REPORT

Geotechnical Investigation

Proposed Building Expansion
Holiday Inn Express and Suites Hotel, Ottawa, Ontario

Submitted to:

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1.0 INTRODUCTION

WSP Canada Inc. (WSP) was retained by Manga Hotels (Nepean) LP c/o Kingslake Projects Inc. (Manga Hotels) to carry out a geotechnical investigation in support of a proposed building expansion of the Holiday Inn Express and Suites Hotel (Holiday Inn Hotel), located at 2055 Robertson Road in Ottawa, Ontario. The investigation and reporting were carried out in general accordance with WSP's proposal No. 2025CA390145, dated July 7, 2025.

The purpose of this investigation was to assess the general subsurface and groundwater conditions within the study area by means of three boreholes and four auger-holes and associated laboratory testing. The subsurface conditions encountered in the current investigation and available project details were used to prepare recommendations related to design aspects of the project, including construction considerations which could influence design decisions. A slope stability assessment was also conducted for the creek slope adjacent to the site to evaluate the (geotechnical) limit of hazard lands.

The reader is referred to the "*Important Information and Limitations of This Report*", which follows the text but forms an integral part of this document.

2.0 SITE AND PROJECT DESCRIPTIONS

Plans are being prepared for the expansion of the Holiday Inn Hotel, located at 2055 Robertson Road in Ottawa, Ontario (see Borehole Location Plan, Figure 1). The site is bounded to the north by an un-named creek, to the south by Robertson Road, and to the east and west by commercial developments.

Prior to the existing hotel development a watercourse crossed the subject site within an approximately 10 m deep valley located along the western site boundary. Within the southern and central portions of the site the creek was routed through a box culvert, and the valley was infilled. The creek valley remains open within the northern portion of the site.

It is understood that the project will include an addition to the north side of the existing hotel building and will include a new six-storey wing containing an additional 30 hotel units. The expansion will increase the building footprint by approximately 220 m². The proposed expansion will not include the addition of new parking spaces; however, it is understood that the existing exterior parking lot will be reconstructed and repaved (in approximately its current configuration) as part of the overall project.

The overall project site, as well as the location of the new addition are shown on Figure 1.

3.0 DESKTOP REVIEW

Golder Associates Ltd. (now a member of WSP Canada Inc.) previously carried out investigations at this site. Relevant previous investigations carried out at this site include:

- Golder Report No. 08-1121-0045 titled: "Geotechnical Investigation, Proposed Holiday Inn Express and Suits Hotel, 45 Robertson Road, Ottawa, Ontario", dated May 2008.
- Golder Technical memorandum No. 08-1121-078 titled: "Shear Wave Velocity Profiling, Holiday Inn Express, 45 Robertson Road, Ottawa, Ontario", dated December 23, 2008.
- Golder Report No. 772037 titled: "Soil Investigation, Proposed Burger King Restaurant and Retail Store, Bells Corners, Ontario", dated April 1977.

The borehole and auger-hole records from the previous investigations are provided in Appendix C, and the corresponding locations of boreholes and auger-holes are also shown on Figure 1.

Based on previous investigations, the subsurface conditions at the site are anticipated to comprise varying thickness of fill material overlying sensitive silty clay, with the inferred bedrock surface varying from about 2 m below existing ground surface (mbgs) within the southern portion of the site to about 7.5 mbgs within the remainder of the site (including the area where the new addition will be located).

Published geological maps indicate that the bedrock at the site consists of sandstone of the Nepean Formation.

4.0 METHOD OF INVESTIGATION

4.1 Field Investigation

The field work for the current geotechnical investigation was carried out on August 28 and August 29, 2025 and included advancing a total of three boreholes (BH25-01 to BH25-03), and four auger-holes (AH25-04 to AH25-07). The approximate borehole and auger-hole locations are shown on Figure 1.

The boreholes and auger-holes were advanced using a CME 11 truck-mounted drill rig supplied and operated by George Downing Estate Drilling Ltd. of Hawkesbury, Ontario. Standard Penetration Tests (SPTs) were carried out in the boreholes at regular depth intervals where possible (typically 0.75 m intervals). In-situ vane testing was carried out within cohesive soil strata encountered in boreholes BH25-01 and BH25-02. Soil samples from boreholes were obtained using a 50 mm outside diameter split-spoon sampler in general accordance with ASTM D1586. Grab soil samples were also collected from auger cuttings.

Boreholes BH25-01 and BH25-02 were advanced to refusal depths of 7.8 m and 7.7 m, respectively. Borehole BH25-01 was extended into the bedrock using rotary diamond drilling technique while retrieving NQ-sized core samples. Borehole BH25-03 was terminated at a depth of 5.2 mbgs. Auger-holes AH25-04 to AH25-07 were advanced to 1.5 m depth below ground surface to provide information for parking lot design and reconstruction.

Monitoring wells were installed in boreholes BH25-01 and BH25-02 to allow for subsequent measurements of stabilized groundwater levels in the soil overburden and bedrock. The monitoring wells consisted of 51 mm outside diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill, and sealed by a section of bentonite hole plug.

Fieldwork was supervised by WSP’s geotechnical staff who logged the boreholes and auger-holes, directed the in-situ testing, and collected the soil and rock samples retrieved during drilling. On completion of the drilling operations, the soil and rock samples were transported to WSP’s Ottawa laboratory for further examination by the project engineer and for possible laboratory testing.

The borehole locations were selected in consultation with Manga Hotels, marked in the field and subsequently surveyed by WSP. The borehole coordinates and existing ground surface elevations were measured using a Trimble R10 GPS survey unit. The geodetic reference system used for the survey is the North American Datum of 1983 (NAD83-CSRS). The borehole coordinates are based on the Universal Transverse Mercator (UTM Zone 18) coordinate system.

4.2 Laboratory Testing

Laboratory testing was completed on selected soil samples. Tests included natural water content, grain size distribution and Atterberg limits on selected soil samples, and Uniaxial Compressive Strength (UCS) on one bedrock core specimen. Two samples of soil were also submitted to Eurofins Environment Testing for chemical analysis related to potential sulphate attack on buried concrete elements, and potential corrosion of buried ferrous elements.

5.0 SUBSURFACE CONDITIONS

5.1 General

The Record of Borehole and Augerhole sheets in Appendix A describe the subsurface conditions at the borehole and auger-hole locations only. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress as well as results of Standard Penetration Tests and, therefore, represent transitions between soil types rather than exact planes of geological change. Within the auger-holes, the subsurface conditions and approximate depths to strata changes were visually logged at the time of drilling, and by further examinations of auger cuttings. Subsurface soil and groundwater conditions will vary between and beyond the borehole and auger-hole locations.

5.2 Summary of Subsurface Stratigraphy

Based on the results of the borehole investigation, the general subsurface stratigraphy within the area of the investigation comprises asphaltic concrete laid on granular base / subbase materials, overlying variable clayey sand and silty clay fill, and then bedrock.

Further descriptions of the soil layers are provided in the subsections below.

5.2.1.1 Pavement Structure

Asphaltic concrete was present at all borehole and auger-hole locations. The thickness of the asphaltic concrete and the pavement sub-structure are provided as follows.

Table 1: Existing Pavement Structure

Borehole / Augerhole No.	Asphalt Thickness (mm)	Granular Base & Subbase Thickness (mm)	Total Thickness (mm)
BH25-01	125	630	755
BH25-02	50	710	760
BH25-03	50	710	760
AH25-04	70	840	910
AH25-05	50	960	1010
AH25-06	100	1200	1300
AH25-07	100	910	1010

It should be noted that it was difficult to distinguish between granular base and subbase material at the time of investigation, as these layers are similar sandy and gravelly materials. Laboratory-measured water content of two selected samples of granular base/ subbase material was 2 % and 6 %.

5.2.1.2 Existing Fill

5.2.1.2.1 Sandy Fill

Fill consisting of gravelly silty sand, silty sand, and gravelly clayey sand was encountered below the pavement structure at boreholes BH25-01, BH25-02 and auger-holes AH25-05 to AH25-07. The fill also contains varying amounts of plastic fines and potential cobbles and boulders. Fill material extended to depths of about 1.37 m (~El. 88.1 m to 88.9 m) at boreholes BH25-01 and BH25-02. Auger-holes AH25-05 to AH25-07 were terminated in sandy fill at 1.52 m (~El. 88.2 m to 89.3 m). The thickness of the sandy fill where fully penetrated was about

0.6 m. SPT ‘N’ values in the sandy fill ranged from 6 and 14 (blows per 0.3 m of penetration) indicating a loose to compact state of packing.

The water content in four selected sample of sandy fill ranged from 10% to 12% based on laboratory tests. Grain size distribution tests carried out on two samples of the sandy fill layer are presented in Figure D1 in Appendix D. The results of the Atterberg limit testing completed on a single sample (of the fines portion) of gravelly clayey sand fill indicate liquid limit of 25, plastic limit of 13, and plasticity index of 12. The Atterberg limit testing results are shown in Figure D2 in Appendix D and indicate the fines fraction of this material is of low plasticity (CL).

5.2.1.2.2 Clayey Fill

Fill consisting of silty clay, sandy silty clay, and silty clay and sand was encountered below the sandy fill in boreholes BH25-01 and BH25-02, and below the pavement structure in borehole BH25-03 and auger-hole AH25-04. Clayey fill contains varying amounts of gravel, wood debris, cobbles and boulders, and extended to depths of about 2.3 mbgs and 2.9 mbgs (~El. 86.4 m to 88.0 m) in boreholes BH25-01 to BH25-03. Auger-hole AH25-04 was terminated within the fill layer at a depth of 1.52 mbgs (~El. 88.0 m). The thickness of the clayey fill layer where fully penetrated ranged from about 0.9 m to 2.1 m. SPT ‘N’ values in the clayey fill typically ranged from 4 to 8 (blows per 0.3 m of penetration) indicating firm to stiff consistency. Very high SPT ‘N’ values (i.e., 104 blows / 0.15 m) can be the result of presence of cobbles and boulders within the fill deposit, rather than the consistency of the soil matrix.

The water content measured in three selected samples of clayey fill ranged from 14% to 28% based on laboratory tests. The results of grain size distribution testing carried out on a single sample of the clayey fill are presented in Figure D1 in Appendix D. The results of the Atterberg limits testing completed on a single sample of the fill material indicate Liquid Limit of 40, Plastic Limit of 15, and Plasticity Index of 25. The Atterberg limits testing results are shown in Figure D2 in Appendix D and indicate the fines fraction of this material is clay of intermediate plasticity (CI).

5.2.1.3 Clayey Sand

A clayey sand deposit with varying amounts of gravel was encountered below the silty clay fill layer at borehole BH25-01. Based on examination of soil samples retrieved from the borehole, there is a possibility that this material was used for valley infill on the site. The clayey sand layer extended to a depth of about 4.6 m (~El. 85.7 m) and had a thickness of about 2.3 m. SPT ‘N’ values ranged from 2 to 10 (blows per 0.3 m of penetration), indicating a very loose to loose state of packing.

The water content measured in two samples of clayey sand was 23% and 30% based on laboratory tests. The results of grain size distribution test carried out on a single sample of this clayey sand are presented in Figure D3 in Appendix D.

5.2.1.4 Silty Clay

Silty clay was encountered below the clayey sand layer at borehole BH25-01, and below the silty clay fill at boreholes BH25-02 and BH25-03. The silty clay also contains varying amounts of sand, gravel, organics, asphalt fragments, and possibly concrete fragments. Based on the examination of soil samples retrieved from boreholes, there is a possibility that this layer is infill material. The silty clay extended to a depth of about 7.8 mbgs (~El. 82.5 m) at borehole BH25-01. Boreholes BH25-02 and BH25-03 were terminated within this material at depths of about 5.2 mbgs to 7.7 mbgs (~ El. 82.4 m to 84.1 m). The thickness of the layer was about 3.2 m where fully penetrated. SPT ‘N’ values ranged from Weight of Hammer (WH) to 5 (blows per 0.3 m of penetration).

In-situ vane testing carried out within the silty clay measured undrained shear strengths ranging from 46 kPa to 96 kPa, which indicates firm to stiff consistency.

The water content measured on seven samples of silty clay ranged from 23 % and 34 % based on laboratory tests. The results of grain size distribution test carried out on two samples of this silty clay are presented in Figure D4 in Appendix D. The results of the Atterberg limit testing completed on five samples of this silty clay material indicate liquid limit ranged from 32 to 35, plastic limit 15 to 18, and plasticity index of 16 to 20. The Atterberg limit testing results are shown in Figure D5 in Appendix D and indicate the fines fraction of this material is clay of intermediate plasticity (CI).

At the location of borehole BH25-02, a 0.14 m thick layer of silt with organics was encountered within the silty clay deposit. The water content measured on a single sample of this material was 45%.

5.2.1.5 Auger Refusal and Bedrock

Refusal to augering was encountered in boreholes BH25-01 and BH25-02 at a depth of 7.8 m (~El. 82.7 m) and 7.7 m (~ El. 82.4 m), respectively.

Borehole BH25-01 was extended through the silty clay deposit into the underlying bedrock using a rotary diamond drilling technique. Recovered bedrock core samples were described as relatively fresh, laminated to medium bedded, grey and black, sandstone bedrock.

The Total Core Recovery (TCR) of bedrock was 100% and the Rock Quality Designation (RQD) was between 78% and 100%, indicating a Good to Excellent quality rock at the location cored.

The result of a UCS test carried out on a single core specimen of the bedrock was 201 MPa, which indicates a very strong rock. The result of the UCS test is provided in Figure D6 in Appendix D. Photographs of the recovered bedrock cores are presented in Appendix B. It is advised that a single UCS test and limited core sampling in one borehole may provide a preliminary indication of bedrock properties for routine foundation design but may not be sufficient to fully assess requirements for construction procedures; contractors should not rely on the limited testing presented above for construction tendering or subsequent claims.

Practical refusal to augering was encountered at depths ranging from 2.1 m to 7.6 m in historic boreholes. It Is not known whether the shallow refusal depths (2.1 m to 2.8 m) encountered in boreholes advanced within southern portion of site (within the footprint of existing hotel building) indicate shallow bedrock, or if the refusal was on cobbles, boulders or similar obstructions (e.g., concrete debris).

The following Table 2 summarizes the refusal or top of bedrock depths and elevations for the current investigation and the previously drilled boreholes.

Table 2: Summary of Refusal or Top of Bedrock Depths

Investigation	Datum	Borehole No.	Ground Surface Elevation (m)	Refusal / Top of Bedrock Depth (m)	Core Length (m)	Refusal / Top of Bedrock Elevation (m)
Current Investigation	Geodetic	25-01	90.27	7.79^{T}	3.13	82.67^{T}
		25-02	90.13	7.71	-	82.42
Previous Investigation (Report No. 08-1121-0045)	Geodetic	08-1A	91.17	2.80^{R}	-	88.37^{R}
		08-1B	91.17	2.13^{R}	-	89.04^{R}
		08-1C	91.17	2.59^{R}	-	88.58^{R}

Investigation	Datum	Borehole No.	Ground Surface Elevation (m)	Refusal / Top of Bedrock Depth (m)	Core Length (m)	Refusal / Top of Bedrock Elevation (m)
Previous Investigation (Report No. 772037)	Local	1	99.7	7.62	-	-
		2	99.3	7.62	-	-
		3	99.8	7.47	-	-
		4	99.0	7.22	-	-

Note: “T” denotes confirmed (by coring) bedrock surface. “R” denotes shallow refusal on presumed cobbles, boulders or other unknown obstructions.

5.3 Groundwater Conditions

The groundwater level in monitoring wells installed at the site was measured on September 18 and September 24, 2025. Results are summarized in Table 3 below.

Table 3: Summary of Groundwater Level

Monitoring Well No.	Geologic Unit	Date	Ground Water Level Depth (m)	Ground Water Level Elevation (m)
BH25-01	Bedrock	September 18, 2025	5.45	84.82
		September 24, 2025	5.09	85.18
BH25-02	Sandy silty clay	September 24, 2025	3.33	86.80

Groundwater levels are expected to fluctuate seasonally and over shorter periods of time. Higher groundwater levels are expected during relatively wet periods of the year, such as spring, after snowmelt events or during periods of heavy rain for example.

5.4 Corrosion Testing

Two samples of soil from boreholes BH25-01 and BH25-02 were submitted to Eurofins Environment Testing for chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The test results are provided in Appendix E and are summarized in Table 4 below.

Table 4: Summary of Corrosion Testing Results

Borehole Number	Sample Number	Depth Intervals (m)	Chlorides (%)	Sulphates (%)	pH	Resistivity (Ohm-cm)
BH25-01	7	3.81 - 4.42	0.035	0.01	7.59	1721
BH25-02	4	2.29 - 2.89	0.308	0.02	7.46	247

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the proposed building expansion based on our interpretation of the borehole information and project requirements.

The information in this portion of the report is provided for planning and design purposes for the guidance of the design engineers and architects. Where comments are made on construction, they are provided only to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities, costs, sequencing and the like.

6.2 Seismic Design

The seismic hazard values associated with the design earthquakes are those established for the National Building Code of Canada (NBC 2020) by the Geological Survey of Canada (GSC). The current seismic hazard maps (referred to as the 6th generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2020.

6.2.1 Seismic Site Classification

The seismic design provisions of the latest Ontario Building Code (OBC) depend, in part, on the shear wave velocity of the upper 30 m of soil and/or rock below founding level.

A site-specific shear wave velocity profiling, using the Multichannel Analysis of Surface Waves (MASW) method, was carried out by Golder Associates Ltd. at the site on December 12, 2008. The results of this testing indicate that this site can be assigned a Site Class C.

6.3 Frost Penetration Depth

All exterior perimeter foundation elements or foundation elements in unheated areas should be provided with a minimum of 1.5 m of earth cover for frost protection purposes. Isolated, unheated exterior footings and/or footings/ pile caps adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 m of earth cover.

Consideration could also be given to insulating the bearing surface with high density Styrofoam insulation as an alternative to earth cover. If frost protection will be used, WSP can provide additional guidance based on the specific application and location.

6.4 Foundation

It is understood that the existing hotel building is supported on a deep foundation. The new addition should similarly be founded on a deep foundation system.

Shallow foundations are not suitable given the heterogeneous nature of the existing fill and the anticipated settlement behavior in the underlying clayey soils. With the shallow foundation option, there will be a differential settlement issue between the existing building and the expansion.

Both driven piles and drilled shafts (caissons) have been considered. Driven piles are commonly used in the area, and piles driven to bedrock will have a relatively high bearing resistance. Driven piles may also be somewhat cheaper to install. The relatively shallow depths to bedrock, however, combined with the existing fill material mean that driven piles will generate very little lateral and uplift resistance, and additional foundation elements (such as retaining walls or rock anchors may be required).

Drilled shafts may be slightly more expensive to install, but the rock sockets which form part of the caisson provide large uplift resistances as well as fixity to resist lateral loads.

Construction of drilled shafts may be challenged by excessive soil sloughing and water seepage which will need temporary steel liners as well as dewatering (e.g., bailing, pumping methods, etc.). Tremie procedures will be required for below water concrete placement, if required. The potential presence of cobbles and boulders within the existing soil deposit should also be considered in selecting the pile boring equipment.

It is likely be difficult to drive steel piles through the potential cobbles and boulders within the existing fill deposit. In addition, significant vibrations will generate during driving, which have potential effect on the existing building and the surrounding development. A vibration monitoring plan should be developed and put in place, if driven piles will be selected for foundation.

Based on the above, the construction feasibility of driven piles and drilled shafts should be confirmed by a qualified specialty contractor prior to selecting the foundation type for final design.

6.4.1 Drilled Shafts

6.4.1.1 Axial Capacity in Compression

Based on the uniaxial compressive strength of the bedrock at this site, and a minimum of 1.0 m long bedrock socket, an unfactored unit ultimate skin friction of 1.5 MPa and unfactored unit ultimate end bearing resistance of 25 MPa can be used in the design.

In accordance with CFEM (2023), geotechnical resistance factor of 0.4 should be applied to the calculated axial ULS values for piles in compression. Hence, the factored unit ultimate skin friction of 0.6 MPa and factored ultimate end bearing resistance of 10 MPa can be used in the design of caisson, socketed minimum of 1.0 m into the bedrock. The geotechnical resistance factor may be increased to 0.5 if a program of dynamic testing is carried out, or 0.6 if a static load test is carried out.

SLS does not apply to caissons end bearing on/in bedrock at this site, since the SLS resistance for 25 mm of settlement is greater than factored axial geotechnical resistance at ULS.

Any geotechnical resistance derived from end-bearing in the rock socket which is contingent on adequate cleaning of the pile bases, as described in Section 6.4.1.5. If the base of the caisson is not cleaned of all cuttings, mud, rock flour, slurry, or other material and is not inspected and confirmed by a qualified engineer, then only a shaft resistance design approach should be used for rock socket. Cleaning the base of piles can be difficult in narrow shafts and consideration should be given to ignoring end bearing for drilled shafts less than approximately 600 mm in diameter.

6.4.1.2 Axial Capacity in Tension

In the case of caissons socketed into sound bedrock, the uplift capacity is determined based on the shaft capacity of the socket.

Based on the uniaxial compressive strength of the bedrock at this site, and a minimum of 1.0 m long bedrock socket, an unfactored unit ultimate skin friction of 1.5 MPa can be used for the caisson capacity in tension.

In accordance with CFEM (2023), geotechnical resistance factor of 0.3 should be applied to the calculated axial ULS values for piles in tension. Hence, the factored unit ultimate capacity of 0.45 MPa can be used in the design of caisson in tension, socketed minimum of 1.0 m into the bedrock.

6.4.1.3 Negative Skin Friction

When the caissons have been installed in or through the soil deposit that is subject to settlement, the resulting relative downward movement of the clay around the piles, as well as in any soil above the clay layers, induces downdrag forces on the piles through negative skin friction.

Downdrag increases the structural loads in the pile and thus should be accounted for when evaluating the structural ultimate limit state of the pile. Downdrag can theoretically increase the settlement of the pile head, however, since the piles are socketed in bedrock (which will not settle significantly) this is not a concern for this particular project. Downdrag has no effect on the ULS geotechnical axial capacity of the pile.

As per Section 9.2.5 of CFEM (2023), the total drag load can be calculated by multiplying the unit ultimate skin friction values by shaft circumference or pile perimeter length and by the length of the pile embedded in settling soil.

In this case, unit ultimate (negative) skin friction of 40 kPa can be used in the design. For this site, the length of pile within the settling soil is taken as the length within the entire overburden soil for conservatism.

6.4.1.4 Lateral Capacity

For drilled shafts socketed into competent bedrock, a fixed condition at the shaft toe may be assumed, provided the socket length and rock properties are sufficient to ensure fixity. This condition (i.e., fixity at the rock socket) may be assumed to be met if the length of the rock socket is the greater of: at least two times the diameter of the drilled shaft, or 1 m.

For the proposed deep foundations, the SLS lateral geotechnical reaction of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equations given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (CFEM 2023) and API (2003). The equations provided, and the associated resistances are based on vertical piles; a modification factor would need to be applied for inclined piles.

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory (as outlined below). However, it should be noted that the response of a pile to lateral loads is highly non-linear and methods that assume linear behaviour (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1 percent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behaviour of the soil should be considered using P-y curves.

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h is the constant of horizontal subgrade reaction (kPa/m) (Table below);

z is the depth (m); and,

B is the pile diameter or width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where: s_u is the undrained shear strength of the soil (kPa); and,

B is the pile diameter/width (m).

Table 5 outlines the ranges for the values of n_h and s_u that may be used in the lateral analysis of the piles at this site. The ranges in values reflect the variability in the subsurface conditions, the soil properties, and the approximate nature of the linear-elastic subgrade reaction analysis.

Table 5: nh and su values for Lateral Load Resistance Calculations

Soil Type	Depth below ground surface (m)	n_h (kPa/m)	s_u (kPa)
Sandy Fill (cohesionless)	0 – 1.4	5,000	-
Clayey Fill (cohesive)	1.4 – 2.3	-	65
Clayey Sand (cohesionless)	2.3 – 4.5	4,250	-
Silty Clay (cohesive)	4.5 – Bedrock	-	60

For piles arranged in closely spaced groups, the pile-soil-pile interaction causes the individual piles in a group to be less effective than a single pile. Theses “group effects” can be incorporated into the design by reducing the calculated coefficient of horizontal subgrade reaction values either in the direction of loading or perpendicular to the direction of loading using a method that modifies the single pile lateral resistance by some factor (i.e., a p-reduction factor).

6.4.1.5 Construction Consideration

Construction of drilled shafts will need to consider the potential for soil sloughing and water seepage and will require temporary steel liners as well as dewatering (e.g., bailing, pumping methods, etc.). Tremie procedures should be assumed for concrete placement (unless the rock sockets are demonstrated to be sufficiently dry during construction). The potential presence of cobbles and boulders within the existing soil deposit should also be considered in selecting the pile boring and casing equipment.

The base of each caisson shall be thoroughly cleaned of any cuttings or other material and inspected and tested. The method of cleaning proposed by the selected piling contractor should be approved by a qualified geotechnical engineer prior to commencement of field works. The cleaned base should be visually inspected (with a camera if necessary, depending on the length and construction setup) by a qualified geotechnical engineer during construction. Should the inspection indicate that loosened material is present at the base, the base would need to be re-cleaned and re-inspected. If an inspection approach is not feasible then a shaft resistance design approach should be used for the rock socket as per Section 9.10.4 of CFEM (2023).

Piles should be spaced at center-to-center spacing of at least three times pile diameter (3D) to minimize pile group effect. A minimum shaft diameter of 400 mm is recommended for drilled shafts to minimize void formation during pouring of the concrete.

If caisson caps are to be included as part of the design at or below the existing ground surface, they should be constructed at a minimum depth of 1.8 m for frost protection purposes, per OPSD 3090.101.

The various soils at this site are sensitive soils and could “flow” into the auger hole during drilled shaft installation if left unsupported. Temporary liners will be required for construction. It should be assumed that these liners will need to be “set” into the underlying bedrock.

It is expected that the temporary liners/casings would be installed using rotation methods. If a vibratory hammer is used, vibration monitoring of the existing hotel building and utilities is recommended. Casing installation through the cobbles and boulders of the fill deposit is expected to require rotary drilling methods, and churn drilling or down-hole hammer techniques may also be required to advance the caisson to the required depth if and where boulders are encountered as well as to form the rock socket in the bedrock. Provisions should be given for the presence of hard cobbles and boulders in selecting suitable drilling equipment to advance pile boring through the obstructions.

Full-time monitoring of caisson installation by a qualified geotechnical inspector is recommended to confirm the proper installation of caisson and base cleaning.

6.4.2 Driven Steel Piles

6.4.2.1 Axial Capacity in Compression

Steel piles driven to refusal on sandstone bedrock may be considered as an alternative foundation option for the proposed building expansion. Both H piles and concrete-filled pipe piles can be considered.

For steel piles driven to refusal on sound bedrock, an unfactored unit ultimate skin friction of 0.04 MPa and unfactored unit ultimate end bearing resistance of 25 MPa can be used in the design. The contribution of shaft friction can also be considered, but it will be very small compared to the end bearing resistance and can reasonably be neglected in the design.

In accordance with CFEM (2023), geotechnical resistance factor of 0.4 should be applied to the calculated axial ULS values for piles in compression. Hence, the factored unit ultimate skin friction of 0.016 MPa and factored ultimate end bearing resistance of 10 MPa can be used in the design of steel piles, driven to sound bedrock.

SLS does not apply to piles end bearing on/in bedrock at this site, since the SLS resistance for 25 mm of settlement is greater than factored axial geotechnical resistance at ULS.

The factored geotechnical resistance of strong bedrock is often higher (sometimes significantly so) than the capacity of the steel section. The design loads should therefore also be limited to the structural capacity of the pile.

6.4.2.2 Axial Capacity in Tension

For steel piles driven to refusal on sound bedrock, an unfactored unit ultimate skin friction of 40 kPa can be used for the caisson capacity in tension.

In accordance with CFEM (2023), geotechnical resistance factor of 0.3 should be applied to the calculated axial ULS values for piles in tension. Hence, the factored unit ultimate capacity of 12 kPa can be used in the design of steel piles in tension.

For driven piles to bedrock, the overburden will offer little resistance to uplift forces. No toe resistance can be mobilized in tension. Hence, rock anchors may be required, where uplift demand exceeds available frictional resistance.

6.4.2.3 Negative Skin Friction

As mentioned in Section 6.4.1.3, the total drag load can be calculated by multiplying the unit ultimate skin friction values by shaft circumference or pile perimeter length and by the length of the pile embedded in settling soil.

In this case, unit ultimate (negative) skin friction of 40 kPa can be used in the design. For this site, the length of pile within the settling soil is taken as the length within the entire overburden soil for conservatism.

Since the steel piles are expected to be driven to refusal on sound bedrock to achieve required resistance, the effect of drag loads on the settlement (or SLS geotechnical axial resistance) of the driven pile will be negligible.

6.4.2.4 Lateral Capacity

The lateral capacity of piles can be calculated as per Section 6.4.1.4.
As piles are driven to refusal without socketing, their lateral capacity is controlled by overburden soil resistance, and no fixity at the rock surface is assumed.If the foundations cannot generate sufficient lateral resistance, then rock anchors can be considered.

6.4.2.5 Construction Consideration

Driven piles must be installed in accordance with OPSS 903.

For the installation of steel piles, consideration must be given to the potential presence of cobbles and boulders within the existing fill deposit. The use of driving shoe/ or flange plates is recommended to minimize damage while penetrating the fill deposit (which is expected to contain boulders and cobbles) and seating on to the sandstone bedrock. Pipe piles are considered to have a higher risk than H-piles for hanging up or being deflected away from their vertical or battered orientation, if cobbles and boulders are encountered during driving.

The installation of steel piles is typically associated with increased noise and ground vibrations, which may affect existing hotel structure, utilities, and nearby development. If the option of driven pile is selected, it is recommended that a vibration monitoring plan be implemented during construction. This plan should include baseline vibration measurements, continuous monitoring during pile driving, and the establishment of threshold limits to ensure potential impacts are identified and managed promptly. Appropriate mitigation measures should also be in place to address any exceedances and minimize disturbance to neighboring properties. The vibration monitoring plan should be prepared by a specialist geotechnical engineer.

The piles will be driven to bedrock through a layer of soil which is known to contain a cobbles and boulders. Piles can deflect or become damaged if they encounter boulders. Piles (both H-piles and pipe piles) should be equipped with pile points (e.g., Titus Standard H Point, or similar) to provide additional protection to the pile tips against damage during driving. Even with this measure, it should be expected that damage may occur to some piles and replacement piles will be required. For piles driven to refusal on bedrock, and as described in OPSS 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then gradually increase the energy over a series of blows to seat the pile.

Provision should be made for restriking all piles at least once to confirm the design set and/or the permanence of the set and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first restrike should receive additional restriking until the design set is met. All restriking should be performed a minimum of 48 hours after the previous set.

Pile driving criteria depend not only on the details of the pile (size, length, load, etc.) but also on the equipment used for installation. Preliminary pile driving criteria should be established prior to construction using wave equation analysis (WEAP or similar) or other approved means and confirmed through a program of dynamic (PDA) testing carried out at an early stage in the piling program. Additional PDA testing should be used to confirm the pile capacities at regular intervals as the project progresses. As a preliminary guideline, the specification should require that at least 10% of the piles be included in the dynamic testing program. CASE method estimates of the capacities should be provided for all piles tested. These estimates should be provided by means of a field report on the day of testing; CAPWAP analyses should be carried out for at least one half of the piles tested, with the results provided no later than three days following testing. The final report should be stamped by an engineer licensed in the province of Ontario. The PDA testing program will justify an increase in the geotechnical resistance factor to 0.5.

The driving energies required to confirm the ultimate geotechnical resistance of the pile (typically the testing is intended to prove a load of twice the design load) will be significantly higher than the energy required to install the pile. This is especially true if very large pile capacities are assumed due to the high geotechnical resistance. Insufficient energy is a common problem in demonstrating the true ultimate capacity of piles during PDA testing, and larger pile driving hammers may be required for testing than are required for installation. It is also likely that the stresses induced in the piles during driving and testing will be limiting factor in pile testing, not the capacity of the bedrock to resist the loading (i.e., it is common to damage or break a pile during driving and/or testing long before the bedrock yields). The ability to test the piles during construction can become a constraint on the pile design if very high capacities are adopted.

The piling specifications should be reviewed by an experienced geotechnical engineer prior to tender, as should the contractor’s submission (shop drawings, equipment, procedures, preliminary set criteria, etc.) prior to construction. Piling operations should be inspected on a full-time basis by geotechnical personnel to monitor the pile locations and plumbness, initial sets, penetrations on restrike, and to check the integrity of the piles following installation.

6.4.3 Rock Anchors

If the foundations cannot generate sufficient uplift or lateral resistance, then rock anchors can be considered.

The design of the rock anchors should be generally carried out in accordance with the guidelines provided in the CFEM and FHWA-IF-99-015 (Section 5.9.2 and other sections). The design and construction of tiedown rock anchors should also follow the specifications in OPSS.MUNI 942.

Rock anchors are typically installed in a borehole that is drilled with air-percussion equipment or with rotary diamond drilling equipment with water circulation; those drilling methods can fairly penetrate through boulder/cobblery ground as well as bedrock. A cased hole would be drilled through the overburden with a socket drilled into the bedrock, the steel anchor inserted, and then the annular space around the bar filled with grout.

Because the rock anchors would be permanent elements of the foundations, a “double corrosion protection” system should be provided.

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

i) failure of the steel tendon or top anchorage

ii) failure of the grout/tendon bond

iii) failure of the rock/grout bond

iv) failure within the rock mass, or rock cone pull-out

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion.

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as *1,000 kPa* for ULS design purposes. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the preliminary resistance is calculated based on the unit weight (undrained) of the potential mass of rock and soil which could be mobilized by the anchor, and resistance to shear of the rock mass. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \varphi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

Where: Qr = Factored uplift resistance of the anchor (kN)
φ = Resistance factor (use 0.4)
γ ’ = Effective unit weight of rock (use 16 kN/m^3 below the groundwater level)
D = Anchor length in m
θ = One-half of the apex angle of the rock failure cone (use 30°)

Where the anchor load is applied at an angle to the vertical, the anchor capacity should be reduced as follows:

$$Q_r{}' = Q_r \cos(\alpha)$$

Where: Qr’ = Factored uplift resistance of the anchor subject to inclined load (kN)
Qr = Factored uplift resistance of the anchor (kN)
α = Angle between the load direction and the vertical

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. In the case of group effects for a series of rock anchors in a rectangle with width “a” and length “b” installed to a depth “D”, the equation for the volume of the truncated trapezoid failure zone would be as follows:

$$V = \frac{4}{3} D^3 sin^2 \mathrm{q} + aD^2 \sin \mathrm{q} + bD^2 \sin \mathrm{q} + abD$$

Where: V = Volume of the truncated trapezoid failure zone (m^3)
D = Depth of anchor group (m)
a = Width of anchor group (m)
b = Length of the anchor group (m)
θ = One-half of the apex angle of the rock failure cone (use 30°)

The ULS factored geotechnical resistance for the truncated trapezoid failure formed by the group of anchors can then be calculated based on the following equation:

$$Q_r = \varphi \gamma' V$$

Where: Qr = Factored uplift resistance of the anchor (kN)
φ = Resistance factor, use 0.4
γ’ = Effective unit weight of rock (use 16 kN/m^3)
V = Volume of truncated trapezoid (m^3)

The method described above does not explicitly consider the tensile strength of the rock that must be overcome prior to mobilization of the weight of the rock mass. If required, the tensile strength of the rock mass can be assessed based on the unconfined compressive strength, recovery, and quality of bedrock core obtained. This assessment, however, requires a detailed understanding of the anchor lengths, geometry, loads, etc. and would need to be completed during detailed design.

It is recommended that proof load tests be carried out on the anchors to confirm their resistance. The proof load tests should be carried out in accordance with OPSS 942 *(Prestressed Soil and Rock Anchors).*

A geotechnical professional should be present during the installation and testing of the anchors. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grouted area with minimum voids. Confirmation of sufficient embedment into the rock beneath the foundations should be carried out to make sure that the anchors are being installed in rock of adequate quality. The anchor holes must be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to promote an adequate bond between the grout and the rock. Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.

6.4.4 Deep Foundation Design Considerations Adjacent to Existing Building Foundations

Piles must be staggered with the existing building foundations to avoid pile bore excavations conflicting with the existing foundations. In addition, a minimum horizontal clear spacing of 1 m should be provided between the outside edge of the existing footing pads and the edge of the new piles.

Even though the new piles are not anticipated to undergo considerable settlements, it is still recommended to connect the new and existing buildings structurally with the use of dowels or provide expansion joints for free movement of the new building structure relative to the existing building structure.

6.4.5 Slab on Grade

Since the building will be supported on pile foundations, the slab-on-grade will only be subjected to light loads such as those from occupants, finishes, and furniture. Under these conditions, it is considered acceptable to place the slab directly on the grade, provided that the subgrade is adequately prepared. Control joints are recommended to minimize cracking and accommodate minor settlement. This recommendation is based on the assumption that no structural loads will be transferred to the slab (i.e., the structure is supported on deep foundations) and that any minor settlement that may occur can be tolerated.

The geotechnical design of a slab-on-grade is typically not governed by bearing resistance, but by maintaining settlement and deformation (particularly differential) of the slab under loading within acceptable limits.

The localized differential settlements (i.e., slab deflections) will depend upon the relative stiffness between the footing and the underlying subgrade. The deflections and the resulting forces and bending moments in the slab to be used in its structural design could be determined by structural analysis using a modulus of subgrade reaction, k_s, for the subgrade.

The modulus of subgrade reaction is not a fundamental soil property, and its value depends, in part, on the size and shape of the loaded area. For the analysis of the contact stress distribution beneath a slab, its value would depend on the size of the areas over which increased/concentrated contact stresses are anticipated (analogous to equivalent footings beneath the columns); the size of these areas is in turn related to the value of the modulus of subgrade reaction, i.e., they are inter-related.

Accordingly, the analysis of the foundation slab should ideally involve an iterative analysis between the determination of the contact stress distribution by the structural engineer and the geotechnical determination of the modulus of subgrade reaction value, until the two are consistent with each other. For a 0.3 m by 0.3 m unit section of the concrete slab, the modulus of subgrade reaction ($K_{0.3}$) may be assumed as 10 MPa/m for concrete slab overlying the existing fill soil and 20 MPa/m for concrete slab overlying the granular engineered fill. Where B (in meters) is the shortest dimension of the loaded area on slab-on-grade.

The design modulus should be adjusted based on the loaded area as outlined in Section 5.4.9.1 of CFEM (2023).

For cohesive soil,

$$K_B = \frac{K_{0.3} * 0.3}{B}$$

K_B = modulus of subgrade reaction for a footing width of B

$K_{0.3}$ = modulus of subgrade reaction for a footing width of 0.3 m

B = foundation width

6.4.5.1 Subgrade Preparation

All unsuitable materials such as topsoil, organics, debris, rootlets, boulders, cobbles, and any wet, weak or disturbed soils should be stripped off from the proposed slab-on-grade footprints. The exposed soil subgrades after excavation should be thoroughly cleaned of debris and loose materials. Any soft or loose zones should be proof rolled and replaced with compacted granular material to provide uniform support. The excavated subgrade should also be visually inspected and approved by a qualified geotechnical consultant prior to placement of engineered fill or slab-on-grade construction. If any soft spots are observed during proof rolling, these should be excavated and replaced with additional Granular A or Granular B Type II.

Any required grade raising of the excavated subgrade to the design slab subgrade level should consist of structural granular engineered fill as per OPSS.MUNI 1010. The structural granular engineered fills should extend laterally and connected to side drainage system to reduce local ponding of water inside the granular engineered fills.

The prepared subgrade should be protected from disturbance of construction traffic, excessive wetting or drying. The prepared subgrade should also be inspected and approved by a qualified geotechnical consultant prior to the installation of granular bedding and concrete slab.

6.4.5.2 Granular Bedding

A minimum of 300 mm thick, clean granular bedding consisting of OPSS.MUNI 1010 Granular A crushed gravel should be installed on the prepared subgrade for the purpose of leveling and facilitating drainage. The granular bedding should be installed in a single lift and compacted to 100% of Standard Proctor Maximum Dry Density (SPMDD) at ±2% of Optimum Moisture Content (OMC).

6.5 Temporary Excavation and Dewatering

6.5.1 Temporary Excavation

Bedrock excavation is not anticipated at the proposed building expansion. In addition, it is the project is not expected to require large or deep excavations (such as for basements). It is expected that excavations will be localized for pile caps, utility trenches, etc.

Excavations of overburden materials are anticipated to be handled using conventional hydraulic excavating equipment. Cobbles and boulders should be expected in the existing fill.

As a minimum requirement, all side slopes of temporary open-cut excavations should conform to the Occupational Health and Safety Act (OHSA) – Regulation for Construction Projects (O. Reg. 213/91). The existing fill materials and native soils above the water table would be classified as Type 3 soils. OHSA indicates that temporary excavations made within these materials above the water table should be developed with side slopes no steeper than 1H:1V. The site overburden soils below the water table would be classified as Type 4 soils, and excavations in these materials should be sloped no steeper than 3H:1V based on OSHA requirements. Large size boulders and cobbles at the excavation side slope faces should be removed for worker safety. Stockpiling and equipment operating should be avoided from the excavation edge to a distance at least equal to the excavation depth to reduce instability of unsupported excavation slopes.

Where the available space limits the above cutback slopes, the temporary excavations can be advanced at steeper slopes and protected by temporary shoring systems or trench boxes designed and installed by the contractor. In addition to the above minimum requirement, all temporary excavations more than 2 m in depth should be properly designed by the contractor.

All permanent excavations should be properly designed by a geotechnical consultant.

6.5.1.1 Temporary Excavation Adjacent to Existing Buildings

Additional care should be taken in conducting temporary excavations adjacent to or within the vicinity of the existing buildings. The temporary excavation may undermine the stability of the existing foundations, grade beams, ground supported slabs, or other grade supported structures including sidewalks and pavements.

As a minimum requirement, excavations should not extend below the bottom of the existing building footings or pile caps where they are immediately adjacent to existing structures unless they are adequately supported (as confirmed by a design prepared by a professional engineer) or the structures are underpinned. If deep excavations are required adjacent to existing structures these areas can be reviewed by WSP during detailed design and additional comments provided as appropriate.

The design of protection systems to support the temporary excavations, as well as any underpinning is typically the responsibility of the contractor.

Open excavations and adjacent structures should be periodically inspected for any signs of instability or movement.

6.5.2 Temporary Dewatering

Based on the groundwater conditions inferred in the monitoring wells, as presented in Section 5.3, excavations deeper than approximately 3 m below ground surface may encounter groundwater, depending on the time of year that construction occurs. The rate of groundwater inflow into the excavations will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavation, the number of working areas being excavated at one time, and the time of year at which the excavation is made. Also, there may be instances where precipitation collects in an open excavation following rainfall and must be rapidly pumped out.

Hydrogeological investigations were not carried out at the project site to estimate the dewatering volume and requirements. According to O.Reg 63/16 and O.Reg 387/04, if the volume of water to be pumped from excavations for the purpose of construction dewatering is greater than 50,000 litres per day and less than 400,000 litres per day, the water taking will need to be registered as a prescribed activity in the Environmental Activity and Sector Registry (EASR) and requires the completion of a "Water Taking Plan". A Permit to Take

Water (PTTW) is required from the Ministry of the Environment Conservation and Parks (MECP) if a volume of water greater than 400,000 litres per day is to be pumped out from an excavation.

Temporary dewatering systems are the Contractor's responsibility, and the rate and volume required for dewatering is dependent on the construction methods and staging chosen by the contractor. In general, however, it is anticipated that the volume of dewatering required in the excavations can be handled, as required, by pumping from properly constructed and filtered sumps located within the excavations. It is understood that extensive excavation will not be required below ground water level for foundation and site services, and hence, it is unlikely that an EASR or PTTW would be required. This assumption should be reviewed during detailed design based on the size and depth of the excavations.

6.6 Site Servicing

6.6.1 Trench Excavation and Subgrade Preparation

The site subsurface stratigraphy within the typical range of depths of underground utility pipes consists of asphaltic concrete over granular pavement structure, overlying existing fill, which is underlain by possible fill consist of clayey sand and silty clay over bedrock. Groundwater may be encountered below the depth of 3.3 m (~El. 86.8 masl) from the existing ground surface. Perched water seepage may also be encountered at shallower depths in the excavations during construction.

Utility trench excavations and temporary dewatering should be carried out in accordance with the general recommendations provided in Section 6.5 of this report.

Where the new underground utility alignments are supported on overburden soil, all the topsoil, organics, rootlets, debris, boulder, cobbles and existing fill containing deleterious materials should be subexcavated and removed from the utility trenches. Due to the significant amount of fill present on this site, it is not feasible to remove entire fill deposit. Where fill material is present below invert level, the base of the trench should be reviewed by a geotechnical engineer. If necessary, fill material may need to be over-excavated and replaced with additional engineered fill consisting of OPSS Granular B Type I or II compacted to at least 95 % of the material's Standard Proctor Maximum Dry Density (SPMDD) using suitable vibratory compaction equipment.

The prepared trench subgrade after excavation or engineered fill replacement of unsuitable materials should be inspected and approved by a qualified geotechnical consultant prior to placement of bedding layer and utility installation. The prepared subgrade should also be protected from disturbance of construction traffic, excessive wetting or drying, or freezing. No more than 15 m of trench should be open in advance of the completed utility installation.

6.6.2 Utility Trench Backfilling

In general, the installation of pipe utilities such as sanitary, storm and watermain should be completed in accordance with OPSS.MUNI 401 and OPSD 802 series drawings for the applicable pipe type and subgrade type. Where applicable, Manufacturer's specifications should be followed for the installation of utilities including gas, electrical and communication.

The bedding and cover materials for rigid pipes, the embedment material for flexible pipes, and the backfill material above the cover or embedment are specified in Section 401.05 of OPSS.MUNI 401. The minimum required thickness and extent of the bedding and cover materials or embedment material should be referenced from the applicable OPSD 802 drawings.

At least 150 mm of Granular A (OPSS.MUNI 1010) should be used for bedding. This can be increased if required based on the actual utilities being installed. The bedding material should extend to the spring line of the utility pipe or conduit. Cover material, from the top of bedding to at least 300 mm above the top of utility, should consist of Granular A or Granular B (Type I or II) material with a maximum particle size of 26.5 mm.

The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the existing soils could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

All the bedding, cover and embedment materials should be placed in maximum loose lifts of 200 mm and compacted to at least 95% of the material's SPMDD at ±2% of Optimum Moisture Content (OMC). The materials should be placed on each side of the utility pipe or conduit simultaneously. At no time should the material levels on each side the pipe or conduit differs by more than 200 mm of uncompacted layer. Additional requirements should be followed in OPSS.MUNI 501.

It may be possible to re-use the excavated inorganic fill soils as trench backfill above the cover or embedment, but the native soils should be carefully separated during excavation. Wet soil, debris, cobbles, and boulders should be separated from the existing fill material. Wet soils may be allowed to dry to an appropriate water content for placement and compaction. The separated fill material should be inspected and approved by a qualified material consultant prior to use as backfill. Suitable engineered fill could be imported if additional material is required to backfill the trench above the cover or embedment. Where the trench will support structures such as pavements and sidewalks, the type of material placed in the frost zone (down to 1.8 m depth) should match the soil exposed on the trench walls for frost heave compatibility. Alternatively, the backfill material should be placed to form frost taper on the trench sides. The trench backfill above the cover or embedment should be placed in maximum loose lifts of 200 mm and compacted to at least 97% of SPMDD at ±2% of OMC.

6.7 Pavement Rehabilitation

It is understood that the existing asphalt pavement is generally performing adequately, and full-depth reconstruction is not warranted at this time. To extend the service life of the pavement and improve surface ride quality, drainage, and durability, the removal of asphalt and resurface treatment is required.
Two options could be considered for the pavement rehabilitation:

- Option 1 – Full Removal of Existing Asphalt

Under this option, the existing asphalt layer shall be fully removed across both the access roadway and parking lot areas. This approach would allow for complete replacement of the surface and binder courses, providing a new, uniform asphalt layer with enhanced durability and performance.

The recommended new asphalt layer is as follows:

Parking areas: 50 mm surface course of Superpave SP 12.5 (or HL 3)

Access road: 50 mm surface course of Superpave SP 12.5 (or HL 3) over 50 mm binder course of Superpave SP 19.0 (or HL 8)

This option provides a robust pavement structure suitable for the expected traffic loading, particularly in the access road areas where heavier vehicle movements and turning stresses are anticipated.

The Superpave 12.5 surface asphalt and Superpave 19.0 binder asphalt shall use Performance Graded Asphalt Cement (PGAC) 58-34. Ontario Traffic Category B should be used in the design of the pavement structure. Asphalt concrete should be in accordance with OPSS 1151 for Superpave mix designs.

- Option 2 – Full removal and Replacement of Existing Asphalt in Parking Area and Partial Removal and Replacement in Access Road

A more targeted rehabilitation strategy involves full removal and replacement of the asphalt layer in the parking areas, where thinner pavement sections may be present, combined with partial removal and resurfacing in the access road areas. In this approach, the entire asphalt layer in the parking areas shall be removed and replaced with a single lift of 50 mm Superpave SP 12.5 (or HL 3).

For the access road, only the upper 50 mm of the existing asphalt shall be milled off, followed by placement of a new 50 mm surface course of Superpave SP 12.5 (or HL 3). This method possibly retains the lower portion of the existing pavement structure while improving surface ride quality and extending service life. A tack coat must be applied to the milled surface prior to paving to ensure proper bonding between the existing and new asphalt layers. The same PGAC 58-34 binder and Traffic Class B criteria apply for this option.

It should be noted that the actual thickness of the existing asphalt pavement may vary on site. While it is anticipated that approximately 50 mm is present in the parking areas and approximately more than 90 mm in the access road, these values should be confirmed during construction. Milling 50 mm in the access road is expected to leave a portion of the existing binder course in place; however, this may require field adjustment based on actual conditions.

6.7.1 Construction Consideration

All rehabilitation works should be carried out in accordance with current OPSS and OPSD. A tack coat should be applied on all milled or prepared surfaces to ensure proper bonding between new and existing layers. Following asphalt removal, the underlying granular base should be inspected and proof rolled. Any weak or unsuitable base materials must be removed and replaced prior to paving to ensure adequate structural support.

In areas where the pavement structure will be fully disturbed as part of site servicing or utility installations, the pavement structure should be reconstructed to match the existing base and subbase thicknesses and configuration. Following backfilling and compaction of the trench or disturbed area, the granular base and subbase materials should be reinstated to their original depths and compacted to meet OPSS requirements. Once the granular structure is restored and proof-rolled to confirm adequate support, the asphalt layer should be placed to match the surrounding pavement structure. For access road areas, this should consist of a binder course of Superpave SP 19.0 or HL 8 and a surface course of Superpave SP 12.5 or HL 3, while for parking lot areas, a single lift of Superpave SP 12.5 or HL 3 is recommended. Proper joint preparation and tack coat application are essential to ensure seamless tie-in between the new and existing pavement.

6.8 Corrosion and Cement Type

Results of chemical analyses for two soil samples submitted to Eurofins Environment Testing were summarized in Section 5.4.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The sulphate results were compared with Table 3 of Canadian Standards Association (CSA A23.1:19) and indicated a low degree of sulphate attack potential on concrete structures at this site. Accordingly, Type GU Portland cement can be considered for buried concrete substructures in contact with existing soils. All imported soils should be tested for soluble sulphate contents. Tables 1 to 4 of CSA-A23.1-19 should be referenced for additional requirements and further information regarding concrete in contact with sulphates. In general, the properties of concrete in contact with soil or groundwater shall meet all the requirements of CSA A23.1:19.

The resistivity test result indicates a severe potential for corrosion of exposed ferrous metal at the site which should be considered in the design of substructures.

6.9 Slope Stability

A slope stability assessment of the creek slope was completed in 2008 as part of the original development of the site. At the request of the City, an update of this assessment and confirmation of the limit of hazard lands is required as part of the proposed addition.

6.9.1 Slope Setback Limits

‘Hazard Lands’ associated with potentially unstable slopes are defined as the table land adjacent to the slope for which there would be an inadequate ‘factor of safety’ against the land being affected by a slope failure. The Hazard Lands, as defined by Ministry of Natural Resources (MNR) guidelines and provincial planning policies, are unsuitable for development with buildings, roadways, parking areas or other infrastructure. In accordance with the MNR guidelines, the setback distance from the crest of an unstable slope to the Limit of Hazard Lands includes three components, as appropriate, namely:

1) A “Stable Slope Allowance”, which is determined as the limit beyond which there is an acceptable factor of safety (i.e., greater than 1.5 under static conditions or 1.1 under seismic conditions) against slope instability. A Stable Slope Allowance must be applied to slopes that do not have at least these minimum factors of safety.

2) An “Erosion Allowance” is used to account for potential future movement of the slope toe, in the table land direction, as a result of erosion along the slope toe/creek bank. This Erosion Allowance is included in the determination of the Limit of Hazard Lands wherever the development could restrict future slope access.

3) An “Erosion Access Allowance”, to allow a corridor by which equipment could travel to access and repair a future slope failure, erosion, etc.

It should be noted that the limits discussed above only account for the geotechnical limitations. It is common to have other environmental setback limits which apply (for example no development within a set distance of a watercourse). It is understood that a minimum setback of 15 m from the crest of the watercourse valley will apply regardless of the geotechnical conditions.

6.9.2 Slope Stability Assessment and Stable Slope Allowance

The evaluation of the stability of a slope depends on several parameters, including:

- The geometry of the slope,
- The ground conditions which form the slope (i.e., the thickness and orientation of the soil/bedrock strata),
- The shear strength parameters of the material which form the slope,
- The unit weight (i.e., density) of the soils which form the slope, and
- The groundwater levels and flow gradients within the slope.

As shown in Figure 1, a slope stability analysis was conducted for slope cross-section A-A. This slope was adjacent to the existing parking lot and was considered to represent the more critical condition (i.e., the location where the development is closest to the slope). The top of slope elevation used in the analyses was established from the topographic survey plan, prepared by Annis, O'sullivan, Vollebekk Ltd., dated August 29, 2025, attached to this report. Beyond the top of slope, geometry was established using previous survey data provided on a Site Plan drawing, prepared by Golder, dated April 21, 2008, attached to this report.

The stability of the slope cross-section was evaluated under both 'static' and 'seismic' loading conditions.

The seismic load imposed on a slope is modeled in a simplified manner by applying a horizontal "pseudo-static" force to the soil mass. The pseudo-static force, Fs is calculated as:

$$F_s = K_h \text{ x } m$$

Where;

K_h = Horizontal seismic co-efficient; and,

M = Mass of soil contained within the failure surface.

The seismic slope stability evaluations were carried out assuming that the design earthquake would correspond to an event with a 2% probability of occurrence in 50 years (i.e., the 2475-year design earthquake). As per the National Building Code of Canada (NBCC 2020), the ground surface PGA for this assessment was taken as 0.33g, corresponding to a Site Class C. For the seismic slope stability analyses, a 'K_h' value of 0.17g, i.e., equal to one-half the ground surface PGA, was used.

The stability of the slopes was evaluated using the commercially available program Slope/W, which is part of the GeoStudio 2023 package, employing the Morgenstern-Price method of analysis. Morgenstern-Price is a general method of slices which is based on the equilibrium of forces and moments acting on each slice of soil mass above the potential failure surface. The factor of safety is defined as the ratio of the magnitude of the forces/moments tending to resist failure to the magnitude of the forces/moments tending to cause failure. Theoretically, a slope with a factor of safety of less than 1.0 will fail and one with a factor of safety of 1.0 or greater will stand. However, because the modeling is not exact and natural variations exist for all of the parameters affecting slope stability, a factor of safety of 1.5 is used to define a stable slope (for static loading conditions). Under seismic loading conditions, a minimum factor of safety of 1.1 is used in a pseudo-static analysis.

Based on available site information and field inspections of slope, it is evident that the slope comprises a substantial amount of fill. There is an existing culvert at the slope location, as well as a concrete headwall and wing walls. These structures would tend to increase the stability of the slope where they are present. They have, however, been conservatively neglected in completing the analysis.

For long-term analysis, drained soil parameters were selected for general clayey fill material (i.e., unit weight of 19.5 kN/m^3, an angle of effective internal friction of 30 degrees, and an effective cohesion of 2 kPa). For short-term analysis, undrained soil parameters were selected for fill (i.e., unit weight of 19.5 kN/m^3, undrained shear strength of 50 kPa). The ground water level was assumed to be at the creek level. This assumption is based on the fact that the culvert is passing through the slope, and the structure and backfill would provide some level of drainage.

The results of the stability analyses carried out for static drained and seismic undrained conditions indicate that the factor of safety against global instability of the existing slope adjacent to the end of the proposed parking lot is greater than 1.5 and 1.1, respectively and hence, the slope is therefore considered to be geotechnically stable. A stable slope allowance is therefore not required. The results of the slope stability analysis (Figure F1 and Figure F2) and site visit photos are provided in Appendix F.

6.9.3 Erosion Allowance

An allowance for erosion is typically included where there is a potential for long-term erosion of the soil at the toe of the slope. This allowance depends on the nature of the soil as well as the flow conditions of the water body.

For the portion of the slope in interest for this assessment, there is a concrete culvert and wing wall retaining the toe of slope, and the flow of water is directed away from the slope. Erosion of the soil along the toe of the creek in the area of interest is therefore unlikely to be a concern. Regardless, a general erosion allowance of 6 m is conservatively assumed for this study.

6.9.4 Access Allowance

No access allowance is considered necessary for this development since the area at the crest of the slope will continue to be a surface parking lot. This use will not limit access to the top of the slope if repairs were to be required.

6.9.5 Summary of Recommended Slope Setback Limits

The setbacks for each condition as listed in the following Table 6 should be applied to the crest of the slopes.

Table 6: Summary of Slope Setback Limits

Stable Slope Allowance (m)	Erosion Allowance (m)	Access Allowance (m)	Total Geotechnical Setback (from Top of Slope) (m)
-	6	-	6

Based on the development plans provided to WSP by client, the slope setback limit is currently (and will remain) 15 m from the crest of slope. The geotechnical setback limit would be 6 m from the crest of the slope. The current 15 m limit therefore satisfies the required geotechnical setback limit.

6.10 Additional Considerations

The soils at this site are sensitive to disturbance from construction, traffic, and frost.

During construction, sufficient subgrade inspections, in-situ density tests, and materials testing should be carried out to confirm that the conditions exposed are consistent with those encountered in the boreholes, and to monitor conformance to the pertinent project specifications. Concrete testing should be carried out in a CCIL certified laboratory.

Piling operation should be inspected on a full-time basis by a geotechnical engineer to monitor the pile locations and plumbness, set criteria, and integrity of the piles following installation. Caisson construction should also be inspected by a geotechnical engineer on a full-time basis.

WSP should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

The slope stability analyses were performed using widely accepted methods based on simplified models and techniques. In addition, slope stability analyses were performed using extrapolation of subsurface data available from current and previous investigation. The above analysis is applicable to the slope adjacent to the north end of the proposed parking lot only. This slope was chosen for evaluation since its stability is more directly relevant to the development. The stability of downstream slopes, beyond the proposed development area, has not been evaluated.

7.0 CLOSURE

We trust this report contains sufficient information for your present purposes. If you have any questions regarding this report or if you have any questions, please reach out to us.

Signature Page

WSP Canada Inc.

LICENSED PROFESSIONAL ENGINEER
K. GAJJAR
100577339
Oct 20,2025
PROVINCE OF ONTARIO

Kinjal Gajjar, P.Eng.
Geotechnical Engineer

Chris Hendry, P.Eng.
Senior Geotechnical Engineer

KG/CH/yj

https://wsponlinecan.sharepoint.com/sites/ca-ca0057064.0145/shared documents/06. deliverables/geotechnical/geotech report/ca0057064.0145-r-rev0-holiday inn expansion-2025'10'20.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: WSP Canada Inc. (WSP) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to WSP 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. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. WSP cannot be responsible for use of this report, or portions thereof, unless WSP is requested to review and, if necessary, revise the 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 WSP'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, WSP 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. Any other use of this report by others is prohibited and is without responsibility to WSP. The report, all plans, data, drawings and other documents as well as all electronic media prepared by WSP are considered its professional work product and shall remain the copyright property of WSP, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of WSP. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of WSP's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to WSP by the Client, communications between WSP and the Client, and to any other reports prepared by WSP 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. WSP can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. 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.

Soil, Rock and Ground Water Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, WSP does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that WSP interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The 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.

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.

Sample Disposal: WSP will dispose of all uncontaminated soil and/or rock samples 90 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, fills 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.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of WSP's report. WSP 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 WSP's report.

During construction, WSP 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 WSP's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in WSP's report. Adequate field review, observation and testing during construction are necessary for WSP 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, WSP'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.

Changed Conditions and Drainage: 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 WSP 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 WSP be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

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. WSP takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

ATTACHMENTS

Figure 1 - Borehole Location Plan

Topographic Plan of Survey, prepared by Annis, O'Sullivan, Vollebekk Ltd., dated August 29, 2025

Figure 2 - Site Plan, prepared by Golder Associates, dated April 21, 2008

KEY MAP

SCALE 1:100,000

LEGEND

AUGERHOLE LOCATION

▚	APPROXIMATE AUGER HOLE FROM PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES - REPORT NO. 08-1121-0045
▚	APPROXIMATE AUGER HOLE FROM CURRENT INVESTIGATION

BOREHOLE LOCATION

⊕	APPROXIMATE BOREHOLE LOCATION FROM CURRENT INVESTIGATION
⊕	APPROXIMATE BOREHOLE LOCATION FROM PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES - REPORT NO. 08-1121-0045
⊕	APPROXIMATE BOREHOLE LOCATION FROM PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES - REPORT NO. 772037
—	WATERCOURSE
↔	SLOPE CROSS-SECTION LINE
- - -	TOP OF SLOPE
- - -	15 m SETBACK LIMIT FROM TOP OF SLOPE
▭	SITE BOUNDARY

0 4 8 16 24 32 40

1:1,000 METRES

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. CONTAINS INFORMATION LICENSED UNDER THE OPEN GOVERNMENT LICENCE - ONTARIO
2. PARCEL BOUNDARY TAKEN FROM CITY OF OTTAWA OPEN DATA.
3. IMAGERY CREDITS: COMMUNITY: SOURCES: ESRI, TOMTOM, GARMIN, FAO, NOAA, USGS, © OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY LIO_IMAGERY/ONTARIO_IMAGERY_WEB_MAP_SERVICE:
4. COORDINATE SYSTEM: NAD 1983 UTM ZONE 18N

CLIENT
MANGA HOTELS (NEPEAN) LP c/o
KINGSLAKE PROJECTS INC.

PROJECT
PROPOSED BUILDING EXPANSION,
HOLIDAY INN EXPRESS AND SUITES, NEPEAN, ON

TITLE
BOREHOLE LOCATION PLAN

CONSULTANT

wsp

YYYY-MM-DD	2025-10-08
DESIGNED	KG
PREPARED	KP
REVIEWED	----
APPROVED	----

PROJECT NO.	CONTROL	REV.	FIGURE
CA0057064.0145	0002	A	1

PATH: S:\Clients\Kingslake_Projects_Inc\Holiday_Inn_Express\99_PROJ\CA0057064_0145\40_PROD\0001_Geotechnical\CA0057064_0145-0001-BG-0000\CA0057064_0145-0001-BG-0000.aprx PRINTED ON: AT: 10:27:18 AM

25mm
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

Notes & Legend

—□—	Denotes	Survey Monument Planted
—■—	"	Survey Monument Found
SIB	"	Standard Iron Bar
SSIB	"	Short Standard Iron Bar
IB	"	Iron Bar
CC	"	Cut Cross
CP	"	Concrete Pin
(WIT)	"	Witness
Meas.	"	Measured
(AOG)	"	Annis, O'Sullivan, Vollebekk Ltd.
(PI)	"	(AOG) Plan dated July 10, 2017
(P2)	"	Plan 4R-24255
(P3)	"	(AOG) Plan dated February 9, 2010
(P4)	"	Plan 4R-4565
(P5)	"	(725) Plan dated February 17, 1986
MF	"	Metal Fence
EOA	"	Edge of Asphalt
EFN	"	East Face Noted
WFN	"	West Face Noted
T/G	"	Top of Grate
□ CB	"	Catch Basin
▦ DI	"	Ditch Inlet
○ MH-ST	"	Maintenance Hole (Storm Sewer)
○ MH-S	"	Maintenance Hole (Sanitary)
○ MH-B	"	Maintenance Hole (Bell)

○ LS	"	Light Standard
∘ AN	"	Anchor
○ UP	"	Utility Pole
— OHW —	"	Overhead Wires
— G —	"	Underground Gas
▭ GM	"	Gas Meter
▢ AC	"	Air Conditioner
∘ B	"	Bollard
△ S	"	Sign
▭ HH	"	Handhole
▭◅	"	Gate
⊛ FP	"	Flag Pole
⊗ WV	"	Water Valve
⌖ FH	"	Fire Hydrant
✿	"	Shrub
◌	"	Deciduous Tree
✳	"	Coniferous Tree
ø	"	Diameter
+ 65.00	"	Location of Elevations
+ 65.00	"	Location of Top of Curb Elevations

Bearings are grid, derived from Can-Net 2016 Real Time Network GPS observations and are referenced to Specified Control Points 01919680005 and 01919750705, MTM Zone 9 (76°30' West Longitude) NAD-83 (original).

For bearing comparisons, a rotation of 0°24'00" counter-clockwise was applied to bearings on plan (P1), (P2) and (P3).

ELEVATION NOTES

1. Elevations shown are geodetic, derived from Ministry of Natural Resources Vertical Control Monument No. 001196530358, having a published elevation of 91.079 metres, and are referred to the CGVD28 geodetic datum.
2. It is the responsibility of the user of this information to verify that the job benchmark has not been altered or disturbed and that it's relative elevation and description agrees with the information shown on this drawing.

UTILITY NOTES

1. This drawing cannot be accepted as acknowledging all of the utilities and it will be the responsibility of the user to contact the respective utility authorities for confirmation.
2. Only visible surface utilities were located.
3. A field location of underground plant by the pertinent utility authority is mandatory before any work involving breaking ground, probing, excavating etc.

TOPOGRAPHIC PLAN OF SURVEY OF

PART OF BLOCK 'A'
REGISTERED PLAN 4M-65
CITY OF OTTAWA

Surveyed by Annis, O'Sullivan, Vollebekk Ltd.

Scale 1 : 200

8.0 6.0 4.0 2.0 0 4 8 Metres

Metric

DISTANCES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048

Surveyor's Certificate

I CERTIFY THAT :

1. This survey and plan are correct and in accordance with the Surveys Act, the Surveyors Act and the regulations made under them.
2. The survey was completed on the 26th day of August, 2025.

AUGUST 29, 2025
Date

Mirel Aradau
Ontario Land Surveyor

ASSOCIATION OF ONTARIO LAND SURVEYORS
PLAN SUBMISSION FORM
V-110947
THIS PLAN IS NOT VALID UNLESS IT IS AN EMBOSSED ORIGINAL COPY ISSUED BY THE SURVEYOR In accordance with Regulation 1026, Section 29 (3).

Ontario Land Surveyors

ANNIS, O'SULLIVAN, VOLLEBEKK LTD.
14 Concourse Gate, Suite 500
Nepean, Ont. K2E 7S6
Phone: (613) 727-0850 / Fax: (613) 727-1079
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Job No. 22838-25 C2 Manga Hotels PtBlk A M-65 O D2

LEGEND:

BH 08-1A	APPROXIMATE BOREHOLE LOCATION IN PLAN, PRESENT INVESTIGATION
● AH 08-2	APPROXIMATE AUGERHOLE LOCATION IN PLAN, PRESENT INVESTIGATION
⌖ BH 1	APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES - REPORT No. 772037
A A	APPROXIMATE CROSS-SECTION LOCATION IN PLAN, REFER TO FIGURE 3 FOR CROSS-SECTION

REFERENCES:

1. BASE DRAWING PROVIDED BY CHAMBERLAIN ARCHITECT SERVICES LIMITED, DRAWING SP1 SITE PLAN, DATED JANUARY 21, 2008.

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

Golder Associates
Ottawa, Ontario, Canada

SCALE	1:400	TITLE	SITE PLAN
DATE	21 April 2008		
DESIGN	K.S.L.		
CAD	M.L.F.		
FILE No. 0811210045-02.dwg	CHECK T.M.S.	PROPOSED HOLIDAY INN OTTAWA, ONTARIO	FIGURE 2
PROJECT No. 08-1121-0045 REV.	REVIEW M.I.C.		

PLOT DATE: May 12, 2008
FILENAME: N:\Active\2008\1121 - Geotechnical\08-1121-0045 Park Holiday Inn Bells Corners\ACAD\0811210045-02.dwg

APPENDIX A

Method of Soil Classification

Abbreviations and Terms used on Records of Boreholes

List of symbols

Lithological and Geotechnical Rock Description Terminology

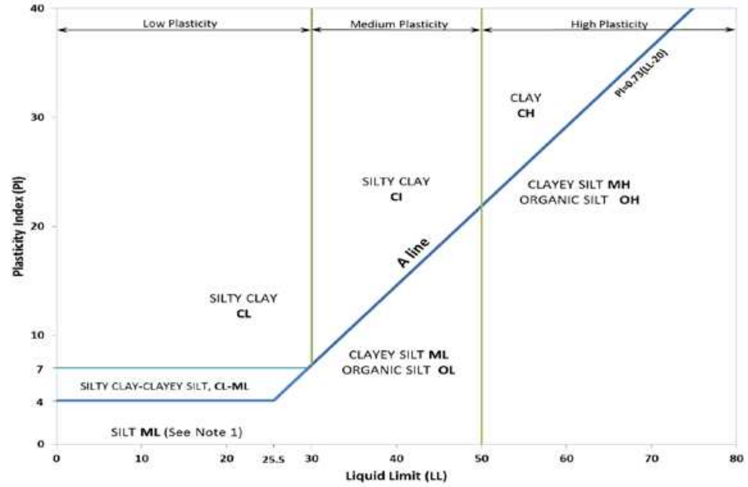
Record of Borehole and Auger-hole Logs-Current Investigation

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators					Organic Content	USCS Group Symbol	Primary Name
				Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)			
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% **(see Note 2)**	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.
Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.
For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.
For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.
A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

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

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	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
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING CLASSIFICATION

Fresh (W1): no visible sign of rock material weathering.

Slightly Weathered (W2): discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

Moderately Weathered (W3): less than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Highly Weathered (W4): more than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Completely Weathered (W5): 100% of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

Residual Soil (W6): all rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90º angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

AXJ	Axial Joint
BD	Bedding
BC	Broken Core
CC	Continuous Core
CL	Closed
CO	Contact
CU	Curved
CT	Coated
FLT	Fault
FOL	Foliation
FR	Fracture
GO	Gouge
IN	Infilled
IR	Irregular
JN	Joint
KV	Karstic Void
K	Slickensided
LC	Lost Core
MB	Mechanical Break
PL	Planar
PO	Polished
RO	Rough
SA	Slightly Altered
SH	Shear
SM	Smooth
SR	Slightly Rough
SY	Stylolite
UN	Undulating
VN	Vein
VR	Very Rough

ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250

PROJECT: CA0057064.0145

LOCATION: N 5019265.02; E 434868.50

RECORD OF BOREHOLE: BH25-01

SHEET 1 OF 3

BORING DATE: August 28, 2025

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg DROP, 762mm

DRILL RIG: Truck mounted/ CME 11

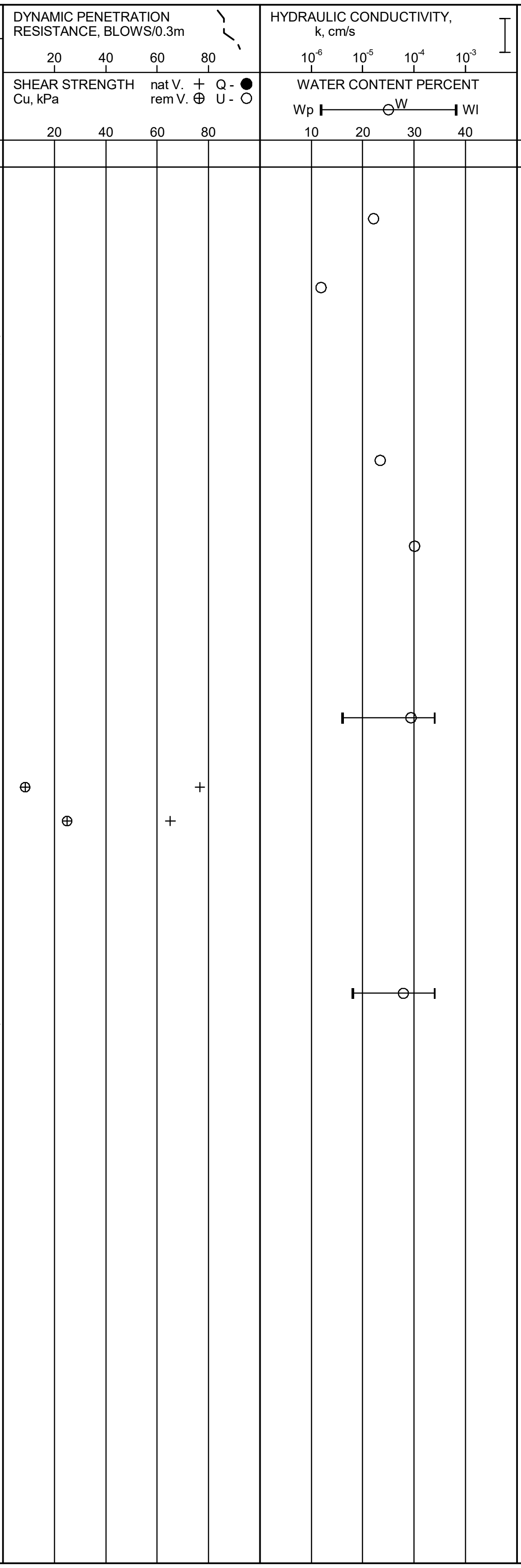
HAMMER TYPE: AUTOMATIC

BORING METHOD: Power Auger, 108 mm I.D. Hollow Stem Augers (0 to about 1.67 m); Wash Bore, 114 mm I.D. Casing (to 7.79 m)

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	90.27
ASPHALT (125 mm)	0.00
FILL - (SP) gravelly SAND; grey to brown; non-cohesive, dry (PAVEMENT STRUCTURE)	0.13
FILL - (SM) gravelly SILTY SAND, some plastic fines; grey to brown, non-cohesive, moist, compact	89.51 0.76
FILL - (CL/CI) SILTY CLAY, some sand to sandy SILTY CLAY, some gravel, contains wood debris, cobbles and boulders; grey to black; cohesive, w<=PL - Boulder at 1.77 m - Auger Refusal on cobbles and boulders at 1.78 m	88.90 1.37
Possible FILL - (SC) CLAYEY SAND, some gravel; grey to brown; cohesive fines, moist, very loose to loose	87.98 2.29
Possible FILL - (CI) Sandy SILTY CLAY, trace to some gravel, grey to brown, cohesive, w>PL, stiff - some gravel - contains pockets of sand, trace rock debris, rootlets from about 7.0 m to 7.6 m depth	85.70 4.57
END OF BOREHOLE	82.48 7.79

SAMPLE NUMBER	TYPE	BLOWS/0.3m	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AS		
2	SS	14	13 49 (38)
3	SS	104/ 0.15	
4	RC		
5	SS	10	9 43 (48)
6	SS	5	
7	SS	2	
8	SS	2	
9	SS	2	
10	SS	5	
11	SS	100/ 0.13	

PIEZOMETER OR STANDPIPE INSTALLATION: Bentonite; Backfill; ▽ September 24, 2025; Bentonite



END OF BOREHOLE

Auger refusal at 7.79 m on inferred (BEDROCK)

Bedrock cored from 7.79 m to 10.92 m

See Record of Drillhole BH 25-01 for rock coring details

NOTE:

1. Water level measured at 5.09 m below ground surface on September 24, 2025.

DEPTH SCALE

1 : 60

LOGGED: BW/IK

CHECKED: AKP/KG

PROJECT: CA0057064.0145

RECORD OF DRILLHOLE: BH25-01

SHEET 3 OF 3

LOCATION: N 5019265.02 ;E 434868.50

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: August 29, 2025

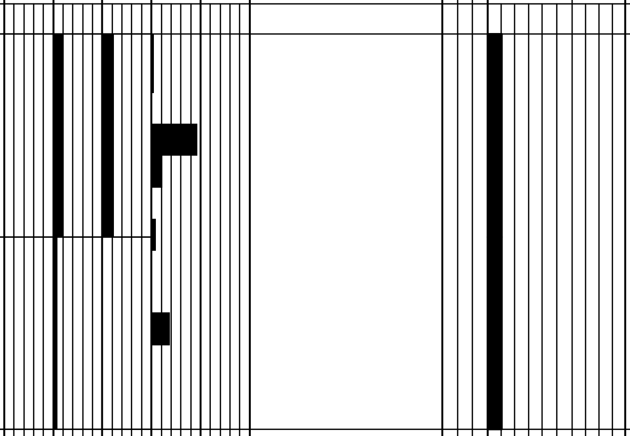
DRILL RIG: Truck mounted/ CME 11

DRILLING CONTRACTOR: Downing Ltd.

DATUM: Geodetic

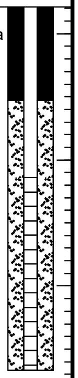
NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN
		Continued from Record of Borehole BH25-01		82.48		
8 9 10		Fresh, laminated to medium bedded, grey and black, crystalline, non to slightly porous, good to excellent quality, very strong, SANDSTONE		7.79	1 2	
11 12 13 14 15 16 17 18 19		END OF DRILLHOLE		79.35 10.92		



FEATURES

PIEZOMETER



DEPTH SCALE

1 : 60

LOGGED: BW/IK

CHECKED: AKP

PROJECT: CA0057064.0145

LOCATION: N 5019271.97; E 434875.47

RECORD OF BOREHOLE: BH25-02

SHEET 1 OF 1

BORING DATE: August 28, 2025

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg DROP, 762mm

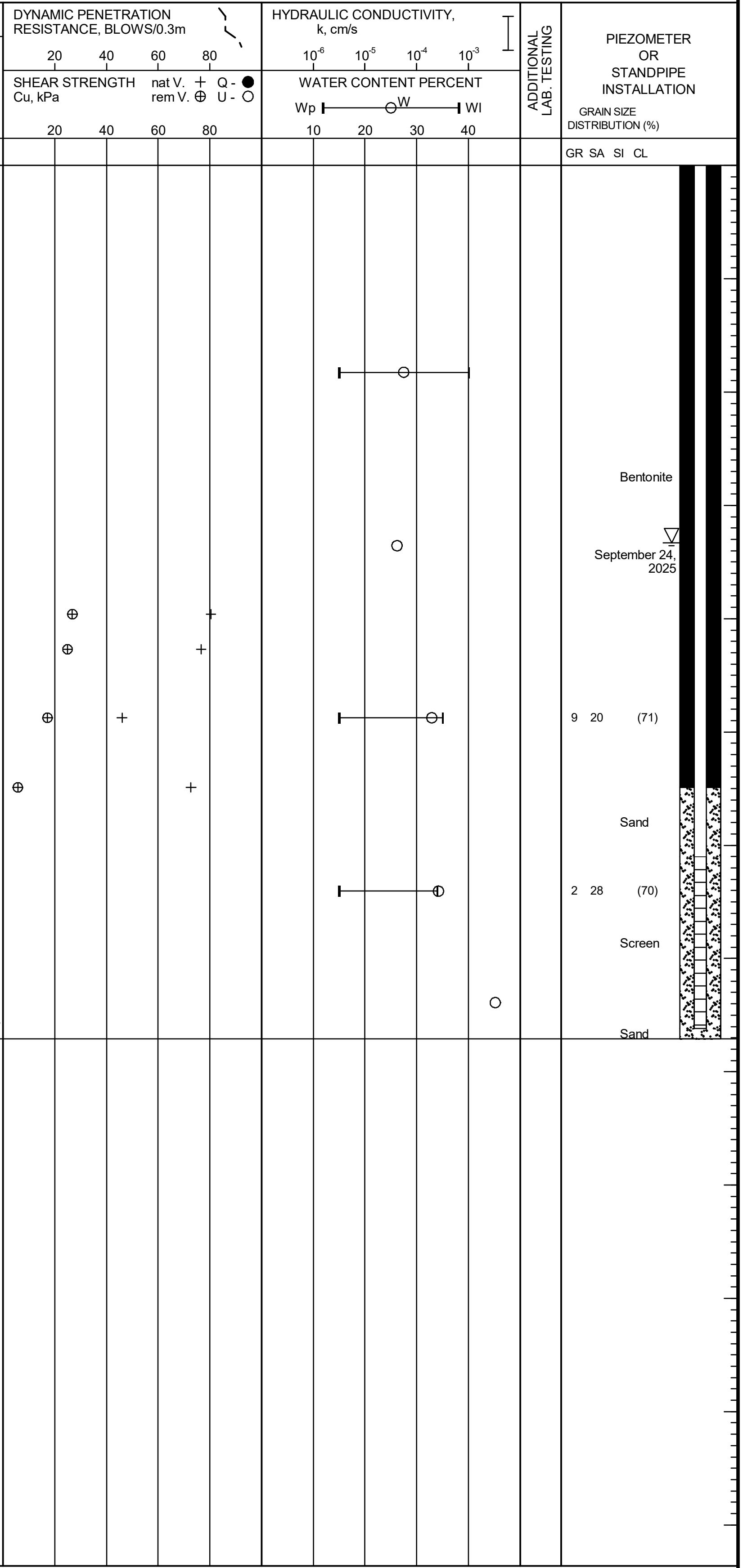
DRILL RIG: Truck mounted/ CME 11

HAMMER TYPE: AUTOMATIC

BORING METHOD: Power Auger
108 mm I.D. Hollow Stem Augers

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	90.13
ASPHALT (50 mm)	0.00
FILL - (SP) gravelly SAND; grey to brown; non-cohesive, dry (PAVEMENT STRUCTURE)	0.05 89.37
FILL - (SM) SILTY SAND, some gravel, some plastic fines; grey to brown; non-cohesive, dry to moist, loose	0.76 88.76
FILL - (CI) SILTY CLAY, some sand to sandy SILTY CLAY, some gravel, contains cobbles and boulders; grey to brown; cohesive, w>PL, firm to stiff	1.37 87.23
Possible FILL - (CI) Sandy SILTY CLAY, trace to some gravel, contains trace asphalt; grey to brown; cohesive, w>PL, firm to stiff - trace organics from about 6.1 m to 6.7 m depth - rootlets from about 6.7 m to 7.3 m depth	2.90 82.81
Possible FILL / Buried TOPSOIL - (OL/OH) SILT with organics; black to brown; moist	7.32
Possible FILL - (CI) Sandy SILTY CLAY, pockets of sand, trace gravel; grey to brown; cohesive, w>PL	7.46 82.42
END OF BOREHOLE Auger refusal at 7.71 m on inferred (BEDROCK)	7.71

SAMPLES NUMBER	TYPE	BLOWS/0.3m	GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	AS	-				
2	SS	6				
3	SS	8				
4	SS	8				
5	SS	3				
6	SS	1	9	20	(71)	
7	SS	1	2	28	(70)	
8A	SS	WH				
8B						
9	SS	100/ 0.51				



DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10 11 12

NOTE:

1. Water level measured at 3.33 m below ground surface on September 24, 2025.

DEPTH SCALE

1 : 60

LOGGED: BW/IK

CHECKED: AKP/KG

PROJECT: CA0057064.0145

RECORD OF BOREHOLE: BH25-03

SHEET 1 OF 1

LOCATION: N 5019318.73; E 434844.39

BORING DATE: August 28, 2025

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg DROP, 762mm

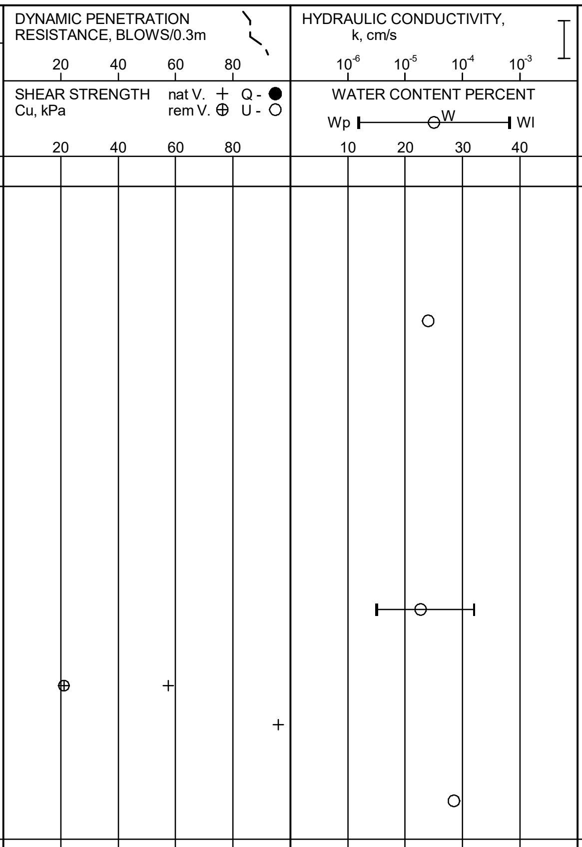
DRILL RIG: Truck mounted/ CME 11

HAMMER TYPE: AUTOMATIC

BORING METHOD: Power Auger
108 mm I.D. Hollow Stem Augers

SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)
GROUND SURFACE		89.28
ASPHALT (50 mm)	■	0.00
FILL - (SP) Poorly graded SAND, some gravel; grey to brown; non-cohesive, dry to moist (PAVEMENT STRUCTURE)	▩	89.23 0.05
FILL - (CL) SILTY CLAY and SAND, trace gravel, contains rootlets; grey to brown; cohesive fines, loose to soft	▩	88.52 0.76
FILL - (CI) Sandy SILTY CLAY, trace to some gravel; grey to brown; cohesive, w~PL, stiff - contains boulder chips	▩	87.76 1.52
Possible FILL - (CI) SILTY CLAY, trace to some sand and gravel; grey to brown; cohesive, w~PL, stiff	▨	86.38 2.90
END OF BOREHOLE		84.10 5.18

SAMPLES NUMBER	TYPE	BLOWS/0.3m	GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	AS					
2	SS	4	4	44	(52)	
3	SS	5				
4	SS	4				
5	SS	3				
6	SS	3				



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10 11 12

DEPTH SCALE

1 : 60

LOGGED: BW/IK

CHECKED: AKP/KG

PROJECT: CA0057064.0145

RECORD OF BOREHOLE: AH25-04

SHEET 1 OF 1

LOCATION: N 5019300.39; E 434862.69

BORING DATE: August 28, 2025

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg DROP, 762mm

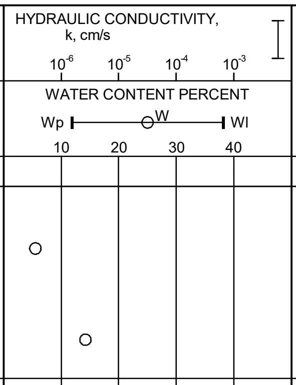
DRILL RIG: Truck mounted/ CME 11

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		89.48					GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0	Power Auger 108 mm I.D. Hollow Stem Augers	ASPHALT (70 mm)	■	0.00 0.07					
		FILL - (SP) gravelly SAND, some silt; grey brown; non-cohesive, dry (PAVEMENT STRUCTURE)	▨	88.57 0.91	1	AS			
1		FILL - (CL) SILTY CLAY and SAND, trace gravel; grey; cohesive fines, moist	▨	87.96 1.52	2	AS			
		END OF BOREHOLE							

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80



DEPTH SCALE

1 : 60

LOGGED: BW/IK

CHECKED: AKP

PROJECT: CA0057064.0145

RECORD OF BOREHOLE: AH25-05

SHEET 1 OF 1

LOCATION: N 5019273.47; E 434841.90

BORING DATE: August 29, 2025

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg DROP, 762mm

DRILL RIG: Truck mounted/ CME 11

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE				SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ├—○W—┤ Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		DESCRIPTION	STRATA PLOT	ELEV.	DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
		GROUND SURFACE		89.78								
0	Power Auger 108 mm I.D. Hollow Stem Augers	ASPHALT (50 mm)	■		0.00 0.05							
		FILL - (SP) Poorly graded SAND, some gravel, trace non-plastic fines; light brown; non-cohesive, dry (PAVEMENT STRUCTURE)	▨	88.77		1	AS					
1		FILL - (SC) gravelly CLAYEY SAND; brown; cohesive fines, dry to moist	▨	88.26	1.01	2	AS			○		
		END OF BOREHOLE			1.52							

2

3

4

5

6

7

8

9

10

11

12

DEPTH SCALE

1 : 60

LOGGED: IK

CHECKED: AKP/KG

PROJECT: CA0057064.0145

RECORD OF BOREHOLE: AH25-06

SHEET 1 OF 1

LOCATION: N 5019230.12; E 434859.51

BORING DATE: August 29, 2025

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg DROP, 762mm

DRILL RIG: Truck mounted/ CME 11

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION; GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		GROUND SURFACE		90.50							
0	Power Auger 108 mm I.D. Hollow Stem Augers	ASPHALT (100 mm)		0.00							
		FILL - (GP) Sandy GRAVEL, trace non-plastic fines; grey to brown; non-cohesive, dry (PAVEMENT STRUCTURE)		0.10	1	AS					
1		FILL - (SC) gravelly CLAYEY SAND; dark brown to grey; cohesive fines, moist		89.38 1.12	2	AS					13 52 (35)
		END OF BOREHOLE		88.98 1.52							
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											

DEPTH SCALE

1 : 60

LOGGED: IK

CHECKED: AKP/KG

PROJECT: CA0057064.0145

RECORD OF BOREHOLE: AH25-07

SHEET 1 OF 1

LOCATION: N 5019199.63; E 434867.86

BORING DATE: August 29, 2025

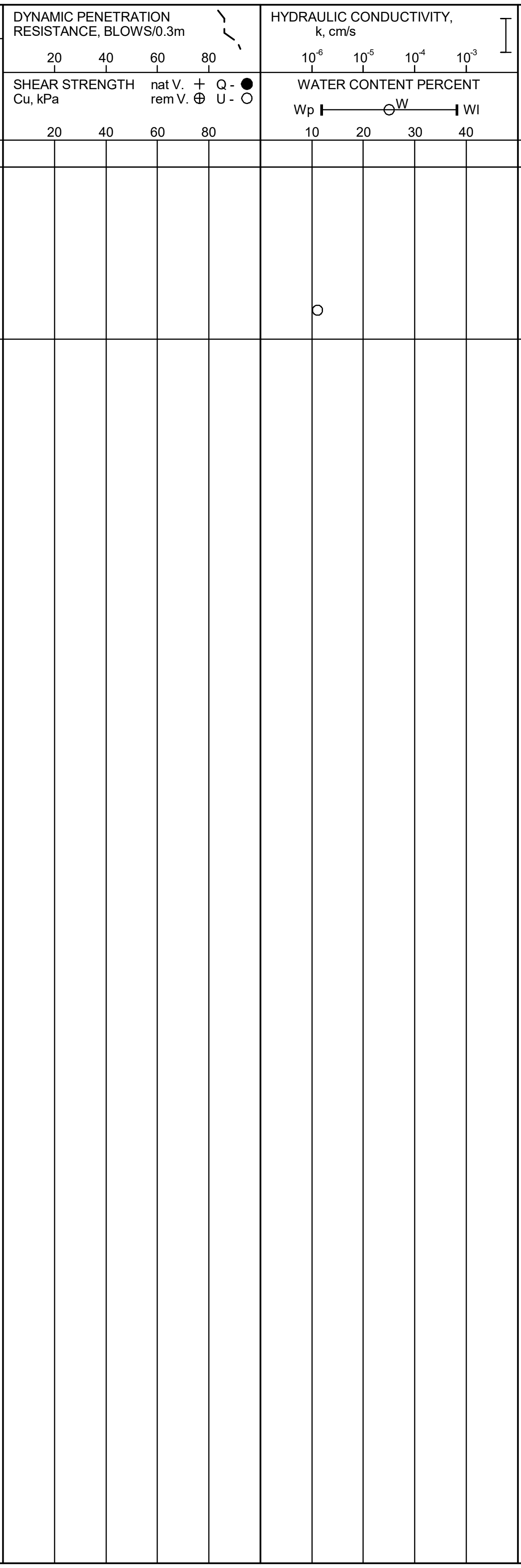
DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg DROP, 762mm

DRILL RIG: Truck mounted/ CME 11

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION GRAIN SIZE DISTRIBUTION (%)
		GROUND SURFACE		90.86					GR SA SI CL
0	Power Auger 108 mm I.D. Hollow Stem Augers	ASPHALT (100 mm)	■	0.00					
		FILL - (GP) Sandy GRAVEL, trace non-plastic fines; grey to brown; non-cohesive, dry (PAVEMENT STRUCTURE)	▨	0.10	1	AS			
1		FILL - (SC) gravelly CLAYEY SAND; dark brown to grey; cohesive fines, moist	▨	89.85 1.01	2	AS			
		END OF BOREHOLE		89.34 1.52					
2									
3									
4									
5									
6									
7									
8									
9									
10									
11									
12									



DEPTH SCALE

1 : 60

LOGGED: lk

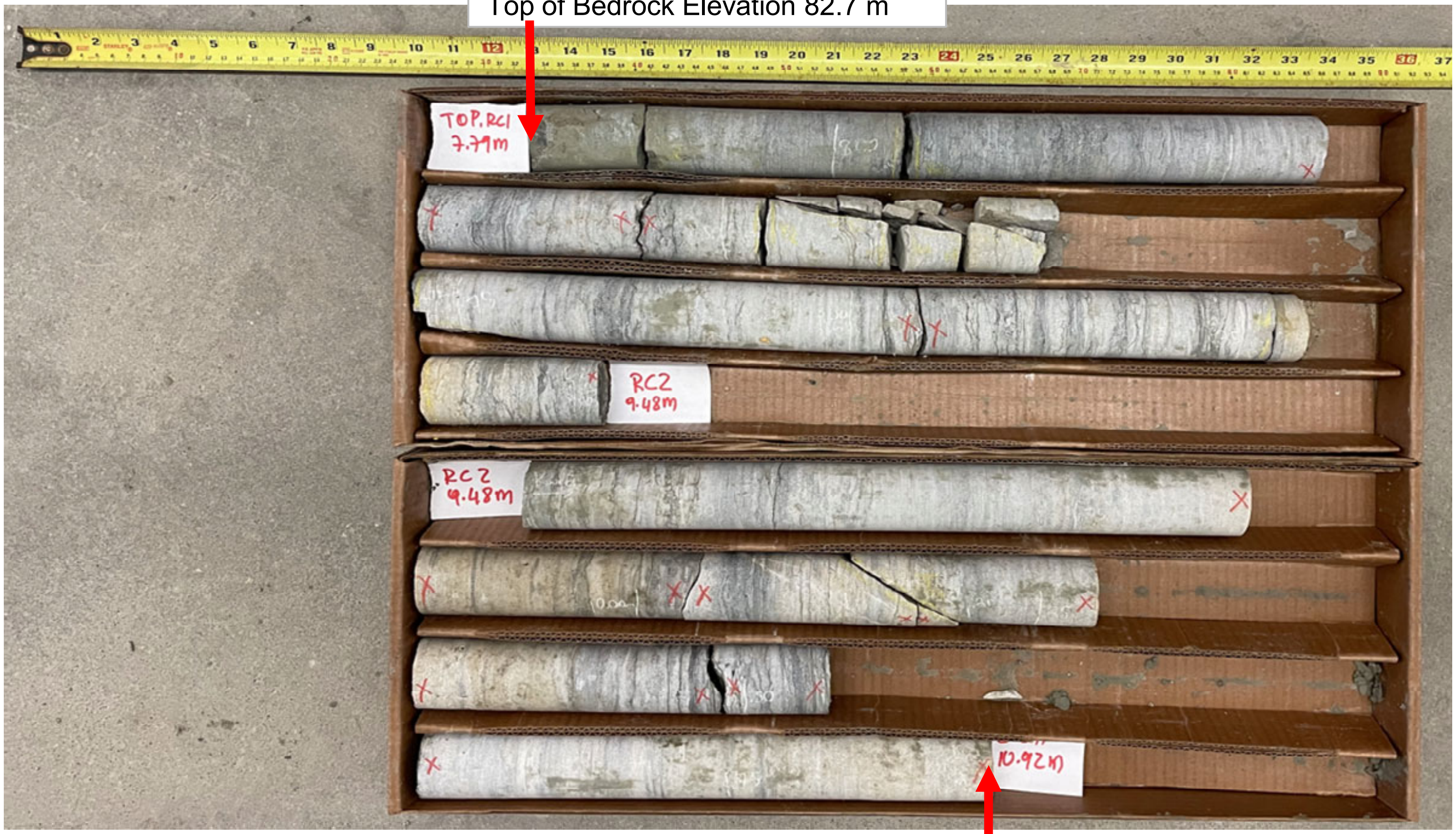
CHECKED: AKP/KG

APPENDIX B

Core Photographs

BH 25-01 (Dry)
Core Box 2 of 2

Top of Bedrock Elevation 82.7 m



Elevation 79.5 m End of Drillhole

Proposed Building Expansion
Holiday Inn Express and Suites Hotel, Ottawa, Ontario

Project No.	CA0057064.0145
Photo Taken:	AKP
Date:	2024-09-09
Checked:	KG
Review:	CH

Figure B1

BH 25-01 (Wet)
Core Box 2 of 2

Top of Bedrock Elevation 82.7 m

Elevation 79.5 m End of Drillhole

Proposed Building Expansion
Holiday Inn Express and Suites Hotel, Ottawa, Ontario

Project No.	CA0057064.0145
Photo Taken:	AKP
Date:	2024-09-09
Checked:	KG
Review:	CH

Figure B2

APPENDIX C

Record of Borehole and Auger-hole Logs - Previous Investigation

TABLE 1

RECORD OF AUGERHOLES

Augerhole Number	Depth (metres)	Description
AH 08-2 (Elev. 90.57m)	0.00 – 0.12	ASPHALTIC CONCRETE
	0.12 – 0.46	Grey crushed stone (FILL)
	0.46 – 1.52	Grey brown silty clay, trace gravel and sandy topsoil (FILL)
	1.52	End of augerhole
		Note: Augerhole dry upon completion
		Sample: 1 — Depth (m): 0.91 – 1.52
AH 08-3 (Elev. 90.40m)	0.00 – 0.12	ASPHALTIC CONCRETE
	0.12 – 0.55	Grey crushed stone (FILL)
	0.55 – 1.16	Brown sand and gravel, with cobbles (FILL)
	1.16 – 1.52	Grey brown silty clay, some gravel (FILL)
	1.52	End of augerhole
		Note: Augerhole dry upon completion
		Sample: 1 — Depth (m): 0.15 – 0.30
AH 08-4 (Elev. 90.29m)	0.00 – 0.03	TOPSOIL
	0.03 – 0.12	Grey crushed stone (FILL)
	0.12 – 0.82	Brown sand and gravel, occasional cobble (FILL)
	0.82 – 1.52	Grey brown silty clay, trace gravel and organic matter (FILL)
	1.52	End of augerhole
		Note: Augerhole dry upon completion
		Sample: 1 — Depth (m): 0.91 – 1.52

PROJECT: 08-1121-0045

RECORD OF BOREHOLE: BH 08-1A

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: April 15, 2008

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		91.17							
0	Power Auger 200mm Diam. (Hollow Stem)	TOPSOIL		0.00							
		Dense brown sandy silt, some gravel, trace clay, occasional cobbles (FILL)		90.90 0.27							
1					1	50 DO	37				
		Compact to very dense brown sand and gravel, some cobbles, trace asphalt (FILL)		89.95 1.22							
					2	50 DO	20				
2											
					3	50 DO	50				
		End of Borehole Auger Refusal		88.37 2.80							
3											
4											
5											

DEPTH SCALE

1 : 25

Golder Associates

LOGGED: D.J.S

CHECKED: K.S.L.

PROJECT: 08-1121-0045

RECORD OF BOREHOLE: BH 08-1B

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: April 15, 2008

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
		Ground Surface		91.17							
0	Power Auger 200mm Diam. (Hollow Stem)	TOPSOIL		0.00							
1		Dense brown sandy silt, some gravel, trace clay, occasional cobbles (FILL)		90.90 0.27							
2		Compact to very dense brown sand and gravel, some cobbles, trace asphalt (FILL)		89.95 1.22							
		End of Borehole Auger Refusal Note: Soil profile inferred from Borehole 08-1A		89.04 2.13							
3											
4											
5											

DEPTH SCALE

1 : 25

Golder Associates

LOGGED: D.J.S

CHECKED: K.S.L.

PROJECT: 08-1121-0045

RECORD OF BOREHOLE: BH 08-1C

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: April 15, 2008

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		91.17							
0	Power Auger 200mm Diam. (Hollow Stem)	TOPSOIL		0.00							
		Dense brown sandy silt, some gravel, trace clay, occasional cobbles (FILL)		90.90 0.27							
1											
		Compact to very dense brown sand and gravel, some cobbles, trace asphalt (FILL)		89.95 1.22							
2											
		End of Borehole Auger Refusal Note: Soil profile inferred from Borehole 08-1A		88.58 2.59							
3											
4											
5											

DEPTH SCALE

1 : 25

Golder Associates

LOGGED: D.J.S

CHECKED: K.S.L.

RECORD OF BOREHOLE 1

LOCATION See Figure 2 BORING DATE MARCH 21, 1977 DATUM LOCAL

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE ELEV'N. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FT.	ELEVATION SCALE
		ASPHALT				
	99.7	GROUND SURFACE				100
POWER AUGER 8" DIAM (HOLLOW STEM)	0.2	CRUSHED STONE				
	0.8	GREY BROWN SILTY CLAY, SOME SAND, TRACE ORGANIC MATTER (FILL)	1	2" DO	19	95
	92.7 7.0	VERY STIFF TO STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)	2	"	22	90
			3	"	14	85
			5	"	5	80
			6	"	3	
	74.7 25.0	END OF HOLE AUGER REFUSAL, PROBABY BEDROCK				75
						70

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

SHEAR STRENGTH Cu., LB./SQ.FT. NAT. V. - + Q. - ● REM.V. - ● U. - O

COEFFICIENT OF PERMEABILITY, k, CM./SEC. 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT Wp W WL

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN R.K.B./DN
CHECKED

RECORD OF BOREHOLE 2

LOCATION See Figure 2
BORING DATE MARCH 21, 1977
DATUM LOCAL
SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.
PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE ELEV'N. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FT.
	99.3	GROUND SURFACE			
POWER AUGER 8" DIAM (HOLLOW STEM)	0.0	COMPACT GREY BROWN SILTY CLAY AND SAND, OCCASIONAL ORGANIC MATTER AND GRAVEL (FILL)	1	2" D.O.	24
			2	"	7
	89.3 10.0	VERY STIFF TO STIFF GREY BROWN **SILTY CLAY** (WEATHERED CRUST)	3	"	15
			4	"	8
			5	"	6
	74.3 25.0	END OF HOLE AUGER REFUSAL, PROBABLY BEDROCK			

ELEVATION SCALE: 100, 95, 90, 85, 80, 75, 70

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

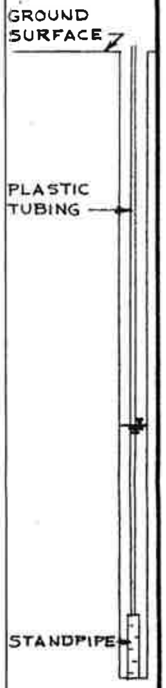
SHEAR STRENGTH Cu., LB./SQ. FT. NAT. V. - + Q. - ● REM.V. - ● U. - ○

COEFFICIENT OF PERMEABILITY, k, CM./SEC. 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT Wp W WL

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



W.L. IN STANDPIPE AT ELEV. 84.4 MAR. 24, 1977

0 / 5 / 10 / 15 Percent axial strain at failure

VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN R.K.B./DN
CHECKED

RECORD OF BOREHOLE 3

LOCATION See Figure 2 BORING DATE MARCH 22, 1977 DATUM LOCAL

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE ELEV'N. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FT.	ELEVATION SCALE
	99.8	GROUND SURFACE					100
POWER AUGER 8" DIAM (HOLLOW STEM)	0.0	BROWN TO GREY BROWN SILTY CLAY, SOME SAND, OCCASIONAL GRAVEL, ORGANIC MATTER AND CONCRETE (FILL)					
							95
				1	2" D.O.	6	
							90
							85
				2	"	7	
							80
				3	"	8	
	75.3 24.5	END OF HOLE AUGER REFUSAL PROBABLY BEDROCK					75
							70

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

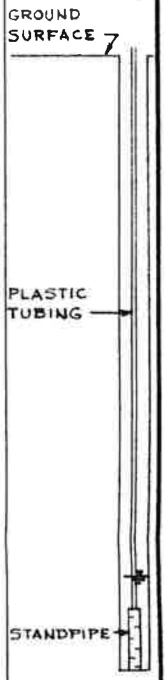
SHEAR STRENGTH Cu., LB./SQ.FT. NAT. V. - + Q. - ● REM.V. - ● U. - ○

COEFFICIENT OF PERMEABILITY, k, CM./SEC. 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT W_p W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



W.L. IN STANDPIPE AT ELEV. 79.0 MAR. 24, 1977

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN DN
CHECKED

RECORD OF BOREHOLE 4

LOCATION See Figure 2 BORING DATE MARCH 21, 1977 - DATUM LOCAL

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE		SAMPLES			ELEVATION SCALE
	ELEV'N. / DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.	
						100
	99.0	GROUND SURFACE				
POWER AUGER 8" DIAM (HOLLOW STEM)	0.0	BROWN SILTY CLAY AND SAND, SOME GRAVEL, OCCASIONAL COBBLES AND ORGANIC MATTER (FILL)	1	2" D.O.	21	95
			2	"	7	
						90
			3	"	18	
						85
			4	"	25	
						80
			5	"	10	
			6	"	14	
	75.3					
	23.7	END OF HOLE SAMPLER REFUSAL, PROBABLY BEDROCK				75
						70

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

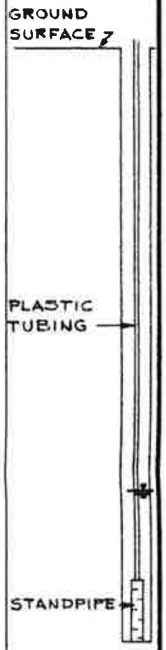
SHEAR STRENGTH Cu., LB./SQ.FT. NAT. V. - + Q. - ● REM.V. - ● U. - O

COEFFICIENT OF PERMEABILITY, k, CM./SEC. 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



W.L. IN STANDPIPE AT ELEV. 81.4 MAR. 24, 1977

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN R.K.B. / DN
CHECKED DK

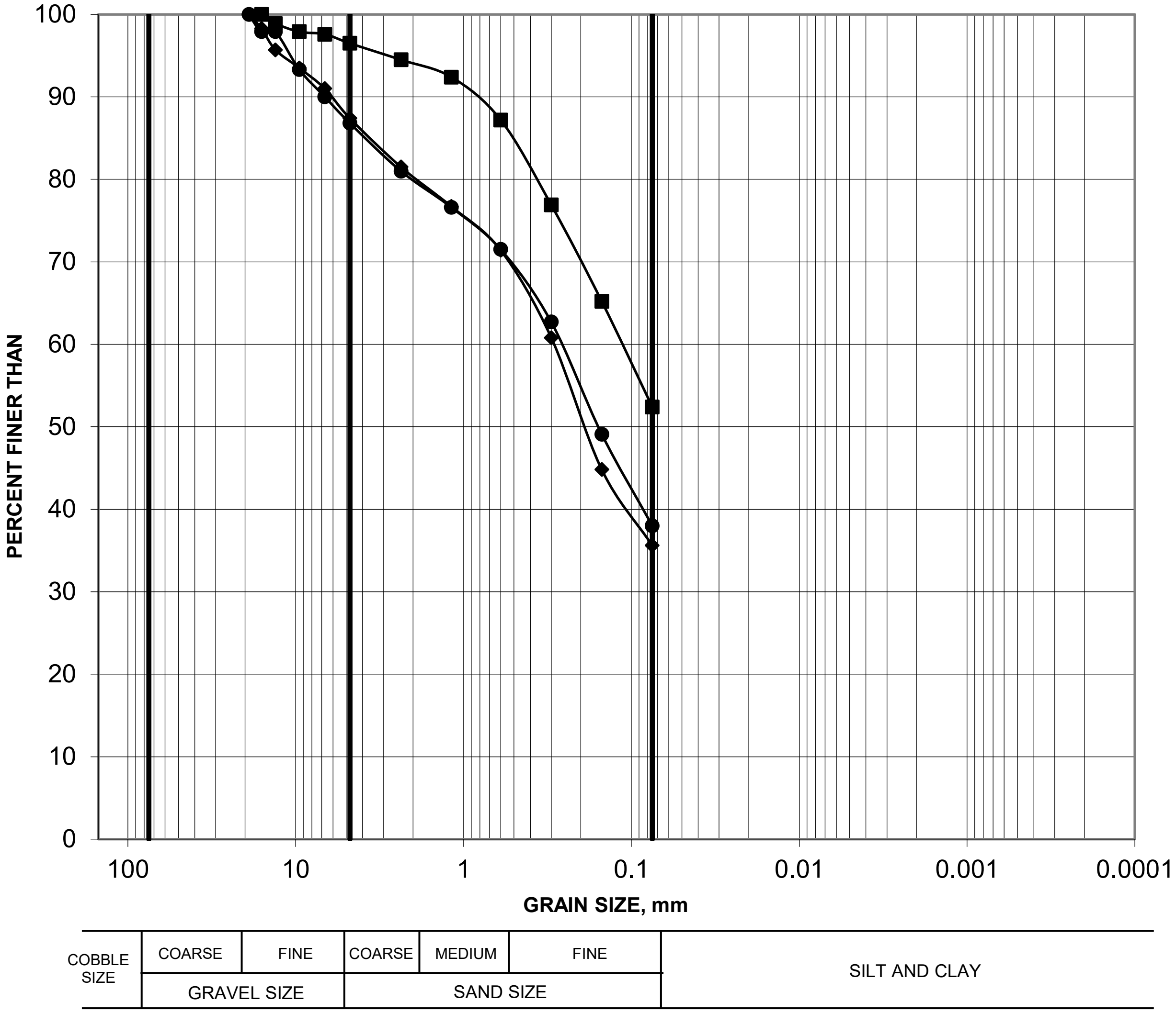
APPENDIX D

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE D1

FILL

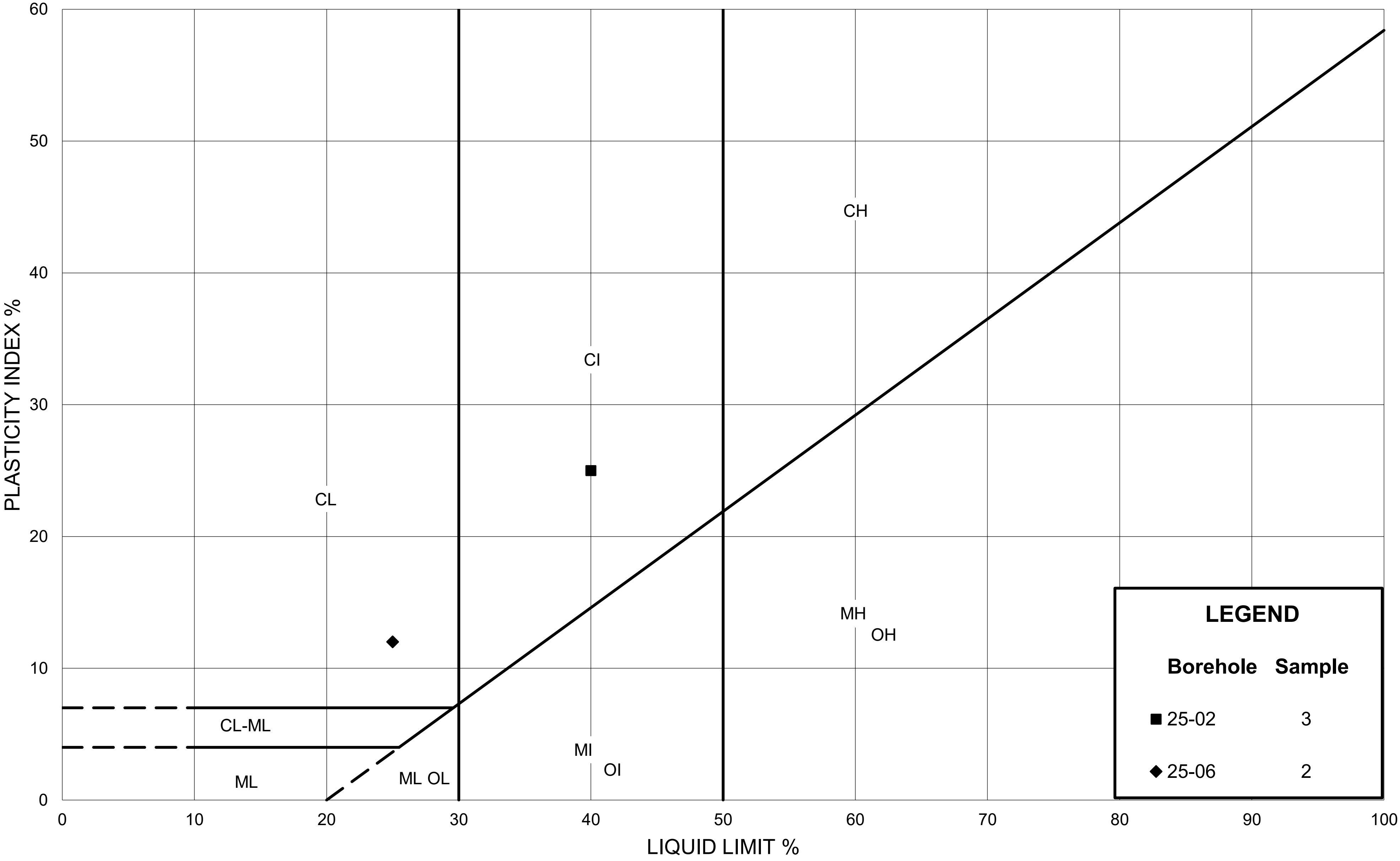


			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
● 25-01	2	0.76-1.37	13	49	38	
■ 25-03	2	0.76-1.37	4	44	52	
◆ 25-06	2	1.12-1.52	13	51	36	

Project: CA0057064.0145

Created by: IK
Checked by: KG

C:\Users\gld_kgajjar\WSP O365\CA0057064.0145 Kingslake EIS and Geotech Inv Holiday INN Ottawa - Project Folders\05. Technical\Geotech\Lab\Lab Figures\



PLASTICITY CHART

FILL

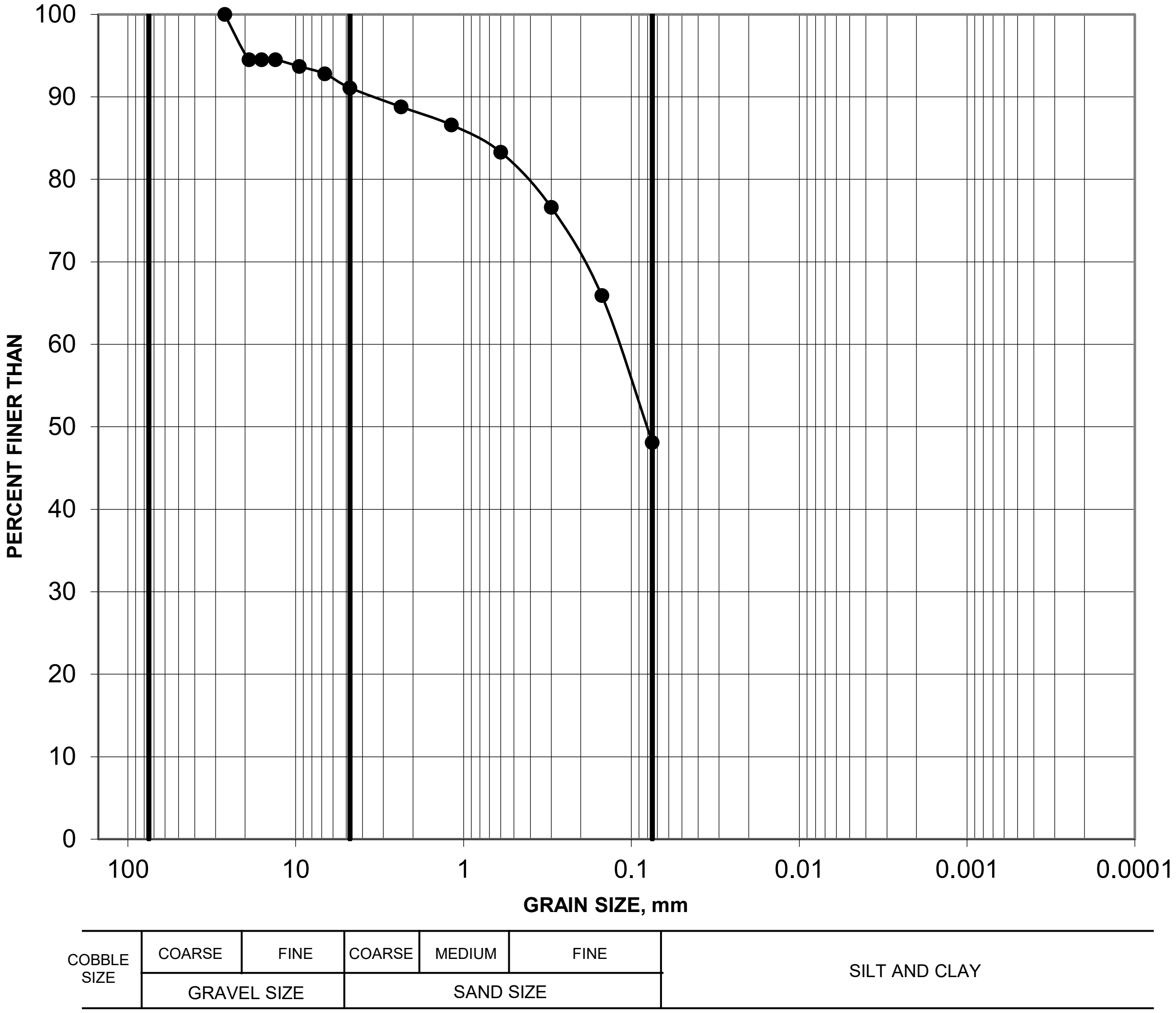
Figure: D2

Project: CA0057064.0145

Created By: IK Checked By: KG

GRAIN SIZE DISTRIBUTION	FIGURE D3

(SC) CLAYEY SAND



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
—●— 25-01	5	2.29-2.90	9	43	48	

Project: CA0057064.0145

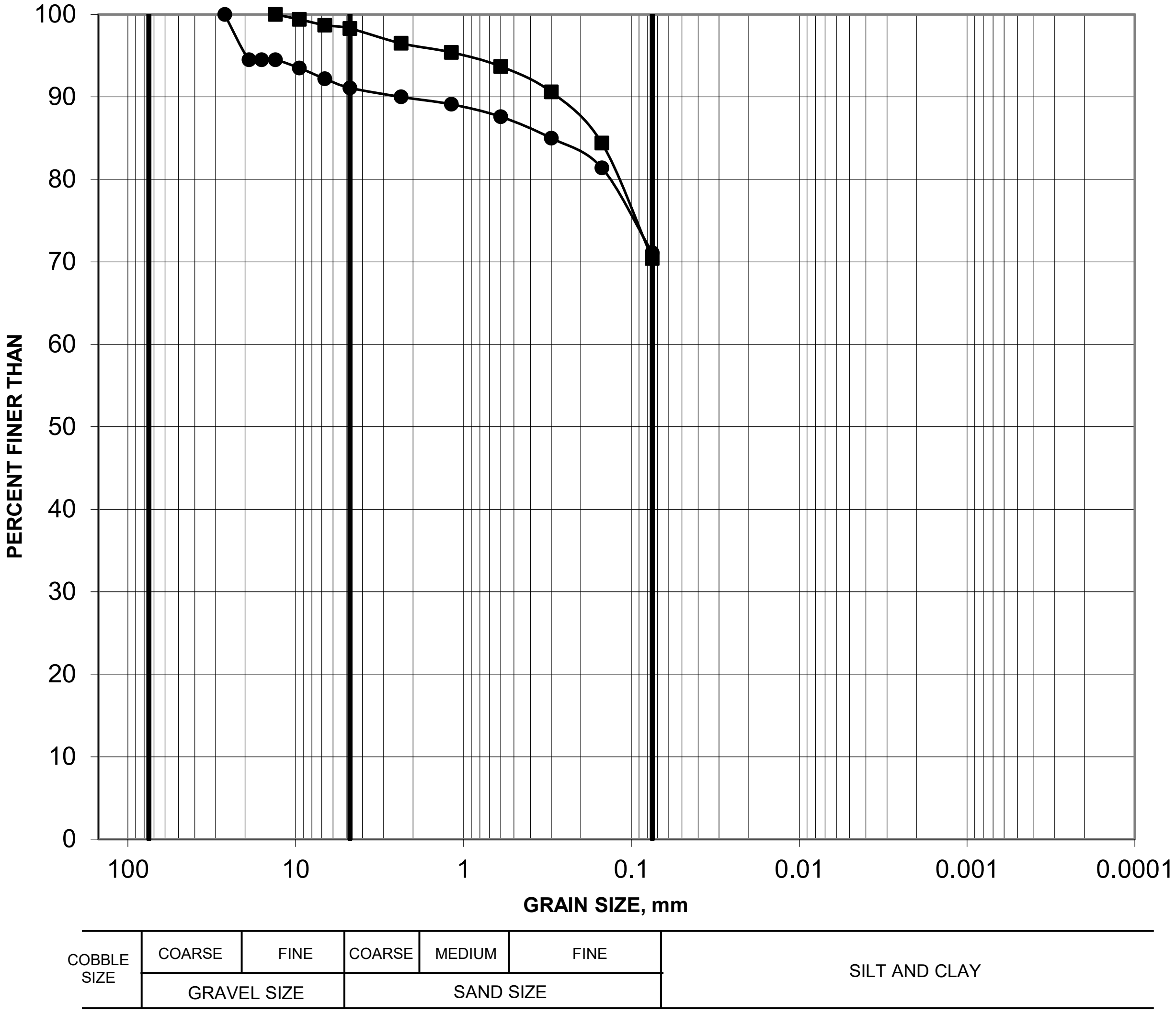
https://wsponlinecan.sharepoint.com/sites/CA-CA0057064.0145/Shared Documents/05. Technical/Geotech/Lab/Lab Figures/

Created by:	IK
Checked by:	KG

GRAIN SIZE DISTRIBUTION

FIGURE D4

(CI) SANDY SILTY CLAY

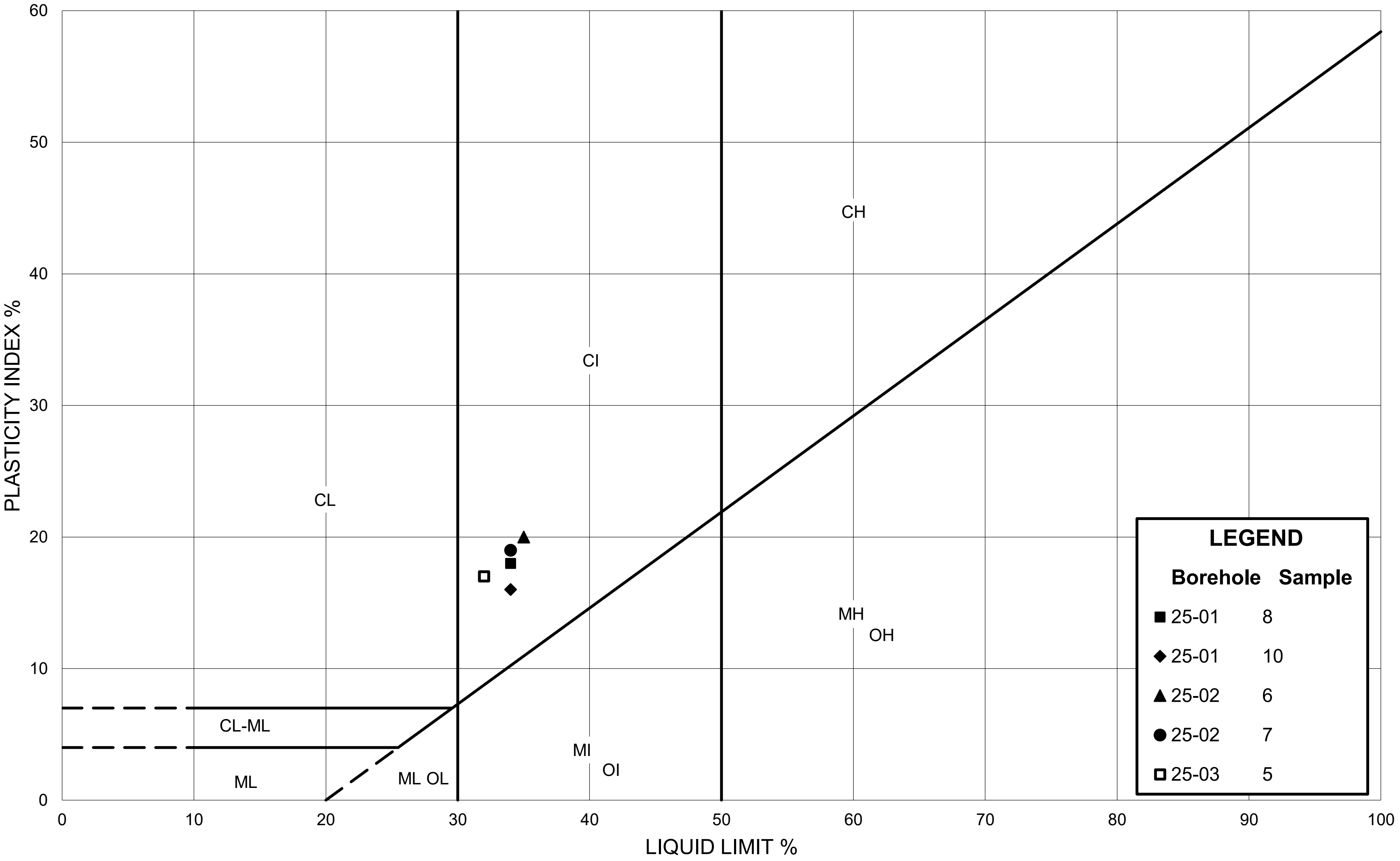


				Constituents (%)			
Borehole		Sample	Depth (m)	Gravel	Sand	Silt	Clay
●	25-02	6	4.57-5.18	9	20	71	
■	25-02	7	6.10-6.71	2	28	70	

Project: CA0057064.0145

Created by: IK
Checked by: KG

https://wsponlinecan.sharepoint.com/sites/CA-CA0057064.0145/Shared Documents/05. Technical/Geotech/Lab/Lab Figures/



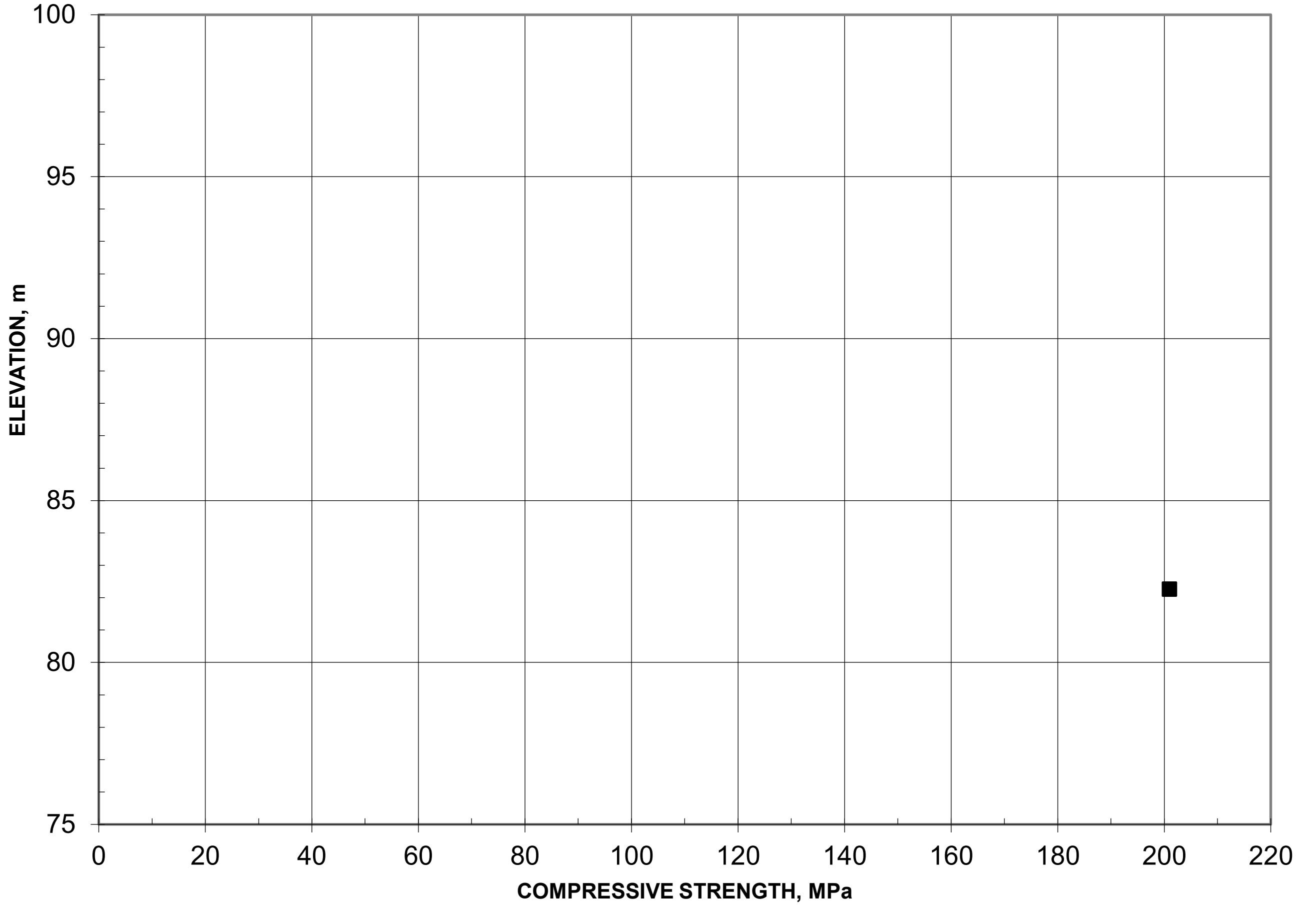
PLASTICITY CHART

SANDY SILTY CLAY (CI) to SILTY CLAY (CI)

Figure:	D5
Project:	CA0057064.0145
Created By:	IK
Checked By:	KG

ASTM D7012 - Method C
UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE
SUMMARY OF LABORATORY TEST RESULTS

FIGURE D6



	Borehole	Depth (m)	L/D	Bulk Density (kg/m^3)	Lithology	UCS (MPa)	Failure Type
■	25-01 RC1	8.0	2.3	2677	Sandstone	201	1

Notes:

Failure Types

1. Well formed cones on both ends
2. Well formed cones on one end, vertical cracks through cap
3. Columnar vertical cracking through both ends
4. Diagonal fracture with no cracking through ends
5. Side fractures at top or bottom
6. Side fractures at both sides of top or bottom

Remarks

- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Time to failure > 2 and < 15 minutes.

Project: CA0057064.0145

wsp

Created by: KG
Checked by: IK

https://wsponlinecan.sharepoint.com/sites/CA-CA0057064.0145/Shared Documents/05. Technical/Geotech/Lab/Lab Figures/

APPENDIX E

Corrosion Test Results

Certificate of Analysis

Client:	WSP Canada Inc. 1931 Robertson Road, Ottawa, Ontario .	Report Number:	3019837
		Date Submitted:	2025-09-23
		Date Reported:	2025-09-29
		Project:	CA0057064.0145
		COC #:	922391
Attention:	Mr. Kinjal Gajjar		
PO#:			
Invoice to:	WSP Canada Inc.		

Dear Kinjal Gajjar:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: ______________________

Patrick Jacques, Chemist

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: https://directory.cala.ca/.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Certificate of Analysis

Client: WSP Canada Inc.
1931 Robertson Road,
Ottawa, Ontario
.
Attention: Mr. Kinjal Gajjar
PO#:
Invoice to: WSP Canada Inc.

Report Number: 3019837
Date Submitted: 2025-09-23
Date Reported: 2025-09-29
Project: CA0057064.0145
COC #: 922391

				Lab I.D.	1782479	1782480
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2025-08-28	2025-08-28
				Sample I.D.	25-01/SA7/10'-12'	25-02/SA4/7.5'-9.5'
Group	**Analyte**	**MRL**	**Units**	**Guideline**		
Anions	Cl	0.002	%		0.035	0.308
	SO4	0.01	%		0.01	0.02
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.58	4.05
	pH	2.00			7.59	7.46
	Resistivity	1	ohm-cm		1721	247

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

Client:	WSP Canada Inc. 1931 Robertson Road, Ottawa, Ontario .	Report Number:	3019837
		Date Submitted:	2025-09-23
		Date Reported:	2025-09-29
		Project:	CA0057064.0145
		COC #:	922391
Attention:	Mr. Kinjal Gajjar		
PO#:			
Invoice to:	WSP Canada Inc.		

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 481901 **Analysis/Extraction Date** 2025-09-26 **Analyst** AsA **Method** C CSA A23.2-4B			
Chloride	<0.002 %	107	75-125
Run No 481966 **Analysis/Extraction Date** 2025-09-27 **Analyst** NaR **Method** Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	90-110
pH	8.44	99	90-110
Resistivity			
Run No 481969 **Analysis/Extraction Date** 2025-09-28 **Analyst** JaJ **Method** AG SOIL			
SO4	<0.01 %	96	70-130

Guideline = *** = Guideline Exceedence**

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

APPENDIX F

Results of Slope Stability Analysis and Site Visit Photographs

Photo 1: Looking North from the Location of Concrete Culvert – August 15, 2025

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Photo 2: Looking South from the Location of Concrete Culvert – August 15, 2025

Photo 3: Looking down to the Culvert Outlet – August 15, 2025

.

Photo 4: Looking side slopes further away from the development – August 15, 2025

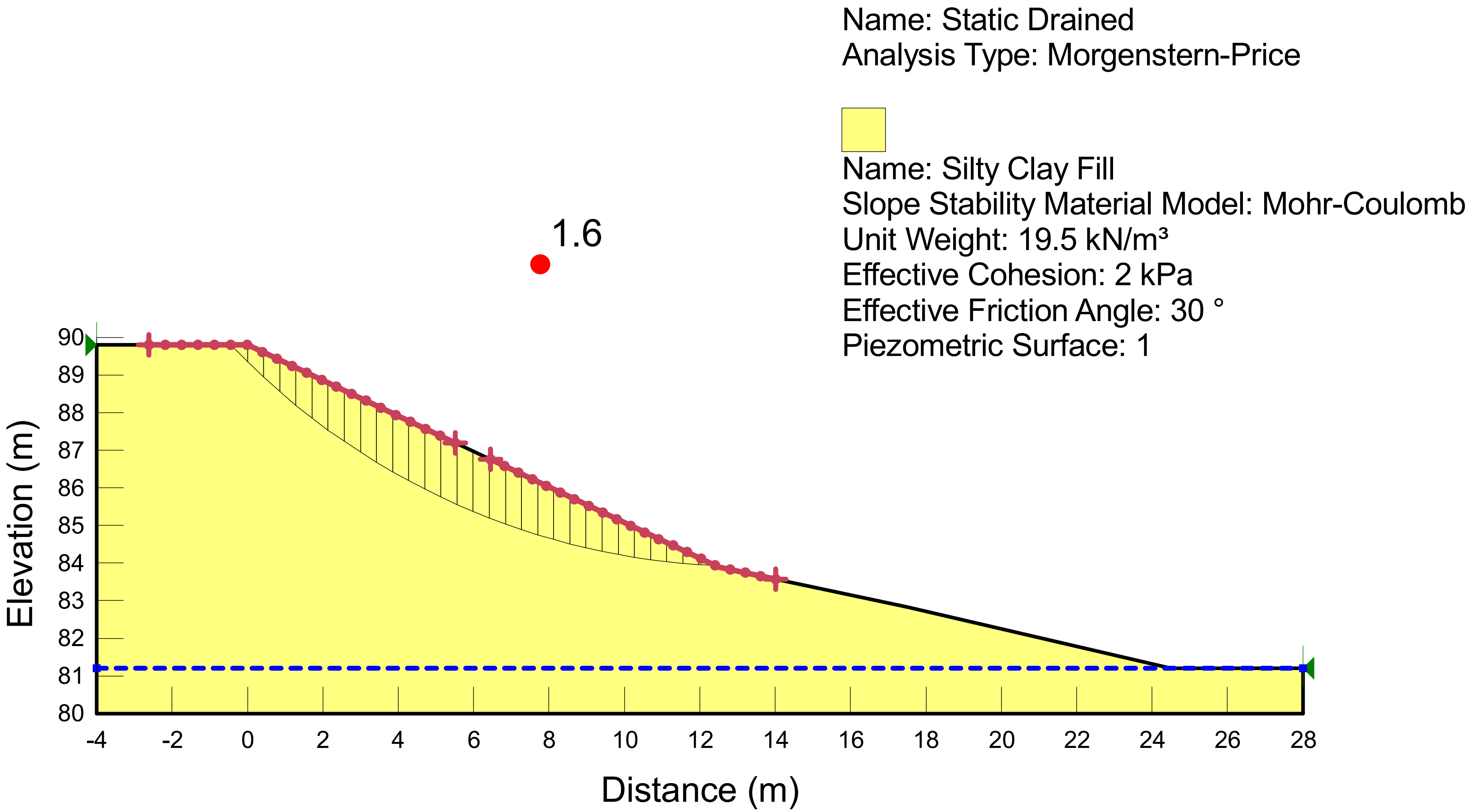

Figure F1 - Static Analysis

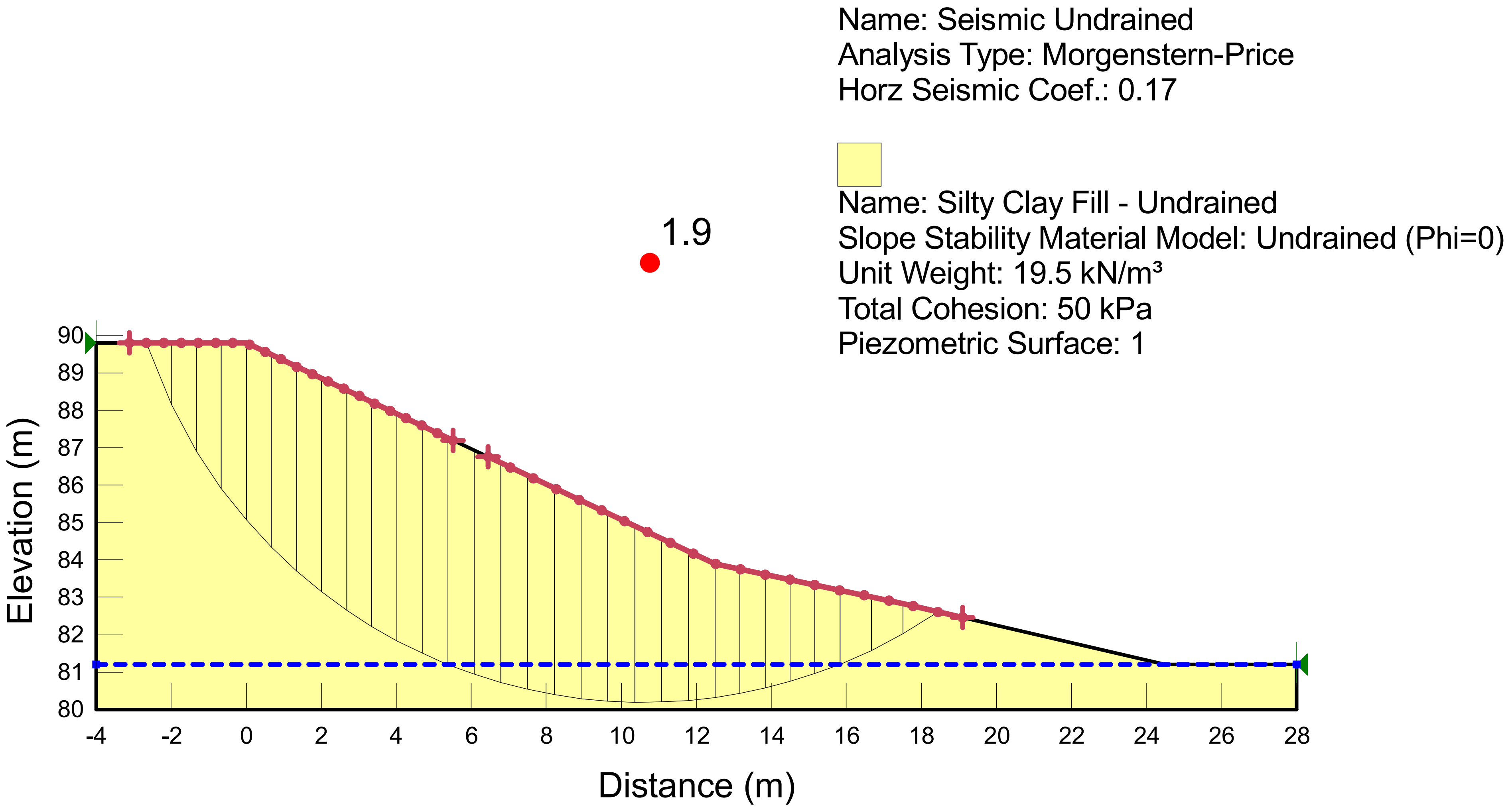

Figure F2 - Seismic Analysis

wsp.com