, respectively. The deep and shallow test hole locations are shown on Figure 1 in yellow and green, respectively. Figure 2 shows cross section A-A' which provides detailed cross-section of the subsurface conditions at the site. Detailed descriptions of the soil conditions with laboratory results encountered across the site can be found on the summary log reports attached in Appendix A. The results of the laboratory testing are attached in Appendix B.

FIGURE 1: TEST HOLE LOCATION PLAN

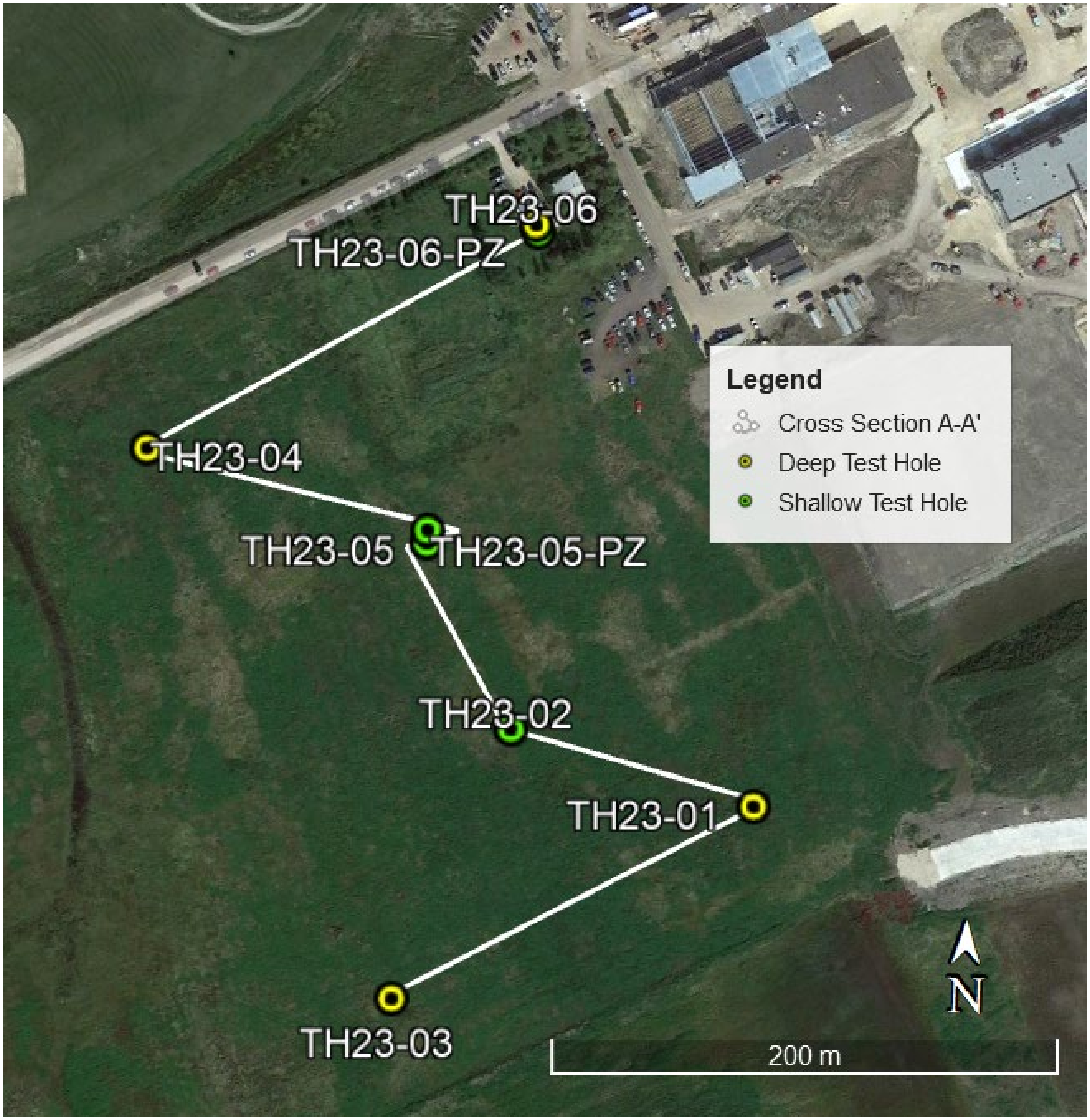


TABLE 1: SUMMARY OF TEST HOLE LOCATIONS AND DEPTHS

Test Hole ID	Approximate UTM Coordinates[1]		Test Hole Ground Elevation (m)	Test Hole Depth (m)
	Northing (m)	Easting (m)		
TH23-01	5,517,488.85	630,644.05	232.61	16.9
TH23-02	5,517,512.00	630,543.00	232.69	6.1
TH23-03	5,517,405.86	630,514.07	232.83	16.8
TH23-04	5,517,612.58	630,391.72	232.85	17.2
TH23-05	5,517,581.79	630,512.25	232.64	6.1
TH23-05-PZ	5,517,578.79	630,514.25	232.64	0.9
TH23-06	5,517,703.35	630,533.45	233.13	16.6
TH23-06-PZ	5,517,701.00	630,534.08	233.11	0.9

Note: Test hole elevation for test hole TH23-02 interpolated from collected survey information.

4.0 STRATIGRAPHY AND GROUNDWATER

4.1 Stratigraphy

In general, the soil stratigraphy across the subject site consisted of a surficial layer of silty clay overlying a deep deposit of high plastic clay, overlying silt till. A detailed description of the soil stratigraphy encountered at the project site is provided below and summary test hole logs are provided in Appendix A, and laboratory testing results are provided in Appendix B.

Clay – A layer of clay was encountered at the surface of all test holes except for TH23-03 and TH23-05. The clay extended to depths ranging from 12.2 to 13.0 m (40 to 42.5 ft) below ground surface (BGS). The clay was dark brown to grey in colour, was damp to moist, and contained trace to and silt, trace sand fine to coarse grained sand, trace fine to coarse gravel, trace gypsum, and trace to moderate oxidation. The clay was frozen to depth ranging from 0.9 to 1.2 m BGS.

The undrained shear strength of the clay was assessed with a pocket Torvane and ranged from 20 to 86 kPa classifying the clay as soft to stiff in consistency, decreasing with depth. Two (2) Atterberg limits tests were completed on samples of the clay indicating a liquid limit of 76 to 102, a plastic limit of 26 to 32, and a plasticity index of 50 to 70, classifying the clay as of high plasticity. One (1) particle size analysis test was completed on a sample of the clay indicating 0% gravel, 1% sand, 41% silt and 58% clay. Two flexible wall hydraulic conductivity tests were conducted, and their results were presented in Section 4.3.2. Twenty-nine moisture content tests were completed on the clay samples indicating moisture contents of 23 to 59%.

Silty Clay(CL) – Silty clay was encountered within all test holes, and was encountered at the surface of test holes TH23-03 and TH23-05, and between depths of 0.45 and 1.8 m BGS in all remaining test holes. The silt clay was frozen to depths ranging from 0.8 to 0.9 m. The silty clay was light brown to brown in colour, dry to moist, soft to stiff, and contained trace to some sand, and was with silt inclusions in test hole TH23-02, and some gravel in TH23-06.

One (1) Atterberg limits test was completed on a sample of the silty clay indicating liquid limit of 26, a plastic limit of 18 and a plasticity index of 8, classifying the silty clay as of low plasticity. One (1) particle size analysis test was completed on a sample of the silty clay indicating 0% gravel, 6% sand, 75% silt, and 19% clay. Five (5) moisture content tests were completed on the silty clay samples indicating moisture contents of 19 to 42%.

Silt Till – was encountered underlying the clay layer within the deep test holes (TH23-01, TH23-03, TH23-04, and TH23-06) at depths ranging from 12.2 to 13.0 m (40 to 42.5 ft) BGS, and extended to the depths explored within these test holes. The silt till was light grey in colour, dry to moist, and contained trace to some fine to medium grained sand, some fine to coarse grained gravel, and trace to with clay.

Standard penetration tests were completed within the silt till indicating uncorrected N-values of 5 blows per 300 mm of penetration to 50 blows per 80 mm of penetration, classifying the silt till as loose to very dense in terms of relative density. One (1) Atterberg limits test was completed on a sample of the silt till indicating a liquid limit of 26, a plastic limit of 13, and a plasticity index of 13. One (1) particle size analysis was completed on a sample of the silt till indicating 7% gravel, 28% sand, 43% silt, and 22% clay. Seven (7) moisture content tests were completed on the silt till samples indicating moisture contents of 9 to 33%.

A geotechnical investigation was also completed by KGS Group at the adjacent school site in 2018, in which the lower clay deposit was classified as silty clay on the 2018 test hole logs. Laboratory testing completed in 2018 consisting of two (2) particle size analysis test and two (2) Atterberg limits indicates this material is in fact a high plasticity fact clay. Meaning that the silty clay layer presented on the 2018 test hole logs was miss classified and is actually a high plasticity clay similar to what was encountered during the 2023 investigation at the proposed South Winnipeg Recreation Campus site.

A secondary supplementary geotechnical investigation was completed at the adjacent school site by KGS Group in 2020. The 2023 test hole logs are in general agreement with the 2020 test hole logs. However, during the 2020 investigation, a layer of silt was found near the surface at the school site, while a layer of silty clay was found near the surface at the recreation campus site. Only moisture content testing was completed on samples of the silt from the 2020 investigation, while moisture content, Atterberg limits, and particle size analysis was completed on a sample of the silty clay during the 2023 investigation. Based on the completed testing the 2023 soil classification of silty clay is considered more accurate.

4.2 Seepage, Sloughing and Groundwater Conditions

Groundwater and sloughing observations were recorded during and upon completion of drilling for each test hole. The observed groundwater seepage and soil sloughing conditions for each test hole are summarized in Table 2 below.

TABLE 2: OBSERVED GROUNDWATER SEEPAGE AND SLOUGHING CONDITIONS

Test Hole ID	Depth of Test Hole (m)	Seepage Layer	Observed Water Seepage Depth/Elevation During Drilling (m)	Depth/Elevation of Water Upon Completion of Drilling (m)	Sloughing Observations Upon Completion of Drilling
TH23-01	16.9	Clay	7.6 / 225.01	4.6 / 228.01	Test hole remained open upon completion of drilling.
TH23-02	6.1	NA	None/Dry	None/Dry	Test hole remained open upon completion of drilling.
TH23-03	16.8	Silt Till	12.8 / 220.03	6.7 / 226.13	Test hole remained open upon completion of drilling.
TH23-04	17.2	NA	None/Dry	None/Dry	Test hole remained open upon completion of drilling.
TH23-05	6.1	NA	None/Dry	None/Dry	Test hole remained open upon completion of drilling.
TH23-05-PZ	0.9	NA	None/Dry	None/Dry	Test hole remained open upon completion of drilling.

TH23-06	16.6	Clay	10.7	None/Dry	Test hole remained open upon completion of drilling.
TH23-06-PZ	0.9	NA	None/Dry	None/Dry	Test hole remained open upon completion of drilling.

In general, groundwater seepage was not observed during drilling and the majority of test holes remained dry upon completion of drilling. However, groundwater seepage was observed from the lower clay materials in test hole TH23-01 and TH23-06, groundwater seepage was also observed from with silt till in test hole TH23-03.

Standpipe piezometers were installed in test holes TH23-01, TH23-03, TH23-04 and TH23-06 such that the screened portion of the piezometer was embedded into the underling silt till, standpipe piezometers were also installed within test holes TH23-05-PZ and TH23-06-PZ such that the screened portion of the piezometer was embedded into the upper silty clay materials. Table 3 below summarizes the groundwater levels recorded in the installed standpipes as recorded between May 12 and 16, 2023, approximately 4 weeks following completion of drilling and are representative of the stabilized groundwater level. Based on the piezometer readings resented below the groundwater elevation within the silt till layer ranges from approximately 227.18 to 229.69 m, and the groundwater elevation within the silty clay ranges from approximately 232.00 to 232.46 m.

TABLE 3: SUMMARY OF RECORDED GROUNDWATER DEPTHS

Test Hole ID	Screened Material Type	Screen Depth Interval (m)	Reading Date	Recorded Depth to Groundwater (m)	Recorded Groundwater Elevation
TH23-01	Silt Till	13.7 – 16.8	May 16, 2032	5.25	227.36
TH23-03	Silt Till	13.7 – 16.8	May 15, 2023	5.49	227.34
TH23-04	Silt Till	15.2 – 16.8	May 15, 2023	5.67	227.18
TH23-05- PZ	Silty Clay	0.3 – 0.9	May 16, 2023	0.64	232.00
TH23-06	Silt Till	13.4 – 16.5	May 12, 2023	3.44	229.69
TH23-06-PZ	Silty Clay	0.3 – 0.9	May 12, 2012	0.65	232.46

Groundwater levels will fluctuate seasonally and following precipitation events; as such, the actual groundwater level at the time of construction could differ from the conditions observed on-site. It is recommended that the groundwater level within the installed standpipes be confirmed prior to construction to allow the contractor to quantify groundwater conditions expected at the project site during construction.

Standpipe piezometers were installed within the test holes for this stage of the project to facilitate rising head hydraulic conductivity testing. In order to gain a sense of the porewater pressures groundwater flow within the different material types and the determine the presence of upward or downward hydraulic gradients its is recommended that the vibrating wire piezometers be installed during the Phase 2 Detailed Investigation and in close proximity to the proposed building location which is unknown at this time.

4.3 Hydraulic Conductivity Estimation

An estimation of the hydraulic conductivity of each of the major soil types (silty clay, clay, silt till) encountered across the project site is provided in the sub sections below. Hydraulic conductivity estimates for the silty clay and silt till materials were provided via in field rising head conductivity testing while hydraulic conductivity for clay materials was determined by laboratory testing.

4.3.1 RISING HEAD TESTING

Rising head tests to estimate the hydraulic conductivity, k, of the soils were performed by KGS Group staff between May 10 and May 18, 2023 in accordance with ASTM D4044/D4044M-15. KGS Group staff installed water level transducers into the bottom of each well to measure water levels within the well during each test. Manual water level measurements were recorded with a water level meter to confirm the accuracy of the data collected from the transducers.

Two of the six (2 of 6) wells were shallow wells (0.9 m below ground surface) screened through an upper silty clay layer. The remaining wells were deeper (16.4 to 16.8 m below ground surface) screened through a silt till layer. Each rising head test involved the following sequence of steps:

- The static water level and depth to well bottom were measured using the water level meter.
- The transducer was installed near the bottom of the well and set to record at a 30 second interval.
- The well was purged until no more water was produced, using plastic tubing and a foot valve (deep wells) or using bailers (shallow wells).
- The plastic tubing or bailer was removed from the well and manual water level readings were recorded every 30 seconds until 10 minutes had elapsed, at which point readings were taken every minute, extending to every 5 minutes and then 10 minutes as elapsed time approached 1 hour.
- The final water level was measured the following day using the water level meter and the transducer was removed from the well to download the data.

The hydraulic conductivity of the soils was calculated from the test data by using the software AQTESOLV. The software plots the transducer data as drawdown versus time beginning from the greatest drawdown (lowest water level in the well) at the end of purging. Well and aquifer geometries were input for each test based on the test-well borehole logs (provided in Appendix A). The ratio of vertical to horizontal conductivity (K_z/K_r) was assumed to be 0.1 for the soil layers. Slight sedimentation of the screened portion of the wells was assumed negligible for this analysis.

Within the software, the Bouwer-Rice (1976) solution technique for slug tests on unconfined aquifers was applied to the plotted data. For the deeper wells, the data allowed for a manual linear approximation of best-fit over the software's recommended head range (see horizontal band in AQTESOLV Reports provided in Appendix C). For the shallower wells, where data was not within the recommended head range, the manual linear approximation was applied to the initial portion of the test for a conservative estimate. Although for the deeper wells the silt till layer is confined by the overlying clay layer, the K results of analysis using the confined aquifer method were comparable to the unconfined K and therefore the more user-friendly unconfined method was used. Well and aquifer geometries, plotted data, and estimated K values are included in the AQTESOLV Reports provided in Appendix C.

4.3.1.1 Silty Clay (CL)

The calculated hydraulic conductivity, k, for the silty clay (CL) layer as determined by rising head testing as described above are summarized on Table 4.

TABLE 4: ESTIMATION OF SILTY CLAY HYDRAULIC CONDUCTIVITY

Test Hole ID	Date of Testing	Material Type	Hydraulic Conductivity, k (cm/sec)
TH23-05-PZ	May 16, 2023	Silty Clay	9.5×10^{-6}
TH23-06-PZ	May 12, 2023	Silty Clay	1.6×10^{-5}

Based on the completed testing, the hydraulic conductivity, k, of the silty clay (CL) can be estimated as 1.0×10^{-5} cm/sec.

4.3.1.2 Silt Till

The calculated hydraulic conductivity, k, for the silt till layer as determined by rising head testing as described above are summarized on Table 5.

TABLE 5: ESTIMATION OF SILT TILL HYDRAULIC CONDUCTIVITY

Test Hole ID	Date of Testing	Material Type	Hydraulic Conductivity, k (cm/sec)
TH23-01	May 16, 2023	Silt Till	$1.2x10^{-4}$
TH23-03	May 15, 2023	Silt Till	$2.6x10^{-4}$
TH23-04	May 15, 2023	Silt Till	$1.2x10^{-4}$
TH23-06	May 12, 2023	Silt Till	$4.4x10^{-6}$

Based on the completed testing, the hydraulic conductivity, k, of the silt till can be estimated as $1x10^{-4}$ cm/sec.

4.3.2 LABORATORY TESTING

During the drilling program undisturbed samples of the clay were collected using 76 mm diameter thin wall Shelby tubes. Shelby tube samples were extruded in the laboratory then tested for hydraulic conductivity using a flexible wall permeameter and in accordance with ASTM D5084. The flexible wall hydraulic conductivity test lab reports are provided within Appendix B.

4.3.2.1 Clay

Results of the laboratory hydraulic conductivity, k, testing are summarized in Table 6.

TABLE 6: SUMMARY OF HYDRAULIC CONDUCTIVITY LAB TESTING

Test Hole ID	Sample Depth (m)	Material Type	Hydraulic Conductivity, k_{20} (cm/sec)
TH23-01	7.6	Clay	$1.5x10^{-8}$
TH23-05	3.0	Clay	$6.3x10^{-9}$

Based on the completed testing, the hydraulic conductivity, k, of the clay can be estimated at $1x10^{-8}$ cm/sec.

4.3.3 SEEPAGE VOLUME ESTIMATION

To calculate the potential volume of seepage within excavations, multiply the side wall area of the excavation by the hydraulic conductivity, k, for the soil type present and by the time the excavation will be open. If multiple soil layers are present within the excavation, seepage volumes for each layer should be calculated, then summated together.

$$\text{Volume (cm}^3\text{) = k (cm/sec) x Excavation Sidewall Area (cm}^2\text{) x Time (sec)}$$

It is understood that a crawl space approximately 83 x 83 m in plan area and 3 m in depth will be present beneath the proposed recreation campus. The soil profile as presented in test hole TH23-06 with a groundwater level at surface would be the worst-case scenario for estimation of groundwater seepage as it has the greatest thickens of the silty clay materials. Based on these assumptions it is estimated that 3 m^3 of groundwater will enter the crawlspace excavation daily. This estimation may be influenced by rainfall events or other season changes in groundwater depth.

4.4 Potential Difficult Ground Conditions

Although not encountered during our investigation, cobbles / boulders are known to exist within till deposits. Cobbles and / or boulders can cause difficulties when advancing bored caissons into till materials. If encountered, the contractor will be required to remove cobbles and / or boulders from within foundation excavations or drilled pile shafts. Cobbles and / or boulders and may also cause driven piles to refuse early before achieving design embedment depth.

Silty clay was encountered across the project site between depths of 0 to 1.8 m, this silty clay (CL) is not considered suitable for the support of foundations, parking lots, roadways or sidewalks. If soft materials such as organics, silt, silty clay, or saturated clays are encountered at the prepared subgrade elevation these materials should be removed and replaced with compacted granular fill. Recommendation on how to remove and replace soft material beneath mat slabs, soil supported floor slabs, and pavement sections are provided in Sections 5.1, 6.1, 7.8 respectively.

5.0 FOUNDATION RECOMMENDATIONS

KGS Group recommends that the proposed recreation campus be founded on deep foundation system comprised of cast-in-place concrete friction or end bearing piles, driven steel H-Piles, or driven prestressed precast concrete piles. Shallow footings are not recommended as a foundation option for the recreation campus due to potential for differential movements resulting from expansive clay soils.

5.1 Limit States Design

The foundation considerations described in this report follow Limit State Design (LSD) guidelines. Limit State Design requires consideration of two (2) main loading states: Ultimate Limit State (ULS) and Serviceability Limit State (SLS). The ULS are primarily concerned with collapse mechanisms of the structure and safety, and the SLS present conditions or mechanisms that restrict or constrain the intended use, function, or occupancy of the structure under expected service or working loads.

For pile foundation design, LSD prescribes Geotechnical Resistance Factors (Φ) that are based upon the method used to evaluate pile capacity to obtain the factored ULS pile capacity values.

The estimated unfactored ULS values provided below represent the nominal (ultimate) geotechnical resistance, R_n. A geotechnical resistance factor (Φ) should be applied to determine the factored geotechnical resistance as presented in the following equation:

$$\Phi R_n \geq \Sigma \alpha_i S_{ni}$$

where:

Φ – geotechnical resistance factor
R_n – nominal (ultimate) geotechnical resistance
$\Sigma \alpha_i S_{ni}$ – summation of the factored overall load effects for a given load combination

Unfactored ULS values shown in the tables below should be multiplied by the appropriate resistance factor to determine the factored geotechnical resistance.

5.2 Cast-In-Place Concrete Friction Piles

Cast-in-place concrete friction piles could be considered a viable option to support the proposed recreation campus. Cast-in-place concrete friction piles should be designed such that they are supported by skin friction with no contribution from end-bearing. Estimates of the ULS and SLS values for skin friction are listed in Table 7 below. The average undrained shear strength within the upper clay materials was approximately 50 kPa based on field torvane measurements. Based on the α-method, and adhesion coefficient (α) of 0.7 was selected for the upper clay layer to estimate shaft skin friction.

TABLE 7: CAST-IN-PLACE CONCRETE PILE SKIN FRICTION DESIGN VALUES

	Approximate Depth Below Grade (m)	Stratigraphic Layer	Serviceability Limit State, SLS (kPa)	Unfactored Ultimate Limit State, ULS (kPa)
Unit Shaft Resistance	0 to 2.5	Clay/Silty Clay	–	-
	≥2.5 to 8.0	Clay	12	35
	≥8.0 to 12.0	Clay	8	25

Notes:

1. Shaft resistance for piles exposed to freezing should neglect the upper 2.5 m of soil due to frost.
2. A Resistance Factor of 0.4 should be applied to the unfactored ULS skin friction values provided in Table 7.
3. The SLS values provided in Table 7 limit settlement to 25 mm.

It is recommended that the maximum pile length be limited to 12 m to prevent sloughing and groundwater seepage issues when advancing drilled shafts through the underlying silt till materials. Cast-in-place concrete friction piles should be at least 8.0 m long to protect against frost jacking.

Based on the observed groundwater seepage and soil sloughing conditions during the drilling investigation shallow groundwater seepage and soil sloughing of the upper silty clay and lower silt till should be expected. It is anticipated that the Contractor will need to provide pumps and temporary full shaft length steel sleeving control groundwater seepage and to maintain the integrity of the drilled shaft excavation. The Contractor may also need to place concrete beneath the groundwater level using tremie or pumping methods. Concrete should be poured immediately upon completion of drilling and inspection. Drilling and concrete placement for the piles should be inspected by experienced geotechnical personnel to verify the soil and encountered conditions are consistent with the findings of this investigation.

5.3 Cast-In-Place End Bearing Piles

Cast-in-place straight shaft concrete friction piles installed into the dense silt till at a minimum depth of 15 m may be considered to support the recreation campus proposed for this project site. A geotechnical resistance factor (Φ) of 0.4 should be applied to the unfactored ULS values listed in Table 8 below. Cast-in-place end bearing piles should have a minimum 900 mm to facilitate removal of cobbles and/or boulders that may be encountered within sandy silt with clay.

TABLE 8: CAST-IN-PLACE CONCRETE PILE END BEARING DESIGN VALUES

Material Type	Serviceability Limit State, SLS (kPa)	Unfactored ULS Capacity (kPa)
End Bearing on Dense Silt Till at a minimum depth of 15 m	500	800

To obtain a sound bearing surface, the base of the cast-in-place piles must be mechanically cleaned, the excavation must be dewatered and all loose or deleterious materials removed from the base of the pile shaft prior to placement of concrete.

5.3.1 ADDITIONAL CAST-IN-PLACE PILE RECOMMENDATIONS

Additional cast-in-place pile recommendations are provided below:

- The spacing between adjacent piles should be a minimum of three-pile diameters center to center. If closer pile spacings are required, KGS Group can review the specific configuration and whether a reduction in capacity is required.
- Due to potential for squeezing of the clay soils, and potential shallow groundwater seepage from the silty clay layer, temporary full shaft length steel sleeves should be used as required during installation of CIP piles to limit groundwater seepage into drilled shafts and to maintain the integrity of the shaft excavation.
- Where seepage is encountered, which cannot be controlled by sleeves, remove water from the pile excavations prior to pouring concrete, or place concrete by tremie methods as required. At all times during removal of the steel sleeve, a head of concrete shall be maintained sufficiently above the sleeve bottom to limit sloughing and seepage as the sleeve is withdrawn.
- To resist tensile forces from frost action acting on piles (frost jacking), all concrete piles shall be reinforced their entire length and be designed by an experienced structural engineer and have a minimum embedment length of 8 m.
- The reinforcement and concrete must be placed immediately following the drilling and inspection of each pile to prevent disturbance to the foundation soil during subsequent construction activity. Where this is not possible on the day of drilling, the pile hole should be refilled, and later redrilled once concrete is ready to place.
- A minimum 150 mm void form should be used below all grade beams and pile caps to protect against potential uplift from frost heave.
- All concrete piles should utilize CSA Type HS sulphate resistant cement.
- If pile foundations are used for unheated structures, a poly wrapped and greased Sonotube should be placed around the upper 2.5 m of the pile shaft to protect against frost jacking.
- Sono tubes should be used to maintain the top of pile at the surface and piles should not be allowed to mushroom.
- Detailed construction records and full-time inspection by experienced geotechnical personnel is recommended throughout construction of foundations to verify the soil and encountered conditions are consistent with the findings of this investigation, and that piles are installed according to the project specifications and meet the intent of the geotechnical design.

5.4 Driven Piles

5.4.1 DRIVEN STEEL H-PILES

Driven steel H-pile capacities were developed based on the soil profile strength parameters and static analysis of the pile section in the GRL Wave Equation Analysis of Piles (GRLWEAP) program. Design capacities are listed for steel H-piles driven to refusal in silt till. Pile lengths may vary as pockets of cobbles and boulders are known to exist in till which could result in variable pile refusal depths.

Driven steel H-piles may be used to support the proposed recreation center. The LSD capacities for driven steel piles advanced to a minimum embedment depth of 16.5 m below grade and embedded into the underling silt till are presented in the table below and consider contributions from both shaft friction and end bearing. The H-pile sections listed are typical for this application. A geotechnical resistance factor (Φ) of 0.4 should be applied to the unfactored ULS values provided in Table 9. KGS Group should be contacted to determine the final refusal criteria once required pile capacities, piles size, and hammer energy have been determined. The SLS resistance values shown in Table 9 table are anticipated to limit foundation settlement to 25 mm. Capacities for additional pile sections can be provided upon request.

TABLE 9: DRIVEN STEEL H-PILE CAPACITIES

Pile Type / Section	Minimum Pile Embedment Length (m)	SLS (kN)	Unfactored ULS (kN)
HP250x85	16.5 m	350	1,100
HP310x79	16.5 m	420	1,300

Based on the geotechnical investigation power auger refusal was encountered at a depth of 16.6 m in test hole TH23-06, but was not encountered in the other test holes, and SPT refusal (SPT N-values over 100) was encountered at depths ranging from 13.7 to 16.5 m. It is anticipated that additional pile capacity could be obtained by driving the piles to refusal, however, it is recommended that additional test hole drilling be completed to determine the depth to power auger refusal across the project site within the footprint of proposed recreation campus. Additional test hole drilling should be completed during the Phase 2 Detailed Investigation.

Pre-boring approximately 3 m below grade should allow for standing of the piles, enhance pile plumbness/alignment, reduce potential ground heave in large pile groups and reduce vibrations induced on the adjacent buildings. Vibration monitoring of the adjacent buildings should be conducted during pile driving. Additional pre-boring will be required if monitored vibrations are above acceptable levels, as determined by the structural engineer.

It is recommended that steel piles be driven with pile shoes to protect the pile from damage from cobbles and boulders when driving into the silt till layer. Driving stresses should not exceed 90% of the nominal yield stress of the steel (F_y). As a minimum, steel piles should meet the requirements of CAN/CSA-G40.20/G40.21, Grade 350 W. The piles should be free from protrusions which could create voids in the soil around the pile during driving.

5.4.2 DRIVEN PRESTRESSED PRECAST CONCRETE PILES

Driven precast concrete piles bearing on the underlying silt till may be used to support the proposed recreation campus building. Below are the estimated factored ULS and SLS pile capacities when driven to practical refusal using a diesel hammer with a rated energy of not less than 40 KJ. End of drive refusal criteria are listed in the Table 10 below for each pile section and should be achieved for three (3) consecutive increments. A geotechnical resistance factor (Φ) of 0.4 should be applied to the unfactored ULS values provided in Table 10.

TABLE 10: DRIVEN PRESTRESSED PRECAST CONCRETE PILE CAPACITIES

Pile Diameter (mm)	SLS Values (kN)	Unfactored ULS Values (kN)	Final Refusal Criteria (blows/25 mm)Note 1
300	445	1,400	5
350	625	1,960	8
400	800	2,400	12

Notes:

1) If higher energies or other types of hammers are used, they should be evaluated to ensure that piles are not overstressed and to determine an appropriate refusal criterion.

Pile penetration depths will depend on localized till conditions and the presence of cobbles and boulders. Based on the geotechnical investigation power auger refusal was encountered at a depth of 16.6 m in test hole TH23-06, but was not encountered in the other test holes, and SPT refusal was encountered at depths ranging from 13.7 to 16.5 m. It is recommended that additional test hole drilling be completed as part of the Phase 2 Detailed Investigation to determine the depth to power auger refusal across the project site.

Piles are typically cast in lengths of up to 22 m (72 ft); however, pile lengths exceeding 18 m (60 ft) require special handling in order to prevent cracking of the piles. The final selection of driven prestressed precast concrete pile lengths will be the responsibility of the piling contractor using the available test hole information provided in this report.

Pre-boring approximately 3 m below grade should allow for standing of the piles, enhance pile plumbness/alignment, reduce potential ground heave in large pile groups and reduce vibrations induced on the adjacent buildings. Vibration monitoring of the adjacent buildings should be completed during pile driving. Additional pre-boring will be required if monitored vibrations are above acceptable levels, as determined by the structural engineer.

If significant squeezing or sloughing of the bore hole occurs during pre-boring, the pre-boring depth may be reduced accordingly. Piles driven within five-pile diameters of each other should be monitored for heave. If heave occurs, these piles should be re-driven to refusal. Careful attention will be required during driving, especially as the pile tip approaches the dense silt till, to ensure practical refusal has been achieved with the proper hammer energy, while avoiding overdriving that would damage the pile.

It should be assumed by the designers that the tensile strength of these precast piles is minimal and they have little capacity to resist bending.

5.4.3 ADDITIONAL DRIVEN PILE RECOMMENDATIONS

Additional driven pile recommendations are provided below:

- The spacing between adjacent piles should be a minimum of three-pile diameters center to center. If closer pile spacings are required, KGS Group can review the specific configuration and whether a reduction in capacity is required.
- All piles driven within five (5) pile diameters of one another should be monitored for heave and where heave is observed the piles should be re-driven to confirm the specified refusal criteria.
- The noted unfactored ULS capacities pertain to geotechnical resistance only. The pile cross-sections must be designed to withstand the design loads, handling stresses and the driving forces during installation.
- Diesel hammers with a maximum energy ratings of 89 to 100 KJ (66 to 75 kip-ft) may be required to install the above pile sections to planned depth. Final drive criteria (set) can be provided once the driving hammer and the maximum energy requirement of a hydraulic hammer will be less.
- During the final set, piles should be driven continuously once driving is initiated to the required refusal criteria.
- A steel follower should not be used for driving of steel piles.
- The uplift capacity of driven pile can be taken as 2/3 of the vertical resistance of the pile, plus the weight of the pile.
- The impacts of vibration from pile driving and its potential effects on existing structures should be evaluated and monitored during driving. Appropriate mitigation measures should be put in place to avoid damage to existing adjacent structures.
- Piles that will be exposed to frost should have a minimum embedment length of 8 m to resist frost jacking.
- A minimum 150 mm void form should be used below all grade beams and pile caps to protect against potential uplift from frost heave.
- Detailed construction records and full-time inspection by experienced geotechnical personnel is recommended throughout construction of foundations to confirm that piles are installed according to the project specifications and meet the intent of the geotechnical design.

5.5 Subgrade Reaction for Laterally Loaded Piles

For preliminary design of laterally loaded piles, the soil response to horizontal loads can be modeled as a series of horizontal springs with uniform stiffness. The response of the soil to the horizontal loading can be estimated using an equivalent spring constant (lateral subgrade reaction modulus), K_s. The lateral subgrade reaction modulus K_s for the soil profile encountered at this project site is listed below in Table 11 below.

It is anticipated that most of the lateral resistance will typically be offered by the upper 5 to 10 m of soil, depending on the location of the applied horizontal load and relative stiffness of the pile and surrounding soil. Any void space around the piles should be filled with cement grout or lean mix concrete to ensure contact with the adjacent soil otherwise the depth with void should be excluded from the lateral capacity of the pile.

The recommended lateral subgrade reaction modulus assumes a linear response to lateral loading and is only applicable to piles subjected to static loads (no dynamic or cyclic loading) that result in small deflections (less than 1% pile diameter). The pile material itself is assumed to be within the elastic range.

A more rigorous lateral pile analysis should be completed during the Phase 2 – Detailed Investigation. The detailed analysis should incorporate the actual soil and section properties of the pile, applied loads, final lateral deflection criteria and a more realistic elastic-plastic model of soil response to loading should be completed to confirm lateral load capacities of the piles.

TABLE 11: LATERAL SUBGRADE REACTION MODULUS

Soil Type	Approximate Depth Range Below Grade (m)	Estimated K_s (kPa/m)
Clay, Firm to Stiff	0.0 to 3.0	20,000z/d
Clay, Firm	3.0 to 9.0	13,000z/d
Clay, Soft	9.0 to 13.0	8,000z/d

Note: z= depth (m), d = pile diameter (m)

It is important to note that the lateral subgrade reaction modulus assumes a linear response to lateral loading and therefore is only appropriate under the following conditions:

- Maximum pile deflection of less than 1% of the pile diameter;
- Static (non-cyclical) loading; and
- Pile material does not reach yield conditions.

If one or more of the above conditions are not met, a more rigorous analysis that includes non-linear behaviour of the piles and surrounding soil is required using programs such as LPILE. As part of detailed design, a lateral pile analysis that incorporates the material and section properties of the piles, final lateral deflection criteria and a more realistic elastic-plastic model of the soil response to loading should be completed.

5.1 Mat Foundations

If some movement can be tolerated lightly to moderately loaded structures may be founded on shallow mat foundation founded on granular fill, overlying the stiff high plasticity clay. Mat foundations will experience some differential movements associated with variations in moisture content and frost related movements. While some movements will likely occur over time, a uniformly prepared subgrade will reduce post-construction differential movement.

To limit frost related movements, mat foundations should be constructed on non-frost susceptible granular materials extending to the depth of frost penetration (2.5 m) below the final exterior surface. The non-frost susceptible fill should extend horizontally a minimum of 600 m beyond the concrete foundation slab edges and contain less than 5% fines (silt and clay) content. Rigid insulation should be installed to further reduce the potential for frost related movements below mat foundations support on granular fill. If some movement can be tolerated partial removal and replacement of 2/3 the depth of the frost zone (1.7 m) with non-frost susceptible granular fill can be considered. It is recommended that a layer of non-woven geotextile be placed over the prepared subgrade to prevent migration of granular backfill materials.

Due to potential for movement, it is recommended that utilities or sensitive mechanical connections to structures founded on mat foundations be flexible and can accommodate potential for movement.

Typically, the coefficient of subgrade reaction is required for a mat foundation design. The estimated vertical modulus of subgrade reaction, based on the findings of the geotechnical investigation and recommendations from the Canadian Foundation Engineering Manual, is presented in Table 12 below.

TABLE 12: VERTICAL SUBGRADE REACTION MODULUS

Supporting Soil Type	Vertical Modulus of Subgrade Reaction, K_{v1} (MPa/m)*
Clay - Stiff	10

Where, K_{v1} is for a 0.3 x 0.3 m (1 x 1 ft) plate modulus. For actual footing of width ‘b’ by length ‘m’ (with b and m in meters) and founded on granular soils, $k_{vb} = (k_{v1}/b)*((m+0.15)/1.5m)$. If the footing is to be installed below the groundwater table k_{vb} should be multiplied by 0.6.

The following support preparations should be completed for this option:

- Sub-excavated as required to the subgrade design elevation, recommended 1.7 to 2.5 m below final ground surface. The exposed native subgrade should be proof rolled with heavy wheeled equipment to detect soft areas. Soft areas should be over-excavated, a minimum of 600 mm and replaced with base or subbase materials compacted to a minimum of 98% of the Standard Proctor Maximum Dry Density (SPMDD) near optimum moisture.
- Excavation and replacement granular fill should extend horizontally a minimum of 0.6 m beyond the concrete mat slab edges.
- Once the bottom of excavation is exposed, the native subgrade surface should be examined and approved by qualified geotechnical personnel, prior to placing engineering granular fill. Bearing soils

that become frozen, dried, or softened should be removed and replaced with granular base course material compacted to 98% SPMDD.

- The contractor must make a conscious effort to protect the finished subgrade surface from getting wet and direct runoff away from the excavation.
- Following examination and approval of the exposed subgrade, a non-woven geotextile should be placed over the subgrade materials then backfilled with new granular materials. Cover geotextile with new granular material ensuring that the placement or handling of the granular material does not damage the geotextile.
- Damaged portions of the geotextile layers must be removed and replaced. Repair patches must provide a minimum of 450 mm overlap to the damaged area in all directions.
- Granular fill should be placed as soon as possible after the placement of the geotextile to prevent disturbance during subsequent construction activity.
- New granular fill materials should conform to City of Winnipeg Specification CW3110-R2 dated November 15, 2022 for Granular A Base Corse (base) or Granular A 50 mm (sub-base) and should be submitted to KGS Group for approval prior to importing to site. Granular fill should be placed in lifts and compacted to 98% Standard Proctor Maximum Dry Density (SPMDD) within 2% of optimum.
- To allow for proper compaction, new granular fill materials must be placed and compacted in a thawed and unfrozen state. Heating and hoarding of granular material stockpiles will be required if construction is to proceed under freezing conditions.
- The entire extent of the controlled granular fill should be capped at the surface with a minimum of 150 mm (6 in) of compacted clay and be sloped away at a minimum 2% grade to provide positive drainage away from the mat foundation and to reduce the potential for water to pond at the surface. Alternatively, capping materials could consist of concrete or asphalt.

KGS Group should be consulted if the proposed mat foundation is to be constructed above existing grade on site grade fill material, so appropriate recommendations and procedures can be provided for the placement of fill below mat foundations.

6.0 INTERIOR SLABS

6.1 Soil Supported Slabs

Interior floor slabs may consist of either a slab-on-grade or structural slab construction. Floor slabs constructed at grade are susceptible to movement due to the swelling of underlying clay-type soils. The magnitude of swell can vary considerably and is based on the natural moisture content and plasticity of the soil. Swell is typically higher for slabs constructed over areas where trees have recently been removed. A slab-on-grade should only be selected if some movements and cracking of the floor slab can be tolerated.

The estimated magnitude of displacement due to swelling of soils below the floor slabs constructed at grade ranges between 25 and 50 mm. If the potential for movement and cracking of the interior slab is unacceptable, the floor should be constructed as a structural slab. Under no circumstance should slab-on-grade construction take place during freezing conditions or on frozen ground or while frozen ground is thawing. The following is recommended for the support of a slab-on-grade interior floor:

- Remove surficial topsoil, fill layers, organics, silt, and soft or unsuitable materials encountered below slabs down to the high plastic clay. Proof rolling and compaction of the clay subgrade soil should be completed using a heavy roller under the supervision of an experienced geotechnical engineer to identify unsuitable or soft areas to achieve a minimum of 95% of the standard Proctor Maximum Dry Density (SPMDD). If unsuitable subgrade soils such as organics, fill, silt or soft clay are encountered, they should be sub-excavated and additional 600 mm and backfilled with compacted granular sub-base to 98% SPMDD.
- The contractor must protect the finished subgrade surface from excessive moisture during construction and promote runoff away from the subgrade.
- Site grading fill placed to raise the site to finished grade (i.e., fill placed above the existing ground surface) can consist of a sub-base granular fill if properly moisture conditioned to within 2% of optimum moisture content and compacted to 98% SPMDD.
- A minimum 150 mm of granular base over 300 mm of sub-base should be placed immediately below the floor slab. All granular fills should be placed in maximum 150 mm thick lifts and compacted to 98% SPMDD. A non-woven geotextile fabric should be placed as a separator between the native subgrade materials and compacted granular fill.
- The granular base course and sub-base course should be well-graded and be free of organics and frozen material supplied in accordance with appliable standard specifications, such as City of Winnipeg Specification CW3110-R2 dated November 15, 2022. Sieve analysis and compaction testing of the granular base and sub-base materials should be conducted by qualified geotechnical personnel to ensure that the materials supplied, and percent compactions are in accordance with design specifications.
- All mechanical services or piping that would be buried within the engineered fill should be designed to accommodate potential slab movement.

6.2 Structural Slabs

Where slab movements cannot be tolerated, structural floor slabs are recommended. Structural floor slabs constructed over a minimum 150 mm void space to reduce the potential of movement due to swelling or frost heave from the underlying soil.

7.0 ADDITIONAL DESIGN CONSIDERATIONS

7.1 Frost Penetration

The depth of frost penetration will vary depending on air temperature, ground cover, the type of any fill material used during development and other factors. The expected depth of frost penetration has been estimated assuming a design freezing index of 2300°C days, taken as the coldest winter over a 10-year period. The estimated maximum depth of frost penetration is 2.5 m assuming bare ground and no insulation cover. The clay soils can heave upon freezing and that consideration must be considered in the foundation design. Good site drainage must also be maintained after development.

Well-graded granular materials should be utilized as structural backfill material as they are less susceptible to the effects of frost heave than the high plastic clay encountered at the site.

Soil in contact with foundation elements can freeze to the foundations and develop adfreeze bonding, which can result in uplift forces. The 4th Edition of the Canadian Foundation Engineering Manual (CFEM 2006) recommends the following adfreeze bond stresses for soil and foundation materials:

- 65 kPa for fine grained soils frozen to wood or concrete;
- 100 kPa for fine grained soils frozen to steel; and
- 150 kPa for saturated gravel frozen to steel.

To calculate the frost uplift force, the adfreeze bond stress should be applied to the perimeter of the pile within the depth of frost. A resistance factor of 0.7 can be used with the unfactored ULS skin friction values on Table 5 to determine the frost uplift resistance of the pile.

The depth of burial (minimum 2.5 m) of water lines or other lines that cannot be allowed to freeze should consider local practice. Shallow lines can be protected using a heat trace or closed cell extruded polystyrene insulation. The amount and extent of insulation required will be dependent on several factors including the thermal regime around the pipe, the depth of burial, surface conditions, and fluid temperature, if present.

7.2 Backfilling

The on-site clay material is considered suitable for general backfilling. Free draining granular material should consist of less than 5% fines (passing the #200 sieve). Backfill materials should be compacted uniformly in maximum 150 mm lifts to at least 95% SPMDD. If granular soil is used as backfill behind below grade walls, the upper 1 m of fill should consist of compacted high plasticity clay, to promote surface runoff and reduce infiltration.

7.3 Lateral Earth Pressure Coefficients

For design purposes the soils may be assigned active, passive and at-rest lateral earth pressure coefficients as shown in Table 13 below.

TABLE 13: EARTH PRESSURE COEFFICIENTS

Material	Bulk Unit Weight (kN/m³)	ϕ'	K_a	K_p	K_o
Silty Clay (CL-ML)	18	20	0.490	2.040	0.658
Clay (CH)	18	17	0.548	1.826	0.708
Well Graded Compacted Granular Fill (GW)	18	35	0.271	3.690	0.426

Live loads or surface surcharge loads should be included if a significant loading is applied within a distance equal to the height of the below grade wall.

7.4 Seismic Site Classification

In accordance with the 2020 National Building Code of Canada (NBCC), Table 4.1.8.4.B, the site class, S, for seismic site designation, X_S, is based on the average properties of soil and rock in the top 30 m. Based on the results of the investigation and on publicly available data on nearby wells, the project site class may be considered to be F, therefore the site designation is X_F.

To obtain the seismic values for the design of buildings in Canada under Part 4 of the 2020 NBCC as prescribed in Article 1.1.3.1. of Division B, use the 2020 NBCC Seismic Hazard Tool available publicly online.

7.5 Cement Type for Foundation Concrete

It is recommended that concrete in contact with soil use a high sulphate-resistant cement (HS or HSb). A maximum water to cement ratio of 0.40 should be specified in accordance with Table 2, CSA A23.1-04 for concrete with very severe sulphate exposure (S1). Concrete which may be exposed to freezing and thawing should be adequately air entrained to improve freeze-thaw durability in accordance with Table 5, CSA A23.1-04.

7.6 Surface and Subsurface Drainage

A permanent subdrain system should be installed around the exterior of below grade walls. Subdrains should be constructed with a perforated weeping tile wrapped with a filter sock and backfilled with drain rock (clean pea gravel or clean crushed rock), placed in a trench directing flows to a central sump pit. It is recommended that the sum pit be a minimum of 0.8 x 0.8 m (2.5 x 2.5 ft) in plan area and have a seal cover suitable support the weight of an adult. Water should be pumped with the sump pit using suitable piping material with a minimum inside diameter of 30 mm, discharge piping should be equipped with a check valve to prevent backflow into the sump pit and should be graded to prevent freezing water from developing within the system. Installation of subsurface drainage should be in accordance with locally accepted practices and should conform to City of Winnipeg By-law No. 4555/87 Section 23.

The final ground elevation around the perimeter of the structures should be sloped a minimum 2% to promote positive drainage away from the perimeter of all structures and to protect against surface water ponding. Roadways, parking lots, unloading areas and landscaping within a zone of approximately 2 m of the exterior perimeter of any structure should be sloped at a minimum gradient of 5% to compensate for future loss of grade that may result from potential settlement. Downspouts should be positively directed away from structures and beyond the backfill zone.

7.7 Temporary Excavations

All trenching and excavations should conform to the latest version of Manitoba Occupational Health and Safety Regulations (OH&S). A side slope of 1H:1V can be used for all excavations that have a maximum depth of 1.5 m. Excavations deeper than 1.5 m should be reviewed and designed prior to construction by an experienced professional engineer with an expertise in geotechnical engineering.

It is anticipated that seepage will occur in excavations below the groundwater table, and from the shallow silty clay layer observed at the site. To maintain safe working conditions and a stable base for construction, water should not be allowed to accumulate within the excavation. A dewatering system consisting of a series of perimeter trenches and sump pumps located at the base of the excavation can likely control the seepage by pumping the water out of the excavation and discharging away from the limits of construction. The rate of seepage is expected to decrease as pumping operations continue.

Open excavation side slopes should be covered to prevent from drying or saturation and surface runoff should be directed away from excavations. Surcharge loads such as soil stockpiles, equipment, etc. should be kept a minimum of 1 m or a distance equal to the depth of excavation away from the edge of excavation, whichever is greater.

If a deep excavation with a shoring system is considered, KGS Group recommends that an excavation and shoring plan should be prepared and submitted by a registered Professional Engineer who is skilled in these designs.

7.8 Pavement Surfacing

Based on the soil conditions encountered during drilling and subject to inspection by qualified geotechnical personnel, the asphalt surfaced parking areas and new access road may be design based on the recommendations in Table 14.

TABLE 14: PAVEMENT SURFACING SECTIONS

Material	Light Traffic Loading (mm)	Heavy Traffic Loading (mm)	Compaction (%)
Asphalt Pavement	50	100	N/A
Granular Base Course (Gradation A)	150	150	100% Standard Proctor
Granular Subbase (50 or 100 mm, Gradation A or B)	350	450	100% Standard Proctor
Subgrade	• Proof-rolled with heavy sheepsfoot roller • Place non-woven geotextile		

The existing topsoil, organics and silty clay should be sub-excavated to the subgrade design elevation and proof-rolled using a heavy sheepsfoot roller to a minimum compaction of 98% SPMDD. The subgrade should be inspected by qualified geotechnical personnel prior to the placement of the overlying granular base. Areas that exhibit unsuitable deflection or unsuitable soils such as organic matter, silts or soft clays should be sub-excavated an additional 600 mm or as directed by the geotechnical personnel and replaced with compacted granular subbase. Non-woven geotextile fabric should be placed as a separator between the clayey subgrade and compacted granular fill.

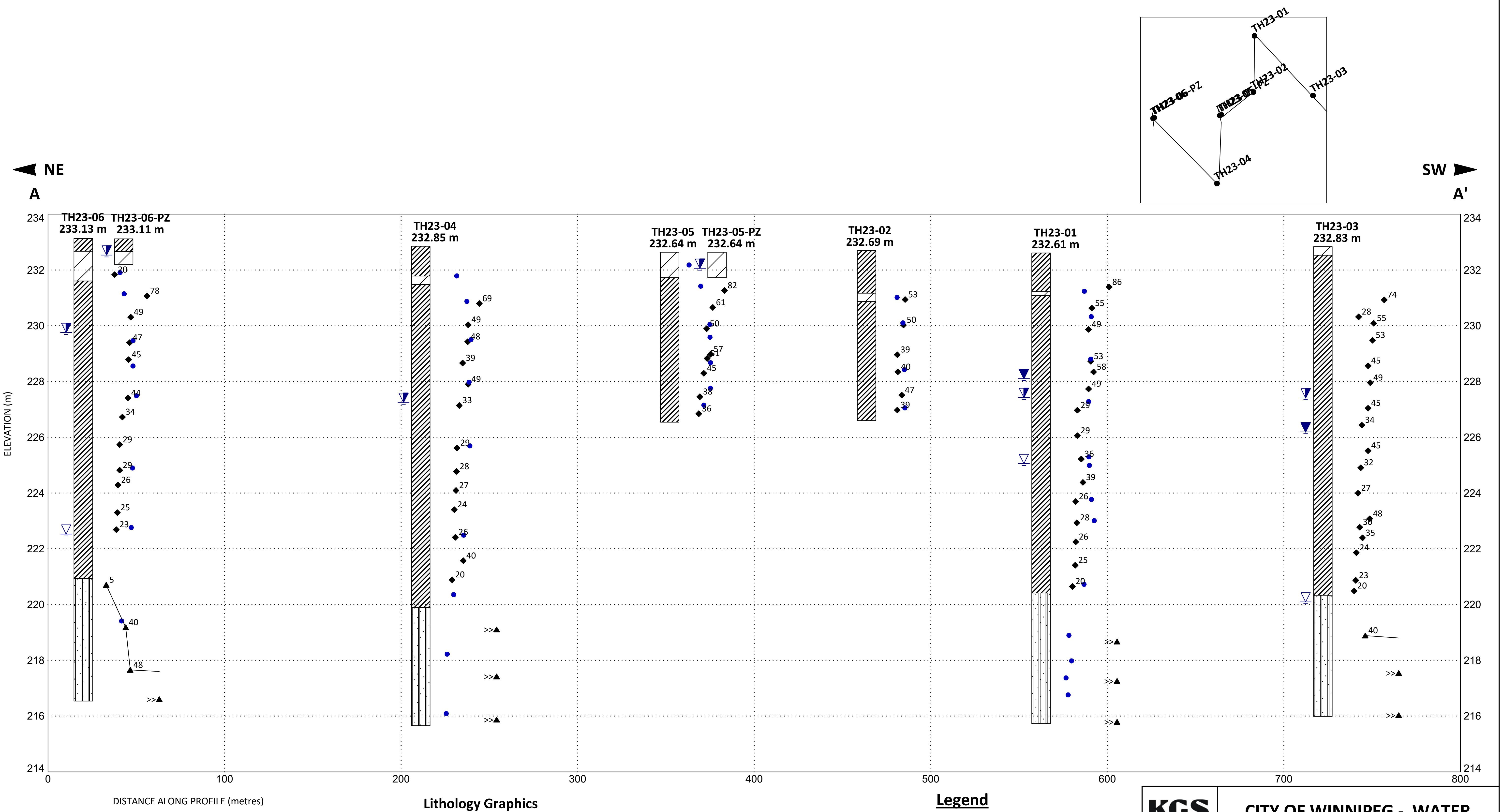
The granular base course (Gradation A) and sub-base (50 or 100 mm, Gradation A or B) should be well-graded and be free of organics and frozen material supplied in accordance with appliable standard specifications, such as City of Winnipeg Specification CW 3110-R22 dated November 2022. Sieve analysis and compaction testing of the granular base and sub-base materials should be conducted by qualified geotechnical personnel to ensure that the materials supplied, and percent compactions are in accordance with design specifications.

7.9 Construction Inspection

KGS Group should be retained to complete the following inspection services during construction:

- Detailed construction records and full-time inspection by experienced geotechnical personnel is recommended throughout construction of foundations to confirm that piles are installed according to the project specifications and meet the intent of the geotechnical design.
- Inspection of driven steel and concrete pile installation by experienced geotechnical personnel is required to ensure that the specified refusal criteria are achieved and will allow for KGS Group to make changes if the soil types encountered at the time of installation are not consistent with those presented within this report.
- Observe proof rolling of sub-grade materials beneath slabs-on-grade, exterior slabs, and pavement sections. Proof rolling inspection by an experienced geotechnical engineer is required to identify unsuitable or soft areas which will need to be sub-excavated and replaced with compacted granular materials.
- Sieve testing of granular base and sub-base materials to ensure that the supplied granular materials meet the graduation requirements specified for these materials.
- Compaction testing of sub-grade materials, and sub-base/base granular materials should be completed to ensure materials are compacted properly and meet the minimum compaction requirements specified for this project.

FIGURES



0 42

Horizontal Scale (meters)
Vertical Exaggeration: 16x

Lithology Graphics

▨ Clay (CH, high plasticity)
◸ Clay (CL, low plasticity)
▥ Silt Till

Legend

TH23-01 — Test Hole ID
Test Hole Lithology —

▽ Water Level During Drilling
▼ Water Level Upon Completion
▼ Water Level Static/Remeasured
▲ SPT, N-Value
◭ DCPT Value
>>▲ values greater than 100
◆ Torvane Cu (kPa)
● Moisture Content (%)

KGS GROUP

CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT

South Winnipeg Recreation Campus

A-A'

Jul 2023

Figure 01

APPENDIX A

Test Hole Logs

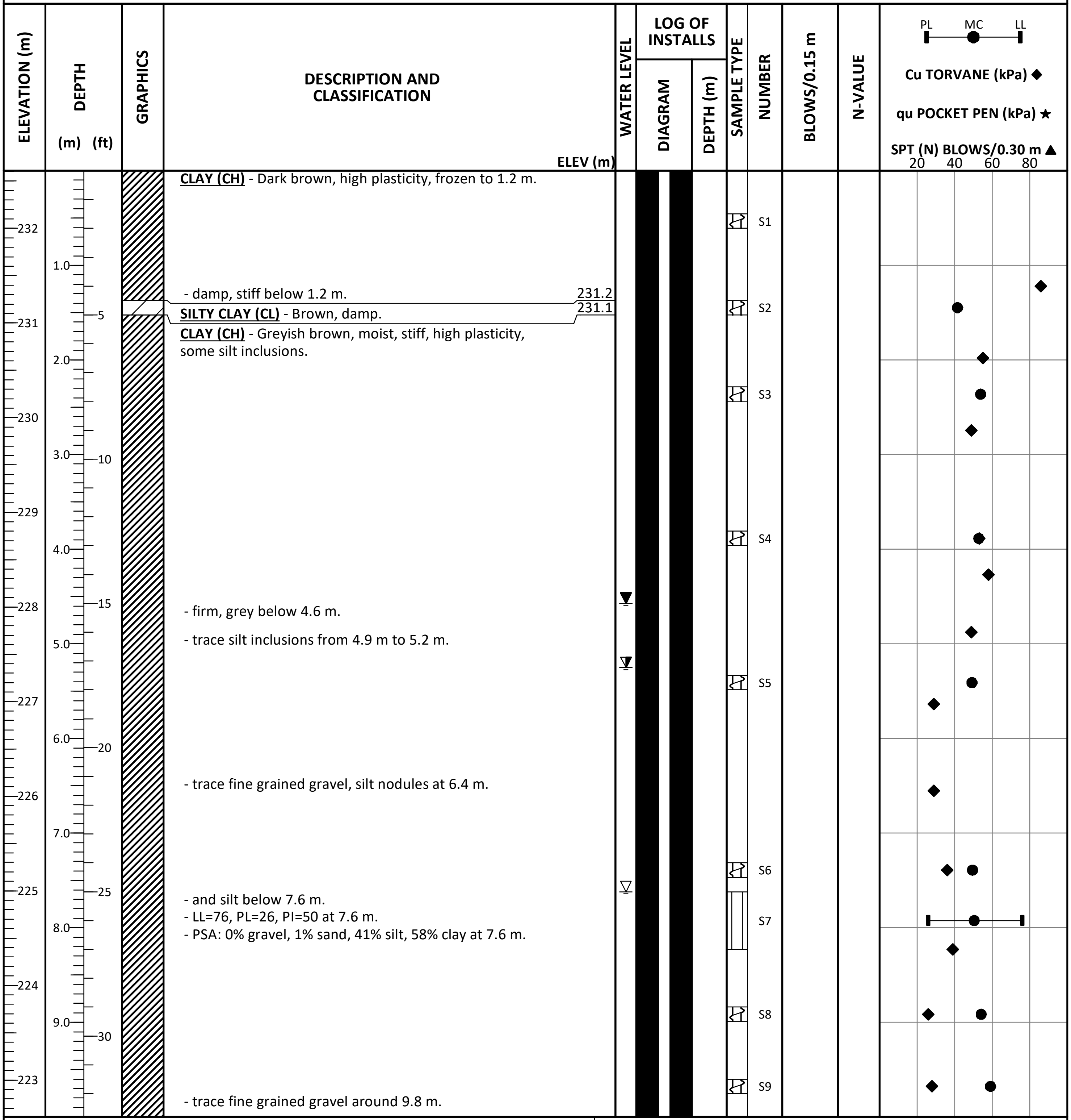
KGS GROUP

TEST HOLE LOG

HOLE NO.
TH23-01

SHEET 1 of 2

CLIENT	**CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT**	**PROJECT NO.**	23-0107-005
PROJECT	**South Winnipeg Recreation Campus**	**SURFACE ELEV.**	232.61 m
LOCATION	Winnipeg, Manitoba	**TOC STICK-UP / ELEV.**	1.07 m / 233.68 m (Standpipe)
DESCRIPTION	SE corner of property, ~300 m South of Cadboro Road	**START DATE**	4-11-2023
DRILL RIG / HAMMER	Mobile B57 Track Mounted Drill Rig with Auto-Hammer	**UTM (m)**	N 5,517,488.85
METHOD(S)	0.0 m to 16.9 m: 125 mm ø SSA		E 630,644.05 Zone 14



CLAY (CH) - Dark brown, high plasticity, frozen to 1.2 m.

- damp, stiff below 1.2 m.

SILTY CLAY (CL) - Brown, damp.

CLAY (CH) - Greyish brown, moist, stiff, high plasticity, some silt inclusions.

- firm, grey below 4.6 m.
- trace silt inclusions from 4.9 m to 5.2 m.
- trace fine grained gravel, silt nodules at 6.4 m.
- and silt below 7.6 m.
- LL=76, PL=26, PI=50 at 7.6 m.
- PSA: 0% gravel, 1% sand, 41% silt, 58% clay at 7.6 m.
- trace fine grained gravel around 9.8 m.

WATER LEVELS ▽	**During Drilling/Digging**	7.62 m
▼	**Upon Completion**	4.57 m
▼	**Remeasured/Static**	5.25 m on 5-16-2023

CONTRACTOR
Paddock Drilling

INSPECTOR
H. SABHARWAL

APPROVED
T. SCHELLENBERG

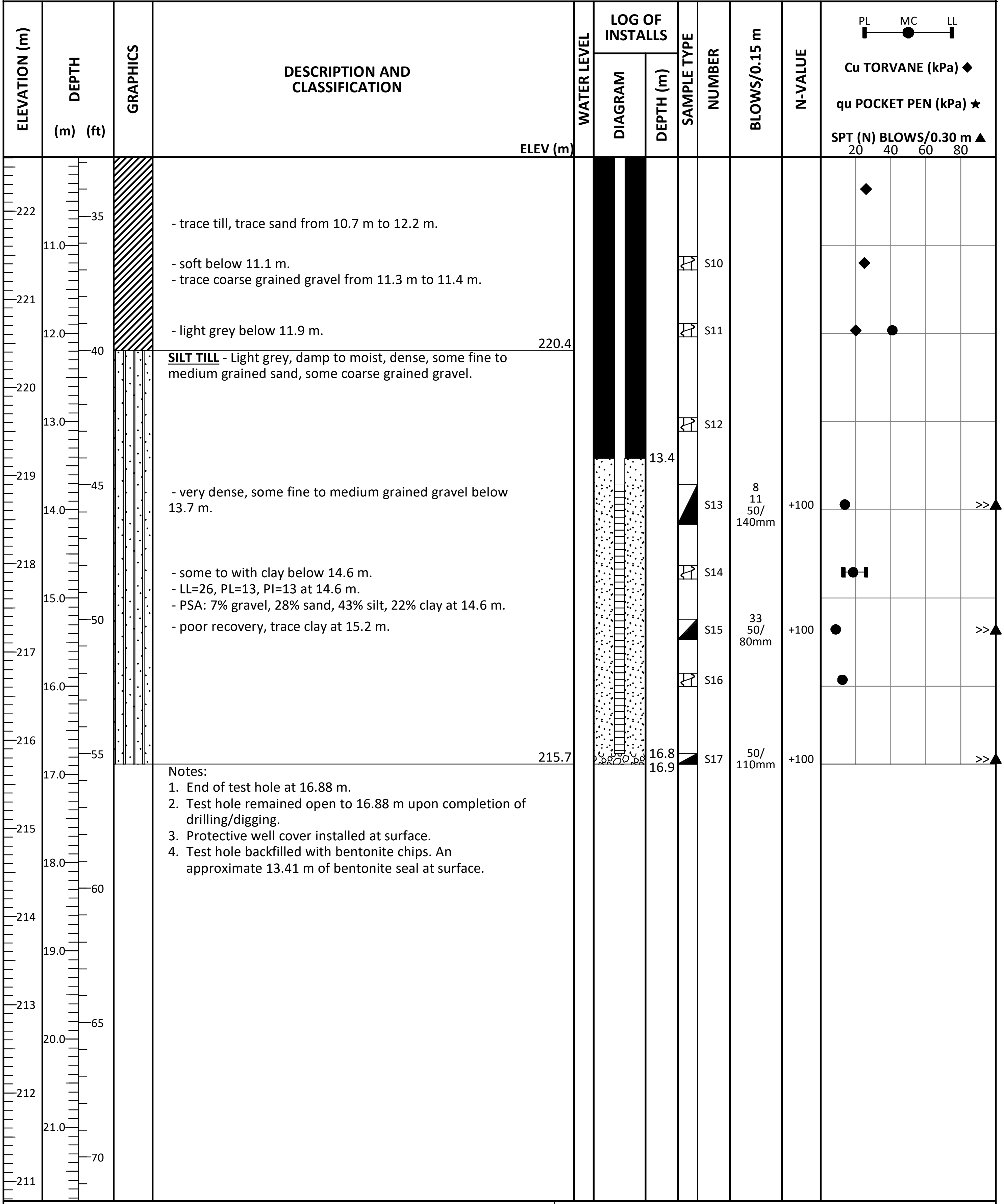
DATE
4-28-2023

KGS GROUP

TEST HOLE LOG

HOLE NO. **TH23-01**

SHEET 2 of 2



- trace till, trace sand from 10.7 m to 12.2 m.

- soft below 11.1 m.
- trace coarse grained gravel from 11.3 m to 11.4 m.

- light grey below 11.9 m.

220.4

SILT TILL - Light grey, damp to moist, dense, some fine to medium grained sand, some coarse grained gravel.

- very dense, some fine to medium grained gravel below 13.7 m.

- some to with clay below 14.6 m.
- LL=26, PL=13, PI=13 at 14.6 m.
- PSA: 7% gravel, 28% sand, 43% silt, 22% clay at 14.6 m.
- poor recovery, trace clay at 15.2 m.

215.7

NUMBER	BLOWS/0.15 m	N-VALUE
S10		
S11		
S12		
S13	8 11 50/ 140mm	+100
S14		
S15	33 50/ 80mm	+100
S16		
S17	50/ 110mm	+100

Notes:
1. End of test hole at 16.88 m.
2. Test hole remained open to 16.88 m upon completion of drilling/digging.
3. Protective well cover installed at surface.
4. Test hole backfilled with bentonite chips. An approximate 13.41 m of bentonite seal at surface.

WATER LEVELS		
∇	During Drilling/Digging	7.62 m
▼	Upon Completion	4.57 m
▼	Remeasured/Static	5.25 m on 5-16-2023

CONTRACTOR
Paddock Drilling

INSPECTOR
H. SABHARWAL

APPROVED
T. SCHELLENBERG

DATE
4-28-2023

KGS GROUP

TEST HOLE LOG

HOLE NO. **TH23-02**

SHEET 1 of 1

CLIENT	**CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT**	**PROJECT NO.**	23-0107-005
PROJECT	**South Winnipeg Recreation Campus**	**SURFACE ELEV.**	232.69 m
LOCATION	Winnipeg, Manitoba	**START DATE**	4-11-2023
DESCRIPTION	Near middle of property, ~200 m South of Cadboro Road	**UTM (m)**	N 5,517,512
DRILL RIG / HAMMER	Mobile B57 Track Mounted Drill Rig with Auto-Hammer		E 630,543 Zone 14
METHOD(S)	0.0 m to 6.1 m: 125 mm ø SSA		

ELEVATION (m) | **DEPTH (m) (ft)** | **GRAPHICS** | **DESCRIPTION AND CLASSIFICATION** | **ELEV (m)** | **WATER LEVEL** | **SAMPLE TYPE** | **NUMBER** | **RECOVERY %**

CLAY (CH) - Dark brown, trace fine to medium grained sand, trace silt nodules, frozen to 1.2 m.

- damp, stiff below 1.2 m.

231.2

SILTY CLAY (CL) - Brown, damp to moist, stiff, with silt inclusions.

230.9

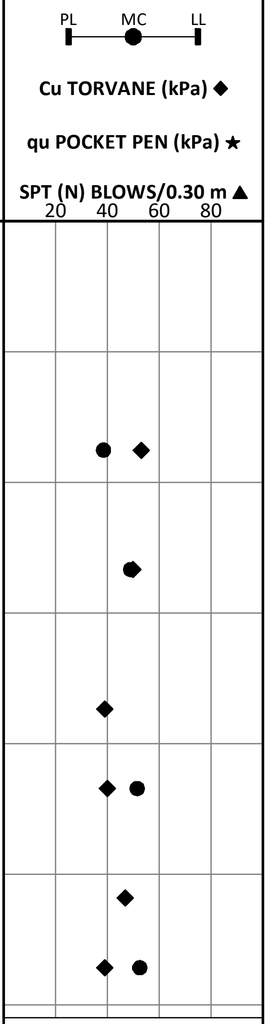
CLAY (CH) - Brown, damp to moist, firm, high plasticity, oxide nodules.

- gypsum inclusions below 2.1 m.
- oxidation below 3.0 m.
- trace fine grained sand at 4.3 m.
- dark grey below 4.6 m.
- trace silt inclusions, trace sand around 5.3 m.

226.6

Notes:
1. End of test hole at 6.10 m.
2. Test hole remained open to 6.10 m upon completion of drilling/digging.
3. Test hole backfilled with auger cuttings.

Number	Recovery %
S1	
S2	
S3	
S4	
S5	
S6	100
S7	



WATER LEVELS ▽ **During Drilling/Digging** Dry
▼ **Upon Completion** Dry

CONTRACTOR **Paddock Drilling**	INSPECTOR **H. SABHARWAL**
APPROVED T. SCHELLENBERG	DATE 4-28-2023

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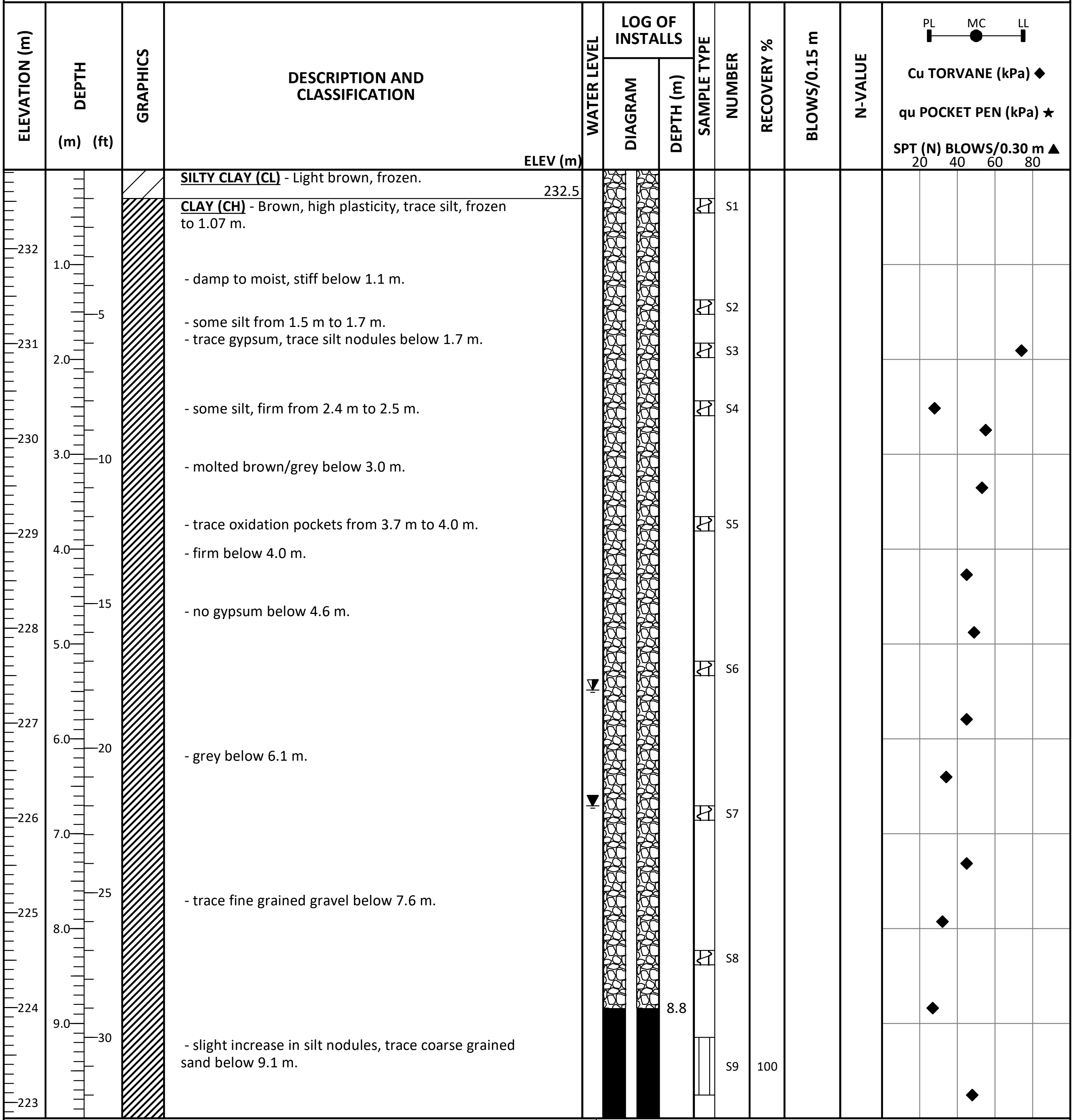
KGS GROUP

TEST HOLE LOG

HOLE NO.
TH23-03

SHEET 1 of 2

CLIENT	**CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT**	**PROJECT NO.**	23-0107-005
PROJECT	**South Winnipeg Recreation Campus**	**SURFACE ELEV.**	232.83 m
LOCATION	Winnipeg, Manitoba	**TOC STICK-UP / ELEV.**	1.00 m / 233.83 m (Standpipe)
DESCRIPTION	SW corner of property, ~300 m South of Cadboro Road	**START DATE**	4-11-2023
DRILL RIG / HAMMER	Mobile B57 Track Mounted Drill Rig with Auto-Hammer	**UTM (m)**	N 5,517,405.86
METHOD(S)	0.0 m to 16.8 m: 125 mm ø SSA		E 630,514.07 Zone 14



SILTY CLAY (CL) - Light brown, frozen.

232.5

CLAY (CH) - Brown, high plasticity, trace silt, frozen to 1.07 m.

- damp to moist, stiff below 1.1 m.
- some silt from 1.5 m to 1.7 m.
- trace gypsum, trace silt nodules below 1.7 m.
- some silt, firm from 2.4 m to 2.5 m.
- molted brown/grey below 3.0 m.
- trace oxidation pockets from 3.7 m to 4.0 m.
- firm below 4.0 m.
- no gypsum below 4.6 m.
- grey below 6.1 m.
- trace fine grained gravel below 7.6 m.
- slight increase in silt nodules, trace coarse grained sand below 9.1 m.

WATER LEVELS		
▽	**During Drilling/Digging**	12.80 m
▼	**Upon Completion**	6.71 m
▼	**Remeasured/Static**	5.49 m on 5-15-2023

CONTRACTOR
Paddock Drilling

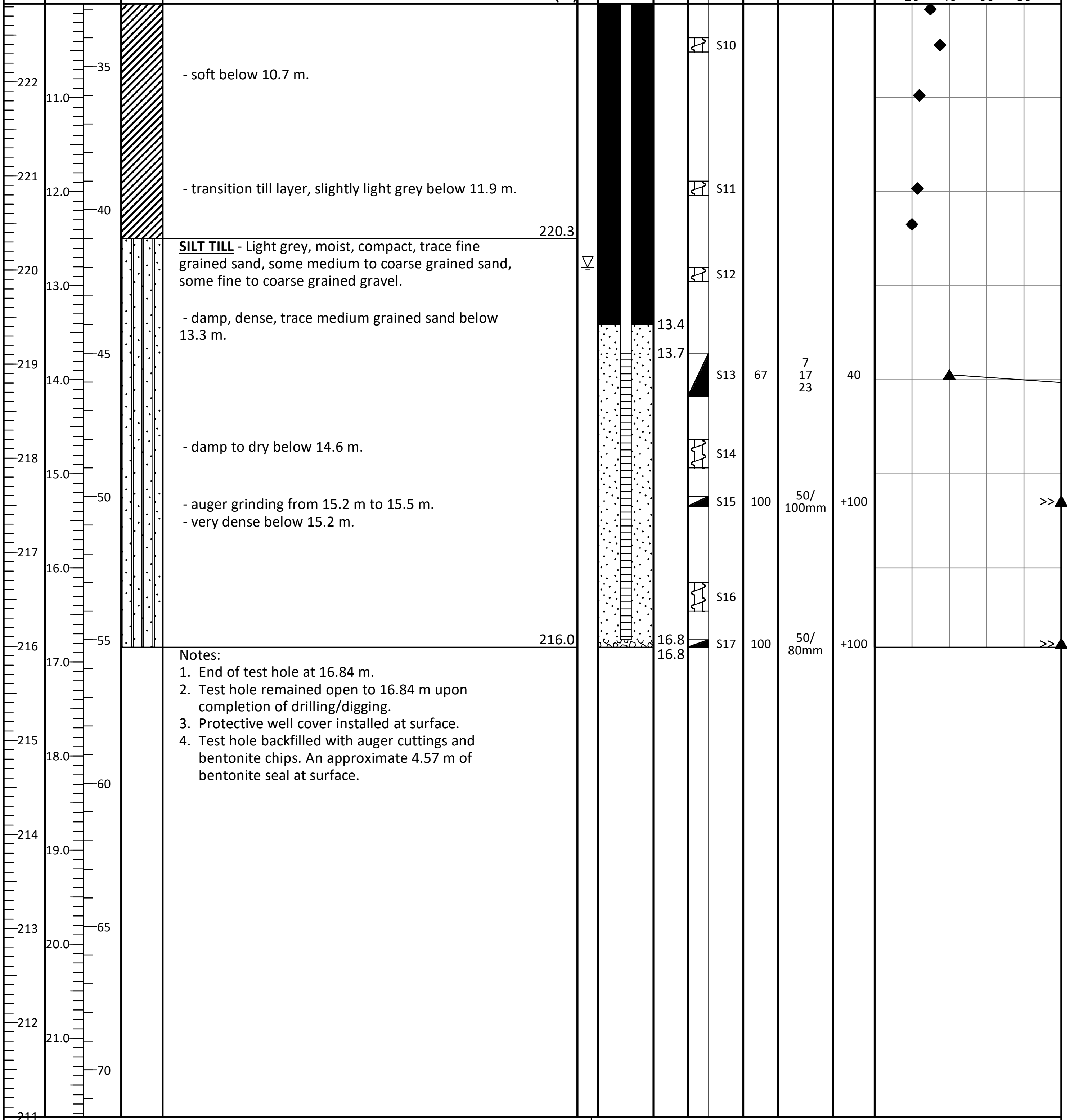
INSPECTOR
C. FRIESEN

APPROVED
T. SCHELLENBERG

DATE
4-28-2023

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	WATER LEVEL	LOG OF INSTALLS DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	BLOWS/0.15 m	N-VALUE
			- soft below 10.7 m.					S10			
			- transition till layer, slightly light grey below 11.9 m.					S11			
			SILT TILL - Light grey, moist, compact, trace fine grained sand, some medium to coarse grained sand, some fine to coarse grained gravel.	220.3	▽			S12			
			- damp, dense, trace medium grained sand below 13.3 m.			13.4 13.7		S13	67	7 17 23	40
			- damp to dry below 14.6 m.					S14			
			- auger grinding from 15.2 m to 15.5 m. - very dense below 15.2 m.					S15	100	50/ 100mm	+100
								S16			
				216.0		16.8 16.8		S17	100	50/ 80mm	+100

Notes:
1. End of test hole at 16.84 m.
2. Test hole remained open to 16.84 m upon completion of drilling/digging.
3. Protective well cover installed at surface.
4. Test hole backfilled with auger cuttings and bentonite chips. An approximate 4.57 m of bentonite seal at surface.



WATER LEVELS		
▽	**During Drilling/Digging**	12.80 m
▼	**Upon Completion**	6.71 m
▼	**Remeasured/Static**	5.49 m on 5-15-2023

CONTRACTOR
Paddock Drilling

INSPECTOR
C. FRIESEN

APPROVED
T. SCHELLENBERG

DATE
4-28-2023

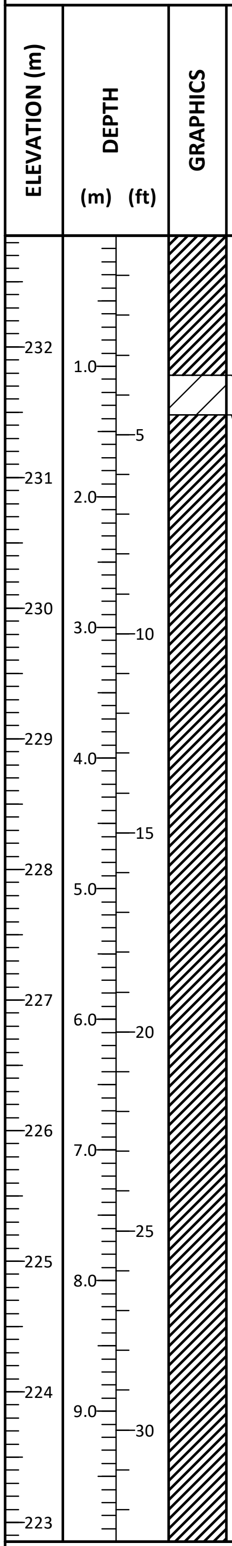
KGS GROUP

TEST HOLE LOG

HOLE NO.
TH23-04

SHEET 1 of 2

CLIENT	**CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT**	**PROJECT NO.**	23-0107-005
PROJECT	**South Winnipeg Recreation Campus**	**SURFACE ELEV.**	232.85 m
LOCATION	Winnipeg, Manitoba	**TOC STICK-UP / ELEV.**	0.98 m / 233.83 m (Standpipe)
DESCRIPTION	NE corner of property, ~50 m South of Cadboro Road	**START DATE**	4-12-2023
DRILL RIG / HAMMER	Mobile B57 Track Mounted Drill Rig with Auto-Hammer	**UTM (m)**	N 5,517,612.58
METHOD(S)	0.0 m to 13.7 m: 125 mm ø SSA - switched due to Sand Blowout 13.7 m to 16.8 m: 150 mm ø HSA		E 630,391.72 Zone 14



DESCRIPTION AND CLASSIFICATION — ELEV (m)

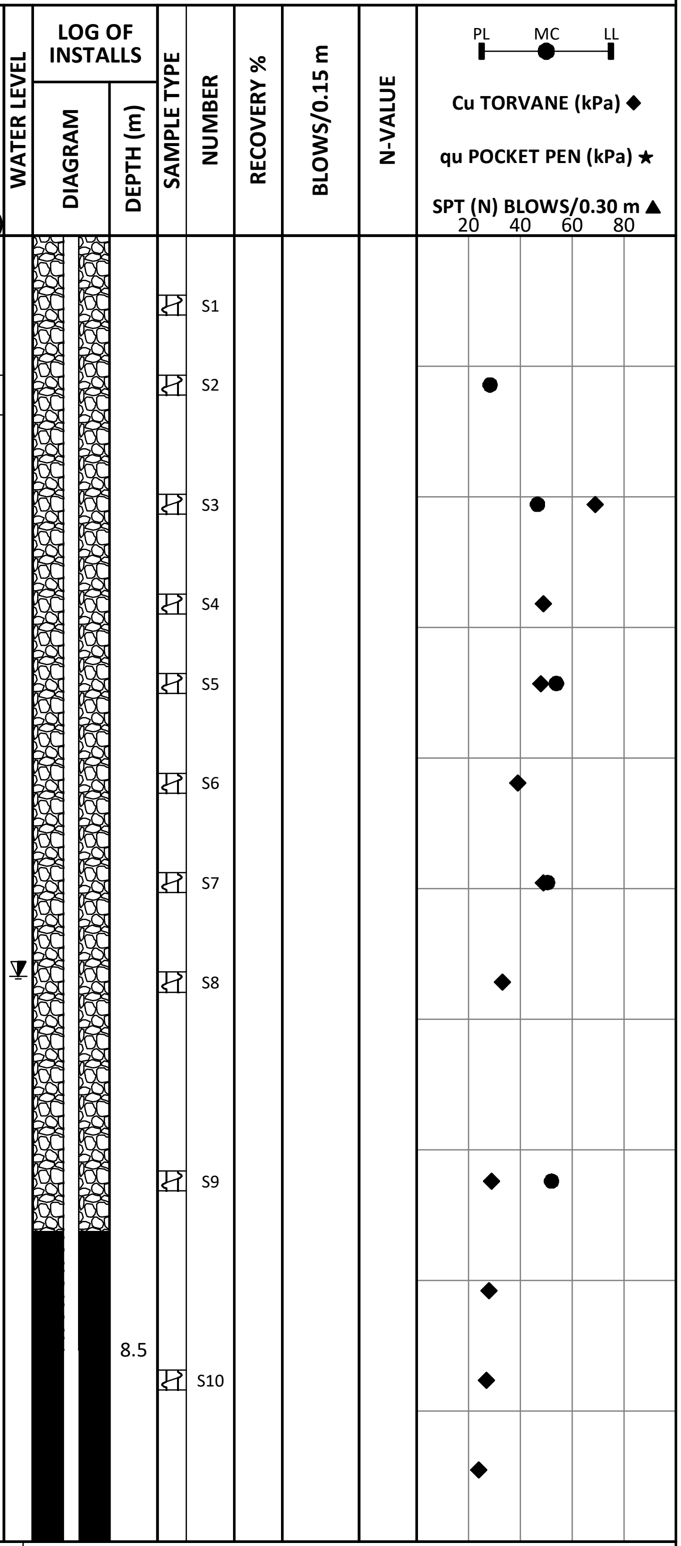
CLAY (CH) - Light brown, trace sand, frozen to 0.91 m.

- damp, stiff below 0.9 m. — 231.8

SILTY CLAY (CL) - Light brownish grey, dry, soft, some sand. — 231.5

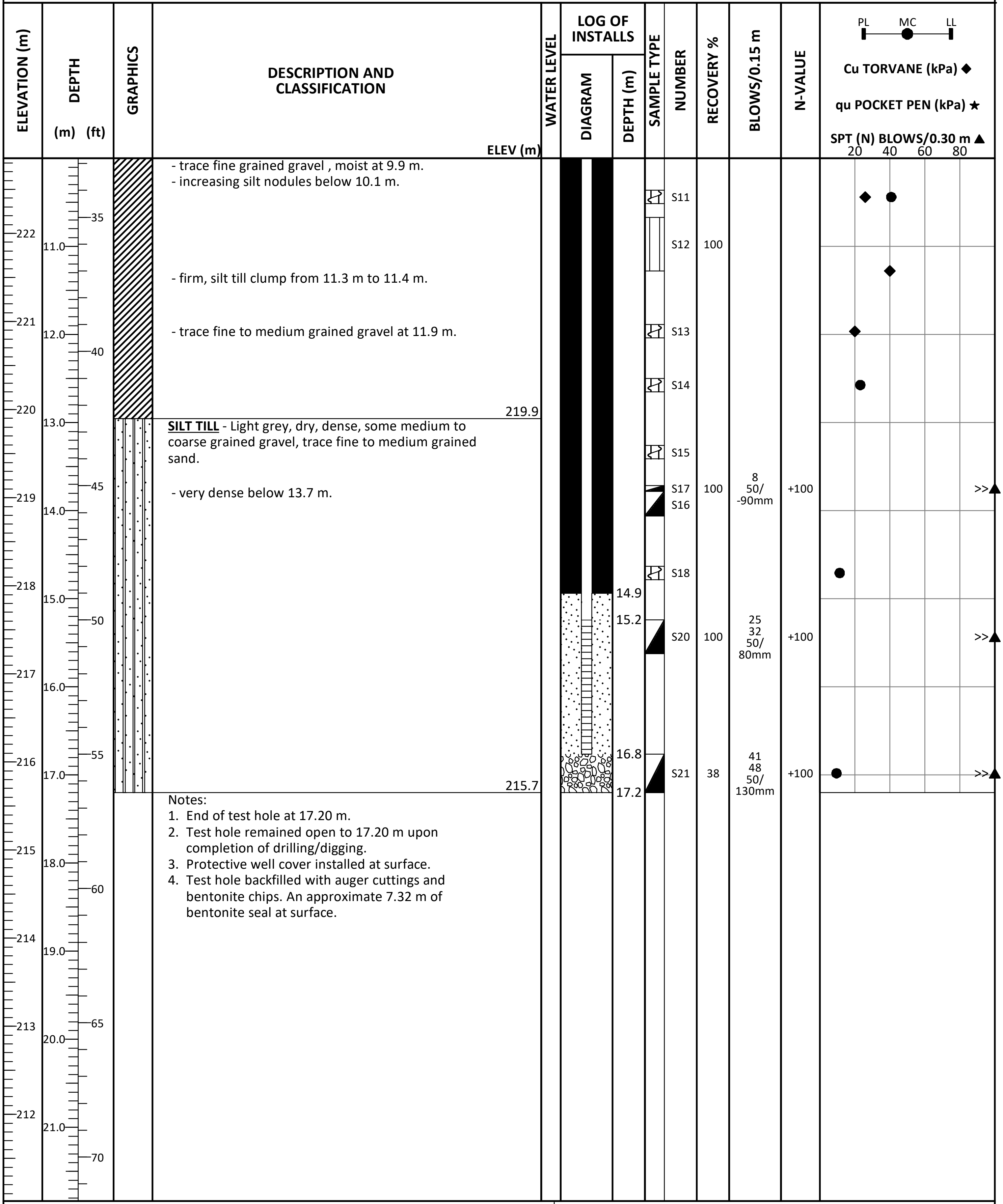
CLAY (CH) - Light brown, damp to moist, stiff, high plasticity, trace silt.

- moderate oxidation, trace silt nodules from 2.1 m to 2.3 m.
- trace gypsum around 2.4 m.
- firm below 2.7 m.
- oxide nodules, trace silt at 4.1 m.
- grey below 5.5 m.
- dark grey, some silt nodules below 6.1 m.
- soft below 9.4 m.



WATER LEVELS	▽ **During Drilling/Digging**	Dry	
	▼ **Upon Completion**	Dry	
	▼ **Remeasured/Static**	5.67 m on 5-15-2023	

CONTRACTOR	INSPECTOR
Paddock Drilling	**H. SABHARWAL**
APPROVED	DATE
T. SCHELLENBERG	4-28-2023



- trace fine grained gravel , moist at 9.9 m.
- increasing silt nodules below 10.1 m.
- firm, silt till clump from 11.3 m to 11.4 m.
- trace fine to medium grained gravel at 11.9 m.

219.9

SILT TILL - Light grey, dry, dense, some medium to coarse grained gravel, trace fine to medium grained sand.

- very dense below 13.7 m.

215.7

Notes:
1. End of test hole at 17.20 m.
2. Test hole remained open to 17.20 m upon completion of drilling/digging.
3. Protective well cover installed at surface.
4. Test hole backfilled with auger cuttings and bentonite chips. An approximate 7.32 m of bentonite seal at surface.

NUMBER	RECOVERY %	BLOWS/0.15 m	N-VALUE
S11			
S12	100		
S13			
S14			
S15			
S17	100	8 50/ -90mm	+100
S16			
S18			
S20	100	25 32 50/ 80mm	+100
S21	38	41 48 50/ 130mm	+100

WATER LEVELS
- ▽ **During Drilling/Digging** Dry
- ▼ **Upon Completion** Dry
- ▼ **Remeasured/Static** 5.67 m on 5-15-2023

CONTRACTOR **Paddock Drilling**

INSPECTOR **H. SABHARWAL**

APPROVED T. SCHELLENBERG

DATE 4-28-2023

KGS GROUP

TEST HOLE LOG

HOLE NO.
TH23-05

SHEET 1 of 1

CLIENT	**CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT**	**PROJECT NO.**	23-0107-005
PROJECT	**South Winnipeg Recreation Campus**	**SURFACE ELEV.**	232.64 m
LOCATION	Winnipeg, Manitoba	**START DATE**	4-12-2023
DESCRIPTION	Near middle of property, ~125 m South of Cadboro Road	**UTM (m)**	N 5,517,581.79
DRILL RIG / HAMMER	Mobile B57 Track Mounted Drill Rig with Auto-Hammer		E 630,513.25 Zone 14
METHOD(S)	0.0 m to 6.1 m: 125 mm ø SSA		

ELEVATION (m) | **DEPTH (m) (ft)** | **GRAPHICS** | **DESCRIPTION AND CLASSIFICATION** | **ELEV (m)** | **WATER LEVEL** | **SAMPLE TYPE** | **NUMBER** | **RECOVERY %**

SILTY CLAY (CL) - Light brown, trace to some sand, frozen to 0.76 m.

- LL=26, PL=18, PI=8 at 0.5 m.
- PSA: 0% gravel, 6% sand, 75% silt, 19% clay at 0.5 m.
- damp below 0.8 m.

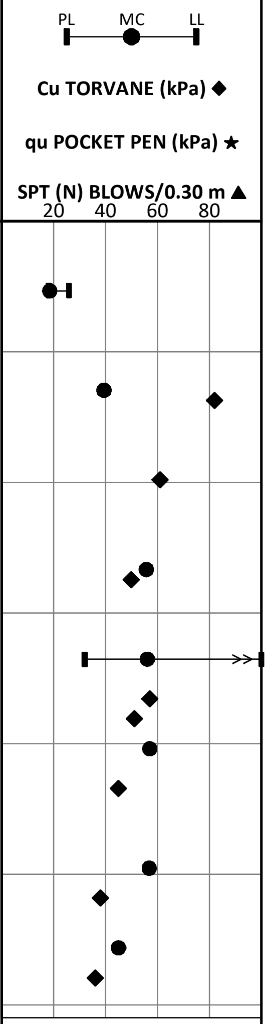
231.7

CLAY (CH) - Dark brown, damp, stiff, high plasticity, trace sand, trace silt nodules.

- minor oxidation at 2.0 m.
- light brown below 2.9 m.
- with silt at 3.0 m.
- LL=102, PL=32, PI=70 at 3.0 m.
- PSA: 0% gravel, 0% sand, 22% silt, 78% clay at 3.0 m.
- trace silt from 3.8 m to 4.3 m.
- firm below 4.3 m.
- silt pockets at 4.4 m.
- grey below 5.5 m.

226.5

NUMBER	RECOVERY %
S1	
S2	
S3	
S4	
S5	100
S6	
S7	
S8	
S9	



Notes:
1. End of test hole at 6.10 m.
2. Test hole remained open to 6.10 m upon completion of drilling/digging.

WATER LEVELS	∇ **During Drilling/Digging**	Dry
	▼ **Upon Completion**	Dry

CONTRACTOR **Paddock Drilling**	INSPECTOR **H. SABHARWAL**
APPROVED T. SCHELLENBERG	DATE 4-28-2023

KGS GROUP

TEST HOLE LOG

HOLE NO.
TH23-05-PZ

SHEET 1 of 1

CLIENT	**CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT**	**PROJECT NO.**	23-0107-005
PROJECT	**South Winnipeg Recreation Campus**	**SURFACE ELEV.**	232.64 m
LOCATION	Winnipeg, Manitoba	**TOC STICK-UP / ELEV.**	0.91 m / 233.55 m (Standpipe)
DESCRIPTION	3 m south of test hole TH23-05	**START DATE**	4-12-2023
DRILL RIG / HAMMER	Mobile B57 Track Mounted Drill Rig with Auto-Hammer	**UTM (m)**	N 5,517,578.79 E 630,514.25 Zone 14
METHOD(S)	0.0 m to 0.9 m: 125 mm ø SSA		

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	WATER LEVEL	LOG OF INSTALLS DIAGRAM	LOG OF INSTALLS DEPTH (m)	SAMPLE TYPE	NUMBER	PL MC LL / Cu TORVANE (kPa) ◆ / qu POCKET PEN (kPa) ★ / SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
		▨	**SILTY CLAY (CL)** - Light brown, trace to some sand, frozen to 0.76 m.				0.3			
232					▼					
			- damp below 0.8 m.	231.7			0.9			
	1.0		Notes: 1. End of test hole at 0.91 m. 2. Test hole remained open to 0.91 m upon completion of drilling/digging. 3. Protective well cover installed at surface. 4. Test hole backfilled with bentonite chips. An approximate 0.30 m of bentonite seal at surface.							

Elevation scale (m): 232, 231, 230, 229, 228, 227, 226, 225, 224, 223
Depth scale (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0
Depth scale (ft): 5, 10, 15, 20, 25, 30

WATER LEVELS	▽ **During Drilling/Digging**	Dry
	▼ **Upon Completion**	Dry
	▼ **Remeasured/Static**	0.64 m on 5-16-2023

CONTRACTOR **Paddock Drilling**	INSPECTOR **H. SABHARWAL**
APPROVED T. SCHELLENBERG	DATE 4-28-2023

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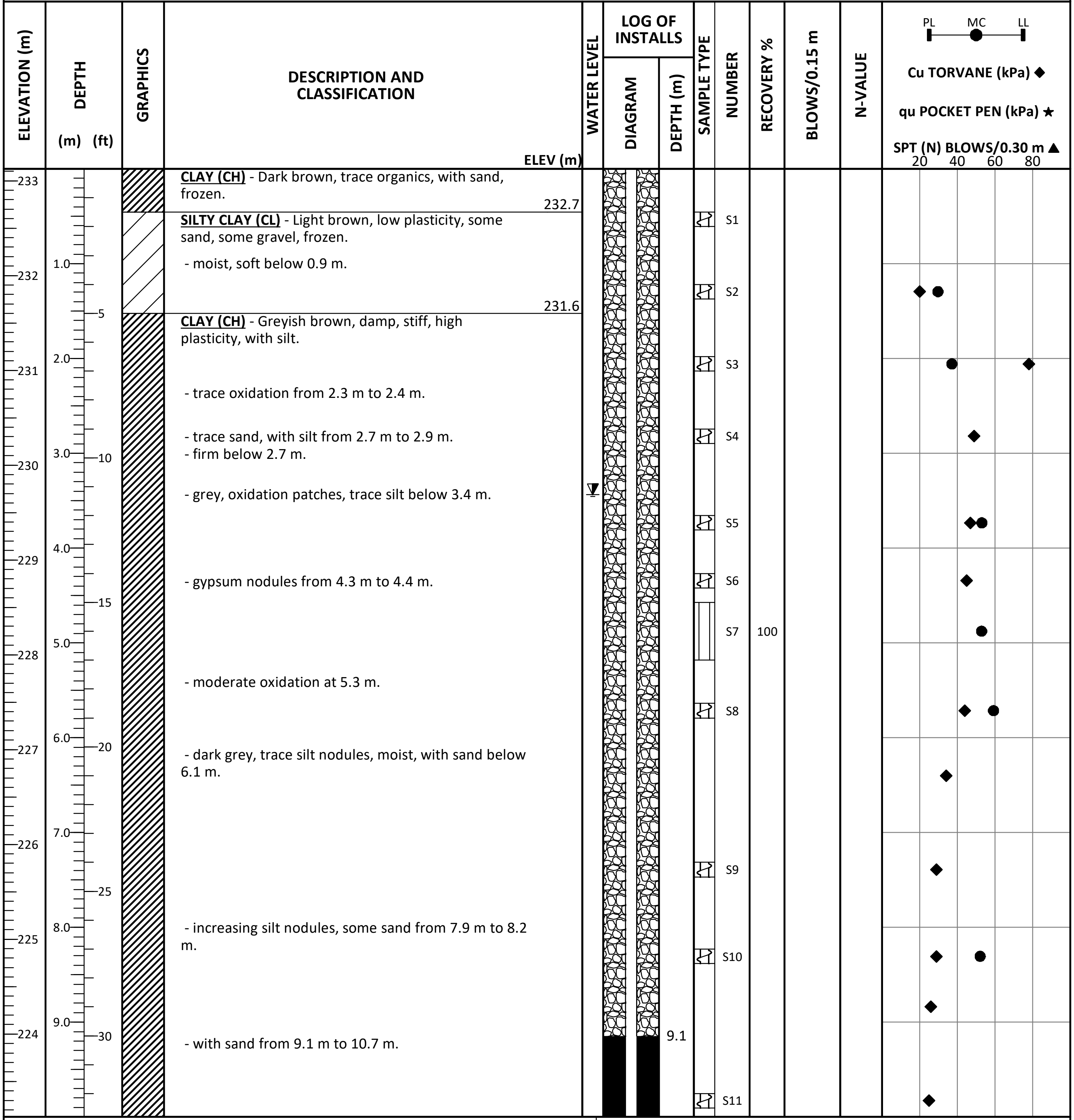
TEST HOLE LOG

HOLE NO.
TH23-06

SHEET 1 of 2

CLIENT	**CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT**	**PROJECT NO.**	23-0107-005
PROJECT	**South Winnipeg Recreation Campus**	**SURFACE ELEV.**	233.13 m
LOCATION	Winnipeg, Manitoba	**TOC STICK-UP / ELEV.**	1.12 m / 234.24 m (Standpipe)
DESCRIPTION	NE corner of property, ~50 m South of Cadboro Road	**START DATE**	4-13-2023
DRILL RIG / HAMMER	Mobile B57 Track Mounted Drill Rig with Auto-Hammer	**UTM (m)**	N 5,517,703.35
METHOD(S)	0.0 m to 10.7 m: 125 mm ø SSA - switched due to wet caving sand 12.2 m to 16.5 m: 250 mm ø HSA		E 630,533.45 Zone 14



DESCRIPTION AND CLASSIFICATION — ELEV (m)

CLAY (CH) - Dark brown, trace organics, with sand, frozen. — 232.7

SILTY CLAY (CL) - Light brown, low plasticity, some sand, some gravel, frozen.

- moist, soft below 0.9 m. — 231.6

CLAY (CH) - Greyish brown, damp, stiff, high plasticity, with silt.

- trace oxidation from 2.3 m to 2.4 m.
- trace sand, with silt from 2.7 m to 2.9 m.
- firm below 2.7 m.
- grey, oxidation patches, trace silt below 3.4 m.
- gypsum nodules from 4.3 m to 4.4 m.
- moderate oxidation at 5.3 m.
- dark grey, trace silt nodules, moist, with sand below 6.1 m.
- increasing silt nodules, some sand from 7.9 m to 8.2 m.
- with sand from 9.1 m to 10.7 m.

WATER LEVELS ▽ **During Drilling/Digging**	10.67 m	
▼ **Upon Completion**	Dry	
▼ **Remeasured/Static**	3.44 m on 5-12-2023	

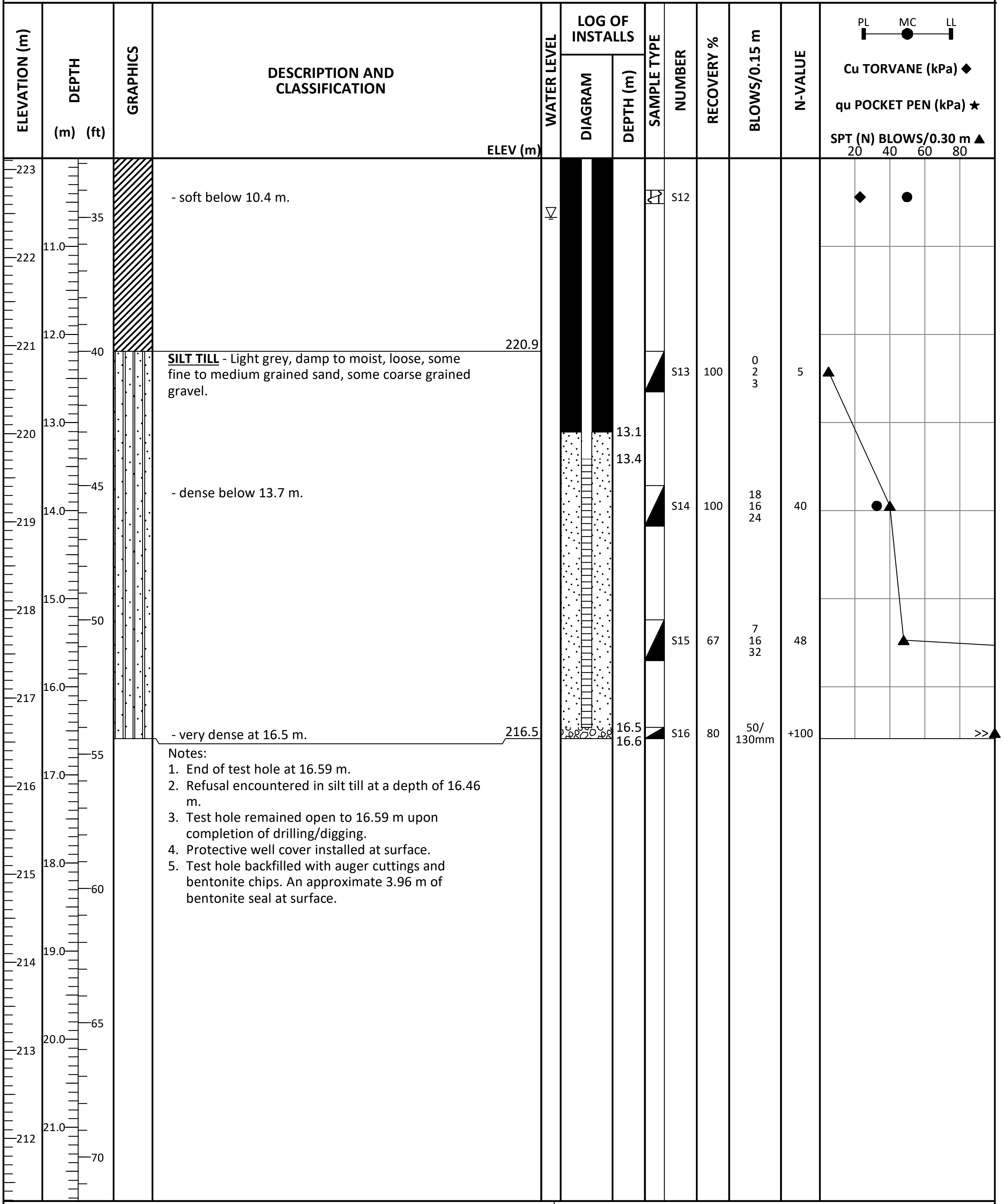
CONTRACTOR	INSPECTOR
Paddock Drilling	**H. SABHARWAL**
APPROVED	DATE
T. SCHELLENBERG	4-28-2023

KGS GROUP

TEST HOLE LOG

HOLE NO.
TH23-06

SHEET 2 of 2



DESCRIPTION AND CLASSIFICATION	ELEV (m)
- soft below 10.4 m.	
SILT TILL - Light grey, damp to moist, loose, some fine to medium grained sand, some coarse grained gravel.	220.9
- dense below 13.7 m.	
- very dense at 16.5 m.	216.5

NUMBER	RECOVERY %	BLOWS/0.15 m	N-VALUE
S12			
S13	100	0 2 3	5
S14	100	18 16 24	40
S15	67	7 16 32	48
S16	80	50/ 130mm	+100

Notes:
1. End of test hole at 16.59 m.
2. Refusal encountered in silt till at a depth of 16.46 m.
3. Test hole remained open to 16.59 m upon completion of drilling/digging.
4. Protective well cover installed at surface.
5. Test hole backfilled with auger cuttings and bentonite chips. An approximate 3.96 m of bentonite seal at surface.

WATER LEVELS		
▽	During Drilling/Digging	10.67 m
▼	Upon Completion	Dry
▼	Remeasured/Static	3.44 m on 5-12-2023

CONTRACTOR
Paddock Drilling

INSPECTOR
H. SABHARWAL

APPROVED
T. SCHELLENBERG

DATE
4-28-2023

KGS GROUP

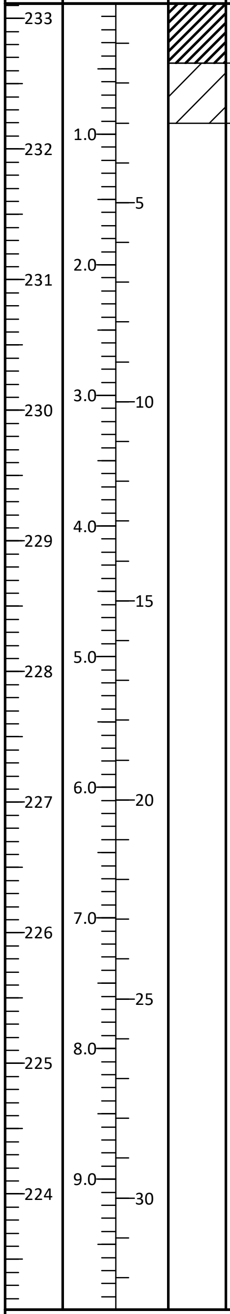
TEST HOLE LOG

HOLE NO.
TH23-06-PZ

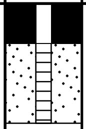
SHEET 1 of 1

CLIENT **CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT**
PROJECT **South Winnipeg Recreation Campus**
LOCATION Winnipeg, Manitoba
DESCRIPTION 2 m north of test hole TH23-06
DRILL RIG / HAMMER Mobile B57 Track Mounted Drill Rig with Auto-Hammer
METHOD(S) 0.0 m to 0.9 m: 125 mm ø SSA

PROJECT NO. 23-0107-005
SURFACE ELEV. 233.11 m
TOC STICK-UP / ELEV. 1.01 m / 234.13 m (Standpipe)
START DATE 4-14-2023
UTM (m) N 5,517,701
E 630,534.08 Zone 14



ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	WATER LEVEL	LOG OF INSTALLS DIAGRAM	LOG OF INSTALLS DEPTH (m)	SAMPLE TYPE	NUMBER	PL MC LL Cu TORVANE (kPa) ◆ qu POCKET PEN (kPa) ★ SPT (N) BLOWS/0.30 m ▲ 20 40 60 80
		▨	**CLAY (CH)** - Dark brown, trace organics, with sand, frozen.	232.7			0.3			
		▨	**SILTY CLAY (CL)** - Light brown, low plasticity, some sand, some gravel, frozen.	232.2	▼		0.9			



Notes:
1. End of test hole at 0.91 m.
2. Test hole remained open to 0.91 m upon completion of drilling/digging.
3. Protective well cover installed at surface.
4. Test hole backfilled with bentonite chips. An approximate 0.30 m of bentonite seal at surface.

WATER LEVELS		
▽	**During Drilling/Digging**	Dry
▼	**Upon Completion**	Dry
▼	**Remeasured/Static**	0.65 m on 5-12-2023

CONTRACTOR
Paddock Drilling

INSPECTOR
H. SABHARWAL

APPROVED
T. SCHELLENBERG

DATE
4-28-2023

KEY TO SYMBOLS

LITHOLOGIC SYMBOLS

▨ Clay (CH, high plasticity)

⧄ Clay (CL, low plasticity)

▥ Silt Till

SAMPLER SYMBOLS

⌇ Auger Grab

▯ Shelby Tube

◪ SPT Split Spoon

WELL CONSTRUCTION SYMBOLS

▩ Slough

◼ Standpipe (bentonite)

▩ Standpipe (cuttings)

░ Standpipe (filter sand)

▤ Screen (filter sand)

ABBREVIATIONS

LL - Liquid Limit
PL - Plastic Limit
PI - Plastic Index
MC - Moisture Content
DD - Dry Density
NP - Non-Plastic
-200 - Percent Passing No. 200 Sieve
TV - Torvane (kPa)
PP - Pocket Penetrometer (kPa)
PSA - Particle Size Analysis
TOC - Top Of Casing

PN - Pneumatic Piezometer
VW - Vibrating Wire Piezometer
PID - Photoionization Detector
ppm - Parts Per Million
▽ Water Level During Drilling
▼ Water Level Upon Completion of Drilling
▼ Water Level Remeasured/Static

KGS GROUP

CLIENT **CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT** **PROJECT NO.** 23-0107-005
PROJECT NAME South Winnipeg Recreation Campus **LOCATION** Winnipeg, Manitoba

APPENDIX B

Laboratory Test Results

SUMMARY OF INDEX TESTS

Sheet 1 of 1

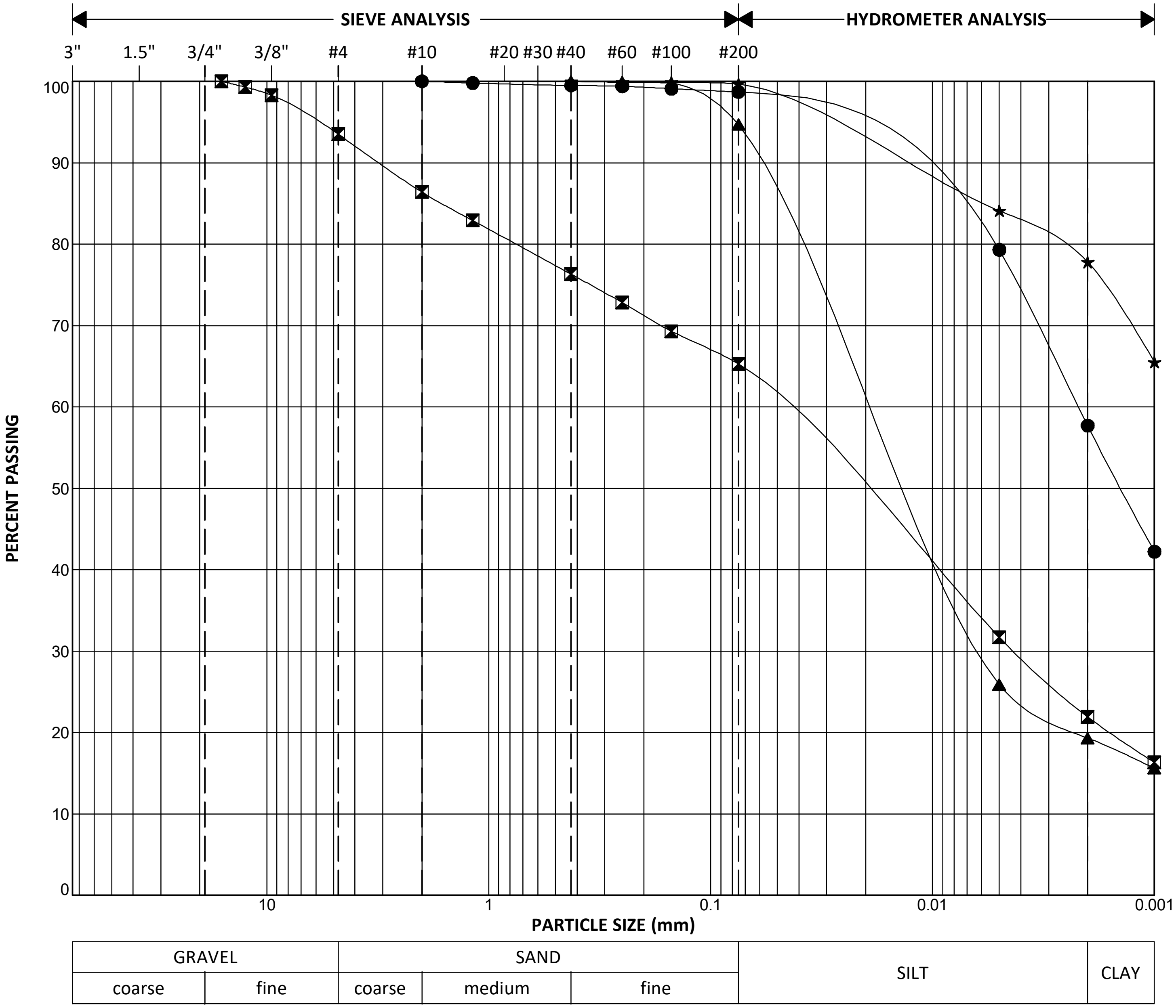
Test Hole ID	Sample No.	Depth (m)	Class-ification	Gravel (%)	Sand (%)	Silt/ Clay (%)	Liquid Limit	Plastic Limit	Plas-ticity Index	Moisture Content (%)	Dry Density (kN/m3)	Specific Gravity	Satur-ation (%)	Void Ratio
TH23-01	S2	1.4	CL							42				
TH23-01	S3	2.3	CH							54				
TH23-01	S4	3.8	CH							53				
TH23-01	S5	5.3	CH							49				
TH23-01	S6	7.3	CH							50				
TH23-01	S7	7.6	CH	0	1	99	76	26	50	50				
TH23-01	S8	8.8	CH							54				
TH23-01	S9	9.6	CH							59				
TH23-01	S11	11.9	CH							41				
TH23-01	S13	13.7	TILL							14				
TH23-01	S14	14.6	TILL	7	28	65	26	13	13	19				
TH23-01	S15	15.2	TILL							9				
TH23-01	S16	15.8	TILL							12				
TH23-02	S2	1.7	CL							39				
TH23-02	S3	2.6	CH							49				
TH23-02	S5	4.3	CH							52				
TH23-02	S7	5.6	CH							53				
TH23-04	S2	1.1	CL							28				
TH23-04	S3	2.0	CH							47				
TH23-04	S5	3.4	CH							54				
TH23-04	S7	4.9	CH							51				
TH23-04	S9	7.2	CH							52				
TH23-04	S11	10.4	CH							41				
TH23-04	S14	12.5	CH							23				
TH23-04	S18	14.6	TILL							11				
TH23-04	S21	16.8	TILL							10				
TH23-05	S1	0.5	CL	0	5	95	26	18	8	19				
TH23-05	S2	1.2	CH							39				
TH23-05	S4	2.6	CH							56				
TH23-05	S5	3.0	CH	0	0	100	102	32	70	56				
TH23-05	S6	4.0	CH							57				
TH23-05	S8	4.9	CH							57				
TH23-05	S9	5.5	CH							45				
TH23-06	S2	1.2	CL							30				
TH23-06	S3	2.0	CH							37				
TH23-06	S5	3.7	CH							53				
TH23-06	S7	4.6	CH							53				
TH23-06	S8	5.6	CH							59				
TH23-06	S10	8.2	CH							52				
TH23-06	S12	10.4	CH							50				
TH23-06	S14	13.7	TILL							33				

* Moisture conditioned and remolded sample.
** Assumed specific gravity.

KGS GROUP

CLIENT **CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT**
PROJECT NAME South Winnipeg Recreation Campus
PROJECT NO. 23-0107-005
LOCATION Winnipeg, Manitoba

GRAIN SIZE DISTRIBUTION

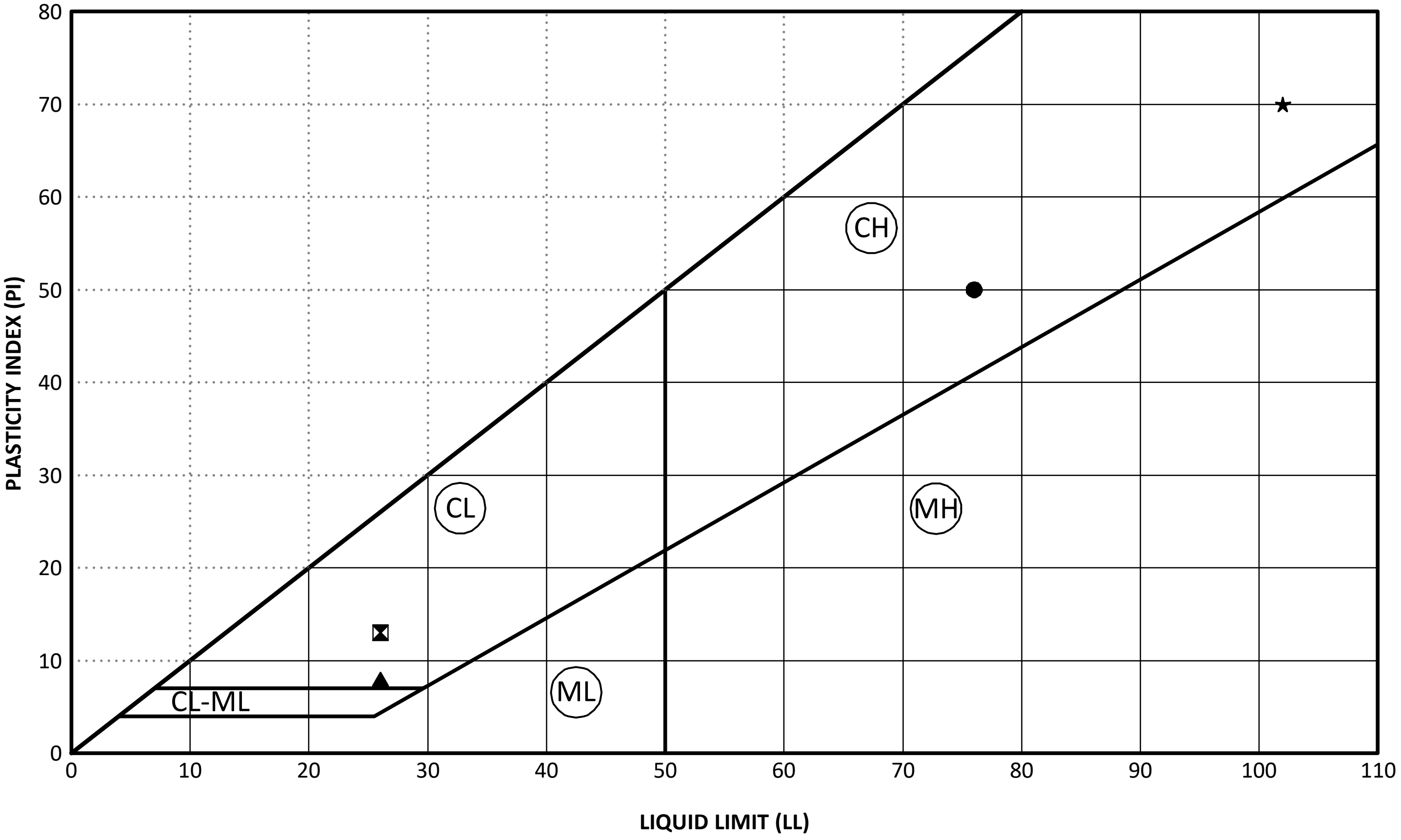


	HOLE	DEPTH (m)	SAMPLE #	GRAVEL (%)	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	Cu	Cc	CLASSIFICATION
●	TH23-01	7.6	S7	0	1	41	58	99			CH
☒	TH23-01	14.6	S14	7	28	43	22	65			TILL
▲	TH23-05	0.5	S1	0	5	75	19	95			CL
★	TH23-05	3.0	S5	0	0	22	78	100			CH

KGS GROUP

CLIENT	**CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT**	**PROJECT NO.**	23-0107-005
PROJECT NAME	South Winnipeg Recreation Campus	**LOCATION**	Winnipeg, Manitoba

ATTERBERG LIMITS



	HOLE	DEPTH (m)	SAMPLE #	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	SILT & CLAY (%)	MC (%)	CLASSIFICATION
●	TH23-01	7.6	S7	76	26	50	1	41	58	99	50	CH
☒	TH23-01	14.6	S14	26	13	13	28	43	22	65	19	TILL
▲	TH23-05	0.5	S1	26	18	8	5	75	19	95	19	CL
★	TH23-05	3.0	S5	102	32	70	0	22	78	100	56	CH

KGS GROUP

CLIENT CITY OF WINNIPEG - WATER AND WASTE DEPARTMENT
PROJECT NAME South Winnipeg Recreation Campus
PROJECT NO. 23-0107-005
LOCATION Winnipeg, Manitoba

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

ASTM D5084 - MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER

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

ATTN: Trevor Schellenberg

PROJECT: South Winnipeg Recreation Centre (23-0107-005)

PROJECT NO. 123316495

REPORT NO. 1

SAMPLE ID: TH23-01, S7, 25'-27'

SOIL DESCRIPTION: Clay, grey, firm, moist, high plasticity, trace silt, trace fine gravel

DATE TESTED: May 5 to May 19, 2023

CONFINING PRESSURE (kPa): 137.9

EFFECTIVE SATURATION STRESS (kPa): 34.5

ASSUMED SPECIFIC GRAVITY: 2.71

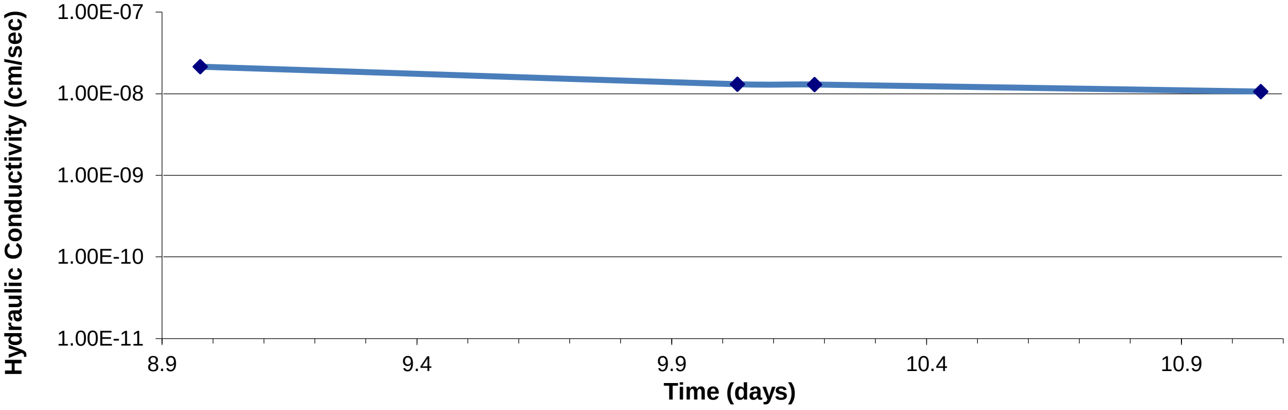
HYDRAULIC GRADIENT: 18.9

TYPE OF PERMEANT LIQUID: De-aired Water

HYDRAULIC CONDUCTIVITY, "k" (cm/s): 1.5E-08

HYDRAULIC CONDUCTIVITY, "k_{20}" (cm/s): 1.5E-08

	Height (mm)	Diameter (mm)	Wet Mass (g)	Dry Density (g/cm^3)	Water Content by Mass (%)	Water Content by Volume (%)	Saturation (%)
Initial Reading	77.6	71.6	548.1	1.168	50.2	58.6	103.0
Final Reading	78.4	71.8	553.5	1.174	48.5	57.0	100.5



COMMENTS:
Tube recovered 42 cm of soil. Test sample portion taken from 10 cm to 30 cm from bottom end of tube.

REPORT DATE: 2023.May.23

REVIEWED BY
GBeauce
Guillaume Beauce, P.Eng.
Geotechnical Engineer - Materials Testing Services

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

ASTM D5084 - MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER

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

ATTN: Trevor Schellenberg

PROJECT South Winnipeg Recreation Centre (23-0107-005)

PROJECT NO. 123316495

REPORT NO. 2

SAMPLE ID: TH23-05, S5, 10'-12'

SOIL DESCRIPTION: Clay, grey, stiff, moist, high plasticity, trace silt

DATE TESTED: May 5 to May 19, 2023

CONFINING PRESSURE (kPa): 137.9

EFFECTIVE SATURATION STRESS (kPa): 34.5

ASSUMED SPECIFIC GRAVITY: 2.71

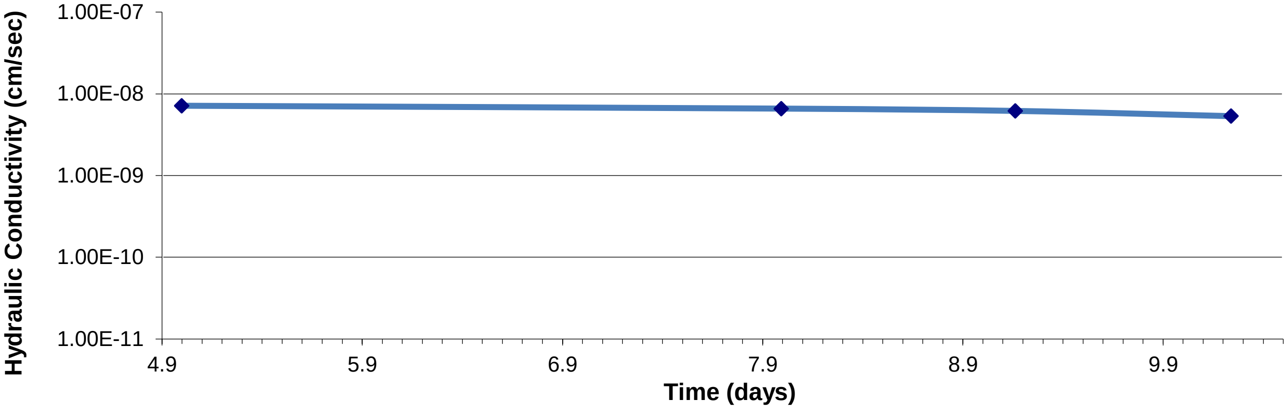
HYDRAULIC GRADIENT: 18.8

TYPE OF PERMEANT LIQUID: De-aired Water

HYDRAULIC CONDUCTIVITY, "k" (cm/s): 6.7E-09

HYDRAULIC CONDUCTIVITY, "k_{20}" (cm/s): 6.3E-09

	Height (mm)	Diameter (mm)	Wet Mass (g)	Dry Density (g/cm^3)	Water Content by Mass (%)	Water Content by Volume (%)	Saturation (%)
Initial Reading	77.2	71.5	532.2	1.104	55.6	61.4	103.7
Final Reading	78.8	72.0	540.1	1.082	55.6	60.2	100.2



COMMENTS:

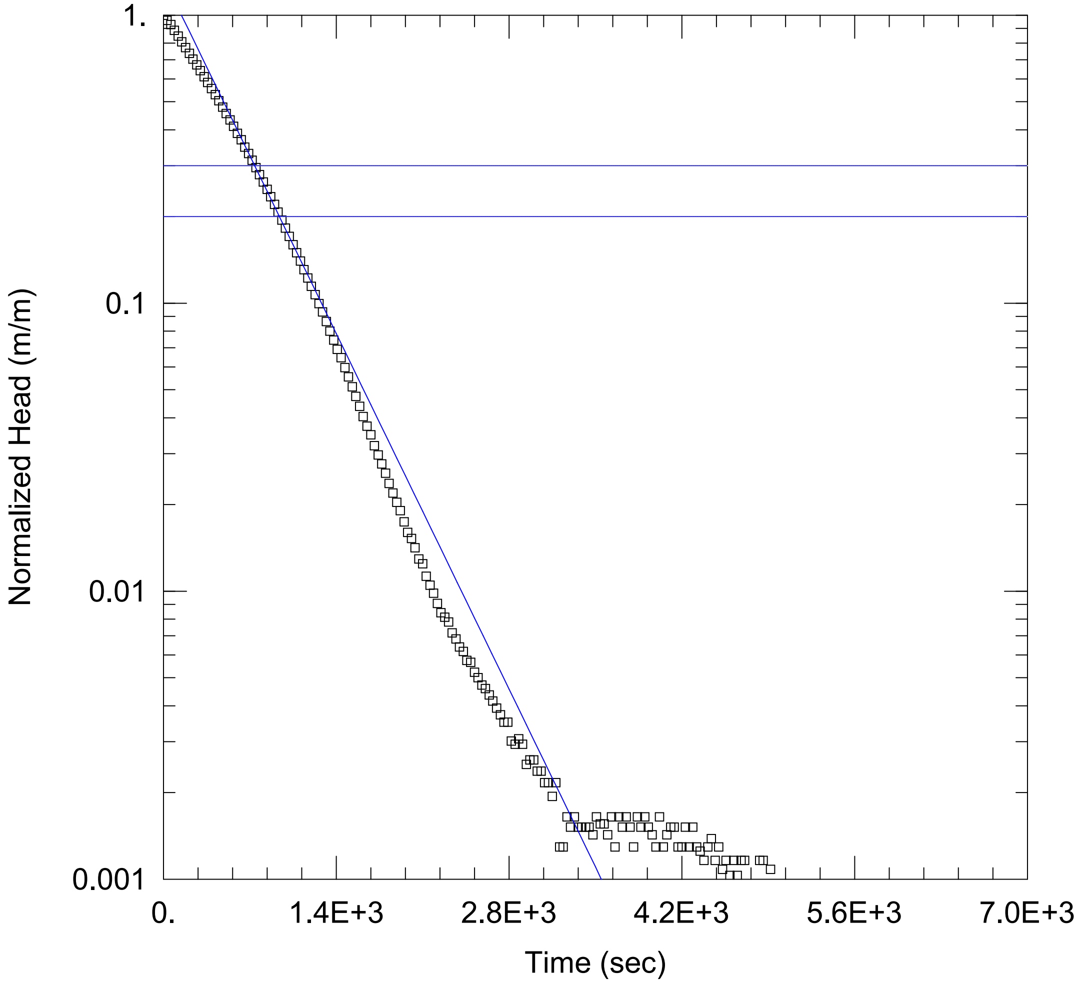
Tube recovered 53 cm of soil. Test sample portion taken from 15 cm to 25 cm from bottom end of tube.

REPORT DATE: 2023.May.23

REVIEWED BY GBeauce
Guillaume Beauce, P.Eng.
Geotechnical Engineer - Materials Testing Services

APPENDIX C

AQTESOLV Results



WELL TEST ANALYSIS

Data Set: C:\...\TH23-01 AQTESOLV results.aqt
Date: 06/02/23 Time: 09:38:16

PROJECT INFORMATION

Company: KGS
Client: City of Winnipeg
Project: 23-0107-005
Location: SWRC
Test Well: TH23-01
Test Date: May 16 2023

AQUIFER DATA

Saturated Thickness: 11.55 m
Anisotropy Ratio (Kz/Kr): 0.1

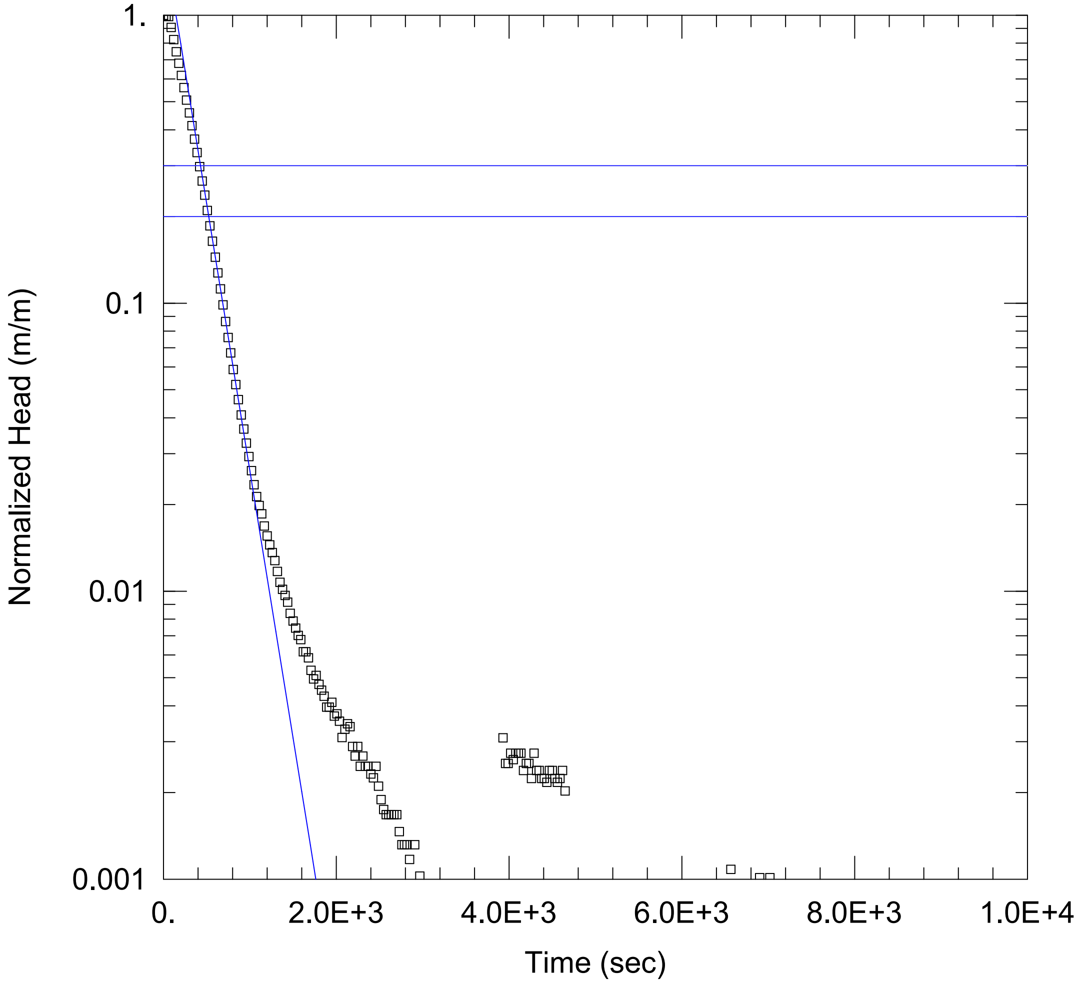
WELL DATA (TH23-01)

Initial Displacement: 9.904 m
Total Well Penetration Depth: 11.55 m
Casing Radius: 0.0254 m
Static Water Column Height: 11.55 m
Screen Length: 3.048 m
Well Radius: 0.0254 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 1.218E-6 m/sec
y0 = 13.32 m



WELL TEST ANALYSIS

Data Set: C:\...\TH23-03 AQTESOLV results.aqt

Date: 06/02/23 Time: 09:43:02

PROJECT INFORMATION

Company: KGS
Client: City of Winnipeg
Project: 23-0107-005
Location: SWRC
Test Well: TH23-03
Test Date: May 15 2023

AQUIFER DATA

Saturated Thickness: 11.3 m
Anisotropy Ratio (Kz/Kr): 0.1

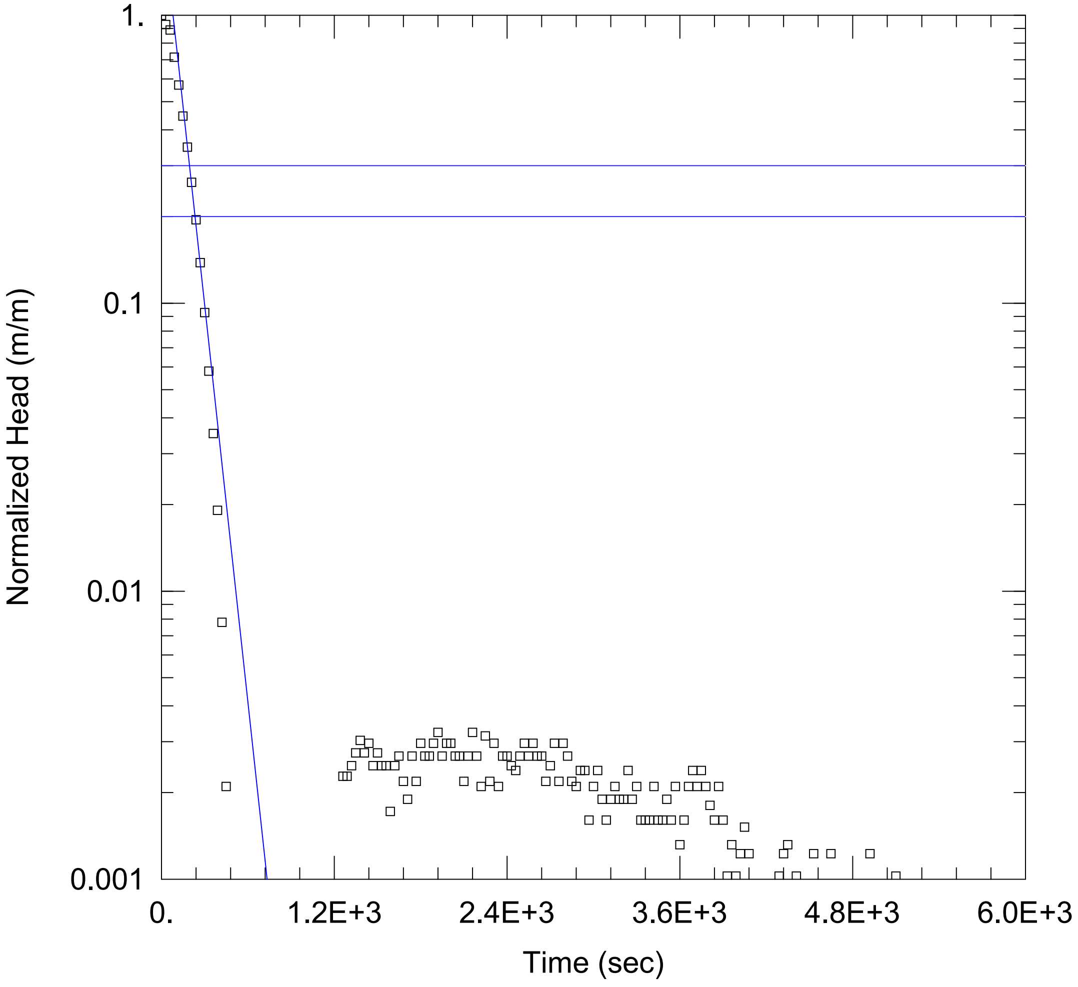
WELL DATA (TH23-03)

Initial Displacement: 8.89 m
Total Well Penetration Depth: 11.3 m
Casing Radius: 0.0254 m
Static Water Column Height: 11.3 m
Screen Length: 3.048 m
Well Radius: 0.0254 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 2.555E-6 m/sec
y0 = 16.63 m



WELL TEST ANALYSIS

Data Set: C:\...\TH23-04 AQTESOLV.aqt
Date: 06/02/23 Time: 09:54:22

PROJECT INFORMATION

Company: KGS
Client: City of Winnipeg
Project: 23-0107-005
Location: SWRC
Test Well: TH23-04
Test Date: May 15 2023

AQUIFER DATA

Saturated Thickness: 11.14 m
Anisotropy Ratio (Kz/Kr): 0.1

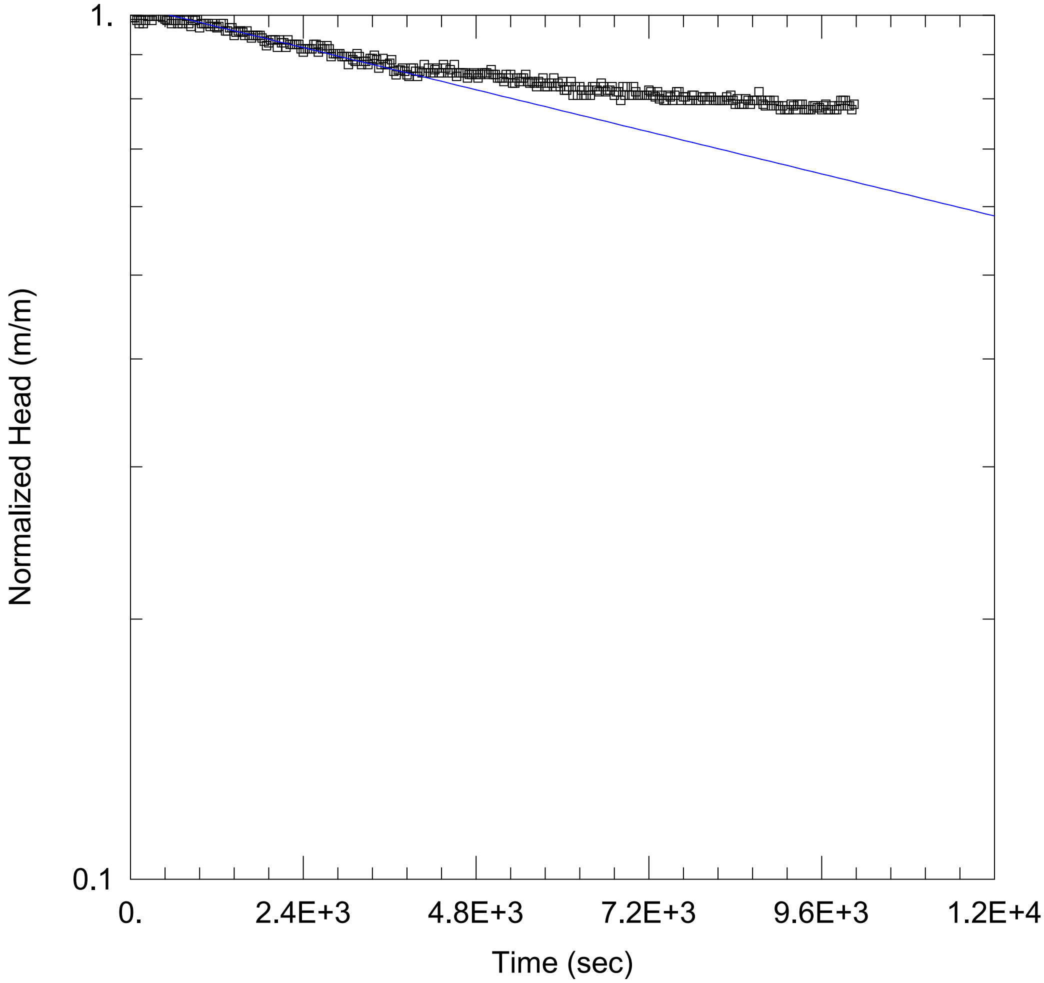
WELL DATA (TH23-04)

Initial Displacement: 4.486 m
Static Water Column Height: 11.14 m
Total Well Penetration Depth: 11.14 m
Screen Length: 1.524 m
Casing Radius: 0.025 m
Well Radius: 0.025 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 1.162E-5 m/sec
y0 = 10.55 m



WELL TEST ANALYSIS

Data Set: C:\...\TH23-05-PZ AQTESOLV.aqt
Date: 06/02/23
Time: 10:47:24

PROJECT INFORMATION

Company: KGS
Client: City of Winnipeg
Project: 23-0107-005
Location: SWRC
Test Well: TH23-05-PZ
Test Date: May 16 2023

AQUIFER DATA

Saturated Thickness: 0.257 m
Anisotropy Ratio (Kz/Kr): 0.1

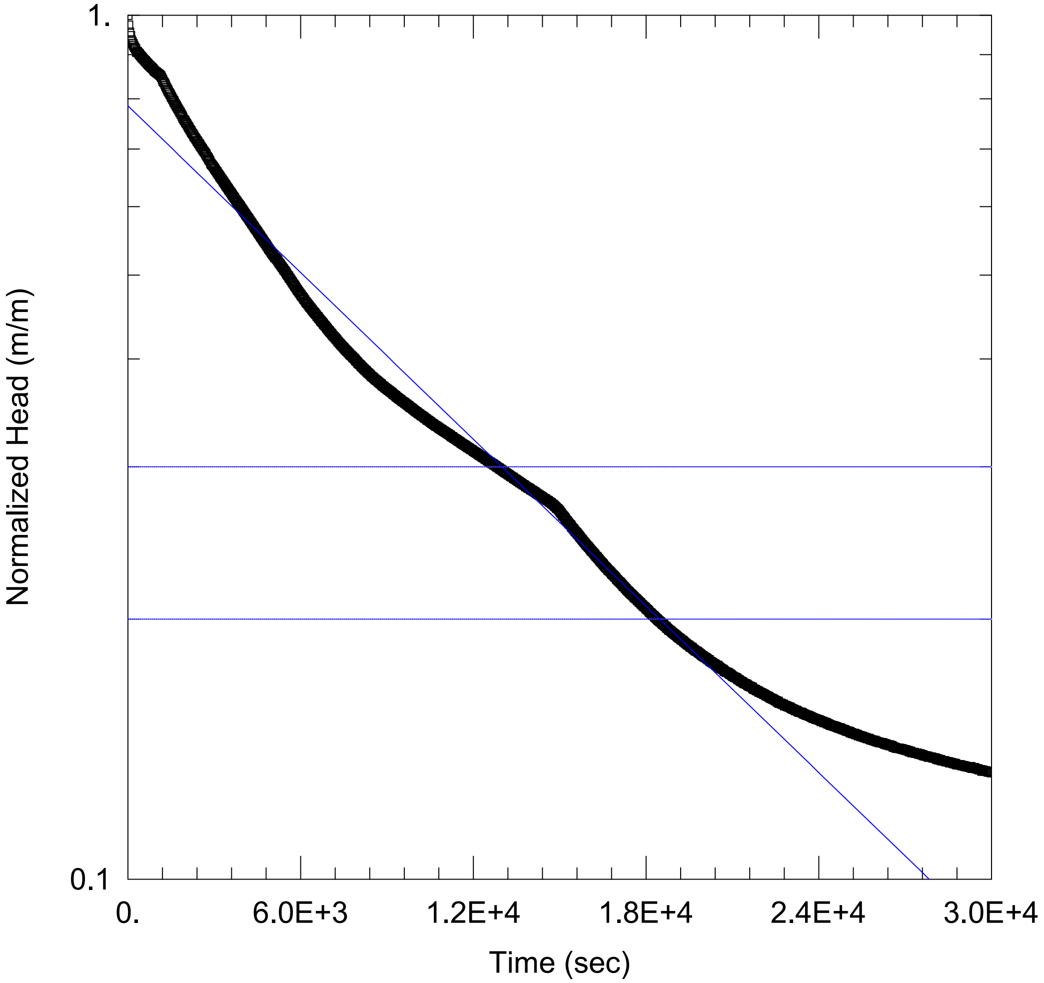
WELL DATA (TH23-05-PZ)

Initial Displacement: 0.168 m
Static Water Column Height: 0.257 m
Total Well Penetration Depth: 0.257 m
Screen Length: 0.257 m
Casing Radius: 0.025 m
Well Radius: 0.025 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 9.494E-8 m/sec
y0 = 0.1723 m



WELL TEST ANALYSIS

Data Set: C:\...\TH23-06 AQTESOLV.aqt
Date: 06/02/23 Time: 10:35:09

PROJECT INFORMATION

Company: KGS
Client: City of Winnipeg
Project: 23-0107-005
Location: SWRC
Test Well: TH23-06
Test Date: May 12 2023

AQUIFER DATA

Saturated Thickness: 12.97 m Anisotropy Ratio (Kz/Kr): 0.1

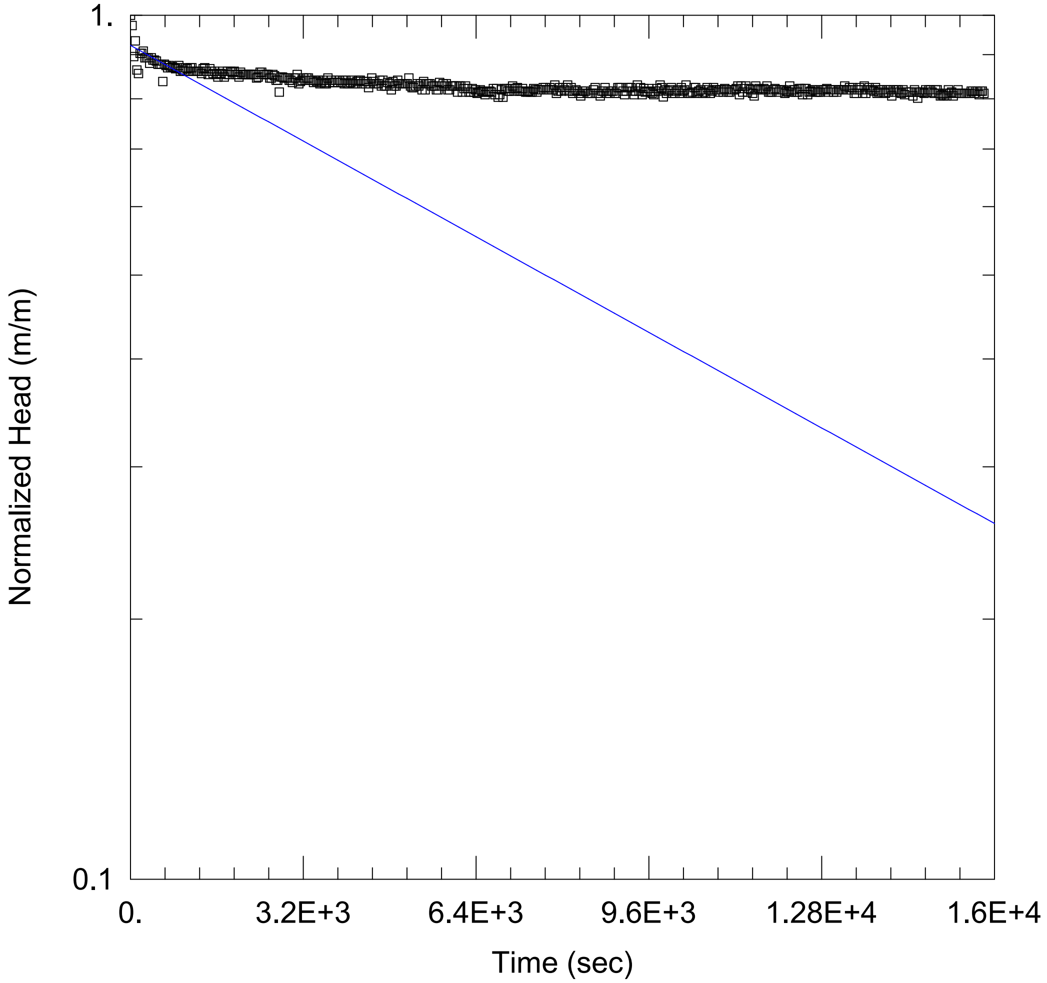
WELL DATA (TH23-06)

Initial Displacement: 12.01 m Static Water Column Height: 12.97 m
Total Well Penetration Depth: 12.97 m Screen Length: 3.048 m
Casing Radius: 0.025 m Well Radius: 0.025 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 4.373E-8 m/sec y0 = 9.427 m



WELL TEST ANALYSIS

Data Set: C:\...\TH23-06-PZ.aqt
Date: 06/02/23 Time: 10:45:06

PROJECT INFORMATION

Company: KGS
Client: City of Winnipeg
Project: 23-0107-005
Location: SWRC
Test Well: TH23-06-PZ
Test Date: May 12 2023

AQUIFER DATA

Saturated Thickness: 0.254 m Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (TH26-06-PZ)

Initial Displacement: 0.243 m
Total Well Penetration Depth: 0.254 m
Casing Radius: 0.025 m
Static Water Column Height: 0.254 m
Screen Length: 0.254 m
Well Radius: 0.025 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 1.629E-7 m/sec y0 = 0.2241 m

KGS
GROUP

Experience in Action