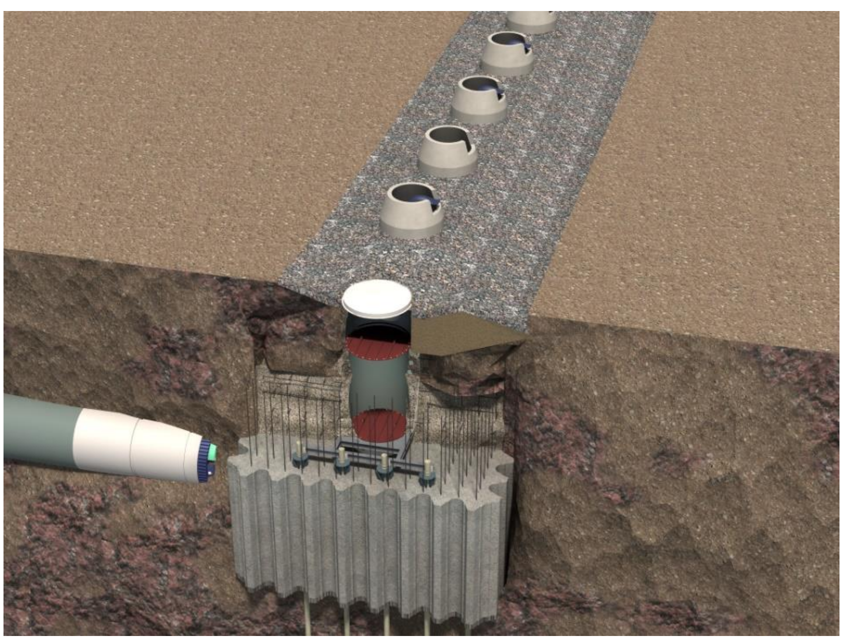
APPENDIX B
GEOTECHNICAL REPORTS

B.1: Geotechnical Data Report

Part A: Report

Annacis Island WWTP New Outfall System

Vancouver Fraser Port Authority
Project and Environmental Review Application



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25 January 2018

GEOTECHNICAL DATA REPORT

AIWWTP Transient Mitigation and Outfall System

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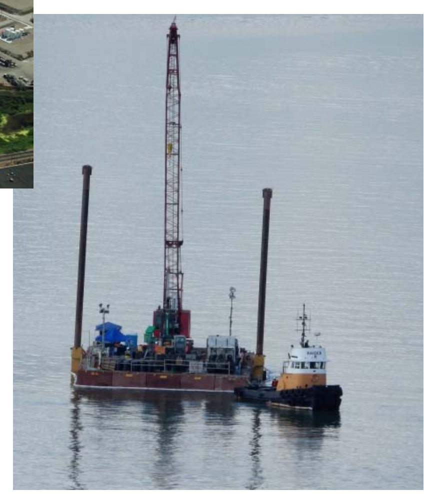

Report Number: 1525010-108-R-Rev0

Distribution:

1 Electronic Copy - CDM Smith Canada ULC
1 Electronic Copy - Golder Associates Ltd.

REPORT

Golder Associates

Record of Issue

Company	Client Contact	Version	Date Issued	Method of Delivery
CDM Smith Canada ULC	Mr. John Newby	Rev A	26 September 2017	SFT Upload
CDM Smith Canada ULC	Mr. John Newby	Rev 0	25 January 2017	SFT Upload

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by CDM Smith Canada ULC (CDM Smith) to provide geotechnical, environmental, and archaeological services for the Annacis Island Wastewater Treatment Plant (AIWWTP) transient mitigation and outfall system from pre-design (Phase A), to detailed design (Phase B) and through to construction (Phase C).

The predesign work generally involved discussions on route and options selection, type of outfall system (i.e. grade-supported, piled, and tunnelled). Golder has carried out geotechnical field investigations in two phases, including on-land and offshore boreholes and cone penetration tests along the proposed conceptual outfall alignments, referred to as the western and central alignments, as part of the conceptual study; this was implemented initially under a separate contract to Black & Veatch Canada, and subsequently under the predesign contract with CDM Smith. The field investigations during the predesign phase were focused on obtaining limited subsurface information along the conceptual alignments to assist with the route and option selection.

A western alignment, located 500 m west of the existing outfall, was initially selected as the preferred alignment following completion of the Phase II geotechnical investigation, and subsequently additional subsurface exploration (Phase III) was completed focusing on this preferred alignment to address data gaps, including hydrogeological testing. In addition, the Phase III investigation included a limited subsurface exploration at the potential future shaft location associated with the conveyance system from the Stage V expansion of the treatment plant to the proposed outfall.

The preferred alignment was subsequently shifted further east to a location some 200 m west of the existing outfall to allow the riser pipe and diffuser system to be located at a location within the river channel where the potential impacts due to sedimentation are minimized. This new preferred alignment is referred to as the “Option 6 Outfall Alignment”, and is the alignment finally selected for detailed design. Additional subsurface exploration (Phase IV) was carried out along the selected final Option 6 outfall alignment corridor.

This Geotechnical Data Report (GDR) contains descriptions of the investigation methods and factual data collected during the field investigations completed to date. It is noted that this report is prepared as a data report only and does not include any interpretation of the subsurface conditions at the site, assignment of design parameters, or geotechnical design and analyses of the proposed outfall system.

Golder was retained to also provide environmental and archeological services for the project. Deliverables from these disciplines are reported under separate cover.

This report should be read in conjunction with **“Important Information and Limitations of This Report”** which is appended following the text of the report. The reader’s attention is specifically drawn to this information, as it is essential that it is followed for the proper use and interpretation of this report.

2.0 SITE CONDITIONS AND PROPOSED DEVELOPMENT

The Annacis Island Wastewater Treatment Plant (AIWWTP) provides secondary treatment of wastewater to a significant number of residents in Metro Vancouver and is located on Annacis Island at 1299 Derwent Way, Delta, BC (see Figure 2-1). The AIWWTP is currently being expanded to increase the secondary treatment capacity and a new outfall is required to augment or replace the existing outfall facilities. The conceptual design recommended two new proposed gravity outfall options following central and western alignments to increase the capacity, as shown on Figure 2-2.

A single outfall located about 200 m west of the existing outfall, referred to as the Option 6 Outfall Alignment, was selected as the final preferred alignment. The proposed alignment traverses underneath the nearby buildings to allow the riser pipe and diffuser system to be located within the river channel at a position where potential impacts due to sedimentation are expected to be minimized. The alignment also traverses under a berthing dolphin supported by piles within the river channel. Based on available information, the tips of the piles for the dolphin are located about 5 m above the proposed tunnel crown. The Option 6 outfall alignment is also shown on Figure 2-2.

The outfall segment from the outfall shaft to the riser shaft, as well as a segment of the effluent conduit leading to the effluent shaft from the outfall shaft, which are together referred to herein as the outfall corridor, will be tunnelled. A new level control gate structure, near the existing Amil Gate, will also be constructed as part of the new outfall system. A riser shaft and a discharge pipe system, with a length of approximately 300 m, will be installed close to the navigational channel within the river to discharge the effluent. The discharge pipe system will be installed below mudline by dredging to the proposed grade.

The ground surface in the area surrounding of the AIWWTP is generally flat, with a nominal grade at El 104.5 m relative to the CVD28GVRD2005 datum. The CVD28GVRD2005 datum refers to geodetic datum plus 100 m. The ground surface remains generally flat or slopes gently towards the Fraser River along the proposed outfall corridor.

The original ground elevation at the Annacis Island site was about El. 100 m CVD28GVRD2005 and the site has been extensively modified through the placement of fill materials and land development for light-industrial and warehouse use. Maintenance dredging is regularly carried out within the river to maintain the navigation channel.

3.0 SITE GEOLOGY

Armstrong (1982[1]) reports that the area in which the subject site is located has been overridden during three or more periods of glaciation and that the area of present day Annacis Island was part of the seafloor at the mouth of the Fraser River prior to the first glaciation. The glaciers then moved into the area during Quaternary time and deposited a mixture of sand and gravel. Following the retreat of the glaciers, primarily fine-grained soils were initially deposited by the meltwater from the glaciers in a marine environment. The fine-grained soils were then subsequently consolidated and subsequent glaciers deposited a mixture of sand and gravel. Glaciofluvial sediments ranging from clayey silt to silty sand, grading into sand with thin layers of gravel with depth, were deposited during the last glaciation, the Fraser Glaciation. Clayey silt with interbedded gravel and gravelly sand was deposited towards the end of the Fraser Glaciation (i.e., Glaciomarine deposit).

The glacial deposits are typically encountered at a depth of 65 metres (m) or more below present ground surface at Annacis Island today. These deposits are hard or very dense due to the glacial loading and are inferred to outcrop within portions of the river channel, based on available information in the vicinity of the Alex Fraser Bridge.

Following the end of the Fraser Glaciation, fine-grained deposits comprising silt and clay were deposited in a marine environment. The silt and clay was then overlain by coarser deposits from the Fraser River, which consist primarily of sand and small amounts of fines. The relative density of the sand generally increases with depth. During flooding, clayey silt to silty clay containing varying amounts of organic material were deposited in areas adjacent to the main river channels; these deposits are referred to as overbank deposits and they are typically weak and compressible. Within the present day Fraser River channel, the overbank deposits were either never present or have subsequently been eroded due to the activity of the river.

The surficial geology of the project site is illustrated in the Surficial Geology Map (GSC No. 1484 A, 1980), as shown on Figure 3-1.

[1] Armstrong JE. 1982. "Geology of the Fraser Lowland with special reference to the area in the vicinity of the Annacis Island Bridge" (App F of Golder Associates report "Geotechnical Investigation Annacis Island Main Span", March 1982)

4.0 SUBSURFACE INVESTIGATIONS

Golder has carried out geotechnical investigations in four different phases along the conceptual, as well as the preferred Option 6 outfall alignments. The results of all investigation phases are included herein. The locations of the test holes put down during the field investigations are shown on Figure 4-1.

The geotechnical investigations included putting down mud-rotary boreholes (BHs), sonic holes (SHs), auger holes (AHs), Cone Penetration Tests (CPTs), Seismic CPTs (SCPTs). Standard Penetration Testing (SPT) and Large Penetration Testing (LPT) were carried within the mud-rotary boreholes. Split-spoon sampling during SPTs and LPTs and thin-wall tube sampling in fine-grained soils were also carried within the mud-rotary boreholes. In situ testing including down-hole shear wave velocity measurements, as well as Nilcon and electric vane shear tests were carried out within the fine-grained deposits. The details of the drilling and in situ testing methodology and the associated standards are provided in the following sections. The details of the field program carried out in each phase are provided in Section 5. Hydrogeological testing was also carried out as part of the Phase III and Phase IV geotechnical investigations and further details can be found in Section 6.

All field work was carried out under the full-time inspection of a member of our geotechnical staff, who identified the borehole and CPT locations in the field, logged the subsurface conditions encountered, and collected representative samples for detailed examination and laboratory testing.

4.1 Mud-Rotary Drilling and Sampling

All mud-rotary boreholes were drilled through the overburden soils underlying the site using a track-mounted drill rig, supplied and operated by Mud Bay Drilling Ltd. (Mud Bay). The locations of the BHs are shown on Figure 4-1. Each BH was drilled to a target depth below ground surface using the mud-rotary drilling technique described below.

- An initial piece of conductor (surface) casing was installed sufficiently into the ground surface to create a seal.
- The borehole was then advanced as follows:
  - A drill bit (tri-cone) attached to a hollow pipe is advanced into the deposit. A drill string comprised of AWJ rods to a maximum depth of 15 m, and subsequently heavier-walled NWJ rods were used for all sampling and testing.
  - The drilling technique involves pumping ‘mud’ through the rotating casing and out the end of the bit in order to lubricate the advancement of the drill, and to bring cuttings to the surface mud circulation tank via the surface casing. The ‘mud’ also prevents the hole from collapsing. Borehole advancement is achieved through the process of fracturing, shearing, and/or displacement depending on the type and consistency of the material encountered.

The mud-rotary drilling technique employed was completed generally in accordance with ASTM D5783-95, Standard Guide for Use of Direct Rotary Drilling with Water-Based Drilling Fluid for Geo-environmental Exploration. Detailed descriptions of the soil conditions encountered in the test holes are presented on the Record of Borehole sheets included in Appendix A.

4.1.1 Split-Spoon Sampling

Split-spoon sampling was completed using an open-ended split-spoon sampler to measure the penetration resistance values during advancement of the sampler, and at the same time, to obtain disturbed soil samples for geotechnical inspection and testing. The split-spoon sampling was completed generally in accordance with ASTM D1586-11, Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils. A 50.8 mm (2 inch) diameter split-spoon sampler was driven to a total penetration depth of 0.61 m (24 inches) or effective refusal per sample; the sampler was driven by a 63.5 kg hammer with a drop height of 762 mm. In specific selected boreholes, a large diameter penetration (LPT) sampler was utilized to allow for increased sampling in coarser materials and to obtain reliable penetration values. It is noted that the North American LPT (NALPT) sampler was utilized and it is generally 76.2 mm in diameter, driven to a total penetration depth of 0.61 m (24 inches) or effective refusal per sample with a hammer mass of 136 kg dropped over a height of 762 mm. The empirical correlation suggested by Daniel et al, 2003[2] can be used for NALPT to establish the equivalent SPT blow counts from the recorded LPT blow counts.

The recorded blow counts for individual test samples are presented on the Record of Borehole sheets in Appendix A. The compactness or consistency reported in the Record of Boreholes is generally based on the recorded blow counts. It is noted that many factors affect the recorded blow count value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), groundwater conditions, and grainsize. As such, the recorded blow count values should be considered only an approximate guide in assigning the compactness term. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction. It is also noted that the compactness was not reported in the Record of Boreholes for certain strata, where significant gravel or cobble-sized particles were encountered and the large split-spoon sampler was used, as the recorded blow counts are not considered representative and an engineering judgement in combination with laboratory testing is required to evaluate the compactness of those strata.

Fine-grained material consistency is described as noted in the Record of Boreholes by observation and the recorded blow count values are also only used as a guide in these deposits.

The split-spoon samples were measured for recovery, bagged, logged and described, and sent to Golder's warehouse for storage and review as required.

Energy measurements were carried out in accordance with ASTM D4633 during SPTs and LPTs within the granular soils underlying the site at four test hole locations to record the energy transfer ratio from the hammer to the drill rods. The results of the energy measurements summarized in two Technical Memoranda included in Appendix B.

4.1.2 Relatively Undisturbed Soil Sampling (Thin-walled Tube Samples)

Thin-walled tube samples of overburden materials were collected by piston sampling methods within cohesive soil deposits at depths selected by the Golder geotechnical inspector. Thin-walled tube samples are generally 0.61 m (24 inches) in length and 76.2 mm (3 inches) in diameter. Stainless steel, sharpened edge tubes were utilized to

[2] Daniel, Chris & Howie, John & Sy, Alex. (2003). A method for correlating large penetration test (LPT) to standard penetration test (SPT) blow counts. Canadian Geotechnical Journal. 40. 66-77. 10.1139/t02-094.

minimize the disturbance of the sample collected. Upon retrieval from the borehole, the ends of the tubes were wax-sealed, and capped for containment and moisture content preservation. The tubes were then placed in purpose-built protective containers to maintain their vertical position during on-site storage and transportation; they were ground-transported with care to reduce the potential exposure to sample handling and transportation-related disturbances. High-end soils laboratory testing was completed on sections of the relatively undisturbed soil samples, after extrusion and visual review of the open-ended tube samples.

All thin-walled tube sampling conducted on-site generally adhered to ASTM D1587/D1587M-15.

4.2 Sonic Drilling

Continuous sonic coring methods were utilized for the investigation purposes of geotechnical and hydrogeological testing and sampling, and polyvinyl chloride (PVC) standpipe monitoring well installation. Sonic drilling for this project was completed using a truck mounted Boart Longyear LS600-ATV2 rig, supplied and operated by Mud Bay. The locations of the SHs are shown on Figure 4-1.

Sonic drilling utilizes a dual-cased single tube core barrel system that employs high frequency mechanical vibration to obtain near-continuous core samples of the soils. The drilling technique involves vibrating the entire drill string at a frequency rate between 50 and 150 cycles per second, adjusted during operation to suit the ground conditions encountered. The technique employs vibration along with low speed rotational motion and downward pressure to advance the drill string. Drill hole advancement is achieved through the process of fracturing, shearing, and/or displacement depending on the type and consistency of the material encountered. The soil enters the core barrel, generally providing 102 mm diameter near-continuous core samples. Upon completion of each drill run, the outer steel casing is advanced to the end of the run, the core barrel and drill rods are removed, and the continuous sonic core sample is vibrated out of the core barrel directly into a plastic sample bag before being transferred into wooden core boxes. The sample is typically highly disturbed, and can be either compressed or expanded during the drilling and retrieval process; soils can also be displaced if the casing becomes blocked during advancement, in which case core is not retrieved from that horizon.

Detailed descriptions of the soil conditions encountered in the sonic holes are included on the sonic hole records in Appendix A, and photographs of sonic hole cores are included in Appendix C.

4.3 Cone Penetration Testing

Static cone penetration testing was carried out utilizing specialized drill rigs supplied by ConeTec Investigations Ltd. (ConeTec) at the various locations as shown on Figure 4-1 throughout the four phases of investigation. A 15-ton compression-type cone with a tip area of 15 cm^2 and a friction sleeve area of 225 cm^2 was used for all soundings. The detailed cone information utilized for the various phases of work can be found on the CPT testing reports provided in Appendix A.

All compression cones are designed with an equal end area friction sleeve and an approximate tip area ratio of 0.80. A pore water pressure filter is located directly behind the cone tip in the u_2 position on the cone. The cone recorded the following parameters at regular depth intervals at each CPT location:

- tip resistance (q_c)

- sleeve friction (f_s)
- dynamic pore water pressure (u)

Cone penetration testing was generally carried out in accordance with ASTM D5778-07. A set of baseline readings were taken prior to, and at the completion of, each sounding to assess temperature shifts and any zero load offsets.

Graphical plots of the CPT data, including the cone tip resistance (q_t), sleeve friction (f_s), dynamic pore pressure (u), and friction ratio (R_f) are presented on the Record of Cone Penetration Tests in Appendix A. An inferred stratigraphic log is also included on the CPT plots for information purposes only. The stratigraphic interpretations shown are based on relationships between q_t, f_s, u and R_f as summarized by Lunne et al. (1997)[3].

4.4 In Situ Testing

At selected borehole locations, in situ Nilcon and electric vane shear tests were carried out within the underlying cohesive soils to collect soil strength data. Dynamic pore pressure dissipation tests were carried out at selected depths during the CPTs. Down-hole shear wave velocity measurements were carried out at selected CPT locations and in select mud-rotary boreholes and sonic holes.

A brief summary of the Nilcon and electric vane testing procedures, pore pressure dissipation tests, and down-hole shear wave velocity measurements is provided in the following sections.

4.4.1 Nilcon Vane Test

Field Nilcon vane testing was performed within the borehole using a Nilcon vane having the capacity to measure undrained shear strength up to 210 kPa. In most cases a 'small' vane measuring 12.7 cm x 5.1 cm with a tapered bottom end was used, especially for the tests carried out at depth. The vane was attached to a steel rod using an adapter, which was coupled with the Nilcon readout apparatus. After initial shearing, remoulded strengths were measured by rotating the vane ten (10) revolutions. The Nilcon vane tests were conducted in accordance with ASTM D2573. The field traces of the Nilcon vane tests are included in Appendix E.

4.4.2 Electric Vane Test

During the Phase IV investigation, due to technical issues with the Nilcon vane at BH16-06, an electrically powered vane was utilized on site to carry out three in situ undrained shear strength tests. The electric vane test is very similar to the Nilcon vane test with the exception that the vane head that is placed at the surface is an electric vane head that records the data digitally. The field traces of the electric vane tests are included in Appendix E.

4.4.3 CPT Pore Water Pressure Dissipation Test

At select depths within the CPT holes, pore water pressure dissipation tests were performed by stopping the advancement of the CPT probe and allowing the pore water pressure sensor to record the dissipation of pore water pressures over time. Pressure recordings were taken at 5-second intervals. Dissipation testing was

[3] Lunne T, Robertson PK, Powell JJM. 1997. "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

performed for a sufficient length of time to allow dissipation of 50 percent of the excess pore water pressure (t_{50}). The dissipation test results are summarized in a report prepared by ConeTec included in Appendix D. Users are responsible for interpretation of the raw dissipation data.

4.4.4 Shear Wave Velocity Measurements

Down-hole shear wave velocity measurements were carried out at selected CPT locations and in selected borehole and sonic holes as described below.

- SCPT Testing

  Shear wave velocity testing was performed in conjunction with the piezocone penetration test in order to collect interval velocities. ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 metres behind the cone tip.

  Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPT data acquisition system. Down-hole shear wave velocity measurements were generally recorded out at 1-m intervals at all SCPT locations.

  The results of the shear wave velocity measurements including shear wave arrival times are summarized in plots prepared by ConeTec, included in Appendix A.

- Geophysical Downhole Seismic Testing

  Downhole seismic testing (DST) is generally conducted using a system comprising a surface source, a downhole tool equipped with a triaxial geophone package, and a data acquisition system. The downhole tool has a triaxial geophone package mounted on an internal block such that the orientation of the geophones can be maintained within the borehole through the use of the built-in fluxgate compass and servo motor system. A motor driven bow spring clamp is used to couple the downhole tool with the borehole wall. The downhole seismic test equipment is in general accordance with the current ASTM D7400 standard.

  Shear waves (Vs) are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. The hammer and beam (or plate) act as a contract trigger that initiates the recording of the seismic wave traces. The beam is generally struck on each end to generate horizontally polarized shear waves. The traces are recorded using an up-hole data acquisition system.

  Geophysical down-hole shear wave measurements were carried out using tri-axial geophones within a purpose-installed PVC casing during the Phase II investigation.

  In an effort to obtain Vs data at depth at reduced cost, downhole shear wave velocity testing was also carried out within the steel sonic casing during the Phase III and IV investigations. The downhole seismic testing was generally carried out in accordance with ASTM D7400; however, due to the use of a steel sonic casing the built in fluxgate compass and servo motor system could not be utilized to maintain the orientation of the geophones. The geophone orientation was maintained by using the motor driven spring clamp to secure the geophone within the casing and subsequently lifting the geophone without releasing the clamp. It was determined, through comparison of the down hole Vs data obtained from the measurements within the sonic casing, paired with SCPT locations and compared with general known Vs trends for the Fraser River sand, that there is an acceptable correlation of Vs data obtained within the sonic steel casing compared to PVC casing.

4.5 Permits and Private Property Access Agreements

The field investigations were carried out on both public and private property; therefore, municipal permits as well as private property access agreements were required for the drilling of the test holes.

A Highway Use Permit was obtained from the Corporation of Delta (Delta) for investigation within public roadways. Where required, Golder retained Valley Traffic Systems based out of Langley, BC to assist with traffic control requirements. Traffic control plans were submitted to Delta prior to initiation of the drilling program. A City Road and Highway Use Permit was required for three of the test hole locations, which was obtained by Golder from Delta as required. Copies of the Highway Use Permit and the Consent to Enter Property form were kept on file and with field staff on site during the field investigations.

In general, access to the test holes located on the private property was negotiated by Metro Vancouver. An access agreement was executed by Metro Vancouver and the private property owner was notified of the proposed testing before commencement of the field program. Contact information and confirmation of acceptance of the agreement was provided to Golder prior to accessing the site.

An access agreement to allow drilling to be carried out in the Fraser River, was obtained and executed by Metro Vancouver during the Phase II investigation; however, this agreement was obtained and executed by Golder during the Phase IV investigation. The access agreement included a property access agreement and an environmental management permit application.

4.6 Utility Clearance

Prior to drilling, Golder completed a BC One Call, and contacted Metro Vancouver (MV) and the Corporation of Delta to confirm the location of buried utility locations. Golder retained Western Utility Locate Services (Western), based in Coquitlam, BC, to assist in the field location of nearby utilities and other potential subsurface obstructions. Western uses electronic locating equipment and ground penetrating radar to identify and confirm the location of utilities in the field. Field inspectors from Western and Golder carried out a site reconnaissance at each proposed test hole location prior to drilling to confirm overhead and underground utility locations, and to mark the test hole locations in the field.

Also, for the purposes of underground utilities clearance, hydro-vacuum clearance of the majority of the test holes, to approximately 2 m below ground surface, was used as the test holes were generally located in a developed, industrial site. The hydro-vacuum clearance hole was completed using a hydro-vacuum truck operated by Badger Daylighting Inc. All boreholes within the AIWWTP required hydro-vacuum clearance and therefore, data in the fill materials, especially within the upper 2 m is generally limited. For geotechnical classification purposes, the sidewall of the hole was observed for visual soil descriptions and limited samples collected using a scoop in the sidewalls for further inspection and laboratory testing.

4.7 Test Hole Location Surveying

Surveying of the onshore test hole locations was carried out by Metro Vancouver and the coordinates were provided to Golder. All test hole locations were surveyed in UTM NAD83 Zone 10 and are presented in ground coordinates. The conversion factor from UTM coordinates to ground coordinates is 1.000397558. Please note that all elevations are in metres and in reference to CVD28GVRD 2005, which is Geodetic datum plus 100 metres. Locations are shown on Figure 4-1 for the test holes completed and are based on the surveyed coordinates.

The offshore borehole and CPT locations were located and surveyed using a Trimble RTK surveying system provided by ConeTec. This survey equipment has an accuracy of approximately 50 mm.

4.8 Test Hole Completion

Upon completion, the mud-rotary, sonic and auger holes, and the CPTs were grouted per the requirements of the British Columbia Groundwater Protection Regulations. With the exception of the mud-rotary boreholes and CPTs carried out during Phase I, all test holes were completed using cementitious grout. The test holes during Phase I were completed using environmentally safe bentonite in accordance with BC Groundwater Protection Regulations.

There were a number of boreholes and sonic holes in which nested standpipe monitoring wells and nested vibrating wire piezometers were installed. The vibrating wire piezometers were installed using cementitious grout; however, the standpipe piezometers were completed utilizing a combination of sand and bentonite chips and pellets. The locations of the standpipes and vibrating wire piezometers are identified on Figure 4-1 and listed in a summary table in Section 6.

Backfill details and installation details are shown on the Record of Borehole sheets in Appendix A.

5.0 GEOTECHNICAL INVESTIGATION PROGRAMS

Test holes were put down at locations based on the scopes of work provided and as discussed by Golder and the client. In general, test hole locations were located in the field based on the following:

- avoidance of buried services
- avoidance of potential conflicts with other working contractors
- traffic safety (distance from intersections, railroad crossings, safe sight distances, etc.)

The field exploration program completed in each phase is described below.

5.1 Phase I Geotechnical Investigation

The Phase I investigation was conducted between 2 and 17 July 2015 consisting of four mud-rotary boreholes (BHs), two Cone Penetration Tests (CPTs), and one Seismic CPT (SCPT) along the western and central outfall alignment corridors as summarized in Table 5-1.

Standard Penetration tests (SPTs) were carried out at regular intervals of depth (1.5 m) within the mud-rotary boreholes to record the in situ relative density and to collect representative soil samples for geotechnical laboratory index testing. Energy measurements were carried out by Golder during the SPTs within the top 30 m at BH15-04 and BH15-14 to record the energy transfer ratio from the hammer to the drill rods.

Nilcon vane shear tests were carried out at selected depths within the fine-grained soils to collect undrained shear strength data. Thin-walled piston tube samples were also collected at selected depths within fine-grained soils to collect undisturbed soil samples for one dimensional consolidation testing.

Table 5-1: Phase I Field Investigation Program Summary

Test Hole	Location Description	Ground Surface Elevation (m)	Depth (m)	Remarks
CPT15-15	AIWWTP Entrance	103.7	55.0	
BH15-13		103.8	54.9	
BH15-14	Derwent Place	104.2	54.9	Energy measurements during SPTs
BH15-04	Fraserview Place	104.1	54.9	Energy measurements during SPTs
CPT15-04		104.0	30.7	
SCPT15-06	Private Parking Lot (1425 Derwent Way)	103.8	48.7	
BH15-05		103.8	54.9	

5.2 Phase II Geotechnical Investigation

The Phase II Investigation, comprising both offshore and on-land investigations, was carried out between 16 September 2015 and 13 October 2015. The offshore investigation consisted of four mud-rotary BHs paired with CPTs/SCPTs, and the on-land investigation consisted of two mud-rotary BHs paired with CPTs as summarized in Table 5-2.

The Phase II investigation focused on geotechnical investigations within the Fraser River and along the shoreline locations of both proposed alignments at the time. Energy measurements were carried out by Golder during the SPTs within sandy soils and during the LPTs in gravelly soils at BH15-01 and BH15-03 to record the energy transfer ratio from the hammer to the drill rods. The results of the energy measurements are summarized in a Technical Memorandum included in Appendix B. Down-hole shear wave velocity measurements were carried out in two of the offshore CPTs and one on-land CPT. Down-hole shear wave measurements were also carried out using tri-axial geophones within a PVC casing installed at BH15-03, located along the western alignment.

Standard Penetration tests (SPTs) were carried out at regular intervals in all mud-rotary boreholes to record the in situ relative density and to collect representative soil samples for geotechnical laboratory index testing. Large penetration tests (LPTs) were carried out at regular interval within gravelly layers, where encountered to record the in situ relative density and to collect representative samples for laboratory testing. Nilcon vane shear tests were carried out at selected depths in fine-grained soils to collect data on the undrained shear strength of the soil. Thin-walled piston tube samples were also collected at selected depths within fine grained soils to collect undisturbed soil samples for advanced laboratory testing such as cyclic and monotonic direct simple shear testing.

Due to poor recovery within the gravelly layer during the drilling of BH15-01, an additional location, BH15-01B was drilled directly adjacent to BH15-01 to allow for additional sampling. The bottom 4.5 m of BH15-01 and BH15-03 were cored utilizing HQ coring techniques to increase the amount of soil recovered in the very dense deposits encountered at depth.

Table 5-2: Phase II Field Investigation Program Summary

Test Hole	Location Description	Ground Surface / River Bed Elevation (m)	Depth (m)	Remarks
SCPT15-01	Fraser River	90.8	34.3	
BH15-01		90.9	46.5	Energy measurements during SPTs and LPTs
BH15-01B		90.8	32.1	
CPT15-02	Fraser River	94.5	37.2	
BH15-02		93.4	36.7	
CPT15-03	Private Parking Lot (401 Fraser View Pl)	104.4	29.0	
BH15-03		104.4	58.2	Energy measurements during SPTs and LPTs and Downhole Vs measurements
SCPT15-11	Fraser River	89.1	55.2	
BH15-09		88.9	80.1	
CPT15-12	Fraser River	92.0	45.0	
BH15-10		91.8	43.6	
SCPT15-13	Private Property (Near 450 Derwent Place)	104.2	58.0	
BH15-11		104.2	58.5	

5.2.1 Phase II Offshore Investigation

The offshore investigation program under Phase II was carried out between 16 September 2015 and 22 September 2015 and consisted of four mud-rotary boreholes paired with CPT/SCPTs, as summarized in Table 5-2. The investigation was carried out round-the-clock with two 12-hour shifts.

A floating spud barge operated by Golder was utilized to provide a suitable drilling platform for the offshore work. The mud-rotary boreholes were put down using drilling equipment supplied and operated by Mud Bay, and the CPTs and SCPTs were advanced using specialized equipment supplied and operated by ConeTec, using Mud Bay's drill rig. The test holes were located using an RTK Trimble GPS unit with an accuracy of about 50 mm supplied by ConeTec. The GPS unit supplied by ConeTec was also used to establish sampling depths during drilling to account for tidal variations.

SCPT15-01 and CPT15-02 were advanced to depths of 34.3 m and 37.2 m, respectively, and terminated within a dense layer upon effective refusal. A gravel layer was encountered at a depth varying from 20 to 25 m at the SCPT/CPT locations, requiring drill outs to further advance the cone through the underlying fine-grained soils before reaching the termination depth. BH15-01 and BH15-02 were terminated within a dense layer at depths of about 46.5 m and 36.7 m below mudline, respectively.

The gravel layer was encountered, along the 'western alignment' overlying a fine-grained deposit at BH15-01 and the sample recovery was poor in the gravelly soils; there was no recovery during the first sample within the fine-grained soils, indicating possible cobbles or larger size materials advancing in front of the drill bit and sampler. The borehole was advanced to the termination depth, with the larger size materials pushed aside as the drill bit advanced through the fine-grained soils. However, the possible influence of the cobbles in the LPTs carried out in the gravelly soils was a concern, and therefore, upon completion of BH15-01, a secondary hole, BH15-01B was drilled approximately 4.0 m away from BH15-01. Sampling was carried out in this supplementary hole only from about 23.8 to 32 m depth below mudline in order to recover appropriate samples and LPT values through the gravelly layer, and to delineate the top of the fine-grained deposit. The borehole was terminated after collecting a sample within the fine-grained soils at a depth of 32 m below mudline.

5.2.2 Phase II Onshore Investigation

The onshore field investigation focused on the foreshore area along the western and central alignments. Work was carried out between 3 and 13 October 2015; the work consisted of two mud-rotary boreholes paired with CPT/SCPTs, as summarized in Table 5-2.

CPT15-03, located along the western alignment, was carried out to practical refusal on a gravelly layer at a depth of about 29 m below ground surface, and the corresponding borehole BH15-03 was terminated at a depth of 58.2 m below ground surface within a dense layer. Coring a length of 8 m into the dense layer was also carried out to collect continuous samples and to carry out down-hole shear wave velocity (V_s) measurements within the dense layer. The paired SCPT15-13 and BH15-11 located along the central alignment were terminated at a depth of about 58 m below ground surface within the fine grained deposit.

5.3 Phase III Geotechnical Investigation

The Phase III investigation was conducted between 20 March 2016 and 29 May 2016; it consisted of five mud-rotary boreholes, four sonic boreholes, three auger holes and five SCPT locations, as summarized in Table 5-3. The test-hole locations are shown on Figure 4-1. The mud-rotary borehole investigation was carried out round-the-clock in two 12-hour shifts, while all other work was carried out utilizing day shifts only.

During the SCPT, tip bearing, sleeve friction, and pore pressure measurements were recorded at 5-cm intervals and down-hole shear wave velocity measurements (Vs) were also carried out at 1-m depth intervals.

Standard Penetration tests (SPTs) were carried out at regular intervals of depth within the mud-rotary boreholes to record the in situ relative density and to collect representative soil samples for geotechnical laboratory index testing. Nilcon vane shear tests were carried out at selected depths within the fine-grained soils to collect undrained shear strength data. Thin-walled piston tube samples were also collected at selected depths within fine-grained soils to collect undisturbed soil samples for advanced laboratory testing.

Near-continuous sonic core was collected and photographed to assist with stratigraphic profiling at the proposed shaft locations. Down-hole shear wave measurements were carried out using tri-axial geophones mounted within the steel sonic casing at SH16-02.

Table 5-3: Phase III Field Investigation Program Summary

Test Hole	Location Description	Ground Surface Elevation (m)	Depth (m)	Remarks
BH16-01	AIWWTP Entrance	104.8	90.5	
SH16-01		105.1	75.0	
BH16-02	Station 0+150	105.9	60.1	
SCPT16-01		105.9	78.0	
BH16-03	Private Parking Lot (1425 Derwent Way)	103.8	77.7	
SH16-02		103.8	76.5	Downhole Vs measurements in sonic casing
SCPT16-05		103.8	64.0	
BH16-04	AIWWTP	104.3	57.9	
SCPT16-03		104.2	63.0	
BH16-05	AIWWTP	103.8	55.6	
SCPT16-04		103.8	70.4	
AH16-01	End of Eaton Place	104.2	12.2	
SCPT16-02		104.2	60.0	
AH16-02	AIWWTP	103.8	9.1	
AH16-03	AIWWTP	105.1	12.2	

BH16-01 was carried out to 90 m depth below ground surface. Sampling at regular intervals was carried out to 76.5 m and the borehole was then advanced to the extent of the available drill rods at site to evaluate if there was any significant stratigraphic change, similar to that encountered at BH16-03. BH16-03 was terminated at 77.7 m within in a very hard silty clay layer.

SCPT16-05 reached practical refusal in a hard, fine-grained deposit at 64 m depth. SCPT16-01 to SCPT16-04 were advanced to a range of depths ranging from 60 to 78 m below ground surface, terminating in fine-grained deposits.

SH16-01 and SH16-02 were advanced to depths of approximately 75 m below ground surface. Due to the refusal of SCPT16-05, located near SH16-02, at a depth of 64 m below ground surface, down-hole shear wave velocity testing was also carried out within the sonic casing to obtain the velocity measurements at depth. Readings were carried out at 2-m intervals from 24 to 48 m depth, and at 1-m intervals from 48 to 75 m depth. Sonic core was collected and retained for detailed logging and photographing. In general, sonic coring was carried out in 1.5-m intervals, except through the sand deposits where using a 3-m interval significantly increased the ability to retain the soil core. SH16-03 and SH16-04 were carried out to install monitoring wells and to confirm the presence and the thickness of the previously noted gravelly layer at depth, encountered at other drilling locations near shore and within the river channel along the western alignment corridor. Sonic core was only logged and retained from approximately 25 to 50 m below ground surface, where the gravel layer was expected to be present.

5.4 Phase IV Geotechnical Investigation

The Phase IV geotechnical investigation was conducted between 21 November 2016 and 19 December 2016, and consisted of three mud-rotary boreholes, three sonic holes, and five SCPT locations as summarized in Table 5-4. The test-hole locations are shown on Figure 4-1.

During the SCPT, tip bearing, sleeve friction, and pore pressure measurements were recorded at 5-cm intervals and down-hole shear wave velocity measurements (Vs) were also carried out at 1-m depth intervals.

Standard Penetration tests (SPTs) were carried out at regular intervals of depth within the mud-rotary boreholes to record the in situ relative density and to collect representative soil samples for geotechnical laboratory index testing. Nilcon and electric vane shear tests were carried out at selected depths within the fine-grained soils to collect undrained shear strength data. Thin-walled piston tube samples were also collected at selected depths within fine-grained soils to collect undisturbed soil samples for advanced laboratory testing.

Near-continuous sonic core was collected and photographed to facilitate stratigraphic profiling at the proposed shaft location.

Down-hole shear wave velocity measurements were carried out using tri-axial geophones mounted within the steel sonic casing at SH16-05, SH16-06 and SH16-07.

Table 5-4: Phase IV Field Investigation Program Summary

Test Hole	Location Description	Ground Surface / River Bed Elevation (m)	Depth (m)	Remarks
BH16-06	Private Parking Lot (1365 Derwent Way)	103.7	60.1	
SH16-05		103.6	90.5	Downhole Vs measurements in sonic casing
SCPT16-06		103.7	62.7	
SCPT16-07	North/West side of Private Parking Lot (1302 Derwent Way)	103.9	66.5	

Test Hole	Location Description	Ground Surface / River Bed Elevation (m)	Depth (m)	Remarks
BH16-07	South/West side of Private Parking Lot (1302 Derwent Way)	103.9	77.7	
SCPT16-08		103.9	76.5	
SH16-06	Nearshore – Private Parking Lot (450 Derwent Place)	104.2	90.2	Downhole Vs measurements in sonic casing
SCPT16-09		104.1	73.7	
SH16-07	AIWWTP – near a potential future shaft location	103.5	90.2	Downhole Vs measurements in sonic casing
BH16-08	Fraser River	87.3	55.5	
SCPT16-10		87.3	51.7	

5.4.1 Phase IV Offshore Geotechnical Investigation

The offshore investigation was carried out between 15 December 2016 and 16 December 2016. The investigation included the drilling of a mud-rotary borehole and the advance of one SCPT at the proposed riser shaft location. The investigation was carried out round-the-clock in two 12-hour shifts. A floating spud barge operated by Golder was utilized to provide a suitable drilling platform for the offshore work. The work was performed during a significant cold spell and high tide volumes, resulting in slower than expected drilling, and steps had to be taken to appropriately care for the crew and undisturbed samples collected during the drilling.

SCPT16-10 was advanced to a depth of 51.7 m below mudline where at it was terminated at refusal due to increased tip resistance and rod bend. No drill-outs were required; however, the hole was carried out in two separate pushes, with a casing installed after the first 15 m to provide additional rod support to allow advance of the CPT through the underlying fine-grained material. SCPT16-10 was terminated within a dense layer at the extent of the push depth.

BH16-08 was advanced to a depth of 55.5 m below mudline. The borehole was terminated upon sufficient confirmation that the dense soils underlying the fine-grained sediments had been reached. Sampling was carried out at 1.5-m depth intervals for the upper 42 m of the borehole; the sample spacing was increased at depth, to be able to reach the target depth in a reasonable time; the target depth was based on the observations noted in the adjacent CPT.

5.4.2 Onshore Geotechnical Investigation

The onshore field investigation was carried out between 21 November 2016 and 14 December 2016, and consisted of two mud-rotary boreholes, three sonic holes, and four SCPT locations as summarized in Table 5-4.

The SCPT tests were carried out along the Option 6 alignment to supplement the geotechnical data for the final outfall design. The tests were spaced at approximately 150 m between the foreshore and the proposed Outfall Shaft location. The tests were advanced to termination in a dense layer at depths below ground surface ranging from 62.7 to 76.5 m. Refusal was observed by noting a rapidly increased tip resistance and/or significant rod bending.

BH16-06 was drilled to 60 m depth below ground surface, with sampling at regular depth intervals of 1.5 m. BH16-07 was advanced to 77.7 m below ground surface, with sampling at regular intervals to 55 m; the borehole was then advanced with limited sampling to confirm the soil conditions at depth and to correlate with the stratigraphy changes at depth as noted in the adjacent SCPT16-08. Both boreholes were terminated in a very hard silty clay layer.

5.5 Results of Field Vane Tests

The results of the field Nilcon vane tests are provided in Table 5-5. The test results are also presented on the Records of Boreholes in Appendix A. The raw traces of the Nilcon and electric shear vane test are included in Appendix E.

Table 5-5: Nilcon Vane Test Results Summary

Test Hole	Test Depth (m)	Vane Size	Shear Strength (kPa)	
			Peak	Remoulded
BH15-01	33.2	Small	150[1]	Not Available
BH15-01B	30.8	Small	133[1]	Not Available
BH15-04	3.9	Small	111[1]	31[1]
BH15-04	44.8	Small	103[2]	208[2]
BH15-05	50.7	Small	160[1]	48[1]
BH15-13	49.5	Small	185[1]	88[1]
BH15-13	53.2	Small	138[1]	86[1]
BH16-01	5.3	Small	66[1]	18[1]
BH16-01	54.0	Small	FVT Capacity Exceeded	Not Successful
BH16-01	57.9	Small	FVT Capacity Exceeded	Not Successful
BH16-01	62.0	Small	FVT Capacity Exceeded	Not Successful
BH16-02	50.2	Medium	FVT Capacity Exceeded	62[1]
BH16-02	54.1	Medium	FVT Capacity Exceeded	70[1]
BH16-02	56.7	Small	142[1]	79[1]
BH16-02	60.4	Small	FVT Exceeded	102[1]
BH16-03	4.0	Medium	33[1]	2.7[1]
BH16-04	3.7	Med	42[1]	9[1]
BH16-04	50.0	Small	113[1]	48[1]
BH16-04	54.0	Small	FVT Capacity Exceeded	101[1]
BH16-04	57.9	Med	169[1]	22[1]
BH16-05	3.5	Med	43[1]	13[1]
BH16-05	45.0	Small	118[1]	27[1]

Test Hole	Test Depth (m)	Vane Size	Shear Strength (kPa)	
			Peak	Remoulded
BH16-05	47.1	Small	120[1]	73[1]
BH16-05	55.6	Small	132[1]	54[1]
SH16-02	51.4	Small	FVT Capacity Exceeded	Not Successful
SH16-02	52.0	Small	129[1]	55[1]
SH16-02	53.4	Small	FVT Capacity Exceeded	113[1]
BH16-06	52.9	Small	83[1]	Not Successful
BH16-07	5.5	Small	87[1]	44[1]
BH16-07	37.2	Small	138[1]	79[1]
BH16-07	60.0	Small	79[1]	74[1]
BH16-07	60.4	Small	221[1]	Not Successful
SH16-06	51.66	Small	146[1]	135[1]
SH16-06	51.97	Small	140[1]	113[1]
SH16-06	54.25	Small	137[1]	102[1]

[1] It should be noted that upon further inspection of the soil samples, and in conjunction with reviewing laboratory test results, thin layers of granular soils were present throughout the marine deposits and as such the Nilcon Vane results are typically higher than expected, and therefore may not be representative of the shear strength of the strata.

[2] Small vane capacity exceeded.

The results of the field Electric vane tests are provided in Table 5-6. The test results are presented on the Record of Borehole sheets in Appendix A.

Table 5-6: Electric Vane Test Results Summary

Test Hole	Test Depth (m)	Shear Strength (kPa)	
		Peak	Remoulded
BH16-06	55.47	Not Successful	47[1]
BH16-06	56.08	107[1]	Not Performed
BH16-06	58.21	137[1]	Not Performed

[1] It should be noted that upon further inspection of the soil samples, and in conjunction with reviewing laboratory test results, thin layers of granular soils were present throughout the marine deposits and as such the Nilcon Vane results are typically higher than expected and therefore may not be representative of the shear strength of the strata.

6.0 HYDROGEOLOGICAL INVESTIGATION

Hydrogeological investigations were carried out as part of the Phase III and Phase IV investigations along the western alignment and the Option 6 outfall alignment corridors, respectively. The purpose was to develop estimates of hydraulic conductivities, water levels, hydraulic gradients, and groundwater velocities within the major soil units. Details on standpipe and vibrating wire piezometers used in the hydrogeological investigations are summarized in Table 6-1 below. The monitoring wells in the vicinity of the shafts were also sampled for analysis of potential gasses present.

Full details of the hydrogeological testing and results are included in Appendix F.

Table 6-1: Phase III and IV Hydrogeological Program Summary

Test Hole	Type of Installation	Depth of Installation (m)	Remarks
BH16-01	Vibrating Wire Piezometer	55	Phase III Investigation
		65	
		75	
BH16-03		52	
		59	
		70	
SH16-01	Standpipe Piezometer (50-mm outer diameter)	10	
		35	
		55	
SH16-02		10	
		45	
		52	
SH16-03		10	
SH16-04		10	
		36.6	
SH16-05		10	Phase IV Investigation
		33	
SH16-06		10	
		33	
SH16-07		10	
		31	

6.1.1 Dissolved Gas Sampling

Select monitoring wells located in the vicinity of the proposed effluent shaft (SH16-01) and outfall shaft (SH16-05) along the Option 6 outfall alignment, as well as a potential shaft location for the future conveyance system (SH16-07) were sampled for analysis of dissolved hydrogen sulfide (H_2S), carbon dioxide, and methane gases. The monitoring wells, screened within the sand deposit as per Table 6-1, were sampled.

Dissolved gas samples were collected in situ using Snap Samplers and were analyzed via laboratory single-stage flash analytical techniques. Further details can be found in Appendix F.

7.0 LABORATORY TESTING

In general, the geotechnical laboratory testing was carried out at Golder's South Burnaby soil mechanics laboratory, unless otherwise noted. All thin-walled piston tubes were sent for x-ray scanning prior to testing to determine the potential sample quality, prior to extrusion.

Selected samples were submitted for laboratory testing based on review of sample composition, sample size and field observations from the geotechnical investigation, with review and approval by the client for the proposed lab schedule prior to testing. The laboratory testing completed includes the tests as noted in Table 7-1.

Table 7-1: Laboratory Testing Standards Summary

Laboratory Test	Standard
Water Content Determination Tests	ASTM D2216
Atterberg Limit Index Tests	ASTM D4318
Particle size distribution tests and/or hydrometer tests	ASTM D422
Specific Gravity	ASTM854-14
Organic Content	ASTM D2974-13
Density (Unit Weight)	ASTM D7263-09
Unconfined Compression Test	ASTM D2166
Consolidation (Test Method B – with rebound)	ASTM D2435-04
Cyclic Simple Shear	N/A
Monotonic Simple Shear	N/A
Petrographic Examination	N/A
Rietveld Method And X-Ray Powder Diffraction	N/A
Soil Abrasion Testing	SAT™*
Miller Number Determination	ASTM G75

*Nilsen B, Dahl F, Holzhaeuser J, Raleigh P. 2007. "New test methodology for estimating the abrasiveness of soils for TBM tunnelling", RETC Proceedings, 104-106.

Summary results of the index testing are included on the Records of Boreholes in Appendix A. Detailed results for each of the Atterberg limit testing, grain size analysis, unconfined compression testing, consolidation tests, monotonic and cyclic simple shear tests are included in Appendix G.

Index testing and general lab testing carried out on selected samples are also summarized in Tables 7-2 included in Appendix G. High end laboratory testing carried out on selected samples are summarized in Table 7-3 included in Appendix G.

Petrographic examinations supplemented with X-Ray diffraction (XRD) were carried to determine the mineralogical composition on soil samples collected from the proposed tunnel horizon and a report summarizing the results is included in Appendix H.

Soil Abrasivity Testing and Miller Number determination were also carried out on representative samples selected in consultation with CDM Smith. SINTEF laboratory in Norway and WRES Inc. in the United States carried out the Soil Abrasivity Testing and Miller Number determination, respectively. The reports summarizing the results of the testing are included in Appendix H.

8.0 CLOSURE

We trust that the contents of this geotechnical data report meet with your immediate project requirements. If you have any questions or need further clarification of the contents, please do not hesitate to contact the undersigned. The report will be finalized following receipt of your comments.

GOLDER ASSOCIATES LTD.

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Principal, Senior Geotechnical Engineer

YEW/VF/nnv/lmk

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Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

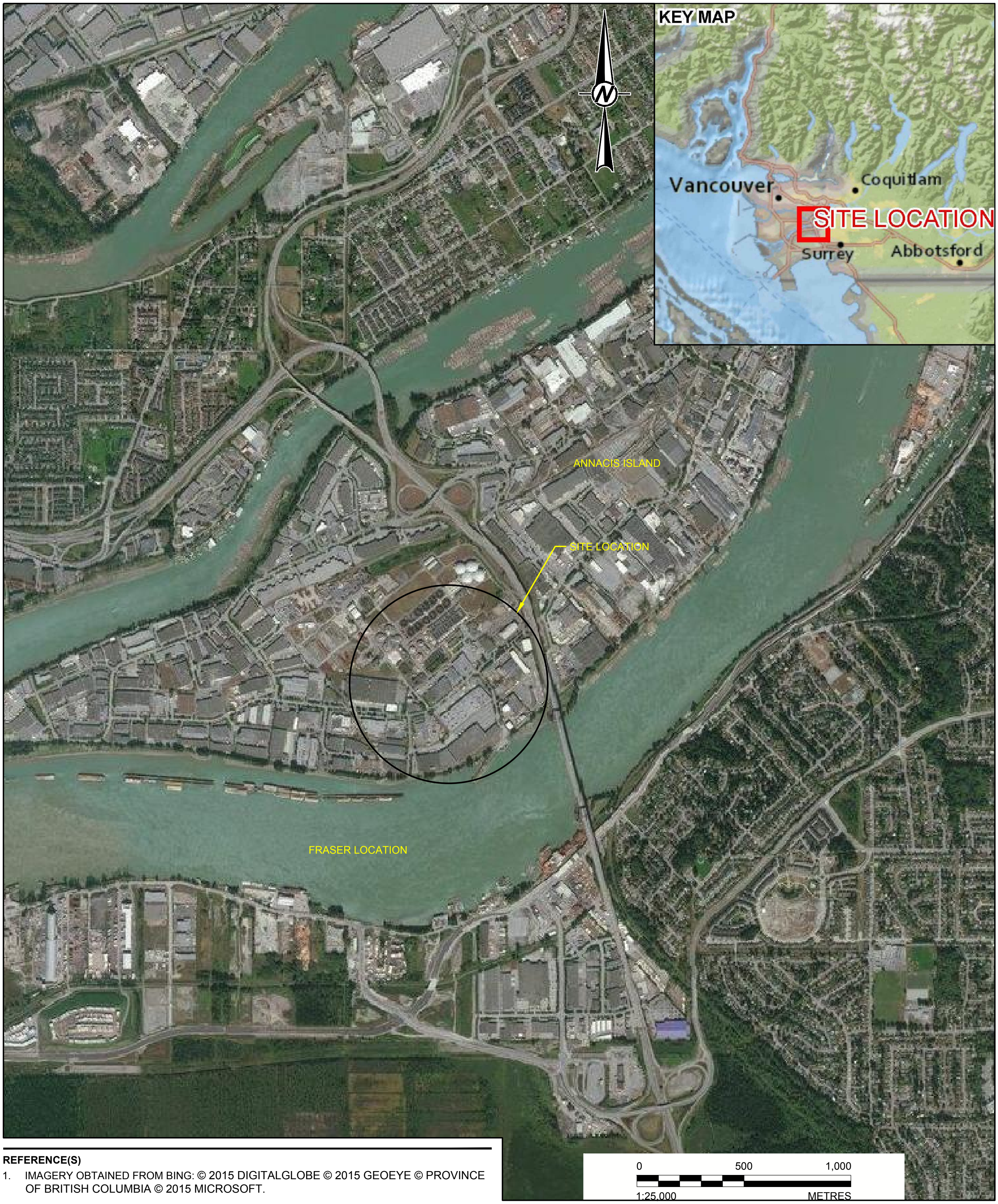
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



REFERENCE(S)

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CLIENT
CDM SMITH CANADA ULC

PROJECT
ANNACIS ISLAND WWTP TRANSIENT MITIGATION AND OUTFALL
DELTA, B.C.

CONSULTANT

Golder Associates

YYYY-MM-DD	2018-01-22
DESIGNED	Y. WITTWER
PREPARED	S. REDDY
REVIEWED	Y. WITTWER
APPROVED	V. FERNANDO

TITLE
PROJECT LOCATION PLAN

PROJECT NO.	PHASE/TASK	REV.	FIGURE
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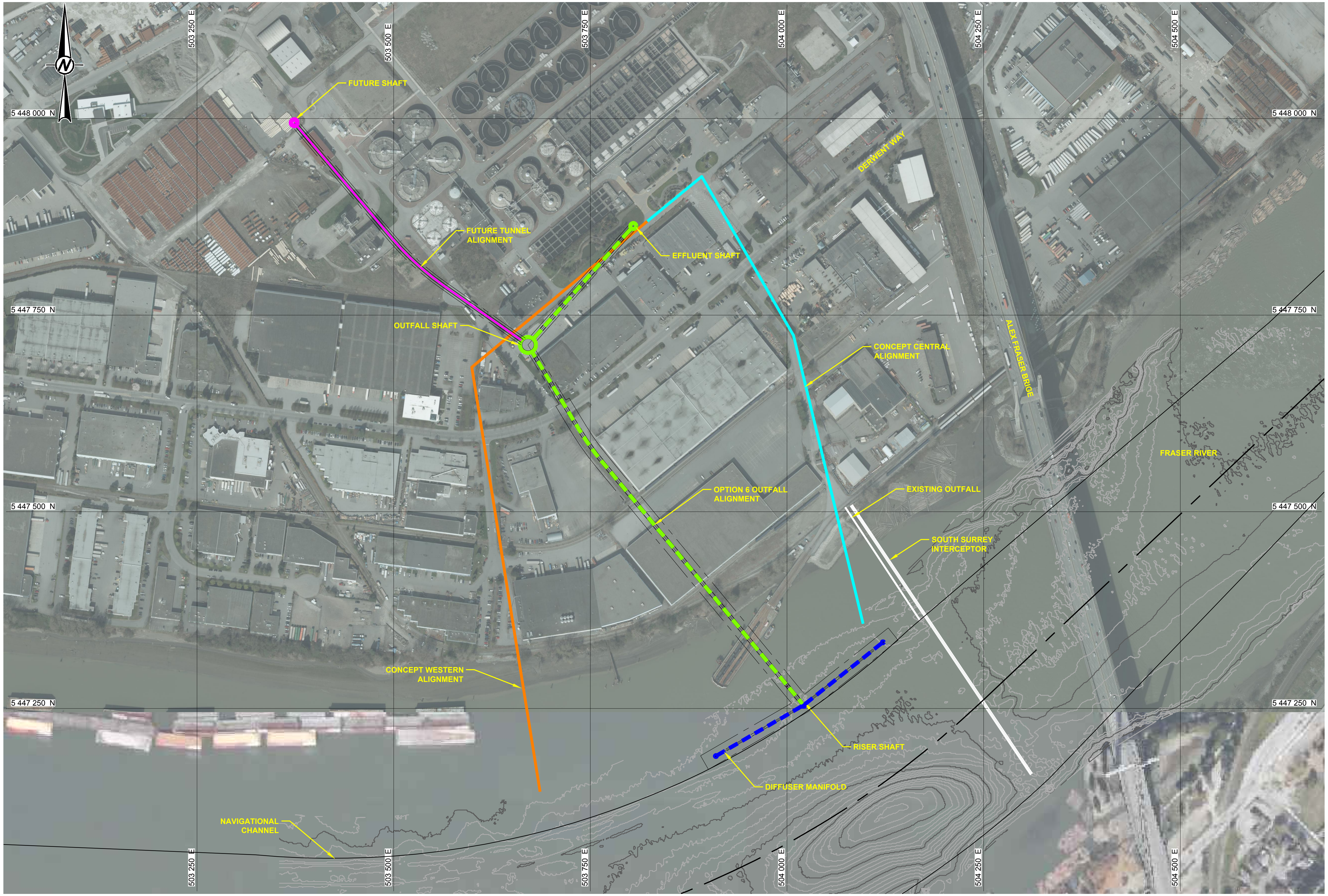

NOTE

1. COORDINATES ARE IN UTM NAD83 ZONE 10 AND CONVERTED TO GROUND LEVEL USING A COMBINED SCALE FACTOR OF 1.000397558.

REFERENCES

1. NAVIGATIONAL CHANNEL AND ORTHOPHOTO PROVIDED BY BLACK & VEATCH FILE: ANNACISNAD27Z10GRND.TIF, ANNACIS-RIVER.TIF, C000A_XXXXX-1.DWG
2. 2016 CCG BATHYMETRY SURVEY AND OPTION 6 ALIGNMENT PROVIDED BY CDM SMITH CANADA ULC ON MAY 19, 2016 FILE: C201072016.DWG
3. 60% DETAIL DESIGN DRAWING OBTAINED FROM CDM SMITH.

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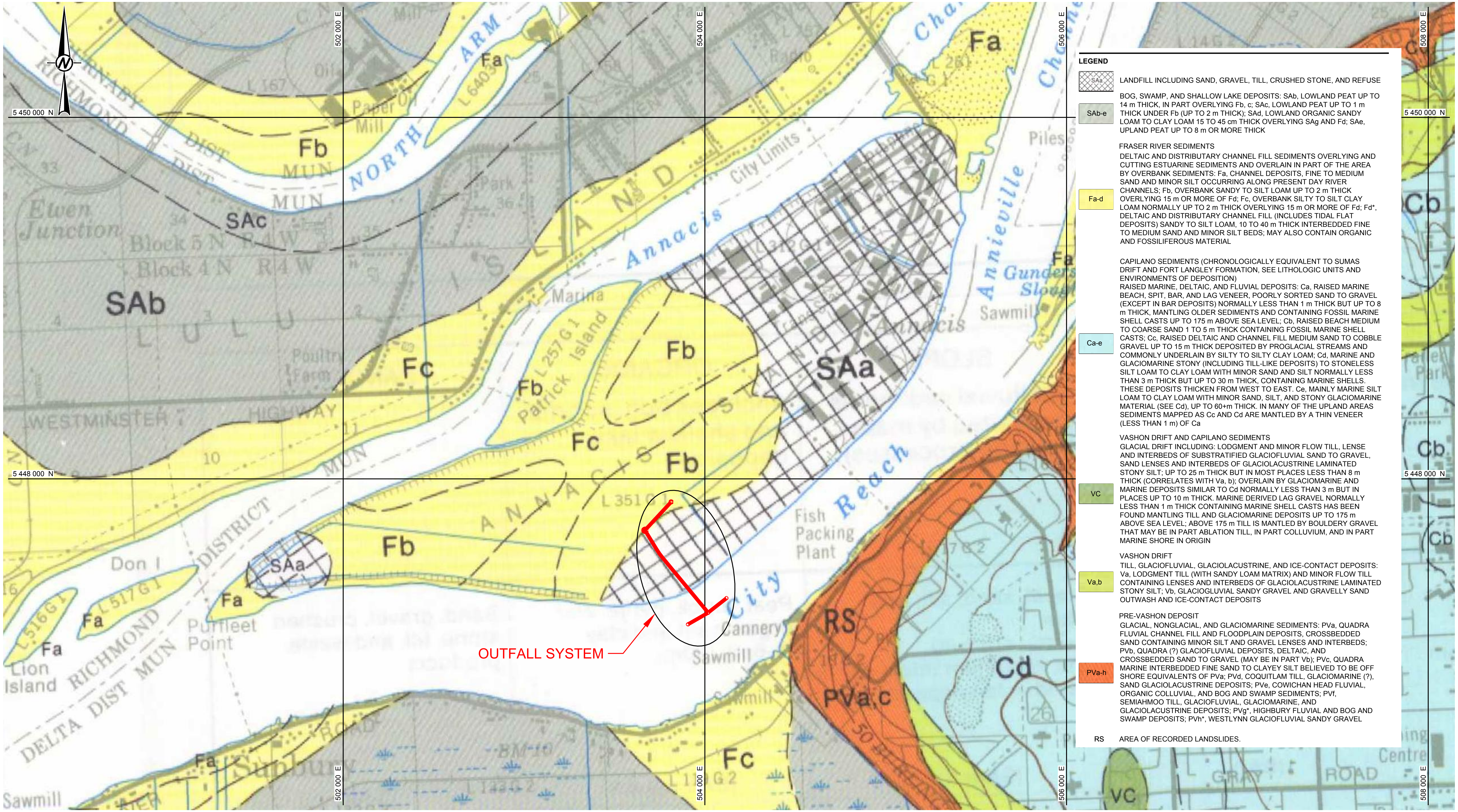
CLIENT CDM SMITH CANADA ULC		PROJECT ANNACIS ISLAND WWTP TRANSIENT MITIGATION AND OUTFALL DELTA, B.C.	
CONSULTANT Golder Associates	YYYY-MM-DD: 2018-01-22	TITLE **CONCEPTUAL AND FINAL OUTFALL ALIGNMENTS**	
	DESIGNED: Y. WITTWER		
	PREPARED: S. REDDY		
	REVIEWED: Y. WITTWER	PROJECT NO.: 1525010	PHASE: 2000
	APPROVED: V. FERNANDO	REV.: 0	FIGURE: 2-2

Path: \\golder.gds\gal\burnaby\CAD-GIS\Client\CDM_Smith\Annacis_Island\99_PROJECTS\1525010_WWTP\02_PRODUCTION\2000\Doc #108-GDR\ | File Name: 1525010-2000-2-2.dwg

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI D

0 25 mm



LEGEND

SAa — LANDFILL INCLUDING SAND, GRAVEL, TILL, CRUSHED STONE, AND REFUSE

SAb-e — BOG, SWAMP, AND SHALLOW LAKE DEPOSITS: SAb, LOWLAND PEAT UP TO 14 m THICK, IN PART OVERLYING Fb, c; SAc, LOWLAND PEAT UP TO 1 m THICK UNDER Fb (UP TO 2 m THICK); SAd, LOWLAND ORGANIC SANDY LOAM TO CLAY LOAM 15 TO 45 cm THICK OVERLYING SAg AND Fd; SAe, UPLAND PEAT UP TO 8 m OR MORE THICK

FRASER RIVER SEDIMENTS

Fa-d — DELTAIC AND DISTRIBUTARY CHANNEL FILL SEDIMENTS OVERLYING AND CUTTING ESTUARINE SEDIMENTS AND OVERLAIN IN PART OF THE AREA BY OVERBANK SEDIMENTS: Fa, CHANNEL DEPOSITS, FINE TO MEDIUM SAND AND MINOR SILT OCCURRING ALONG PRESENT DAY RIVER CHANNELS; Fb, OVERBANK SANDY TO SILT LOAM UP TO 2 m THICK OVERLYING 15 m OR MORE OF Fd; Fc, OVERBANK SILTY TO SILT CLAY LOAM NORMALLY UP TO 2 m THICK OVERLYING 15 m OR MORE OF Fd; Fd*, DELTAIC AND DISTRIBUTARY CHANNEL FILL (INCLUDES TIDAL FLAT DEPOSITS) SANDY TO SILT LOAM, 10 TO 40 m THICK INTERBEDDED FINE TO MEDIUM SAND AND MINOR SILT BEDS; MAY ALSO CONTAIN ORGANIC AND FOSSILIFEROUS MATERIAL

Ca-e — CAPILANO SEDIMENTS (CHRONOLOGICALLY EQUIVALENT TO SUMAS DRIFT AND FORT LANGLEY FORMATION, SEE LITHOLOGIC UNITS AND ENVIRONMENTS OF DEPOSITION)
RAISED MARINE, DELTAIC, AND FLUVIAL DEPOSITS: Ca, RAISED MARINE BEACH, SPIT, BAR, AND LAG VENEER, POORLY SORTED SAND TO GRAVEL (EXCEPT IN BAR DEPOSITS) NORMALLY LESS THAN 1 m THICK BUT UP TO 8 m THICK, MANTLING OLDER SEDIMENTS AND CONTAINING FOSSIL MARINE SHELL CASTS UP TO 175 m ABOVE SEA LEVEL; Cb, RAISED BEACH MEDIUM TO COARSE SAND 1 TO 5 m THICK CONTAINING FOSSIL MARINE SHELL CASTS; Cc, RAISED DELTAIC AND CHANNEL FILL MEDIUM SAND TO COBBLE GRAVEL UP TO 15 m THICK DEPOSITED BY PROGLACIAL STREAMS AND COMMONLY UNDERLAIN BY SILTY TO SILTY CLAY LOAM; Cd, MARINE AND GLACIOMARINE STONY (INCLUDING TILL-LIKE DEPOSITS) TO STONELESS SILT LOAM TO CLAY LOAM WITH MINOR SAND AND SILT NORMALLY LESS THAN 3 m THICK BUT UP TO 30 m THICK, CONTAINING MARINE SHELLS. THESE DEPOSITS THICKEN FROM WEST TO EAST. Ce, MAINLY MARINE SILT LOAM TO CLAY LOAM WITH MINOR SAND, SILT, AND STONY GLACIOMARINE MATERIAL (SEE Cd), UP TO 60+m THICK. IN MANY OF THE UPLAND AREAS SEDIMENTS MAPPED AS Cc AND Cd ARE MANTLED BY A THIN VENEER (LESS THAN 1 m) OF Ca

VC — VASHON DRIFT AND CAPILANO SEDIMENTS
GLACIAL DRIFT INCLUDING: LODGMENT AND MINOR FLOW TILL, LENSE AND INTERBEDS OF SUBSTRATIFIED GLACIOFLUVIAL SAND TO GRAVEL, SAND LENSES AND INTERBEDS OF GLACIOLACUSTRINE LAMINATED STONY SILT; UP TO 25 m THICK BUT IN MOST PLACES LESS THAN 8 m THICK (CORRELATES WITH Va, b); OVERLAIN BY GLACIOMARINE AND MARINE DEPOSITS SIMILAR TO Cd NORMALLY LESS THAN 3 m BUT IN PLACES UP TO 10 m THICK. MARINE DERIVED LAG GRAVEL NORMALLY LESS THAN 1 m THICK CONTAINING MARINE SHELL CASTS HAS BEEN FOUND MANTLING TILL AND GLACIOMARINE DEPOSITS UP TO 175 m ABOVE SEA LEVEL; ABOVE 175 m TILL IS MANTLED BY BOULDERY GRAVEL THAT MAY BE IN PART ABLATION TILL, IN PART COLLUVIUM, AND IN PART MARINE SHORE IN ORIGIN

Va,b — VASHON DRIFT
TILL, GLACIOFLUVIAL, GLACIOLACUSTRINE, AND ICE-CONTACT DEPOSITS: Va, LODGMENT TILL (WITH SANDY LOAM MATRIX) AND MINOR FLOW TILL CONTAINING LENSES AND INTERBEDS OF GLACIOLACUSTRINE LAMINATED STONY SILT; Vb, GLACIOGLUVIAL SANDY GRAVEL AND GRAVELLY SAND OUTWASH AND ICE-CONTACT DEPOSITS

PVa-h — PRE-VASHON DEPOSIT
GLACIAL, NONGLACIAL, AND GLACIOMARINE SEDIMENTS: PVa, QUADRA FLUVIAL CHANNEL FILL AND FLOODPLAIN DEPOSITS, CROSSBEDDED SAND CONTAINING MINOR SILT AND GRAVEL LENSES AND INTERBEDS; PVb, QUADRA (?) GLACIOFLUVIAL DEPOSITS, DELTAIC, AND CROSSBEDDED SAND TO GRAVEL (MAY BE IN PART Vb); PVc, QUADRA MARINE INTERBEDDED FINE SAND TO CLAYEY SILT BELIEVED TO BE OFF SHORE EQUIVALENTS OF PVa; PVd, COQUITLAM TILL, GLACIOMARINE (?), SAND GLACIOLACUSTRINE DEPOSITS; PVe, COWICHAN HEAD FLUVIAL, ORGANIC COLLUVIAL, AND BOG AND SWAMP SEDIMENTS; PVf, SEMIAHMOO TILL, GLACIOFLUVIAL, GLACIOMARINE, AND GLACIOLACUSTRINE DEPOSITS; PVg*, HIGHBURY FLUVIAL AND BOG AND SWAMP DEPOSITS; PVh*, WESTLYNN GLACIOFLUVIAL SANDY GRAVEL

RS — AREA OF RECORDED LANDSLIDES.

LEGEND

PROPOSED OUTFALL ALIGNMENT

0 500 1,000
1:20,000 METRES

REFERENCE(S)

1. SURFICIAL GEOLOGY MAP OBTAINED FROM GEOLOGICAL SURVEY OF CANADA MAP 1484 A (NEW WESTMINSTER), 1980.

CLIENT	CDM SMITH CANADA ULC
PROJECT	ANNACIS ISLAND WWTP TRANSIENT MITIGATION AND OUTFALL DELTA, B.C.
CONSULTANT	Golder Associates
YYYY-MM-DD	2018-01-22
DESIGNED	Y. WITTWER
PREPARED	S. REDDY
REVIEWED	Y. WITTWER
APPROVED	V. FERNANDO
TITLE	SURFICIAL GEOLOGY MAP
PROJECT NO.	1525010
PHASE	2000
REV.	0
FIGURE	3-1

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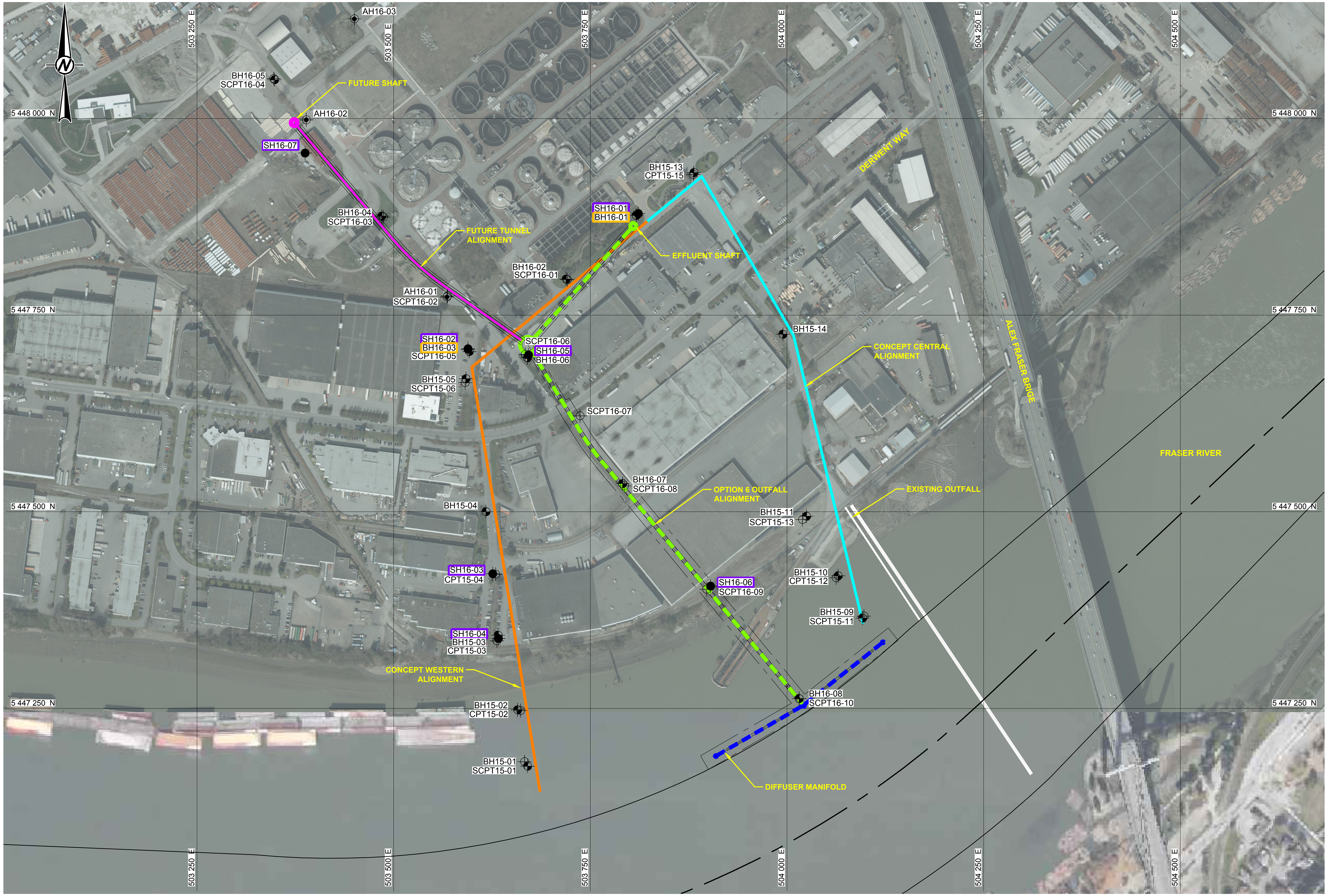

LEGEND

- SONIC HOLE LOCATION
- BOREHOLE LOCATION
- AUGERHOLE LOCATION
- (SEISMIC) CONE PENETRATION TEST LOCATION
- VIBRATING WIRE PIEZOMETER
- STANDPIPE PIEZOMETER

NOTE

1. COORDINATES ARE IN UTM NAD83 ZONE 10 AND CONVERTED TO GROUND LEVEL USING A COMBINED SCALE FACTOR OF 1.000397558.

REFERENCES

1. NAVIGATIONAL CHANNEL AND ORTHOPHOTO PROVIDED BY BLACK & VEATCH FILE: ANNACISNAD27Z10GRND.TIF, ANNACIS-RIVER.TIF, C000A_XXXXX-1.DWG
2. 2016 CCG BATHYMETRY SURVEY AND OPTION 6 ALIGNMENT PROVIDED BY CDM SMITH CANADA ULC ON MAY 19, 2016 FILE: C201072016.DWG
3. 60% DETAIL DESIGN DRAWING OBTAINED FROM CDM SMITH.

0 100 200

1:2,500 METRES

CLIENT	CDM SMITH CANADA ULC
PROJECT	ANNACIS ISLAND WWTP TRANSIENT MITIGATION AND OUTFALL DELTA, B.C.
CONSULTANT	Golder Associates
YYYY-MM-DD	2018-01-22
DESIGNED	Y. WITTWER
PREPARED	S. REDDY
REVIEWED	Y. WITTWER
APPROVED	V. FERNANDO
TITLE	TESTHOLE LOCATION PLAN
PROJECT NO.	1525010
PHASE	2000
REV.	0
FIGURE	4-1

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