KGS GROUP
CONSULTING ENGINEERS

PROGRESSIVE REAL ESTATE

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## NORMAND AVENUE PROPERTY DEVELOPMENT
## PRELIMINARY GEOTECHNICAL ASSESSMENT REPORT

**FINAL**

KGS Group 18-3405-001

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# TABLE OF CONTENTS

1.0 INTRODUCTION ..... 1

2.0 BACKGROUND REVIEW ..... 2
2.1 SITE CONDITIONS ..... 2
2.1.1 FLOOD PROTECTION LEVEL ..... 3
2.1.2 PROPOSED DEVELOPMENT ..... 3

3.0 INVESTIGATION PROGRAM ..... 5
3.1 TEST HOLE DRILLING AND SOIL SAMPLING PROGRAM ..... 5
3.2 INSTRUMENTATION ..... 6
3.3 SITE STRATIGRAPHY ..... 7
3.4 GROUNDWATER CONDITIONS ..... 8
3.5 SLOPE INCLINOMETER RESULTS ..... 10
3.6 POTENTIAL DIFFICULT GROUND CONDITIONS ..... 10

4.0 SLOPE STABILITY ANALYSIS ..... 12
4.1 CONCEPTUAL STABILITY ANALYSIS ..... 12
4.2 PRELIMINARY STABILITY ANALYSIS ..... 14

5.0 FLOODPROOFING CONSIDERATIONS ..... 17
5.1 PRELIMINARY COST OF FLOODPROOFING ..... 18
5.2 ADDITIONAL FLOODPROOFING CONSIDERATIONS ..... 19

6.0 FOUNDATION CONSIDERATIONS ..... 21
6.1 CAST-IN-PLACE CONCRETE PILES ..... 22
6.2 DRIVEN PRE-STRESSED PRE-CAST CONCRETE PILES ..... 24
6.3 ADDITIONAL PILE FOUNDATION RECOMMENDATIONS ..... 26

7.0 SITE DEVELOPMENT CONSIDERATIONS ..... 28
7.1 FLOOR SLABS ..... 28
7.2 FROST PENETRATION ..... 29
7.3 BACKFILL ..... 30
7.4 TEMPORARY CONSTRUCTION EXCAVATIONS AND SHORING ..... 30
7.5 SITE SURFACE DRAINAGE ..... 31
7.6 SITE SUBSURFACE DRAINAGE ..... 31
7.7 TYPE OF CEMENT FOR CONCRETE MIX ..... 32
7.8 SEISMIC SITE CLASSIFICATION ..... 32

8.0 REGULATORY AND ENVIRONMENTAL CONSIDERATIONS ..... 33

9.0 CONCLUSIONS ..... 36

10.0 RECOMMENDATIONS ..... 38

11.0 STATEMENT OF LIMITATIONS ..... 39
11.1 THIRD PARTY USE OF REPORT ..... 39
11.2 GEOTECHNICAL INVESTIGATION STATEMENT OF LIMITATIONS ..... 39

TABLES
FIGURES
APPENDICES

## LIST OF TABLES

1 Approximate Test Hole Coordinates and Depths
2 Piezometric Monitoring Data
3 Estimated Unit Shaft (Skin) Friction Resistance for Limit States Design
4 Estimated End Bearing Resistance for Limit States Design
5 Allowable Load Capacities for Pre-stressed Pre-cast Driven Concrete Piles

## LIST OF FIGURES

1 Test Hole Location Plan
2 Critical Section Slope Stability Sensitivity Analysis Results

## LIST OF DRAWINGS

18-3405-001 G01 Proposed Flood Protection Works Preliminary Plan and Sections

## LIST OF APPENDICES

A Test Hole Logs
B Conceptual Slope Stability Analysis Results
C Preliminary Slope Stability Analysis Results
D Floodway Fringe Area Regulations

# 1.0 INTRODUCTION

Kontzamanis Graumann Smith MacMillan Inc. (KGS Group) was authorized by Progressive Real Estate to complete a preliminary geotechnical investigation, conceptual flood protection system, and foundation assessment for the proposed development at Normand Avenue.

The proposed development is located along the north side of Normand Avenue, west of St. Mary's Road. It consists of five (5) two-storey townhouse blocks, two (2) four-storey apartment blocks, and three (3) six-storey apartment blocks, all with below grade parking as illustrated in Drawing 18-3405-001 G01. The proposed development is bordered to the north by Henteleff Park and Normand Creek, and is located within the Designated Floodway Fringe Area. The development must therefore satisfy the requirements of the Designated Floodway Fringe Area Regulation (Regulation 266/91) in a manner that also safeguards it from potential bank movements along Normand Creek. KGS Group was therefore tasked with developing flood protection alternatives and foundation recommendations for the proposed development that also satisfy typical slope stability criteria.

KGS Group's scope of work for this assignment included the following:

- Review background information;
- Describe site conditions;
- Perform a drilling and sampling program;
- Complete a preliminary slope stability assessment of Normand Creek;
- Identify offset limits from Normand Creek that would result in no significant slope stability improvement requirements;
- Identify potential slope stability improvement options to accommodate building proximal to Normand Creek;
- Prepare a conceptual flood protection plan and provide preliminary construction costs for flood proofing options;
- Complete an engineering analysis and design to evaluate alternative foundation design options.

## 2.0 BACKGROUND REVIEW

KGS Group has reviewed drawings of the proposed development prepared by Affinity Architecture Inc., Designated Floodway Area maps of the proposed development area provided by the City of Winnipeg, 2011 LiDAR data, and updated topographic data provided by Barnes and Duncan. KGS Group first visited the site on June 20, 2018 to examine the site and we have completed numerous follow-up visits to complete geotechnical drilling investigations and obtain supplemental topographic survey data.

### 2.1 SITE CONDITIONS

The proposed development is 7.5 acres± in size and located west of St. Mary's Road. The site itself is bounded by public greenspace and residential development on its south and west sides and by Henteleff Park to the north. There are six (6) existing one or two storey dwellings at the site as well as numerous garages, sheds and other secondary structures. The Normand Creek channel runs parallel to the north side of the proposed development and meanders across the northern property line onto the proposed development for a length of 110 m± at the middle third of the site, as illustrated in Drawing 18-3405-001 G01. The creek channel turns north towards the Red River partway through the site such that the western third of the site is well offset from the Creek channel.

The Normand Creek channel does not always contain waterflow as evidenced by the channel being dry at the time of KGS Group's first site inspection (June 20, 2018). The Normand Creek channel varies in its overall dimensions from one end of the site to the other. On the east end, the creek channel bottom varies from Elev. 227.1 m to 227.5 m and the bank varies in height from 3.3 m to 3.6 m. Creek channel slopes in this area vary from 2.8H:1V to 3.4H:1V. Towards the west end of the site, the creek channel bottom deepens to Elev. 226.1 m, the overall bank height increases to up to 5 m. The creek channel slopes also steepen to 2H:1V. The overall geometry of the creek bank is steeper on the west side of the creek channel before it turns north; however, there did not appear to be evidence of recent slope movements in this area at the time of the site walkthrough. There appeared to be greater evidence of potential slope instability (i.e. toe slumping, backwards leaning trees, ground subsidence) on the east side of the site, which may explain the shallower overall slopes in this area. The creek itself is lined with

regular mature trees and has a discontinuous chain link fence along parts of the edge of bank. Observations of miscellaneous fill and concrete debris within the creek channel also indicate that the creek has experienced previous disruption.

### 2.1.1 FLOOD PROTECTION LEVEL

The flood protection level (FPL) for the site is Elev. 231.85 m which is up to 1.75 m± higher than the edge of bank of the Normand Creek channel (the apparent edge of bank elevation varies from Elev. 230.1 m to 231 m). Ground elevation is lowest near the west side of the site at Elev. 229.75 m± (2.1 m below FPL) approximately 150 m west of Normand Creek.

### 2.1.2 PROPOSED DEVELOPMENT

The proposed development consists of five (5) two-storey townhouses (Buildings C-1 to C-5), two (2) four-storey apartment blocks (Buildings B-1 and B-2), and three (3) six-storey apartment blocks (Buildings A-1 to A-3) as illustrated in Drawing 18-3405-001 G01. The four-storey and six-story apartment blocks are located on the east side of the proposed development in close proximity to the Normand Creek channel. The two-story townhouses are proposed on the west side of the site, further offset from the Normand Creek channel. The four-storey and six-storey apartment blocks are proposed to have underground parking.

The four-storey and six-storey apartments would be founded on deep piles, end-bearing on till due to proximity of these structures to the creek channel. The two-story townhouses would be founded on either friction piles or end-bearing piles based on economy. Further discussion on foundation considerations for the proposed development is provided in the Slope Stability Analysis (Section 4.0) and Foundation Considerations (Section 6.0) sections of this report.

The main floor elevation of the proposed four-storey and six-storey apartments is proposed to be at Elev. 232.85 m (1 m above FPL), consistent with Schedule C of the Designated Floodway Fringe Area Regulation. The main floor elevation of the two-storey townhouses is proposed to be at Elev. 232.15 m (0.3 m above FPL), consistent with Schedule A of the Designated Floodway Fringe Area Regulations. The proposed development would satisfy the intent of the

Designated Floodway Fringe Area Regulations by means of a combination of implementation of Schedule A and Schedule C. Further discussion on floodproofing for the proposed development is provided in the subsequent floodproofing considerations section of this report (Section 5.0).

# 3.0 INVESTIGATION PROGRAM

## 3.1 TEST HOLE DRILLING AND SOIL SAMPLING PROGRAM

A drilling and sampling program was completed at the site of the proposed development on August 7, 2018. Drilling services were provided by Maple Leaf Drilling Ltd. and the investigation proceeded under continuous KGS Group supervision. The investigation consisted of drilling three (3) test holes (TH18-01, TH18-02 and TH18-03) to refusal in till. Test hole refusal depth varied from 15.3 m± to 17.8 m± below grade. TH18-01 was drilled within the footprint of Building A-1, closest to St. Mary's Road. TH18-02 was drilled in the footprint of building B-1, at the top of the bank near the highest, steepest section of the Normand Creek channel. TH18-03 was drilled within the footprint of the townhouse C-2 on the west side of the development.

The test holes were completed using a Mobile Drill B54X track mounted drill rig equipped with 125 mm diameter solid stem continuous flight augers. Representative disturbed soil samples were obtained in each test hole at 1.5 m (5 ft) intervals, or at any change in soil strata. Soil samples were collected directly off the auger flights and visually classified in the field in accordance with the modified Unified Soil Classification System (USCS). Cohesive samples were tested with a Field Torvane to evaluate consistency and estimate the undrained shear strength. Standard Penetration Tests (SPTs) was performed in the till at 1.5 m (5 ft) intervals to determine the relative in-situ density.

Upon completion of drilling, the test holes were examined for indications of sloughing, squeezing and seepage and then backfilled to grade. Test hole locations are shown in plan on Figure 3.1 and on Drawing 18-3405-001 G01. Approximate UTM coordinates (Zone 14) for the test holes are provided in Table 3.1. Summary soil logs incorporating all field observations are provided in Appendix A.

**FIGURE 1**
**TEST HOLE LOCATION PLAN**




**TABLE 3.1**
**APPROXIMATE TEST HOLE COORDINATES AND DEPTHS**

| Test Hole ID | Approximate UTM Coordinates | | Elevation [m] | | |
|---|---|---|---|---|---|
| | Northing | Easting | Ground | Top of Till | Auger Refusal |
| TH18-01 | 5,519,179 | 635,695 | 231.99 | 217.2 | 214.16 |
| TH18-02 | 5,519,061 | 635,492 | 230.93 | 215.4 | 214.42 |
| TH18-03 | 5,518,978 | 635,341 | 229.83 | 216.6 | 214.46 |

## 3.2 INSTRUMENTATION

Six (6) pneumatic piezometers, two (2) Casagrande-tipped standpipe piezometers and one (1) slope inclinometer were installed in the test holes after completion of drilling as follows:

- TH18-01: Installed one (1) pneumatic piezometer and one (1) standpipe piezometer.
- TH18-02: Installed three (3) pneumatic piezometers and one (1) slope inclinometer
- TH18-03: Installed two (2) pneumatic piezometers and one (1) standpipe piezometer.

Pneumatic piezometers were attached directly to the standpipe piezometers in test holes TH18-01 and TH18-03. These test holes were backfilled with sand surrounding the Casagrande tips and with bentonite chips to grade. In test hole TH18-02, the pneumatic piezometers were attached directly to the slope inclinometer and the test hole was backfilled completely with cement-bentonite grout. All instrumentation details are illustrated in the test hole logs in Appendix A.

## 3.3 SITE STRATIGRAPHY

The general stratigraphy at the site was interpreted to consist of clay fill over glacio-lacustrine clay of high plasticity over glacial till. The composition of the uppermost soil layer (typically fill) varied across the site. Test hole TH18-01 was observed to have the most complex arrangement of shallow soil layers, test hole TH18-02 was observed to have the most consistent layer of clay fill, and test hole TH18-03 did not appear to have any fill at surface.

***Sand and Gravel Fill***

Sand and Gravel Fill was encountered at grade in TH18-01 and extended to a depth of 0.5 m±. The granular fill was dark grey, damp, compact, and varied in gradation from medium to coarse grained sand to fine grained gravel. This fill was also observed to have significant clay content.

***Topsoil***

Topsoil was encountered below the sand and gravel fill in TH18-01. The topsoil was observed to be black, loose, damp, and contained organics. The approximate thickness of the topsoil layer was 0.1 m±

***Clay Fill***

Clay fill was observed at grade in TH18-02 to a depth of approximately of 3 m± and as a 0.5 m± thick layer below topsoil in TH18-01. In TH18-02, the clay fill was dark brown, friable, damp, and contained trace medium to coarse grained sand as well as trace fine to coarse grained gravel. There was an apparent increase in moisture content with depth and increase in silt and gypsum nodules. In TH18-03, the clay fill was black, damp to moist, very stiff, and of intermediate to high plasticity. Trace fine grained gravel and trace rootlets were also observed.

***Silt***

A silt layer was encountered at a depth of 1.4 m in TH18-01. The silt was tan in colour, moist, soft, and of low plasticity. The approximate thickness of the silt layer was 0.4 m±.

***Clay (CH)***

High plasticity clay of lacustrine origin was encountered below the fill layers in test holes TH18-01 and TH18-0-2 and from below ground surface in test hole TH18-03. The clay extended to approximately 13.3 m± to 15.5 m± below grade (Elevation 215.4 m to 217.2 m). The clay was brown to dark brown in colour, damp to moist, stiff, of high plasticity, contained trace silt and gypsum inclusions, and trace fine to coarse grained gravel.

The undrained shear strength of the silty clay deposit, as determined from the field Torvane on disturbed auger cutting samples, was found to decrease with depth ranging from 45 to >100 kPa in the upper portions of the clay and 15 kPa to 45 kPa in the lower clay.

***Silt Till***

Glacial silt till was observed below the high plasticity clay at approximately 13.3 m± to 15.5 m± depth below grade (Elevation 215.4 m to 217.2 m). The silt till was light brown to grey, moist to wet, compact to dense, contained some fine to coarse grained sand, and contained trace fine to coarse grained gravel. The relative density of the till was observed to increase with depth. Power auger refusal was achieved in the dense silt till at 15.4 m± and 17.8 m± depth below grade (Elevation 214.16 m to 214.46 m).

Uncorrected blow counts “N” from Standard Penetration Tests (SPT) in the till varied from 7 blows per 300 mm at the transition from the overlying silty clay to greater than 50 blows per 300 mm within the dense till.

## 3.4 GROUNDWATER CONDITIONS

Seepage and sloughing conditions were monitored throughout the drilling investigation. Minor test hole sloughing was observed in the test holes after completion of drilling (varying from 0.2 m± to 0.5 m± above the fully drilled test hole depths). Groundwater infiltration was observed when drilling test hole TH18-03 below a depth of 5.2 m± (Elev. 224.6 m) and the groundwater

level was at the same depth in the test hole following completion of the drilling and prior to backfilling the test hole. A trace amount of water was found in TH18-01 immediately after drilling and TH18-02 was dry immediately after drilling.

Six (6) pneumatic piezometers and two (2) standpipes were installed at the site with a protective above-ground casing. Table 3.2 summarizes groundwater monitoring information to date.

**TABLE 3.2**
**PIEZOMETRIC MONITORING DATA**

| Test Hole | TH18-01 | | TH18-02 | | | TH18-03 | | |
|---|---|---|---|---|---|---|---|---|
| Ground Elevation [m] | 231.99 | | 230.93 | | | 229.83 | | |
| Serial No. | 037845 | STP | 037700 | 037936 | 037846 | 037701 | 037847 | STP |
| Tip Elevation [m] | 224.37 | 215.84 | 224.99 | 217.37 | 214.93 | 223.73 | 219.16 | 214.59 |
| Monitoring Zone | Silty Clay | Silt Till | Silty Clay | Silty Clay | Silt Till | Silty Clay | Silty Clay | Silt Till |
| Data | Piezometric Elevation [m] | | | | | | | |
| 09-Aug-18 | 224.37 | 224.30 | 225.21 | 225.11 | 225.46 | 226.32 | 225.13 | 217.51 |
| 13-Aug-18 | 224.41 | 224.33 | 225.29 | 225.40 | 225.25 | 226.32 | 225.33 | 219.57 |
| 17-Aug-18 | 224.41 | 224.24 | 225.29 | 225.54 | 225.32 | 226.53 | 225.13 | 221.14 |
| 24-Aug-18 | 224.37 | 224.27 | 225.36 | 225.40 | 225.75 | 226.18 | 225.61 | 222.77 |
| 27-Aug-18 | 224.52 | 224.28 | 225.39 | 225.40 | 225.68 | 226.18 | 226.03 | 223.22 |

Although the groundwater monitoring period to date is relatively short (20 days), the groundwater levels measured in most of the pneumatic piezometers and the standpipe piezometer in TH18-01 now appear to fluctuate minimally with relatively consistent readings. This suggests that most of the piezometers may have stabilized and are now reading representative groundwater levels at the time of the readings. At the same time, the standpipe piezometer installed in TH18-03 and the lower pneumatic piezometer (037847) may still be developing equilibrium with the actual porewater pressures at the location of the instruments; this is based on the observed continual and substantial rate of change in the readings during the short monitoring period. The readings taken to date in the lower piezometers in TH18-03 may therefore not yet be representative of the true porewater pressures at the time of the readings at

the instrument locations. Based on the piezometric readings in instruments that are considered stabilized, the highest groundwater levels are located on the west side of the site (near TH18-03) and the lowest are on the east side of the site (near TH18-01). Where readings are considered stabilized, there currently appears to be a static gradient at the site with respect to upward / downward groundwater movement.

Groundwater levels will also fluctuate seasonally and following precipitation events. Due to the relatively short groundwater monitoring period, the actual groundwater levels at the time of construction may differ from the conditions documented in this report.

### 3.5 SLOPE INCLINOMETER RESULTS

A slope inclinometer was installed at the top of the slope at TH18-02 to a depth of 16.31 m below ground surface into the till. Only baseline readings have been taken to date and monitoring will continue into the fall.

### 3.6 POTENTIAL DIFFICULT GROUND CONDITIONS

Observed seepage, squeezing and sloughing conditions throughout the drilling investigation were considered minor. Groundwater inflows and subsequent sloughing may have to be dealt with during construction depending on the type of foundation being installed and whether groundwater conditions differ from those encountered during the investigation at the test hole locations. In the event that water inflows are encountered, water inflows will have to controlled through means as determined by the foundation contractor, which may include such methods as pumping equipment of sufficient capacity and / or through the use of casing, or sheet piling or shallow temporary cut slopes on excavation faces. Shoring or temporary cut slopes greater than 1.5 m deep or below the water table must be designed by a qualified geotechnical engineer.

In KGS Group’s experience, sporadic and irregular zones of cobbles, boulders and/or granular layers have been frequently encountered within till deposits and may be encountered at this site. These zones can and should be expected to be water bearing, which may cause difficulties when advancing bored piles or caissons through the till. The cobbles and boulders would need

to be removed when encountered within foundation excavations. Contractors should anticipate and have readily available full length sleeves for the installation of any bored piles.

# 4.0 SLOPE STABILITY ANALYSIS

Conceptual and preliminary slope stability analyses were completed for the proposed property development prior to creating the proposed flood protection configuration illustrated in Drawing 18-3405-001 G01. These early iterations of slope stability analyses were focused on defining a development offset line from the Normand Creek channel such that if the four-storey and six-storey apartment structures (and any associated fill) could be located behind the development offset line, no slope stabilization works would be required to achieve an adequate factor of safety against the impacts of potential slope movements would be achieved. The criteria for the slope stability modelling was that a final development offset line would ensure that the proposed property development would demonstrate a minimum estimated safety factor (FS) of 1.3 under normal groundwater conditions and an estimated FS of 1.1 under extreme, fully saturated groundwater conditions, and without slope stabilization works. Analytically, an estimated FS of 1.0 is unstable.

Ultimately, it was found that the location of a development offset line, assuming typical soil shear strength parameters, would preclude the feasibility of the proposed development without incorporating a tie-back sheetpile wall, embedded in till, as illustrated in Drawing 18-3405-001 G01.

The stability analyses were conducted using the two-dimensional computer modeling program SLOPE/W, developed by GeoSlope International Ltd. from Calgary, Alberta. The modeling program is capable of analyzing numerous iterations of both circular and composite slip surfaces to identify analytically the worst-case slip surface. The Morgenstern-Price Method of Slices with a half sine wave inter-slice force function was applied to all calculations reported herein. Static groundwater conditions were assumed in the analysis.

## 4.1 CONCEPTUAL STABILITY ANALYSIS

A conceptual level slope stability analysis was first completed prior to the drilling investigation using: 2011 LiDAR data, estimated depths to till, and groundwater conditions that varied from full bank saturation to 2 m below grade. The goal was determine if the proposed development could accommodate a best case offset line (Case 1), a worst case offset line (Case 2), or

neither cases with the understanding that the results would be refined if necessary once the drilling investigation was complete. The results of the analysis were presented to Progressive Real Estate on August 2, 2018. A sketch illustrating the proposed development modifications at this time, assuming that Case 1 conditions would prevail across the entire site, is included in Appendix B (Sheet 1). A plan illustrating the locations of cross-sections used to establish the Case 1 and Case 2 offset lines is included in Appendix B – Sheet 2 and the slope stability cross-sections are included in Appendix B – Sheets 3 to 10.

In total, eight (8) cross-sections were analyzed, two (2) soil shear strength conditions were considered (Case 1 and Case 2), and two (2) groundwater conditions were examined. The locations of the cross-sections are illustrated in Appendix B – Sheet 2 along with critical safety adjacent to each cross-section. The critical safety factors in Sheet 2 illustrate that the most critical bank geometry occurs at the location of Section 5 (Appendix B – Sheet 6). Both the Case 1 and Case 2 offset lines illustrate the point where fill could be placed up to Elev. 232 m for their respective assumed soil strength conditions and still achieve a minimum FS of 1.1 under extreme saturated conditions and FS of 1.3 under the assumed normal groundwater conditions at the time of the conceptual study (2 m below grade).

The Case 1 model assumed intact soil strength parameters ($\phi$' = 15° and c' = 5 kPa) and the Case 2 model assumed residual strength shear strength parameters ($\phi$' = 10° and c' = 5 kPa). Fill was modelled as a surcharge load of 20 kN/m$^3$. Normal groundwater conditions were assumed to be 2 m below grade and establishment of offsets based on this condition was required to satisfy a minimum FS of 1.3. Due to uncertainties in groundwater levels at this time, a second fully saturated condition was analyzed to establish fill offsets for this extreme condition. This extreme condition was required to satisfy an FS of 1.1.

Satisfying the criteria for the extreme saturated groundwater condition (minimum FS = 1.1) was found to govern the establishment of the fill offset lines for both Case 1 and Case 2 shear strength parameters. Only extreme saturated conditions are illustrated in the Appendix B cross-sections for that reason. Depending on the creek geometry of a particular cross-section, the difference in offset between Case 1 (best case) and Case 2 (worst case) conditions varied from 10 m to 15 m.

## 4.2 PRELIMINARY STABILITY ANALYSIS

After the geotechnical investigation was completed on August 7, 2018 and the site was thoroughly examined for signs of slope instability, KGS Group refined the stability analyses and the associated effective soil shear strength parameters that were originally assumed in the conceptual stability analysis. Additional topographic surveying was also completed to examine differences in the site geometry at the locations of slope stability cross-sections between 2011 and 2018.

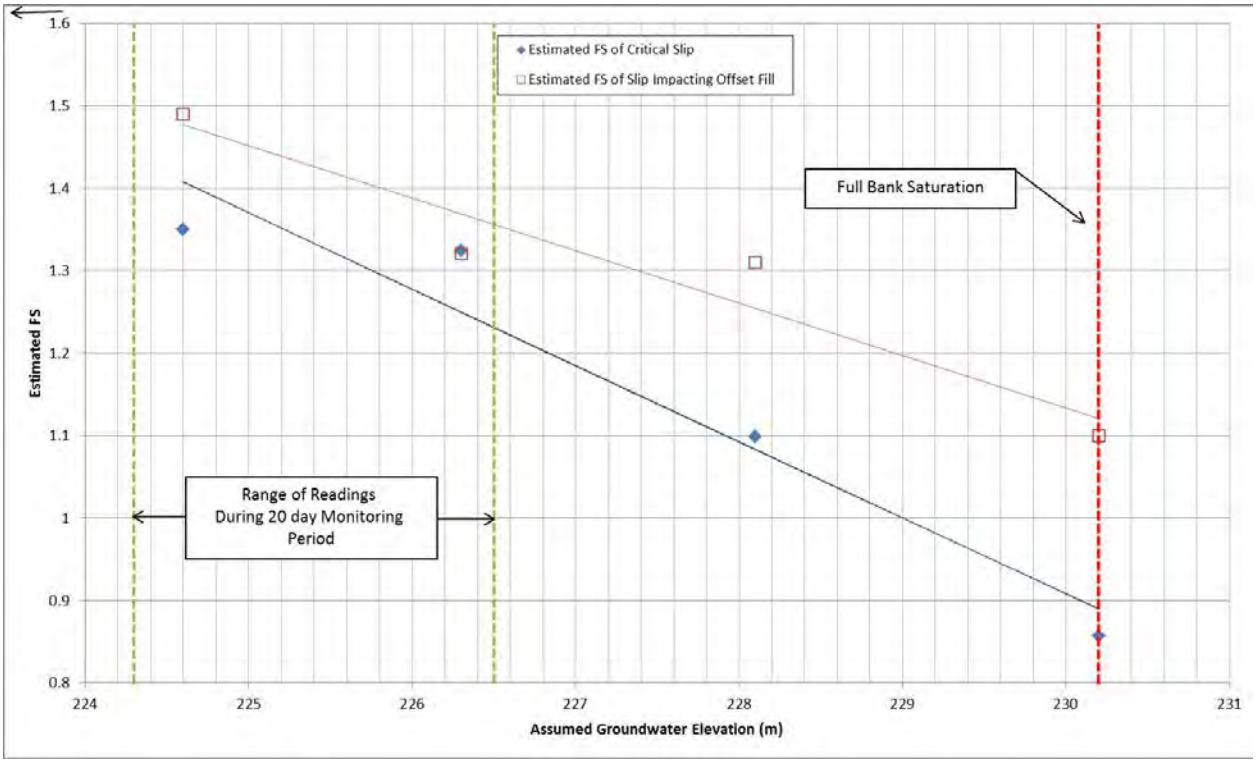
The models were refined relative to the conceptual analysis as follows:

- The intact (Case 1) effective shear strength parameters were refined to be $\phi'$ = 14° and c' = 5 kPa, consistent with shear strength parameters assumed for lacustrine clay based on extensive soil shear strength testing for design of the Red River Floodway Expansion and numerous other projects within the City of Winnipeg.
- Case 1 (intact strength) conditions were determined to be applicable to cross-sections 2, 3, 4, 5 and 6 where investigation of the Creek shoreline and comparison of survey data from 2011 and 2018 did not appear to indicate that there were signs of slope movements.
- A combination of Case 1 / Case 2 conditions (residual clay / intact clay distinction) were determined to be applicable to cross-sections 7, 8 and 9 where there did appear to be evidence of slope movements based on site observations, or where there appeared to be differences between 2011 and 2018 LiDAR data.
- Where significant differences did exist between 2011 and 2018 ground surfaces, cross-sections were updated to incorporate the results of the 2018 topographic survey.

The results of the preliminary stability analysis were communicated to Progressive Real Estate on August 13, 2018. A sketch illustrating a single recommended fill offset line for the proposed development is included in Appendix C – Sheet 1. A sensitivity analysis has also been completed varying groundwater levels at the location of the critical cross-section (Section 5 – Appendix B – Sheet 6) to better understand what safety factors may be based on the measured groundwater levels to date. The groundwater sensitivity analysis results are

illustrated in Figure 2. Groundwater levels at the site varied from Elev. 224.3 m to 226.5 m during the 20-day monitoring period. Assuming these groundwater levels in the stability analysis at this location, the corresponding safety factor of a critical slip surface varies from 1.25 to 1.35 and the safety factor of the first slip surface impacting fill beyond the slope stability offset line varies from 1.32 to 1.49.

**FIGURE 2**
**CRITICAL SECTION SLOPE STABILITYSENSITIVITY ANALYSIS RESULTS**




At the same time that the preliminary slope stability analysis was updated, it had become apparent that the concept of a development offset for this site was not feasible for the property development. Similarly, common slope stabilization alternatives such as rockfill shear key stabilization were not determined to be feasible for the proposed development as they would cause too great of a disturbance to the existing creek and they could not be consistently implemented in an effective manner due to Normand Creek meandering on both sides of the northern property line. Therefore, Appendix C – Sheet 1 also illustrates an early concept of the sheetpile retaining wall solution currently presented in Drawing 18-3405-001 for providing flood

protection consistent with the requirements of the Floodway Fringe Area Regulations. The means currently proposed for flood-proofing the proposed development are elaborated upon in Section 5.0.

Key concepts from the preliminary slope stability analysis that should be carried forward into detailed design of the proposed development are illustrated on Drawing 18-3045-001 G01 and summarized as follows:

- The proposed four-storey and six-storey apartment blocks should be founded on deep piles, end-bearing on till in order not to transfer loads to the Normand Creek bank.
- The proposed two-storey townhouses can be founded on cast-in-place friction piles in clay should they be desirable from the perspective of economy.
- Sheetpiles will be required to retain fill at the required FPL around the proposed development. These sheetpiles should be driven to refusal in the underlying till wherever they encroach upon the property development offset lines provided in Appendix C – Sheet 1. In these areas, sheetpile tie-back details and backfill requirements would be finalized as part of detailed design in order to prevent excessive deflection of the sheetpiles.
- There may be opportunities in the detailed design of the proposed development to optimize the type of retaining walls proposed near the proposed two-storey townhouses on the west side of the development and also on the east side of the proposed development. These are areas well-removed from Normand Creek, with and there is therefore much less concern with creek bank stability.

## 5.0 FLOODPROOFING CONSIDERATIONS

The property is located in the Designated Floodway Area (also known as the Floodway Fringe Area) and all development must therefore adhere to the requirements of Manitoba Regulation 266/91. This means that because this property is located outside the Primary Line of Defence (PLD), it must be developed at or above the Flood Protection Level (FPL). The Floodway Fringe Area Regulations are included in Appendix D for reference. The development will satisfy the intent of the floodproofing criteria through a combination of the implementation of Schedule A – Reference Regulation Section 7(2) – and Schedule C – Reference Regulation Section 7(4).

Several alternative flood protection preliminary design options were considered for the proposed development including: clay dike, double-sided segmental block wall dike, sheet pile wall, composite dike, flood wall, combination dike and site grading. In most cases, however, numerous options were precluded due to the following constraints:

- There is 15 ft (4.6m) to 18 ft (5.5 m) between the proposed structures and the northern property line. This relatively short distance limits the types of flood protection configurations that can be used to satisfy the intent of Floodway Regulation Schedule A. Schedule A requires a 4.6 m wide bench around the structure to provide access for emergency diking. Currently, the main floor elevation of the two-storey townhouses is recommended to be 0.3 m above FPL to satisfy these requirements. On Drawing 18-3405-001 G01, the intent is to provide a sheetpile retaining wall around the development that will allow for construction of a 4.6 m wide bench. There may be opportunities to optimize the type of retaining wall proposed in this location as part of detailed design as the townhouses are not proximal to the Normand Creek channel, which minimizes slope stability concerns. A mechanically stabilized earth (MSE) wall for example, may be a more cost-effective solution in this area.

- Based on the results of the preliminary slope stability assessment (Section 4.0), both the four-storey apartment blocks (B-1 and B-2) and one (1) of the six-storey apartment blocks (A-3), are located within the recommended offset line for fill placement. A

sheetpile wall, driven to refusal in the underlying till with appropriate backfill and tie-back considerations, is therefore required to ensure that future potential creek slope instability does not negatively impact the development. It is understood that Progressive Real Estate also wishes to avoid developing within Normand Creek itself, though a small portion of the Normand Creek channel, within the property limits, may require reshaping as a function of installing the proposed sheetpile wall. Moving away from the Normand Creek channel places the required location of a sheetpile retaining wall closer to the apartment buildings in some areas. The result is the that buildings A-3, B-1 and B-2 cannot satisfy the 4.6 m wide bench requirement of Schedule A, and the default must therefore be that the main floor elevation of these three (3) structures should be 1 m higher than FPL (consistent with Schedule C). This may also influence the main floor elevation of the other two (2) six-storey apartment blocks (A-1 and A-2) for aesthetic reasons (i.e. it may be preferable to provide a consistent main floor elevation 1 m higher than FPL for all four-storey and six-storey apartment structures).

- In between the proposed buildings, a minimum 4.6 m wide corridor must be provided such that an emergency dike can be constructed between the four-storey and six-storey apartment buildings. As part of detailed design, these in-between abutment areas should plan for the event that a 1 m high sandbag dike with 1.6 m base width should be able to abut against the buildings. The area in blue in Drawing 18-3405-001 G01 illustrates where a minimum 4.6 m wide bench is currently supplied for emergency diking in transition areas between structures that satisfy the floodproofing intents of Schedule A and Schedule C.

## 5.1 PRELIMINARY COST OF FLOODPROOFING

Drawing 18-3405-001 G01 illustrates that a sheetpile driven into dense till and protruding above ground to the site FPL (Elev. 231.85 m) will be approximately 18 m in total length (stick-up and embedment). For preliminary planning purposes, a typical cost to install a sheetpile of this length will range from $7,500 to $12,500 per lineal metre, depending on final tie-back requirements which are to be determined during detailed design. The minimum span of this deep sheetpile wall should extend across the limits of the slope stability offset line illustrated in Appendix C – Sheet 1. Near the peripheries of the proposed development, mechanically

stabilized earth (MSE) walls with adequate cut-off depths into high plasticity lacustrine clay can be considered during design to optimize flood-proofing costs. Walls of this nature for the retained fill heights anticipated typically cost between $500 to $700 per square metre of wall face.

## 5.2 ADDITIONAL FLOODPROOFING CONSIDERATIONS

Requirements of Manitoba Regulation 266/91 include, but are not limited to the requirements listed below. The regulations are included in Appendix D for reference.

Every structure other than an accessory structure shall meet the following requirements:

- If a structure with a basement supported by fill, then the main floor elevation must be 0.3 m above applicable FPL.
- If a structure with no basement supported by fill, then the main floor elevation must not be less than the applicable FPL and the top of the fill must not be more than 0.3 below the applicable FPL.
- If a structure is supported by piles, then the elevation of a floor containing finished or occupied space must be at least 1.0 m above applicable FPL.
- All windows, exterior doors and other exterior openings, concluding openings between a house and an attached garage, shall be located above the applicable FPL.
- The electrical service and panel board shall be located above the main floor.
- The potable water shut-off valve shall be located on the main floor with no take-off fittings on the service pipe side of the valve and shall have a dual check valve type backflow preventer to provide protection against possible contamination of the potable water supply.
- If the structure has floor space below the applicable FPL, the drain between the structure and a septic or holding tank or a common sanitary sewer line shall have a backwater valve.

A primary dyke and an extension of a primary dyke shall:

- be located entirely within the Floodway Fringe Area;
- be located, designed and constructed to the standard and elevation approved by the City Administrator and The Dyking Commissioner but in no case shall the elevation be less than the FPL or the top width be less than 9.2 m at the FPL;
- have adequate permanent works, as approved by the City administrator, for the removal of water as a result of internal drainage and seepage within the protected area; and
- have measures, as approved by the City administrator, that are operable during flood conditions to close openings through the primary dyke.

## 6.0 FOUNDATION CONSIDERATIONS

Deep foundations are recommended for all buildings in the proposed development. Shallow foundations are not considered a suitable foundation alternative. Shallow footing foundations bearing directly on either fill or overburden clay should be expected to perform poorly due to the expansive nature of the clay (the ability of the clay to swell or shrink under varying seasonal groundwater conditions), the potential differential settlements that would occur as a result. Furthermore, slope stability concerns are also present depending on the structure, as discussed in Section 4.0.

End-bearing, pre-stressed, pre-cast concrete piles driven to refusal in the till are recommended for all four-storey and six-storey apartment structures. End-bearing cast-in-place concrete piles in the till are also an option. The townhouses do not need to be founded on deep end-bearing piles, though they could be, if so desirable. Cast-in-place concrete friction piles (installed a minimum 7.5 m depth into native ground) would be recommended for the townhouses.

The foundation considerations described in this report follow Limit States Design (LSD) guidelines. Limit States Design requires consideration of two (2) main loading states: Ultimate Limit States and Serviceability Limit States. The Ultimate Limit States (ULS) are primarily concerned with collapse mechanisms of the structure and safety, and the Serviceability Limits States (SLS) present conditions or mechanisms that restrict or constrain the intended use, function or occupancy of the structure under expected service or working loads. The ULS values represent the nominal (ultimate) geotechnical resistance, $R_n$. A Geotechnical Resistance Factor ($\Phi$) of 0.4 should be applied to determine the factored geotechnical resistance as presented in in the following equation:

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

where:

| | |
|---|---|
| $\Phi$ | 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 |

## 6.1 CAST-IN-PLACE CONCRETE PILES

Cast-in-place concrete friction piles may be designed based upon the estimated Ultimate Limit State (ULS) and Serviceability Limit State (SLS) skin friction values provided in Table 3 for each test hole. A Geotechnical Resistance Factor (Φ) of 0.4 has been applied to the estimated average factored resistance values. Piles that are designed to be friction piles should be designed to resist the load by shaft resistance only. The contribution from end bearing should be considered nominal and should be disregarded in the pile capacity calculations.

**TABLE 3**
**ESTIMATED UNIT SHAFT (SKIN) FRICTION RESISTANCE FOR LIMIT STATES DESIGN***

| Location | Depth Below Finished Subgrade | Serviceability Limit State, SLS (kPa) | Unfactored Ultimate Limit State, ULS (kPa) |
|---|---|---|---|
| TH18-01 | 0 to 2.5 m | – | – |
| | 2.5 to 9.0 m | 11 | 36 |
| | Below 9.0 m | 9 | 28 |
| TH18-02 | 0 to 2.5 m | – | – |
| | 2.5 to 7.5 m | 11 | 36 |
| | 7.5 to 11.0 m | 9 | 31 |
| | Below 11.0 m | 6 | 20 |
| TH18-03 | 0 to 2.5 m | – | – |
| | 2.5 to 4.5 m | 10 | 33 |
| | 4.5 to 11.0 m | 12 | 40 |
| | Below 11.0 m | 6 | 20 |

*Values apply for piles installed in the native silty clay only, and not fill or topsoil.

For design purposes, the upper 2.5 m of pile length below final ground elevation of piles potentially exposed to frost should be neglected when determining pile capacities. For piles not exposed to frost the upper 2.0 m of pile length below final grade should be neglected. It should be noted that this applies to piles installed in the native silty clay only, and that fill or organic material should be assumed to provide no support.

To resist tensile pile uplift due to frost action and / or swelling of the clay, all piles should have full depth reinforcing steel designed by a qualified structural engineer.

The placement of fill to raise the development to the Flood Protection Level (FPL) will result in consolidation of the underlying clay, most notably in the location of the two-storey townhouses where the required fill amounts to be 2.0 m± in height. The resulting downward movement of the clay around the piles will induce down drag forces on the piles due to negative skin friction. If drive pre-cast concrete driven piles are used, the down drag on the piles can be reduced by the application of bituminous or other coatings like bentonite slurry on the piles before installation. A low-friction sleeve such as a poly-wrapped sono-tube can also minimize the downdrag on the piles and are recommended through all fill material. It should be noted that the lateral resistance capacity of the piles may also be affected when these methods are used to reduce the downdrag.

Cast-in-place straight shaft or belled concrete caissons end bearing on the dense silt till could also be used to support the loading. The estimated average end bearing values for Limit States Design of the piles are provided in 4.

**TABLE 4**
**ESTIMATED END BEARING RESISTANCE FOR LIMIT STATES DESIGN**

| Material Type | Serviceability Limit State, SLS (kPa) | Unfactored Ultimate Limit State, ULS (kPa) |
|---|---|---|
| End Bearing on Dense Till | 600 | 3,900 |

The base of end bearing cast-in-place piles must be mechanically cleaned prior to placement of concrete to obtain a sound bearing surface and ensure that all deleterious material has been removed from the bearing surface. It is recommended that geotechnical personnel are on site to confirm end bearing piles are founded on dense silt till during pile installation.

The potential exists for squeezing of the boreholes during the installation of the cast-in-place concrete piles at the site. Temporary steel sleeves should be used as required during pile installation to maintain the drill shaft in a clean and dry state. Concrete should be poured

immediately following the drilling of each shaft. Should heavy groundwater inflow be encountered, concrete placement should be completed using tremie or pump-in methods. It is recommended that geotechnical personnel are on site during drilling and concrete placement for the piles to confirm end bearing piles are founded on dense silt till and ensure proper installation of the piles.

## 6.2 DRIVEN PRE-STRESSED PRE-CAST CONCRETE PILES

Hexagonal, pre-stressed pre-cast driven concrete end bearing piles are used extensively in Winnipeg. Piles may be designed using the Limit State Design (LSD) values for Serviceability Limit State (SLS) and unfactored Ultimate Limit State (ULS) capacities shown on Table 5 when driven to practical refusal on the underlying till.

**TABLE 5**
**ALLOWABLE LOAD CAPACITY FOR PRE-STRESSED PRE-CAST DRIVEN CONCRETE PILES**

| Pile Diameter | Serviceability Limit State (SLS) Values (kN) | Unfactored Ultimate Limit State (ULS) Values (kN) | Final Refusal Criteria (Blows per 25 mm)* |
|---|---|---|---|
| 300 mm | 450 | 1,250 | 5 |
| 350 mm | 625 | 1,750 | 8 |
| 400 mm | 800 | 2,200 | 12 |

* If higher energies or other types of hammers are used, they should be evaluated to ensure that piles are not overstressed and suitable refusal criteria to be determined.

The final penetration resistances indicated on Table 5 should be achieved at least three (3) times for the final resistance. The recommended SLS value would limit settlement to 25 mm. A geotechnical resistance factor ($\Phi$) of 0.4 should be applied to determine the factored geotechnical resistance as presented in the equation above. The resistance factor can be increased with load testing including dynamics methods using a Pile Driving Analyzer (PDA) and a static load test, according to the criteria discussed in subsequent sections.

It is expected the pile tip elevations will vary considerably (several metres) in depth. The actual depth of pile penetration will depend on localized till conditions and thickness. Power auger refusal was encountered dense silt till between 15.4 m± and 17.8 m± depth below grade. Piles

can typically be cast in lengths ranging from 10 m to 22 m. However, pile lengths exceeding 18 m require special handling in order to prevent cracking of the piles.

Pre-boring approximately 3.0 m to 4.0 m below grade is recommended to allow for standing of the piles, enhance pile plumbness/alignment and to reduce ground vibration and potential ground heave in large pile groups. If significant squeezing or sloughing of the bore hole occurs during pre-boring then the pre-boring depth may be reduced accordingly. To minimize potential rebound or pile heave during driving, the spacing between adjacent piles should be a minimum of three (3) pile diameters from center-to-center. If heave occurs, these piles should be re-driven to refusal. Careful attention will be required during driving, especially as the pile approaches refusal, to avoid breaking the pile.

The hammer and pile section combination should be reviewed by KGS Group prior to mobilizing to the site. In lieu of pile load testing, the required final drive criteria (final set) will be established at the start of driving based on a Wave Equation Analysis (GRLWEAP) analysis for the selected pile hammer and pile section combination.

It is anticipated that a pile hammers with a maximum energy rating on the order of 60 kN-m (45 kip-ft) will be required to drive the above pile sections to practical refusal and concurrently achieve the ULS capacities listed in Table 5. However, the type of hammer (i.e. diesel versus hydraulic) will also factor in the final pile capacity achieved. A Pile Driving Analyzer (PDA) is recommended as it will confirm the effectiveness of the hammer during driving to achieve the design values and also allow for the use of an increased Geotechnical Resistance Factor of 0.5. If a hammer with a lower energy rating is used, then it should be expected that the ULS values will also be reduced.

Reduction in pile load carrying capacity due to group action is not necessary for the precast concrete piles when driven to practical refusal, since the pile design relies mainly on the base-resistance developed at the tip. The design capacity of a pile group is the sum of the allowable capacity of the individual pile.

It should be assumed by the designers that the tensile strength of pre-stressed pre-cast piles is minimal and they have little capacity to resist bending. The age of the pile should be specified to be at least seven (7) days old prior to driving.

## 6.3 ADDITIONAL PILE FOUNDATION RECOMMENDATIONS

Additional considerations for design and construction of the pile foundations are provided below:

- The spacing between adjacent friction piles should be a minimum of three (3) pile diameters (or bell diameter, if applicable), as measured from center-to-center. In addition to piles acting individually, piles can act as a group when closely spaced, less than three (3) pile diameters apart. Group action occurs when the soil between adjacent piles is dragged down and shaft resistance develops around the perimeter of the group only. If it is necessary to space the piles closer than three (3) pile diameters apart, the capacity of the piles acting as a group will need to be evaluated once final geometry and spacing of the piles is known.
- All piles should be provided with adequate steel reinforcement designed by a qualified structural engineer
- To resist tensile forces and to limit potential for pile uplift due to frost action and/or swelling of the clay, all piles should have a minimum length of 7.6 m in the native silty clay. Piles should also have reinforcing steel over a minimum embedded length of 6 m and 8 m for piles located in heated and unheated areas, respectively. Reinforcing steel should be designed by a qualified structural engineer.
- For belled piles end bearing on competent till, the elevation of the base of the pile must be selected so that it is at least 500 mm into the competent underlying till to ensure that the desired capacity can be developed.
- Full-length steel sleeves should be maintained on site and utilized as required when seepage is encountered, which cannot be controlled by sleeves during piling, removal of the water from the pile holes prior to pouring concrete or placing concrete by tremie methods may be 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 into the pile hole from the adjacent ground.
- Reinforcement and concrete should be placed immediately following the drilling 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 re-drilled, once concrete is ready to place.
- All driven piles should be driven continuously to their required design length once driving is initiated. The till surface elevation, thickness and consistency can be irregular and it is expected that the pile tip elevations of end bearing piles could vary up to several metres

in depth. The actual depth of pile penetration will depend on localized till conditions and thickness.

- The final selection of driven concrete pile lengths will be the responsibility of the piling contractor using the available test hole information provided in this report.
- Detailed construction records and full time inspection by qualified geotechnical personnel is recommended throughout construction of foundations to ensure that the design capacities indicated in this report are achieved. Inspection of the bearing surface by qualified geotechnical personnel is required to ensure that the specified bearing capacity is achieved.
- A Geotechnical Resistance Factor of 0.5 could be utilized if dynamic load testing using a Pile Driving Analysis (PDA) is undertaken on a minimum of 5% of the piles driven at the site. The dynamic testing should be utilized to verify that the ULS geotechnical values are achieved at the end of drive and to determine a final drive criteria (blow count). Additional testing may be required if a pile does not achieve the final criteria or confirm to pile integrity as pile damage is suspected. Final pile capacities will be based on results of Case Pile Wave Analysis Program (CAPWAP) analysis.
- A Geotechnical Resistance Factor of 0.6 could be utilized if a static load test is undertaken on a minimum of one (1) pile at the site, in addition to the dynamic load testing.
- If PDA or static load testing is undertaken, it should be completed under the supervision of an experienced geotechnical engineer and KGS Group should review the results of any testing and pile capacities.
- All concrete piles should utilize CSA Type HS sulphate resistant cement.
- A minimum 150 mm void form should be used below all grade beams and pile caps to protect against potential uplift from swelling clay, silt, and potential frost heave below perimeter grade beams.

# 7.0 SITE DEVELOPMENT CONSIDERATIONS

## 7.1 FLOOR SLABS

Floor slabs may consist of either a slab-on-grade or structural slab construction. A slab-on-grade floor should only be selected if some differential movement and cracking of the floor slab is tolerable. Differential movements up to 100 mm are routinely observed for slab-on-grade floor slabs in residential construction. Otherwise, a structural floor slab supported on piles and grade beams should be used. The following is recommended if a slab-on-grade floor design is selected:

- Sub-excavate the surficial soils (if required) to the subgrade design elevation and proof-roll compact the native soil subgrade to identify any soft spots. If a soft spot is encountered, it should be excavated a further 600 mm and replaced with a compacted granular to 98% Standard Proctor Maximum Dry Density (SPMDD) compacted in 150 mm thick lifts. A non-woven geotextile should also be placed on the subgrade prior to sub-base placement.
- A minimum 150 mm thick layer of granular base and 300 mm thick layer of sub-base should be placed immediately below the slab. All granular should be placed in maximum 150 mm thick lifts and compacted to 98% SPMDD.
- The granular fill should be well-graded free-draining and include organic-free and non-frozen aggregate conforming to the City of Winnipeg Specifications (CW3110). Sieve analysis and compaction testing of the granular base and subgrade materials should be conducted by qualified geotechnical personnel to ensure that the materials supplied and percent compactions are in accordance with design specifications.
- Shallow silt was identified at the site below existing grade to a depth of 1.5 m in TH18-01 during the geotechnical investigation. Additional excavation of these layers and replacement with a compacted granular sub-base is recommended wherever these unsuitable foundation layers are encountered.
- Depending on the elevations of the foundations, provisions for groundwater control in the vicinity of the foundations may be required. The system should include a perimeter and under-floor weeping tile system around the perimeter of the foundations and under the foundation floor leading to a facility sump pit.
- Some movement, potential cracking, and/or differential settlement of the concrete slab is likely to occur with grade supported slabs due to the expansive (swelling and shrinking) nature of the silty clay soil.
- All mechanical services or piping that would be buried within the engineered fill should be designed to accommodate potential slab movement.

- A minimum 150 mm void space should be constructed under all structural elements including pile caps and grade beams to accommodate the expansive nature of the underlying soil.

Both seasonal movement and differential settlement and associated potential cracking of the concrete slab may occur over time with grade supported slabs. This alternative should be selected only if some movement and differential settlement is acceptable. Differential settlements of 50 mm to 100 mm do routinely occur for floor slabs poured directly on grade. Where these potential movements are deemed unacceptable a structural slab supported on intermediate piles should be utilized. Under no condition should slab-on-grade construction proceed in freezing conditions or on frozen ground or while frozen ground is thawing out.

## 7.2 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 2680ºC days, taken as the coldest winter over a 10 year period. The estimated maximum depth of frost penetration in Winnipeg is 2.5 m assuming bare ground and no insulation cover. The clay soils can heave upon freezing and must be considered in the foundation design. Positive site drainage must also be maintained after development.

Well-graded granular materials with less than 5% fines should be used as backfill material because they are less susceptible to the effects of frost heave than fine grained silt and clay materials. Polystyrene insulation can be used as a thermal insulator to minimize any effects that frost could potentially have on foundations or slabs along with a minimum 150 mm void form below grade beams or pile caps.

Soil in contact with foundation elements can freeze to the foundations and develop adfreeze bonding, which can result in uplift forces. The 4$^{th}$ 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.
- 150 kPa for saturated gravel frozen to steel.

The depth of burial of water lines or other lines that cannot be allowed to freeze should consider local practice. In general, it is recommended that water lines be buried a minimum of 2.5 m below ground or finished grade to reduce the risk of freezing. Shallower 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.

## 7.3 BACKFILL

Both the exterior and the interior of parkade grade beams and walls as well as fill throughout the site to comply with the Flood Protection Level (FPL) would need to be properly backfilled with compacted controlled fill to reduce settlement of the fill and surface structures such as pavements and sidewalk slabs.

It is recommended that all material placed as fill be a clean granular material with less than 5% fines (passing the #200 sieve), up to a design elevation which is 600 mm below final grade. Low permeable clay should be placed for the final 600 mm. The granular backfill should be compacted uniformly in maximum 150 mm lifts to a density of at least 98% SPMDD. In addition, the base of the walls should be provided with a filter protected drainage system to prevent hydrostatic pressure build up against walls. Where drainage is not provided, the hydrostatic pressure against the wall should be assumed with a groundwater level to be at the surface.

## 7.4 TEMPORARY CONSTRUCTION EXCAVATIONS AND SHORING

Construction excavation details were not available during preparation of this report. Any excavation deeper than 1.5 m (5 ft) should be reviewed and designed prior to construction by an experienced professional engineer with an expertise in geotechnical engineering.

The silt and clayey silt soils encountered may be susceptible to sloughing from wetting and mechanical disturbance. Localized groundwater inflows into the excavation may require temporary pumping and shoring, depending on the size, depth, and extent of the excavation. KGS Group recommends that the side slopes of all open excavation be covered to prevent saturation of the soil and all surface runoff should be directed away from excavations. All surcharge loads such as stockpiled soil, equipment, etc. should be kept a minimum of 10 m away from the edge of excavations. All excavations should be carried out in accordance with applicable Workplace Health and Safety rules and regulations.

## 7.5 SITE SURFACE DRAINAGE

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 pavements 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.

## 7.6 SITE SUBSURFACE DRAINAGE

A permanent subdrain system should be installed around the exterior of below grade walls and below grade floor slabs to collect groundwater and direct it into a central high capacity sump pit(s). Internal drainage should also be given consideration, with a perforated weeping tile wrapped with filter sock in a granular (pea gravel or crushed rock) trench directing flows to a central sump pit(s). Interior weeping tile should be located a minimum of 300 mm below the underside of the prepared subgrade elevation for the slab. All granular fill placed within the weeping tile trench should be free draining. Exterior weeping tile trenches should be capped a minimum of 0.6 m thick cohesive soil (clay).

## 7.7 TYPE OF CEMENT FOR CONCRETE MIX

All foundation concrete should be made with high sulphate-resistant cement (HS or HSb), and all cast-in-place piles and pile caps should have a minimum specified 28 day compressive strength of 35 MPa and class of exposure of S-1 corresponding to very severe sulphate attack. 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 4, CSA A23.1-04.

## 7.8 SEISMIC SITE CLASSIFICATION

Based on the results of the test hole drilling program and KGS Group's knowledge of the geology in the area the design can assume National Building Code of Canada (NBCC) 2015 Site Class E conditions for seismic site response.

## 8.0 REGULATORY AND ENVIRONMENTAL CONSIDERATIONS

Based on KGS Group's professional experience and understanding of the project, federal environmental approvals will not be required for the project. The proposed project will not require approval by the Canadian Environmental Assessment Agency (CEA Agency), as it is not a designated project under the CEAA 2012 annexed regulation SOR/2012-147A "Regulations Designating Physical Activities," and it is not being constructed on federal land. Regulatory approval by Transport Canada is not required as Normand Creek is not identified on the List of Scheduled Waters under the *Navigation Protection Act*. The project will not require approval by the Fisheries and Oceans Canada under the *Fisheries Act* as the proposed project activities are not likely to cause serious harm to fish that are part of a fishery, or to fish that support such a fishery. A buffer zone is proposed between the project site and nearby waterbodies including Normand Creek, which appears to be an ephemeral creek that is predominantly wet following rain events and during the spring freshet due to back flow from the Red River. As the creek is ephemeral, dry or frozen most of the year, an authorization under the *Fisheries Act* will not be required.

Provincial approvals will also not be required. Through past discussions with the Manitoba Sustainable Development Environmental Approvals Branch, it has been determined that projects such as that proposed are not considered a development under the Classes of Development Regulation (Reg. 164/88) of *The Environment Act* (C.C.S.M. c. E125) and environmental licensing is not required.

While a portion of the proposed site development is located near the upper bank of the Normand Creek channel, permission to undertake work would not be required under the City of Winnipeg Waterway By-law. The regulated area of the Red River includes the riverbed and lands extending 106.7 m from the Normal Summer Water Level on each side of the river and the project area is located outside of this zone.

While we do not anticipate the need to prepare regulatory submissions to the authorities noted above, we recommend that an assessment be completed to determine the impact of the proposed project on the existing environment. This assessment should consist of a desktop survey of biological and heritage conditions. If the need for regulatory approvals is identified

during the project then the detailed design drawings showing mitigation measures and/or compensation plans would be included with any required submissions.

As part of a biological assessment, the Conservation Data Centre (a branch of Manitoba Sustainable Development) would be contacted to identify the potential for any species of conservation concern in the project area to assess potential project effects.

The Historic Resources Branch (HRB) of the Manitoba Department of Sport, Culture and Heritage would also be contacted and provided with the location and description of the proposed works to find out if there is concern relating to impacts to heritage resources. A request would be submitted to the HRB for a screening-level evaluation of the project and for them to cross-reference the project location with the provincial heritage resources database. Under Section 12(2) of *The Heritage Resources Act*, if the Minister of Sport, Culture and Heritage has reason to believe that heritage resources or human remains are known, or thought likely to be present, on lands that are to be developed, then a Heritage Resources Impact Assessment (HRIA) and mitigation is required to be conducted, if necessary, prior to the project's start. Given the proximity of the proposed project to the Red River, an active riverine feature and historical watercourse route, the HRB may have concerns relating to the disturbance of heritage resources. Early settlers, including Métis families, settled along the banks of the Red River not long after the Red River Settlement was established in 1812 and several Métis families farmed in the area. If a HRIA is required an archaeological consultant would be contracted who would obtain a heritage permit to conduct a heritage investigation at the site. The investigation would determine if there are heritage resources within the areas to be developed and a report would be submitted to the HRB as a requirement of the heritage permit. Based on the consultant's report the HRB would make a recommendation on the project. In the event of a discovery of an archaeological/heritage resource during project construction, activities in that area of the project will be temporarily postponed and the area cordoned off until the item encountered can be examined by an appropriately trained individual.

Several design measures are proposed to mitigate potential project effects typical of a project of this type. To mitigate concerns relating to sediment-laden runoff entering nearby waterways winter construction, erosion control for disturbed bank slopes, and re-vegetation of all disturbed areas is proposed. Construction when the ground is frozen typically results in less disturbance

to the ground surface, however, it is anticipated that groundwater may rise through the excavation and eventually flow onto the ground surface. This groundwater seepage through an active construction area may cause erosion and sediment-laden runoff. Groundwater must be controlled during construction to help prevent sediments from entering Normand Creek and the Red River. All excavated materials should be hauled off site immediately. Silt fencing should be erected and maintained directly downslope of any disturbed areas to prevent sediment migration into nearby watercourses. These mitigation measures should be maintained until vegetation is established on the disturbed areas. Together with winter construction this will help to prevent erosion and sediment-laden runoff from entering both Normand Creek and the Red River and from causing harm to fish.

Winter construction can also mitigate concerns relating to disturbance of nesting migratory birds. In Canada, migratory birds have their nesting habitat protected by *The Migratory Birds Convention Act* (MBCA). The MBCA offers protection for migratory bird species, their eggs, and their nests. In compliance with the MBCA, no listed migratory bird species may be captured, injured, taken or disturbed, or nests damaged, destroyed, removed, or disturbed. Environment and Climate Change Canada publishes technical information on general nesting periods to support the planning of activities in order to reduce the risk of detrimental effects to migratory birds. Winnipeg lies within Migratory Bird Zone B4 where the general nesting period is mid-April to late-August during which clearing activities should be avoided. Conducting tree and ground clearing between September and March can avoid disturbance of most nesting migratory birds. If clearing activities need to be conducted during the migratory bird nesting period, bird biologists would need to conduct a nesting bird survey to identify the presence of active nests. Such a survey would use a scientifically sound approach that considers available bird habitats, which migratory bird species are likely to be encountered in such habitats, and the time periods when they are likely to be present. Bird surveys would be undertaken during the pre-construction stage to determine the presence or absence of nesting activity by protected birds and additional surveys can be conducted immediately prior to construction, if necessary. Occupied nests located within the construction area would be protected with a buffer zone determined by a setback distance appropriate to the species, the level of the disturbance and the landscape context. No construction activities should be conducted within buffer zones until young birds have fledged and permanently left the vicinity of the nest.

## 9.0 CONCLUSIONS

The following are the primary conclusions from this assignment:

- The proposed development consists of five (5) two-storey townhouses, two (2) four-storey apartment blocks, and three (3) six-storey apartment blocks. All structures, with the exception of the townhouses, are proposed to have below grade parking. The Normand Creek channel meanders in close proximity to the development on the north side.

- The property is located within the Designated Floodway Fringe area and the flood protection level (FPL) for the property is Elev. 231.85 m. This elevation is up to 2.1 m above grade at the current top of bank area and up to 5.8 m above grade at the deepest portion of the Normand Creek channel.

- The stratigraphy at the location of the proposed development was interpreted to consist of clay fill over lacustrine silty clay over glacial till. Till was encountered at approximately 13.3 m± to 15.5 m± depth below grade (Elevation 215.4 m to 217.2 m). Power auger refusal occurred within the dense silt till at Elevation 214.16 m to 214.46 m.

- Observed seepage, squeezing and sloughing conditions throughout the drilling investigation were considered minor. Groundwater inflows and subsequent sloughing may have to be dealt with during construction depending on the type of foundation being installed and whether groundwater conditions differ from those encountered during the investigation at the test hole locations.

- During a short (20 day) groundwater monitoring period, groundwater at the site appears to vary from Elev. 224.3 m to 226.5 m with higher groundwater levels located on the west side of the site. A static groundwater gradient currently appears to be present at individual monitoring locations with respect to upward / downward groundwater movement. Some instruments are still stabilizing. Groundwater levels will fluctuate seasonally and following precipitation events and the actual groundwater levels at the time of construction could differ from those reported in this report.

- Based on the results of a preliminary slope stability assessment and discussions with Progressive Real Estate during the preliminary design, it will not be feasible to construct the proposed development upslope of an analytically determined slope stability offset line. Conventional slope stabilization alternatives such as rockfill columns cannot be implemented consistently across the site within the property limits and outside of the Creek banks. The proposed development therefore requires some alternate means of soil retention, such as the sheetpile retaining wall illustrated in Drawing 18-3405 G01, in order to satisfy floodproofing requirements.

- The proposed development can satisfy the requirements of Floodway Fringe Area Regulation (266/91) through a combination of implementation of Schedule A and Schedule C, as illustrated in Drawing 18-3405-001 G01 by incorporating a sheetpile retaining walls. This sheetpile retaining wall is also necessary to protect the development

from potential slope movements wherever the development must encroach beyond the recommended slope stability development offset line.

- Wherever the development encroaches beyond the slope stability offset line, the proposed sheetpile wall must be driven to refusal in the underlying till. Lesser sheetpile embedment depths, or alternate retaining wall solutions can be considered to retain fill for the portions of the proposed development outside the limits of the slope stability offset line.

- The estimated preliminary cost to install sheetpiling at the site varies from $7,500 to $12,500 per lineal meter of sheetpile. If in detailed design, mechanically stabilized earth walls can be implemented to provide more economical soil retention solutions outside of areas where bank stability is a concern, typical costs for these details at this site will vary from $500 to $750 per square meter of wall face.

- Driven pre-stressed pre-cast concrete piles, cast-in-place concrete friction piles or end bearing piles may be used to support the building foundation loads. The four-storey and six-storey apartment buildings must be founded on end-bearing piles while the two-storey townhouses can be founded on friction piles, if they provide a more economical piled foundation solution.

## 10.0 RECOMMENDATIONS

Based on our assessment the following recommendations are made:

- Continue to monitor the geotechnical instrumentation installed at the site (eight (8) piezometers and one (1) slope inclinometer) at regular intervals during detailed design and construction
- Should municipal land use approvals be granted, proceed to detailed design of the proposed development under the assumption that flood protection can be provided for the proposed development, consistent with the requirements of Floodway Fringe Area Regulation (Regulation 266/91). Based on the proximity of the Normand Creek to several of the apartment structures in the proposed development, a combination of Schedule A and Schedule C must be implemented to satisfy the regulations.
- Sheetpiles driven to refusal in till, and with appropriate tie-back considerations, must be implemented wherever the development crosses the recommended slope stability offset line discussed in this report. Tie-back requirements for the sheetpile wall should be finalized as part of detailed design.
- Mechanically stabilized earth walls should be explored in detailed design as more economical soil retention alternatives for flood-proofing the proposed development wherever the proposed development is sufficiently offset from the Normand Creek channel, as dictated by the preliminary slope stability analysis in Section 4.0.
- The four-storey and six-storey apartment structures must be founded on deep piles that are end bearing on till. The two-storey townhouses can be founded on either friction or end bearing piles, as dictated by economy.
- A geotechnical resistance factor $\Phi$, of 0.4 should be applied for the design of the foundations for the proposed development if no further confirmation of ULS capacity is undertaken. Numerous other foundation recommendations and considerations are discussed in Section 6.0 of this report.
- Should municipal land use approvals be granted, and as part of detailed design, an assessment should be completed to determine the potential impact of the proposed project on the existing environment, and to determine appropriate mitigation measures, as required. This assessment would consist of a desktop survey of biological conditions. Specifically, the Conservation Data Centre (a branch of Manitoba Sustainable Development) would be contacted to identify the potential for any species of conservation concern in the project area. Enquiries would also be made to the Historic Resources Branch.
- Should the project progress to the detailed design phase, consider the environmental mitigations measures proposed in Section 8.0. Certain development aspect activities, such as tree clearing within the winter months may augment the environmentally responsible manner under which the project will proceed with less potential for scheduling setbacks.

# 11.0 STATEMENT OF LIMITATIONS

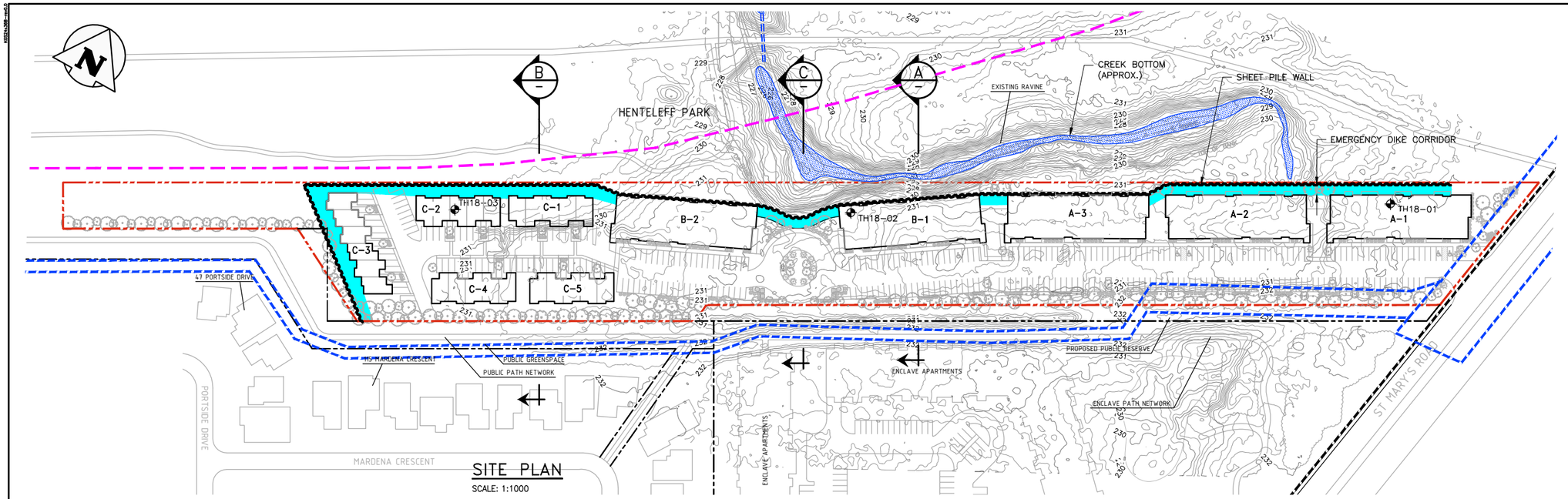
## 11.1 THIRD PARTY USE OF REPORT

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

## 11.2 GEOTECHNICAL INVESTIGATION STATEMENT OF LIMITATIONS

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

# DRAWINGS




## SITE PLAN
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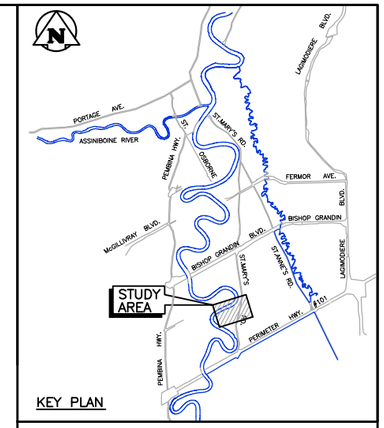
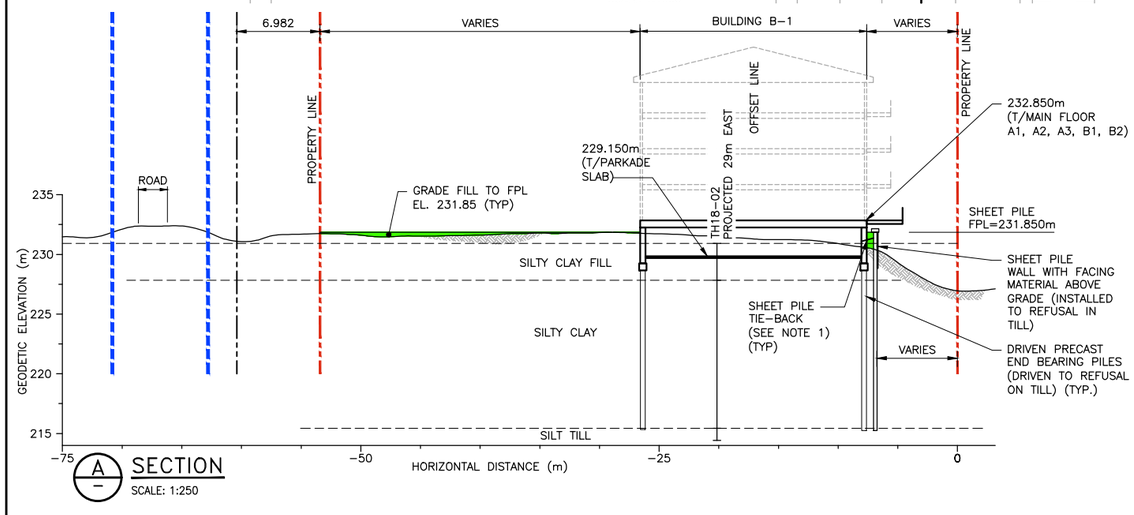
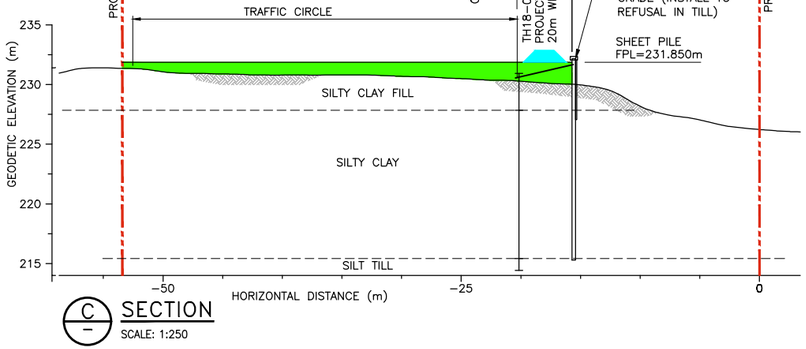


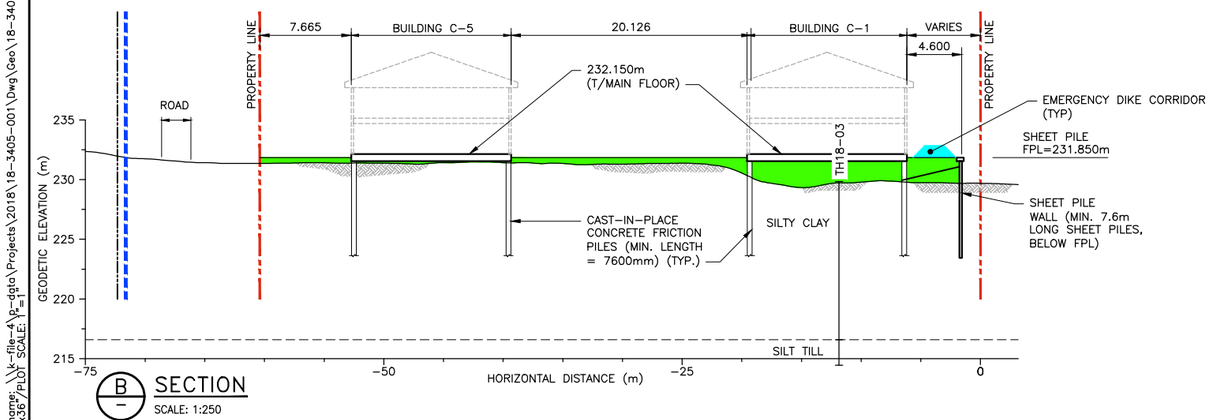

KEY PLAN




A SECTION
SCALE: 1:250




C SECTION
SCALE: 1:250




B SECTION
SCALE: 1:250

LEGEND:

- 232 — LiDAR CONTOUR (m)
- PROPERTY LINE
- LIMITS OF PROPOSED PROPERTY DEVELOPMENT
- FLOODWAY LINE
- PRIMARY LINE OF DEFENSE (PLD)
- TH18-01 TESTHOLE (KGS GROUP, AUGUST 2018)
- CREEK BOTTOM (APPROX.)
- INFILL TO RAISE TO FPL
- EMERGENCY DIKE CORRIDOR

**FLOOD PROTECTION LEVEL**
**FPL = EL. 231.85m**

**NOTES:**

1. SHEET PILE TIE-BACK AND BACKFILL REQUIREMENTS TO BE FINALIZED AS PART OF DETAILED DESIGN

**PRELIMINARY**
NOT TO BE USED FOR CONSTRUCTION

**REFERENCE:**
AFFINITY ARCHITECTURE INC. DWG NO 18004 SHT NO A1.1

| NO. | YY/MM/DD | DESCRIPTION | DESIGN BY | DESIGN CHECK |
|---|---|---|---|---|
| 0 | 18/08/27 | ISSUED WITH FINAL REPORT | CMR | BPA |

REVISIONS / ISSUE

CLIENT: PROGRESSIVE REAL ESTATE

PROJECT:
NORMAND AVE. PROPERTY DEVELOPMENT
PRELIMINARY GEOTECHNICAL
ASSESSMENT PROJECT

DWG. DESCRIPTION:
PROPOSED FLOOD PROTECTION WORKS
PRELIMINARY PLAN AND SECTIONS

KGS GROUP CONSULTING ENGINEERS

| | |
|---|---|
| DESIGN BY: CMR | DATE (YY/MM/DD): 18/08/17 |
| DESIGN CHECK: | DATE: |
| DRAWN BY: PwD | DATE: 18/08/17 |
| DWG CHECK: | DATE: |

DWG. NO. 18-3405-001 G01 REV: 0

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
KGS Group
No. 245

Filename: \\k-file-4\p-dgta\Projects\2018\18-3405-001\Dwg\Ceo\18-3405-001_G01 - Tab:Rev0 Plotted By: PDeffner 18/08/27 [Mon 11:00am]
24 X36 PLOT SCALE: 1 = 1

# APPENDIX A

# TEST HOLE LOGS

KGS GROUP

# SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH18-01**



**CLIENT** PROGRESSIVE REAL ESTATE

**PROJECT** Normand Avenue Property Development

**SITE** Proposed Normand Ave Property Development

**LOCATION** Within footprint of building A-1

**DRILLING METHOD** 125 mm ø Solid Stem Auger, Mobile B37x Track Mounted Drill Rig

JOB NO. 18-3405-001
GROUND ELEV. 231.99
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED 8/7/2018
UTM (m) N 5,519,179
E 635,695

| ELEVATION (m) | DEPTH (m) | DEPTH (ft) | DESCRIPTION AND CLASSIFICATION |
|---|---|---|---|
| 231.5 | | | **SAND AND GRAVEL FILL** - Dark grey, damp, compact, medium to coarse grained sand, fine grained gravel, with clay, trace coarse grained gravel, trace clay. |
| 231.3 | | | **TOPSOIL** - Black, damp, loose, with organics. |
| 231 | 1 | | **CLAY FILL** - Black, damp to moist, very stiff, intermediate to high plasticity, trace fine to coarse grained sand, trace fine grained gravel, trace rootlets. |
| 230.8 | | | |
| 230.5 | | 5 | **CLAY** - Brown, moist, stiff, high plasticity. |
| 230.2 | | | **SILT** - Tan, moist, soft, no plasticity. |
| 230 | 2 | | **CLAY** - Brown, moist, stiff, high plasticity. |
| | | | - Trace gypsum inclusions at 2.1 m. |
| 229 | 3 | 10 | - Trace silt nodules below 3.0 m. |
| 228 | 4 | | |
| | | 15 | |
| 227 | 5 | | - Grey below 5.6 m. |
| 226 | 6 | 20 | |
| | | | - Firm below 6.7 m. |
| 225 | 7 | | |
| | | 25 | |
| 224 | 8 | | |
| 223 | 9 | 30 | |
| | | | - Trace medium grained sand below 9.8 m. |
| 222 | | | |

PIEZ. LOG: P at 7.5 m / 7.6 m

| SAMPLE TYPE NUMBER | DEPTH (m) | RECOVERY % |
|---|---|---|
| S1 | | |
| S2 | | |
| S3 | | |
| S4 | | |
| S5 | | |
| S6 | | |
| S7 | | 100 |
| S8 | 7.5 / 7.6 | |
| S9 | | |

SPT (N) blows/0.15 m ▲
DYNAMIC CONE (N) blows/ft △
20 40 60

Cu POCKET PEN (kPa) ★
Cu TORVANE (kPa) ◆
20 40 60 80
PL MC LL
%
20 40 60 80

SAMPLE TYPE: Auger Grab | Shelby Tube | Split Spoon

| CONTRACTOR | INSPECTOR | APPROVED | DATE |
|---|---|---|---|
| Maple Leaf Drilling Ltd. | C. FRIESEN | CMR | 8/23/18 |

GEOTECHNICAL-SOIL LOG U:\FMS\18-3405-001\NORMAND AVE.GPJ

KGS GROUP

# SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH18-01**



| ELEVATION (m) | DEPTH (m) | DEPTH (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.15 m ▲ / DYNAMIC CONE (N) blows/ft △ | Cu POCKET PEN (kPa) ★ / Cu TORVANE (kPa) ◆ / PL MC LL % |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 221 | 11 | 35 | | - Trace fine to coarse grained gravel below 11.3 m. | | | S10 | | | | |
| 220 | 12 | 40 | | | | | S11 | | | | |
| 219 | 13 | 45 | | | | | S12 | | | | |
| 218 | 14 | | | | | | | | | | |
| 217.2 | | | | | | | | | | | |
| 217 | 15 | 50 | | **SILT TILL** - Tan, moist, compact, some fine to coarse grained sand, some fine to coarse grained gravel. | | 15.2 | S13 | | | | |
| | | | | - Light grey below 15.2 m. | | | S14 | 100 | | ▲ 3, 3, 4 | |
| 216 | 16 | | | | | 15.8 | | | | | |
| | | | | - Damp, dense below 16.5 m. | | 16.5 | S15 | | | | |
| | | 55 | | - Coarse grained sand layer (~ 75 mm thick) at 16.8 m. | | 16.8 | S16 | 72 | | ▲ 15, 21, 18 | |
| 215 | 17 | | | | | | | | | | |
| 214.2 | | | | | | 17.8 | S17 | | | | |
| 214 | 18 | 60 | | **POWER AUGER REFUSAL AT 17.83 m** | | | | | | | |
| 213 | 19 | 65 | | Notes:<br>1. Test hole open to 17.37 m after drilling. Trace water in the test hole after drilling.<br>2. Test hole sloughed in to 16.76 m prior to installing standpipe piezometer.<br>3. Installed casagrande standpipe piezometer at 16.15 m below grade with a stick-up of 1.02 m.<br>4. Installed PN 037845 at 7.62 m below grade.<br>5. Backfilled test hole with silica sand from 16.76 to 15.24 m and bentonite chips from 15.24 m to grade. | | | | | | | |
| 212 | 20 | | | | | | | | | | |
| 211 | 21 | 70 | | | | | | | | | |

SAMPLE TYPE: Auger Grab | Shelby Tube | Split Spoon

| CONTRACTOR | INSPECTOR | APPROVED | DATE |
|---|---|---|---|
| Maple Leaf Drilling Ltd. | C. FRIESEN | CMR | 8/23/18 |

**KGS GROUP**

# SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH18-02**



**CLIENT** PROGRESSIVE REAL ESTATE

**PROJECT** Normand Avenue Property Development

**SITE** Proposed Normand Ave Property Development

**LOCATION** Within footprint of building B-1

**DRILLING METHOD** 125 mm ø Solid Stem Auger, Mobile B37x Track Mounted Drill Rig

JOB NO. **18-3405-001**
GROUND ELEV. **230.93**
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED **8/7/2018**
UTM (m) N **5,519,061**
E **635,492**

| ELEVATION (m) | DEPTH (m) | DEPTH (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | SAMPLE NUMBER | RECOVERY % |
|---|---|---|---|---|---|---|---|---|---|
| 230 | 1 | 5 | | **SILTY CLAY FILL** - Dark brown, damp, friable, trace medium to coarse grained sand, trace fine to coarse grained gravel. | | | | S1 | |
| 229 | 2 | | | - Moist, with clay, trace silt nodules, trace gypsum nodules below 2.4 m. | | | | S2 | |
| 228 / 227.9 | 3 | 10 | | **CLAY** - Dark grey, moist, stiff, high plasticity, trace organics. | | | | | |
| 227 | 4 | | | - Brown below 4.2 m. | | | | S3 | |
| 226 | 5 | 15 | | | | | | | |
| 225 | 6 | 20 | | | P | 5.9<br>6.1 | | S4 | |
| | | | | | | | | S5 | 98 |
| 224 | 7 | | | - Mottled brown and grey below 6.7 m. | | | | S6 | |
| 223 | 8 | 25 | | - Grey, firm below 7.6 m. | | | | S7 | 100 |
| 222 | 9 | 30 | | | | | | S8 | |
| 221 | | | | | | | | | |

SPT (N) blows/0.15 m ▲
DYNAMIC CONE (N) blows/ft △
20 40 60

Cu POCKET PEN (kPa) ★
Cu TORVANE (kPa) ◆
20 40 60 80
PL MC LL
%
20 40 60 80

SAMPLE TYPE Auger Grab | Shelby Tube | Split Spoon

| CONTRACTOR | INSPECTOR | APPROVED | DATE |
|---|---|---|---|
| Maple Leaf Drilling Ltd. | C. FRIESEN | CMR | 8/23/18 |

GEOTECHNICAL-SOIL LOG U:\FMS\18-3405-001\NORMAND AVE.GPJ

KGS GROUP

# SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH18-02**



| ELEVATION (m) | DEPTH (m) | DEPTH (ft) | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | SAMPLE NUMBER | RECOVERY % | SPT (N) blows/0.15 m ▲ / DYNAMIC CONE (N) blows/ft △ | Cu POCKET PEN (kPa) ★ / Cu TORVANE (kPa) ◆ / PL MC LL % |
|---|---|---|---|---|---|---|---|---|---|---|
| 220 | 11 | 35 | | | | S | S9 | | | |
| 219 | 12 | 40 | - Soft below 11.9 m. | | | S | S10 | | | |
| 218 | 13 | 45 | - Trace fine grained gravel below 13.1 m. | | 13.4 | S | S11 | | | |
| 217 | 14 | | | P | 13.6 | | | | | |
| 216 | 15 | 50 | | | | S | S12 | | | |
| 215.4 | | | **SILT TILL** - Light grey, moist, compact, some fine to medium grained sand, trace fine grained gravel. | | 15.8 | | | | | |
| 215 | 16 | | - Dense below 15.8 m. | P | 16.0 | S | S13 | | | |
| | | | | | 16.3 | | | | | |
| 214.4 | | | | | 16.5 | S | S14 | 100 | ▲ 50 *SPT refused 50 mm into 1st set | |
| | | | **POWER AUGER REFUSAL AT 16.51 m.** | | | | | | | |
| 214 | 17 | 55 | | | | | | | | |
| 213 | 18 | 60 | | | | | | | | |
| 212 | 19 | | | | | | | | | |
| 211 | 20 | 65 | | | | | | | | |
| 210 | 21 | 70 | | | | | | | | |

Notes:
1. Test hole open to 16.31 m after drilling. Test hole dry after drilling.
2. Installed slope inclinometer at 16.31 m below grade with a stick-up of 0.84 m.
3. Installed PN 037700 at 5.94 m below grade, PN 037936 at 13.56 m below grade, and PN 037846 at 16.00 m below grade.
4. Backfilled test hole with bentonite cement grout from 16.31 m to grade.

SAMPLE TYPE: Auger Grab | Shelby Tube | Split Spoon

| CONTRACTOR | INSPECTOR | APPROVED | DATE |
|---|---|---|---|
| Maple Leaf Drilling Ltd. | C. FRIESEN | CMR | 8/23/18 |

GEOTECHNICAL-SOIL LOG U:\FMS\18-3405-001\NORMAND AVE.GPJ

KGS GROUP

# SUMMARY LOG

REFERENCE NO.

HOLE NO. **TH18-03**



**CLIENT** PROGRESSIVE REAL ESTATE

**PROJECT** Normand Avenue Property Development

**SITE** Proposed Normand Ave Property Development

**LOCATION** Within footprint of building C-2

**DRILLING METHOD** 125 mm ø Solid Stem Auger, Mobile B37x Track Mounted Drill Rig

JOB NO. **18-3405-001**
GROUND ELEV. **229.83**
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED **8/7/2018**
UTM (m) N **5,518,978**
E **635,341**

| ELEVATION (m) | DEPTH (m) | DEPTH (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % |
|---|---|---|---|---|---|---|---|---|---|
| 229 | 1 | 5 | | **CLAY** - Grey, damp to moist, stiff, intermediate to high plasticity, trace coarse grained sand, trace fine grained gravel, trace rootlets. | | | | | |
| | | | | - Trace gypsum nodules between 0.9 and 1.5 m. | | | | S1 | |
| 228 | 2 | | | - Brown, trace silt nodules below 1.8 m. | | | | S2 | |
| 227 | 3 | 10 | | - Firm to stiff below 3.2 m. | | | | S3 | |
| 226 | 4 | 15 | | - Stiff below 4.6 m. | | | | | |
| 225 | 5 | | | - Slight groundwater infiltration into test hole when drilling below 5.2 m. | | | | S4 | |
| 224 | 6 | 20 | | - Grey below 6.1 m. | P | 5.9<br>6.1 | | S5 | |
| 223 | 7 | | | | | | | | |
| 222 | 8 | 25 | | | | | | S6 | |
| 221 | 9 | 30 | | | | | | S7 | |
| 220 | | | | | | | | | |

SPT (N) blows/0.15 m ▲
DYNAMIC CONE (N) blows/ft △
20 40 60

Cu POCKET PEN (kPa) ★
Cu TORVANE (kPa) ◆
20 40 60 80
PL MC LL
%
20 40 60 80

SAMPLE TYPE Auger Grab | Shelby Tube | Split Spoon

CONTRACTOR **Maple Leaf Drilling Ltd.** INSPECTOR **C. FRIESEN** APPROVED CMR DATE 8/23/18

GEOTECHNICAL-SOIL LOG U:\FMS\18-3405-001\NORMAND AVE.GPJ

KGS GROUP

# SUMMARY LOG

REFERENCE NO.

HOLE NO.
**TH18-03**



| ELEVATION (m) | DEPTH (m) | DEPTH (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.15 m ▲ / DYNAMIC CONE (N) blows/ft △ | Cu POCKET PEN (kPa) ★ / Cu TORVANE (kPa) ◆ / PL MC LL % |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 219 | 11 | 35 | | | | 10.5 | S | S8 | | | |
| | | | | | | 10.7 | | | | | |
| | | | | - Firm below 11.3 m. | | | | | | | |
| 218 | 12 | 40 | | - Trace silt nodules below 11.6 m. | | | S | S9 | | | |
| 217 | 13 | | | | | | | | | | |
| 216.6 | | | | **SILT TILL** - Light grey, moist, compact, some fine to coarse grained sand. | | | S | S10 | | | |
| 216 | 14 | 45 | | - Trace fine grained gravel below 13.7 m. | | | | | | | |
| | | | | - Dense below 14.3 m. | | 14.3 | | | | | |
| 215 | 15 | | | | | 14.9 | S | S11 | | | |
| | | 50 | | | | 15.2 | | | | | |
| 214.4 | | | | | | 15.4 | S | S12 | 100 | ▲ 52 *SPT refused 125 mm into 1st set | |
| | | | | **POWER AUGER REFUSAL AT 15.37 m** | | | | | | | |
| 214 | 16 | | | Notes:<br>1. Test hole open to 15.24 m after drilling. Water level at approximately 5.21 m below grade after drilling.<br>2. Installed casagrande standpipe at 15.24 m below grade with a stick-up of 1.09 m.<br>3. Installed PN 037701 at 6.10 m below grade and PN 037847 at 10.67 m below grade.<br>4. Backfilled test hole with silica sand from 15.24 to 14.33 m and bentonite chips from 14.33 m to grade. | | | | | | | |
| 213 | 17 | 55 | | | | | | | | | |
| 212 | 18 | 60 | | | | | | | | | |
| 211 | 19 | | | | | | | | | | |
| 210 | 20 | 65 | | | | | | | | | |
| 209 | 21 | 70 | | | | | | | | | |

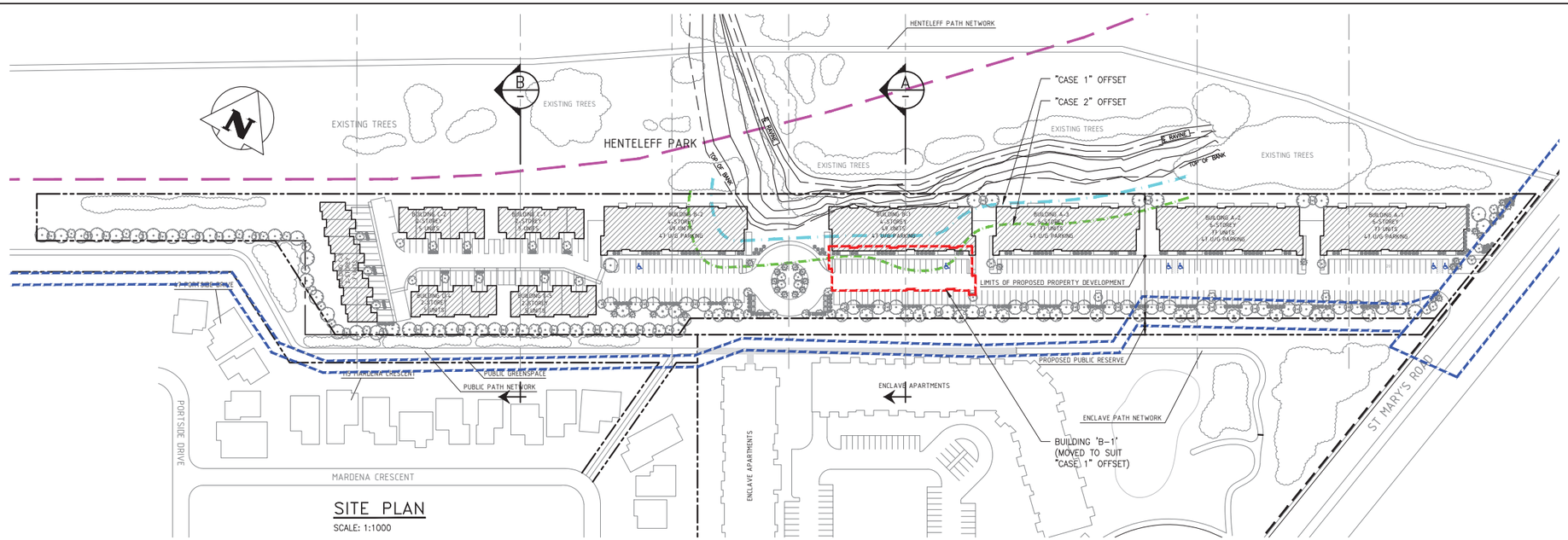
SAMPLE TYPE ⊠ Auger Grab ▯ Shelby Tube ⊠ Split Spoon

| CONTRACTOR | INSPECTOR | APPROVED | DATE |
|---|---|---|---|
| Maple Leaf Drilling Ltd. | C. FRIESEN | CMR | 8/23/18 |

GEOTECHNICAL-SOIL LOG U:\FMS\18-3405-001\NORMAND AVE.GPJ

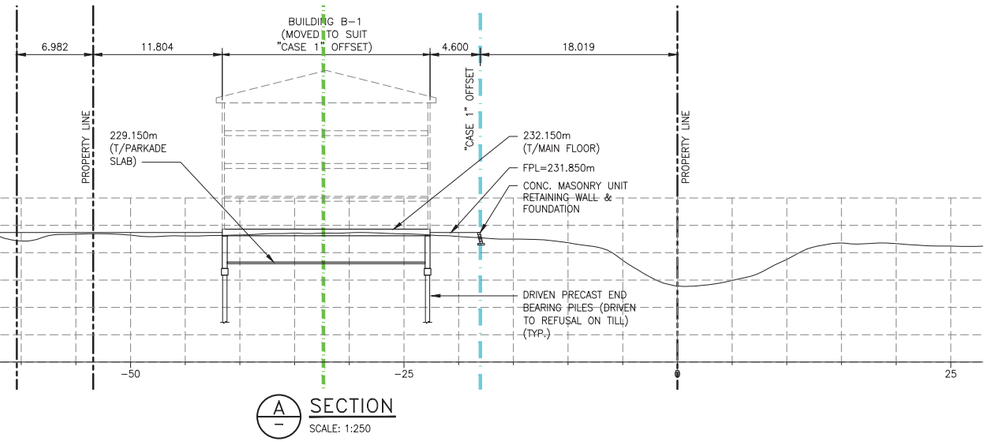
# APPENDIX B

# CONCEPTUAL SLOPE STABILITY ANALYSIS RESULTS



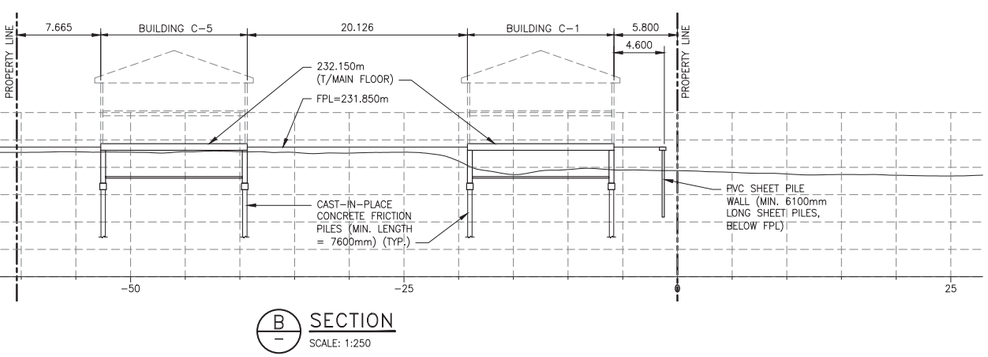

## SITE PLAN

SCALE: 1:1000



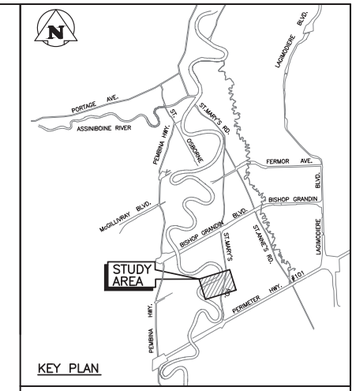

A SECTION

SCALE: 1:250




B SECTION

SCALE: 1:250




## LEGEND:

- CASE 1 OFFSET BUFFER ZONE (SEE NOTE 1 AND 3)
- CASE 2 OFFSET BUFFER ZONE (SEE NOTE 2 AND 3)
- 232 LIDAR CONTOUR (m)
- PROPERTY LINE
- FLOODWAY LINE
- PRIMARY LINE OF DEFENCE (PLD)

**FLOOD PROTECTION LEVEL**
**FPL = EL. 231.85m**

## NOTES:

1. CASE 1 OFFSET BUFFER ZONE ASSUMES INTACT SOIL SHEAR STRENGTH PARAMETERS.
2. CASE 2 OFFSET BUFFER ZONE ASSUMES RESIDUAL SOIL SHEAR STRENGTH PARAMETERS.
3. APPLICABILITY OF CASE 1 OR CASE 2 CONDITIONS IS DEPENDANT ON RESULTS OF GEOTECHNICAL SITE INVESTIGATIONS AND LABORATORY SOIL TESTING.
4. OFFSET BUFFER ZONES ARE BASED ON ACHIEVING A SAFETY FACTOR (FS) OF 1.3 UNDER ASSUMED NORMAL CONDITIONS AND FS OF 1.1 UNDER FULLY SATURATED CONDITIONS WITH FILL PLACEMENT UP TO EL. 232m UPSLOPE OF THESE LINES.
5. OBSERVED EVIDENCE OF HISTORIC SLOPE MOVEMENT IN THIS AREA.
6. THIS LOT IS LOCATED IN THE DESIGNATED FLOODWAY AREA OF FLOODWAY FRINGE AREA. AnY NEW DEVELOPMENT IN THE FLOODWAY AREA AND FLOODWAY FRINGE AREA IS GOVERNED BY MANITOBA REGULATION 266/91 AND THE CITY OF WINNIPEG CHARTER. PROPERTIES OUTSIDE THE PRIMARY LINE OF DEFENSE (PLD) MUST BE DEVELOPED AT OR ABOVE THE FLOOD PROTECTION LEVEL (FPL), AND MUST CONFORM TO REGULATION 266/91.

## REFERENCE:

AFFINITY ARCHITECTURE INC. DWG NO 18004 SHT NO A1.1

| NO. | YY/MM/DD | DESCRIPTION | DESIGN BY | DESIGN CHECK |
|---|---|---|---|---|
| A | 18/08/02 | ISSUED FOR INFORMATION | | |

REVISIONS / ISSUE

CLIENT: PROGRESSIVE REAL ESTATE

PROJECT:
NORMAND AVE. PROPERTY DEVELOPMENT

NORMAND AVENUE, WINNIPEG

DWG. DESCRIPTION:
STRUCTURAL
OFFSET BUFFER ZONE
PLAN & SECTIONS

AUTHENTICATION FOR CURRENT REVISION

ENG. STAMP

KGS GROUP CONSULTING ENGINEERS

| | | | |
|---|---|---|---|
| DESIGN BY: | RJL | DATE (YY/MM/DD): | 18/08/01 |
| DESIGN CHECK: | | DATE: | |
| DRAWN BY: | MBB | DATE: | 18/08/01 |
| DWG CHECK: | | DATE: | |

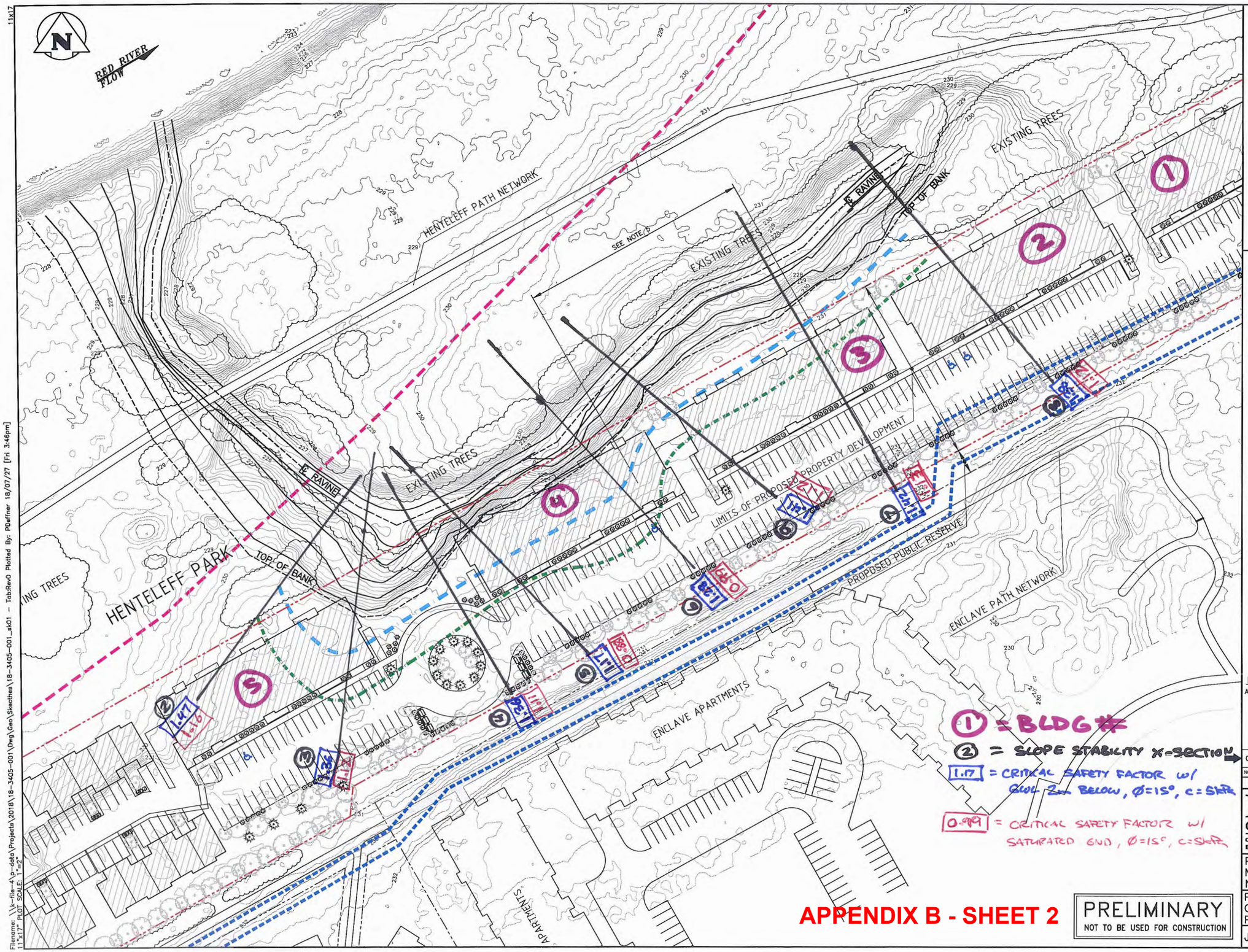
DWG. NO. 18-3405-001 S01

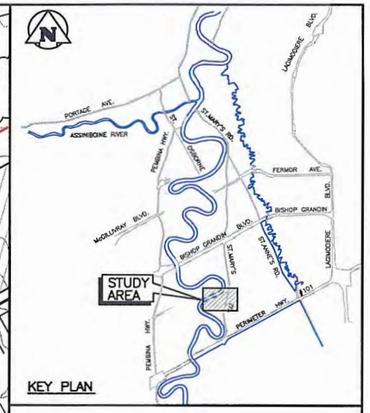
REV: A

**PRELIMINARY**
NOT TO BE USED FOR CONSTRUCTION

Filename: U:\PMS\18-3405-001\18-3405-001_S01 - Tab:S01 Plotted By: mbrischuk 18/08/02 [Thu 12:33pm]
24"x36" PLOT SCALE: 1"=1"

**APPENDIX B - SHEET 1**




KEY PLAN

LEGEND:

- CASE 1 OFFSET BUFFER ZONE (SEE NOTE 1 AND 3)
- CASE 2 OFFSET BUFFER ZONE (SEE NOTE 2 AND 3)
- 232 LIDAR CONTOUR (m)
- PROPERTY LINE
- FLOODWAY LINE
- PRIMARY LINE OF DEFENCE (PLD)

FLOOD PROTECTION LEVEL
FPL = EL. 231.85m

NOTES:

1. CASE 1 OFFSET BUFFER ZONE ASSUMES INTACT SOIL SHEAR STRENGTH PARAMETERS.
2. CASE 2 OFFSET BUFFER ZONE ASSUMES RESIDUAL SOIL SHEAR STRENGTH PARAMETERS.
3. APPLICABILITY OF CASE 1 OR CASE 2 CONDITIONS IS DEPENDANT ON RESULTS OF GEOTECHNICAL SITE INVESTIGATIONS AND LABORATORY SOIL TESTING.
4. OFFSET BUFFER ZONES ARE BASED ON ACHIEVING A SAFETY FACTOR (FS) OF 1.3 UNDER ASSUMED NORMAL CONDITIONS AND FS OF 1.1 UNDER FULLY SATURATED CONDITIONS WITH FILL PLACEMENT UP TO EL. 232m UPSLOPE OF THESE LINES.
5. OBSERVED EVIDENCE OF HISTORIC SLOPE MOVEMENT IN THIS AREA.
6. THIS LOT IS LOCATED IN THE DESIGNATED FLOODWAY AREA OF FLOODWAY FRINGE AREA. ANY NEW DEVELOPMENT IN THE FLOODWAY AREA AND FLOODWAY FRINGE AREA IS GOVERNED BY MANITOBA REGULATION 266/91 AND THE CITY OF WINNIPEG CHARTER. PROPERTIES OUTSIDE THE PRIMARY LINE OF DEFENSE (PLD) MUST BE DEVELOPED AT OR ABOVE THE FLOOD PROTECTION LEVEL (FPL), AND MUST CONFORM TO REGULATION 266/91.

REFERENCE:

AFFINITY ARCHITECTURE INC. DWG NO 18004 SHT NO A1.1

10 0 10 20 30 40m

SCALE: 1:1000 11x17

| NO. | YY/MM/DD | DESCRIPTION | ISSUED BY | CHECK BY |
|---|---|---|---|---|
| 0 | 18/07/27 | ISSUED FOR INFORMATION | CMR | BPA |

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

PROGRESSIVE REAL ESTATE

PRELIMINARY GEOTECHNICAL ASSESSMENT
NORMAND AVENUE PROPERTY DEVELOPMENT
NORMAND AVE, WINNIPEG

PRELIMINARY SLOPE STABILITY
OFFSET BUFFER ZONES FOR
PROPERTY DEVELOPMENT

JULY 2018 | SKETCH 01 | REV. 0

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

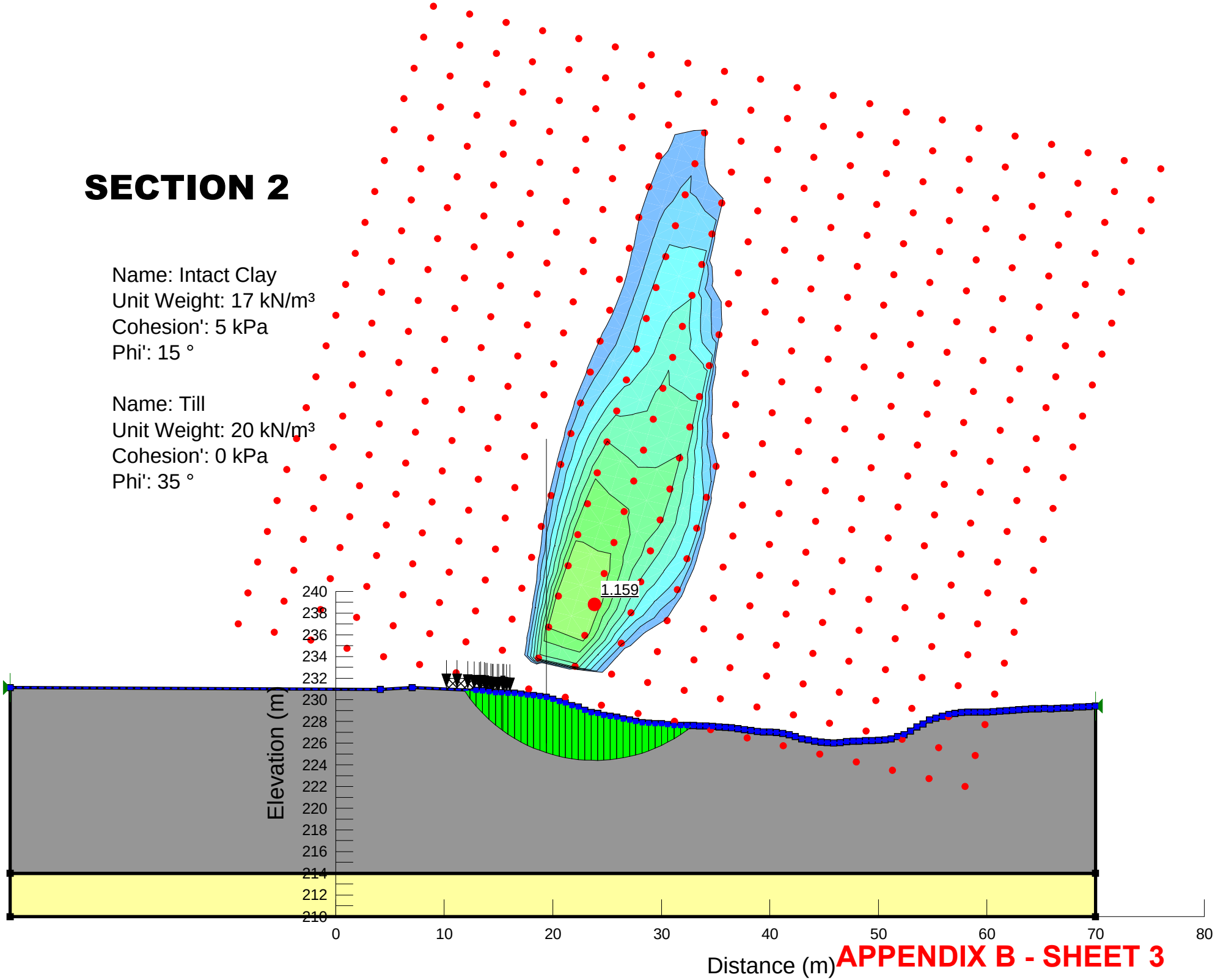
**APPENDIX B - SHEET 2**

Filename: \\f-file-4\p-data\Projects\2018\18-3405-001\Dwg\Geo\Sketches\18-3405-001_sk01 - TabRevO Plotted By: PDeffner 18/07/27 [Fri 3:46pm]
11"x17" PLOT SCALE: 1"=2'

# SECTION 2

Name: Intact Clay
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 15 °

Name: Till
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °

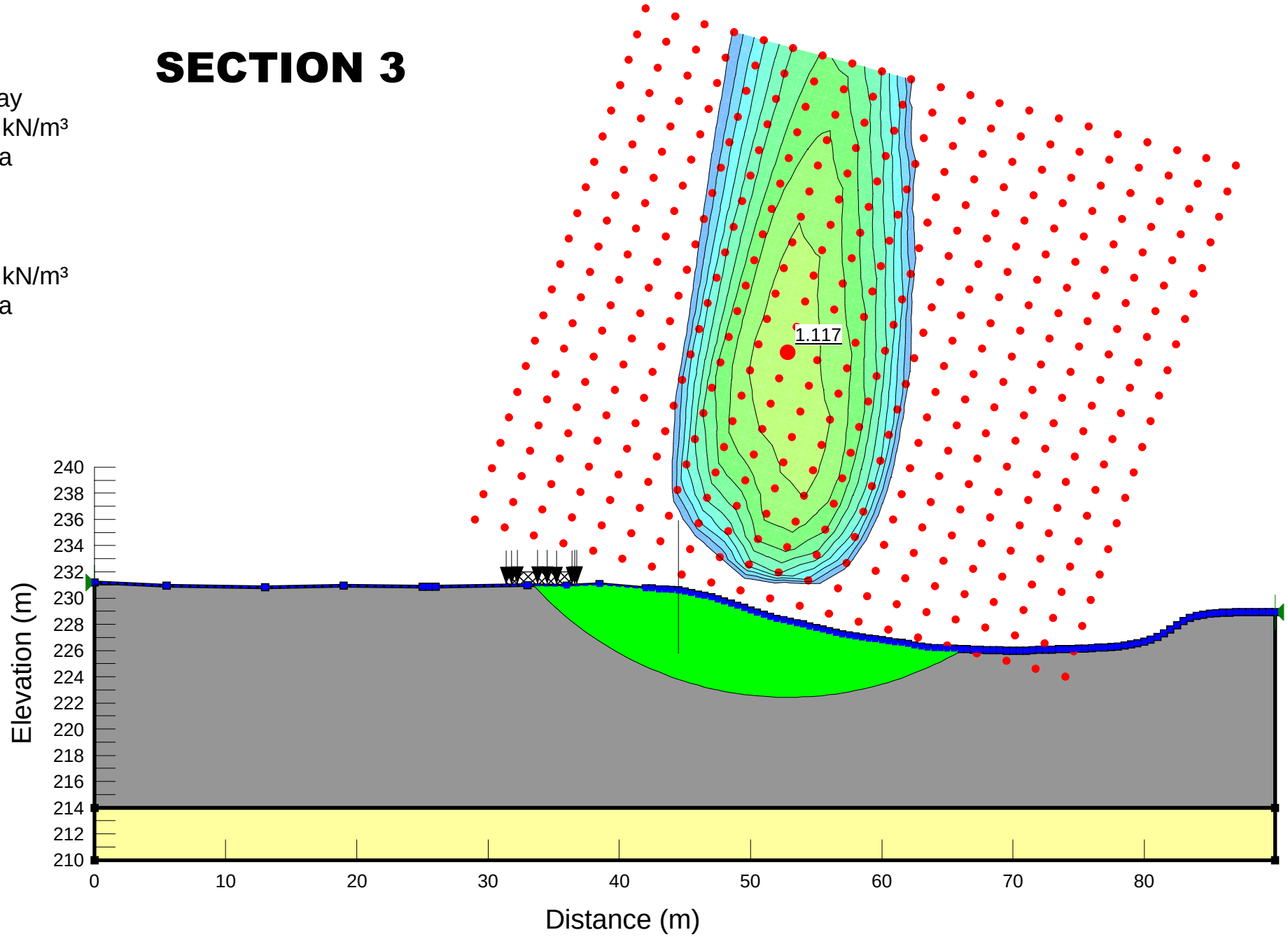



APPENDIX B - SHEET 3

# SECTION 3

Name: Intact Clay
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 15 °

Name: Till
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °

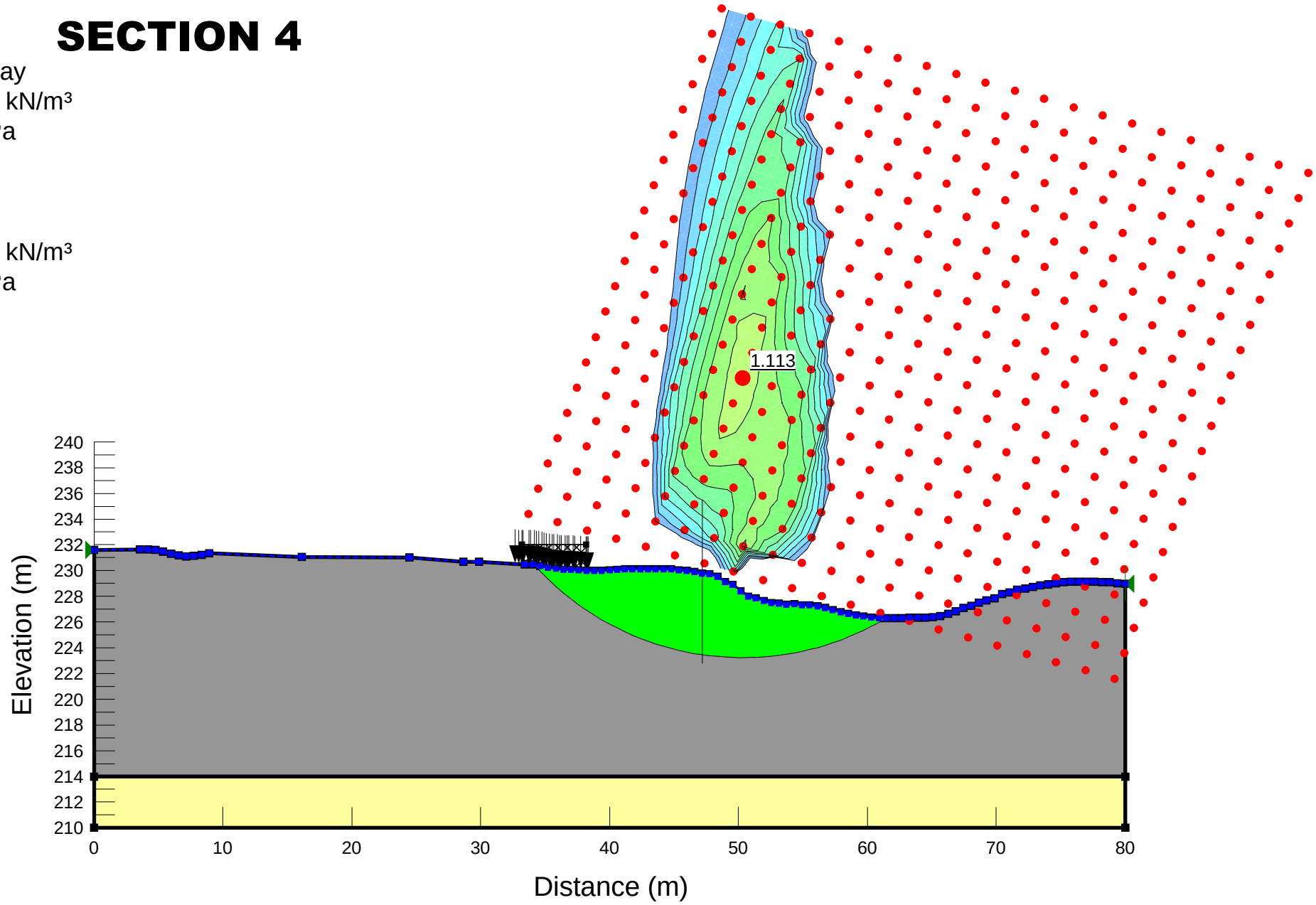



APPENDIX B - SHEET 4

# SECTION 4

Name: Intact Clay
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 15 °

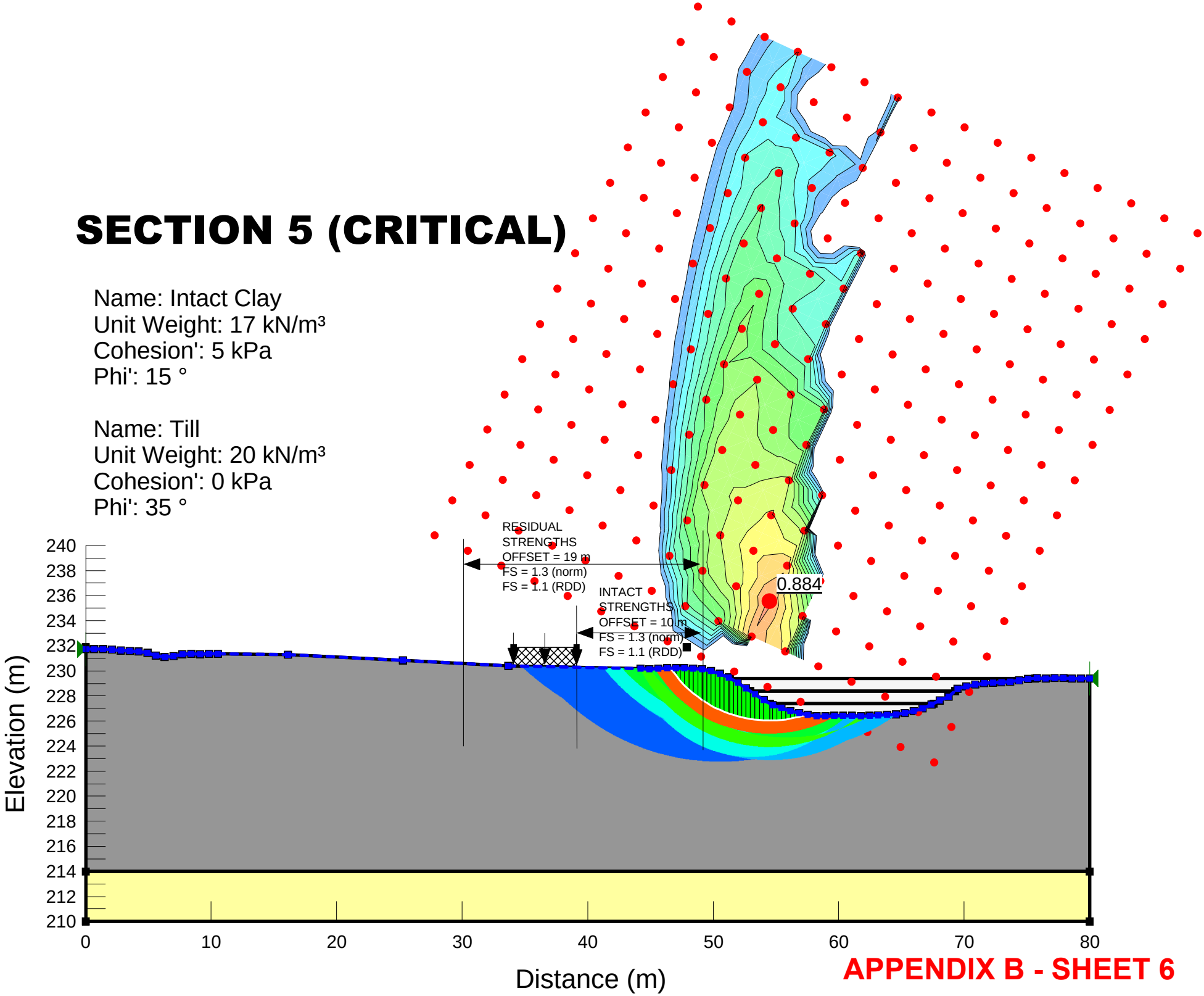
Name: Till
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °



# SECTION 5 (CRITICAL)

Name: Intact Clay
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 15 °

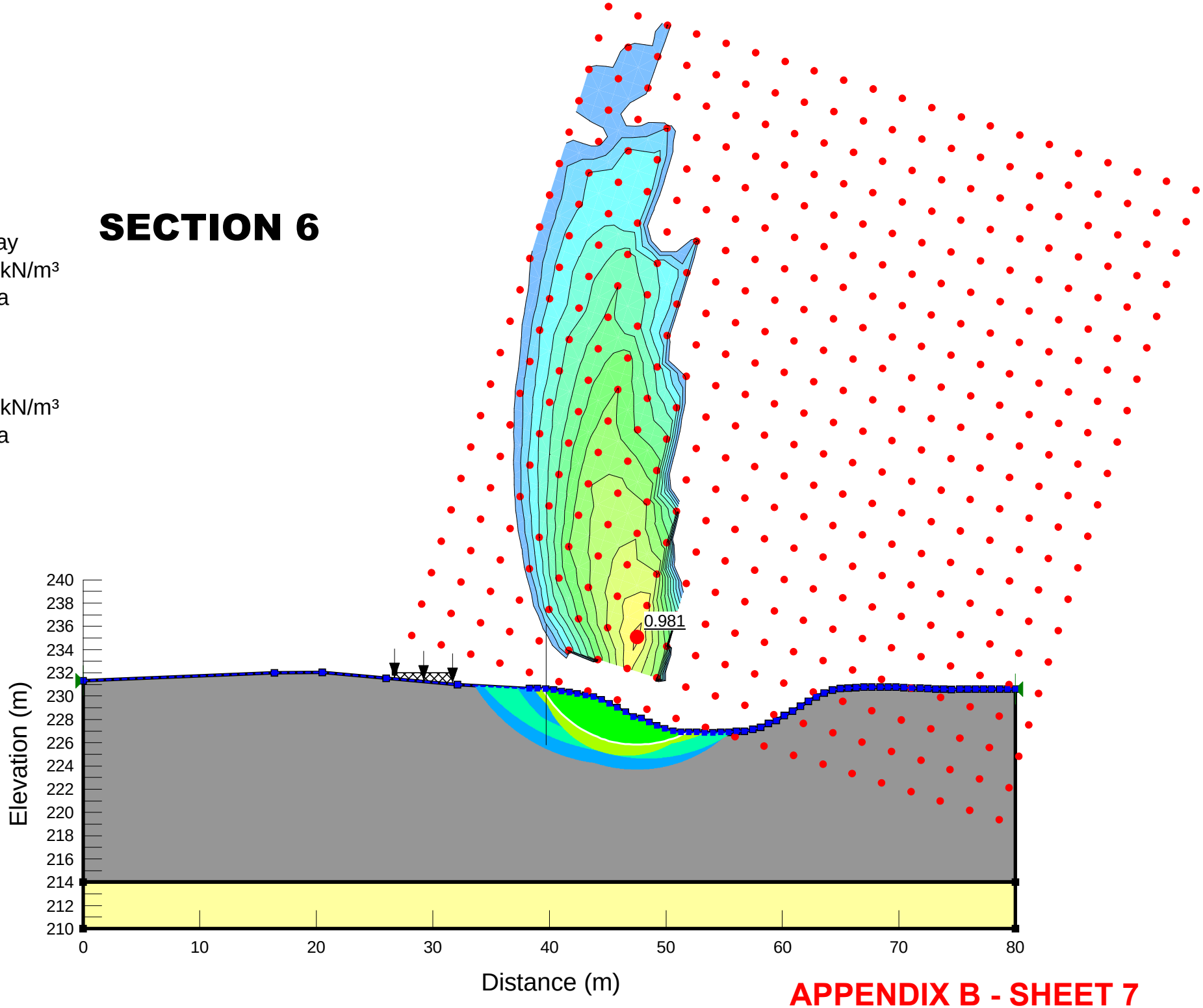
Name: Till
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °



# SECTION 6

Name: Intact Clay
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 15 °

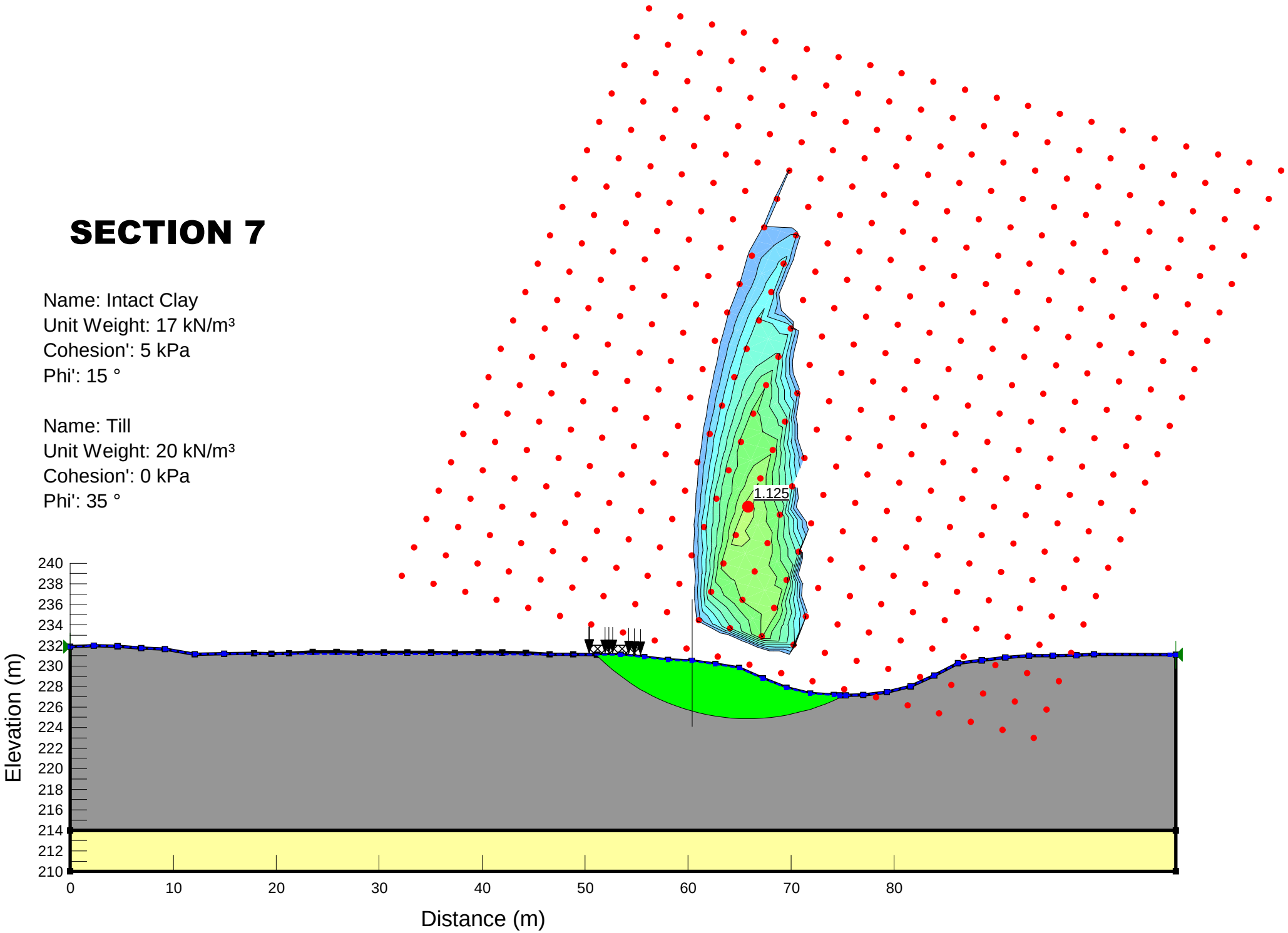
Name: Till
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °



# SECTION 7

Name: Intact Clay
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 15 °

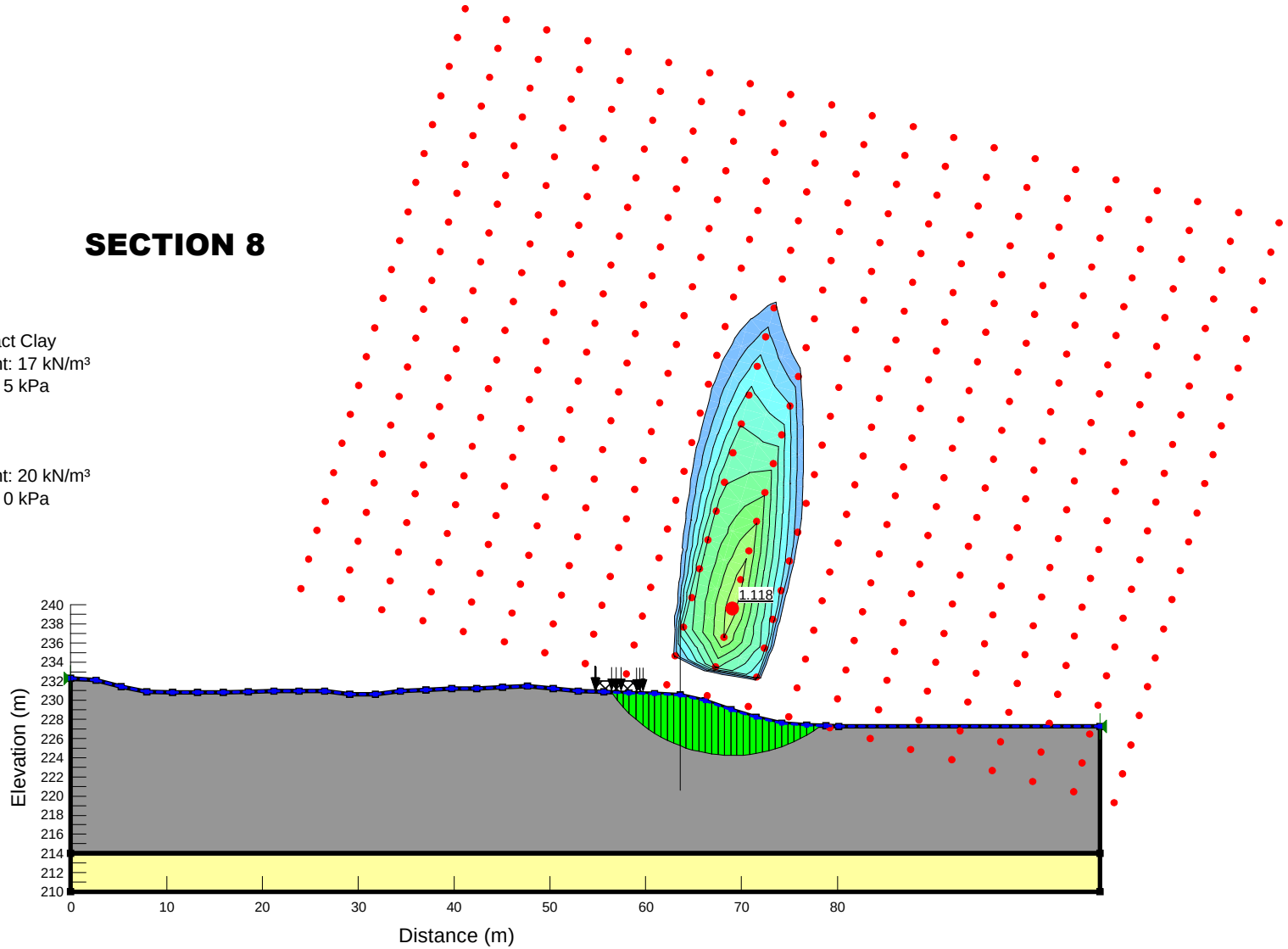
Name: Till
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °



# SECTION 8

Name: Intact Clay
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 15 °

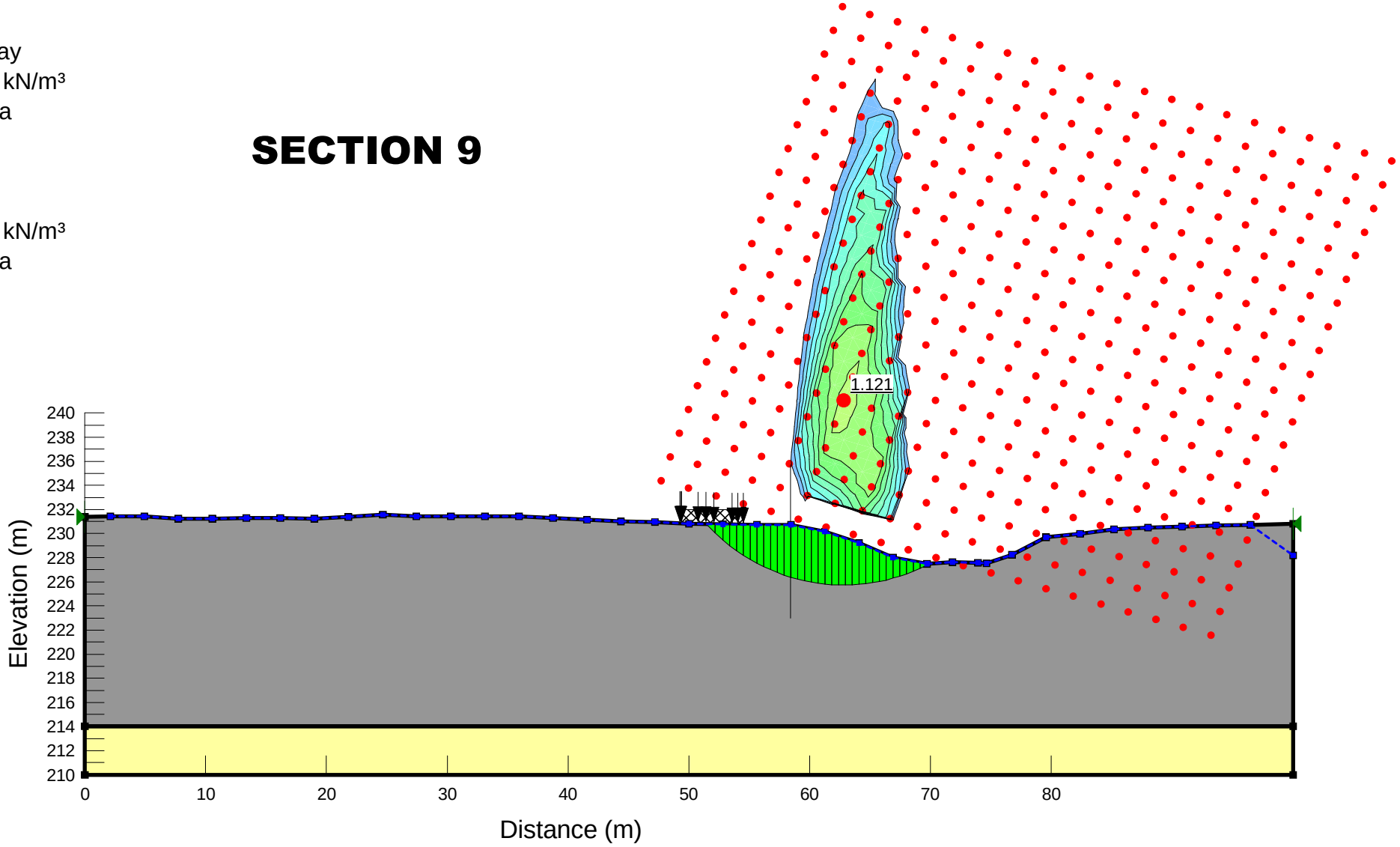
Name: Till
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °



Name: Intact Clay
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 15 °

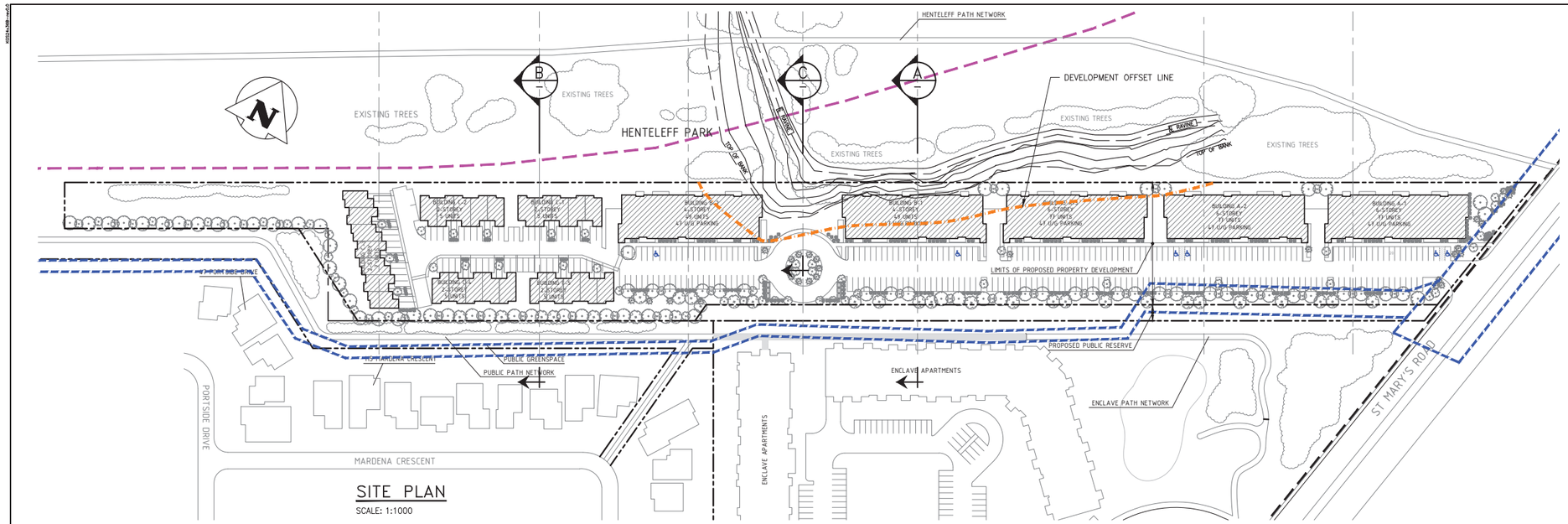
Name: Till
Unit Weight: 20 kN/m³
Cohesion': 0 kPa
Phi': 35 °

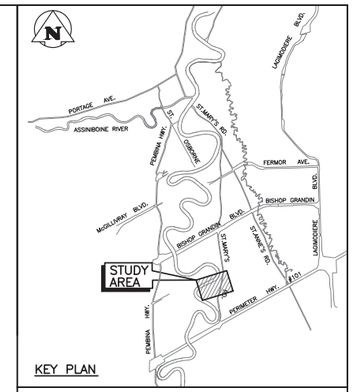
# SECTION 9



# APPENDIX C

# PRELIMINARY SLOPE STABILITY ANALYSIS RESULTS




LEGEND:

- DEVELOPMENT OFFSET LINE
- 232 — LiDAR CONTOUR (m)
- PROPERTY LINE
- FLOODWAY LINE
- PRIMARY LINE OF DEFENCE (PLD)

FLOOD PROTECTION LEVEL
FPL = EL. 231.85m

NOTES:

1. OBSERVED EVIDENCE OF HISTORIC SLOPE MOVEMENT IN THIS AREA.
2. THIS LOT IS LOCATED IN THE DESIGNATED FLOODWAY AREA OF FLOODWAY FRINGE AREA. AnY NEW DEVELOPMENT IN THE FLOODWAY AREA AND FLOODWAY FRINGE AREA IS GOVERNED BY MANITOBA REGULATION 266/91 AND THE CITY OF WINNIPEG CHARTER. PROPERTIES OUTSIDE THE PRIMARY LINE OF DEFENSE (PLD) MUST BE DEVELOPED AT OR ABOVE THE FLOOD PROTECTION LEVEL (FPL), AND MUST CONFORM TO REGULATION 266/91.

REFERENCE:

AFFINITY ARCHITECTURE INC. DWG NO 18004 SHT NO A1.1

| NO. | YY/MM/DD | DESCRIPTION | DESIGN BY | DESIGN CHECK |
|---|---|---|---|---|
| B | 18/08/13 | RE-ISSUED FOR INFORMATION | | |
| A | 18/08/02 | ISSUED FOR INFORMATION | | |

REVISIONS / ISSUE

CLIENT: PROGRESSIVE REAL ESTATE

PROJECT:
NORMAND AVE. PROPERTY DEVELOPMENT
NORMAND AVENUE, WINNIPEG

DWG. DESCRIPTION:
STRUCTURAL
OFFSET BUFFER ZONE
PLAN & SECTIONS

AUTHENTICATION FOR CURRENT REVISION
ENG. STAMP

KGS GROUP CONSULTING ENGINEERS

| | | | |
|---|---|---|---|
| DESIGN BY: | RJL | DATE (YY/MM/DD): | 18/08/01 |
| DESIGN CHECK: | | DATE: | |
| DRAWN BY: | MBB | DATE: | 18/08/01 |
| DWG CHECK: | | DATE: | |

DWG. NO. 18-3405-001 S01 REV: B

Filename: U:\FMS\18-3405-001\18-3405-001_S01 - Tab:S01 Plotted By: mbrischuk 18/08/13 [Mon 4:53pm]
24"x36" /PLOT SCALE: 1"=1'

APPENDIX C - SHEET 1

# APPENDIX D

# FLOODWAY FRINGE AREA REGULATIONS

As of 2018-08-22, this is the most current version available. It is current for the period set out in the footer below. It is the first version and has not been amended.

Le texte figurant ci-dessous constitue la codification la plus récente en date du 2018-08-22. Son contenu était à jour pendant la période indiquée en bas de page. Il s'agit de la première version; elle n'a fait l'objet d'aucune modification.

THE CITY OF WINNIPEG CHARTER
(S.M. 2002, c. 39)

# Designated Floodway Fringe Area Regulation

CHARTE DE LA VILLE DE WINNIPEG
(c. 39 des L.M. 2002)

# Règlement sur la zone limite désignée du canal de dérivation

Regulation 266/91
Registered December 17, 1991

Règlement 266/91
Date d'enregistrement : le 17 décembre 1991

TABLE OF CONTENTS

Section

1 Definitions
2 Application
3 Designated areas
4 Determination of flood protection level
5 Application to construct
6 Establishment of reference mark
7 Floodproofing criteria
8 Additional floodproofing criteria
9 Accessory structures
10 Exceptions
11 Private dykes
12 Protection by primary dykes
13 Criteria for primary dykes
14 Type of flood protection
15 Inspection
16 Notice of compliance
17 Building codes, bylaws, regulations and Acts applicable
18 Repeal
19 Coming into force

SCHEDULES

TABLE DES MATIÈRES

Article

1 Définitions
2 Application
3 Zones désignées
4 Niveau de protection contre les inondations
5 Demande de permis de construire
6 Établissement d'un niveau repère
7 Critères de prévention des inondations
8 Critères de prévention additionnels
9 Constructions annexes
10 Exceptions
11 Digues privées
12 Digues primaires
13 Critères applicables aux digues primaires
14 Type de protection contre les inondations
15 Inspection
16 Avis de conformité
17 Codes du bâtiment, arrêtés, règlements et lois applicables
18 Abrogation
19 Entrée en vigueur

ANNEXES

**Definitions**

**1** In this regulation,

"**Act**" means *The City of Winnipeg Act*; (« *Loi* »)

"**accessory structure**" means a structure described in section 9; (« construction annexe »)

"**Dyking Commissioner**" means The Dyking Commissioner appointed under *The Dyking Authority Act*; (« commissaire des digues »)

"**flood protection level**" means the flood protection level determined under section 4; (« niveau de protection contre les inondations »)

"**hazardous material**" includes a flammable, explosive or toxic material and a buoyant heavy object; (« matériaux dangereux »)

"**minister**" means the Minister of Natural Resources; (« ministre »)

"**permit**" means a permit issued under a by-law passed under subsection 472(1) of the Act; (« permis »)

"**structure**" means a building, storage tank or drilled well, and includes an addition to any of those things, but does not include

(a) an unenclosed playing field, ice rink or similar open-air structure used for a recreational purpose,

(b) a building that is incidental or subordinate to an open-air structure used for a recreational purpose, including a change room and washroom,

(c) playground equipment,

(d) an unenclosed swimming pool,

(e) a fence,

(f) a storage shed or similar structure under 10m$^2$, or

(g) an open-air building, including a cattle shed, used for an agricultural purpose. (« construction »)

**Définitions**

**1** Les définitions qui suivent s'appliquent au présent règlement.

« **commissaire des digues** » Le commissaire des digues nommé en vertu de la *Loi sur l'administration des digues*. ("Dyking Commissioner")

« **construction** » Bâtiments, réservoirs de stockage, puits forés et leurs rajouts, à l'exception

a) des terrains de jeu et des patinoires et autres constructions à ciel ouvert du même genre utilisées à des fins récréatives;

b) des bâtiments accessoires ou annexes à une construction à ciel ouvert utilisée à des fins récréatives, notamment les vestiaires et les toilettes;

c) du matériel de terrain de récréation;

d) des piscines non couvertes;

e) des clôtures;

f) des remises ou autres constructions semblables de moins de 10 m$^2$;

g) des bâtiments à ciel ouvert, y compris les abris à bestiaux, utilisés à des fins agricoles. ("structure")

« **construction annexe** » Construction visée à l'article 9. ("accessory structure")

« ***Loi*** » La *Loi sur la Ville de Winnipeg*. ("Act")

« **matériaux dangereux** » Matériaux inflammables, explosifs ou toxiques et objets lourds flottables. ("hazardous material")

« **ministre** » Le ministre des Ressources naturelles. ("minister")

« **niveau de protection contre les inondations**» Le niveau de protection contre les inondations déterminé conformément à l'article 4. ("flood protection level")

« **permis** » Permis délivré aux termes d'un arrêté pris en vertu du paragraphe 472(1) de la *Loi*. ("permit")

**Application**
**2** This regulation applies in The City of Winnipeg to the designated floodway fringe area and to parcels of land in the designated floodway area that meet the criteria set out in section 494.3(3) of the Act.

**Designated areas**
**3** The designated floodway fringe area and designated floodway area are the floodway fringe and floodway areas identified on the Interim Flood Risk Maps, as designated on February 15, 1980, under the Canada-Manitoba Flood Damage Reduction Agreements, including any amendments thereto, filed at the head office of the Water Resources Branch of the Department of Natural Resources in Winnipeg.

**Determination of flood protection level**
**4** The minister shall determine a flood protection level that is the maximum static water level that occurs during flooding conditions of a certain frequency, as determined by the minister, plus a specific minimum freeboard allowance.

**Application to construct**
**5** An application for a permit to construct a structure shall be made in a form prescribed by council and shall include such of the following information as the designated employee may require:

(a) plans and specifications of the structure;

(b) a plan drawn to scale showing the location of the proposed structure on its site;

(c) a copy of the certificate of title respecting the site;

(d) a plan of survey certified by a Professional Engineer or Manitoba Land Surveyor and referenced to Geodetic Survey of Canada datum showing

(i) the existing and proposed ground elevations on the site where the structure is to be constructed, and

**Application**
**2** Le présent règlement s'applique à la zone limite désignée du canal de dérivation située dans les limites de la Ville de Winnipeg, ainsi qu'aux parcelles de bien-fonds dans la zone désignée du canal de dérivation conformes aux critères énoncés au paragraphe 494.3(3) de la *Loi*.

**Zones désignées**
**3** La zone limite désignée du canal de dérivation et la zone désignée du canal de dérivation constituent la zone limite du canal de dérivation et la zone du canal de dérivation indiquées sur les cartes provisoires des risques d'inondation établies le 15 février 1980 conformément à l'entente Canada-Manitoba modifiée sur la réduction des dommages dûs aux inondations, lesquelles cartes ont été déposées au bureau principal de la Direction des ressources hydriques du ministère des Ressources naturelles à Winnipeg.

**Niveau de protection contre les inondations**
**4** Le ministre détermine le niveau de protection contre les inondations, lequel correspond au niveau statique maximum de l'eau prévu en cas d'inondations survenant à une fréquence reconnue, ainsi qu'il a été déterminé par le ministre, en plus d'une marge minimale pour la hauteur de franc-bord.

**Demande de permis de construire**
**5** Les demandes de permis de construire sont présentées au moyen de la formule prévue à cette fin par le conseil et sont accompagnées des documents énoncés ci-après que l'employé désigné exige :

a) les plans et devis de la construction;

b) un plan à l'échelle indiquant l'endroit où la construction sera située sur l'emplacement;

c) une copie du titre de propriété du bien-fonds;

d) un plan d'arpentage certifié conforme par un ingénieur ou par un arpenteur-géomètre du Manitoba et contenant les renvois aux Levés géodésiques du Canada indiquant

(i) les cotes de nivellement actuelle et projetée de la surface de l'emplacement où la construction sera érigée,

(ii) the floor elevations of the structure;

(e) such other information as the designated employee considers necessary for the purpose of considering the application.

**Establishment of reference mark**

**6(1)** Where a permit holder makes a written request for the establishment of a reference mark, a designated employee shall, within 15 days after the receipt of the request, establish a reference mark at or near the site of the structure or proposed structure indicating the flood protection level applicable to the site and structure.

**6(2)** Upon the written request of a permit holder, a designated employee shall re-establish a reference mark established under subsection (1).

**Floodproofing criteria**

**7(1)** Every structure, other than an accessory structure referred to in section 9, shall be constructed on a site raised by fill or supported by piles.

**7(2)** Where a structure constructed on a site raised by fill has a basement or cellar,

(a) the site shall be raised by impervious fill in accordance with the requirements illustrated in Schedule A;

(b) the elevation of the main floor shall be at least 30 cm above the applicable flood protection level; and

(c) the structure is situated on pervious soil, the design of the structure shall be certified by a Professional Engineer as capable of withstanding hydrostatic and uplift pressures by a static water level at the flood protection level.

(ii) la cote de nivellement du plancher de la construction;

e) les autres renseignements dont l'employé désignée estime devoir tenir compte dans l'examen de la demande.

**Établissement d'un niveau repère**

**6(1)** Sur réception d'une demande écrite d'établissement d'un niveau repère présentée par le titulaire d'un permis, l'employé désigné établit, dans les 15 jours qui suivent la réception de la demande, un niveau repère indiquant le niveau de protection contre les inondations à l'emplacement ou près de l'emplacement de la construction ou du projet de construction.

**6(2)** Sur réception d'une demande écrite présentée par le titulaire d'un permis, l'employé désigné établit de nouveau le niveau repère prévu au paragraphe (1).

**Critères de prévention des inondations**

**7(1)** Toutes les constructions, à l'exception des constructions annexes visées à l'article 9, sont érigées sur des emplacements remblayés ou sur pilotis.

**7(2)** Dans le cas des constructions érigées sur des emplacements remblayés et qui sont dotées d'un sous-sol ou d'une cave :

a) l'emplacement est remblayé au moyen de matériaux de remplissage imperméables, conformément aux exigences indiquées à l'annexe A;

b) la cote de nivellement du rez-de-chaussée est au moins 30 cm au-dessus du niveau de protection contre les inondations;

c) si la construction est érigée sur un sol perméable, un ingénieur doit attester qu'elle est conçue de manière à résister à la pression hydrostatique et à la sous-pression qui s'exercent lorsque le niveau hydrostatique correspond au niveau de protection contre les inondations.

**7(3)** Where a structure constructed on a site raised by fill has no basement or cellar,

(a) the site shall be raised by fill in accordance with the requirements illustrated in Schedule B;

(b) the elevation of the main floor shall be not less than the applicable flood protection level; and

(c) the top of the fill shall be not more than 30 cm below the applicable flood protection level.

**7(4)** Where a structure is supported by piles,

(a) the design of the foundation shall be certified by a Professional Engineer;

(b) the structure shall be supported in accordance with the requirements illustrated in Schedule C or an equivalent support system;

(c) the structure shall be so constructed as not to be buoyant when the surface of any flood waters is higher than the bottom of the horizontal members supporting the structure; and

(d) the elevation of a floor containing finished or occupied space shall be at least 1.0 m above the applicable flood protection level.

**Additional floodproofing criteria**

**8** In addition to the floodproofing criteria set out in section 7, every structure, other than an accessory structure referred to in section 9, shall meet the following requirements:

(a) all windows, exterior doors and other exterior openings, including openings between a house and an attached garage, shall be located above the applicable flood protection level;

**7(3)** Dans le cas des constructions érigées sur un emplacement remblayé et qui ne sont pas dotées d'un sous-sol ou d'une cave :

a) l'emplacement est remblayé conformément aux exigences indiquées à l'annexe B;

b) la cote de nivellement du rez-de-chaussée n'est pas au-dessous du niveau de protection contre les inondations;

c) la surface du remblayage n'est pas plus de 30 cm au-dessous du niveau de protection contre les inondations.

**7(4)** Dans le cas des constructions érigées sur pilotis :

a) la conception de la fondation est attestée par un ingénieur;

b) les travaux sont effectués conformément aux exigences indiquées à l'annexe C ou à des exigences similaires;

c) la construction est érigée de manière à ce qu'aucune poussée hydrostatique ne soit exercée lorsque la surface des eaux de crues s'élève au-dessus de la face inférieure des membres horizontaux sur lesquels repose la construction;

d) la cote de nivellement du plancher des locaux finis ou occupés est au moins 1,0 mètre au-dessus du niveau de protection contre les inondations.

**Critères de prévention additionnels**

**8** En plus des critères énoncés à l'article 7, toutes les constructions, à l'exception des constructions annexes visées à l'article 9, doivent répondre aux exigences suivantes :

a) les fenêtres, les portes extérieures et les autres ouvertures extérieures, y compris les ouvertures entre une habitation et un garage attenant, sont toutes au-dessus du niveau de protection contre les inondations;

(b) the electrical service and panelboard shall be located above the main floor unless the existing service and panelboard located below the main floor within a structure existing before August 15, 1981 is being replaced or added to in the same location;

(c) the potable water shut-off valve shall be located on the main floor with no take-off fittings on the service pipe side of the valve and shall have a dual check valve type backflow preventer to provide protection against possible contamination of the potable water supply;

(d) if the structure has floor space below the applicable flood protection level, the drain between the structure and a septic or holding tank or a common sanitary sewer line shall have a backwater valve.

**Accessory structures**

**9(1)** An accessory structure shall comply with the floodproofing criteria set out in this section.

**9(2)** Where an accessory structure is a livestock barn, granary, farm machinery shed, attached garage or other building used for the storage of agricultural produce, or a workshop or shed used for the storage of immovable equipment or material or hazardous material,

(a) the floor elevation shall be not more than 30 cm below the applicable flood protection level; and

(b) the elevation of the top of the fill shall be not more than 60 cm below the applicable flood protection level.

**9(3)** Where an accessory structure is a detached garage,

(a) the floor elevation shall be not more than 1.5 m below the applicable flood protection level;

b) la boîte de dérivation et le tableau de distribution sont installés au-dessus du rez-de-chaussée, à moins qu'il s'agisse remplacer une boîte ou un tableau existant ou d'ajouter une boîte ou un tableau supplémentaire au-dessous du rez-de-chaussée dans une construction érigée avant le 15 août 1981;

c) la soupape d'arrêt d'eau potable se trouve au niveau du rez-de-chaussée, elle ne comporte aucun raccord de prise d'eau en amont et elle est munie d'un dispositif anti-vide à clapet de non-retour double servant à empêcher toute contamination possible de la source d'eau potable;

d) si la construction a une superficie de plancher au-dessous du niveau de protection contre les inondations, le branchement d'égout entre la construction à la fosse septique, le bac à eaux usées ou l'égout sanitaire public doit être muni d'un clapet anti-refoulement.

**Constructions annexes**

**9(1)** Les constructions annexes doivent être conformes aux critères de prévention des inondations énoncées dans le présent article.

**9(2)** Les halles aux bestiaux, les greniers à céréales, les hangars à machines agricoles, les garages attenants ou autres bâtiments servant au stockage de produits agricoles, les ateliers et les remises servant à abriter de l'équipement ou du matériel agricole fixe ou des matières dangereuses doivent satisfaire aux exigences suivantes :

a) la surface du plancher n'est pas plus de 30 cm au-dessous du niveau de protection contre les inondations;

b) la surface du remblai n'est pas plus de 60 cm au-dessous du niveau de protection contre les inondations.

**9(3)** Si la construction annexe est un garage isolé :

a) la surface du plancher n'est pas plus de 1,5 m au-dessous du niveau de protection contre les inondations;

(b) if constructed of wood, wood by-products or other material susceptible to water damage, the garage shall be supported by a foundation constructed of water resistant material, and the elevation of the top of the foundation shall be not more than 50 cm below the applicable flood protection level; and

(c) any immovable equipment or material or hazardous material stored in the structure shall be stored 1.0 m above the floor level.

**9(4)** Where an accessory structure is a storage tank for fuel oil, gasoline or any other liquid or solid, the storage tank shall

(a) be designed, and the installation certified, by a Professional Engineer;

(b) be situated above the applicable flood protection level or buried underground;

(c) be anchored so as to prevent flotation; and

(d) have the vent and filler pipes extend above the applicable flood protection level.

**9(5)** Where an accessory structure is a drilled well, the well casing shall

(a) extend upward at least to the applicable flood protection level; or

(b) be sealed at the top.

**Exceptions**

**10(1)** If a designated employee is reasonably satisfied that, because of the size of a building site, compliance with the requirements of Schedule A or B is impossible or impractical, the designated employee

(a) may vary the berm width and side slope requirements of Schedule A or B; and

(b) may require, as a condition of the variance, that retaining walls be constructed and maintained in accordance with the issued permit to ensure slope protection and prevent water run-off on to adjoining property.

b) si le garage a été construit en bois, en produits du bois ou en un autre matériau susceptible de se détériorer sous l'effet de l'eau, sa fondation est en un matériau imperméable et la surface de la fondation n'est pas plus de 50 cm au-dessous du niveau de protection contre les inondations;

c) l'équipement et le matériel fixe ainsi que les matières dangereuses entreposés dans le garage sont placés au moins un mètre au-dessus de la surface du plancher.

**9(4)** Si la construction annexe est un réservoir de stockage de mazout, d'essence ou de tout autre liquide ou solide, le réservoir :

a) est conçu et son installation est attestée par un ingénieur;

b) est installé au-dessus du niveau de protection contre les inondations ou il est enterré;

c) est assujetti de manière à ne pas pouvoir flotter;

d) est muni de tuyaux de remplissage et d'aération dont l'orifice s'élève au-dessus du niveau de protection contre les inondations.

**9(5)** Si la construction annexe est un puits foré, l'extrémité supérieure du tubage est, selon le cas :

a) prolongée jusqu'au niveau de protection contre les inondations ou jusqu'à un niveau supérieur;

b) scellée.

**Exceptions**

**10(1)** Si l'employé désigné estime, à cause des dimensions de l'emplacement d'une construction, qu'il est impossible ou particulièrement difficile de répondre aux exigences de l'annexe A ou B, il peut :

a) modifier les exigences de l'annexe A ou B en ce qui a trait à la largeur de la risberme et à la pente latérale;

b) exiger, comme condition de la modification, la construction et l'entretien de murs de soutènement en conformité avec le permis délivré afin de maintenir la stabilité de la pente et d'empêcher l'écoulement des eaux sur les biens-fonds contigus.

**10(2)** If a designated employee is reasonably satisfied that compliance with the floodproofing criteria is impossible or impractical or would result in a structure that would significantly detract in appearance from neighbouring properties, the designated employee may vary the floodproofing criteria in respect of

(a) a new structure to be constructed on one of a small number of remaining building sites, on the only remaining building site or on newly subdivided building sites in an area that is almost fully developed with buildings;

(b) proposed work that constitutes reconstruction of, or an addition or accessory to, a lawfully existing building; or

(c) the replacement of a structure that is destroyed by fire or other peril.

**10(3)** A designated employee may vary the floodproofing criteria after completion of a foundation by no more than 10 cm in the elevation of foundations, finished floors and fills and in the width of berms.

**Private dykes**
**11** No person shall construct a dyke for flood protection of an existing structure unless it is constructed in accordance with the requirements illustrated in Schedule D.

**Protection by primary dykes**
**12** A structure protected by a primary dyke or an extension of a primary dyke is deemed to comply with the floodproofing criteria.

**Criteria for primary dykes**
**13** A primary dyke and an extension of a primary dyke shall

(a) be located entirely within the floodway fringe area;

**10(2)** Si l'employé désigné estime qu'une construction ne peut pas ou peut difficilement être érigée conformément aux critères de prévention des inondations ou ne s'harmoniserait pas avec les bâtiments environnants, il peut modifier les critères de prévention des inondations à l'égard des constructions suivantes :

a) une nouvelle construction érigée sur le dernier ou l'un des derniers terrains à bâtir ou sur un terrain nouvellement loti dans un quartier presque complètement aménagé;

b) un projet de reconstruction d'un bâtiment existant ou d'une construction annexe à ce bâtiment;

c) le remplacement d'une construction détruite par le feu ou un autre sinistre.

**10(3)** L'employé désigné peut modifier de 10 cm au maximum les critères de prévention des inondations d'une fondation déjà construite en ce qui concerne la cote de nivellement des fondations, du plancher fini et de la surface du remblai, ainsi que la largeur de la risberme.

**Digues privées**
**11** Il est interdit d'ériger une digue dans le but de protéger une construction existante contre les inondations à moins que la digue ne soit conforme aux exigences illustrées à l'annexe D.

**Digues primaires**
**12** Les constructions protégées par des digues primaires ou par le prolongement d'une digue primaire sont réputées conformes aux critères de prévention des inondations.

**Critères applicables aux digues primaires**
**13** Les digues primaires et tout prolongement d'une digue primaire doivent répondre aux exigences suivantes :

a) elles sont situées à l'intérieur de la zone limite du canal de dérivation;

(b) be located, designed and constructed to the standard and elevation approved by the city administrator and The Dyking Commissioner but in no case shall the elevation be less than the flood protection level or the top width be less than 9.2 m at the flood protection level;

(c) have adequate permanent works, as approved by the city administrator, for the removal of water as a result of internal drainage and seepage within the protected area; and

(d) have measures, as approved by the city administrator, that are operable during flood conditions to close openings through the primary dyke.

b) elles sont situées, conçues et construites conformément aux normes et aux cotes de nivellement approuvées par l'administrateur de la ville et le commissaire des digues, mais elles ne doivent en aucun cas être moins élevées que le niveau de protection contre les inondations, et la largeur de la digue ne doit pas être inférieure à 9,2 mètre au niveau de protection contre les inondations;

c) elles sont pourvues d'un système de pompage, approuvé par l'administrateur de la ville, pour l'élimination de l'eau qui s'accumule à l'intérieur de la zone protégée par suite du drainage interne;

d) elles sont pourvues de dispositifs, approuvés par l'administrateur de la ville, servant à obstruer, pendant la crue des eaux, les canalisations qui traversent la digue primaire.

**Type of flood protection**
**14** Where a new structure is proposed for construction, the city administrator and The Dyking Commissioner may determine whether the structure must conform with the floodproofing criteria or the criteria for primary dykes.

**Type de protection contre les inondations**
**14** L'administrateur de la ville et le commissaire des digues peuvent déterminer si une nouvelle construction projetée doit être conforme aux critères de prévention des inondations ou aux critères applicables aux digues primaires.

**Inspection**
**15** A designated employee may at any stage of, and after the completion of, the construction of a structure make an inspection to determine whether the structure complies with this regulation.

**Inspection**
**15** L'employé désigné peut procéder à l'inspection d'une construction, à toute étape de son érection ou après son érection, afin de déterminer si elle est conforme au présent règlement.

**Notice of compliance**
**16(1)** Where a structure described in a permit is completed and inspected, a designated employee shall, upon a request of the owner, issue to the owner a written notice indicating whether the structure complies with this regulation.

**16(2)** Where a designated employee carries out an inspection under section 15 and determines that the structure does not comply with this regulation, the city may register in the Land Titles Office, in accordance with *The Real Property Act*, a notice to that effect against the land on which the structure is located.

**Avis de conformité**
**16(1)** Après l'érection et l'inspection d'une construction décrite dans un permis, l'employé désigné délivre, au propriétaire qui en fait la demande, un avis écrit indiquant si la construction est conforme ou non au présent règlement.

**16(2)** Lorsqu'un employé désigné procède à une inspection aux termes de l'article 15 et détermine que la construction n'est pas conforme au présent règlement, la ville peut enregistrer au Bureau des titres fonciers, conformément à la *Loi sur les biens réels*, un avis de non-conformité à l'égard du bien-fonds sur lequel la construction a été érigée.

**Building codes, by-laws, regulations and Acts applicable**

**17** The requirements under this regulation are in addition to requirements under any applicable building code, by-law, regulation or Act of the Legislature.

**Repeal**

**18** Manitoba Regulation 439/88 R is repealed.

**Coming into force**

**19** This regulation comes into force on proclamation of sections 494.3 and 494.4 of *The City of Winnipeg Amendment Act*, S.M. 1991-92, c. 15.

**Codes du bâtiment, arrêtés, règlements et lois applicables**

**17** Les exigences énoncées dans le présent règlement s'ajoutent aux exigences des codes du bâtiment, des arrêtés, des règlements et des lois de l'Assemblée législative applicables.

**Abrogation**

**18** Le *règlement du Manitoba* 439/88 R est abrogé.

**Entrée en vigueur**

**19** Le présent règlement entre en vigueur à la date de proclamation des articles 494.3 et 494.4 de la *Loi modifiant la Loi sur la Ville de Winnipeg*, c. 15 des *L.M. de 1991-92*.

SCHEDULES 1 TO D

[These schedules are not available online. Copies can be purchased by calling Statutory Publications at 945-3101.]

ANNEXES A À D

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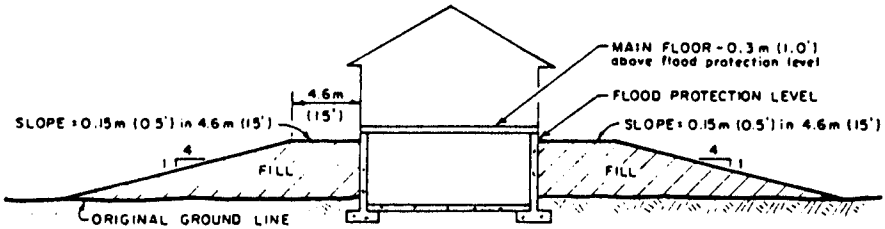
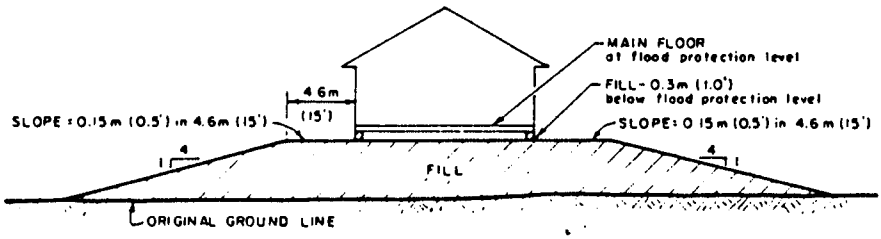
# SCHEDULE A

## STRUCTURE WITH A BASEMENT OR CELLAR



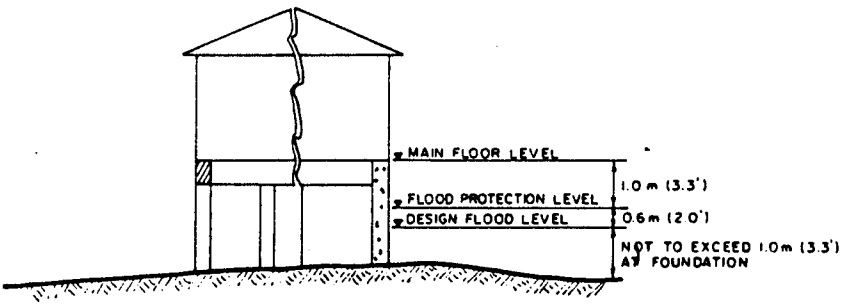

# SCHEDULE B

## STRUCTURE WITH NO BASEMENT OR CELLAR




# SCHEDULE C

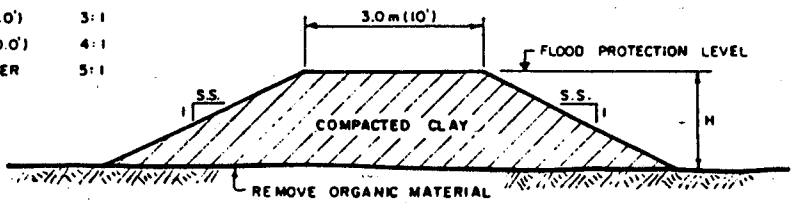
## ELEVATED STRUCTURE




# SCHEDULE D

## DYKE CROSS-SECTION

| DYKE HEIGHT (H) | SIDE SLOPE (SS) |
|---|---|
| 0.3m (1.0') TO 1.5m (5.0') | 3:1 |
| 1.5m (5.0') TO 3.0m (10.0') | 4:1 |
| 3.0m (10.0') OR GREATER | 5:1 |



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