Geotechnical Investigation
Courtyard Marriott Hotel
Maritime Way
City of Ottawa (prev. Kanata), Ontario

Prepared for:

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Summary

The purpose of this report is to present the results of a geotechnical investigation undertaken at the site of the proposed Courtyard Marriott Hotel to be located at Maritime Way, in the City of Ottawa, Ontario. The proposed hotel building will comprise of a six (6) storey high basementless building with associated access roads, parking areas and underground services.

The fieldwork for this investigation was undertaken in two stages. The initial stage comprised the excavation of fourteen test pits throughout the site to depths ranging between 0.15 and 2.4 m and backfilling nine (9) of these test pits with sand. The second stage comprised the drilling of five (5) boreholes within the proposed building envelope to depths ranging between 11.6 to 13.7 m, Elevations 83.5 and 81.6 m.

The investigation revealed the site to be underlain by crusher-run limestone (150 mm minus) extending to depths ranging between 0.15 to 1.8 m in Borehole Nos. 1 to 5 and in Test Pit Nos. 6 to 9, 11, 12 and 13. In Test Pit Nos. 10 and 14, reworked silty clay and with organic topsoil was contacted surficially and extended to depth ranging between 1.5 to 1.8 m.

The crusher-run in Boreholes 1 to 3, 5 and in Test Pit Nos. 6 to 8 is underlain by an original topsoil 100 to 900 mm thick.

The reworked fill material in Test Pit 10 and 14, the crusher-run limestone in Test Pit 9 and the topsoil in the remaining test pits or borehole are underlain by silty clay deposit which extends to the maximum depths investigated of 0.75 to 2.3 in the test pits and to depths of 5.2 to 12.2 in Borehole Nos. 1 to 5.

The consistency of the silty clay is very stiff to soft, based on the undrained shear strength of the silty clay of 140 to 24 kPa. The natural moisture content and unit weight of the clay range from 31 to 60 percent and 17.6 to 18.6 kN/m^3, respectively. One dimensional oedometer test performed on one silty clay sample indicated that pre-consolidation pressure of the silty clay is 113 kPa and its recompression and compression indices are 0.021 and 1.091, respectively.

The silty clay in Borehole Nos. 1 and 3 to 5 is underlain by sitly clay to silty sand till deposit which extends to the maximum depth investigated of 11.6 to 13.7 m i.e. Elevation 83.5 to 81.6 m. The relative density of the silty clay to silty sand till is very loose to dense.

Refusal to augers or to dynamic cone penetration test/split spoon sampler was met in all the boreholes at depths of 11.6 m to 13.7 m i.e. Elevations 83.5 to 81.6 m. It is not known whether this refusal was met on boulders in the till layer or on bedrock surface.

The groundwater table at the site was established at depths of 0.2 m to 0.7 m below the existing ground surface, i.e. Elevation 95.3 m to 94.5 m approximately 25 days following the completion of the fieldwork.

The investigation has revealed the site to be underlain by 6.3 to 9.1 m silty clay deposit, which is prone to consolidation settlements under additional loads. Preliminary computation indicated that placement of 1.5 m of fill at the site in combination with 1.0 m of lowering of ground water table and footing loads will results in additional load on the clay, which will exceed its over-consolidation pressure. This would results in settlements of the structure above the tolerable limits of 25 mm total and 19 mm differential if conventional spread and strip footings are used at the site. Therefore spread and strip types of foundations are not recommended for the site.

Based on the above, it is recommended that the proposed building should be founded on pile foundations. Closed end steel pipe or steel H piles driven to practical refusal on the bedrock surface anticipated at depths of 12.0 to 14.0 m i.e. Elevations 83.0 to 81.0 m are considered to be the most suitable type of foundations for the proposed structure. The allowable load on a 324 mm OD by 12 mm wall thickness pipe pile or HP310 x 110 steel H pile driven to practical refusal is 1119 kN (126 tons) and 1084 kN (121 tons), respectively as detailed in the report. The allowable load on the piles for the type of pile used must be confirmed by using the pile driving analyzer or by a pile load test conducted in accordance with the requirements of ASTM D1143 prior to commencement of production piling. Settlements of the structure founded on piles designed according to the previous recommendations are expected to be well within the normally tolerated limits of 25 mm total and 19 mm differential movements. The rock fill may require excavation and removal from the vicinity of the piles to facilitate installation and minimize damage to the piles. In any event, the piles tip would require reinforcement prior to driving.

The investigation has revealed that the original topsoil was not removed from the majority of the site prior to placement of the crusher-run limestone fill. Therefore, it is recommended that the granular fill and the original topsoil should be removed from the building envelope and the excavation backfilled with engineered fill to the underside of the floor slab as recommended in the main body of the report. In the parking areas and access roadways, the topsoil can be left in-place if future maintenance of these facilities can be tolerated due to consolidation settlement of the topsoil under the weight of the fill and traffic loading.

Computations undertaken indicated that the consolidation settlements of the clay will be within tolerable limits for a total grade raise of 1.5 m provided the building is founded on piles. Therefore, the maximum grade raise at the site should be limited to 1.5 m inclusive of the fill that has already been placed. Based on the above, an additional 0.6 to 1.1 m of fill can be placed at the site in the building area.

The lowest floor slab of the proposed building may be constructed as slab-on-grade provided is set on a bed of 200 mm clear stone placed on engineered fill subgrade prepared as per the recommendations provided in the report.

Excavations for installation of underground services at the site are expected to extend to a maximum depth of 4.0 m below the existing ground surface. These excavations will extend through the rock fill and into the silty clay. They are expected to be up to 3.8 m below the groundwater table. The excavations at the site may be undertaken as 'open cut' provided they are cut back at 45 degree above the groundwater table and at a slope of 2H to 1V below the groundwater table. It should be possible to collect the water entering the excavation in perimeter ditches and to remove it by pumping from sumps. Due to the high water table at the site and the free drainage nature of the rock fill, large quantity of water is expected to flow into the open excavation and therefore localized drainage pits and continuous pumping may be required during the construction process.

The backfill in footing and service trenches located beneath the building should be compacted to 98 percent of standard Proctor density. The on-site rock fill is not expected to be suitable for backfilling of service and footing trenches beneath the building. The native clay is expected to be too wet for adequate compaction and should be discarded. It can be used however for general grading purposes in the landscape areas. Therefore, it is expected that the majority of the backfill material would have to be imported and should conform to OPSS Granular 'B' material.

The pavement structure thickness for access roads may consist of 90 mm of asphaltic concrete underlain by 150 mm of Granular 'A' base and 400 mm of Granular 'B' sub-base. Parking areas should be provided with 65 mm of asphaltic concrete underlain by 150 mm of Granular 'A' base and 300 mm of Granular 'B' sub-base. The recommended pavement structure is based on the assumption that the subgrade comprise of crusher-run limestone rock fill. The base and sub-base materials should be compacted to 100 percent of Standard Proctor density. The asphaltic concrete should be compacted to 97 percent of the Marshall Density.

Normal Portland cement may be used in the subsurface concrete at this site.

The site classification is 'Class D' for Seismic Site Response in accordance with Table 4.1.8.4A of the National Building Code (NBC) 2006 edition.

The above and other related considerations are discussed in greater detail in the body of the report.

Introduction

The purpose of this report is to present the results of a geotechnical investigation undertaken at the site of the proposed Courtyard Marriott Hotel to be located at Maritime Way, in the City of Ottawa, Ontario. The proposed hotel building will comprise of a six (6) storey high basementless building with associated access roads, parking areas and underground services.

This work was authorized by Mr. Joe Lischwe of Concord Purchaseco Inc. on January 22, 2008.

The investigation was undertaken to:

(a) Establish the geotechnical and groundwater conditions at the site;

(b) Establish the maximum feasible grade raise at the site for construction of the proposed hotel on spread and strip footing foundations if possible;

(c) Make recommendations regarding foundation alternatives feasible at the site, founding depths, and allowable bearing pressure of founding soils;

(d) Discuss recommendations on excavation conditions anticipated including possible effects of groundwater during construction;

(e) Estimate the anticipated settlements (total and differential);

(f) Comment on backfilling requirements and suitability of on-site soils for backfilling purposes;

(g) Comment on sub-surface concrete requirements;

(h) Provide recommendations regarding pavement structure thicknesses for access roads and parking areas; and,

(i) Classify the site for seismic site response in accordance with 2006 edition of Ontario Building Code.

The comments and recommendations given in this report are based on the assumption that the above-described design concept will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations or it may require additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

Procedure

The fieldwork for this investigation was undertaken in two stages. The initial stage was completed on January 28, 2008 and comprised the excavation of fourteen (14) test pits throughout the site using a mechanical shovel to depths ranging between 0.15 and 2.4 m and backfilling nine (9) of these test pits with sand and the remaining test pits with the excavated material. The second stage of the fieldwork was completed on January 29 and 30, 2008 and comprised the drilling of five boreholes at the locations of Test Pit Nos. 1 to 5 to maximum depths of 11.6 to 13.7 m

Therefore, for the purpose of this report, the fieldwork will be identified to comprise of five (5) boreholes, i.e. Borehole Nos. 1 to 5 and nine (9) test pits i.e. Test Pit Nos. 6 to 14.

The locations of the boreholes and test pits and their geodetic elevations were established by representatives of Fairhall Moffat Woodland acting as a sub-contractor to Trow Associates Inc. The locations of the boreholes/test pits are shown on Site Plan, Figure No. 1.

Standard penetration tests were performed in all the boreholes at 0.75 m to 1.5 m depth intervals and soil samples retrieved using the split barrel sampler. In-situ field vane tests were performed at regular depth intervals and the undrained shear strength of the silty clay established. In addition, undisturbed thin walled tube samples of the silty clay were also obtained at various depths. In the test pits, however, the sampling program comprised of collection of grab samples from the various soil strata encountered. A dynamic cone penetration test was also performed in Boreholes No. 2 from a depth of 7.0 m to refusal at 11.9 m.

Refusal to augers or to dynamic cone penetration test/split spoon sampler was encountered in all the boreholes at depths of 11.6 m to 13.7 m, i.e. Elevation 83.5 m to 81.6 m.

All the soil samples were visually examined in the field, logged and identified. On completion of the fieldwork, all the soil samples were transported to the Trow laboratory in the City of Ottawa where they were examined by a geotechnical engineer and borehole logs prepared. Laboratory testing comprised of performing moisture content on all samples, unit weight, grain size analyses, and pH and sulphate tests on selected soil samples. In addition, one dimensional oedometer test was performed on one selected silty clay sample.

Water levels were measured in the open boreholes on completion of drilling. In addition, long-term groundwater monitoring installations consisting of 19 mm diameter PVC (polyvinyl chloride) pipes was installed in Borehole Nos. 1 to 4. The installation configuration is documented on the corresponding borehole logs.

Site Description

The site under consideration is a vacant parcel of land located at Maritime Way, in the City of Ottawa (previously City of Kanata), Ontario. It is bordered by Campeau Drive to the north and by Kanata Drive to the south.

The site is covered by 150 mm minus rock fill, which was placed recently by the current developer of the land. The exception to this is in the vicinity of Test Pits 10 and 14 where reworked silty clay mixed with topsoil, trees, branches, etc. was encountered. The ground surface elevations at the locations of the boreholes and test pits ranged between Elevation 96.2 m and 94.8 m.

Subsurface Soil and Groundwater Conditions

A detailed description of the geotechnical conditions encountered in the five (5) boreholes and nine (9) test pits is given on the borehole and test pit logs, Figures 2 to 15 inclusive. The borehole and test pit logs depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted.

Blast-shattered rock (150 mm minus)

A layer of rock fill (150 mm minus), which extends to depths ranging between 0.2 to 1.8 m, i.e. Elevation 95.3 to 93.7 m was encountered in Borehole Nos. 1 to 5 and in Test Pit Nos. 6 to 9, 11 to 13. The rock fill comprised of 150 mm of crusher-run limestone. In Test Pit No. 11 refusal to excavation was encountered at 0.15 m depth on possible frozen rock fill or boulder.

Reworked Silty Clay (Fill)

A layer of reworked silty clay mixed with topsoil, tree branches, sand was encountered surficially in Test Pit Nos. 10 and 14. This layer extends to depths ranging between 1.5 to 1.8 m, i.e. Elevation 93.8 to 93.7 m.

Original Topsoil

The rock fill in Boreholes Nos. 1 to 3 and 5, and in Test Pit Nos. 6 to 8 is underlain by an organic topsoil layer which extends to the top of the native soil contacted at depths of 0.9 to 2.1 m i.e. Elevations 94.4 to 93.5 m.

Silty Clay

The topsoil in Borehole Nos. 1 to 3 and 5 and in Test Pit Nos. 6 to 8, the rock fill in Test Pit Nos. 9, 12, 13 and the reworked silty clay in Borehole No.4 and Test Pit Nos. 10 and 14 are underlain by native silty clay deposit which extends to the maximum depth investigated in all the test pits of 0.75 m to 2.4 m, i.e. Elevation 94.1 to 93.8 m and to depths of 5.2 to 12.2 m, i.e. Elevations 90.0 to 83.0 m in the boreholes.

The undrained shear strength of the silty clay varies from 140 to 24 kPa indicating a very stiff to soft consistency. Its natural moisture content and unit weight vary from 31 to 60 percent and 17.6 to 18.6 kN/m^3, respectively. Grain size analyses conducted on clay samples from Borehole No. 3 (5.3 m to 5.8 depths) and from Test Pit No. 12 (0.2 to 0.8 m depth) are shown on Figure Nos. 16