Appendix A

Borehole Log Excerpts

Kingston, Ontario
Search and Rescue Dock
Project #: R.106283.001

Groundwork Engineering Limited

GEOTECHNICAL REPORT

COAST GUARD SEARCH & RESCURE BUILDING

PORTHSMOUTH OLYMPIC HARBOUR

KINGSTON, ONTARIO

Prepared for:

Fisheries and Oceans Canada

Prepared by:

Groundwork Engineering Limited

Project No. 17004-13

August 2017

3443 Princess St, Kingston, ON K7P 3A5 mburger@groundengineer.ca

613-634-1789(tel) *www.groundengineer.ca*

Contents

1.0 Introduction

Groundwork Engineering Limited (**GWEL**) was retained by Fisheries and Oceans Canada (**DFO**) to conduct a geotechnical investigation at Portsmouth Olympic Harbour, Kingston, Ontario. This report has been prepared solely and exclusively for the Client for the purpose of providing geotechnical information for footing design for a proposed Search & Rescue Building.

2.0 Site and Project Description

The site is located along the eastern side of Portsmouth Olympic Harbour, Kingston, Ontario (Figure 1).

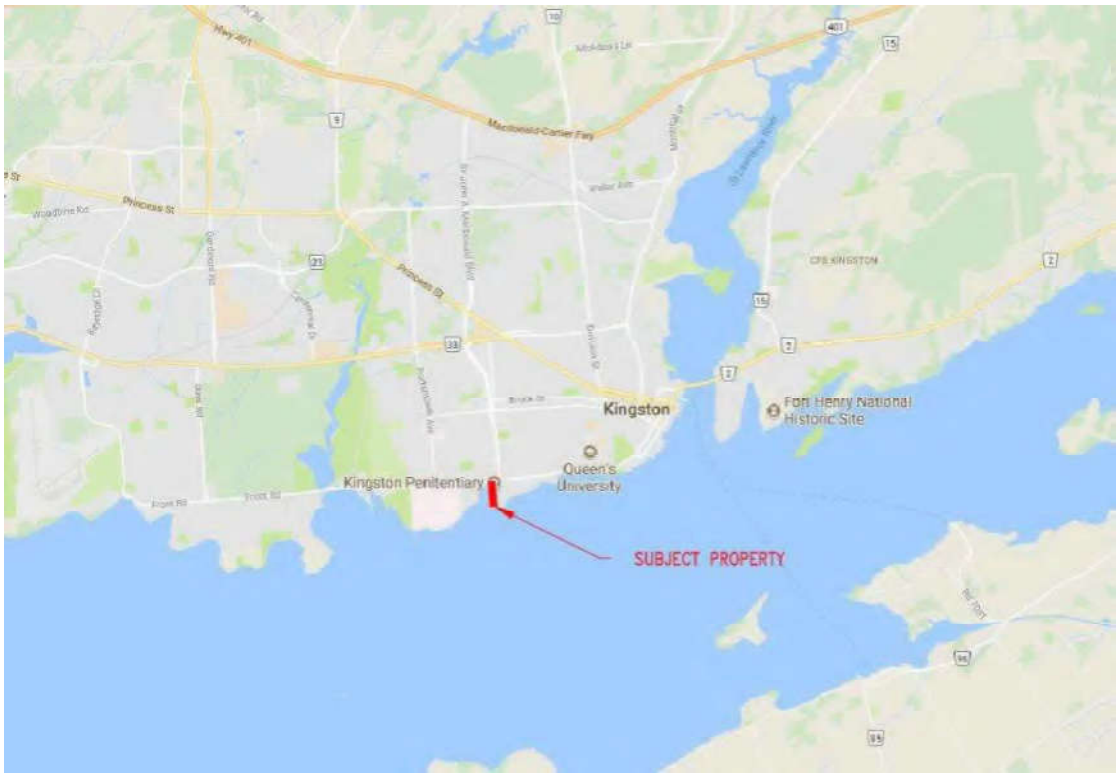

Figure 1. Site Location

The intent of the investigation was to study and report on the subsurface soil and ground water conditions in boreholes drilled at the site and to provide geotechnical engineering information for the design of building foundations, floor slabs on grade, pavements, site servicing, fixed dock foundations and other elements of the development. A bathymetric survey was also completed

to determine the depth of the bottom of the harbor for design of a permanent dock. A topographic survey was also completed along the proposed location of the utility corridor from the proposed building northerly to King St. Existing services on the west side of the former Kingston Penitentiary site were located in order to provide information for water and sanitary servicing options.

3.0 Method of Investigation

3.1 General

The scope of this geotechnical investigation included drilling sixteen (16) boreholes, two within the proposed building footprint to a depth of 6m or refusal, two along the gabion basket wall, drilled to rock with a 1.5m rock core, and twelve (12) located along the proposed utility corridor evenly spaced and to a depth of 2.5m.

Prior to drilling of the boreholes, locates were obtained from the relevant authorities and utility providers. Underground utilities were marked and the borehole locations were selected to avoid the utilities while still being representative of the site.

Drilling of the boreholes was undertaken on August 9th, 10th, and 11th using a truck mounted CME 55 drill rig. The boreholes were advanced using continuous flight augers. Standard penetration testing and sampling was carried out at regular intervals. Split-spoon samples were examined and logged on site. The number of drops required to drive the sampler 0.3m was recorded as the “N” value.

Representative samples were packaged and labelled for laboratory sampling. After the drilling, sampling and logging was completed, the boreholes were backfilled using drill cuttings and bentonite pellets.

A piezometer was installed in borehole 1 (BH1) to monitor groundwater elevations after drilling was completed.

All field activities were performed under the constant supervision of GWEL technical staff.

3.2 Borehole Location

All borehole locations were laid out in the field by GWEL. Borehole locations and elevations were collected by GWEL with GPS survey equipment. The location of the boreholes can be seen in Appendix A. The benchmark for the site was Ontario Benchmark Station 0011975U501 with an elevation of 75.989m. The following table summarizes the borehole ground surface elevations:

ID	Northing (m)	Easting (m)	Elevation (m)
BH1	4896936.871	378989.608	77.88
BH2	3896960.711	378985.415	77.71
BH3	4896952.830	379979.755	76.98
BH4	4896969.150	378977.523	76.93
BH5	4896984.944	378993.074	78.03
BH6	4896984.944	378993.074	77.78
BH7	4897023.856	378990.457	76.49
BH8	4897053.910	378986.555	76.87
BH9	4897083.968	378983.722	76.96
BH10	4897114.127	378983.271	77.91
BH11	4897143.748	378983.124	77.68
BH12	4897176.771	378980.330	77.81
BH13	4897206.463	378981.469	78.12
BH14	4897233.989	378978.517	78.88
BH15	4897264.800	378976.929	79.60
BH16	4897290.636	378972.761	81.53

Table 1. Borehole Elevations

3.3 Bathymetric Survey

A bathymetric survey was conducted along the shore in the location of the proposed fixed dock. Soundings were taken on a grid of approximately 2m x 2m. The bottom edge of the existing gabion wall was taken as the shoreline. Along portions of the shoreline a concrete cap poured on rock boulders was observed just above the waterline. The harbour bottom was observed to be rocky with boulders. Results of the survey can be found in Appendix A.

3.4 Topographic Survey

A topographic survey was conducted from the proposed building location northerly to King St along the proposed utility corridor. Measurements were taken on a grid of approximately 1m x 1m. Results of the survey can be found in Appendix A.

3.5 Existing Services

Existing water and sanitary service mains were exposed on the former Kingston Penitentiary property immediately east of the proposed building. The watermain was found to be located 1.78m below finished grade, with a top of pipe elevation of 76.91m. An unknown SDR PVC pipe was found above the watermain with a top of pipe elevation of 77.52m. Three 50mmØ conduits were discovered to be empty and approximately 0.5m below finished grade.

Two manholes and one sanitary sewer main was investigated using a Flexiprobe sewer camera. Manhole 2 was found to be buried under asphalt and granular with a top of grate elevation of 78.16m. The lid of Manhole 2 was found to be covered in concrete and could not be removed during the investigation. Manhole 1 was found at grade with a top of grate elevation of 78.71m. The inverts for Manhole 1 were:

- N INV: 76.60m
- E INV: 76.67
- S INV: 76.55
- W INV: 76.86

A 250mm Ø PVC pipe runs 64.5m from Manhole 1 to Manhole 2 with an approximate slope of 1.30%. An existing site services sketch can be found in Appendix A.

4.0 Subsurface Conditions

4.1 General

Details on the subsurface conditions encountered during the geotechnical investigation are presented on individual borehole logs attached to this report as Appendix B. The soil descriptions given in this report are based on current geotechnical practice as per the Canadian Foundation Engineering Manual, 4th Edition. The stratigraphic boundaries shown have been inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil type to another. These boundaries should not be interpreted to represent actual planes of geologic change. The subsurface conditions are confirmed at the borehole location only and will vary between and beyond the locations.

4.2 Topsoil

A surface layer of topsoil, ranging in thickness from 0.11m to 0.76m, was discovered in BH1, BH3 to BH10, and BH12 to BH16.

- BH1 from 0.00 to 0.44 meters below ground surface (mbgs) (77.88m – 77.44m)
- BH3 from 0.00 mbgs to 0.36 mbgs (76.98m – 76.62m)
- BH4 from 0.00 mbgs to 0.30 mbgs (76.93m – 76.63m)
- BH5 from 0.00 mbgs to 0.19 mbgs (78.06m – 77.87m)
- BH6 from 0.00 mbgs to 0.24 mbgs (77.78m – 77.54m)
- BH7 from 0.00 mbgs to 0.33 mbgs (76.49m – 76.16m)
- BH8 from 0.00 mbgs to 0.51 mbgs (76.87m – 76.36m)
- BH9 from 0.00 mbgs to 0.76 mbgs (76.96m – 76.20m)
- BH10 from 0.00 mbgs to 0.21 mbgs (77.91m – 77.70m)
- BH12 from 0.00 mbgs to 0.13 mbgs (77.87m – 77.74m)
- BH13 from 0.00 mbgs to 0.25 mbgs (78.12m – 77.87m)
- BH14 from 0.00 mbgs to 0.18 mbgs (78.88m – 78.70m)
- BH15 from 0.00 mbgs to 0.23 mbgs (79.60m – 79.37m)
- BH16 from 0.00 mbgs to 0.11 mbgs (81.53m – 81.42m)

4.3 Gravel

A surface layer of gravel, 0.61m in thickness was encountered in BH 2.

- BH2 from 0.00 mbgs to 0.61 mbgs (77.71m – 77.10m)

4.4 Asphalt

A surface layer of asphalt, 0.16m in thickness was encountered in BH11.

- BH11 from 0.00 mbgs to 0.16mbgs (77.68m – 77.52m)

4.5 Heterogeneous Fill

Layers of heterogeneous mixed fill consisting of sand, silty clay soil, gravel, wood, concrete and brick, cobbles and boulders ranging in thickness from 6.8m in BH3 to 0.65m in BH16 was encountered in BH1 – BH16.

- BH1 from 0.44 mbgs to 6.89 mbgs (77.88m – 70.99m)
- BH2 from 0.61 mbgs to 5.79 mbgs (77.10m – 72.10m)

- BH3 from 0.36 mbgs to 7.16 mbgs (76.62m – 69.82m)
- BH4 from 0.30 mbgs to 1.77 mbgs and 3.36 mbgs to 5.03 mbgs (76.63m – 75.16m, 73.57m – 71.90m)
- BH5 from 0.19 mbgs to 1.83 mbgs (78.06m– 76.23m)
- BH6 from 0.24 mbgs to 3.05 mbgs (77.54m – 74.73m)
- BH7 from 0.33 mbgs to 3.08 mbgs (76.49m – 73.41m)
- BH8 from 0.51 mbgs to 3.16 mbgs (76.87m – 73.71m)
- BH9 from 0.76 mbgs to 2.14 mbgs (76.20m – 74.82m)
- BH10 from 0.21 mbgs to 3.08 mbgs (77.70m – 74.83m)
- BH11 from 0.16 mbgs to 3.07 mbgs (77.52m – 74.61m)
- BH12 from 0.13 mbgs to 3.53 mbgs (77.74m – 74.34m)
- BH13 from 0.25 mbgs to 3.06 mbgs (77.87m – 75.06m)
- BH14 from 0.18 mbgs to 3.05 mbgs (78.88m – 75.83m)
- BH15 from 0.23 mbgs to 1.07 mbgs (79.60m – 78.53m)
- BH16 from 0.11 mbgs to 0.76 mbgs (81.53m – 80.77m)

A layer of black sand was observed in BH1 and BH2 1.27mbgs.

The SPT 'N' value of this material was recorded as 1-38 blows per 300mm of penetration indicating a very loose to stiff consistency. The high range in relative density is due to the variability of material found in the fill and the likely irregular placement of the fill.

Advancement of a dynamic cone penetrometer was completed in BH3 through loose material with voids commencing at 6.13 mbgs to refusal on anticipated bedrock at 7.16 mbgs.

4.6 Concrete

A concrete abutment, 1.59m in thickness, was encountered in BH4.

- BH4 from 1.77 mbgs to 3.36 mbgs (75.16m – 73.57m)

4.7 Bedrock

Bedrock refusal was encountered in seven boreholes:

Borehole	Meters Below Ground Surface (m)	Elevation (m)
BH2	5.79	71.92
BH3	7.16	69.82
BH4	5.03	71.90
BH5	1.83	76.23
BH9	2.13	74.83
BH15	1.07	78.53
BH16	0.76	80.77

Table 2. Bedrock Elevations

Rock cores, minimum 1.5m long, were obtained in boreholes 3 and 4 using HQ sized diamond core barrels to prove bedrock. The purpose of taking a rock core is to classify the bedrock on site and provide information to aid in structural design. Rock core summaries can be found in Appendix C.

The rock in BH3 consisted of 800mm soft brown grey silt stone with thin layers of brown mudstone over 400mm of hard grey dolostone over 500mm of hard grey limestone. The rock in BH4 consisted of 400mm of soft grey siltstone with thin layers of brown mudstone over 600mm of soft brown mudstone over 600mm of hard grey limestone. Discontinuities consist largely of bedding planes separating the contacts between the different sedimentary rocks. The discontinuity spacing varies from close to moderately close.

Rock Quality Designation, or RQD, is an indirect measure of the number of fractures within a rock mass. The RQD for both core samples was determined to be between 50 to 75% indicating a rock of fair quality.

A sample of the hard grey limestone found at depth was taken from each core and tested for unconfined compressive strength. The rock in BH3 and BH4 exhibited compressive strengths of 55.6 MPa and 62.4 MPa respectively and therefore can be classified as strong. Results can be found in Appendix D.

5.0 Groundwater

A piezometer was installed in BH1 to monitor groundwater levels on site. Groundwater was encountered during the investigation in ten (10) boreholes. These observations are recorded in

the borehole logs. After the investigation was completed the boreholes were observed to be filling with water.

The water level in the piezometer was measured on August 14th, 2017 and again on August 23rd, 2017. The water level was measured to be 2.52 mbgs (75.36masl) on August 14th, and 2.39 mbgs (75.49 masl) on August 23rd.

Groundwater levels can fluctuate greatly and be located at different elevations depending on the seasonal and the atmospheric conditions – i.e. heavy rains, spring thaw, dry spells, etc. Groundwater levels at this site will be highly influenced by Lake Ontario water levels. Higher than normal lake levels have been recorded during the spring and summer of 2017. The weekly mean for Lake Ontario at Kingston for the week ending 21/08/2017 as noted on the Great Lakes Website was 75.39m.

It is noteworthy that if over 50,000L per day is to be pumped during construction, a Permit to Take Water will be required from the Ontario Ministry of the Environment. Further hydrogeological studies may be required in order to fulfill the requirements for obtaining such a permit, in addition to quantifying the amount of pumping to be expected during construction.

6.0 Laboratory Results

Select samples collected in the field were transported to Caduceon Environmental Laboratories Kingston, Ontario laboratory for testing. Chemical analysis in accordance with O.Reg 153 was carried out on two samples of interest; BH1 Sample #1 taken at 1.27 mbgs, and BH2 Sample #1 taken at 1.27 mbgs. Test results can be found in Appendix E.

Moisture content testing was carried out on all samples. The moisture content of the sand ranged from 6.5% in BH1 to 9.7% in BH2.

The samples were found to be in exceedance of Table 1 Background Soil Site Condition Standards for Industrial/Commercial Property Use for zinc, lead, copper, antimony, benzene, ethylbenzene, toluene, xylene, PHC F2 in BH1 and lead, barium, arsenic, PHC F2 and naphthalene in BH2. The samples were found to be in exceedance of Table 3 Soils in Non-

Potable Groundwater Condition Standards for Industrial/Commercial Property Use for copper, lead and zinc in BH1 and for Arsenic and Lead in BH2. Any excess fill that is required to be hauled off site must be disposed of at an appropriate receiving site.

7.0 Seismic Hazard Site Classification

The Ontario Building Code 2012 (Code) stipulates the methodology for earthquake design analysis, as set out in Section 4.1.8. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.4.4.A of the Code. The classification is based on the determination of the average shear velocity of the top 30 metres of the site stratigraphy, where shear wave velocity measurements have been taken or alternatively estimated on the basis of rational analysis of undrained shear strength or penetration resistance.

Based on the results of the boreholes taken the site designation for seismic analysis is best represented by Site Class C according to Table 4.1.8.4.A of the Code and National Building Code of Canada.

8.0 Discussion

The following discussion provides our interpretation of the geotechnical data obtained from the field investigation. The interpretation and discussion is specific to the design of building foundations, fixed dock foundations, backfill, slab on grade, excavations, pavement structure and servicing.

Any comments made regarding the construction aspects are provided only in as much as they may impact on design considerations. Contractors bidding on or undertaking work at the site should make their own assessment regarding the nature and adequacy of the factual information, as it affects their construction techniques, scheduling, equipment selection and the like.

Contractors should submit a detailed excavation, shoring, and dewatering plan for review prior to the start of construction. Construction stages should be clearly defined, any required

structural members and connection details should be clear and understood, and it should be ensured that any required inspections can be carried out easily in the field. All working drawings for temporary works should be prepared by a qualified professional engineer licensed in Ontario.

8.1 Existing Fill

There are risks inherent to developing property underlain by fill placed without engineering control such as at the subject site. The primary concern relates to the potential for post-construction settlement.

Engineered fill placed under careful control may be a very dense material, more uniform, more rigid, and stronger than most natural deposits. However, when not placed under controlled conditions, it may be a heterogeneous mass of rubbish, debris, and loose soil of many types. Fill placed on site would likely have been placed without any engineering control. However, the fill has been in place for decades and although creep-type movements within the fill are likely still occurring, large movements are not expected without the addition of new load. Unfortunately, there is no practical way, short of complete removal and replacement engineered fill, that one can remove all risk of settlement. While it is a feasible option, construction of foundations bearing on or in the non-engineered fill carries an inherent risk of significant post-construction settlements.

Conventionally designed shallow foundations founded at typical depths would be bearing on the fill. If spread footings are founded in the fill, post-construction settlement of the foundations should be anticipated and would have to be managed by designing connections and a superstructure that can accommodate these movements. The designer and owner would also have to consider future owners or users of this site that may be affected.

If the serviceability or aesthetics of the proposed structure would be compromised by the anticipated settlements, it would be necessary to support the structure on engineered fill placed directly on native soils or bedrock. This may become impractical or uneconomical due to the high permeability of the site fills and the water control measures that would be required to

excavate down to the native materials.

A more practical alternative may be a piled foundation bearing directly on or in bedrock. This system may be comprised of driven piles, bored piles, micro piles, or other deep foundation system. This approach would address concerns related to settlement by taking the loads down to bedrock.

With consideration of the above, based on our field investigation and our understanding of the proposed design requirements, we are providing recommendations for shallow foundations on the existing fills as well as driven and drilled pile foundations to support the fixed dock.

8.2 Dewatering

Ground water was encountered in the fill approximately 2.4 mbgs during the investigation. Due to the proximity of Lake Ontario it is anticipated that ground water seepage will be heavy in any excavation below the adjacent lake level.

The existing fills on site are of relatively high permeability and will thus allow water to move through them at a relatively high rate. The effort required to dewater excavations below the groundwater level will be high and may require the construction of cofferdams and a large pumping effort. Design and construction methods that avoid or minimize excavation below the water table should be considered and may prove to be the most practical and economical approach.

Good construction practices include diverting surface water away from excavations; this may be accomplished through the use of ditches and swales. To remove water that does enter, the base of excavations should be shaped to drain to one or more sumps and pumped, as required. Any water discharged from site should meet all applicable regulatory requirements including those related to erosion and sedimentation control.

A further hydrogeological study would be required in order to quantify groundwater flow volumes and/or the need to obtain a Permit to Take Water (PTTW) from the Ontario Ministry of the Environment.

8.3 Excavations

We recommend that all unsuitable materials (sod, rootmat, topsoil, soils containing large quantities of organic matter) be removed from below the footprint of pile caps, shallow foundations, and structural fills to expose the existing granular fills.

After removal of the required materials, the exposed soil surface should be re-graded, compacted and inspected by qualified geotechnical personnel prior to any placement of fill, formwork, or concrete.

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health & Safety Act and Regulations for Construction Projects. The OHSA regulations require that if workmen must enter an excavation deeper than 1.2 m, the excavation must be suitably sloped and/or braced in accordance with the OHSA requirements. The heterogeneous fill found on site can be considered as Type 3 soil. OHSA specifies maximum slope of the excavations for Type 3 soil from the bottom of the excavation to be 3 horizontal to 1 vertical.

Stockpiles of excavated materials must be kept from the edge of any excavation to avoid slope instability. It is therefore important to make sure to keep a distance from the edge at least equal to the depth of the excavation. This distance is also applicable for the passage of heavy machinery near excavations. This condition must be respected at all times unless specific studies are conducted for each case.

8.4 Foundations

8.4.1 Shallow Foundations

Spread footings could be placed directly on the existing fill, however, they would be subject to the inherent risks of developing sites with non-engineered fill discussed previously. We recommend over excavation of minimum 1.0m beneath the required footing elevation. The subgrade is to be compacted and inspected for soft areas. The over-excavation is to be replaced with engineered fill. The final construction beneath

footings should consist of at least 150mm of well compacted (100% SPMDD) OPSS Granular “A” base material.

Conventional spread and strip footings located on the engineered fill may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 200 kPa and a bearing resistance of 135 kPa at Serviceability Limit States (SLS) provided they are a minimum of 2.0 m wide with a minimum soil cover of 1.5 m. This preliminary value was estimated based on assumed conditions and should be checked once the final geometry and loading conditions are determined to assess the effect of slopes, inclined loads, or other conditions that affect the bearing resistance.

The total settlement of spread footings is difficult to predict due to the heterogeneous nature of the fill. Based on the conditions observed on the borehole logs we anticipate the total settlement that can be expected under SLS conditions for a 2.0 m wide footing designed and constructed using the procedures above would be less than 35mm with differential settlements of less than 19mm.

These values are larger than typical design values for most structures. Furthermore, there is the inherent risk of unknown conditions and unpredictable conditions in non-engineered fill so the response of the structure to settlements beyond this value, although not anticipated, need to be considered.

8.4.2 Driven Piles

Driven steel piles are an option for building foundation design and the permanent dock foundation. The presence of boulders, large debris, or other obstructions within the fill is a potential concern for driven foundations. For this reason, the use of drive low-displacement steel H-piles are preferred to high-displacement piles such as close-ended steel pipe piles.

Driven low-displacement piles may be designed using a ULS factored geotechnical axial compressive resistance of 70 MPa based on the cross-sectional area of the steel. The factored compressive axial resistances of several H-pile sections are provided in Table

3; we would be pleased to review other sections upon your request. In accordance with the Canadian Highway Bridge Design Code (CAN/CSA S6-14, 2014) Clause 6.9.1 this includes a resistance factor of 0.4.

The compressive resistance will be achieved through a combination of end-bearing and shaft friction. To achieve this resistance, the piles should penetrate the overburden and may also penetrate into bedrock.

Pile Type	Factored Axial Resistance (Compression)
HP 360 x 132	1175 kN
HP 310 x 125	1115 kN
HP 310 x 110	990 kN

Table 3. Factored Axial Compressive Resistance at ULS

Driven piles are anticipated to provide only a very limited amount of resistance against uplift. If uplift resistance is required consideration should be given to rock anchors or drilled piles.

The resistance of pile groups may be calculated as the sum of the individual pile capacities provided that the centre-to-centre spacing of the piles is at least three pile diameters. The expected settlement of piles driven to refusal on or in bedrock at the serviceability limit state (SLS) loads is negligible and may be estimated as the elastic deformation of the pile.

Piles should be driven with a hammer having a minimum rated energy of 275 Joules/cm^2 of steel cross-sectional area. Practical refusal in bedrock should be taken as a pile penetration of less than 25 mm for ten blows at the rated energy for four consecutive 25 mm increments. The contractor should provide full details on the method of installation and equipment to the geotechnical engineer prior to starting the work.

The boreholes indicate that bedrock is relatively shallow, particularly in boreholes BH3 and 4 which were advanced on the shore. Due to this, there is an increased risk of

overstressing the piles as the preponderance of the driving energy is transferred to the pile tip. Dynamic monitoring (i.e. Pile Driving Analyzer System) should be carried out on the initial pile installations to assess the driving stresses, that the hammer is operating within normal efficiencies, and that the estimated resistance provided for design is achieved at the set criteria. As a minimum, dynamic pile monitoring should be performed on one pile at end of initial drive and at the beginning of re-strike at each abutment. We recommend that the driving energy should initially be at the low end of the hammer's rated energy and that dynamic monitoring is used to evaluate stresses prior to driving at the higher energies that may be required to mobilize the required resistance.

To evaluate the potential for relaxation to occur following initial driving, at least two piles at each abutment should be re-tapped a minimum of 24 hours after initial driving refusal. If relaxation occurs, all piles should be re-driven to the refusal criteria and the cycle repeated until the refusal criteria can be achieved during the re-tap. If significant relaxation continues to occur, dynamic pile monitoring could be used to determine if the required resistance is being developed.

Full-time inspection by qualified geotechnical personnel is recommended during pile installation.

8.4.3 Drilled Piles

Drilled piles (including micropiles) are another practical option to support the building foundations and the dock. Drilled piles can be advanced through obstructions in the fill and will provide uplift resistance, if required. Construction using drilled piles and a void forms below the foundations to provide protection against frost heave would be a good option while limiting the depth of excavation.

Rock-socketed piles rely on the bond between the grout and the rock to develop their capacity. Design of rock-socketed piles should be based on the factored resistance of the socket which is a function of the socket diameter, socket length, bond stress, and installation method. Based on the types and quality of bedrock encountered at the site

during our investigation, the following factored bond stresses are recommended for use in design of gravity-grouted rock sockets:

- Axial Compression 600 kPa
- Axial Tension 450 kPa

These values include a resistance factor of 0.4 for piles in compression and 0.3 for piles in tension in accordance with the Canadian Highway Bridge Design Code (CAN/CSA S6-14, 2014). The design bond length should begin below any weathered or highly fractured portion of the bedrock surface. For this site, we recommend that the design bond length not include the upper 0.8 m of bedrock. Steel casing should be firmly set into the bedrock. Socket lengths should generally be kept between 3.0 m and 8.0 m.

As indicated above, rock-socketed piles provide capacity in tension that can be used to resist uplift forces. The uplift resistance should also consider pulling a cone or wedge of rock and soil. For design, the cone can be taken as a 60-degree apex from the base of the socket. If a series of piles are used, the uplift may mobilize a wedge of rock and soil which splays outwards from the base of the piles at 30 degrees. Submerged unit weights should be used for materials below the groundwater table. A resistance factor of 0.8 is typically applied to the submerged unit weight. The following submerged unit weights may be used in this analysis:

- Existing fills $\gamma' = 8.5$ kN/m^3
- Limestone Bedrock $\gamma' = 16$ kN/m^3

For drilled piles socketed into bedrock, the factored geotechnical axial compressive resistance at ultimate limit states (ULS) is presented below in Table 4 for varying socket diameters and lengths. Group capacities for compressive loads can be taken as the sum of the individual pile capacities provided that the centre-to-centre spacing between the bond zone of adjacent piles is at least three pile diameters. The settlement at the serviceability limit state (SLS) of socketed piles installed as described herein is expected to be negligible and may be estimated based on the elastic compression of the pile.

Socket Length	Pile Diameter		
	203 mm	254 mm	305 mm
	Factored Axial Resistance (Compression)		
5m	1910 kN	2390 kN	2870 kN
6m	2290 kN	2870 kN	3440 kN
7m	2680 kN	3350 kN	4020 kN
8m	3060 kN	3830 kN	4590 kN

Table 4. Factored Socket Resistance at ULS for Drilled Piles in Bedrock

Placement of grout or concrete should be performed promptly after drilling of the socket is complete. Installation of the piles should be closely monitored by personnel having experience with rock-socketed piles. Comparison of bedrock elevations should be carried out on an ongoing basis to check that the socket length is as designed. Compressive strength testing of grout or concrete used in the socket and pile shaft should also be completed. Since the pile base will be installed under the water line correct techniques for placement of concrete below water are required.

In order to confirm the bond stress and the contractor's installation methods, we recommend verification testing be performed on at least one pile at each abutment. Verification testing may be either carried out on a sacrificial pile installed specifically for the test or on a production pile. The test load should be at least two times the design load. If verification testing is performed on a production pile, the pile should be designed with a structural capacity at least 1.25 times the maximum test load and it should not be failed or overloaded during testing. Good practice dictates that a plan should be developed prior to testing to replace the pile in the case that it does fail during testing. As an alternative, PDA testing may be used as a verification test.

8.5 Frost Protection

A permanent soil cover of 1.5 m or its equivalent in thermal insulation is required for frost protection of all foundations at the Site. If thermal insulation is being considered,

appropriate products and placement instructions are best obtained from individual manufacturers.

8.6 Floor Slabs on Grade

Topsoil, asphalt, loose gravel and any fill to a depth of 600mm must be removed from all areas to be developed as floor slabs on grade. The proposed finished floor elevation is such that up-fill will be required to achieve the required subgrade elevation beneath the floor slab on grade. On this basis any up-fill required should consist of OPSS Type II Granular “B” material placed and uniformly compacted in 200mm thick lifts to at least 98% Standard Proctor Maximum Dry Density (SPMDD).

Excavations within the building envelope should be backfilled with OPSS Type II Granular “B” material. Use of granular material as outlined above, however will not guarantee that settlement will not take place. All backfill within the building envelope should be placed in 200mm thick lifts with each lift uniformly compacted to at least 100% SPMDD. Excavations outside the building envelope under landscaped areas can be backfilled with native fill. Excavations outside the building envelope under paved areas should be backfilled with OPSS Type II Granular “B” material.

Irrespective of the structural design, the final construction beneath floor slabs on grade within building envelopes should consist of at least 150mm of well compacted (100% SPMDD) OPSS Granular “A” base material or clear crushed stone. For buildings where moisture control is critical, clear stone should be specified rather than Granular “A” and consideration should also be given to including a polyethylene sheet beneath the slab to help damp-proof the floors for such applications. The slabs should be free floating and should not be tied into the foundation walls or grade beams. Saw cut control joints should be incorporated into the slabs along column lines and at regular intervals to control shrinkage cracking. Interior load bearing walls should not be founded on the slab but on spread footings as outlined above.

8.7 Earth Pressure Design Parameters

The following unfactored values may be used for the design of retaining structures:

Parameter	EXISTING GRAVEL FILL	STRUCTURAL FILL (Compacted Granular B)
Effective Angle of Internal Friction, degrees	32	34
Effective Cohesion, kPa	0	0
Total Unit Weight, kN/m^3	18.5	22
Submerged Unit Weight, kN/m^3	8.5	12
Coefficient of Active Earth Pressure(a)	0.31	0.28
Coefficient of Passive Earth Pressure(a)	3.25	3.54
Coefficient of At-Rest Earth Pressure(a)	0.41	0.44
Friction Factor, Soil/Steel Interface	0.25	0.35
Friction Factor, Soil/ Mass Concrete Interface	0.45	0.50
Friction Factor, Soil/ Formed or Pre-Cast Concrete Interface	0.30	0.35

(a) Coefficients of earth pressure presented in table assume a frictionless wall with a vertical back face and a horizontal back slope. Values can be provided for different conditions upon request.

Table 5. Unfactored Geotechnical Parameters

8.8 Engineered Fill

Engineered Fill application may be required on this project in order to provide a level base in cases of over-excavation.

For any fill operation to be considered Engineered Fill, the following criteria must be satisfied:

- Engineered Fill should consist of uniform, homogeneous material. The fill material should also be free of organics, deleterious materials (i.e. building debris such as bricks, metal etc.). Materials meeting Ontario Provincial Standard Specification, OPSS Granular B Type I or II specifications would be considered a suitable Engineered Fill material;
- Prior to the placement of Engineered Fill, it must be evaluated for suitability in the Geotechnical Laboratory. Samples should be provided to the Geotechnical Engineer and submitted for Standard Proctor and grain size analysis;
- Engineered fill must be compactable, and of a suitable moisture content such that it is within +/- 2.0% of its optimum moisture content, as determined through laboratory testing;
- Engineered Fill must be placed under the continuous supervision of a Geotechnical Engineer or their designate;
- Each layer of material should be placed in maximum 0.2m lifts, and uniformly

compacted with heavy compaction equipment suitable for the type of fill used, to 100% of its SPMDD;

• Field density tests must be taken by the Geotechnical Engineer on each lift of Engineered Fill. Any Engineered Fill which is tested and found to be out of specification shall be either removed, reworked or retested; and

• Engineered fill placed underneath foundations must extend laterally a minimum of 1.5 D from the outside edge of footings, where D is the depth of Engineered Fill placed.

If unshrinkable fill is used underneath foundations, it must also extend a minimum of 1.5 D from the outside edge of footings, as mentioned above.

8.9 Pavements

8.9.1 Subgrade Considerations

Topsoil, asphalt and fill must be removed to subgrade elevations from all areas to be developed as parking areas and the surface proof-rolled prior to placement of granular sub-base.

8.9.2 Flexible Pavements

The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedure must be maintained to ensure that uniform subgrade moisture and density conditions are maintained as much as practically possible and that the natural subgrade is not disturbed and weakened.

The following pavement component thicknesses are provided for the design of flexible pavements. Control of surface water is an important factor in achieving good pavement performance. Grading adjacent to pavement areas should be designed so that water is not allowed to pond adjacent to the outside edges of the pavement. The subgrade must be free of depressions and sloped (preferably at a minimum grade of two percent) to provide effective drainage toward subgrade drains. To intercept excess subsurface water within the pavement structure granular materials, perforated sub-drains with suitable outlets and catch basins should be installed below the pavement area's subgrade if adequate overland flow drainage is not provided (i.e. ditches and swales). The surface of the pavement should be properly graded to direct runoff water towards suitable drainage features.

In the following table Light Duty Areas refer to areas that are occupied by car parking and light traffic while Heavy Duty Areas refers to areas that experience heavy truck traffic, emergency routes, delivery routes, etc. Where asphalt paving areas are proposed the following component thicknesses should be adhered to:

Pavement Layer	Compaction Requirements	Light Duty Areas	Heavy Duty Areas
Surface Course HL3 (OPSS 1150) (PG58-34)	97% of Marshall Relative Density (OPSS 310)	50mm	40mm
Base Course HL8 (OPSS 1150) (PG58-34)	97% of Marshall Relative Density (OPSS 310)	-----	80mm
Granular A Base	100% Standard Proctor Maximum Dry Density	150mm	150mm
Granular B Type II Sub-base (OPSS 1010)	100% Standard Proctor Maximum Dry Density	200mm	300mm

Table 6. Minimum Asphaltic Concrete Pavement Structure New Construction

8.10 Site Servicing

The bedding material for site services should consist of an approved free draining, well graded granular material (such as granular “A”) compatible with the size, class and type of pipe and consistent with local standards as may be applicable. Care will be required to ensure that any softened or disturbed soil is removed prior to placing pipe bedding. Bedding should be placed and uniformly compacted in 200mm thick lifts and compacted to at least 95% of SPMDD.

Selective re-use of the excavated soil as trench backfill may be feasible provided that any excessively wet or frozen material is excluded, and that minor post construction settlement is tolerable. Excavated rock may be used as backfill provided that rock fragments are less than 150mm in size in all directions.

All service trench backfill materials should be placed in 300mm thick lifts with each lift uniformly compacted to at least 95% of SPMDD. The upper 1m of backfill beneath any paved or hard standing areas should be uniformly compacted to at least 98% of SPMDD.

9.0 Statement of Qualifications and Limitations

It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document or its use by a third party beyond the client specifically listed in the report is neither intended nor authorized by Groundwork Engineering Limited. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report.

The recommendations provided in this report are based on subsurface data obtained at the specific test locations only. Boundaries between zones presented on the borehole logs are often not distinct but transitional and were interpreted. Experience indicates that the subsurface soil and groundwater conditions can vary significantly between and beyond the test locations. For this reason, the recommendations given in this report are subject to a field verification of the subsurface soil conditions at the time of construction.

The report recommendations are applicable only to the project described in the report. Any changes to the project will require a review by Groundwork Engineering Limited., to ensure compatibility with the recommendations contained in this project.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we may be of further services to you, please do not hesitate to contact the undersigned.

Martin Burger
M.Eng. P.Eng.