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Geotechnical Investigation
Proposed Lot Severances
930 Smith Road

Ottawa, Ontario

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Submitted to:

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Proposed Lot Severances
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June 6, 2024
Project: 100812.001

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the proposed lot severance to be located at 930 Smith Road in Ottawa, Ontario.

The purpose of the investigation was to identify the general subsurface and groundwater conditions at the site by means of a limited number of boreholes and, based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

2.0 BACKGROUND

2.1 Project Description

Based on preliminary plans and information provided to GEMTEC Consulting Engineers and Scientists Limited (GEMTEC), it is understood that the existing parcel of land will be severed into up to seven land parcels.

It is anticipated that a single-family home will be constructed on each land parcel in the future. Details of the proposed buildings were not available at the time of preparing this report but are likely to be typical two storey timber framed houses with a partial or full basement level. Each lot will have access to Smith Road at the south or east side of the site.

The site is currently in use as agricultural land. The site is approximately rectangular in shape with plan dimensions of about 330 metres by 180 metres. The site is relatively flat, with a slope running approximately northwest to southeast along the south and west side of the site. A more detailed description of the sloping ground at the site is provided in Section 2.3 of this report. The site location and configuration are shown on the Site Plan, Figure 1.

It is understood that the City of Ottawa has mapped the area in which the severed lots will be located as an area of unstable slopes due in part to the proximity of McKinnon's Creek. One of the conditions on the Pre-Consultation Meeting is to assess if the site is suitable, or can be made suitable, for development based on the proximity to the potentially unstable slopes of McKinnon's Creek. As such, a slope stability assessment is required to determine the factor of safety against global stability and the safe setback distance (i.e., the limit of hazard lands) for the proposed development of the land severances.

GEMTEC has also completed a hydrogeological investigation at the site. The results of that investigation are provided in the following report:

- Draft report titled "Hydrogeological Investigation & Terrain Analysis, Proposed Residential Severances, 930 Smith Road, Ottawa, Ontario" dated March 28, 2023 (Report No. 100812.001).

2.2 Site Geology

Surficial geology maps of the Ottawa area indicate that the site is underlain by thick deposits of sensitive silty clay. Bedrock geology maps of the area show that the overburden deposits are underlain by shale bedrock of the Billings formation. Drift thickness mapping indicates that the bedrock surface is expected at depths of about 15 to 25 metres.

2.3 Description of Slopes, Southwest Corner of Site and McKinnon's Creek

2.3.1 Slope on Southwest Corner of Site

A site reconnaissance was carried out on July 5, 2021, by a member of GEMTEC's engineering staff.

On that date the geometry of the slope along the south side of the property was measured at a total of three locations using a Trimble R10 GPS survey instrument. The cross sections were positioned at the site by GEMTEC personnel at key locations based on slope geometry and height. The locations of the cross sections considered are provided on Figure 1. The geometries of the cross sections considered are summarized in Table 2.1.

In general, the slopes are vegetated with grass, shrubs, and small to large trees. No signs of erosion or overall slope instability (i.e., rotational failures) were observed along the slopes.

Table 2.1 – Slope Cross Section Height and Slope Inclination, Southwest of Site

Cross Section	Slope Height (metres)	Overall inclination from horizontal (degrees)
A-A	4.9	17
B-B	4.9	18
C-C	5.1	16

2.3.2 McKinnon's Creek

A site reconnaissance was carried out on October 25, 2023, by a member of GEMTEC's engineering staff.

On that date the geometry of the slope along the west side of McKinnon's Creek was measured at a total of five locations using a combination of precision GPS and hand survey instruments. The cross sections were positioned at the site by GEMTEC personnel at key locations based on slope geometry and height. The locations of the cross sections considered are provided on Figure 1. The geometries of the cross sections considered are summarized in Table 2.2.

In general, the slopes along McKinnon's are vegetated with shrubs, and small to large trees. In this area, erosion has resulted in steep side slopes devoid of vegetation, and ongoing toe erosion in the form of sloughing of the creek channel is evident.

Signs of overall slope instability (i.e., previous rotational failures, fallen and leaning trees, and near vertical slopes) were observed along the creek.

Table 2.2 – Slope Cross Section Height and Slope Inclination, McKinnon's Creek

Cross Section	Slope Height (metres)	Overall inclination from horizontal (degrees)
D-D	7.9	19 to 39
E-E	8.5	13 to 34
F-F	8.0	17 to 29
G-G	8.4	9 to 63
H-H	8.5	11 to 41

3.0 SUBSURFACE INVESTIGATION

On August 12, 2021 a total of four boreholes (numbered 21-01, 21-02A, 21-02B, and 21-03) were advanced at the site using a rubber tire ATV mounted hollow stem auger drill rig supplied and operated by CCC Geotechnical and Environmental Drilling of Ottawa, Ontario.

The boreholes were advanced to depths ranging from about 6.1 to 8.2 metres below ground surface.

Standard penetration tests were carried out in boreholes 21-01, 21-02A, and 21-03 and samples of the soils encountered were recovered using a 50-millimetre diameter split barrel sampler. Borehole 21-02B was advanced adjacent to borehole 21-02A to obtain one relatively 'undisturbed' sample of the silty clay deposit. In-situ vane shear testing was carried out, where possible, in the boreholes to measure the undrained shear strength of the silty clay.

Well screens were sealed in the overburden at all borehole locations, except borehole 20-02B, to measure the groundwater levels and to allow for hydraulic conductivity testing.

The fieldwork was supervised throughout by a member of our engineering staff who directed the drilling operations, logged the samples and carried out the in-situ testing. Following the fieldwork, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were tested for water content, Atterberg limits, shrinkage limits, and grain size distribution testing. One sample of the soil recovered from borehole 20-02A was sent to an accredited laboratory for basic chemical testing relating to corrosion of buried concrete and steel.

The borehole locations were positioned in the field by GEMTEC personnel using our Trimble R10 GPS survey instrument. The ground elevations at the boreholes were also determined using our Trimble R10 GPS survey instrument. The elevations are referenced to geodetic datum.

Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix A. The results of the laboratory tests are provided on the borehole logs and in Appendix B. The results of chemical testing completed the soil sample are provided in Appendix C. The approximate locations of the test holes are shown on the Site Plan, Figure 1.

4.0 SUBSURFACE CONDITIONS

4.1 General

The borehole logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of drilling, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the test locations may vary from the conditions encountered in the boreholes. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities in the area.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and GEMTEC does not guarantee descriptions as exact but infers accuracy to the extent that is common in current geotechnical practice.

The following presents an overview of the subsurface conditions encountered in the boreholes advanced during this investigation.

4.2 Topsoil

A layer of topsoil was encountered at the ground surface at the borehole locations with a thickness ranging from about 130 to 180 millimetres.

4.3 Silty Sand

A native deposit of silty sand was encountered below the topsoil in borehole 21-02 with a thickness of about 150 millimetres.

4.4 Silty Clay

Native deposits of silty clay were encountered in all of the boreholes. The silty clay was not fully penetrated in all the boreholes, with the exception of borehole 21-03. At borehole 21-03 the base of the silty clay was encountered at a depth of 5.3 metres. At the other locations the silty clay was proven to depths ranging from about 7.3 to 8.2 metres below ground surface.

The upper part of the silty clay in the boreholes is weathered to a grey brown crust. The weathered silty clay crust has a thickness ranging from about 2.8 to 5.2 metres and extends to depths ranging from about 3.1 to 5.3 metres below the existing ground surface.

Standard penetration tests carried out in the weathered silty clay crust gave N values ranging from 4 to 16 blows per 0.3 metres of penetration. The results of the in-situ testing reflect a stiff to very stiff consistency.

A grain size distribution test was undertaken on one sample of the weathered silty clay crust from borehole 21-01. The results are provided in Appendix B and are summarized in Table 4.1.

Table 4.1 – Summary of Grain Size Distribution Test (Weathered Crust)

Borehole ID	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
21-01	3	1.5 – 2.1	0	1	21	78

The results of the Atterberg limit tests carried out on samples of the weathered silty clay crust are provided in Appendix B. The results are summarized in Table 4.2. This testing indicates that the samples of weathered silty clay have a medium plasticity. The water content of the weathered silty clay ranges from about 25 to 51 percent which is typically below the measured liquid limit value.

Table 4.2 – Summary of Atterberg Limit Test Results (Weathered Crust)

Borehole ID / Sample No.	Depth (metres)	Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index
21-01 / 2	0.8 to 1.4	33	55	29	26
21-01 / 4	2.3 to 2.9	38	58	30	28

Borehole ID / Sample No.	Depth (metres)	Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index
21-01 / 6	3.8 to 4.4	27	52	27	25
21-02A / 2	0.8 to 1.4	37	53	24	29
21-02A / 4	2.3 to 2.9	49	57	25	32
21-03 / 3	1.5 to 2.1	37	56	23	33

Below the weathered zone, the silty clay is grey in colour. Boreholes 21-01 and 21-02A/B were terminated within the grey silty clay at depths ranging from 6.7 to 8.3 metres below ground surface.

Standard penetration tests carried out in the grey silty clay gave N values of Static Weight of Hammer "WH" to 2 blows per 0.3 metres of penetration. In-situ vane shear strength tests carried out in the grey silty clay gave undrained shear strengths ranging from about 40 to 65 kilopascals, which indicate a firm to stiff consistency, generally decreasing with depth to a local minimum value.

The silty clay has a sensitivity ranging from about 5 to 14. Based on Table 4.2 "Soil Sensitivity (data from Rankka et al. 2004)." from the Canadian Foundation Engineering Manual 5th Edition (2003), is considered to have a low to medium sensitivity (i.e., sensitivity less than 30).

The results of Atterberg limit tests carried out on one sample of the grey silty clay are provided in Appendix B. The results are summarized in Table 4.3. This testing indicates that the sample of grey silty clay tested has a medium plasticity. The water content of the grey silty clay ranges from about 61 to 81 percent, which is above the liquid limit.

Table 4.3 – Summary of Atterberg Limit Test Results (Grey Silty Clay)

Borehole ID / Sample No.	Depth (metres)	Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index
21-02A / 6	4.6 to 5.2	77	57	37	20
21-03 / 7	4.6 to 5.2	63	57	28	29

4.5 Glacial Till

A deposit of glacial till was encountered below the silty clay in borehole 21-03. The glacial till was not fully penetrated in the borehole but was proven to depth of about 6.1 metres below ground surface.

The glacial till is a heterogeneous mixture of all grain sizes, which at this site, can be described as grey silty sand with some gravel and some clay. Although not encountered in the borehole directly, the glacial till deposits in this area are known to contain cobbles and boulders.

One standard penetration test carried out in the glacial till deposit gave an N value of 28 blows per 0.3 metres of penetration, which indicates a compact relative density.

One grain size distribution test was undertaken on a sample of the glacial till from borehole 21-03. The results are provided in Appendix B and are summarized in Table 4.4.

The water content of one sample of the glacial till was about 9 percent.

Table 4.4 – Summary of Grain Size Distribution Test (Glacial Till)

Borehole ID	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
21-03	8	5.3 – 5.9	16	44	24	16

4.6 Auger Refusal

Auger refusal was encountered in borehole 21-02A at a depth of about 8.2 metres below ground surface (elevation of about 72.7 metres, geodetic). Auger refusal may represent the upper surface of bedrock or the presence of cobbles and boulders.

4.7 Groundwater Levels

Standpipe piezometers (monitoring wells) were installed in the overburden at boreholes 21-01, 21-02A, and 21-03. The groundwater levels measured in the wells are summarized in Table 4.5. The groundwater levels should be anticipated to vary seasonally and may be higher during wet periods of the year such as the early spring or following periods of precipitation.

Table 4.5 – Groundwater Depth and Elevation

Borehole ID.	Ground Surface Elevation (metres)	Groundwater Depth (metres)	Groundwater Elevation (metres)	Date of Reading
21-01	78.94	3.7	75.2	September 13, 2021
21-02A	80.95	2.1	78.9	September 13, 2021
21-03	80.08	> 6.1	< 74.0	September 13, 2021

4.8 Hydraulic Test Results

In-situ hydraulic conductivity testing was carried out on September 13, 2021, in the well screens installed in boreholes 21-01 and 20-02A by members of GEMTEC's hydrogeological team. At these locations the well screens were installed in the silty clay unit.

Falling head testing was completed by inserting a slug with a known displacement (0.45 or 0.60 metre). The water level change was monitored manually using a water level meter and electronically using a VanEssen Diver Datalogger, recording at 0.5 minute intervals. The falling head tests (i.e., inserting a slug) recorded a recovery of about 73 percent at 21-01 and about 7 percent at 21-02A. As these are slow to very slow recovery times levels rising head tests were not performed.

The hydraulic conductivities for the silty clay were calculated from the data obtained in the falling head testing using the Hvorslev solution in an unconfined aquifer. A summary of the recovery measurements made during the hydraulic testing and the estimated value of hydraulic conductivity are provided in Table 4.6. The detailed results of the hydraulic testing are provided in Appendix D.

Based on clay rich soil type at the screened interval at the tested wells 21-01 (silty clay) and 21-02A (silty clay to clayey silt) the recovery response may be considered very slow but reasonable for the encountered soil type. In areas within the site where a saturated granular soil layer is encountered, higher hydraulic conductivity values should be expected.

Table 4.6 – Summary of Falling Head and Rising Head Test Results

Borehole	Geological Material Tested	Static Groundwater Depth (metres)	Falling Head Test	Calculated hydraulic conductivity, *k* (m/s)	General Comments
21-01	Silty clay	3.7	73 percent in 90 minutes	1.0×10^{-7}	Recovery was too slow
21-02A	Silty clay	3.0	7 percent in 60 minutes	3.0×10^{-8}	Recovery was too slow

4.9 Soil Chemistry Relating to Corrosion

The results of chemical testing on one soil sample recovered from borehole 21-02A are provided in Appendix C and are summarized in Table 4.7.

Table 4.7 – Summary of Corrosion Testing

Parameter	Borehole 21-02A Sample No. 3 Depth: 1.5 to 2.1 m
Chloride Content (ug/g)	53
Resistivity (Ohm.m)	47.5
pH	6.71
Sulphate Content (ug/g)	57

5.0 GEOTECHNICAL GUIDELINES

5.1 General

The information in the following sections is provided for the guidance of the design engineers and is intended for the design of this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions. The implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site from materials from offsite sources are outside the terms of reference for this report and have not been addressed.

5.2 Preliminary Site Grade Raise Restrictions

The site is underlain by deposits of sensitive silty clay, which has a limited capacity to support loads imposed by grade raise fill material and foundations for the houses. The placement of fill material on this site must therefore be carefully planned and controlled so that the stress imposed by the fill material does not result in excessive consolidation of the silty clay deposit. Concrete slabs, granular base materials, overall grade raise are considered grade raise filling. Groundwater lowering also results in a stress increase on the underlying sensitive silty clay deposit.

Based on the results of the subsurface investigation, the maximum thickness of any grade raise filling should be limited to about **1.2 metres** above original grade.

The grade raise restriction for the site has been calculated to limit the total settlement of the ground to about 25 millimetres in the long term. For design purposes, we have made the following assumptions:

- The groundwater lowering due to the development at this site will be at most 0.5 metres below the underside of footing elevation;
- The unit weight of the grade raise material used in the vicinity of the structures is not greater than 21.5 kilonewtons per cubic metre; and,
- The grade raise fill material used below the structures, where required, will be composed of compacted granular material having a unit weight of 21.5 kilonewtons per cubic metre.

If heavier grade raise fill material is used, the maximum grade raise will have to be reduced accordingly. Conversely light weight fill materials (e.g. clear stone) could be used to increase the thickness of grade raise fill that could be achieved. The use of expanded polystyrene (EPS) blocks which are specifically manufactured for this purpose could also be used to raise the grade, in combination with native or imported material. The use of light weight fill below the garage and any porches may also be required.

Further guidelines on the use of EPS blocks (if required) could be provided as more details of the proposed buildings (i.e., foundation levels and loading) and site grading are available.

5.3 Proposed Buildings

5.3.1 Excavation

The excavations for the foundations should be taken through topsoil to expose undisturbed native silty clay. The sides of the excavations should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the shallow native overburden deposits can be classified as Type 3 and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical extending upwards from the base of the excavation.

Based on our previous experience, groundwater inflow from the silty clay deposits into the excavations should be relatively small and controlled by pumping from filtered sumps within the excavations. It is not expected that short term pumping during excavation will have any significant effect on nearby structures and services.

5.3.2 Foundation Design

The native silty clay deposits are considered suitable for the support of houses, on the proposed lots, founded on conventional spread footing foundations.

In areas where proposed founding level is above the level of the native soil, or where subexcavation of disturbed material is required below proposed founding level, imported granular material (engineered fill) should be used. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor maximum dry density. In areas where groundwater inflow is encountered, pumping should be carried out from sumps in the excavation during placement of the engineered fill. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.3 metres beyond the footings and then down and out from this point at 1 horizontal to 1 vertical, or flatter. The excavations for the residential dwellings should be sized to accommodate this fill placement. The engineered fill should be placed in accordance with the site grade raise restrictions.

Spread footings founded on or within native undisturbed silty clay deposits, or on a pad of compacted granular material above native, undisturbed soil should be sized using an allowable bearing pressure of 75 kilopascals. Provided that any loose or disturbed soil is removed from the bearing surfaces, and the grade raise restriction provided above are adhered to, the settlement of the footings should be less than 25 millimetres.

5.3.3 Seismic Site Class

Based on the results of the investigation, it is anticipated that the proposed foundations will be supported on a deposit of stiff to very stiff weathered silty clay crust or a pad of engineered fill constructed on the weathered crust.

As per Table 4.1.8.4.A. of the 2012 Ontario Building Code (OBC), Site Class E only applies if the soil has at least 3 metres with all of the following characteristics: plasticity index greater than 20, moisture content greater than 40 percent, and undrained shear strength less than 25 kilopascals.

Since the measured undrained shear strengths at the site are greater than 25 kilopascals, all three conditions, as described above, are not met, and therefore, a seismic Site Class E is not applicable to the site.

Based on the results of the in-situ shear vane testing and the standard penetration testing, the proposed lot severances should be designed for seismic Site Class D.

There is no potential for liquefaction of the overburden deposits at this site.

5.3.4 Frost Protection of Foundations

All exterior footings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated (unheated) footings that are located in areas that are to be cleared of snow should be provided with at least 1.8 metres of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Further details regarding the insulation of foundations could be provided, if necessary.

5.3.5 Backfill and Drainage

5.3.5.1 Basement Foundation Walls

In accordance with the Ontario Building Code, the following alternatives could be considered for drainage of the basement foundation walls:

- Damp proof the exterior of the foundation walls and backfill the walls with free draining, non-frost susceptible sand or sand and gravel such as that meeting OPSS requirements for Granular B Type I or II. OR
- Damp proof the exterior of the foundation walls, install an approved proprietary drainage material on the exterior of the foundation walls and backfill the walls with native material or imported soil.

Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable compaction equipment. Where future landscaped areas will exist next to the proposed structure and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value.

A perforated drain should be installed around the basement area at the level of the bottom of the footings. The drain should outlet by gravity to a storm sewer or to a sump pit from which the water is pumped to a suitable outlet.

5.3.5.2 Garage Foundation Walls and Isolated Piers

To avoid adfreeze and possible jacking (heaving) of the foundation walls, the interior and exterior of the garage foundation walls should be backfilled with free draining, non-frost susceptible sand or sand and gravel such as that meeting OPSS requirements for Granular B Type I or II. The backfill within the garage should be compacted in maximum 300 millimetres thick lifts to at least 95 percent of the standard Proctor dry density value using suitable vibratory compaction equipment.

The backfill against isolated (unheated) walls or piers should consist of free draining, non-frost susceptible material, such as sand or sand and gravel meeting OPSS Granular B Type I or II requirements. Other measures to prevent frost jacking of these foundation elements could be provided, if required.

5.3.6 Lateral Earth Pressures

Foundation walls that are backfilled with granular material such as that meeting OPSS Granular B Type I or II requirements should be designed to resist "at rest" earth pressures calculated using the following formula:

$P_o = 0.5\ K_o\ \gamma\ H^2$

where;

- P_o: Static “At Rest” thrust (kilonewtons per metre);
- γ: Moist material unit weight (kilonewtons per cubic metre);
- K_o: “At Rest” earth pressure coefficient;
- H: Wall height (metre).

Seismic shaking can increase the forces on the foundation walls. The total “At Rest” thrust acting on the walls (P_{oe}) during a seismic event is composed of a static component (P_o) and a dynamic component (P_e), that is:

$P_{oe} = P_o + P_e$

The dynamic at rest thrust component (P_e), which acts only during seismic loading conditions, should be calculated using the following formula:

$P_e = 0.5\ (K_{oe} – K_o)\ \gamma\ H^2$

where;

- P_e: Total “At Rest” thrust (kilonewtons per metre);
- γ: Moist material unit weight (kilonewtons per cubic metre);
- K_o “At Rest” earth pressure coefficient;
- K_{oe}: Dynamic “At Rest” earth pressure coefficient;
- H: Wall height (metre).

The static thrust component (P_o) acts at a point located H/3 above the base of the wall. During seismic shaking, the dynamic at rest thrust component (P_o) acts at a point located about 0.6H above the base of the wall.

For design purposes, the parameters provided in Table 5.1 can be used to calculate the thrust acting on (non-yielding) walls during static and seismic loading conditions.

Heavy construction traffic should not be allowed to operate adjacent to foundation walls for the proposed buildings (within about 2 metres horizontal) during construction, without the approval of the designers.

Table 5.1 – Summary of Design Parameters (Building Foundation Walls)

Parameter	OPSS Granular B Type I	OPSS Granular B Type II
Material Unit Weight, γ (kilonewtons per cubic metre)	22	22
Estimated Friction Angle (degrees)	34	38
"At Rest" Earth Pressure Coefficient, K_o, assuming horizontal backfill behind the structure	0.44	0.38
Dynamic "At Rest" Earth Pressure Coefficient, K_{oe}, assuming horizontal backfill behind the structure	0.51[1]	0.45[1]

Notes:

1) According to the 2015 National Building Code of Canada, the peak ground acceleration (PGA) for this site is 0.32 for Site Class D. The dynamic at rest earth pressure coefficient was calculated using the method suggested by Mononobe and Okabe, assuming a horizontal seismic coefficient, kh, of 0.32 (taken as the corrected PGA) and assuming that the vertical seismic coefficient, kv, is zero.

5.3.7 Basement Floor Slabs

To provide predictable settlement performance of basement slabs, all topsoil, loose soil, or debris should be removed from the slab area. The base of the floor slab should consist of at least 200 millimetres of 19 millimetre clear crushed stone. Any necessary grade raise fill should consist of either 19 millimetre clear crushed stone or OPSS Granular B Type II. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular B Type II material. Since the source of recycled material cannot be determined or controlled, it is suggested that any imported Granular B Type II materials be composed of 100 percent crushed rock only.

The clear crushed stone should be nominally compacted in maximum 300 millimetre thick lifts with at least 2 passes of a diesel plate compactor. The Granular B Type II should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment.

In areas where the subgrade consists of silty sand, a suitable nonwoven geotextile should be placed over the subgrade prior to the placement of clear stone to prevent ingress of fines into voids in the clear stone and possible settlement/cracking of the slab. If clear crushed stone is used below the floor slab, underfloor drains are not considered essential provided that drains are installed to link any hydraulically isolated areas in the basement. Where drains are installed the drains should outlet by gravity to a sump from which the water is pumped or drained by gravity to a sewer.

The ACI 302.1R-04 “Guide for Concrete Floor and Slab Construction” should be referenced for design purposes.

A polyethylene vapour retarder is recommended below the floor slabs.

5.3.8 Effects of Agricultural Tile Drains

It is likely that some of the agricultural fields within the subject site are tile drained. Any agricultural tile drains encountered within the house excavations could be a source of significant volumes of water, which could impact on the basements of the houses. It is suggested that any drainage tiles that are within about 2 metres horizontal distance to the dwellings be removed and the excavation for the tiles backfilled with compacted silty clay to prevent any water flow through the tiles or trench. The silty clay could be compacted with the bucket of the excavator. Any drainage tiles that are below proposed footings should be removed. The ends of the drains should be severed at least 2 metres outside of the proposed basement foundations to reduce the potential for post construction groundwater inflow into the basements. The excavations for the removal of tiles should be backfilled with compacted silty clay as described above.

5.3.9 Corrosion of Buried Concrete and Steel

According to Canadian Standards Association (CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate in the soil sample recovered from borehole 21-02A can be classified as low. For low exposure conditions, any concrete that will be in contact with the native soil or groundwater could be batched with General Use (GU) type cement. The effects of freeze thaw in the presence of de-icing chemical (sodium chloride) near the houses should be considered in selecting the air entrainment and the concrete mix proportions for any exposed concrete.

Based on the resistivity and pH of the soil samples tested the soil can be generally classified as nonaggressive toward unprotected steel. It is noted that the corrosivity of the soil could vary throughout the year due to the application sodium chloride for de-icing.

5.4 Sensitive Marine Clay – Effects of Trees

The site is underlain by silty clay, a material which is known to be susceptible to shrinkage with a change/reduction in moisture content. Research by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada has shown that trees can cause a reduction of moisture content in the silty clays in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations, or hard surfaced areas. Therefore, deciduous tree planting should be carried in accordance with the guidelines identified in the City of Ottawa document titled: “Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines”.

The City of Ottawa Tree Planting Guidelines indicates that sensitive marine clay soils with a modified plasticity index of less than 40 percent are considered to have a low/medium potential for soil volume change. Clay soils with a modified plasticity index that exceeds 40 percent are considered to have a high potential for soil volume change.

As part of the geotechnical investigation, soil samples at about 150 metre spacing were tested in our laboratory to determine the Atterberg limits for the sensitive marine clay. A summary of the test results is provided in Table 5.2.

Table 5.2 – Summary of Modified Plasticity Index

Borehole ID / Sample No.	Shrinkage Limit[3] (%)	Plastic Limit[1] (%)	Liquid Limit[1] (%)	Plasticity Index[1] (%)	Modified Plasticity Index[2] (%)
21-01 / 2	-	55	29	26	26
21-02A / 2	16.7	53	24	29	29
21-03 / 3	-	56	23	33	33

Notes:

1) Calculated in accordance with ASTM D4318.
2) The modified plasticity index (PI_m) was calculated using the following formula, where PI is the plasticity index determined in accordance with ASTM D4318: $PI_m = PI \times$ (% passing the 425 micrometre sieve / 100).
3) Calculated in accordance with ASTM D4943, which was discontinued in 2017 by the ASTM Sponsoring Committee responsible for the standard.

The modified plasticity index of the samples tested ranges from about 26 to 33 percent. As such, the potential for soil volume change, as defined by the City of Ottawa, is low/medium. For this site, the low/medium potential clay soils encompass the entire site.

In accordance with the City of Ottawa Tree Planting Guidelines, tree planting restrictions apply where clay soils with low/medium potential for volume change are present between the underside of footing and a depth of 3.5 metres below finished grade (refer to the City of Ottawa document titled: “Tree Planting in Sensitive Marine Soils - 2017 Guidelines”).

According to the City of Ottawa 2017 Tree Planting Guidelines, the tree to foundation setbacks within the lots can be reduced to 4.5 metres for small to medium sized trees (i.e., trees with a mature height of less than 14 metres) with further information and recommendations on planting trees near foundations provided in the City of Ottawa Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines.

6.0 SLOPE STABILITY ANALYSIS

6.1 General

The purpose of this stability assessment is to establish the 'Erosion Hazard Limit' for the site, in relation to the slope on the southwest corner of the site, and along McKinnon's Creek to the east of the site, as described by the Ministry of Natural Resources. This limit constitutes a safe setback for any proposed development at the site with respect to slope stability. The Erosion Hazard Limit was determined based on the Natural Hazard Policies set forth in Section 3.1 of the Provincial Policy Statements of the Planning Act of Ontario. Current regulations restrict development within the Erosion Hazard Limit.

The slope stability analyses were carried out at Section 'C-C', 'D-D', and 'G-G' using Slope/W, a two-dimensional limit equilibrium slope stability program. The slope stability assessment was carried out on a limited number of slope cross sections due to the similarity of the cross sections (i.e., there were three distinct cross sections to assess).

The results of the slope stability analyses are provided in Appendix E and F.

6.2 Soil Strength Parameters

The soil conditions used in the stability analyses were based, in part, on the results of the boreholes advanced across the site. The slope stability analyses were carried out using silty clay strength parameters based on site specific studies in the Ottawa area. To determine the existing factor of safety against overall rotational failure, the slope stability analyses were carried out using drained soil parameters, which reflect long term conditions. Undrained shear strength parameters were used for seismic loading conditions. Table 6.1 summarizes the soil parameters used in the analyses.

Table 6.1 – Slope Stability Soil Strength Parameters

Soil Type	Effective Angle of Internal Friction, φ (degrees)	Effective Cohesion, c' (kilopascals)	Undrained Shear Strength, S_u (kilopascals)	Unit Weight, γ (kN/m^3)
Grade Raise Fill	34	0	0	21.5
Weathered Silty Clay Crust	35	5	75	18.0
Grey Silty Clay	35	5	50	16.5

The results of a stability analysis are highly dependent on the assumed groundwater conditions. No information is available on the long-term groundwater levels throughout the year; however, as a conservative approach for the static model, we have assumed full hydrostatic saturation with the groundwater level at ground surface and groundwater flow horizontally towards the slope. For the seismic model, we have assumed a groundwater elevation of about 77.9 metres, corresponding to the underside of the weathered crust.

The slope stability analyses were carried out using soil parameters, groundwater conditions and a slope profile that attempt to model the slopes in question but do not exactly represent the actual conditions. For the purposes of this study, a computed factor of safety of less than 1.0 to 1.3 is considered to represent a slope bordering on failure to marginally stable, respectively; a factor of safety of 1.3 to 1.5 is considered to indicate a slope that is less likely to fail in the long term and provides a degree of confidence against failure ranging from marginal (1.3) to adequate (1.4 and greater) should conditions vary from the assumed conditions. A factor of safety of 1.5, or greater, is considered to indicate adequate long-term stability for static conditions.

For the seismic analysis, a computed factor of safety of less than 1.0 is considered to represent a slope bordering on failure; a factor of safety of 1.0 to 1.1 is considered to represent a marginally stable slope. For seismic conditions a factor of safety of 1.1, or greater, is considered to indicate adequate stability subject to the design earthquake event.

6.3 Slope on Southwest Corner of Site

6.3.1 Existing Conditions

The slope stability analyses indicates that the existing slopes, in their current configurations, have a factor of safety against overall rotational failure of about 2.2 under static conditions.

Based on the results of the analyses, the slopes along the south side of the site are considered stable under "worst case" conditions.

6.3.2 Setback Requirements

For unstable slopes, the distance from the unstable slope to the safe setback line is called 'Erosion Hazard Limit'. In accordance with the Ministry of Natural Resources (MNR) Technical Guide "Understanding Natural Hazards" dated 2001, the Erosion Hazard Limit consists of three components, those being: (1) Stable Slope Allowance, (2) Toe Erosion Allowance, and (3) Erosion Access Allowance.

The Stable Slope Allowance, as described in the MNR procedures, encompasses the area where a factor of safety of less than 1.5 against overall rotational failure is calculated. The slope stability analyses indicate that the existing slopes, in their current configurations, have a factor of safety greater than 1.5. As such, the Stable Slope Allowance described in the MNR procedures is not required.

In accordance with the MNR documents, a minimum Toe Erosion Allowance is not required, as there was no waterbody at the toe of the slope, and, therefore, no evidence of erosion at the toe of the slope.

The MNR procedures also include the application of a 6 metre wide Erosion Access Allowance to allow for access by equipment to repair a possible failed slope. The construction of driveways within this 6 metre wide zone is, in our opinion, permitted as driveways would not prevent repair/stabilization work to be carried out, should it be required. However, structures should not be constructed within this zone which could impeded construction access.

Based on the above information, the Erosion Hazard Limit for the slopes along the south and west side of the site will be 6 metres, as measured from the crest of the slope. From the survey of the slope cross sections, and the topographic mapping at the site, it is assumed that the top of the slope is at about elevation 78.0 metres.

6.3.3 Future Conditions

As the lot is severed and developed placement of fill material on the upper portion of the slopes may be carried out. The addition of grade raise fill at the crest of the slope will impact the factor of safety against global instability, to some extent. A slope stability model was completed with the addition of grade raise fill and house loading with the following assumptions:

- Grade raise fill of 1.2 metres was added to the crest of the slope model. The grade raise fill material was assumed to be compacted engineered fill, sloped at about 3 horizontal to 1 vertical from the crest of the existing slope; and,
- A surcharge load of 75 kilopascals was added to the ground surface at the grade raise fill to act as the loading from the proposed house. The surcharge load was applied up to the edge of the grade raise fill.

Based on the results of the analysis the slopes, with the addition of grade raise fill and house foundations located up to about 3.6 metres from the crest of the slope, will have a factor of safety against global stability of 1.6. As such, the slope is considered stable from a geotechnical point of view with the addition of grade raise fill and foundation loading up to the edge of the grade raise fill. However, in Section 6.3.2, above, a minimum set back of 6 metres is required for the Erosion Access Allowance, and therefore, the minimum setback for construction of proposed houses should be 6 metres from the crest of the existing slope.

6.4 McKinnon's Creek

6.4.1 Existing Conditions

The slope stability analyses indicates that the existing slopes, in their current configurations, have a factor of safety against overall rotational failure of less than 1.0 under static conditions on Cross Sections D-D and G-G.

Under seismic conditions the slopes have a factor of safety against overall rotational failure of greater than 1.1.

Based on the results of the analyses, the slopes along the McKinnon's Creek are considered to be potentially unstable under "worst case" conditions.

6.4.2 Geotechnical Hazard Limit

The Stable Slope Allowance, as described in the MNR procedures, encompasses the area where a factor of safety of less than 1.5 against overall rotational failure is calculated. The Stable Slope Allowance described in the MNR procedures extends about 15 metres horizontally from the crest of the slope (as shown by the red zone in Figures F7 and F10, in Appendix F).

In accordance with the MNR documents, a minimum Toe Erosion Allowance of between 8 to 15 metres is required for soft/firm cohesive soils (i.e., silty clay). Given that erosion along the toe of the slope, and previous slope failures were observed along McKinnon's Creek, a Toe Erosion Allowance of 15 metres should be applied. The Toe Erosion Allowance is applied only where the watercourse is located within the 15 metres from the toe of the slope and can be reduced an equivalent distance of the toe of slope to the watercourse, with the balance to be applied in the determination of the Erosion Hazard Limit.

A summary of the distance between the toe of the slope and the watercourse, and the resulting required Toe Erosion Allowance is provided in Table 6.2, below.

Table 6.2 Summary of Toe Erosion Allowance

Slope Cross Section	Required Toe Erosion Allowance (metres)	Approximate Distance Between Toe of Slope and Watercourse (metres)	Balance of Erosion Allowance Applied from Crest (metres)
D-D	15	0	15
E-E	15	29	0
F-F	15	12	3
G-G	15	0	15
H-H	15	12	3

The MNR procedures also include the application of a 6 metre wide Erosion Access Allowance to allow for access by equipment to repair a possible failed slope.

Based on the above information, the Erosion Hazard Limit for the slope along McKinnon's Creek, as measured from the toe of the existing slope, is summarized in Table 6.3. The construction of driveways within this Erosion Hazard Limit is, in our opinion, permitted as driveways would not prevent repair/stabilization work to be carried out, should it be required, or any risk to global instability of the overall slopes. However, grade raise filling and structures should not be constructed within the Erosion Hazard Limit which could impeded construction access and negatively impact the stability of the slopes.

From the survey of the slope cross sections, the top of the slope is at about the east edge of the existing Smith Road. A summary of the Erosion Hazard Limit for each cross section is provided in Table 6.3, below, and are shown on the cross sections provided in Figures F1 to F5.

Table 6.3 Summary of Erosion Hazard Limit

Slope Cross Section	Stable Slope Allowance (metres)	Toe Erosion Allowance (metres)	Erosion Access Allowance (metres)	Erosion Hazard Limit (metres)
D-D	15	15	6	36
E-E	15	0	6	21
F-F	15	3	6	24
G-G	15	15	6	36
H-H	15	3	6	24

Based on the results of the slope stability assessment carried out for McKinnon's Creek, it is understood that the proposed houses and any grade raise filling will be located outside of the limit of hazard lands and therefore the additional loading from the houses and grade raise fill will not have a negative impact on the stability of McKinnon's Creek.

6.4.3 Setback Requirements from McKinnon's Creek

It is understood that, based on Section 4.9.3, policy 2 of the City of Ottawa Official Plan, the minimum setback from a surface water feature will be the greater of the following setbacks:

- The conservation authority's hazard limit (including the geotechnical hazard limit);

- The geotechnical hazard limit based on the City of Ottawa's Slope Stability Guidelines for Development Applications. It is assumed that this geotechnical hazard limit, as described in Section 6.4.2, above, is the same as the conservation authority's geotechnical hazard limit;
- 30 metres from the top of bank. Since the top of bank was not measured, it was conservatively taken as approximately elevation 76 metres, which is located about 2 metres above the approximate location of the creek; and,
- 15 metres from the stable top of slope. It is assumed that the stable top of slope is defined as the "stable slope allowance" as described in Section 6.4.2, above.

It is also understood that a 27 metre setback, applied from the property limit, for development will be provided as a condition of development by Hierarchy Development and Design Inc.

The above setbacks are provided on Figure 2 and, as provided by the City of Ottawa Official Plan, the minimum setback from McKinnon's Creek should be taken as the greater of the setbacks.

6.4.4 Potential for Cyclic Softening

An assessment of the potential for cyclic softening (i.e., liquefaction like behaviour) in the silty clay soils was carried out, using the method developed by Idriss and Boulanger (2007). The method developed by Idriss and Boulanger (2007) includes an assessment of the cyclic stress ratio (CSR), which is cyclic shear stresses resisting cyclic softening, and the cyclic resistance ratio (CRR), which is the CSR that is required to trigger a cyclic softening event in the silty clay.

The factor of safety against cyclic softening during an earthquake is the ratio of the CRR to the CSR. Where the factor of safety is greater than 1.0, the silty clay deposits are not considered to be susceptible to cyclic softening.

Based on the results of the assessment, the silty clay soils at this site have a factor of safety greater than 1.5, and therefore, are not considered to undergo cyclic softening during the design earthquake event.

6.5 Potential for Retrogressive Earth Flow Sliding

The City of Ottawa has provided high level screening criteria to assess the potential where retrogressive earth flow slide failure may occur along the slopes. The following are the criteria to assess the potential for retrogressive earth flow slide failures:

i. The height of the slope must be greater than 8 metres;
ii. The top and bottom of the slope are to be determined where the slope has a gradient of less than 14 percent over a distance of greater than 15 metres; and,
iii. At least 35 to 40 percent of the slope height above the critical failure surface must consist of sensitive marine clay.

Based on the comments provided by the City of Ottawa, if one of the above criteria is not met, the slope is not considered to be at risk of retrogressive earth flow slide.

6.5.1 Thickness of Clay along McKinnon's Creek

A site reconnaissance was carried out on April 26 and 29, 2024, by a member of GEMTEC's engineering staff. On those days, a series of shallow hand excavated test pits were advanced at each of the previously measured slope cross sections (cross sections D-D to H-H). The test pits were advanced to assess the subsurface conditions along the slope to assess the elevation of the underside of the silty clay layer.

The test pits were advanced, starting just below the crest of the slope to the toe of the slope, and advanced to depths of up to about 0.6 metres below the existing ground surface. The glacial till deposit was encountered within the test pits at elevations ranging from about 76.9 to 77.5 metres at slope cross sections D-D to G-G. At slope cross section H-H, the glacial till was not directly encountered in the test pits but was exposed on the banks of the creek at an elevation of about 73.2 metres.

The test pit locations, where the underside of the silty clay/top of glacial till layer was encountered, are provided on the slope cross sections in Appendix F. As discussed above, the test pits were generally advanced along the previously measured cross sections, and are not shown on the Site Plan for clarity.

Table 6.4, below, summarizes the elevation of the crest of the slope, the underside of the silty clay layer, and the overall thickness of the silty clay deposit along the slope of McKinnon's Creek.

Table 6.4 – Summary of Silty Clay Thickness Along McKinnon's Creek

Slope Cross Section	Approximate Elevation of Slope Crest (metres)	Approximate Elevation of Underside of Silty Clay Deposit (metres)	Approximate Thickness of Silty Clay Deposit
D-D	81.2	77.5	3.7
E-E	81.5	76.8	4.7
F-F	81.4	76.9	4.5
G-G	80.8	77.3	3.5
H-H	80.5	73.2	7.3

The underside of the silty clay layer was also encountered in borehole 21-03 (south of cross-section H-H) at an elevation of 74.8 metres, or about 5.3 metres below ground surface.

6.5.2 McKinnon Creek Slope Height

The heights of the slope were assessed at 16 locations along McKinnon's Creek, based on the 2019 LiDAR imaging provided by the Geospatial Analytics, Technology and Solutions department of the City of Ottawa and criteria ii) provided above (i.e., where the slope has a gradient of less than 14 percent), between the north and south property limits of 930 Smith Road. The locations of the cross sections as well as contour lines, are provided on Figure 2.

The slope profiles at each cross-section location are provided on Figures 3, 4, and 5. As described above, per criteria ii), the elevation of the top and bottom of the slope is provided for each slope profile as the point where the slope has a gradient of less than 14 percent. Table 6.5, below, summarizes the elevation of the top and bottom of the slope and the slope height at each cross-section location.

Table 6.5 Summary of Slope Heights

Slope Cross Section	Top of Slope Elevation (metres)	Bottom of Slope Elevation (metres)	Slope Height (metres)
I-I	80.9	73.7	7.2
J-J	80.9	73.7	7.2
K-K	81.0	76.0	5.0
L-L	81.1	76.0	5.1
M-M	81.0	75.7	5.3
N-N	81.0	75.3	5.7
O-O	80.9	74.0	6.9
P-P	81.0	73.2	7.8
Q-Q	80.8	72.9	7.9
R-R	80.4	72.8	7.6
S-S	80.2	74.1	6.1
T-T	80.3	73.8	6.5
U-U	79.8	73.9	5.9

Slope Cross Section	Top of Slope Elevation (metres)	Bottom of Slope Elevation (metres)	Slope Height (metres)
V-V	79.3	77.1	2.2
W-W	79.2	76.5	2.7
X-X	79.0	75.9	3.1

Based on the table above, the slope has a height ranging from about 2.2 to 7.9 metres.

6.5.3 Results of Retrogressive Earth Flow Sliding Potential

Since the slope does not meet criteria i and ii, as described above (i.e., slope height, as defined by the top and bottom of slope at the elevation where the gradient is less than 14 percent, is less than 8 metres), the slope along the west side of McKinnon's Creek would be considered to have a low risk of retrogressive landslide failure, as per the high-level screening criteria from the City of Ottawa.

7.0 ADDITIONAL CONSIDERATIONS

7.1 Maintenance of Existing Vegetation

The existing vegetation and trees along the slopes should be maintained, to ensure the stability of the slope is not affected. As part of the overall site grading for any future development, no additional surface water should be directed towards the slope. This could cause erosion of the slope and could also negatively affect the stability of the slope. Final plans and finished grades for any proposed development adjacent to the slope should be reviewed by a geotechnical engineer to ensure that the guidelines provided on this report have been interpreted as intended.

7.2 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, etc.) will cause ground vibration on and off the site. The vibrations will attenuate with distance from the source but may be felt at nearby structures. The magnitude of the vibrations will be much less than that required to cause damage to the nearby structures or services in good condition.

7.3 Monitoring Well Abandonment

All monitoring wells installed as part of this investigation should be decommissioned by a licensed well technician. The well abandonment could be carried out in advance of or during construction.

7.4 Disposal of Excess Soil

It is noted that the professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination, including naturally occurring source of contamination, are outside the terms of reference for this report. This report does not constitute a Phase II Environmental Site Assessment (ESA) nor does it constitute a contaminated material management plan.

7.5 Design Review and Construction Observation

The engagement of the services of the geotechnical consultant as the plans for the proposed severances and structures is recommended to confirm the geotechnical guidelines and recommendations provided in this report remain applicable and are interpreted as intended.

The geotechnical consultant should be engaged during construction to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. The subgrade surfaces for the houses, services, and roadways should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

8.0 CLOSURE

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

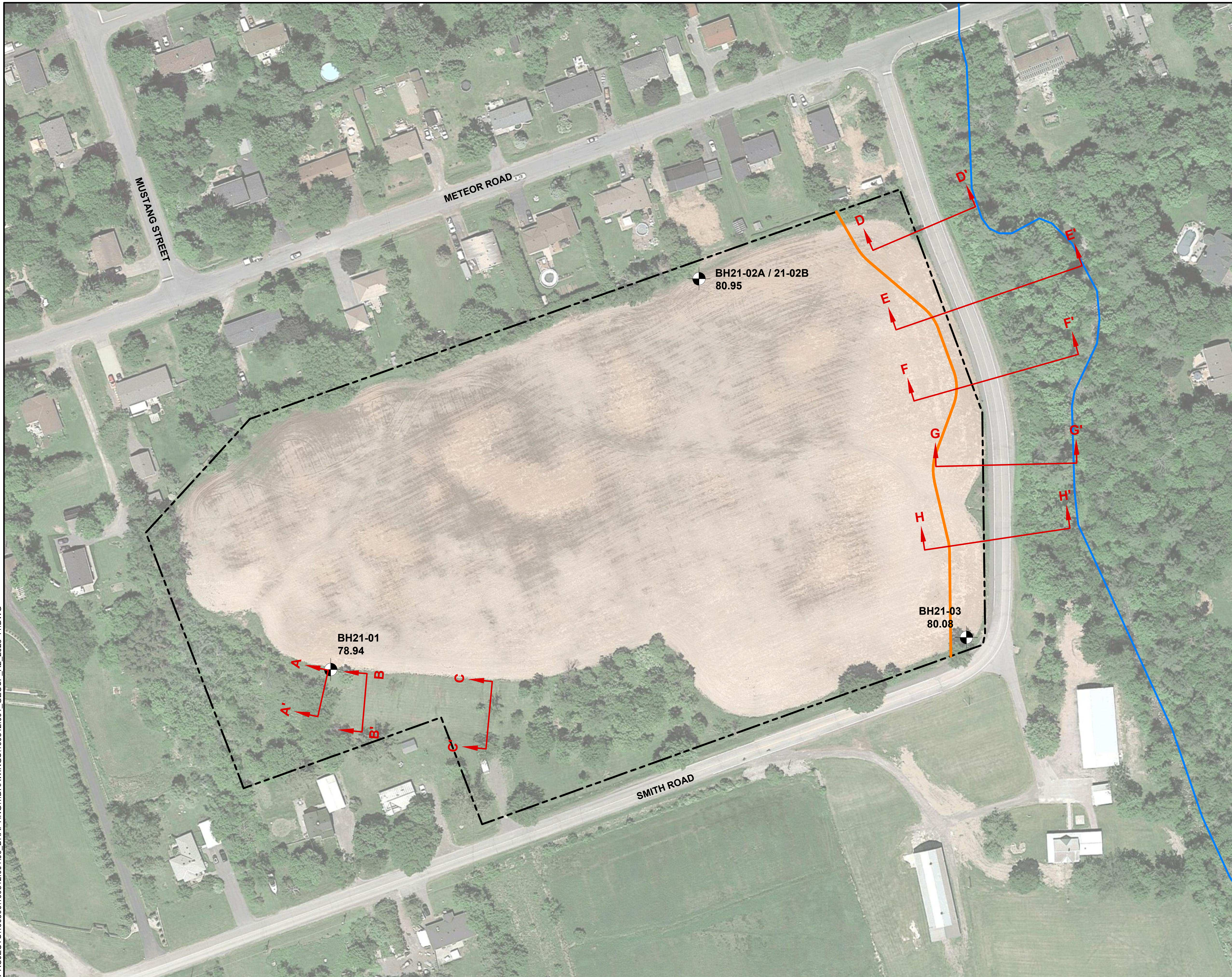
Alex Meacoe, P.Eng.
Senior Geotechnical Engineer

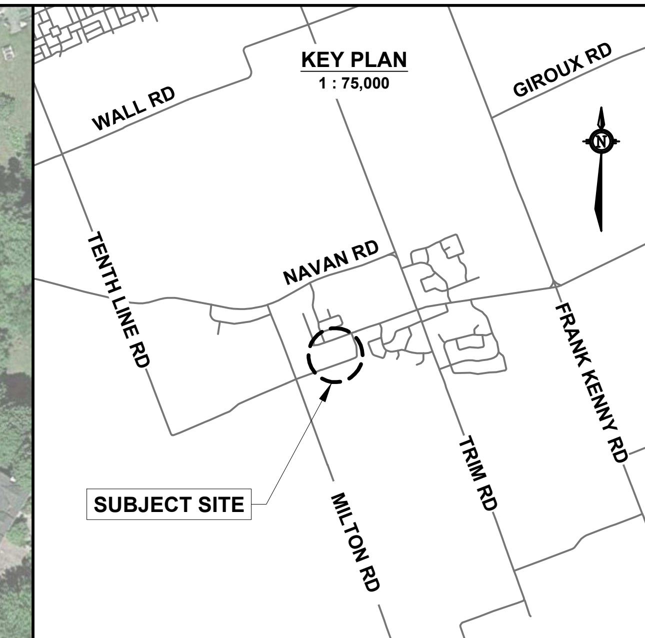
LICENSED PROFESSIONAL ENGINEER
W. A. MEACOE
100162115
Jun 7, 2024
PROVINCE OF ONTARIO

Bill Cavers, P.Eng.
Principal Geotechncial Engineer

REFERENCES

- Ross W. Boulanger, I. M. Idriss (2007). Evaluation of Cyclic Softening in Silts and Clays. Journal of Geotechnical and Geoenvironmental Engineering.





LEGEND

BH # — BOREHOLE ID

XX.XX — GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

BOREHOLE LOCATION (current investigation by GEMTEC)

A A' — SLOPE CROSS SECTION LOCATION

APPROXIMATE SITE BOUNDARY

APPROXIMATE LOCATION OF WATERCOURSE

APPROXIMATE EROSION HAZARD LIMIT

Scale

1:1500

0 30 60 90m

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Drawing	SITE PLAN
Client	HIERARCHY DEVELOPMENT & DESIGN INC.
Project	100812.001
Drwn by	S.L.
Chkd by	W.A.M.
	GEOTECHNICAL INVESTIGATION PROPOSED LOT SEVERANCES 930 SMITH ROAD OTTAWA, ONTARIO
Date	NOVEMBER 2023
Rev.	2
	FIGURE 1

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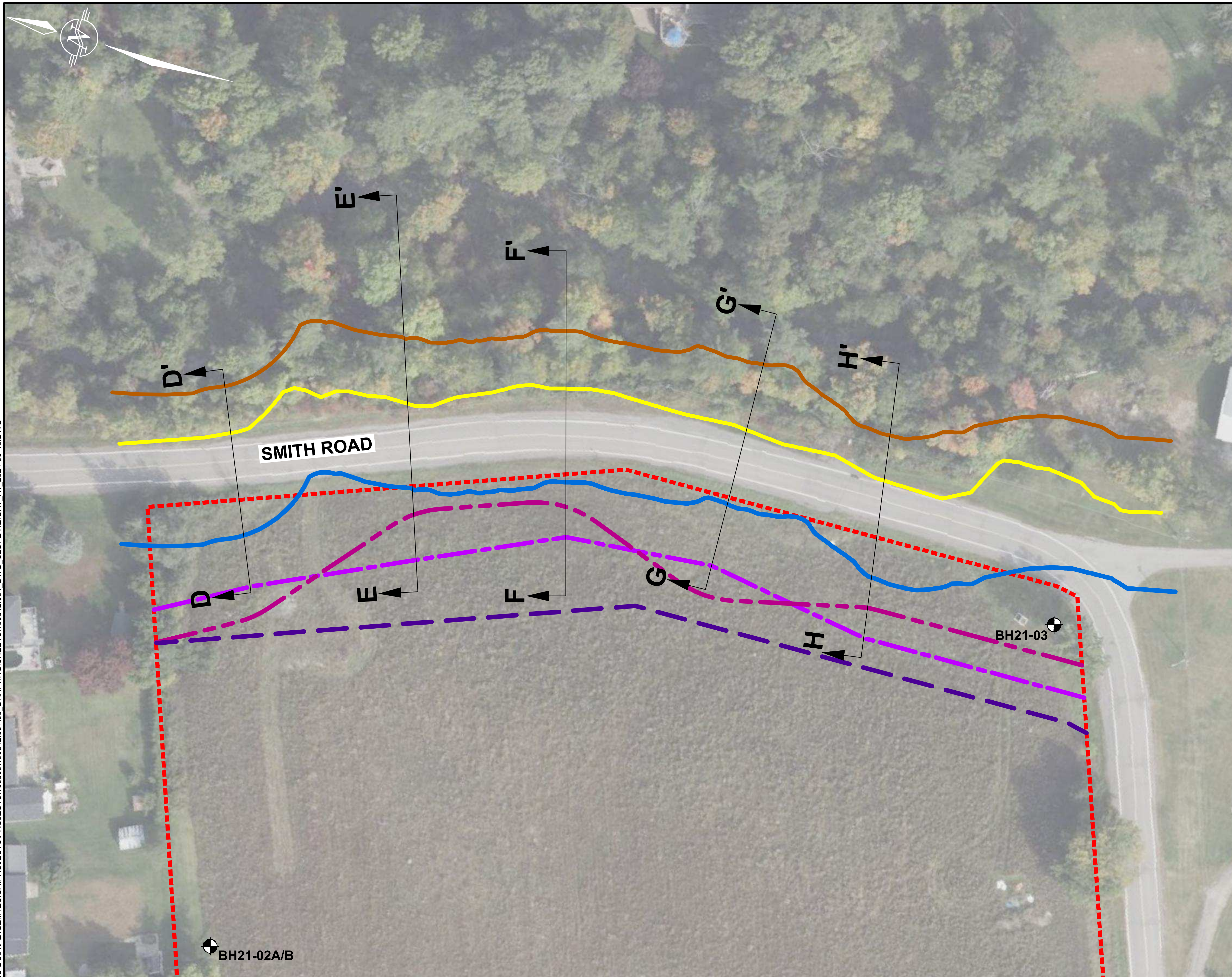

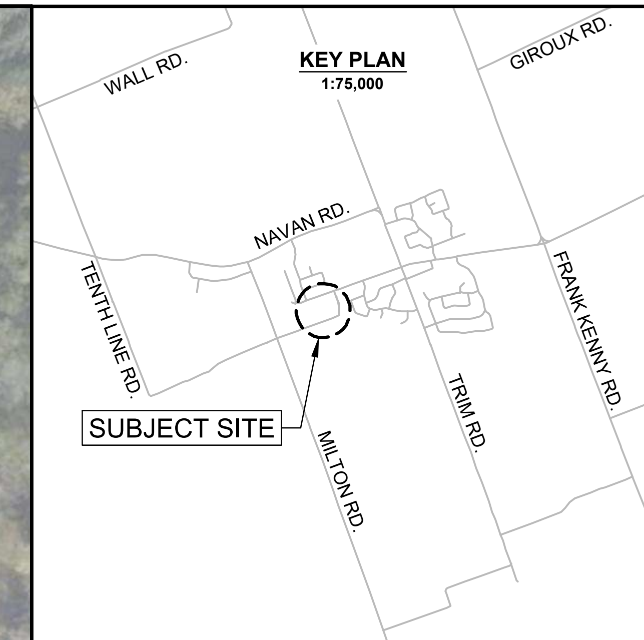

LEGEND

EXISTING BOREHOLE LOCATION
APPROXIMATE SITE BOUNDARY
TOP OF SLOPE
TOP OF BANK
30m OFFSET FROM TOP OF BANK
GEOTECHNICAL HAZARD LIMIT
15m OFFSET FROM STABLE TOP OF SLOPE
27m OFFSET FROM PROPERTY LINE
A A' CROSS SECTION LOCATION

NOTE: Contours derived from 2020 LiDAR DEM provided by the City of Ottawa and are shown in metres.

Scale
1:750
0 15 30 45m

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Drawing
SETBACK LIMITS FROM WATERCOURSE

Client
HIERARCHY DEVELOPMENT & DESIGN INC.

Project	GEOTECHNICAL INVESTIGATION PROPOSED LOT SEVERANCES 930 SMITH ROAD OTTAWA, ONTARIO
100812.001	
Drwn by: AGSD	Chkd by: WAM
Date: JUNE, 2024	Rev. 1
FIGURE 2	

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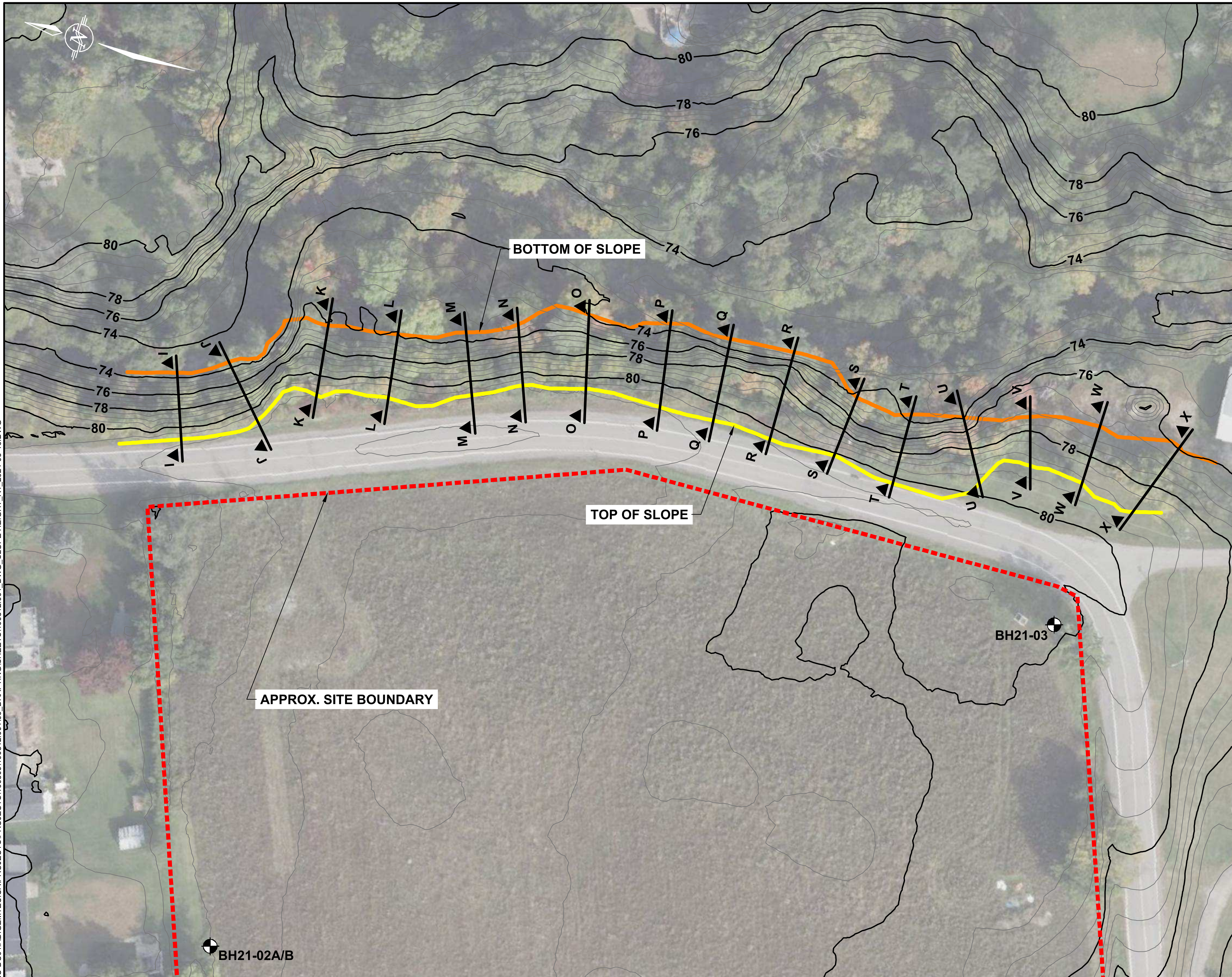

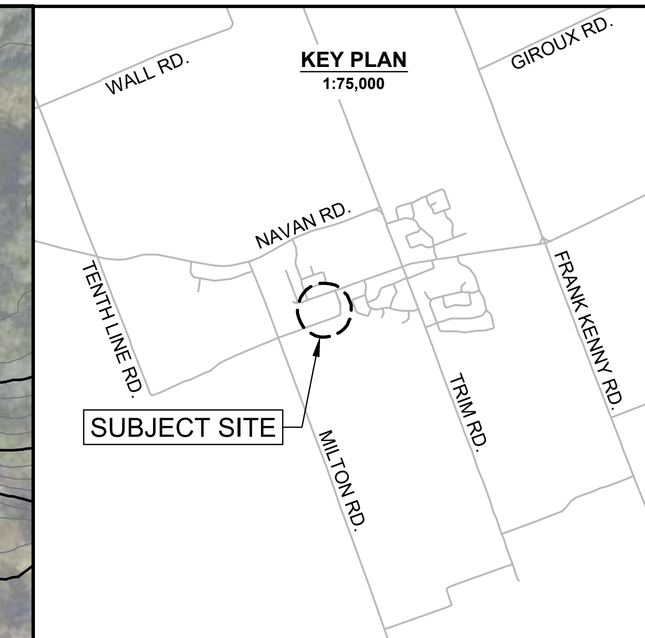

LEGEND

- EXISTING BOREHOLE LOCATION
- TOP OF SLOPE
- BOTTOM OF SLOPE
- APPROXIMATE SITE BOUNDARY
- 80 EXISTING GRADE CONTOUR
- A A CROSS SECTION LOCATION

NOTE: Contours derived from 2020 LiDAR DEM provided by the City of Ottawa and are shown in metres.

Scale 1:750

0 15 30 45m

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Drawing: MCKINNON'S CREEK CROSS SECTION LOCATION PLAN

Client: HIERARCHY DEVELOPMENT & DESIGN INC.

Project		GEOTECHNICAL INVESTIGATION PROPOSED LOT SEVERANCES 930 SMITH ROAD OTTAWA, ONTARIO
100812.001		
Drwn by: AGSD	Chkd by: WAM	
Date: JUNE, 2024	Rev.: 1	FIGURE 3

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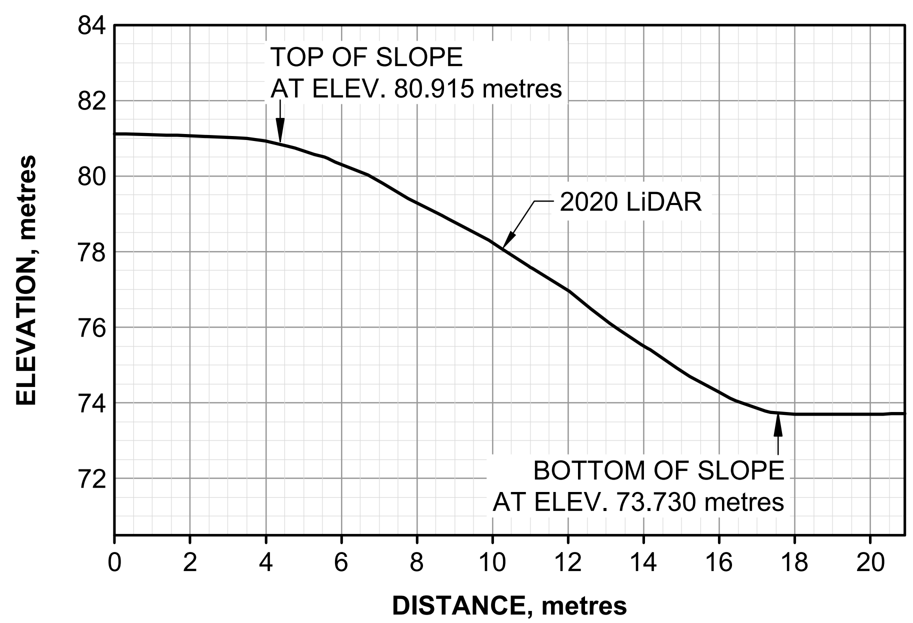
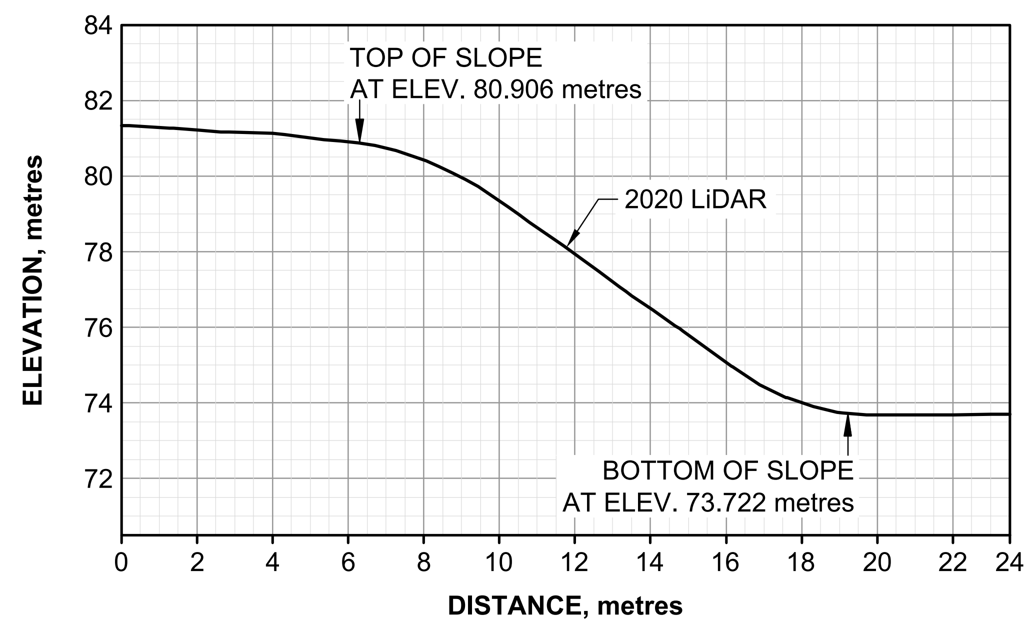
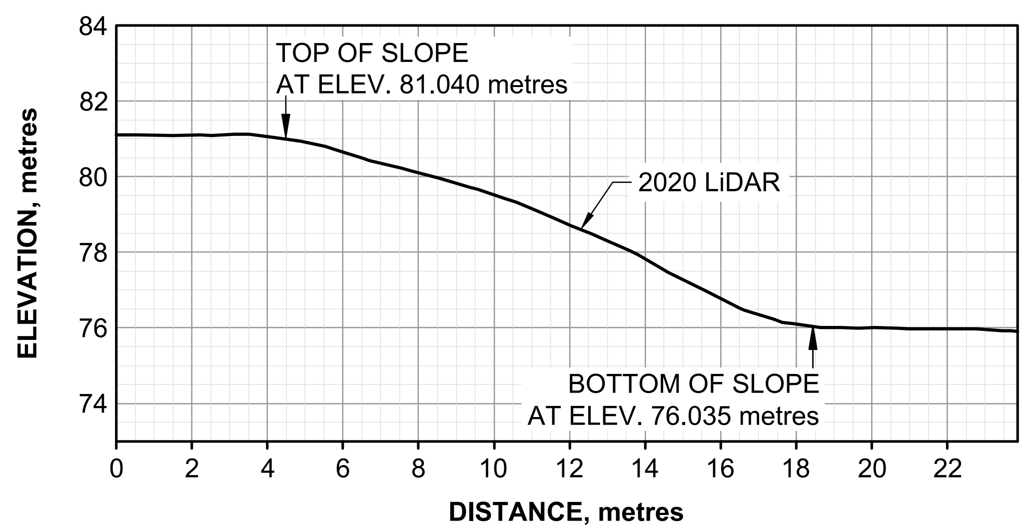
SECTION I-I



SECTION J-J



SECTION K-K



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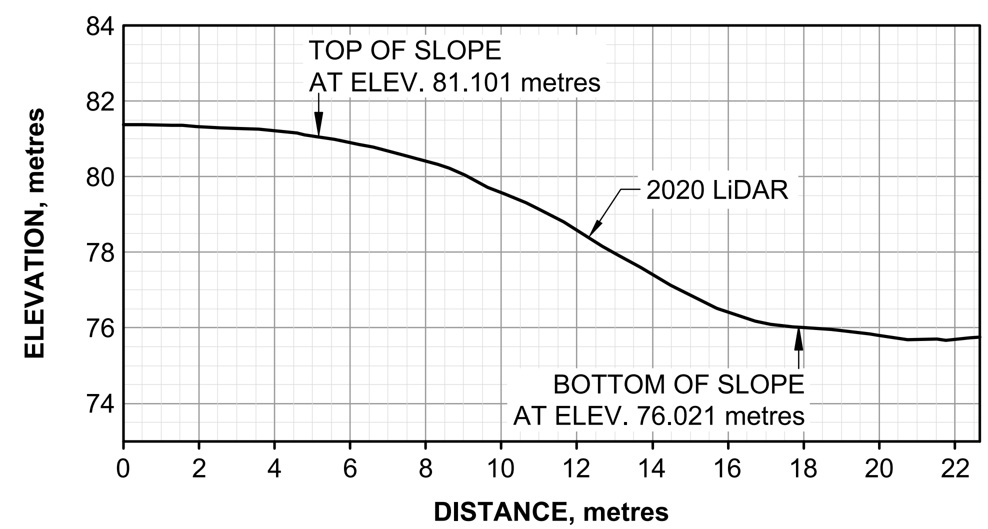
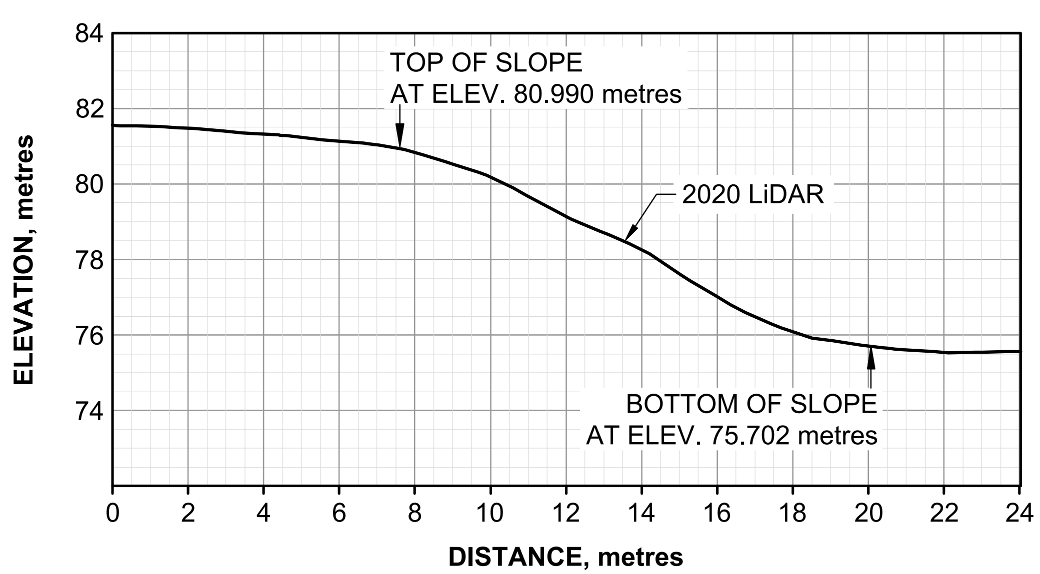
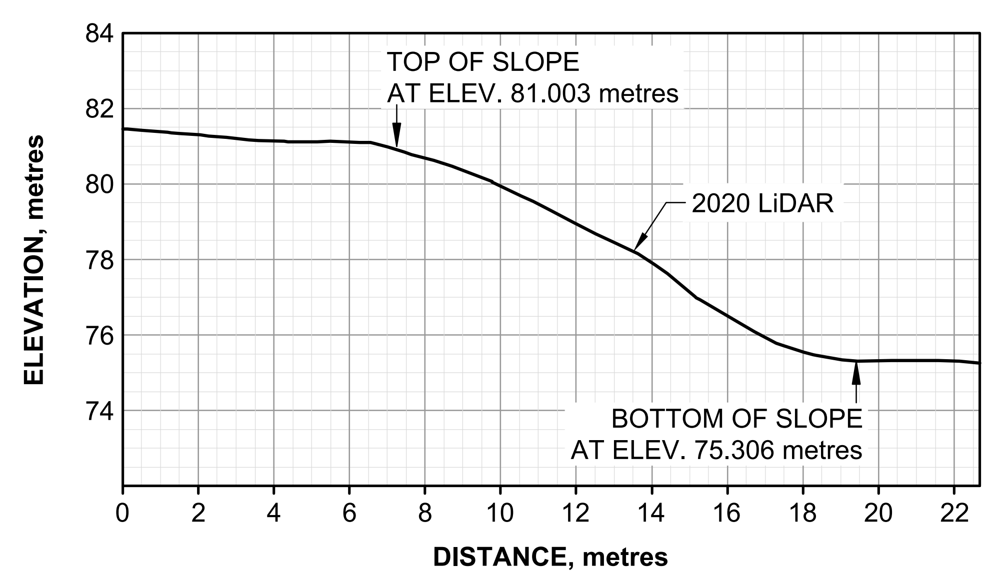


SECTION M-M



SECTION N-N



KEY PLAN
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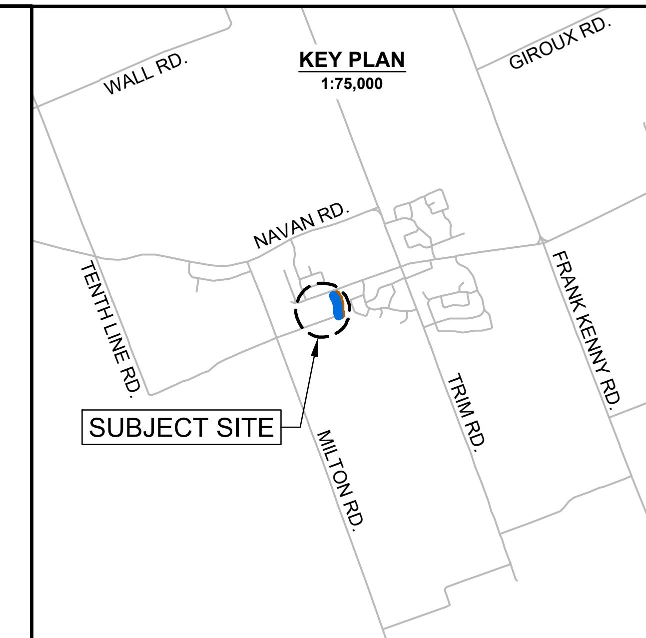

NOTE: Contours derived from 2020 LiDAR DEM provided by the City of Ottawa and are shown in metres.

Scale
1:200
0 4 8 12m

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Ottawa, ON K2K 2A9
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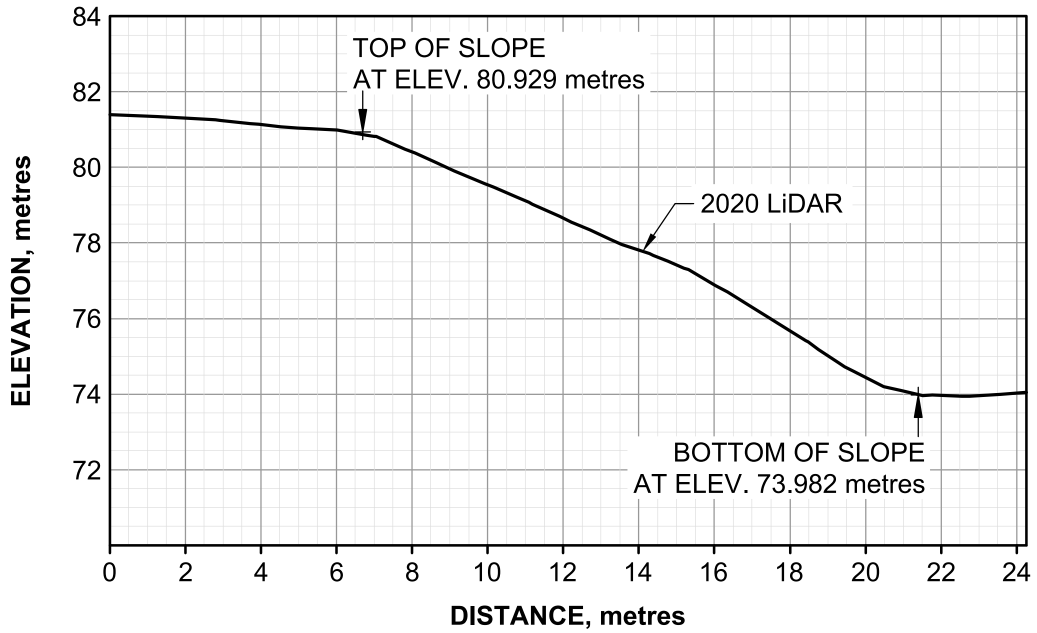
www.gemtec.ca
ottawa@gemtec.ca

Drawing	SECTIONS (1 of 3)
Client	HIERARCHY DEVELOPMENT & DESIGN INC.
Project 100812.001	GEOTECHNICAL INVESTIGATION PROPOSED LOT SEVERANCES 930 SMITH ROAD OTTAWA, ONTARIO
Drwn by AGSD / Chkd by WAM	
Date JUNE, 2024	Rev. 1 — FIGURE 4

SECTION O-O



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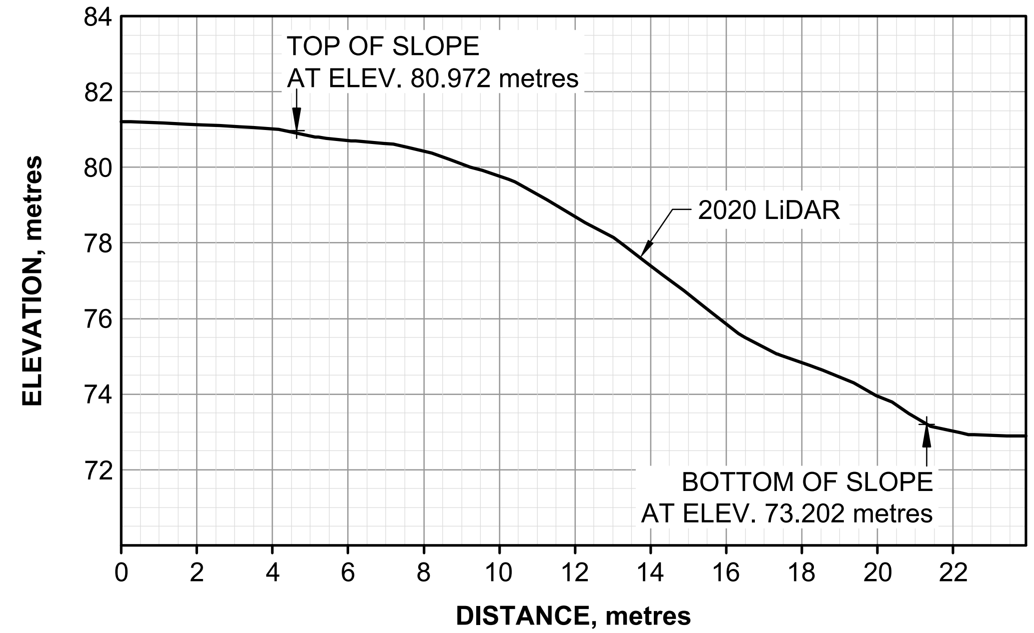

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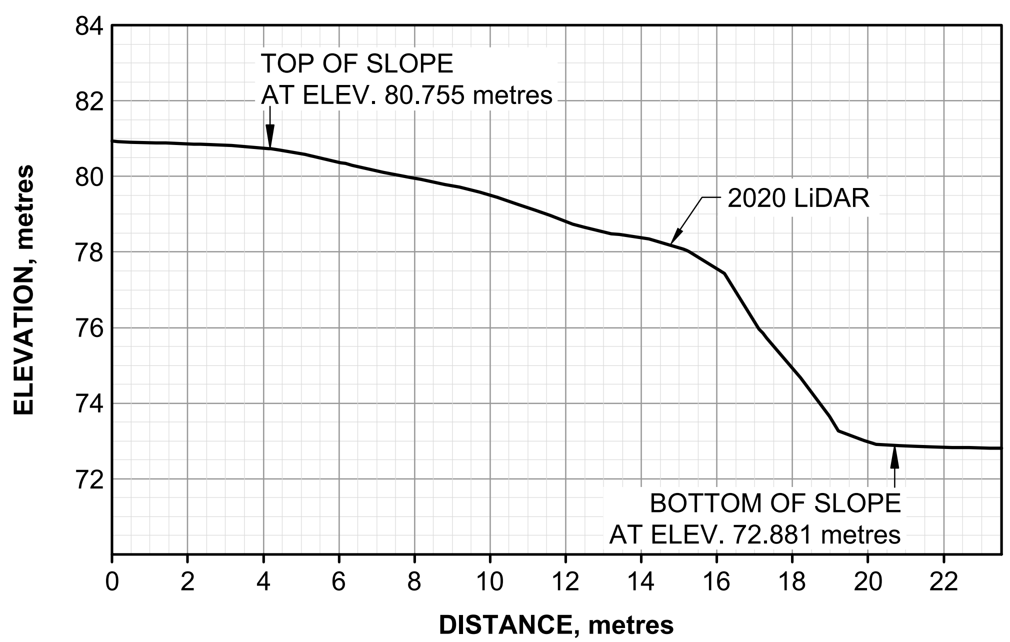

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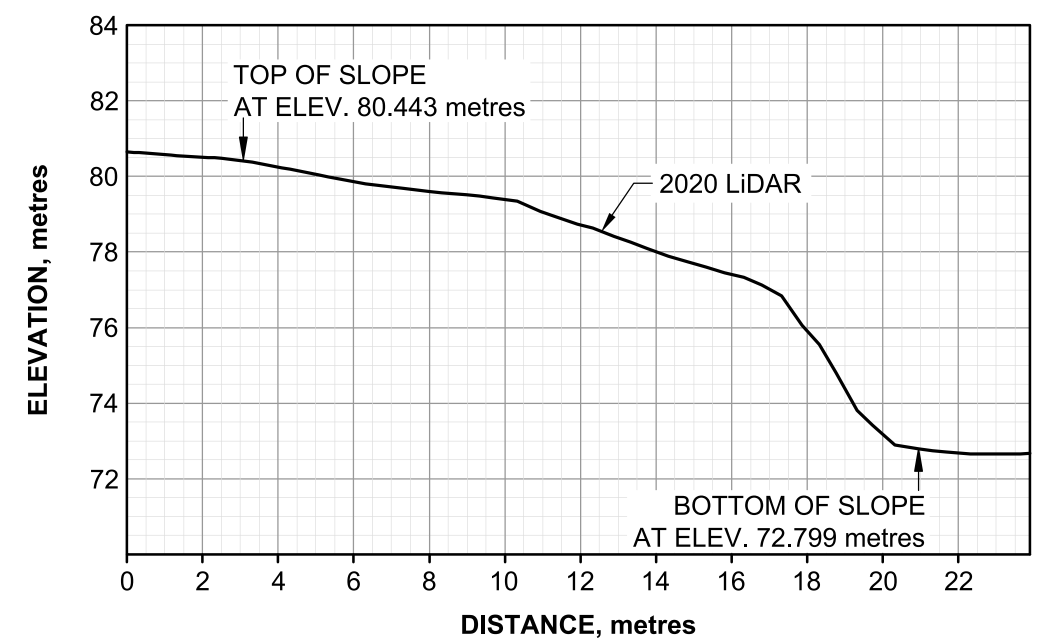

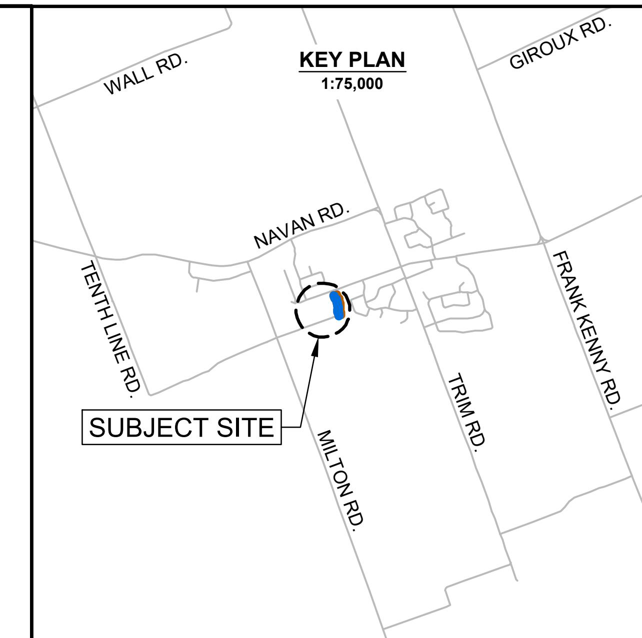

NOTE: Contours derived from 2020 LiDAR DEM provided by the City of Ottawa and are shown in metres.

Scale
1:200
0 4 8 12m

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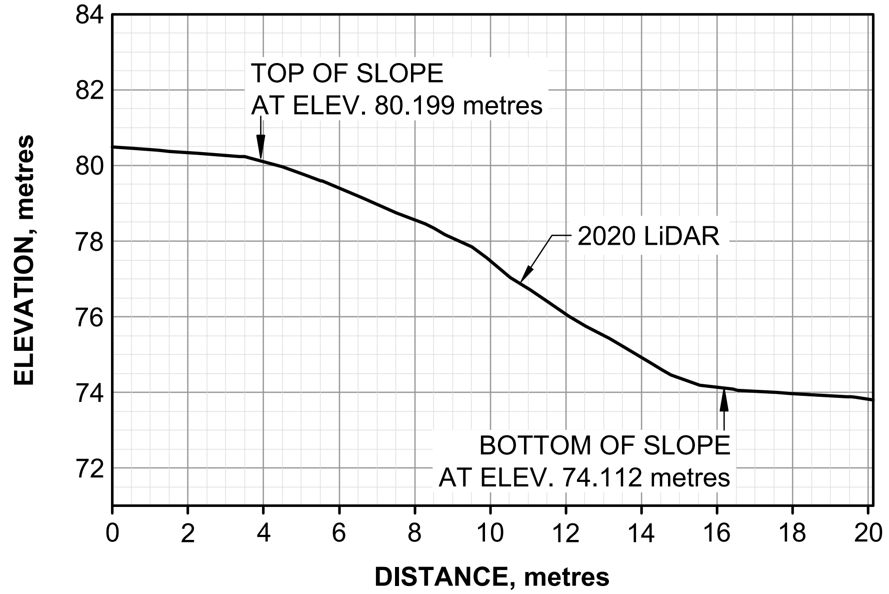
Drawing	SECTIONS (2 of 3)
Client	HIERARCHY DEVELOPMENT & DESIGN INC.
Project: 100812.001	GEOTECHNICAL INVESTIGATION PROPOSED LOT SEVERANCES 930 SMITH ROAD OTTAWA, ONTARIO
Drwn by: AGSD	Chkd by: WAM
Date: JUNE, 2024	Rev. 1
	FIGURE 4

SECTION S-S



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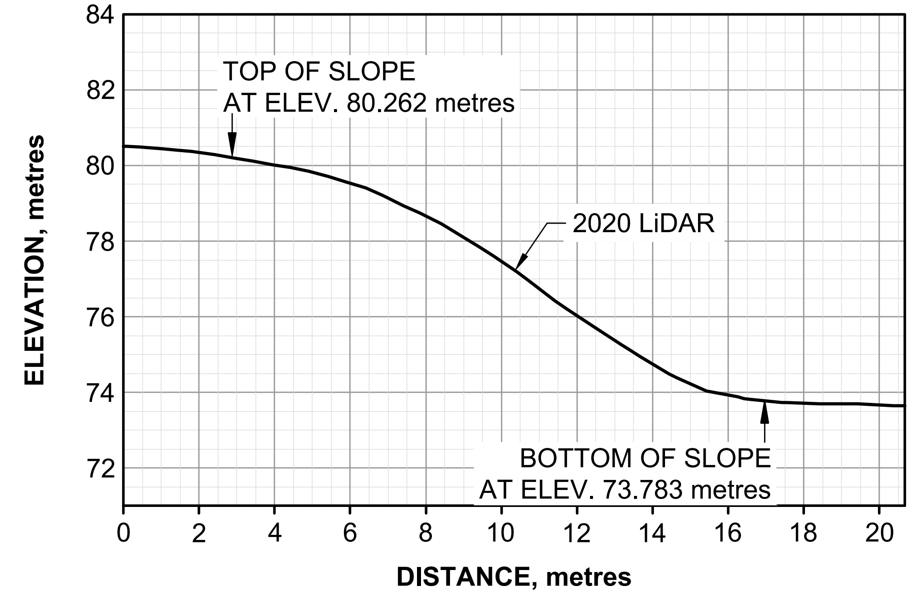

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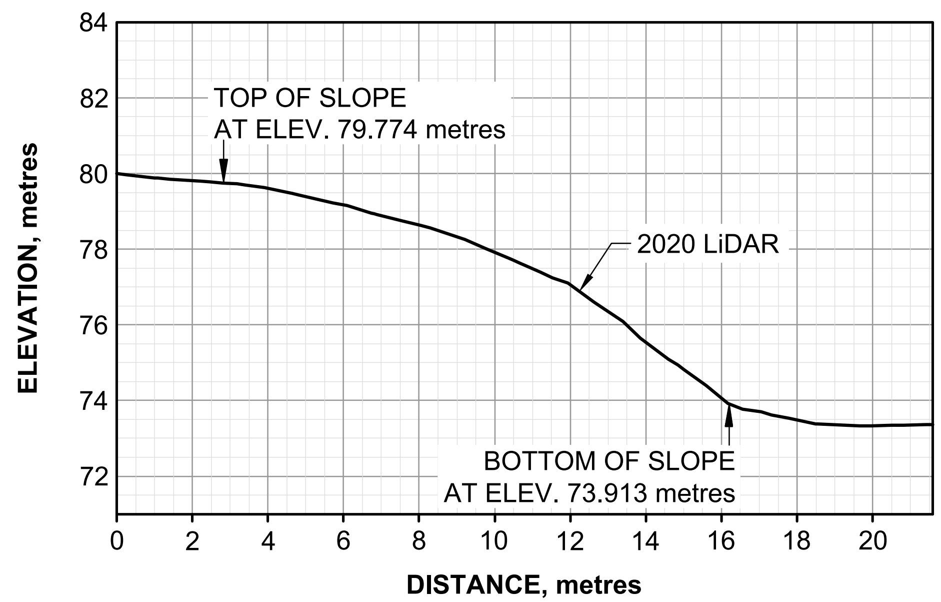

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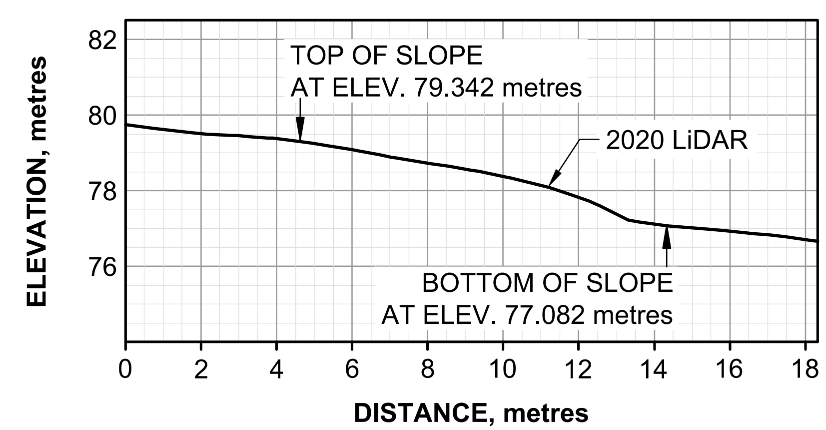

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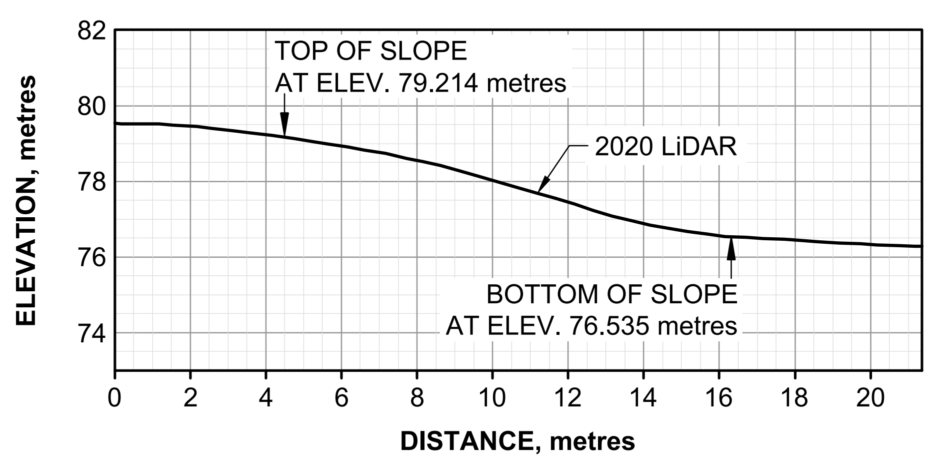
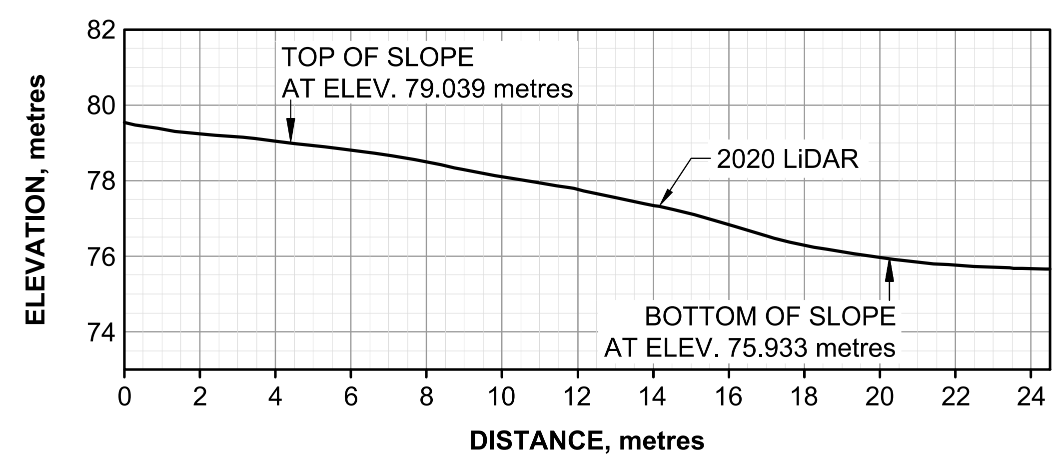


SECTION X-X



KEY PLAN
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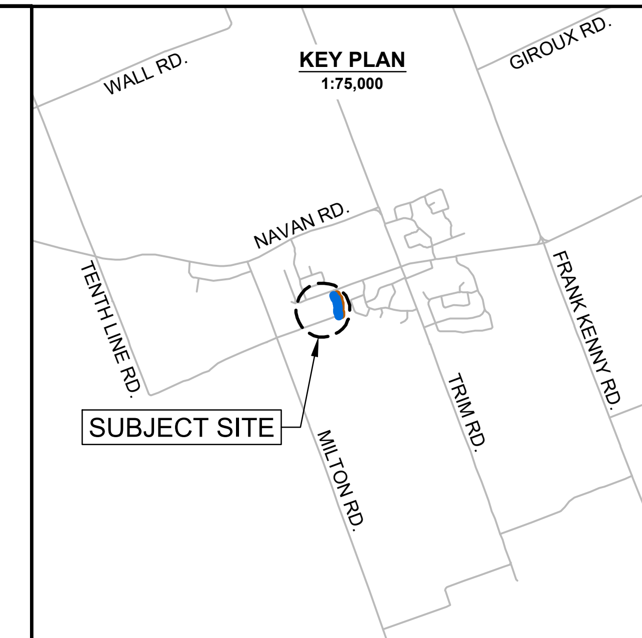

NOTE: Contours derived from 2020 LiDAR DEM provided by the City of Ottawa and are shown in metres.

Scale
1:200
0 4 8 12m

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Drawing	SECTIONS (3 of 3)
Client	HIERARCHY DEVELOPMENT & DESIGN INC.
Project	100812.001
Drwn by	AGSD
Chkd by	WAM
	GEOTECHNICAL INVESTIGATION PROPOSED LOT SEVERANCES 930 SMITH ROAD OTTAWA, ONTARIO
Date	JUNE, 2024
Rev.	1
	FIGURE 6

\\HC-DC01.AD.GEMTEC.CA\PROJECTS\PROJECTS\100800\100812.001\06_DRAFTING\SHEETS\100812.001_DWG_SLOPE HEIGHT_R1_2024-05-16.DWG

APPENDIX A

Record of Borehole Sheets
List of Abbreviations and Symbols
Boreholes 21-01, 21-02A, 21-02B, and 21-03

ABBREVIATIONS AND TERMINOLOGY USED ON RECORDS OF BOREHOLES AND TEST PITS

SAMPLE TYPES	
AS	Auger sample
CA	Casing sample
CS	Chunk sample
BS	Borros piston sample
GS	Grab sample
MS	Manual sample
RC	Rock core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled open shelby tube
TP	Thin-walled piston shelby tube
WS	Wash sample

SOIL TESTS	
w	Water content
PL, w_p	Plastic limit
LL, w_L	Liquid limit
C	Consolidation (oedometer) test
D_R	Relative density
DS	Direct shear test
G_S	Specific gravity
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	Organic content test
UC	Unconfined compression test
γ	Unit weight

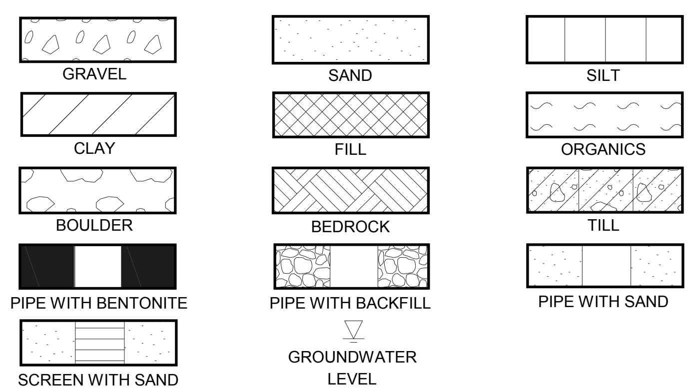
PENETRATION RESISTANCE

Standard Penetration Resistance, N
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 millimetres (30 in.) required to drive a 50 mm split spoon sampler for a distance of 300 mm (12 in.). For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

WH	Sampler advanced by static weight of hammer and drill rods
WR	Sampler advanced by static weight of drill rods
PH	Sampler advanced by hydraulic pressure from drill rig
PM	Sampler advanced by manual pressure

COHESIONLESS SOIL Compactness		COHESIVE SOIL Consistency	
SPT N-Values	Description	Cu, kPa	Description
0-4	Very Loose	0-12	Very Soft
4-10	Loose	12-25	Soft
10-30	Compact	25-50	Firm
30-50	Dense	50-100	Stiff
>50	Very Dense	100-200	Very Stiff
		>200	Hard



GRAIN SIZE

0.01 | 0.1 | 1.0 | 10 | 100 | 1000mm

SILT CLAY	SAND			GRAVEL	COBBLE	BOULDER
	Fine	Medium	Coarse			

0.08 | 0.4 | 2 | 5 | 80 | 200

DESCRIPTIVE TERMINOLOGY

(Based on the CANFEM 4th Edition)

0 | 10 | 20 | 35

TRACE	SOME	ADJECTIVE	noun > 35% and main fraction
trace clay, etc	some gravel, etc.	silty, etc.	sand and gravel, etc.

RECORD OF BOREHOLE 21-01

CLIENT: Hierarchy Development & Design Inc.
PROJECT: Geotechnical Investigation, Proposed Lot Severances, 830 Smith Road, Ottawa, Ontario
JOB#: 100812.001
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 12 2021

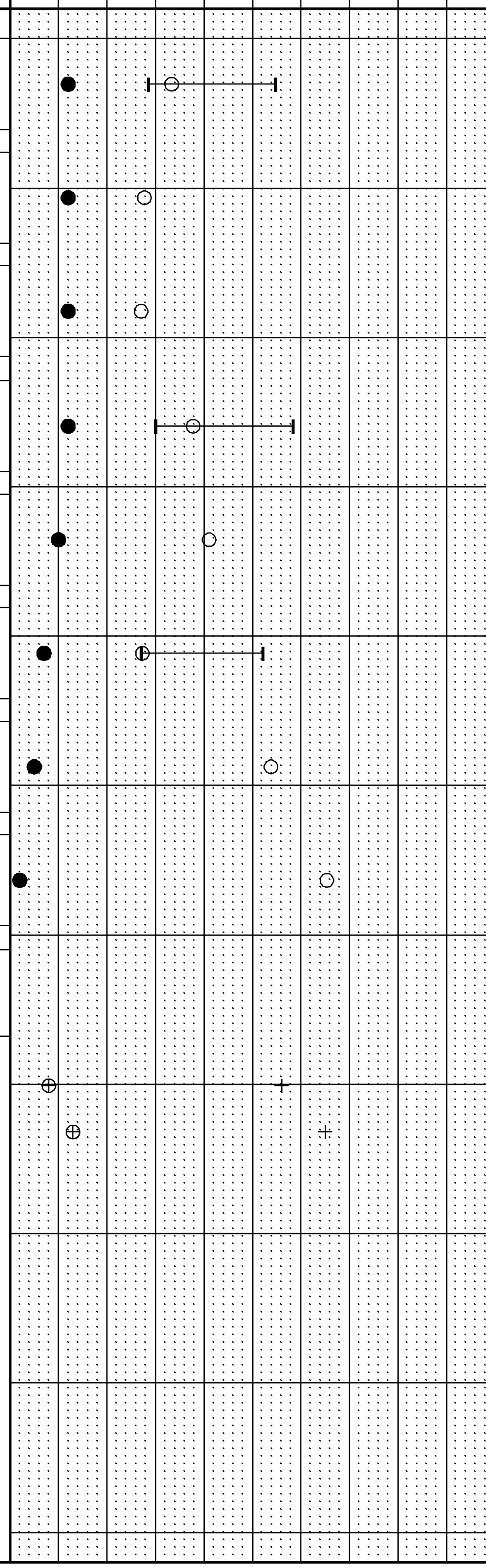
BORING METHOD: Power Auger, Hollow Stem Auger (210mm OD)

SOIL PROFILE

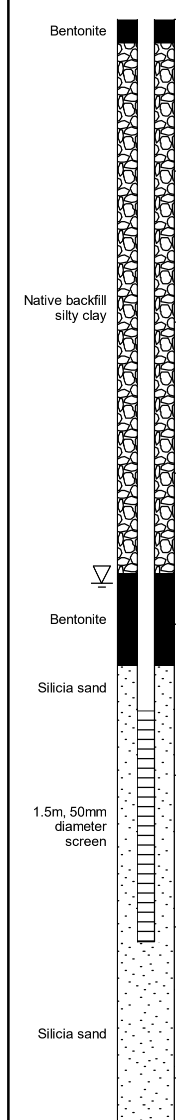
DESCRIPTION	ELEV. (m)	DEPTH (m)
Ground Surface	78.94	
TOPSOIL	78.76	0.18
Very stiff to stiff, grey brown SILTY CLAY (WEATHERED CRUST)	73.61	5.33
Stiff, grey SILTY CLAY	71.62	7.32
End of borehole		

SAMPLES

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	200	12	
2	SS	610	12	
3	SS	610	12	MH
4	SS	610	12	
5	SS	610	10	
6	SS	610	7	
7	SS	610	5	
8	SS	610	2	
9	TO	580	PH	



PIEZOMETER OR STANDPIPE INSTALLATION



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
21/09/13	3.7 ▽	75.2

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LOGGED: A.N.
CHECKED: W.A.M.

RECORD OF BOREHOLE 21-02A

CLIENT: Hierarchy Development & Design Inc.
PROJECT: Geotechnical Investigation, Proposed Lot Severances, 830 Smith Road, Ottawa, Ontario
JOB#: 100812.001
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 12 2021

BORING METHOD: Power Auger, Hollow Stem Auger (210mm OD)

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	80.95						
	TOPSOIL	80.82 / 0.13						Bentonite
	Very loose to loose, brown SILTY SAND	0.28	1	SS	560	4		
1	Very stiff to stiff, grey brown SILTY CLAY (WEATHERED CRUST)		2	SS	610	8		
2			3	SS	610	6		
			4	SS	610	4		Native backfill
3	Firm grey SILTY CLAY to CLAYEY SILT	77.90 / 3.05	5	SS	610	2		
4								
5			6	SS	610	WH		
6								Bentonite
			7	SS	610	WH	MH	Silicia sand
7								1.5m, 50mm diameter screen
8			8	SS	560	WH		
	End of borehole Auger refusal	72.77 / 8.18						
9								
10								

PENETRATION RESISTANCE (N), BLOWS/0.3m
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA: + NATURAL ⊕ REMOULDED
WATER CONTENT, %: W_P — W — W_L
10 20 30 40 50 60 70 80 90

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
21/09/13	2.1 ∇	78.9

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LOGGED: A.N.
CHECKED: W.A.M.

RECORD OF BOREHOLE 21-02B

CLIENT: Hierarchy Development & Design Inc.
PROJECT: Geotechnical Investigation, Proposed Lot Severances, 830 Smith Road, Ottawa, Ontario
JOB#: 100812.001
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 12 2021

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	PENETRATION RESISTANCE (N), BLOWS/0.3m / DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH (Cu), kPA + NATURAL ⊕ REMOULDED / WATER CONTENT, % W_P W W_L (10 20 30 40 50 60 70 80 90)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger; Hollow Stem Auger (210mm OD)	Ground Surface		80.95							
		TOPSOIL		80.82 / 0.13							Backfilled with auger cuttings
		Very loose to loose, brown SILTY SAND		80.67 / 0.28							
		Very stiff to stiff, grey brown silty clay (WEATHERED CRUST)									
3		Firm grey SILTY CLAY		77.90 / 3.05							
6					1	TO	550	PH			
		End of borehole Note: Soil statigraphy inferred from Borehole 21-02A		74.30 / 6.65							

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CHECKED: W.A.M.

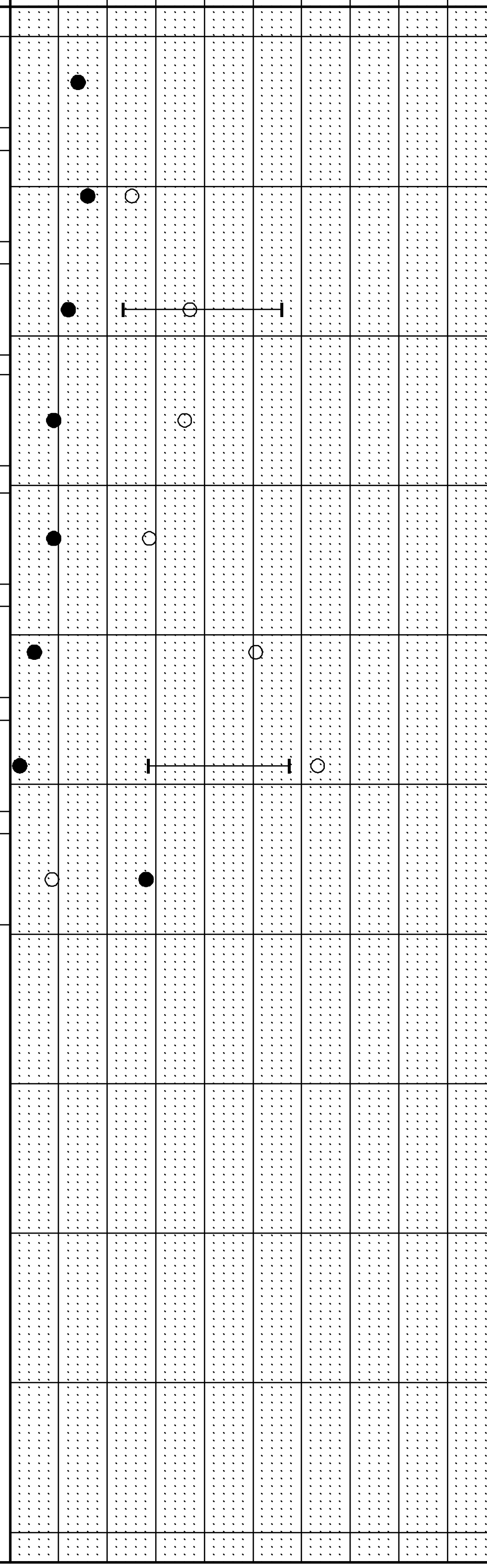
RECORD OF BOREHOLE 21-03

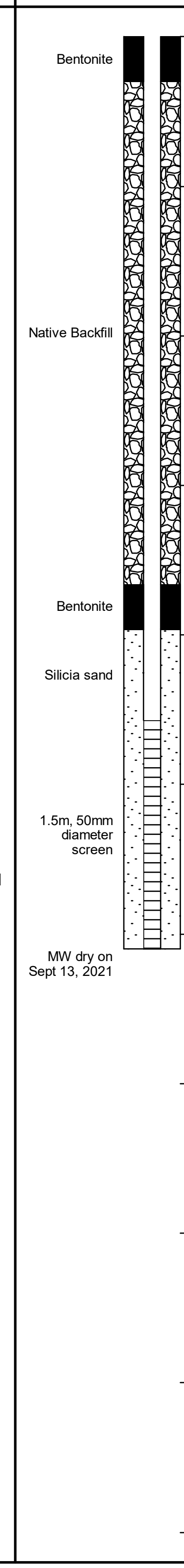
CLIENT: Hierarchy Development & Design Inc.
PROJECT: Geotechnical Investigation, Proposed Lot Severances, 830 Smith Road, Ottawa, Ontario
JOB#: 100812.001
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 12 2021

BORING METHOD: Power Auger — Hollow Stem Auger (210mm OD)

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	80.08					
TOPSOIL	79.90 / 0.18					
Very stiff to stiff, grey brown SILTY CLAY (WEATHERED CRUST)		1	SS	400	14	
		2	SS	610	16	
		3	SS	610	12	
		4	SS	610	9	
		5	SS	610	9	
		6	SS	610	5	
	75.20 / 4.88	7	SS	610	2	
Stiff to firm, grey SILTY CLAY	74.75 / 5.33					
Compact, grey SILTY SAND, some gravel, some clay, with cobbles and boulders (GLACIAL TILL)		8	SS	225	28	MH
End of borehole	73.98 / 6.10					





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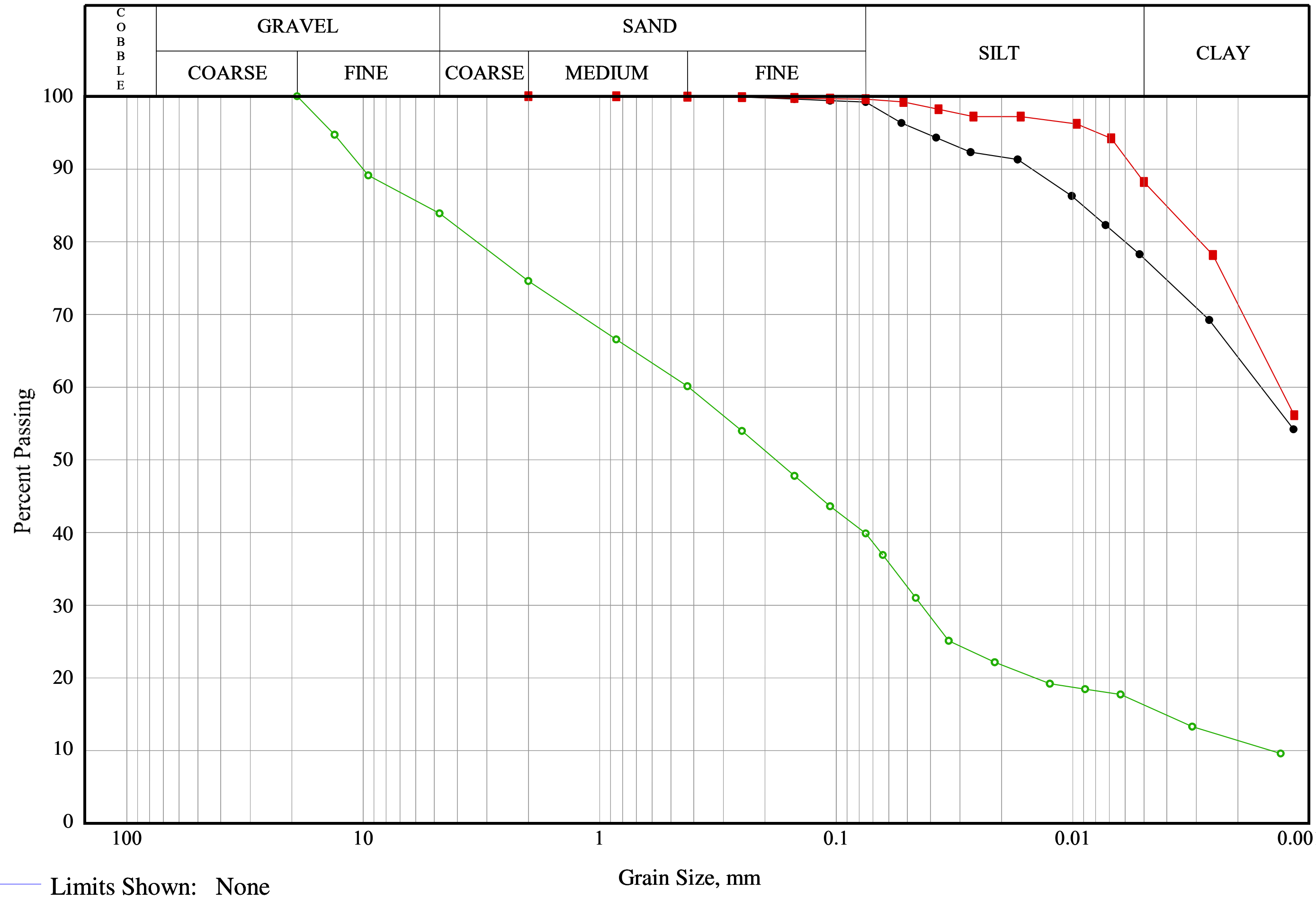
LOGGED: A.N.
CHECKED: W.A.M.

APPENDIX B

Geotechnical Laboratory Testing
Grain Size Distribution Testing
Plasticity Index Testing

GEMTEC Consulting Engineers and Scientists	Client: Hierarchy Development & Design	Soils Grading Chart (T88)
	Project: Geotechnical, Hydrogeological, and Erosion and Sedime	
	Project #: 100812001	

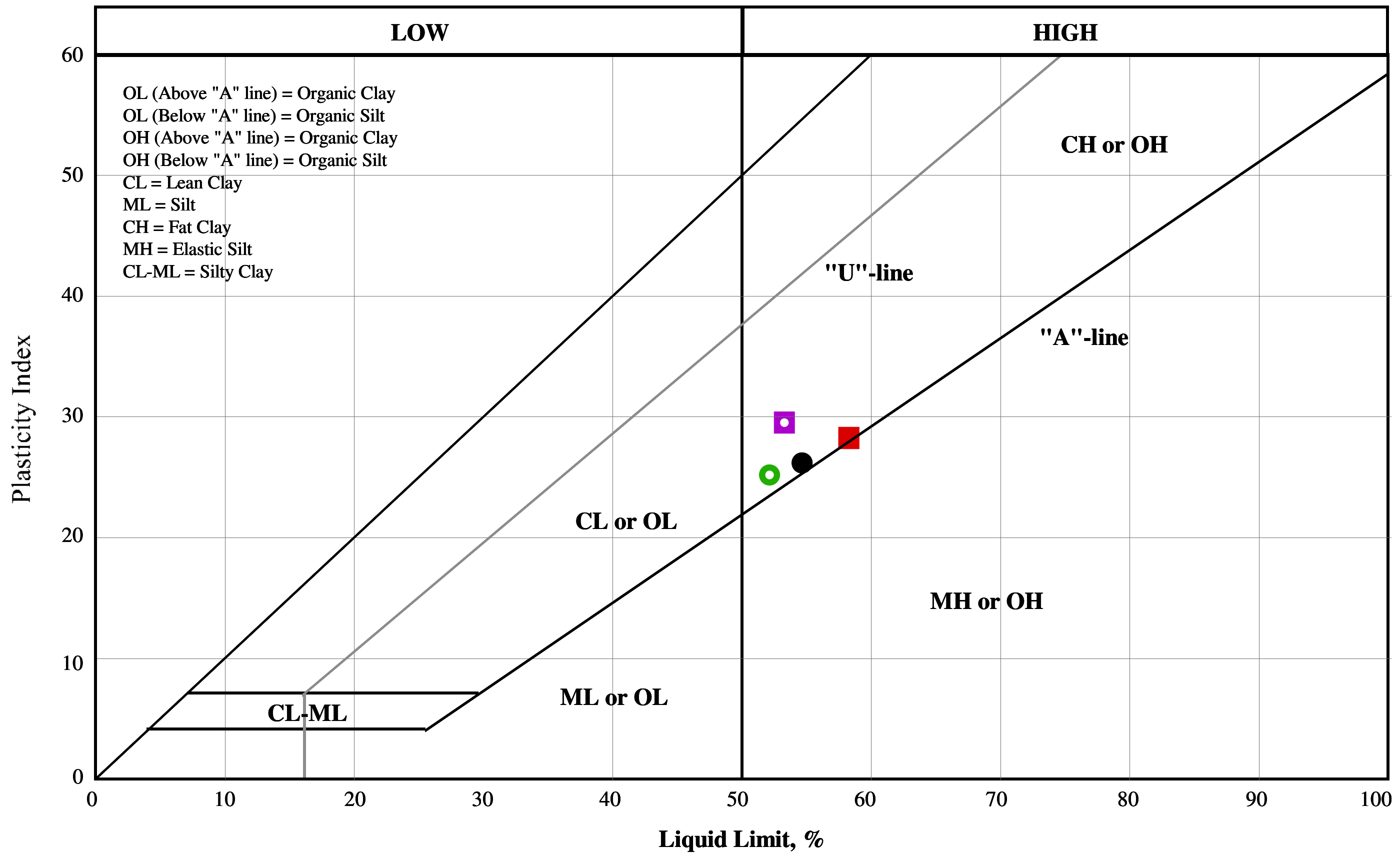


Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
●(black)	SILT CLAY (WEATHERED CRUST)	21-01	SA 3	1.52-2.13	0.0	0.8	21.5	77.7
■(red)	SILTY CLAY	21-02A	SA 7	6.09-6.71	0.0	0.4	11.4	88.2
○(green)	GLACIAL TILL	21-03	SA 8	5.33-5.94	16.1	44.0	23.6	16.3

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
●(black)	Silty clay , trace sand	N/A	---	---	---	---	0.00	0.01	20.9
■(red)	Clay , some silt , trace sand	N/A	---	---	---	---	0.00	0.00	11.4
○(green)	Silty sand , some gravel, some clay	N/A	0.00	0.00	0.04	0.18	0.42	5.51	23.6

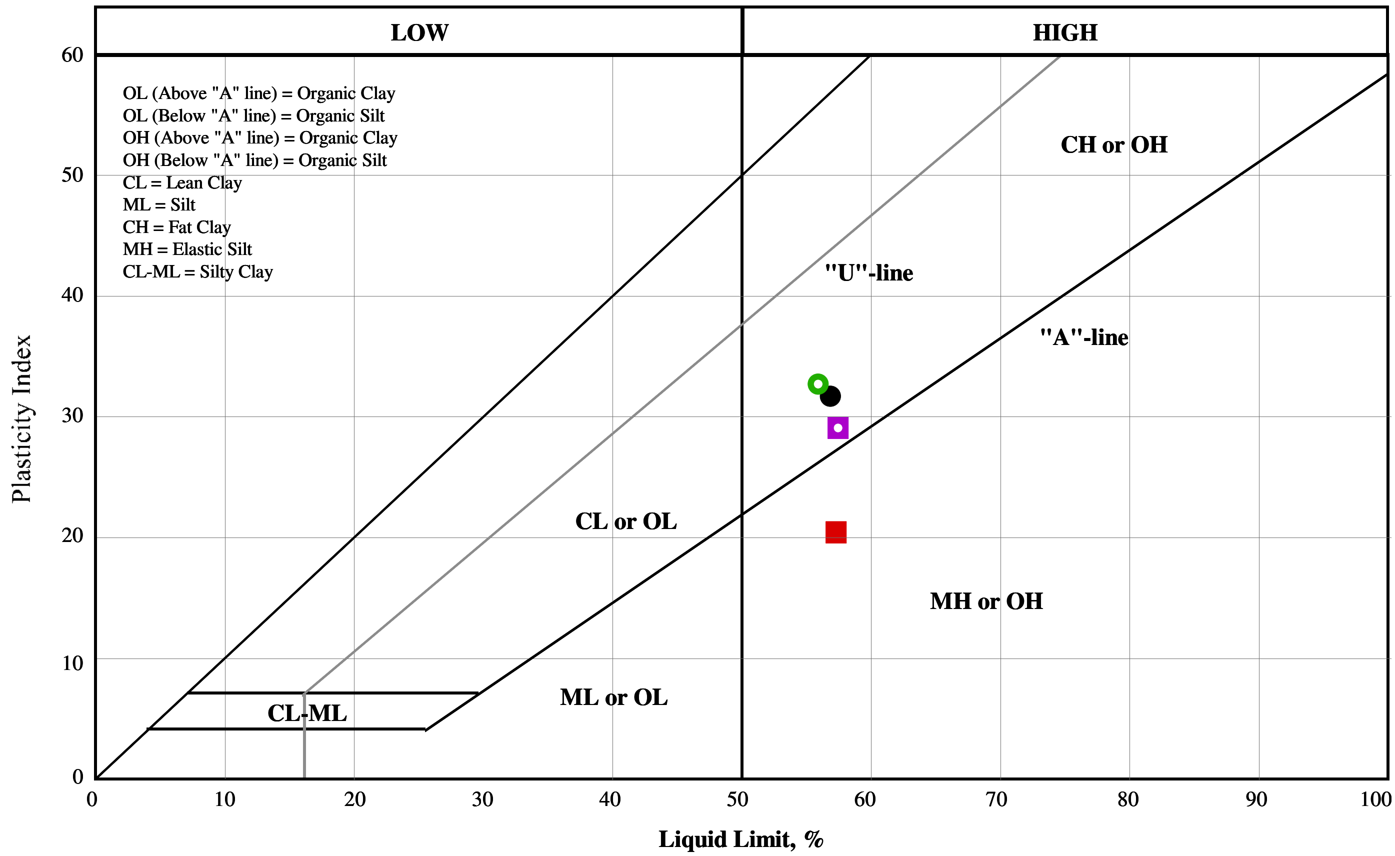
GEMTEC Consulting Engineers and Scientists		
Client:	Hierarchy Development & Design	Plasticity Chart (D4318)
Project:	Geotechnical, Hydrogeological, and Erosion and Sedime	
Project #:	100812001	



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	21-01	SA 2	0.76-1.37	54.7	28.5	26.2	☐	33.33
■	21-01	SA 4	2.28-2.89	58.3	30.0	28.2	☐	37.76
○	21-01	SA 6	3.81-4.42	52.1	27.0	25.2	☐	27.29
▣	21-02A	SA 2	0.76-1.37	53.3	23.8	29.5	☐	36.61

Note: More information available upon request

GEMTEC Consulting Engineers and Scientists	Client: Hierarchy Development & Design	Plasticity Chart (D4318)
	Project: Geotechnical, Hydrogeological, and Erosion and Sedime	
	Project #: 100812001	



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	21-02A	SA 4	2.28-2.89	56.8	25.1	31.7	☐	49.08
■	21-02A	SA 6	4.57-5.18	57.3	36.9	20.4	☐	76.57
○	21-03	SA 3	1.52-2.13	55.9	23.2	32.7	☐	37.03
▣	21-03	SA 7	4.57-5.18	57.4	28.4	29.1	☐	63.33

Note: More information available upon request

GEMTEC www.gemtec.ca	Shrinkage Limit ASTM D4943

Volume of Shrinkage Dish

Mass of Glass Plate (g):	37.33
Mass of Shrinkage Dish (g) (m):	20.70
Mass of Shrinkage Dish, Plate, Grease and Water (g):	75.40
Mass of Water (g):	17.37
Volume of Shrinkage Dish:	17.0

Test Specimen

Specimen No:	**1**
Mass of Shrinakge Dish, m (g):	20.81
Mass of Shrinkage Dish and Wet Soil, m_w (g):	48.71
Mass of Shrinkage Dish and Dry Soil, m_d (g):	38.45
Mass of Wax-Coated Soil in Air, m_{sxa} (g):	18.36
Mass of Wax-Coated Soil in Water, m_{sxw} (g):	7.9

Calculated Shrinkage Limit

Specimen No:	**1**
Mass of Dry Soil, m_s (g):	17.64
Water Content of Soil when Placed in Dish, w (%):	58.16
Mass of Water Displaced by Wax-Coated Soil, m_{wsx} (g):	10.46
Volume of Dry Soil and Wax, V_{dx} (cm^3):	10.46
Mass of Wax, m_x (g):	0.72
Volume of Wax, V_x (cm^3):	0.80
Volume of Dry Soil, V_d (cm^3):	9.66
Shrinkage Limit, SL	16.55

Specific Gravity of Wax = 0.908 at15.5°C

Specific Gravity of Wax = 0.900 at 20°C

Density of Water (g/cm^3) = 1.000 (g/cm^3)

Project No: 100812.001	**Tested By: K.N**
Project Name: 908 Smith Road, Ottawa, Ont	**Checked By: K.S.**
Date Tested: Sept 15, 2021	**Sample No: 21-02A SA 3**
Sample Date: Sept 3, 2021	**Source: N/A**
Remarks:	**Depth: 1.52-2.13 m**

APPENDIX C

Chemical Analysis of Soil Samples
Samples Relating to Corrosion
(Paracel Laboratories Ltd. Order No. 2134503)

Certificate of Analysis

Client: **GEMTEC Consulting Engineers and Scientists Limited**

Client PO:

Report Date: 25-Aug-2021

Order Date: 18-Aug-2021

Project Description: 100812.001

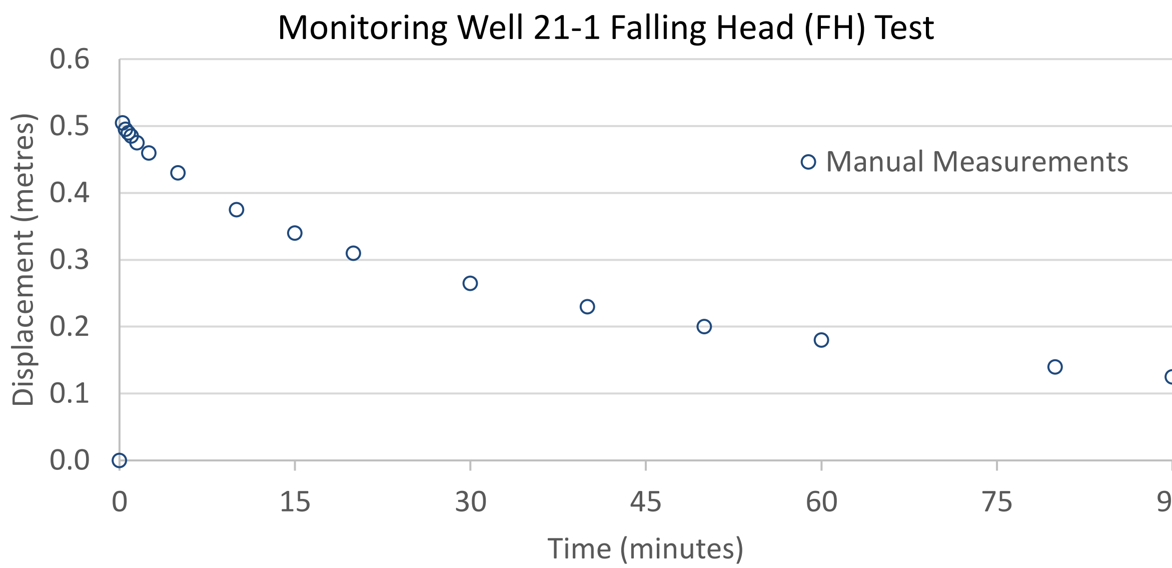
	Client ID:	BH21-2A SA3	-	-	-
	Sample Date:	18-Aug-21 15:43	-	-	-
	Sample ID:	2134503-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	68.9	-	-	-
General Inorganics					
Conductivity	5 uS/cm	210	-	-	-
pH	0.05 pH Units	6.71	-	-	-
Resistivity	0.10 Ohm.m	47.5	-	-	-
Anions					
Chloride	5 ug/g dry	53	-	-	-
Sulphate	5 ug/g dry	57	-	-	-

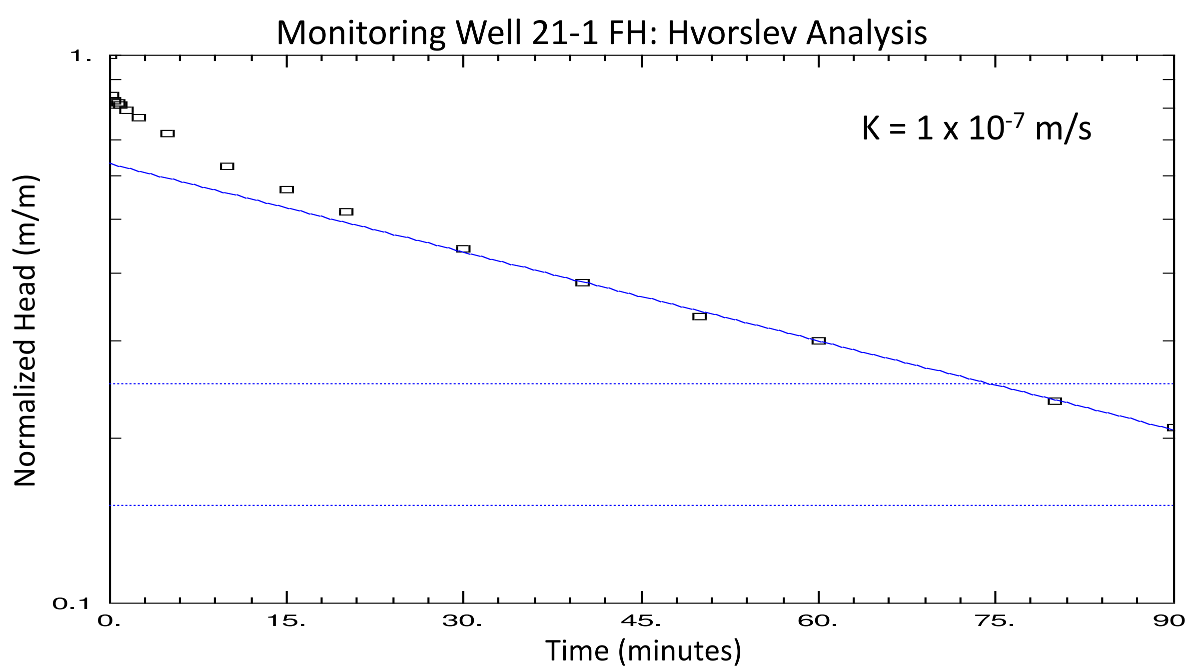
APPENDIX D

Hydraulic Conductivity Test Results
Figure D1 and D2

Slug Test Results

FIGURE D1





Well Data:

Displacement observed (slug size): 0.51 metres (0.60 m)

Well Depth: 7.32 metres

Screen Length: 3.05 metres

Well Radius: 0.0255 metres

Aquifer Data

Saturated Thickness: 3.59 metres

Anisotropy Ratio (Kz/Kr): 0.1

Aquifer Model: Unconfined, Hvorslev

Static Water Level: 3.73 metres bgs

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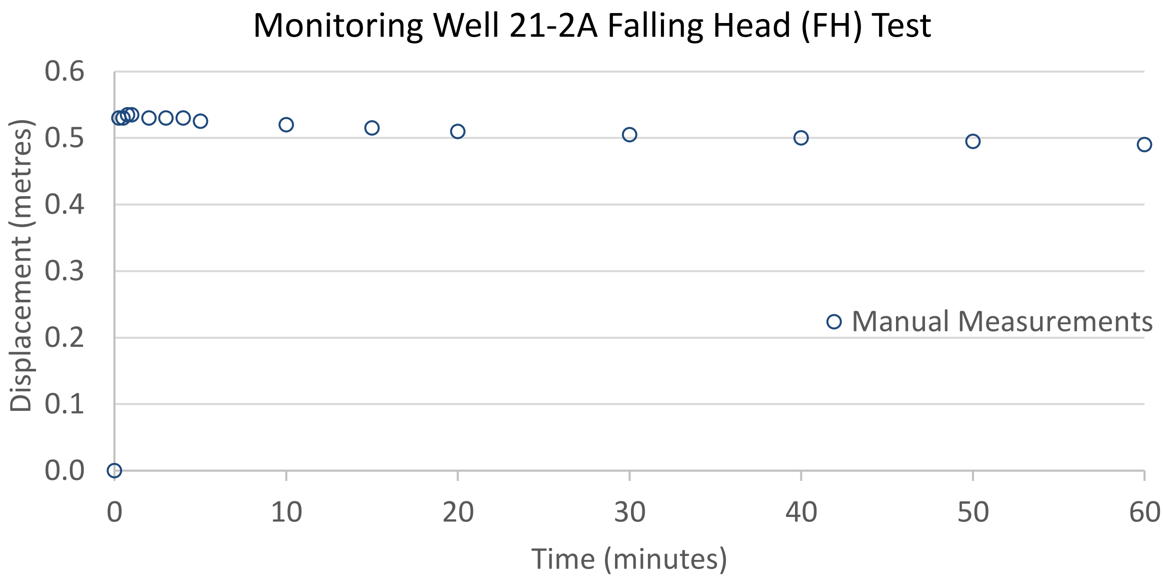
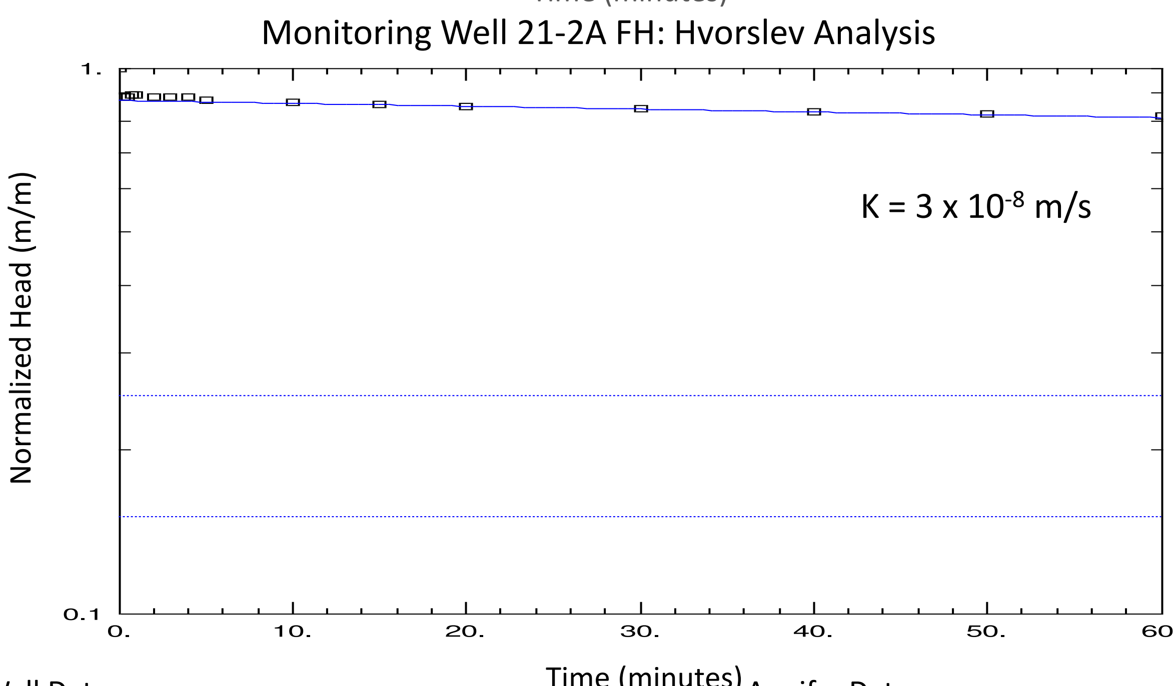
Date: Jan. 31, 2023

Project: 100812.001

Slug Test Results

FIGURE D2





Well Data:
Displacement observed (slug size): 0.53 metres (0.60 m)
Well Depth: 8.18 metres
Screen Length: 1.52 metres
Well Radius: 0.0255 metres

Aquifer Data
Saturated Thickness: 6.10 metres
Anisotropy Ratio (Kz/Kr): 0.1
Aquifer Model: Unconfined, Hvorslev
Static Water Level: 2.08 metres bgs

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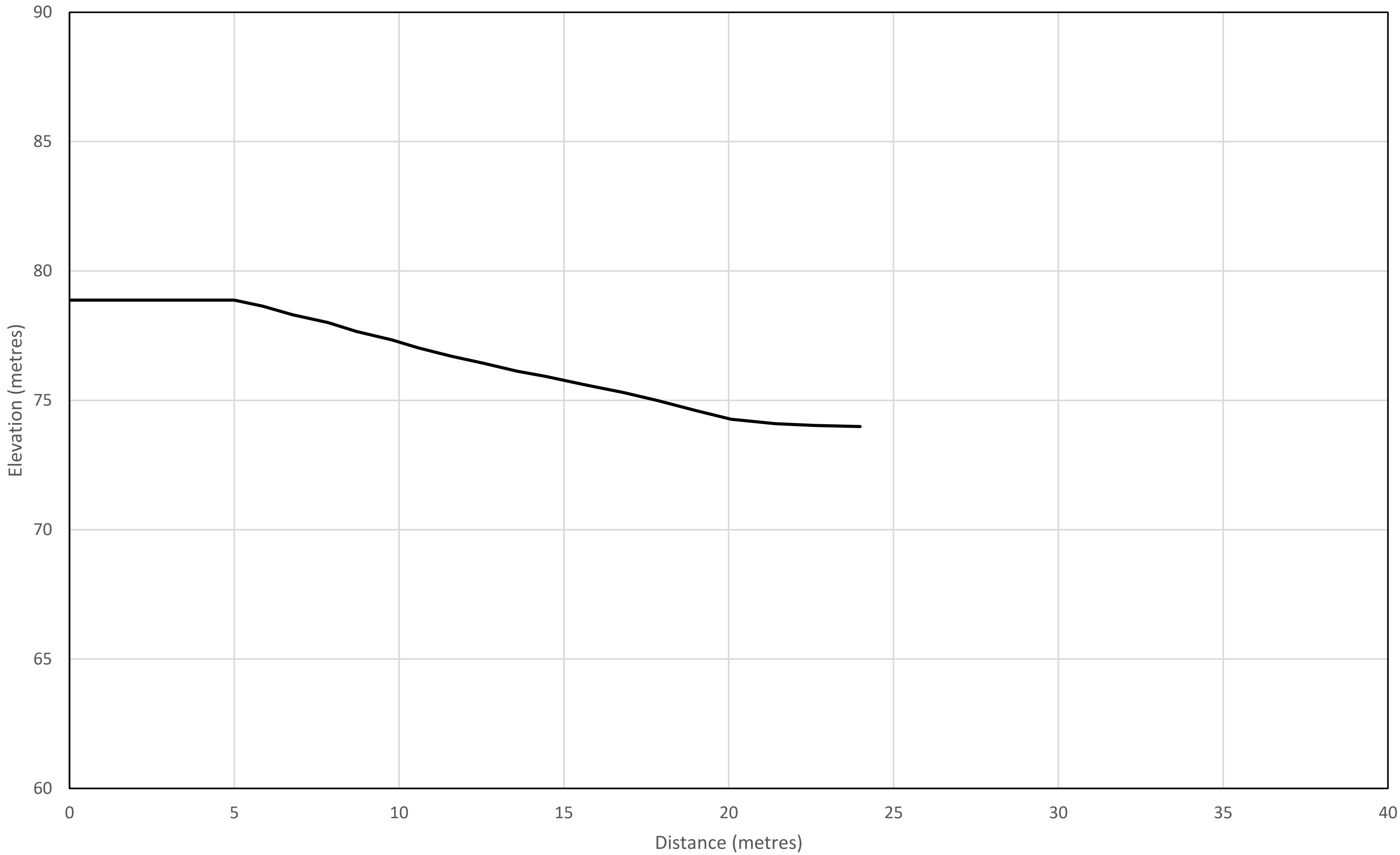
Date: Jan. 31, 2023

Project: 100812.001

APPENDIX E

Slope Stability Analysis – Southwest Corner of Site
Figure E1 – Cross Section A-A
Figure E2 – Cross Section B-B
Figure E3 – Cross Section C-C
Figure E4 and E5 –Slope Stability Analysis Cross Section C-C



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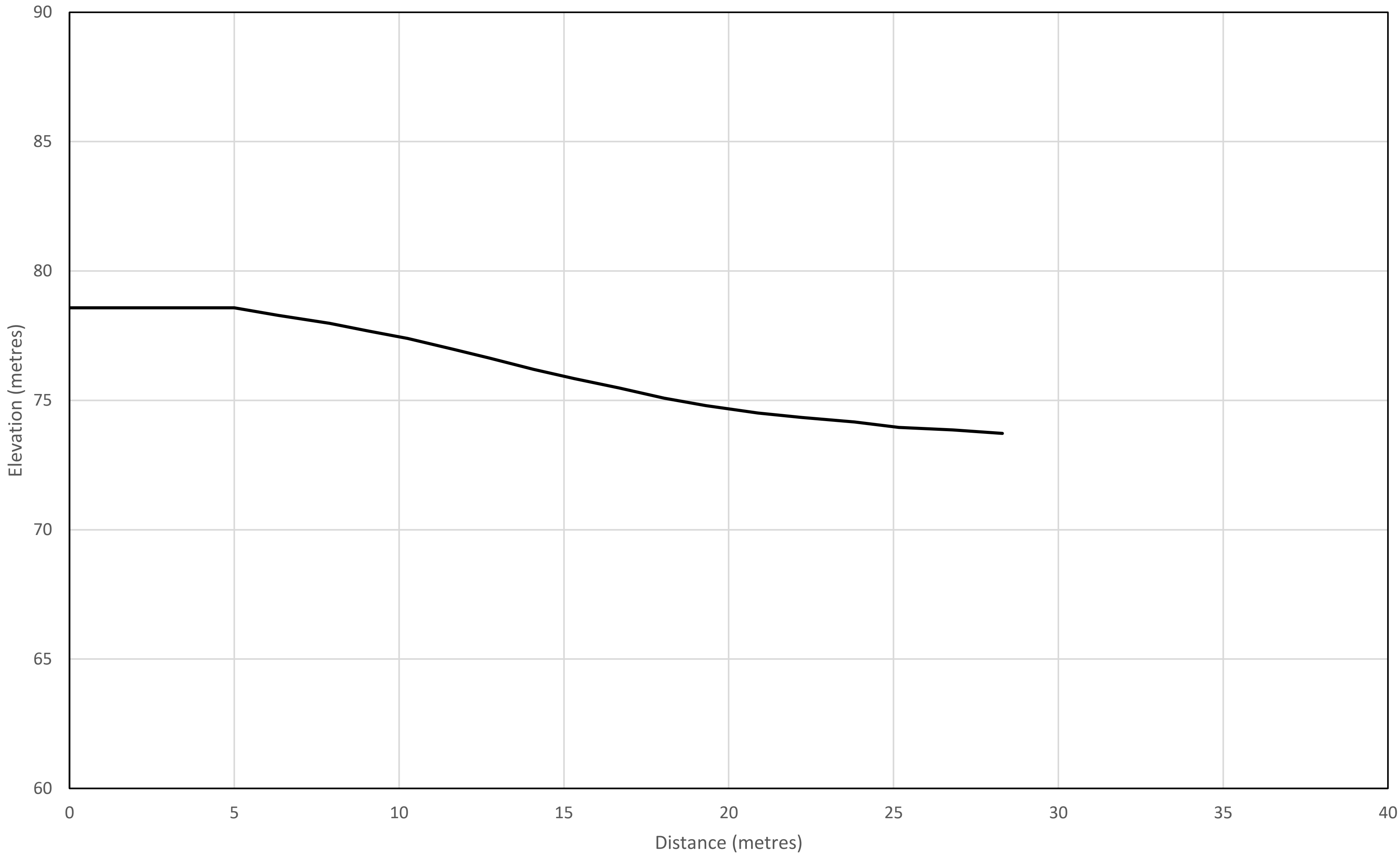
Slope Cross Section A-A

930 Smith Road

Ottawa, Ontario

Project No.	100812.001
Drawn:	WAM
Date:	21/02/2023

Figure E1



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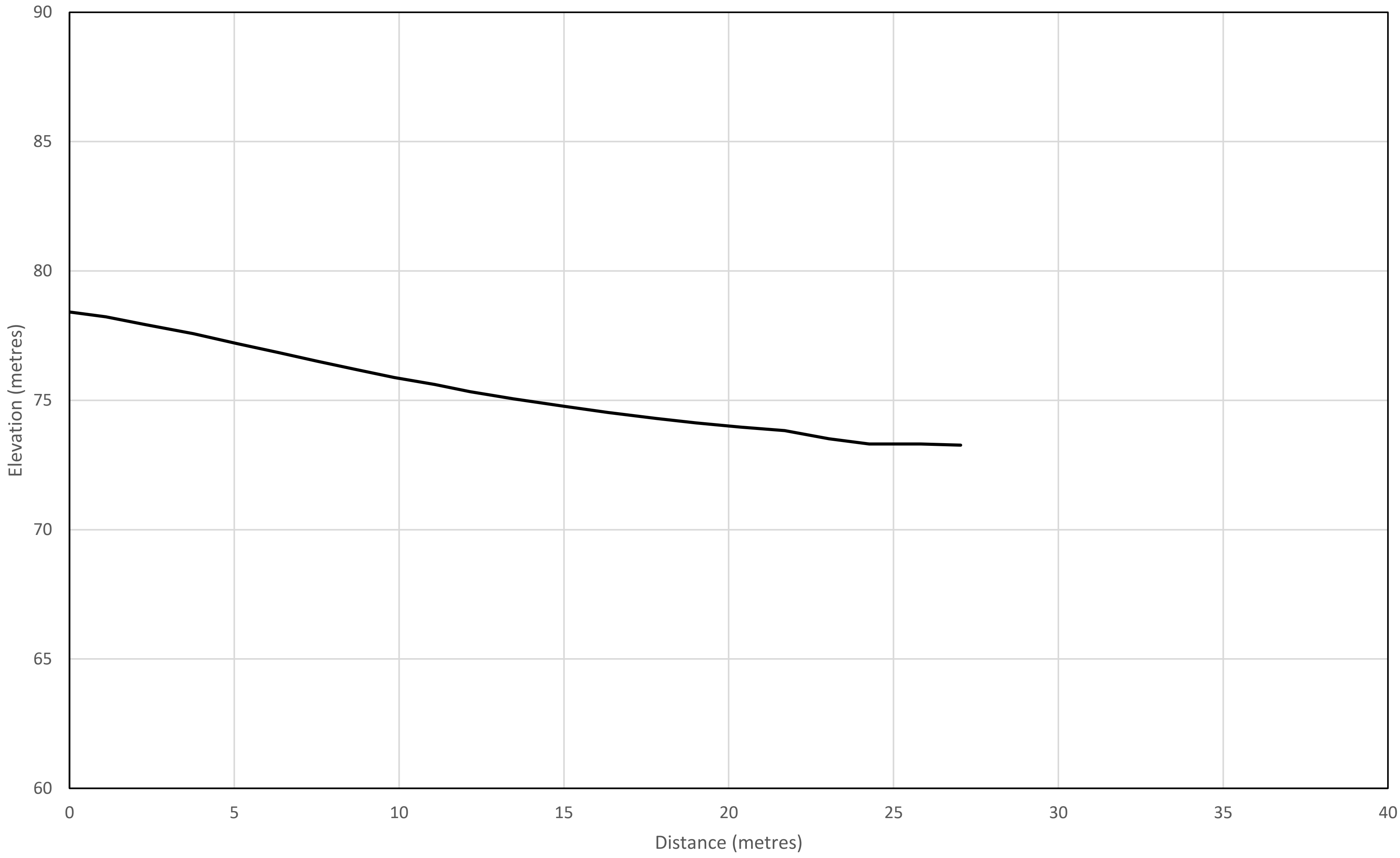
Slope Cross Section B-B

930 Smith Road

Ottawa, Ontario

Project No.	100812.001
Drawn:	WAM
Date:	21/02/2023

Figure E2



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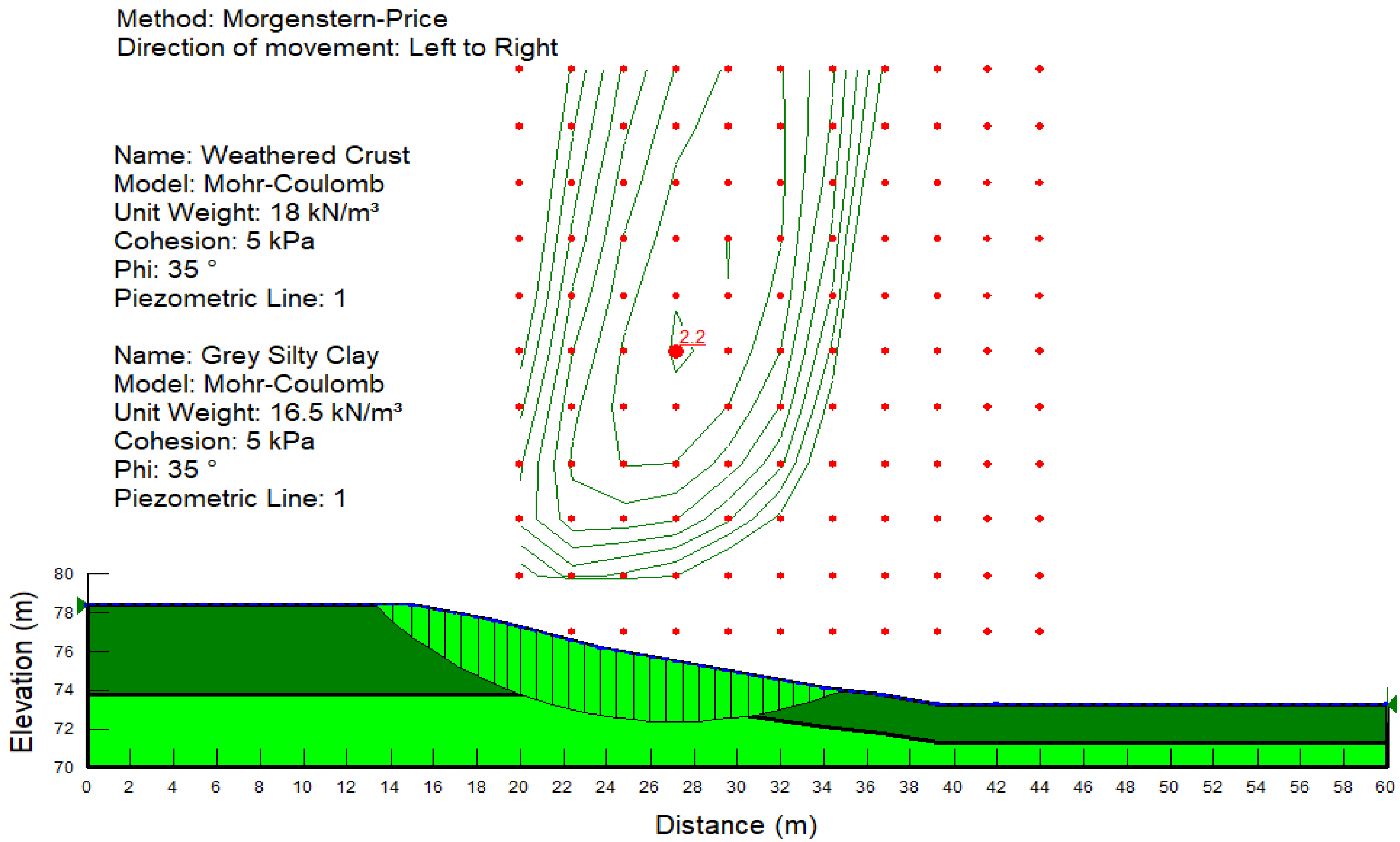
Slope Cross Section C-C

930 Smith Road

Ottawa, Ontario

Project No.	100812.001
Drawn:	WAM
Date:	21/02/2023

Figure E3



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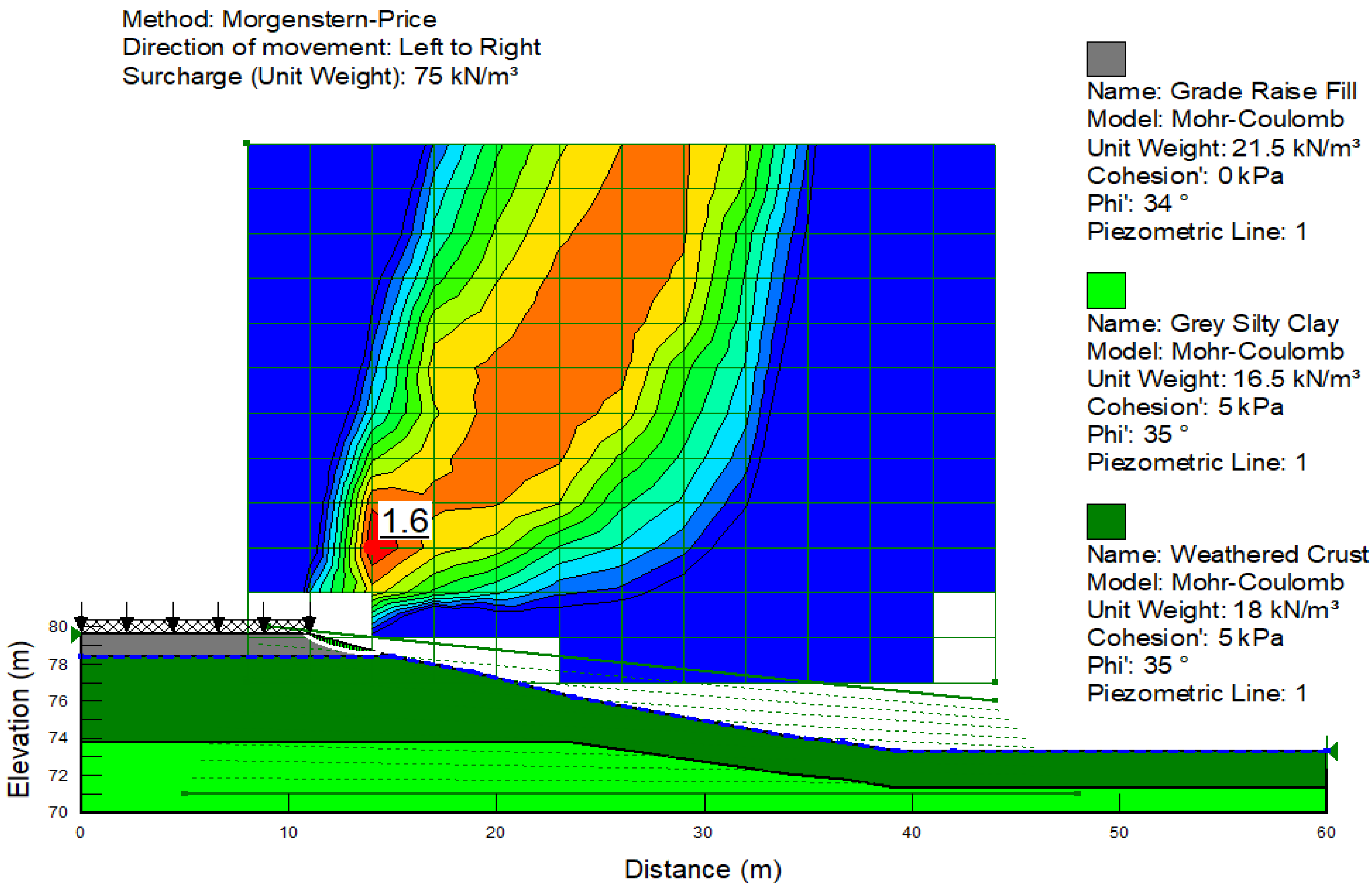
Slope Stability Analysis - Cross Section C-C

930 Smith Road

Ottawa, Ontario

Project No.	100812.001
Drawn:	WAM
Date:	21/02/2023

Figure E4



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Slope Stability Analysis - Cross Section C-C

930 Smith Road

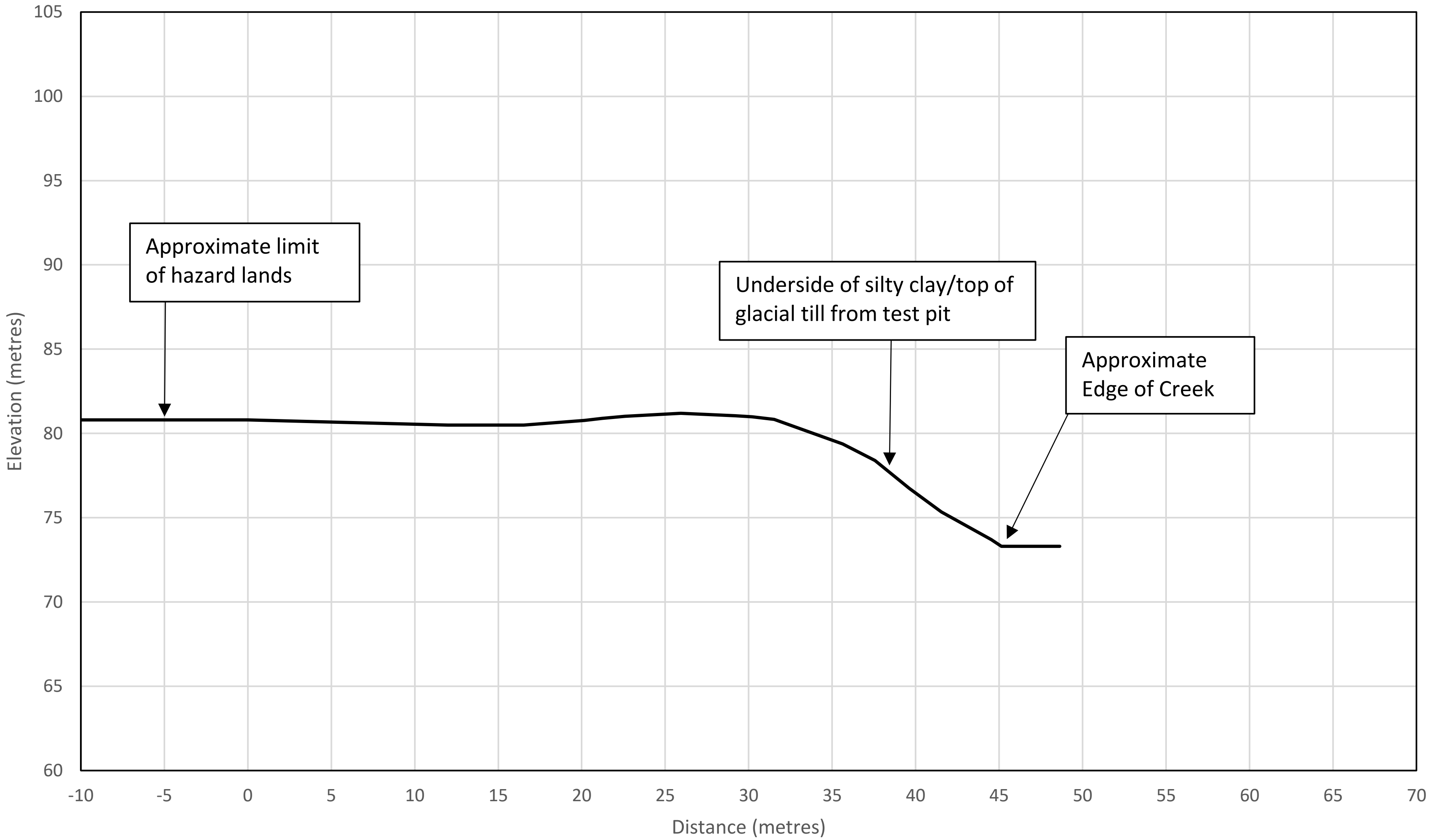
Ottawa, Ontario

Project No.	100812.001
Drawn:	WAM
Date:	01/11/2023

Figure E5

APPENDIX F

Slope Stability Analysis – McKinnon's Creek
Figure F1 – Cross Section D-D
Figure F2 – Cross Section E-E
Figure F3 – Cross Section F-F
Figure F4 – Cross Section G-G
Figure F5 – Cross Section H-H
Figure F6 to F11 –Slope Stability Analysis Cross Sections D-D and G-G



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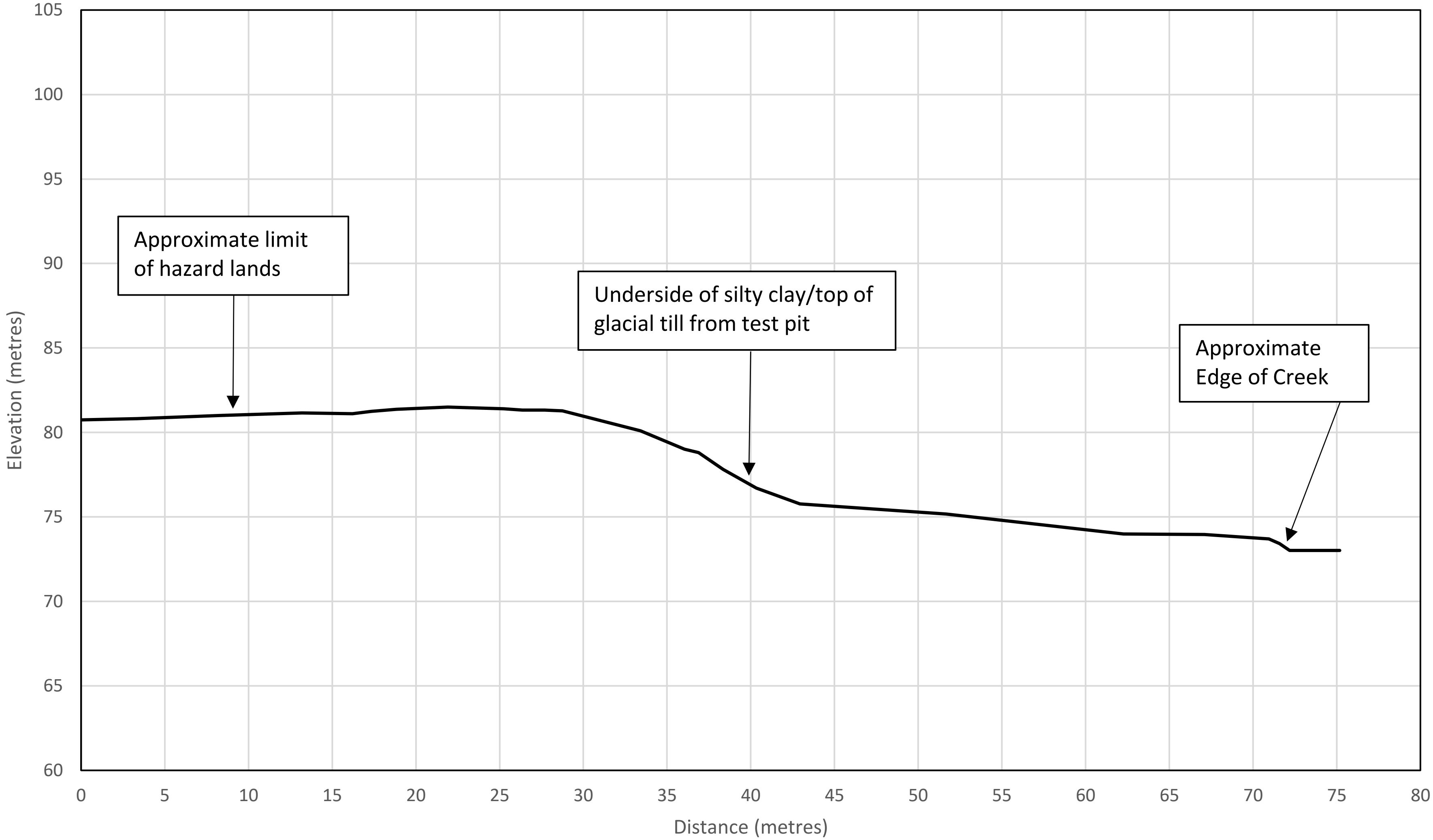
Slope Cross Section DD

Proposed Lot Severances

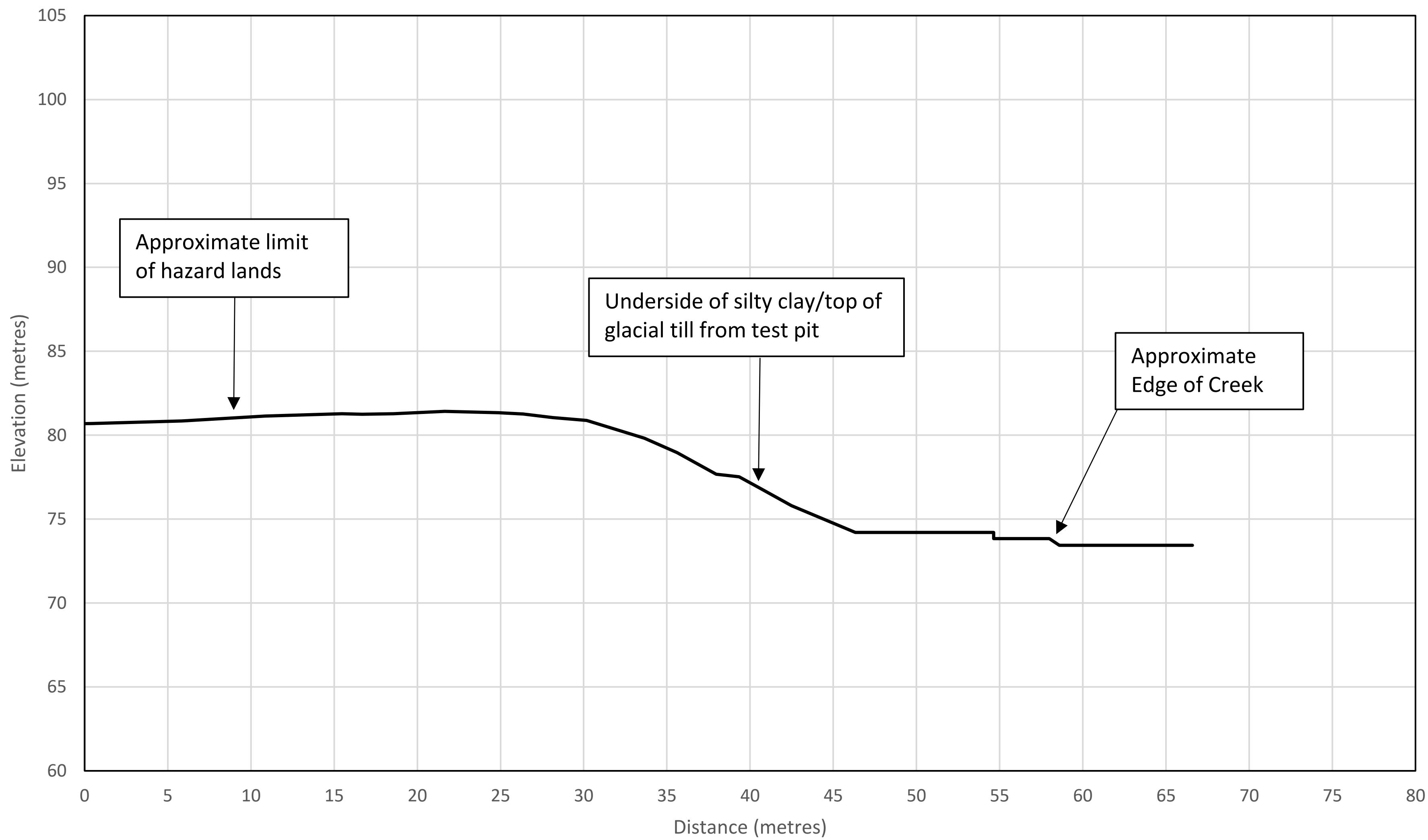
930 Smith Road, Navan

Project No.	100812.001
Drawn:	WAM
Date:	06/06/2024

Figure F1



GEMTEC Consulting Engineers and Scientists	**Slope Cross Section EE** **Proposed Lot Severances** **930 Smith Road, Navan**	Project No. 100812.001 Drawn: WAM Date: 06/06/2024	**Figure F2**



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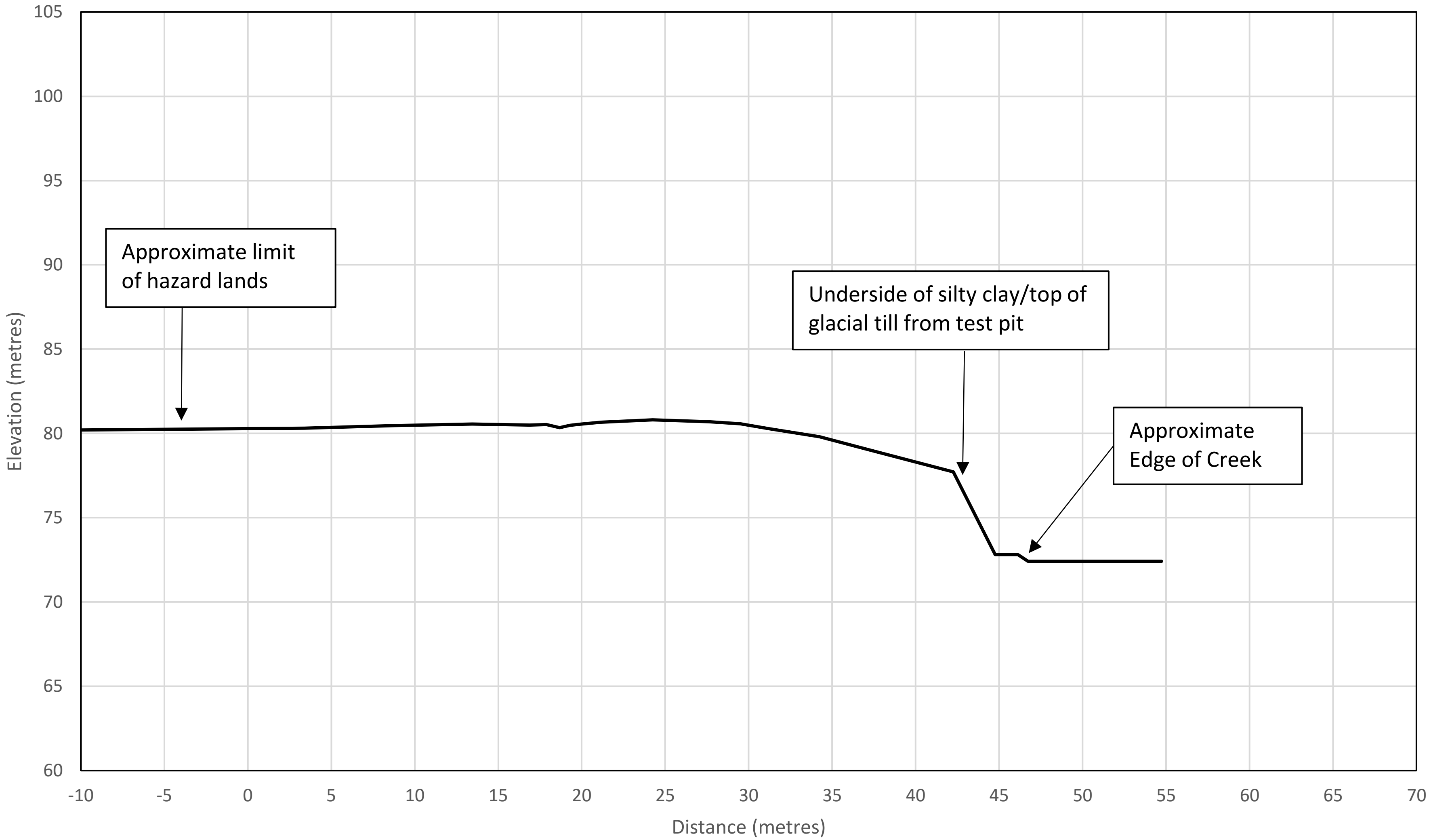
Slope Cross Section FF

Proposed Lot Severances

930 Smith Road, Navan

Project No.	100812.001
Drawn:	WAM
Date:	06/06/2024

Figure F3



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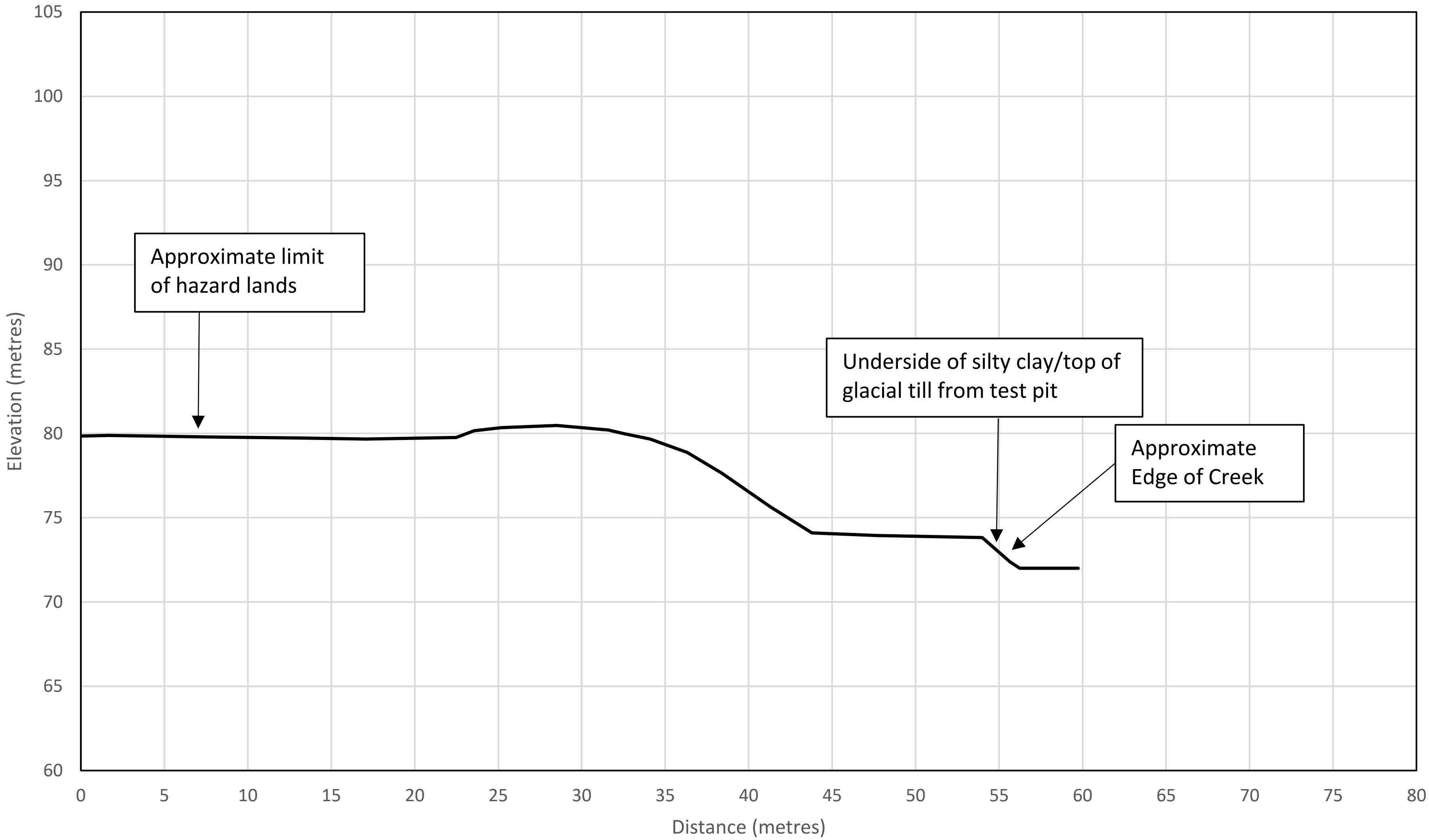
Slope Cross Section GG

Proposed Lot Severances

930 Smith Road, Navan

Project No.	100812.001
Drawn:	WAM
Date:	06/06/2024

Figure F4



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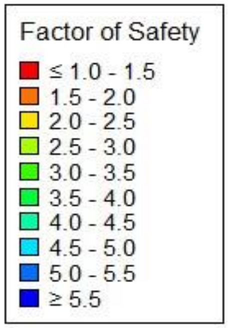
Slope Cross Section HH

Proposed Lot Severances

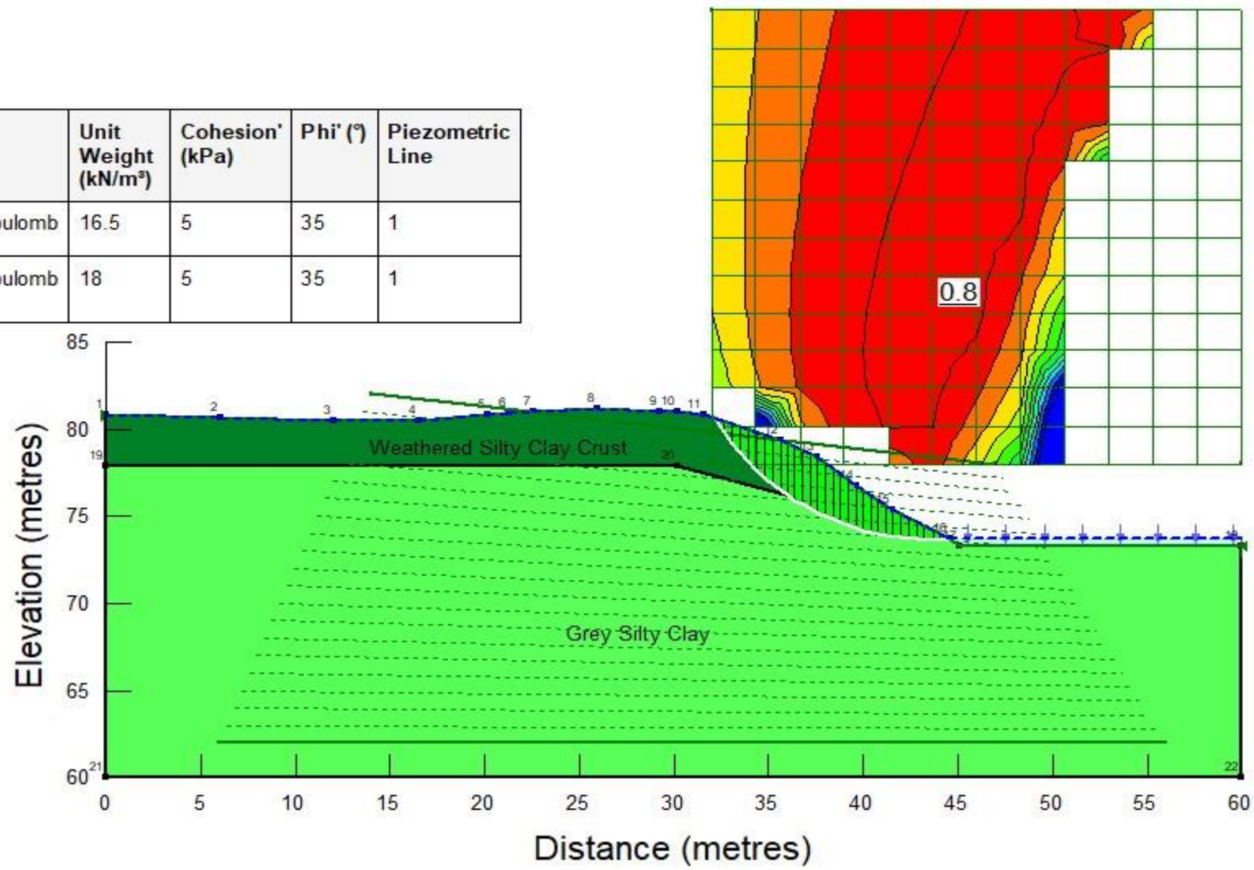
930 Smith Road, Navan

Project No.	100812.001
Drawn:	WAM
Date:	06/06/2024

Figure F5



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Grey Silty Clay	Mohr-Coulomb	16.5	5	35	1
	Weathered Silty Clay Crust	Mohr-Coulomb	18	5	35	1



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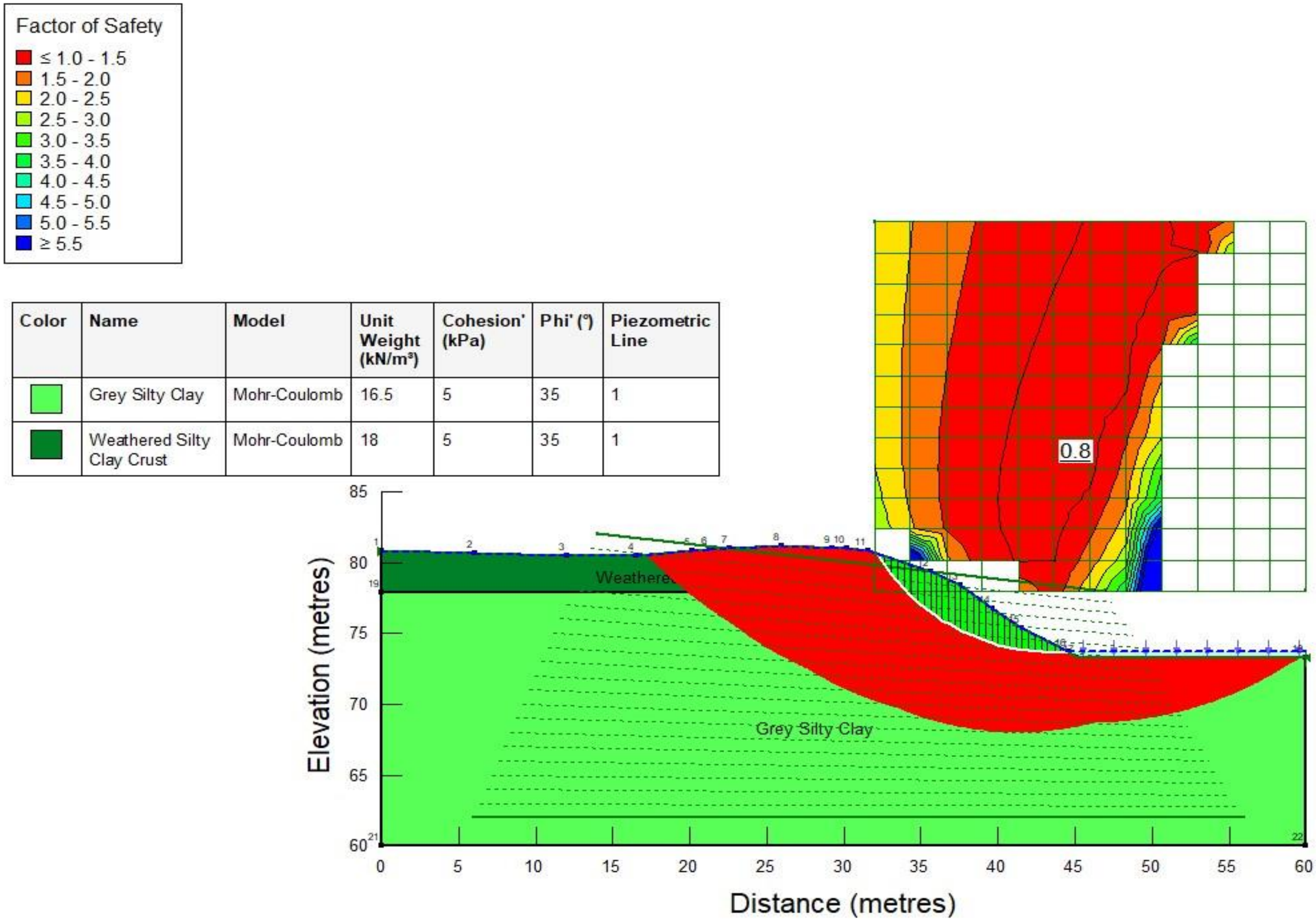
Static Slope Stability - Cross Section DD

Proposed Lot Severances

930 Smith Road, Navan

Project No.	100812.001
Drawn:	WAM
Date:	06/06/2024

Figure F6



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Grey Silty Clay	Mohr-Coulomb	16.5	5	35	1
	Weathered Silty Clay Crust	Mohr-Coulomb	18	5	35	1

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Static Slope Stability - Cross Section DD

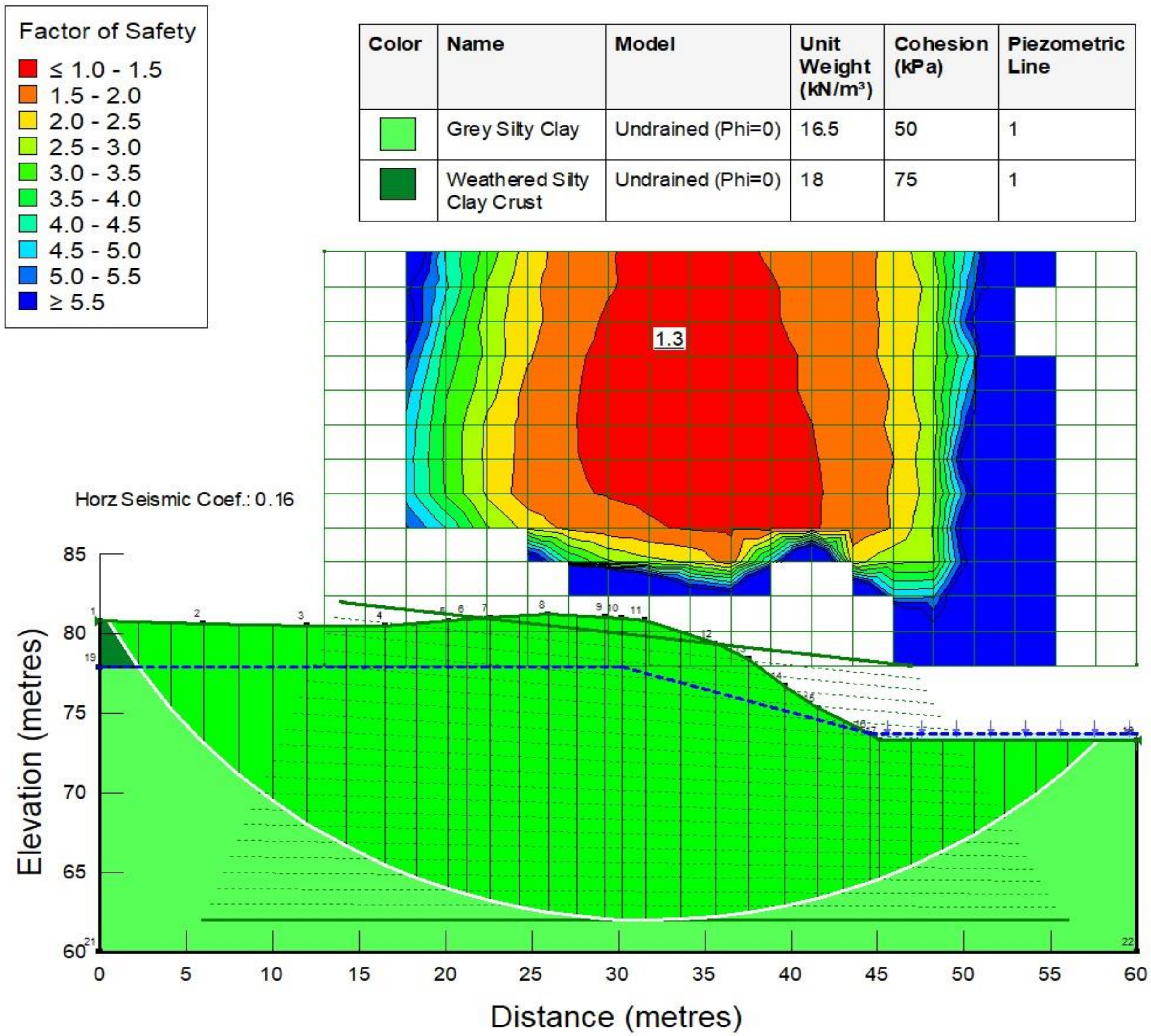
Proposed Lot Severances

930 Smith Road, Navan

Project No.	100812.001
Drawn:	WAM
Date:	06/06/2024

Figure F7

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Piezometric Line
	Grey Silty Clay	Undrained (Phi=0)	16.5	50	1
	Weathered Silty Clay Crust	Undrained (Phi=0)	18	75	1



GEMTEC
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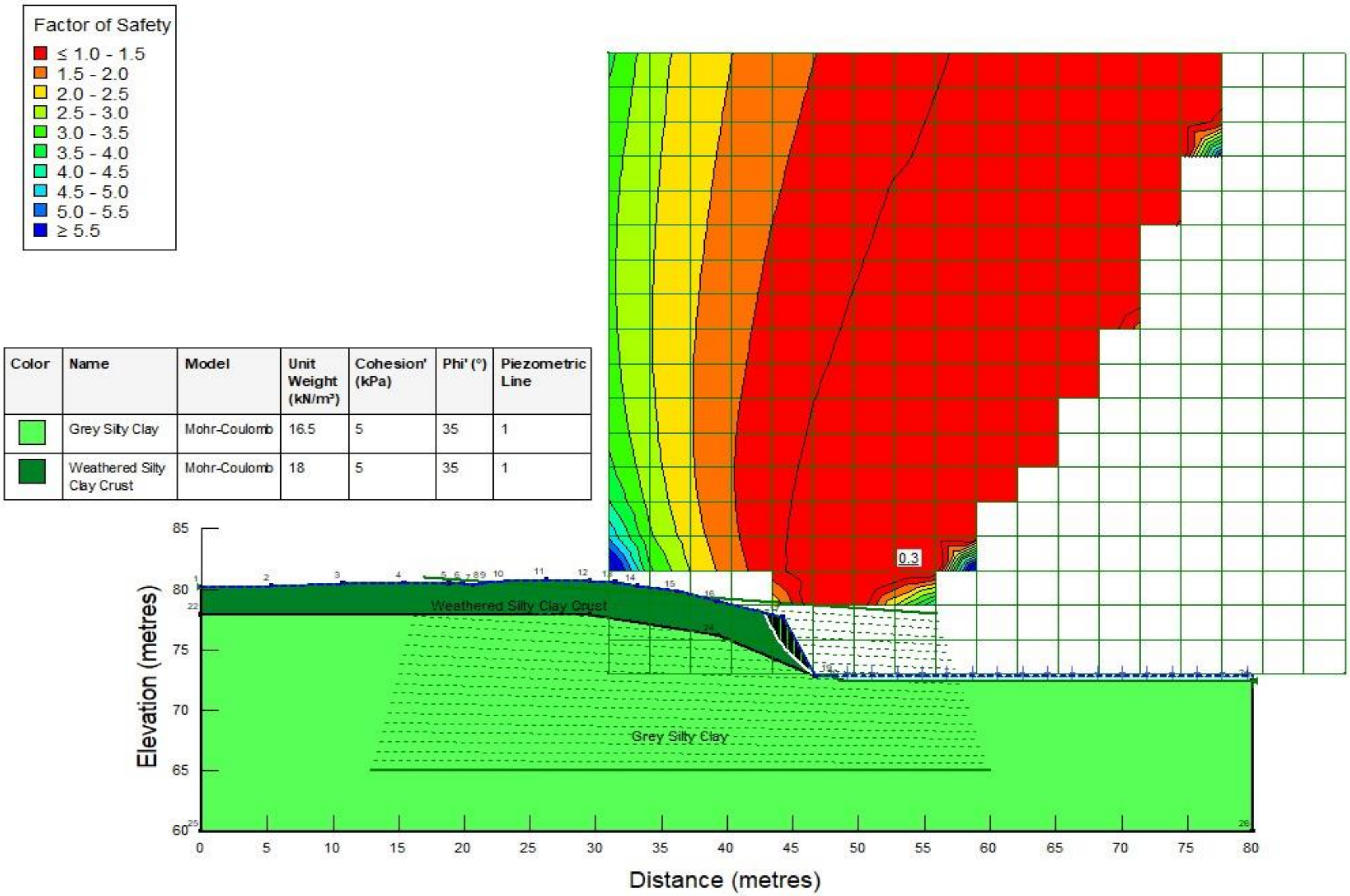
Seismic Slope Stability - Cross Section DD

Proposed Lot Severances

930 Smith Road, Navan

Project No.	100812.001
Drawn:	WAM
Date:	06/06/2024

Figure F8



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Grey Silty Clay	Mohr-Coulomb	16.5	5	35	1
	Weathered Silty Clay Crust	Mohr-Coulomb	18	5	35	1

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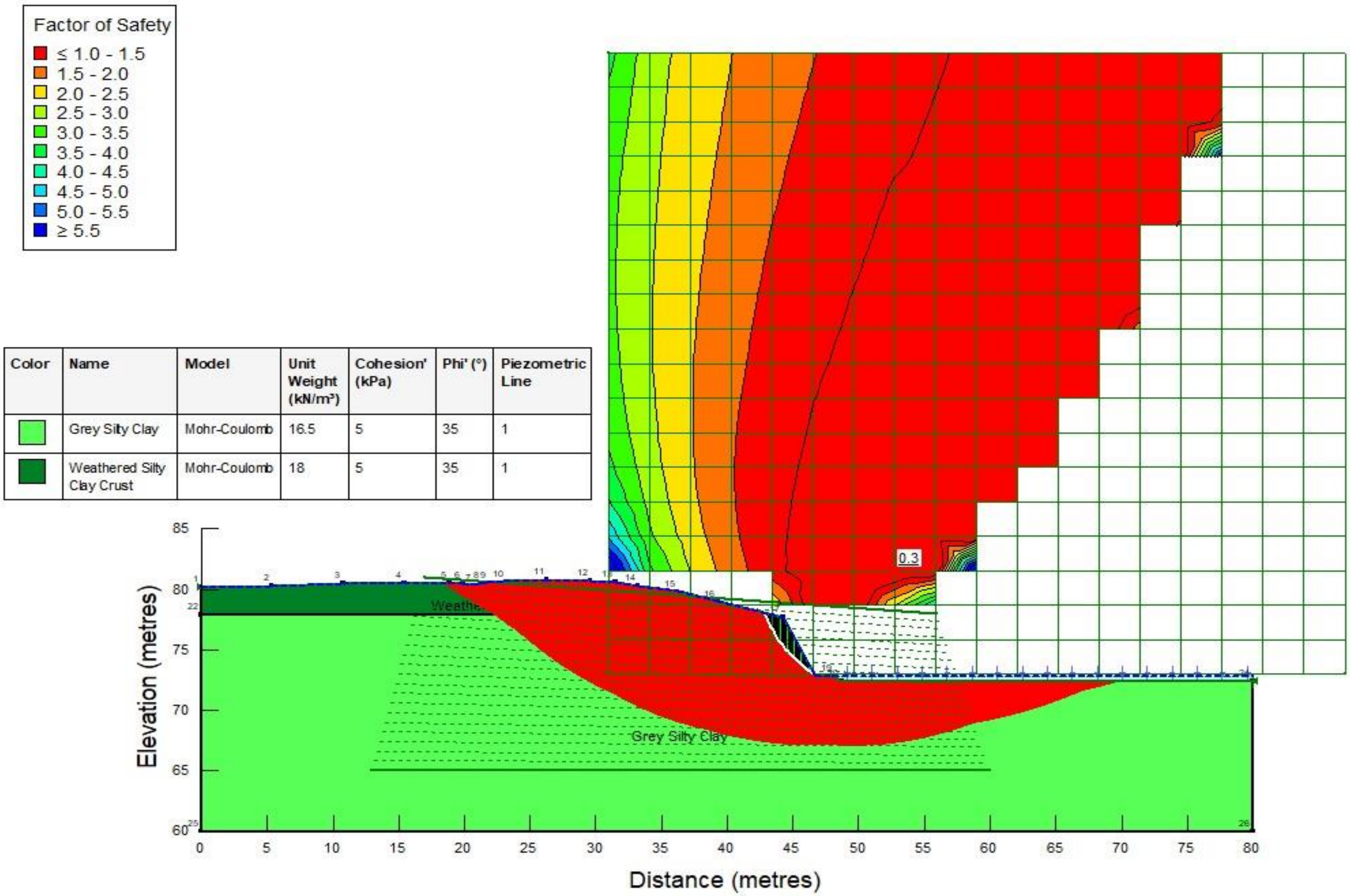
Static Slope Stability - Cross Section GG

Proposed Lot Severances

930 Smith Road, Navan

Project No.	100812.001
Drawn:	WAM
Date:	06/06/2024

Figure F9



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Grey Silty Clay	Mohr-Coulomb	16.5	5	35	1
	Weathered Silty Clay Crust	Mohr-Coulomb	18	5	35	1

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Static Slope Stability - Cross Section GG

Proposed Lot Severances

930 Smith Road, Navan

Project No.	100812.001
Drawn:	WAM
Date:	06/06/2024

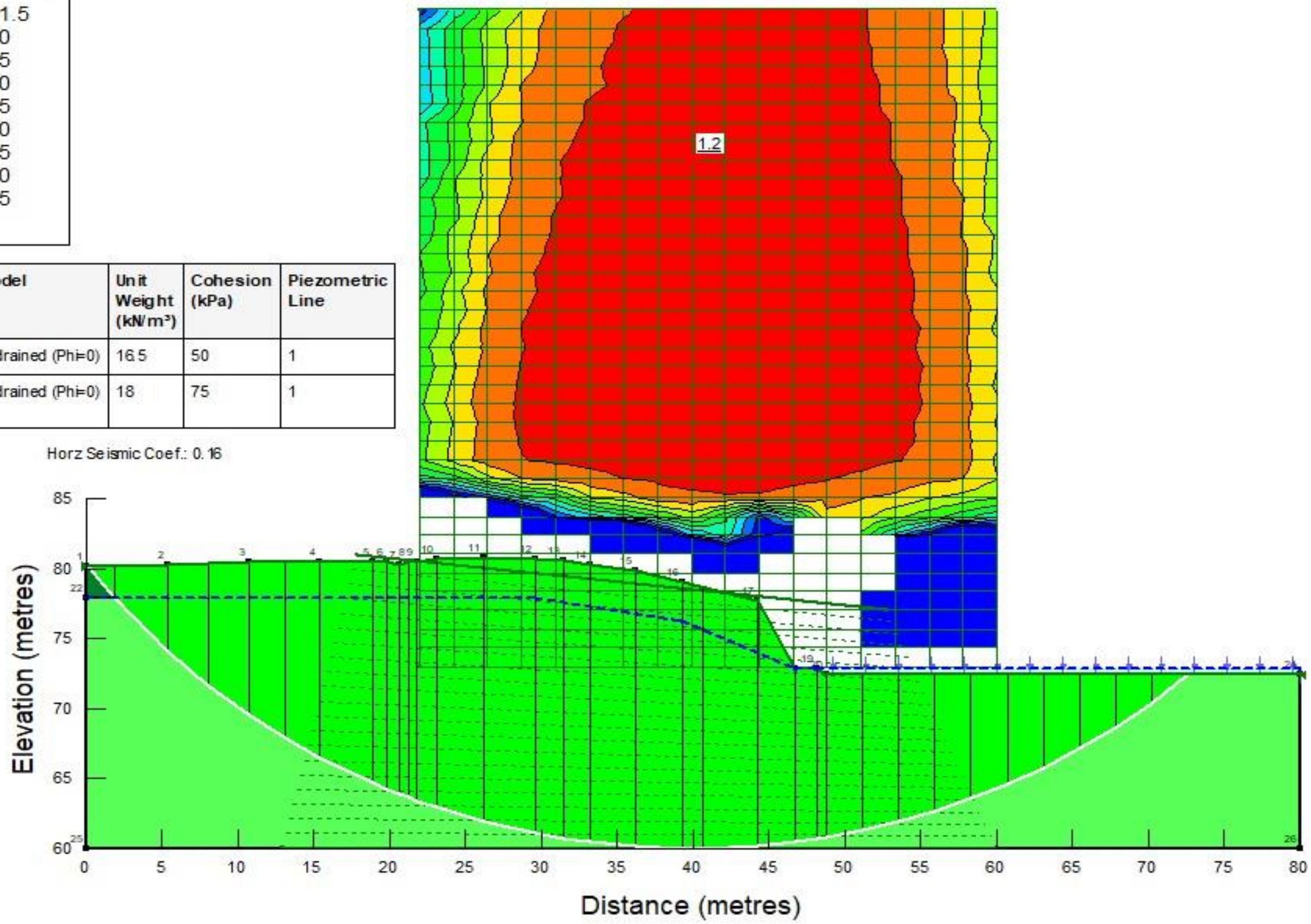
Figure F10

Factor of Safety

- ≤ 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 2.5
- 2.5 - 3.0
- 3.0 - 3.5
- 3.5 - 4.0
- 4.0 - 4.5
- 4.5 - 5.0
- 5.0 - 5.5
- ≥ 5.5

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Piezometric Line
	Grey Silty Clay	Undrained (Phi=0)	16.5	50	1
	Weathered Silty Clay Crust	Undrained (Phi=0)	18	75	1

Horz Seismic Coef.: 0.16



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Seismic Slope Stability - Cross Section GG

Proposed Lot Severances

930 Smith Road, Navan

Project No.	100812.001
Drawn:	WAM
Date:	06/06/2024

Figure F11

experience • knowledge • integrity

civil	civil
geotechnical	géotechnique
environmental	environnementale
field services	surveillance de chantier
materials testing	service de laboratoire des matériaux

expérience • connaissance • intégrité