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Geotechnical Investigation
Proposed Industrial Development
1319 Johnston Road

Ottawa, Ontario

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Submitted to:

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Proposed Industrial Development
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Ottawa, Ontario

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the proposed Industrial development to be located at 1319 Johnston Road in the City of 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.

This report is subject to the Conditions and Limitations of This Report, which follows the text of the report, and which are considered an integral part of the report.

2.0 BACKGROUND

2.1 Project Description

Plans are being prepared for a proposed industrial development to be located at 1319 Johnston Road in Ottawa, Ontario. The following is known about the project and site:

- The site is located northeast of Bank Street and Johnston Road, and south of the existing rail line;
- The site is currently a commercial property with some trees throughout and gravel access roads;
- The site is irregular in shape with a total area of about 11 acres and measures about 130 by 190 metres in plan;
- The existing Sawmill Creek runs approximately north-south along the west border of the site, and the existing ditch runs approximately east-west along the north border of the site.
- The proposed development consists of eight building with footprints ranging from about 1,200 to 2,500 square metres. It is assumed that the building will be one storey in height and of slab on grade construction (i.e., no basement level); and,
- A proposed underground stormwater storage tank is to be located at the center of the site.

As part of the geotechnical investigation;

- An assessment of the geotechnical setback requirement for the existing Sawmill Creek and ditch are to be carried out. A site reconnaissance visit was specifically carried out for this purpose; and,
- A global stability assessment was carried out for the two retaining walls that are proposed for the site. One retaining wall is located north of Building A with a height of up to about 2.2 metres, and one is located south of Buildings F, G, and H with a height of up to about 0.4 to 1.5 metres.

2.2 Site Geology

A review of surficial geology maps of the Ottawa area and reported well records by the Ministry of the Environment, Conservation and Parks of Ontario (MECP) indicates that the site is underlain by silty clay over glacial till. Bedrock geology maps indicate that the bedrock is comprised of shale of the Billings Formation at depths ranging from about 5 to 15 metres below ground surface.

3.0 METHODOLOGY

3.1 Geotechnical Investigation

The fieldwork for geotechnical investigation was carried out on June 12 and June 13, 2023. During that time, eight boreholes, numbered 23-01 to 23-08, inclusive, were advanced at the locations shown on the Site Plan, Figure 1. Details of the boreholes are provided in Table 3.1.

Table 3.1 – Borehole Details

Borehole ID	Approximate Ground Surface Elevation (metres)	Approximate Borehole Depth (metres)	Monitoring Well
23-01	82.8	5.1	✓
23-02	81.9	8.4	–
23-03	82.1	5.6	✓
23-04	83.2	4.5	–
23-05	82.0	6.1	–
23-06	81.9	3.0	–
23-07	82.5	5.7	✓
23-08	82.5	7.1	–

The boreholes were advanced using a truck mounted hollow stem auger drill rig supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario.

One test pit (numbered 24-03) was advanced at the site on May 2, 2024, using a rubber tire backhoe supplied and operated by Glenn Wright Excavating of Ottawa, Ontario. The test pit was advanced to a depth of about 2.1 metres below the existing ground surface adjacent to borehole 23-03 for additional sampling of the silty clay deposit for geotechnical classification testing associated with tree planting guidelines.

The fieldwork was supervised by a member of our engineering staff who directed the drilling operations, logged the boreholes and samples, observed the conditions in the test pit, and carried out the in-situ testing. Standard penetration tests were carried out in the boreholes and samples of the soils encountered were recovered using a 50-millimetre diameter split barrel sampler.

Three monitoring wells were installed in boreholes 23-01, 23-03, and 23-07 to measure the groundwater levels.

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, grain size distribution, and plasticity index, where applicable.

Two recovered soil samples were sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

The borehole locations were selected by GEMTEC personnel and positioned at the site relative to existing site features. The locations and ground surface elevations of the boreholes were surveyed using our Trimble R10 GPS survey instrument. The elevation is referenced to geodetic datum NAD83 (CSRS) Epoch 2010, vertical network CGVD28.

3.2 Site Reconnaissance at Sawmill Creek and Existing Ditch

A site reconnaissance was carried out on April 17, 2024, by a member of the GEMTEC engineering staff.

At the time of the site visit, the geometry of the existing Sawmill Creek and ditch were measured at a total of 11 locations using hand surveying equipment. The cross sections were positioned at the site by GEMTEC personnel. The locations of the cross sections considered are provided on Figure 1. Cross sections of the banks are provided in Appendix D.

The geometries of the cross sections considered are summarized below in Table 3.2, below.

Table 3.2 – Summary of Cross Section Height and Inclination

Watercourse	Cross Section	Bank Height (metres)	Overall inclination from horizontal (degrees)
Sawmill Creek	A-A	0.9	Near vertical
Sawmill Creek	B-B	0.9	15 to near vertical
Sawmill Creek	C-C	0.7	Near vertical
Sawmill Creek	D-D	0.9	Near vertical
Sawmill Creek	E-E	0.8	25
Sawmill Creek	F-F	0.9	20

Watercourse	Cross Section	Bank Height (metres)	Overall inclination from horizontal (degrees)
Sawmill Creek	G-G	0.8	Near vertical
Ditch	H-H	0.9	25
Ditch	I-I	1.2	35
Ditch	J-J	1.3	25 to 50
Ditch	K-K	0.9	35

It should be noted that cross sections C-C and D-D were measured at the location of existing gabion baskets along the creek.

In general, the banks along the east side of Sawmill Creek are vegetated with grass, shrubs, and also contain concrete debris and gabion baskets, with large trees located at the north end of the creek. Minor to moderate erosion was observed along the creek. No signs of overall instability (i.e., rotational failures) were observed at the creek.

The banks along the south side of the existing ditch are vegetated with grass, shrubs, small to large trees, and granular fill. Minor to no erosion was observed along the ditch. No signs of overall instability were observed at the ditch.

4.0 SUBSURFACE CONDITIONS

4.1 General

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 classification testing are provided on the Record of Borehole Sheets and in Appendix B. The results of the chemical analysis are provided in Appendix C.

The following sections provide a description of the subsurface conditions encountered in the boreholes advanced as part of this investigation.

It should be noted that test pit 24-03 was advanced solely for obtaining a sample of the silty clay for shrinkage limit testing, and as such, no test pit log is presented. The subsurface conditions encountered in the test pit are similar to those encountered in borehole 23-03, and therefore are not discussed in the following sections, with the exception of the results of the shrinkage limit testing.

4.2 Fill Material

A layer of fill material was encountered at the ground surface in all the boreholes. The fill material extends to depths ranging from about 0.6 to 2.3 metres below the existing ground surface.

The fill material generally consists of gravel and sand, with some silt, over silty sand, with various contents of clay and gravel. The fill material also contains organics, cobbles, and boulders.

Standard penetration tests carried out in the fill material of silty sand gave N values ranging from 6 to 41 blows per 0.3 metres of penetration, which indicates a loose to dense relative density.

A thin layer of topsoil was encountered within the fill material in borehole 23-03. The layer of topsoil was encountered at a depth of about 0.8 metres below ground surface and has a thickness of about 0.1 metres.

Grain size distribution tests were carried out on two sample of the fill material from boreholes 23-04 and 23-07. The results are provided in Appendix B and are summarized in Table 4.1.

Table 4.1 – Summary of Grain Size Distribution Tests (Fill Material)

Borehole ID	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt and Clay (%)
23-04	1	0 – 0.4	47	40	13
23-07	3	0.8 – 1.4	10	66	24

The measured water contents of five samples of the fill material ranged from about 2 to 15 percent.

4.3 Silty Clay

Native deposits of silty clay and clay and silt, herein referred to silty clay, were encountered below the fill material in boreholes 23-01, 23-02, 23-03, 23-07, and 23-08. The silty clay extends to depths ranging from about 1.5 to 2.3 metres below the existing ground surface.

The full depth of the silty clay encountered has been weathered to a grey brown crust. Standard penetration tests carried out in the weathered silty clay gave N values ranging from 4 to 7 blows per 0.3 metres of penetration, which probably indicates a stiff to very stiff consistency.

Grain size distribution tests were carried out on two samples of the silty clay. The results are provided in Appendix B and are summarized in Table 4.2.

Table 4.2 – Summary of Grain Size Distribution Tests (Silty Clay)

Borehole ID	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
23-03	2	1.0 – 1.4	0	5	25	70
23-07	4	1.5 – 2.1	3	14	36	47

Atterberg limits test were carried out on four samples of the silty clay. The results are provided in Appendix B and are summarized in Table 4.3.

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

Borehole ID	Sample Number	Sample Depth (metres)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Water Content (%)
23-01	3	1.5 – 2.1	37	20	17	30
23-02	3	0.8 – 1.4	34	18	16	27
23-03	3	1.5 – 2.0	31	17	14	31
23-08	4	1.5 – 2.0	35	17	18	23

The measured water contents of six samples of the silty clay ranged from about 23 to 41 percent.

One shrinkage limit test was carried out one sample of the silty clay, in general accordance with ASTM D4943 (which was discontinued in 2017 by the ASTM Sponsoring Committee responsible for the standard). The modified plasticity index (PI_m) was also calculated for the silty clay samples using the following formula and the results of the Atterberg limits and grain size distribution testing described previously:

$$PI_m = PI \text{ x } (\% \text{ passing the 425 micrometre sieve } / \ 100)$$

The test and calculation results are provided in Appendix B and are summarized in Table 4.4.

Table 4.4 – Summary of Modified Plasticity Index (Silty Clay)

Test Hole ID / Sample No.	Shrinkage Limit (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Modified Plasticity Index (%)
23-01 / 3	-	37	20	17	16

Test Hole ID / Sample No.	Shrinkage Limit (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Modified Plasticity Index (%)
23-02 / 3	-	34	18	16	16
23-03 / 3	-	31	17	14	13
24-03 / 1	15	-	-	-	-
23-08 / 3	-	35	17	18	18

4.4 Glacial Till

Native deposits of glacial till were encountered below the silty clay and/or fill material, where encountered, in all the boreholes.

The glacial till is a heterogeneous mixture of all grain sizes, which at this site can be described as silty clayey sand, with some gravel, over gravelly silty sand to sand. The glacial till also contains cobbles and boulders. Standard penetration tests carried out in the glacial till gave N values ranging from 2 to greater than 50 blows per 0.3 metres of penetration, which indicates a very loose to very dense relative density. These low values may also indicate the presence of zones of glacial till with increased fine-grained (sand, silt, and clay) content which may be more sensitive to disturbance from the upward flow of groundwater into the augers.

Grain size distribution tests were carried out on four samples of the glacial till. The results are provided in Appendix B and are summarized in Table 4.5.

Table 4.5 – Summary of Grain Size Distribution Tests (Glacial Till)

Borehole ID	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
23-03	4	2.3 – 2.9	12	46	21	21
23-03	7	4.6 – 5.2	11	77	12 (combined)	
23-07	6	3.1 – 3.7	10	52	18	20
23-07	7	3.8 – 4.4	31	44	25 (combined)	

The measured water content of selected samples of the glacial till material ranged from about 7 to 20 percent.

4.5 Auger Refusal and Bedrock

Practical auger refusal was encountered in boreholes 23-01, 23-03, 23-04, 23-06, and 23-07, at depths ranging from about 3.0 to 5.7 metres below the existing ground surface. Practical auger refusal can occur on cobbles and boulders and may not necessarily be representative of the upper surface of the bedrock.

Possible highly weathered shale bedrock was encountered in boreholes 23-2, 23-05, and 23-08, at depths ranging from about 4.8 to 8.0 metres below the existing ground surface. A summary of the refusal and bedrock depths and elevations are provided in Table 4.6.

Table 4.6 – Summary of Auger Refusal and Bedrock Depths

Borehole ID	Depth to Refusal (metres)	Depth to Bedrock (metres)	Elevation of Refusal / Bedrock (metres)
23-01	5.1	–	77.7
23-02	–	8.0	73.9
23-03	5.6	–	76.5
23-04	4.5	–	78.8
23-05	–	4.8	77.2
23-06	3.0	–	78.9
23-07	5.7	–	76.8
23-08	–	6.9	75.6

The measured water content of one sample of the shale bedrock was about 11 percent.

4.6 Groundwater Level

The groundwater levels were observed within the open boreholes at depths ranging from about 2.1 to 4.0 metres below the existing ground surface upon completion of augering. Borehole 23-06 was observed dry upon completion of augering.

Groundwater seepage was observed in the test pit at a depth of about 0.9 to 1.1 metres below the existing ground surface upon completion of excavating.

The groundwater levels were measured in the monitoring wells on June 28, 2023, and May 2, 2024, and are summarized in Table 4.7.

Table 4.7 – Summary of Groundwater Levels

Borehole ID	Groundwater Depth (metres)	Groundwater Elevation (metres)	Date of Reading
23-01	2.3	80.5	June 28, 2023
	2.4	80.4	May 2, 2024
23-03	2.1	80.0	June 28, 2023
	2.1	80.0	May 2, 2024
23-07	2.9	79.6	June 28, 2023
	3.0	79.5	May 2, 2024

The groundwater level may be higher during wet periods of the year such as the early spring or following periods of precipitation.

4.7 Chemistry Relating to Corrosion

Two sample of the soil obtained from boreholes 23-03 and 23-06 were sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel. The results of chemical testing are provided in Appendix C and summarized in Table 4.8.

Table 4.8 – Summary of Corrosion Testing

Parameter	Borehole 23-03 Sample 3	Borehole 23-06 Sample 3
Conductivity (µS/cm)	341	451
Resistivity (Ohm.m)	29.3	22.2
pH	7.63	7.68
Chloride Content (µg/g)	<10	<10
Sulphate Content (µg/g)	234	465

5.0 RECOMMENDATIONS AND GUIDELINES

5.1 General

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 Grade Raise Restrictions

The subsurface conditions at the site consist of fill material over native deposits of silty clay over glacial till. Based on the results of the subsurface investigation, the maximum thickness of any grade raise filling should be limited to 2.0 metres above the existing surface grade.

5.3 Excavation

5.3.1 General

The excavations for the proposed industrial development will likely be carried out through the fill material, silty clay, and possibly into the upper portion of the glacial till (i.e., the glacial till of silty clayey sand).

The sides of the excavations within the overburden soils 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 soils at this site above the groundwater level can be classified as Type 3 soils, and, as such an allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation slopes. The very loose to loose soils would be classified as Type 4 soils, and, as such an allowance should be made for 3 horizontal to 1 vertical, or flatter, excavation slopes.

Excavation of the fill material, silty clay, and the glacial till should not present significant constraints. Cobbles and boulders should be anticipated in the fill material and glacial till which may lead to increased excavation effort and slower progress. As such, allowance should be made for removal of boulders during excavation. Additional engineered fill material may be required to fill any voids left from the removal of boulders in glacial till.

5.3.2 Base of Excavation and Subgrade Protection

As excavation is carried out, there is potential for zones of glacial till to be encountered which are very sensitive to disturbance from construction traffic, ponded water etc., and which may soften/ loosen rapidly. Further, potential exists for some upward groundwater flow to occur at the base of the excavation also leading to softening of the glacial till. As such, some unavoidable disturbance and softening to the subgrade surface is likely to occur. To reduce the effects of these occurrences, GEMTEC recommends the following:

- Where possible, construction works should be staged to allow for protection of the subgrade to be completed within a working day. Excavations to final levels may be carried out in staged areas where practical;
- Construction traffic over the unprotected subgrade surface should be avoided wherever practical;

- Geotechnical personnel should be available at the time of excavation to the final subgrade surface to carry out inspections as soon as practical and allow for backfilling to commence. Full time supervision of the excavation is preferable to prevent delay;
- Following approval, the exposed surface at the base of excavation could be protected with a mat of coarse angular rock fill, or a mud mat of low strength concrete, as a protective layer. Installation of a non-woven Class II geotextile separator layer may be required; and,
- Over-excavation or subexcavation should be avoided or minimised wherever practical as deepening the excavation below the groundwater level may present additional constraints.

To further reduce the risk of disturbance of soils during excavation, the groundwater level should be kept below the excavation base where possible (noting the additional comments provided in Section 6.5). Notwithstanding, some disturbance and loosening of the materials could occur, and allowance should be made for subexcavation and additional engineered fill material placement.

5.3.3 Excavations Adjacent to the Existing Storm Trunk Sewer

It is understood that buildings B, D, and G as well as the underground stormwater storage tank are to be located adjacent to the easement for the existing storm trunk sewer, and that excavations for the footings may extend into the easement.

Based on the grading plan prepared by Robinson Land Development (Robinson), and provided in Drawing No. 23034-GR1, titled “Grading Plan” (Project Number 23034) Revision No. 3, dated March 24, 2024, it is understood that;

- the proposed underside of footing elevations of buildings B, D, G ranges from about 81.6 to 82.1 metres;
- the proposed invert elevation of the underground stormwater storage tank is about 80.8 metres;
- The existing storm trunk sewer runs approximates northeast to southwest across the site, with invert elevations ranging from about 78.2 to 78.4 metres; and,
- The storm trunk sewer has a total easement width of about 9 metres.

Based on the subsurface conditions encountered during the investigation, it is anticipated that the excavations for the proposed building and stormwater storage tank will extend to elevations ranging from about 80.7 to 81.3 metres (i.e., to the native undisturbed soils and with the addition of engineered fill, if required).

We understand that temporary excavations are permitted within the easement of the storm truck sewer. Based on the above, the excavation will remain above the obvert of the trunk sewer. Therefore, the excavations are not expected to undermine the existing storm trunk sewer, and standard sloped excavations can be used for this project (i.e., shored excavations will not be required for this purpose).

Based on the underside of footing elevations of the proposed buildings, and the offset of the buildings to the storm sewer, it is anticipated that the existing storm trunk sewer will be located above the zone of influence of the footings. The zone of influence of the footings is considered to extend down and out from the edge of the footings at 1 horizontal to 1 vertical, see Figure 2.

Based on the offset from the buildings along with the footing elevations and sewer invert elevation, the sewer is not expected to be located within the zone of influence of the proposed buildings, and therefore, the addition of the new buildings will not have significant negative impact on the existing storm sewer from foundation loading, provided the above recommendations are followed.

However, consideration can be given to lowering the footings, as much as practical, to lower the zone of influence as much as possible below the existing sewer.

It is recommended that the location of the existing storm trunk sewer be confirmed, so that it's position is definitively established relative to the proposed buildings and underground storage tank on site.

5.4 Groundwater Management

Based on the results of the investigation, it is anticipated that the groundwater inflow into excavations for site services could be handled by pumping from within the excavations. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services. Suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review.

It is noted that the water level was measured at depths ranging from about 2.1 to 2.9 metres below the existing ground surface. Depending on the depths of foundations, proposed services, and groundwater levels at the time of construction, an Environmental Activity and Sector Registry (EASR) in accordance with Environmental Protection Act Part II or a Category 3 Permit To Take Water (PTTW) may be required.

5.5 Foundation Design

Based on the subsurface conditions which were encountered during the investigation, it is considered that the proposed buildings could be founded on spread footings bearing on or within the native overburden deposits, with some limitations on the glacial till. The fill material is not considered suitable for the support of the proposed structures. Therefore, any fill or deleterious material, if encountered, should be removed from the proposed building areas.

For exterior strip footings founded on or within native, undisturbed deposits of silty clay, compact or better glacial till, or on a pad of compacted granular fill above native, undisturbed soil should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 150 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 300 kilopascals.

Zones of very loose to loose glacial till may be encountered at the underside of foundation level. Where present, subexcavation and replacement of a portion of these soils is recommended from below the foundation loaded area. A minimum thickness of 600 millimetres of compacted engineered fill should be placed between the underside of the footings and any very loose to loose glacial till. Foundations supported on a pad of engineered fill in this manner may be sized using an SLS and ULS value of 75 kilopascals and 150 kilopascals, respectively. The SLS and ULS values could be increased, however, additional subexcavation and replacement may be required depending on the size of foundations proposed.

In areas where the underside of footing level is above the level of the native soil or where subexcavation of soil is required, the grade below the proposed buildings could be raised with granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II. The granular material should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the material’s standard Proctor maximum dry density value using suitably sized compaction equipment. To provide adequate spread of load below the footings, the granular material should extend at least 0.5 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

Provided that the subgrade surface and engineered fill are prepared as described in this report, the post construction total and differential settlement of the footings at SLS should be less than 25 and 15 millimetres, respectively.

For adjacent footings founded at different elevations, we recommend that the underside of the adjacent lower footing not encroach within a zone extending 0.5 metres horizontally beyond the underside of the upper footing and then down and out from this point at 1 horizontal to 1 vertical, or flatter.

Given the level of information on the proposed structures available at this time, the above recommendations should be considered to be preliminary. More detailed assessment of soil bearing capacity and potential foundation settlements can be provided once further details of the foundation design and structural configurations are known.

5.6 Frost Protection

All exterior footings, adjacent to heated areas, should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated exterior footings should be provided with at least 1.8 metres of earth cover for frost protection purposes. The required depth of frost protection can be reduced by the thickness of any engineered fill beneath the foundations. 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.

5.7 Seismic Site Class and Liquefaction Potential

Based on the results of the investigation, it is anticipated that the proposed foundations will be supported on deposit of silty clay and/or glacial till or on a pad of engineered fill constructed on the silty clay and/or glacial till.

Based on Table 4.1.8.4.A. of the 2012 Ontario Building Code, the seismic site class can be determined based on the Average Standard Penetration Resistance or the Soil Undrained Shear Strength. Based on the results of the standard penetration carried out as part of this investigation, it is recommended that seismic Site Class D be used for the design of structures in the industrial development.

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

5.8 Slab on Grade Support

The fill material is not considered suitable for support of the slab on grade. To prevent long term settlement of the floor slab, all fill material and any organic material, if encountered, should be removed from below the proposed slab to expose the native overburden deposits. The subgrade surface should then be proof rolled with suitable compaction equipment under dry conditions and any noted soft or disturbed areas should be sub-excavated, subject to inspection of the geotechnical engineer.

The grade within the proposed buildings could then be raised where necessary, with material meeting OPSS requirements for Granular A and Granular B Type I or II. The granular base for the proposed slab on grade should consist of at least 150 millimetres of OPSS Granular A.

OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the floor slab be composed of virgin material only, for environmental reasons.

All imported granular materials placed below the proposed floor slab should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the material’s standard Proctor maximum dry density value.

Underfloor drainage is not considered necessary provided that the floor slabs are above the finished exterior ground surface level. If any areas of the buildings are to remain unheated during the winter period, thermal protection of the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary.

The floor slabs should be wet cured to minimize shrinkage cracking and slab curling. The slab should be saw cut to about one third of the thickness of the slab as soon as curing of the concrete permits, in order to minimize shrinkage cracks.

Proper moisture protection with a vapour retarder should be used for floor slabs where the floor will be covered by moisture sensitive flooring materials or where moisture sensitive equipment, products or environments will exist. The “Guide for Concrete Floor and Slab Construction”, ACI 302.1R-04 should be considered for the design and construction of vapour retarders below the floor slabs.

5.9 Backfill and Drainage

The native deposits at this site are frost susceptible and should not be used as backfill against foundations. To avoid frost adhesion and possible heaving, the foundations should be backfilled with imported, free-draining, non-frost susceptible granular material such as that meeting the requirements of OPSS Granular A, or Granular B Type I or II.

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 material’s standard Proctor maximum dry density value using suitable vibratory compaction equipment. Light walk behind compaction equipment should be used next to the foundation walls to avoid excessive compaction induced stress on the foundation walls.

Where future landscaped areas will exist next to the proposed structures and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the material’s standard Proctor maximum dry density value. Where areas of hard surfacing (concrete, sidewalks, pavement, etc.) abut the proposed structures, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible fill material to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from 1.5 metres below finished grade to the underside of the granular subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter. Further, we recommend that downspouts outlet in such a way as to prevent saturation of soils below hard surfaced areas.

The frost susceptible native soils could be considered for foundation wall backfill purposes in soft landscaped areas provided that a suitable bond break is applied to the surface of the foundations to prevent frost jacking. A suitable bond break could consist of at least 2 layers of 6 MIL polyethylene sheeting or a proprietary plastic drainage system. It is also pointed out that the native soils at this site can be impacted by changes in moisture content and this could affect the ability to compact this material to the required density.

Perimeter foundation drainage is not considered necessary for a slab on grade structure provided that the floor slab level is above the finished exterior ground surface level.

5.10 Site Services

Details of the proposed services to be installed as part of the works were not available to GEMTEC. As such relatively generic guidelines are provided only.

5.10.1 Excavation

The overburden excavations for the site services will be carried out through fill material, silty clay and into the deposits of glacial till, and likely below the groundwater level. Refer to Section 5.3 for further comments.

In areas where space constraints dictate, the sides of the service trenches could be supported by a tightly fitting, braced steel trench box, which is specifically designed for this purpose. To advance the trench box, even boulders that partially intrude into the sides of the excavation must be removed, which may result in a wider and deeper excavation than anticipated.

It is recommended that no excavated material be stockpiled within 5 metres of the edge of the excavation.

Groundwater seepage into excavations is expected and should be controlled, as necessary, by pumping from within the excavations. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services.

5.10.2 Pipe Bedding and Cover

The bedding for service pipes should consist of at least 150 millimetres of crushed stone meeting OPSS requirements for Granular A. Cover material, from spring line to at least 300 millimetres above the top of the pipes, should consist of granular material, such as that meeting OPSS Granular A.

In areas where the subsoil is disturbed or where unsuitable material (such as fill material) exists below the pipe subgrade level, the disturbed/unsuitable material should be removed and replaced with a subbedding layer of compacted granular material, such as that meeting OPSS Granular B Type I or II. To provide adequate support for the pipes in the long term in areas where subexcavation of material is required below design subgrade level, the excavations should be sized to allow for a 1 horizontal to 1 vertical spread of granular material down and out from the bottom of the pipes.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A. The granular bedding and subbedding materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the material's standard Proctor maximum dry density value using suitably sized compaction equipment.

The use of clear crushed stone as a bedding, subbedding, or cover material should not be permitted on this project.

5.10.3 Trench Backfill

The backfill materials within the zone of seasonal frost penetration (i.e., 1.8 metres below finished grade) should match the materials exposed on the trench walls. This will reduce the potential for differential frost heaving between the area over the trench and the adjacent roadway. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I or II or imported OPSS Select Subgrade Material.

To minimize future settlement of the backfill and achieve an acceptable subgrade for any roadways, curbs, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the material's standard Proctor maximum dry density value, using suitable compaction equipment.

The specified density for compaction of the backfill materials may be reduced where the trench backfill is not located below or in close proximity to existing or future areas of hard surfacing and/or structures.

5.11 Internal Roadway Construction

5.11.1 Subgrade Preparation

In preparation for construction of the internal roadways in the project area, any soft, wet, deleterious material should be removed from the subgrade surface. If needed, the grade below the roadway could then be raised with compacted granular material such as that meeting OPSS specifications for Select Subgrade Material, Granular B Type I, II, or III and/or reuse of existing fill material which meets OPSS Granular B Type I. Prior to placing granular material, the subgrade surface should be proof rolled with a large steel drum roller (i.e., 10 tonne or larger) under dry conditions.

In areas where abrupt changes in the frost susceptibility of the subgrade materials are encountered, frost tapers and/or some subexcavation of materials may be required to prevent future localized differential frost heaving of the pavement structure. The frost taper and subexcavation requirements should be assessed at the time of construction by geotechnical personnel.

The roadway subgrade surface should be made smooth and crowned or sloped prior to placing the granular materials to promote drainage of the roadway base and subbase materials.

Grade raise fill material placed below the roadway should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the material's standard Proctor maximum dry density value using suitable vibratory compaction equipment.

5.11.2 Pavement Design

The following minimum pavement structure is suggested for internal roadways and parking lots at this site, assuming that these pavements will be used as light vehicle (i.e., primarily passenger vehicle) traffic:

- 50 millimetres of Superpave 12.5 Traffic Level B; over
- 150 millimetres of base (OPSS Granular A); over
- 300 millimetres of subbase (OPSS Granular B Type II).

In the absence of detailed traffic data, the thickness of asphaltic concrete and OPSS Granular B Type II subbase should be increased for heavy truck traffic, fire access, and garbage collection, etc., as follows:

- 100 millimetres of asphaltic concrete, comprising
    - 40 millimetres of Superpave 12.5 Traffic Level D; over
    - 60 millimetres of Superpave 19.0 Traffic Level D; over
- 150 millimetres of base (OPSS Granular A); over
- 450 millimetres of subbase (OPSS Granular B Type II).

5.11.3 Effects of Subgrade Disturbance

If the roadway subgrade surface becomes disturbed or wetted due to construction operations or precipitation, or the granular pavement materials are to be used by construction traffic (i.e., if the granular pavement materials are placed during installation of the sewers, watermains, and laterals), the Granular B Type II thicknesses provided above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase. The contractor should be responsible for providing suitable access for construction equipment.

The required thickness of the subbase materials will depend on a number of factors, including contractor workmanship and schedule, contractor methodology, soil types and weather conditions, and should be assessed by geotechnical personnel at the time of construction. In our opinion, the preferred approach from a geotechnical point of view is to:

- Proof roll the subgrade conditions at the time of construction under the supervision of experienced geotechnical personnel; and,
- Adjust the thickness of the subbase material and include a woven geotextile separator, as required. Unit rate allowances should be made in the contract for subexcavation and replacement with OPSS Granular B Type II.

5.11.4 Granular Material Placement

All imported granular materials should be placed in maximum 200 millimetre hick lifts and should be compacted to at least 99 percent of the material's standard Proctor maximum dry density value using suitable vibratory compaction equipment.

5.11.5 Asphaltic Cement

Performance graded PG 58-34 asphaltic cement is recommended for local roadways while performance graded PG 64-34 asphalt is recommended heavy truck traffic, fire access, and garbage collection, etc.

5.11.6 Pavement Transitions

The new asphalt will abut the existing asphalt at Johnston Road. The following transition treatment is suggested to improve the performance of the joint between the new and the existing pavements:

- Neatly saw cut the existing asphaltic concrete;
- Remove the asphaltic concrete and slope the bottom of the excavation within the existing granular base and subbase at 1 horizontal to 1 vertical, or flatter, to avoid undermining of the existing asphaltic concrete;
- To avoid cracking of the asphaltic concrete due to an abrupt change in the thickness of the roadway granular materials where new pavement areas join with the existing pavements, the granular depths should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the existing pavement structure; and,
- Remove (mill off) 50 millimetres of the existing asphaltic concrete to a distance of 300 millimetres at the joint and tack coat the asphaltic concrete at the joint in accordance with the requirements in OPSS 310.

5.11.7 Pavement Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long-term performance of the pavement at this site. Where feasible, catch basins should be provided with minimum 3-metre-long perforated stub drains which extend in at least two directions from each catch basin at the pavement subgrade level. In addition, where catch basins are not feasible, we recommend that swales or ditches be implemented to promote drainage around the road surface areas. The granular base and subbase materials should be crowned and extend horizontally to the ditches or swales. Where possible, the bottom of the swales/ditches should be at least about 0.3 metres below the bottom of the Granular B Type II.

5.12 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the sample of the weathered silty clay recovered from borehole 23-03 was 234 micrograms per gram. The measured sulphate concentration in the sample of the glacial till recovered from borehole 23-06 was 465 micrograms per gram. According to the Canadian Standards Association “Concrete Materials and Methods of Concrete Construction” (CSA A23.1-14 Table 3), the degree of sulphate exposure stemming from the soils is negligible (less than 0.10 percent). Therefore, any concrete in contact with the soil at this site could be batched with General Use (GU) cement. However, the effects of freeze thaw in the presence of de-icing chemicals (sodium chloride) use on the roadway should be considered in selecting the air entrainment and the concrete mix proportions for any concrete.

Based on the resistivity and pH of the tested soil samples, the weathered silty clay and glacial till can be classified as nonaggressive towards unprotected steel. The manufacturer of any buried steel elements that will be in contact with the soil and groundwater should be consulted to ensure that the durability of the intended product is appropriate. It is noted that the corrosivity of the soil and groundwater could vary throughout the year due to the application of sodium chloride for de-icing.

5.13 Sensitive Marine Clay – Effects of Trees

Portions of the site are 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.

The modified plasticity index of the samples of the weathered silty clay provided in Table 4.4 ranges from about 13 to 18 percent. As such, the potential for soil volume change, as defined by the City of Ottawa, is low/medium.

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”) – as is likely the case at this site.

Refer to the City of Ottawa 2017 Tree Planting Guidelines for further information and recommendations on planting trees near foundations for soils with low/medium potential for soil volume change provided in the City of Ottawa Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines.

5.14 Setback Requirements for Sawmill Creek and Ditch

Based on the Slope Stability Guidelines for Development Applications in the City of Ottawa" (dated 2012), any ground which is inclined steeper than about 11 degrees from horizontal and greater than 2 metres in height has the potential for instability. Since the overall height of the banks of the existing Sawmill Creek and the ditch are less than 1.5 metres high, the banks are not considered to have the potential for instability, and therefore, the limit of hazard lands should not apply to the watercourses.

However, the following provides recommendations on the setbacks, if required.

The purpose of this assessment is to establish the 'Erosion Hazard Limit' for the east banks of Sawmill Creek and the south banks of the ditch at the site. 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.

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: (1) Stable Slope Allowance, (2) Toe Erosion Allowance, and (3) Erosion Access Allowance.

Based on the document titled "Slope Stability Guidelines for Development Applications in the City of Ottawa" (dated 2012), any ground which is inclined steeper than about 11 degrees from horizontal and greater than 2 metres in height has the potential for instability. Based on the shallow inclinations and heights of less than 1.5 metres high, the banks along Sawmill Creek and the existing ditch are considered to be stable, from a geotechnical point of view, and therefore the Stable Slope Allowance, as described in the MNR procedures, does not apply.

In accordance with the MNR documents, a minimum Toe Erosion Allowance of between 5.0 to 8.0 metres is required for stiff cohesive soil (clays) and coarse cohesionless materials. Given the banks of the creek and ditch are less than 1.5 metres in height, as well as gabion baskets and granular material was present along portions of the banks of Sawmill Creek and the ditch, a Toe Erosion Allowance of 5.0 metres can be used.

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

Based on the low height (i.e., generally less than 1.0 metre) of the banks, relatively small equipment could likely be used for repairs.

Based on the above information, the Erosion Hazard Limit for the banks along the Sawmill Creek and ditch would be 11 metres, as measured from the crest of the bank.

Based on the results of the environmental impact statement (EIS), in accordance with the City of Ottawa's Official Plan policies, a minimum 30 metre setback from top of bank is required and is considered sufficient to protect fish habitat within Sawmill Creek. A minimum 15 metre setback is required from top of bank of the ditch to protect fish habitat within the tributary.

It is considered that the required setbacks from the watercourses as described in the EIS are not additional to the Erosion Hazard Limit, since the setbacks identified by the EIS are for fishery protection and are greater than the Erosion Hazard Limit which sets a limiting distance from the top of bank for development. Application of the 15 and 30 metre setbacks as identified by the EIS will therefore also provide the required separation distance from top of bank for erosion hazards.

6.0 GLOBAL STABILITY ANALYSIS – PROPOSED RETAINING WALLS

6.1 General

The purpose of this global stability assessment is to determine the factor of safety against global instability of the proposed retaining walls located north east of Building A and south of Buildings F, G, and H.

A series of analyses were carried out using Slope/W, a two-dimensional limit equilibrium slope stability program. The software determines a factor of safety for possible failure surfaces as the ratio of the available shear strength along the surface and the shear strength required to maintain equilibrium. The Morgenstern-Price method was used in the analyses. A discussion of the required values of factor of safety is provided later in this report.

Static and simplified seismic (or pseudo-static) analyses were carried out to model long term and seismic loading conditions, respectively. An earthquake with a return period of 2,475 years (i.e., probabilities of exceedance of 2 percent over a 50-year period) was considered for the simplified seismic analyses.

6.2 Analysis Inputs

The global stability analyses were carried out using soil parameters, groundwater conditions that attempt to model the proposed retaining wall but do not exactly represent the actual conditions.

Elevated groundwater levels were used in the analysis as a conservative approach, in general accordance with standard practice for slope stability analyses.

6.3 Soil Strength Parameters

The global stability analyses were carried out using strength parameters assessed from the results of the previous geotechnical investigation. To assess the existing factor of safety against overall static rotational failure in long term conditions, the global stability analyses were carried out using drained soil parameters. To assess the existing factor of safety against overall static rotational failure during seismic conditions (i.e., earthquake loading), undrained parameters were assigned to the silty clay layer.

Table 6.1 summarizes the soil parameters used in the analyses.

Table 6.1 – Slope Stability Soil Strength Parameters

Soil Type	Unit Weight, γ (kN/m^3)	Effective Angle of Internal Friction, φ (degrees)	Cohesion, c (kilopascals)	Undrained Shear Strength, S_u, (kilopascals)
Retaining Wall	24	90	500	0
Engineered Fill	22	34	0	0
Existing Fill	20	32	0	0
Weathered Silty Clay Crust	18	35	5	75
Glacial Till	22	36	0	0

The soil properties of the retaining wall were given such that the model would not fail through the proposed retaining wall to assess the global stability of the retaining wall.

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, we have assumed the groundwater level at an elevation of about 81.1 metres for long term and seismic conditions.

It is also understood that the retaining wall will be supporting driving lanes and parking areas. As such, a surcharge load of 12 kilopascals was applied to the ground surface of the parking area to account for vehicular traffic.

6.4 Results of Assessment

6.4.1 Required Factor of Safety

For the purposes of this study, for static conditions, a factor of safety of 1.5 or greater, is considered an acceptable factor of safety which allows for a degree of uncertainty. A factor of safety of 1.3 to 1.5 is considered to be marginally suitable, depending on the risk tolerance of the owner.

A computed factor of safety of between 1.3 and greater than 1.0 is generally not considered an acceptable factor of safety for long term conditions. While a factor of safety of 1.0 (or less) is considered to represent a slope which is potentially unstable.

For seismic/dynamic conditions, a factor of safety of 1.1, or greater, is considered to indicate adequate stability under the design earthquake event. For this assessment the design earthquake loading is based on an acceleration of 0.15 g (which corresponds to half the peak ground acceleration based on a Site Class D, as per the 2015 National Building Code of Canada).

The selection of an acceptable Factor of Safety depends in part on the level of risk of failure which is considered acceptable, and also on regulatory requirements where applicable.

6.4.2 Findings of Assessment

The results of the slope stability analyses are provided in Appendix D on Figures D12 to D15, and are summarized in Table 6.2, below.

Table 6.2 – Summary of Factor of Safety

Retaining Wall Location	Assessment	Global Factor of Safety
Building A	Static	1.7
Building A	Seismic	2.2
Buildings F, G, and H	Static	2.2
Buildings F, G, and H	Seismic	2.7

The results indicate that the retaining walls at the site have a global factor of safety against instability greater than 1.5 and 1.1 under static and seismic conditions, respectively. As such, the proposed retaining walls at the site are considered stable, from a global stability point of view.

7.0 ADDITIONAL CONSIDERATIONS

7.1 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. Assuming that any excavating is carried out in accordance with the guidelines in this report, the magnitude of the vibrations will be much less than that required to cause damage to the nearby structures or services in good condition. Precondition surveys of the adjacent structures should be considered.

7.2 Winter Construction

Most of the soils at this site are highly frost susceptible and prone to significant ice lensing. In order to carry out the work during freezing temperatures, the excavation should be opened for as short a time as practicable and the excavations should be carried out only in lengths that allow all of the construction operations, including backfilling, to be fully completed in one working day. The materials on the sides of the trenches should not be allowed to freeze. In addition, the backfill should be excavated, stored, and replaced without being disturbed by frost or contaminated by snow or ice.

7.3 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have not been assessed. Consultation on this matter can be provided upon request.

7.4 Abandonment of Monitoring Wells

All monitoring wells installed as part of this investigation should be decommissioned by a licensed well technician in accordance with Ontario Regulation 903, as amended by Ontario Regulation 128/03. The well abandonment could be carried out in advance of or during construction.

7.5 Billings Formation Shale

The site is within a region where Billings Formation shale is mapped as the bedrock type underlying the soils. In the Ottawa area, problems associated with swelling bedrock are mainly associated with the Billings Formation. This bedrock formation contains pyrite which is known to breakdown to sulfides when exposed to air (oxygen) under some combinations of conditions, which can result in relatively rapid deterioration of the shale and heaving in the longer term.

Swelling bedrock can present a significant risk to the works at the site and additional measures below may be required during design and construction stages to reduce the risk of swelling from effecting the proposed and existing nearby structures, as preliminary guidance:

- Where possible exposure of the bedrock to oxygen should be avoided. If this cannot be avoided the bedrock surface should be covered by a mud-slab as soon as practical upon exposure (but not later than within 24 hours of first exposure);
- Dewatering of the Billings Formation shale can also cause swelling of the shale to occur by exposing the shale to oxygen. Further it should be considered that radius of influence of dewatering activities can extend beyond the perimeter of the site can trigger swelling of shale on adjacent sites within the zone of influence of the dewatering operations;

- Re-use of shale bedrock should not be considered due to the potential for the shale to swell once exposed to air. The swelling could cause damage to overlying structures including floor slabs, foundations, utilities, and roadways; and,
- The oxidation reaction of pyrite in expansive shale can produce sulphuric acid. The sulphites can corrode unprotected buried steel elements, including, for example rock anchors. Any concrete in contact with the shale may also require use of sulphate resistant cement.

It should be noted that the above measures are intended to reduce not eliminate the risk associated with swelling bedrock conditions. Further discussion on risks and mitigation measures associated with Billings Formation shale can be provided if required.

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.

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W. Alex Meacoe

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LICENSED PROFESSIONAL ENGINEER
W. A. MEACOE
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May 13, 2024
PROVINCE OF ONTARIO

FZ/SG/WAM/DC

Enclosures
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CONDITIONS AND LIMITATIONS OF THIS REPORT

1. **Standard of Care:** GEMTEC has prepared this report in a manner consistent with generally accepted engineering or environmental consulting practice in the jurisdiction in which the services are provided at the time of the report. No other warranty expressed or implied is made.

2. **Copyright:** The contents of this report are subject to copyright owned by GEMTEC, save to the extent that copyright has been legally assigned by us to another party or is used by GEMTEC under license. To the extent that GEMTEC owns the copyright in this report, it may not be copied without our prior written agreement for any purpose other than the purpose indicated in this report. The methodology (if any) contained in this report is provided to the Client in confidence and must not be disclosed or copied to third parties without the prior written agreement of GEMTEC. Disclosure of that information may constitute an actionable breach of confidence or may otherwise prejudice our commercial interests.

3. **Complete Report:** This report is of a summary nature and is not intended to stand alone without reference to the instructions given to GEMTEC by the Client, communications between GEMTEC and the Client and to any other reports prepared by GEMTEC for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. GEMTEC cannot be responsible for use of portions of the report without reference to the entire report.

4. **Basis of Report:** This Report has been prepared for the specific site, development, design objectives and purposes that were described to GEMTEC by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document, subject to the limitations provided herein, are only valid to the extent that this report expressly addresses the proposed development, design objectives and purposes. Any change of site conditions, purpose or development plans may alter the validity of the report and GEMTEC cannot be responsible for use of this report, or portions thereof, unless GEMTEC is requested to review any changes and, if necessary, revise the report.

5. **Time Dependence:** If the proposed project is not undertaken by the Client within 18 months following the issuance of this report, or within the timeframe understood by GEMTEC to be contemplated by the Client, the guidance and recommendations within the report should not be considered valid unless reviewed and amended or validated by GEMTEC in writing.

6. **Use of This Report:** The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without GEMTEC's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, GEMTEC may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process.

   Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety, and equipment capabilities.

7. **No Legal Representations:** GEMTEC makes no representations whatsoever concerning the legal significance of its findings, or as to other legal matters touched on in this report, including but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation and change. Such interpretations and regulatory changes should be reviewed with legal counsel.

8. **Decrease in Property Value:** GEMTEC shall not be responsible for any decrease, real or perceived, of the property or site’s value or failure to complete a transaction, as a consequence of the information contained in this report.

9. **Reliance on Provided Information:** The evaluation and conclusions contained in this report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations. information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in this report as a result of misstatements, omissions, misrepresentations. or fraudulent acts of the Client or other persons providing information relied on by us. We are entitled to rely on such representations, information

and instructions and are not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

10. **Investigation Limitations:** Site investigation programs are a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions but even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions.

    The data derived from the site investigation program and subsequent laboratory testing are interpreted by trained personnel and extrapolated across the site to form an inferred geological representation and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Conditions between and beyond the borehole/test hole locations may differ from those encountered at the borehole/test hole locations and the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies. Accordingly, GEMTEC does not warrant or guarantee the exactness of the subsurface descriptions.

    Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

    In addition, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

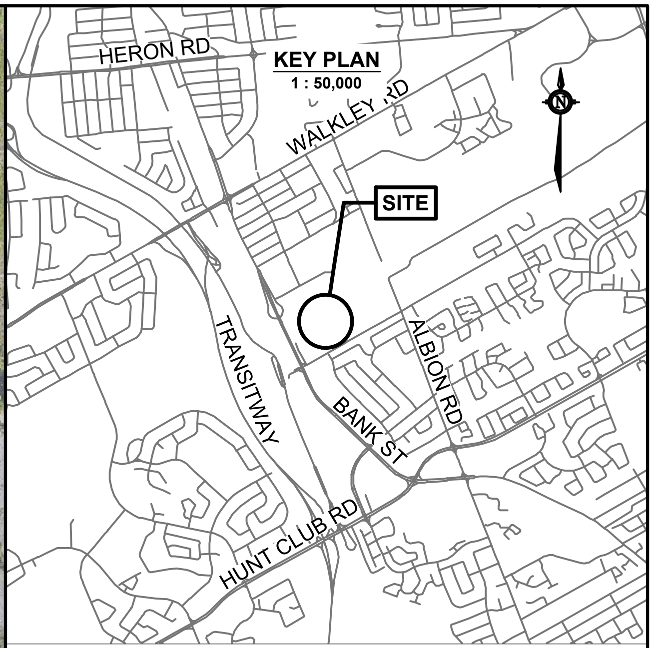
11. **Sample Disposal:** GEMTEC will dispose of all uncontaminated soil and/or rock samples 60 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fill materials or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

12. **Follow-Up and Construction Services:** All details of the design were not known at the time of submission of GEMTEC's report. GEMTEC should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of GEMTEC's report.

    During construction, GEMTEC should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of GEMTEC's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in GEMTEC's report. Adequate field review, observation and testing during construction are necessary for GEMTEC to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, GEMTEC's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

13. **Changed Conditions:** Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that GEMTEC be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that GEMTEC be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

14. **Drainage:** Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. GEMTEC takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



LEGEND

BH # ← BOREHOLE ID
XX.XX ← GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

BOREHOLE LOCATION (current investigation by GEMTEC)

APPROXIMATE SITE BOUNDARY

A A' **CROSS SECTION LOCATION**

GENERAL NOTE(S)

1. Coordinate system: NAD83, UTM ZONE18N.
2. Contains information licensed under the Open Government Licence – Ontario.
3. Maps Data: Google, @2023 CNES / Airbus, First Base Solutions, Maxar Technologies.
4. Geographic dataset source: Ontario GeoHub.

SCALE
1:1500
0 30 60 90m

DRAWING
SITE PLAN

CLIENT
2079 ARTISTIC PLACE GP INC.

PROJECT
GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL DEVELOPMENT
1319 JOHNSTON ROAD
OTTAWA, ONTARIO

DRAWN BY	CHECKED BY
S.L.	W.A.M.
PROJECT NO.	REVISION NO.
101481.008	1
DATE	FIGURE NO.
MAY 2024	**FIGURE 1**

GEMTEC
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Tel: (613) 836-1422
www.gemtec.ca
ottawa@gemtec.ca

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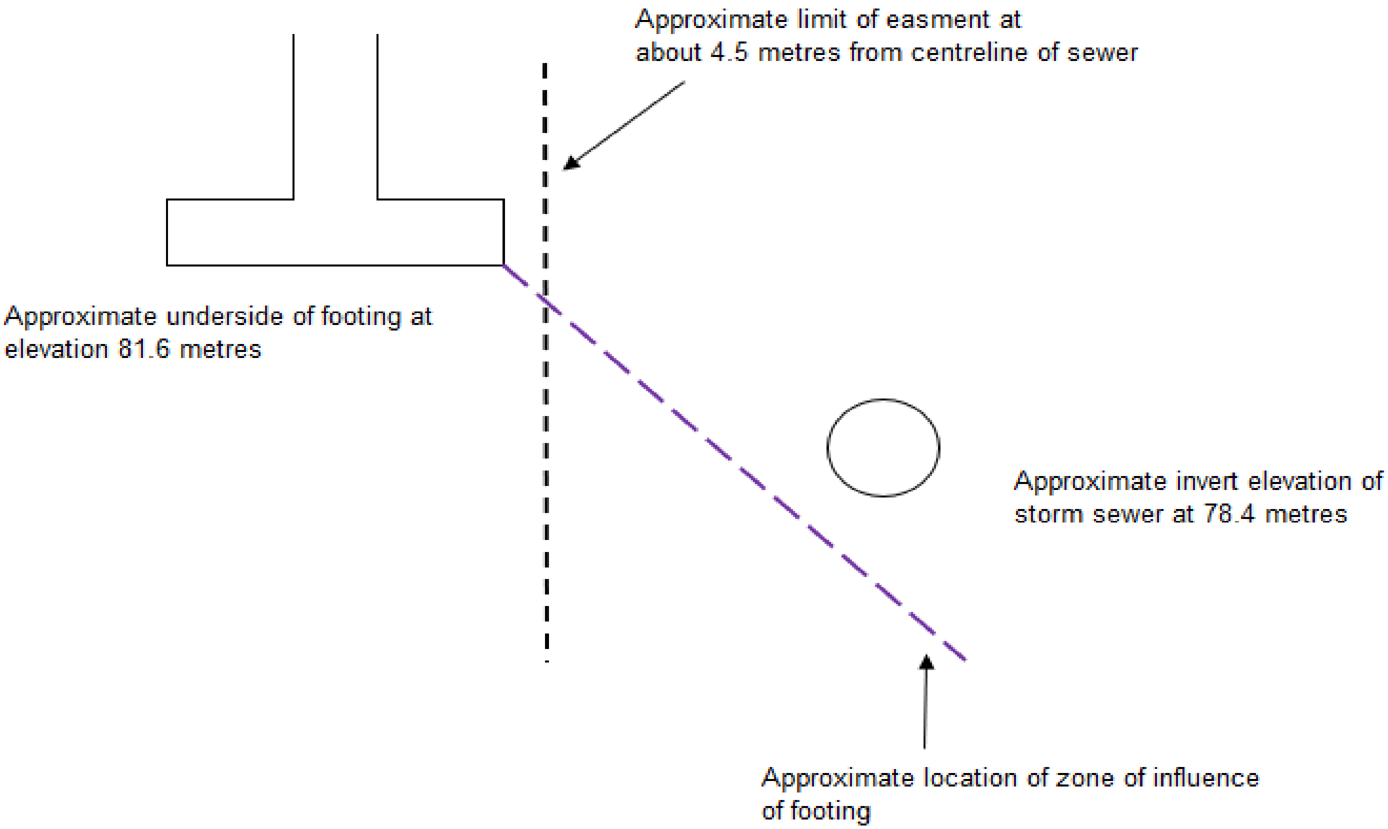

Not to Scale

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Approximate Location of Footings to Storm Sewer

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	10/05/2024

Figure 2

APPENDIX A

Record of Borehole Sheets
List of Abbreviations and Symbols
Boreholes 23-01 to 23-08

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

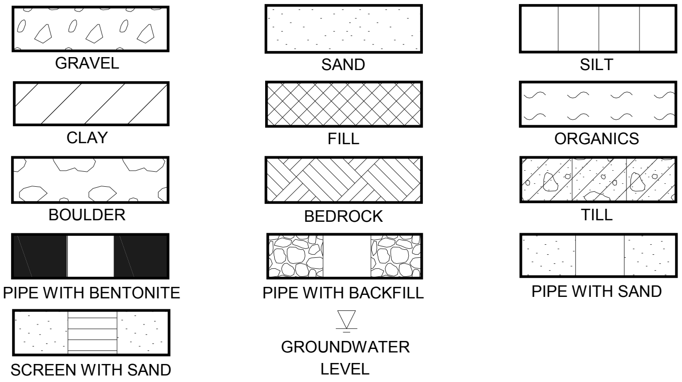
PENETRATION RESISTANCE

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.

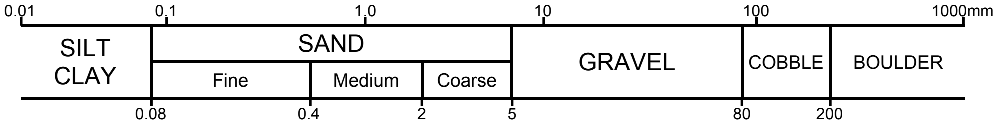
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

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



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.

GEMTEC

descriptive terms.pub
Modified May 2018

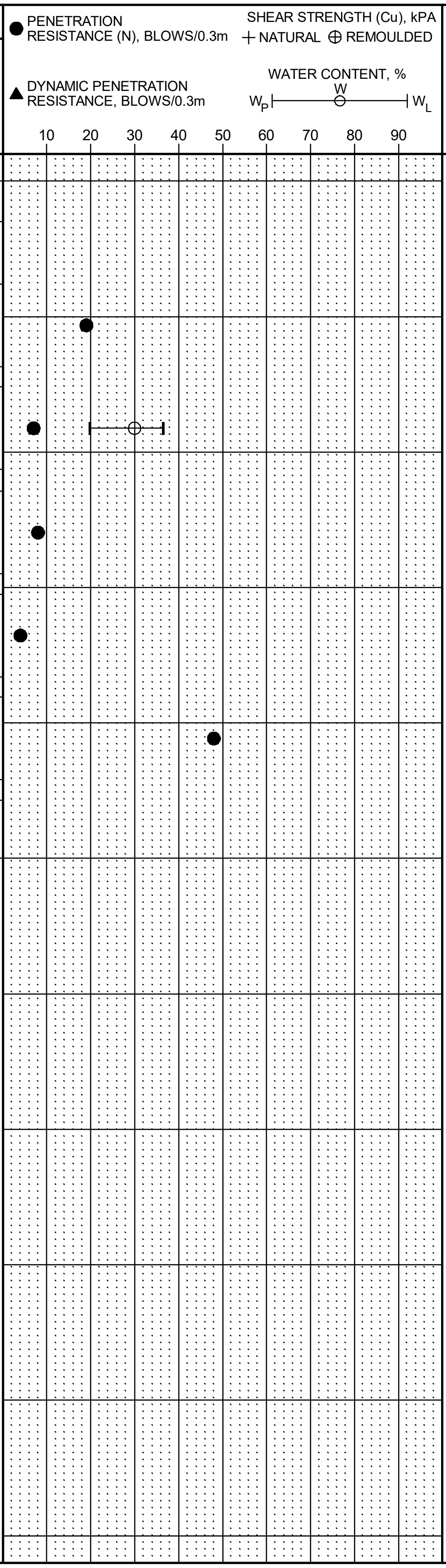
RECORD OF BOREHOLE 23-01

CLIENT: 2079 Artistic Place GP Inc.
PROJECT: Geotechnical Investigation, Proposed Industrial Development, 1319 Johnston Road, Ottawa, Ontario
JOB#: 101481.008
LOCATION: See Site Plan, Figure 1

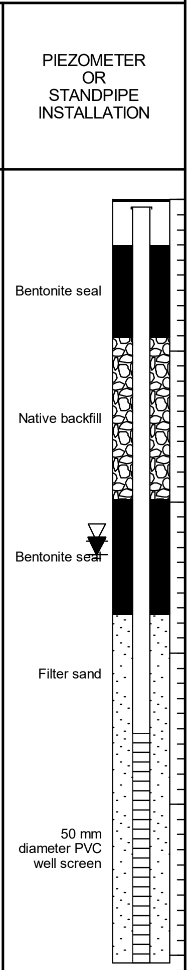
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 13 2023

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
Ground Surface	82.75				
Grey brown gravel and sand, some silt, with cobbles and boulders (FILL MATERIAL)		1	GS		
Stiff to very stiff, grey brown SILTY CLAY (WEATHERED CRUST)	81.71 1.04	2	SS	255	19
		3	SS	305	7
Loose, grey brown SILTY CLAYEY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)	80.46 2.29	4	SS	560	8
		5	SS	330	4
Dense to very dense, grey brown gravelly SILTY SAND, with cobbles and boulders (GLACIAL TILL)	78.79 3.96	6	SS	305	48
		7	SS	430	>50
End of Borehole Auger Refusal	77.70 5.05				



ADDITIONAL LAB. TESTING



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)		ELEV. (m)
23/06/28	2.3	▽	80.5
24/05/02	2.4	▼	80.4

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LOGGED: AN
CHECKED: FZ

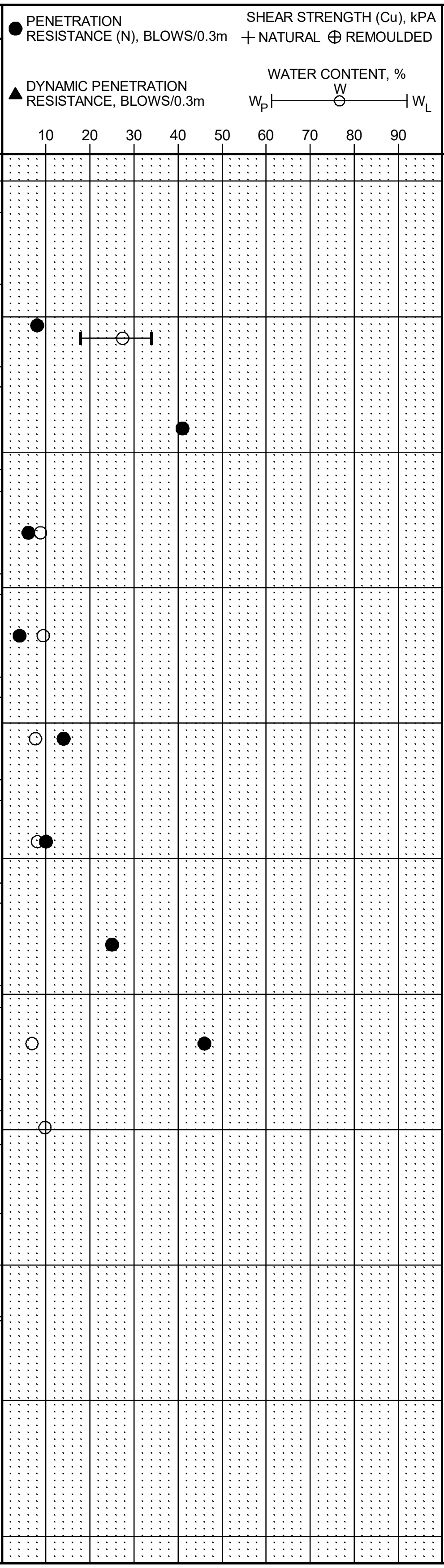
RECORD OF BOREHOLE 23-02

CLIENT: 2079 Artistic Place GP Inc.
PROJECT: Geotechnical Investigation, Proposed Industrial Development, 1319 Johnston Road, Ottawa, Ontario
JOB#: 101481.008
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 12 2023

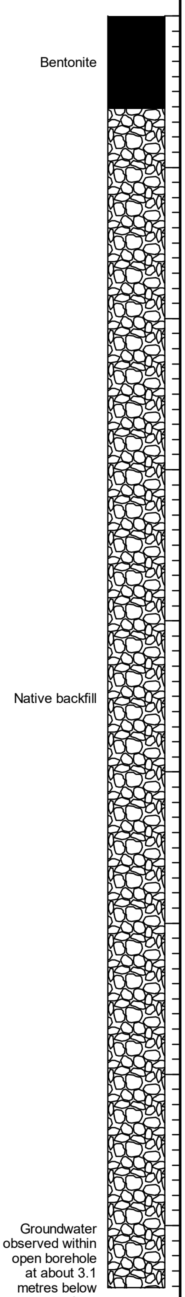
BORING METHOD: Power Auger, Hollow Stem Auger (210mm OD)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
Ground Surface	81.87				
Grey brown gravel and sand, some silt, with cobbles (FILL MATERIAL)		1	GS		
		2	GS		
Stiff to very stiff, grey brown SILTY CLAY (WEATHERED CRUST)	80.93 0.94	3	SS	405	8
Dense to loose, grey brown SILTY CLAYEY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)	80.35 1.52	4	SS	380	41
		5	SS	255	6
		6	SS	305	4
		7	SS	355	14
Compact to very dense, grey brown gravelly SILTY SAND, with cobbles and boulders (GLACIAL TILL)	77.30 4.57	8	SS	180	10
		9	SS	405	25
		10	SS	510	46
		11	SS	255	>50
		12	SS	380	>50
Possible highly weathered SHALE BEDROCK	73.92 7.95				
End of Borehole	73.46 8.41	~~13~~	~~SS~~	~~25~~	~~>50~~



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



Groundwater observed within open borehole at about 3.1 metres below ground surface upon completion of augering

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LOGGED: AN
CHECKED: FZ

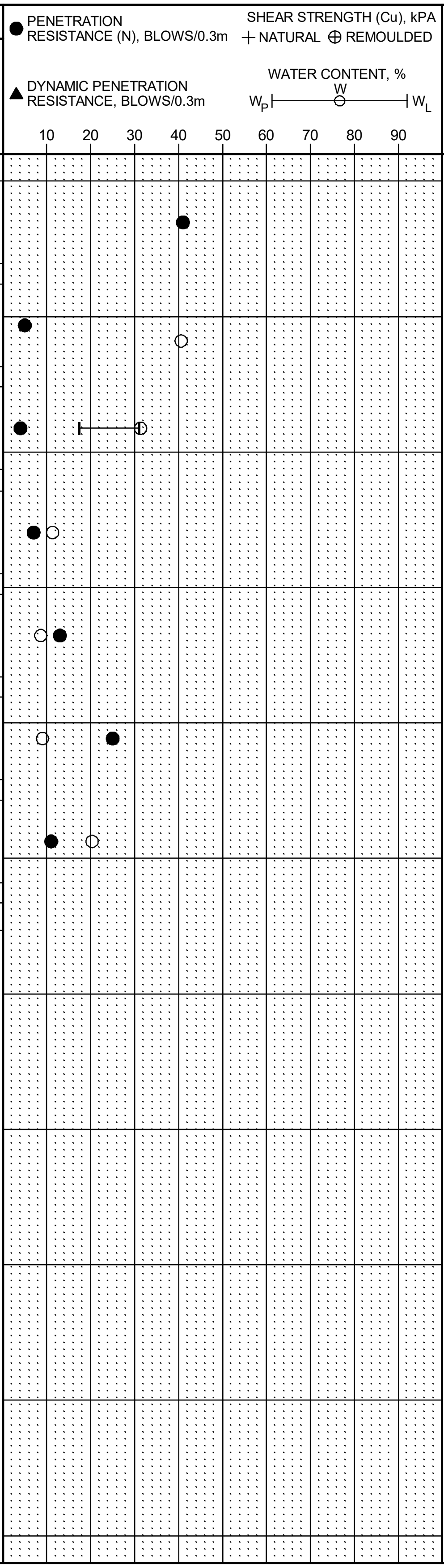
RECORD OF BOREHOLE 23-03

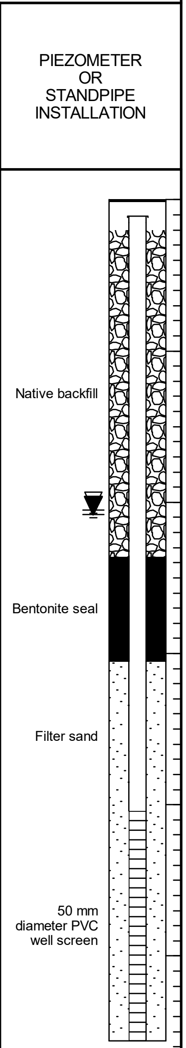
CLIENT: 2079 Artistic Place GP Inc.
PROJECT: Geotechnical Investigation, Proposed Industrial Development, 1319 Johnston Road, Ottawa, Ontario
JOB#: 101481.008
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 13 2023

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

SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	82.05					
Dense, grey brown gravel and sand, some silt (FILL MATERIAL)		1	SS	510	41	
Topsoil (FILL MATERIAL)	81.29 0.76					
Grey brown silty sand, some gravel (FILL MATERIAL)	0.89					
Stiff to very stiff, grey brown SILTY CLAY, trace sand (WEATHERED CRUST)	0.99	2	SS	455	5	MH
		3	SS	585	4	
Loose to compact, grey brown SILTY CLAYEY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)	80.02 2.03	4	SS	455	7	MH
		5	SS	485	13	
		6	SS	280	25	
Compact to very dense, grey brown SAND, some gravel, some silt, with cobbles and boulders (GLACIAL TILL)	77.48 4.57	7	SS	610	11	M
		8	SS	205	>50	
End of Borehole Auger Refusal	76.49 5.56					





GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
23/06/28	2.1 ▽	80.0
24/05/02	2.1 ▼	80.0

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LOGGED: AN
CHECKED: FZ

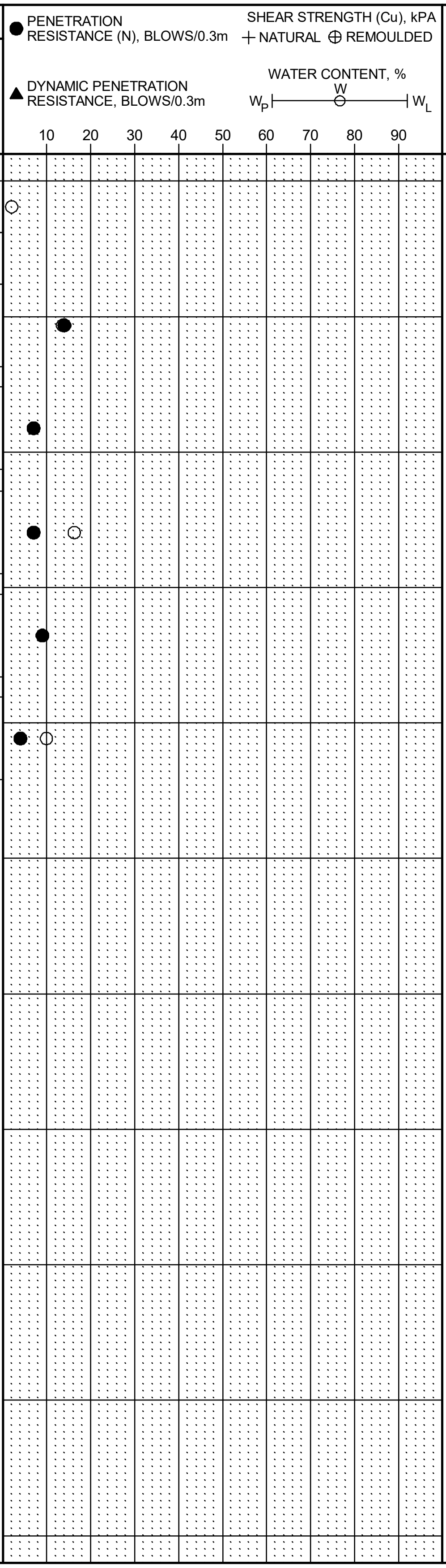
RECORD OF BOREHOLE 23-04

CLIENT: 2079 Artistic Place GP Inc.
PROJECT: Geotechnical Investigation, Proposed Industrial Development, 1319 Johnston Road, Ottawa, Ontario
JOB#: 101481.008
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 12 2023

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	83.21						
	Grey brown gravel and sand, some silt, with cobbles (FILL MATERIAL)		1	GS			M	
			2	GS				
1	Grey brown silty sand to silty clay, some gravel, with organics (FILL MATERIAL)	82.45 0.76	3	SS	330	14		
2			4	SS	150	7		Native backfill
	Loose, grey brown SILTY CLAYEY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)	80.92 2.29	5	SS	150	7		
3			6	SS	560	9		
4			7	SS	380	4		Groundwater observed within open borehole at about 3.2 metres below ground surface upon completion of augering
	End of Borehole Auger Refusal	78.76 4.45						



LOGGED: AN
CHECKED: FZ

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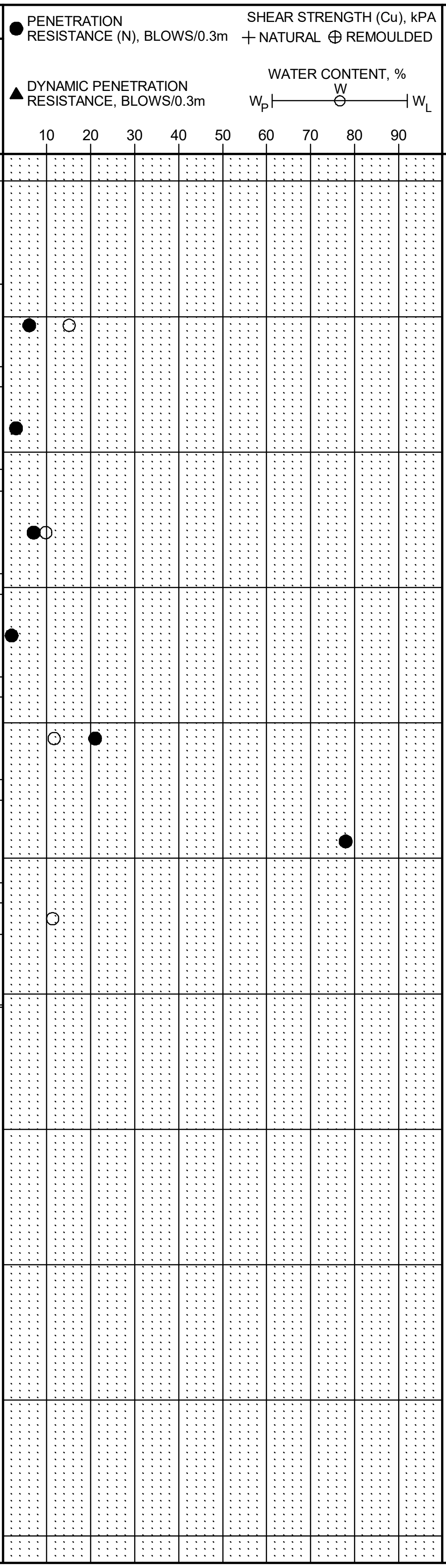
RECORD OF BOREHOLE 23-05

CLIENT: 2079 Artistic Place GP Inc.
PROJECT: Geotechnical Investigation, Proposed Industrial Development, 1319 Johnston Road, Ottawa, Ontario
JOB#: 101481.008
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 12 2023

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

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
Ground Surface	81.99				
Grey brown gravel and sand, some silt, with cobbles (FILL MATERIAL)		1	GS		
Loose, grey brown silty sand, some clay, some gravel, with organics (FILL MATERIAL)	81.23 0.76	2	SS	205	6
Very loose to loose, grey brown SILTY CLAYEY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)	80.42 1.57	3	SS	180	3
		4	SS	50	7
		5	SS	180	2
Compact, grey brown gravelly SILTY SAND, with cobbles and boulders (GLACIAL TILL)	78.18 3.81	6	SS	355	21
Possible highly weathered SHALE BEDROCK	77.19 4.80	7	SS	430	78
		8	SS	230	>50
End of Borehole	75.89 6.10	~~9~~	~~SS~~	~~0~~	~~>50~~



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Native backfill	0 – 0.6 m
Bentonite	0.6 – 0.9 m
Native backfill	0.9 – 4.4 m
Bentonite	4.4 – 5.6 m
Native backfill	5.6 – 6.1 m

Groundwater observed within open borehole at about 2.1 metres below ground surface upon completion of augering

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LOGGED: AN
CHECKED: FZ

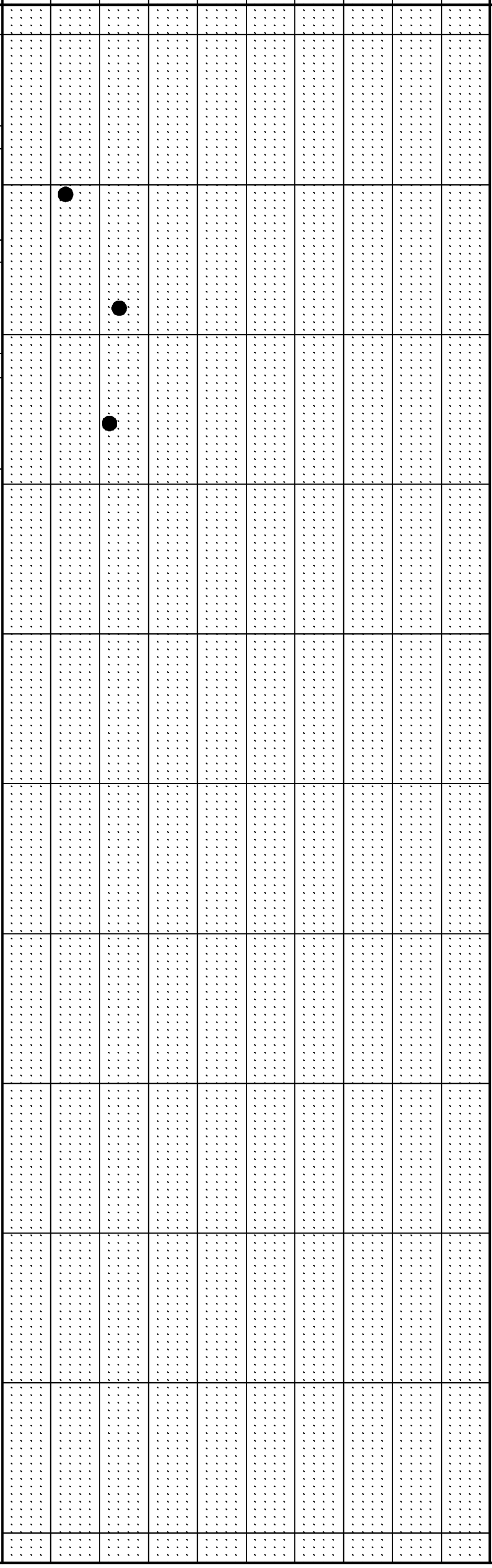
RECORD OF BOREHOLE 23-06

CLIENT: 2079 Artistic Place GP Inc.
PROJECT: Geotechnical Investigation, Proposed Industrial Development, 1319 Johnston Road, Ottawa, Ontario
JOB#: 101481.008
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 12 2023

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

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	81.93					
	Grey brown gravel and sand, some silt, with cobbles (FILL MATERIAL)		1	GS			
	Compact, grey brown SILTY CLAYEY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)	81.32 0.61					Native backfill
1			2	SS	405	13	
2			3	SS	560	24	
			4	SS	25	22	
3	End of Borehole Auger Refusal	78.93 3.00					No groundwater observed within open borehole upon completion of augering



ADDITIONAL LAB. TESTING

LOGGED: AN

CHECKED: FZ

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RECORD OF BOREHOLE 23-07

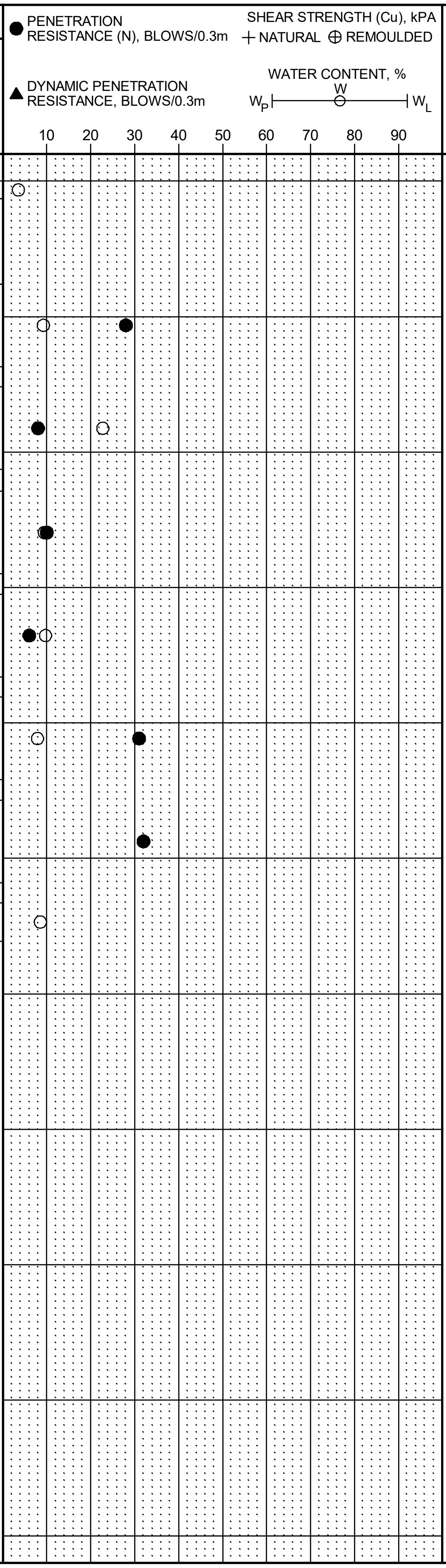
CLIENT: 2079 Artistic Place GP Inc.
PROJECT: Geotechnical Investigation, Proposed Industrial Development, 1319 Johnston Road, Ottawa, Ontario
JOB#: 101481.008
LOCATION: See Site Plan, Figure 1

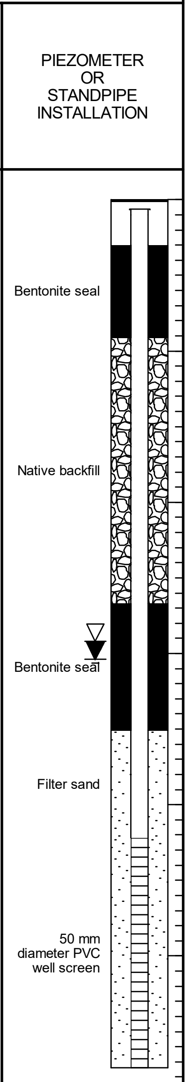
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 12 2023

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

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	82.49
Grey brown gravel and sand, some silt (FILL MATERIAL)	
Grey brown clayey sand, some silt, some gravel (FILL MATERIAL)	82.36 0.13
Compact, grey brown silty sand, trace gravel (FILL MATERIAL)	81.73 0.76
Stiff to very stiff, grey brown CLAY and SILT, some sand (WEATHERED CRUST)	80.74 1.75
Loose, grey brown CLAYEY SAND, some silt, trace gravel, with cobbles and boulders (GLACIAL TILL)	80.53 1.96
Dense to very dense, grey brown gravelly SILTY SAND, with cobbles and boulders (GLACIAL TILL)	78.68 3.81
End of Borehole Auger Refusal	76.75 5.74

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	GS			
2	GS			
3	SS	380	28	M
4	SS	430	8	MH
5	SS	535	10	
6	SS	305	6	MH
7	SS	455	31	M
8	SS	535	32	
9	SS	280	>50	





GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)		ELEV. (m)
23/06/28	2.9	▽	79.6
24/05/02	3.0	▼	79.5

LOGGED: AN

CHECKED: FZ

RECORD OF BOREHOLE 23-08

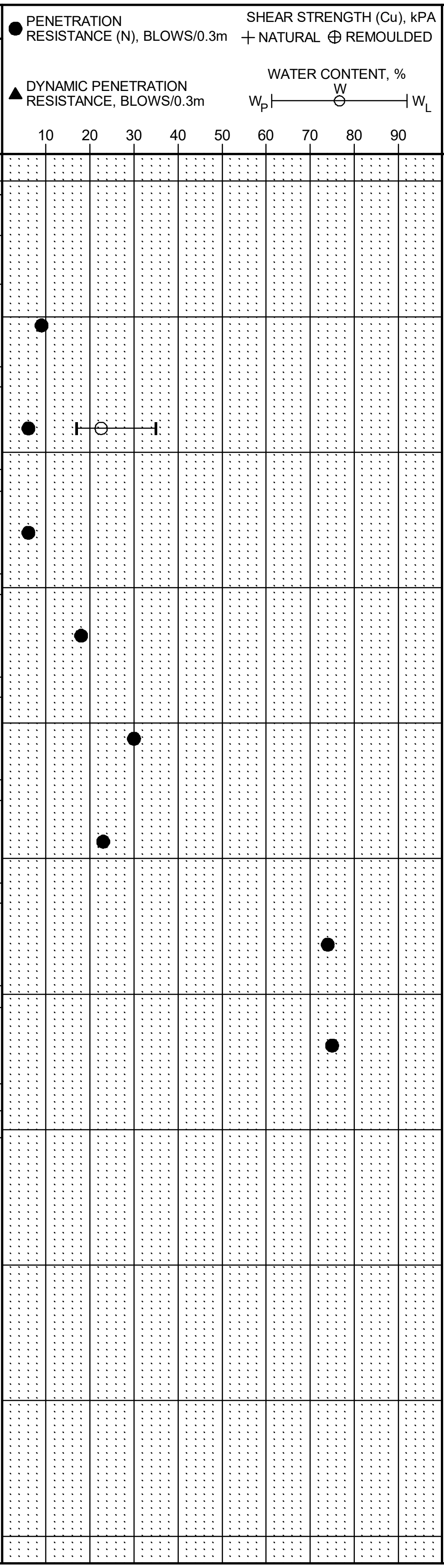
CLIENT: 2079 Artistic Place GP Inc.
PROJECT: Geotechnical Investigation, Proposed Industrial Development, 1319 Johnston Road, Ottawa, Ontario
JOB#: 101481.008
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 13 2023

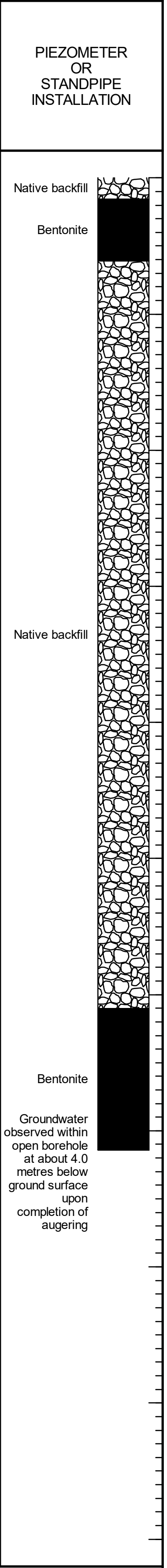
BORING METHOD: Power Auger, Hollow Stem Auger (210mm OD)

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
0	Ground Surface	82.50
0	Grey brown gravel and sand, some silt (FILL MATERIAL)	
1	Loose, grey brown silty sand, some clay, some gravel, with organics (FILL MATERIAL)	81.74 0.76
2	Stiff to very stiff, grey brown SILTY CLAY, some sand (WEATHERED CRUST)	80.98 1.52
2	Loose, grey brown SILTY CLAYEY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)	80.52 1.98
3	Compact to very dense, grey brown gravelly SILTY SAND, with cobbles and boulders (GLACIAL TILL)	79.32 3.18
7	Possible highly weathered SHALE BEDROCK	75.64 6.86
7	End of Borehole Auger Refusal	75.36 7.14

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	GS		
2	GS		
3	SS	205	9
4	SS	455	6
5	SS	0	6
6	SS	430	18
7	SS	610	30
8	SS	430	23
9	SS	610	74
10	SS	405	75
11	SS	205	>50



ADDITIONAL LAB. TESTING



Groundwater observed within open borehole at about 4.0 metres below ground surface upon completion of augering

GEMTEC Consulting Engineers and Scientists

LOGGED: AN
CHECKED: FZ

APPENDIX B

Laboratory Test Results

Report to: 2079 Artistic Place GP Inc.
Project: 101481.008 (May 13, 2024)

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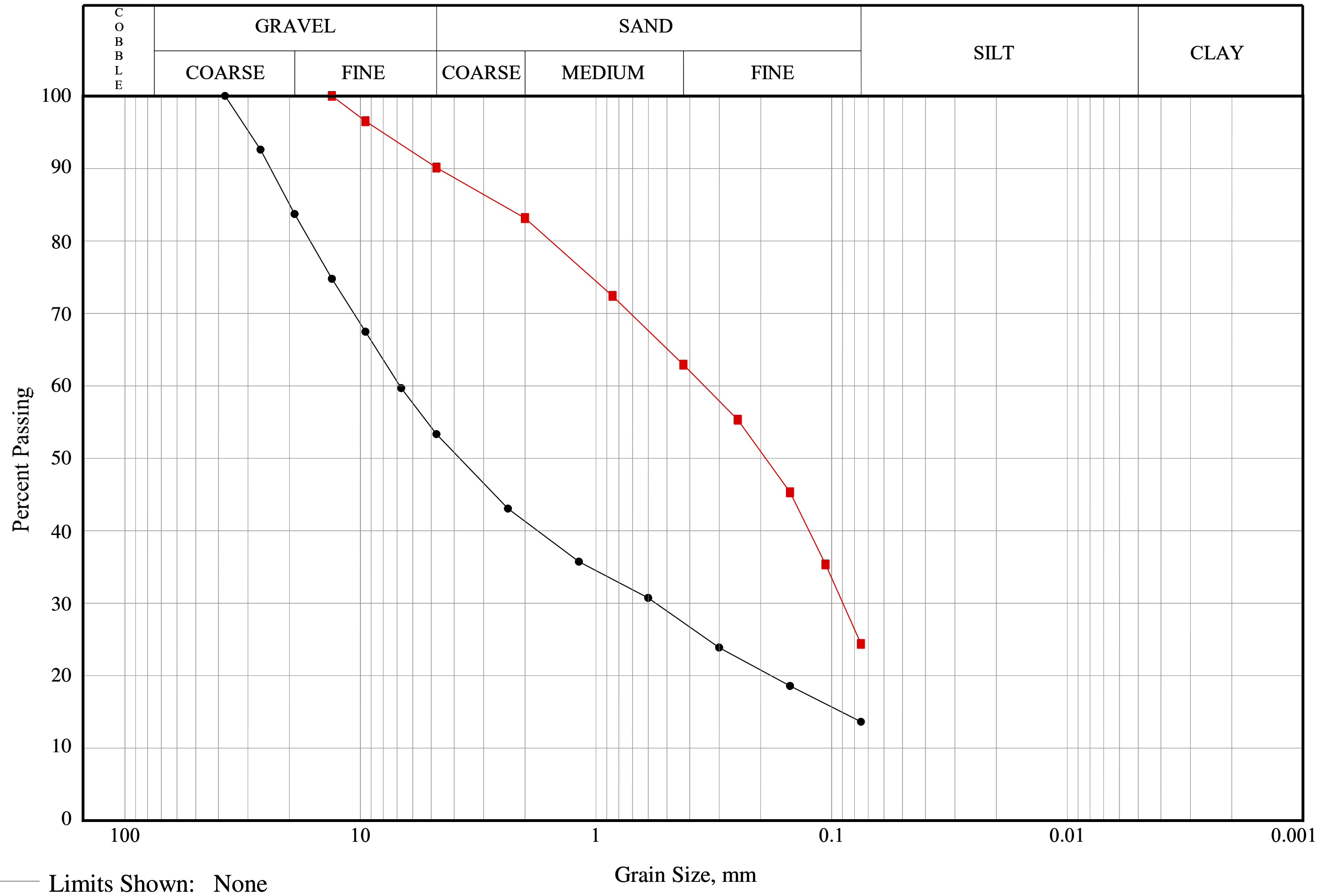
Consulting Engineers
and Scientists

Client: 2079 Artistic Place GP Inc.
Project: Geotechnical and Environmental Services, Proposed Indu
Project #: 101481008

Soils Grading Chart

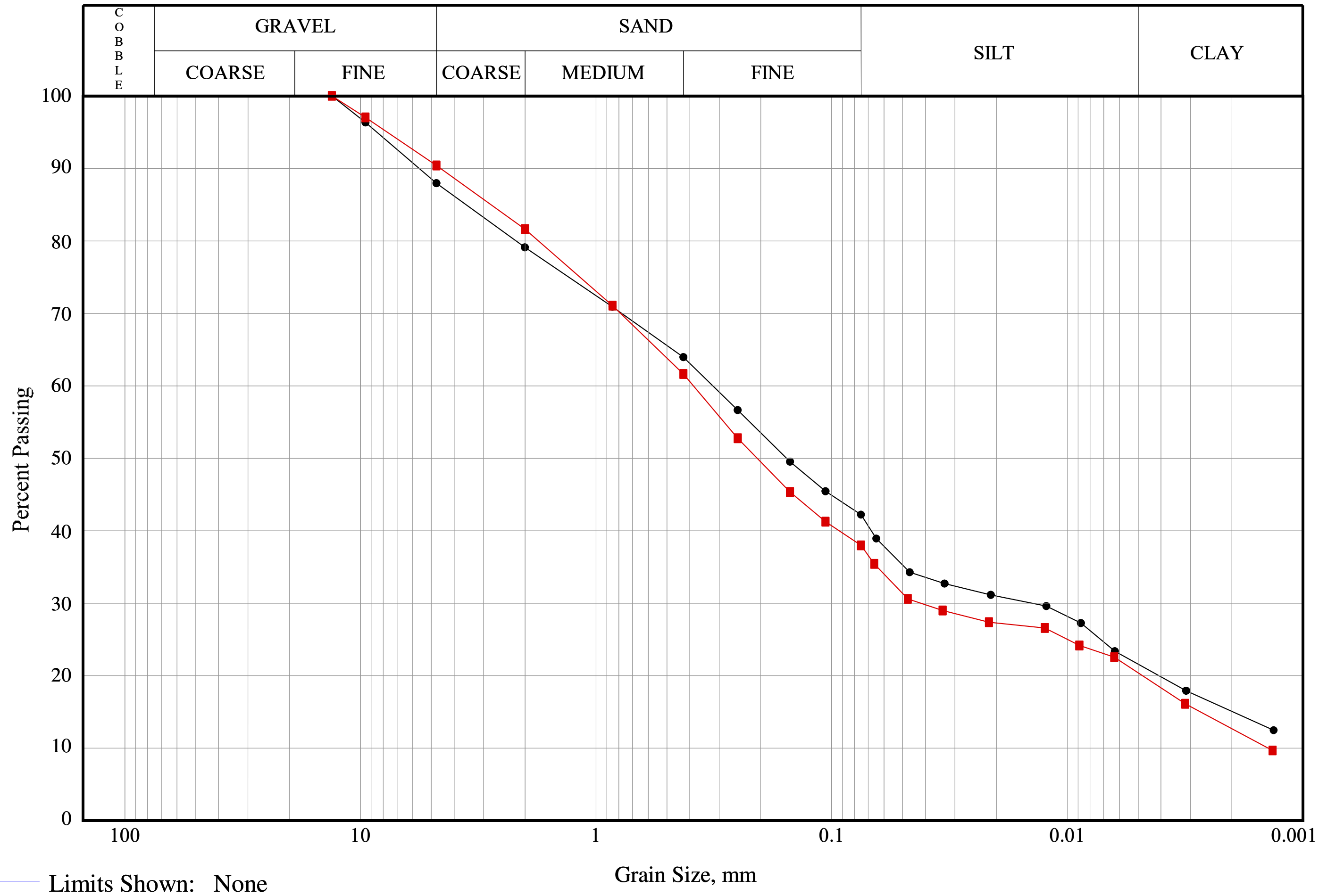


Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	FILL MATERIAL	23-04	SA 01	0.0-0.38	46.7	39.7	13.6	
—■—	FILL MATERIAL	23-07	SA 03	0.76-1.37	9.9	65.8	24.4	

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—	Gravel and sand , some silt	N/A	---	0.09	0.56	3.79	6.80	19.95	---
—■—	Silty sand , trace gravel	N/A	---	---	0.09	0.19	0.35	2.52	---

GEMTEC Consulting Engineers and Scientists			Soils Grading Chart (LS-702/ ASTM D-422)
	Client:	2079 Artistic Place GP Inc.	
	Project:	Geotechnical and Environmental Services, Proposed Indu	
	Project #:	101481008	



Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
●	GLACIAL TILL	23-03	SA 04	2.28-2.89	12.0	45.7	20.6	21.6
■	GLACIAL TILL	23-07	SA 06	3.05-3.66	9.6	52.4	17.6	20.4

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
●	Silty clayey sand , some gravel	N/A	---	0.00	0.01	0.16	0.32	3.56	20.6
■	Clayey sand , some silt , trace gravel	N/A	0.00	0.00	0.04	0.21	0.39	2.79	17.6

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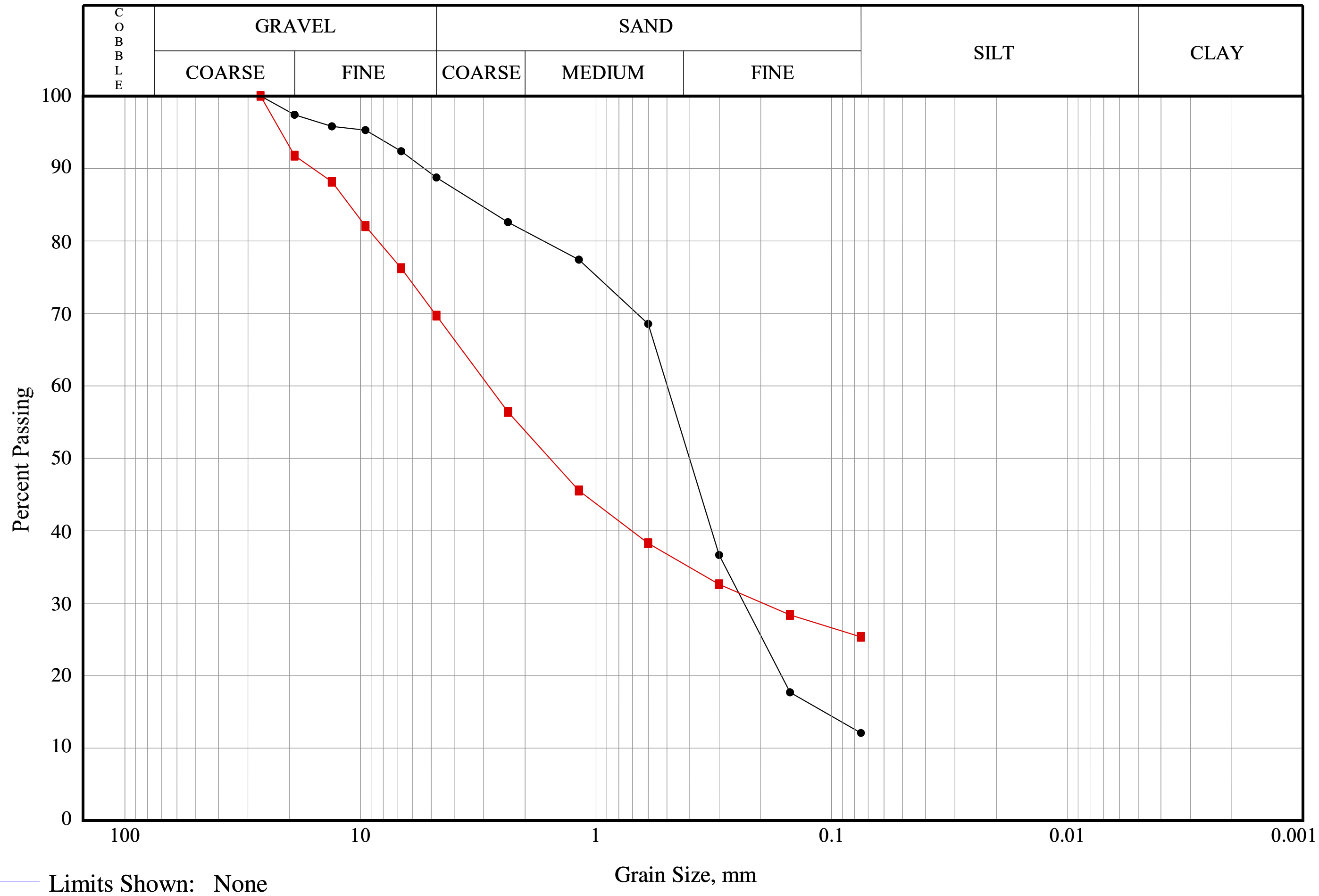
Consulting Engineers
and Scientists

Client: 2079 Artistic Place GP Inc.
Project: Geotechnical and Environmental Services, Proposed Indu
Project #: 101481008

Soils Grading Chart



Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	GLACIAL TILL	23-03	SA 07	4.57-5.18	11.3	76.7	12.1	
—■—	GLACIAL TILL	23-07	SA 07	3.81-4.42	30.3	44.3	25.3	

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—	Sand , some gravel, some silt	N/A	---	0.11	0.24	0.40	0.50	3.11	---
—■—	Gravelly silty sand	N/A	---	---	0.20	1.57	2.86	11.14	---

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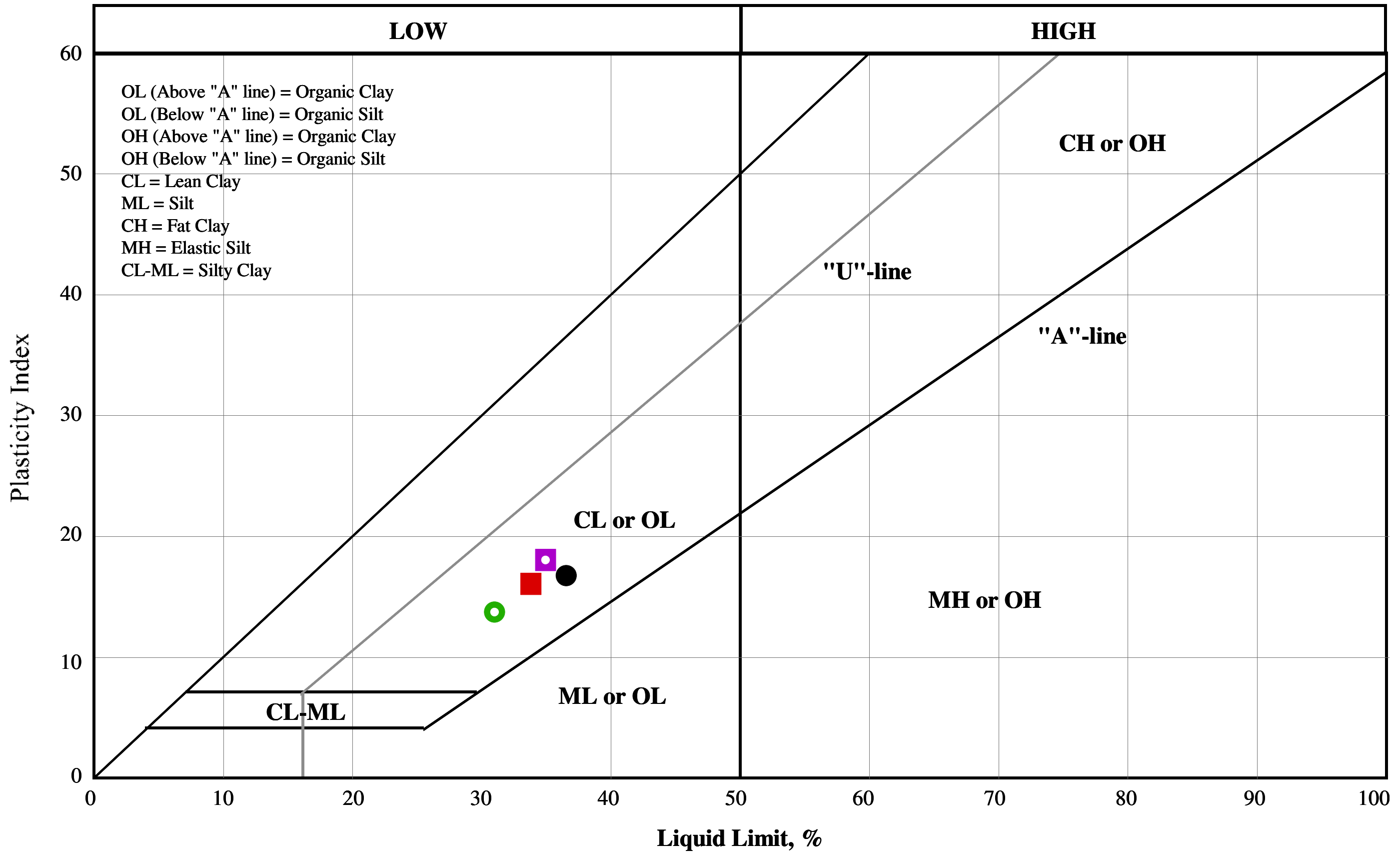
Client: 2079 Artistic Place GP Inc.
Project: Geotechnical and Environmental Services, Proposed Indu
Project #: 101481008

Plasticity Chart (D4318)



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	23-01	SA 03	1.52-2.13	36.5	19.8	16.7	☐	29.95
🟥	23-02	SA 03	0.76-1.37	33.8	17.8	16.1	☐	27.41
○	23-03	SA 03	1.52-2.03	31.0	17.3	13.7	☐	31.35
▣	23-08	SA 04	1.52-1.98	34.9	16.9	18.0	☐	22.53

Note: More information available upon request

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Shrinkage Limit

ASTM D4943

Volume of Shrinkage Dish

Mass of Shrinkage Dish (g) (m):	**20.88**	**20.82**
Mass of Shrinkage Dish and Grease(g) (m):	**20.89**	**20.90**
Mass of Glass Plate (g):	**37.35**	**37.35**
Mass of Shrinkage Dish, Plate, Grease and Water (g):	**75.60**	**75.47**
Mass of Water (g):	**17.36**	**17.22**
Volume of Shrinkage Dish:	**17.36**	**17.22**

Test Specimen

Specimen Dish:	**SL2**	**SL8**
Mass of Shrinakge Dish, m (g):	**20.88**	**20.82**
Mass of Shrinkage Dish and Grease, m_{dxg} (g):	**20.89**	**20.87**
Mass of Shrinkage Dish and Wet Soil, m_w (g):	**53**	**53.62**
Mass of Shrinkage Dish and Dry Soil, m_d (g):	**44.74**	**45.31**
Mass of Wax-Coated Soil in Air, m_{sxa} (g):	**24.56**	**25.08**
Mass of Wax-Coated Soil in Water, m_{sxw} (g):	**11.5**	**11.5**

Calculated Shrinkage Limit

Specimen Dish:	**SL2**	**SL8**
Mass of Dry Soil, m_s (g):	**23.86**	**24.49**
Water Content of Soil when Placed in Dish, w (%):	**34.62**	**33.93**
Mass of Water Displaced by Wax-Coated Soil, m_{wsx} (g):	**13.06**	**13.58**
Volume of Dry Soil and Wax, V_{dx} (cm^3):	**13.06**	**13.58**
Mass of Wax, m_x (g):	**0.70**	**0.59**
Volume of Wax, V_x (cm^3):	**0.78**	**0.66**
Volume of Dry Soil, V_d (cm^3):	**12.28**	**12.92**
Shrinkage Limit, SL:	**13.34**	**15.82**
Average Shrinkage Limit, $Sl_{avg.}$:	**14.58**	

Specific Gravity of Wax = 0.908 at15.5°C

Specific Gravity of Wax = 0.900 at 20°C

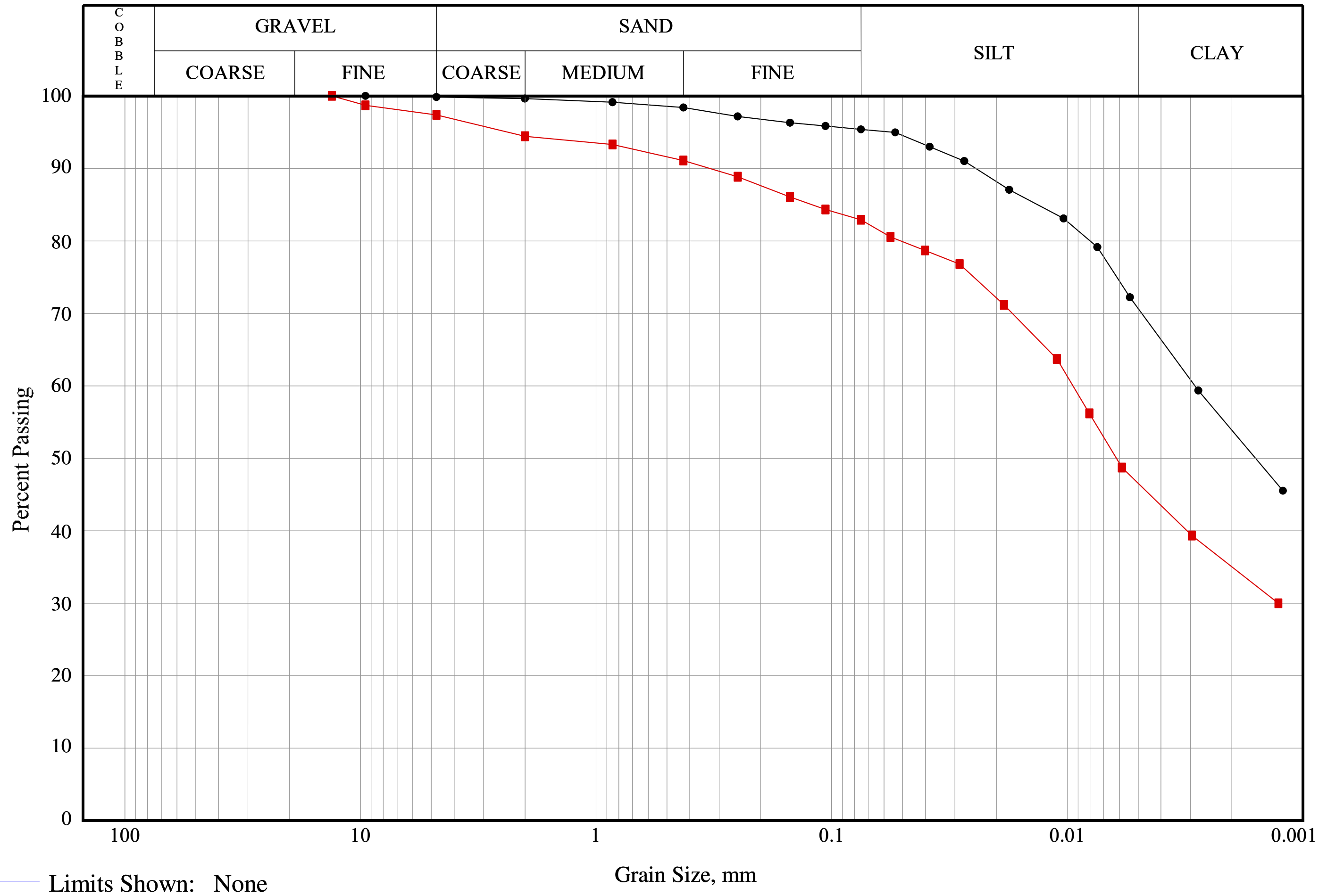
Density of Water (g/cm^3) = 1.000 (g/cm^3)

Project No.: 101481.008	**Tested By: K. Neil**
Project Name: 1319 Johnston Road, Ottawa	**Checked By: K.Smith**
Date Tested: May 6, 2024	**Sample No: TP24-01 SA1**
Sample Date:	**Source:**
Remarks:	**Depth: 1.1-2.0**

GEMTEC
Consulting Engineers and Scientists

Client:	2079 Artistic Place GP Inc.
Project:	Geotechnical and Environmental Services, Proposed Indu
Project #:	101481008

Soils Grading Chart (LS-702/ ASTM D-422)



Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
●	WEATHERED CRUST	23-03	SA 02	0.99-1.37	0.2	4.5	24.7	70.7
■	WEATHERED CRUST	23-07	SA 04	1.52-2.13	2.6	14.5	36.4	46.5

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
●	Silty clay , trace gravel, trace sand	N/A	---	---	---	0.00	0.00	0.01	23.2
■	Clay and silt , some sand , trace gravel	N/A	---	---	0.00	0.01	0.01	0.12	36.4

APPENDIX C

Chemical Analysis of Soil Sample
Sample Relating to Corrosion
(Paracel Laboratories Ltd. Order No. 2324486)

Certificate of Analysis

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

Client PO:

Report Date: 20-Jun-2023

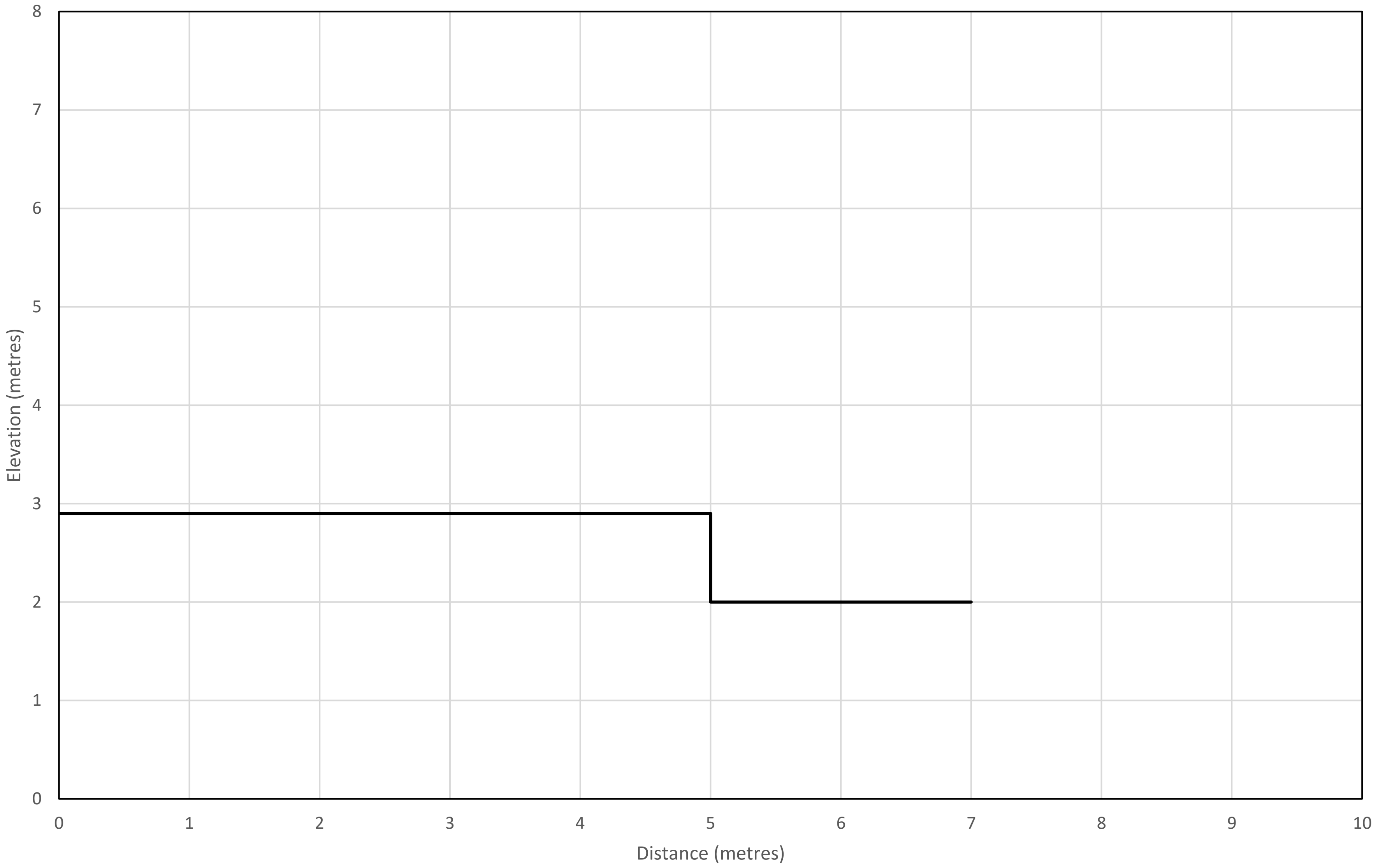
Order Date: 15-Jun-2023

Project Description: 101481.008

	Client ID:	BH23-03 SA-3 Depth 5'-7'	BH23-06 SA-3 Depth 5'-7'	-	-
	Sample Date:	13-Jun-23 01:30	12-Jun-23 02:15	-	-
	Sample ID:	2324486-01	2324486-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	77.2	90.3	-	-
General Inorganics					
Conductivity	5 uS/cm	341	451	-	-
pH	0.05 pH Units	7.63	7.68	-	-
Resistivity	0.1 Ohm.m	29.3	22.2	-	-
Anions					
Chloride	10 ug/g dry	<10	<10	-	-
Sulphate	10 ug/g dry	234	465	-	-

APPENDIX D

Stability Assessment
Figures D1 to D7 – Sawmill Creek Cross Sections
Figures D8 to D11 – Existing Ditch Cross Sections
Figures D12 to D15 – Retaining Wall Stability Assessment



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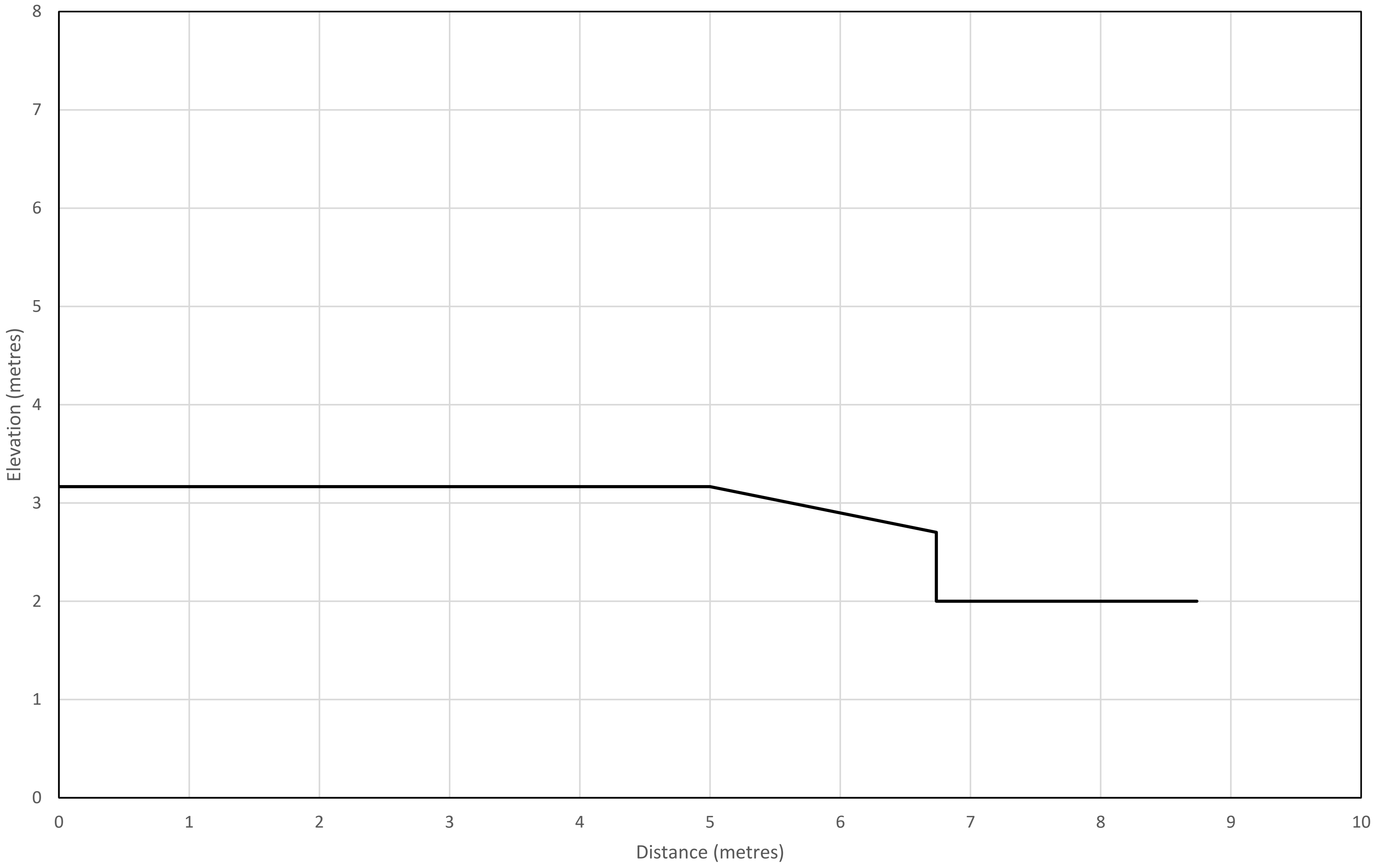
Sawmill Creek - Cross Section A-A

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D1



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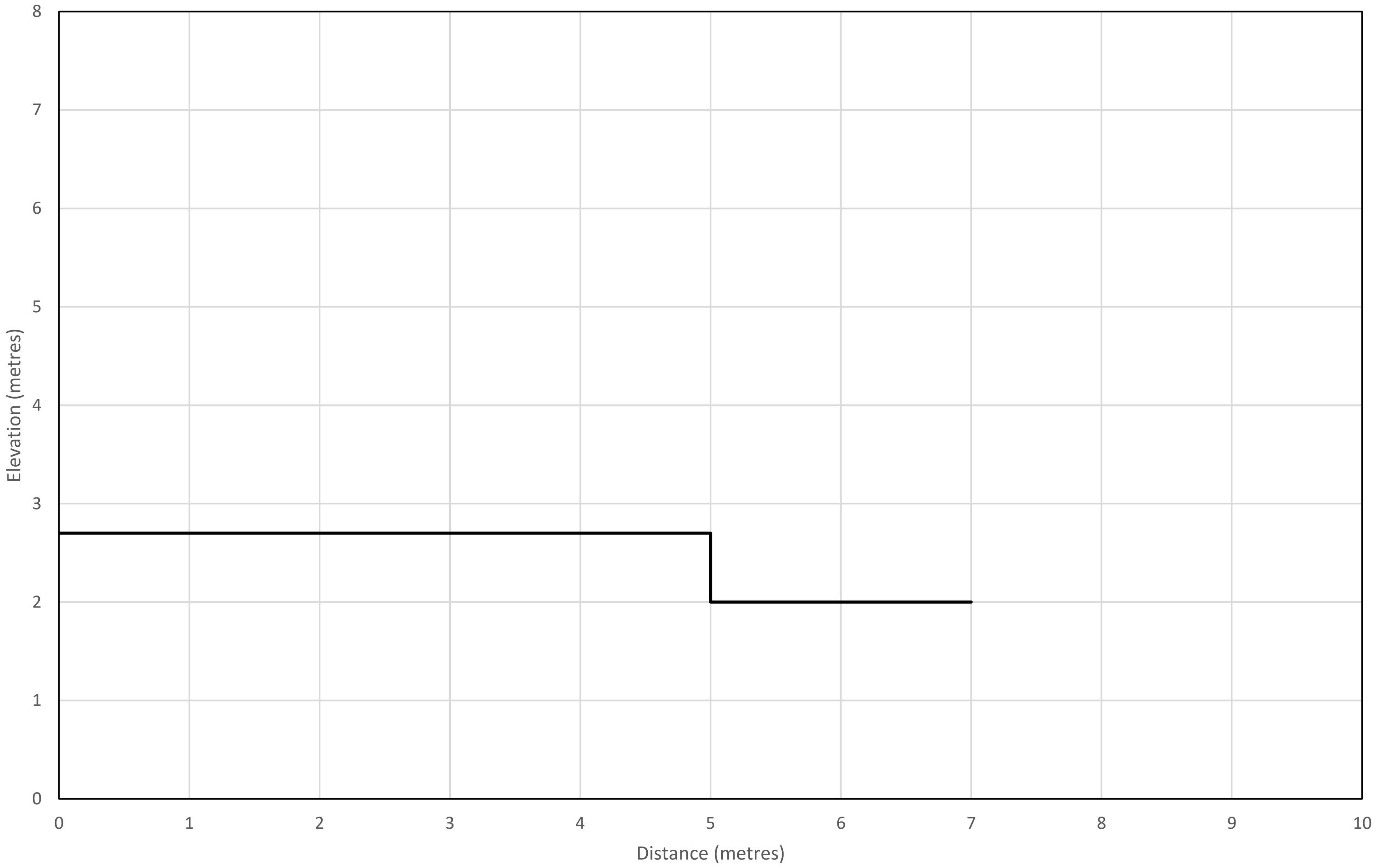
Sawmill Creek - Cross Section B-B

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D2



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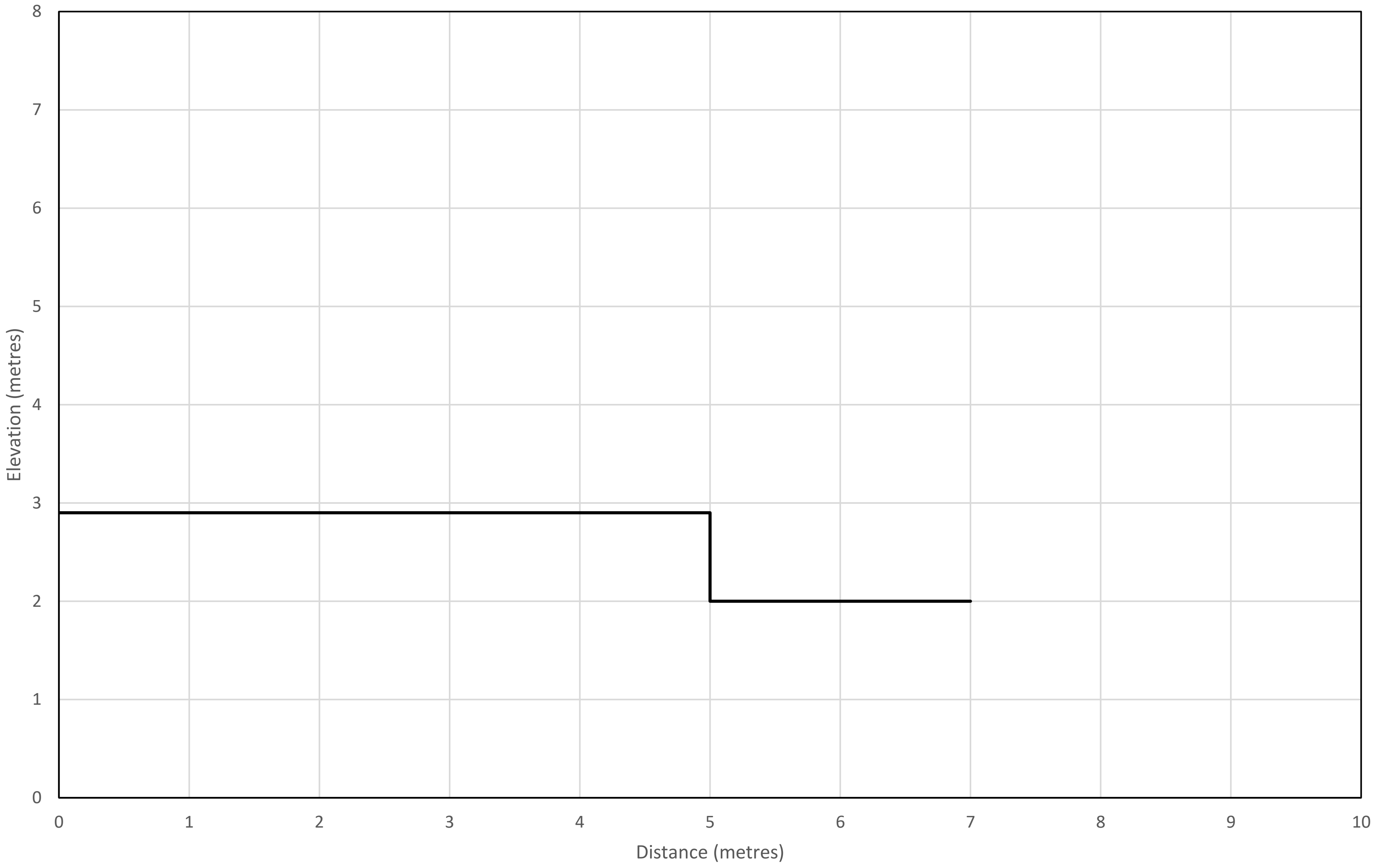
Sawmill Creek - Cross Section C-C

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D3



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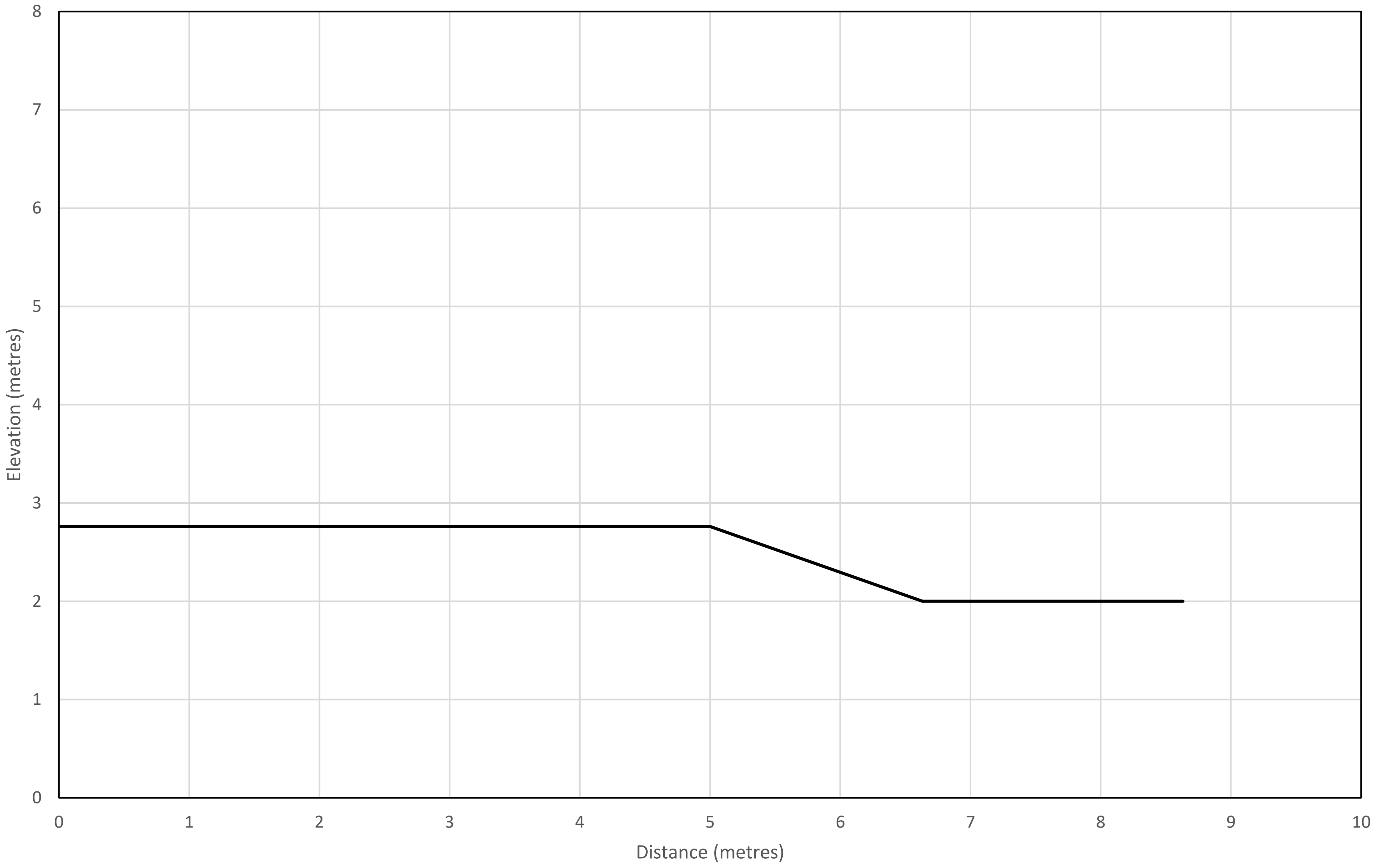
Sawmill Creek - Cross Section D-D

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D4



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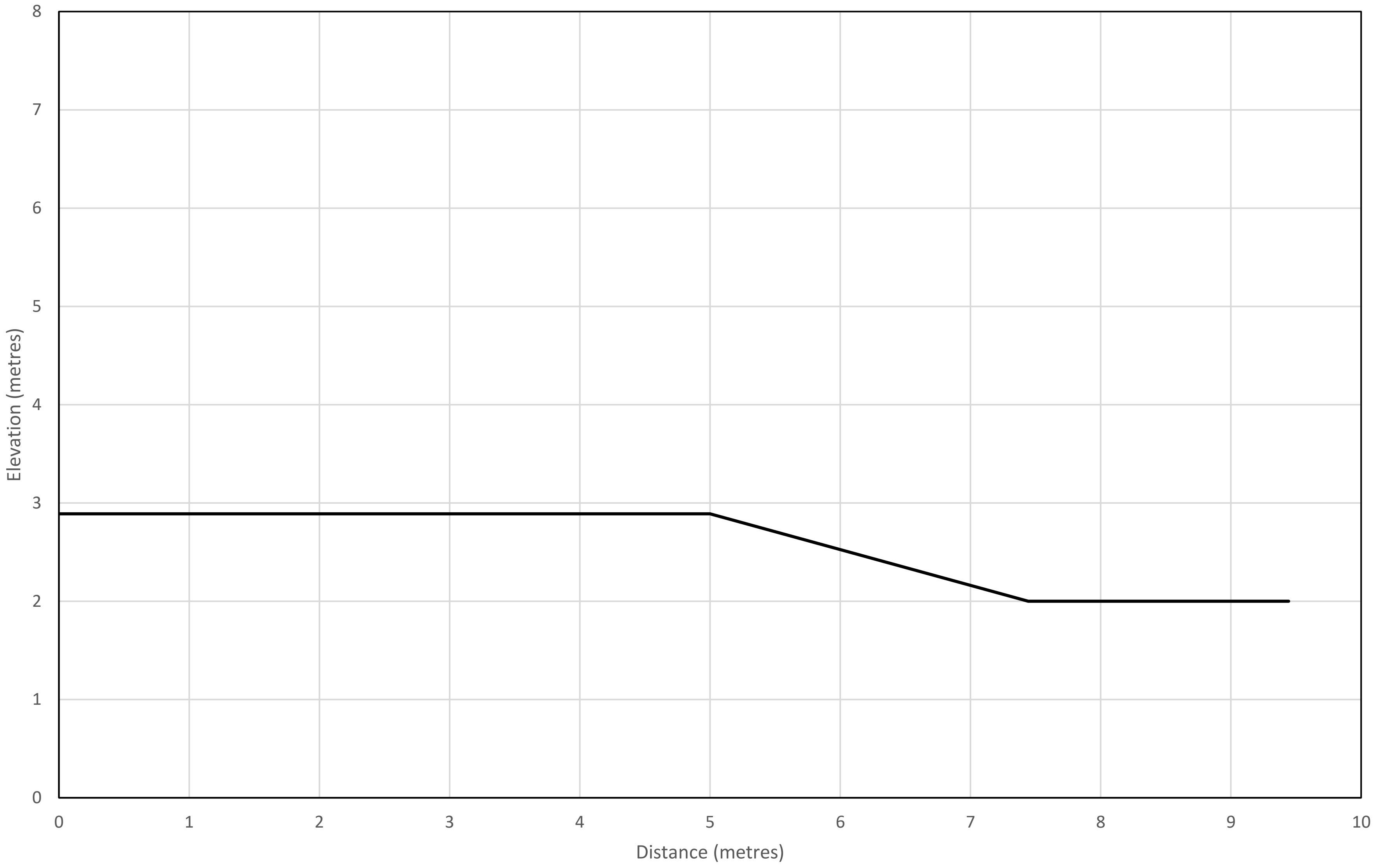
Sawmill Creek - Cross Section E-E

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D5



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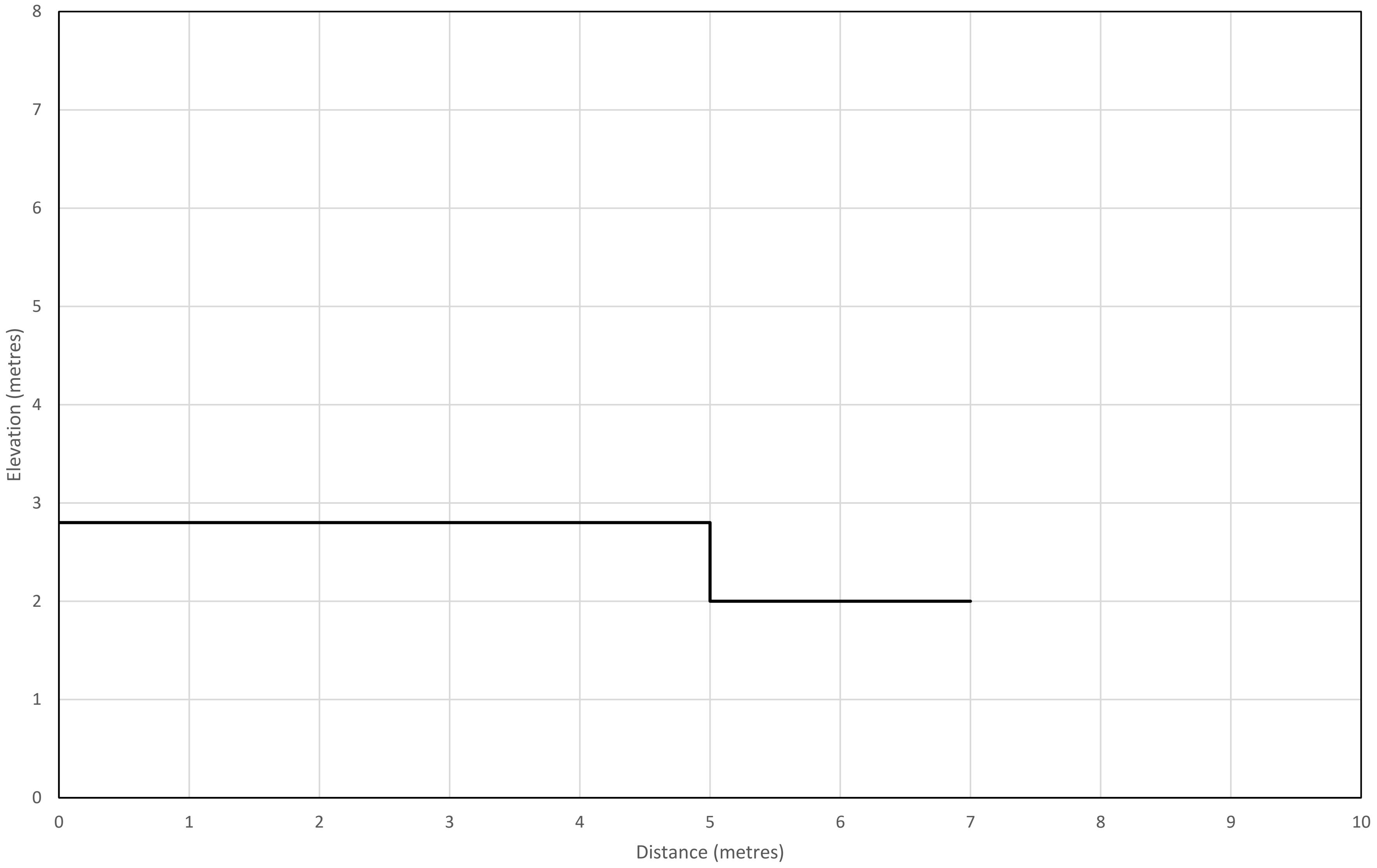
Sawmill Creek - Cross Section F-F

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D6



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Sawmill Creek - Cross Section G-G

Industrial Development, 1319 Johnston Road

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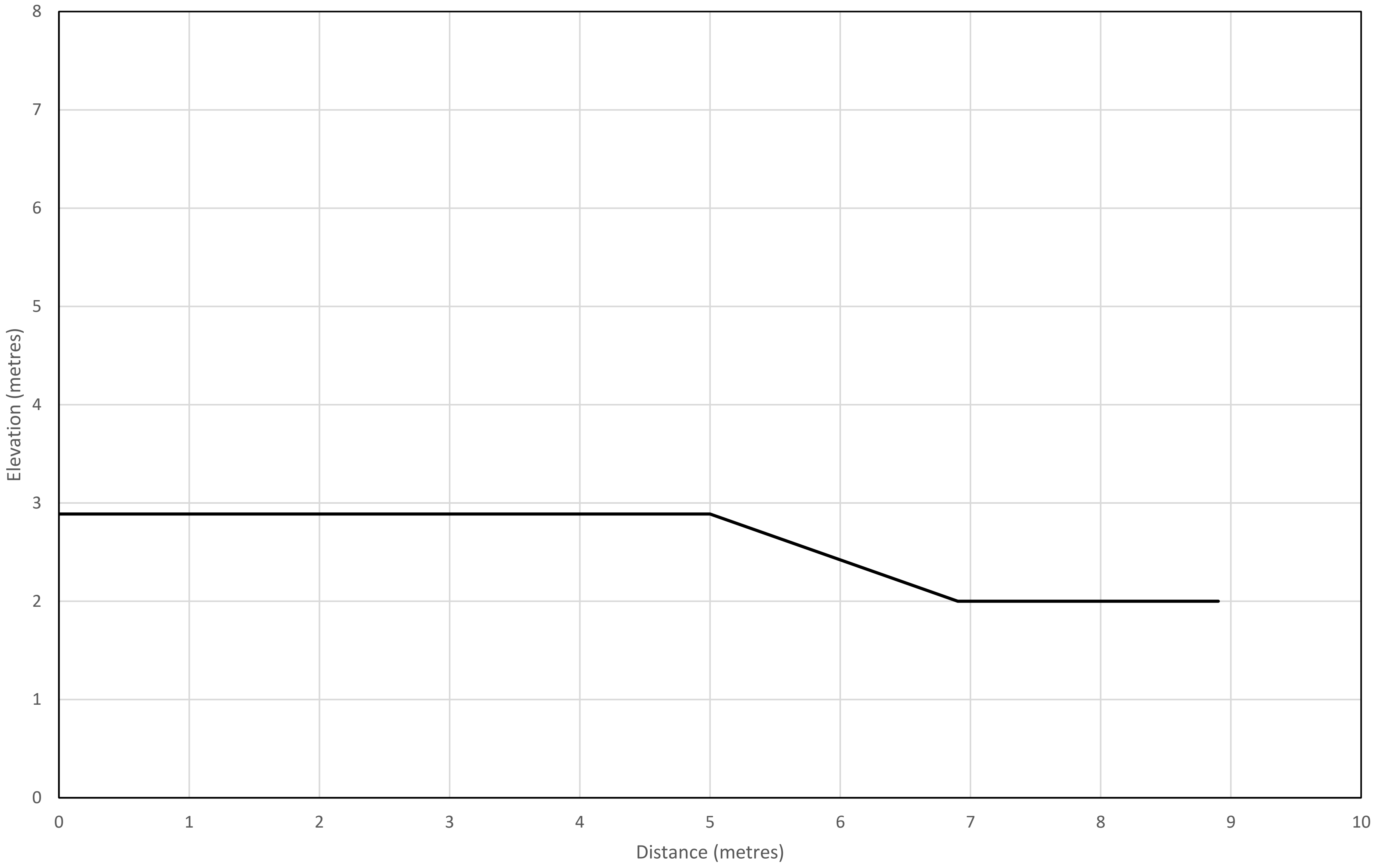
Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D7



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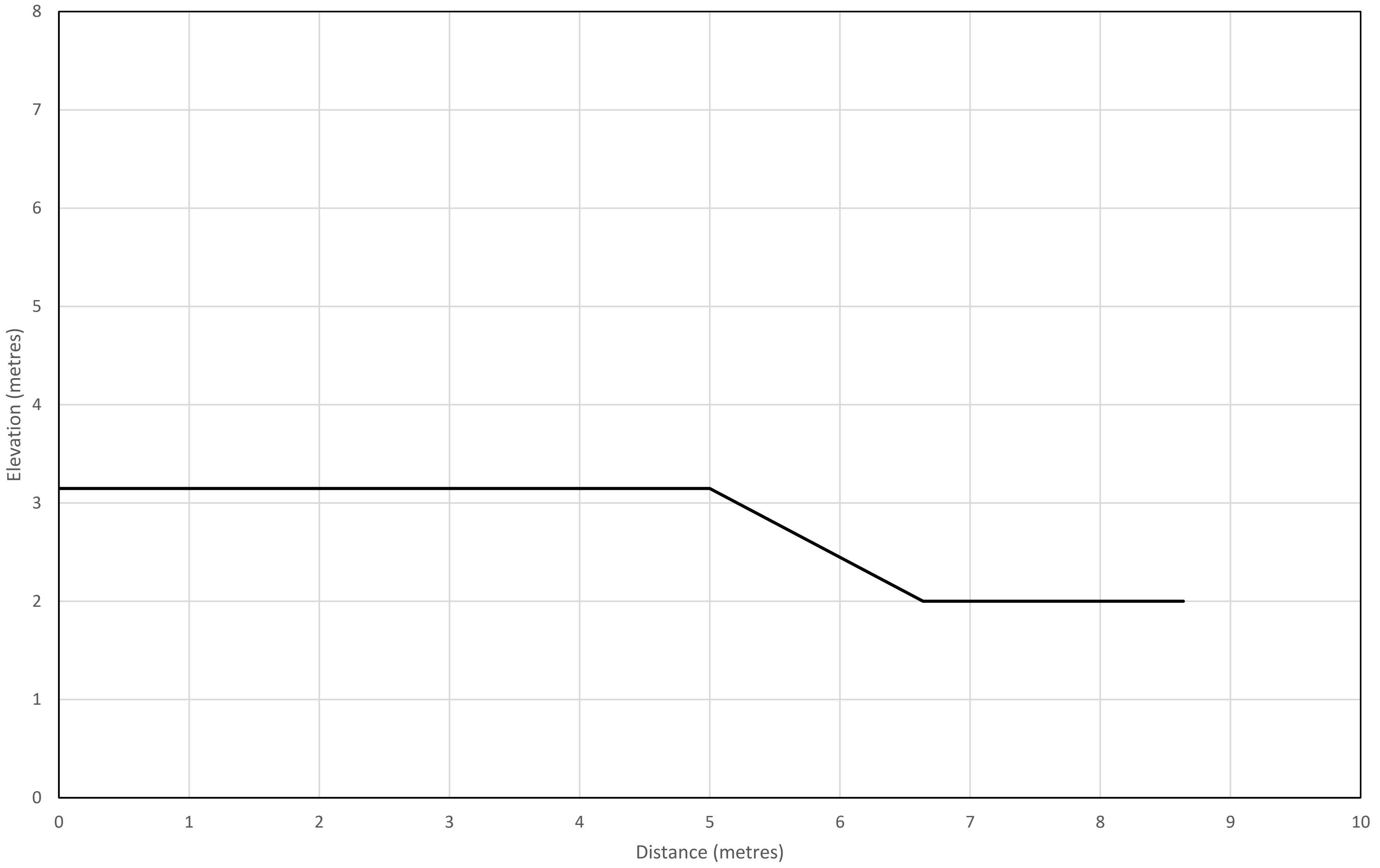
Existing Ditch - Cross Section H-H

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D8



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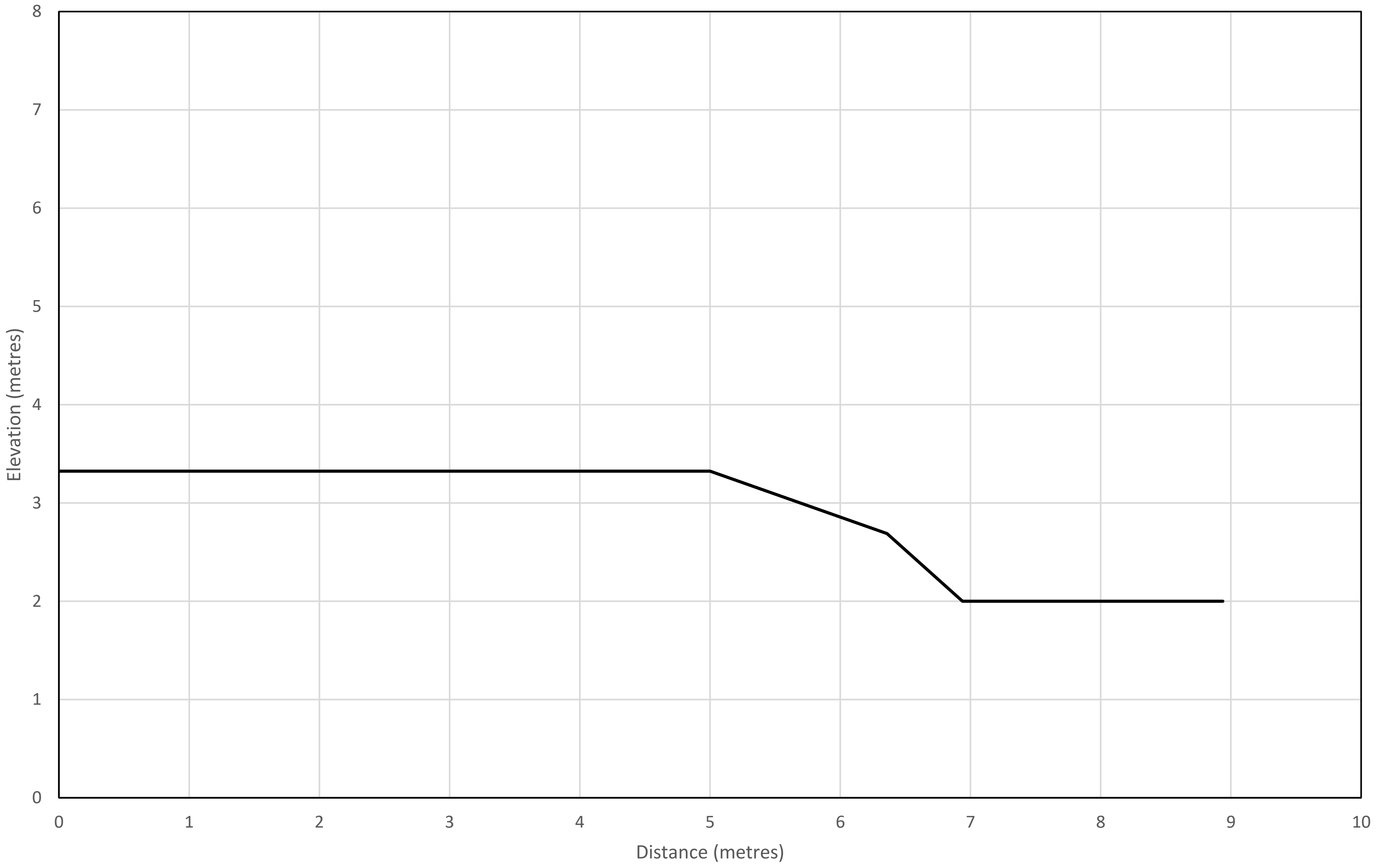
Existing Ditch - Cross Section I-I

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D9



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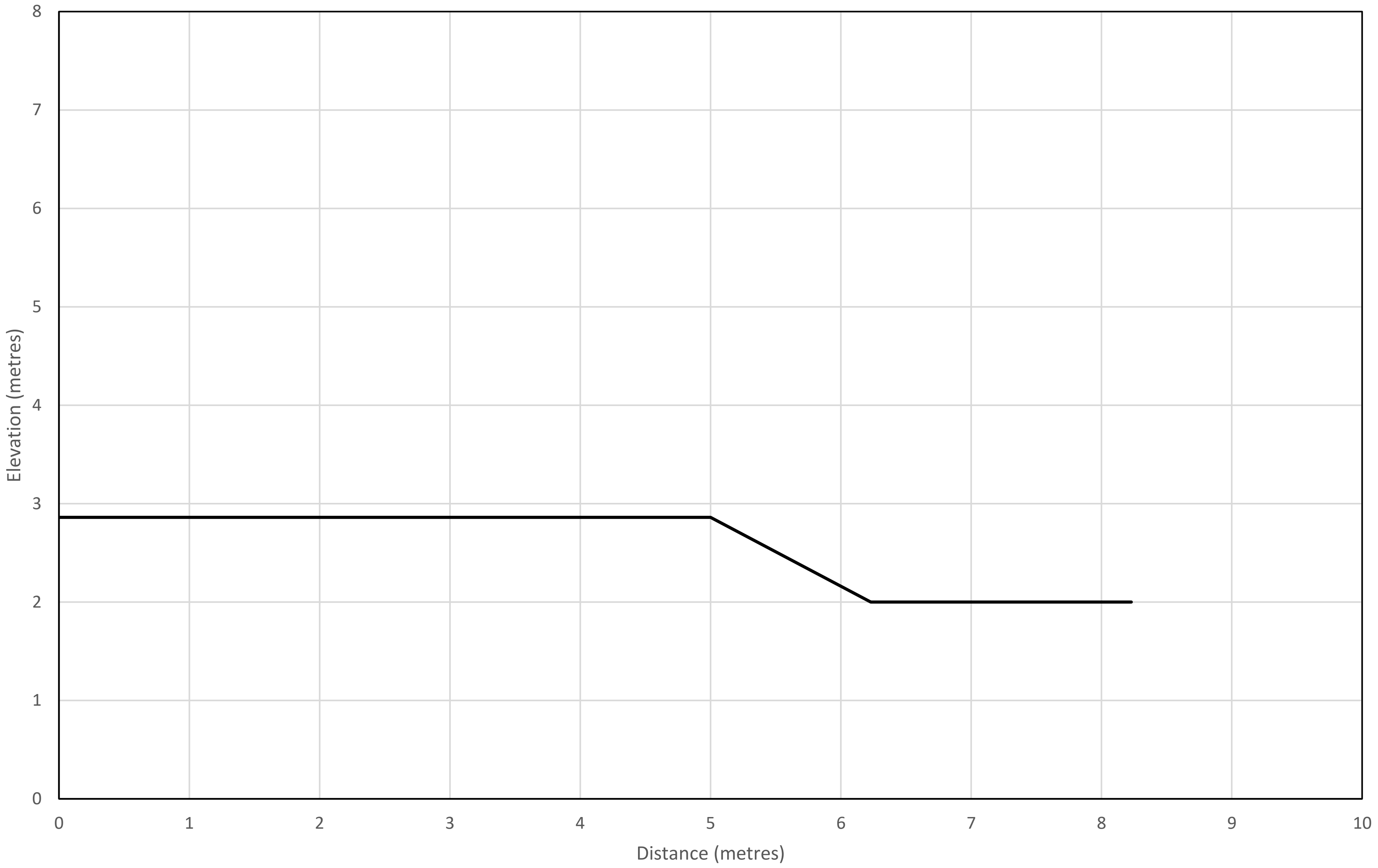
Existing Ditch - Cross Section J-J

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D10



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Existing Ditch - Cross Section K-K

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

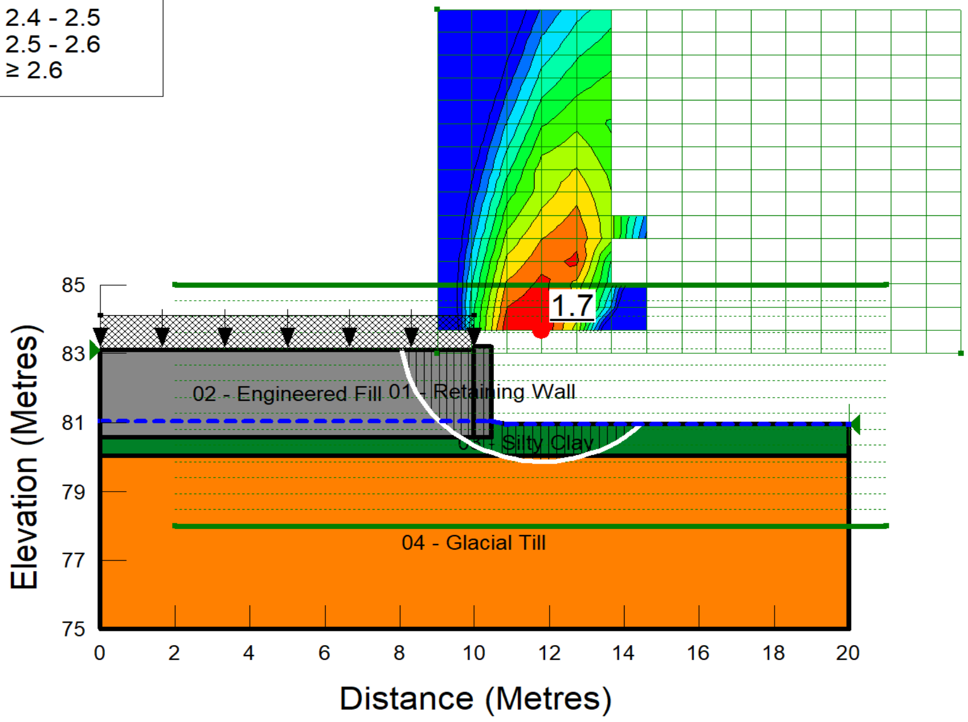
Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D11

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
■	01 - Retaining Wall	Mohr-Coulomb	24	500	90	0	1
■	02 - Engineered Fill	Mohr-Coulomb	22	0	34	0	1
🟩	03 - Silty Clay	Mohr-Coulomb	18	5	35	0	1
🟧	04 - Glacial Till	Mohr-Coulomb	22	0	36	0	1

Factor of Safety

🟥	1.7 - 1.8
🟧	1.8 - 1.9
🟨	1.9 - 2.0
🟩	2.0 - 2.1
🟩	2.1 - 2.2
🟩	2.2 - 2.3
🟩	2.3 - 2.4
🟦	2.4 - 2.5
🟦	2.5 - 2.6
🟦	≥ 2.6



Name: Slope Stability Static
Method: Morgenstern-Price
Direction of movement: Left to Right
Surcharge (Unit Weight): 12 kN/m³
Horz Seismic Coef.: 0

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Static Slope Assessment - Ret. Wall at Bldg A

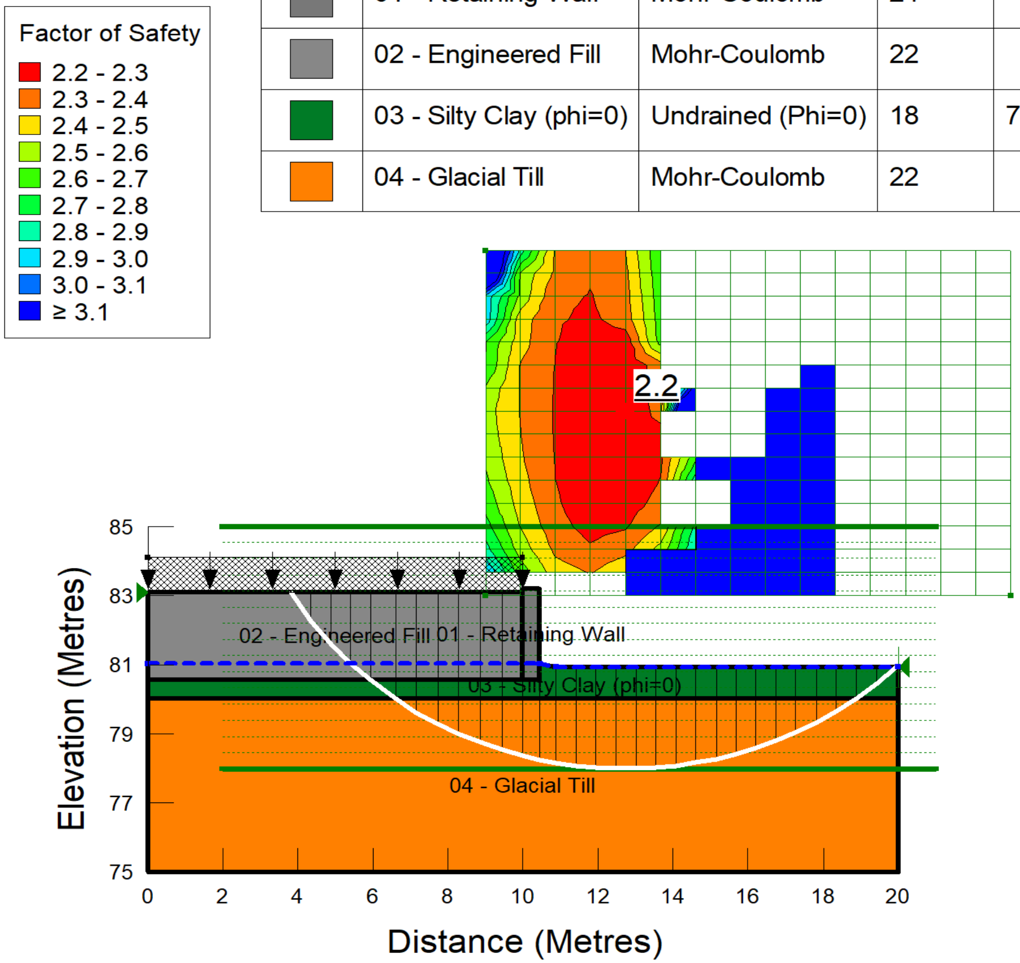
Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D12

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
⬛	01 - Retaining Wall	Mohr-Coulomb	24		500	90	0	1
⬛	02 - Engineered Fill	Mohr-Coulomb	22		0	34	0	1
🟩	03 - Silty Clay (phi=0)	Undrained (Phi=0)	18	75				1
🟧	04 - Glacial Till	Mohr-Coulomb	22		0	36	0	1



Name: Slope Stability Seismic
Method: Morgenstern-Price
Direction of movement: Left to Right
Surcharge (Unit Weight): 12 kN/m³
Horz Seismic Coef.: 0.153

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Seismic Slope Assessment - Ret. Wall at Bldg A

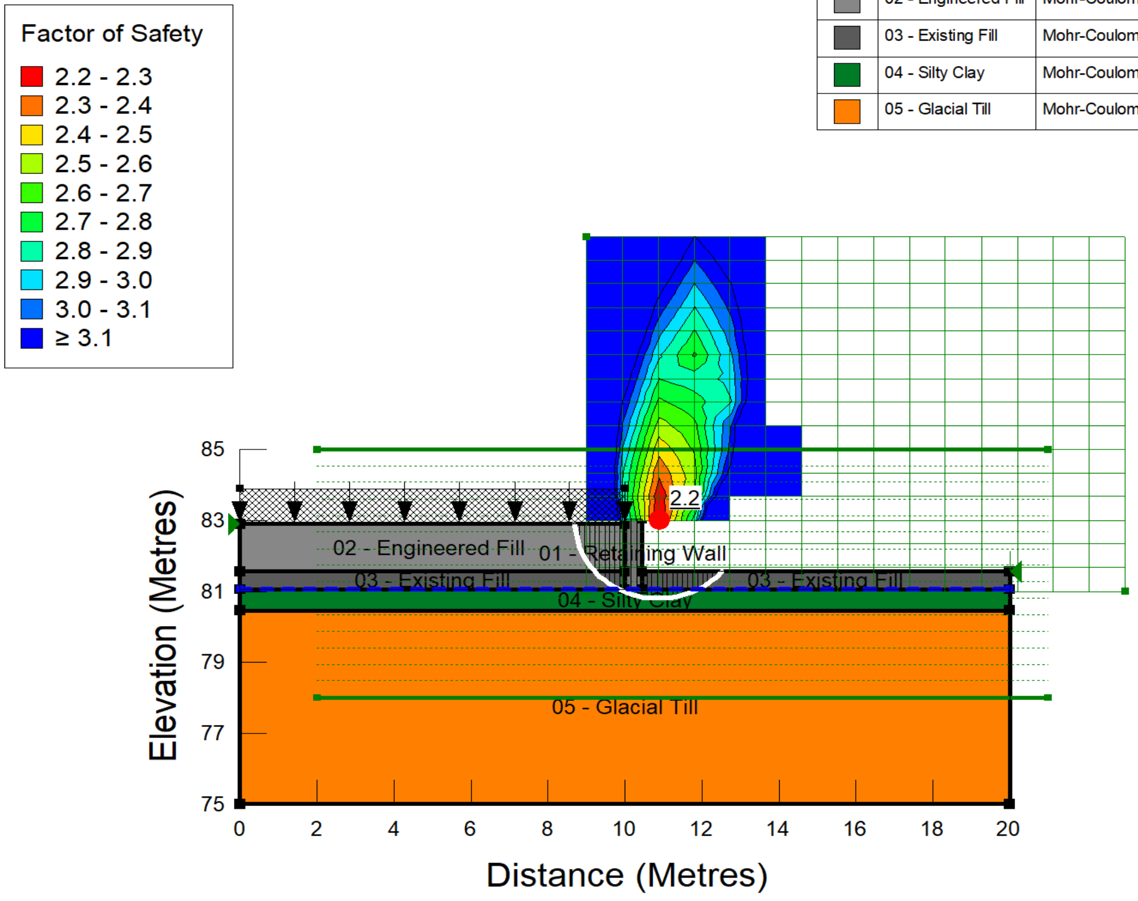
Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D13

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
◼	01 - Retaining Wall	Mohr-Coulomb	24	500	90	0	1
◼	02 - Engineered Fill	Mohr-Coulomb	22	0	34	0	1
◼	03 - Existing Fill	Mohr-Coulomb	20	0	32	0	1
🟩	04 - Silty Clay	Mohr-Coulomb	18	5	35	0	1
🟧	05 - Glacial Till	Mohr-Coulomb	22	0	36	0	1



Name: Slope Stability Static
Method: Morgenstern-Price
Direction of movement: Left to Right
Surcharge (Unit Weight): 12 kN/m³
Horz Seismic Coef.: 0

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Consulting Engineers and Scientists

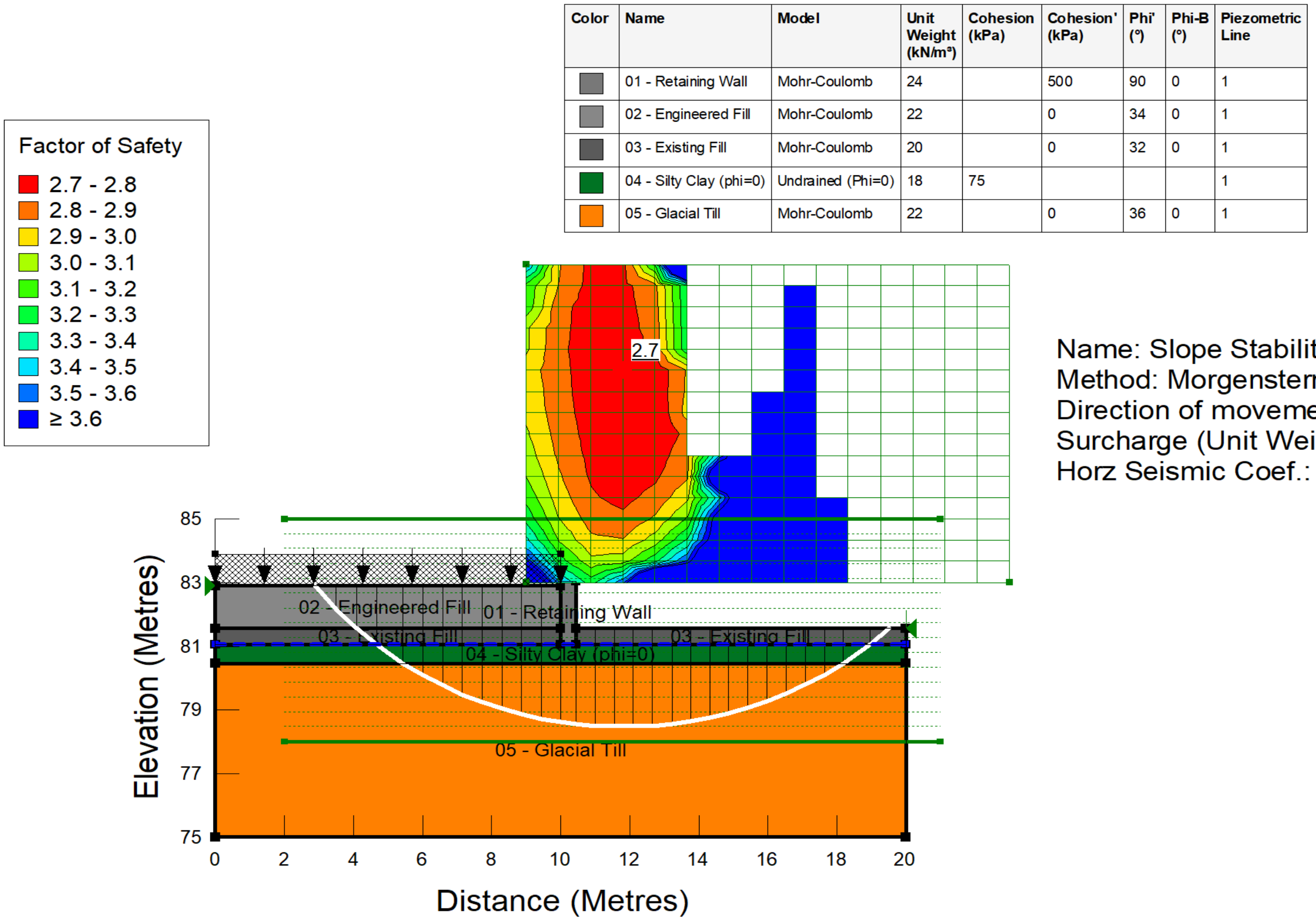
Static Slope Assessment - Ret. Wall at Bldg F, G, and H

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D14



Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
■	01 - Retaining Wall	Mohr-Coulomb	24		500	90	0	1
■	02 - Engineered Fill	Mohr-Coulomb	22		0	34	0	1
■	03 - Existing Fill	Mohr-Coulomb	20		0	32	0	1
🟩	04 - Silty Clay (phi=0)	Undrained (Phi=0)	18	75				1
🟧	05 - Glacial Till	Mohr-Coulomb	22		0	36	0	1

Name: Slope Stability Seismic
Method: Morgenstern-Price
Direction of movement: Left to Right
Surcharge (Unit Weight): 12 kN/m³
Horz Seismic Coef.: 0.153

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Seismic Slope Assessment - Ret. Wall at Bldg F, G, and H

Industrial Development, 1319 Johnston Road

Ottawa, Ontario

Project No.	101481.008
Drawn:	WAM
Date:	09/05/2024

Figure D15

experience • knowledge • integrity

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

expérience • connaissance • intégrité