MEMO

TO: Diamond Schmitt Architects and KWC Architects
FROM: WSP Canada Inc.
SUBJECT: Revised Supplementary comments to the final geotechnical report Rev. 2
DATE: **July 21, 2020**

In the comments from the site Site Plan Application Plan (City of Ottawa File Number: D07-12-19-0205) it was requested that WSP Canada Inc. (WSP) geotechnical provide updates to the geotechnical report. The following sections are to complete the geotechnical report, dated December 2019.

ROCK ANCHORS

Rock anchor specifications, as detailed in the structural drawings dated March 31 and April 6, 2020, have been reviewed and are consistent with the requirements detailed in the geotechnical report dated December 2019 as well as the content of this memo.

SEISMIC SITE CLASSIFICATION

Multichannel analysis of surface waves (MASW) has been carried out on site. The aim of MASW testing is to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of Rayleigh surface waves ("ground roll"). The dispersion properties are measured as a change in phase velocity with frequency. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The Vs30 values calculated for the minimum and the maximum envelopes ranged from 189 to 2160m/s. Based on the average Vs30 values (as determined through the MASW method) and table 4.1.8.4.A of the National Building Code of Canada, 2015 Edition, the investigated area is site class "B" (760 < VS30 ≤ 1500 m/s), however as this value is not to be applied to if there is more than 3 m of soil between the rock surface, a site classification of "C" has been applied. For foundations placed within 3 meters of the underlying bedrock, on either engineered fill or the native soil, a site classification of "B" could be applied.

The shear-wave velocity measurement for seismic site classification from Geophysics GRP International Inc. has been included as an attachment to this memo.

GRADE RAISE

It is understood that a grade raise of up to 3.0 m is being proposed. Given that the building will be supported on deep foundations, a grade raise of up to 3.0 m will not cause settlement of the proposed building.

Underlying the surface in all the boreholes is a layer of fill which extends to depths ranging from 1.4 m to 6.9 m below the existing ground surface. The density of this fill was highly variable and ranged from a loose to very dense state of packing. Underlying the fill in the northwest section of the site, as well as borehole BH19-7 a layer of silty clay was encountered which may experience minor settlement with additional loading. However, in the southern section of the site, where the majority of the grade raise is proposed, the silty clay deposit was not encountered and the underlying sand and gravel or glacial till can accept a grade raise of 3.0 m.

Prior to the placement of any additional fill, unsuitable materials such as organic soils, frozen soils, etc. should be stripped and the underlying subgrade inspected by a qualified geotechnical engineering. Additional compaction and densification may be required as well as the removal of localized areas of unsuitable material which will be replaced with suitable approved fill compacted to 95%. All additional material needs to be approved prior to placement.

LATERAL EARTH PRESSURES

The active earth pressure, Ka, can be calculated as follow:

$$Ka = [(1 - \sin \emptyset)/(1 + \sin \emptyset)]$$

The passive earth pressure K_p is the inverse of the active earth pressure, K_a.

DEEP FOUNDATIONS

The soil pile interactions went into several rounds of soil data and relevant pile calculations. Considering similar piling experience within the same type of rock for the university main garage building, the following pile information and loading conditions were eventually agreed upon;

Pile size: 245 DIAMx13thk

- Factored Lateral Pile Load per pile (At underside of pile cap/top of pile): 110kN-120kN
- Anticipated Pile capacity of 1600 kN
- Maximum Lateral Displacement at top of Pile/Underside of Pile Cap: 21mm-32mm
- Factored Stiffness at top of pile/underside of pile cap for a pinned pile to the pile cap: 6.0kN/mm-12kN/mm (It takes 6kN to 12KN to move the soil 1mm at the top of the pile)

These displacement values for the soil given the cyclical/dynamic seismic loading are to be considered acceptable and this can be supported by the fact that piles will be installed in the native soil (Sand Layer/Till Layer) and not in the built up/95% compacted granular B, as excavation will only be to the base of the pile cap and piles will be installed from there.

Design Factor for Steel Pipe Piles

WSP has reviewed the pile driving analysis for the Carleton University Parking Garage P18, founded in similar soil/bedrock conditions in near proximity to the proposed Residence. Based on the dynamic capacity testing (PDA testing) carried out at this site, a design factor of 0.6 is considered appropriate for this Site as the piles constructed at Garage P18 have demonstrated that similar piles are able to carry this loading. Therefore, the allowable loading for a pile can be calculated by multiplying the capacity of the steel by 0.6 design factor.

Please note, this design factor has taken into account the soil corrosivity impacts on the proposed piles.

Anticipated Pile Capacity

The anticipated pile capacity of the piles for the proposed residence will be in the order of 1600 kN. pile testing will be required in order to verify that this loading can be achieved.

It is expected that the spacing for the piles will be greater than or equal to 3.5 d and therefore no group reduction factor will be required. If the spacing will be less that 3.5 d, WSP can provide further guidance.

Pile Uplift

Bedrock Uplift Criteria for piles:
The unfactored ultimate unit shear for a rock socket, q_s, is governed by 0.05 x the 28 day concrete compressive strength of the concrete, f'c. Based on the assumption that a concrete with an f'c value of 35MPa or greater will be used, an uplift value of 1.5 MPa can be assumed.

Uplift conditions:
For pile groups with a spacing of 2.5 d a reduction factor of 0.65 should be applied. This factor will increase in a linear manner and will have a value of 1 with a spacing of 6d.

Slab on Grade

24kPa at SLS below the slab on grade may be used. A modulus of subgrade reaction of 10,000 KN/m3 can be used, as long as the subgrade is compacted properly and 200 - 300mm of well-graded crushed sand and gravel meeting the requirements of OPSS Granular A is used under the slab as the soil at this elevation is considered fill material.

Modulus of Subgrade – Wall basement

The lateral modulus of subgrade for the soil adjacent to the basement wall can be calculated using Broms method mentioned in the report. Typically, the lateral modulus will increase along the depth of the wall until it reaches approximately 11000 - 12000 KN/m3 at 3 m depth from the ground surface. An average value may be considered between 7000 – 10,000 KN/m3 along the wall.

Concrete/Soil Friction

Sliding can be resisted between the concrete and soil using 0.4 friction factor relative to vertical loads. This friction factor is provided at elevation of 61.00, which could be suitable to silty clay and clayey Sand.

Soil Corrosivity

The corrosivity in the soil ranged from severe (389 ohm-cm) to moderate (2270 ohm-cm). A corrosive soil is anticipated, especially the silty clay. Class S-3 for any concrete works at the 4 meters just below ground surface only (The reason is that the top 4 meters material is a mixed fill)

Liquefaction Potential

The soils at the site are not considered to be susceptible to seismic liquefaction, as it is mainly silty clay and Till material while the sandy soils under the ground water table exhibits a medium to very dense state of packing with an average SPT counts ranges from 18 to more than 45.

COMMENTS FROM THE SITE PLAN APPLICATION PLAN

WSP geotechnical was asked to provide comment on the Site Servicing and Stormwater Management Design Brief (the Design Brief), from a geotechnical perspective.

As is fully described in the geotechnical report, the general site conditions encountered consists of fill overlying a layer of glacial till. Lying between these layers in about half of the boreholes a layer of silty clay which in turn was underlain by sand and gravel with cobbles/boulders also encountered. The glacial till extended to the depth of refusal between 7.6 m to 12.6 m below the existing ground surface. The bedrock depth was confirmed through coring, it was encountered at depths 10.7 to 11.8 m in depth.

This Design Brief, dated March 13, 2020, was provided to WSP on March 20th, 2020 and has been reviewed. It is understood that the proposed building will be serviced by dual 250 mm diameter water services connected to the relocated 400mm diameter watermain east of the proposed building and the invert elevations for the watermains range from 63.89 to 60.49 m. The design criteria in the report indicates that unless otherwise insulated that the minimum depth of cover will be 2.4 m. Based on the stratigraphy encountered during the borehole investigations, this would place the invert of the watermains on either the silty clay or sand and gravel deposit.

It is also understood that twenty-one Precast Concrete Maintenance (PCM) holes, as per OPSD 0701.0100 or 0701.0110, for both the storm and sanitary sewer systems are to be installed, with diameters of either 1200 or 1500 mm. It is understood that a 600-mm diameter concrete pipes will be connected the PCM storm sewer structures and 200mm diameter PVC pipes will be connected to the sanitary service PCM. The invert elevations of these structures range from 63.4 m to 58.25 m where silty clay, sand and gravel and glacial till were encountered. As with all structures, the subgrade for the PCM should be inspected prior to the installation.

Based on the water levels recorded in November 2019, the water table ranged in elevation between 59.5 m and 60.2 m. Additional water levels were taking in March 2020 and were found to be below the November 2019 water levels. The requirement of the joints for the sanitary service within the design specifications are listed as a minimum hydrostatic pressure of 345 kPa (50 psi) without leakage, list as equivalent to 35 m of hydrostatic head. Given these design requirements, WSP agrees with the Design Brief that these pipes are suitable for installation under the water table. The minimum cover of 2.4 m over the proposed watermains is also considered suitable for pipes installed within a zone which may be below the water table. The storm system, consisting of concrete pipes is also considered acceptable for installation below the groundwater table given the proposed depth of cover listed in the Design Brief.

Based on the drawings in Appendix F provided by Aco Systems Ltd., it is understood that northern and southern detention tanks are to be constructed using ACO StormBrixx for as surface water retention. The north detention tank is listed is having an invert elevation of 63.62 m and top elevation of 64.62 m. The south detention tank is listed is having an invert elevation of 60.85 m and top elevation of 62.6 m. Again, the highest water levels recorded in November 2019, recorded the water table ranging in elevation between 59.5 m and 60.2 m. Based on these recorded values, there is no concern about uplift forces for the northern tank. The invert of the southern tank is approximately 650 mm above the highest recorded water level. Given that the surface elevation at the location of the southern tank will be approximately 65.5 m at this location, approximately 2.9 m of fill will be placed above the retention structure. This amount of fill would provide sufficient weight to counter uplift forces applied on the structure. How the structure itself will respond to these forces not commented upon. It should however be noted that that based on the closest boreholes to the proposed retention tanks, the founding level of the retention structures may be sitting upon granular fill material and this surface will require inspection from a qualified geotechnical engineer/technician prior to it being approved. Over excavation may also be required.

Of note, references are made to the Geotechnical Investigation Report (NO. 191-12948-00 dated December, 2019). This memo is to be complemented by an addendum to this geotechnical report. Also, of note, the soil description within the Design Report have not mentioned of layer of silty clay which was encountered in about half the boreholes at the site.

ADDITIONAL DESIGN REVIEW

WSP was provided with the plans by Read Jones Christoffersen Ltd (RJC) entitled CU NSR Foundation Permit Structural-S200 and CU NSR Foundation Permit Structural-S701 both dated April 7th, 2020 in order to review the design of the proposed tunnel from a geotechnical perspective. These documents have been included with this design memo. Within document S200 it outlines that the design requirements for the tunnel slab as well as the providing instruction to refer to general notes and typical details for tunnel geotechnical requirements and well as to coordinate with the geotechnical consultant (WSP). Based on the proposed details, the tunnel foundation will be approximately 5.85 m below the existing ground surface. The geotechnical investigation carried out in 2019 did not have boreholes along the proposed alignment of the tunnel, but based on the general soils information within the building footprint, the USL and SLS values for a tunnel foundation placed on either the sand and gravel deposit or the glacial till deposit can be taken to be 200 kPa and 100 kPa respectively. Prior to the placement of the tunnel foundation the bearing soil will have to be inspected by a qualified geotechnical engineer and this assumption be verified. It should be understood by any contractor taking on the excavation of the tunnel foundation that localized area along the proposed alignment may not be suitable and require over excavation. These areas will be placed with an approved granular fill compacted to 100% of the materials standard proctor maximum dry density (SPMDD) value.

The document, Retaining wall design OTT.124933.0001-RJC-20200505-AKP-CUNSR Retaining Wall Design, by RJC was provided in order to review and provide comment that the retaining wall has been designed as per the geotechnical report. This document has been included as part of this memo. The design has been reviewed from a geotechnical perspective. As a general comment, the retaining wall design refers back to the geotechnical report dated December 2019 and the subsequent memos and as such is consistent with the geotechnical recommendations. The required bearing capacity will need to be verified during the excavation by a qualified geotechnical engineer. It should also be noted that over excavation may be required if soil which does not meet the required bearing capacity is encountered and granular fill may be required to be placed and compacted to 100% of the SPMDD value.

Regarding the galvanized steel casing, the document 190444600_CU NSR Casing Detail_11May2020 was provided to WSP and is included with this memo. From a geotechnical perspective, a minimum clearance of 300 mm of cover is required for all pipe installations and the inclusion of a steel casing needs to take this in consideration.

Also of note, regarding the matter of clay seal for pipe trenches, it is the recommendation of WSP that clay seals should be installed as per City Of Ottawa Standard Detail S8. This document is attached as part of this memo.

CLOSURE

In preparation for this report, numerous documents were reviewed and reference. These documents are provided as an attachment to this document. Other than the topics covered above, the recommendations of the geotechnical report dated December 2019 are still applicable.

WSP Canada Inc.

Report prepared by:

Daniel Wall	**Mohamed Elsayed**
Intermediate Geotechnical Engineer, M.Eng, P. Eng.	**Senior Geotechnical Engineer, M.Eng., P.Eng**

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Attachments:

Carleton University New Student Residence – Grading Plan (03/13/20)

GPR19-01875_WPS Canada_Carleton University

CU NSR Foundation Permit Structural-S200

CU NSR Foundation Permit Structural-S701

OTT.124933.0001-RJC-20200505-AKP-CUNSR Retaining Wall Design

190444600_CU NSR Casing Detail_11May2020

City Of Ottawa Standard Detail S8

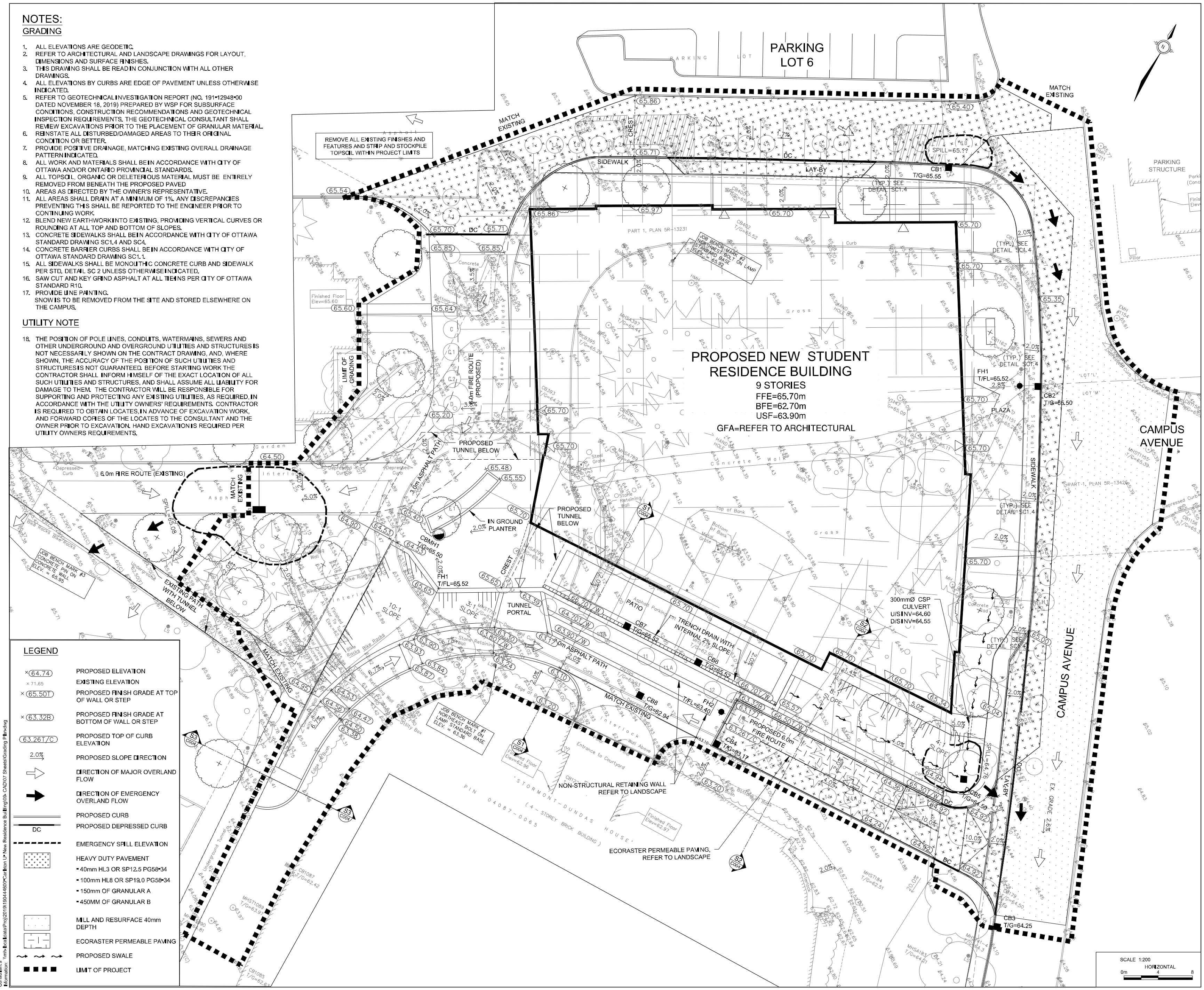
NOTES:

GRADING

1. ALL ELEVATIONS ARE GEODETIC.
2. REFER TO ARCHITECTURAL AND LANDSCAPE DRAWINGS FOR LAYOUT, DIMENSIONS AND SURFACE FINISHES.
3. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH ALL OTHER DRAWINGS.
4. ALL ELEVATIONS BY CURBS ARE EDGE OF PAVEMENT UNLESS OTHERWISE INDICATED.
5. REFER TO GEOTECHNICAL INVESTIGATION REPORT (NO. 191-12948-00 DATED NOVEMBER 18, 2019) PREPARED BY WSP FOR SUBSURFACE CONDITIONS, CONSTRUCTION RECOMMENDATIONS AND GEOTECHNICAL INSPECTION REQUIREMENTS. THE GEOTECHNICAL CONSULTANT SHALL REVIEW EXCAVATIONS PRIOR TO THE PLACEMENT OF GRANULAR MATERIAL.
6. REINSTATE ALL DISTURBED/DAMAGED AREAS TO THEIR ORIGINAL CONDITION OR BETTER.
7. PROVIDE POSITIVE DRAINAGE, MATCHING EXISTING OVERALL DRAINAGE PATTERN INDICATED.
8. ALL WORK AND MATERIALS SHALL BE IN ACCORDANCE WITH CITY OF OTTAWA AND/OR ONTARIO PROVINCIAL STANDARDS.
9. ALL TOPSOIL, ORGANIC OR DELETERIOUS MATERIAL MUST BE ENTIRELY REMOVED FROM BENEATH THE PROPOSED PAVED
10. AREAS AS DIRECTED BY THE OWNER'S REPRESENTATIVE.
11. ALL AREAS SHALL DRAIN AT A MINIMUM OF 1%. ANY DISCREPANCIES PREVENTING THIS SHALL BE REPORTED TO THE ENGINEER PRIOR TO CONTINUING WORK.
12. BLEND NEW EARTHWORK INTO EXISTING, PROVIDING VERTICAL CURVES OR ROUNDING AT ALL TOP AND BOTTOM OF SLOPES.
13. CONCRETE SIDEWALKS SHALL BE IN ACCORDANCE WITH CITY OF OTTAWA STANDARD DRAWING SC1.4 AND SC4.
14. CONCRETE BARRIER CURBS SHALL BE IN ACCORDANCE WITH CITY OF OTTAWA STANDARD DRAWING SC1.1.
15. ALL SIDEWALKS SHALL BE MONOLITHIC CONCRETE CURB AND SIDEWALK PER STD. DETAIL SC 2 UNLESS OTHERWISE INDICATED.
16. SAW CUT AND KEY GRIND ASPHALT AT ALL TIE INS PER CITY OF OTTAWA STANDARD R10.
17. PROVIDE LINE PAINTING. SNOW IS TO BE REMOVED FROM THE SITE AND STORED ELSEWHERE ON THE CAMPUS.

UTILITY NOTE

18. THE POSITION OF POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWING, AND, WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK THE CONTRACTOR SHALL INFORM HIMSELF OF THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES, AND SHALL ASSUME ALL LIABILITY FOR DAMAGE TO THEM. THE CONTRACTOR WILL BE RESPONSIBLE FOR SUPPORTING AND PROTECTING ANY EXISTING UTILITIES, AS REQUIRED, IN ACCORDANCE WITH THE UTILITY OWNERS' REQUIREMENTS. CONTRACTOR IS REQUIRED TO OBTAIN LOCATES, IN ADVANCE OF EXCAVATION WORK, AND FORWARD COPIES OF THE LOCATES TO THE CONSULTANT AND THE OWNER PRIOR TO EXCAVATION. HAND EXCAVATION IS REQUIRED PER UTILITY OWNERS REQUIREMENTS.



LEGEND

64.74	PROPOSED ELEVATION
71.65	EXISTING ELEVATION
65.50T	PROPOSED FINISH GRADE AT TOP OF WALL OR STEP
63.32B	PROPOSED FINISH GRADE AT BOTTOM OF WALL OR STEP
63.26T/C	PROPOSED TOP OF CURB ELEVATION
2.0%	PROPOSED SLOPE DIRECTION
	DIRECTION OF MAJOR OVERLAND FLOW
	DIRECTION OF EMERGENCY OVERLAND FLOW
	PROPOSED CURB
DC	PROPOSED DEPRESSED CURB
	EMERGENCY SPILL ELEVATION
	HEAVY DUTY PAVEMENT • 40mm HL3 OR SP12.5 PG58-34 • 100mm HL8 OR SP19.0 PG58-34 • 150mm OF GRANULAR A • 450MM OF GRANULAR B
	MILL AND RESURFACE 40mm DEPTH
	ECORASTER PERMEABLE PAVING
	PROPOSED SWALE
	LIMIT OF PROJECT

Consultant's Information: \\mh-localdata\Proj\2019\190444600\Carleton U New Residence Building\09- CAD\07 Sheets\Grading Plan.dwg

CONTRACTOR MUST CHECK & VERIFY ALL DIMENSIONS ON THE JOB.

DO NOT SCALE DRAWINGS.

THIS DRAWING IS NOT TO BE USED FOR CONSTRUCTION UNTIL SIGNED BY THE ARCHITECT.

MORRISON HERSHFIELD
200-2932 BASELINE ROAD, OTTAWA, ON K2H 1B1

SITE

KEY PLAN

PRINCE OF WALES · COLONEL BY · BRONSON · BREWER · RIDEAU RIVER

ISSUED

No.	Date	Description
1	13/12/2019	ISSUED FOR SITE PLAN APPROVAL
2	30/01/2020	ISSUED FOR 100% DD
3	13/03/2020	REISSUED FOR GRADING PLAN APPROVAL

LICENSED PROFESSIONAL ENGINEER
J. G. FOCKES
100202520
Mar 13, 2020
PROVINCE OF ONTARIO

CARLETON UNIVERSITY NEW STUDENT RESIDENCE

CARLETON UNIVERSITY
1125 COLONEL BY DRIVE
OTTAWA ON
K1S 5B6

GRADING PLAN

Scale: 1:200
Project No: 190444600
Date: 03/13/20

C003

GEOPHYSICS GPR INTERNATIONAL INC.

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Longueuil (Québec)
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January 6th , 2020

GPR Ref.: GPR-19-01875

Daniel Wall, P.Eng.
Geotechnical Engineer
WSP Canada Inc.
2611 Queensview Dr., Suite 300
Ottawa (ON) K2B 8K2

RE: Shear-Wave Velocity Sounding for Site Class Determination at Campus Avenue, Carleton University, Ottawa, ON.

Dear Mr. Wall:

Geophysics GPR International Inc. has been requested by WSP Canada Inc. to carry out a shear-wave velocity measurement for seismic site classification at the above site in Ottawa. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads.

The MASW surveys were performed on December 6th, 2019.

The investigation included the multi-channel analysis of surface waves (MASW) and the Extended SPatial AutoCorrelation (ESPAC) methods.

The following paragraphs describe the survey design, the principles of the test method, the methodology for interpreting the data, and provide a culmination of the results in table format.

METHODS PRINCIPLES

MASW Survey

The *Multi-channel Analysis of Surface Waves* (MASW) and the *Extended SPatial AutoCorrelation* (ESPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface waves ("ground roll"). The MASW is considered an "active" method, as the seismic signal is induced at known location and time in the geophones spread axis. Conversely, the ESPAC is considered a "passive" method, using the low frequency "noises" produced far away. The method can also be used with "active" seismic source records. The dispersion properties are expressed as a change of phase velocities with frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding). Figure 3 schematically outlines the basic operating procedure for the MASW method.

Figure 4 illustrates an example of one of the MASW/ESPAC records, the corresponding spectrogram analysis and resulting 1D V_S model. The ESPAC method allows deeper Vs soundings, but generally with a lower resolution for the surface portion. Its dispersion curve can then be merged with the higher frequency one from the MASW to calculate a more complete inversion.

Seismic Refraction Survey

The method consists in measuring the propagation delays of the direct and refracted seismic waves (P and/or S) produced by an artificial source in the axis of a seismic linear spread. The seismic velocities of the materials can be directly calculated, then the refractors depths.

SURVEY DESIGN

The geometry of an MASW survey is similar to that of a seismic refraction investigation (i.e. 24 geophones in a linear array). The fundamental principle involves intentionally generating an acoustic wave at the surface and digitally recording the surface waves from the moment of source impact with a linear series of geophones on the surface. This is referred to as an "active source" method. A sledgehammer was used as the primary energy source with traces being recorded at 6 locations: approximately 20 m off both ends and at both ends of the spread. Data were collected with geophones spacing of 3 m and 1m for a total of 8 shot records.

The theoretical maximum depth of penetration (34.5 m) is half of the maximum seismic array length (69 m), in practice the maximum depth of penetration is often influenced by the geology.

The seismic records counted 12,000 data, sampled at 250 µs for the MASW surveys, and 16,000 data, sampled at 62.5 µs for the seismic refraction. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records. Unlike the refraction method, which allows producing a result point beneath each geophone, the shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length. The seismic records were made with a Geometrics Geode Seismograph, and the geophones were 4.5 Hz.

Interpretation Method and Accuracy of Results

MASW Surveys

The main processing sequence involved plotting, picking, and 1-D inversion of the MASW shot records using the SeisimagerSW™ software package. In theory, all MASW shot records should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation and localized surface variations. The results of the inversion process are inherently non-unique and the final model must be judged to be geologically realistic. The inversion modelling also assumes that all layering is flat/horizontal and laterally uniform.

The results of the MASW tests are presented in chart format as Figure 5. The chart presents the 1-D shear wave velocity values from the inversion models of the seismic records.

The Vs30 values for the soundings are presented in Table 1. The Vs30 values are based on the harmonic mean of the shear wave velocities over the upper 30 m. The Vs30 value is calculated by dividing the total depth of interest (e.g. 30 m) by the sum of the time spent in each velocity layer up to that depth. This harmonic mean value reflects the equivalent single layer response.

The estimated error in the average Vs30 value determined through MASW tests is typically +/-10 to 15% for overburden sites. The shear-wave velocities modelled through the MASW method within bedrock have a higher estimated error.

Seismic Refraction surveys

The General Reciprocal Method was used, with signal sources at both ends of the seismic spreads, to consider seismic wave propagation for two opposite directions. The seismic wave's arrival times were identified for each geophone. The measurements were realised to calculate the rock depth (using P waves).

More detailed descriptions of these methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015.

CONCLUSION

The approximate location of the shear-wave sounding is indicated in Figures 1 and 2.

The shear-wave models are presented in Figure 5. The results are summarized in Table 1. The background seismic noise levels at this site were low. The quality of the seismic records and the resulting dispersion curves was good; the shear-wave velocities for the bedrock were constrained by the MASW and refraction methods indicate bedrock between depths of 12 m and 13 m.

Borehole data from previous studies indicate bedrock around depths of 10.7 m and 17.8 m below grade. Simple seismic refraction calculation reached the depth to a competent bedrock where a compressional wave velocity of approximately 4100 m/s. The MASW models have been constrained to fit with the seismic refraction data with consideration for the nearby borehole data.

Table 1. Calculated Vs30 values (m/s) from the MASW data

Depth	Vs		
	Min.	Median	
(m)	(m/s)	(m/s)	(m/s)
0	219.2	227.1	233.1
0.9	181.7	186.4	189.8
2.0	176.2	202.3	231.1
3.3	231.7	301.2	352.6
4.8	730.2	1012.5	1418.2
6.5	984.6	1190.6	1439.4
8.3	1234.3	1322.3	1465.9
10.4	1383.5	1432.2	1509.7
12.6	2083.5	2085.2	2138.2
15.1	2098.3	2100.0	2150.3
17.7	2105.7	2107.4	2155.1
20.5	2123.0	2124.6	2160.0
23.5	2132.8	2134.5	2155.0
26.6	2150.0	2151.6	2155.5
30			

Vs30 (m/s)	836.2
Site class	C*

* Conditional on the NBC 2015 Commentary ‘J’ requirements

Based on the average Vs30 values (0 to 30 m below grade) as determined through the MASW method, and table 4.1.8.4.A of the National Building Code of Canada, 2015

Edition, site class “C” (360 < VS30 ≤ 760 m/s) could be considered for the investigated site; however, this site class could be superseded by the presence of peat (indicated in historic boreholes) and/or other sensitive soils. Sites with more than 3 m of soft soils may require application of seismic site class ‘E’ or ‘F’ based on the geotechnical data and liquefaction risk analysis.

The use of site class “B” is conditional on the requirements of Commentary “J” sentence 100, specifically, *“Site Classes A and B, are not to be used if there is more than 3 m of soil between the rock surface and the bottom of the spread footing or mat foundation, even if the computed average shear wave velocity is greater than 760m/s*”.

As noted, the site classification provided in this report is based solely on the Vs30 value as derived from the MASW method and it can be superseded by other geotechnical information. This geotechnical information includes, but is not limited to, the presence of sensitive and/or liquefiable soils, peat, more than 3m of soft clays, high moisture content, etc. The reader is referred to section 4.1.8.4 of the National Building Code of Canada, 2015 Edition for more information on the requirements for site classification.

The V_S values calculated are representative of the in-situ materials and are not corrected for the total and effective stresses.

The interpretation of the seismic data and preparation of this report was performed by Andrés Rincón, M.Sc., and reviewed by Lhoucin Taghya, P.Geo.

Lhoucin Taghya, P.Geo.
Geophysicist

PROFESSIONAL GEOSCIENTIST
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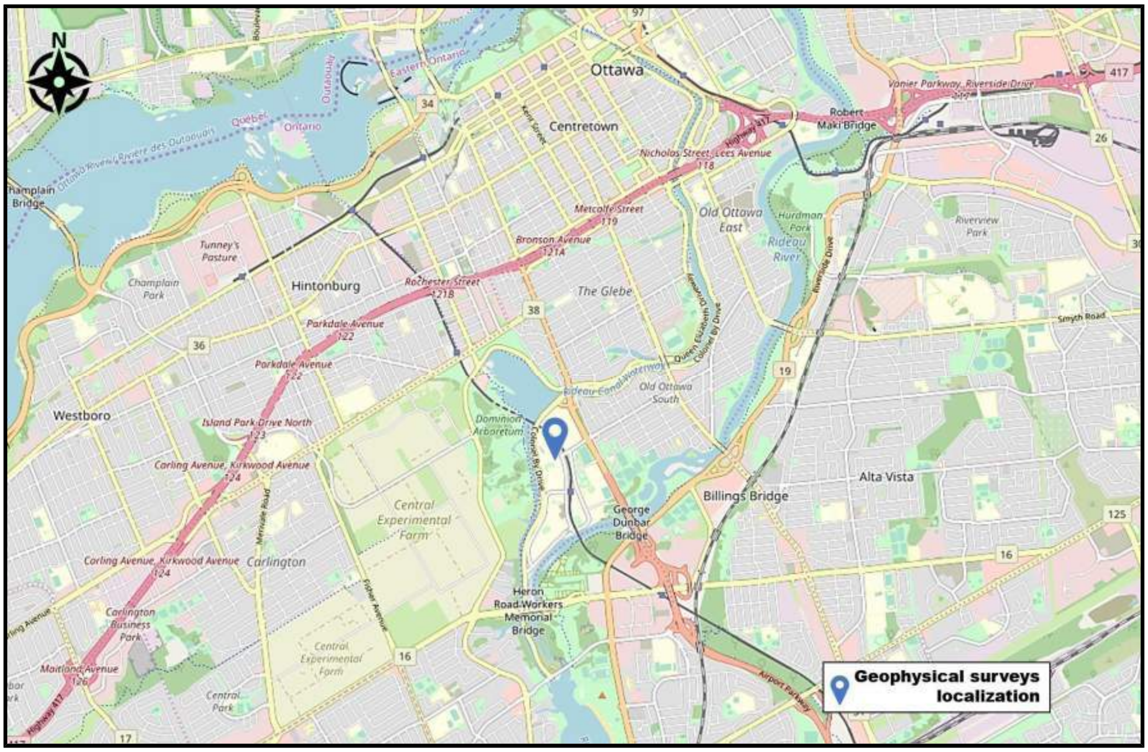


Figure 1: Regional location of the Site
(source: *OpenStreetMap™*)

Figure 2: Location of the seismic spreads
(source: *Google Earth™*)

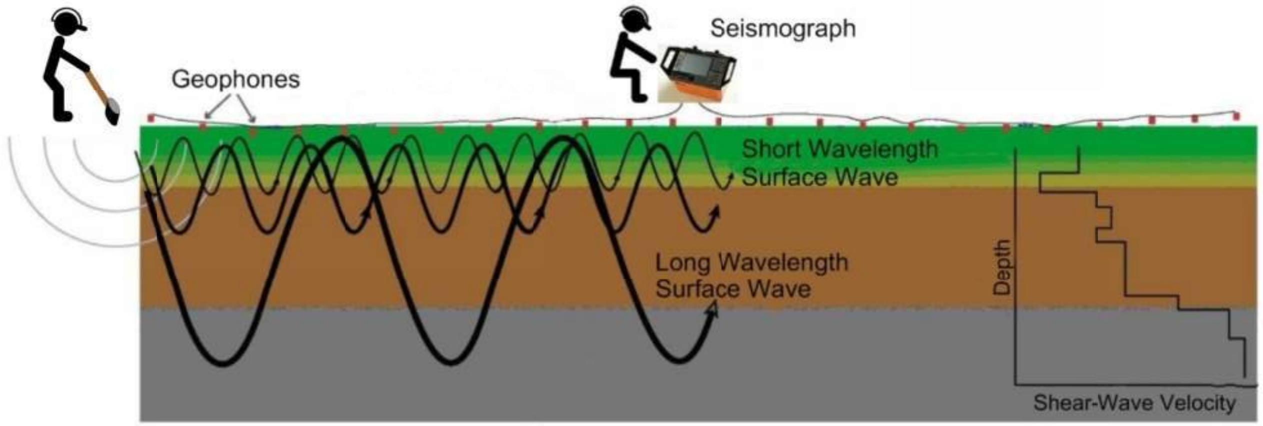


Figure 3: MASW Operating Principle

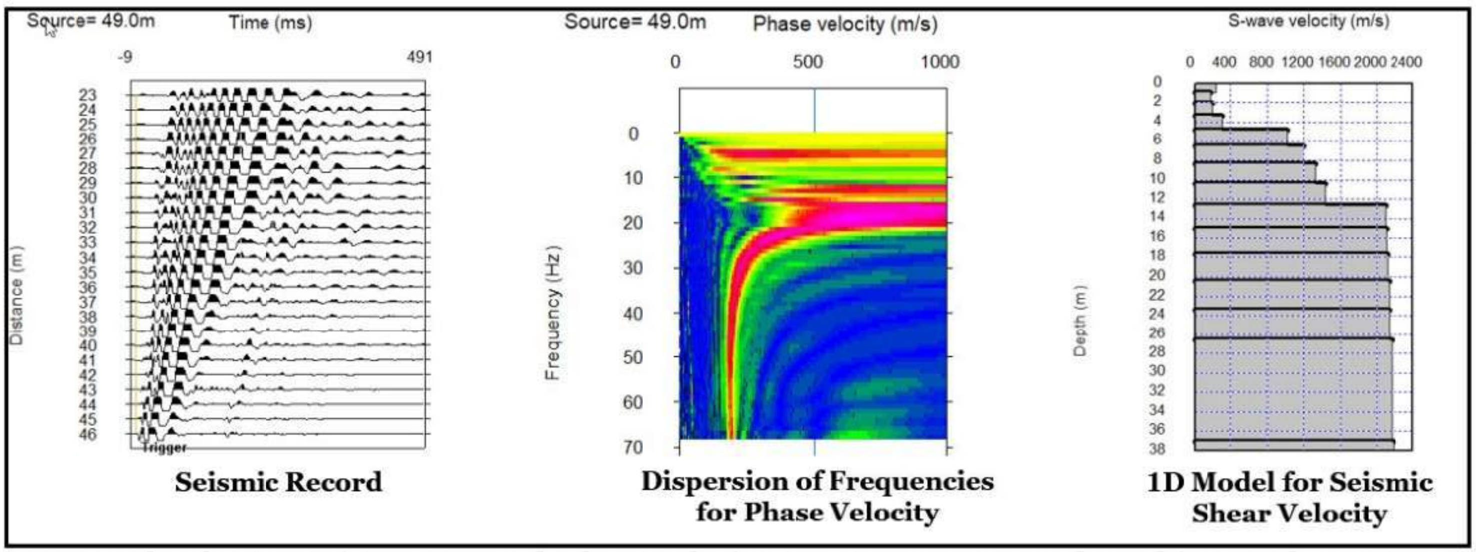


Figure 4: Example of a MASW/ESPAC record, Phase Velocity - Frequency curve and resulting 1D Shear Wave Velocity Model

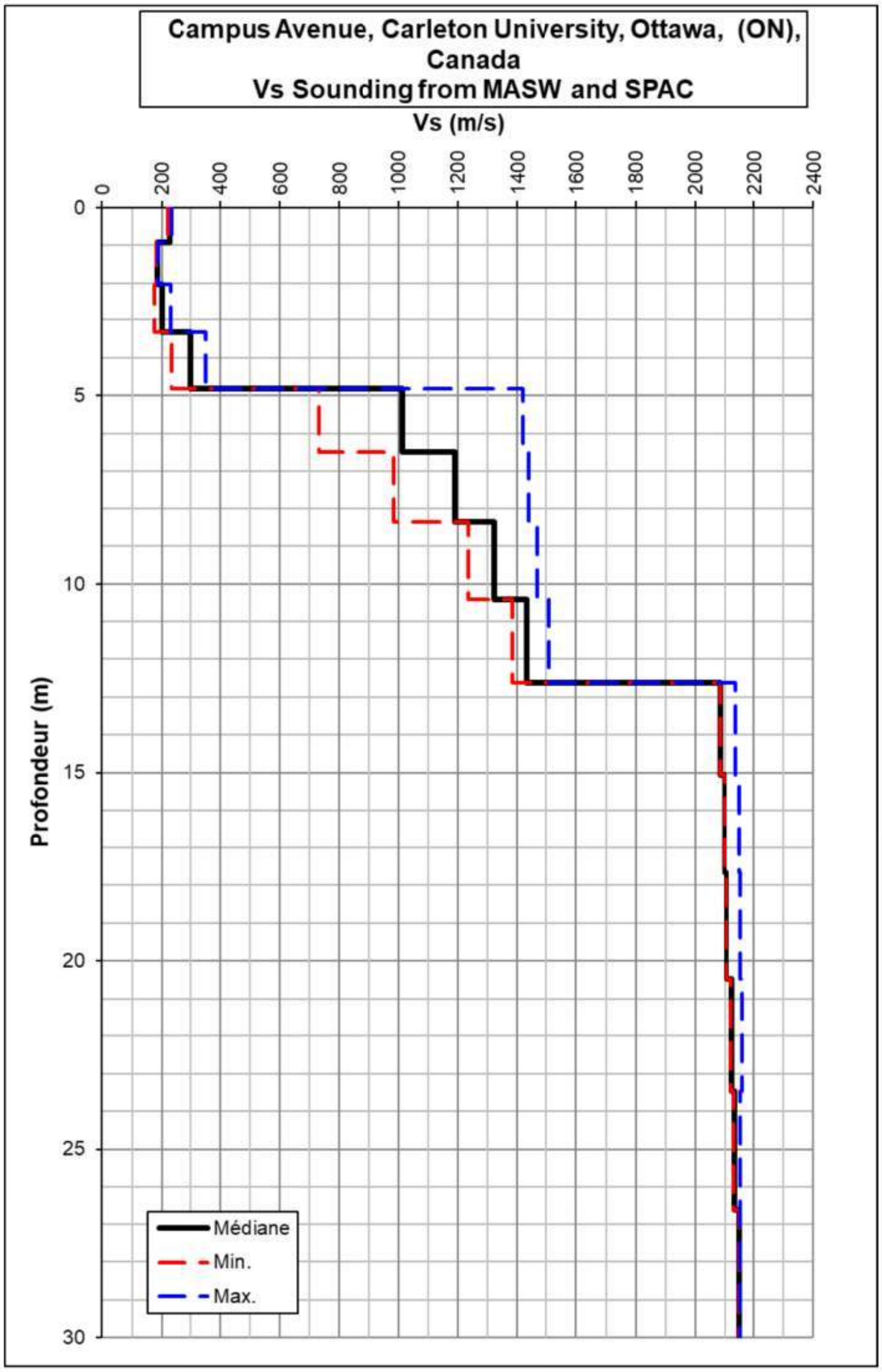
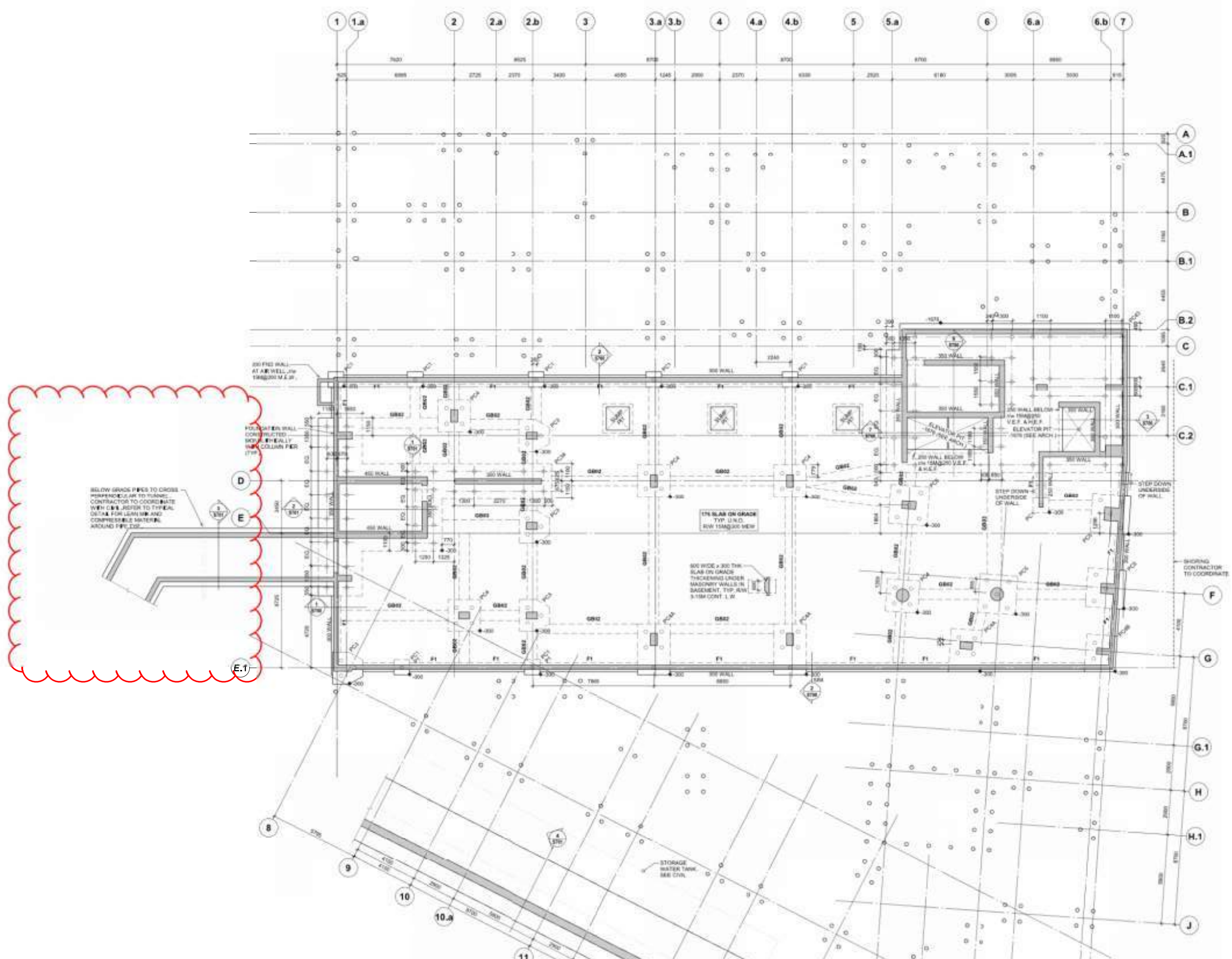


Figure 5: MASW Shear-Wave Velocities Sounding



TYPE	SUPERIMPOSED DEAD LOAD (kPa)			LIVE LOAD (kPa)	COMMENTS
	COMPONENTS		TOTAL		
STORAGE AREAS, CORRIDORS, LOBBIES, MECHANICAL AREAS AND ELECTRICAL AREAS	PARTITIONS (INCLUDED IN LIVE LOAD) FLOORING LEVELING SCREED	0.2 0.15	1.35	7.2	
STAIRS			0	4.8	

BASEMENT/FOUNDATION PLAN

1 : 100

NOTES:

1. REFER TO ARCHITECTURAL DRAWINGS FOR TOP OF SLAB ELEVATIONS, SLOPES AND DIMENSIONS. TOP OF STRUCTURAL SLAB IS 0 mm FROM FINISHED FLOOR DATUM ELEVATION EXCEPT AS CROSSED AND NOTED ON PLAN. FINISHED FLOOR DATUM 62.20
2. PILING CONTRACTOR TO DESIGN PILES FOR LOADS NOTED. REFER TO GEOTECHNICAL REPORT #191-18248-00 PREPARED BY WSP, DATED DECEMBER 2019 (REPORT), MARCH 31, 2020 (MEMO) AND APRIL 6, 2020 (MEMO) FOR ADDITIONAL INFO.
3. CENTRE ALL CAPS, PILASTERS, AND FOOTINGS UNDER COLUMNS EXCEPT NOTED ON PLAN.
4. MAINTAIN SLAB-ON-GRADE THICKNESS AS SHOWN AT ALL LOCATIONS UNLESS NOTED OTHERWISE. SLAB ON GRADE TO BE SUPPORTED ON SOIL CAPABLE OF SUSTAINING SLS: 24kPa, SUBGRADE MODULUS: 3000kN/m³.
5. SEE GENERAL NOTES FOR CONTROL JOINT REQUIREMENTS U.N.O. ON PLAN. C.J. CONTROL JOINTS MAY BE REPLACED WITH CONSTRUCTION JOINTS.
6. THE SHORING SYSTEM WHERE SHOWN CONCEPTUALLY IS ONLY TO CLARIFY THE INTERFACE BETWEEN STRUCTURE AND THE EXCAVATION SHORING SYSTEM USED. REFER TO EXCAVATION SHORING DRAWINGS FOR LOCATION, SIZE AND DETAILS OF THE SOLDIER PILES, CAISSONS, TIEBACKS, ETC.
7. REFER TO THE FOLLOWING DRAWINGS:
   - GENERAL NOTES - S100s
   - FOOTING/GRADE BEAM SCHEDULE - S600s
   - PILE CAP SCHEDULE - S600s
   - STUD RAIL SCHEDULE - S600s
   - COLUMN SCHEDULE - S300s
   - BEAM SCHEDULE - S500s
   - SHEAR WALL ELEVATIONS/SCHEDULE - S400s
   - COUPLING BEAM SCHEDULE - S400s
   - ZONE SCHEDULE - S410
8. TOP OF PILE CAP ELEVATION NOTED ON PLAN RELATIVE TO FINISHED FLOOR DATUM.
9. REFER TO ARCHITECTURAL, GEOTECHNICAL AND CIVIL FOR BELOW SLAB ON GRADE AND FOUNDATION WALL/RETAINING WALL DRAINAGE.
10. SEE PLAN S200R FOR REINFORCING

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ISSUED

No.	Date	Description
1	2020-01-30	100% DESIGN DEVELOPMENT
2	2020-03-31	ISSUED FOR COORDINATION
3	2020-04-07	ISSUED FOR TENDER

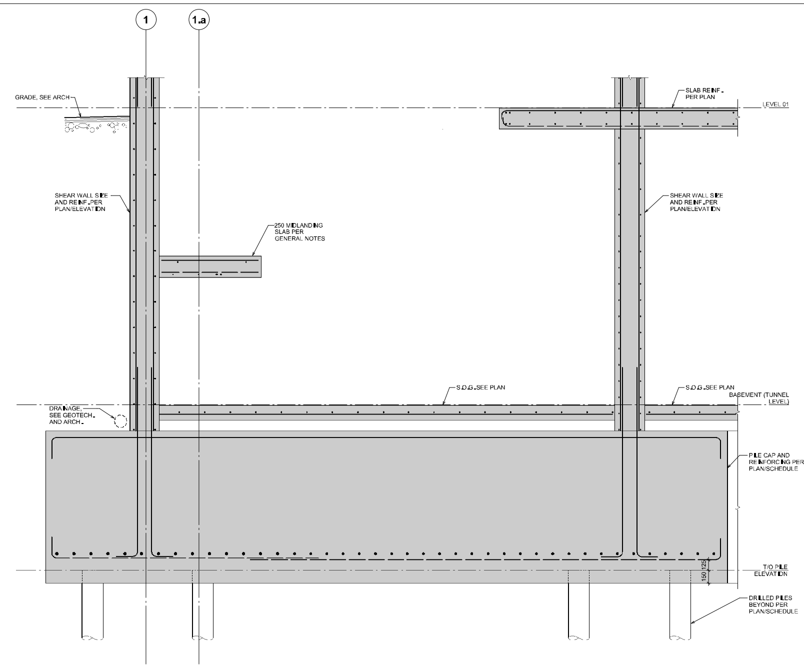
Diamond Schmitt Architects

CARLETON UNIVERSITY

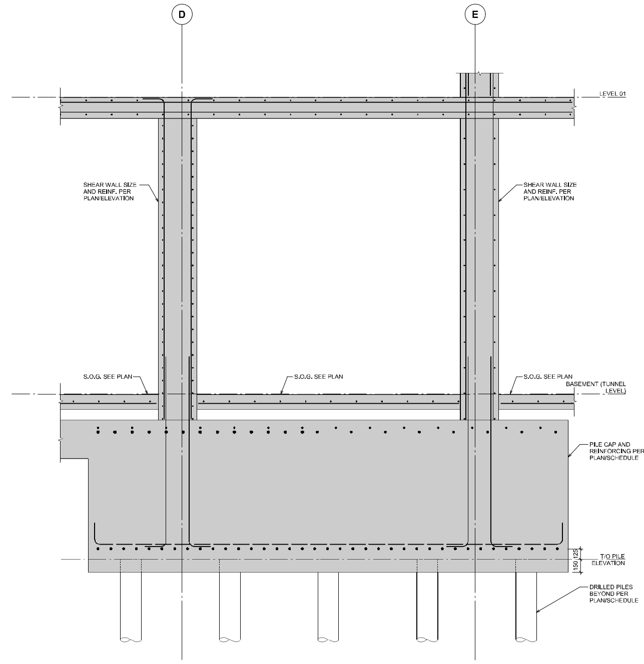
BASEMENT/FOUNDATION PLAN - FRAMING PLAN

Scale: As indicated
Project No: OTT.121632.0001
Date: Issue Date

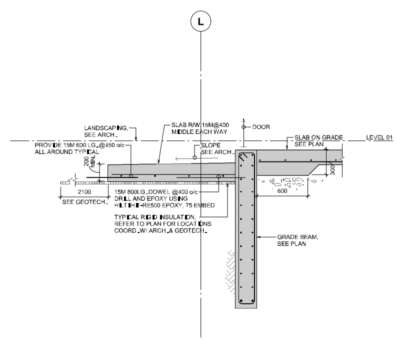
S200



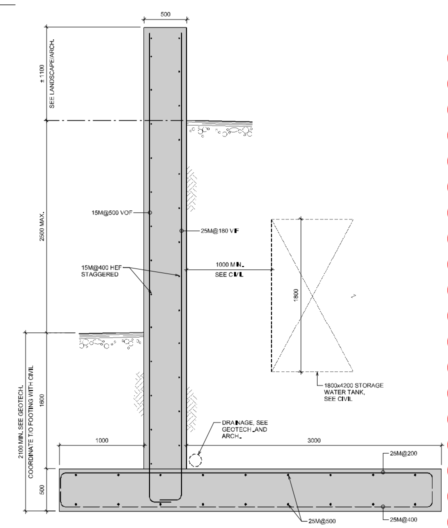
2 / S701 — 1 : 20



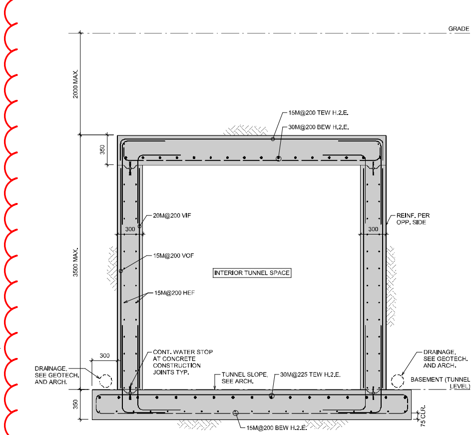
1 / S701 — 1 : 20



5 / S701 TYPICAL FROST SLAB AT DOOR
1 : 20



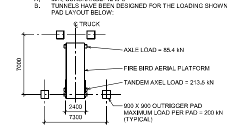
4 / S701 RETAINING WALL SECTION
1 : 20



3 / S701 TUNNEL SECTION
1 : 20

NOTES:

1. REFER TO GENERAL NOTES AND TYPICAL DETAILS FOR TUNNEL GEOTECHNICAL REQUIREMENTS. CONTRACTOR TO COORDINATE WITH GEOTECHNICAL CONSULTANT.
2. TUNNEL SLAB HAS BEEN DESIGNED FOR THE FOLLOWING SURCHARGE:
   A. MIN. SURCHARGE: 12 kPa
   B. TUNNELS HAVE BEEN DESIGNED FOR THE LOADING SHOWN ON THE OUTRIGGER PAD LAYOUT BELOW:



   C. CONTRACTORS CONSTRUCTION LOADS MUST NOT EXCEED THE ABOVE DESIGN LOADS. DESIGN LOADS MAY ONLY BE APPLIED AFTER CONCRETE REACHES ITS DESIGN STRENGTH.

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Read Jones Christoffersen Ltd.
Engineers
rjc.ca

343 Preston Street, 11th Floor
Ottawa, ON K1S 1N4 Canada
tel 343-291-1081
fax

ISSUED

No.	Date	Description
1	2020-03-31	ISSUED FOR COORDINATION
2	2020-04-07	ISSUED FOR PERMIT

2020-04-07
2020-04-07

Diamond Schmitt Architects

CARLETON UNIVERSITY

FOUNDATION SECTIONS

Scale: As indicated
Project No: OTT.124933.0001
Date: Issue Date

S701

May 05, 2020

Jenny Kluke
Development Review, Central Branch
City of Ottawa
110 Laurier Avenue West
Ottawa, ON K1P 1J1

Dear Jenny Kluke:

RE: Consolidation of Engineering Related Comments
1125 Colonel By Drive – New Student Residence Building
File Number: D07-12-19-0205
Consultant File Number: 190444600 **RJC No.: OTT.124933.0001**

RJC has completed the structural design of the proposed retaining wall at the above-noted site, per the below noted city request:

- A retaining wall is proposed to overcome the significant difference in grade between the site and the Stormont-Dundas House and is over 1m in height. As per *City of Ottawa Slope Stability Guidelines for Development Applications* an engineering report is required to be prepared by a qualified engineer for any retaining walls 1m or greater in height that addresses the global stability of the wall. An Internal Compound Stability (ICS) analysis from a professional Geotechnical Engineer/ Structural Engineer licensed in the Province of Ontario is required to check for global stability. The retaining wall is to have a factor of safety of at least 1.5 for static conditions (as calculated through SLIDE) and 1.1 for seismic conditions. The report shall provide structural details of the retaining wall and account for the load from the adjacent underground storage tank. The retaining wall design is required prior to planning approval not at the time of building permit application submission as suggested.

Please refer to the attached structural sketch SSK-S01 for the structural design meeting the above noted city request. Refer to civil drawing C003 for retaining wall extents and soil grades.

Should you have any questions or concerns, please do not hesitate to contact the undersigned at 343-291-1081.

Yours truly,

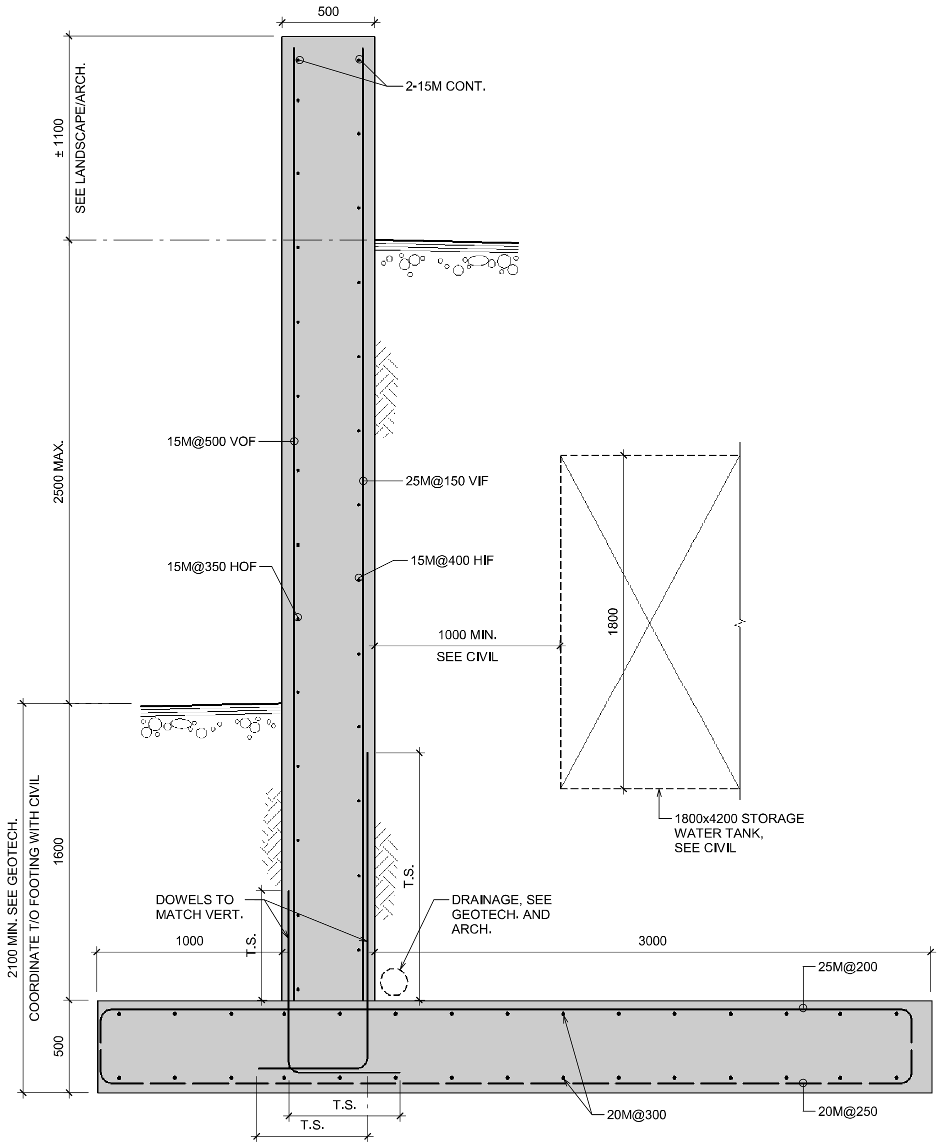
Read Jones Christoffersen Ltd.

Prepared by: Reviewed by:

Alaina Polkki, E.I.T.
Engineering Intern
Structural Engineering

Sean Keating, P.Eng.
Regional Manager/Project Engineer
Structural Engineering

LICENSED PROFESSIONAL ENGINEER
S. C. KEATING
100153857
20200505
PROVINCE OF ONTARIO



<u>RETAINING WALL NOTES:</u>

1. RETAINING WALLS ARE DESIGNED IN ACCORDANCE WITH THE RECOMMENDATIONS OF THE GEOTECHNICAL REPORT BY WSP (PROJECT #: 191-19248-00) DATED DECEMBER 2019 (REPORT), MARCH 2020 (MEMO), AND APRIL 2020 (MEMO), PLUS A 4.8 kPa LATERAL LOAD ALLOWANCE FOR A VERTICAL SURCHARGE OF 12 kPa. REFER TO GEOTECHNICAL REPORT FOR ALL GEOTECHNICAL REQUIREMENTS.
2. RETAINING WALLS TO BE SUPPORTED ON SOIL CAPABALE OF SUSTAINING:
   SLS: 150 kPa
   ULS: 175 kPa
   SUBGRADE MODULUS: 6000 kN/m3
3. RETAINING WALLS ARE DESIGNED FOR A FREE DRAINING AND WELL DRAINED BACKFILL. SEE ARCHITECTURAL AND CIVIL SPECIFICATIONS AND DRAWINGS FOR DRAINAGE REQUIREMENTS.
4. SEE ARCHITECTURAL DRAWINGS AND SPECIFICATIONS FOR DAMPROOFING OR WATERPROOFING REQUIREMENTS.
5. SEE ALSO ARCHITECTURAL AND CIVIL/LANDSCAPING DWGS FOR EXTENT OF RETAINING STRUCTURES AND LOCATION RELATIVE TO SITE.
6. BACKFILL MATERIALS AND METHODS TO BE REVIEWED BY SOILS CONSULTANT TO ENSURE COMPLIANCE TO THE RECOMMENDATIONS AS NOTED IN THE GEOTECHNICAL REPORT.
7. DESIGN AND FIELD REVIEW OF BACKFILL IS BY SOILS CONSULTANT AND NOT BY READ JONES CHRISTOFFERSEN.
8. UNLESS NOTED OTHERWISE, ALL RETAINING WALLS BELOW GRADE AND ALL EXTERIOR WALLS EXPOSED TO THE WEATHER ABOVE GRADE SHALL HAVE CONTROL JOINTS. CONSTRUCTION JOINT MAY REPLACE CONTROL JOINT WHERE REQUIRED. THE LOCATION OF CONTROL JOINTS IN EXPOSED CONCRETE WALLS SHALL BE SUBMITTED TO THE ARCHITECT FOR REVIEW.

<u>CONCRETE NOTES:</u>

1. COMPRESSIVE STRENGTH OF 35 MPa (MIN.)
2. CONCRETE EXPOSURE CLASS S-3.
3. PROVIDE TYPE HS CEMENT.
4. PROVIDE CONCRETE MIX DESIGN SHOP DRAWING TO RJC FOR REVIEW PRIOR TO CONSTRUCTION.
5. 40mm CLEAR COVER TO REINFORCING U.N.O.

D07-12-19-0205

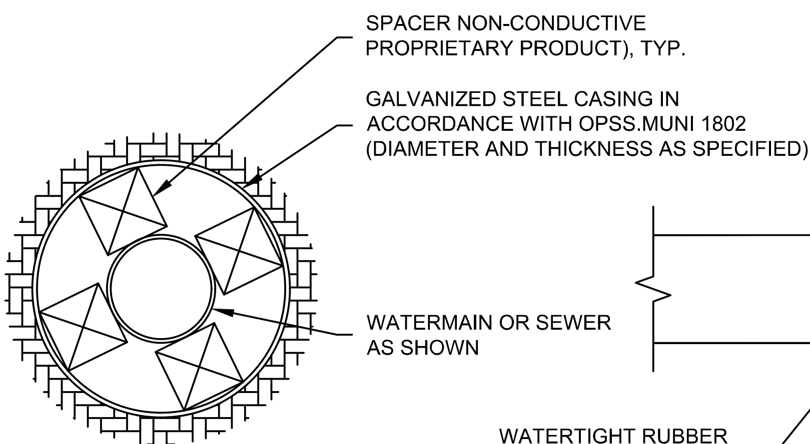
#18030

rjc Engineers

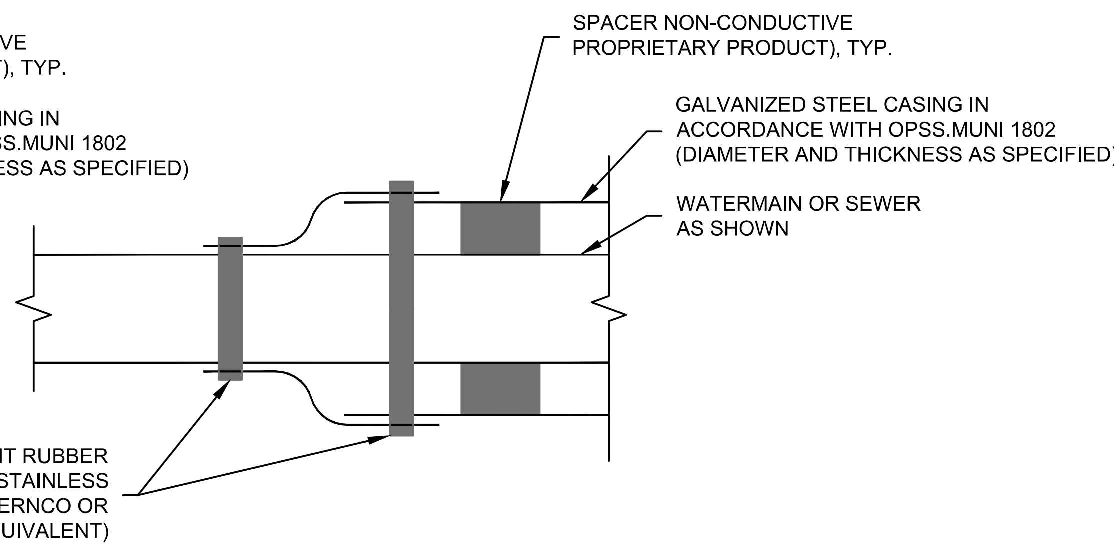
Project Name
CARLETON UNIVERSITY NEW STUDENT RESIDENCE

Sketch Title
RETAINING WALL TYPICAL DETAIL

Dwg. Ref.	**C001, C003**
Scale	**1:40**
Date	**2020/05/05**
Project No.	**OTT.124933.0001**
Sketch Number	Rev.
SSK-S01	**00**



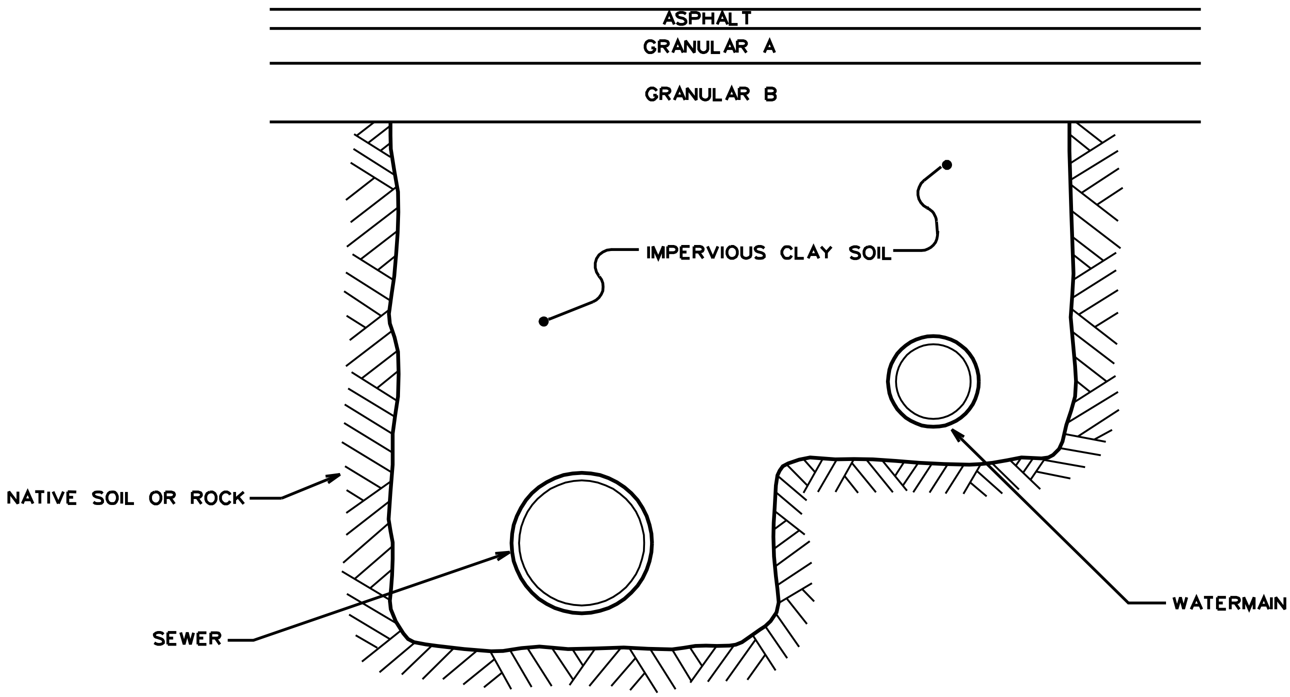
CASING CROSS SECTION
N.T.S.



CASING END DETAIL
N.T.S.

CASING NOTES:

1. SUBMIT SHOP DRAWINGS FOR CASING, CATHODIC PROTECTION, COUPLERS, PROPRIETARY SPACERS.
2. INSTALL CATHODIC PROTECTION ON CASING IN ACCORDANCE WITH REQUIREMENTS FOR WATERMAINS AS SPECIFIED ELSEWHERE.
3. BEDDING, SURROUND AND BACKFILL TO BE AS SPECIFIED FOR WATERMAINS.
4. USE SINGLE LENGTH OF CASING PIPE (JOINTS ARE NOT PERMITTED).
5. FOR WATERMAINS RUN TRACER WIRE OUTSIDE CASING.



NOTES:

1. CLAY SEAL TO EXTEND FROM BOTTOM OF TRENCH EXCAVATION TO UNDERSIDE OF ROAD STRUCTURE.
2. CLAY SEAL TO EXTEND FULL TRENCH WIDTH TO EXISTING NATIVE SOILS WITH A MINIMUM THICKNESS OF 1.0m ALONG PIPES.
3. CLAY SEAL TO BE LOCATED SO THAT NO PIPE JOINTS ARE WITHIN THE CLAY SEAL MATERIAL.

Ottawa	CLAY SEAL FOR PIPE TRENCHES	DATE: MAY 2001
		REV. DATE: MARCH 2006
		DWG. No.: S8