HORIZON ENGINEERING INC

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April 7, 2015

Our File: 112-3155

KLAUS FUERNISS ENTERPRISES INC.
c/o Art Phillips & Associates Inc.
521 Bridgeman Road
Gibsons, BC V0N 1V1

Attn: Art Phillips

Re: Proposed "The George" Mixed Use Development
377, 385 & 407 Gower Point Road, 397 & 689 Winn Road, and Winn Road Right-of-Way, Gibsons, BC
Geotechnical Investigation Report (Revised)

Attached, please find copies of the revised geotechnical investigation report for the proposed "The George" development to be located at the above-stated addresses. This report summarizes the results of the subsurface investigations carried out on September 19 and 24, 2012, April 11, 15, and 16, 2014, December 9 and 10, 2014, and January 12, 2015 at the subject site, available background information, and our engineering analyses and recommendations.

We trust this is sufficient for your current requirements. Please contact us if you have any questions, or if we can provide additional service.

Sincerely,

HORIZON ENGINEERING INC

Karim Karimzadegan, M.A.Sc., P.Eng.
Principal

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GEOTECHNICAL INVESTIGATION REPORT
(REVISED)

for the

PROPOSED "THE GEORGE" MIXED USE DEVELOPMENT

at

377, 385 & 407 Gower Point Road, 397 & 689 Winn Road, and Winn Road Right-of-Way, Gibsons, BC

Our File: 112-3155

April 7, 2015

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EXECUTIVE SUMMARY

PART A - BACKGROUND INFORMATION AND TECHNICAL DATA

This document reports on the results of the field investigations carried out in September 2012, April 2014, December 2014, and January 2015, as well as subsequent engineering analyses. In addition, it provides geotechnical design and construction recommendations for the proposed development. A preliminary, feasibility-level version of this report was originally issued on November 26, 2012. On January 18, 2014, we were informed by the Client that the subject site had been expanded to include the property at 407 Gower Point Road, which is located adjacent to the south property lines at 397 and 689 Winn Road, at the south portion of the site. Access to 407 Gower Point Road was unavailable until December 2014 to carry out a supplementary subsurface investigation; therefore, our original recommendations were extrapolated to the new proposed south property line at the time of publishing an updated report at the Client's request, which was issued on February 12, 2014. Subsequently, our February 2014 report was reviewed by Levelton Consultants Ltd. and Waterline Resources Inc. on behalf of the Town of Gibsons. Three supplementary subsurface investigations were carried out following our review of questions raised by the Town's consultants in order to collect data to support the rezoning and Development Permit stage recommendations provided in this report.

The subject site is located on the waterfront in the town of Gibsons, BC, and includes the properties at 377, 385, and 407 Gower Point Road, 397 and 689 Winn Road, and the Winn Road right-of-way between 385 and 397 Winn Road. In addition, we understand that the site includes an existing water lease area, which extends into the harbour east of the site. We understand that the proposed development comprises a new, multi-level hotel, conference centre, and residential development called "The George", to be partially supported over two to three parkade levels that daylight to the east. We also understand that the eastern portion of the site will comprise new, at-grade or above-grade amenities including a café, retail space, meeting room, and a 'seawalk', as well as gangways, an over-water restaurant building, and a marina with a fuel dock. We understand that a fuel storage tank is proposed to be located within the southwest portion of the parkade to provide service to the marina. We also understand that approximately 2.5 to 4.0 metres (8 to 13 feet) of dredging is proposed to be carried out within the existing water lease area in the harbour.

Four phases of subsurface investigations were carried out at the subject site. Preliminary subsurface investigations were carried out on September 19 and 24, 2012 that involved excavating two test pits, drilling three auger holes, and advancing four Dynamic Cone Penetration Tests (DCPTs) at the site. A supplementary subsurface investigation was carried out on April 11, 15, and 16, 2014 that involved drilling six auger holes, advancing six DCPTs, and drilling three sonic boreholes at the site. A second supplementary subsurface investigation was carried out on December 9 and 10, 2014 that involved drilling three sonic boreholes at the site. A third supplementary subsurface investigation was carried out on January 12, 2015 that involved advancing seven WildCat Cone Penetration tests in the foreshore area of the site.

The soil stratigraphy encountered during the subsurface investigations is summarized following:

- Surficial soils (fill, peat, sand, silty sand to sandy silt to silt) with variable total thicknesses of approximately 2 to 6 metres (7 to 21 feet) were encountered beneath the current grades, which were inferred to be unsuitable as bearing layers.

- Till-like, silty sand to silt and sand soils were encountered below the surficial soils. These materials were found to be dense to very dense (when boreholes terminated in this soil type) and 2.7 metres (9 feet) thick).
- Sand to sand and gravel to gravel soils were encountered below the till-like soils (where present, and beneath the surficial soils elsewhere). These materials were found to be compact to very dense and (when boreholes terminated in this soil type) 6 metres (21 feet) thick.

The subsurface investigations carried out in the foreshore and harbour areas generally encountered approximately 3 to 4 metres (10 to 13 feet) of loose seabed sediments overlying the aforementioned natural silty sand to sandy silt to silt. Approximately 1.5 metres (4 to 5 feet) of fill materials were inferred to be encountered near the surface at the north foreshore portion of the site, presumed to overly the loose seabed sediments that were encountered elsewhere in the foreshore and harbour areas.

During the subsurface investigations, non-artesian groundwater was encountered at depths of 0.9 to 3.7 metres (3 to 12 feet) within the test pits and auger holes. The presence of non-artesian groundwater conditions could not be determined during sonic drilling, as water was used as a drilling fluid. Perched groundwater within the more permeable surficial soil horizons is expected to be daylighted during excavation at the site. We expect that non-artesian groundwater levels at the site may be tidally influenced, particularly at the east portion of the property.

During the supplementary subsurface investigations, artesian groundwater was encountered at depths of approximately 4.6 to 14.9 metres (15 to 49 feet) below adjacent grades (or the seabed) within the sonic boreholes. Artesian groundwater was inferred to have stabilized at elevations ranging from approximately -2.0 metres (-6 feet 7 inches) at the eastern water least portion of the site to approximately 11.3 metres (37 feet 1 inch) at the northwest portion of the site.

Standpipe piezometers were installed at three of the borehole locations during the supplementary subsurface investigations in order to facilitate groundwater monitoring within the upper portion of the Gibsons Aquifer. The standpipes were completed to depths of 7.3 to 15.2 metres (24 to 50 feet) below adjacent existing grades and were fitted with analog pressure gauges or water level dataloggers (transducers) to collect water level monitoring data.

PART B - DISCUSSIONS AND RECOMMENDATIONS

From a geotechnical viewpoint, the subject site is considered to be suitable for development of the type proposed, and the Gibsons Aquifer is envisaged to <u>not be negatively impacted by the proposed development</u> provided that the recommendations in this report are incorporated into the design and construction. The conclusions and recommendations presented in the previous sections of this report are summarized below.

The Town of Gibsons and the subject site are underlain by the Gibsons Aquifer, which is a confined aquifer comprising sand and gravel that provides drinking water for the town. The confining Gibsons Aquitard is inferred to comprise variable thicknesses of sand, peat, silty sand to sandy silt to silt, and localized till-like materials within the subject site.

Artesian groundwater pressures have been observed within the Gibsons Aquifer. Hydraulic connections have been observed between the Gibsons Aquifer and the ocean at the central portion of the site and between the Gibsons Aquifer and Town Well #1 at the west portion of the site.

A computer model was generated to analyse the site and subsurface conditions during and after construction of the proposed development based on existing information, published literature, and engineering judgement. The results of this modelling work indicate that the proposed excavation should not advance below a geodetic elevation of 5.0 metres (16 feet 5 inches) at the northwest portion of the site in order to ensure that the underlying Gibsons Aquifer is not compromised (even temporarily) due to excavation of the overlying materials. Note that zero geodetic is equal to 3.02 metres (9.9 feet) above Chart Datum (average lower low tide)in Gibsons. The results of these analyses are based on conservative soil strength properties. Therefore, there is an inherent Factor of Safety (which may be of the order of approximately two) in the deformation analysis results.

At the southwest, southeast, and northeast portions of the site, we recommend that the proposed excavation not advance below 0.5 metre (1 foot 8 inches) below existing grades in order to ensure that the Gibsons Aquifer is not compromised due to excavation of the overlying materials. Deeper excavation at the southwest portion of the site is not recommended due to the proximity of the Gibsons Aquifer to the existing site grades.

All habitable spaces are recommended to be constructed at or above a Flood Construction Level (FCL) of approximately 5.33 metres (17 feet 6 inches), which takes into account potential effects of sea level rise and storm and tsunami waves during the design life of the proposed building. We envisage that habitable spaces could be constructed below the FCL if a sea dike is constructed around the building, which would be designed to protect the building from rising sea levels and future storm events. The proposed marina and over-water restaurant should be constructed at or above the FCL since they would otherwise be unprotected from the design flood conditions.

We envisage that the lowest proposed top of slab elevations would be approximately 6.3 metres (20.5 feet) at the west portion of the site and 3.1 metres (10.2 feet) at the east portion of the site. Accordingly, we envisage that the proposed footing elevations would be approximately 5.4 metres (17.5 feet) at the west portion of the site and 2.2 metres (7.2 feet) at the east portion of the site. Therefore, we envisage that the proposed excavation would be approximately 5.1 to 5.8 metres (17 to 19 feet) deep below adjacent existing grades at the northwest portion of the site. Excavation at the southwest, northeast, and southeast portions of the site is envisaged to be less than approximately 0.5 metres (1.6 feet) deep below adjacent existing grades.

We envisage that the proposed finished floor elevation for the proposed café, retail space, meeting room, seawalk, and over-water restaurant at the east portion of the site would be approximately 5.3 metres (17.3 feet), which is consistent with the recommended FCL. The currently proposed lowest parkade floor elevation is below the FCL; therefore, a sea dike is envisaged to be required as part of the proposed development. The sea dike would comprise gravity structures, constructed above the natural boundary such that the final grades will be raised above the proposed FCL.

We recommend that foundations for the entire building footprint are supported on conventional strip and pad foundations or on a raft foundation. Due to the presence of loose and compressible subgrade materials (which are judged to be unsuitable for supporting shallow foundations in their current state), ground improvement is recommended beneath proposed foundations such that suitable bearing is achieved.

We recommend that footings proposed at the east portion of the site, where excavation is not required, are lowered to the existing grades after the proposed ground improvement measures are complete. In these areas, floor slabs are recommended to be designed as suspended slabs.

The soil profile at the footprint area of the proposed building is considered to be locally potentially liquefiable; however, we envisage that after implementation of the proposed deep soil mixing ground improvement measures, the potential for liquefaction beneath the proposed building foundations would be eliminated.

We envisage that deep soil mixing (which includes mixing existing soil with cement grout using large mechanical equipment) may be the preferred method of ground improvement at the subject site.

Non-artesian groundwater is expected to be daylighted during excavation at the site, which we envisage would be managed with conventional drainage measures.

Due to the naturally high non-artesian water levels expected at the site, we recommend that the below-grade portions of the building be designed as a waterproof structure. It is envisaged that an in-ground infiltration system would be installed at the eastern portion of the site to disperse intercepted groundwater into the existing, natural, subsurface peat and sand to silty sand materials.

A methane venting system is recommended to be constructed beneath any portion of the building that is being constructed at or above existing grades where unsaturated organic materials, such as peat, remain below.

If subexcavation of settlement-susceptible materials in proximity to the shoreline is judged to be impractical for subgrade preparation at the proposed landscaping sidewalk and seawalk footprint areas, we recommend that these structures be supported by shallow foundations constructed on soil-cement columns following ground improvement or by piles. Alternatively, these structures could be designed as 'floating' sidewalks supported on a geogrid-reinforced earth slab.

We recommend that all foreshore development structures, including the over-water restaurant building, docks, and boat slips, be supported by drilled pipe pile foundations. We expect that insufficient resistance may be encountered above the Gibsons Aquifer to provide suitable pile capacity for the proposed structures; however, the materials that were inferred to comprise the aquifer are expected to provide suitable end bearing and/or frictional resistance for the proposed piles. We envisage that installing drilled pipe piles would not result in "leakage" of artesian groundwater from the aquifer around the piles; however, a detailed monitoring program should be implemented during pile installation to detect any breach of the aquifer, if it were to occur. If a passive approach to pile driving into the aquifer is found to be unacceptable, we envisage that piles could be fully sealed, as required, to prevent artesian groundwater from potentially leaking out around the proposed piles.

We understand that dredging is proposed to be carried out near the shoreline within the west and north portions of the existing water lease area in the harbour. The soils that are proposed to be removed during dredging are inferred to generally comprise seabed sediments (and fill materials at the north foreshore area), though we envisage that the proposed dredging excavation may locally intercept the underlying Gibsons Aquitard materials. The seabed sediments were observed to comprise materials that are inferred to be not be significantly more dense than water; therefore, we envisage that "blowout" of the underlying aquitard materials following removal of overlying seabed sediments during dredging would not be expected to occur. Although a hydraulic connection between the ocean and the underlying aquifer is envisaged to exist in the subject area, we recommend that dredging be limited to the seabed sediments to reduce the risk of impacting the aquifer. We recommend that dredging of seabed sediments be carried out by means of hydraulic dredging; where denser / harder seabed and/or fill sediments are encountered near the surface, we envisage that mechanical dredging may be required.

We understand that a 75,000 litre fuel tank is proposed to be constructed at the southwest portion of the site, which will service the proposed marina. We recommend that the tank be supported by soil-cement columns, as previously recommended for the building foundation.

As bulk excavation approaches the final excavation elevations, we recommend that regular surveying by a British Columbia Land Surveyor be carried out within the excavation to confirm that the lowest recommended excavation elevations are not exceeded. We recommend that the excavation be carried out in stages (in plan view). At each stage, the ground surface should be surveyed and monitored such that any potential signs of heaving and upward groundwater seepage are detected, respectively. Based on the available information and the computer modelling described in this report, we envisage there to be no risk of ground heaving or upward groundwater seepage into the excavation if our recommendations in this report are implemented.

We envisage that there will be insufficient room for sloping on the northwest, north, and south sides of the proposed excavation at the northwest portion of the site. It is envisaged that temporary excavation support using tied-back shotcrete shoring will be suitable. A preliminary slope stability analysis was carried out on the temporary shoring system proposed at the west property line in this area, which indicated that this system would be stable during construction.

PART A - BACKGROUND INFORMATION AND TECHNICAL DATA

1.0 INTRODUCTION

This document reports on the results of the field investigations carried out on September 19 and 24, 2012, April 11, 15, and 16, 2014, December 9 and 10, 2014, and January 12, 2015, as well as subsequent engineering analyses. In addition, it provides geotechnical design and construction recommendations for the proposed development. This report is prepared in conformance with our proposed scope of services dated August 20, 2012 and April 4, 2014 and in response to questions from the Town's consultants (Waterline Resources Inc. and Levelton Consultants Ltd.), discussed on March 28, 2014. Authorization to Proceed was received on August 24, 2012 and April 5, 2014.

A preliminary, feasibility-level version of this report was originally issued on November 26, 2012. On January 18, 2014, we were informed by the Client that the subject site had been expanded to include the property at 407 Gower Point Road, which is located adjacent to the south property lines at 397 and 689 Winn Road, at the south portion of the site. Access to 407 Gower Point Road was unavailable until December 2014 to carry out a supplementary subsurface investigation; therefore, our original recommendations were extrapolated to the new proposed south property line at the time of publishing an updated report at the Client's request, which was issued on February 12, 2014.

Subsequently, our February 2014 report was reviewed (by Levelton Consultants Ltd. and Waterline Resources Inc.) on behalf of the Town of Gibsons. The resulting review raised questions that are referenced in Section 4.0 below. Three supplementary subsurface investigations were carried out in April and December 2014 and January 2015 following comments from the Town's consultants in order to collect data to support the rezoning and Development Permit stage recommendations provided in this report.

2.0 SITE DESCRIPTION

The subject site is located on the waterfront in the town of Gibsons, BC, as shown on Figure 1, which is attached following the text of this report. The subject site includes the properties at 377, 385, and 407 Gower Point Road, 397 and 689 Winn Road, and the Winn Road right-of-way between 385 and 397 Winn Road. In addition, we understand that the site includes an existing water lease area, which extends into the harbour east of the site, as shown on Figure 2.

The site is bounded by Gower Point Road to the west, Gibsons Harbour (Howe Sound) and existing docks to the east, an adjacent residential property to the south, and an adjacent park to the north. Topography in the vicinity of and within the site is generally sloping gently down to the east-southeast toward the ocean, though a moderately steep slope exists at the northwest portion of the site.

At the times of our site visits, the site was occupied by several houses and marina-related buildings that are expected to be founded generally at grade. Several large, above-grade fuel tanks were observed at the northwest portion of the site, which we understand supplies fuel to the marina at the northeast portion of the site via several buried pipes. Several paved surface parking lots, landscaping, and treed areas occupied the balance of the site. Within the existing water lease area, several pile-supported floating docks (including a fuel dock) were observed within the central and southern portions of the existing water lease area. The existing Gibsons Marina to the southeast of the subject site and the government marina to the northeast of the site also include many pile-supported docks and boat slips.

3.0 PROPOSED DEVELOPMENT

3.1 General

Based on the architectural drawings referenced in Section 4.0, we understand that the proposed development comprises a new, multi-level hotel, conference centre, and residential development called "The George", to be partially supported over two to three parkade levels that daylight to the east. We also understand that the eastern portion of the site will comprise new, at-grade or above-grade amenities including a café, retail space, meeting room, and a 'seawalk', as well as gangways, an over-water restaurant building, and a marina with a fuel dock. We understand that a fuel storage tank is proposed to be located within the southwest portion of the parkade to provide service to the marina.

We understand that all existing structures would be demolished as part of the proposed development, and we assume that the proposed development would have a design life of approximately 50 to 75 years, as required by the BC Building Code 2012.

We also understand, based on the drawings from Balanced Environmental referenced in Section 4.0, that approximately 2.5 to 4.0 metres (8 to 13 feet) of dredging is proposed to be carried out within the existing water lease area in the harbour.

3.2 Expected Loads

The expected structural loads associated with the proposed development are currently unknown. We have assumed that structural loads will be consistent with those used for similar projects in our previous experience.

4.0 BACKGROUND INFORMATION

We have been provided with the following documents:

Drawings for the proposed development:

- architectural sketches of the P1, P2, and P3 levels, prepared by Omicron Architecture Engineering Construction Ltd., received February 3, 2015, which show the proposed development in plan view;
- topographic site survey plans prepared by Larry W. Penonzek, BC Land Surveyor, dated October 5, 2012, August 25, 2013, and January 28, 2015;
- float layout and dredge cut plan prepared by Balanced Environmental Services Inc., received May 14, 2014;

- architectural drawings prepared by Omicron Architecture Engineering Construction Ltd., dated October 18 and December 20, 2013, which show the proposed development in plan, elevation, section, and rendered 3D views;

- landscaping drawings prepared by PMG Landscape Architects, dated September 24, 2013; and

- drawings prepared by LB Petroleum & Environmental Consulting, dated September 10, 2012, which pertain to the proposed fuel tank to be installed at the site as part of the proposed development.

Peer review DRAFT reports:

- "Hydrogeological Review of 'The George' Geotechnical Investigation Report - DRAFT", prepared by Waterline Resources Inc. and dated June 25, 2014; and

- "Geotechnical Review: Horizon Engineering Inc. Report - June 5, 2014, Proposed George Hotel Development, Gibsons - DRAFT" report (issued for Client review), prepared by Levelton Consultants Ltd. and dated June 23, 2014.

Background information:

- Town Well #1 pumping records (December 2014 and January 2015), provided by the Town of Gibsons on January 28 and 29, 2015;

- marine electrical conductivity survey of a portion of the Gibsons Harbour provided by Waterline Resources Inc., received on April 9, 2014;

- "Aquifer Mapping Study - Town of Gibsons, British Columbia" report, prepared by Waterline Resources Inc. and dated May 13, 2013;

- "Aquifer Mapping Study Findings" document, prepared by Waterline Resources Inc. and dated May 8, 2012, which includes a schematic map and cross section through the Gibsons area and provides general information regarding the expected geology and aquifer extents near the subject site;

- "Waterfront Development: Gower Point Road, Gibsons, BC, Geotechnical Investigation" report, prepared by Thurber Engineering Ltd. and dated February 8, 2007, which describes a geotechnical investigation for a proposed development located approximately 60 metres (200 feet) south of the subject site's south property line (note that figures are not included);

- "Geotechnical Investigation: 377 Gower Point Road" report, prepared by Geo Tac Tics Engineering Ltd. and dated March 19, 2004, which provides geotechnical subsurface information and foundation recommendations for a proposed mixed use development at the north portion of the subject site that was similar in scope to the currently proposed development;

- "Groundwater Supply Testing: Town of Gibsons, British Columbia" report prepared by Piteau Associates and dated December 19, 2000;

- "Groundwater Supply Reconnaissance: Town of Gibsons, British Columbia - Interim Update" memorandum prepared by Piteau Associates and dated August 30, 1999;

- "Production Well 1 - Pump Test Results, Town of Gibsons, British Columbia" report by Piteau Associates, dated August 29, 1997;

- "Production Well No. 2 - Dougal Park" report, prepared by Bullock, Guelpa & Associates Ltd. and dated January 21, 1976; and

- "Report 1 - Detailed Well Record" BC Ministry of Environment well log for Well #19896, which we understand to be Town Well 1, dated April 1, 1966.

5.0 SUBSURFACE CONDITIONS

5.1 Site Investigations

Four phases of subsurface investigations were carried out at the subject site, as described in Section 1.0. Preliminary subsurface investigations were carried out on September 19 and 24, 2012 that involved excavating two test pits (TP12-1 and TP12-2), drilling three auger holes (AH12-1 through AH12-3), and advancing four Dynamic Cone Penetration Tests (DCPT12-1 through DCPT12-4) at the site. A supplementary subsurface investigation was carried out on April 11, 15, and 16, 2014 that involved drilling six auger holes (AH14-1 through AH14-6), advancing six DCPTs (DCPT14-1 through DCPT14-6), and drilling three sonic boreholes (BH14-1 through BH14-3) at the site. A second supplementary subsurface investigation was carried out on December 9 and 10, 2014 that involved drilling three sonic boreholes (BH14-4 through BH14-6) at the site. A third supplementary subsurface investigation was carried out on January 12, 2015 that involved advancing seven WildCat Cone Penetration tests (WC15-1 through WC15-7) in the foreshore area of the site.

The test pits and test holes were located in the vicinity of the proposed development, as shown on Figure 2. All auger holes, DCPTs, and test pits, as well as boreholes BH14-3, BH14-4, BH14-5, and BH14-6, were located within or adjacent to the subject site. WildCat holes WC15-1 through WC15-7 were located in the foreshore area, and boreholes BH14-1 and BH14-2 were located east of the shoreline, within the existing water lease area, and were drilled from a barge.

The test pits and test holes were logged to depths ranging from 2.4 to 15.2 metres (8 to 50 feet) below the existing grades (or below the seabed in the cases of boreholes BH14-1 and BH14-2), while the WildCat holes were advanced to depths of 1.3 to 3.5 metres (4 feet 3 inches to 11 feet 5 inches) below the existing grades. Select soil samples were retrieved from the test pits, auger hole flights, and sonic borehole core barrels, and returned to our office for further examination. The site investigations were directed and supervised by engineers with our office. A representative of the Town of Gibsons witnessed all of the aforementioned test pit and drilling activities at the site, and a hydrogeologist retained by Horizon Engineering attended the supplementary subsurface investigation carried out in December 2014. Test pit excavation was carried out by N.B. Contracting of Gibsons, auger drilling and DCPT testing were carried out by Uniwide Drilling of Burnaby, BC, sonic drilling was carried out by Mud Bay Drilling Co. Ltd. of Surrey, BC, and WildCat testing was carried out by HE Testing & Monitoring Ltd. of North Vancouver, BC. Utility locate searches were conducted by Western Utility Locate on September 19, 2012, April 11, 2014, and December 9, 2014, prior to the respective subsurface investigations, also directed and supervised by an engineer from our office.

As described in our original Proposed Scope of Services, we were provided with information prior to carrying out the 2012 site investigations indicating that Gibsons is underlain by a confined aquifer with artesian groundwater pressures and that this aquifer was not permitted to be penetrated by deep foundations, nor during the 2012 subsurface investigations. On that basis, we agreed to provide recommendations for the proposed foundations based on assumed subsurface conditions below the extent of the subsurface investigations and based on previous subsurface investigation results. These recommendations were provided in our previous Geotechnical Investigation Reports issued in November 2012 and February 2014, as previously described. Further characterization of the materials confining the aquifer (referred to as the "aquitard") was required by the scope of the April and December 2014 supplementary subsurface investigations to confirm that a suitable minimum aquitard thickness exists within the subject site for rezoning and Development Permit application, as is discussed in detail later in this report. This scope was also recommended by the aforementioned peer review reports, which were received in March 2014. Therefore, the auger drilling and DCPT subsurface investigation carried out on April 11, 2014 endeavoured to characterize the depth to the top of the aquitard, particularly at the west portion of the site, while the sonic drilling subsurface investigations carried out on April 15 and 16 and December 9 and 10, 2014 endeavoured to confirm the aquitard thickness and artesian groundwater pressures within the underlying aquifer at the test hole locations.

5.2 Soil Conditions

The soil stratigraphy encountered during the subsurface investigations is described following and on the auger hole, borehole, test pit, and WildCat Cone Penetration logs attached in Appendix B. In addition, five geotechnical sections are included on Figures 2 and 3. Laboratory testing, including sieve analyses, moisture content testing, and Liquid Limit, Plastic Limit, and Plasticity Index, were carried out on select soil samples. Laboratory testing results are included in Appendix D.

5.2.1 Sand Fill

At the locations of auger holes AH12-1, AH14-1 through AH14-6, boreholes BH14-3 through BH14-6, and test pits TP12-1 and TP12-2, approximately 0.2 to 2.7 metres (8 inches to 9 feet) of fill materials were encountered beneath asphalt, grass, or gravel at the surface. These materials generally comprised grey to brown, fine to coarse grained sand with trace gravel to gravelly, trace to no silt, and trace debris (bricks, nails, and/or plastic) and organics (roots, rootlets, and/or decomposed organics) at some hole locations. These materials were found to be moist to wet and were inferred to be very loose to dense and fill.

5.2.2 Peat

At all the auger hole, borehole, and test pit locations (with the exception of AH14-4, BH13-1, and BH13-2), approximately 0.1 to 1.8 metres (6 inches to 6 feet) of dark brown peat was encountered beneath the overlying fill or asphalt. This material generally comprised fine grained sand to silt and was mostly organic, fibrous, odorous, and moist to wet. This material was inferred to be very loose to loose / very soft to stiff and natural. The peat horizon is inferred to represent the natural ground surface prior to placement of overlying fill materials.

5.2.3 Sand

At the locations of auger holes AH12-1 through AH12-3, test pits TP12-1 and TP12-2, borehole BH14-1, and boreholes BH14-3 through BH14-6, approximately 0.1 to at least 1.7 metres (6 inches to at least 5 feet 8 inches) of sand materials were encountered, generally beneath the peat. These materials generally comprised grey to dark grey, fine to coarse grained sand with trace to some gravel and trace to no silt. These materials were found to be moist to wet and were inferred to be loose to dense.

5.2.4 Silty Sand to Sandy Silt to Silt

At all the auger hole, borehole, and test pit locations (with the exception of auger hole AH12-2 and test pit TP12-2), approximately 0.8 to 3.4 metres (2 feet 6 inches to 11 feet) of silty sand to sandy silt to silt materials were encountered, generally beneath the overlying peat or sand. These materials generally comprised grey to brown, fine to medium grained sand with trace to some gravel and trace coarse grained sand. These materials were found to be moist to wet and were inferred to be very loose to very dense (soft to very stiff where silt).

5.2.5 Till-Like Silty Sand to Silt and Sand

At the location of test pit TP12-1, auger holes AH12-1 through AH12-3, auger holes AH14-1, AH14-2, and AH14-4 through AH14-6, and boreholes BH14-4 through BH14-6, the sand to silty sand to sandy silt to silt materials were found to be underlain by grey to brown, fine to medium grained silty sand to silt and sand with some to no gravel and trace to no coarse grained sand. This material was measured to be at least 0.05 to 2.7 metres (at least 2 inches to 9 feet) thick, was found to be moist to wet and dense to very dense, and was inferred to be till-like. In accordance with the aforementioned restrictions on the subsurface investigation extent, the auger holes, DCPT soundings, and test pits were terminated once confirmation of the presence of till-like materials was obtained.

Based on the background information described in Section 4.0, a previous subsurface investigation was carried out by Geo Tac Tics Engineering at the north portion of the subject site. Three auger holes penetrated into this till-like material, which provide an estimation of its minimum thickness, as described in Section 5.3.5 below.

5.2.6 Sand to Sand and Gravel to Gravel

At the locations of boreholes BH14-1 through BH14-3, the silty sand to sandy silt to silt materials were found to be underlain by grey, fine to coarse grained gravelly sand to sand and gravel to gravel with occasional cobbles. At the locations of boreholes BH14-4 through BH14-6, the till-like silty sand to silt and sand materials were found to be underlain by grey, fine to coarse grained sand to sand and gravel to gravel with occasional cobbles. This material was measured to be at least 0.1 to 6.4 metres (at least 6 inches to 21 feet) thick, was found to be wet, and was inferred to be compact to very dense. These materials are inferred to comprise the Gibsons Aquifer, as described in Section 6.3.

5.2.7 WildCat Cone Penetration Test Results

Based on the observed consistency of the subsurface materials at the locations of WildCat Cone Penetration Tests WC15-1 through WC15-3, the stratigraphy at the south foreshore portion of the site (i.e., south of the proposed over-water restaurant location) was inferred to comprise three distinct horizons, as described below:

- loose to dense / stiff to hard soils, inferred to comprise shallow seabed sediments, to depths of 0.65 metre (2 feet 2 inches) at the location of WC15-1, 1.1 metres (3 feet 7 inches) at the location of WC15-2, and 1.6 metres (5 feet 3 inches) at the location of WC15-3,

- very loose to loose / soft to stiff soils, inferred to comprise buried seabed sediments, to depths of 3.3 metres (10 feet 8 inches) at the location of WC15-1, 3.3 metres (10 feet 10 inches) at the location of WC15-2, and 3.2 metres (10 feet 6 inches) at the location of WC15-3, and

- compact / stiff soils, inferred to comprise natural silty sand to sandy silt to silt (as described in Section 5.2.4) to depths of at least 3.5 metres (11 feet 5 inches) at the location of WC15-1, 3.3 metres (10 feet 8 inches) at the location of WC15-2, and 3.3 metres (10 feet 10 inches) at the location of WC15-3

Based on the observed consistency of the subsurface materials at the locations of WildCat Cone Penetration Tests WC15-4 through WC15-7, the stratigraphy at the north foreshore portion of the site (i.e., north of the proposed over-water restaurant location) was inferred to comprise two distinct horizons, as described below:

- very loose to loose / soft to firm soils, inferred to comprise shallow seabed sediments, to depths of 0.5 metre (1 foot 8 inches) at the location of WC15-5, 0.2 metre (8 inches) at the location of WC15-6, and 1.1 metre (3 feet 7 inches) at the location of WC15-7, and

- compact to dense / stiff to hard soils, inferred to comprise fill materials, to depths of 1.3 metres (4 feet 3 inches) at the location of WC15-4, 1.6 metres (5 feet 3 inches) at the location of WC15-5, 1.4 metres (4 feet 7 inches) at the location of WC15-6, and 1.4 metres (4 feet 7 inches) at the location of WC15-7.

5.3 Soil Conditions from Background Information

The soil stratigraphy understood to be encountered during the Geo Tac Tics Engineering drilling program and during the drilling of Town Well 1, as referenced in Section 4.0, is described following and on the borehole logs attached in Appendix C. It is noteworthy that the log for Town Well 1 is a hydrogeological log and does not include geotechnical data (such as density information). Horizon Engineering accepts no responsibility for the accuracy of test hole logs provided by others. The test holes provided in Thurber Engineering's report for a site located approximately 60 metres (200 feet) south of the subject site's south property line are excluded from this discussion due to their distance from the site.

5.3.1 Silt, Sand, and Gravel Fill

At the locations of BH04-08, BH04-09, and Town Well 1, approximately 0.6 to 3.2 metres (2 feet to 10 feet 6 inches) of fill materials were encountered at the surface. These materials generally comprised brown silt, sand, and gravel fill that were found to be moist to wet and loose to very dense, and were inferred to be fill.

5.3.2 Topsoil and Peat

At the locations of BH04-07, BH04-08, BH04-09, and Town Well 1, approximately 0.6 to 0.9 metre (2 to 3 feet) of dark brown peat was encountered beneath the overlying fill or topsoil materials. This material was noted to be fibrous and contain large roots at the location of BH04-07. The test hole logs indicate that this material was found to be loose to very dense.

5.3.3 Sand

At the locations of BH04-07 and BH04-08, approximately 0.6 to 1.7 metres (2 feet to 5 feet 7 inches) of sand materials were encountered beneath the peat. These materials generally comprised yellow-brown silty fine grained sand to grey medium grained sand, both with trace gravel. Organics were noted to be observed at the location of BH04-08. These materials were found to be wet and very loose to very dense.

5.3.4 Sandy Silt

At the locations of BH04-07, BH04-08, BH04-09, and Town Well 1, approximately 2.3 to 5.2 metres (7 feet 7 inches to 17 feet) of sandy silt materials were encountered beneath the overlying peat or sand. These materials generally comprised yellow-grey to grey to yellow-brown, fine grained sandy silt with trace gravel. The log for Town Well 1 describes these materials as "cobbles interfilled with silty fine sand" and "boulders interspaced with compact sandy silt". These materials were found to be moist to wet and were inferred to be soft to very stiff.

5.3.5 Till-Like Silty Sand

At the locations of BH04-07, BH04-08, and BH04-09, the sandy silt materials were found to be underlain by grey silty sand with gravel and cobbles, interbedded with lenses of fine grained sand and silt at the locations of BH04-07 and BH04-09. This material was measured to be at least 3.7 to 5.5 metres (at least 12 feet 2 inches to 18 feet) thick, extending to depths of at least 9.1 to 12.2 metres (at least 30 to 40 feet) below site grades. This material was found to be moist and dense to very dense, and was inferred to be till-like.

5.3.6 Sand to Sandy Gravel to Sand and Gravel

At the location of Town Well 1, the silty sand materials were found to be underlain by horizons of the following materials:

- medium to coarse grained sandy gravel with isolated layers of silt,
- coarse grained sand and gravel,
- coarse grained sand with some gravel,

- medium to coarse grained sand with occasional seams of gravel, and
- medium to coarse grained sand.

This material was measured to be 18.9 metres (62 feet) thick, extending down to at least 25.3 metres (83 feet) below adjacent site grades, and was found to be wet.

5.3.7 Sand to Silty Sand to Silt

At the location of Town Well 1, the sand to sandy gravel to sand and gravel materials were found to be underlain by horizons of the following materials:

- fine to medium grained silty sand,
- fine to medium grained sand,
- silt with peat stringers,
- fine to medium grained sand with some silt,
- medium grained silty sand,
- fine grained silty sand,
- fine grained sand, and
- fine grained sand with trace silt.

This material was measured to be at least 16.8 metres (55 feet) thick, extending down to at least 42.1 metres (138 feet) below adjacent site grades.

5.4 Groundwater Conditions

5.4.1 Non-Artesian Groundwater Conditions

During the subsurface investigations, non-artesian groundwater was encountered at depths of 0.9 to 3.7 metres (3 to 12 feet) within the test pits and auger holes. Perched groundwater was also observed within the surficial fill materials at the test pit locations. The non-artesian groundwater levels understood to have been encountered during Geo Tac Tics Engineering's drilling program and during the drilling of Town Well 1 ranged from 0.6 to 2.1 metres (2 feet to 6 feet 11 inches) below adjacent grades. The presence of non-artesian groundwater conditions could not be determined during sonic drilling, as water was used as a drilling fluid.

Perched groundwater within the more permeable surficial soil horizons is expected to be daylighted during excavation at the site. We expect that non-artesian groundwater levels at the site may be tidally influenced, particularly at the east portion of the property.

5.4.2 Artesian Groundwater Conditions

During the supplementary subsurface investigations, artesian groundwater was encountered at the following depths within the sonic boreholes, which were sealed prior to demobilizing the drilling equipment from the site. (Note that all elevations described in this report are geodetic.)

- 6.5 metres (21 feet 6 inches) below the seabed at the location of BH14-1; stabilized at an elevation of approximately -2.0 metres (-6 feet 7 inches),
- 5.5 metres (18 feet) below the seabed at the location of BH14-2; inferred to have stabilized at an elevation of approximately 3.2 metres (10 feet 6 inches),

- 4.6 metres (15 feet) below adjacent grades at the location of BH14-3; inferred to have stabilized at an elevation of approximately 4.5 metres (14 feet 9 inches),
- 14.9 metres (49 feet) below adjacent grades at the location of BH14-4; inferred to have stabilized at an elevation of approximately 11.3 metres (37 feet 1 inch),
- 7.3 metres (24 feet) below adjacent grades at the location of BH14-5; inferred to have stabilized at an elevation of approximately 5.8 metres (19 feet), and
- 7.3 metres (24 feet) below adjacent grades at the location of BH14-6; inferred to have stabilized at an elevation of approximately 7.8 metres (25 feet 7 inches).

Artesian groundwater is understood to have been encountered at a depth of 6.4 metres (21 feet) below adjacent grades at the location of Town Well 1, which is understood to have stabilized at an elevation of approximately 14.9 metres (48 feet 11 inches).

Although the test holes provided in Thurber Engineering's report for a site located approximately 60 metres (200 feet) south of the subject site's south property line were excluded from the soil conditions discussion in Section 5.3 due to their distance from the site, it is noteworthy that two of the holes (TH06-8 and TH06-9), which were located near the centre of the site, are understood to have encountered "slight artesian conditions". At a standpipe installed at the location TH06-9, we understand that "a stabilized [water] level would have been about 1 metre [3 feet 3 inches] above [the] ground surface." However, the report states that "it appears that the [artesian] water is sourced in the [sand and] gravel layer[s] below the peat...", which is understood to have been encountered between 1.6 and 4.0 metres (5 feet 3 inches and 13 feet 3 inches) below adjacent grades. These sand and gravel layers are understood to be underlain by firm to very stiff silt materials to depths of at least 7.6 metres (25 feet) below adjacent grades.

5.4.3 Groundwater Monitoring

Standpipe piezometers were installed at the locations of BH14-3 through BH14-6 during the supplementary subsurface investigations in order to facilitate groundwater monitoring within the upper portion of the Gibsons Aquifer. The standpipes were completed to depths of 7.3 to 15.2 metres (24 to 50 feet) below adjacent existing grades. The standpipe at BH14-3 was fitted with an analog pressure gauge, while water level dataloggers (transducers) were installed within each of the standpipes at BH14-4 through BH14-6 following standpipe installation. An additional datalogger was installed at the site to collect barometric data, which was used during processing of the water level monitoring data. Piezometer completion details are provided on the attached test hole logs.

The artesian pressures measured within approximately 0.3 to 0.5 metre (1 foot to 1.5 feet) below the adjacent grades were observed to be as follows:

- at BH14-3, approximately 28 kPa (4 psi) at the time of standpipe installation on April 16, 2014, and approximately 34 kPa (5 psi) on May 12, 2014, December 10, 2014, and January 12, 2015;
- at BH14-4, approximately 14 to 34 kPa (2 to 5 psi) in the period between December 10, 2014 and January 12, 2015;
- at BH14-5, approximately 30 to 41 kPa (4 to 6 psi) in the period between December 10, 2014 and January 12, 2015; and

- at BH14-6, approximately 12 to 22 kPa (2 to 3 psi) in the period between December 10, 2014 and January 12, 2015.

Further discussion regarding artesian pressures in the vicinity of the subject site is provided in Section 6.3.

PART B - DISCUSSIONS AND RECOMMENDATIONS

6.0 HYDROGEOLOGY

6.1 General

Waterline Resources Inc. recently completed a comprehensive Aquifer Mapping Study of the Gibsons Aquifer in 2013, which is referenced in Section 4.0. The discussion that follows in this section is based largely on information published in that report.

6.2 Regional Hydrogeology

The Town of Gibsons is underlain by the Gibsons Aquifer, which is a confined aquifer comprising sand and gravel that provides drinking water for the town. A significant portion of the recharge to the Gibsons Aquifer occurs via mountain block recharge on Mt. Elphinstone (located northwest of the town) or recharge through creekbeds in the aquifer catchment area. Over most of its areal extent, the Gibsons Aquifer is confined by low hydraulic conductivity (low permeability), till-like soils that are collectively termed the Gibsons Aquitard. In some areas, the Gibsons Aquitard is overlain by the unconfined alluvial deposits of the shallower Capilano Aquifer. In the vicinity of the subject site, artesian pressures have been observed in wells and standpipes installed into the Gibsons Aquifer, including the four town water supply wells. The Gibsons Aquifer is understood to naturally discharge to the seabed beneath Gibsons Harbour.

The aforementioned Waterline report identifies the primary hydrogeologic units in the Gibsons area as comprising the following, listed in descending stratigraphic order:

- unconfined surficial Capilano Aquifer;
- low permeability Gibsons Aquitard;
- high permeability Gibsons Aquifer; and
- bedrock.

The Capilano Aquifer and Gibsons Aquitard are generally thicker in the Upper Gibsons area than at the subject site. The Capilano Aquifer has a thickness of up to 9 metres (30 feet) in the Gibsons area; however, the Waterline report indicates that it is not present in the vicinity of the town wells, nor in the project area. In the Upper Gibsons area, the Gibsons Aquitard has a thickness of up to 30 metres (100 feet). Waterline's report indicates that its thickness diminishes substantially near the harbour. Recent subsurface investigations carried out at the subject site have confirmed this, as described in Section 5.2.

The Gibsons Aquifer has a maximum estimated thickness of approximately 120 metres (400 feet), extends from the base of Mt. Elphinstone to Howe Sound, and is present throughout the project area. The Gibsons Aquifer provides the majority of the town's water supply via four supply wells, labelled Town Well #1 through Town Well #4. Town Well #3, located approximately 300 metres (980 feet) southwest of the subject site in Dougall Park, operates continuously. Town Wells #1 and #4, located on Gower Point Road approximately 35 and 120 metres (115 and 400 feet) west and north of the subject site, respectively, supplement Town Well #3 as required. Town Well #2, located approximately 270 metres (890 feet) southwest of the subject site in Dougall Park, is a backup well for Town Well #3.

6.3 Local Hydrogeology

At the subject site, recent subsurface investigations have intersected the Gibsons Aquifer at six borehole locations (i.e., BH14-1 through BH14-6), as described in Section 5.2.6, the locations of which are shown on Figure 2. At three of these locations (i.e., BH14-4, BH14-5, and BH14-6), the Gibsons Aquifer was found to be overlain by approximately 1 to 3 metres (3 to 9 feet) of till-like soil, as described in Section 5.2.5. At all six of the BH14-series borehole locations, a horizon of lower-strength silty sand to sandy silt to silt was encountered above the till materials, where present, or directly overlying the Gibsons Aquifer where the till material was absent, for a combined low-permeability aquitard thickness of 1.2 to 4.1 metres (4 to 13.5 feet). The silty sand to sandy silt to silt materials were observed to be overlain by discontinuous sand pockets, as described in Section 5.2.3, which may be remnants of the Capilano Aquifer. At most test hole locations, peat was encountered above the sand and silty sand to sandy silt to silt materials during drilling, as described in Section 5.2.2. At the western portion of the subject site, fill materials were noted near the surface, as described in Section 5.2.1. Beneath the harbour, loose seabed sediments were encountered over the underlying sand and silty sand materials. Because the sand and peat materials encountered above the silty sand to sandy silt to silt materials were found to be discontinuous, these materials have been combined with the silty sand to sandy silt to silt and till-like materials to form the "inferred Gibsons Aquitard", as illustrated on the legends shown on Figures 2 and 3. It is noteworthy that these components of the inferred Gibsons Aquitard were treated as separate soils with different hydraulic conductivities during the seepage analyses, as described in Section 8.3. In general, an aquitard is a low permeability material, specifically, a material that is significally lower in permeability than the aquifer. At the subject site, the specific geological deposits termed the "Gibsons Aquitard" includes a number of sedimentary deposits that have different geotechnical properties.

During and following drilling of the aforementioned boreholes, artesian pressures were observed within the Gibsons Aquifer, as described in Section 5.4.2. Artesian pressures were not observed in any of the overlying materials. At the locations of BH14-4, BH14-5, and BH14-6, hydraulic heads in the Gibsons Aquifer were recorded using dedicated transducers, installed following drilling, for a period of 33 days. The hydraulic heads observed at these wells for the period of December 10, 2014 to January 12, 2015 are shown on Figures 4 through 6. Tidal elevations, daily precipitation amounts, and average daily pumping rates for nearby Town Well #1 are also shown on these figures. It is noteworthy that a direct correlation between the measured hydraulic head and the tidal elevations in Howe Sound is evident at the location of BH14-5, which indicates a strong hydraulic connection between the Gibsons Aquifer and the ocean (see Figure 5). This borehole is located approximately 60 metres (200 feet) from the harbour (high water mark) at the southern portion of the subject site, and the top of the Gibsons Aquifer was observed at an elevation of approximately -3.2 metres (-10.5 feet) at this location. At the location of BH14-6, the correlation between tidal elevations and measured hydraulic head is muted relative to that observed at BH14-5 (see Figure 6), presumably due to the greater distance to the shoreline and greater vertical separation between the harbour and the top of the Gibsons Aquifer. This borehole is located on Gower Point Road near the southwest corner of the subject site, approximately 120 metres (400 feet) from the harbour, and the top of the Gibsons Aquifer was observed at an elevation of approximately 1.7 metres (5.5 feet) at this location. At the location of BH14-4, the correlation between tidal elevations and measured hydraulic head is further muted relative to BH14-6 (see Figure 4), presumably due to the greater vertical separation between the harbour and the top of the Gibsons Aquifer. This borehole is located near Gower Point Road at the west-central portion of the subject site, approximately 75 metres (250 feet) from the harbour, and the top of the Gibsons Aquifer was observed at an elevation of approximately 1.2 metres (4 feet) at this location.

Although tidal effects can be seen at the location of BH14-4, the more pronounced time series signature at this location is a sharp decline in hydraulic head that occurs approximately every 34 hours. This head drop appears to be due to the periodic pumping of Town Well #1, which is located approximately 45 metres (150 feet) from BH14-4. Between pumping periods, the static water head elevation in the upper portion of the Gibsons Aquifer at the location of BH14-4 was observed to be approximately 12.8 metres (42 feet), or approximately 2.7 metres (9 feet) above the adjacent ground surface. Although BH14-6 is located farther from the shoreline than BH14-4, the static water head elevation at this location is approximately 8.3 metres (27 feet), or approximately 4.5 metres (15 feet) lower than that at BH14-4. The average static water head elevation at the location of BH14-5 is approximately 6.2 metres (20 feet), or approximately 6.6 metres (22 feet) lower than that at BH14-4. The lower hydraulic heads at the locations of BH14-5 and BH14-6 indicate that the hydraulic connectivity between the Gibsons Aquifer and Howe Sound is higher in the vicinity of BH14-5 and BH14-6 than in the vicinity of BH14-4. This connectivity could be due to a higher hydraulic conductivity within the Gibsons Aquifer in the BH14-5 and BH14-6 areas compared to the BH14-4 area, or a reduced thickness of confining materials (i.e., aquitard) between the ocean and the aquifer at the locations of BH14-5 and BH14-6. This latter possibility is consistent with the greater till-like material thickness observed at the location of BH14-4 compared to at the locations of BH14-5 and BH14-6, as well as with the stratigraphic mapping presented in Waterline's 2013 report, which notes that the Gibsons Aquitard is absent in the Bluff area, south of the existing marina that is located south of the subject site.

7.0 POTENTIAL GEOTECHNICAL CHALLENGES

Based on the results of our site investigations and our review of the background information described in Section 4.0, we consider the subject site to be geotechnically complex. We are aware of the following potential geotechnical challenges at the site, each of which is addressed in the subsequent sections of this report.

7.1 Artesian Groundwater Pressures

As described in Sections 5.1 and 6.2, Gibsons is underlain by a confined aquifer with artesian groundwater pressures, capped by an aquitard comprising low-permeability soils, including till-like materials in some areas. Based on the background information provided in Section 4.0, we understand that the following potential geotechnical challenges are associated with potential ground heave and/or "blowout" of the aquitard materials following excavation of overlying soils (within the building footprint and/or dredging areas) or puncturing the aquifer with deep foundations (within the building footprint and/or marina areas):

- formation of an uncontrolled sinkhole,
- depressurization of the aquifer,
- ground settlement following depressurization, and
- contamination of the aquifer, which is a drinking water source for Gibsons, either by sea water or toxic materials.

Further discussion on addressing artesian groundwater challenges is provided in Sections 8.3, 10.0, 12.0, and 17.0.

7.2 High Non-Artesian Groundwater Levels

As described in Section 5.4.1, non-artesian groundwater was encountered during the subsurface investigations at depths of 0.9 to 3.7 metres (3 to 12 feet) within the test pits and auger holes. As illustrated on Figures 2 and 3, most of these water levels are higher than the proposed excavation elevation; therefore, significant groundwater is expected to be daylighted during excavation at the site. Potential geotechnical challenges associated with high non-artesian water levels include:

- temporary construction dewatering,
- permanent foundation drainage or waterproofing, and
- potential impact on adjacent grades following dewatering (temporary and permanent).

Further discussion on addressing non-artesian groundwater challenges is provided in Sections 10.0 and 14.0.

7.3 Sea Level Rise

Based on the background information provided in Section 4.0, we understand that the marine high water elevation (which typically coincides with the natural boundary) at the subject site is 2.15 metres (7 feet). We envisage that the lowest proposed top of slab elevation for the proposed building may be near the current high water elevation. The Town of Gibsons' Official Community Plan (2005) and a document published by the Association of Professional Engineers and Geoscientists (APEGBC, 2012) indicate that sea level could rise up to 0.82 to 1.0 metre (2 feet 8 inches to 3 feet 3 inches) along the BC coast by the year 2100. This could pose the following geotechnical challenges to the proposed development:

- flooding of the lower parkade level,
- inundation of the foundation drainage system (if applicable) during flooding, and
- structural problems with the slab-on-grade resulting from uplift during flooding.

Further discussion on addressing sea level rise is provided in Section 9.2.

7.4 Tsunami Hazard

A tsunami is a series of ocean waves generated by sudden displacements in the sea floor, landslides, or volcanic activity. In Gibsons, the most probable cause of a tsunami is due to sudden displacements of the sea floor caused by a large earthquake. We understand that a tsunami in the Gibsons area could be expected to be less than approximately 2.0 metres (6 feet 7 inches) in height (Clague et al., 2001). A tsunami could pose the following potential geotechnical challenges to the proposed development:

- short term flooding of the lower parkade level, retail spaces, seawalks, and over-water restaurant, and
- structural damage and physical injury due to wave impact.

Further discussion on tsunami hazard is provided in Section 9.3.

7.5 Loose and Compressible Soil

As described in Sections 5.2 and 5.3, significant thicknesses of loose and compressible soils were encountered at the test pit and test hole locations. If these soils are subjected to surcharge loads, such as from the proposed foundations or slabs-on-grade, they could be expected to undergo significant settlement. If these soil types are subjected to external loads:

- foundations of the proposed building placed directly over this compressible soil may experience settlements that may impact safety and performance of buildings, or

- slabs-on-grade and other settlement sensitive structures may undergo significant settlement that may aversely impact performance of the building.

Further discussion on loose and compressible soil is provided in Sections 10.0, 12.0, and 16.0.

7.6 Liquefaction

Liquefaction is the loss of shear strength of soil due to an increase in pore water pressure resulting from cyclic shear stress. This condition can occur in loose, saturated, granular deposits during an earthquake of sufficient magnitude. Potentially liquefiable soils are judged to be present within the subject site, which poses a potential geotechnical challenge, as liquefaction of soils supporting building loads (both shallow or deep) can result in foundation failure and significant settlement. Further discussion on liquefaction is provided in Sections 11.0 and 13.4.

7.7 Impact of Dredging of Foreshore Area

Based on the background information provided in Section 4.0 and as discussed in Section 3.1, we understand that approximately 2.5 to 4.0 metres (8 to 13 feet) of dredging (depth estimated above the proposed dredge cut toe) is proposed to be carried out near the shoreline within the existing water lease area in the harbour. We envisage that dredging is proposed to facilitate moorage of boats within the proposed marina areas that are currently too shallow. Potential geotechnical challenges associated with dredging include:

- "blowout" of the aquitard materials following removal of overlying soils during dredging, as described in Section 7.1, and
- stability of submarine excavation slopes.

Further discussion on dredging is provided in Section 17.3.

7.8 Methane Buildup

As described in Sections 5.2 and 5.3, a horizon of peat was encountered during the subsurface investigation at the majority of the site. If unsaturated organic materials such as peat remain beneath a structure, methane that is released during decomposition of organic material can build up beneath the slab-on-grade. Methane is extremely flammable when enclosed and concentrated; therefore, its presence beneath a structure poses a potential geotechnical hazard.

Further discussion on methane buildup is provided in Section 15.0.

8.0 COMPUTER MODELLING

8.1 General

A computer model was generated to analyse the site and subsurface conditions during and after construction of the proposed development based on existing information, published literature, and engineering judgement. Although this type of analysis may not be required at the re-zoning stage for the majority of projects, Horizon Engineering carried out such modelling and analysis in order to provide a reasonable estimate of ground behaviour due to construction of the proposed structure. In addition, the results of this analysis shows a conservative approach for the prediction of stresses and deformations.

The computer modelling was carried out using a commercially available software program called GeoStudio 2007, which comprises several modules that can be used for both seepage and deformation analyses. A seepage analysis was carried out to establish a steady state groundwater condition (as the initial condition for the deformation analyses), and stress and deformation analyses, based on the configuration of the proposed development, were carried out to estimate changes in subsurface stress conditions as excavation progresses and to determine potential ground movements at each stage.

For the above analyses, we have used Section A, which is shown on Figure 2, as the reference for slope geometry and subsurface soil modelling. This section represents the area where excavation for the proposed building would be greatest at the subject site.

Eight soil types were assigned to represent the expected subsurface soil conditions, as described in Section 8.2. Soil properties were estimated based on in-situ testing, laboratory testing, published literature, and our previous experience. It should be noted that due to the nature of this project and limitations on the subsurface investigations that were carried out, a rigorous sampling and laboratory testing program to develop a complex soil model was not practicable nor feasible at the time of executing the subsurface investigations. We envisage that the analyses carried out are sufficient to provide a reasonable order of magnitude result, which is considered to be conservative. Engineering judgement was used to interpret the results of the analyses and to provide recommendations for the design and construction of the proposed development from a geotechnical point of view.

8.2 Soil Properties

In general, an elastic-plastic soil model was used for the analyses. Strength and deformation parameters were estimated based on the soil density/consistency values measured by Dynamic Cone Penetration Test (DCPT) and Standard Penetration Test (SPT) results during the subsurface investigations, which were confirmed with published literature. Strength and deformation parameters for the till-like materials were also estimated based on typical values and published information (Klohn, 1965). The soil properties used for computer modelling are summarized below.

Table 1 : Soil Properties Used for Computer Modelling

SOIL TYPE	UNIT WEIGHT	FRICTION ANGLE	UNDRAINED SHEAR STRENGTH	YOUNG'S MODULUS	DILATION ANGLE	PERMEABILITY
	(kN/m^3)	(degrees)	(kPa)	(MPa)	(degrees)	(metres/second)
Fill	17.5	40.0	-	35.1	10.0	1.0E-5 to 5.0E-5
Peat	12.3	0.0	59	5.2	-	1.0E-6
Sand	18.3	44.3	-	31.3	12.0	1.0E-4 to 5.0E-5
Silty Sand	19.0	43.0	-	33.5	8.0	1.0E-6 to 1.0E-5
Till	23.6	45.0	150 to 300	60.0 to 120	15.0	1.0E-8 to 5.0E-7
Gravel	21.6	45.0	-	40.9	15.0	2.6E-3 to 1.0E-1
Seabed Sediments	13.5	34.1	-	13.9	5.0	1.0E-2
Seabed Silty Sand	15.7	37.1	-	19.3	5.0	1.0E-4

8.3 Seepage Analysis (Initial State)

In order to establish artesian groundwater pressures and a steady state condition prior to carrying out deformation analyses, a seepage analysis was carried out. This seepage analysis was carried out using a commercially available software program called SEEP/W (Version 2007), which mathematically models the physical process of water flowing through porous media.

For the purpose of this seepage analysis, we have applied boundary conditions at the east and west ends of the section with 3.2 and 14.9 metres (10 feet 6 inches and 48 feet 11 inches) of total head pressure, respectively. Where the existing grade elevations are less than 2.2 metres (7 feet 2 inches), 2.2 metres (7 feet 2 inches) of total head pressure was applied to simulate the high water level described in Section 7.3. Elsewhere, a potential seepage surface (total head pressure of zero) was applied. After these boundary conditions were applied, the model was executed for the steady state condition.

For the seepage analysis, we have used ranges of permeability (hydraulic conductivity) values for some of the soil materials, as listed in Table 1 above. However, for the steady state condition, there are no significant changes in the estimated pore water pressure generated within the model. Based on this modelled steady state condition, a flux of groundwater into the west portion of the proposed excavation was estimated, which is described in Section 14.3. An artesian groundwater pressure was established at the underside of the Gibsons Aquitard with a range of 130 to 160 kPa (2,715 to 3,550 psf). It is noteworthy that these pressures appeared to be affected by the shape of the aquitard's lower limit; however, for this analysis, the effect of the soil layer shapes was not explored. The resultant pore water pressure at the location of borehole BH14-4 within the aquifer was estimated to be 12.8 metres (42 feet) of head, which is in agreement with the observed water pressure measurement at the standpipe piezometer installed at this borehole location, as described in Section 6.3. The groundwater pressures established with the seepage analysis were used as initial loading conditions for the subsequent deformation analyses, which are described below.

8.4 Deformation Analyses

The deformation analyses were carried out using a commercially available program called Sigma/W (Version 2007) for the purpose of determining the expected magnitudes of ground deformation at the base of the proposed excavation that could result from the effects of the underlying artesian groundwater pressures within the Gibsons Aquifer. The model was established with the output of the previously described seepage analysis as the initial condition. The deformation analyses were carried out in seven stages to simulate the progressive bulk excavation, and ground movements were calculated at each stage. Each stage simulates approximately 1.0 metre (3 feet) of vertical excavation across the site, starting at the existing grades at the west property line. Effective, drained parameters were applied for all soil types for these analyses.

We have used "coupled" models for the deformation analyses, which involve solving equilibrium equations and seepage continuity equations simultaneously. This requires an additional coupling function related to permeability changes based on the pore water pressure status. In order to obtain the coupling hydraulic properties for all soil materials, we have used built-in functions provided by Sigma/W that are estimated based on the saturated permeability, water contents, and residual water contents.

In the numerical model, the simulated excavation was carried out in seven vertical stages. The temporal duration of the each stage of excavation was set to be within 5 to 10 days, which is expected to be within the rate of excavation. Based on the sensitivity analysis, we found that the minimum period of 5.5 days will be required to provide stable seepage conditions (stable pore pressures) In order to reduce the risk of being less conservative due to a premature deformation at each step, each stage was divided into five sub-stages, and the deformation trends of each sub-stage were also reviewed. In addition to the above, we also carried out a deformation analysis with a longer excavation stage duration to confirm that the aforementioned duration assumption was reasonable.

The aforementioned deformation model was analysed with boundary conditions with fixed horizontal coordinates at both ends of the model and fixed vertical coordinates at the bottom of the model. Ideally, the model used for numerical analyses should be sufficiently large such that the area of interest within the model is located with an adequate setback distance from the model boundaries. Under the aforementioned boundary conditions, it should be noted that minor lateral deformations were observed. We also carried out the analyses with a boundary condition with fixed horizontal and vertical coordinates along the base of the model.

Based on the results of the deformation modelling with the aforementioned ranges of hydraulic conductivities, Young's Modulus, and boundary conditions, we observed that no traceable ground heave or subsequent tension cracks at the excavation surface are expected until the excavation extends to a depth of 5.0 metres (16 feet 5 inches) below the existing grade at the west portion of the proposed building footprint area, which corresponds to an elevation of approximately 5.0 metres (16 feet 5 inches).

Based on the aforementioned analyses, excavation below an elevation of approximately 5.0 metres (16 feet 5 inches) appears to have a tendency for local ground heaving. For example, at the "stage six excavation" (which corresponds to one additional metre of excavation to an elevation of 4.0 metres / 13 feet 1 inch at the bottom of the excavation), the model indicates that the deformation due to the uplift pressure could be significant, resulting in an unacceptable magnitude of ground heaving. Therefore, we recommend that the proposed excavation not advance below 5.0 metres (16 feet 5 inches, or 16.4 feet) geodetic elevation in order to ensure that the Gibsons Aquifer is not compromised due to excavation of the overlying materials.

It must be noted that the results of these analyses are based on conservative soil strength properties. As a result, there is an inherent Factor of Safety (which may be of the order of approximately two) in the deformation analysis results. Therefore, the actual safety margin for excavation is around 2.0 metres. Section 21.2 of this report provides more information regarding excavation details. The computer modelling results are summarized in Appendix E.

9.0 FLOOD HAZARDS

9.1 General

Based on the site's proximity to the ocean, we envisage that the development is subject to flood hazards due to storm waves, tsunamis, and rising sea levels that are expected to occur as a result of climate change during the design life of the building.

9.2 Flood Construction Level

The Flood Construction Level (FCL) is defined as the design flood level plus an allowance for freeboard (APEGBC, 2012). By definition, the underside of all wooden floor systems or tops of all concrete slabs for habitable areas must be located at or above the FCL. It is noteworthy that "habitable areas" are defined in The British Columbia Ministry of Water, Land, and Air Protection's Flood Hazard Area Land Use Management Guidelines (May 2004) as "Any room or space within a building or structure that is or can be used for human occupancy, commercial sales, or storage of goods, possessions, or equipment (including furnaces), which would be subject to damage if flooded." Therefore, we envisage that the lowest level of the proposed building must be located at or above the FCL, as it is intended to be used for vehicle storage.

In coastal areas, the FCL is defined as the high water elevation plus allowances for storm surge, wave runup, and freeboard to address uncertainties in the prediction (Ausenco Sandwell, 2011). We are not aware of a hydraulic engineering assessment in the vicinity of the site that would indicate the recommended FCL for the design flood; therefore, we envisage that the following FCL estimate would be suitable for the proposed development.

As described in Section 7.3, we understand that sea level could rise up to 0.82 to 1.0 metre (2 feet 8 inches to 3 feet 3 inches) along the BC coast by the year 2100. As described in Section 3.1, we assume that the proposed development would have a minimum design life of approximately 50 years. Assuming that the proposed development is completed in 2015 and assuming that sea level rise occurs linearly, we therefore assume that sea level at the site could rise 0.53 metre (1 foot 9 inches) by 2065 (i.e., 65% of the expected sea level rise from 2000 to 2100). The provincial guidelines for future (i.e., post sea level rise) FCL recommend that the following be added to the future high water elevation to establish the future FCL (Ausenco Sandwell, 2011):

- 1.4 metres (4 feet 7 inches) for storm surge allowance (assuming that the value indicated for Vancouver Harbour is relevant to Gibsons Harbour, as it is judged to be reasonable based on proximity and similar degrees of protection from the Strait of Georgia),
- 0.65 metre (2 feet 2 inches) for wave runup, and
- 0.6 metre (2 feet) for freeboard.

As discussed in Section 7.3, we understand that the current high water and natural boundary elevation at the subject site is 2.15 metres (7 feet). Adding 0.53 metre (1 foot 9 inches) for sea level rise by 2065 and the above allowances for storm surge, wave runup, and freeboard, we envisage the FCL at the end of the building's design life in approximately 2065 would be approximately 5.33 metres (17 feet 6 inches).

We understand that the above may be considered to be a conservative estimation of FCL. Therefore, if a more accurate estimate of the FCL is required, we recommend that the above FCL recommendations be reviewed by a hydraulic engineer such that site specific recommendations can be provided.

We envisage that habitable spaces could be constructed below the FCL if a "sea dike" is constructed around the building (including the north and south sides, where applicable), which would be designed to protect the building from rising sea levels and future storm events. This sea dike could comprise a water-tight wall or other gravity structure that could be incorporated into the proposed seawalk structure (a conceptual sketch is provided on Figure 2). Based on existing information, it appears that grades at the locations of the proposed sea dike would be raised up to 3.3 metres (10.8 feet). The proposed marina and over-water restaurant should be constructed at or above the FCL since they would otherwise be unprotected from the design flood conditions. Details of the proposed sea dike could be provided under separate cover after the above-stated recommendations are reviewed by the design team, when more information regarding landscaping design is available, and, if necessary, following review of the FCL by a hydraulic engineer.

9.3 Tsunami Hazard

As described in Section 7.4, we understand that a tsunami in the Gibsons area could be expected to be less than approximately 2.0 metres (6 feet 7 inches) in height (Clague et al., 2001). The aforementioned recommended FCL exceeds this height above the future expected high water elevation; therefore, we envisage that the proposed development would not be subjected to a tsunami hazard provided that the requirements for FCL are met.

10.0 BUILDING FOUNDATION CONCEPT

10.1 Proposed Building Grades

Based on the aforementioned architectural sketches, we envisage that the lowest proposed top of slab elevations would be approximately 6.3 metres (20.5 feet) [geodetic] at the west portion of the site and 3.1 metres (10.2 feet) at the east portion of the site. Allowing 0.9 metre (3 feet) from the top of slab elevations to the underside of footing elevations, we envisage that the proposed footing elevations would be approximately 5.4 metres (17.5 feet) at the west portion of the site and 2.2 metres (7.2 feet) at the east portion of the site. The extent of the lowest proposed excavation area is shown on Figure 2.

Based on existing grades indicated on the site survey plan referenced in Section 4.0, we envisage that the proposed excavation would be approximately 5.1 to 5.8 metres (17 to 19 feet) deep below adjacent existing grades at the northwest portion of the site. Excavation at the southwest, northeast, and southeast portions of the site is envisaged to be less than approximately 0.5 metres (1.6 feet) deep below adjacent existing grades.

Based on the architectural drawings, we envisage that the proposed finished floor elevation for the proposed café, retail space, meeting room, seawalk, and over-water restaurant at the east portion of the site would be approximately 5.3 metres (17.3 feet), which is noted to be consistent with the FCL recommended in Section 9.2. It is noteworthy that the currently proposed lowest finished floor elevation of 3.1 metres (10.2 feet) is approximately 2.2 metres (7.3 feet) below the FCL; therefore, the sea dike proposed in Section 9.2 is envisaged to be required to facilitate construction of habitable spaces below the FCL.

10.2 Recommended Building Grades

As described in Section 8.0, we recommend that the proposed excavation not advance below 5.0 metres (16 feet 5 inches, or 16.4 feet) geodetic elevation at the northwest portion of the site. At the balance of the site, we recommend that the proposed excavation not advance below 0.5 metre (1 foot 8 inches, or 1.6 feet) below existing grades in order to ensure that the Gibsons Aquifer is not compromised due to excavation of the overlying materials. Deeper excavation at the southwest portion of the site is not recommended due to the proximity of the Gibsons Aquifer to the existing site grades, as observed at the location of BH14-6 (see Section D on Figure 3).

Notwithstanding the Flood Construction Level recommendations provided in Section 9.2 and allowing 0.9 metre (3 feet) from the top of slab elevation to the underside of footing elevation, we recommend that the lowest proposed top of slab (finished floor) elevation not be lower than approximately 5.9 metres (19.4 feet) at the northwest portion of the site, which is consistent with the aforementioned proposed building grades, as described in Section 10.1. As illustrated on Figures 2 and 3, (in plan view and sections) the proposed excavation is expected to daylight approximately 30 metres (100 feet) east of the west property line at the north portion of the site.

10.3 Recommended Building Foundation Concept

The subgrade materials that are expected to be exposed at the northwest portion of the site are envisaged to comprise the following, sequentially exposed from east to west:

- sand fill (as described in Section 5.2.1),
- peat (as described in Section 5.2.2),
- sand (as described in Section 5.2.3), and
- silty sand to sandy silt to silt (as described in Section 5.2.4).

The subgrade materials that are expected to be exposed at the balance of the site are envisaged to comprise the aforementioned materials. Till-like silty sand to silt to sand materials (as described in Section 5.2.5) are not expected to be exposed at the proposed foundation elevations anywhere at the site.

We recommend that foundations for the entire building footprint are supported on conventional strip and pad foundations or on a raft foundation. (Foundation type would be determined during the detailed design stage.) Due to the presence of loose and compressible subgrade materials (which are judged to be unsuitable for supporting shallow foundations in their current state, as described in Section 7.5), either the proposed foundation loads should be transferred down to a suitable bearing strata (either the till-like silty sand to silt and sand or compact to very dense sand to sand and gravel to gravel) or some form of ground improvement should be implemented. The use of deep foundations may be appropriate for this site, but due to the potential impact of deep foundations on the aquifer during or after installation, ground improvement is judged to be the more suitable option to provide support for the proposed foundations. Further discussion on ground improvement techniques is provided in Section 12.0.

Where excavation is not required to reach the design foundation elevation at the east portion of the site, subexcavation of the aforementioned loose and compressible materials in order to construct foundations (or place Engineered Fill) directly on the underlying competent (i.e., dense to very dense and non-compressible) materials is not recommended for the following reasons. Firstly, subexcavation is expected to be impractical at most of the footprint area both due to the significant thickness of soils requiring excavation and due to challenges with significant groundwater inflow that would be expected as a result of the high non-artesian groundwater levels observed at the site, as well as the site's proximity to the ocean. Secondly, there is known to be negligible competent soil overlying the aquifer at some portions of the site, as shown on Figures 2 and 3; therefore, removal of the confining aquitard materials (including the sand, peat, silty sand to sandy silt to silt, and till-like materials, as described in Section 6.3) is not recommended in these areas. For these reasons, we recommend that footings at the east portion of the site, where excavation is not required, are lowered to the existing grades after the proposed ground improvement measures are complete. In these areas, floor slabs are recommended to be designed as suspended slabs, as surcharging the underlying loose and compressible soils with fills and slab surcharge loads is not recommended.

11.0 LIQUEFACTION ASSESSMENT

As described in Section 7.6, potentially liquefiable soils are judged to be present within the subject site. In order to inform discussions regarding the liquefaction susceptibility of subsurface soils, we have carried out liquefaction triggering analyses using the data collected from the subsurface investigations.

Dynamic Cone Penetration Test blow count (N) values, which are indicated on the test hole logs attached in Appendix B, were converted to equivalent Standard Penetration Test (SPT) $N_{(60)}$ blow count values in accordance with the methods and relationships proposed by Sy and Campanella (1993). The $N_{(60)}$ values were then corrected for overburden pressure to produce equivalent $(N_1)_{60}$ values.

The liquefaction analysis is based on comparing the cyclic stresses caused by the design magnitude earthquake with the inferred resistance of the soil to these stresses. The cyclic stresses that are generated are defined as the Cyclic Stress Ratio (CSR), while the resistance parameter is termed the Cyclic Resistance Ratio (CRR). The CSR is derived from correlations to the BC Building Code design magnitude earthquake (i.e., 1:2475 year event, or a 2% chance of exceedance in 50 years) and soil profile characteristics using the simplified equations proposed by Seed and Idriss. The CRR can be calculated in accordance with the method proposed by the "Task Force Report: Geotechnical Design Guidelines for Buildings on Liquefiable Sites in Accordance with NBC 2005 for Greater Vancouver Region" (Greater Vancouver Liquefaction Task Force Report, 2007). The method uses the inferred SPT $(N_1)_{60}$ blow count values and the soil properties to determine a CRR. The Factor of Safety is defined as the ratio between the expected soil resistance to cyclic stresses (CRR) and the stress induced by the earthquake (CSR); therefore, where the CSR exceeds the CRR, the Factor of Safety is less than one and liquefaction is predicted. The zones within the auger holes that are expected to be located below the proposed foundation elevations that are envisaged to be potentially liquefiable are as follows:

Table 2: Liquefaction Assessment Results

HOLE	DEPTH BELOW EXISTING GRADES TO POTENTIALLY LIQUEFIABLE SOILS (WHERE PRESENT BELOW THE PROPOSED FOUNDATION ELEVATION)	MAXIMUM THICKNESS OF POTENTIALLY LIQUEFIABLE ZONES	FACTOR OF SAFETY RANGE
AH12-1	3.0 to 4.0 metres (10 to 13 feet)	0.9 metre (3 feet)	0.4 to 0.7
AH12-2	1.8 to 2.1 metres (6 to 7 feet)	0.3 metre (1 foot)	0.6
AH12-3	1.5 to 1.8 metres (5 to 6 feet) and 2.4 to 4.3 metres (8 to 14 feet)	1.8 metres (6 feet)	0.1 to 1.0
AH14-1	-	-	-
AH14-2	5.2 to 5.8 metres (17 to 19 feet) and 6.1 to 6.7 metres (20 to 22 feet)	0.6 metre (2 feet)	0.5 to 0.9
AH14-3	-	-	-
AH14-4	-	-	-
AH14-5	2.1 to 2.4 metres (7 to 8 feet) 3.4 to 4.0 metres (11 to 13 feet)	0.6 metre (2 feet)	0.3 to 0.6
AH14-6	1.8 to 5.2 metres (6 to 17 feet)	3.4 metres (11 feet)	0.1 to 0.8

Further details are provided on the table attached in Appendix F. The analyses indicate that the soil profile at the footprint area of the proposed building is considered to be potentially liquefiable in zones ranging from approximately 0.3 to 3.4 metres (1 to 11 feet) thick, and with Factors of Safety ranging from 0.1 to 1.0.

The ground improvement techniques discussed in Sections 10.0 and 12.0, which are recommended in order to support the proposed foundations, will be designed to mitigate the liquefaction hazard beneath the proposed foundations. Therefore, subsequent to implementation of the proposed ground improvement measures, improved subgrade materials under the proposed footings are not expected to liquify during the design seismic event.

12.0 GROUND IMPROVEMENT

As described in Section 10.0, we recommend that foundations for the entire building footprint are supported on conventional strip and pad foundations or a raft foundation, constructed upon improved soil, as the natural loose and compressible subgrade materials are judged to be unsuitable for supporting conventional shallow foundations in their current state, as described in Section 7.5.

We envisage that jet grouting or deep soil mixing may be feasible ground improvement options at the subject site. These ground improvement methods have been successfully implemented at similar projects by local contractors. In general, jet grouting involves drilling into the unsuitable subgrade materials and introducing hydraulic energy in order to erode the soil. Grout is then introduced into the hole under pressure to penetrate the surrounding natural soil and form a solidified "soil-cement" column. Deep soil mixing involves drilling into the unsuitable subgrade materials with large diameter drilling equipment and subsequently introducing grout into the hole (not under pressure), which is mechanically mixed with the disturbed soil to create a soil-cement column that is approximately 1.0 to 1.5 metre (3 to 5 feet) in diameter.

Both jet grouting and deep soil mixing produce columns of improved soil, or "soil-cement", which typically have an unconfined compressive strength of approximately 700 kPa (15,000 psf). Both ground improvement processes would terminate at the surface of compact to dense soil, which we would expect to comprise the silty sand to sandy silt to silt as described in Section 5.2.4 (where compact to very dense), the till-like silty sand to silt and sand described in Section 5.2.5, or the sand to sand and gravel to gravel described in Section 5.2.6. It is noteworthy that neither method involves removing soil from the drillholes (unlike during the subsurface investigation described in Section 5.4.2, where artesian groundwater was observed at the surface); therefore, we envisage that the aquifer-confining effects of the aquitard materials would not be compromised during either process. In the event that till-like soils are not encountered over the aquifer, artesian groundwater is not expected to enter the drillholes for the same reason. However, we envisage that injection of pressurized grout into the soils above the aquifer (depending on grout pressures) may penetrate into lower strata and be an environmentally problematic and/or controversial process; therefore, we envisage that deep soil mixing would be the preferred method of ground improvement at the subject site.

A specialized contractor that is experienced with deep soil mixing should carry out the ground improvement work at the site. For foundation applications, columns are typically installed adjacent and overlapping such that a continuous soil-cement "wall" is constructed for the support of strip footings. Ground improvement is only required at discrete foundation locations; therefore, precise determination of foundation locations would be required prior to commencing ground improvement work. To simplify the ground improvement process, and for the reason described in Section 13.4, it is ideal if foundations are configured in a grid pattern. As there is not envisaged to be a minimum practical depth for the deep soil mixing process, we envisage that this method of ground improvement can be carried out at all portions of the site. Typically, the limitation of deep soil mixing is the density / consistency of the subsurface soil. Based on the in-situ soil testing data collected during the subsurface investigations that show penetration blow counts generally below a value of twenty, we expect that deep soil mixing would be feasible at the subject site.

13.0 BUILDING FOUNDATION DESIGN RECOMMENDATIONS

13.1 Recommended Static Design Bearing Pressure

We envisage that the soil-cement columns resulting from ground improvement works described in Section 12.0 are suitable materials to support the proposed foundations. At this time, we suggest that a bearing pressure of approximately 290 kPa (6,000 psf) be used for Serviceability Limit State (SLS) design for footings constructed upon these materials. This recommendation will be confirmed at a later stage of the project after details pertaining to ground improvement are finalized and after structural engineering details for the proposed building foundations are available.

Any loosened, softened, organic, disturbed, or otherwise deleterious material should be removed prior to footing construction. Foundation subgrades should be protected from freezing. In addition, groundwater and rainwater runoff should be directed to temporary sumps, and footing subgrades should be kept free of standing water.

Horizon Engineering should be provided with an opportunity to review the exposed subgrade prior to footing construction or concrete pouring.

13.2 Recommended Typical Footing Characteristics

It is recommended that foundations be placed at least 0.45 metre (18 inches) below any slab-on-grade and exterior grades for confinement and frost protection. Foundations should step not steeper than 1.0 vertical to 2.0 horizontal.

13.3 Expected Settlement

The total settlement of footings, under static loading and designed in accordance with the above recommendations should be less than 25 mm (1 inch). Differential settlement would be expected to be less than 19 mm over 9.0 metres (3/4 of an inch over a span of 30 feet) or 0.002 radians angular distortion.

13.4 Seismic Considerations

Based on our preliminary liquefaction assessment described in Section 11.0, we envisage that up to 3.4 metres (11 feet) of potentially liquefiable soils are present below the footprint area of the proposed building. Therefore, based on the BC Building Code 2012, the site would be categorized as a Site Class F if no ground improvement is implemented.

We envisage that after implementation of the proposed deep soil mixing ground improvement measures (as discussed in Section 12.0), the potential for liquefaction beneath the proposed building foundations would be eliminated. The layout of the proposed ground improvement measures will be determined at the detailed design stage; however, we envisage that the proposed soil-concrete columns forming the subgrade for the footings will create a grid of underground cut-off walls. This grid is envisaged to provide suitable confinement for any potential lateral spreading of liquefied (unimproved) soils. In addition, we envisage that the stiffness of the proposed soil-cement columns will be suitable to resist lateral pressures from adjacent liquified soil.

After implementation of the proposed ground improvement system and based on the BC Building Code 2012, the subject site is judged to be categorized as Site Class D. The site-specific, peak horizontal ground acceleration for the design magnitude seismic event with a 2% probability of exceedance in 50 years is 0.434g (NBCC 2010). The recommended spectral accelerations are presented following:

Table 3: NBCC 2010 Design Ground Motions for 2% Probability of Exceedance in 50 Years

Sa(0.2) (g)	Sa(0.5) (g)	Sa(1.0) (g)	Sa(2.0) (g)	PGA (g)
0.891	0.624	0.329	0.171	0.434

Based on the above design spectral response accelerations and Tables 4.1.8.4.B and 4.1.8.4.C on Division B - Part 4 of the BC Building Code 2012, values of the acceleration and velocity based site coefficients (Fa and Fv) would be 1.1 and 1.2, respectively.

An ultimate bearing capacity of 2.0 times the Serviceability Limit State design bearing pressure can be used for Ultimate Limit State design requirements for footings.

14.0 GROUNDWATER CONSIDERATIONS

14.1 General

As described in Sections 5.4.1 and 7.2, non-artesian groundwater was encountered during the subsurface investigations at depths of 0.9 to 3.7 metres (3 to 12 feet) at the test pit and auger hole locations. As illustrated on Figures 2 and 3, most of these water levels are significantly higher than the proposed excavation elevation; therefore, groundwater is expected to be daylighted during excavation at the site.

14.2 Temporary Excavation Dewatering

We expect that the sand fill, sand, and silty sand to sandy silt to silt materials that are expected to be encountered during excavation at the northwest portion of the site could be drained during excavation without causing dewatering-related off-site settlement. However, we recommend that the peat materials not be allowed to drain freely, as dewatering-related settlement could be expected to occur in these materials otherwise. Therefore, we recommend that weep holes be implemented in the proposed shoring system discussed in Section 21.3 with the exception of where peat materials are exposed. In these areas, the shoring system could be designed to withstand hydrostatic pressures to avoid drainage of the peat horizon. We envisage that the volume of water expected to be encountered during excavation at the subject site could be managed with conventional drainage measures.

14.3 Permanent Waterproofing Considerations

Due to the naturally high non-artesian water levels expected at the site with an inferred general flow direction of west to east, we recommend that the below-grade portions of the building be designed as a waterproof structure. We envisage that typical foundation and underslab drainage systems would be either omitted or revised such that water ingress into the below-grade portions of the building would be unlikely. We also recommend installation of emergency sump pumps inside the building to dewater the below-grade portions of the building, as required. The top elevation of any foundation walls that support the proposed building should be at or above the FCL (see Section 9.2) and estimated maximum groundwater levels, which are expected to be as shallow as 0.9 metre (3 feet) below existing grades. These walls should be constructed of concrete and have no construction joints or openings lower than the FCL and the maximum groundwater level. Walls and slabs built below these elevations should be waterproofed in accordance the BC Building Code

2012. No unnecessary electrical or mechanical works should be constructed below these elevations. We recommend that covenants be registered with the land title that indemnifies the Town of Gibsons and Horizon Engineering of any potential damages to the proposed development due to construction below the water table and/or the FCL, if applicable. In addition, the requirements of this report for construction and maintenance of the structure must be included in the proposed covenants.

We envisage that the services of a qualified professional must be retained to provide the details of the proposed waterproofing systems.

It is envisaged that an in-ground infiltration system would be designed to disperse intercepted groundwater into the existing, natural, subsurface peat and sand to silty sand materials using pre-case segmental tank modules (such as a Brentwood water tank system, or approved equivalent). This system would be installed at the eastern (i.e., downstream) portion of the site, possibly beneath the eastern portion of the proposed building, which is envisaged to be constructed above existing grades. Based on an estimated groundwater inflow during excavation of approximately 5 to 7 L/minute (1.3 to 1.8 Usgpm) per 1 metre (3 feet) of excavation length, as determined in the seepage analyses discussed in Section 8.3, and an average soil hydraulic conductivity of $5E^{-5}$ m/second ($1.6E^{-4}$ ft/second), we estimate that the proposed infiltration system will be approximately 2 metres (6.5 feet) wide and approximately 10 metres (33 feet) long, with a volume of approximately 10 m^3 (350 ft^3).

15.0 METHANE VENTING SYSTEM

As described in Sections 5.2, 5.3, and 6.8, a horizon of peat was encountered during the subsurface investigations at the majority of the site. If unsaturated organic materials, such as peat, remain beneath a structure, methane that is released during decomposition of organic material can build up beneath the slab-on-grade. Methane is highly flammable when enclosed and concentrated; therefore, a methane venting system is recommended to be constructed where organic soils are present below. As discussed in Section 10.1 and as illustrated on Figures 2 and 3, excavation at the southwest, northeast, and southeast portions of the site is envisaged to be less than approximately 0.5 metres (1.6 feet) deep below adjacent existing grades; therefore, the peat encountered near the surface in these areas is expected to remain beneath the building. We recommend that a methane venting system be installed beneath any portion of the building that is being constructed at or above existing grades where the peat materials were not removed (unless the building is suspended above the ground, which would allow for adequate venting).

A methane collection system would typically be comprised of 0.1 metre (4 inches) diameter, rigid, perforated, PVC pipe that should be installed in the underslab gravel layer with perforations facing upward and spaced 3.0 to 4.5 metres (10 to 15 feet) apart. These pipes should vent upwards to the exterior of the building above the roof elevation. They should be sloped to drain upward and suitably capped to allow ventilation without water ingress.

16.0 LANDSCAPING SIDEWALKS AND SEAWALK

Based on the architectural drawings, we understand that landscaping features, including sidewalks, planters, and a 'seawalk', are proposed as part of the at-grade development at the site, including near the shoreline at the east portion of the site. Subexcavation of settlement-susceptible materials is expected to be challenging in proximity to the shoreline. Subgrade preparation would be expected to include staged removal of peat and any other unsuitable subgrade materials during periods of low tide and replacing them with Engineered Fill. This approach would require special attention to groundwater fluctuations and surface water management. If this approach is judged to be impractical, we recommend that the proposed landscaping sidewalks and seawalk structures be supported by shallow foundations constructed on soil-cement columns following ground improvement as discussed in Section 12.0 or by piles as discussed in Section 17.0. The approximate ground raise for construction of sidewalk is estimated to be 3.3 metres (10.8 feet). The extension of the proposed sidewalk with the raised FCL requires a suitable transition to the neighbouring properties with lower elevations which would be part of the architectural and landscaping detailed design stage.

We understand that a sanitary sewer utility line exists beneath the seawalk and that access to this utility is required to be maintained. We envisage that removable concrete and wood panels could be supported by the aforementioned soil-cement columns or piles. Design details and future maintenance requirements would be provided at the detailed design stage of the project after reviewing the landscape architects' design drawings.

An alternative design concept could be to construct these structures as 'floating' sidewalks supported on a geogrid-reinforced earth slab to minimize potential settlement of the overlying sidewalks. These structures could comprise a geogrid-reinforced earth mat approximately 0.3 metre (12 inches) thick, constructed at a depth of approximately 0.5 metre (18 inches) and with a width of 1.5 times the proposed sidewalk width. Filter fabric should separate the geogrid-reinforced earth mat from the overlying Engineered Fill materials. Future maintenance of landscaping features constructed as 'floating' would be expected to be required.

17.0 FORESHORE DEVELOPMENT

17.1 General

As described in Section 3.1, we understand that the proposed development includes an over-water restaurant building and a marina comprising docks, boat slips, and a fuel dock, all to be located within the existing water lease area east of the subject site's east property line, as shown on Figure 2.

17.2 Proposed Foundation Concept

We recommend that all foreshore development structures be supported by pile foundations, comprised of drilled pipe piles. As described in Section 5.2, the soils encountered at the locations of boreholes BH14-1 and BH14-2 comprised approximately 2.9 to 4.0 (9.5 to 13 feet) of very soft / very loose seabed sediments overlying 1.4 to 3.5 metres (4.5 to 11.5 feet) of sand to silty sand that was inferred to be compact to dense. The Gibsons Aquifer was encountered at depths of 6.4 and 5.3 metres (21.0 and 17.5 feet) below the seabed, respectively. Based on the above, we expect that insufficient resistance may be encountered above the aquifer to provide suitable pile capacity for the proposed structures. The sand and gravel to gravelly sand materials that were inferred to comprise the aquifer were inferred to be compact to dense; therefore, these materials are expected to provide suitable end bearing and/or frictional resistance for the proposed piles.

In order to provide an estimate of the compressive capacity of the proposed piles, we carried out an analysis based on SPT and DCPT blow counts obtained during the subsurface investigations. We envisage that pipe piles should be extended down to the comparatively dense soils below the seabed where sufficient compressive and lateral capacities are expected to be available. At this stage, we do not have information regarding the magnitude of pile foundation loads. We envisage that pipe piles with typical diameters of 300 to 400 millimetres (12 to 16 inches) would be suitable for the proposed structures. The range of compressive service capacities for the piles were calculated to be approximately 330 kN, 445 kN, and 600 kN (75 kips, 100 kips, and 135 kips) for 300 mm, 350 mm, and 400 mm (12 inch, 14 inch, and 16 inch) diameter piles, respectively. Combined compressive and lateral capacities of piles would be determined using a commercially available software program called LPILE with application of a suitable soil / pile constitutive model. This modelling would be carried out at a later stage of the project when more information regarding the type of structures and magnitudes of compressive and lateral loads are available.

It is noteworthy that more than one hundred piles have been installed in the Gibsons Harbour to date, and we envisage that many, if not most of them, likely penetrate into the Gibsons Aquifer. It is also quite likely that the majority of these piles have been driven into the aquifer throughout the past several decades without any consideration for artesian water pressures, aquifer contamination, pile sealing, or monitoring. In Waterline Resources' Aquifer Mapping Study report (referenced in Section 4.0), we note that Section 7.3.5 describes the following:

> *"Recent development work initiated by the Gibsons Harbour Port Authority in the Gibsons waterfront..."* [inferred to be located within approximately 150 metres / 500 feet of the subject marina development] "...*has included driving steel pilings into the seabed and through the Gibsons Aquifer. Waterline was contracted to assist in monitoring of the pile driving program and evaluate the concern with breaching the Vashon Till aquitard and creating a conduit for the known artesian flow from the Gibsons Aquifer. Preliminary results indicate that fresh water leakage was detected both inside and outside the pile. However, no significant adverse impact to the aquitard or Gibsons Aquifer was observed. The Town's engineering department completed the monitoring as additional pilings were driven through the Vashon Till Aquitard and the Gibsons Aquifer, and into the underlying bedrock. Although some indication of fresh water leakage was observed, no major uncontrolled breach of the Gibsons Aquitard was indicated during the pile driving program. Ongoing monitoring is recommended to confirm these initial findings."*

We envisage that installing drilled pipe piles would not result in "leakage" of artesian groundwater from the Gibsons Aquifer around the piles. Based on the discussion above and on the premise that a hydraulic connection appears to exist between the ocean and the Gibsons Aquifer (as discussed in Section 6.3), we envisage that future discussions with the Town of Gibsons and with Waterline Resources may determine that a passive approach to pile driving into the aquifer may be found to be acceptable based on successfully using this approach in the same area. Regardless, a detailed monitoring program should be implemented during pile installation to detect any breach of the aquifer, if it were to occur. In the event of any breach, the piles would be sealed as discussed below.

However, if a passive approach to pile driving into the aquifer is found to be unacceptable, we envisage that piles could be fully sealed, as required, to prevent artesian groundwater from potentially leaking out around the proposed piles, using methods similar to those that were successfully used during the supplementary subsurface investigation when artesian groundwater was encountered.

17.3 Proposed Foreshore Dredging

As discussed in Sections 3.1 and 7.7, we understand that approximately 2.5 to 4.0 metres (8 to 13 feet) of dredging (depth estimated above the proposed dredge cut toe) is proposed to be carried out near the shoreline within the existing water lease area in the harbour. Based on the aforementioned dredging drawings referenced in Section 4.0, we understand that dredging is proposed at the west and north portions of the existing water lease area. The dredging area is understood to commence at elevations of approximately 0 and -0.5 to -1.5 metres (0 and -1.5 to -5.0 feet) at the north and south portions of the dredging area, respectively, extending to an elevation of approximately -5.5 metres (-18 feet), as shown schematically on Figures 2 and 3.

The dredging plans include a section that indicates an excavation slope of 1.0 vertical to 2.0 horizontal, which we envisage would be carried out largely underwater. We envisage that reinforcement of the dredge cutslope with riprap may be required to provide stability to the slope, which should be extended up above the high water level to provide erosion protection. If the dredge cutslope is not reinforced by riprap material, we expect that regular maintenance would be required to remove sloughed material from the toe of the slope. Recommendations for dredge cutslope stability and riprap design will be provided under separate cover at the detailed design stage.

As described in Section 5.2.7, at the south foreshore portion of the site (i.e., south of the proposed over-water restaurant location), approximately 3.2 to 3.3 metres (10 feet 6 inches to 10 feet 10 inches) of very loose to loose / soft to stiff seabed sediments (loose to dense / stiff to hard near the surface) were inferred to be encountered over compact / stiff soils that were inferred to comprise natural silty sand to sandy silt to silt (as described in Section 5.2.4). At the north foreshore portion of the site (i.e., north of the proposed over-water restaurant location), approximately 0.2 to 1.1 metres (8 inches to 3 feet 7 inches) of very loose to loose / soft to firm shallow seabed sediments were inferred to be encountered over at least 1.3 to 1.6 metres (4 feet 3 inches to 5 feet 3 inches) of compact to dense / stiff to hard soils that were inferred to comprise fill materials. Based on the observed subsurface conditions at the north foreshore portion of the site, as well as on the observed steeper beach slope angle in this area, we envisage that fill materials were placed over the underlying seabed sediments during development of Winegarden Park, which is located north of the subject site and immediately upslope of the north portion of the proposed marina area (see Figure 2). Accordingly, we envisage that the very loose to loose / soft to stiff seabed sediments that were observed near the surface at the south foreshore portion of the site are likely to be present beneath the aforementioned fill materials at the north foreshore portion of the site.

As indicated on Figures 2 and 3, the soils that are proposed to be removed during dredging are inferred to generally comprise seabed sediments (and fill materials at the north foreshore area, as described above), though we envisage that the proposed dredging excavation may locally intercept the underlying silty sand to sandy silt to silt materials (inferred to comprise the Gibsons Aquitard, as described in Section 6.3). The seabed sediments were observed at the locations of boreholes BH14-1 and BH14-2 to comprise very soft / very loose materials that are inferred to be not be significantly more dense than water. As a result, we envisage that "blowout" of the underlying aquitard materials following removal of overlying seabed sediments during dredging, as described in Section 7.1, would not be expected to occur, as these materials are not expected to be providing a confining effect to the underlying Gibsons Aquifer. Although a hydraulic connection between the ocean and the underlying aquifer is envisaged to exist in the subject area (as described in Section 6.3), we recommend that dredging be limited to the seabed sediments, as described above, to reduce the risk of impacting the aquifer, as the potential effect of removing a portion of the aquitard materials over a large area is currently unknown.

We recommend that dredging of seabed sediments be carried out by means of hydraulic (i.e., suction) dredging, which is a method that is envisaged to be incapable of removing the natural silty sand to sandy silt to silt (i.e., Gibsons Aquitard) materials that underlie the seabed sediments. Where compact to dense / stiff to hard seabed and/or fill sediments are encountered near the surface, we envisage that mechanical dredging may be required.

18.0 FUEL TANK

Based on information provided by the Client, we understand that a 75,000 litre fuel tank is proposed to be constructed at the southwest portion of the site, which will service the proposed marina. Based on the architectural drawings, we understand that this fuel tank will be approximately 12 metres (40 feet) in length, approximately 4.8 metres (16 feet) in diameter, and founded within the building at an elevation of approximately 5.2 metres (17 feet). We recommend that the tank be supported by soil-cement columns, as previously recommended for the building foundation in Section 12.0.

19.0 ENGINEERED FILL

Within the context of this report, Engineered Fill should consist of select, clean, well-graded granular material with less than 5% fines content and 100% passing a 150 mm sieve. Engineered Fill should extend beyond the footprints of foundations, where applicable, a distance equal to its thickness. We recommend a design Serviceability Limit States bearing capacity of 100 kPa (2,000 psf) for footings constructed upon Engineered Fill materials, if applicable. Engineered Fill should be placed in suitable lifts (generally 0.3 metre / 1 foot loose thickness or less) and compacted to the equivalent of at least 100% of its Maximum Dry Density determined in accordance with ASTM D698 (Standard Proctor). Field density testing should be carried out to ensure the compaction criteria are achieved and these test results should be forwarded to Horizon for review. In addition, Horizon Engineering should be given the opportunity to review the Engineered Fill material type and placement and compaction procedures.

We consider the sand fill materials described in Section 5.2.1 and the sand materials described in Section 5.2.3 to be suitable for re-use as Engineered Fill provided that cobbles and boulders larger than 0.15 metre (6 inches) in diameter and all organic and debris materials are removed prior to compaction. The material should be stockpiled/treated such that it has a moisture content within 2% of optimum at the time of final placement.

The remaining materials that we envisage would be excavated as part of the proposed development generally consisted of silty or organic soils. Materials such as these are typically not recommended for re-use as Engineered Fill due to the potential difficulty of placement and achieving suitable compaction. Fine-grained soil may be suitable for landscaping purposes and where support of settlement-sensitive structures is not required; however, we envisage that the peat materials would not be suitable for landscaping applications unless significant settlement is tolerable. It should be noted that fine-grained soil may be moisture sensitive and susceptible to water softening; therefore, this soil should only be placed under dry weather and site conditions.

20.0 LATERAL EARTH PRESSURES AND BACKFILL MATERIALS

20.1 General

The earth pressure on basement and retaining walls depends on a number of factors including the backfill material, surcharge loads, backfill slope, drainage, rigidity of the foundation wall, and method of construction, including sequence and degree of compaction. As discussed in Section 14.3, we recommend that the below grade portions of the building be designed as a waterproof structure; therefore, backfill materials behind these walls need not necessarily be free draining. However, Engineered Fill materials are recommended for backfilling purposes to minimize surficial settlement. If the retained soil is sloping, the design slope geometry should be provided to Horizon Engineering in order to revise the following lateral earth pressure recommendations, which assume that the retained slope is horizontal. Since it is recommended that the foundation walls be waterproof, hydrostatic pressures should be added to these lateral earth pressures.

20.2 Static Design

For foundation and retaining walls that will be backfilled with compacted granular material such as sand and gravel and which can move 0.2% of the wall height, or about 0.25 inch between floor slabs, then locally, the condition is presumed to be unrestrained and it is recommended that the wall be designed to resist a **35 x h** (psf) [**5.6 x h** kPa] triangular earth pressure distribution where **h** is the distance from the top of the wall measured in feet (metres).

Where the backfill material will be required to support settlement sensitive structures, compaction to greater than 100% of its Standard Proctor Maximum Dry Density would be required. In this circumstance, a compaction earth pressure of 400 psf [19 kPa] uniform pressure distribution should be used in the top approximate 4.1 metres (13.5 feet). Elsewhere, backfill should be compacted to at least 95% of its Standard Proctor Maximum Dry Density.

20.3 Seismic Design

For seismic loading conditions, the effect of earthquake shaking can be assumed to add an additional triangular pressure to the top of the wall, which decreases to zero at the base of the wall. Based on the Mononobe-Okabe method (Mononobe and Matsuo, 1929; Okabe, 1924) the seismic surcharge pressure can be assumed to be **25 x (H-h)** (psf) [**3.9 x (H-h)** kPa] where **h** is the distance from the top of the wall and **H** is the total wall height measured in feet (metres).

Although the Mononobe-Okabe method is suggested in the 4th edition of the Canadian Foundation Engineering Manual (2006), the equations do not account for the stiffness of the structure, nor the soil-structure interaction. If a more accurate determination of seismic earth pressure is required, more rigorous analytical methods such as finite element analysis to account for soil-structure interaction should be carried out. We would be pleased to provide additional information regarding this type of engineering service if requested.

Seismic earth pressures are not added to the compaction earth pressure; therefore, only the greater of either the seismic or compaction earth pressures would be recommended at the corresponding depth.

20.4 Surcharge Loading Due to Vehicles

Surcharge loads from adjacent streets and parking areas can be assumed to be equivalent to an additional 0.6 metre (2 feet) of soil supported against the wall.

21.0 SITE PREPARATION AND TEMPORARY EXCAVATION

21.1 General

It is judged that the materials encountered during the site investigation would be readily excavated using conventional hydraulic excavation equipment in good repair. In the event that large boulders are encountered that require splitting for removal, boulders which cannot be ripped and have a volume in excess of 1m^3 should be defined as "rock" for contractual purposes. Volumes should be quantified in-situ (i.e., before being split) by the architect, owner, or Horizon Engineering.

21.2 Temporary Excavation

As described in Section 10.1, we understand that the proposed excavation would be approximately 5.1 to 5.8 metres (17 to 19 feet) deep below adjacent existing grades at the northwest portion of the site. Excavation at the southwest, northeast, and southeast portions of the site is envisaged to be less than approximately 0.5 metres (1 foot 8 inches) deep below adjacent existing grades. As discussed in Sections 8.0 and 10.2, and as shown schematically on the attached sections on Figures 2 and 3, we recommend that the proposed excavation not advance below 5.0 metres (16 feet 5 inches) geodetic elevation at the northwest portion of the site and 0.5 metre (1 foot 8 inches) below existing grades at the balance of the site in order to ensure that the Gibsons Aquifer is not compromised due to excavation of the overlying materials. We recommended that a British Columbia Land Surveyor be engaged in order to provide a reference point during construction and to assist the contractor and Horizon's representative in order to prevent the excavation from extending below the recommended elevations.

In general, it is recommended that unshored excavation slopes be maintained with an inclination no steeper than 1 vertical to 1.5 horizontal in the fill, peat, sand, and silty sand to sandy silt to silt materials. Unshored excavation slopes should be protected with a layer of 6 mil polyethylene sheeting, secured in place to resist wind action and maintained regularly. Grade adjacent to the excavation should be sloped to direct surface runoff away from the excavation slopes, and the excavation area should be kept free from standing water by installing a temporary sump pump. In addition, it is recommended that excavated spoil and construction materials be stockpiled no closer than 1.5 metre (5.0 feet) to the crest of the excavation slopes and covered with 6 mil polyethylene sheeting. Any signs of instability such as tension cracks, excessive sloughing, or ground movements should be reported to Horizon Engineering immediately.

As bulk excavation approaches the final excavation elevations (5.0 metres / 16 feet 5 inches geodetic at the northwest portion of the site and 0.5 metre (1 foot 8 inches) below existing grades at the balance of the site), we recommend that regular surveying by a British Columbia Land

Surveyor be carried out within the excavation to confirm that these lowest recommended excavation elevations are not exceeded. Survey data should be collected in a grid pattern with adjacent points not more than approximately 3 metres (10 feet) apart, and with bulk excavation stages not more than approximately 0.5 metre (1 foot 8 inches) in thickness below an elevation of approximately 3 metres (10 feet). A survey should be carried out following demolition of all structures at the site such that a grid of lowest proposed excavation elevations at the northeast, southeast, and southwest portions of the site can be established prior to commencement of stripping and bulk excavation. Survey data should be forwarded to Horizon Engineering in a timely fashion for review.

In general, we recommend that the excavation be carried out in stages (in plan view). At each stage, the ground surface should be surveyed and monitored such that any potential signs of heaving and upward groundwater seepage are detected, respectively. In the unlikely event that ground heaving and/or upward groundwater seepage are observed, the affected area would be limited with a staged excavation and remediation would be manageable. Potential remediation works may include temporary backfilling until further recommendations can be provided. It is noteworthy that based on the available information and the computer modelling described in Section 8.0, we envisage there to be no risk of ground heaving or upward groundwater seepage into the excavation if our recommendations in this report are implemented.

21.3 Temporary Excavation Support

Based on the aforementioned drawings and the proximity of the proposed underground structures to the property lines at the northwest portion of the site, we envisage that there will be insufficient room for sloping on the northwest, north, and south sides of the proposed excavation at the northwest portion of the site. Therefore, temporary excavation support is envisaged to be required in these areas. It is envisaged that excavation support using tied-back shotcrete shoring will be suitable, fitted with weep holes as described in Section 14.2 to allow drainage of the retained materials where recommended. This system requires that the owner obtain permission to encroach with anchors onto adjacent properties. If such permission is not obtained, an alternate shoring system would be required. Typically, such non-encroaching shoring systems would be expected to negatively impact shoring, formwork, and concrete contract costs and schedules as well as potentially compromise the building envelope, which may not be feasible in conjunction with a waterproof foundation wall.

A commercially available limit equilibrium slope stability analysis program (GeoStudio 2007, Version 7.23) was used to carry out a preliminary slope stability analysis for the temporary shoring system proposed at the west property line at the northwest portion of the site, under static ground conditions. A Morgenstern-Price method of analysis was used to search for the most critical potential circular failure surfaces that could influence the proposed excavation slope. Section A, as shown on Figure 2, was selected to represent the subject excavation slope, and soil strengths were selected as presented in Section 8.2. Slope stability analyses indicated that the proposed tied-back shotcrete shoring system would be stable under static conditions, with a Factor of Safety of approximately 2.1, which is judged to be suitable. Slope stability analysis results are presented in Appendix E. Detailed shoring design analyses will be carried out, and excavation shoring drawings, including sediment and erosion control requirements if required, will be prepared upon receipt of detailed design drawings for the proposed development at a later stage of the project.

Preconstruction assessments of adjacent structures should be carried out prior to commencement of excavation. If single-sided forms will be used for below-grade foundation walls, post-grouting of anchors will be required. Removal of shotcrete and anchors near street grades may be required on Town of Gibsons property.

21.4 Ground Improvement

We recommend that ground improvement works commence following bulk excavation and shoring installation. If ground improvement works are to be carried out below the high water mark (i.e., for the seawalk, as discussed in Section 16.0), we envisage that a level construction pad or platform may be required from which to operate the machinery, and scheduling complications may arise due to tidal fluctuations. The environmental engineer should be consulted with regard to carrying out deep soil mixing in the foreshore area, as we envisage that there may be restrictions with regard to working and utilizing soil additives such as grout in proximity to or within the marine environment. Typically, grout pressures utilized during deep soil mixing are not high and are not envisaged to be capable of penetrating beyond the diameter of each column (i.e., 1.0 to 1.5 metres / 3 to 5 feet). Therefore, we do not expect the proposed ground improvement system to adversely impact groundwater or the aquifer during installation.

22.0 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical viewpoint, the subject site is considered to be suitable for development of the type proposed, and the Gibsons Aquifer is envisaged to not be negatively impacted by the proposed development provided that the recommendations in this report are incorporated into the design and construction. The conclusions and recommendations presented in the previous sections of this report are summarized below:

- The Town of Gibsons and the subject site are underlain by the Gibsons Aquifer, which is a confined aquifer comprising sand and gravel that provides drinking water for the town. The confining Gibsons Aquitard is inferred to comprise variable thicknesses of sand, peat, silty sand to sandy silt to silt, and localized till-like materials within the subject site.

- Artesian groundwater pressures have been observed within the Gibsons Aquifer. Hydraulic connections have been observed between the Gibsons Aquifer and the ocean at the central portion of the site and between the Gibsons Aquifer and Town Well #1 at the west portion of the site.

- A computer model was generated to analyse the site and subsurface conditions during and after construction of the proposed development based on existing information, published literature, and engineering judgement. The results of this modelling work indicate that the proposed excavation should not advance below a geodetic elevation of 5.0 metres (16 feet 5 inches) at the northwest portion of the site in order to ensure that the underlying Gibsons Aquifer is not compromised (even temporarily) due to excavation of the overlying materials. The results of these analyses are based on conservative soil strength properties. Therefore, there is an inherent Factor of Safety (which may be of the order of approximately two) in the deformation analysis results.

- At the southwest, southeast, and northeast portions of the site, we recommend that the proposed excavation not advance below 0.5 metre (1 foot 8 inches) below existing grades in order to ensure that the Gibsons Aquifer is not compromised due to excavation of the overlying materials. Deeper excavation at the southwest portion of the site is not recommended due to the proximity of the Gibsons Aquifer to the existing site grades.

- All habitable spaces are recommended to be constructed at or above a Flood Construction Level (FCL) of approximately 5.33 metres (17 feet 6 inches), which takes into account potential effects of sea level rise and storm and tsunami waves during the design life of the proposed building. We envisage that habitable spaces could be constructed below the FCL if a sea dike is constructed around the building, which would be designed to protect the building from rising sea levels and future storm events. The proposed marina and over-water restaurant should be constructed at or above the FCL since they would otherwise be unprotected from the design flood conditions.
- We envisage that the lowest proposed top of slab elevations would be approximately 6.3 metres (20.5 feet) at the west portion of the site and 3.1 metres (10.2 feet) at the east portion of the site. Accordingly, we envisage that the proposed footing elevations would be approximately 5.4 metres (17.5 feet) at the west portion of the site and 2.2 metres (7.2 feet) at the east portion of the site. Therefore, we envisage that the proposed excavation would be approximately 5.1 to 5.8 metres (17 to 19 feet) deep below adjacent existing grades at the northwest portion of the site. Excavation at the southwest, northeast, and southeast portions of the site is envisaged to be less than approximately 0.5 metres (1.6 feet) deep below adjacent existing grades.

- We envisage that the proposed finished floor elevation for the proposed café, retail space, meeting room, seawalk, and over-water restaurant at the east portion of the site would be approximately 5.3 metres (17.3 feet), which is consistent with the recommended FCL. The currently proposed lowest parkade floor elevation is below the FCL; therefore, a sea dike is envisaged to be required as part of the proposed development.

- We recommend that foundations for the entire building footprint are supported on conventional strip and pad foundations or on a raft foundation. Due to the presence of loose and compressible subgrade materials (which are judged to be unsuitable for supporting shallow foundations in their current state), ground improvement is recommended beneath proposed foundations such that suitable bearing is achieved.

- We recommend that footings proposed at the east portion of the site, where excavation is not required, are lowered to the existing grades after the proposed ground improvement measures are complete. In these areas, floor slabs are recommended to be designed as suspended slabs.

- The soil profile at the footprint area of the proposed building is considered to be locally potentially liquefiable; however, we envisage that after implementation of the proposed deep soil mixing ground improvement measures, the potential for liquefaction beneath the proposed building foundations would be eliminated.

- We envisage that deep soil mixing may be the preferred method of ground improvement at the subject site.

- Non-artesian groundwater is expected to be daylighted during excavation at the site, which we envisage would be managed with conventional drainage measures.

- Due to the naturally high non-artesian water levels expected at the site, we recommend that the below-grade portions of the building be designed as a waterproof structure. It is envisaged that an in-ground infiltration system would be installed at the eastern portion of the site to disperse intercepted groundwater into the existing, natural, subsurface peat and sand to silty sand materials .

- A methane venting system is recommended to be constructed beneath any portion of the building that is being constructed at or above existing grades where unsaturated organic materials, such as peat, remain below.

- If subexcavation of settlement-susceptible materials in proximity to the shoreline is judged to be impractical for subgrade preparation at the proposed landscaping sidewalk and seawalk footprint areas, we recommend that these structures be supported by shallow foundations constructed on soil-cement columns following ground improvement or by piles. Alternatively, these structures could be designed as 'floating' sidewalks supported on a geogrid-reinforced earth slab.

- We recommend that all foreshore development structures, including the over-water restaurant building, docks, and boat slips, be supported by drilled pipe pile foundations. We expect that insufficient resistance may be encountered above the Gibsons Aquifer to provide suitable pile capacity for the proposed structures; however, the materials that were inferred to comprise the aquifer are expected to provide suitable end bearing and/or frictional resistance for the proposed piles. We envisage that installing drilled pipe piles would not result in "leakage" of artesian groundwater from the aquifer around the piles; however, a detailed monitoring program should be implemented during pile installation to detect any breach of the aquifer, if it were to occur. If a passive approach to pile driving into the aquifer is found to be unacceptable, we envisage that piles could be fully sealed, as required, to prevent artesian groundwater from potentially leaking out around the proposed piles.

- We understand that dredging is proposed to be carried out near the shoreline within the west and north portions of the existing water lease area in the harbour. The soils that are proposed to be removed during dredging are inferred to generally comprise seabed sediments (and fill materials at the north foreshore area), though we envisage that the proposed dredging excavation may locally intercept the underlying Gibsons Aquitard materials. The seabed sediments were observed to comprise materials that are inferred to be not be significantly more dense than water; therefore, we envisage that "blowout" of the underlying aquitard materials following removal of overlying seabed sediments during dredging would not be expected to occur. Although a hydraulic connection between the ocean and the underlying aquifer is envisaged to exist in the subject area, we recommend that dredging be limited to the seabed sediments to reduce the risk of impacting the aquifer. We recommend that dredging of seabed sediments be carried out by means of hydraulic dredging; where denser / harder seabed and/or fill sediments are encountered near the surface, we envisage that mechanical dredging may be required.

- We understand that a 75,000 litre fuel tank is proposed to be constructed at the southwest portion of the site, which will service the proposed marina. We recommend that the tank be supported by soil-cement columns, as previously recommended for the building foundation.

- As bulk excavation approaches the final excavation elevations, we recommend that regular surveying by a British Columbia Land Surveyor be carried out within the excavation to confirm that the lowest recommended excavation elevations are not exceeded. We recommend that the excavation be carried out in stages (in plan view). At each stage, the ground surface should be surveyed and monitored such that any potential signs of heaving and upward groundwater seepage are detected, respectively. Based on the available information and the computer modelling described in this report, we envisage there to be no risk of ground heaving or upward groundwater seepage into the excavation if our recommendations in this report are implemented. It should be noted that after completion of excavation to the design elevations and confirming that there is no impact on the aquifer, subsequent construction activities can proceed; specifically, ground improvement and construction of the foundation.

- We envisage that there will be insufficient room for sloping on the northwest, north, and south sides of the proposed excavation at the northwest portion of the site. It is envisaged that temporary excavation support using tied-back shotcrete shoring will be suitable. A preliminary slope stability analysis was carried out on the temporary shoring system proposed at the west property line in this area, which indicated that this system would be suitably stable under static conditions.

23.0 REVIEW

In accordance with the 2012 edition of the BC Building Code's, *Letters of Assurance* program, the Geotechnical Engineer of Record is obligated to carry out field reviews. For this project, geotechnical field reviews should be completed for the following items or during the following stages of construction:

<u>Geotechnical - Temporary</u>
7.1 Excavation
7.2 Shoring

<u>Geotechnical - Permanent</u>
8.1 Bearing capacity of the soil (including ground improvement)
8.2 Geotechnical aspects of deep foundations
8.3 Compaction of engineered fill
8.4 Structural considerations of soil, including slope stability and seismic loading
8.5 Backfill

Thus, the Geotechnical Engineer of Record must be given the opportunity to review temporary excavation, shoring installation, ground improvement, and pile installation, confirm the foundation subgrades, and review the suitability, placement, and compaction of Engineered Fill and backfill.

It is the responsibility of the Client to ensure that Horizon Engineering is contacted to carry out the aforementioned field reviews during construction. The BC Building Code Schedule C-B Letters of Assurance cannot be completed without having carried out the required field reviews.

24.0 CLOSURE

This report has been prepared for the sole use of Klaus Fuerniss Enterprises Inc., Art Phillips & Associates Inc., and other consultants for this project. Any use or reproduction of this report for other than the stated intended purpose is prohibited without the written permission of Horizon Engineering Inc.

We are pleased to be of assistance to you on this project and we trust that our comments and recommendations are both helpful and sufficient for your current purposes. If you would like further details or require clarification of the above, please do not hesitate to contact us.

For
HORIZON ENGINEERING INC

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA K. KARIMZADEGAN #27156

April 2015

Karim Karimzadegan, M.A.Sc., P.Eng.
Principal

For
HORIZON ENGINEERING INC

Pamela Bayntun, P.Eng.
Project Engineer

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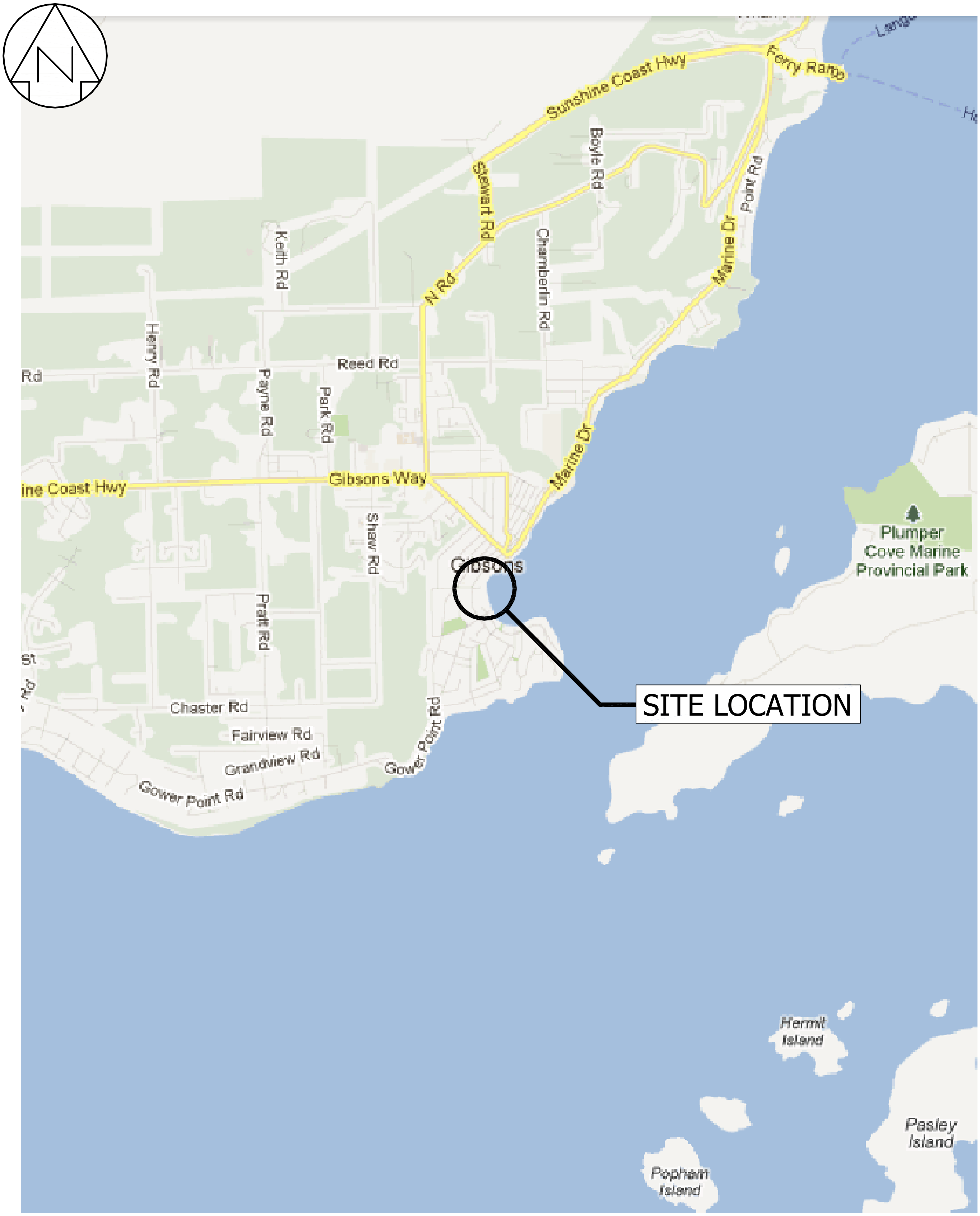
APPENDIX A

FIGURES



Klaus Fuerniss Enterprises Inc.
c/o Art Phillips & Associates Inc.

PROPOSED "THE GEORGE" DEVELOPMENT
377, 385 & 407 Gower Point, 397 & 689 Winn Road, & Winn Road Right-Of-Way, Gibsons, BC

SITE LOCATION PLAN

HORIZON ENGINEERING INC

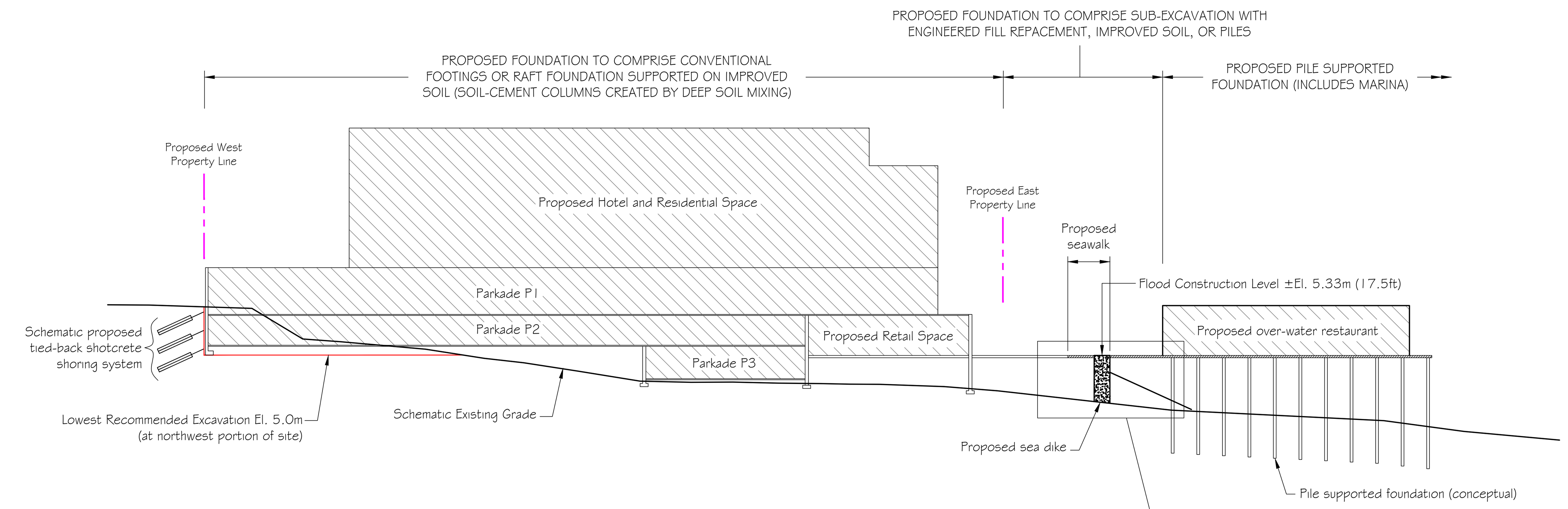
Scale: NTS	Job No: 112-3155	Date: JUN/2014	FIGURE: 1
Des: PB	Dwn: CD	Chk: KK	Rev:

4-Jun-14, 11:57:05 AM, Bronson, N:\2012 Projects\112-3155 GIB 385 Gower Point Rd _The George\Drawings\112-3155 Site Location Plan.dwg

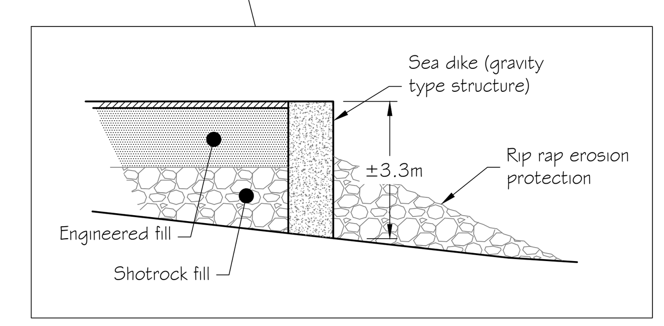
LEGEND:
- Water Level
- FILL / SEABED SEDIMENTS
- PEAT
- SAND
- SILTY SAND TO SANDY SILT TO SILT
- TILL-LIKE SILTY SAND TO SILT AND SAND
- GRAVELLY SAND TO SAND AND GRAVEL TO GRAVEL

INFERRED GIBSONS AQUITARD
INFERRED GIBSONS AQUIFER

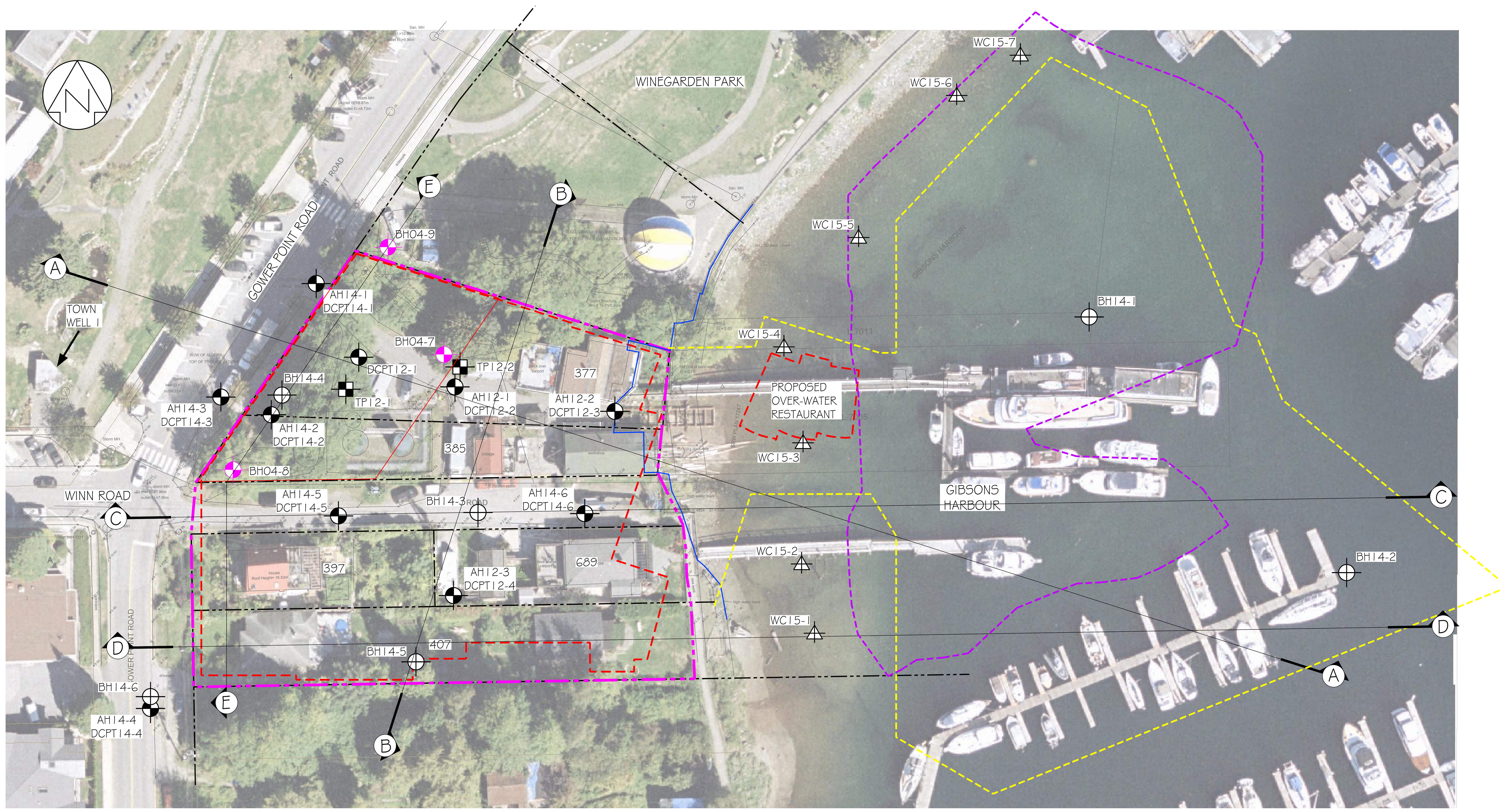
SECTION A
SCALE: 1:300



SCHEMATIC SECTION
SCALE: NTS



SCHEMATIC SEA DIKE DETAIL



TEST HOLE LOCATION PLAN
SCALE: 600

LEGEND:
- Section
- Approximate site boundary
- Approximate high water mark
- Current property lines
- Approximate location of test pit
- Approximate location of auger hole and/or Dynamic Cone Penetration Test
- Approximate location of Geo Tac Tics borehole
- Approximate location of sonic borehole
- Approximate location of Wildcat Cone Penetration Test
- Approximate location of proposed building
- Existing water lease & proposed marina area
- Approximate proposed limit of dredging
- Approximate limit of lowest recommended excavation (El. 5.0) (at northwest portion of site)

REFERENCE DRAWINGS:
Larry W. Penonzek BC Land Surveyor; Topographic Survey, dated: Jan 28, 2015
Balanced Environmental; Dredging Plan, received May 14, 2014
Sunshine Coast Regional District; 2009 Orthophotos

NO	BY	REVISION	DATE
5			
4			
3			
2			
1	KK	Issued for Development Permit	7 April, 2015

HORIZON ENGINEERING INC
114 - 2433 Dollarton Highway
North Vancouver, BC, V7H 0A1
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PROPOSED "THE GEORGE" DEVELOPMENT
377, 385, 407, 409 Gower Point Rd.
397 & 689 Winn Rd. & Winn Rd. Right-of-Way
Gibsons, BC

TEST HOLE LOCATION PLAN AND SECTIONS

SCALE	DATE
AS SHOWN	JAN/2015
DRAWN: BB/CD	CHECKED: KK
DESIGN: PB	ISSUED:
FIGURE: 2	
JOB NUMBER: 112-3155	

SECTION C
SCALE: 1:300

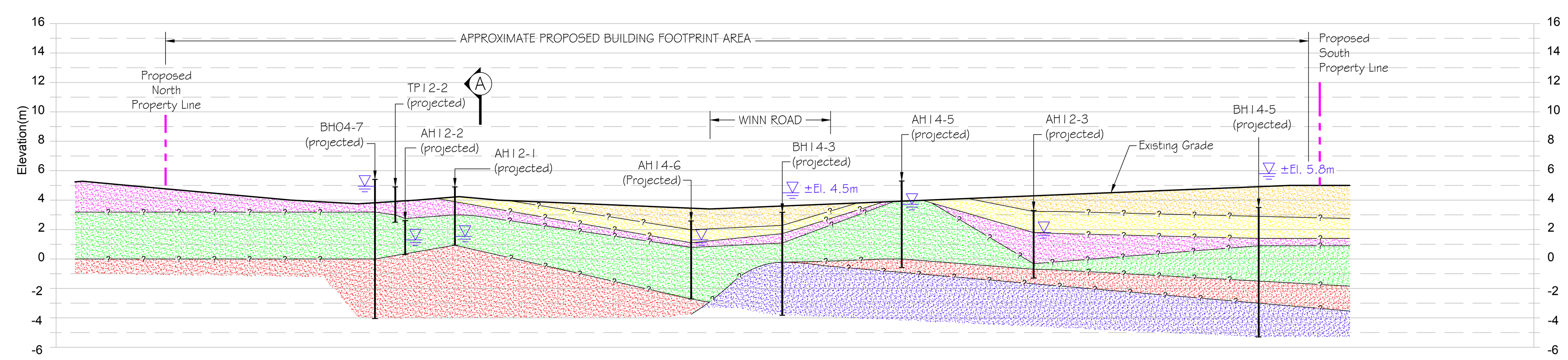
LEGEND:

- Water Level
- FILL / SEABED SEDIMENTS
- PEAT
- SAND
- SILTY SAND TO SANDY SILT TO SILT
- TILL-LIKE SILTY SAND TO SILT AND SAND
- GRAVELLY SAND TO SAND AND GRAVEL TO GRAVEL

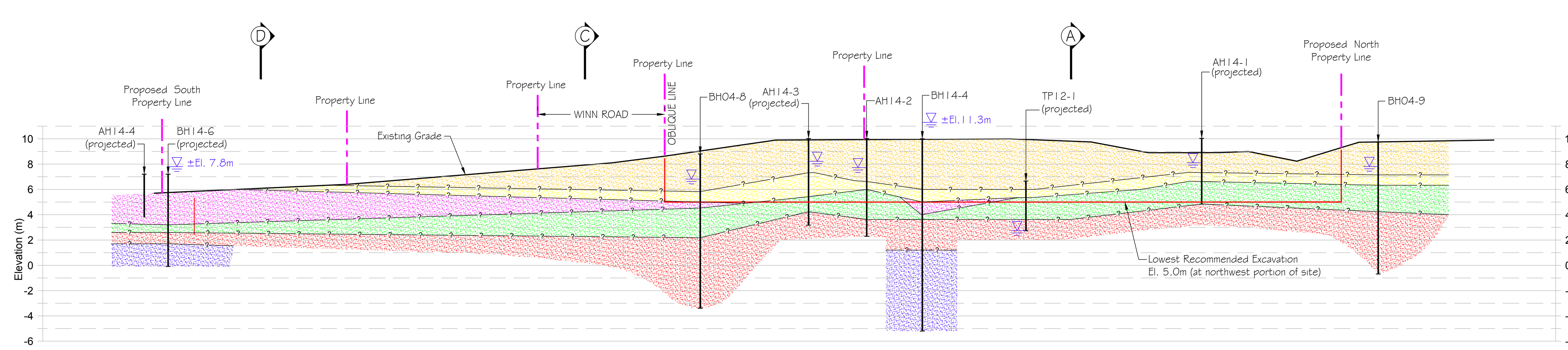
INFERRED GIBSONS AQUITARD (PEAT; SAND; SILTY SAND TO SANDY SILT TO SILT; TILL-LIKE SILTY SAND TO SILT AND SAND)

INFERRED GIBSONS AQUIFER (GRAVELLY SAND TO SAND AND GRAVEL TO GRAVEL)

SECTION D
SCALE: 1:300



SECTION B
SCALE: 1:300



SECTION E
SCALE: 1:300

NO	BY	REVISION	DATE
5			
4			
3			
2			
1	KK	Issued for Development Permit	7 April, 2015

HORIZON ENGINEERING INC
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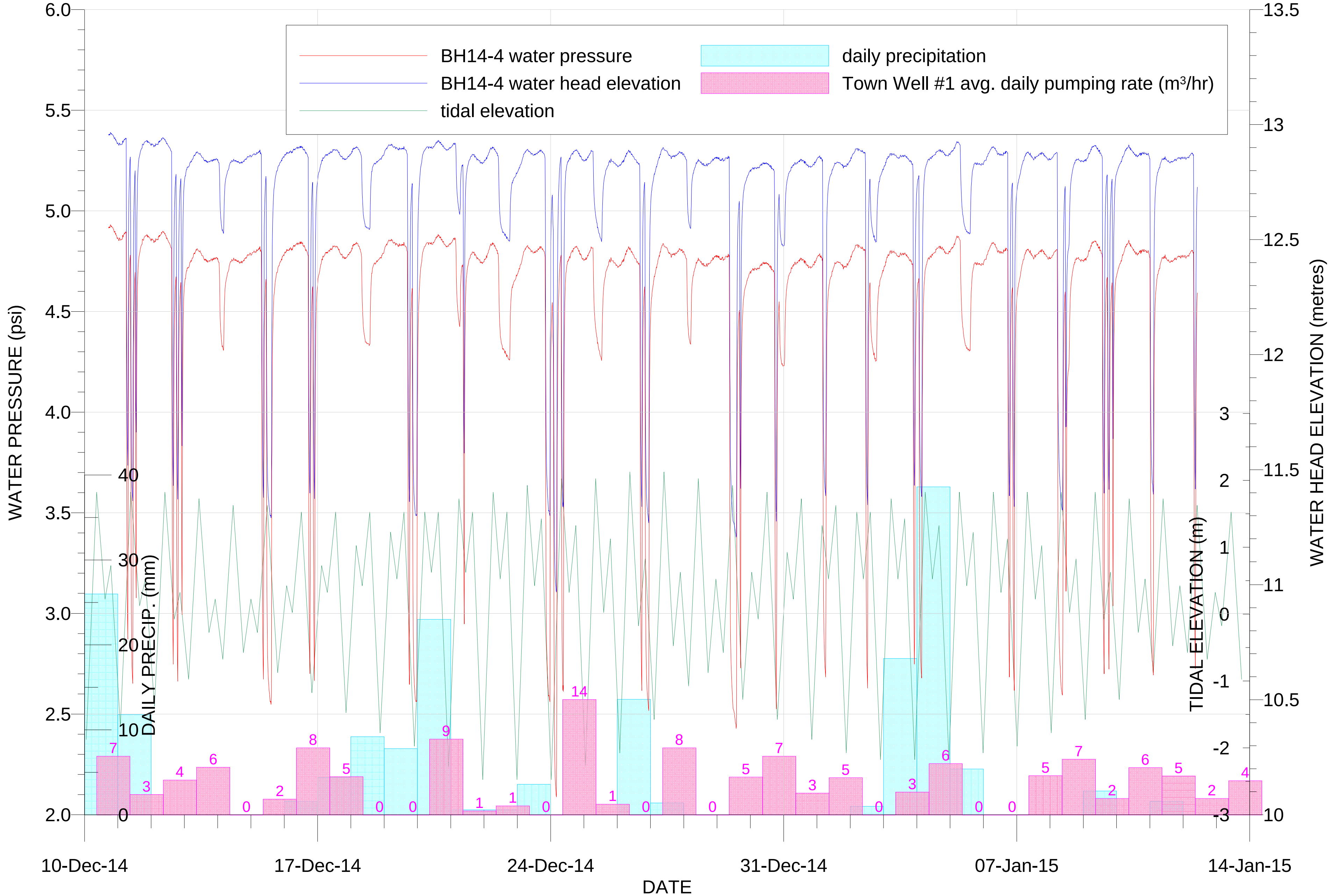
PROPOSED "THE GEORGE" DEVELOPMENT
377, 385, 407, 409 Gower Point Rd.
397 & 689 Winn Rd. & Winn Rd. Right-of-Way
Gibsons, BC

SECTIONS B TO E

SCALE	DATE
AS SHOWN	JAN/2015
DRAWN: BB/CD	CHECKED: KK
DESIGN: PB	ISSUED:
FIGURE: 3	
JOB NUMBER: 112-3155	



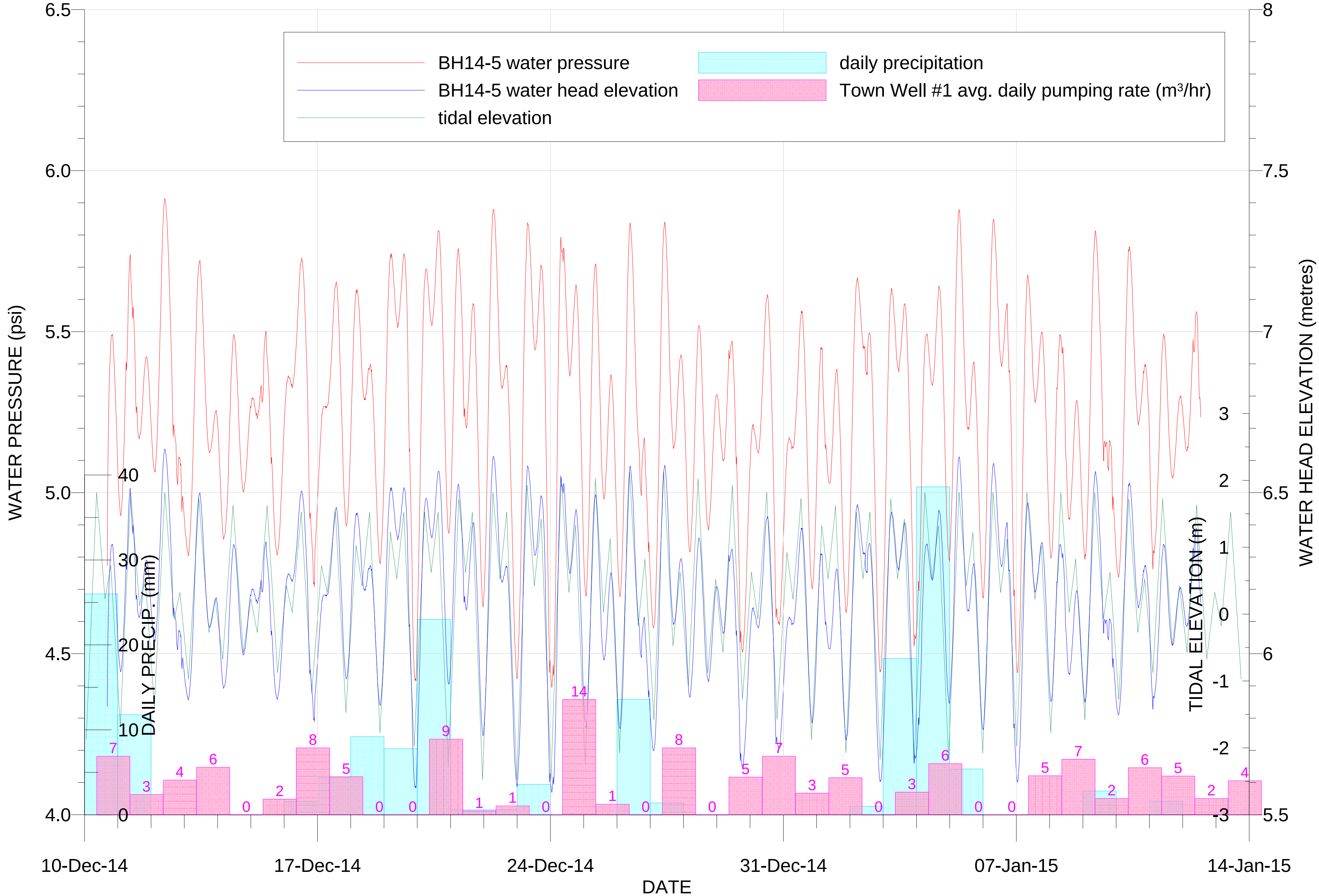
HORIZON ENGINEERING INC

KLAUS FUERNISS ENTERPRISES INC.
PROPOSED "THE GEORGE" DEVELOPMENT
WATER LEVEL MONITORING PROGRAM

BH14-4 WATER LEVEL MONITORING DATA

Scale:	File No:	Date:	FIGURE:
NTS	112-3155	JAN/15	4

Des:	Dwn:	Chk:	Rev:
PB	PB	KK	



HORIZON ENGINEERING INC

KLAUS FUERNISS ENTERPRISES INC.
PROPOSED "THE GEORGE" DEVELOPMENT
WATER LEVEL MONITORING PROGRAM

BH14-5 WATER LEVEL MONITORING DATA

Scale:	File No:	Date:	FIGURE:
NTS	112-3155	JAN/15	5

Des:	Dwn:	Chk:	Rev:
PB	PB	KK	

HORIZON ENGINEERING INC	KLAUS FUERNISS ENTERPRISES INC. PROPOSED "THE GEORGE" DEVELOPMENT WATER LEVEL MONITORING PROGRAM	BH14-6 WATER LEVEL MONITORING DATA	Scale: NTS; File No: 112-3155; Date: JAN/15; Des: PB; Dwn: PB; Chk: KK; Rev:; FIGURE: 6

APPENDIX B

TEST HOLE DATA

TEST HOLE DATA

TEST HOLE	COLLAR LOCATION [1] (metres)		COLLAR ELEVATION [2]		TOTAL DEPTH		DEPTH TO PERCHED GROUNDWATER		STATIC ARTESIAN GROUNDWATER HEIGHT ABOVE GROUND/SEABED		STATIC ARTESIAN GROUNDWATER ELEVATION	
	Northing	Easting	feet	metres	feet	metres	feet	metres	feet	metres	feet	metres
AH12-1	5472033.94	463134.23	16.1	4.92	14.0	4.3	9.0	2.7	-	-	-	-
AH12-2	5472027.36	463169.03	8.9	2.71	8.0	2.4	5.0	1.5	-	-	-	-
AH12-3	5471990.96	463136.35	10.3	3.14	15.0	4.6	5.0	1.5	-	-	-	-
AH14-1	5472049.76	463111.66	32.2	9.83	17.0	5.2	7.5	2.3	-	-	-	-
AH14-2	5472026.70	463101.24	33.2	10.11	25.0	7.6	7.5	2.3	-	-	-	-
AH14-3	5472030.19	463091.29	33.4	10.17	22.0	6.7	6.0	1.8	-	-	-	-
AH14-4	5471969.45	463077.46	23.5	7.15	11.0	3.4	3.0	0.9	-	-	-	-
AH14-5	5472006.74	463114.49	17.4	5.32	15.0	4.6	3.5	1.1	-	-	-	-
AH14-6	5472007.82	463160.28	8.3	2.54	19.0	5.8	3.0	0.9	-	-	-	-
BH14-1	5472053	463265	-8.2	-2.5	21.5	6.6	-	-	1.6	0.5	-6.6	-2.0
BH14-2	5472002	463316	-23.0	-7.0	18.0	5.5	-	-	33.5	10.2	10.5	3.2
BH14-3	5472007.18	463141.25	10.4	3.18	24.0	7.3	-	-	4.3	1.3	14.7	4.5
BH14-4	5472030.58	463103.34	33.1	10.09	50.0	15.2	-	-	3.9	1.2	37.0	11.3
BH14-5	5471977.95	463129.78	10.0	3.04	29.0	8.8	-	-	9.0	2.7	19.0	5.8
BH14-6	5471970.22	463077.29	23.7	7.22	24.0	7.3	-	-	1.9	0.6	25.6	7.8
TP12-1	5472033.85	463119.56	20.4	6.22	13.0	4.0	12.0	3.7	-	-	-	-
TP12-2	5472036.54	463141.43	14.8	4.52	8.0	2.4	-	-	-	-	-	-
BH04-7			18.0	5.5	30.0	9.1	2.0	0.6	-	-	-	-
BH04-8			29.0	8.8	40.0	12.2	6.6	2.0	-	-	-	-
BH04-9			32.0	9.8	35.0	10.7	6.9	2.1	-	-	-	-
TW-1			44.3	13.5	138.0	42.1	-	-	4.6	1.4	48.9	14.9

N:\2012 Projects\112-3155 GIB 385 Gower Point Road - The George\drilling programs\[drillhole data.xlsx]test hole data rams\[drillhole data.xlsx]test hole data

NOTES:

1. All collar locations are based on site survey plan dated Jan. 28/15 except BH14-1 and BH14-2, which are based on GPS survey data.
2. All collar elevations are based on site survey plan dated Jan. 28/15 except BH14-1 and BH14-2, which are estimated based on bathymetric survey plan.

Auger Hole LOG

Auger Hole No.: AH12-1

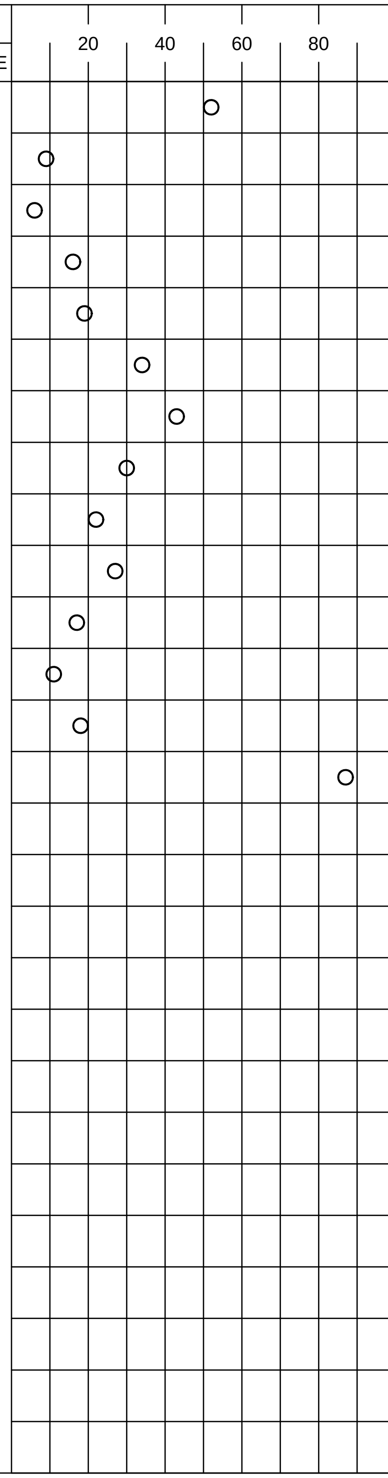
LOGGED BY: PB ON: 24 / 09 / 2012 REVIEWED BY: KK COLLAR ELEVATION: 4.92m METHOD: truck mounted auger drill rig

Type of Test	TYPE -- Type of sample
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon
● Becker Denseness Test (BDT)	S -- Shelby tube
▲ Number of blows - Standard Penetration (SPT)	G -- Grab
■ Moisture Content (% of dry weight)	O -- Other (specify)
> Plastic limit	
< Liquid limit	∇ Ground water level

Notes: N 5472033.94, E 463134.23
backfilled with bentonite and hole
spoil

Depth (ft)	DESCRIPTION	Depth
0	**ASPHALT**	0.2
0.2	**FILL - SAND** (grey) fine to coarse grained, some gravel, moist, loose	1
1	**PEAT** (dark brown) fine grained sand to silt, mostly organic, fibrous, odourous, moist - inferred to be loose / soft	2.5
2.5	**SAND** (grey) fine grained, trace to some medium grained sand, trace to some silt, trace gravel, moist to wet, compact - inferred to be natural	6
6	**SILTY SAND** (grey-brown) fine to medium grained, trace gravel, moist to wet, compact to dense - water at 9 feet	12.8
12.8	**TILL-LIKE SILTY SAND** (grey-brown) fine to medium grained, some gravel, wet, very dense - inferred to be till-like	14
14	Test hole terminated at 14 feet	

SAMPLE DCPT	TYPE
52	
9	G 1
6	
16	G 2
19	
34	
43	
30	
22	G 3
27	
17	
11	
18	
87	G 4



Piezometer / Comments / Additional Testing

∇
water level 9 feet during drilling

very dense soil at 12'10"; DCPT terminated at 14 feet

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

Auger Hole LOG

Auger Hole No.: AH12-2

LOGGED BY: PB ON: 24 / 09 / 2012 REVIEWED BY: KK COLLAR ELEVATION: 2.71m METHOD: truck mounted auger drill rig

Type of Test	TYPE -- Type of sample	Notes:
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon	N 5472027.36, E 463169.03
● Becker Denseness Test (BDT)	S -- Shelby tube	backfilled with hole spoil
▲ Number of blows - Standard Penetration (SPT)	G -- Grab	
■ Moisture Content (% of dry weight)	O -- Other (specify)	
> Plastic limit		
< Liquid limit	∇ Ground water level	

Depth (ft)	DESCRIPTION	Depth	DCPT	TYPE	Piezometer / Comments / Additional Testing
0	**CONCRETE**				
	PEAT (dark brown) fine grained sand to silt, mostly organic, fibrous, odourous, moist - inferred to be very loose / very soft - several thin horizons of silt throughout	0.3	1		
			2		
			0		
			1	G 1	
			0		∇
5	**SAND** (grey) medium to coarse grained, trace fine grained sand, some gravel to gravelly, wet, compact - water at 5 feet	5	19	G 2	water level 5 feet during drilling
			10		
	TILL-LIKE SILTY SAND (grey) fine to medium grained, some gravel, wet, very dense - inferred to be till-like - limited sample returned - difficult to advance drill, significant grinding	7.5	100	G 3	very dense soil at 7'6"; DCPT terminated at 7'10" (effective refusal)
	Test hole terminated at 8 feet	8			

Depth scale: m 0–8; ft 0, 5, 10, 15, 20, 25. DCPT plot scale: 20, 40, 60, 80.

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

Auger Hole LOG

Auger Hole No.: AH12-3

LOGGED BY: PB ON: 24 / 09 / 2012 REVIEWED BY: KK COLLAR ELEVATION: 3.14m METHOD: truck mounted auger drill rig

Type of Test	TYPE -- Type of sample	Notes:
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon	N 5471990.96, E 463136.35
● Becker Denseness Test (BDT)	S -- Shelby tube	backfilled with bentonite and hole
▲ Number of blows - Standard Penetration (SPT)	G -- Grab	spoil
■ Moisture Content (% of dry weight)	O -- Other (specify)	
> Plastic limit		
< Liquid limit	∇ Ground water level	

Depth (ft)	DESCRIPTION	Depth	DCPT	TYPE	Piezometer / Comments / Additional Testing
0	**ASPHALT**				
	PEAT (dark brown) fine grained sand to silt, trace rounded gravel, mostly organic, fibrous, odourous, moist - inferred to be very loose / very soft	0.2	0 0 0 1 1	G 1	
5	**SAND** (grey) medium to coarse grained, some fine grained sand, trace silt, some gravel, wet, loose to compact - water at 5 feet	5	9 32 31 18 19	G 2	∇ water level 5 feet during drilling
10	(no sample - washed off auger rods during retrieval)	10	7 3		
	SILT (light grey) wet, firm	12	4 7	G 3	
	TILL-LIKE SILTY SAND (grey-brown) fine to medium grained, some gravel, wet, very dense - inferred to be till-like	14.5	56	G 4	very dense soil at 14'6"
15		15	71		DCPT terminated at 16 feet
	Test hole terminated at 15 feet				

Depth scale: m 0–8; ft 0, 5, 10, 15, 20, 25. DCPT plot scale: 20, 40, 60, 80.

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

HORIZON ENGINEERING INC

Auger Hole LOG

Auger Hole No.: AH14-1

LOGGED BY: PB ON: 11 / 04 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 9.83m METHOD: truck mounted auger drill rig

Type of Test
- ○ Dynamic Cone Penetrometer Test (DCPT)
- ● Becker Denseness Test (BDT)
- ▲ Number of blows - Standard Penetration (SPT)
- ■ Moisture Content (% of dry weight)
- > Plastic limit
- < Liquid limit

TYPE -- Type of sample
- SPT -- Split spoon
- S -- Shelby tube
- G -- Grab
- O -- Other (specify)

∇ Ground water level

Notes: N 5472049.76, E 463111.66
backfilled with excavation spoil

Depth (ft)	DESCRIPTION	Depth	DCPT	TYPE	Piezometer / Comments / Additional Testing
0	**ASPHALT**	0.2	19		
	FILL - SAND (brown) fine to coarse grained sand, some gravel, moist, compact to very dense - no recovery 0-3 feet due to granular soil falling off auger during retrieval - inferred to be fill		40		
			102		102
			42		
			26		
5			22		
	(inferred boulder at 6.5 feet; moved hole 3 feet and drilled out to 7.5 feet)	6.5	225		225
	FILL - SAND (grey) fine to coarse grained sand, some gravel to gravelly, trace silt, wet, loose - inferred to be fill due to density - wet at 7.5 feet	7.5			∇ perched water at 7.5 feet during drilling
			9		
	PEAT (dark reddish brown) fine grained sand to silt, trace rounded gravel, mostly organic, fibrous, odourous, moist, loose to compact / stiff to very stiff	9	12	G 1	
10			29		
	SILTY SAND (grey) fine to medium grained sand, some silt to silty, trace to some coarse grained sand, trace gravel, wet, compact to very dense - limited sample returned 10-15 feet (washed off during retrieval) - inferred to be natural - density increasing with depth	11	26		
			31		
			37	G 2	
			34		
15			40		
			69		
	VERY DENSE SOIL INFERRED BASED ON DCPT DATA	17 / 17.1	113		113
	Auger hole terminated at 17 feet		118		118
					DCPT effective refusal at 19'0"

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

Auger Hole LOG

Auger Hole No.: AH14-2

LOGGED BY: PB ON: 11 / 04 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 10.11m METHOD: truck mounted auger drill rig

Type of Test	TYPE -- Type of sample
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon
● Becker Denseness Test (BDT)	S -- Shelby tube
▲ Number of blows - Standard Penetration (SPT)	G -- Grab
■ Moisture Content (% of dry weight)	O -- Other (specify)
> Plastic limit	
< Liquid limit	∇ Ground water level

Notes: N 5472026.70, E 463101.24
backfilled with excavation spoil

Depth (ft)	DESCRIPTION
0	**FILL - SAND** (greyish brown) fine to coarse grained, some gravel to gravelly, trace decomposed organics, moist, compact to dense - inferred to be fill
3	**FILL - SAND** (brown) fine to medium grained, trace to some silt, trace to some gravel, moist, loose to compact - inferred to be fill - reddish brown at 4.5 feet
5	**FILL - SAND** (brown) fine to coarse grained, some gravel, moist, loose to compact - inferred to be fill
6.5	**FILL - SAND** (brown) fine to medium grained, trace to some coarse grained sand, trace silt, moist to wet, very loose to loose - inferred to be fill - wet at 7.5 feet
9	**SILT** (grey) some fine grained sand, wet, soft to firm - inferred to be natural - very limited sample returned
11	**PEAT** (dark brown) fine grained sand to silt, trace rounded gravel, mostly organic, roots and wood fragments, fibrous, odourous, moist, loose / stiff
12.5	**SILTY SAND** (greyish brown) fine to coarse grained, some gravel, wet, loose to compact
14	**SILTY SAND** (light grey) fine to coarse grained, trace gravel, moist to wet, compact to very dense - wet 14 to 16 feet, moist 16 to 20 feet, wet 20 to 25 feet - grinding 21 to 25 feet but not difficult to penetrate - generally compact 14 to 21 feet, very dense 21 to 25 feet
25	Auger hole terminated at 25 feet

SAMPLE DCPT	TYPE
23	
29	
35	
29	
9	G 1
16	
6	
4	
3	G 2
4	
4	
5	
4	G 3
12	G 4
15	
22	
18	
25	
55	
15	G 5
17	
50	G 6
72	
59	
80	
63	
68	

Piezometer / Comments / Additional Testing:
∇ perched water at 7.5 feet during drilling

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

Auger Hole LOG

Auger Hole No.: AH14-3

LOGGED BY: PB ON: 11 / 04 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 10.17m METHOD: truck mounted auger drill rig

Type of Test	TYPE -- Type of sample	Notes
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon	N 5472030.19, E 463091.29
● Becker Denseness Test (BDT)	S -- Shelby tube	backfilled with excavation spoil
▲ Number of blows - Standard Penetration (SPT)	G -- Grab	
■ Moisture Content (% of dry weight)	O -- Other (specify)	
> Plastic limit		
< Liquid limit	∇ Ground water level	

Depth (ft)	DESCRIPTION	Depth	DCPT	TYPE	Piezometer / Comments / Additional Testing
0	**FILL - SAND** (brown) fine to coarse grained, some gravel, moist, loose - inferred to be fill		5		
3	**FILL - SAND** (brown) fine to medium grained, some coarse grained sand, trace to some gravel, dry to moist, compact - inferred to be fill	3	20 12		
5	**FILL - SAND** (brown) fine to coarse grained, trace to some gravel, moist to wet, loose to compact - moist 5 to 6 feet, wet 6 to 9 feet - inferred to be fill	5	15 9 7 6		∇ perched water at 6 feet during drilling
9	**PEAT** (dark reddish brown) fine grained sand to silt, trace rounded gravel, mostly organic, wood fragments, fibrous, odourous, wet, loose to very loose / soft to firm	9	6 3		
10.5	**SAND AND ORGANICS** (light brown with black inclusions) fine to medium grained sand, some silt, approximately 50% inclusions of decomposed organics, some wood fragments, moist, loose	10.5	6 8 5 5	G 1	
15	**SILTY SAND** (grey) fine to medium grained, trace coarse grained sand, grace gravel, wet, compact to dense - inferred to be natural	15	5 28 43		
18	**SAND** (grey) coarse grained, trace fine to medium grained, trace silt, wet, dense to very dense	18	45 57		
20	- very limited sample returned (no sample returned - washed off auger flights) - very dense	20	63		
22	Auger hole terminated at 22 feet	22			

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

Auger Hole LOG

Auger Hole No.: AH14-4

LOGGED BY: PB ON: 11 / 04 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 7.15m METHOD: truck mounted auger drill rig

Type of Test	TYPE -- Type of sample
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon
● Becker Denseness Test (BDT)	S -- Shelby tube
▲ Number of blows - Standard Penetration (SPT)	G -- Grab
■ Moisture Content (% of dry weight)	O -- Other (specify)
> Plastic limit	
< Liquid limit	⊽ Ground water level

Notes: N 5471969.45, E 463077.46
backfilled with excavation spoil

Depth (ft)	DESCRIPTION	Depth	DCPT	TYPE	Piezometer / Comments / Additional Testing
0	**ASPHALT**	0.3	1		
	FILL - SAND (brown) fine to medium grained, some coarse grained sand, some gravel, moist, very loose to loose - inferred to be fill		7		
2		2	22		⊽
3	**FILL - SAND** (brown) medium grained, trace gravel, moist to wet, compact - inferred to be fill	3	3		perched water at 3 feet during drilling
3.5	**FILL - SANDY SILT** (dark brown) fine grained sand, trace gravel, wet, soft	3.5	7	G 1	
5	**SILTY SAND** (grey mottled reddish brown) fine to medium grained, trace coarse grained sand, trace gravel, moist to wet, loose to very dense - wet 3.5 to 10 feet, moist 10 to 11 feet - loose 3.5 to 5 feet, compact 5 to 7 feet, dense 7 to 9 feet, very dense 9 to 11 feet - inferred to be natural		11		
			20		
			47		
			41	G 2	
			65		
10			57		
11	Auger hole terminated at 11 feet	11			

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

HORIZON ENGINEERING INC

TESTHOLE LOG TEST HOLE LOGS.GPJ HORIZON.GDT 15/1/29

Auger Hole LOG

Auger Hole No.: AH14-5

LOGGED BY: PB ON: 11 / 04 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 5.32m METHOD: truck mounted auger drill rig

Type of Test
- ○ Dynamic Cone Penetrometer Test (DCPT)
- ● Becker Denseness Test (BDT)
- ▲ Number of blows - Standard Penetration (SPT)
- ■ Moisture Content (% of dry weight)
- > Plastic limit
- < Liquid limit

TYPE -- Type of sample
- SPT -- Split spoon
- S -- Shelby tube
- G -- Grab
- O -- Other (specify)

∇ Ground water level

Notes: N 5472006.74, E 463114.49
backfilled with excavation spoil

Depth (ft)	DESCRIPTION
0 – 0.2	**ASHPHALT**
0.2 – 2	**FILL - SAND** (brown) fine to medium grained, some coarse grained sand, some gravel, trace rootlets, moist - inferred to be loose to compact - inferred to be fill
2 – ~4.6	**PEAT** (dark brown) fine grained sand to silt, trace rounded gravel, mostly organic, roots and wood fragments, fibrous, odourous, moist - inferred to be loose / stiff
~4.6 – 13	**SILTY SAND** (grey) fine to medium grained, some silt to silty, trace coarse grained sand, trace gravel, moist to wet, loose to dense - inferred to be natural - localized thin seams of fine to coarse grained sand from 3 to 5 feet and 8 to 9 feet - wet at 3.5 feet - grinding at 7 to 9 feet
13 – 15	**SAND** (grey) fine to medium grained, some silt, trace coarse grained sand, trace gravel, moist to wet, very dense - limited sample returned
15	Auger hole terminated at 15 feet

Depth (ft)	DCPT	TYPE	Piezometer / Comments / Additional Testing
0.2			
1		G 1	Moisture content ~62%
2		G 2	
	12		∇ perched water at 3.5 feet during drilling
	27		
	19		
	6		
	40		
	41		
	25		
	25		
	9		
	16		
13	77	G 3	
	86		
15			

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

HORIZON ENGINEERING INC

Auger Hole LOG

Auger Hole No.: AH14-6

LOGGED BY: PB ON: 11 / 04 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 2.54m METHOD: truck mounted auger drill rig

Type of Test	TYPE -- Type of sample
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon
● Becker Denseness Test (BDT)	S -- Shelby tube
▲ Number of blows - Standard Penetration (SPT)	G -- Grab
■ Moisture Content (% of dry weight)	O -- Other (specify)
> Plastic limit	
< Liquid limit	∇ Ground water level

Notes: N 5472007.82, E 463160.28

backfilled with excavation spoil

Depth (ft)	DESCRIPTION	Piezometer / Comments / Additional Testing
0	**FILL - SAND** (brown) fine to coarse grained, some gravel, moist, compact to dense - inferred to be fill	
2	**PEAT** (dark brown) fine grained sand to silt, trace rounded gravel, mostly organic, wood fragments, fibrous, odourous, moist - inferred to be very loose / soft	∇ water at 3 feet during drilling
3	**SAND AND PEAT** (grey and dark brown) fine to medium grained sand with trace gravel and lenses of peat (as above), wet, very loose to loose - wet at 3 feet	
6	**SILTY SAND** (grey) fine to coarse grained sand, some silt to silty, trace to some gravel, occasional cobbles, wet, very loose to compact - inferred to be natural	
12	**SANDY SILT** (grey) fine grained sand, trace medium grained sand, trace coarse grained sand, trace gravel, wet, stiff to very stiff, low plasticity	
15	**SILTY SAND** (grey) fine to medium grained sand, trace coarse grained sand, trace gravel, wet, loose to very dense - loose 15 to 17 feet, very dense 17 to 19 feet	
19	Auger hole terminated at 19 feet	

Depth interval (ft)	SAMPLE DCPT	SAMPLE TYPE
0–1	38	
1–2	11	
2–3	3	
3–4	3	
4–5	4	
5–6	5	
6–7	2	
7–8	8	
8–9	9	G 1
9–10	16	
10–11	10	
11–12	11	
12–13	8	
13–14	19	
14–15	12	G 2
15–16	7	
16–17	6	
17–18	68	
18–19	120	

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

HORIZON ENGINEERING INC

Borehole LOG

Borehole No.: BH14-1

LOGGED BY: PB ON: 15 / 04 / 2014 REVIEWED BY: KK COLLAR ELEVATION: seabed METHOD: track mounted sonic rig (on barge)

Type of Test
- ○ Dynamic Cone Penetrometer Test (DCPT)
- ● Becker Denseness Test (BDT)
- ▲ Number of blows - Standard Penetration (SPT)
- ■ Moisture Content (% of dry weight)
- > Plastic limit
- < Liquid limit

TYPE -- Type of sample
- SPT -- Split spoon
- S -- Shelby tube
- G -- Grab
- O -- Other (specify)

∇ Ground water level

Notes: N 5472053, E 463265
Depth datum is seabed. Water used as drilling fluid. Artesian groundwater pressures encountered; hole sealed with bentonite as described below.

Depth (m / ft)	DESCRIPTION	Depth	SPT	TYPE	SPT blows (20 / 40 / 60 / 80)	Piezometer / Comments / Additional Testing
0 / 0	**SILT AND SAND** (dark grey) fine to medium grained sand, trace organics (wood fragments), trace seashells, wet, odourous - inferred to be very loose / very soft - sample recovery approximately 2 feet - inferred to be natural			G 1, 4		slough backfill 0 to 3 feet
1 /	**SILT** (grey) some fine grained sand, trace gravel, occasional cobbles, wet - inferred to be very soft	3.5		G 2		
3 / 10	**SAND** (dark grey) fine to medium grained, trace seashells, wet - inferred to be compact	9.5		G 3		bentonite chips backfill 3 to 17.5 feet
	SILTY SAND (grey) fine grained sand, trace medium grained sand, trace coarse grained sand, trace gravel, wet, compact to dense - inferred to be compact 11 to 12 feet, inferred to be dense 12 to 21 feet - artesian groundwater encountered following drilling of 11 to 21.5 foot run but prior to retrieving core barrel - rising water level confirmed with electrical conductivity probe to be fresh water - low artesian pressure observed (water level rose ~1.3 metre in ~15 minutes in 6 inch casing) - water level inferred to have stabilized at ~ -2.0 metres elevation	11	14	SPT 5	▲ 14	
			32		▲ 32	
				G 6		
				G 7		
				G 8		coated bentonite pellets backfill 17.5 to 21.5 feet
				G 9		hole confirmed to be sealed following backfilling
6.4 / 21	**GRAVELLY SAND** (grey) medium to coarse grained sand, fine to coarse subrounded gravel, some fine grained sand, trace silt, wet - inferred to be compact to dense	21				
	Borehole terminated at 21.5 feet	21.5				

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

Borehole LOG

Borehole No.: BH14-2

LOGGED BY: PB ON: 15 / 04 / 2014 REVIEWED BY: KK COLLAR ELEVATION: seabed METHOD: track mounted sonic rig (on barge)

Type of Test
- ○ Dynamic Cone Penetrometer Test (DCPT)
- ● Becker Denseness Test (BDT)
- ▲ Number of blows - Standard Penetration (SPT)
- ■ Moisture Content (% of dry weight)
- > Plastic limit
- < Liquid limit

TYPE -- Type of sample
- SPT -- Split spoon
- S -- Shelby tube
- G -- Grab
- O -- Other (specify)

∇ Ground water level

Notes: N 5472002, E 463316
Depth datum is seabed. Water used as drilling fluid. Artesian groundwater pressures encountered; hole sealed with bentonite as described below.

Depth (ft)	DESCRIPTION	Depth	SPT	TYPE	Blows (20 / 40 / 60 / 80)	Piezometer / Comments / Additional Testing
0	(No sample returned - very soft / very loose soil not retained in core barrel)					pressure grout backfill 0 to 15 feet
13	**SILTY SAND** (grey) fine grained, trace medium grained sand, trace coarse grained sand, trace gravel, occasional cobbles, wet, compact - trace to some gravel below 16.5 feet - inferred to be natural - artesian groundwater encountered at 18 feet before retrieving core barrel - low artesian pressure observed (~1 psi at 29.5 to 31.5 feet above seabed) - water level inferred to have stabilized at ~3.2 metres elevation - rising water level confirmed with electrical conductivity probe to be fresh water - electrical conductivity prove confirmed only saline water observed outside casing adjacent to borehole	13	11	SPT 1	▲	
			14	SPT 2	▲	bentonite chips backfill 15 to 18 feet
				G 3		
17.5	**SAND AND GRAVEL** (grey) medium to coarse grained sand, some silt to silty (may be from horizon above), wet - very limited sample (end of run) - inferred to be compact to dense	17.5		G 4		hole confirmed to be sealed following backfilling
18	Borehole terminated at 18 feet	18				

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

Borehole LOG

Borehole No.: BH14-3

LOGGED BY: PB ON: 16 / 04 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 3.18m METHOD: track mounted sonic rig

Type of Test	TYPE -- Type of sample
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon
● Becker Denseness Test (BDT)	S -- Shelby tube
▲ Number of blows - Standard Penetration (SPT)	G -- Grab
■ Moisture Content (% of dry weight)	O -- Other (specify)
> Plastic limit	
< Liquid limit	⊽ Ground water level

Notes: N 5472007.18, E 463141.25
Water used as drilling fluid. Artesian groundwater pressures encountered; hole sealed with bentonite & pressure grounted as described below.

Depth (m / ft)	DESCRIPTION	Depth	SPT	TYPE
0 / 0	**FILL - SAND** (greyish brown) fine to coarse grained, some gravel, trace to some silt, trace to some organics (wood fragments and decomposed organics), wet - inferred to be fill - inferred to be loose to very loose			
1	**PEAT** (dark brown) fine grained sand to silt, trace rounded gravel, mostly organic, wood fragments, fibrous, odourous, wet - inferred to be loose / stiff	4		G 1
5		5.5		G 2
	SANDY SILT (grey) fine grained sand, trace gravel, trace organics (wood fragments), moist - inferred to be natural - inferred to be stiff to very stiff	6		G 3
2	**SAND** (dark grey) coarse grained, some medium grained sand, trace to some gravel, trace fine grained sand, trace silt, wet - inferred to be loose	8		G 4
3 / 10	**SILTY SAND** (grey) fine to medium grained, trace coarse grained sand, trace gravel, wet - inferred to be loose to compact	9.5		G 5
4	**SAND** (grey) fine grained, some medium grained sand, some silt, some coarse grained sand, trace gravel, trace organics (wood fibres), moist - locally some gravel - inferred to be compact	12		
15			29	SPT 6
			30	
5	**GRAVELLY SAND** (grey) medium to coarse grained sand, some fine grained sand, some subrounded to subangular gravel, occasional cobbles, wet, compact - locally trace to some silt at 15.5 feet - artesian groundwater encountered at 15 feet during SPT - rising water level confirmed with electrical conductivity probe to be fresh water - low artesian pressure observed when casing at 14 feet and SPT to 16 feet (flow 3L/min, water level stabilized 1.28m above ground)	16.5		G 7
6 / 20			47	
			46	
7	**GRAVEL** (grey) fine to coarse gravel, trace medium to coarse grained sand, occasional cobbles, wet, compact - low artesian pressure observed when casing at 19 feet (flow 3.5L/min inside casing, water level stabilized 1.30m above ground in casing) - SPT 19-21 (pushing a rock, infer soil to be compact to dense)	22		G 8
25 / 8		24		

Chart scale: 20 40 60 80

Piezometer / Comments / Additional Testing

steel 2" pipe 1 to 6 feet

pressure grout backfill 1 to 9 feet

solid 2" PVC pipe 6 to 14 feet

bentonite pellets backfill 9 to 13 feet

natural slough backfill 13 to 24 feet (plus 1/2 bag filter sand)

screened 2" PVC pipe 14 to 24 feet

during piezometer installation, moderate artesian pressure observed when casing at 6.5 feet (pumping 76L/min out of hole to take pressure off fresh grout) hole confirmed to be sealed following piezometer installation

TESTHOLE LOG TEST HOLE LOGS.GPJ HORIZON.GDT 15/1/29

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

Borehole LOG

Borehole No.: BH14-3

LOGGED BY: PB ON: 16 / 04 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 3.18m METHOD: track mounted sonic rig

Type of Test
- ○ Dynamic Cone Penetrometer Test (DCPT)
- ● Becker Denseness Test (BDT)
- ▲ Number of blows - Standard Penetration (SPT)
- ■ Moisture Content (% of dry weight)
- > Plastic limit
- < Liquid limit

TYPE -- Type of sample
- SPT -- Split spoon
- S -- Shelby tube
- G -- Grab
- O -- Other (specify)

∇ Ground water level

Notes: N 5472007.18, E 463141.25
Water used as drilling fluid. Artesian groundwater pressures encountered; hole sealed with bentonite & pressure grounted as described below.

Depth (m)	Depth (ft)	DESCRIPTION	Symbol	Depth	SAMPLE SPT	SAMPLE TYPE	20 / 40 / 60 / 80	Piezometer / Comments / Additional Testing
9	30	**SAND AND GRAVEL** (grey) medium grained sand, some coarse grained sand, occasional rounded cobbles, wet - inferred to be compact to dense - low artesian pressure observed when casing at 24 feet (water level rose 3.5m inside casing in 3 hours 9 minutes) - water level stabilized at ~4.6 metres elevation Borehole terminated at 24 feet						standpipe readings: Apr. 16/14 - 4 psi, May 12/14 - 5 psi, Dec. 10/14 - 5.5 psi, Jan. 12/15 - 5 psi
10								
11	35							
12	40							
13								
14	45							
15	50							
16								

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

Borehole LOG

Borehole No.: BH14-4

LOGGED BY: PB ON: 09 / 12 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 10.09m METHOD: track mounted sonic rig

Type of Test
- ○ Dynamic Cone Penetrometer Test (DCPT)
- ● Becker Denseness Test (BDT)
- ▲ Number of blows - Standard Penetration (SPT)
- ■ Moisture Content (% of dry weight)
- > Plastic limit
- < Liquid limit

TYPE -- Type of sample
- SPT -- Split spoon
- S -- Shelby tube
- G -- Grab
- O -- Other (specify)

∇ Ground water level

Notes: N 5472030.58, E 463103.34
Water used as drilling fluid. Artesian groundwater pressures encountered; hole sealed with bentonite and grounted as described below.

Depth (ft)	DESCRIPTION	Depth	SPT	TYPE	Piezometer / Comments / Additional Testing
0	**TOPSOIL** (dark brown) fine to medium grained sand, some coarse grained sand, organics (roots and rootlets), moist - inferred to be loose	0.3			
	FILL - SAND (brown) fine to coarse grained sand, some gravel to gravelly, trace silt, trace organics (rootlets), moist - inferred to be loose to compact - inferred to be fill	5			black iron riser pipe 1 to 6 feet
5	**FILL - SAND** (grey-brown) fine to coarse grained sand, trace gravel, moist - inferred to be loose to compact - inferred to be fill	5.5 / 7			
	FILL - SILTY SAND (brown) fine to medium grained sand, trace gravel, trace organics (wood fragments), trace debris (nail, plastic), occasional cobbles, moist - inferred to be loose - inferred to be fill	9		G 1	
10	**FILL - SAND** (brown) fine to medium grained sand, trace silt, trace gravel, trace organics (roots and rootlets), moist - inferred to be very loose to loose - inferred to be fill	11.5		G 2	solid 2" PVC 6 to 44 feet
	SANDY SILT (grey) fine to medium grained sand, some gravel, trace coarse grained sand, wet - inferred to be soft to firm - inferred to be natural	13		G 3	cementatious grout 1 to 39 feet
15	**SILTY SAND** (brown) fine to medium grained, some gravel to gravelly, trace coarse grained sand, occasional cobbles, wet - inferred to be loose to compact	15 / 16		G 4 / G 5	
	PEAT (dark brown) fine grained sand to silt, trace rounded gravel, mostly organic, roots and wood fragments, fibrous, odourous, moist - inferred to be loose / stiff	19	13		
20	**SAND** (grey) fine grained sand, some silt to silty, trace coarse grained sand, trace gravel, moist - inferred to be loose to compact		100		SPT effective refusal at 20'4" (100 blows in 1 inch)
	SILTY SAND (light brown) fine grained sand, some medium grained sand, trace coarse grained sand, trace gravel, moist - inferred to be compact to dense				
25	**SILTY SAND** (grey) fine to medium grained sand, moist, compact to very dense - compact 19 to 20 feet - very dense 20 to 23.5 feet - boulder 23.5 to 26.5 feet	26.5			

Depth scale (m): 0, 1, 2, 3, 4, 5, 6, 7, 8. Chart scale: 20, 40, 60, 80.

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

Borehole LOG

Borehole No.: BH14-4

LOGGED BY: PB ON: 09 / 12 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 10.09m METHOD: track mounted sonic rig

Type of Test
- ○ Dynamic Cone Penetrometer Test (DCPT)
- ● Becker Denseness Test (BDT)
- ▲ Number of blows - Standard Penetration (SPT)
- ■ Moisture Content (% of dry weight)
- > Plastic limit
- < Liquid limit

TYPE -- Type of sample
- SPT -- Split spoon
- S -- Shelby tube
- G -- Grab
- O -- Other (specify)

∇ Ground water level

Notes: N 5472030.58, E 463103.34
Water used as drilling fluid. Artesian groundwater pressures encountered; hole sealed with bentonite and grounted as described below.

Depth (m / ft)	DESCRIPTION	Depth	SPT	TYPE	SPT blows (20, 40, 60, 80)	Piezometer / Comments / Additional Testing
	SILTY SAND (grey) fine to coarse grained sand, trace gravel, wet - inferred to be dense to very dense - very low recovery, as most of silt component washed out in drilling fluid *(Continued)*					
9 m	**SAND AND GRAVEL** (grey) medium grained, trace coarse grained sand, trace fine grained sand, trace silt, wet, very dense	29	112	G 6	▲112	
30 ft		30	75	G 7	▲	
	SILTY, GRAVELLY SAND (grey) fine to medium grained, trace coarse grained sand, moist, very dense	32		G 8		
10 m	**SAND** (grey) medium grained, some coarse grained sand, some fine grained sand, trace silt, trace gravel, wet - inferred to be compact	33		G 9		
		34				
35 ft	**GRAVEL** (grey) some coarse grained sand, trace medium grained sand, occasional cobbles, wet - inferred to be compact			G 10		
11 m	**SAND** (grey) medium grained, some fine grained sand, trace coarse grained sand, trace silt, trace gravel, wet - inferred to be compact	37		G 11		
		38		G 12		
		39	27	G 13	▲	
12 m / 40 ft	**GRAVEL** (grey) trace coarse grained sand, trace medium grained sand, trace fine grained sand, trace silt, wet - inferred to be compact		28		▲	
		41				bentonite seal 39 to 43 feet
13 m	**SAND AND GRAVEL** (grey) fine to medium grained sand, trace coarse grained sand, trace silt, occasional cobbles, wet - inferred to be compact					filter sand 43 to 49 feet
45 ft	**SAND** (grey) medium grained, some fine grained sand, some coarse grained sand, some gravel, trace silt, wet, compact					screened 2" PVC 44 to 49 feet
14 m	**(NO RECOVERY)** - driller thinks that core barrel is pushing cobble - inferred soil is loose sand					SPT refusal at 49 feet (inferred to be bouncing on a cobble)
15 m	**SAND** (grey) fine to medium grained, trace coarse grained sand, trace silt, trace gravel, wet - inferred to be compact	49 / 49.5	100	SPT 14 / SPT 15	▲	artesian pressure observed at 49 feet (static head ~1.17m /3'10" above grade, flow 0.04 L/s)
50 ft / 16 m	**SAND** (grey) medium grained, some coarse grained sand, some gravel, trace fine grained sand, wet - inferred to be compact	50				
	Borehole terminated at 50 feet					

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

Borehole LOG

Borehole No.: BH14-5

LOGGED BY: PB ON: 10 / 12 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 3.04m METHOD: track mounted sonic rig

Type of Test	TYPE -- Type of sample
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon
● Becker Denseness Test (BDT)	S -- Shelby tube
▲ Number of blows - Standard Penetration (SPT)	G -- Grab
■ Moisture Content (% of dry weight)	O -- Other (specify)
> Plastic limit	
< Liquid limit	∇ Ground water level

Notes: N 5471977.95, E 463129.78
Water used as drilling fluid. Artesian groundwater pressures encountered; hole sealed with bentonite and grounted as described below.

DESCRIPTION

FILL - SAND (dark brown)
fine to coarse grained, some gravel, trace silt, trace organics (roots), moist
- inferred to be loose
- inferred to be fill

FILL - SAND (brown)
fine to coarse grained, some gravel, moist
- inferred to be loose
- inferred to be fill

PEAT (dark brown)
fine grained sand to silt, trace rounded gravel, mostly organic, roots and wood fragments, fibrous, odourous, moist, very loose / very soft
- some fine to coarse grained sand 5 to 7 feet
- odourous
- inferred to be natural

SILTY SAND (grey)
fine to medium grained, wet
- inferred to be loose

SAND (grey)
medium to coarse grained, trace fine grained sand, trace gravel, wet
- inferred to be loose

SILT AND SAND TO SANDY SILT(grey)
fine grained sand, some medium grained sand, trace coarse grained sand, trace gravel, occasional cobbles, wet

- inferred to be very loose to loose / soft to stiff

SILT AND SAND (grey to light brown)
fine grained sand, trace medium grained sand, trace coarse grained sand, occasional cobbles, wet, very dense

SANDY GRAVEL (grey)
medium grained sand, trace coarse grained sand, trace fine grained sand, trace silt, wet
- inferred to be dense to very dense

SANDY GRAVEL (dark grey)
medium to coarse grained sand, some cobbles, trace fine grained sand, trace silt, wet
- inferred to be dense to very dense

SAND (grey)
medium grained, some coarse grained sand, some gravel, trace fine grained sand,

Layer depths (ft)

1, 2, 7, 7.5, 8, 16.5, 20.5, 23, 24, 26

SAMPLE

SPT	TYPE
	G 1
1	
2	
	G 2
3	SPT 3
8	
7	SPT 4
9	
	G 5
57	SPT 6
71	
	G 7
	G 8
36	G 9
63	

Chart scale: 20 40 60 80

Depth scale: m 0–8; ft 0, 5, 10, 15, 20, 25

Piezometer / Comments / Additional Testing

black iron riser pipe 1 to 6 feet

solid 2" PVC 6 to 24 feet

cementatious grout 1 to 19.5 feet

bentonite seal 19.5 to 23 feet

pea gravel 23 to 29 feet

screened 2" PVC 24 to 29 feet

Borehole LOG

Borehole No.: BH14-5

LOGGED BY: PB ON: 10 / 12 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 3.04m METHOD: track mounted sonic rig

Type of Test
○ Dynamic Cone Penetrometer Test (DCPT)
● Becker Denseness Test (BDT)
▲ Number of blows - Standard Penetration (SPT)
■ Moisture Content (% of dry weight)
> Plastic limit
< Liquid limit

TYPE -- Type of sample
SPT -- Split spoon
S -- Shelby tube
G -- Grab
O -- Other (specify)

▽ Ground water level

Notes: N 5471977.95, E 463129.78
Water used as drilling fluid. Artesian groundwater pressures encountered; hole sealed with bentonite and grounted as described below.

Depth m	Depth ft	DESCRIPTION	Symbol	Depth	SAMPLE SPT	SAMPLE TYPE	20 40 60 80	Piezometer / Comments / Additional Testing
		trace silt, wet, dense to very dense						
		GRAVEL (grey) trace coarse grained sand, wet - inferred to be dense to very dense *(Continued)*		27.5		G 10		artesian pressure observed at 24 feet (2.7 psi at 1m/3'3" above grade, static head ~2.7m /9'0" above grade, flow 6.6 USgpm)
9	30	**SAND AND GRAVEL** (grey) medium grained sand, trace coarse grained sand, trace fine grained sand, trace silt, wet - inferred to be dense to very dense		29				
		Borehole terminated at 29 feet						
10								
	35							
11								
12	40							
13								
	45							
14								
15	50							
16								

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

Borehole LOG

Borehole No.: BH14-6

LOGGED BY: PB ON: 10 / 12 / 2014 REVIEWED BY: KK COLLAR ELEVATION: 7.22m METHOD: track mounted sonic rig

Type of Test	TYPE -- Type of sample
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon
● Becker Denseness Test (BDT)	S -- Shelby tube
▲ Number of blows - Standard Penetration (SPT)	G -- Grab
■ Moisture Content (% of dry weight)	O -- Other (specify)
> Plastic limit	
< Liquid limit	∇ Ground water level

Notes: N 5471970.22, E 463077.29
Water used as drilling fluid. Artesian groundwater pressures encountered; hole sealed with bentonite and grounted as described below.

Depth (ft)	DESCRIPTION	Depth	SAMPLE SPT	SAMPLE TYPE	SPT N (▲)	Piezometer / Comments / Additional Testing
0	**FILL - SAND** (brown to grey) fine to coarse grained, moist - fine to coarse grained sand to 2.5 feet - fine to medium grained sand 2.5 to 3 feet - inferred to be loose - inferred to be fill					
3	**PEAT** (dark brown) fine grained sand to silt, trace rounded gravel, mostly organic, roots and wood fragments, fibrous, odourous, moist - inferred to be very loose / very soft - inferred to be natural	3	2	G 1	2	black iron riser pipe 1 to 6 feet
3.5		3.5	5		5	
	SANDY SILT (grey) fine to medium grained sand, trace coarse grained sand, trace gravel, moist, soft to firm					solid 2" PVC 6 to 19 feet
7	**SAND TO SILT AND SAND** (grey) fine to coarse grained, trace to some silt, wet - fine grained sand, trace medium grained sand, trace coarse grained sand, trace gravel 7 to 8.5 feet - fine to coarse grained sand 8.5 to 9.5 feet - inferred to be loose to compact	7	13	G 2 / SPT 3	13	cementatious grout 1 to 15.5 feet
			16		16	
9.5	**SILT AND SAND** (grey) fine grained sand, trace medium grained sand, trace coarse grained sand, trace gravel, wet, compact / stiff to very stiff	9.5				
13	**SANDY SILT** (grey) fine grained sand, trace medium grained sand, trace coarse grained sand, trace gravel, occasional cobbles, wet - inferred to be firm	13	5	G 4	5	SPT - bouncing on cobble at 15'4" - N value artificially high
15	**SILT AND SAND** (grey-brown) fine to medium grained, trace gravel, trace coarse grained sand, wet, dense - brown 17 to 18 feet	15	41	G 5	41	bentonite seal 15.5 to 18 feet
18	**SAND** (grey) coarse grained, some medium grained sand, some gravel, trace coarse grained sand, wet - inferred to be very dense	18		G 6		filter sand 18 to 24 feet
19		19	72		72	
	SANDY GRAVEL (grey) some coarse grained sand, trace medium grained sand, trace fine grained sand, trace silt, wet - inferred to be dense to very dense		91	G 7	91	screened 2" PVC 19 to 24 feet
24	Borehole terminated at 24 feet	24				artesian pressure observed at 24 feet (static head 0.6m /1'10" above grade, flow 1.3 USgpm)

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

Test Pit LOG

Test Pit No.: TP12-1

LOGGED BY: PB ON: 19 / 09 / 2012 REVIEWED BY: KK COLLAR ELEVATION: 6.22m METHOD: excavator

Type of Test	TYPE -- Type of sample	Notes
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon	N 5472033.85, E 463119.55
● Becker Denseness Test (BDT)	S -- Shelby tube	backfilled with excavation spoil
▲ Number of blows - Standard Penetration (SPT)	G -- Grab	
■ Moisture Content (% of dry weight)	O -- Other (specify)	
> Plastic limit		
< Liquid limit	∇ Ground water level	

DESCRIPTION

Depth (ft)	Description
0 – 0.7	**FILL - SAND** (grey) medium to coarse grained sand, some gravel to gravelly, wet - inferred to be compact - perched seepage observed
0.7 – 1.5	**SILTY SAND** (grey) fine grained, trace medium grained sand, moist - inferred to be very loose - inferred to be natural
1.5 – 4	**PEAT** (dark brown) fine grained sand to silt, mostly organic, fibrous, odourous, moist - inferred to be very loose / very soft
4 – 5	**SAND** (grey) medium to coarse grained, some gravel, wet - inferred to be dense to very dense - some seepage observed
5 – 9	**SILTY SAND** (grey) moist - inferred to be compact to dense
9 – 12	**SAND** (grey) fine to medium grained, moist - inferred to be compact to dense
12 – 13	**SAND** (grey) medium to coarse grained, wet - inferred to be compact - significant seepage (inferred to be perched on surface of till below)
13 – 13.2	**INFERRED TILL-LIKE SOIL** - unable to push excavator bucket below 13 feet - inferred to be very dense

Test pit terminated at 13 feet at inferred surface of till

SAMPLE / DCPT

Depth (ft)	Sample (DCPT)	Type	Piezometer / Comments / Additional Testing
0–1	37		DCPT12-1 data shown (located adjacent to TP12-1)
1–2	1		
2–3	0		
3–4	120		
4–5			DCPT refusal at 3'10" so augered to 5 feet to resume DCPT
5–6	13		
6–7	11		
7–8	39	G 1	
8–9	45		
9–10	35	G 2	
10–11	20		
11–12	12		∇
12–13	11		water level 12 feet during drilling
13–14	65	G 3	very dense soil at 13'3", DCPT refusal at 14 feet

TESTHOLE LOG TEST HOLE LOGS.GPJ HORIZON.GDT 15/1/29

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

Test Pit LOG

Test Pit No.: TP12-2

LOGGED BY: PB ON: 19 / 09 / 2012 REVIEWED BY: KK COLLAR ELEVATION: 4.52m METHOD: excavator

Type of Test	TYPE -- Type of sample	Notes
○ Dynamic Cone Penetrometer Test (DCPT)	SPT -- Split spoon	N 5472036.54, E 463141.43
● Becker Denseness Test (BDT)	S -- Shelby tube	backfilled with excavation spoil;
▲ Number of blows - Standard Penetration (SPT)	G -- Grab	surface grade is approximately 5 feet
■ Moisture Content (% of dry weight)	O -- Other (specify)	lower than TP12-1
> Plastic limit		
< Liquid limit	∇ Ground water level	

Depth (m)	Depth (ft)	DESCRIPTION	Depth	SAMPLE TYPE	20 / 40 / 60 / 80	Piezometer / Comments / Additional Testing
0	0	**ASHPHALT**	0.2			
		FILL - SAND (grey) medium to coarse grained, some gravel to gravelly, trace debris (bricks), moist - inferred to be loose - perched seepage observed	1			
		PEAT (dark brown) fine grained sand to silt, mostly organic, fibrous, odourous, moist - inferred to be very loose / soft	2.25			
1	5	**SAND** (grey) fine to medium grained, trace gravel, occasional cobbles, moist - inferred to be compact to dense - brown below 3.5 feet		G 1		
2		Test pit terminated at 8 feet due to significant groundwater inflow resulting from intercepted drainage trench	8			
3	10					
4	15					
5						
6	20					
7	25					
8						

HORIZON ENGINEERING INC

PROJECT: PROPOSED 'THE GEORGE' HOTEL, GIBSONS

JOB NO.: 112-3155

SHEET 1 of 1

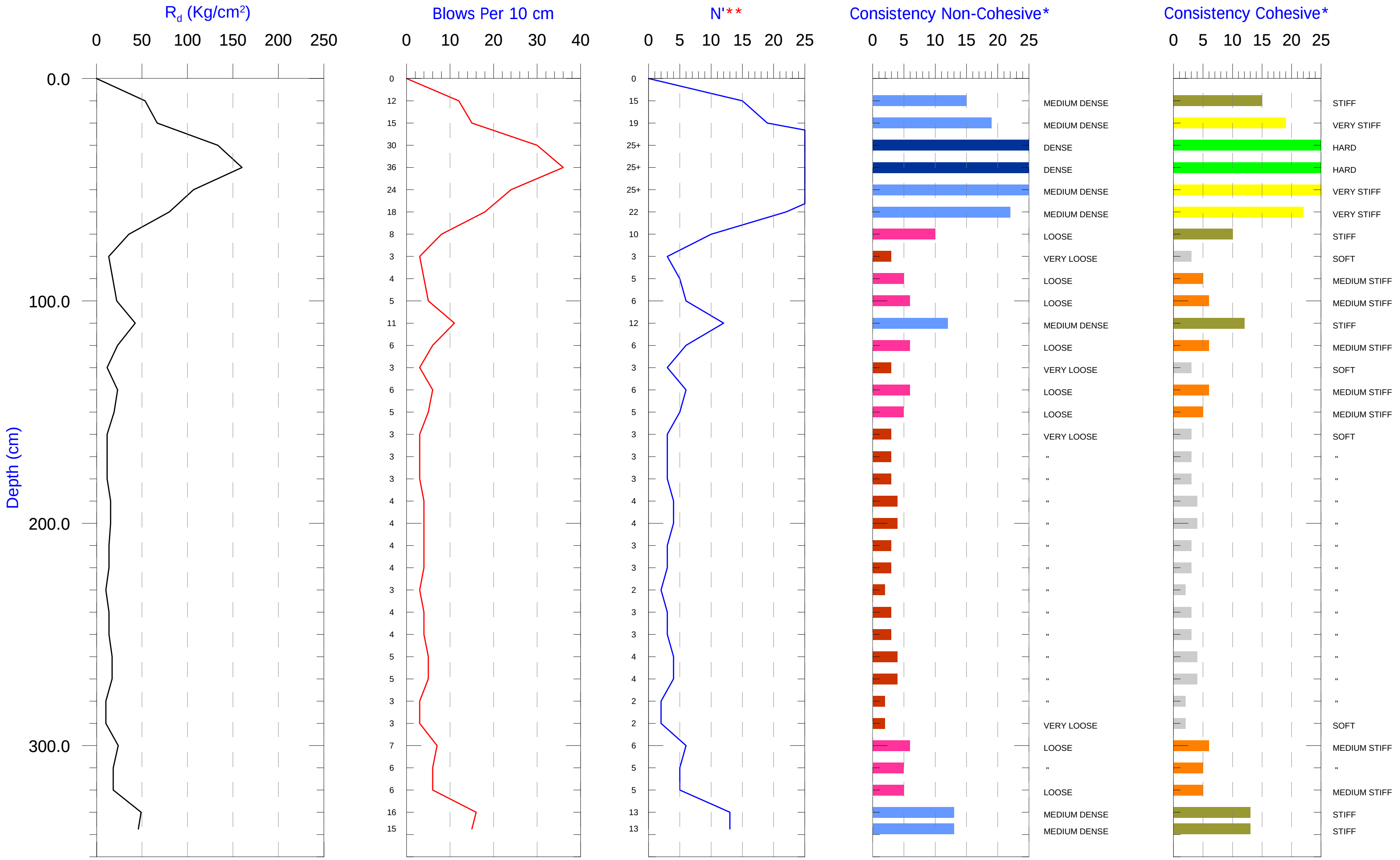
Wildcat Cone Penetration

Job No: 112-3155
Date: 12-Jan-15
Site: 385 Gower Pt. Road
Gibsons, BC

Operator: CC/GC
Test Hole: WC15-1
Max Depth (cm): 347.5
Coordinates: 49° 23.966'N 123° 30.425'W

Comments: Terminated at inferred loose to medium dense strata
24.5m east of back property fence at 407 Gower Point Road

Data & Results



* Inferred Soil Description as per Manufacture Manual
** Wildcat does not correlate N' beyond 25+ blows.

HE Testing & Monitoring
102-173 Forester St, North Vancouver, BC

HE Testing

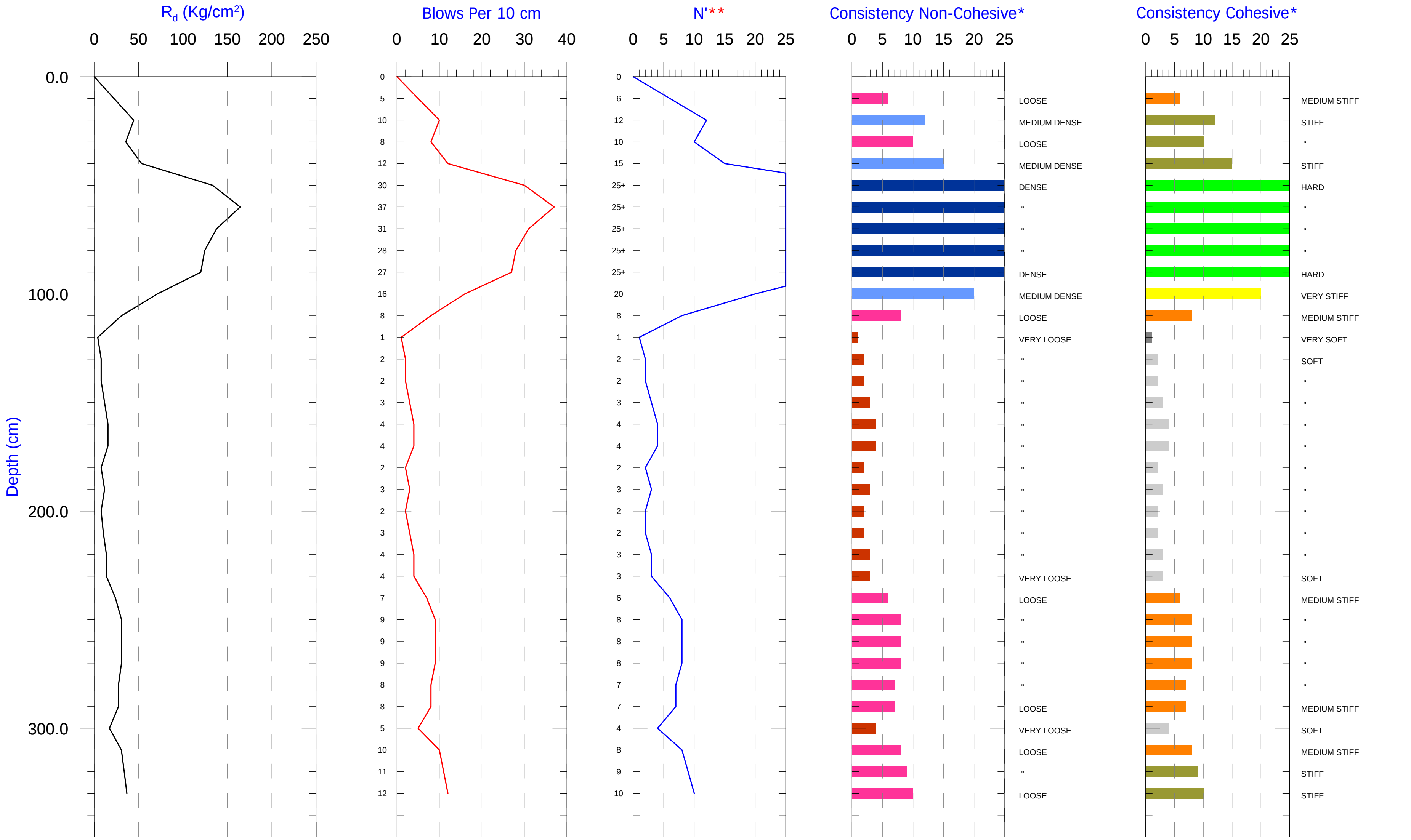
Wildcat Cone Penetration

Job No: 112-3155
Date: 12-Jan-15
Site: 385 Gower Pt. Road
Gibsons, BC

Operator: CC/GC
Test Hole: WC15-2
Max Depth (cm): 330
Coordinates: 49° 23.973'N 123° 30.427'W

Comments: Terminated at inferred loose to medium dense strata
(18.5m north of WC15-1; 27.5m east of dock walkway entrance)

Data & Results



* Inferred Soil Description as per Manufacture Manual
** Wildcat does not correlate N' beyond 25+ blows.

HE Testing & Monitoring
102-173 Forester St, North Vancouver, BC

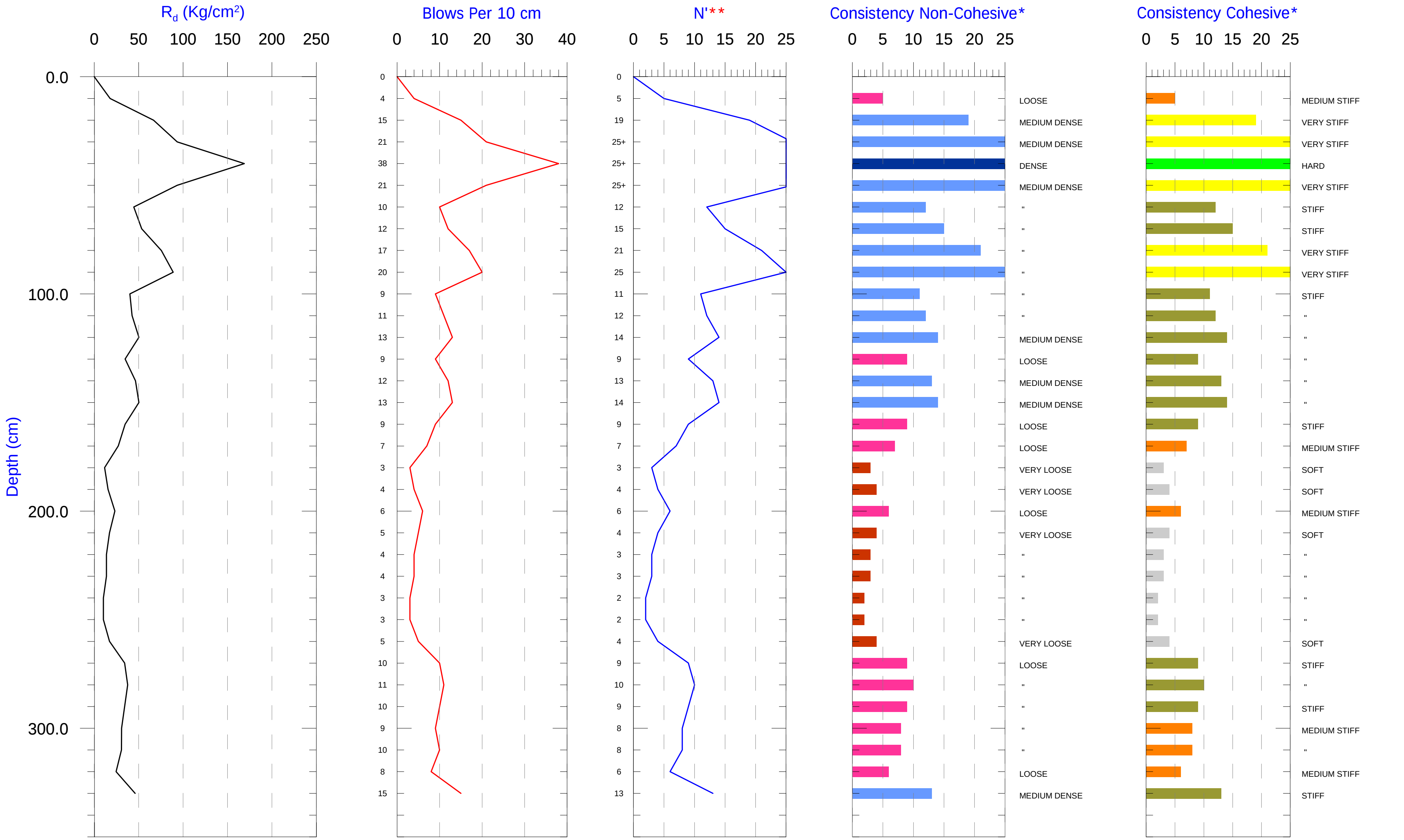
Wildcat Cone Penetration

Job No: 112-3155
Date: 12-Jan-15
Site: 385 Gower Pt. Road
Gibsons, BC

Operator: CC/GC
Test Hole: WC15-3
Max Depth (cm): 330
Coordinates: 49° 23.986'N 123° 30.426'W

Comments: Terminated at inferred loose to medium dense strata
(18.5m north of WC15-2; 30m east of large bldg at end of Winn Rd)

Data & Results



* Inferred Soil Description as per Manufacture Manual
** Wildcat does not correlate N' beyond 25+ blows.

HE Testing & Monitoring
102-173 Forester St, North Vancouver, BC

HE TESTING

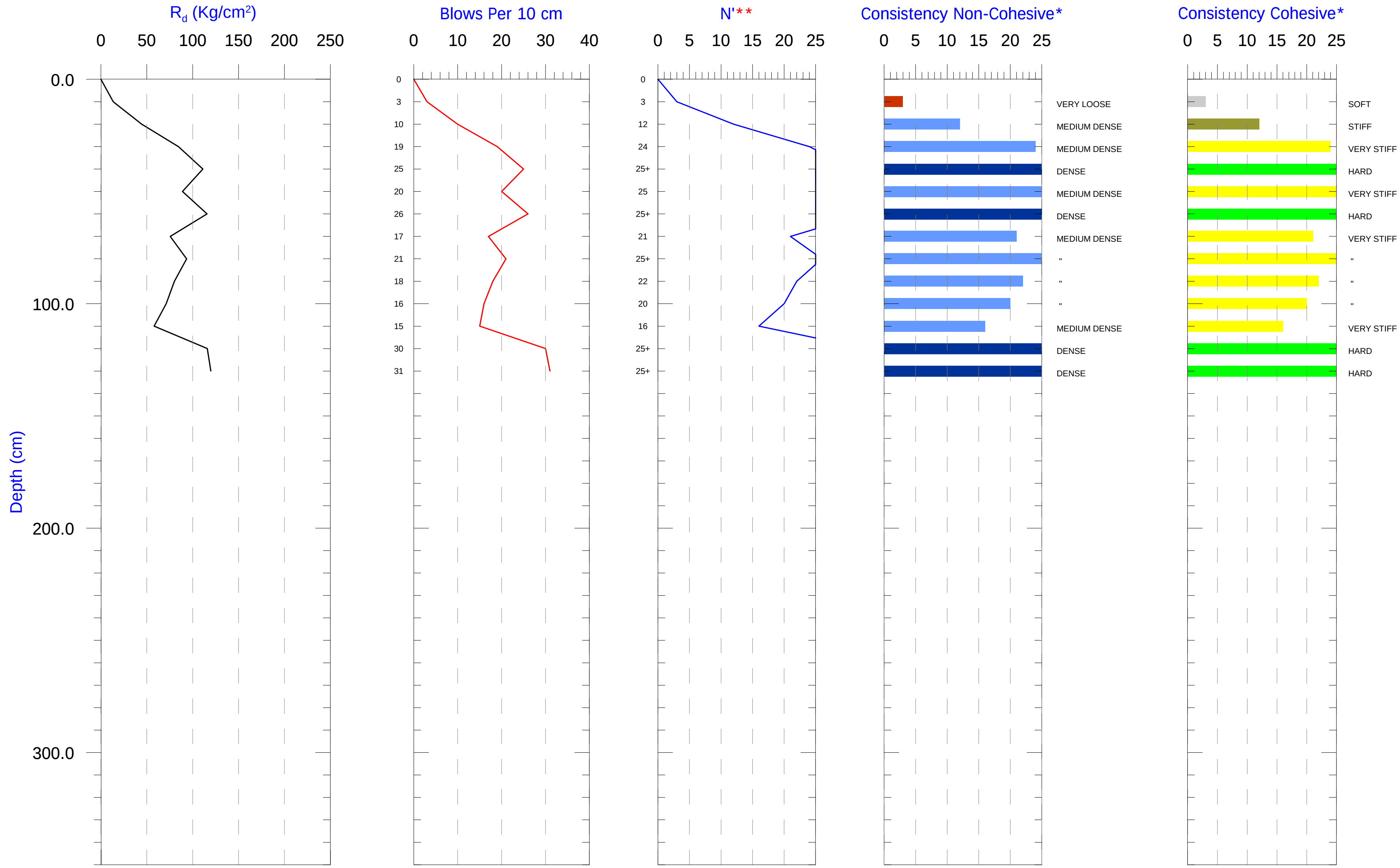
Wildcat Cone Penetration

Job No: 112-3155
Date: 12-Jan-15
Site: 385 Gower Pt. Road
Gibsons, BC

Operator: CC/GC
Test Hole: WC15-4
Max Depth (cm): 130
Coordinates: 49° 23.996'N 123° 30.429'W

Comments: Terminated at inferred medium dense to dense strata
(20m north of WC15-3; 25.5m east of entrance of dock walkway)

Data & Results



* Inferred Soil Description as per Manufacture Manual
** Wildcat does not correlate N' beyond 25+ blows.

HE TESTING

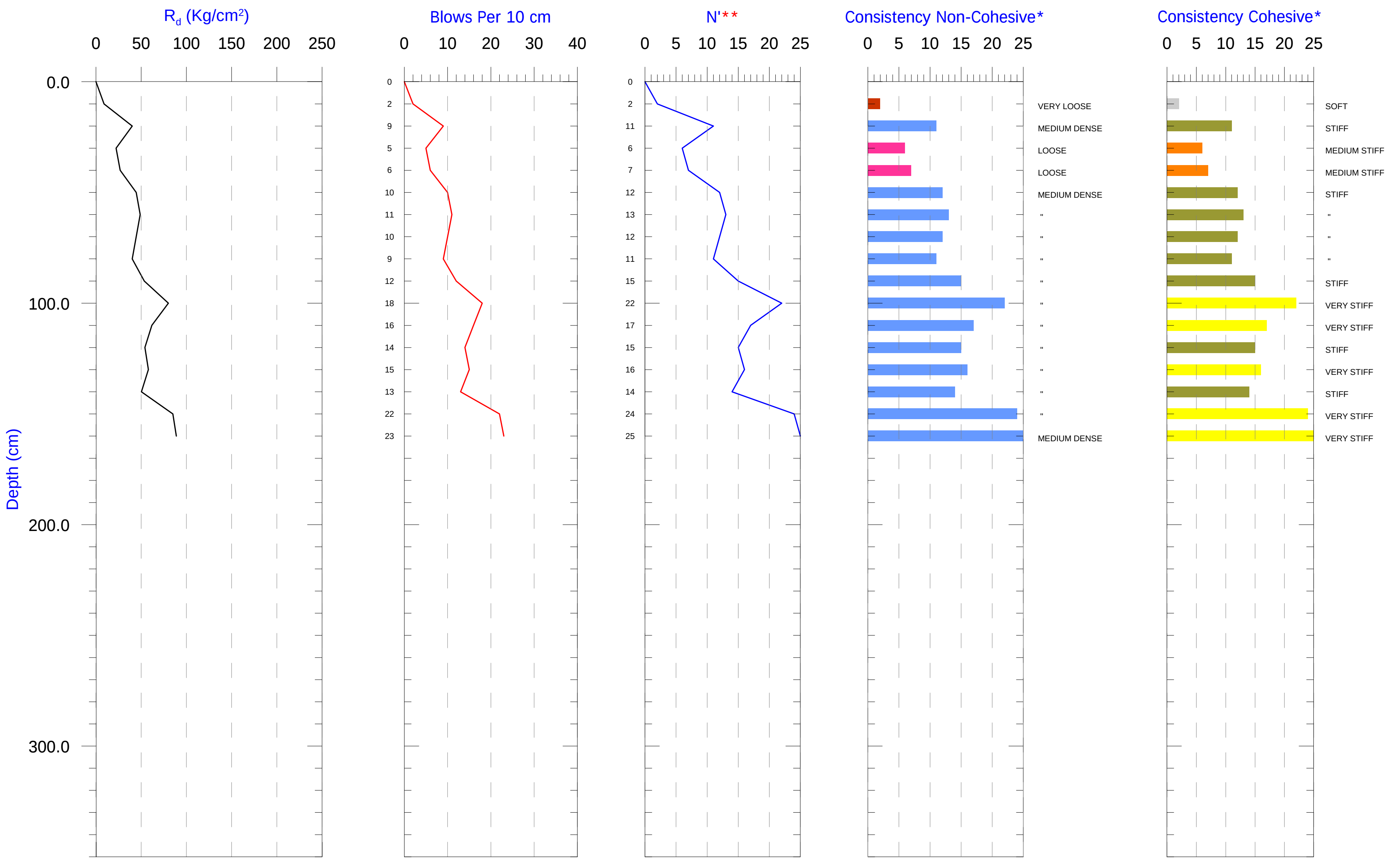
Wildcat Cone Penetration

Job No: 112-3155
Date: 12-Jan-15
Site: 385 Gower Pt. Road
Gibsons, BC

Operator: CC/GC
Test Hole: WC15-5
Max Depth (cm): 160
Coordinates: 49° 24.008'N 123° 30.416'W

Comments: Terminated within medium dense strata
(20m south of WC15-6 and 23m north of park walkway wall)

Data & Results



* Inferred Soil Description as per Manufacture Manual
** Wildcat does not correlate N' beyond 25+ blows.

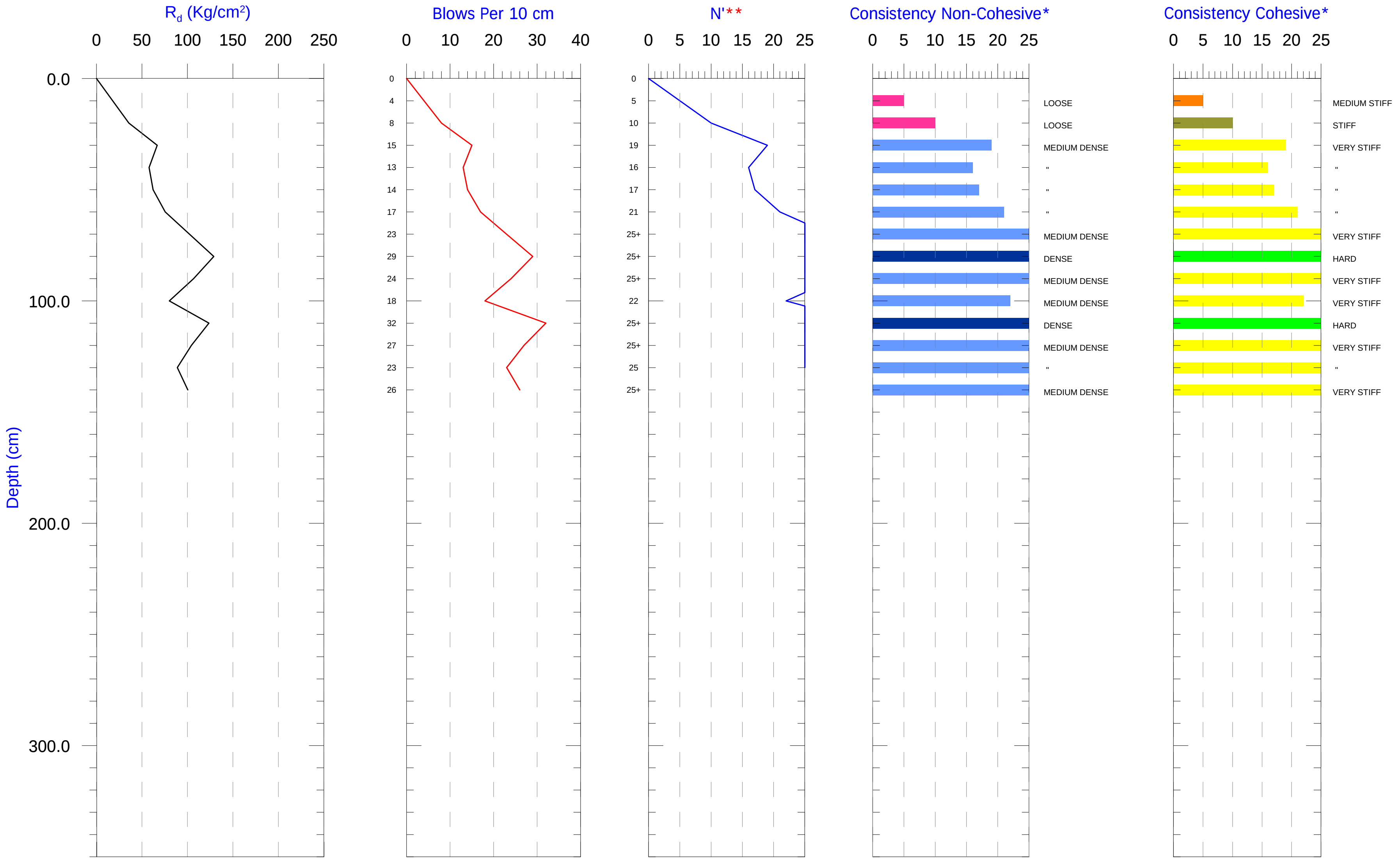
Wildcat Cone Penetration

Job No: 112-3155
Date: 12-Jan-15
Site: 385 Gower Pt. Road
Gibsons, BC

Operator: CC/GC
Test Hole: WC15-6
Max Depth (cm): 140
Coordinates: 49° 24.023'N 123° 30.399'W

Comments: Terminated within inferred medium dense strata (20m south of WC15-7 and 21.5m east of park walkway wall)

Data & Results



* Inferred Soil Description as per Manufacture Manual
** Wildcat does not correlate N' beyond 25+ blows.

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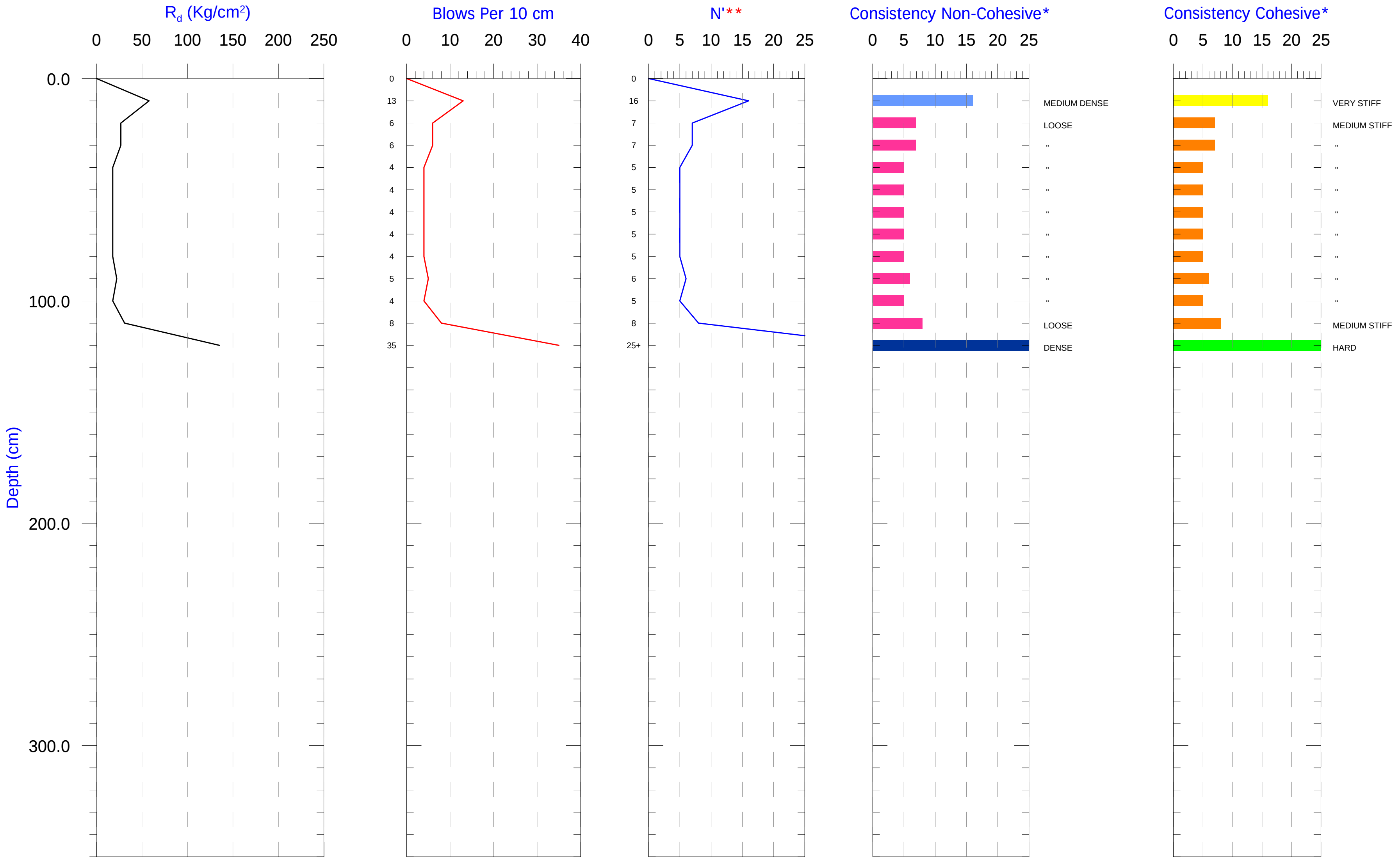
Wildcat Cone Penetration

Job No: 112-3155
Date: 12-Jan-15
Site: 385 Gower Pt. Road
Gibsons, BC

Operator: CC/GC
Test Hole: WC15-7
Max Depth (cm): 140
Coordinates: 49° 24.028'N 123° 30.388'W

Comments: Terminated at dense strata
(11.5m south of east dock gangway; 16.5m east of park walkway wall)

Data & Results



* Inferred Soil Description as per Manufacture Manual
** Wildcat does not correlate N' beyond 25+ blows.

HE Testing & Monitoring
102-173 Forester St, North Vancouver, BC

APPENDIX C

BACKGROUND INFORMATION

BOREHOLE RECORD

BH04-07

CLIENT: KLAUS FUERNISS ENTERPRISES — PROJECT No. KF - 01
PROJECT: KLAUS FUERNISS ENTERPRISES — DATUM: NAD — NORTHING:
LOCATION: 377 Gower Point Road, Gibsons, BC — ELEVATION: — EASTING:
DRILLING DATE: Mar. 4, 2004 — DRILLING CO.: Mud Bay Drilling — DRILLING METHOD: Solid Stem Auger

Legend: ☐ Insitu Shear Vane (kPa) ■ Remoulded Shear Vane (kPa) △ Pocket Penetrometer (kPa) — 50kPa 100kPa 150kPa 200kPa
W_P W W_L Moisture Content & Atterberg Limits
● Standard Penetration Test, blows/0.3m — 10 20 30 40 50 60 70 80 90

DEPTH (m)	USC	SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	MOISTURE CONTENT (%)	DEPTH (ft)
0	TP	Sod and dark brown topsoil				0
	PT	Dark brown peat, fibrous, large roots				2
1	SM	Yellow brown silty fine SAND; trace gravel; medium dense; wet	GS	7-1	21	4
2			GS	7-2J		6
	SP	Grey medium SAND, some silt; trace gravel; compact; wet				8
3	SM / ML	Yellow grey sandy SILT; trace gravel; soft; wet				10
		grades to stiff at 12 feet				12
4	ML	grades to soft and plastic at 14 feet				14
5						16
	TL	Grey silty sand with gravel and cobbles (GLACIAL TILL); very dense; moist				18
6						20
7						22
	TL	interbedded with lenses of fine sand and silt				24

SPT values plotted (blows/0.3m, approx.): 72, 40, 3, 26, 38, 50, 73, >>90, >>104, >>150

Sample Type: GS - Grab Sample SPT - Standard Penetration Test ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite, Pea Gravel, Drill Cuttings, Sand

Logged by: MME
Reviewed by: MME
Date: Mar. 4 and 5, 2004

ENGINEERING LTD.
RR 22, 1209 Roberts Creek Road
Roberts Creek, B.C.
V0N 2W2

BOREHOLE RECORD

BH04-07 cont'd

CLIENT	KLAUS FUERNISS ENTERPRISES	PROJECT No.	KF - 01
PROJECT	KLAUS FUERNISS ENTERPRISES	DATUM	NAD
		NORTHING	
LOCATION	377 Gower Point Road, Gibsons, BC	ELEVATION	
		EASTING	
DRILLING DATE	Mar. 4, 2004	DRILLING CO.	Mud Bay Drilling
DRILLING METHOD	Solid Stem Auger		

DEPTH (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLES TYPE	SAMPLES NUMBER	MOISTURE CONTENT (%)	☐ Insitu Shear Vane (kPa) ◻ Remoulded Shear Vane (kPa) △ Pocket Penetrometer (kPa) 50kPa 100kPa 150kPa 200kPa; Wp W WL Moisture Content & Atterberg Limits; ● Standard Penetration Test, blows/0.3m 10 20 30 40 50 60 70 80 90	DEPTH (ft)
8								26
								28
9								30
			End of Hole at 30 feet					
								32
10								34
11								36
								38
12								4(
13								4:
								4·
14								4·
								4
15								5

Sample Type: GS - Grab Sample SPT - Standard Penetration Test
ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite, Pea Gravel, Drill Cuttings, Sand

Logged by: MME
Reviewed by: MME
Date: Mar. 4 and 5, 2004

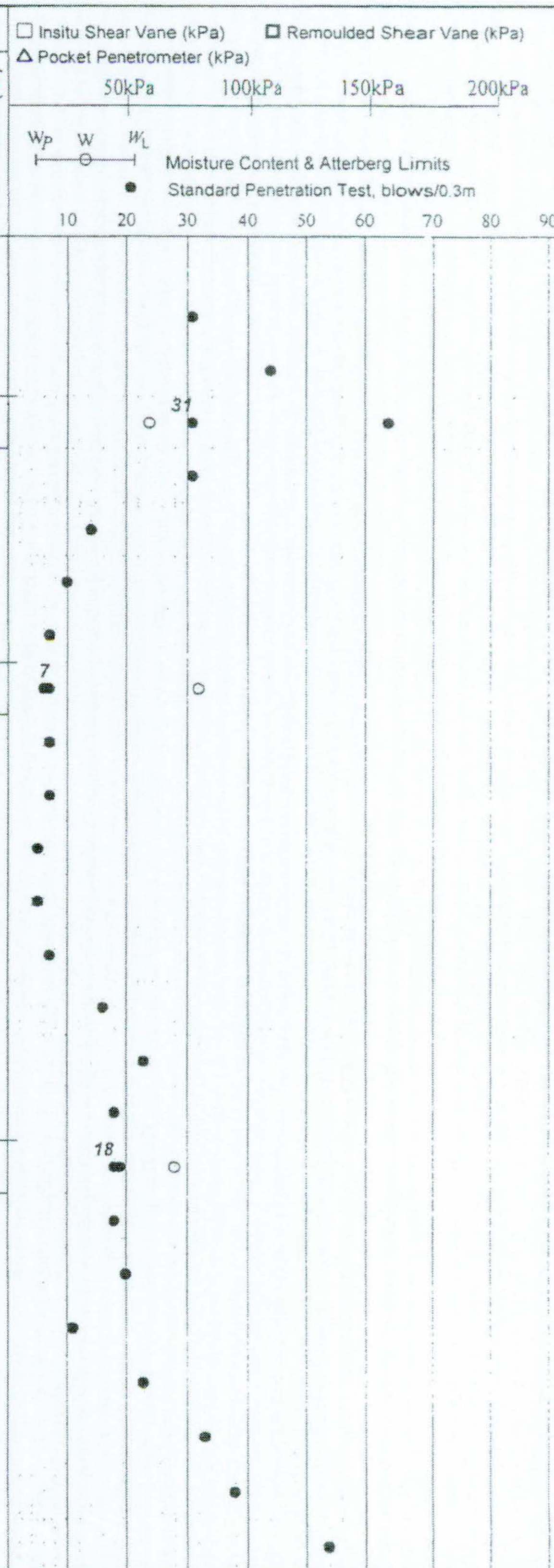
ENGINEERING LTD.
RR 22, 1209 Roberts Creek Road
Roberts Creek, B.C.
V0N 2W2

BOREHOLE RECORD

BH04-08

CLIENT	KLAUS FUERNISS ENTERPRISES	PROJECT No.	KF - 01
PROJECT	KLAUS FUERNISS ENTERPRISES	DATUM	NAD
		NORTHING	
LOCATION	377 Gower Point Road, Gibsons, BC	ELEVATION	
		EASTING	
DRILLING DATE	Mar. 4, 2004	DRILLING CO.	Mud Bay Drilling
DRILLING METHOD	Solid Stem Auger		

DEPTH (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLES TYPE	NUMBER	MOISTURE CONTENT (%)	DEPTH (ft)
0	FL		Brown silt, sand and gravel FILL; compact; moist	GS	8-1	24	0
1.4	FL		Brown silt, trace sand and gravel (FILL); stiff; grades to wet.	AS	8-2J	32	
3.2	PT		Dark brown PEAT, fibrous; compressed				
3.8	SM		Yellow brown silty SAND, trace gravel, inclusions of organic; loose to compact; wet				
4.4	ML		Grey fine sandy SILT, trace gravel; stiff (TV=0.3 tsf); wet	AS	8-3J	28	
6.7			Grey silty sand with gravel and cobbles (GLACIAL TILL); dense to very dense; moist				



Depth (m): 0 1 2 3 4 5 6 7
Depth (ft): 0 2 4 6 8 10 12 14 16 18 20 22 24

Sample Type: GS - Grab Sample SPT - Standard Penetration Test ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite, Pea Gravel, Drill Cuttings, Sand

Logged by: MME
Reviewed by: MME
Date: Mar. 4 and 5, 2004

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RR 22, 1209 Roberts Creek Road
Roberts Creek, B.C.
V0N 2W2

BOREHOLE RECORD

BH04-08 cont'd

CLIENT: KLAUS FUERNISS ENTERPRISES
PROJECT No.: KF - 01
PROJECT: KLAUS FUERNISS ENTERPRISES
DATUM: NAD
NORTHING:
LOCATION: 377 Gower Point Road, Gibsons, BC
ELEVATION:
EASTING:
DRILLING DATE: Mar. 4, 2004
DRILLING CO.: Mud Bay Drilling
DRILLING METHOD: Solid Stem Auger

☐ Insitu Shear Vane (kPa) ☐ Remoulded Shear Vane (kPa)
△ Pocket Penetrometer (kPa)
50kPa 100kPa 150kPa 200kPa
Wp W WL — Moisture Content & Atterberg Limits
● Standard Penetration Test, blows/0.3m
10 20 30 40 50 60 70 80 90

DEPTH (m)	USC	SOIL DESCRIPTION	SAMPLES TYPE	NUMBER	MOISTURE CONTENT (%)	DEPTH (ft)
8	TL					26
9						28, 30
10	TL	Grey medium sand with silt, gravel and cobbles (GLACIAL TILL); very dense (very hard drilling)				32, 34
11	TL					36
11.7			AS	8-4J	10	38
12		End of hole at 40 feet.				40
13						42, 44
14						46
15						48, 50

SPT: >>100

Sample Type: GS - Grab Sample SPT - Standard Penetration Test ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite, Pea Gravel, Drill Cuttings, Sand

Logged by: MME
Reviewed by: MME
Date: Mar. 4 and 5, 2004

ENGINEERING LTD.
RR 22, 1209 Roberts Creek Road
Roberts Creek, B.C.
V0N 2W2

BOREHOLE RECORD

BH04-09

CLIENT: KLAUS FUERNISS ENTERPRISES — PROJECT No. KF - 01
PROJECT: KLAUS FUERNISS ENTERPRISES — DATUM: NAD — NORTHING:
LOCATION: 377 Gower Point Road, Gibsons, BC — ELEVATION: — EASTING:
DRILLING DATE: Mar. 5, 2004 — DRILLING CO. Mud Bay Drilling — DRILLING METHOD: Solid Stem Auger

□ Insitu Shear Vane (kPa) ■ Remoulded Shear Vane (kPa)
△ Pocket Penetrometer (kPa)
50kPa 100kPa 150kPa 200kPa

W_P W W_L Moisture Content & Atterberg Limits
● Standard Penetration Test, blows/0.3m

10 20 30 40 50 60 70 80 90

DEPTH (m)	USC	SOIL DESCRIPTION	TYPE	NUMBER	MOISTURE CONTENT (%)
0	FL	Brown silty sand and gravel fill, boulders at 3.5 feet grades siltier with depth			
~2.1			AS	9-1	15
~2.6	PT	Dark brown PEAT, fibrous, compressed			
~3.2	ML	Yellow brown sandy SILT, trace clay and gravel; soft; wet			
~3.8			AS	9-2J	25
~4.3	ML	Yellow grey fine sandy SILT, trace gravel; firm; moist			
~5.5	TL	Grey silty sand, gravel and cobbles (GLACIAL TILL); dense; moist	AS	9-3J	23
~6.1		Grey silty sand, gravel and boulders; very dense (hard drilling); moist grades with lenses of fine sand and silt			

Sample Type: GS - Grab Sample SPT - Standard Penetration Test ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite, Pea Gravel, Drill Cuttings, Sand

Logged by: MME
Reviewed by: MME
Date: Mar. 4 and 5, 2004

ENGINEERING LTD.
RR 22, 1209 Roberts Creek Road
Roberts Creek, B.C.
V0N 2W2

BOREHOLE RECORD

BH04-09 cont'd

CLIENT: KLAUS FUERNISS ENTERPRISES — PROJECT No. KF - 01

PROJECT: KLAUS FUERNISS ENTERPRISES — DATUM: NAD — NORTHING:

LOCATION: 377 Gower Point Road, Gibsons, BC — ELEVATION: — EASTING:

DRILLING DATE: Mar. 5, 2004 — DRILLING CO.: Mud Bay Drilling — DRILLING METHOD: Solid Stem Auger

☐ Insitu Shear Vane (kPa) ■ Remoulded Shear Vane (kPa) △ Pocket Penetrometer (kPa)

50kPa 100kPa 150kPa 200kPa

W_P W W_L Moisture Content & Atterberg Limits

● Standard Penetration Test, blows/0.3m

10 20 30 40 50 60 70 80 90

DEPTH (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLES TYPE	SAMPLES NUMBER	MOISTURE CONTENT (%)	DEPTH (ft)
8–10.7	TL						26–35
10.7			End of hole at 35 feet.				35

Depth (m) scale: 8, 9, 10, 11, 12, 13, 14, 15

Depth (ft) scale: 26, 28, 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50

Sample Type: GS - Grab Sample SPT - Standard Penetration Test ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite Pea Gravel Drill Cuttings Sand

Logged by: MME
Reviewed by: MME
Date: Mar. 4 and 5, 2004

ENGINEERING LTD.
RR 22, 1209 Roberts Creek Road
Roberts Creek, B.C.
V0N 2W2

BRITISH COLUMBIA

Report 1 - Detailed Well Record

Well Tag Number: 19896

Owner: VILLAGE OF GIBSONS

Address:

Area:

WELL LOCATION:
NEW WESTMINSTER Land District
District Lot: Plan: Lot:
Township: Section: Range:
Indian Reserve: Meridian: Block:
Quarter:
Island:
BCGS Number (NAD 83): 092G043122 Well: 1

Class of Well:
Subclass of Well:
Orientation of Well:
Status of Well: New
Well Use: Unknown Well Use
Observation Well Number:
Observation Well Status:
Construction Method: Drilled
Diameter: 10.0 inches
Casing drive shoe:
Well Depth: 138 feet
Elevation: 0 feet (ASL)
Final Casing Stick Up: inches
Well Cap Type:
Bedrock Depth: feet
Lithology Info Flag: N
File Info Flag: N
Sieve Info Flag: N
Screen Info Flag: N

Site Info Details:
Other Info Flag:
Other Info Details:

Construction Date: 1966-04-01 00:00:00.0

Driller: Rural Well Drillers
Well Identification Plate Number:
Plate Attached By:
Where Plate Attached:

PRODUCTION DATA AT TIME OF DRILLING:
Well Yield: 40 (Driller's Estimate) Gallons per Minute (U.S./Imperial)
Development Method:
Pump Test Info Flag: N
Artesian Flow: .01 U.S. Gallons per Minute
Artesian Pressure (ft):
Static Level:

WATER QUALITY:
Character:
Colour:
Odour:
Well Disinfected: N
EMS ID:
Water Chemistry Info Flag: Y
Field Chemistry Info Flag:
Site Info (SEAM): N

Water Utility: N
Water Supply System Name:
Water Supply System Well Name:

SURFACE SEAL:
Flag: N
Material:
Method:
Depth (ft):
Thickness (in):

WELL CLOSURE INFORMATION:
Reason For Closure:
Method of Closure:
Closure Sealant Material:
Closure Backfill Material:
Details of Closure:

Screen from	to feet	Type	Slot Size	
Casing from	to feet	Diameter	Material	Drive Shoe

GENERAL REMARKS:
PUMPED AT 110US GPM, FOR 2 HRS. WATER LEVEL STABLIZED AT 40 FT. AFTER .5 HR. WELL FLOWED AGAIN IN 2 MIN 40 SECS. AFTER STOPPING PUMP

LITHOLOGY INFORMATION:
From 0 to 2 Ft. Fill
From 2 to 4 Ft. Soft organic top soil
From 4 to 8 Ft. Cobbles interfilled with silty fine sand

```
From     8 to    21 Ft.   Boulders - interspaced with compact
From     0 to     0 Ft.   sandy silt - few isolated layers of
From     0 to     0 Ft.   gravel and sand
From    21 to    26 Ft.   Sandy gravel medium to coarse, few
From     0 to     0 Ft.   isolated layers of silt (3"-6" thick),
From     0 to     0 Ft.   W.B. flowing
From    26 to    33 Ft.   Sandy gravel (medium to coarse) W.B.
From    33 to    39 Ft.   Coarse sand and gravel. 2" to 4" and
From     0 to     0 Ft.   larger W.B.
From    39 to    42 Ft.   Coarse sand and fine gravel mostly sand
From     0 to     0 Ft.   (W.B.)
From    42 to    64 Ft.   Sand, medium to coarse with occasional
From     0 to     0 Ft.   seams of gravel
From    64 to    76 Ft.   Medium to coarse sand (W.B.)
From    76 to    83 Ft.   Coarse sand some fine gravel
From    83 to    96 Ft.   Silty sand fine to medium compact
From    96 to    98 Ft.   Sand fine to medium (W.B.)
From    98 to   101 Ft.   Compact silty sand
From   101 to   108 Ft.   Compact silt with peat stringers
From   108 to 110.5 Ft.   Sand fine to med., some silt
From 110.5 to   113 Ft.   Compact silty sand
From   113 to   115 Ft.   Silty sand medium
From   115 to   117 Ft.   Light grey fine silty sand
From   117 to   132 Ft.   Fine grey sand (very little silt) W.B.
From   132 to   138 Ft.   Fine grey sand with a little silt
```


- Return to Main
- Return to Search Options
- Return to Search Criteria

TOPOGRAPHIC PLAN OF LOTS 1 AND 2, BLOCK A, DL 686, PLAN 14197, AND LOTS 1 AND A, DL 685, PLAN 5579

SCALE 1 : 300

NOTE

THE TEST WELL LOCATIONS SHOWN ON THIS PLAN ARE THE ONLY FEATURES THAT ARE CONFIRMED AS OF JANUARY 28, 2015. ALL OTHER INFORMATION SHOWN IS FROM PREVIOUS SURVEYS AND MAY NOT BE CURRENT.

TEST WELL LOCATIONS

AH14-3	N	2030.19	E	3091.29	El.	10.17
BH14-4	N	2030.58	E	3103.34	El.	10.09
DCPT121	N	2037.62	E	3119.99	El.	6.82
TP12-1	N	2033.85	E	3119.56	El.	6.22
AH12-1	N	2033.94	E	3134.23	El.	4.92
TP12-2	N	2036.54	E	3141.43	El.	4.52
well lid (otheres)	N	2032.48	E	3136.29	El.	4.60
AH14-1	N	2049.76	E	3111.66	El.	9.83
BH04-8	N	2023.24	E	3099.22	El.	10.08
AH14-2	N	2026.70	E	3101.24	El.	10.11
BH14-6	N	1970.22	E	3077.29	El.	7.22
AH14-4	N	1969.45	E	3077.46	El.	7.15
AH14-5	N	2006.74	E	3114.49	El.	5.32
BH14-3	N	2007.18	E	3141.25	El.	3.18
AH12-3	N	1990.96	E	3136.35	El.	3.14
BH14-5	N	1977.95	E	3129.78	El.	3.04
AH14-6	N	2007.82	E	3160.28	El.	2.54
AH12-2	N	2027.36	E	3169.03	El.	2.71

Note

Geodetic elevation of the high tide line determined from tide charts and field observation on Feb. 23, 2005 is 2.15 metres.
High tide used from tables = 5.1 metres (16.7 feet).
Bench marks set were hub numbers 4853, 4854 and BM as shown.

LEGEND

Elevations are relative and datum is referred to Gibsons Town Datum control monument 94H1221 = 11.54m

- Spot Elevation
- Lamp Post
- Fire Hydrant
- Tree
- HP Hydro Pole
- Catch Basin
- san. MH Sanitary/Storm Manhole
- AH14-1 Monitor well location

NOTE:
Diameters of pipes shown were determined from data from the Town of Gibsons and from field inspection which was hampered by high volumes. A furthur inspection is required for any costruction design.

Revised Jan 28, 2015.

Revised Aug 26, 2013.

OCTOBER 5, 2012

504T - 1504

Larry W. Penonzek
B. C. Land Surveyor
Box 505 Gibsons, B.C.
VON 1V0 Fx. 886-2553
Phone 604-886-2531

SAN MH
LID EL. 6.07
Invert El. 4.62

APPENDIX D

LABORATORY TESTING RESULTS

HE TESTING & MONITORING LTD.

102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Labortory Liquid Limit, Plastic Limit and Plasticity Index of Soil

Reference Number: 112-3155

Client

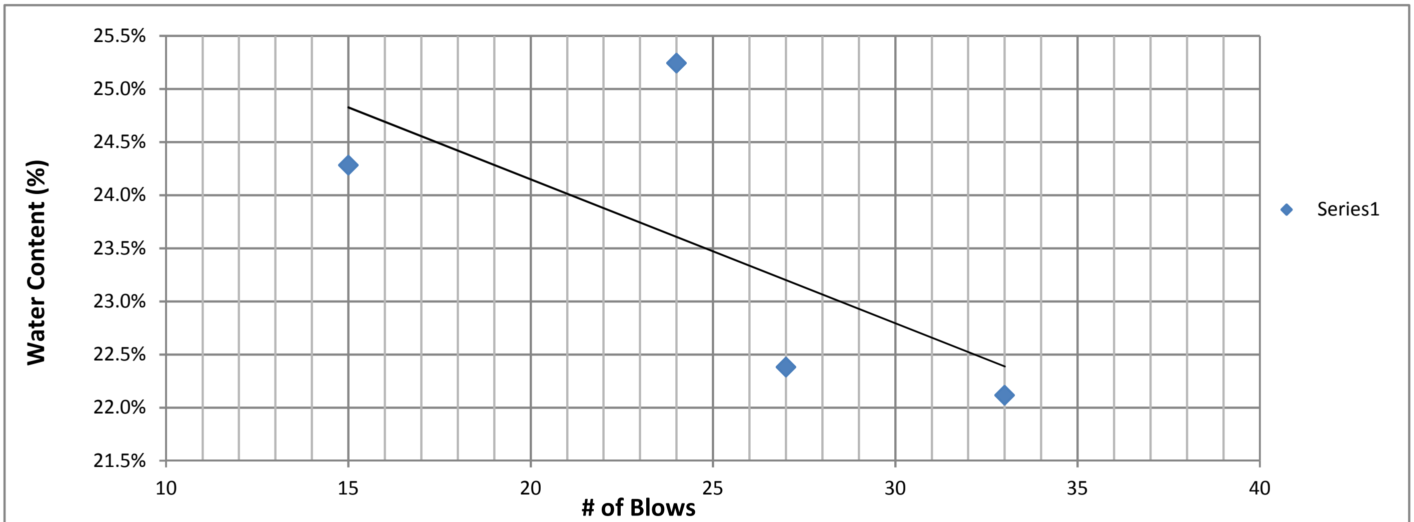
Horizon Engineering Inc.
114-2433 Dollarton Hwy
North Vancouver, BC
V7H 0A1

Location: 385 Gower Pt. Rd., Gibsons, BC
Project: 112-3155
Collected by: Pamela Bayntun
Sample Date: 15-Apr-14
Receive Date: 20-May-14
Sample ID AH14-6 - G2

Data & Results

Technician: CC
Date Tested: May-20-14

Data Point	Container No.	No. of Blows	Container + Wet Soil (g)	Container + Dry Soil (g)	Wt of Container (g)	Wet Soil (g)	Dry Soil (g)	Water Content (%)
1	M103	33	10.30	9.13	3.84	6.46	5.29	22.12%
2	M102	27	11.49	10.08	3.78	7.71	6.30	22.38%
3	M101	24	10.26	8.96	3.81	6.45	5.15	25.24%
4	M104	15	9.46	8.36	3.83	5.63	4.53	24.28%
5								
6								



Plastic Limit (PL)	17.73%
Liquid Limit (LL)	23.48%

Plastic Index = LL-Pl	5.75%

Comments low plasticity

Tested By DW

102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca

HE TESTING & MONITORING LTD.

Laboratory Determination of Water Content of Soil and Rock by Mass

Reference Number: 112-3045

Client

Horizon Engineering Inc.
114-2433 Dollarton Hwy
North Vancouver, BC
V7H 0A1

Location: **381 Gower Pt. Rd., Gibsons, BC**
Project: 112-3155
Collected by: Pamela Bayntun
Sample Date: April-15-14
Receive Date: May-14-14

Test Method

Test Type: ASTM D2216
Number of Samples: 5

Data & Results

Technician: Clive
Date Tested: May 16-20, 2014

Sample #	Test #	Container #	Container + Wet Soil (g)	Container + Dry Soil (g)	Weight of Water (g)	Weight of Container (g)	Dry Soil (g)	Water Content (%)
AH14-1 - G1	1	M12	206.23	112.5	93.73	8.56	103.94	**90.2**
AH14-2 - G3	1	M9	203.33	112.48	90.85	8.53	103.95	**87.4**
AH14-5 - G1	1	M18	209.57	132.48	77.09	8.94	123.54	**62.4**
AH14-6 - G2	1	M4	225.83	184.11	41.72	8.56	175.55	**23.8**
AH14-2 - G5	1	Pan	1252.3	1118.65	133.65	444.02	674.63	**19.8**

Tested By: DW

Reviewed By: CC

102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca

HE TESTING & MONITORING LTD.

Laboratory Determination of Water Content of Soil and Rock by Mass

Reference Number: 112-3155

Client

Horizon Engineering Inc.
114-2433 Dollarton Hwy
North Vancouver, BC
V7H 0A1

Location: 381 Gower Pt. Road, Gibsons, BC
Project: 112-3155
Collected by: Pamela Bayntun
Sample Date: December-10-14
Receive Date: December-17-14

Test Method

Test Type: ASTM D2216
Number of Samples: 2

Data & Results

Technician: Jason
Date Tested: December-30-14

Sample #	Test #	Container #	Container + Wet Soil (g)	Container + Dry Soil (g)	Weight of Water (g)	Weight of Container (g)	Dry Soil (g)	Water Content (%)
BH14-4 - G3	1	M12	72	24	48	3	21	**228.6**
BH14-5 - G3	1	M9	84	29	55	8	21	**261.9**

Tested By: JT

Reviewed By: CC

HE TESTING & MONITORING LTD.
102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

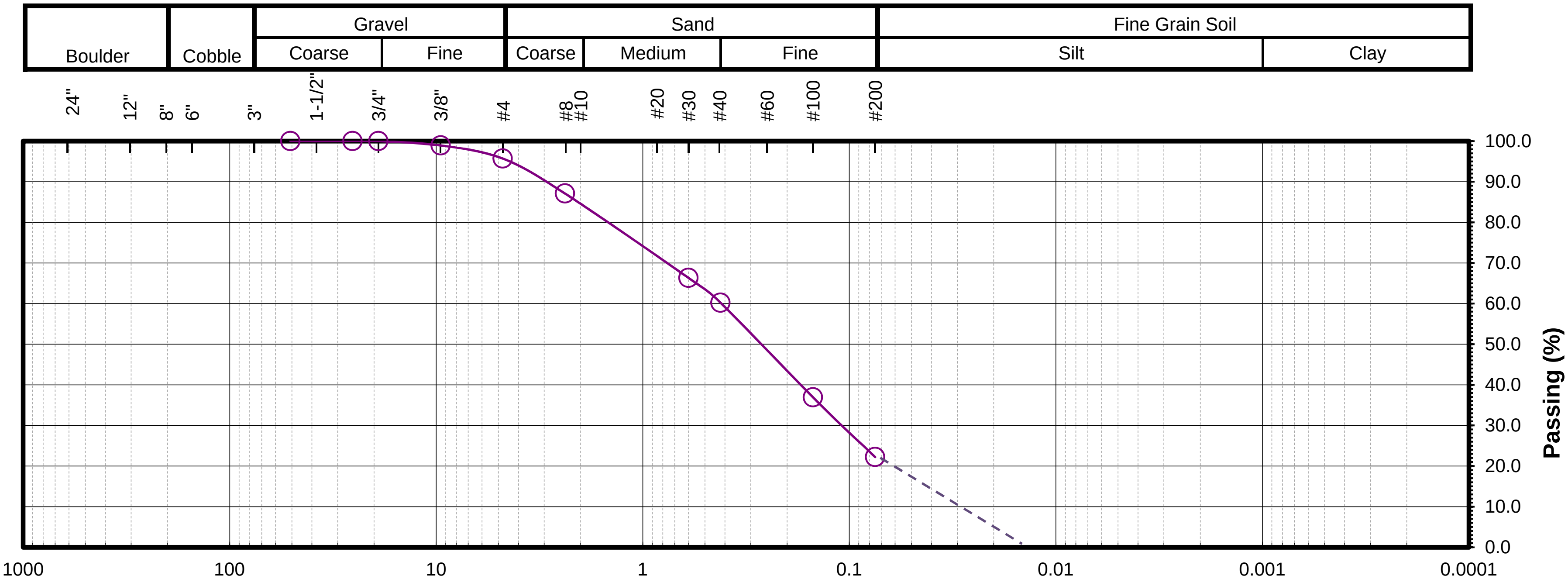
Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: AH14-1 - G2

Sample Date: 15-Apr-14
Receive Date: 14-May-14
Collected by: Pam

Data & Results

Technician: ***Clive***
Sieve Type: Dry Sieve
Date Tested: May 16-20, 2014



Coefficient of Uniformity and Curvature

C_u	C_C
13.87	0.94

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.42	0.11	0.03	3.01

Comments: **SILTY SAND, fine to medium grained, trace coarse grained sand, trace gravel (D10 value extrapolated)**

Tested By DW

102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

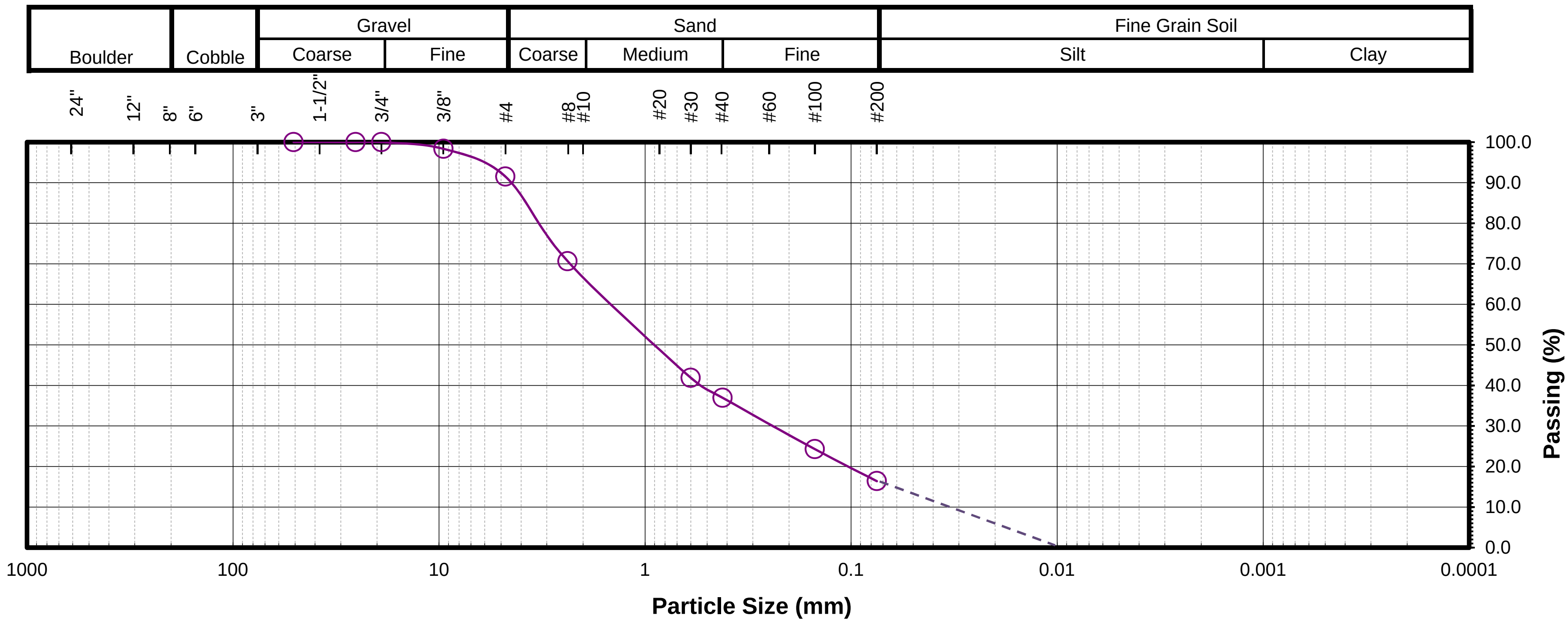
Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: AH14-2 - G5

Sample Date: 15-Apr-14
Receive Date: 14-May-14
Collected by: Pam

Data & Results

Technician: *Clive*
Sieve Type: Dry Sieve
Date Tested: May 16-20, 2014



Coefficient of Uniformity and Curvature

C_u	C_c
40.84	1.14

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
1.43	0.24	0.04	4.53

Comments: SAND, fine to coarse grained, some silt, trace gravel
(D10 value extrapolated)

Tested By DW

TESTING & MONITORING LTD.
102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

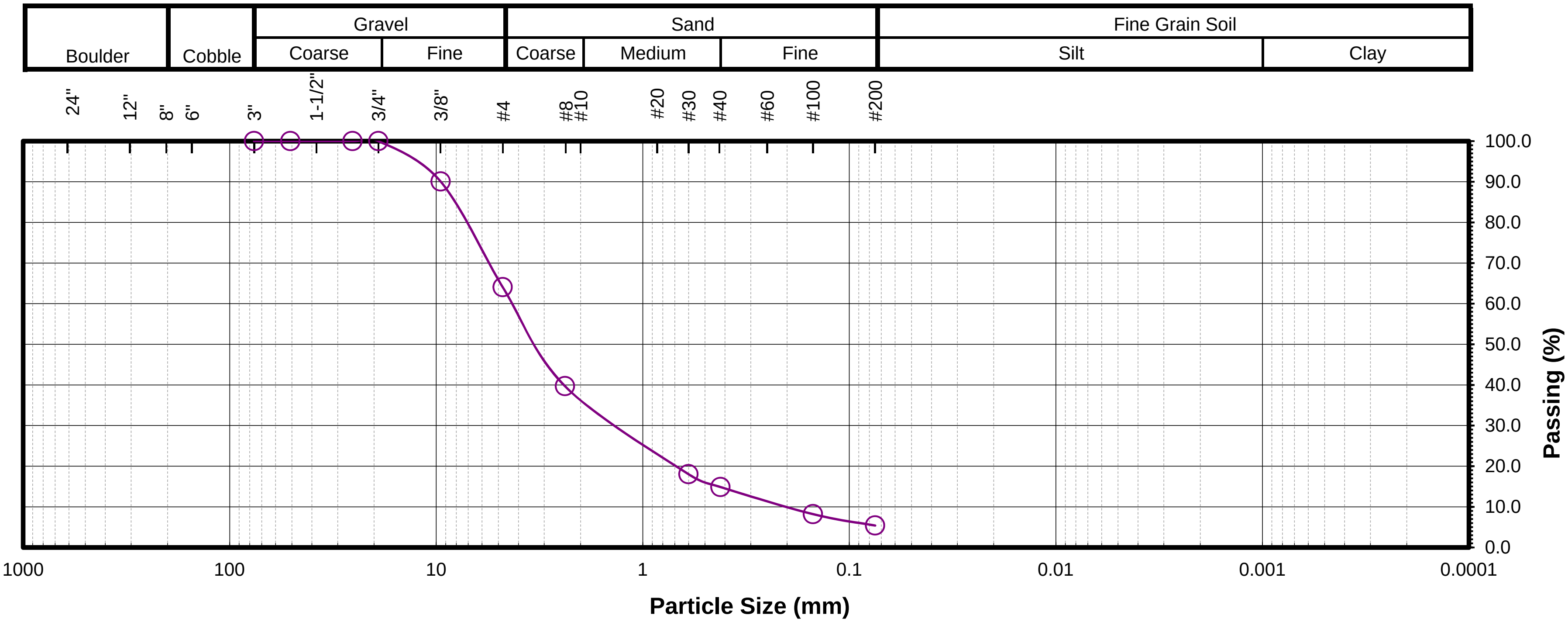
Client

Horizon Engineering	**Project:** 112-3155	**Sample Date:** 15-Apr-14
114-2433 Dollarton Hwy	**Location:** 385 Gower Point Rd., Gibsons, BC	**Receive Date:** 14-May-14
North Vancouver, BC, V7H 0A1	**Sample Label:** AH14-4 - G2	**Collected by:** Pam

Data & Results

Technician: ***Clive*** **Date Tested:** May 16-20, 2014

Sieve Type: Dry Sieve



Coefficient of Uniformity and Curvature

C_u	C_c
21.41	1.96

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
4.24	1.28	0.20	9.48

Comments: GRAVELLY SAND, medium to coarse grained, trace silt

Tested By DW

ESTING & MONITORING LTD.

102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

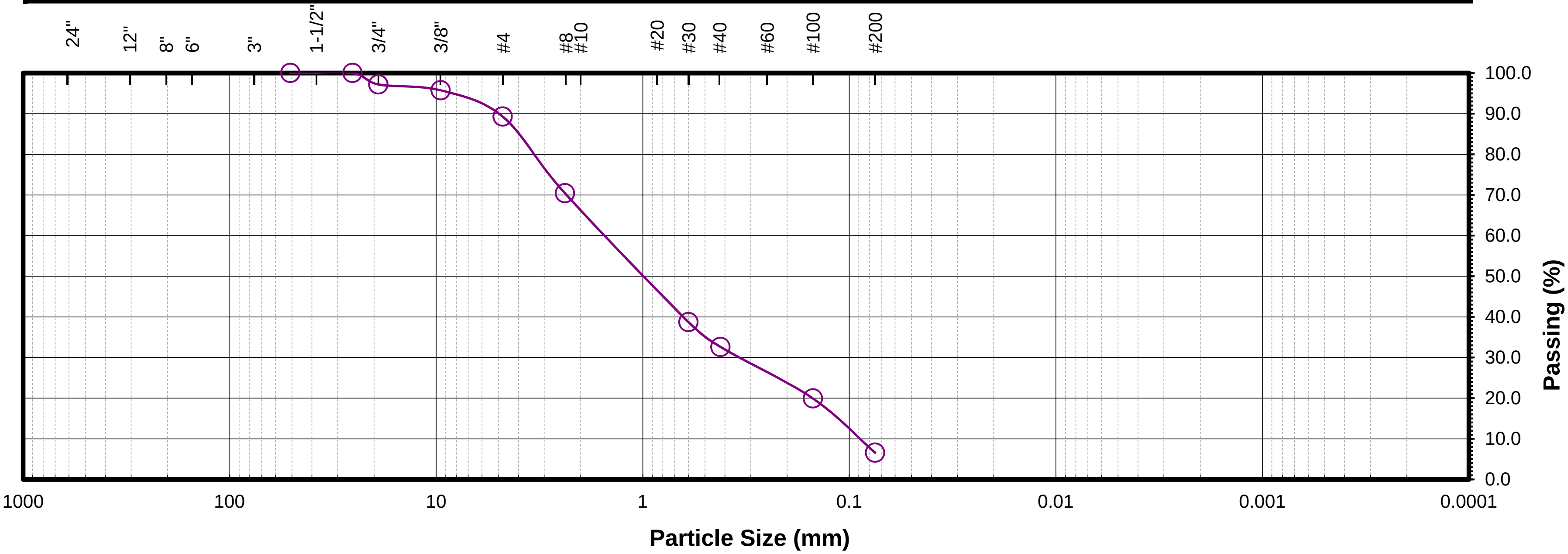
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: AH14-5 - G2

Sample Date: 15-Apr-14
Receive Date: 14-May-14
Collected by: Pam

Data & Results

Technician: *Clive*
Sieve Type: Dry Sieve
Date Tested: May 16-20, 2014

Boulder	Cobble	Gravel		Sand			Fine Grain Soil	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
16.90	0.85

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
1.51	0.34	0.09	5.15

Comments: SAND, fine to coarse grained, trace silt, trace gravel

Tested By DW

102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

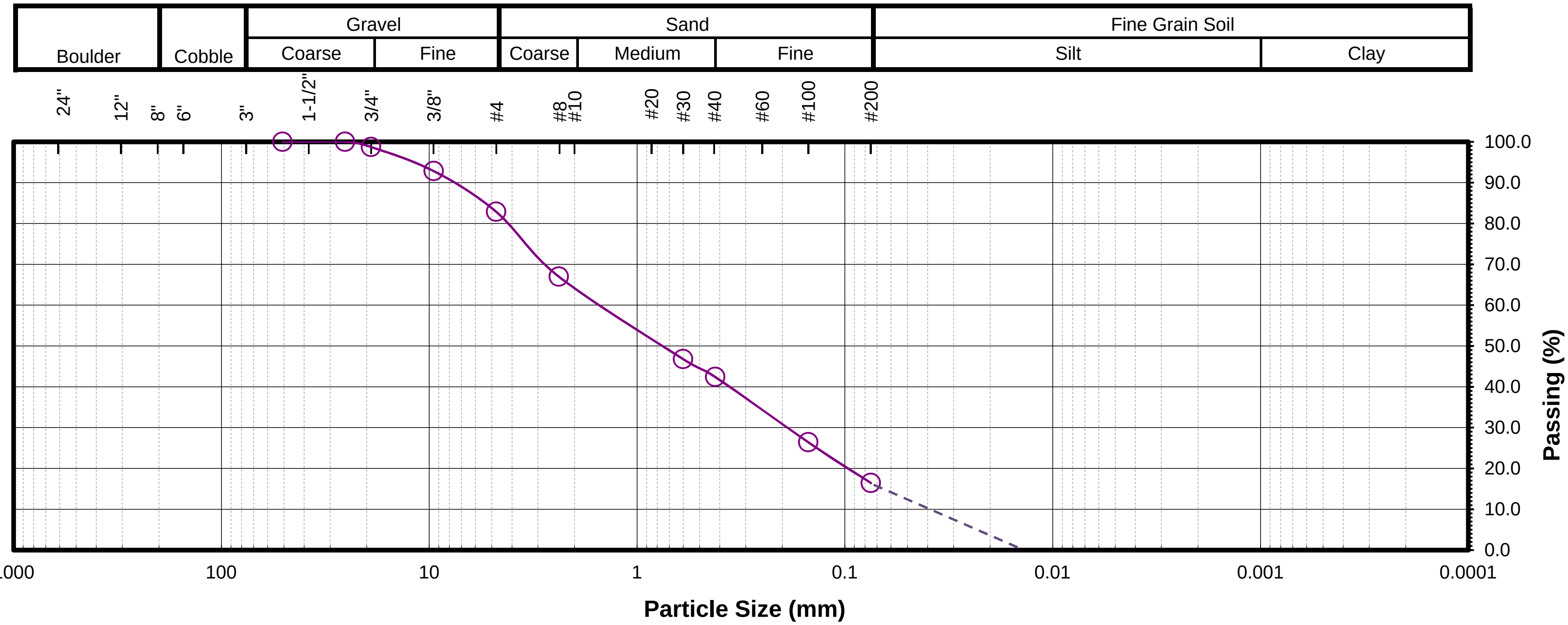
Client

Horizon Engineering	**Project:**	112-3155	**Sample Date:**	15-Apr-14
114-2433 Dollarton Hwy	**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	14-May-14
North Vancouver, BC, V7H 0A1	**Sample Label:**	AH14-6 - G1	**Collected by:**	Pam

Data & Results

Technician: ***Clive*** **Date Tested:** May 16-20, 2014

Sieve Type: Dry Sieve



Coefficient of Uniformity and Curvature

C_u	C_c
36.95	0.60

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
1.48	0.19	0.04	7.81

Comments: **SAND, fine to medium grained, some coarse grained sand, some silt, some gravel (D10 value extrapolated)**

Tested By DW

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102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

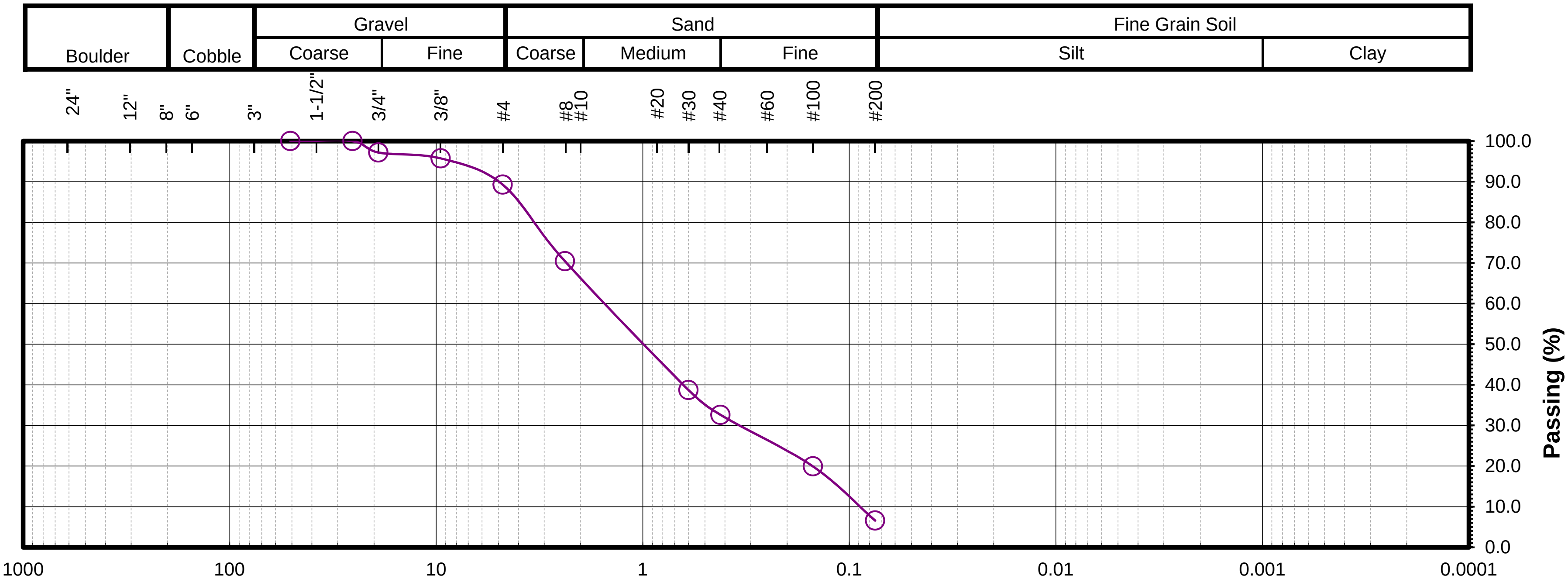
Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-1 - G8

Sample Date: 15-Apr-14
Receive Date: 14-May-14
Collected by: Pam

Data & Results

Technician: ***Clive***
Sieve Type: Dry Sieve
Date Tested: May 16-20, 2014



Coefficient of Uniformity and Curvature

C_u	C_C
16.90	0.85

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
1.51	0.34	0.09	5.15

Comments: **SAND, fine to coarse grained, trace gravel, trace silt**

Tested By DW

H-E TESTING & MONITORING LTD.

102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

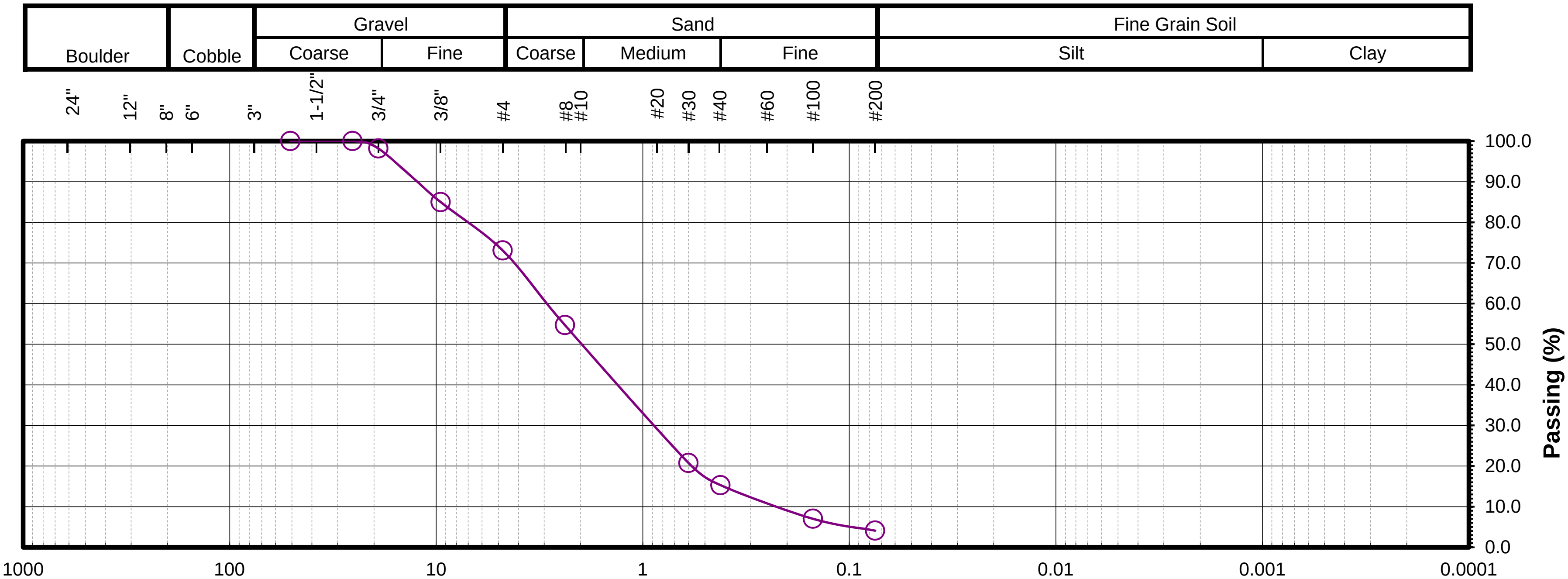
Client

Horizon Engineering	**Project:** 112-3155	**Sample Date:** 15-Apr-14
114-2433 Dollarton Hwy	**Location:** 385 Gower Point Rd., Gibsons, BC	**Receive Date:** 14-May-14
North Vancouver, BC, V7H 0A1	**Sample Label:** BH14-1 - G9	**Collected by:** Pam

Data & Results

Technician: *Clive* **Date Tested:** May 16-20, 2014

Sieve Type: Dry Sieve



Coefficient of Uniformity and Curvature

C_u	C_C
13.36	1.21

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
2.91	0.87	0.22	12.38

Comments: GRAVELLY SAND, medium grained sand, some fine grained sand, some coarse grained sand, trace silt

Tested By DW

102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

HE TESTING & MONITORING LTD.

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

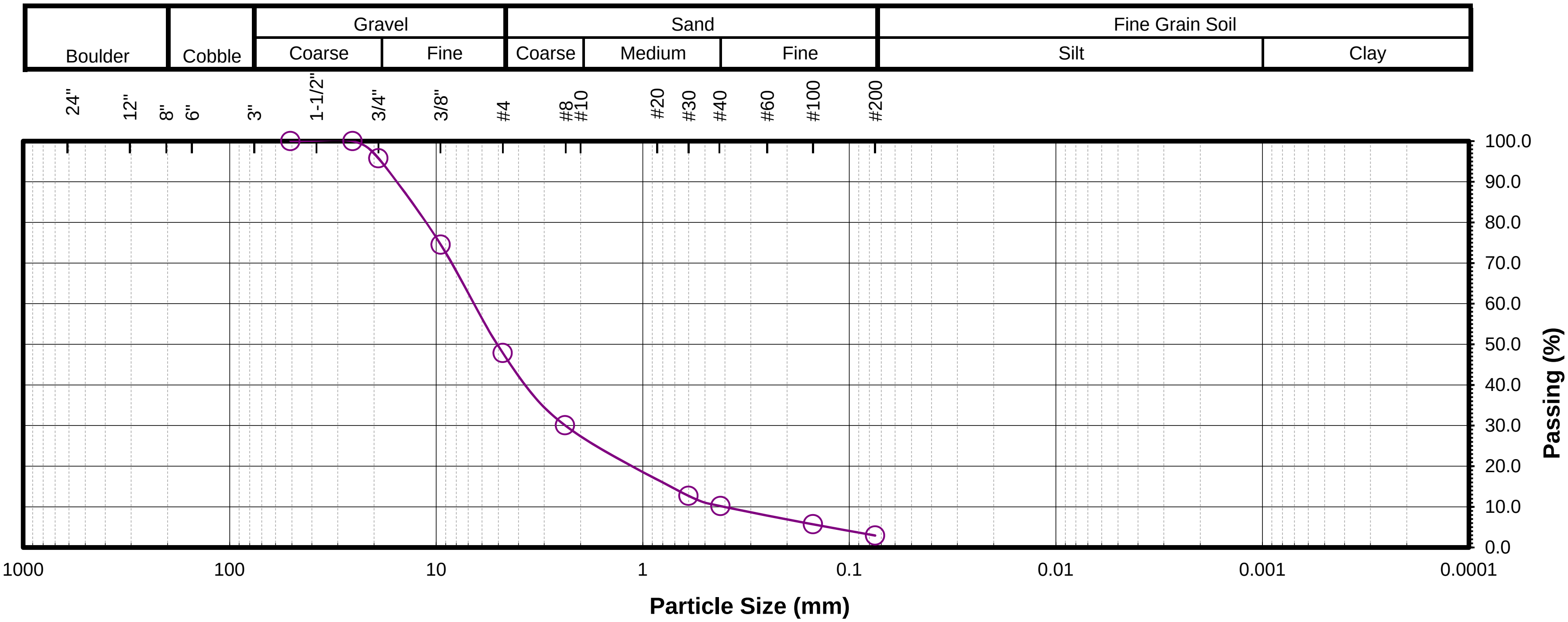
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-2 - G4

Sample Date: 15-Apr-14
Receive Date: 14-May-14
Collected by: Pam

Data & Results

Technician: *Clive*
Sieve Type: Dry Sieve

Date Tested: May 16-20, 2014



Coefficient of Uniformity and Curvature

C_u	C_c
16.23	2.14

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
6.52	2.37	0.40	15.77

Comments: SAND AND GRAVEL, medium to coarse grained sand, trace fine grained sand, trace silt

Tested By DW

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102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

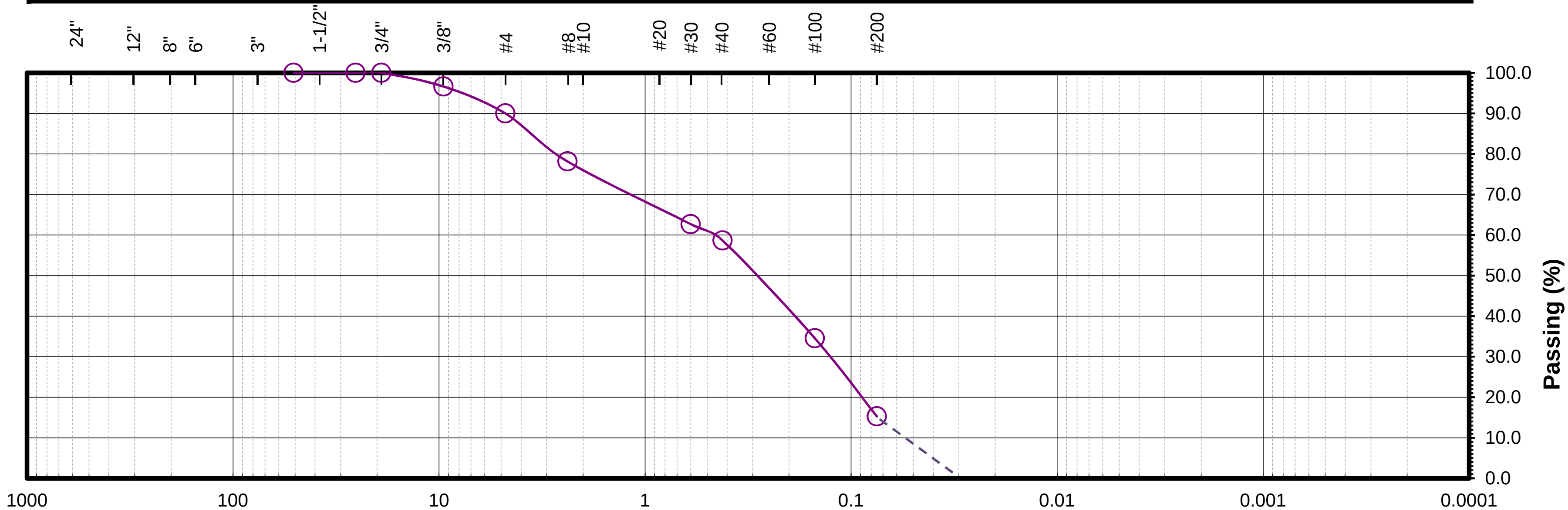
Horizon Engineering		**Project:**	112-3155	**Sample Date:**	15-Apr-14
114-2433 Dollarton Hwy		**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	14-May-14
North Vancouver, BC, V7H 0A1		**Sample Label:**	BH14-3 - G4	**Collected by:**	Pam

Data & Results

Technician: *Clive* **Date Tested:** May 16-20, 2014

Sieve Type: Dry Sieve

Boulder	Cobble	Gravel		Sand			Fine Grain Soil	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_C
7.89	0.57

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.47	0.13	0.06	4.77

Comments: SAND, fine grained, some medium grained sand, some coarse grained sand, some silt, trace gravel (D10 value extrapolated)

Tested By DW

TESTING & MONITORING LTD.

102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

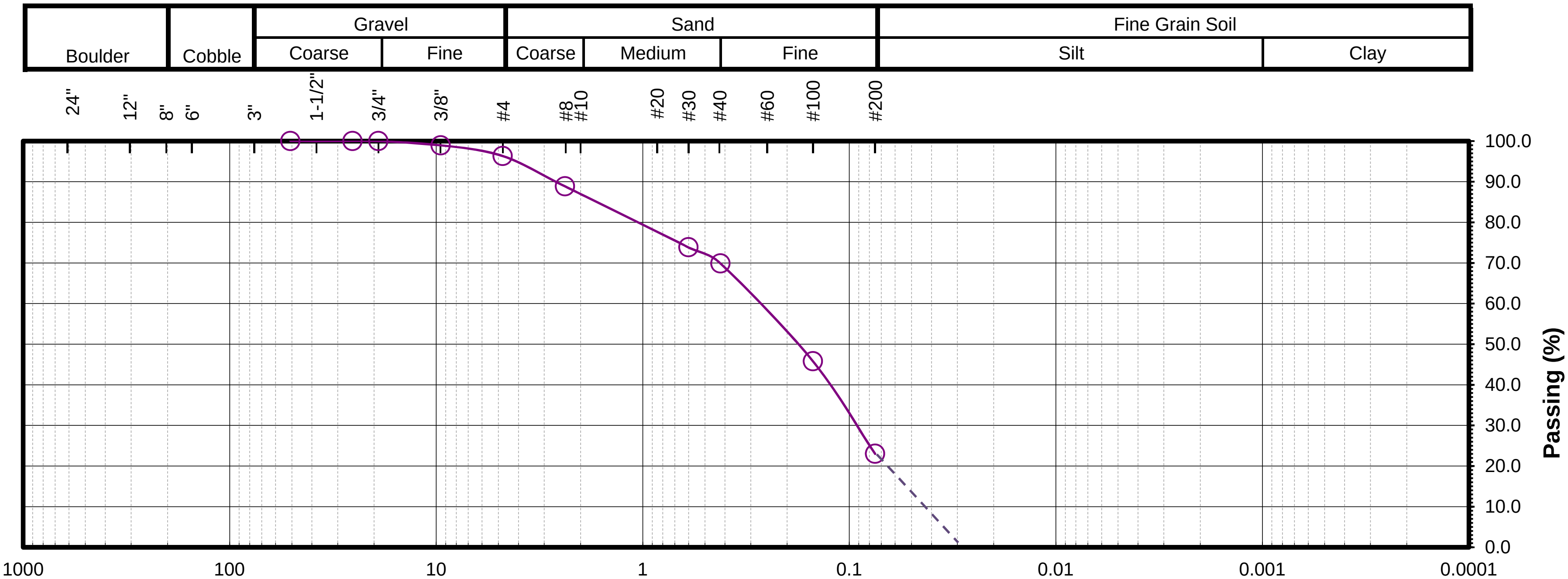
Client

Horizon Engineering	**Project:** 112-3155	**Sample Date:** 15-Apr-14
114-2433 Dollarton Hwy	**Location:** 385 Gower Point Rd., Gibsons, BC	**Receive Date:** 14-May-14
North Vancouver, BC, V7H 0A1	**Sample Label:** BH14-3 - G5	**Collected by:** Pam

Data & Results

Technician: ***Clive*** **Date Tested:** May 16-20, 2014

Sieve Type: Dry Sieve



Coefficient of Uniformity and Curvature

C_u	C_C
6.89	0.78

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.28	0.09	0.04	2.64

Comments: SILTY SAND, fine grained sand, some medium grained sand, trace coarse grained sand, trace gravel (D10 value extrapolated)

Tested By DW

HE TESTING & MONITORING LTD.
102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

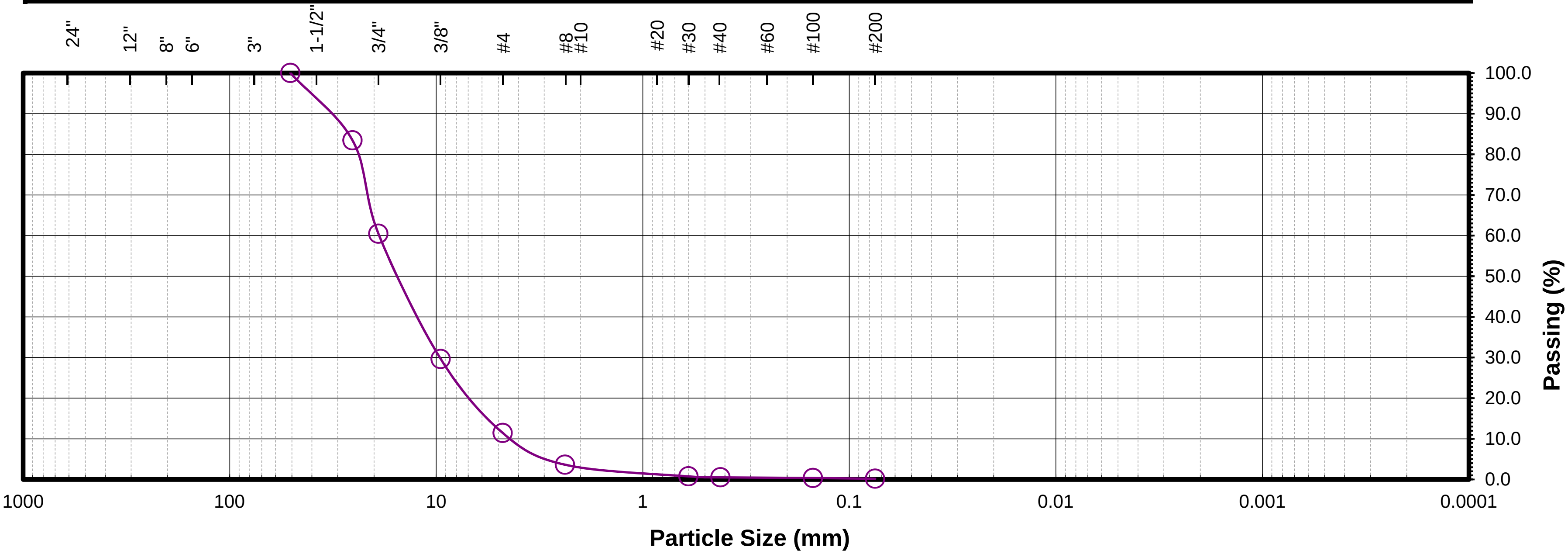
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-3 - G7

Sample Date: 15-Apr-14
Receive Date: 14-May-14
Collected by: Pam

Data & Results

Technician: *Clive*
Sieve Type: Dry Sieve
Date Tested: May 16-20, 2014

Boulder	Cobble	Gravel		Sand			Fine Grain Soil	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
4.50	1.16

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
18.86	9.59	4.19	33.45

Comments: SANDY GRAVEL, coarse grained sand, trace medium grained sand

Tested By DW

HE TESTING & MONITORING LTD.

102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

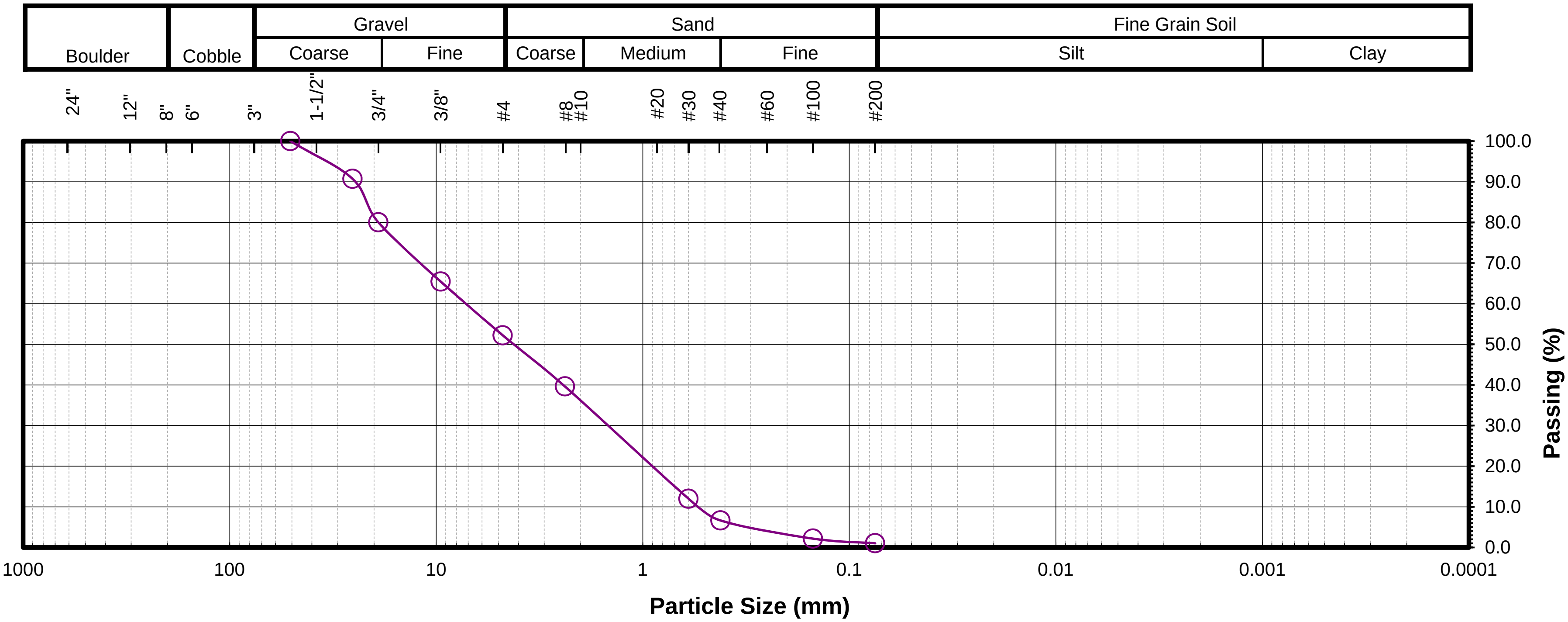
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-3 - G8

Sample Date: 15-Apr-14
Receive Date: 14-May-14
Collected by: Pam

Data & Results

Technician: *Clive*
Sieve Type: Dry Sieve

Date Tested: May 16-20, 2014



Coefficient of Uniformity and Curvature

C_u	C_c
13.61	0.58

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
7.16	1.47	0.53	24.91

Comments: SAND AND GRAVEL, medium grained sand, some coarse grained sand, trace fine grained sand, trace silt

Tested By DW

TESTING & MONITORING LTD.
102-173 Forester St
North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-4 - G1

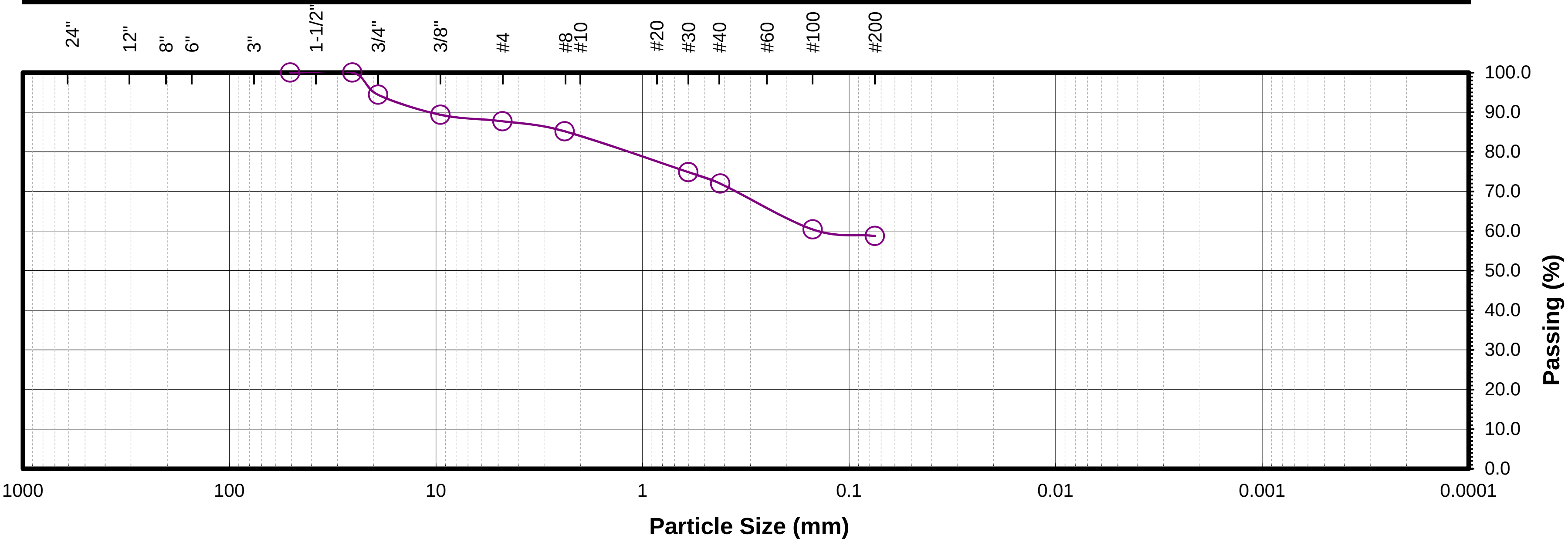
Sample Date: 9-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: *Jason*
Sieve Type: Washed Sieve

Date Tested: 05-Jan-15

Boulder	Cobble	Gravel - 12.3%		Sand - 28.9%			Fine Grain Soil - 58.8%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.13			0.72

Comments: SANDY SILT, fine to medium grained sand, some gravel, trace coarse grained sand

Tested By JT

HE Testing & Monitoring Ltd.
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North Vancouver, BC, V7H 2M9
info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

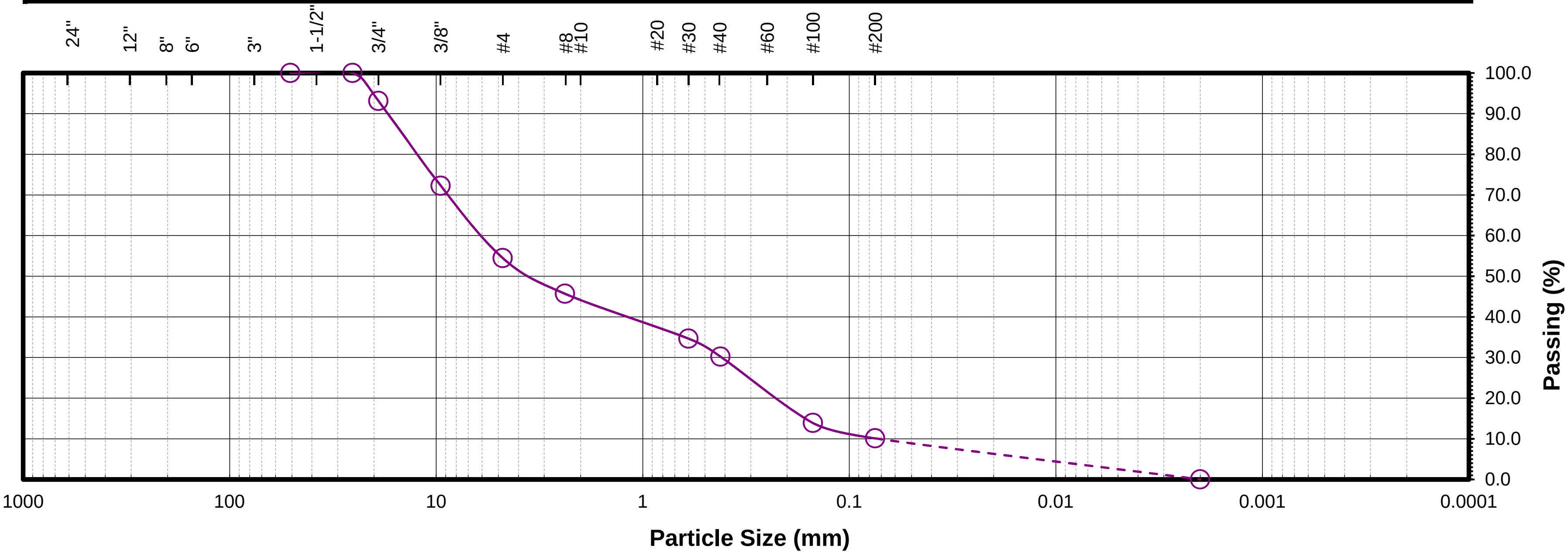
Horizon Engineering		**Project:**	112-3155	**Sample Date:**	9-Dec-14
114-2433 Dollarton Hwy		**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1		**Sample Label:**	BH14-4 - G2	**Collected by:**	Pam

Data & Results

Technician: ***Jason*** | **Date Tested:** 06-Jan-15

Sieve Type: Washed Sieve

Boulder	Cobble	Gravel - 45.6%		Sand - 44.3%			Fine Grain Soil - 10.1%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
78.77	0.39

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
5.91	0.42	0.075	18.34

Comments: **SAND AND GRAVEL, fine to medium grained sand, trace to some silt, trace coarse grained sand (D10 value extrapolated)**

Tested By JT

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Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

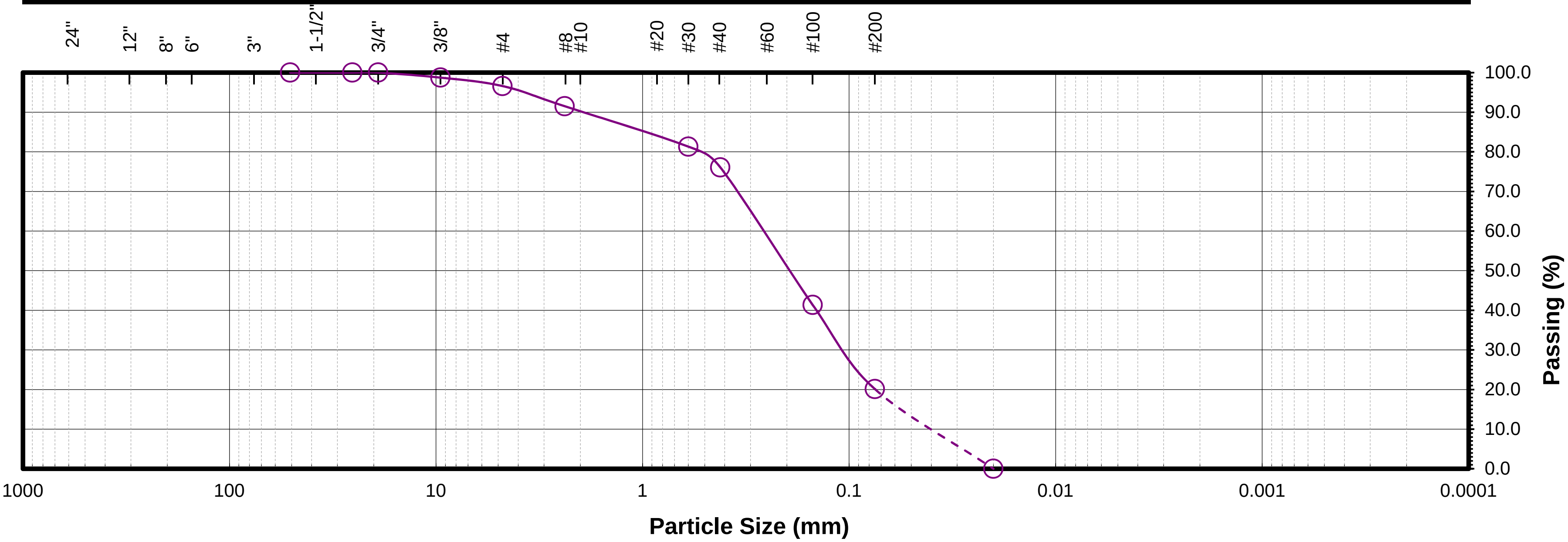
Horizon Engineering		**Project:**	112-3155	**Sample Date:**	9-Dec-14
114-2433 Dollarton Hwy		**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1		**Sample Label:**	BH14-4 - G4	**Collected by:**	Pam

Data & Results

Technician: ***Jason*** **Date Tested:** Dec. 24, 2014

Sieve Type: Dry Sieve

Boulder	Cobble	Gravel - 3.4%		Sand - 76.5%			Fine Grain Soil - 20.1%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
6.52	1.03

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.26	0.10	0.04	1.94

Comments: SAND, fine grained, some silt to silty, some medium grained sand, trace coarse grained sand, trace gravel (D10 value extrapolated)

Tested By JT

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Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

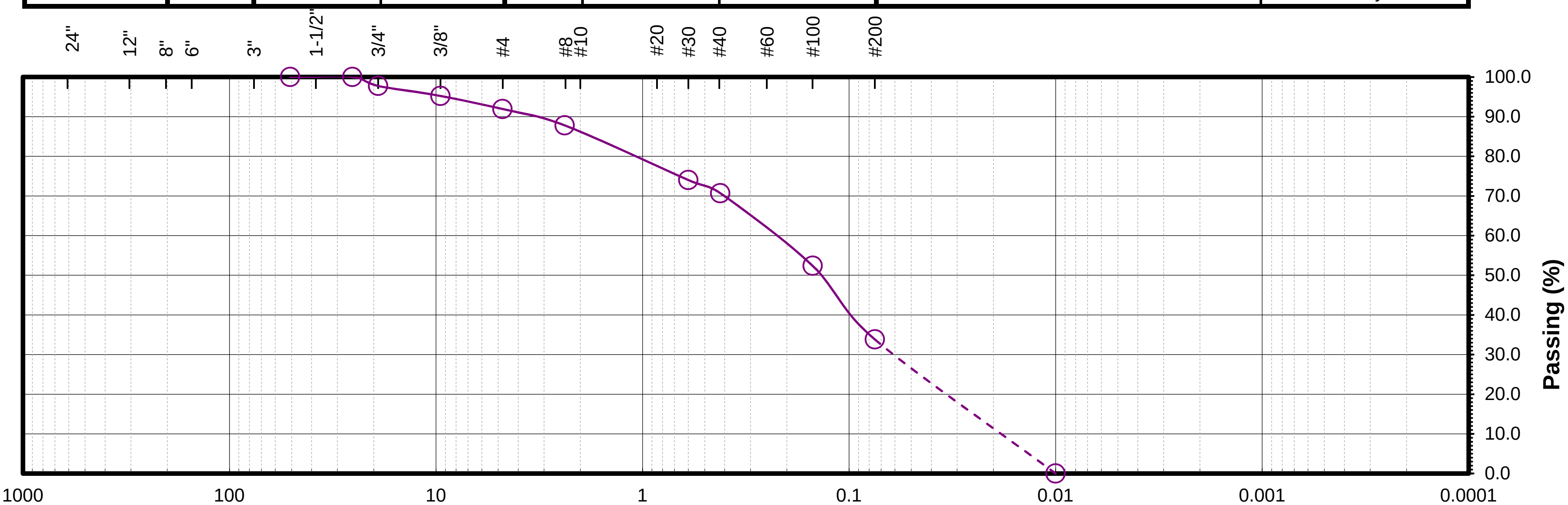
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-4 - G5

Sample Date: 10-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: ***Jason***
Sieve Type: Washed Sieve
Date Tested: Dec. 29, 2014

Boulder	Cobble	Gravel - 8.1%		Sand - 58.1%			Fine Grain Soil - 33.8%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
12.79	0.87

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.23	0.06	0.018	3.45

Comments: SILTY SAND, fine grained, some medium grained sand, trace coarse grained sand, trace gravel
(D10 and D30 values extrapolated)

Tested By JT

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Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-4 - G6

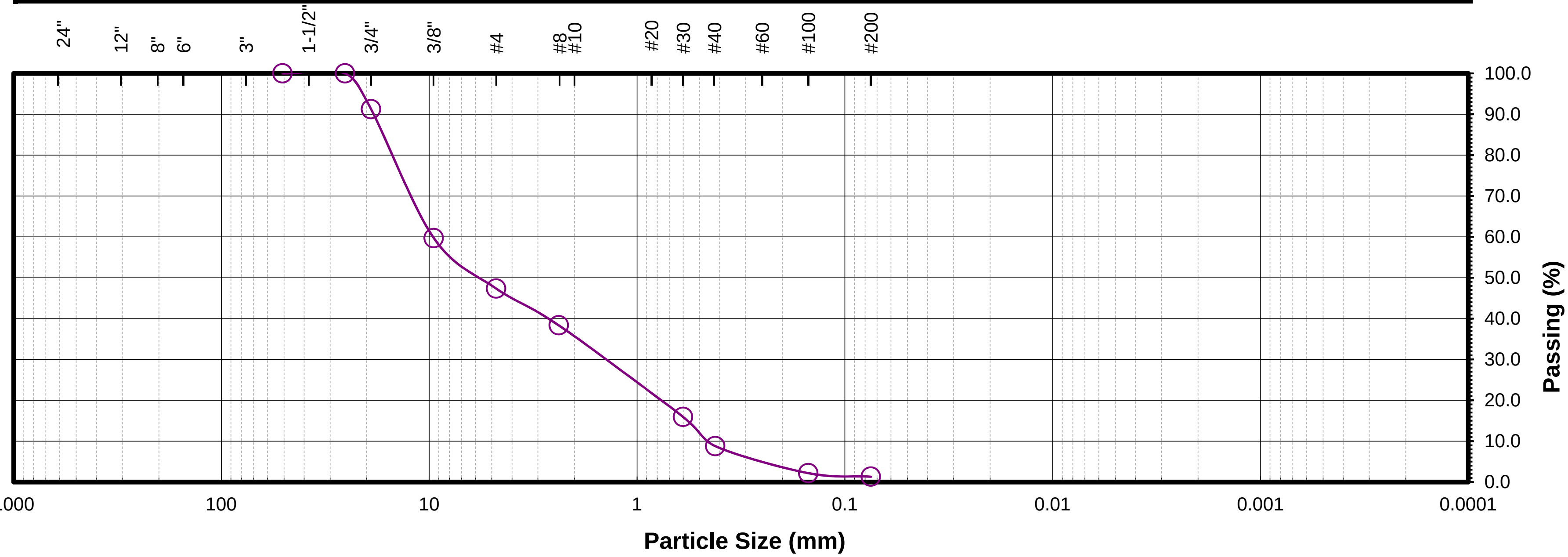
Sample Date: 9-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: *Jason*
Sieve Type: Dry Sieve

Date Tested: Dec 17-19, 2014

Boulder	Cobble	Gravel - 52.7%		Sand - 46%			Fine Grain Soil - 1.3%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
21.44	0.47

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
9.57	1.42	0.45	18.54

Comments: SAND AND GRAVEL, medium grained sand, trace coarse grained sand, trace fine grained sand, trace silt

Tested By JT

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info@hetesting.ca
Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

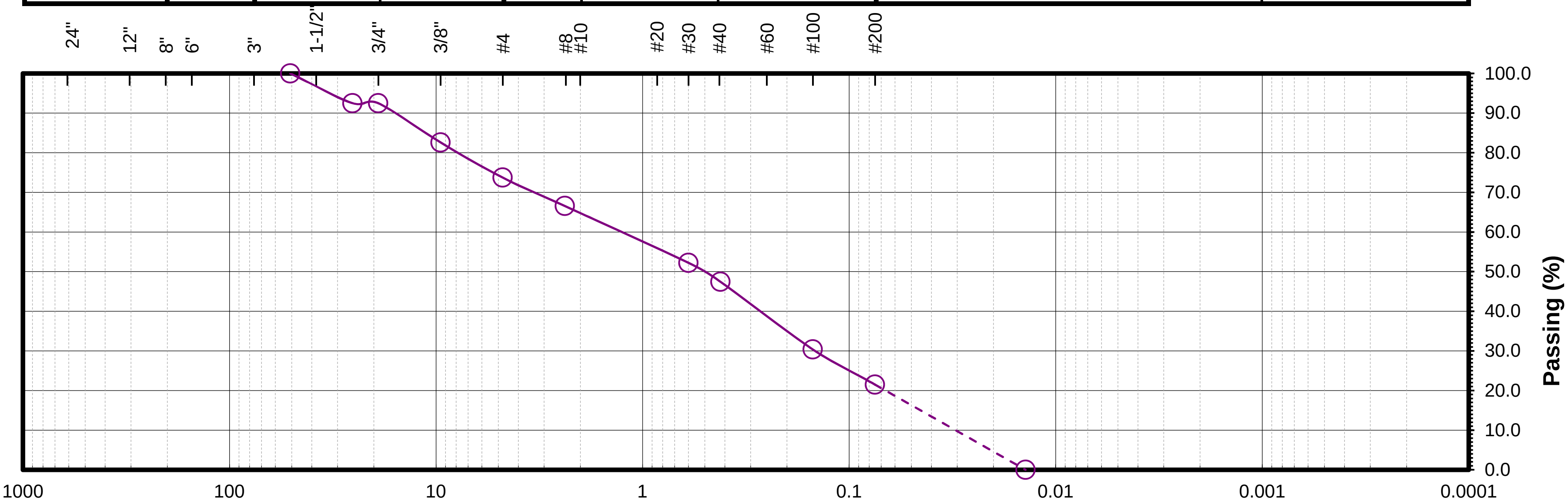
Horizon Engineering	**Project:**	112-3155		**Sample Date:**	9-Dec-14
114-2433 Dollarton Hwy	**Location:**	385 Gower Point Rd., Gibsons, BC		**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1	**Sample Label:**	BH14-4 - G7		**Collected by:**	Pam

Data & Results

Technician: ***Jason*** **Date Tested:** 31-Dec-14

Sieve Type: Washed Sieve

Boulder	Cobble	Gravel - 26.3%		Sand - 52.2%			Fine Grain Soil - 21.5%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
42.26	0.56

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
1.27	0.146	0.030	1.68

Comments: **SILTY, GRAVELLY SAND, fine to medium grained sand, trace coarse grained sand (D10 value extrapolated)**

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Sieve Analysis

Reference Number

Client

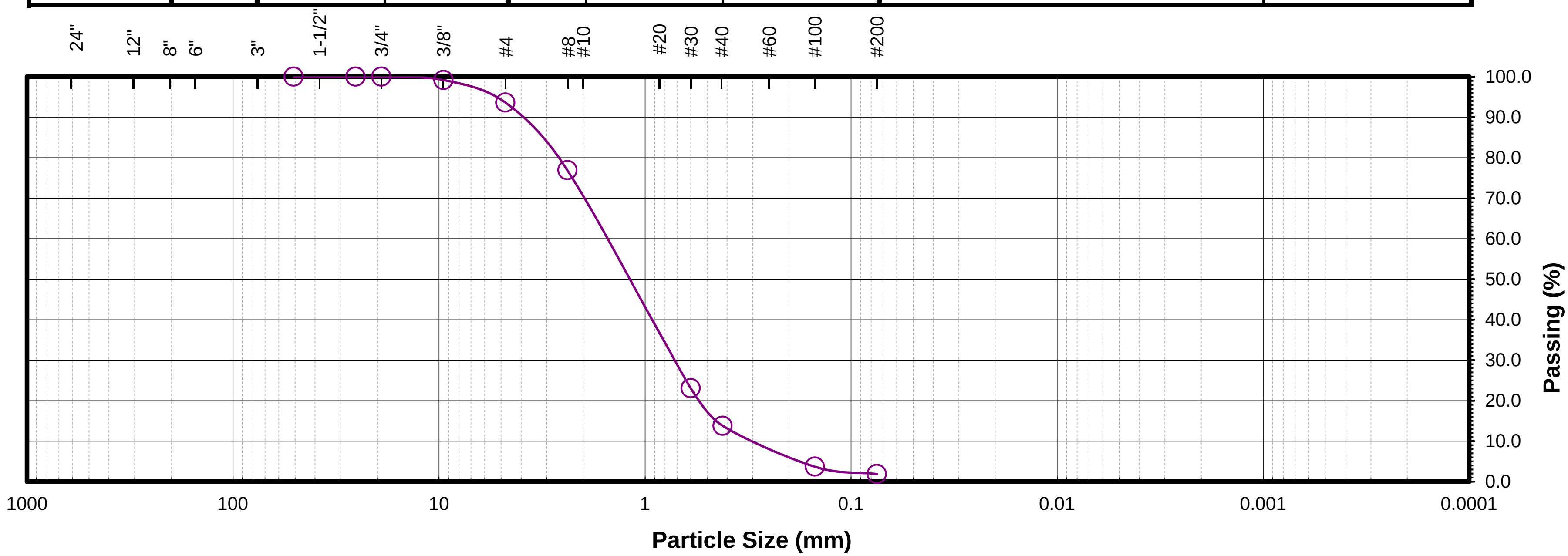
Horizon Engineering	**Project:**	112-3155	**Sample Date:**	10-Dec-14
114-2433 Dollarton Hwy	**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1	**Sample Label:**	BH14-4 - G8	**Collected by:**	Pam

Data & Results

Technician: ***Clive*** **Date Tested:** Dec 17-19, 2014

Sieve Type: Dry Sieve

Boulder	Cobble	Gravel - 6.4%		Sand - 91.7%			Fine Grain Soil - 1.9%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
5.42	1.17

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
1.54	0.72	0.29	4.10

Comments: SAND, medium grained, some coarse grained sand, some fine grained sand, trace gravel, trace silt

Tested By JT

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Sieve Analysis

Reference Number

Client

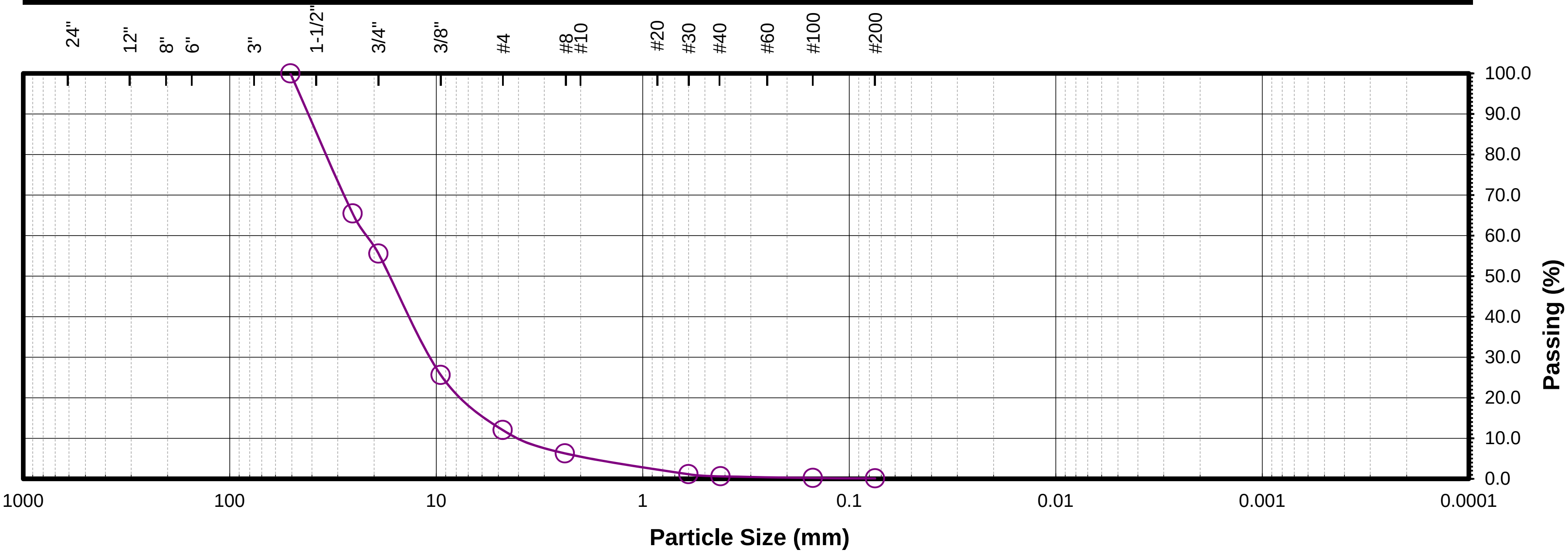
Horizon Engineering	**Project:**	112-3155	**Sample Date:**	9-Dec-14
114-2433 Dollarton Hwy	**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1	**Sample Label:**	BH14-4 - G9	**Collected by:**	Pam

Data & Results

Technician: ***Clive*** **Date Tested:** Dec 17-19, 2014

Sieve Type: Dry Sieve

Boulder	Cobble	Gravel - 88%		Sand - 10.9%			Fine Grain Soil - 0.1%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
5.82	1.37

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
21.68	10.53	3.73	41.57

Comments: GRAVEL, some coarse grained sand, trace medium grained sand

Tested By JT

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Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering		**Project:**	112-3155	**Sample Date:**	9-Dec-14
114-2433 Dollarton Hwy		**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1		**Sample Label:**	BH14-4 - G10	**Collected by:**	Pam

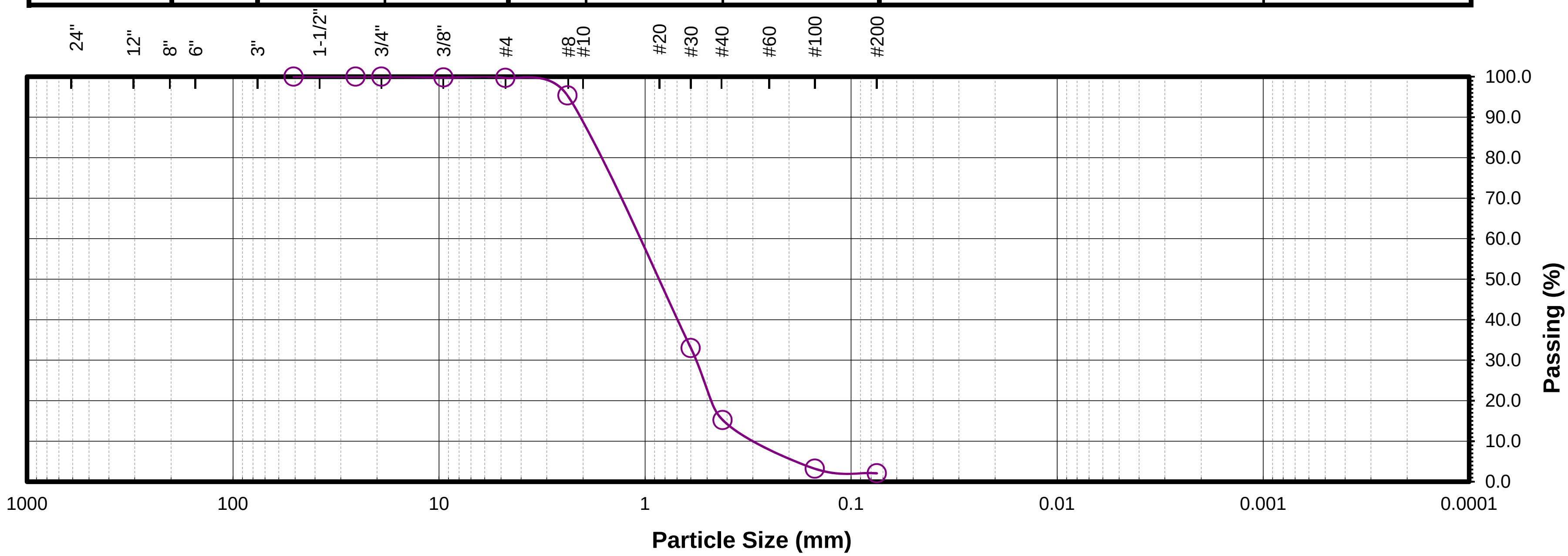
Data & Results

Technician: ***Jason***

Date Tested: Dec. 29, 2014

Sieve Type: Washed Sieve

Boulder	Cobble	Gravel - 0.3%		Sand - 97.6%			Fine Grain Soil - 2.1%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
4.06	1.09

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
1.09	0.57	0.27	2.11

Comments: SAND, medium grained, some fine grained sand, trace coarse grained sand, trace silt, trace gravel

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Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

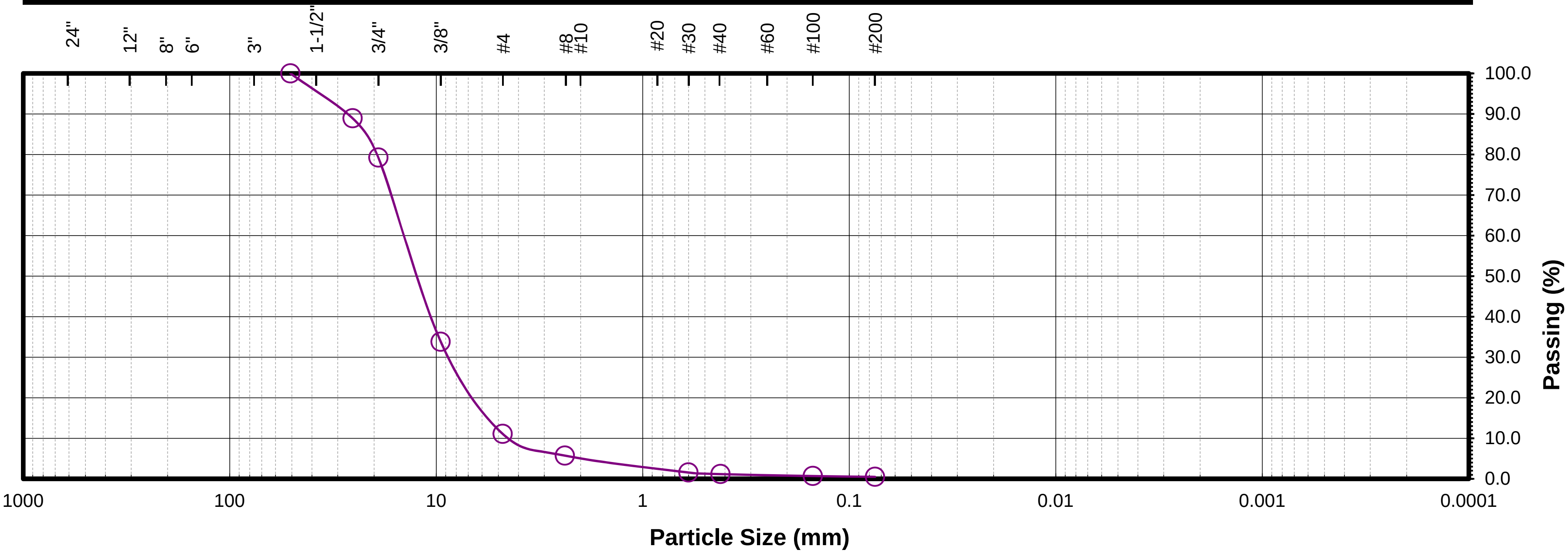
Horizon Engineering		**Project:**	112-3155	**Sample Date:**	10-Dec-14
114-2433 Dollarton Hwy		**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1		**Sample Label:**	BH14-4 - G11	**Collected by:**	Pam

Data & Results

Technician: *Clive* **Date Tested:** Dec 17-19, 2014

Sieve Type: Dry Sieve

Boulder	Cobble	Gravel - 88.9%		Sand - 10.6%			Fine Grain Soil - 0.5%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
3.43	1.22

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
14.20	8.47	4.14	27.16

Comments: GRAVEL, trace coarse grained sand, trace medium grained sand, trace fine grained sand, trace silt

Tested By JT

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Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

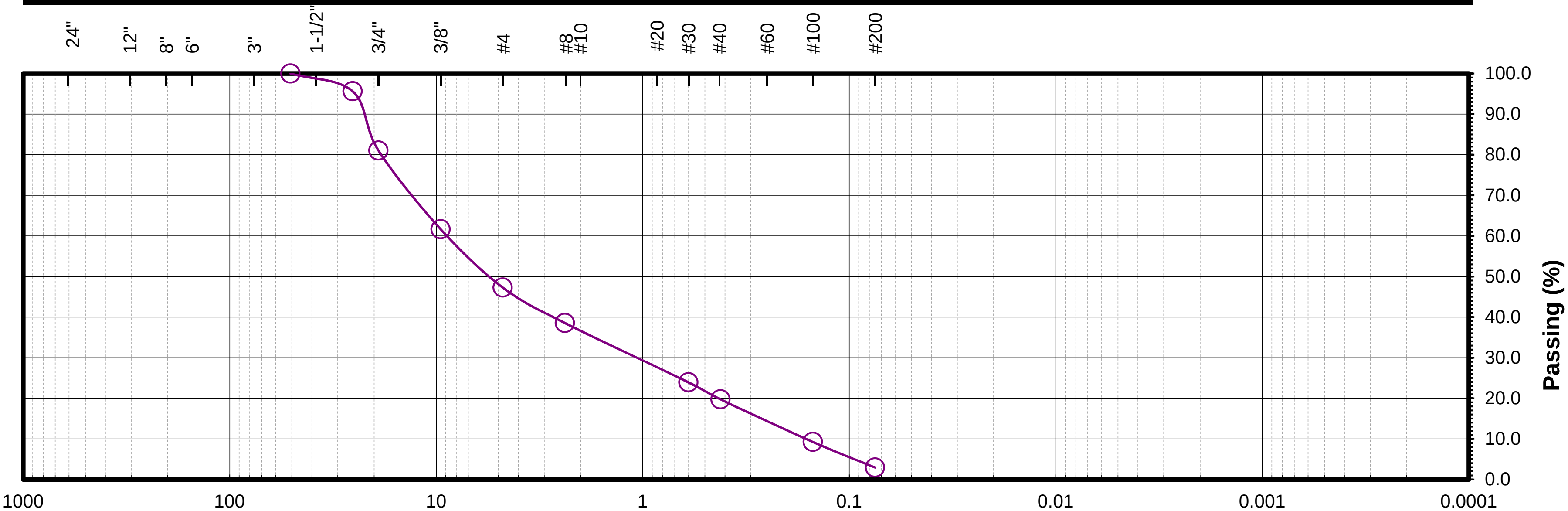
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-4 - G12

Sample Date: 9-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: *Clive*
Sieve Type: Dry Sieve
Date Tested: Dec 17-19, 2014

Boulder	Cobble	Gravel - 52.7%		Sand - 44.4%			Fine Grain Soil - 2.9%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
54.38	0.80

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
8.79	1.06	0.16	22.73

Comments: SAND AND GRAVEL, fine to medium grained sand, trace coarse grained sand, trace silt

Tested By JT

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Sieve Analysis

Reference Number

Client

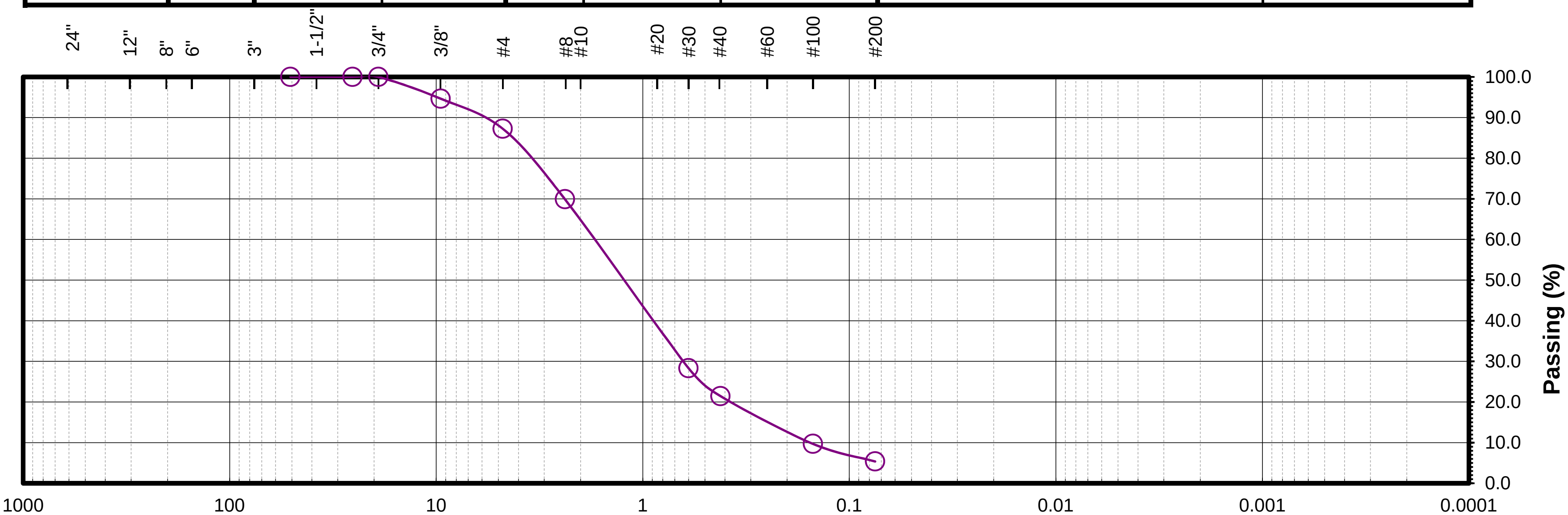
Horizon Engineering		**Project:**	112-3155	**Sample Date:**	9-Dec-14
114-2433 Dollarton Hwy		**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1		**Sample Label:**	BH14-4 - G13	**Collected by:**	Pam

Data & Results

Technician: *Jason* **Date Tested:** Dec. 29, 2014

Sieve Type: Dry Sieve

Boulder	Cobble	Gravel - 12.8%		Sand - 81.8%			Fine Grain Soil - 5.4%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
11.13	1.52

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
1.71	0.63	0.15	6.16

Comments: SAND, medium grained, some fine grained sand, some coarse grained sand, some gravel, trace silt

Tested By JT

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Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-4 - G14

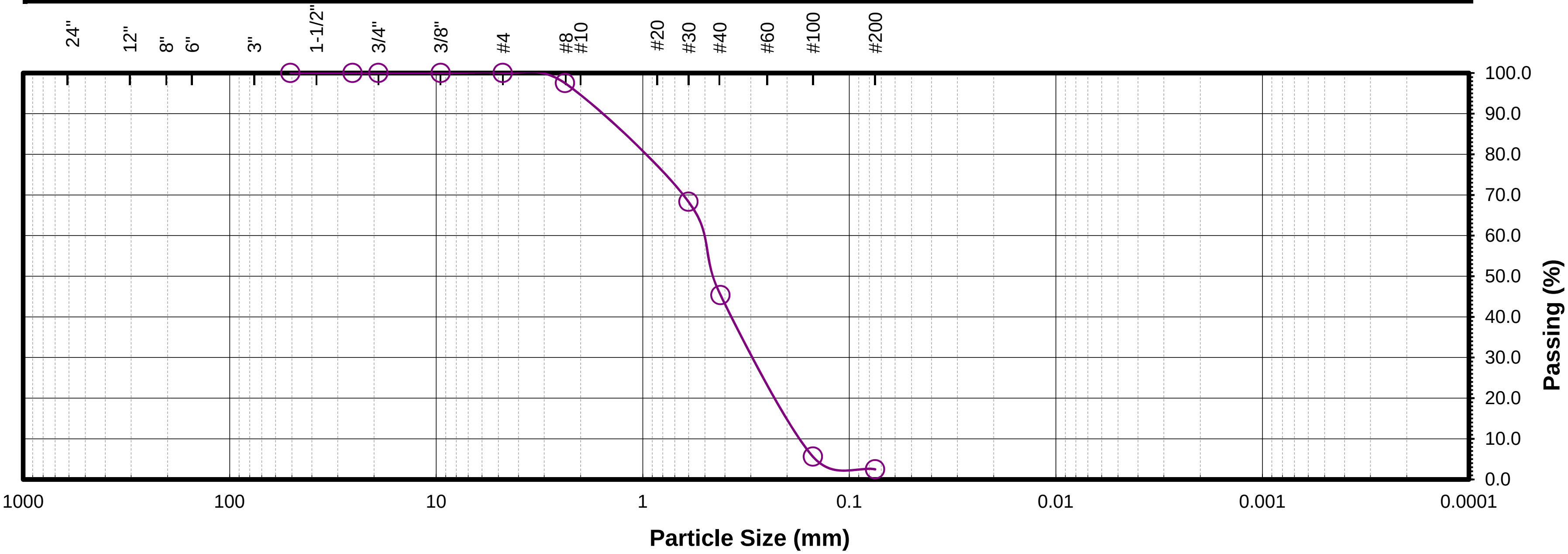
Sample Date: 9-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: *Jason*
Sieve Type: Dry Sieve

Date Tested: Dec. 30, 2014

Boulder	Cobble	Gravel - 0.0%		Sand - 97.5%			Fine Grain Soil - 2.5%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
3.14	0.90

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.53	0.28	0.17	1.67

Comments: SAND, fine to medium grained, trace coarse grained sand, trace silt

Tested By JT

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Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

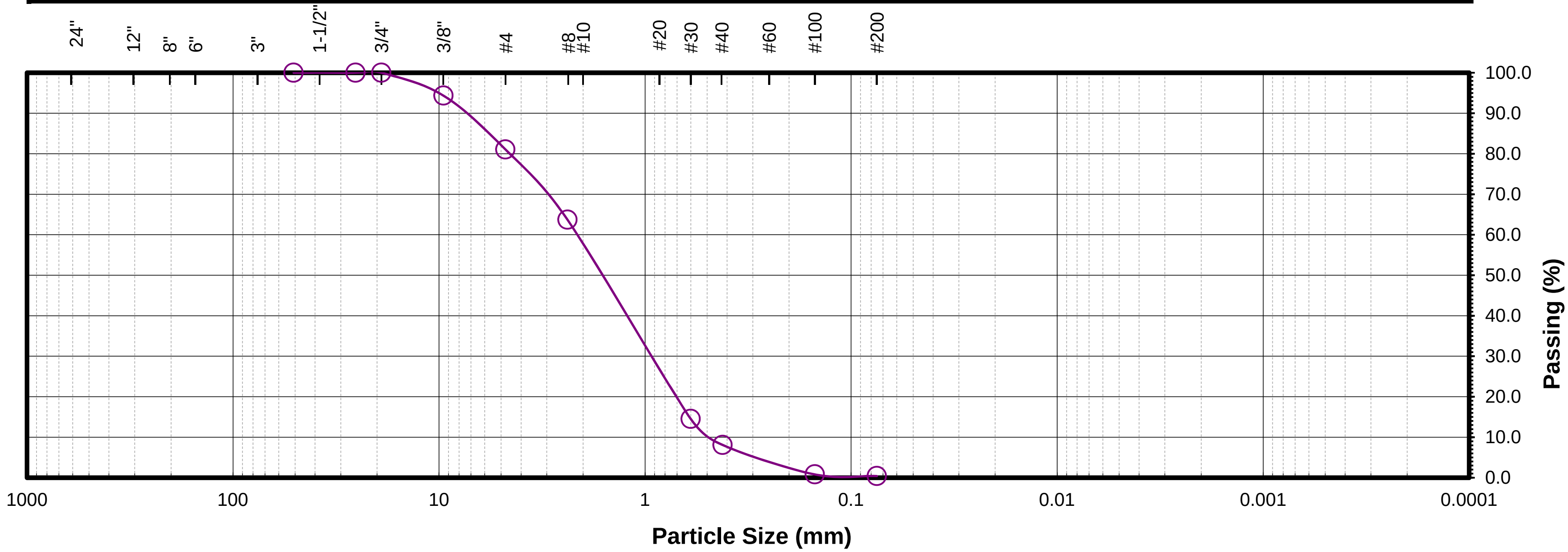
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-4 - G15

Sample Date: 9-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: ***Jason***
Sieve Type: Dry Sieve
Date Tested: Dec. 30, 2014

Boulder	Cobble	Gravel - 19.0%		Sand - 80.6%			Fine Grain Soil - 0.4%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
4.59	0.85

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
2.15	0.93	0.47	7.58

Comments: SAND, medium grained, some coarse grained sand, some gravel, trace fine grained sand

Tested By JT

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Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

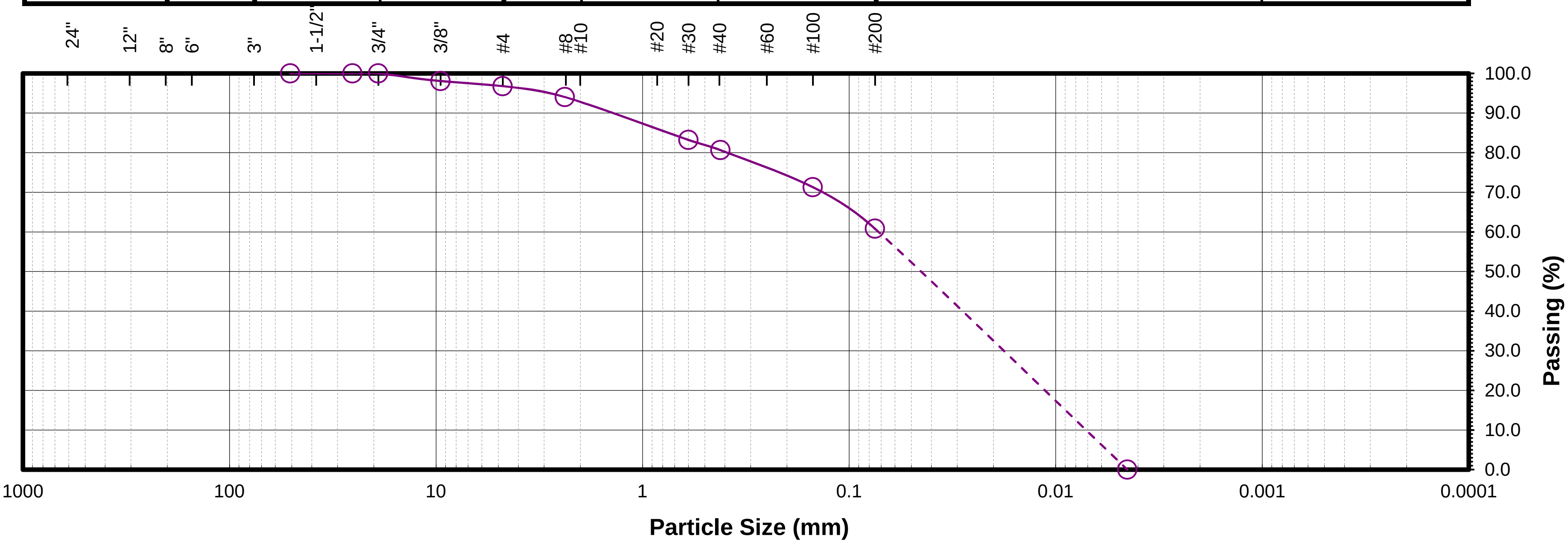
Horizon Engineering	**Project:**	112-3155		**Sample Date:**	9-Dec-14
114-2433 Dollarton Hwy	**Location:**	385 Gower Point Rd., Gibsons, BC		**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1	**Sample Label:**	BH14-5 - G2		**Collected by:**	Pam

Data & Results

Technician: ***Jason*** **Date Tested:** Jan. 6, 2015

Sieve Type: Washed Sieve

Boulder	Cobble	Gravel - 3.2%		Sand - 36.0%			Fine Grain Soil - 60.8%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
8.75	0.64

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.07	0.019	0.008	1.42

Comments: **SILT AND SAND, fine grained sand, some medium grained sand, trace coarse grained sand, trace gravel (D10, D30, and D60 values extrapolated)**

Tested By JT

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Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

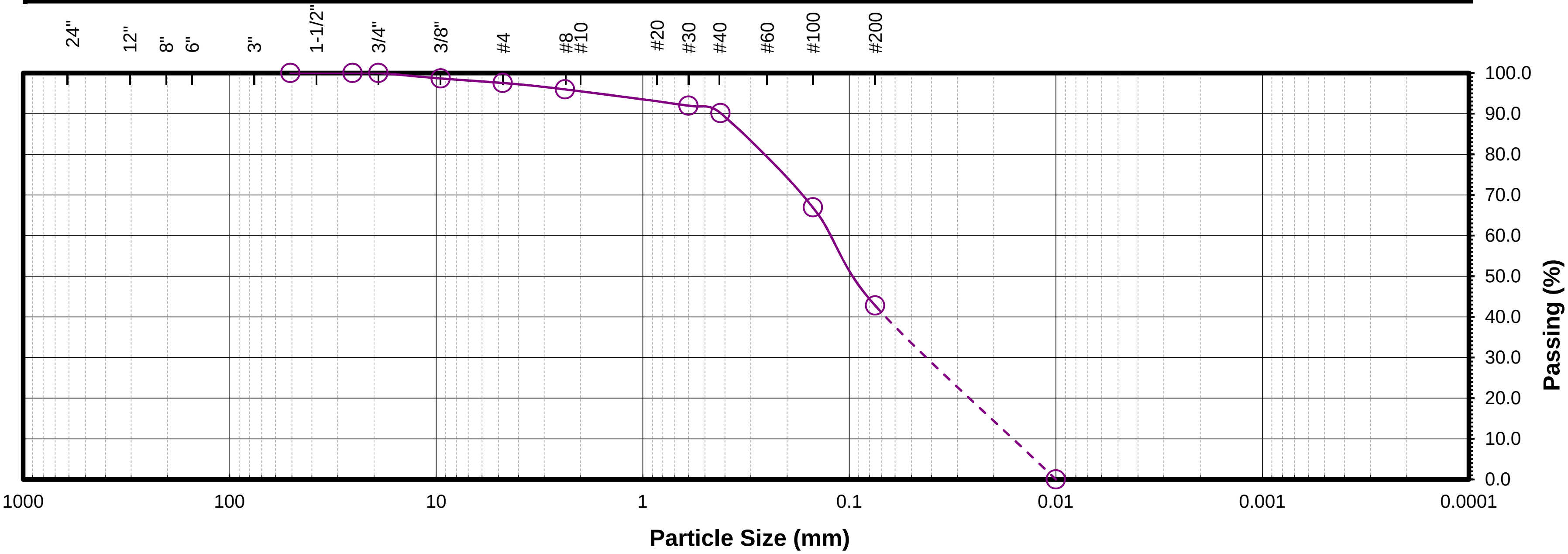
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-5 - G3

Sample Date: 10-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: *Jason*
Sieve Type: Washed Sieve
Date Tested: 06-Jan-15

Boulder	Cobble	Gravel - 2.4%		Sand - 54.8%			Fine Grain Soil - 42.8%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
7.68	0.94

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.12	0.04	0.02	0.42

Comments: SAND AND SILT, fine grained sand, trace medium grained sand, trace coarse grained sand, trace gravel
(D10 value extrapolated)

Tested By JT

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Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

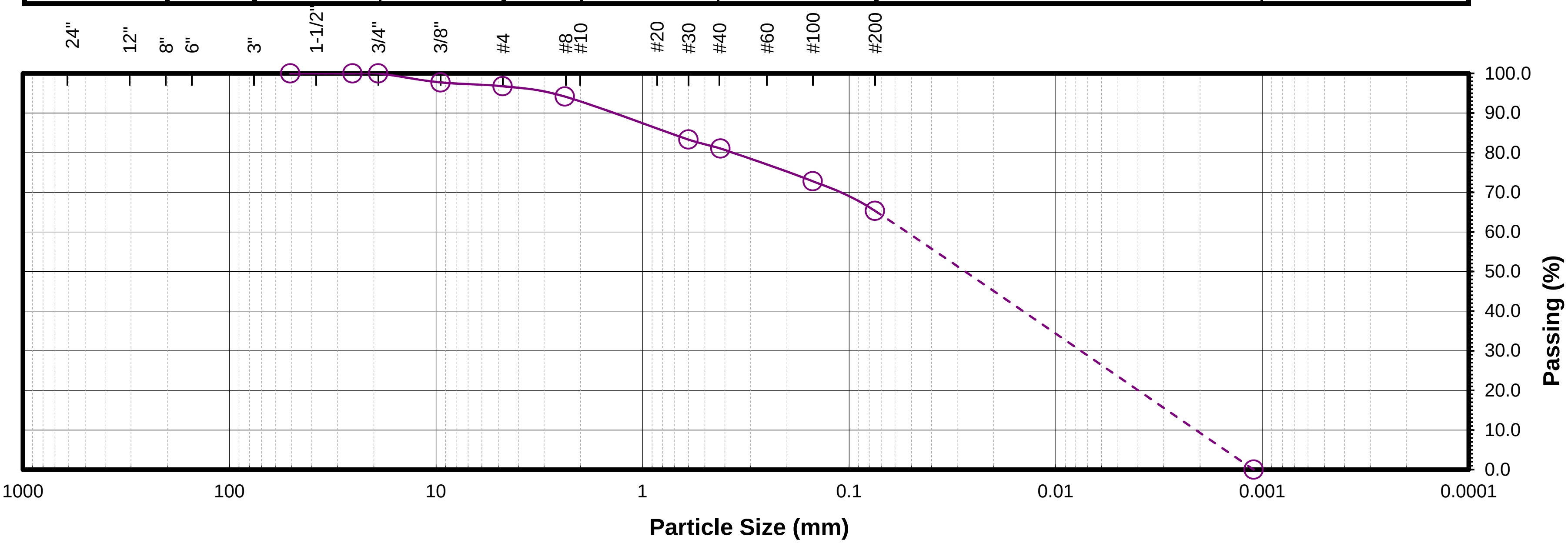
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-5 - G4

Sample Date: 10-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: *Jason*
Date Tested: Jan. 5, 2014
Sieve Type: Washed Sieve

Boulder	Cobble	Gravel - 3.2%		Sand - 31.5%			Fine Grain Soil - 65.3%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_C
25.00	0.72

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.05	0.0085	0.002	1.40

Comments: SANDY SILT, fine to medium grained sand, trace coarse grained sand, trace gravel
(D10, D30, and D60 values extrapolated)

Tested By JT

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Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-5 - G5

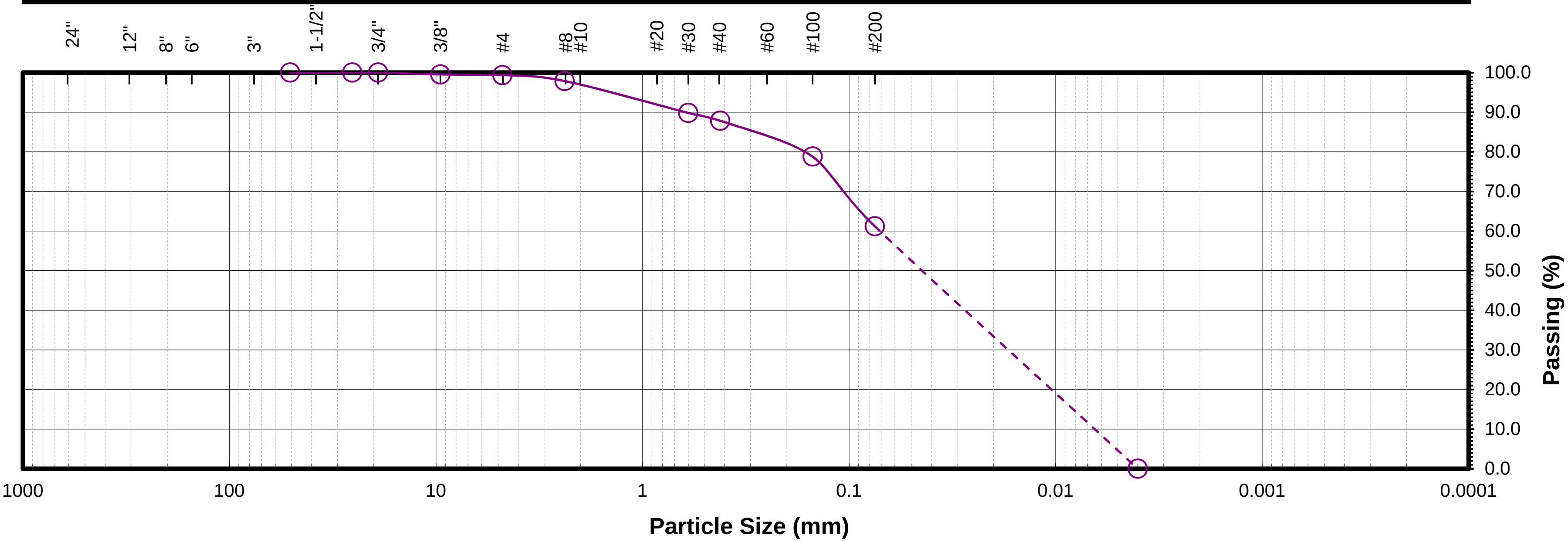
Sample Date: 9-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: *Jason*
Sieve Type: Washed Sieve

Date Tested: 05-Jan-15

Boulder	Cobble	Gravel - 0.7%		Sand - 38.1%			Fine Grain Soil - 61.2%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
10.00	0.59

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.07	0.017	0.007	0.62

Comments: **SILT AND SAND, fine grained sand, trace medium grained sand, trace coarse grained sand, trace gravel (D10, D30, and D60 values extrapolated)**

Tested By JT

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Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

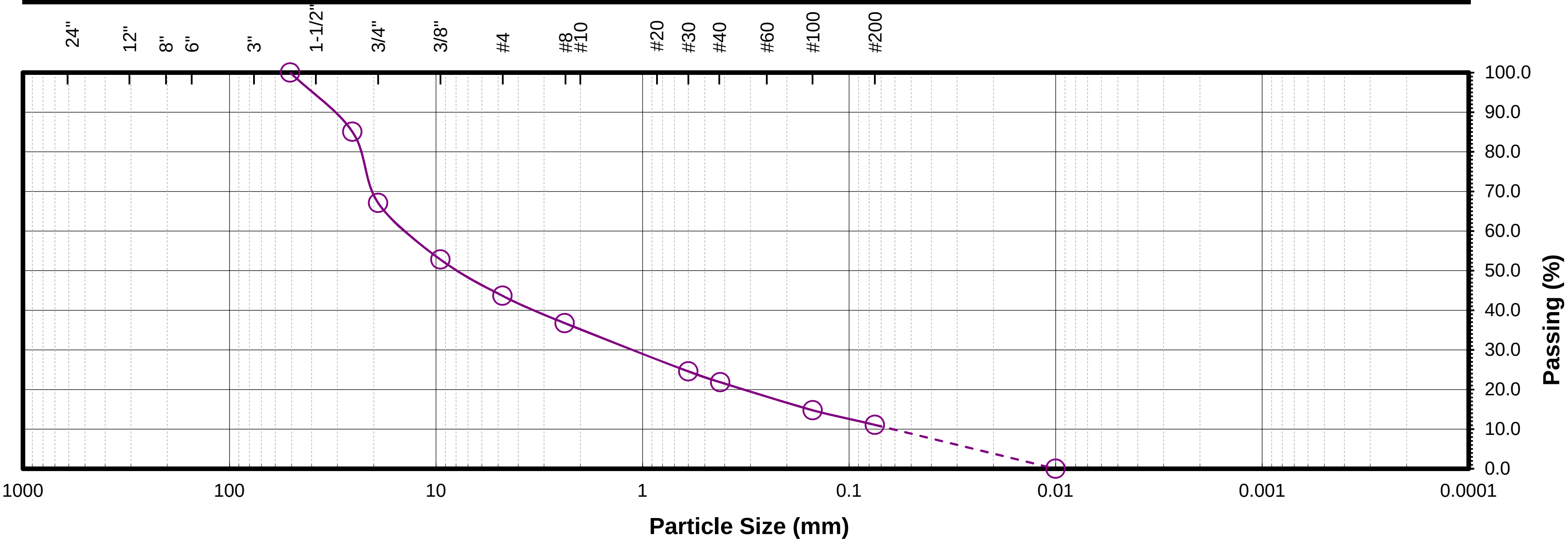
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-5 - G6

Sample Date: 10-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: ***Jason***
Sieve Type: Washed Sieve
Date Tested: Dec. 29, 2014

Boulder	Cobble	Gravel - 56.3%		Sand - 32.6%			Fine Grain Soil - 11.1%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
224.86	1.52

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
13.49	1.11	0.06	31.93

Comments: SANDY GRAVEL, fine to medium grained sand, some silt, trace coarse grained sand
(D10 value extrapolated)

Tested By JT

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Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

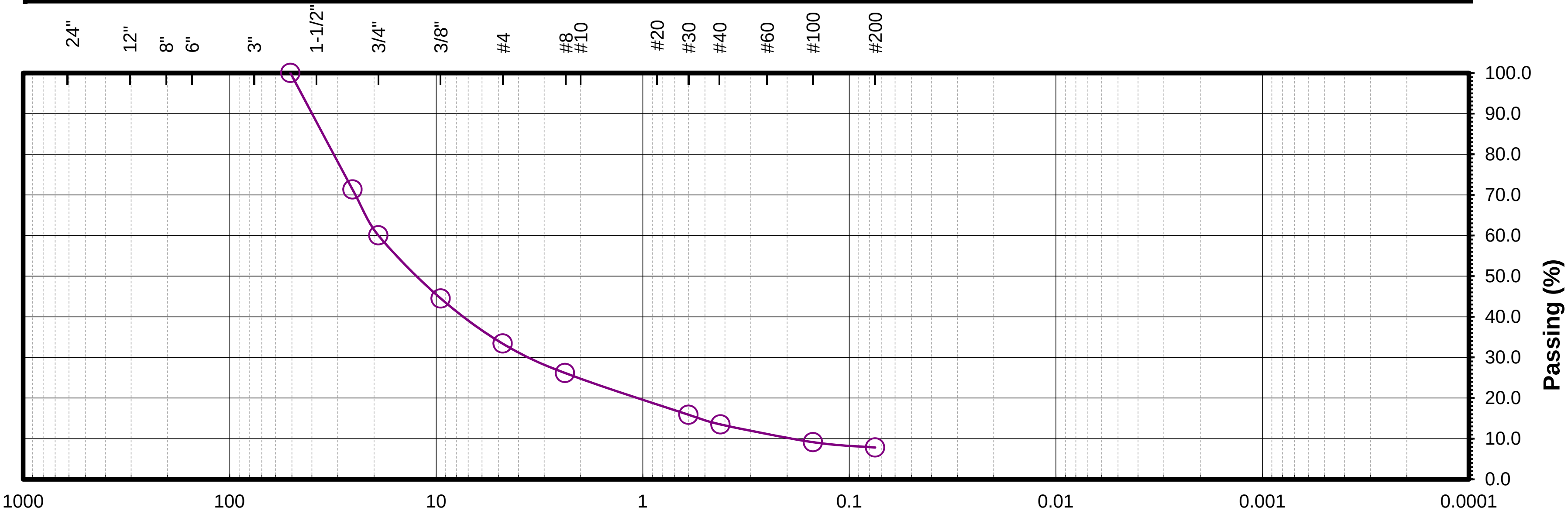
Horizon Engineering	**Project:**	112-3155	**Sample Date:**	10-Dec-14
114-2433 Dollarton Hwy	**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1	**Sample Label:**	BH14-5 - G7	**Collected by:**	Pam

Data & Results

Technician: *Jason* **Date Tested:** Dec. 30, 2014

Sieve Type: Washed Sieve

Boulder	Cobble	Gravel - 66.6%		Sand - 25.6%			Fine Grain Soil - 7.8%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_C
103.21	3.38

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
19.02	3.44	0.18	39.90

Comments: SANDY GRAVEL, medium grained sand, trace fine grained sand, trace coarse grained sand, trace silt

Tested By JT

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Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

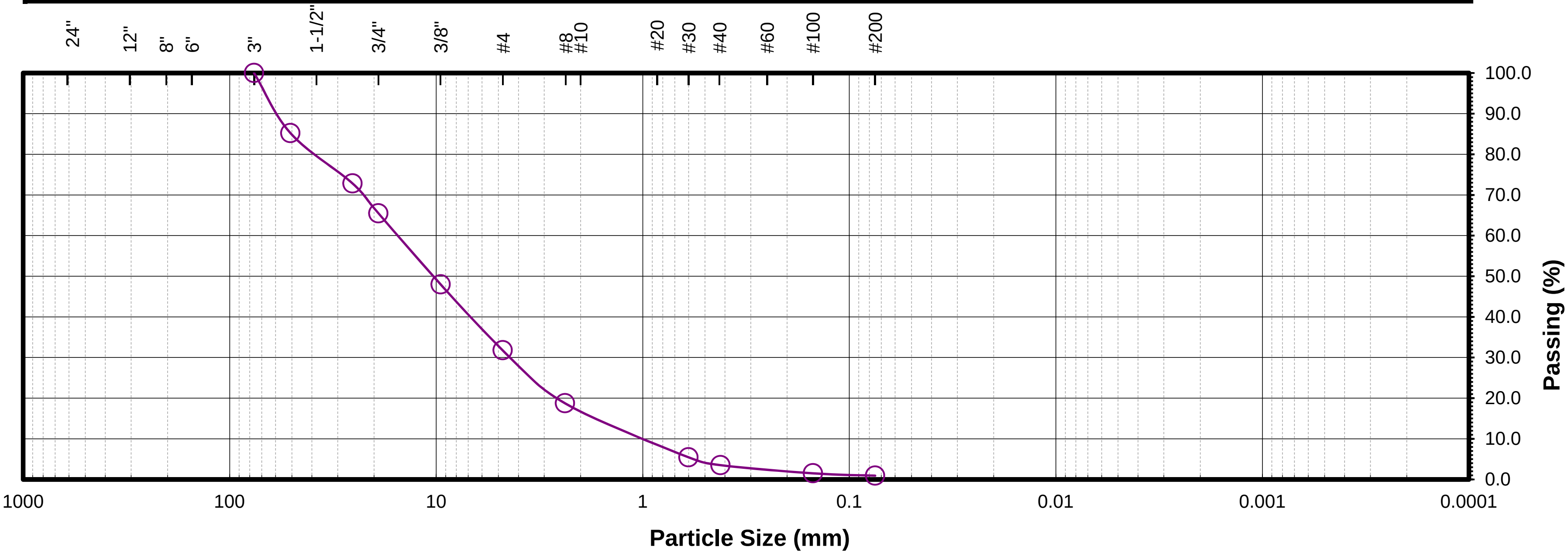
Horizon Engineering	**Project:**	112-3155	**Sample Date:**	10-Dec-14
114-2433 Dollarton Hwy	**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1	**Sample Label:**	BH14-5 - G8	**Collected by:**	Pam

Data & Results

Technician: *Jason* **Date Tested:** Dec. 24, 2014

Sieve Type: Dry Sieve

Boulder	Cobble	Gravel - 68.2%		Sand - 30.9%			Fine Grain Soil - 0.9%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
15.92	1.27

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
15.33	4.33	0.96	57.94

Comments: SANDY GRAVEL, medium to coarse grained sand, trace fine grained sand, trace silt

Tested By JT

ESTING & MONITORING LTD.

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Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

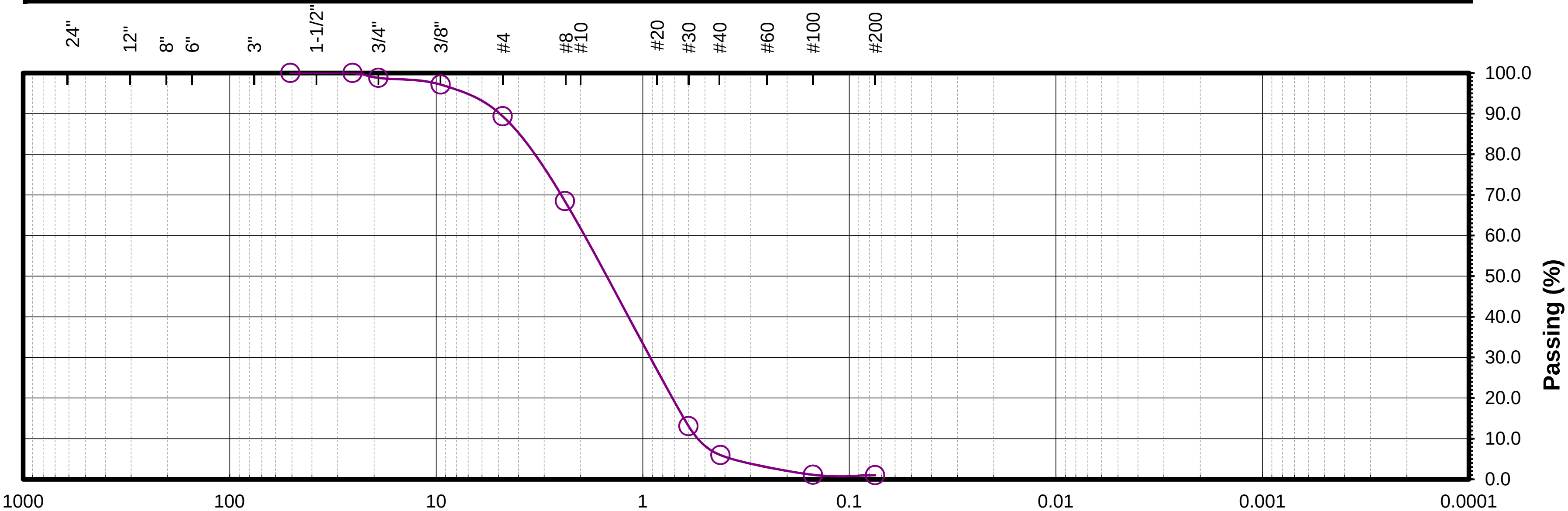
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-5 - G9

Sample Date: 10-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: *Jason*
Date Tested: Dec. 29, 2014
Sieve Type: Washed Sieve

Boulder	Cobble	Gravel - 10.7%		Sand - 88.3%			Fine Grain Soil - 1.0%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
3.75	0.84

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
1.93	0.91	0.51	5.05

Comments: SAND, medium grained, some coarse grained sand, some gravel, trace fine grained sand, trace silt

Tested By JT

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Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-5 - G10

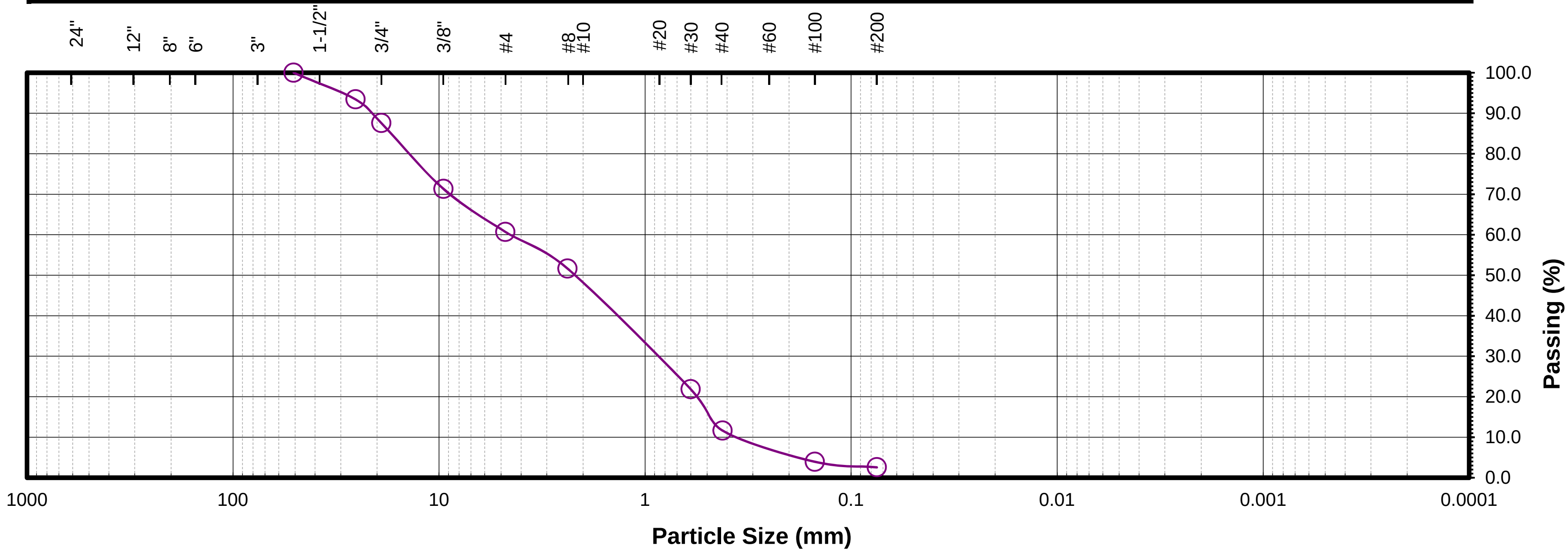
Sample Date: 10-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: ***Clive***
Sieve Type: Dry Sieve

Date Tested: Dec 17-19, 2014

Boulder	Cobble	Gravel - 39.3%		Sand - 58.1%			Fine Grain Soil - 2.6%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
13.33	0.50

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
4.51	0.87	0.34	21.47

Comments: SAND AND GRAVEL, medium grained sand, trace coarse grained sand, trace fine grained sand, trace silt

Tested By JT

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HE TESTING & MONITORING LTD.

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

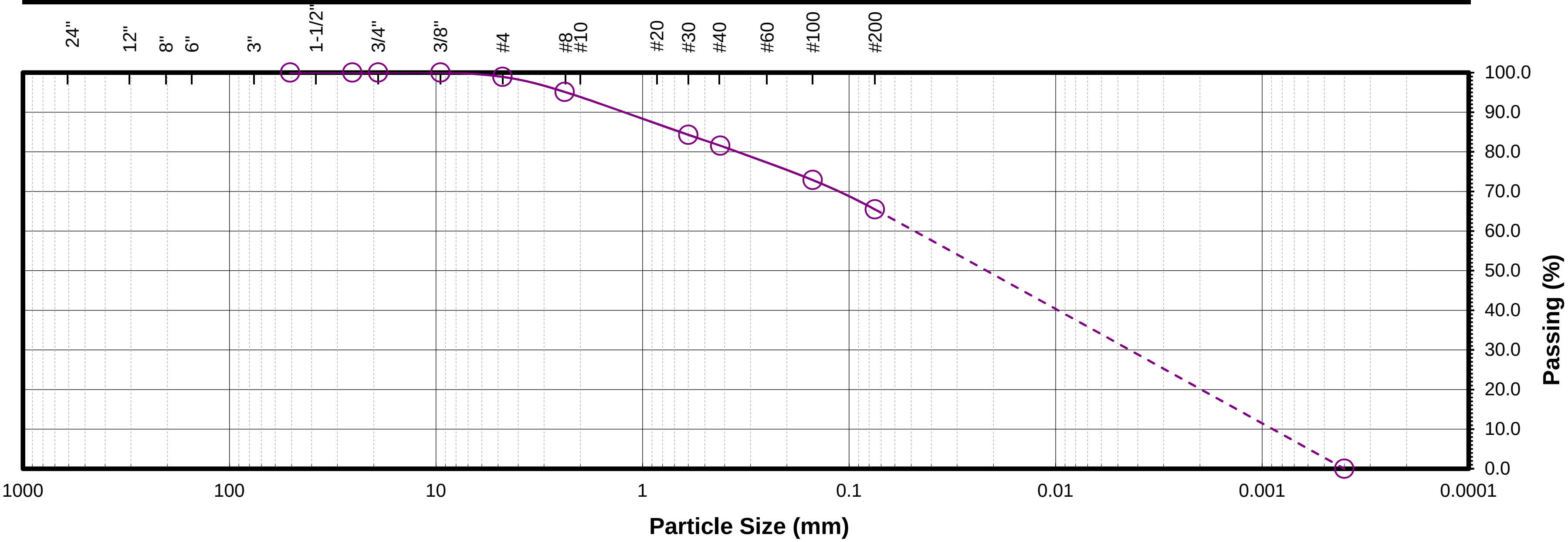
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-6 - G1

Sample Date: 10-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: *Jason*
Sieve Type: Washed Sieve
Date Tested: Dec. 30, 2014

Boulder	Cobble	Gravel - 1.1%		Sand - 33.4%			Fine Grain Soil - 65.5%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
54.44	0.44

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.049	0.0044	0.0009	1.24

Comments: SANDY SILT, fine to medium grained sand, trace coarse grained sand, trace gravel
(D10, D30 and D60 values extrapolated)

Tested By JT

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Sieve Analysis
Reference Number

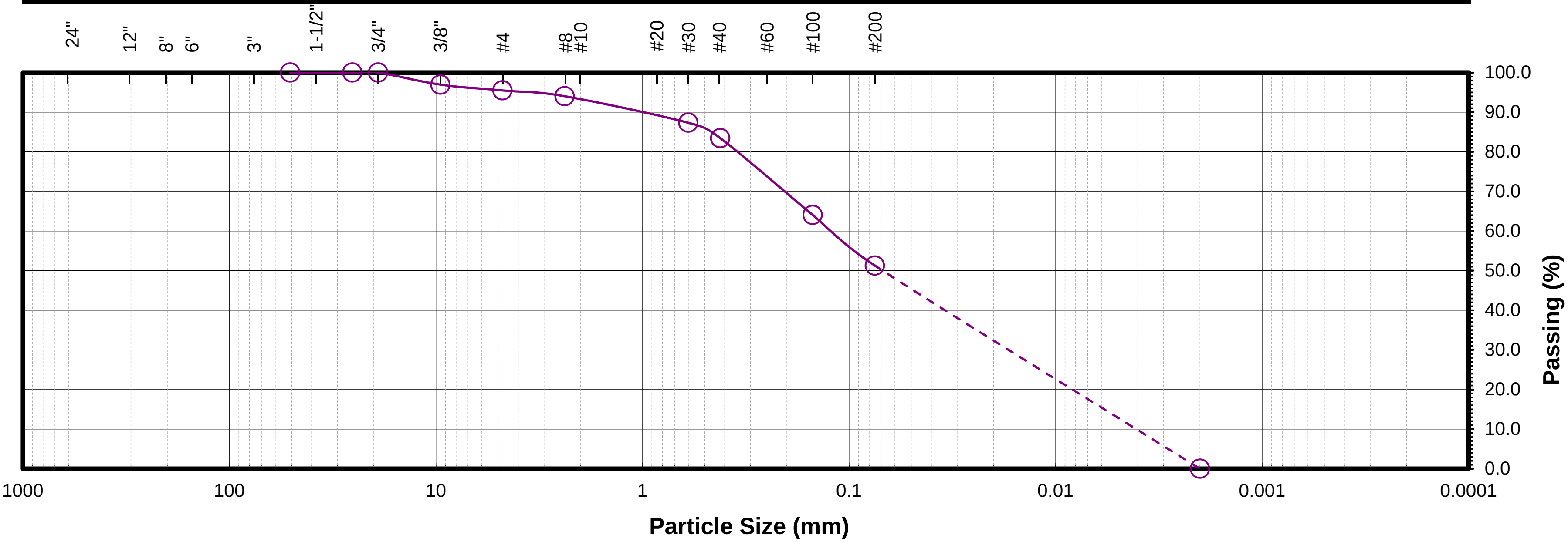
Client

Horizon Engineering	**Project:**	112-3155	**Sample Date:**	10-Dec-14
114-2433 Dollarton Hwy	**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1	**Sample Label:**	BH14-6 - G2	**Collected by:**	Pam

Data & Results

Technician: ***Jason*** **Date Tested:** Dec. 30, 2014
Sieve Type: Washed Sieve

Boulder	Cobble	Gravel - 4.5%		Sand - 44.2%			Fine Grain Soil - 51.3%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
30.08	0.64

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.12	0.018	0.004	1.04

Comments: **SILT AND SAND, fine grained sand, trace medium grained sand, trace coarse grained sand, trace gravel (D10 and D30 values extrapolated)**

Tested By JT

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Sieve Analysis

Reference Number

Client

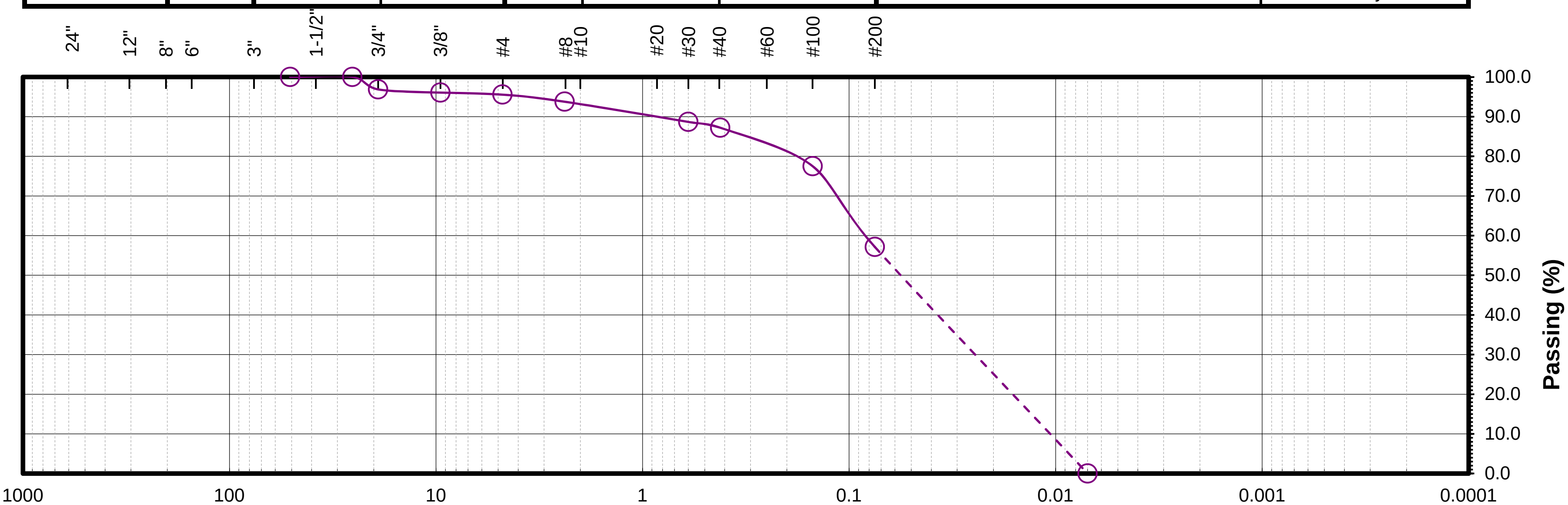
Horizon Engineering	**Project:**	112-3155	**Sample Date:**	9-Dec-14
114-2433 Dollarton Hwy	**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1	**Sample Label:**	BH14-6 - G3	**Collected by:**	Pam

Data & Results

Technician: ***Jason*** **Date Tested:** 05-Jan-15

Sieve Type: Washed Sieve

Boulder	Cobble	Gravel - 4.4%		Sand - 38.5%			Fine Grain Soil - 57.1%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
8.27	0.76

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.08	0.025	0.01	0.86

Comments: **SILT AND SAND, fine grained sand, trace medium grained sand, trace coarse grained sand, trace gravel (D10 and D30 values extrapolated)**

Tested By JT

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Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

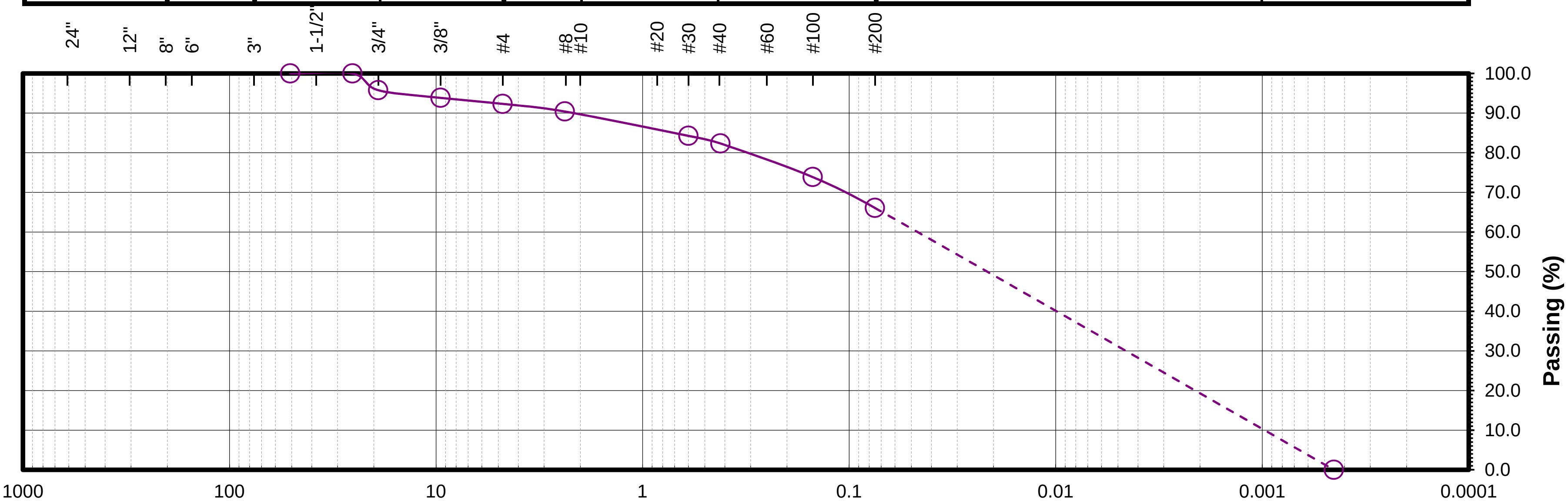
Project:	112-3155	**Sample Date:**	9-Dec-14
Location:	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
Sample Label:	BH14-6 - G4	**Collected by:**	Pam

Data & Results

Technician: ***Jason*** **Date Tested:** 31-Dec-14

Sieve Type: Washed Sieve

Boulder	Cobble	Gravel 7.7%		Sand - 26.2%			Fine Grain Soil - 66.1%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
47.00	0.43

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.047	0.005	0.001	2.18

Comments: **SANDY SILT, fine grained sand, trace medium grained sand, trace coarse grained sand, trace gravel**
(D10, D30, and D60 values extrapolated)

Tested By JT

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Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-6 - G5

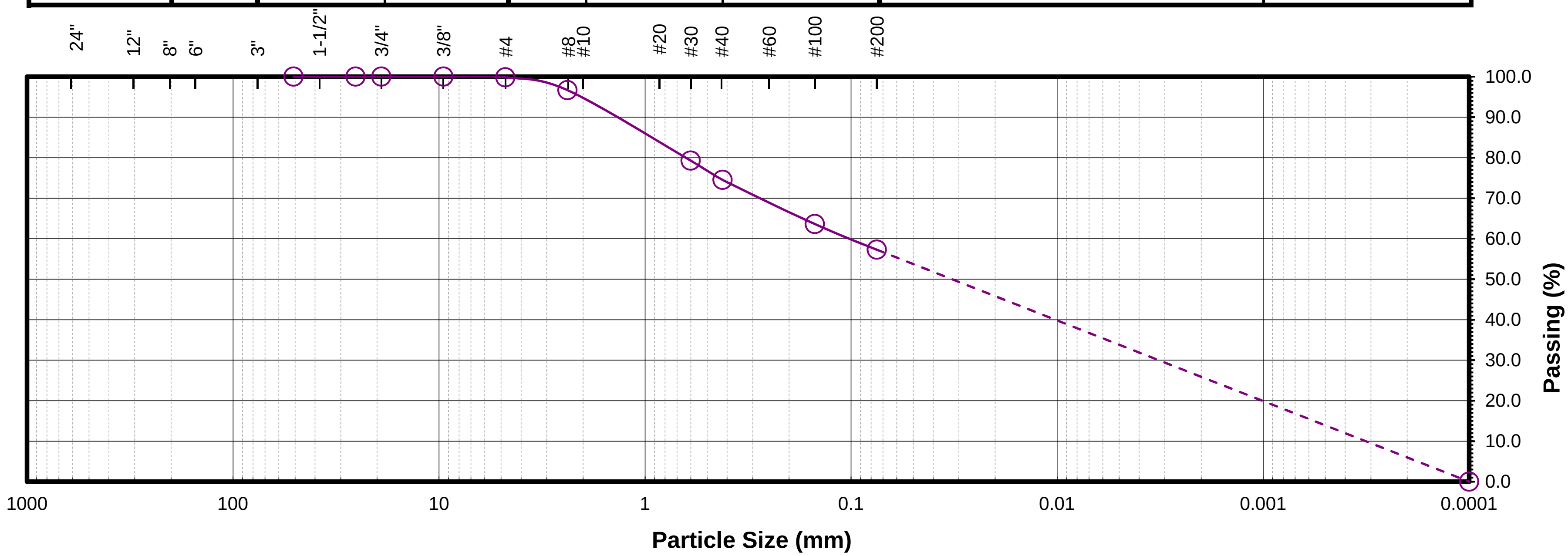
Sample Date: 10-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: *Jason*
Sieve Type: Washed Sieve

Date Tested: 06-Jan-15

Boulder	Cobble	Gravel - 0.2%		Sand - 42.5%			Fine Grain Soil - 57.3%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
336.81	0.34

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
0.10	0.00	0.00	1.40

Comments: **SILT AND SAND, fine to medium grained sand, trace coarse grained sand (D10 and D30 values extrapolated)**

Tested By JT

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Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

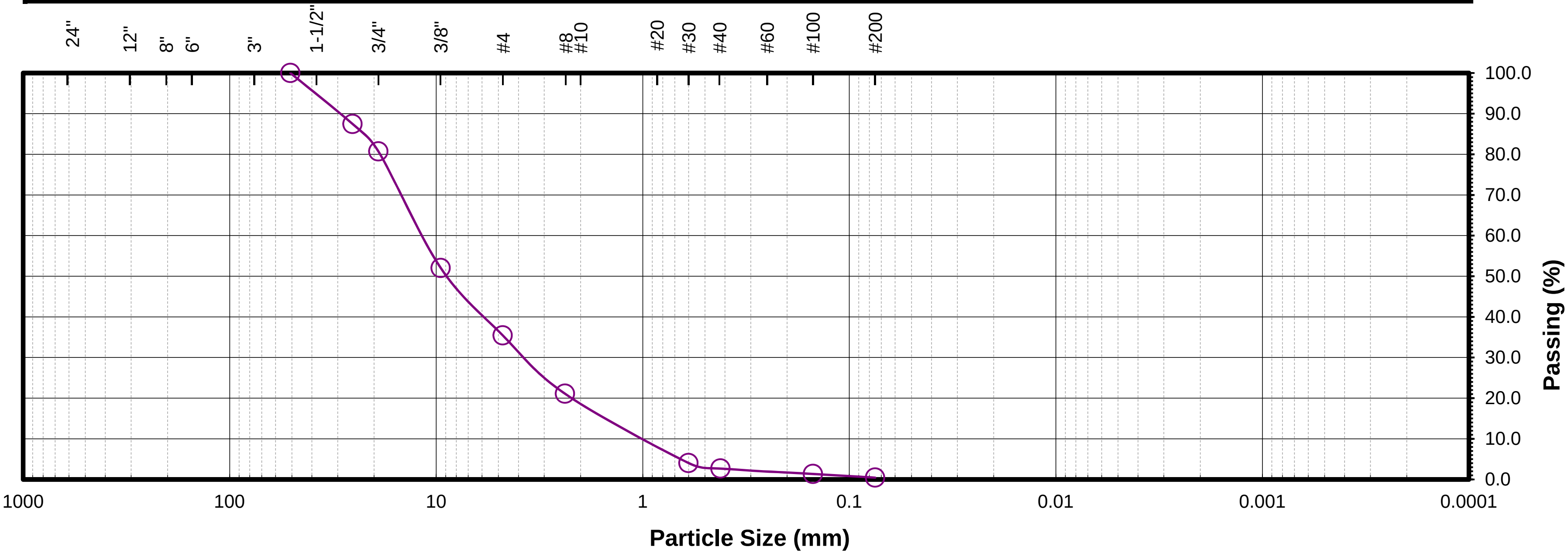
Horizon Engineering		**Project:**	112-3155	**Sample Date:**	10-Dec-14
114-2433 Dollarton Hwy		**Location:**	385 Gower Point Rd., Gibsons, BC	**Receive Date:**	17-Dec-14
North Vancouver, BC, V7H 0A1		**Sample Label:**	BH14-6 - G6	**Collected by:**	Pam

Data & Results

Technician: *Jason* **Date Tested:** 30-Dec-14

Sieve Type: Dry Sieve

Boulder	Cobble	Gravel - 64.6%		Sand - 35.0%			Fine Grain Soil - 0.4%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
11.87	1.20

D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
11.54	3.66	0.97	29.25

Comments: SANDY GRAVEL, medium to coarse grained sand, trace fine grained sand

Tested By JT

TESTING & MONITORING LTD.

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Tel: 604-770-1002
Fax: 604-770-1004

Sieve Analysis

Reference Number

Client

Horizon Engineering
114-2433 Dollarton Hwy
North Vancouver, BC, V7H 0A1

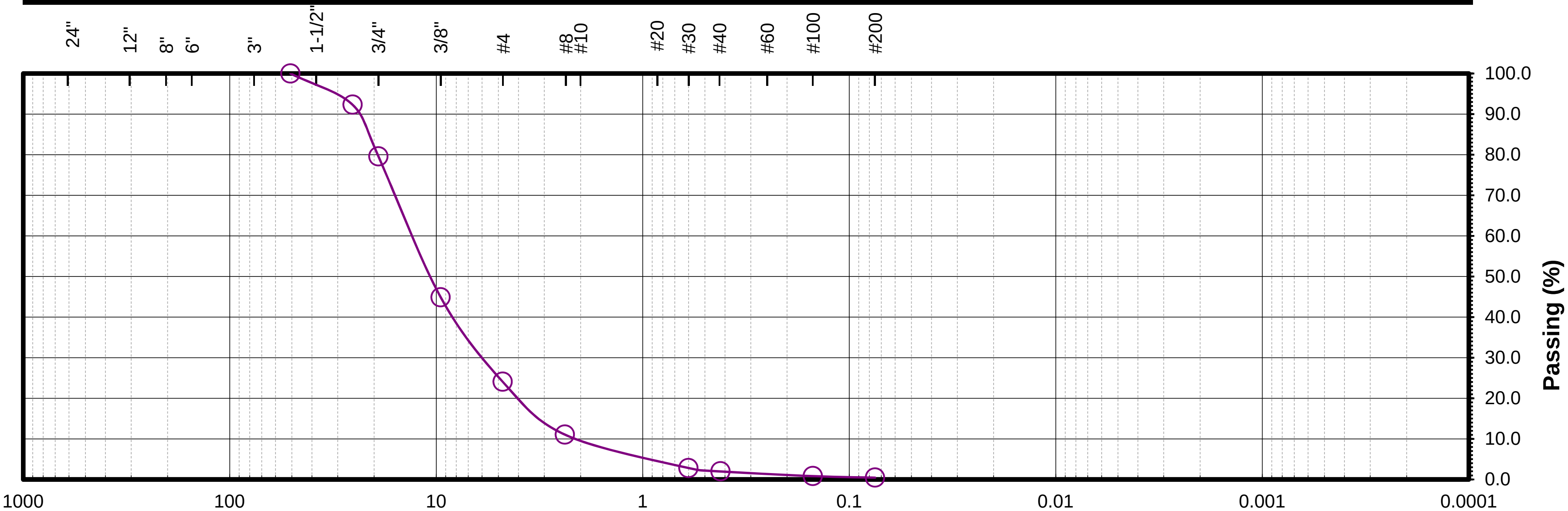
Project: 112-3155
Location: 385 Gower Point Rd., Gibsons, BC
Sample Label: BH14-6 - G7

Sample Date: 10-Dec-14
Receive Date: 17-Dec-14
Collected by: Pam

Data & Results

Technician: ***Clive***
Sieve Type: Dry Sieve
Date Tested: Dec 17-19, 2014

Boulder	Cobble	Gravel - 75.9%		Sand - 23.6%			Fine Grain Soil - 0.5%	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Coefficient of Uniformity and Curvature

C_u	C_c
6.43	1.31

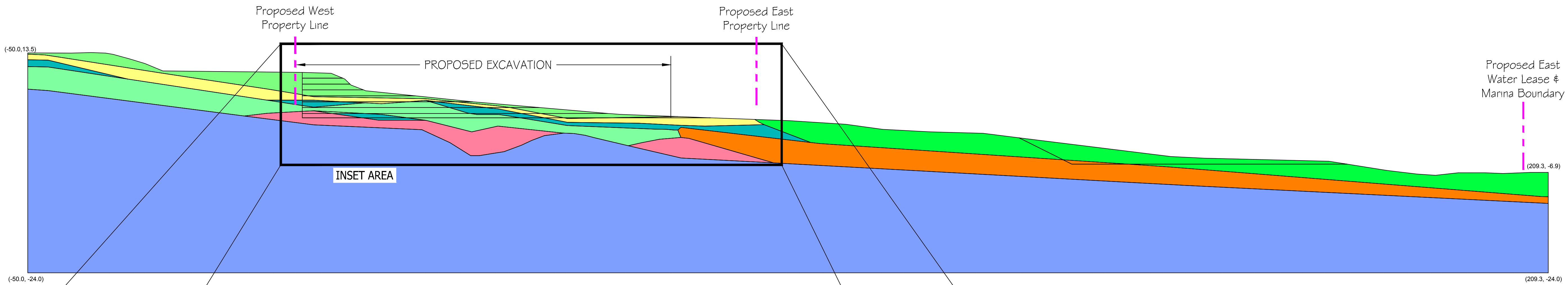
D60 (mm)	D30 (mm)	D10 (mm)	D90 (mm)
12.88	5.80	2.00	24.11

Comments: SANDY GRAVEL, some coarse grained sand, trace medium grained sand, trace fine grained sand, trace silt

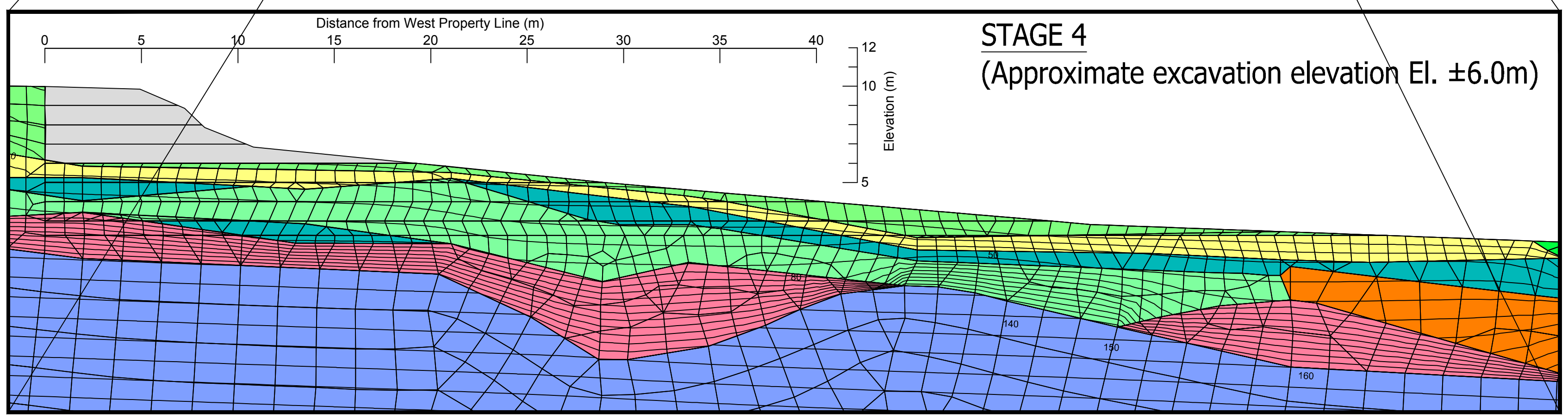
Tested By JT

APPENDIX E

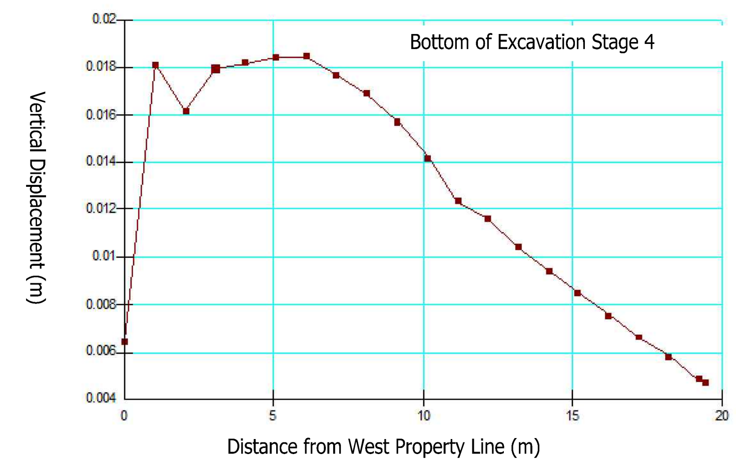
COMPUTER MODELLING RESULTS

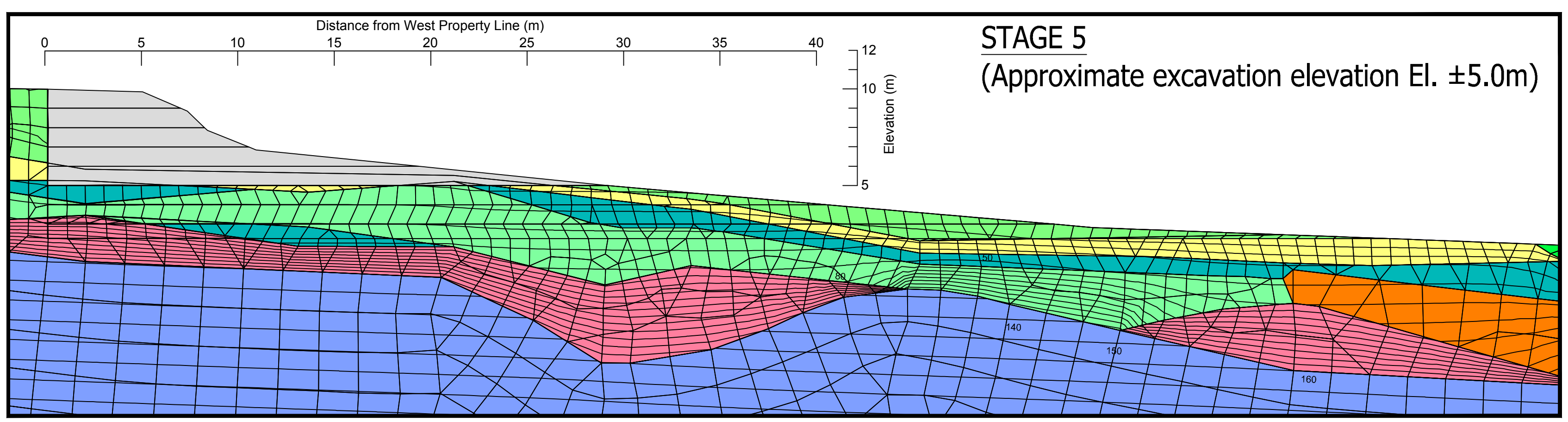


OVERALL MODEL

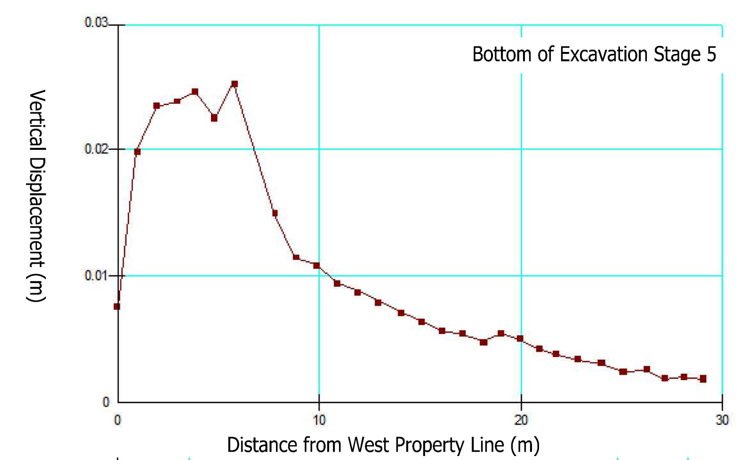


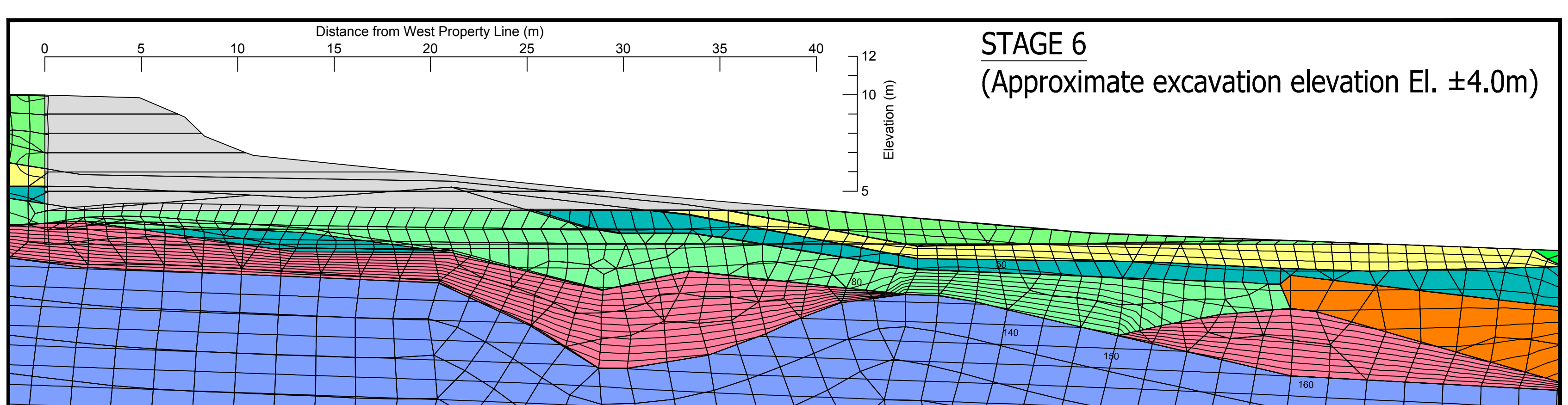
STAGE 4
(Approximate excavation elevation El. ±6.0m)



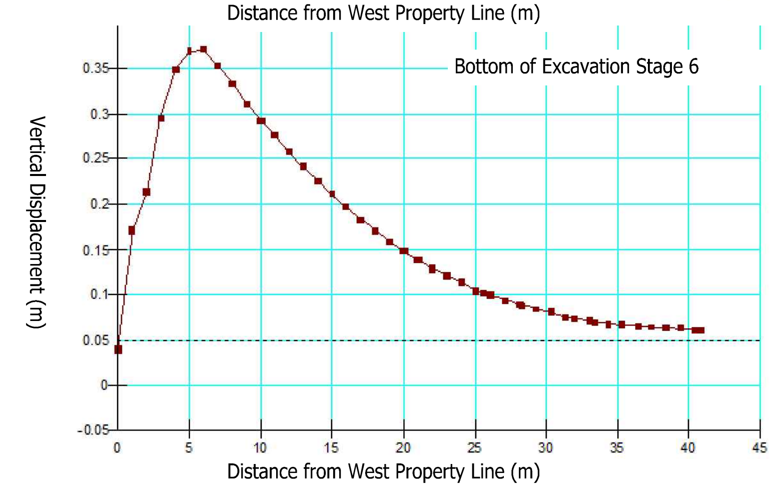


STAGE 5
(Approximate excavation elevation El. ±5.0m)





STAGE 6
(Approximate excavation elevation El. ±4.0m)



REFERENCE DRAWINGS
N/A

NO	DATE	REVISION
5		
4		
3		
2		
1		

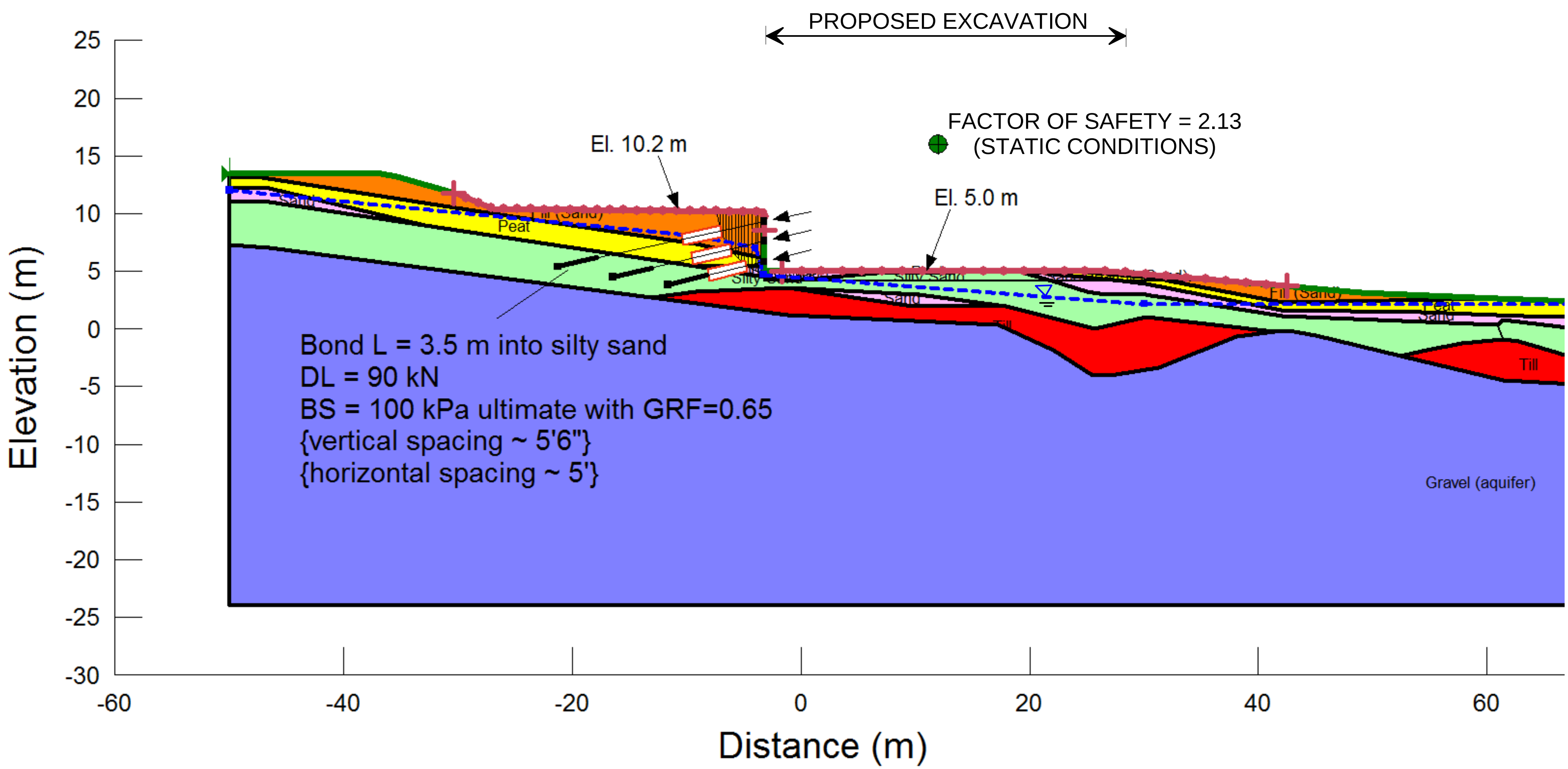
HORIZON ENGINEERING INC
114 - 2433 Dollarton Highway
North Vancouver, BC, V7H 0A1
Phone 604-990-0546
Fax 604-990-0583

Klaus Fuerniss Enterprises Inc.
c/o Art Phillips & Associates Inc.

PROPOSED "THE GEORGE" DEVELOPMENT
Gibsons, BC

COMPUTER MODELLING
OVERALL MODEL
and STAGES 4, 5 & 6

SCALE N.T.S.	DATE FEB/2015
DRAWN BB	CHECKED KK
DESIGN HS	ISSUED
FIGURE E1 OF 1	
JOB NUMBER 112-3155	



HORIZON ENGINEERING INC

KLAUS FUERNISS ENTERPRISES INC.
PROPOSED "THE GEORGE" DEVELOPMENT
TEMPORARY EXCAVATION SUPPORT: SHORING DESIGN

SLOPE STABILITY ANALYSIS RESULTS SECTION A

Scale: NTS	File No: 112-3155	Date: FEB/15	FIGURE: E2
Des: PB	Dwn: PB	Chk: KK	Rev:

APPENDIX F

LIQUEFACTION ASSESSMENT RESULTS

LIQUEFACTION ASSESSMENT

DEPTH		AH12-1		AH12-2		AH12-3		AH14-1		AH14-2		AH14-3		AH14-4		AH14-5		AH14-6	
(feet)	(metres)	SOIL TYPE	FOS	SOIL TYPE	FOS	SOIL TYPE	FOS	SOIL TYPE	FOS	SOIL TYPE	FOS	SOIL TYPE	FOS	SOIL TYPE	FOS	SOIL TYPE	FOS	SOIL TYPE	FOS
1	0.3	Sand		Peat		Peat		Peat		Sand		Sand		Sand		Sand		Sand	
2	0.6	Peat		Peat		Peat		Peat		Sand		Sand		Sand		Sand		Sand	
3	0.9	Peat		Peat		Peat		Peat		Sand		Sand		Sand		Sand		Peat	
4	1.2	Sand		Peat		Peat		Peat		Sand		Sand		Sand		Sand		Peat	
5	1.5	Sand	1.50	Peat	0.04	Peat		Peat		Sand	0.60	Sand	0.96	Sand	0.50	Sand	4.19	Peat	
6	1.8	Sand	3.31	Sand	1.62	Sand	0.62	Sand	1.91	Sand	1.01	Sand	1.06	Sand	0.73	Sand	1.45	Peat	
7	2.1	Sand	2.96	Sand	0.61	Sand	3.22	Sand	3.15	Sand	0.30	Peat		Sand	1.31	Sand	0.32	Sand	0.10
8	2.4	Sand	2.30	Sand	2.88	Sand	2.87	Sand	2.80	Sand	0.18	Peat		Sand	2.80	Sand	2.83	Sand	0.41
9	2.7	Sand	1.08			Sand	0.95	Peat		Sand	0.12	Peat		Sand	2.57	Sand	2.59	Sand	0.43
10	3.0	Sand	1.31			Sand	0.93	Peat		Clay		Peat		Sand	2.38	Sand	1.28	Sand	0.76
11	3.4	Sand	0.69			Sand	0.27	Peat		Clay		Peat		Sand	2.24	Sand	1.17	Sand	0.41
12	3.7	Sand	0.40			Sand	0.10	Sand	1.20	Peat		Peat				Sand	0.33	Sand	0.43
13	4.0	Sand	0.65			Sand	0.13	Sand	2.04	Peat		Peat				Sand	0.60	Sand	0.28
14	4.3	Sand	1.94			Sand	0.27	Sand	1.96	Sand	0.47	Peat				Sand	1.95	Sand	0.83
15	4.6					Sand	1.88	Sand	1.88	Sand	0.58	Peat				Sand	1.88	Sand	0.46
16	4.9					Sand	1.82	Sand	1.82	Sand	0.87	Sand	0.17					Sand	0.25
17	5.2							Sand	1.77	Sand	0.65	Sand	1.73					Sand	0.20
18	5.5							Sand	1.72	Sand	0.94	Sand	1.69					Sand	1.71
19	5.8							Sand	1.68	Sand	1.68	Sand	1.65					Sand	1.67
20	6.1									Sand	0.54	Sand	1.61						
21	6.4									Sand	0.60	Sand	1.58						
22	6.7									Sand	1.57	(outside excavation)							
23	7.0									Sand	1.54								
24	7.3									Sand	1.52								
25	7.6									Sand	1.49								
26	7.9									Sand	1.47								
27	8.2									Sand	1.45								

Notes:
1. FOS = Factor of Safety
2. Red line indicates approximate proposed foundation elevation (El. 5.0m / 16.4 ft)
3. Liquefaction is predicted where FOS < 1.0 (red shading)