Cascade Geotechnical Ltd.

GEOTECHNICAL ASSESSMENT

THE NEW MONACO DEVELOPMENT

PEACHLAND, B.C.

Submitted to: New Monaco Enterprise Corporation

Submitted by: Cascade Geotechnical Ltd.
Kelowna, B.C.

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1.0 INTRODUCTION

Cascade Geotechnical Ltd. (Cascade) has been retained by the New Monaco Enterprise Corporation to conduct a geotechnical assessment of the proposed New Monaco development site. The development will include the construction of roads, underground parking, commercial/retail and residential development, and associated underground services. The purpose of the assessment was to gain an understanding of the soil and bedrock properties, and groundwater conditions, and determine the general stability within the project area. Cascade has carried out a geotechnical investigation of the property. This report summarizes our investigation findings and presents recommendations with respect to the design and construction of the development.

The scope of work for this project was presented in our proposal, P13-0411, dated July 19, 2013. In summary, we proposed to carry out a test pit and borehole investigation of the soil, bedrock, and groundwater profile within the proposed development, and an assessment the bedrock conditions. Following the field and laboratory work, a geotechnical report would be prepared that presented the results of the investigation and recommendations for design and construction of the project.

Authorization to proceed with the assessment was received from Mr. Mark Holland, of New Monaco Enterprise Corporation, by email on August 12, 2013.

2.0 PROJECT AND SITE DESCRIPTION

We understand the proposed development may include the construction of 2800 units, 250,000 sq.ft. of commercial/retail development with some underground parking, a 100 room hotel, a six storey parkade, and residential development. The development will have three distinct levels with about 10 m elevation difference between each one, with one terrace midway for Level 2. Access to the site will be off Highway 97.

The site is located east of the Peachland town centre and is situated between Highway 97C (Okanagan Connector) and Highway 97. This phase of the development is located at the east end of the site and encompasses approximately 30 acres. The property is south facing and drains to Highway 97, has an existing orchard over much of the site, and an abandoned house and several small structures.

The site is characterized by gentle to moderate rolling terrain over much of the property, and by steep bedrock bluffs in the southwest corner. The west side of the subject site is bordered by Drought Creek and a gully. At the time of our investigation, approximately 60% of the site was being used as an orchard, and the remaining area was undeveloped, sparsely treed, and vegetated with indigenous grasses and shrubs.

3.0 OFFICE REVIEW

Cascade reviewed the geotechnical report prepared for the site by Golder Associates (Golder), entitled "Geotechnical Investigation, New Monaco Development", dated July 13, 2011. The geotechnical assessment conducted by Golder in 2011 included the entire development area, about 125 hectares. The subject property was identified in the report as being the east portion of Site B, which is on the east side of the Drought Creek gully. The geotechnical investigation in the subject property included the excavation of nine test pits to depths ranging from 1.0 to 4.8 m below ground elevation, and the advancement of two boreholes to between 12.2 to 18.3 m below ground elevation.

The Golder report presented the following comments:

- The soil conditions generally consisted of topsoil and silty sand / sandy silt underlain by interlayered granular deposits. Inferred glacial till was encountered at a depth of 4.0 m at one test pit location, and bedrock was encountered between 1.0 and 11.3 m below ground elevation. Groundwater was encountered in one test pit at a depth of 2.9 m.
- It was concluded that the site was geotechnically safe for development. Off-site geotechnical hazards, such as rockfalls, have been controlled by the construction of Highway 97C.
- Due to the relatively shallow depth of the overburden at the site, Golder did not recommend the on-site disposal of groundwater without a site specific assessment.

4.0 INVESTIGATION PROGRAM

4.1 Field Reconnaissance

A site reconnaissance was carried out on August 15, 2013, by the author of this report and Dr. Dwayne Tannant, P.Eng. The purpose of the reconnaissance was to determine the general condition of the site and identify any bedrock related conditions that might affect the method of construction or constraints to development.

4.2 Subsurface Investigation

A test pit program was carried out on August 26, 2013, under the supervision of the author of this report. A tracked Hitachi EX-120 excavator contracted from On The Mark Locates, of Kelowna was used to dig test pits. Eleven test pits were excavated to depths ranging from 0.75 to 4.0 m below ground elevation. As part of the test pit program, four percolation tests were conducted for the design of infiltration pits. The results of the percolation tests are discussed in Section 6.8.3, and the approximate locations of the test pits are presented on Figure No. 1.

A borehole program was carried out on August 26 to 28, 2013, under the supervision of the author of this report. A truck mounted portable drill rig also contracted from On The Mark Locates was used to drill the boreholes. Eleven boreholes were advanced using a combination of solid stem augers and ODEX drilling. The boreholes were advanced to depths ranging from 5.2 to 14.3 m below ground elevation. The approximate locations of the boreholes are presented on Figure No. 1, in Appendix A.

Classification and index tests were conducted in our laboratory on samples collected from the test pits and boreholes to aid in the determination of engineering properties. Natural moisture content values were determined and grain size tests were performed on the recovered samples. The laboratory test results are provided in Appendix C.

5.0 GEOLOGY AND SOIL CONDITIONS

5.1 Bedrock Geology

Bedrock outcrops are concentrated along the western and southern parts of the site. Three rock units are mapped in the immediate region of the proposed development. Okanagan Batholith rock types exist on the south part of the site and form some of the higher outcrops. These consist of granodiorite and granite, light grey weathering, massive, medium to coarse grained, equigranular to porphyritic, unfoliated to weakly foliated.

At the north side of the site (primarily exposed along Highway 97C) the rock consists of Trepanier Rhyolite. Trepanier Rhyolite is exposed on all sides of the intersection between Highway 97 and Highway 97C and consists of rhyolite lava, breccia, and minor sandstone. Near Drought Creek and along Highway 97, exposures of Springbrook Formation occur and consist of conglomerate and a polymictic breccia. The fault running along Drought Creek delineates some of the boundaries between the rock types.

A dominant steeply dipping joint set exists in most of the rock exposures and the joint set strikes roughly 030° to 040°. This orientation is sub-parallel to the steeply dipping Drought Creek fault. This fabric in the rock mass is frequently associated with the orientation of steep bluff faces and ridges in the bedrock. It is possible that the depth of the overburden above the bedrock surface may be influenced by the shape of buried ‘ridges’ and ‘valleys’ running in a direction north-northeast to south-southeast. Therefore, overburden depths may vary more quickly in directions roughly perpendicular to these structures.

The exposed surface of the bedrock at all locations contains small fractures associated with weathering since the last glacial period. The more extensive fracturing is expected to penetrate a few metres into the rock mass. The freshest exposures of rock are in the rock cuts along Highway 97.

For a more detailed explanation of the bedrock conditions please refer to the report prepared by Dr. Dwayne Tannant, P.Eng., in Appendix D.

5.2 Soil Conditions

The soil conditions encountered on the site generally consisted of TOPSOIL underlain by natural, sandy SILT, SAND or SAND and GRAVEL, which in turn is underlain by SAND and GRAVEL TILL and/or BEDROCK. The soil conditions can be described as the following:

Topsoil

The topsoil layer generally ranged between 100 and 400 mm in thickness at the test pit locations and consisted of sandy silt. The topsoil was medium brown, loose and moist to wet.

Silt

The silt was generally sandy with varying amounts of gravel, and was stiff/compact, dry to damp and light greyish brown. The silt ranged between 1.5 to 3.0 m in thickness.

Sand

The sand gravel deposits were generally fine to medium grained, had varying amounts of silt and gravel, and were dry to damp, compact, and light brown to light grey.

Sand and Gravel

The sand gravel (to gravel and sand) deposits were generally fine to coarse grained, had varying amounts of silt and cobbles/boulders, and were damp, compact, and light to medium brown.

For a more detailed description of the soils encountered during our investigation, please refer to the test pit and borehole logs in Appendix B.

It should be noted that geological conditions are innately variable and are seldom spatially uniform. At the time of the report, information on the stratigraphy at the project site was available at 22 discrete test pit locations. In order to develop recommendations from this information, it is necessary to make some assumptions concerning conditions other than at the test pit locations. Adequate monitoring should be provided during construction to check that these assumptions are reasonable.

5.3 Drainage Conditions

At the time of our investigation, the property was being used as an orchard and irrigation systems were being used. Seepage or groundwater was encountered at eight test locations situated in the centre of the property. Table 1 below present the locations and depths where seepage or groundwater was encountered.

Table 1 – Depth to Groundwater/Seepage

Location	Depth
TP-02	1.2 m
TP-04	3.2 m
BH-01	3.5 m
BH-02	7.3 m
BH-05	0.3 m
BH-06	2.1 m
BH-07	2.9 m
BH-11	6.7 m

Seepage or groundwater was often encountered at the surface of bedrock, till, or at the surface of a silty layer of soil, and may be perched in some areas. The groundwater will fluctuate seasonally or after heavy rainfall events, and may currently be influenced by irrigation of the orchard.

5.4 Drought Creek

The Drought Creek channel drains from upslope of Highway 97C to a culvert installed under the highway. At the time of our assessment, there was not any surface flow in the culvert. Water daylights in the channel about 100 m downslope (south) of the culvert, then disappears into the ground in an area where there appears to be a check dam installed. The water daylights again downstream of the check dam and continues down the channel to a culvert installed under Highway 97. It was noted that white PVC pipe has been installed at several locations along the creek channel, likely for irrigation purposes.

The creek channel is generally heavily vegetated and is incised downstream of the check dam. The upper half of the channel has a 15 to 20% average grade, increasing to 25 to 30% the bottom half. The channel side slopes consisted of sand and gravel deposits which generally ranging between 40 and 60%, with some localized areas having 60 to 70% slopes. Frequent bedrock outcrops were observed on both sides of the creek, both at the creek elevation and at the crest of the side slopes.

Cascade did not observe any significant indications of slope instability, such as slumping, erosion or excessively leaning trees, along the creek channel or upslope of the creek.

6.0 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

6.1 General

The proposed development will include the construction of roads, underground parking and a six storey parkade, commercial/retail and residential development, and associated underground services. The development will have three distinct elevation levels, and access to the site will be off Highway 97.

The site is characterized by gentle to moderate rolling terrain over much of the site, and by steep bedrock bluffs in the southwest corner of the property. At the time of our investigation approximately 60% of the site was being used as an orchard, and the remaining area was undeveloped, sparsely treed, and vegetated with indigenous grasses and shrubs.

A previous geotechnical assessment conducted by Golder Associates determined that the site was geotechnically suitable for development. However, a site specific assessment for the on-site disposal of groundwater was recommended.

The soil conditions at the test locations generally consisted of topsoil underlain by natural sandy silt, sand or sand and gravel, which in turn is underlain by sand and gravel till and/or bedrock. Groundwater was encountered at eight test locations in the centre of the property, and was often encountered at the surface of bedrock, till, or at the surface of a silty layer of soil.

Cascade Geotechnical Ltd. concludes that, from a geotechnical perspective, the New Monaco site is well suited for the proposed development and can be geotechnically safe for the intended use. However, when determining final site grades, we recommend the items in the following subsections be included in the site preparation, grading, drainage, and building setback distances.

6.2 Site Preparation

All organic topsoil or vegetation, fill, trees and stumps, old foundations and irrigation pipes, and any loose, soft, wet, weathered or disturbed soils should be removed from within the proposed building lot areas and areas to be paved to expose either the natural undisturbed compact sand or sand and gravel material, stiff/compact silt, or bedrock. The exposed subgrades, as described above, should be inspected by a geotechnical engineer prior to any structural fill placement.

In a situation where the building pad is situated over the natural overburden or structural fill, and bedrock, then we recommend that the bedrock surface be over-blasted a minimum of 1.0 metre below the foundation elevation and be stripped to expose the competent bedrock surface. The blast rock can then be placed and compacted to form the base of the building pad. This will reduce the risk of differential settlement of the foundation.

We recommend that, in areas where a foundation subgrade consists of silt, the silt should be over-excavated by at least 0.3 m and replaced with granular structural fill or blast rock fill.

Where structural fill is required to bring building lots up to grade, the fill should extend beyond the foundation footprint area for a distance at least equal to the depth of the fill placed below the footing. The structural fill bank should be over-built, then trimmed back, so that the entire bank has been compacted to the specifications below.

Structural fill in building lots should generally consist of 200 mm minus granular material with less than 8% passing the #200 sieve size, and should be compacted to a minimum of 95% Modified Proctor Maximum Dry Density (MPMDD) in accordance with ASTM D1557, and within 2% of the optimum moisture content. The reuse of the on-site soil deposits as structural fill is discussed in Section 6.5.

The structural fill should be placed in lifts not exceeding 0.3 m. We recommend that the compaction of each lift of structural fill be determined with in situ density tests prior to placing subsequent lifts.

Structural fill placed on slopes greater than 30% should be “keyed” into the slope at each lift. The key should extent at least 1.5 m into the natural slope.

6.3 Permanent Slopes

Permanent cut and fill slopes in the natural or compacted sand or sand and gravel structural fill residential lots should be graded no steeper than 2.0(H):1.0(V). Structural fill consisting of compacted angular blast rock should be graded no steeper than 1.5(H):1.0(V).

Permanent cut and fill slopes for road construction should be graded no steeper than 1.5(H):1.0(V) for the natural soils or compacted sand or sand and gravel fill.

Permanent slopes in bedrock should be sloped no steeper than 0.25(H):1.0(V). For additional recommendations about bedrock cuts, blasting and catchment areas please refer to the report prepared by Dr. Dwayne Tannant, P.Eng., in Appendix D.

6.4 Underground Utility Installation

Temporary trenches for underground utilities should be excavated at a slope no steeper than 1.0(H):1.0(V) for the compact sand or sand and gravel to a maximum depth of 1.2 m. Trenches deeper than 1.2 m should be reviewed by a professional engineer in accordance with Work Safe BC Guidelines. The soil conditions and the stability of the excavation should be reviewed by Cascade prior to workers entering the excavation. Alternatively, service line trenches or excavations can be shored.

Temporary cuts should be cleaned of cobbles and boulders prior to workers entering the excavation.

Seepage will likely be encountered during excavation of the deep bench cuts and in trenches. Cascade should be contacted in order to evaluate the stability of the cuts in soils and bedrock trenches and drainage.

To maintain the stability of the trench, all materials excavated from the trench should be placed a minimum distance away from the excavation, equal to the depth of the excavation.

Excavated trenches left open for longer than a few days should be covered with polyethylene sheeting to prevent surface saturation during wet weather conditions, or drying out during warm weather conditions.

All work conducted in and around excavations should be carried out in accordance with requirements specified by Work Safe BC Guidelines.

All services should be bedded as per District of Peachland standards. General trench backfill should be placed in maximum 0.3 m thick lifts, and each lift should be compacted to a minimum of 95% of Modified Proctor maximum dry density.

6.5 Reuse of Existing Soil

Any topsoil, other organic soil or silt on this site is considered unsuitable for use as structural fill in building areas. We consider the natural sand or sand and gravel to be suitable as structural fill in the foundation areas.

We consider the natural sand, sand and gravel and sandy silt to be suitable as subgrade material in areas to be paved and as general trench backfill. Blast rock would also be considered suitable for reuse as subgrade fill or trench fill.

Natural soils, fill or blast rock material proposed for reuse as structural fill, road subgrade/subbase or trench backfill should be confirmed by Cascade prior to placement.

6.6 General Foundation Recommendations

The building areas should first be prepared as outlined above. Foundations supported on the natural, compact sand, sand and gravel, silt and sand, or on granular structural fill which has been compacted to a minimum of 95% of Modified Proctor maximum dry density, as outlined above, may be designed on the basis of an allowable bearing pressure of 150 kPa.

A minimum of 0.6 m of soil cover must be provided above the bottom of all exterior footings in order to ensure adequate protection from frost.

The settlement of foundations designed for a maximum allowable bearing pressures presented above should be within the normally acceptable limits of up to about 25 mm total, post-construction settlement, and up to about 20 mm of post-construction differential settlement over a horizontal distance of about 10 m.

6.7 Slab-On-Grade Floors

Cascade recommends that granular structural fill placed below slab-on-grade floors be compacted to a minimum of 95% of Modified Proctor maximum dry density. The floor slabs should be underlain by polyethylene sheeting to prevent the migration of moisture through the slab.

6.8 Site Drainage

6.8.1 During Construction

Surface drainage protection of the site should be maintained during grading and after construction. Precautions should be taken during the performance of site clearing, excavations, and grading to protect the work site from ponding by poor or improper surface drainage. Temporary provisions should be made to direct surface drainage away from and off the work site.

6.8.2 Perimeter Drainage and Storm Water

We recommend a foundation drainage system be installed around the perimeter of all the foundations where a floor slab is lower in elevation than the surrounding ground surface. The perimeter drainage system should include a minimum 100 mm diameter perforated PVC pipe covered with a minimum 0.3 m of drain rock. The drain rock should be covered with filter fabric prior to backfilling against the foundation walls.

We recommend that foundation drainage systems, and storm water from road catch basins and roof downspouts, be drained to the storm sewer system, to Drought Creek if allowed, or to infiltration pits. If infiltration pits are used, the actual size and suitable location of the pits should be determined on a site-by-site basis. Infiltration pits should be located a minimum of 5 m away from the building foundations and from the crest of slopes. Cascade should be provided with the proposed location of infiltration pits in order to provide recommendations for construction.

Sidewalks, paved or landscaped areas within a zone of approximately 2 m of the exterior perimeter of any buildings should be sloped to drain water away from the structure at a minimum gradient of 2%.

Drainage considerations established during design and construction should be maintained for the life of the development. Property owners should be made aware that altering drainage patterns can be detrimental to slope stability and foundation performance.

6.8.3 Infiltration Pit Design

For the design of the infiltration pits, four percolation tests were conducted, and four grain size analyses was performed on sand or sand and gravel samples obtained during the test pit program. The grain size analyses used Hazen's formula, which calculates the approximate coefficient of permeability of sand (K). Table 2 below presents the results of the percolation tests and sieve analyses:

Table 2 – Coefficient of Permeability for Infiltration Pits

Test Location	Depth (m)	K - Coefficient of Permeability (m/s^2)
TP-01	1.8	4.8×10^{-4}
TP-03	1.5	5.2×10^{-5}
TP-04	0.6	1.3×10^{-4}
TP-07	1.8	6.2×10^{-4}
TP-09	0.3	5.7×10^{-3}
TP-11	1.8	4.0×10^{-3}
Between Golder TP11-02 and BH-04	0.6	5.6×10^{-4}
BH-02	0.5	3.7×10^{-5}

We recommend that a factor of safety of 2 be used to the above K values for the design of the infiltration pits.

6.9 Building Setbacks

The required setback from the crest of slopes will depend on the size of the building, and whether the subgrade consists of compacted sand and gravel (or blast rock) structural fill, or sound, unfractured bedrock. As general rule, buildings four storeys or less should be located no closer than 3.0 m from the crest of slopes. Buildings four storeys or higher will likely require a greater setback distance and should be evaluated on a case-by-case basis.

6.10 Test Pits

The test pits were excavated at the approximate locations as indicated on Figure 1. In the situation where a portion of any proposed building area or paved area is found to be located over a test pit area, we recommend that the test pit area be over excavated and the loose soil replaced with granular structural fill compacted to a minimum of 95% of Modified Proctor maximum dry density.

6.11 Slope Stability

Cascade has carried out static and dynamic analyses of the project area at five cross sections using an interactive slope stability software. The locations of the cross sections are provided on Figure 2 in Appendix A. The static analyses of the cross sections indicate that the slopes on the subject property have a factor of safety of greater than 1.5.

The dynamic analyses were carried out using empirical pseudo-static equilibrium analysis using methods presented by Bray and Travasarou (2009), as presented in the APEGBC "Guidelines for Legislated Landslide Assessments for Proposed Residential Developments in BC", revised in May of 2010 (the Guidelines). For the analysis, a magnitude M=7 earthquake and a spectral response acceleration for a period of 0.5s, Sa(0.5), was used for a 2% in 50 year seismic event (1:2,475).

The results of the analyses indicate that the slopes have a factor of safety greater than 1.0, indicating that the slopes would have a displacement of less than 15 cm, as per the Guidelines.

7.0 PAVEMENT SECTION

We recommend areas to be paved be prepared as described in Section 6.2. Our design method for the asphalt pavement takes into account the anticipated traffic loading conditions, or Equivalent Single Axle Loads (ESALs), and the strength of the subgrade soil immediately below the pavement section. The classification of the pavement structure is determined by the estimated number of ESAL's over the service design life. The strength of the subgrade is measured in terms of the California Bearing Ratio (CBR) testing procedures.

Based on the soil conditions encountered during the test pit investigation, a road subgrade can consist of either sandy silt, or sand (sand and gravel) subgrade. We anticipate that a sandy silt subgrade to have a CBR value between 5 and 10, and a sand subgrade to have a CBR value greater than 10. Table 3 below presents the recommended asphalt pavement structure for a Local Road on a silt or sand subgrade.

Table 3 – Asphalt Pavement Structure

Materials	Road Type	
	Silt Subgrade	Sand and Gravel Subgrade
Asphalt pavement	50 mm	50 mm
Base course - 20 mm minus crushed gravel and sand	100 mm	100 mm
Select granular sub-base (SGSB)	200 mm	-

The pavement structure presented above assumes that the exposed natural granular deposits are a suitable subbase material, and that suitable base and subbase materials will be compacted to a minimum of 95% of Modified Proctor maximum dry density, and within 2% of optimum moisture content. We recommend that the suitability of the natural deposits as a subbase material be verified prior to placement of base course material.

The actual compaction of all granular materials placed should be confirmed with in situ density testing. The gradation of all materials used in construction should be tested prior to construction to confirm they conform to District of Peachland gradation specifications.

8.0 REVIEWS AND CONSTRUCTION INSPECTIONS

It is recommended that Cascade be given the opportunity to review details of the design and specifications related to the geotechnical aspects of this project prior to construction. Past experience has shown that this action may prevent inconsistencies that may lead to disputes.

It is recommended that geotechnical / materials engineering field services, such as observation of bearing surface and backfill, and testing of soil density be performed to ensure that the requirements of this report are followed.

9.0 LIMITATIONS

The recommendations contained in this report have been prepared for the proposed development described in Section 1.0 of this report, and can be relied upon by the New Monaco Enterprise Corporation and their agents, and the District of Peachland. Should the intended use for the property, at any time, vary from our understanding of the project, Cascade should be given the opportunity to review the project to ensure that our recommendations are both accurate and sufficient. Please also refer also to the attached Geotechnical Report - General Conditions.

We have also included in Appendix E, the “Appendix D: Landslide Assessment Assurance Statement” form, which is a requirement for hazard assessment reports (from “Guidelines for Legislated Landslide Assessments for Proposed Residential Development in British Columbia, Revised May, 2010”, prepared by The Association of Professional Engineers and Geoscientists of British Columbia).

10.0 CLOSURE

We trust that you will find our recommendations sufficient at this time. If you require any additional details, please do not hesitate to contact us.

Yours truly,

CASCADE GEOTECHNICAL LTD.

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