

## **Appendix C**

### **Geotechnical Report**

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13 April 2017

## GEOTECHNICAL DESIGN BRIEF

# Conveyance for North Shore Wastewater Treatment Plant Project

**Submitted to:**  
AECOM  
3292 Production Way  
Burnaby, BC  
V5A 4R4

REPORT



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Golder 2015 Lions Gate Secondary WWTP Sewer Conveyance Report

### APPENDIX B

AECOM 2013 Technical Brief - Conveyance Alternatives



## 1.0 INTRODUCTION

AECOM Canada Inc. (AECOM) has retained Golder Associates Ltd. (Golder) to conduct a subsurface investigation and provide geotechnical and archaeological services as input to the indicative design of the proposed NSWWTP (formerly known as Lions Gate Secondary Wastewater Treatment Plant (LGSWWTP)) Sewer Conveyance and Pump Station (Conveyance and Pump Station) project located in North Vancouver, BC.

As part of the owner's team, Golder has prepared this design brief which provides geotechnical engineering input to the indicative design of the proposed Conveyance and Pump Station project. We understand that the project is to be procured as Design Build. This document is intended to summarize our evaluation of the geotechnical aspects of the project for the owner and their consultants, as input to the overall evaluation of feasibility and cost of the project. This report provides a summary of expected subsurface conditions underlying the site, based on published information, historical investigations, and our 2014 geotechnical investigation. Our 2015 report (*Geotechnical Investigation and Design Report, Lions Gate Secondary Wastewater Treatment Plant Sewer Conveyance*, No. 1406018-012-Rev0, dated June 11, 2015) is attached herein as Appendix A. We also include as Appendix B, AECOM's 2013 Technical Brief (*Conveyance Alternatives Technical Brief No. 456928\_WBG103013092847VBC*, dated December 2013). We also provide comments and recommendations as input to the indicative design of the proposed pump station and conveyance sewers. A geotechnical investigation has been carried out. The factual results of that investigation will be presented under separate cover.

This report should be read in conjunction with the "**Important Information and Limitation of This Report**" which is attached following the text of this report. The reader's attention is specifically drawn to this information as it is essential for the proper use and interpretation of this report.

## 2.0 SITE AND PROPOSED DEVELOPMENT

The overall site comprises a linear corridor that extends from the proposed NSWWTP at 1st Street and Philip Avenue in North Vancouver to the outfall of the existing Lions Gate WWTP (see Figure 1). From the proposed secondary plant location, the site extends west on 1<sup>st</sup> Street to its western terminus before diverting south to a GVRD right-of-way parallel to the CN Rail tracks. The site extends west along the right-of-way past Capilano Road to Bridge Road. At Bridge Road the site extends southwest, crossing two CN Rail tracks, and into a MoTI-controlled area below the Lions Gate Bridge, before diverting southwest to the existing Lions Gate WWTP outfall.

The site comprises a mixture of commercial and industrial land. The ground surface along the site comprises asphalt pavement, granular fill or vegetation. The elevation of the site ranges between about 2.4 and 4.4 m. All elevations in this report are with reference to geodetic datum. The GVRD right-of-way is some 8 to 9 m wide and extends between the CN Rail right-of-way to the south and Squamish Nation reserve land to the north. The MoTI-controlled area south of the CN tracks is 60 m wide and extends between the Lions Gate WWTP property to the west and Crown Lands to the east.

The Conveyance and Pump Station project is part of the overall NSWWTP project. The purpose of the Conveyance and Pump Station project is to connect the new plant to the existing sewerage system through a pump station, forcemain and existing and proposed influent gravity sewers, and to convey secondary treated effluent from the proposed new plant to the outfall for discharge. The pump station will be located on Fraser Vancouver Port Authority land which has federal jurisdiction.



Based on conceptual design drawings provided by AECOM dated October 2016 and the proposed pump station site and conveyance right-of-way plan drawings by AECOM dated February 2017, the Conveyance and Pump Station project comprises the following elements:

- a new pump station
- a new 1200 mm inside diameter gravity main (the Hollyburn Interceptor Diversion) connecting the existing Hollyburn Interceptor sewage to the new pump station
- a new 900 mm diameter forcemain extending from the new pump station to the proposed NSWWTP
- a new 2100 mm diameter low pressure gravity sewer extending from the proposed NSWWTP to the existing outfall

As part of the project, an existing 450 mm diameter District of North Vancouver gravity sewer, which extends south on Capilano Road, will be slip-lined as a 600 mm diameter gravity sewer within the existing 1674 mm diameter North Vancouver Interceptor (NVI), extending parallel to the rail right-of-way, and which is to be decommissioned. Refer to Figures 2A to 2F.

The following table provides the diameter and invert elevations for each of the proposed sewers.

**Table 1: Diameter and Invert Elevation of Proposed Sewers**

	Nominal Diameter	Range of Invert Elevation (geodetic datum)
Hollyburn Interceptor Diversion (gravity)	1,200 mm	-3.8 m to -4.1 m
Forcemain	900 mm	+2.0 m to -0.8 m
Low Pressure Effluent Sewer (the Conveyance)	2,100 mm	-0.2 m to -2.3 m

### 3.0 GEOTECHNICAL PROGRAM

The geotechnical program carried out by Golder comprises the following tasks:

- Review of information
- Conceptual analysis and engineering
- Reporting

The following sections provide the results of the individual tasks above.



## **4.0 REVIEW OF INFORMATION**

We collected and reviewed available information related to the geotechnical aspects of the site. The purpose of this review was to enhance our understanding of the subsurface soil and groundwater conditions along the alignment, described below. The information reviewed included published information from the Geological Survey of Canada (GSC), information from our project archives, including our 2014 geotechnical investigation for this project and information from the MoTI's investigations for the upgrades to the Lions Gate Bridge. The following sections summarize the information collected. These sources are discussed separately in the following sections.

We also obtained preliminary design information from AECOM, including drawings of the proposed piping and pump station.

### **4.1 Surficial Geology**

Review of published surficial geological information from the Geological Survey of Canada (GSC Map 1486A, 1976) indicates that the site is underlain by mountain stream and marine deltaic sediments. They are indicated to be up to 15 m or more in thickness. These correspond to alluvial deposits of the Lower Capilano and generally consist of medium to coarse gravel and minor sand, and include cobbles.

### **4.2 Historical Investigations**

Golder conducted a geotechnical investigation of the new pump station and conveyance site for MV in 2014 (Golder 2015). This investigation consisted of a series of sonic boreholes and Becker Penetration Tests drilled along the proposed conveyance route and in the area of the proposed pump station.

The results of this investigation indicates that the Conveyance and Pump Station site is generally underlain by deposits of gravel, sand and cobbles to depths of 10 m to 15 m. This coarse-grained layer also contains discontinuous interlayers of sand or sand / silt mixtures. Becker Penetration blow counts recorded within this deposits vary significantly over a broad range from relatively low to high blow counts. This is inferred to reflect the range of particles sizes from finer- to coarser-grained materials. Overall, this layer is inferred to be loose to compact. Underlying the coarse-grained deposits is an extensive deposit of sand with interbeds of gravel and silt which extends beyond the depth of our investigation. The sand deposits are inferred to be compact based on the results of penetration testing. Historical investigations by others indicate that the sand layer extends to 55 m or more in the western part of the site; to 30 m or more in the central area, and to some 75 m to 90 m in the eastern part of the site. Dense glacial and interglacial sediments underlie the sand deposits. The groundwater table is anticipated to be at an approximate geodetic elevation of +1 m to +2 m, some 1 m to 2.5 m below the ground surface over much of the site. The groundwater table is expected to fluctuate noticeably in response to tidal fluctuations and precipitation.

We also reviewed reports in our project archives from investigations conducted in the site area. These include geotechnical investigations for the Digester No. 4 refurbishment and the Sludge Thickener No. 2 at the existing Lions Gate WWTP site. Review of these reports indicates that these sites are underlain by surficial granular fill followed by sand and gravel deposits with cobbles extending to about 12 m in depth. Underlying these coarse-grained soils are deposits of sand with minor gravel.



We also obtained and reviewed information from geotechnical investigations carried out by the MoTI for the Lions Gate Bridge between 1994 and 1997. During these investigations, a total of 13 boreholes were advanced by Becker Penetration Tests or using mud-rotary techniques, along with Standard Penetration Tests. The boreholes were drilled within the area controlled by the MoTI under the Lions Gate Bridge. The results of these investigations indicate that this part of the site is underlain by 15 m of sand, gravel and cobbles, followed by 40 m of sand with some gravel. The upper 15 m of granular sediments are loose to compact and the underlying sandy deposits are indicated to be compact to dense. Underlying the sand is a stiff clay layer 7 m in thickness at depths in excess of 55 m at the North Viaduct near the existing rail track. Underlying the sandy deposits or clay are inferred glacial till-like deposits which extend to 70 to 100 m below ground surface (elevation -68 m to -98 m). Tertiary bedrock consisting of shale and sandstone is inferred to range in depth from about 70 m to 100 m at the bridge in the vicinity of the proposed sewer alignment.

Within the central part of the site, subsurface information related to the design and construction of the First Narrows Pressure Tunnel shaft indicates that this area is underlain by sand and gravel with cobbles extending to a depth of some 20 m. These coarse-grained deposits are underlain by sand with minor gravel extending to about 30 m depth or more. Dense glacial till-like soils underlie the sands.

At the east end of the site where the proposed new treatment plant will be located, Golder conducted a geotechnical investigation as input to the indicative design of the plant. The investigation consisted of sonic boreholes and Becker Penetration Tests. The results of the investigation indicate that the site is underlain by surficial sand and gravel fill also containing cobbles, followed by natural sediments consisting of sand, gravel and cobbles extending to some 15 m in depth. These coarse-grained soils were found to be interbedded with finer-grained deposits of clay, silt and sand. Underlying the coarse-grained sediments are extensive deposits of interbedded sand and silt with minor gravel extending to 75 m to 90 m or more. The extent of these sediments was not determined conclusively as the boreholes were terminated within this layer, although the presence of possible glacial till-like and hard clays consistent with interglacial sediments, was noted within the deeper boreholes.

The general stratigraphy inferred from the various previous geotechnical investigations is generally consistent with the published information from the GSC and indicate the presence of an alluvial fan comprising coarse-grained granular sediments extending across the site.

## 5.0 SEISMICITY AND GROUND MOTIONS

### 5.1 Seismic Design Criteria

We understand that the conveyance sewer and appurtenances are classified as “Critical Infrastructure” by MV. We understand that MV is considering a design earthquake return period of 975 years, and we have carried out analysis for both return period events. The pump station is classified as “Essential Infrastructure”. Seismic design for these structures is governed by the NBCC National Building Code of Canada, since it is located on federal land, with a design earthquake return period of 2,475 years. Our seismic assessment was carried out using seismic ground motion parameters under the 2015 National Building Code of Canada which is expected to be adopted within the new version of the BC Building Code.



The following sections provide the results of ground motion and liquefaction trigger analyses, and include discussions on the potential consequences of liquefaction for the proposed infrastructure.

## 5.2 Firm Ground Parameters

Site-specific seismic ground motion parameters for the subject site were obtained from the interactive website maintained by the Geological Survey of Canada (GSC), and are summarized in Table 2. They correspond to Site Class C conditions, or for soil profiles where the average shear wave velocity of the upper 30 m varies between 360 m/s and 760 m/s.

**Table 2: Site-Specific Probabilistic Firm-Ground Motion Parameters (Site Class C)**

Return Period	PHGA	Sa (0.2s)	Sa (0.5s)	Sa (1.0s)	Sa (2.0s)
2,475-Years (2% probability of exceedance in 50 years)	0.353	0.813	0.720	0.408	0.248
975-Years (5% probability of exceedance in 50 years)	0.248	0.570	0.504	0.280	0.166

Note: In Table 2, PHGA refers to peak horizontal ground acceleration; Sa refers to spectral acceleration for a given period.

The above seismic hazard parameters were derived from probabilistic hazard model developed by the GSC based on the results of their extensive work. This approach is industry standard and has been adopted in the National and BC Building Codes as accepted practice for design of buildings. We did not carry out a detailed hazard assessment which may consider the proximity of known or potential faults to the site with recorded seismicity over many years. We consider the probabilistic approach to be adequate for the indicative design of the proposed Conveyance and Pump Station project.

Seismic ground motion parameters that correspond to horizontal shaking for the site have been evaluated using the 5th generation seismic hazard model developed on a regional basis by the Natural Resources Canada (NRCAN) for use in the 2015 National Building Code of Canada (NBCC) for a return period of 2,475 years (equivalent to having a 2% chance of being exceeded in 50 years). The input ground motions were applied at firm-ground which has been characterized based on in-situ measurements as having a shear wave velocity varying from 360 to 760 m/s (i.e., Site Class C). The ground motions were propagated upwards towards the ground surface using 1D ground equivalent linear response analysis model, considering the influence of overburden soils underlying the site.

## 5.3 Site-Specific Ground Response Analysis

Site specific, one-dimensional equivalent linear total stress ground response analyses were carried out to evaluate the acceleration profile, spectral accelerations and cyclic stress ratio at three locations along the site, taking into account the subsurface soil and groundwater conditions encountered during the investigation. The ground response analysis was carried out using the commercially available computer program SHAKE 2000 (developed by GeoMotions LLC). The potential for liquefaction was evaluated using methods developed by



Idriss and Boulanger (2014). The outcome of these analyses was used to evaluate the liquefaction potential of the site soils under the design ground motions corresponding to a return period of 2,475 years for the pump station and 975 years for the sewer pipes.

The ground response analysis presented herein was carried out using the input ground motions developed for the Lions Gate Secondary WWTP project. A series of six earthquake records were scaled to the seismic hazard at the subject site. The time-histories corresponding to these records were spectrally matched to the site-specific response spectrum corresponding to the Class C soil conditions as presented in Section 5.2. A total of 11 horizontal and vertical spectrally-matched acceleration time-histories were developed for ground response analyses. Based on the results of de-aggregation of the seismic hazard, the acceleration time-histories developed correspond to the three distinct types of earthquakes anticipated at the subject site as identified previously in Section 2.1. Of the 11 time-histories, 2 time-histories correspond to shallow crustal earthquakes, 4 correspond to deep inslab earthquakes, and the remaining 5 correspond to megathrust earthquakes.

The analyses were carried out at three different areas representing the eastern portion, the central portion and the western portion of the site. The SHAKE analysis carried out by Golder for the proposed NSWWTP site (Golder 2013) was used to analyze the ground motions in the eastern portion of the alignment. The analysis for the central portion of the alignment assumes the till-like soils at 30 m depth, based on previous investigation for the First Narrows Pressure Tunnel shaft. The analysis for the western portion of the alignment assumes the bedrock at 70 m depth, based on the historical investigations carried out by MoTI along the Lions Gate Bridge corridor.

The analyses were carried out with small-strain shear modulus ( $G_{max}$ ) derived from the shear wave velocity profile developed from the data obtained during the investigation. The shear wave profile was developed utilizing the data obtained from the BPT soundings. The BPT results in the form of  $(N_1)_{60}$  profiles are presented in Figure 4. The correlation between the small-strain shear modulus and the shear wave velocity is as follows:

$$G_{max} = \rho V_s^2$$

where

- $G_{max}$  = the maximum shear modulus,
- $\rho$  = the density of soil, and
- $V_s$  = the shear wave velocity of the soil

The small-strain shear modulus for granular soils may also be obtained from the penetration test results using a correlation by Seed and Idriss (1970) in which:

$$G_{max} = 22(k_2)_{max} P_a (\sigma'_m / P_a)^{1/2}$$

where

- $(k_2)_{max}$  =  $20[(N_1)_{60}]^{1/3}$  for granular soil,
- $P_a$  = atmospheric pressure, and
- $\sigma'_m$  = mean effective stress

The shear wave velocity of the intact glacial till-like soils was assumed to be 360 m/s which is consistent with the lower end of the range of shear wave velocity for Site Class C in the NBCC. The shear wave velocity for the bedrock is assumed to be 760 m/s which is consistent with the lower end of the range of shear wave velocity for Site Class B (soft rock) in the NBCC.



A design shear wave velocity profile was created by evaluating the data and comparing with the results of the sonic borehole logs and the Becker Penetration Tests. The design shear wave velocity profiles used for ground response analyses at the three sections are shown in Figure 5A to 5C.

### 5.4 Liquefaction Susceptibility

The cyclic stress ratio (CSR) profile, defined as the ratio of cyclic shear stress and overburden stress induced by the design earthquake, was computed using the simplified procedure developed by Seed. No numerical analyses were carried out as part of this work. The CSR profile from ground motion response analysis was developed to account for the potential for liquefaction and considering the soil conditions at the site. The cyclic resistance ratio (CRR), defined as the ratio between the cyclic shear strength and overburden stress, was computed using the empirical correlation by Idriss and Boulanger (2014) with the equivalent  $(N_1)_{60}$  values. The soil is considered to be potentially liquefiable when the CRR is equal to or less than the CSR. Based on the computed CSR and the CRR profiles, the interbedded sand, silt and silty sand unit between the upper and lower coarse-grained granular units, is potentially liquefiable. The upper portion of the underlying sand deposits, up to about 25 m depth, is also susceptible to liquefaction. The upper and lower gravel, sand and cobbles are considered to be less susceptible to soil liquefaction due to the coarse particle size and high hydraulic conductivity which allow excess porewater pressure to dissipate before the onset of liquefaction. The extent of liquefaction at different segments of the proposed alignment under the 975-year return period and earthquake is shown in Figure 6A to 6C. The extent of liquefaction at the proposed pump station location, considering the 2,475-year return period earthquake, is shown in Figure 6D.

### 5.5 Consequences of Liquefaction

#### 5.5.1 Liquefaction-Induced Re-Consolidation Settlement

Saturated liquefiable soils are expected to generate excess porewater pressure under cyclic loading due to earthquake shaking. The post-liquefaction re-consolidation settlements for the three sections of the pipe alignment and at the proposed pump station locations were estimated based on the CSR profile obtained from the ground response analyses, and considering the aggregate thickness of the liquefied zones. Post-liquefaction re-consolidation settlements resulting from soil liquefaction were estimated using the empirical relations proposed by Ishihara and Yoshimine (1992) utilizing the results of one-dimensional ground response analysis.

It should be noted that our seismic analysis for the central portion of the conveyance sewer is based on information from a single borehole. Based on our analysis, liquefaction is confined to relatively thin layers. However, it should be recognized that there is variation in the subsurface conditions and there may be other areas within the central part of the site that may potentially be susceptible to liquefaction under the 975-year return period earthquake. The table below provides the estimated liquefaction-induced re-consolidation settlement at various portion of the conveyance sewer under both the 975-year return period earthquake, along with the estimated liquefaction-induced re-consolidation settlement at the proposed pump station under the 2,475 year return period earthquake.



**Table 3: Estimated Liquefaction-Induced Re-Consolidation Settlement (in mm)**

Location	Design Earthquake Return Period	Minimum	Maximum
Eastern portion (West 1 <sup>st</sup> Street)	975 years	150	225
Central portion	975 years	75	100
Western portion (near existing plant)	975 years	250	400
Pump Station	2,475 years		400

Abrupt differential settlement may occur at the joints between the conveyance sewer and a fixed connection such as the new wastewater treatment plant. The conveyance sewer should be designed to accommodate such displacements.

### 5.5.2 Earthquake-Induced Lateral Displacements

Lateral ground displacements can occur during earthquake shaking in soil masses located near free faces, such as river banks, or at sloped sites. This results from lateral spread towards the slope face due to loss of strength and inertial forces. The magnitude of lateral ground displacements is related to the geometry of the site, the level of seismic acceleration and the soil strength.

The lateral ground displacement was estimated by using the empirical method by Newmark (1965). Most areas of the site are relatively far from Burrard inlet, in excess of 350 m to 700 m, with the exception of the western areas of the site near the existing plant and at the proposed pump station location where the pump station and the tie-in to the existing outfall will be located within 100 m of Burrard Inlet.

The results of our analysis under the 975-year return period indicate that flow slide conditions, characterized by more than several metres of lateral displacement, are expected within 20 m of Burrard Inlet (see Figure 7A). Outside of this zone, relatively large but decreasing lateral displacements on the order of 0.15 to 1.0 m are expected up to a distance of 300 m from the slope face. Under the 2475-year return period analysed for the proposed pump station, lateral displacements on the order of 0.3 to 0.5 m are expected at the proposed pump station location, some 100 m away from the Burrard Inlet (see Figure 7B). It should be noted that portions of the proposed conveyance sewer, including the tie in to the outfall, are located within 20 m of the shoreline; analysis indicates that they would be susceptible to very large flow slide displacements as a result of the design seismic event.

In addition to vertical and lateral displacements, liquefaction results in temporary loss of strength and stiffness in the liquefied soil. If such soils are at or below structures which derive support from them, this can result in substantial reduction in bearing capacity as discussed in Section 6.4.



## 6.0 GEOTECHNICAL DESIGN CONSIDERATIONS

The results of our review of published and historical investigation information indicates that the Conveyance and Pump Station site is generally underlain by coarse-grained alluvial deposits of sand, gravel and cobbles extending to some 10 to 15 m in depth. These are followed by sand and gravel deposits which are inferred to extend at least 30 m or more below the ground surface. Groundwater is within about 2 m of ground surface, at about elevation +1 m.

Based on preliminary civil drawings from AECOM, the proposed effluent pipe will consist of polyurethane coated/lined steel, and the forcemain and the Hollyburn Interceptor will be plastic pipe (HDPE). Based on preliminary structural drawings from AECOM, the pump station will be founded about 11 m or more below ground surface. Design and installation of the pipes and the pump station will be influenced by the subsurface soil and groundwater conditions. The site conditions will also impact the design and construction of the proposed facilities. The site is constrained by existing features, such as the Lions Gate Bridge, the CN Rail lines, roadways, as well as buried utilities; design and constructability factors relative to these facilities need to be considered, and will impact cost and feasibility. Deep installations, and those conflicting with existing important infrastructure, will require consideration of trenchless installation, particularly at crossings of the rail lines and for the Hollyburn Interceptor. Installation of the pump station to the depth proposed within the inferred subsurface conditions will be challenging.

The following geotechnical design recommendations are provided as input to the indicative design of the pump station and conveyance.

### 6.1 Post-Construction Settlement

Post-consolidation settlements along the pipe and below the pump station are not expected to be a major concern, since the soils are not generally highly compressible and excavation of the soil necessary to install the pipe or the pump station will result in an effective unloading of the soil below the pipe.

### 6.2 Pipe Flotation

The potential for pipe flotation under design flood conditions, or conditions where the pipe may not be full, should be assessed. Under static conditions, it may be assumed that the moist unit weight of granular backfill above the water table is  $20 \text{ kN/m}^3$ , the buoyant unit weight below the water table is  $10 \text{ kN/m}^3$ , and that the unit weight for water displaced by the pipe is  $9.8 \text{ kN/m}^3$ .

Provided that the pipe bedding and trench backfill materials around the pipe are clean granular materials that are placed and compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D1557), the risk of liquefaction occurring around the pipes is considered to be low; liquefaction may still occur below and adjacent to the trench, and sandy material may migrate into the trench. The risk of flotation under liquefied conditions around the pipe (a conservative assumption) can be assessed, if desired, using the material parameters identified in Table 3, below in Section 6.4, except that the unit weight of the material displaced by the pipe should be assumed to be equivalent to the saturated unit weight of backfill material, or about  $20 \text{ kN/m}^3$ .



### **6.3 Pump Station Siting**

As currently envisaged, the proposed pump station would be located south of the North Cable Bent of the Lions Gate Bridge. The pump station would be founded some 12 m or more below the ground surface. Based on review of available information, we understand that the North Cable Bent is founded on shallow spread footings which carry significant foundation loads. Due to prior rotation and settlement of the North Cable Bent foundation, discovered in the late 1970s, remediation was carried out to improve the foundation. The two spread footings were tied together with a grade beam and a series of 9 m long piles were installed around each footing to improve their load-bearing capacity. It is noted that further deformation of the pier occurred while undertaking the remediation, indicating the sensitivity of the pier to construction activities.

Design and installation of the proposed pipes and the pump station should take into consideration the subsurface soil and groundwater conditions, the depth of installation and the presence of existing features, including the existing bridge. Removal of soil due to excavation will result in reduction in confining stress of the subsoils in the area of the excavation. This can result in reduction in the strength and bearing capacity of soils at nearby bridge pier foundations. This, as well as vibrations and/or loss of ground from construction activities, could result in settlement and/or lateral displacement of footings and, in cases where the excavation is very close, instability of the footing. The zone of influence is related to pier foundation geometry and support type, loads from the bridge and/or the proposed pump station, and the subsurface conditions. Under seismic loading conditions, modifications to the ground or loading in the vicinity of bridge foundations could have measurable impacts on performance, even at significant horizontal distances. This represents a risk to the bridge and places constraints on pump station siting and design that requires further geotechnical and structural evaluation to fully resolve. Detailed analysis with suitable data on subsurface conditions and loads would be required to evaluate the full potential impacts of pump station construction on bridge performance, if any, particularly under seismic conditions.

At this preliminary stage, given the importance of the bridge, and considering constructability as well as design impacts, we recommend a minimum preliminary setback of 30 m be established, pending additional analysis. The setback should extend from the nearest point on a given pier foundation to the nearest point on the proposed shoring envelope for the proposed pump station, as an initial preliminary requirement.

### **6.4 Hollyburn Interceptor Diversion and Conveyance Siting**

The Hollyburn Interceptor Diversion will be a 1200 mm diameter gravity sewer founded approximately 7 m below the ground surface. This sewer would extend along the western side of the Lions Gate Bridge alignment. The location of the sewer, based on preliminary routing drawings from AECOM, would likely be approximately 10 m west of the western edge of the pier foundations. To limit the risk of additional loading imposed on the sewer from future development, the sewer should be located at least 7 m from the property line to the west, equalling its proposed depth. If this is not possible, then structural evaluation of the potential impact of loads from future development on adjacent Squamish FN property should be taken into consideration.

The proposed conveyance sewer will be founded some 4 m to 5 m below ground surface over much of its installation. It will be located adjacent to existing infrastructure, including the CN Rail tracks and the Lions Gate Bridge. It is recommended that adjacent to the CN Rail tracks, the conveyance pipe be installed as far from the CN Rail right-of-way as possible to facilitate installation and avoid possible negative impacts on CN property.



Where the contractor considers it necessary to facilitate installation and avoid negative impacts, the pipe should be installed using suitably design shoring. It is recommended that the alignment of the conveyance sewer adjacent to the Lions Gate Bridge be offset as far to the east as possible within the available right-of-way to minimize the potential for impact on the Lions Gate Bridge footings. Where considered necessary by the contractor, and subject to all requirements by MoTI, the pipe should be installed using shored excavation.

### 6.5 Pump Station Design Recommendations

For the calculation of static lateral earth pressure, we recommend a unit weight of 20 kN/m<sup>3</sup> for the native soils. A design groundwater depth of 2 m below the existing grade may be used. Based on information provided by AECOM, we understand that the below-grade pump station is considered to be a rigid structure and therefore at-rest earth conditions would apply to the pump station wall under static conditions. An at-rest lateral earth pressure coefficient of 0.45 is recommended for the static design. An allowable bearing capacity of 200 kPa is recommended for the static design of individual footings assuming settlement in the order of 25 mm are acceptable; however, assuming that the pump station will be supported on a heavily-reinforced base slab, allowable bearing pressures will more likely be controlled by deformations.

The modulus of subgrade reaction may be used to estimate structural deformation taking into consideration the compressibility of the structural slab and the underlying soil. The modulus of subgrade reaction is not a fundamental soil property. In addition to the deformation characteristics of the subgrade, it is also dependent on the geometry and stiffness of the structural member in contact with the subgrade. Therefore, we recommend the following relationship (Vesic and Saxena, 1970) be used in the determination of the modulus of subgrade reaction for structural analysis and design.

$$k_s = 0.91 \frac{[E_s(1 - \nu_c^2)]^{\frac{1}{3}}}{[E_c(1 - \nu_s^2)]^{\frac{1}{3}}} \frac{E_s}{(1 - \nu_s^2)h}$$

where,

- $k_s$  = Modulus of subgrade reaction
- $E_s$  = Young's modulus of soil subgrade
- $E_c$  = Young's modulus of structural element (slab)
- $\nu_s$  = Poisson's ratio of soil subgrade
- $\nu_c$  = Poisson's ratio of structural element
- $h$  = Thickness of structural element

The following range of soil parameters are recommended for substitution in the above equation:

- $E_s$  = 100 to 150 MPa and
- $\nu_s$  = 0.35



Under seismic conditions, the soils around the pump station and below the water table are considered potentially liquefiable. During seismic shaking it is expected that the pump station would move together with the ground, although they may be out of phase. Based on this, use of the Mononobe-Okabe equation to estimate the earth pressure coefficient under seismic conditions for the pre-liquefaction case is considered appropriate for preliminary design purposes. Prior to the onset of soil liquefaction, a lateral earth pressure coefficient of 0.41 is recommended for the seismic design. See Figure 3.

Under liquefied conditions, an equivalent fluid unit weight of 20 kN/m<sup>3</sup> for the liquefied soil (below the water table) should be used to calculate lateral earth pressure. An allowable bearing capacity of 100 kPa is recommended for the liquefied soils below the pump station.

**Table 4: Geotechnical Design Parameters – Pump Station**

Parameter Description	Value	Units
Unit weight of native soil, $\gamma_t$	20	kN/m <sup>3</sup>
Unit weight of backfill, $\gamma_t$	20	kN/m <sup>3</sup>
Buoyant unit weight of native soil, $\gamma'$	9.7	kN/m <sup>3</sup>
Buoyant unit weight of backfill, $\gamma'$	10.2	kN/m <sup>3</sup>
Friction angle of native soil ( $\phi'$ )	33	Degrees
Friction angle of backfill ( $\phi'$ )	34	Degrees
Design groundwater depth below existing ground surface	1.5	m
Coefficient of earth pressure at rest ( $K_0$ )	0.45	
Allowable bearing pressure (kN/m <sup>2</sup> ) (Factor of Safety = 3)	200	kPa
Ultimate Limit State (ULS) Factored Bearing Resistance (kN/m <sup>2</sup> ) – Resistance factor $\Phi = 0.5$ (Canadian Foundation Engineering Manual 2006)	300	kPa
Allowable bearing pressure under seismic loading conditions	100	kPa
Base Friction Coefficient ( $\tan \delta$ ) – For concrete poured over crushed gravel base	0.55	

## 7.0 GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

Excavation, shoring and dewatering methodologies for the sewer pipes and the pump station will be impacted by the depth of installation, the subsurface conditions and the presence of existing sensitive features. The following sections discuss excavation, shoring and dewatering methodologies for the sewers and pump station.



## **7.1 Installation by Open Cut**

The invert elevation of the proposed 900 mm diameter forcemain generally ranges from approximately +0.5 to +2.0 m along most of the alignment, with the exception of the CN Rail crossing where it is at approximately -0.8 m. This would require excavation up to about 2 m to 4 m deep, but typically at or slightly below the natural groundwater table for the majority of the site. Along the CN right-of-way, the forcemain will be as close as approximately 2.8 m from the property line. This would result in excavation side slopes of approximately 1 horizontal to 1 vertical, or possibly steeper, which are expected to be feasible, however, a temporary encroachment into CN property may need to be considered. Alternatively, the use of vertically shored excavation may need to be carried out to avoid encroachment.

The invert elevation of the effluent conveyance sewer ranges overall from about -0.3 m to -2.3 m. Below the Lions Gate Bridge, the pipe invert ranges from about -1 m to -2.3 m with excavation depths of 4 m to 6 m. Parallel to the CN rail tracks, the invert elevation is about -0.3 m to -2.3 m, with excavation depths of about 5 m. The deeper parts of these two segments of the installation are in the same area and correspond to the trenchless crossing of the CN Rail tracks. Along 1<sup>st</sup> Street the invert elevation ranges between about -1 m and -1.5 m with excavation depths of 4 m to 4.5 m. During our 2014 investigation for the conveyance and the investigation for the new secondary treatment plant, groundwater elevation was encountered at about +0.7 m to +1.8 m, with an overall average of approximately +1.2 m. Excavation and groundwater management will be key issues for the installation of the effluent sewer due to the depth of excavation, the presence of adjacent surface and below ground features, and the relatively shallow groundwater table. The excavations are expected to be up to 1 m to 2.5 m below the groundwater, and at deeper installations up to about 3 m below groundwater.

Based on review of historical construction records obtained from Metro Vancouver, the North Vancouver Interceptor (NVI) was installed by open excavation at slopes of 1 horizontal to 1 vertical. It appears that the excavation was conducted, at least partially, in the wet. The NVI is located along the same corridor and at a similar invert elevation as the proposed effluent sewer over much of its alignment. It is considered feasible to install most of the conveyance sewer using open excavation techniques, with provision for shoring and dewatering. Excavation in the wet may also be considered where installation extends below the groundwater and dewatering is considered undesirable.

Where space is constrained, or where excavations are deeper, vertical excavation with suitable forms of shoring compatible with the coarse-grained soils and shallow groundwater within the excavation, would be required. It is recommended that the alignment of the conveyance sewer adjacent to the Lions Gate Bridge be offset as far to the east as possible to minimize the potential for impact on the Lions Gate Bridge footings.

Given the coarse-grained nature of the soils and their high permeability, use of dewatering wells will likely be necessary where excavations extend more than 1 m to 2 m below the groundwater level. Groundwater inflow rates are expected to be significant wherever excavation is conducted below the groundwater level, given the coarse-grained nature of the soils. Where excavations are in excess of 5 m in depth and/or are more than 2 m below the groundwater, it may be necessary to consider using continuous shoring, such as secant piles, or trenchless methodologies (discussed below in more detail under Section 7.2) to install the sewer. If continuous shoring is utilized, it will likely be necessary to extend it at least a distance below the base of the excavation equal to its depth in order to reduce groundwater inflow rates. Alternatively, excavation in the wet may be considered.



The impacts of dewatering on existing nearby grade-supported structures include the potential for settlement. This should be evaluated by proponents as part of their dewatering designs. Given the very high hydraulic conductivity of the soils, groundwater volumes will be large. The requirements, environmental and logistical, of discharge and disposal will all need to be properly addressed in the design and construction methodology prepared by the proponent. Although outside of our scope of work, water quality should be evaluated given the proximity of the trench to existing industrial land and the ocean.

Monitoring of nearby grade-supported features is recommended as part of excavation and dewatering design and execution. Of particular note are the Lions Gate Bridge and the CN Rail tracks. Settlement, tilt and displacement of all nearby bridge footings should be monitored throughout excavation, shoring, dewatering and tunnelling activities, in order to confirm deformation and displacements are within acceptable tolerances. It is emphasized that the bridge piers in the area of the proposed sewer installations are heavily loaded and are founded on shallow spread footings and grade beams, with the exception of the North Cable Bent which has additional piles, as discussed above. Proponents may have to consider limiting length of open trench and phased excavation along this corridor, as well as localized use of shoring at and near the piers.

Along the CN Rail tracks, the effluent conveyance pipe will be as close as 5 m to the CN property line. Where space permits, installation by open excavation is considered to be feasible with suitable dewatering and shoring, or alternatively, in the wet. A temporary encroachment into CN property may need to be considered. Alternatively, use of shoring could be considered where space is constrained.

## 7.2 Pump Station

As currently envisaged at this conceptual stage, the proposed pump station would be founded some 11 m or more below the ground surface, and some 9 m to 10 m below the groundwater level. Excavation to install the pump station to this depth within the coarse saturated granular soils underlying the site will require continuous shoring. Given the coarse-grained nature of the granular deposits, installation of sheet piles is not expected to be practical, and is not recommended. Given the size of the excavation, its depth and the coarse-grained nature of the granular soils, it is not considered technically feasible, or advisable, to draw down the water table with the use of pumping wells, even with the use of adequate continuous shoring. The volumes of water would be excessive and the water would likely have elevated salt content requiring extensive treatment facilities. Furthermore, aggressive groundwater drawdown could result in loss of ground and/or settlement of nearby grade-supported structures, including the bridge. Ground freezing is similarly not considered feasible, due to the coarse-grained nature of the subsoils and the possibility of encountering salt water.

Excavation into granular soils below the water is sometimes achieved with the use of caisson sinking methodology. This involves advancing a pre-cast or cast-in-place concrete structure, usually with a reinforced shoe. Water jetting, or bentonite can be used to facilitate excavation and advance of the caisson. Given the size, depth and the irregular shape of the proposed pump station, together with the coarseness of the subsoils, we would consider this to be a very challenging and risky approach at this site.

We anticipate that excavation would most likely have to be carried out in the wet, followed by placement of a tremie concrete plug over the entire footprint of excavation. Alternatively, techniques such as jet grouting might be considered to form a seal prior to excavation. In that case, the seal would likely be installed first and designed to



extend out beyond the footprint of the vertical shoring elements, secant piles for example. The piles could then be drilled into the jet grout plug thereby forming a better seal between the plug and the shoring. If the plug were to extend far enough outside the piles to form a lip, the system could mobilize soil self weight in the uplift resistance. In order to provide sufficient resistance to uplift pressure from groundwater, it is expected that a plug of at least 3 m in thickness would be required, depending on its configuration strength. As such, excavations of up to 14 m below the ground surface could be required to install the shoring system. Once the ballast is properly sealed and tied in to the shoring, the water would be drained to permit construction of the pump station.

### 7.3 Trenchless Installation

The invert elevation of the Hollyburn Interceptor Diversion ranges from approximately -3.8 m to -4.1 m, some 7 m to 8 m below the existing ground surface. Due to the depth of installation, the presence of coarse-grained particles including cobbles and possibly boulders, and the proximity of the bridge, it will be difficult or not feasible to install this pipe using open or shored excavation for its entire length. Trenchless installation is therefore recommended for this installation. Where trenchless installation is not practical, open trench installation of this pipe may be considered. This would require continuous shoring and likely the inclusion of a sealed excavation base, similar to that proposed for the pump station.

The forcemain and the effluent conveyance pipe both cross the CN Rail lines at Bridge Road which will similarly require trenchless installation. Outside of the CN Rail crossing, it also may be considered preferable by a contractor to install the conveyance pipe using trenchless methodology, subject to suitable evaluation of the depth of cover as well as other considerations. Given the coarse-grained soils and the relatively shallow groundwater and the potential for face instability and ground loss, use of open-face tunnelling is not recommended. Equipment with full face support, and the capacity to process cobbles and boulders, is recommended. It should be noted that the cobbles and boulders in this area are inferred to be derived from Pre-Tertiary granodiorite rock, and can be very strong, with unconfined compressive strength on the order of 250 to 350 MPa, or more.

In order to reduce the potential for heave or settlement of the rail lines, the effluent and forcemain pipes should be installed at least 2 times their respective diameter below the tracks. Use of a two-pass system such that the pipes are installed within a casing will be required for the CN crossing. The diameter of the casing pipes should be sufficient to install the sewer pipes to the specified line and grade, considering the accuracy of the trenchless technique used. Use of steel or concrete may be considered, subject to CN Rail requirements. The required casing pipe diameter for the Hollyburn Interceptor and the forcemain is expected be about 1.8 m, considering the ground conditions. The annulus between the casing and the product pipe would be backfilled with grout. The susceptibility to grout – in particular the heat of hydration – of the plastic pipe for the forcemain, or polyurethane coating for the effluent pipe, should be properly evaluated by the contractor.

Subject to the requirements of MoTI on settlement and displacement tolerances of the bridge structure, it may be preferable to install the conveyance pipe entirely by trenchless methods within the area below the bridge. However, given the relatively shallow depth of the pipe along this corridor, there will likely be some surface expression of the microtunnelling. It may be necessary to consider deepening the conveyance pipe though this corridor in order to reduce such effects, if trenchless installation is considered preferable.



Trenchless entry/exit shafts will need to be installed at locations either side of the rail crossing and where significant pipe bends are required. The placement and configuration of the shafts will be the responsibility of the designer, however, the location and protection of nearby existing features, including the Lions Gate Bridge and the rail lines should be taken into consideration. Entry shafts are anticipated to be about 5 m by 12 m in overall plan dimension. Exit or retrieval shafts are expected to be up to about 5 m by 5 m in plan dimension. The trenchless shafts will require suitable continuous shoring, such as that described above for the pump station.

Monitoring for displacements and settlement will be required to confirm ground deformation due to the trench excavation is within an acceptable tolerances, and to identify potential loss of ground. Based on the concept drawings by AECOM, up to five entry/exit shafts may be required.

### 8.0 CLOSURE

We trust that this report provides sufficient information for your immediate requirements. If you have any questions or comments, or require further inputs, please do not hesitate to contact us.

#### GOLDER ASSOCIATES LTD.

Chung Ho Cheng, PEng  
Geotechnical Engineer

Anthony Fuller, PEng  
Associates, Project Manager

Reviewed by:

Trevor P. Fitzell, PEng  
Principal

CHC/ALF/TPF/nnv

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## **IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT**

**Standard of Care:** Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

**Basis and Use of the Report:** This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder can not be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

**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.

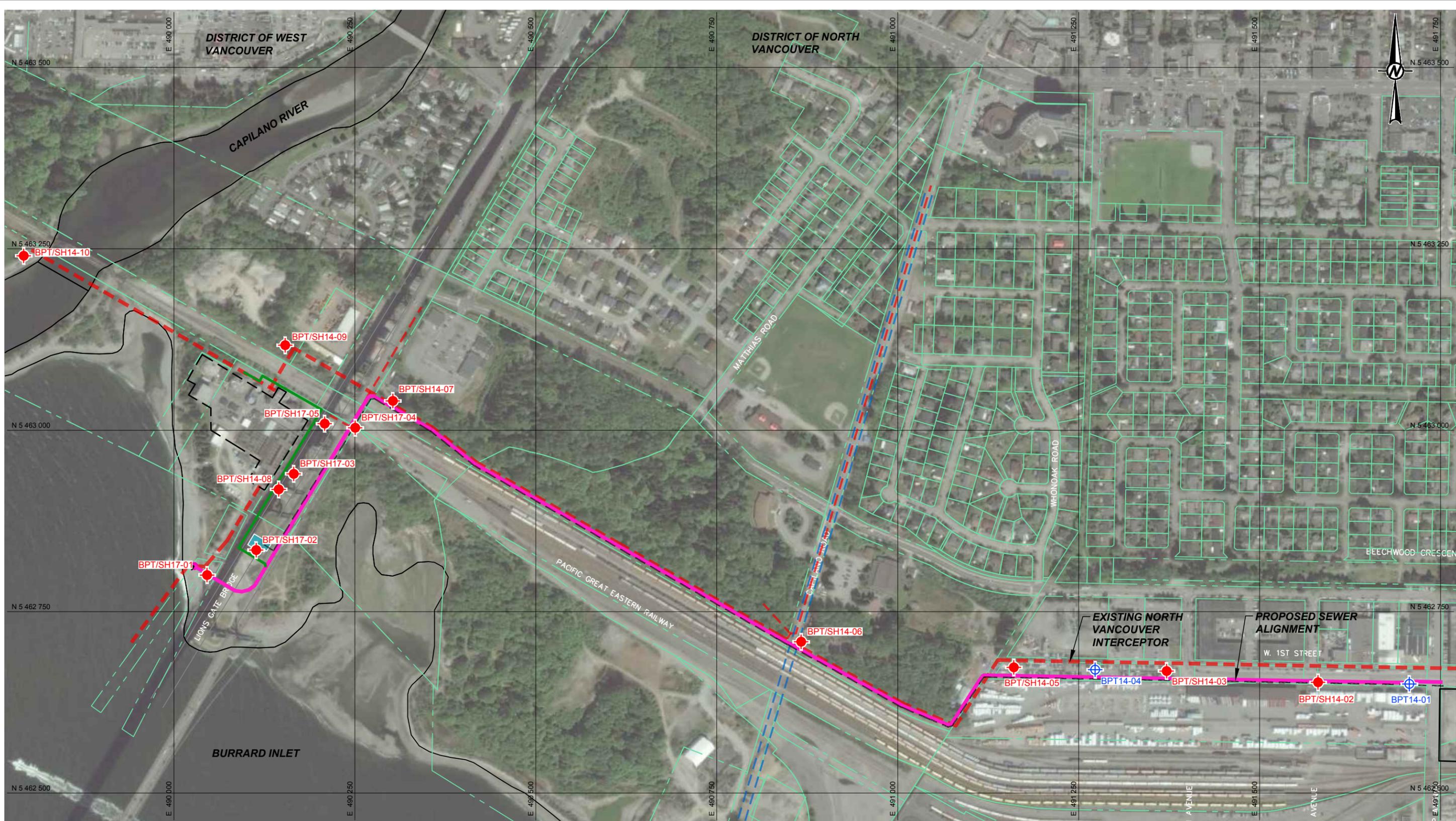
**Sample Disposal:** Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater 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.



**LEGEND**

 BECKER PENETRATION TEST / SONIC HOLE LOCATION  
 BECKER PENETRATION TEST LOCATION

**NOTE**

1. BECKER PENETRATION TEST AND SONIC HOLE SIZE ARE NOT TO SCALE.  
 2. THIS DRAWINGS IS BASED ON UTM GRID COORDINATES SYSTEM FOR PLANNING AND DISCUSSION PURPOSES ONLY.

**REFERENCE**

1. ORTHOPHOTO RETRIEVED FROM BING UNDER MICROSOFT LICENSE. 2012 DIGITALGLOBE IMAGE COURTESY OF USGS, 2012 GEOEYE. RETRIEVED DATE: MAY 29, 2014.
2. BASE DRAWING PROVIDED BY METRO VANCOUVER IN UTM GROUND COORDINATES SYSTEM WITH A COMBINED SCALE FACTOR OF 0.9996 AND ELEVATION ARE IN METERS REFERENCED TO GEODETIC DATUM. CAD DRAWING "X10001S3189.dwg" DATED OCTOBER, 2014.
3. BASE DRAWING PROVIDED BY AECOM RECEIVED ON DECEMBER 5, 2016. CAD FILE: X-001-LGWWTP CONVEYANCE SKETCH\_1 DRAWING NUMBER X-001.



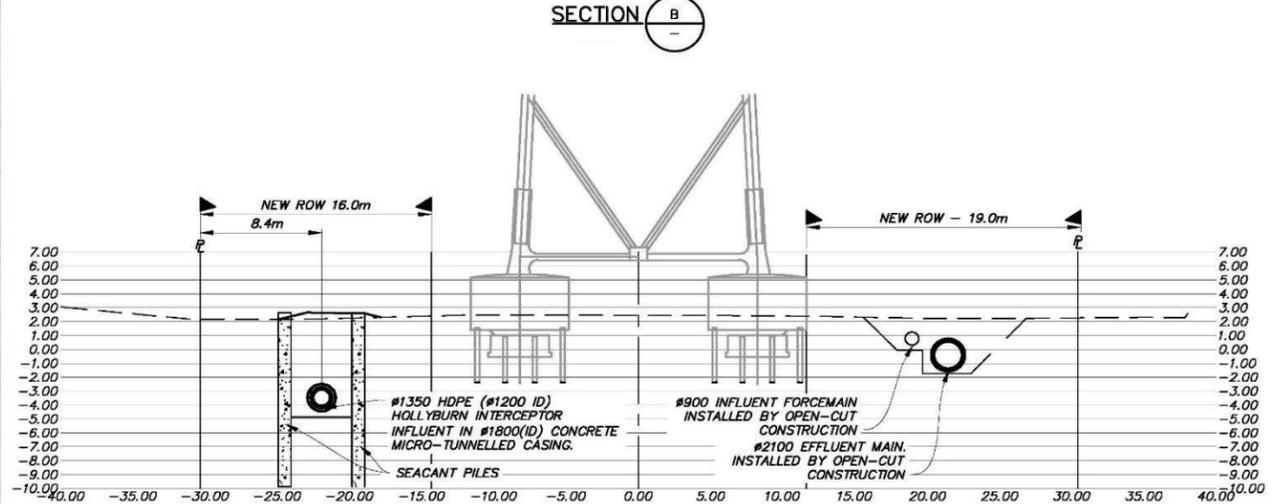
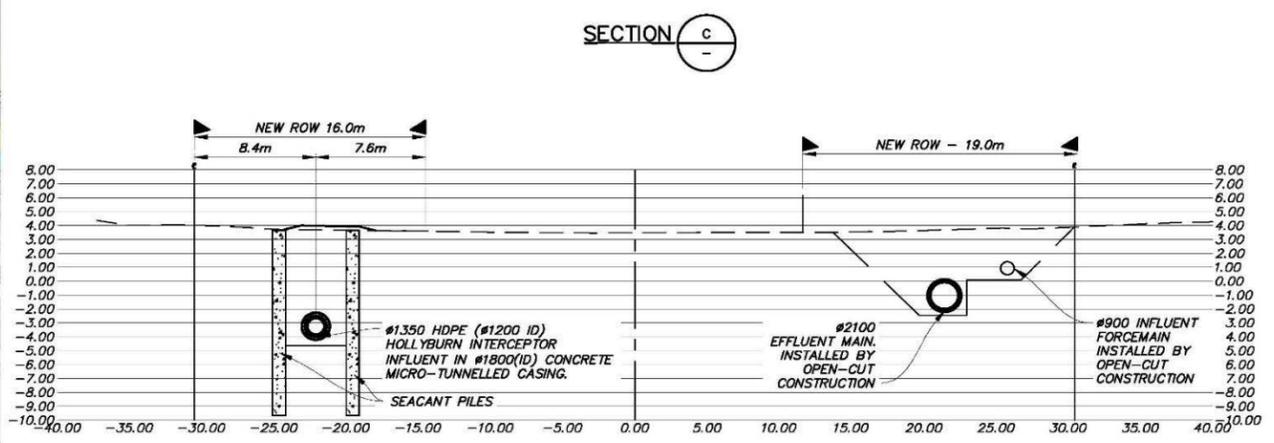
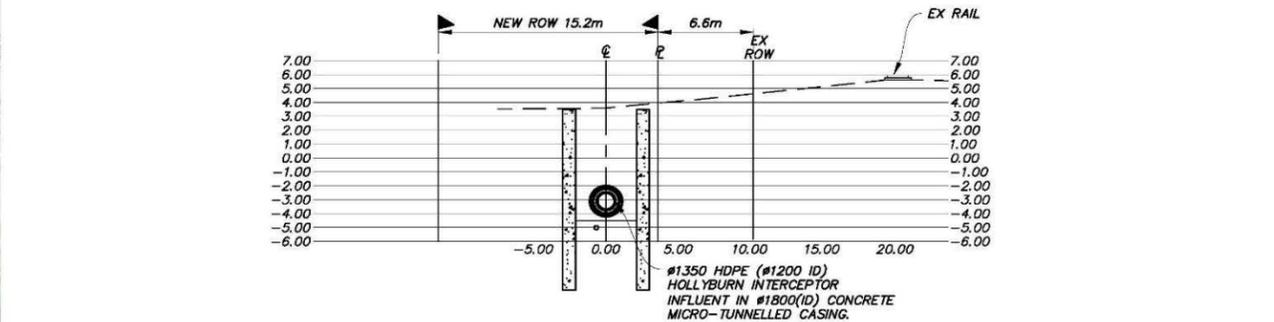
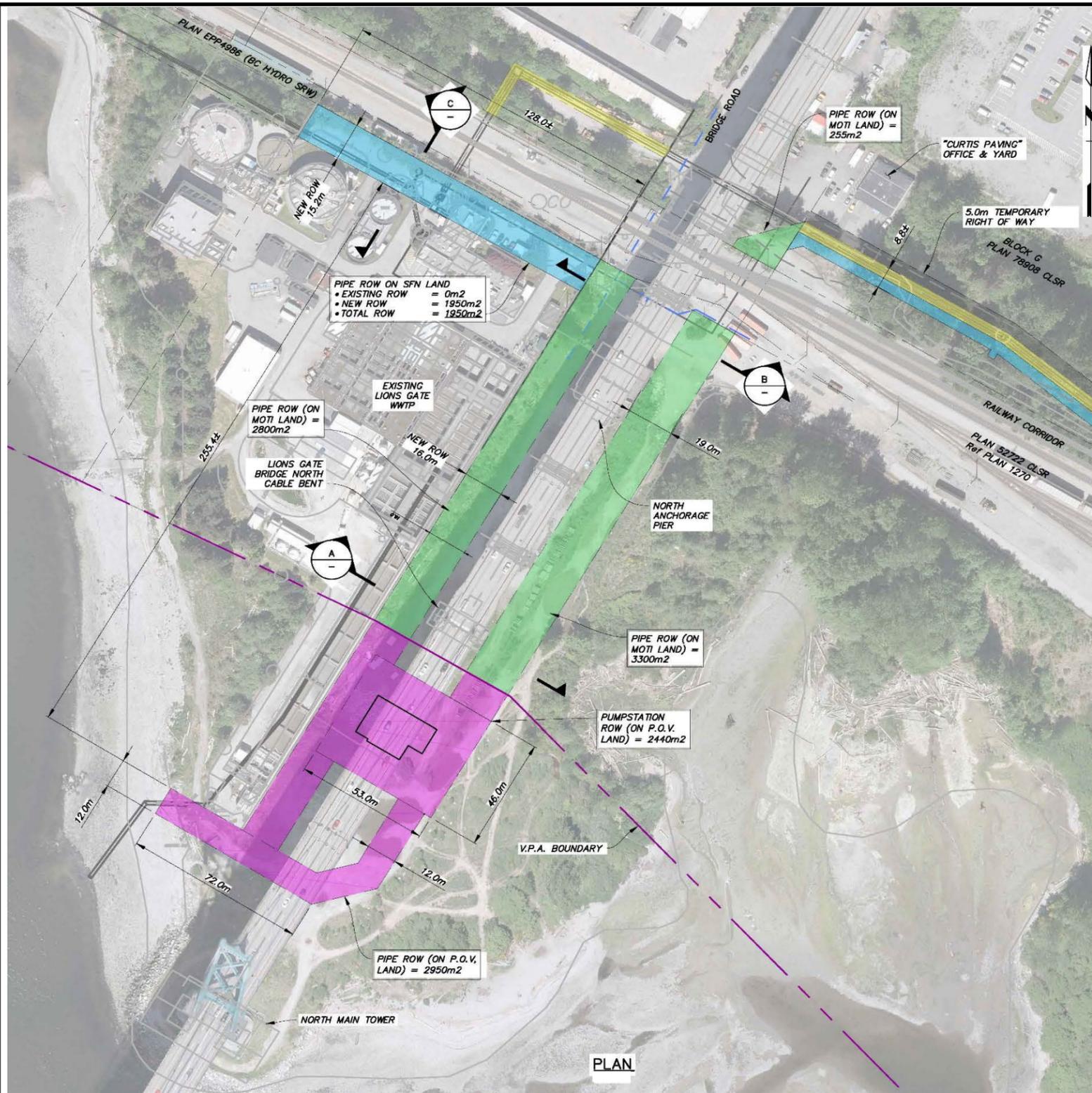
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CONSULTANT	YYYY-MM-DD 2017-04-12
	PREPARED S. REDDY
	DESIGN C. H. CHENG
	REVIEW C. H. CHENG
	APPROVED A. FULLER



PROJECT			
LIONS GATE SEWER SECONDARY WWTP CONVEYANCE AND PUMP STATION NORTH VANCOUVER, B.C.			
TITLE			
TEST HOLE LOCATION PLAN			
PROJECT No.	PHASE	Rev.	FIGURE
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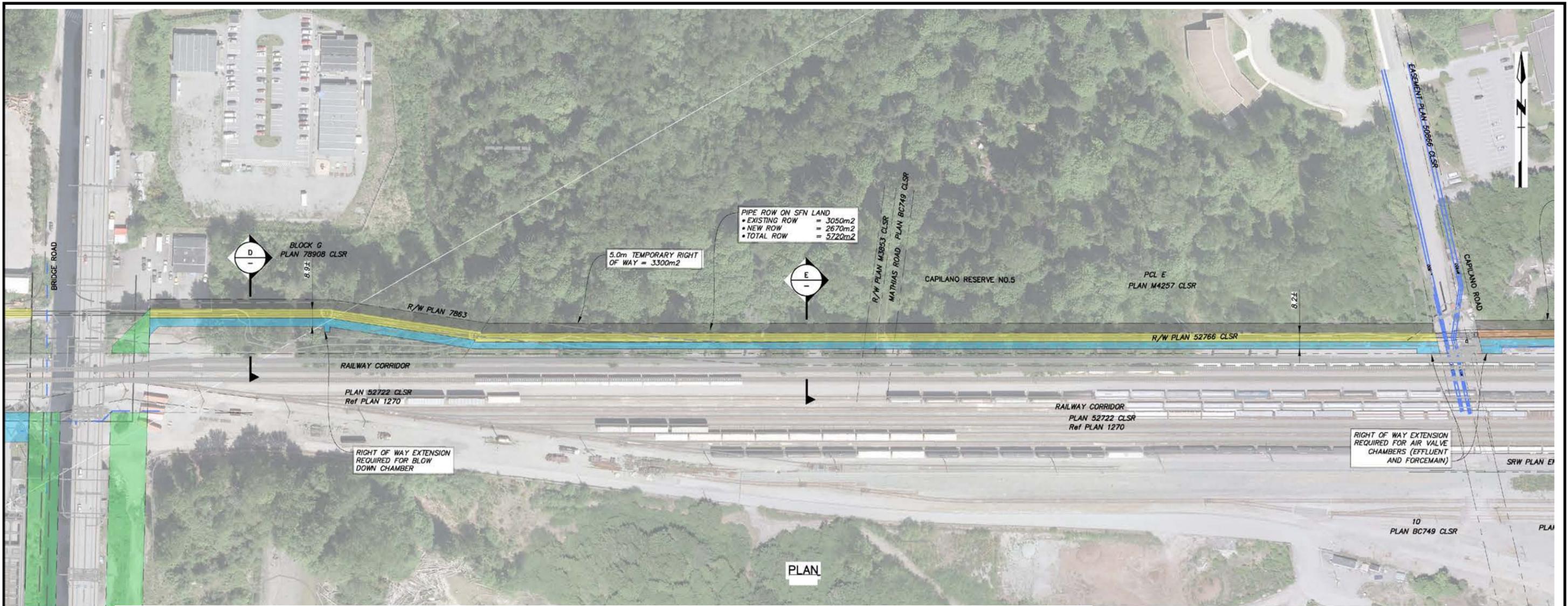
**LEGEND**

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	RIGHT OF WAY WITHIN PORT OF VANCOUVER LAND		EXISTING NM RIGHT OF WAY AND NEW DNV RIGHT OF WAY
	EXISTING METRO VANCOUVER RIGHT OF WAY		TEMPORARY CONSTRUCTION ACCESS RIGHT OF WAY

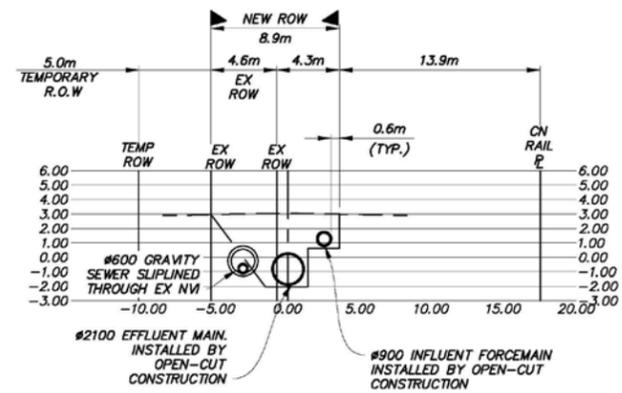
NOTE:  
THIS ROW PLAN IS DRAFT, PREPARED FOR DISCUSSION PURPOSES ONLY. ROW BOUNDARY, AREAS MAY BE ADJUSTED IN FUTURE

Reference  
Conveyance for Lions Gate Secondary WWTP – Proposed Pump Station Site and Conveyance Right of Way Plan by AECOM dated February 2017

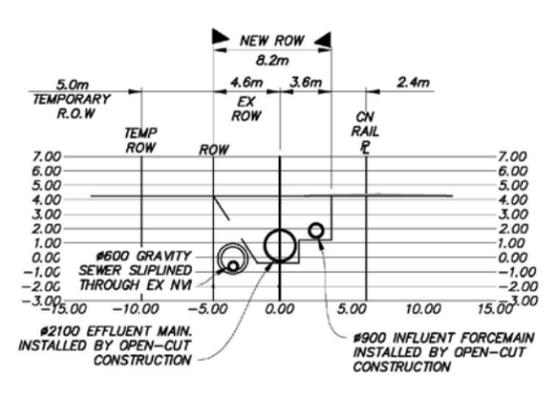
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REVIEW	ALF	10FEB17			



PLAN



SECTION D



SECTION E

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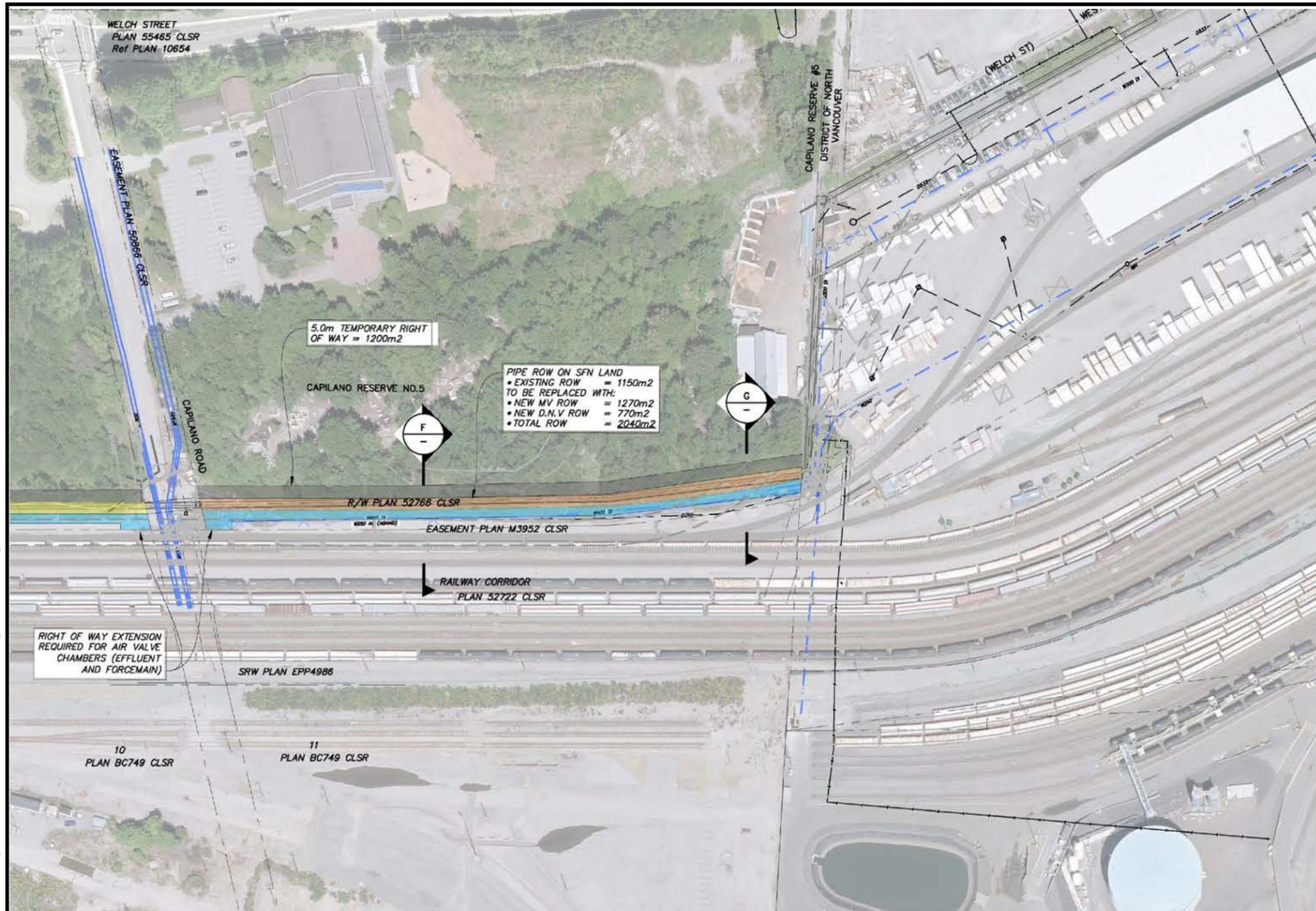
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	EXISTING METRO VANCOUVER RIGHT OF WAY		EXISTING NM RIGHT OF WAY AND NEW DNV RIGHT OF WAY
	TEMPORARY CONSTRUCTION ACCESS RIGHT OF WAY		

NOTE:  
THIS ROW PLAN IS DRAFT, PREPARED FOR DISCUSSION PURPOSES ONLY. ROW BOUNDARY, AREAS MAY BE ADJUSTED IN FUTURE

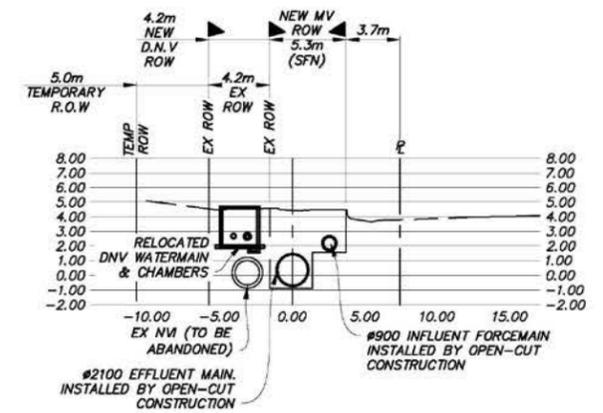
Reference  
Conveyance for Lions Gate Secondary WWTP – Proposed Pump Station Site and Conveyance Right of Way Plan by AECOM dated February 2017

PROJECT	AECOM				
LIONS GATE SWWTP CONVEYANCE AND PUMP STATION NORTH VANCOUVER, BC					
TITLE	RIGHT OF WAY PLAN				
	PROJECT No.	1544193	PHASE No.	1000	
	DESIGN	BDA	08FEB17	SCALE	NTS
	CADD			REV.	0
	CHECK	CHC	08FEB17	Figure 2B	
	REVIEW	ALF	10FEB17		

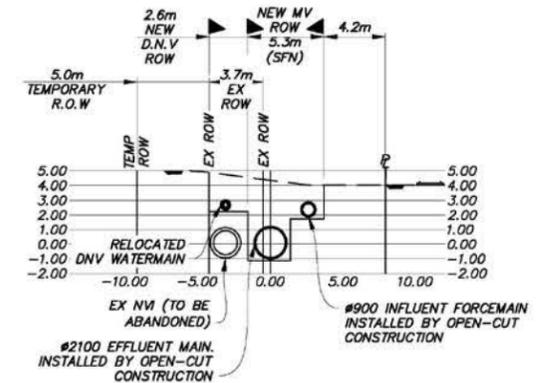
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PLAN



SECTION G



SECTION F

LEGEND

- RIGHT OF WAY WITHIN MOTI LAND
- RIGHT OF WAY WITHIN PORT OF VANCOUVER LAND
- EXISTING METRO VANCOUVER RIGHT OF WAY
- NEW RIGHT OF WAY WITHIN SQUAMISH FIRST NATION LAND
- EXISTING NVI RIGHT OF WAY AND NEW DNV RIGHT OF WAY
- TEMPORARY CONSTRUCTION ACCESS RIGHT OF WAY

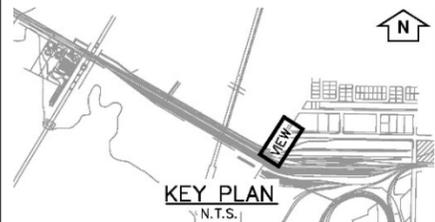
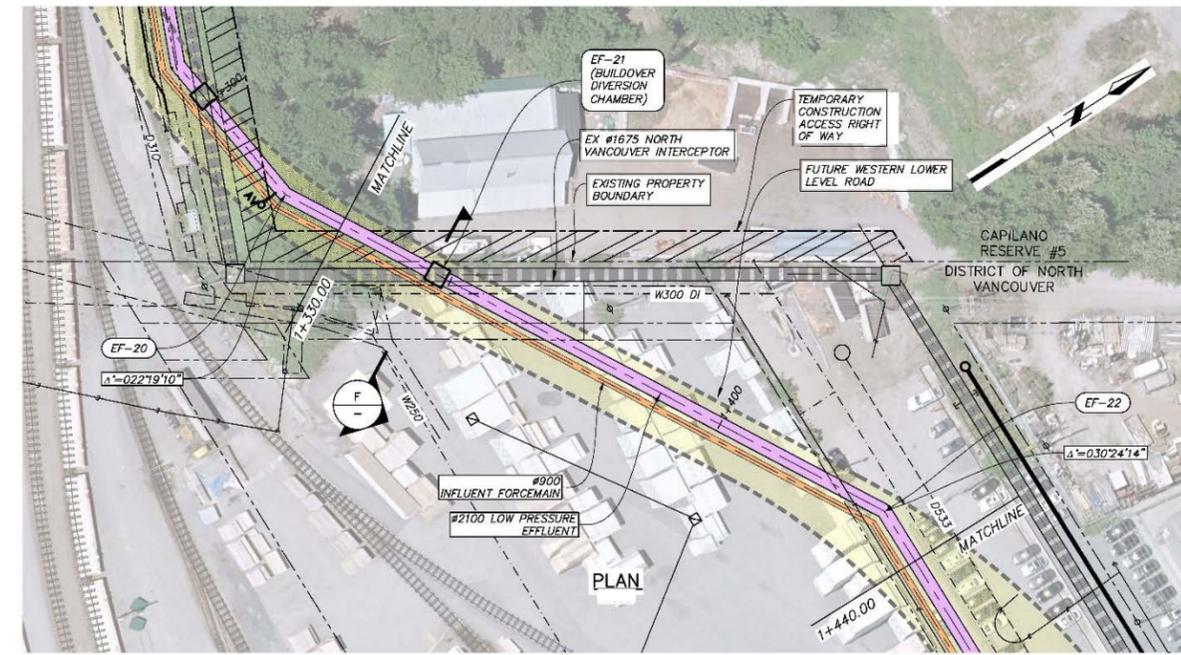
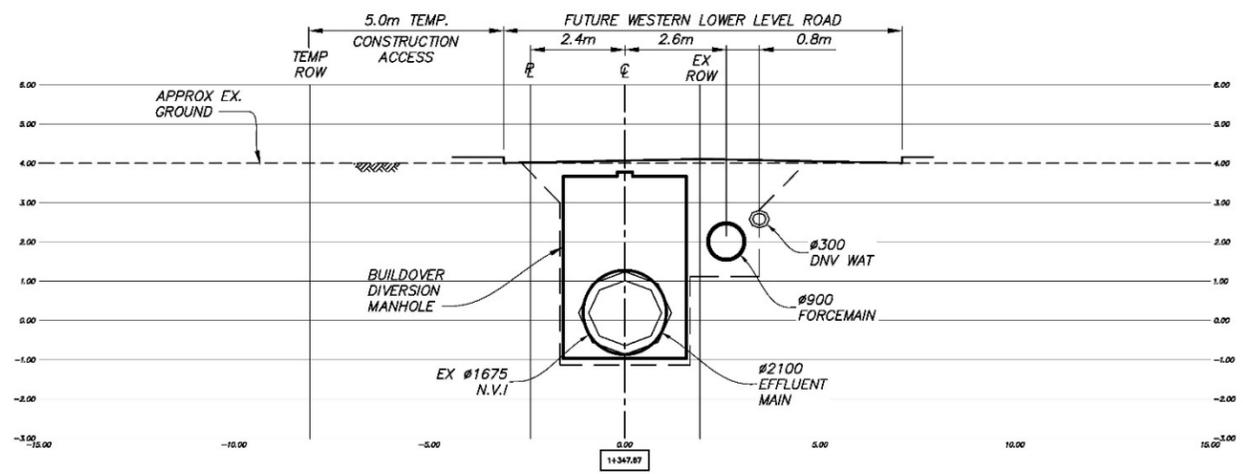
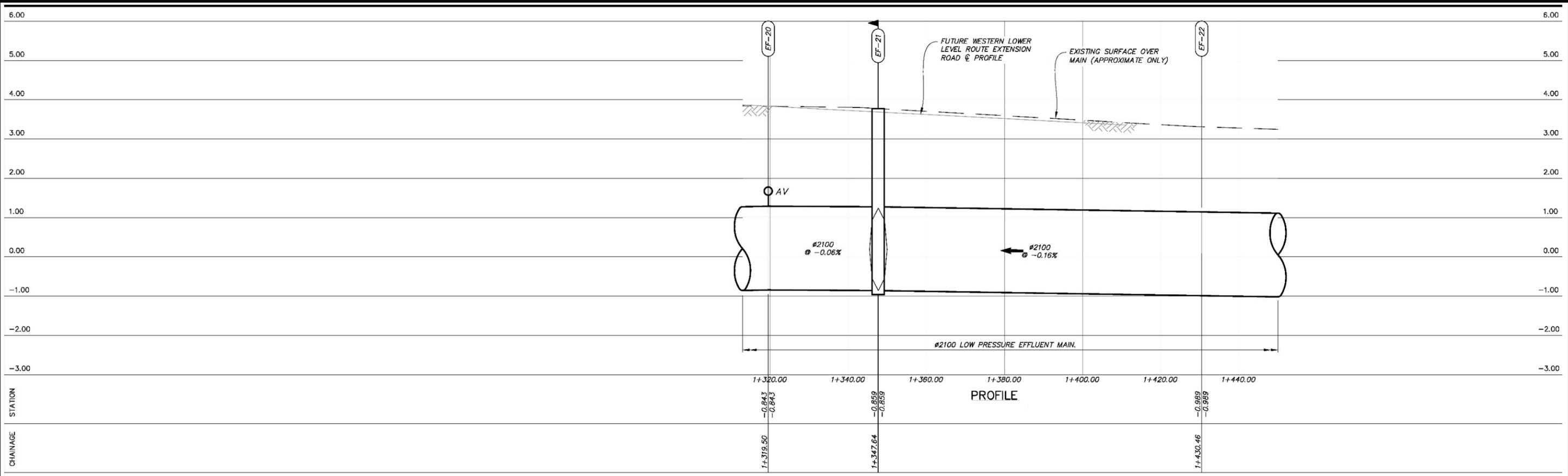
NOTE:  
THIS ROW PLAN IS DRAFT, PREPARED FOR DISCUSSION PURPOSES ONLY. ROW BOUNDARY, AREAS MAY BE ADJUSTED IN FUTURE

Reference

Conveyance for Lions Gate Secondary WWTP – Proposed Pump Station Site and Conveyance Right of Way Plan by AECOM dated February 2017

PROJECT		AECOM			
LIONS GATE SWWTP CONVEYANCE AND PUMP STATION NORTH VANCOUVER, BC					
TITLE		RIGHT OF WAY PLAN			
	PROJECT No.	1544193	PHASE No.	1000	
	DESIGN	BDA	08FEB17	SCALE	NTS
	CADD			REV.	0
	CHECK	CHC	08FEB17	Figure 2C	
	REVIEW	ALF	10FEB17		

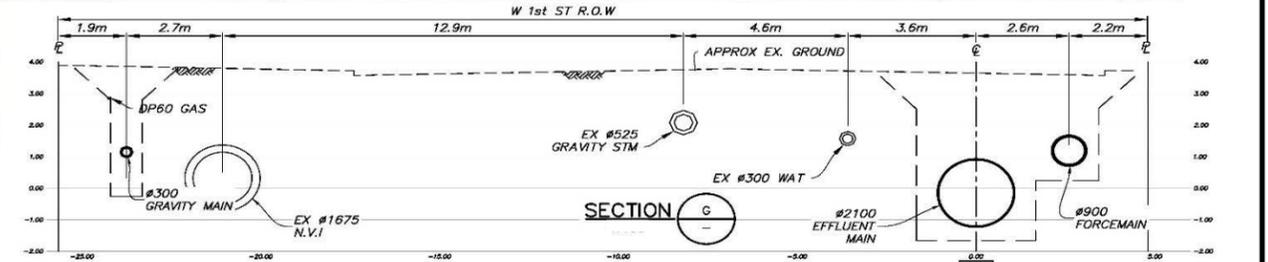
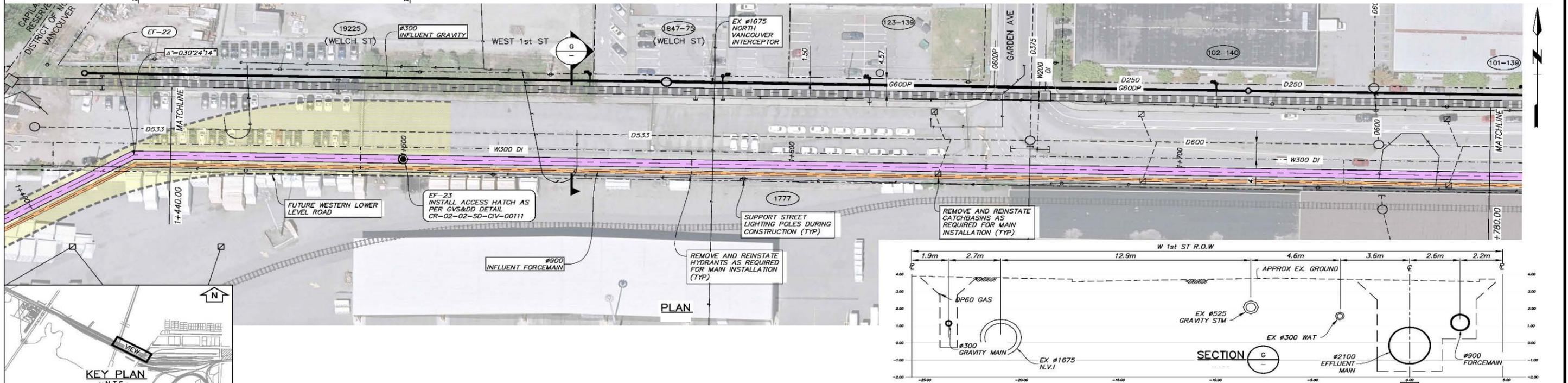
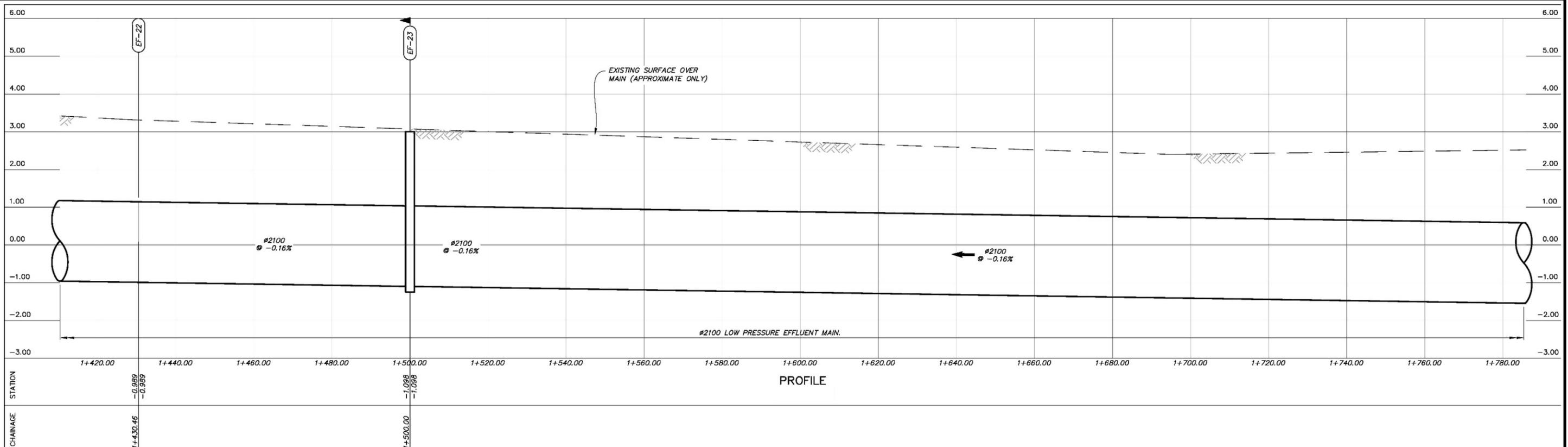
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DRAWING SYMBOLS			
	PIPELINE		CATCH BASIN
	TRAVERSE HUB		FIRE HYDRANT
	MONUMENT		SEWER HOUSE CONNECTIONS
	IRON PIN FOUND		SEWER
	LEAD PLUG FOUND		STORM DRAIN
	MANHOLE		GAS MAIN
	UTILITY POLE		CULVERT
	GUY/ANCHOR POLE		STORM HOUSE CONNECTION
	UTILITY VALVE BOX		UNDERGROUND TEL. CABLES
	GAS VALVE BOX		WATER MAIN
	RAILWAY SIGNAL		WATER HOUSE CONNECTION
	LAMP STANDARD		UG ELECT. CABLES
	HOUSE NUMBER		DITCH
	PAVEMENT		SIDEWALK
	FENCE		GRAVEL SURFACE
	ELECT. PULL BOX		SIGN
	HEDGE		TREE - CONIF/DEC
	CAPPED SERVICE PIPE		PROPOSED PERMANENT EASEMENT
	AIR VALVE		EXISTING PROPERTY LINE
	GRADE BREAK		TEMP. WORKING SPACE
	BLOWDOWN		

Reference  
 Conveyance for LGSWWTP – Effluent Outfall Plan and Profile by  
 AECOM dated October 2016

PROJECT		AECOM	
LIONS GATE SWWTP CONVEYANCE AND PUMP STATION		NORTH VANCOUVER, BC	
TITLE			
EFFLUENT OUTFALL PLAN AND PROFILE			
PROJECT No. 1544193		PHASE No. 1000	
DESIGN	BDA	08FEB17	SCALE NTS
CADD			REV. 0
CHECK	CHC	08FEB17	
REVIEW	ALF	10FEB17	
			Figure 2D

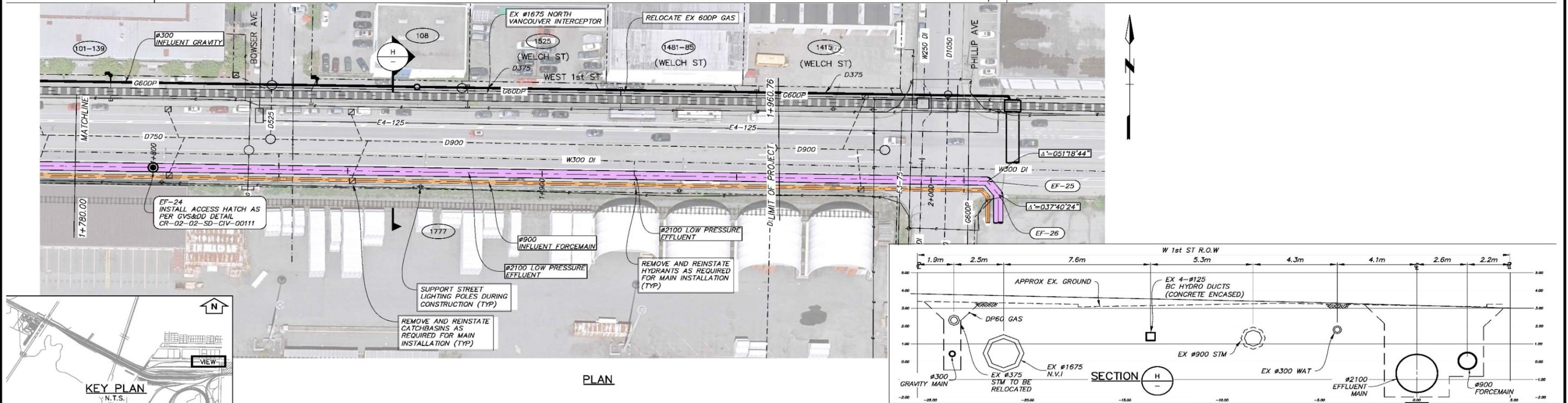
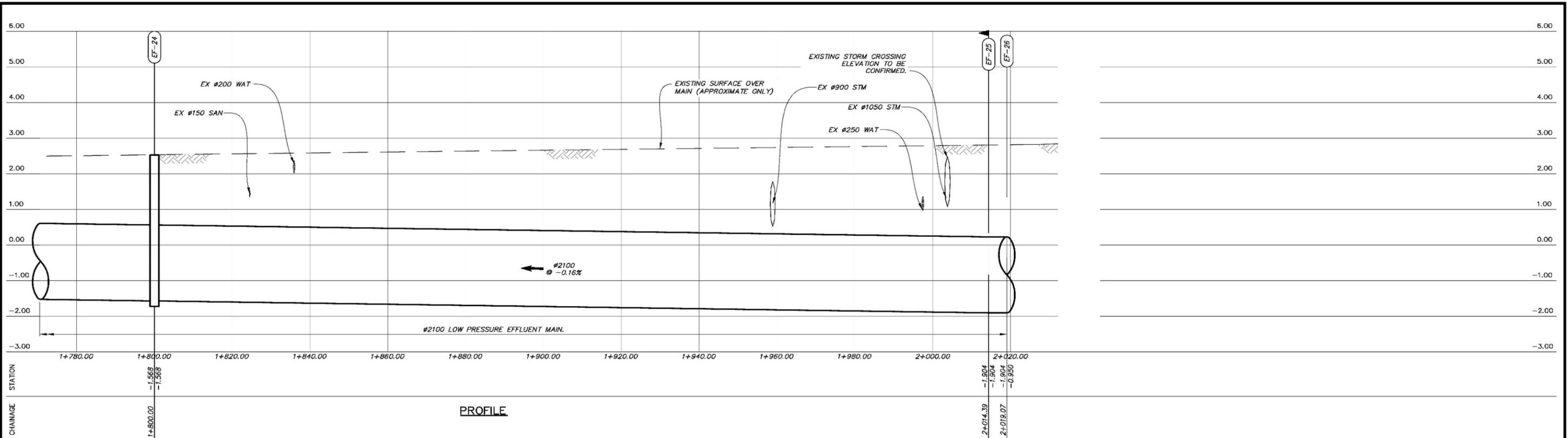


<ul style="list-style-type: none"> <li>△ HUB</li> <li>⊙ MON</li> <li>⊙ IP</li> <li>⊙ LP</li> <li>○ MANHOLE</li> <li>⊙ UTILITY POLE</li> <li>⊙ GUY/ANCHOR POLE</li> <li>⊙ UTILITY VALVE BOX</li> <li>⊙ GAS VALVE BOX</li> <li>⊙ RAILWAY SIGNAL</li> <li>⊙ LAMP STANDARD</li> </ul>	<ul style="list-style-type: none"> <li>□ CATCH BASIN</li> <li>⊙ H FIRE HYDRANT</li> <li>— SH — SEWER HOUSE CONNECTIONS</li> <li>— S — SEWER</li> <li>— D — STORM DRAIN</li> <li>— G — GAS MAIN</li> <li>— C — CULVERT</li> <li>— DH — STORM HOUSE CONNECTION</li> <li>— T — UNDERGROUND TEL. CABLES</li> <li>— W — WATER MAIN</li> <li>— WH — WATER HOUSE CONNECTION</li> <li>— E — UG ELECT. CABLES</li> </ul>	<ul style="list-style-type: none"> <li>2112 HOUSE NUMBER</li> <li>— DITCH</li> <li>— PAVEMENT</li> <li>— SIDEWALK</li> <li>— FENCE</li> <li>— GRAVEL SURFACE</li> <li>— ELECT. PULL BOX</li> <li>— SIGN</li> <li>— HEDGE</li> <li>— TREE — CONIF/DEC</li> <li>— CAPPED SERVICE PIPE</li> </ul>	<ul style="list-style-type: none"> <li>--- PROPOSED PERMANENT EASEMENT</li> <li>--- EXISTING PROPERTY LINE</li> <li>--- TEMP. WORKING SPACE</li> <li>AV AIR VALVE</li> <li>BD BLOWDOWN</li> <li>— GRADE BREAK</li> </ul>
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Reference  
 Conveyance for LGSWWTP – Effluent Outfall Plan and Profile by  
 AECOM dated October 2016

PROJECT		AECOM	
LIONS GATE SWWTP CONVEYANCE AND PUMP STATION		NORTH VANCOUVER, BC	
TITLE			
EFFLUENT OUTFALL PLAN AND PROFILE			
PROJECT No. 1544193		PHASE No. 1000	
DESIGN	BDA	08FEB17	SCALE NTS
CADD	CHC	08FEB17	REV. 0
CHECK	ALF	10FEB17	
REVIEW			
		Figure 2E	

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DRAWING SYMBOLS			
	PIPELINE		CATCH BASIN
	TRAVERSE HUB		FIRE HYDRANT
	MONUMENT		SEWER HOUSE CONNECTIONS
	IRON PIN FOUND		SEWER
	LEAD PLUG FOUND		STORM DRAIN
	MANHOLE		GAS MAIN
	UTILITY POLE		CULVERT
	GUY/ANCHOR POLE		STORM HOUSE CONNECTION
	UTILITY VALVE BOX		UNDERGROUND TEL. CABLES
	GAS VALVE BOX		WATER MAIN
	RAILWAY SIGNAL		WATER HOUSE CONNECTION
	LAMP STANDARD		UG ELECT. CABLES
	HOUSE NUMBER		DITCH
	PAVEMENT		SIDEWALK
	FENCE		GRAVEL SURFACE
	ELECT. PULL BOX		SIGN
	HEDGE		TREE - CONIF/DEC
	CAPPED SERVICE PIPE		PROPOSED PERMANENT EASEMENT
	EXISTING PROPERTY LINE		TEMP. WORKING SPACE
	AIR VALVE		GRADE BREAK
	BLOWDOWN		

Reference

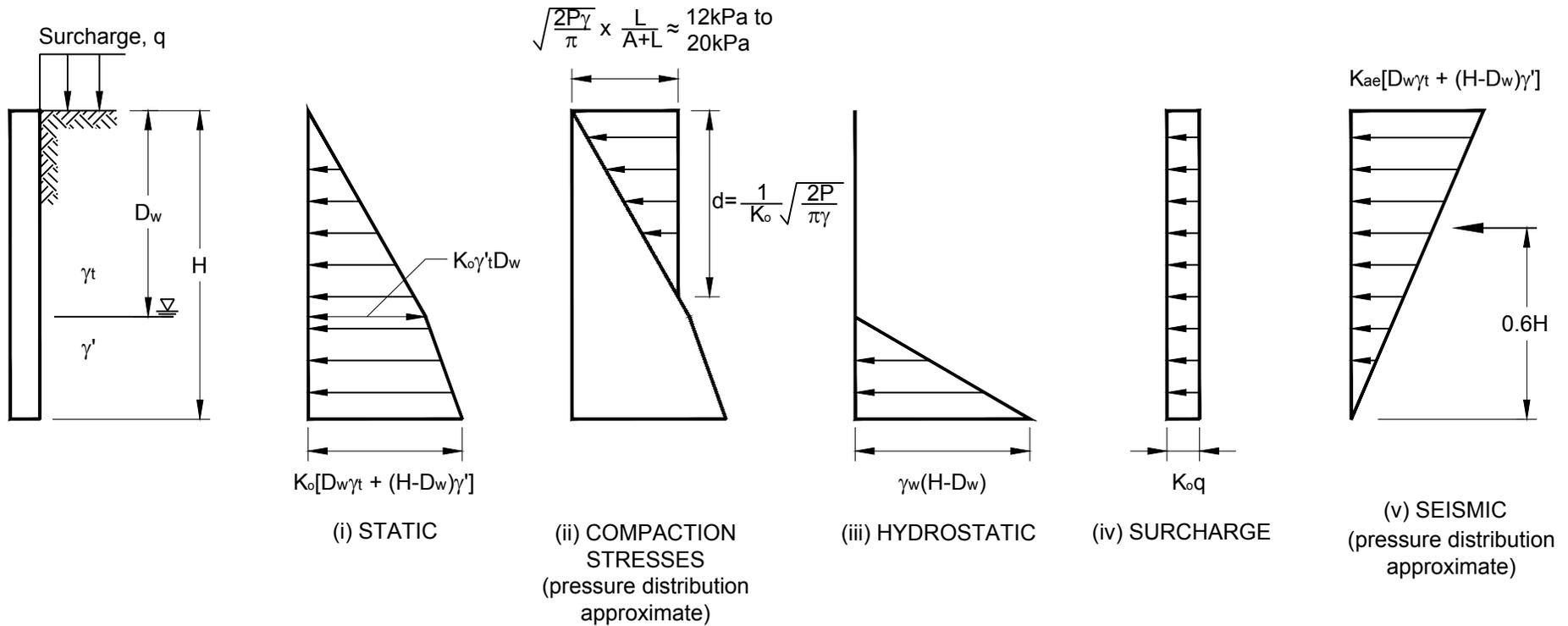
Conveyance for LGSWWTP – Effluent Outfall Plan and Profile by AECOM dated October 2016

PROJECT  
**AECOM**  
 LIONS GATE SWWTP CONVEYANCE AND PUMP STATION  
 NORTH VANCOUVER, BC

TITLE  
**EFFLUENT OUTFALL PLAN AND PROFILE**

	PROJECT No.	1544193	PHASE No.	1000	
	DESIGN	BDA	08FEB17	SCALE	NTS
	CADD			REV.	0
	CHECK	CHC	08FEB17	<b>Figure 2F</b>	
	REVIEW	ALF	10FEB17		

### Free Wall - Level Ground Surface



$\gamma_w$  = Water Unit Weight  
 $\gamma_t$  = Total Unit Weight  
 $\gamma'$  = Buoyant Unit Weight  
 $K_0$  = At-Rest Co-efficient of Lateral Earth Pressure  
 $q$  = Surcharge

$D_w$  = Depth to Water Table  
 $H$  = Height of Wall  
 $d$  = Depth Over Which Compaction Stresses are Applicable  
 $a$  = Peak Horizontal Acceleration  
 $g$  = Acceleration Due to Gravity

$P$  (Roller Load) =  $\frac{\text{Dead Weight of Roller} + \text{Centrifugal Force}}{\text{Width of Roller}}$

**NOTES**

- FOR STATIC CONDITIONS: CONSIDER THE COMBINED ACTION OF (i)+(iii)+(iv) FOR OVERTURNING, SLIDING AND BEARING, AND CONSIDER THE COMBINED ACTION OF (i)+(ii)=(iii)=(iv) FOR STRUCTURAL DESIGN.
- FOR SEISMIC CONDITIONS: CONSIDER THE COMBINED ACTION OF (IV)+(V). CONSIDER (IV) FOR DEAD LOADS ONLY.

CLIENT  
**AECOM**

CONSULTANT



YYYY-MM-DD    2017-03-14

PREPARED    H. KING

DESIGN    C. H. CHENG

REVIEW    C. H. CHENG

APPROVED    A. FULLER

PROJECT

**LIONS GATE SECONDARY WWTP SEWER CONVEYANCE AND PUMP STATION  
NORTH VANCOUVER. B.C.**

TITLE

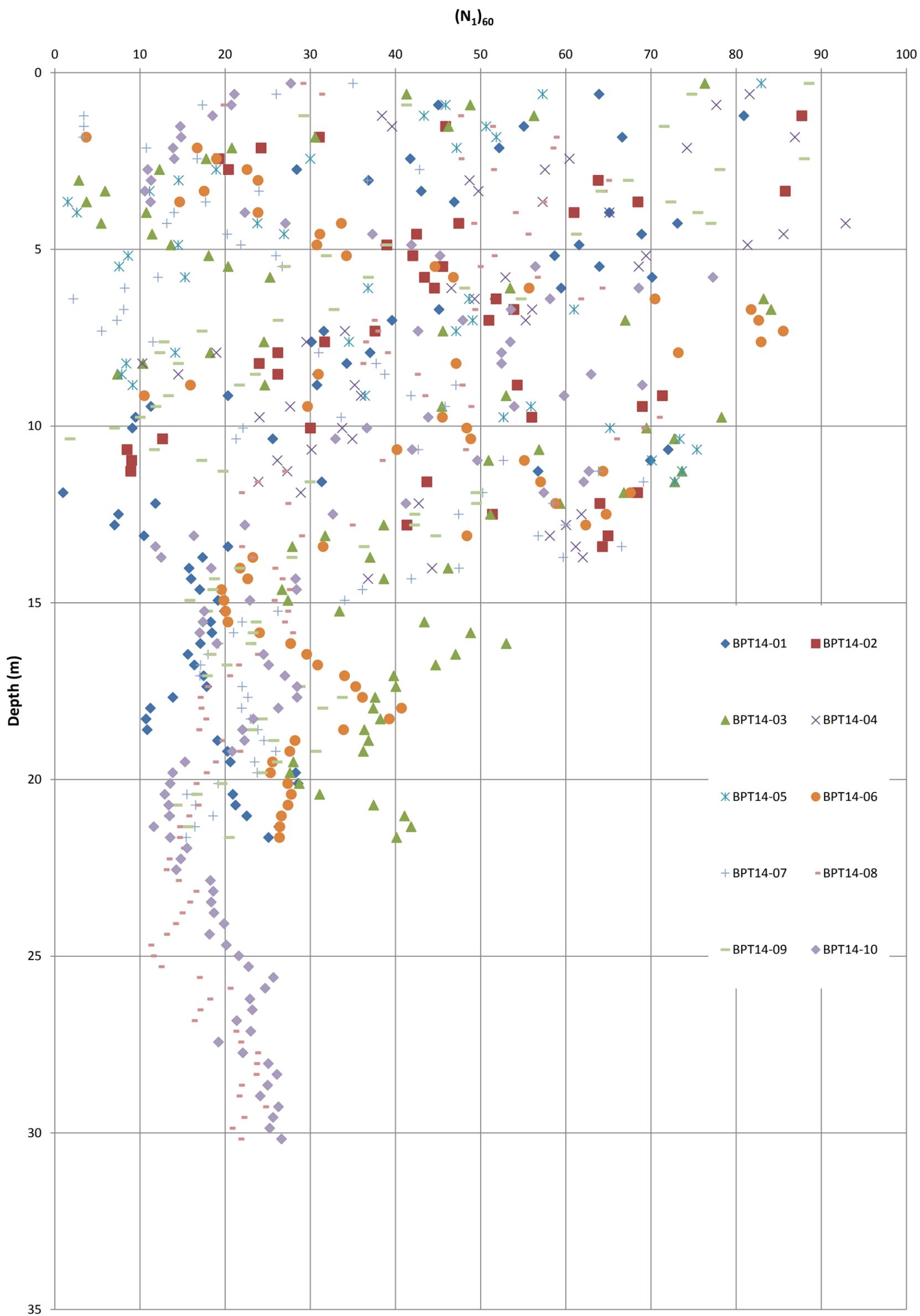
**LATERAL EARTH PRESSURE DIAGRAMS**

PROJECT No.  
**1544193**

PHASE  
**1000**

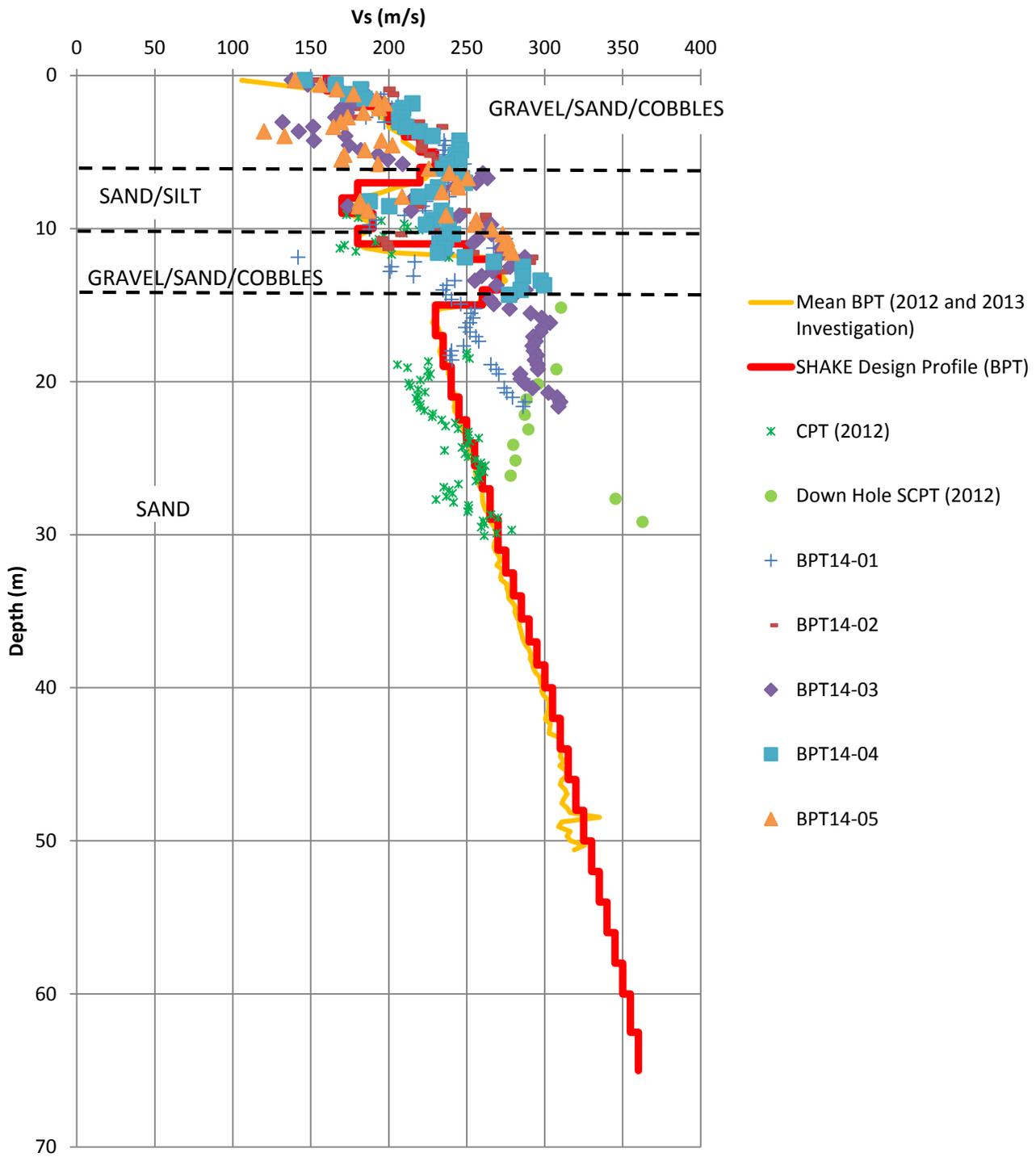
Rev.  
**0**

FIGURE  
**3**



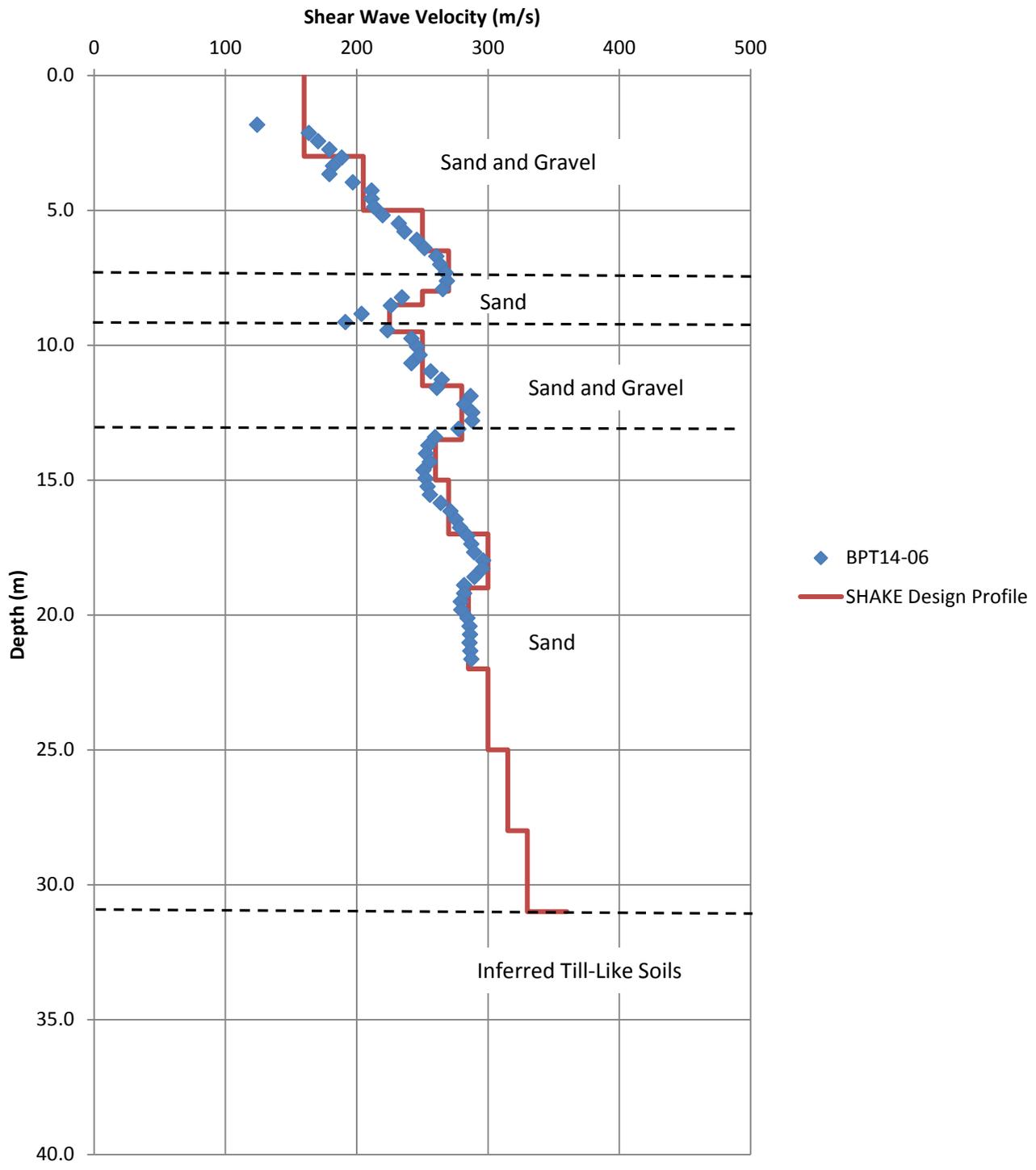
PROJECT		<b>AECOM</b>	
<b>LIONS GATE SWWTP CONVEYANCE &amp; PUMP STATION NORTH VANCOUVER VANCOUVER, B.C.</b>			
TITLE			
<b>INTERPRETED <math>(N_1)_{60}</math> PROFILE</b>			
PROJECT No. 1544193		PHASE No. 1000	
DESIGN	CHC	26JAN17	SCALE NTS    REV. 0
CADD	--		
CHECK	CHC	31JAN17	<b>FIGURE 4</b>
REVIEW	ALF	31JAN17	





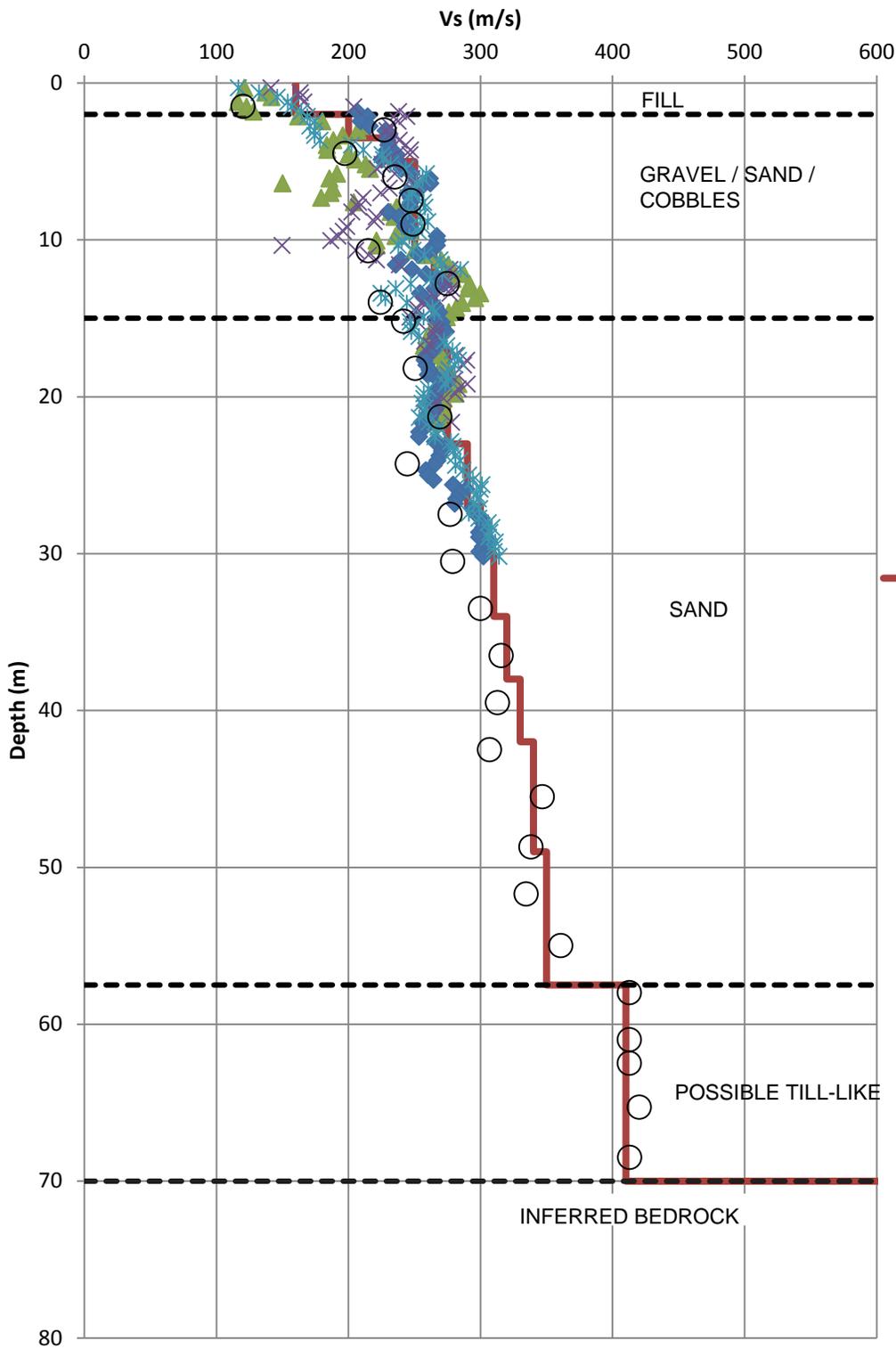
PROJECT		AECOM			
		LIONS GATE SWWTP CONVEYANCE & PUMP STATION NORTH VANCOUVER VANCOUVER, B.C			
TITLE		SHEAR WAVE VELOCITY AND DESIGN PROFILE, EASTERN PORTION OF ALIGNMENT			
		PROJECT No. 1544193		PHASE No. 1000	
DESIGN	CHC	26JAN17	SCALE	REV. 0	
CADD	--				
CHECK	CHC	31JAN17	<b>FIGURE 5A</b>		
REVIEW	ALF	31JAN17			





PROJECT		AECOM	
LIONS GATE SWWTP CONVEYANCE & PUMP STATION NORTH VANCOUVER VANCOUVER, B.C			
TITLE		SHEAR WAVE VELOCITY AND DESIGN PROFILE, CENTRAL PORTION OF ALIGNMENT	
PROJECT No. 1544193		PHASE No. 1000	
DESIGN	CHC	26JAN17	SCALE
CADD	--		REV. 0
CHECK	CHC	31JAN17	FIGURE 5B
REVIEW	ALF	31JAN17	

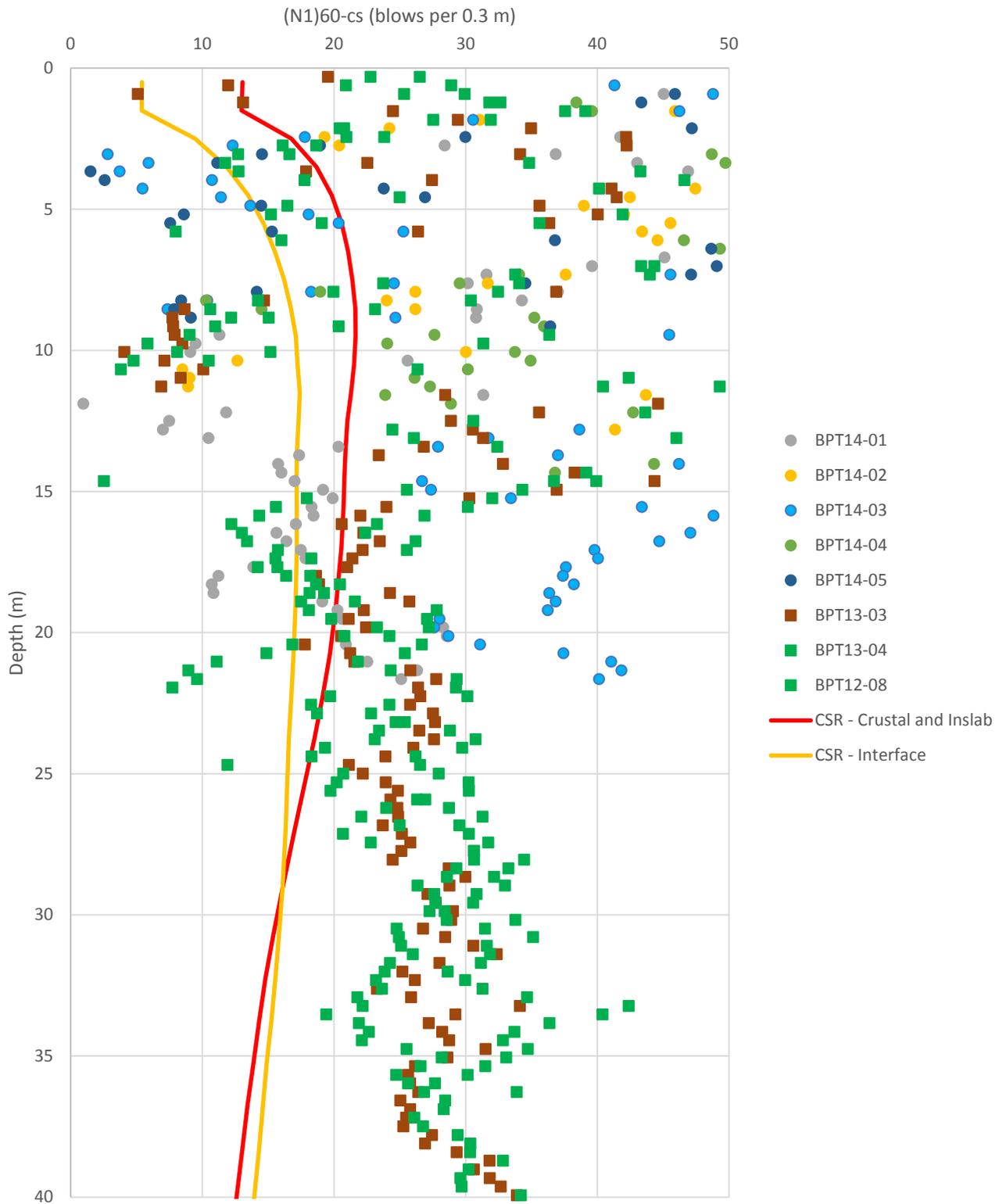




- SHAKE Design Profile
- ▲ BPT14-07
- ◆ BPT14-08
- × BPT14-09
- \* BPT14-10
- TH94-9

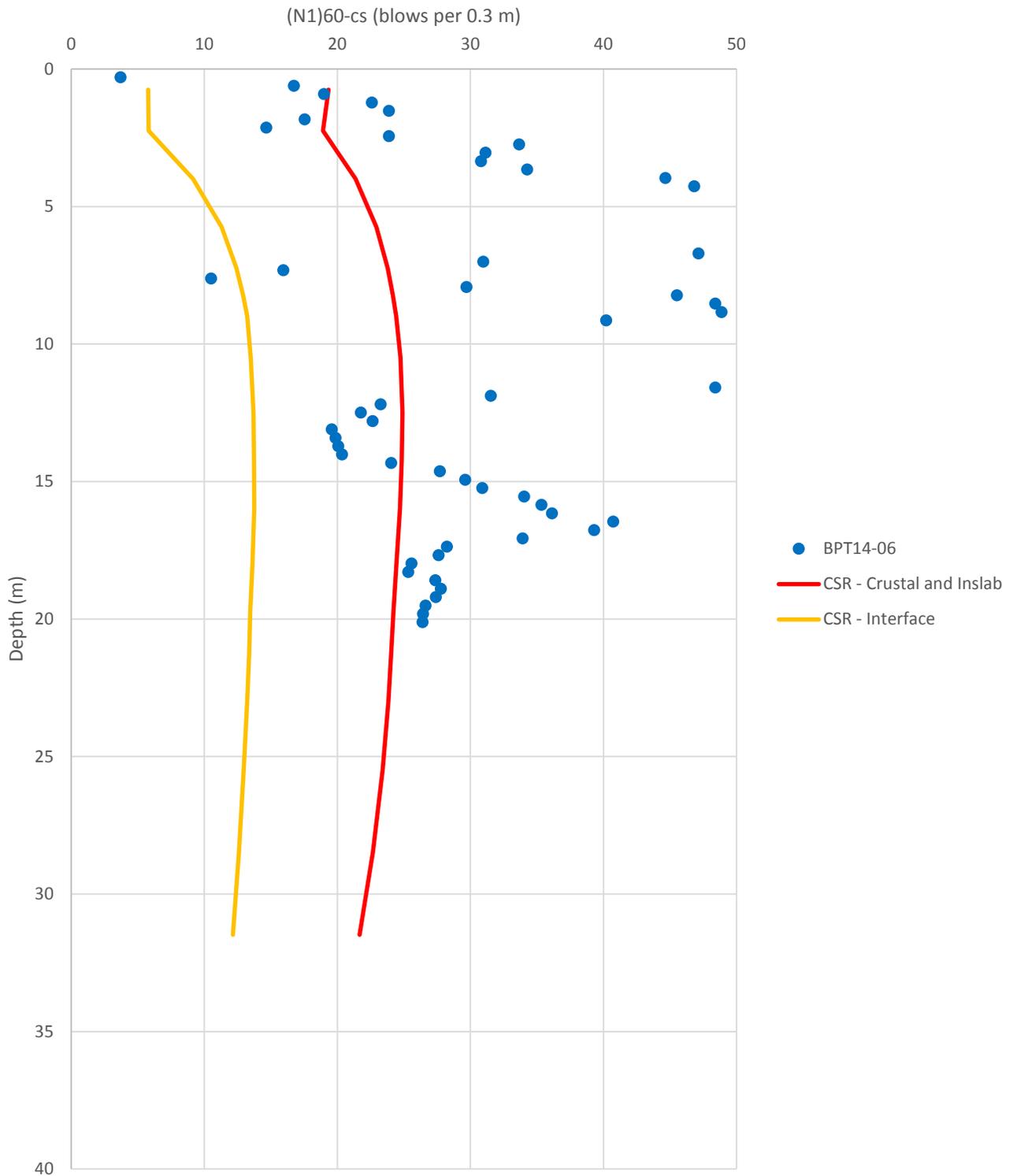
PROJECT		AECOM			
		LIONS GATE SWWTP CONVEYANCE & PUMP STATION NORTH VANCOUVER VANCOUVER, B.C			
TITLE		SHEAR WAVE VELOCITY AND DESIGN PROFILE, WEST PORTION OF ALIGNMENT			
		PROJECT No. 1544193		PHASE No. 1000	
DESIGN	CHC	26JAN17	SCALE	REV. 0	
CADD	--				
CHECK	CHC	31JAN17	<b>FIGURE 5C</b>		
REVIEW	ALF	31JAN17			



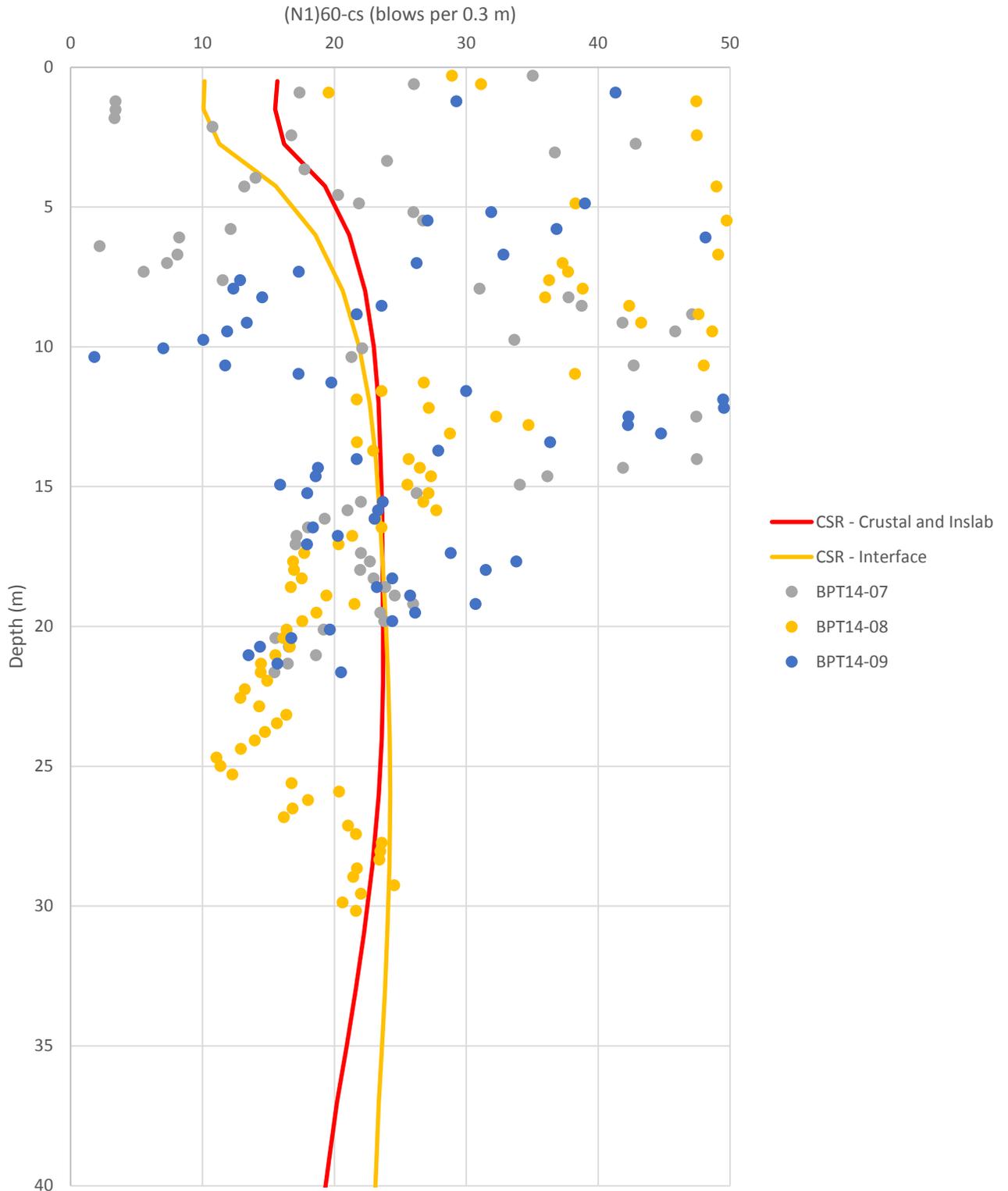


PROJECT		AECOM	
LIONS GATE SWWTP CONVEYANCE & PUMP STATION NORTH VANCOUVER VANCOUVER, B.C			
TITLE		LIQUEFACTION SUSCEPTIBILITY 975 YEAR EAST PORTION OF ALIGNMENT	
PROJECT No. 1544193		PHASE No. 1000	
DESIGN	CHC	26JAN17	SCALE
CADD			REV. 0
CHECK	CHC	31JAN17	<b>FIGURE 6A</b>
REVIEW	ALF	31JAN17	

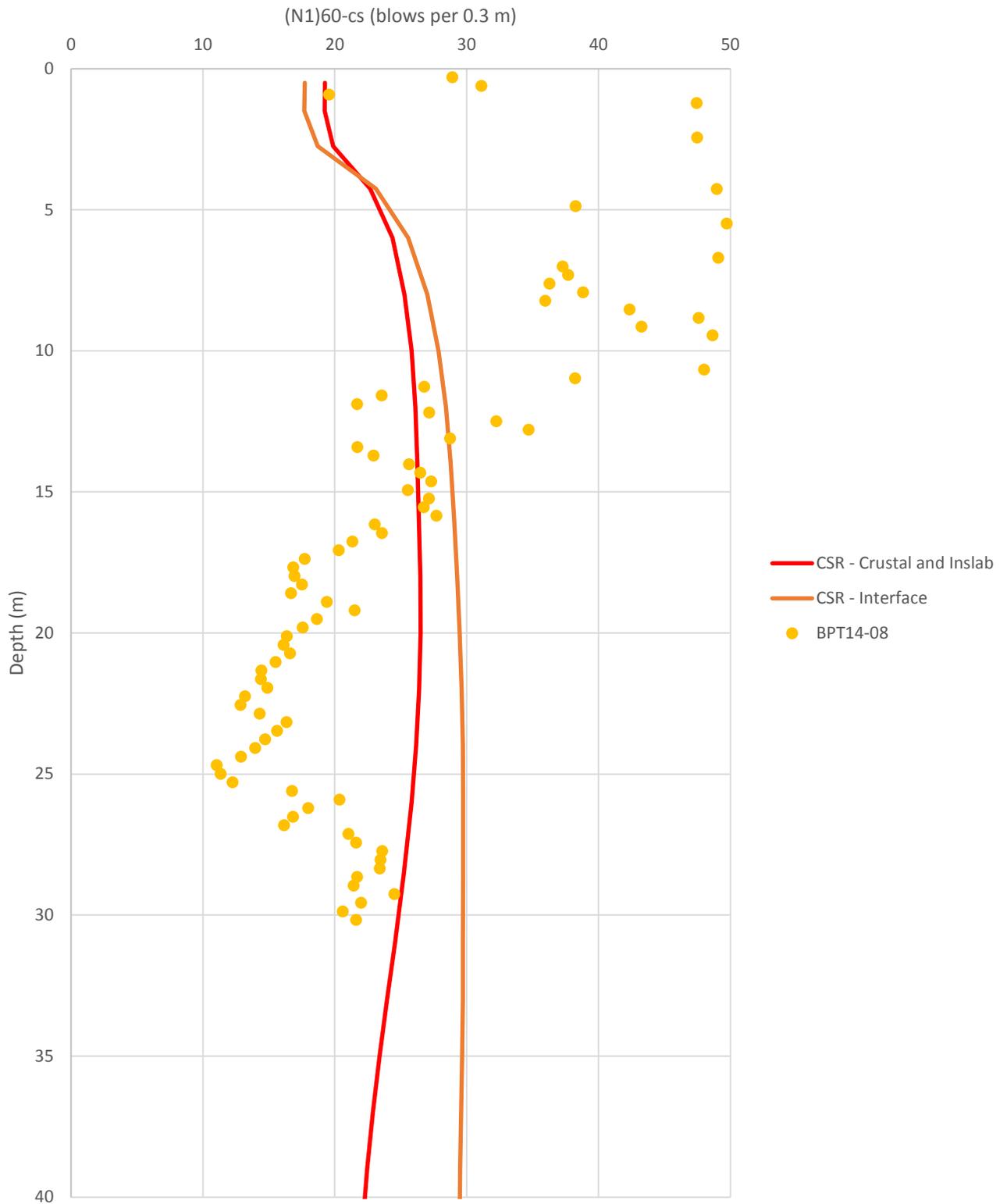




PROJECT		<b>AECOM</b>	
<b>LIONS GATE SWWTP CONVEYANCE &amp; PUMP STATION NORTH VANCOUVER VANCOUVER, B.C</b>			
TITLE		<b>LIQUEFACTION SUSCEPTIBILITY, 975 YEAR CENTRE PORTION OF ALIGNMENT</b>	
	PROJECT No. 1544193		PHASE No. 1000
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	CADD	--	REV. 0
	CHECK	CHC 31JAN17	<b>FIGURE 6B</b>
REVIEW	ALF 31JAN17		



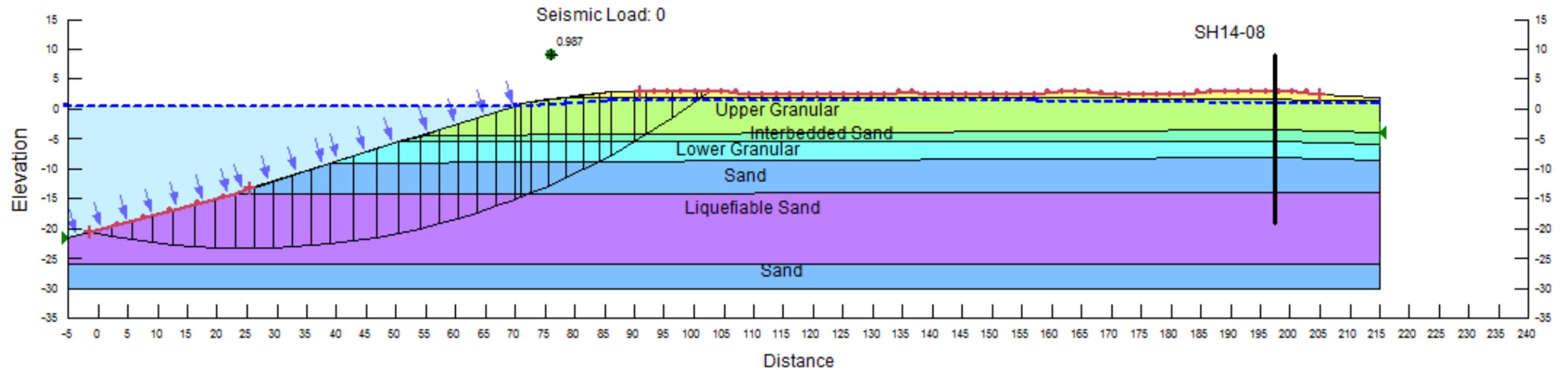
PROJECT		<b>AECOM</b>			
		<b>LIONS GATE SWWTP CONVEYANCE &amp; PUMP STATION NORTH VANCOUVER VANCOUVER, B.C</b>			
TITLE		<b>LIQUEFACTION SUSCEPTIBILITY, 975 YEAR WEST PORTION OF ALIGNMENT</b>			
	PROJECT No. 1544193		PHASE No. 1000		
	DESIGN	CHC	26JAN17	SCALE	REV. 0
	CADD	--			
	CHECK	CHC	31JAN17	<b>FIGURE 6C</b>	
REVIEW	ALF	31JAN17			



PROJECT		<b>AECOM</b>			
<b>LIONS GATE SWWTP CONVEYANCE &amp; PUMP STATION NORTH VANCOUVER VANCOUVER, B.C</b>					
TITLE		<b>LIQUEFACTION SUSCEPTIBILITY, 2475 YEAR PROPOSED PUMP STATION</b>			
PROJECT No. 1544193			PHASE No. 1000		
DESIGN	CHC	26JAN17	SCALE	REV. 0	
CADD	--				
CHECK	CHC	31JAN17	<b>FIGURE 6D</b>		
REVIEW	ALF	31JAN17			



Name: FILL Unit Weight: 18 kN/m<sup>3</sup> Phi: 30 ° Piezometric Line: 1  
 Name: Upper Granular Unit Weight: 20 kN/m<sup>3</sup> Phi: 34 ° Piezometric Line: 1  
 Name: Interbedded Sand Unit Weight: 19 kN/m<sup>3</sup> Phi: 30 ° Piezometric Line: 1  
 Name: Lower Granular Unit Weight: 22 kN/m<sup>3</sup> Phi: 34 ° Piezometric Line: 1  
 Name: Sand Unit Weight: 19 kN/m<sup>3</sup> Phi: 33 ° Piezometric Line: 1  
 Name: Liquefiable from 16m to 30m Unit Weight: 19 kN/m<sup>3</sup> Tau/Sigma Ratio: 0.13 Minimum Strength: 0 Piezometric Line: 1

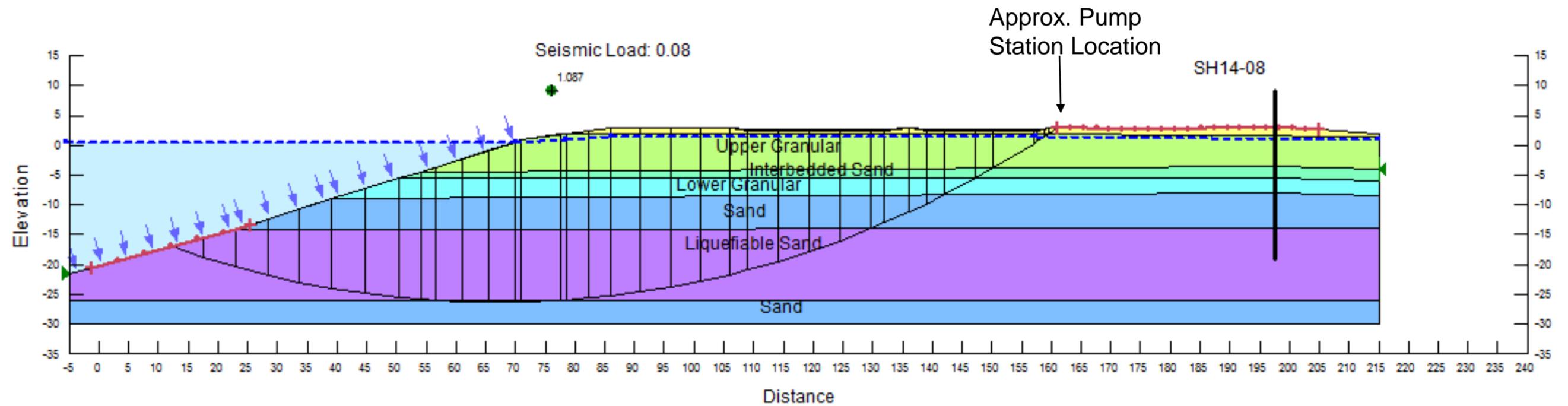


Factor of Safety <1, Flow Slide

PROJECT		AECOM	
LIONS GATE SWWTP CONVEYANCE & PUMP STATION NORTH VANCOUVER VANCOUVER, B.C			
TITLE			
SEISMIC SLOPE STABILITY ANALYSIS, 975 YEAR BURRARD INLET			
PROJECT No. 1544193		PHASE No. 1000	
DESIGN	CHC	26JAN17	SCALE NTS
CADD	--		REV. 0
CHECK	CHC	31JAN17	FIGURE 7A
REVIEW	ALF	31JAN17	



Name: FILL Unit Weight: 18 kN/m<sup>3</sup> Phi: 30 ° Piezometric Line: 1  
 Name: Upper Granular Unit Weight: 20 kN/m<sup>3</sup> Phi: 34 ° Piezometric Line: 1  
 Name: Interbedded Sand Unit Weight: 19 kN/m<sup>3</sup> Phi: 30 ° Piezometric Line: 1  
 Name: Lower Granular Unit Weight: 22 kN/m<sup>3</sup> Phi: 34 ° Piezometric Line: 1  
 Name: Sand Unit Weight: 19 kN/m<sup>3</sup> Phi: 33 ° Piezometric Line: 1  
 Name: Liquefiable from 16m to 30m Unit Weight: 19 kN/m<sup>3</sup> Tau/Sigma Ratio: 0.13 Minimum Strength: 0 Piezometric Line: 1



PROJECT		AECOM		
LIONS GATE SECONDARY WWTP SEWER CONVEYANCE NORTH VANCOUVER AND WEST VANCOUVER, B.C.				
TITLE				
SEISMIC SLOPE STABILITY ANALYSIS, 2475 YEAR PROPOSED PUMP STATION				
PROJECT No. 1544193		PHASE No. 1000		
DESIGN	CHC	26JAN17	SCALE NTS	REV. 0
CADD	--			
CHECK	CHC	31JAN17	<b>FIGURE 7B</b>	
REVIEW	ALF	31JAN17		

