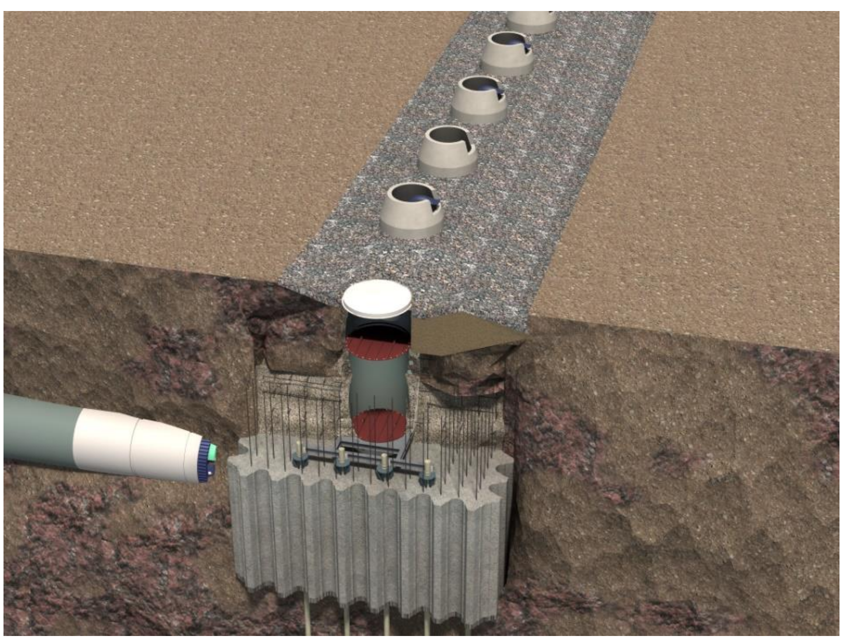
APPENDIX B
GEOTECHNICAL REPORTS

B.4: 2D Ground Response Analysis

Annacis Island WWTP New Outfall System

Vancouver Fraser Port Authority
Project and Environmental Review Application



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04 December 2017

SEISMIC DEFORMATION ANALYSES

AIWWTP Transient Mitigation and Outfall System

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REPORT

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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).

This report summarizes the results of the two-dimensional ground deformation analyses carried out to assess the potential liquefaction of the site soils and the resulting lateral spreading a under the design ground motions consistent with the 2010 National Building Code of Canada (NBCC) and 2015 NBCC. In addition, the lateral and vertical displacement profiles, the stiffness and strength parameters and the load resistance curves were provided as input to the soil-structure interaction (SSI) analyses of the on-land and riser shafts and the outfall and effluent tunnels; these inputs are summarized herein.

It is noted that the geotechnical input parameters required for SSI analyses were only provided for the 2010 NBCC design ground motions in accordance with the project scope. The analyses associated with the 2015 NBCC were carried out for comparison purposes only.

The factual results of the field investigation program completed for the subject development, along with interpretations of the subsurface stratigraphy, are presented in the Geotechnical Interpretive Report (GIR) dated 31 May 2017. Golder was also retained to 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. Two alternative gravity outfall options following central and western alignments were proposed to increase the capacity during conceptual design, as shown on Figure 2-1.

A single outfall located about 200 metres (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. The Option 6 outfall alignment is shown on Figure 2-1.

The outfall conduit 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 design grade.

The site is located on the floodplain of the Fraser River. The ground surface in the area surrounding the AIWWTP is generally flat, with a nominal grade at elevation (EL) 104.5 m (CGVD28-GVRD datum). The ground surface is generally flat or slopes gently towards the Fraser River along the proposed outfall corridor.

Prior to development, the ground at the Annacis Island site was at approximately EL 100 m. Since then, 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. There is a known anomaly in the river bed called Mungo's Hole. It is located near the southern edge of the Fraser River, across from the project site. Mungo's Hole is a 30-m deep erosional feature that developed after numerous upstream construction activities.

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the subject site were established based on the results of the field investigations carried out along the western, central and final Option 6 outfall alignment corridors, with a specific focus on the Option 6 alignment corridor. The results indicate that the site is underlain by fill (Unit 1) overlying overbank deposits (Unit 2) comprising clayey silt and organic silt, which is followed by a Fraser River sand deposit (Unit 3). The Fraser River sand deposit, in turn, is underlain by an extensive marine sequence (Unit 4) comprising interlayered fine sand and clayey silt to silty clay, followed by a glacio-marine deposit (Unit 7).

The glacio-marine deposit was encountered at depths ranging from 60 to 80 m below ground surface on land, while the deposit was encountered at a depth of about 55 m below mudline in the offshore area. The glacio-marine deposit is inferred to be underlain by a glacial deposit comprising till-like soils. The till-like soils were encountered in the offshore area at a depth of 80 m below mudline near the Option 6 outfall alignment.

A stratigraphic profile along the Option 6 outfall alignment, including a proposed tunnel segment leading to a future shaft that will connect to the Stage V expansion, was developed and is shown on Figure 3-1. The stratigraphic profile was developed considering the test holes put down during the supplementary investigation completed along the Option 6 outfall alignment, as well as the test holes put down as part of the previous investigations along the conceptual alignments. In addition, a stratigraphic profile along the effluent tunnel leading to the effluent shaft from the outfall shaft was also developed, and is shown on Figure 3-1. All elevations shown on Figures 3-1 are with respect to the CGVD28-GVRD datum, which is geodetic datum plus 100 metres.

The natural groundwater level at the site is expected to vary with the water level in the river, change in season, and amount of precipitation. Based on available information, the groundwater levels on land vary between Elevations 100 m and 101 m relative to the CVGD28-GVRD datum.

4.0 SITE HAZARD PARAMETERS

The following sections briefly summarize the design Site Class C acceleration spectra and the input time-histories used in the two-dimensional (2D) ground deformation analyses. Further details on the design acceleration spectra and the time histories related to the 2010 NBCC and 2015 NBCC ground motions can be found in the Technical Memorandum "Seismic Design Criteria and Performance Expectation – AIWWTP Transient Mitigation and Outfall System" dated 08 July 2016.

4.1 Design Acceleration Response Spectra 2010 NBCC and 2015 NBCC

The site-specific hazard parameters based on both the 4th and 5th generation seismic hazard models were obtained from Natural Resources Canada and they are summarized in Table 4-1. The parameters correspond to a "reference ground condition" referred to as Site Class C. The Site Class C in the 4th generation hazard maps are defined as having an average shear wave velocity (Vs) varying between 360 m/s and 760 m/s within the upper 30 m, while it is defined in the 5th generation hazard maps by an average Vs of 450 m/s within the upper 30 m. Note that the spectral values up to 2 seconds are available for the ground motions associated with the 2010 NBCC, while they are available up to 10 seconds for the 2015 NBCC ground motions.

Table: 4-1: Site-Specific Probabilistic Firm-Ground Motion Parameters (Site Class C)

Return Period (2,475 Years)	PHGA	Sa (0.2s)	Sa (0.5s)	Sa (1.0s)	Sa (2.0s)
2010 NBCC [4th generation model]	0.51 g	1.03 g	0.68 g	0.34 g	0.17 g
Subduction Earthquake [4th generation model]	0.16 g	0.37 g	0.31 g	0.17 g	0.09 g
2015 NBCC [5th generation model]	0.36 g	0.84 g	0.75 g	0.42 g	0.25 g
Subduction Earthquake [5th generation model]	0.14 g	0.0.29 g	0.34 g	0.27 g	0.19 g

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

The 4th generation seismic hazard model considers the shallow crustal and deep inslab earthquake sources in the probabilistic seismic hazard assessment (PSHA) and the interface (subduction) earthquake is considered separately using a deterministic approach. However, the subduction earthquake sources are incorporated into the PSHA for the 5th generation hazard model. In addition, for the 5th generation model, the expected earthquake magnitude of a subduction event is determined to be M9 with an epicentral distance of 120 km, compared to M8.2 and 160 km in the 4th generation hazard model.

Although the PSHA for the 5th generation model incorporates the subduction earthquake sources, design spectra associated with the crustal and inslab, and the subduction earthquake sources, were considered separately to develop the applicable input time-histories. Spectrally matching time-histories over the full period range (from PHGA to 10 seconds) is not recommended as this would result in "modified" earthquake records that are very different from those observed from past earthquakes, resulting in increased displacement demand at long periods.

The 2,475-yr Uniform Hazard Response Spectra and the subduction earthquake spectra provided by NRCan for the 4th and 5th generation seismic hazard models, as summarized in Table 4-1, are shown in Figure 4-1.

4.2 Acceleration Time Histories – 2010 NBCC and 2015 NBCC

Consistent with the seismic ground motions that have been used for the Stage V expansion, three sets of ground motions were developed for the 2010 NBCC ground motions, with each set comprising two single-component time-histories to represent the crustal and inslab earthquakes and one ground motion comprising two single-component time-histories to represent the subduction earthquakes. The time histories were matched to the design Site Class C spectra consistent with the 2010 NBCC ground motions, and they are shown on Figures 4-2a and 4-2b, for the crustal and inslab, and interface earthquakes, respectively.

Dr. Tuna Onur was retained to develop the applicable time histories based on the site-specific ground motion parameters consistent with the 2015 NBCC. A total of 11 single-component acceleration time-histories were developed to represent the crustal and inslab earthquakes, and they were spectrally matched to the 2015 NBCC UHRS (Site Class C) over a period range extending from PHGA to about 2.0 seconds, as shown on Figure 4-3a and b. A total of five single-component acceleration time-histories were developed to represent the interface earthquakes, and they were spectrally matched to the interface spectrum as shown on Figure 4-3c.

Further details on the earthquake acceleration time histories related to the 2010 NBCC and 2015 NBCC can be found in the Technical Memorandum “Seismic Design Criteria and Performance Expectation – AIWWTP Transient Mitigation and Outfall System” dated 08 July 2016.

5.0 GROUND RESPONSE ANALYSES

One and two-dimensional (1D & 2D) ground response analyses were carried out as input to the design of the new outfall system. 1D ground response analyses were carried out to assess the liquefaction potential of the site soils based on the simplified method of analyses under the design ground motions consistent with the 2010 NBCC and the 2015 NBCC. The 2D ground response (or deformation) analyses were carried out to further evaluate the extent of liquefaction and the resulting permanent ground deformations along the final Option 6 outfall alignment for “as-is” ground conditions.

The results of the 1D ground response analyses are summarized under separate cover. The following sections summarize the details of the 2D ground deformation analyses undertaken at various phases during the course of the outfall system design. The displacement profiles, stiffness and strength parameters, and the load resistance curves (i.e., p-y, t-z, Q-z curves) provided based on the results of the 1D and 2D ground response analyses, as input to the soil-structure interaction analyses carried out by others, are also summarized herein.

5.1 Analyses Methodology

Detailed ground response analyses were carried out to evaluate the permanent ground displacements under the design ground motions using the 2D finite difference computer code FLAC2D (Version 7.00.424), developed by Itasca Consulting Ltd. FLAC allows the domain of interest to be modeled by elements or zones. Each element or zone behaves according to a prescribed stress-strain law in response to the applied forces and/or boundary conditions. In addition, via subroutines, the program allows the implementation of specific constitutive relations to appropriately model phenomena such as liquefaction and the associated softening and strength reductions in soils.

The site soils were modeled as Mohr-Coulomb materials to numerically simulate the characteristic behaviour of the different materials under static loading conditions.

The user-defined constitutive model UBCSAND, which is capable of capturing the liquefaction potential of granular soils, was used to numerically simulate the characteristic behaviour of the existing fill and sand layers (Units 1 & 3) under seismic loading conditions. The primary input parameters for the UBCSAND model are as follows:

- normalized standard penetration test (SPT) blow counts $(N_1)_{60}$ corrected for fines content (i.e., $(N_1)_{60cs}$)
- unit weight and internal friction angle (ϕ)
- hydraulic conductivity

The organic silt/clayey silt layer (Unit 2) that is considered not liquefiable under the design ground motions was modeled as a non-linear Mohr-Coulomb material using a hyperbolic stress-strain relation and modified Masing's rule incorporated in the user-defined routine UBCHYST. The following input parameters were utilized to capture the dynamic behaviour of fine-grained soils using the UBCHYST model under dynamic loading:

- unit weights and undrained shear strength
- small strain shear modulus
- modulus reduction and damping curves
- hydraulic conductivity

Both the UBCSAND and UBCHYST models were developed by Prof. Peter M. Byrne and his colleagues at the University of British Columbia (UBC).

The marine deposits (Units 4 & 7) underlying the site, which are also considered not liquefiable under the design ground motions, were modeled as Mohr-Coulomb materials with an equivalent linear approach (ELA) to simulate the non-linear cyclic behaviour. The model includes shear modulus degradation with shear strain and shear strain-dependent damping using a three-parameter hysteretic model built into FLAC.

The site is also expected to undergo additional settlements after shaking due to excess pore water pressure dissipation in the liquefied zones (i.e., post-seismic consolidation of liquefied materials). The post seismic consolidation settlements were estimated based on the following two approaches:

i) Approach 1: Utilizing the results of the 1D ground response analyses (i.e., SHAKE2000) to establish the Factor of Safety (FoS) against liquefaction with depth at the individual CPT locations along the Option 6 outfall alignment, the volumetric strains with depth were estimated based on the empirical chart (volumetric strain vs FoS) developed by Yoshimine (1992), considering the FoS computed with depth.

ii) Approach 2: Utilizing the FLAC results to obtain the shear strain profiles along the tunnel alignment below the tunnel invert due to seismic shaking, the volumetric strains were estimated based on the empirical chart (volumetric strain vs shear strain) developed by Yoshimine (1992), considering the computed shear strain profiles.

Approach 1 was generally used to estimate the post-seismic consolidation settlements along the shafts and tunnels. Approach 2 was considered to assess the settlements along the outfall tunnel for comparison purposes.

5.2 Analyses Stages

The detailed ground deformation analyses were carried out in three phases during the course of the project. Table 5-1 summarizes the details of the analyses completed in the three phases for the outfall design, which are discussed in the following sections.

Table 5-1: Summary of Analyses

Variable	Analyses Stages		
	Initial Phase	**Subsequent Phase**	**Final Phase**
Stratigraphic Profile	Sections A-A' & C-C' – Initial Analyses	Section A'A'	Section A-A' – Adjusted for data along Option 6 Alignment
	Section A –A'– Follow Up Analyses		
Design Ground Motions	2010 NBCC – Initial Analyses	2010 NBCC	2010 and 2015 NBCC
	2015 NBCC – Follow Up Analyses		
Design SPT Profile	33^{rd} Percentile $(N_1)_{60cs}$	50^{th} Percentile $(N_1)_{60cs}$	

5.2.1 Initial Phase

The SPT $(N_1)_{60cs}$ design profiles corresponding to the 33^{rd} percentile values, as per the Task Force guidelines (2007)[1], and the design ground motions corresponding to a return period of 2,475 years consistent with the 2010 NBCC, were considered in the initial phase of the analyses. The SPT $(N_1)_{60cs}$ design profiles were established based on the subsurface data available at that time, along the conceptual western and central outfall alignment corridor, prior to completing the field program along the preferred final Option 6 outfall alignment corridor.

The analyses were carried out considering a section along the Option 6 outfall alignment extending through Mungo's Hole (i.e., Section A-A'), and also a section through the edge of Mungo's hole to exclude the possible effect of the Mungo's Hole (i.e., Section C-C'), as shown on Figure 3-1. It is noted that both sections are generally perpendicular to the river bank. The lateral ground deformations are expected to be influenced by the variations in the soil parameters as well as the input ground motions with respect to their polarity; therefore, the ground deformation analyses were carried out for the following cases:

a) Case 1: Design SPT $(N_1)_{60cs}$ profile based on combined penetration resistance data from both onshore and offshore areas.

b) Case 2: Design SPT $(N_1)_{60cs}$ profile same as Case 1 with the input ground motions applied with reversed polarity.

[1]Geotechnical Guidelines For Buildings On Liquefiable Sites in Greater Vancouver, dated 08 May 2007

c) Case 3: Design SPT $(N_1)_{60cs}$ profiles considering the penetration resistance data from the on-land and offshore areas separately.

d) Case 4: Design SPT $(N_1)_{60cs}$ profile same as Case 3 with the input ground motions applied with reversed polarity.

The design SPT $(N_1)_{60cs}$ profiles used in the analyses noted above are shown on Figure 5-1. The initial phase also included follow-up analyses based on the seismic hazard parameters associated with the 2015 NBCC to assess the potential impact associated with the updated seismic hazard parameters. They were carried out corresponding to Case 1 for comparison purposes.

Technical Memoranda presenting a summary of the results of the analyses carried out for the design ground motion parameters consistent with both the 2010 NBCC and 2015 NBCC were issued on 15 September 2016 and 05 January 2017, respectively; they are included in Appendices A and B, respectively.

5.2.2 Subsequent Phase

Following the initial phase of the analyses, and based on input from the peer reviewers (Drs. Dharma Wijewickreme and Liam Finn retained by CDM), further analyses were carried out based on the SPT $(N_1)_{60cs}$ design profiles corresponding to the 50th percentile data rather than the 33rd percentile data, considering the potential impact of the $(N_1)_{60cs}$ profiles adopted on the predicted ground deformations. The analyses were carried out for the design ground motions consistent with the 2010 NBCC only.

It is noted that the recent studies carried out by Boulanger and Montgomery (2016) [2] indicated that the representative $(N_1)_{60cs}$ for use in uniform models can range from the 30th to the 70th percentile of the stochastic $(N_1)_{60cs}$ distributions, depending on the intensity of shaking and the variations in the soil parameters, topography, etc. The studies also indicated that the 50th percentile data may be representative for the uniform model considering the design ground motions associated with the site.

The analyses were previously carried out for four cases as noted in Section 5.2.1 considering variations in the soil parameters in the on-land and offshore areas. The subsequent analyses to assess the sensitivity of the design profiles of $(N_1)_{60cs}$ were carried out corresponding to Case 1 only. Figure 5-2 shows a comparison between the design $(N_1)_{60cs}$ profiles associated with the 33rd and 50th percentile values, and the difference is about two to three blow counts.

[2]Nonlinear deformation analyses of an embankment dam of a spatially variable liquefiable deposit. Ross W Boulanger, Jack Montgomery "Soil Dynamics and Earthquake Engineering", July 2016

5.2.3 Final Phase

The SPT $(N_1)_{60cs}$ design profiles obtained previously from a statistical evaluation were re-evaluated with the incorporation of the field investigation results from the Option 6 outfall alignment corridor during the final phase. The statistical evaluation was carried out for the following cases:

a) Scenario 1: All the SPT and CPT data along the Option 6 alignment.

b) Scenario 2: All the data obtained at the project site (including the SPT and CPT data from both western and central alignments that were not considered as part of Scenario 1).

The results of the statistical evaluation for the 33rd and 50th percentile SPT $(N_1)_{60cs}$ values are shown on Figures 5-3 and 5-4. The on-land and offshore data sets have been analysed separately as well as combined, to assess their impact on the design profiles, as shown on Figures 5-3 and 5-4.

It is noted that there is no significant difference in the design profiles between Scenarios 1 and 2 based on the data sets corresponding to the on-land area, as shown on Figures 5-3 and 5-4. However, there are differences in the design profiles corresponding to the offshore area, especially within the upper 8 m where the soils are relatively looser along the Option 6 alignment (Scenario 1) compared to the overall data (Scenario 2). In addition, the lower portion of the sand deposit along the Option 6 alignment is relatively looser compared to the overall data in the offshore area (i.e., EL 69 m to EL 65 m). Further details on the data associated with Scenarios 1 and 2 can be found in the Geotechnical Interpretive Report dated 31 May 2017.

Based on the above, it was considered that the $(N_1)_{60cs}$ design profiles associated with Scenario 1 were representative for the detailed ground response analyses. Additional analyses were therefore carried out considering the following cases:

a) 2010 NBCC with the 50th percentile $(N_1)_{60cs}$ design profiles based on Scenario 1, considering the results of the statistical evaluation as noted above.

b) 2015 NBCC with the $(N_1)_{60cs}$ design profiles the same as item a).

It is noted that the analyses were carried out based on the SPT $(N_1)_{60cs}$ design profiles corresponding to the 50th percentile data based on input from the peer reviewers and the recent studies carried out by Boulanger and Montgomery (2016) as noted previously.

5.3 FLAC^{2D} Models and Dynamic Analyses

Following completion of the single element calibration and the 1D comparison of FLAC and SHAKE results as discussed above, 2D models were generated in the computer code FLAC as a collection of single elements along the soil profile. The ground response was simulated in the different soil zones as earthquake loading progressed while complying simultaneously with equilibrium, strain compatibility, boundary conditions, and the prescribed stress-strain laws.

The finite difference models developed for the 2D ground response analyses during the initial phase are shown on Figures 5-5 and 5-6 for the sections extending south through Mungo's hole (Section A-A') and through the edge of Mungo's Hole (Section C-C'), respectively. It is noted that the model developed for Section A-A' was only considered in the follow up analyses carried out in the initial phase as well as in the subsequent and final phases (see Table 5-1). In addition, the stratigraphic profile along Section A-A' was adjusted to reflect the results of the investigation carried out along the Option 6 outfall alignment corridor and the updated model is shown on Figure 5-7.

It is noted that the lower model boundary extends to the Class C ground conditions. Competent ground comprising Pleistocene deposits was not encountered directly along the final preferred alignment; however, it was observed at one location just to the west of the proposed outfall shaft location. In addition, there are limited site-specific shear wave velocity measurements available at depth from which to evaluate the Class C conditions along the alignment; therefore, a correlation developed by Hunter (1995) for the Fraser River Delta was used to establish the Class C ground profile across the site for the purpose of modelling. Figure 5-8 illustrates the Class C ground profile established along Section A-A' based on the correlation in the initial phase.

The analyses were conducted using the effective-stress approach, where generation, distribution and dissipation of excess pore water pressure of sand-like layers during earthquake shaking were accounted for.

The 2D ground response analyses were carried out for a series of six spectrum-compatible earthquake records representing the crustal and inslab seismic sources, and two earthquake records representing the interface event, as input motions under the design ground motions consistent with the 2010 NBCC. The analyses were carried out for a total of sixteen earthquake records representing the crustal, inslab and interface seismic sources as input motions under the design ground motions consistent with the 2015 NBCC. These spectrum-compatible ground motions correspond to the Class C ground condition, excluding the effects of overburden soils; therefore, a compliant base boundary condition with an elastic zone was used to model the Class C ground conditions and to apply the input motions at the base. An average shear wave velocity of 450 m/s was used for the Class C ground conditions. For analysis purposes, the water level was considered at CGVD-28GVRD elevation 101 m.

The coupled stress-flow response was simulated by taking into account the dissipation and/or redistribution of pore pressures that occur with time and the strains caused by such changes in pore pressure. Two mechanical effects are considered in this case: (i) changes in pore pressure induced by volume changes, and (ii) changes in effective stresses caused by pore pressure changes. The effects induced by the pore pressure dissipation/ redistribution process, seepage forces, and the loading conditions were accounted for at every step of the calculation, capturing the coupled stress-flow response. The groundwater flow formulation in FLAC is based on Darcy's Law for an anisotropic porous medium and the Continuity Equations. The characteristic hydraulic conductivity values were established for each soil unit based on comparing grain size data and correlating to the results of the hydrogeological tests. The input parameters used in the analyses are summarized in Table 5-2.

Table 5-2: Summary of Engineering Parameters used in FLAC Analyses

Soil Unit	SPT$(N_1)_{60cs}$ (blows/0.3 m)			Unit Wt. (kN/m3)	ϕ' (deg.)	Su (kPa)	K_h (m/s)	K_h/K_v
	Initial Phase	Subsequent Phase	Final Phase					
Unit 1	20	20	20	19	35	-	1.3E-4	10
Unit 2	-	-	-	17	--	40	3.0E-7	5
Unit 3	Fig 5-1	Figs. 5-2	Fig 5-4	19	35	-	1.3E-4	10
						-	1.3E-4	10
Unit 4	-	-	-	19	--	0.22-0.26σ'v	4.0E-7	5
Unit 7	-	-	-			-	4.0E-7	5

5.4 Model Calibration

The parameters for the UBCSAND model were obtained by carrying out a series of single element simple shear test simulations using the computer code FLAC and the user-defined UBCSAND model. The parameters of the UBCSAND constitutive model were adjusted in order to predict triggering of liquefaction in approximately 15 cycles of loading, at a cyclic stress ratio equal to the characteristic cyclic resistance ratio (CRR) value based on the CRR vs. the $(N_1)_{60cs}$ equation recommended by Idriss & Boulanger (2014). In addition, the overburden correction factors recommended by Idriss & Boulanger (2014) were used to calibrate the element behaviour for stress levels other than 100 kPa. When following this methodology, a correction factor larger than unity was applied for confining stress levels less than 100 kPa.

It is noted that the model calibration was undertaken during both the initial and final phases of the analyses. The model calibration in the initial phase was focused on the design $(N_1)_{60cs}$ values less than 18 blows/0.3 m. However, further calibration was carried out during the final phase to extend the calibration of the design, $(N_1)_{60cs}$ values up to 25 blows/0.3 m, because the 50th percentile design profiles considered in the final phase of the analyses ranged up to 24 blows/0.3 m.

Results from the single element models completed in the initial and the final phase of the analyses are plotted on Figures 5-9 and 5-10, respectively in terms of CRR vs. Number of Cycles required for liquefaction (Ncyc). A good agreement was achieved between the Idriss & Boulanger (2014) CRR vs. $(N_1)_{60cs}$ chart data and model predictions, as shown on Figures 5-9 and 5-10.

The effects of confining stress have also been recalibrated with the triggering calibration for the $(N_1)_{60cs}$ values higher than 18 blows/0.3 m, as noted above. A comparison of the UBCSAND model vs. the correlation developed by Idriss and Boulanger (2008), for the effects of confining stresses, was completed during the initial and the final phases of the analyses, as shown on Figures 5-9 and 5-10, respectively. A good agreement can be noted between the correlation and the model predictions.

A comparison of the stress path and excess pore pressure response of an element test with results from a laboratory cyclic simple shear test (Byrne et al. 2004) is also shown on Figure 5-9. The results indicate that the UBCSAND model can adequately capture the load deformation response observed in the laboratory, both prior to and following triggering of liquefaction.

Typical modulus reduction and damping curves available from literature for Unit 2, and those computed from FLAC using the UBCHYST model, are shown on Figure 5-11. The modulus reduction and damping curves from literature and those computed using the built-in model utilizing ELA, for Units 4 and 7, are shown on Figure 5-12.

5.4.1 Model Calibration – Pre-liquefaction

Following completion of the single element calibration, 1D models were generated in the computer code FLAC, without considering the effects of liquefaction, to assess the response of the user defined models as noted above. The results of the 1D ground response analyses carried out using the computer program SHAKE2000 were used for comparison purposes and the profiles of maximum cyclic shear stress ratio vs. depth computed from the two programs at a typical location are shown on Figure 5-13, where a good agreement can be noted indicating that the pre-liquefaction response of the site soils can also be adequately captured with the user defined models.

5.5 Results of the Analyses – Initial Phase

The ground response analyses associated with the crustal and inslab motions predicted soil liquefaction, defined herein as zones developing an excess pore pressure ratio (i.e., a ratio between the excess pore water pressure and the overburden stress) of 85% or more at the end of shaking, through the entire sand deposit along the outfall alignment corridor for all the cases considered in the analyses. The extent of liquefaction is generally limited to the upper 10 m of the sand deposit under the subduction event. The extent of liquefaction predicted for Cases 1 through 4 at the shaft locations and along the tunnel are shown in Figures 5-14 through 5-19 for all eight input ground motion time-histories.

The computed permanent lateral ground displacement and vertical settlement profiles predicted at the shaft locations for Cases 1 through 4 are presented in Figures 5-20 through 5-23, while the displacement and settlement profiles along the tunnel for the same cases are shown on Figures 5-24 through 5-27.

It is noted that the lateral displacement and settlement profiles for the effluent shaft were established from the second FLAC model extending through the edge of the Mungo's hole, and the analyses were carried out only for Case 1. The average displacement profiles associated with Case 1 at the future shaft location and effluent shaft are shown on Figure 5-28. As shown on Figure 5-28, there is no significant difference in the displacement and settlement profiles; therefore, the displacement and settlement profiles established for the future shaft location were also used for the effluent shaft in the follow-up analyses.

The results are summarized in Table 5-3 and indicate the following:

- The maximum permanent lateral displacement resulting from the crustal and inslab ground motions is computed to be in the order of 0.3 m at the future and effluent shafts, while the lateral displacement is computed to be in the order of 0.4 m at the outfall shaft. The lateral displacement due to the subduction event is computed to be in the order of 0.05 m at the on-land shafts.
- The maximum permanent vertical settlements at the on-land shafts are computed to be in the order of 0.8 m due to the crustal and inslab motions, and in the order of 0.4 m due to the subduction event.

- The maximum permanent lateral displacement resulting from the crustal and inslab ground motions is computed to be in the order of 1.8 m at the riser shaft, and in the order of 0.4 m due to the subduction event.
- The maximum permanent vertical settlement at the riser shaft is computed to be in the order of 0.6 m due to the crustal and inslab motions, and in the order of 0.2 m due to the subduction event.
- The maximum permanent lateral displacements resulting from the crustal and inslab ground motions vary up to 1.2 m along the tunnel alignment and generally insignificant further away in-land from the foreshore area, while those from the subduction event along the tunnel alignment are computed to be insignificant.
- The permanent vertical settlements along the tunnel vary from 0.05 m to 0.2 m, with the maximum settlement occurring at the river bank and within the river channel.

Table 5-3: Permanent Lateral and Vertical Displacement

Type of Ground Motion	Outfall Shaft		Future and Effluent Shafts		Riser Shaft		Tunnel Alignment	
	Maximum Displacement (m)							
	Lateral	Vertical	Lateral	Vertical	Lateral	Vertical	Lateral*	Vertical**
Crustal and Inslab	0.40	0.80	0.30	0.80	1.80	0.60	Up to 1.20	0.05 – 0.20
Subduction	0.05	0.40	0.05	0.40	0.40	0.20	N/A***	N/A***

*The lateral displacement of the proposed tunnel varies along the alignment and are generally insignificant further inland

**The settlement varies along the alignment with the maximum occurring at the river bank and within the river channel

***The lateral displacement and vertical settlements are predicted to be insignificant under the 2010 NBCC subduction ground motions for the tunnel alignment

5.5.1 Input to Soil-Structure Interaction Analyses

The following recommendations were provided on how to most effectively utilize the results of the 2D ground response analyses as input to the soil-structure interaction (SSI) analyses that were undertaken by the structural design engineers.

- **On-land Shafts**: Utilize the mean permanent lateral displacement and vertical settlement profiles computed from Case 1 associated with crustal and inslab ground motions. In addition, use the displacement profiles associated with Case 1 – Loma Prieta NS and Case 2 – Chi NS together with the mean settlement profiles to assess the sensitivity of the forces with respect to the functional requirements under the design ground motions. The computed mean, as well as the displacement profiles associated with Case 1 – Loma Prieta NS and Case 2 – Chi NS, are shown in Figures 5-29 and 5-30.
- **Riser Shaft**: Utilize the mean permanent lateral displacement and vertical settlement profiles computed from Case 1. In addition, use the displacement profiles associated with Case 1 – Landers NS and Case 1 – Landers EW together with the mean settlement profiles to assess the sensitivity of the forces with respect to the functional requirements under the design earthquake. The computed mean as well as the displacement profiles associated with Case 1 – Landers NS and EW are shown in Figure 5-31.

- **Tunnel**: Utilize the mean permanent lateral displacement and vertical settlement profiles computed from Case 1. In addition, use the displacement profiles associated with Case 1 – Landers EW and Case 1 – Landers NS together with the mean settlement profiles to assess the sensitivity of the forces with respect to the functional requirements under the design earthquake. The computed mean as well as the displacement profiles associated with Case 1 – Landers NS and EW are shown in Figure 5-32.

It should be noted that the lateral displacement profiles induced by different earthquake input ground motions varied along the shafts and at the tunnel alignment. Consequently, if the soil stiffness parameters established at the end of shaking together with the displacement and settlement profiles are to be used as input in the SSI analyses, the displacement and settlement profiles and the stiffness parameters corresponding to all eight records associated with Case 1 should be considered. However, the stiffness and strength parameters as requested by the structural design engineers were provided for all four cases, in the event that sensitivity analyses are considered necessary to assess the performance of the structures for various cases with respect to the functional requirements under the design ground motions.

Direction of Movements and Resultant Movements

Evidence from past earthquakes for sites underlain by liquefiable soils indicates that liquefaction-induced ground movements primarily occur in a direction perpendicular to the river banks, or other topographic features. These studies focus on visible, shallow or surficial movements. We are unaware of studies on displacement patterns in soils where liquefaction has occurred at depths in the order of 20 m to 30 m below ground surface.

Considering that the design earthquake scenario for the proposed outfall is a rare event and that the cross section analyzed is approximately perpendicular to the river bank, it was recommended that the permanent displacements transverse to the tunnel alignment be taken as 50% of the computed mean permanent displacements in the longitudinal direction. The transverse movements could occur in either the upstream or the downstream direction. The same recommendation was provided for the riser shaft. The lateral displacements in the transverse direction for the on-land shafts were recommended to be in the same order of the displacement in the longitudinal direction.

5.5.2 Results – 2015 NBCC

The analyses for the seismic hazard parameters associated with the 2015 NBCC were carried out to assess the sensitivity to the updated seismic hazard parameters, and they were carried out only considering the SPT$(N_1)_{60}$ profile consistent with Case 1 for comparison purposes. A comparison was made between the excess pore pressure ratios (Ru) and the lateral displacements predicted previously at the shaft locations using the 2010 NBCC parameters and those predicted for the motions using the 2015 NBCC parameters. The results of the analyses are as noted below.

- Outfall Shaft

  The sensitivity analysis indicates that the entire sand deposit at the on-land shaft locations may liquefy under the 2010 NBCC crustal and inslab design ground motions, while liquefaction is limited to the upper 20 m under the 2015 NBCC crustal and inslab motions (see Figure 5-33). The upper 12 m is expected to liquefy under the 2010 NBCC interface ground motions, while the upper 25 m of the sand deposit is expected to liquefy under the 2015 NBCC interface ground motions (see Figure 5-33).

The average lateral displacement profiles at the outfall shaft location under the 2010 NBCC crustal and inslab design ground motions are generally higher than those predicted under the 2015 NBCC crustal and inslab design ground motions (see Figure 5-34). The lateral displacement profiles are generally higher under the 2015 NBCC interface ground motions compared to the 2010 NBCC motions (see Figure 5-34). In addition, the lateral displacements predicted under the 2015 NBCC interface ground motions are generally similar or higher (i.e., up to 20%) compared to those predicted under the 2010 NBCC crustal and inslab motions.

The computed lateral displacements, considering the 16 earthquake records comprising both crustal and inslab, and the interface sources, associated with the 2015 NBCC are also shown on Figure 5-34.

- Future Shaft (and Effluent Shaft)

  The entire sand deposit may liquefy under the 2010 NBCC crustal and inslab design ground motions, while the liquefaction is limited to the upper 15 m under the 2015 NBCC crustal and inslab motions (see Figure 5-35). The upper 10 m is expected to liquefy under the 2010 NBCC interface ground motions, while the entire sand deposit may liquefy when subjected to the 2015 NBCC interface ground motions (see Figure 5-35).

  Similar to the outfall shaft location, the average lateral displacement profiles at the future shaft location under the 2010 NBCC crustal and inslab design ground motions are generally higher than those predicted under the 2015 NBCC crustal and inslab design ground motions (see Figure 5-36). Also, similarly, the lateral displacement profiles are generally higher under the 2015 NBCC interface ground motions compared to the 2010 NBCC interface motions (see Figure 5-36). Furthermore, the lateral displacements predicted under the 2015 NBCC interface ground motions are generally higher (i.e., up to 50%) compared to those predicted under the 2010 NBCC crustal and inslab motions.

  The computed lateral displacements, considering the 16 earthquake records comprising crustal and inslab, and the interface sources, associated with the 2015 NBCC are also shown on Figure 5-36.

- Riser Shaft

  The entire sand deposit may liquefy under both the 2010 NBCC and 2015 NBCC crustal and inslab design ground motions. The liquefaction is limited to the upper 15 m under the 2010 NBCC interface motions, while it extends through the entire deposit under the 2015 interface motions (See Figure 5-37).

  The average lateral displacement profiles at the riser shaft location under the 2010 NBCC crustal and inslab design ground motions are generally higher than those predicted under the 2015 NBCC crustal and inslab design ground motions (see Figure 5-38). The lateral displacement profiles are generally higher under the 2015 NBCC interface ground motions compared to the 2010 NBCC interface ground motions (see Figure 5-38). In addition, the displacements predicted under the 2015 NBCC interface motions are generally similar to those predicted under the 2010 NBCC crustal and inslab motions.

  The computed lateral displacements, considering the 16 earthquake records comprising crustal and inslab, and the interface sources, associated with the 2015 NBCC are also shown on Figure 5-38.

Discussion

The lateral displacements associated with the design input for the soil-structure interaction analyses are estimated to be similar for the riser shaft under both the 2010 NBCC and 2015 NBCC motions, and they are estimated to be higher (up to 50%) for the 2015 NBCC compared to the 2010 NBCC for the on-land shafts. It is also noted that the governing earthquake source associated with the design displacement profiles is found to be different under the 2010 NBCC and 2015 NBCC motions, in that the crustal and inslab earthquake motions govern the extent of liquefaction and lateral displacements for the 2010 NBCC, while they were governed by the interface earthquake motions under the 2015 NBCC

5.6 Results of the Analyses – Subsequent Phase

This section presents the results of the 2D ground response analyses completed for the design $(N_1)_{60cs}$ profile corresponding to the 50th percentile values. The analyses were carried out to assess the sensitivity of the liquefaction potential of the site soils, and the resulting permanent ground deformations, to variations in the design SPT $(N_1)_{60cs}$ values under the 2010 NBCC design ground motions.

The results are presented in terms of a comparison between the excess pore pressure ratios (Ru) and lateral displacements predicted previously at the shaft locations using the 33rd percentile values and those predicted for the 50th percentile values. The results indicate the following:

- The results of the previous analyses considering the design $(N_1)_{60cs}$ profile corresponding to the 33rd percentile values indicated that the entire sand deposit at the shaft locations could potentially liquefy under the 2010 NBCC 2,475-year design ground motions. The extent of liquefaction for the design $(N_1)_{60cs}$ profile corresponding to the 50th percentile values is also found to be similar to that for the 33rd percentile values, as shown in Figures 5-39 through 5-41 for the on-land and riser shafts.

  The computed excess pore pressure ratios considering the six earthquake records comprising crustal and in-slab sources associated with the 2010 NBCC are also shown on Figures 5-39 through 5-41 for the design $(N_1)_{60cs}$ profiles corresponding to both the 33rd and 50th percentile values.

- The average lateral displacement profiles considering the design $(N_1)_{60cs}$ profile corresponding to the 33rd percentile values are slightly higher than those for the 50th percentile values, as sown on Figures 5-42 and 5-43 at the outfall and future shaft locations, respectively. However, the average lateral displacements at the riser shaft location are generally about 30 percent lower for the design profile corresponding to the 50th percentile values compared to those for the 33rd percentile values, as shown on Figure 5-44.

  The computed lateral displacements, considering the six earthquake records are also shown on Figures 5-42 through 5-44 for the design $(N_1)_{60cs}$ profiles corresponding to both the 33rd and 50th percentile values.

Discussion

The previous 2D ground deformation analyses were carried out for the design $(N_1)_{60cs}$ profiles corresponding to the 33rd percentile values as per the Task Force guidelines. The use of the 33rd percentile values has been the local practice for assessment of the potential liquefaction of granular soils and the resulting lateral ground displacements. Considering the recent studies based on the stochastic $(N_1)_{60cs}$ distributions, further analyses were carried out to assess the impact on the predicted lateral spreading considering the 50th percentile $(N_1)_{60cs}$ values.

The results of the analyses indicate that the differences in the predicted lateral displacements between the two design $(N_1)_{60}$ profiles at the on-land shaft locations are insignificant. However, the predicted lateral displacements were generally 30 percent lower at the riser shaft location for the 50th percentile $(N_1)_{60cs}$ values compared to that of the 33rd percentile $(N_1)_{60cs}$ values. It is noted that the predicted lateral displacements at the on-land shaft locations are generally smaller; hence, an appreciable difference as seen at the riser shaft location could not be realized with the increase in the penetration values at the on-land shaft location. It is also noted that the relatively lower predicted lateral displacements at the riser shaft for the 50th percentile $(N_1)_{60cs}$ values may not actually result in a comparable reduction in the predicted lateral deflection of the shaft; this is because the higher $(N_1)_{60cs}$ values result in increased strength of the sand, which would likely increase the lateral load imposed on the shaft.

5.7 Results of the Analyses – Final Phase

This section presents the results of the 2D ground response analyses completed for the design $(N_1)_{60cs}$ profile analyses considering the 50th percentile $(N_1)_{60cs}$ values corresponding to Scenario 1 with the offshore and on-land data separately. The analyses were carried out for the design ground motions consistent with both the 2010 NBCC and 2015 NBCC. The results associated with the 2010 ground motions are presented below and those associated with the 2015 ground motions are presented in Section 5.7.1.

The predicted zones of liquefaction computed in the FLAC model for selected earthquake records under the design ground motions consistent with the 2010 NBCC and 2015 NBCC are shown on Figures 5-45 through 5-47.

The excess pore pressure ratios (Ru) and lateral displacements predicted at the shaft locations are also presented in terms of a comparison between the 33rd and 50th percentile design profiles considered in the initial and subsequent phases, and those predicted for the 50th percentile values considered in the final phase, as shown on Figures 5-48 through 5-53 for the six input ground motion time-histories representing the crustal and inslab seismic sources. No analyses were carried out for the interface event as it was not found to be governing the design based on the results of the initial phase analyses.

The results indicate that the extent of liquefaction is generally consistent in all cases from the initial to the final phases (see Figures 5-48 through 5-50). However, the lateral displacement at the riser shaft location reduced by about 20 percent with the combined data set corresponding to the 50th percentile $(N_1)_{60cs}$ values considered in the subsequent phase (See Figure 5-53). The displacements were further reduced by another 10 percent from the initial prediction with the revised 50 percentile $(N_1)_{60cs}$ design profiles considering the on-land and offshore data sets separately (i.e., Scenario 1). A reduction in the order of 85 percent was also observed in the displacement from the initial prediction at the outfall shaft location with the revised design profiles. It is also noted that the predicted lateral displacements at the future shaft location are generally small; hence, an appreciable difference as seen at the riser shaft location could not be realized with the revised $(N_1)_{60cs}$ design profiles.

5.7.1 Results – 2015 NBCC

The results of the analyses carried out for the seismic hazard parameters consistent with the 2015 NBCC are presented in this section. The analyses were carried out for a total of 16 earthquake records including 11 crustal and inslab and five interface (subduction) ground motions. The results are presented in terms of a comparison between the excess pore pressure ratios (Ru) and lateral displacements predicted at the shaft locations using the ground motions consistent with the 2010 NBCC and 2015 NBCC based on the design profiles considered in the final phase.

- On-land Shafts

  Except for the lower 10 to 15 m, the sand deposit at the on-land shaft locations may liquefy under the 2010 NBCC crustal and inslab design ground motions and the 2015 NBCC interface ground motions (see Figures 5-54 and 5-55). However, the liquefaction is limited to the upper 15 m under the 2015 NBCC crustal and inslab motions . The computed excess pore pressure ratios considering the six earthquake records associated with the 2010 NBCC and the 16 earthquake records associated with the 2015 NBCC are also shown on Figures 5-54 and 5-55.

  The average lateral displacements at the on-land shaft locations are generally small, with a maximum of about 0.25 m under the design ground motions consistent with both the 2010 NBCC and 2015 NBCC. In addition, the displacements under the 2015 interface motions are higher compared to those corresponding to the crustal and inslab motions consistent with both the 2010 NBCC and 2015 NBCC (see Figures 5-56 and 5-57). The computed lateral displacements considering the six earthquake records associated with the 2010 NBCC and the 16 earthquake records associated with the 2015 NBCC are shown on Figures 5-56 and 5-57.

- Riser Shaft

  The entire sand deposit may liquefy under the 2010 NBCC crustal and inslab design ground motions, as shown on Figure 5-58. The extent of liquefaction is expected to be similar under the interface ground motions consistent with the 2015 NBCC. However, the liquefaction is limited to the upper 10 m under the 2015 crustal and inslab motions.

  The average lateral displacement profiles at the riser shaft location under the crustal and inslab design ground motions consistent with the 2010 NBCC are generally higher than those predicted under the 2015 NBCC crustal and inslab design ground motions (see Figure 5-59). In addition, the displacements predicted under the 2015 interface motions are generally 30% higher than those predicted under the 2010 crustal and inslab motions.

  The computed lateral displacements, considering the six earthquake records associated with the 2010 NBCC and the six earthquake records associated with the 2015 NBCC are also shown on Figure 5-59.

Discussion

The lateral displacements associated with the 2015 NBCC ground motions are generally higher compared to those estimated under the 2010 NBCC motions, and this is considered to be significant at the riser shaft given the magnitude of the predicted displacement. It is also noted, as found in the previous analyses phases, that the crustal and inslab earthquake motions govern the extent of liquefaction and lateral displacements under the 2010 NBCC motions, while they were governed by the interface earthquake motions under the 2015 NBCC motions.

5.7.2 Input to Soil-Structure Interaction Analyses

The geotechnical input parameters required for the soil-structure interaction (SSI) analyses were only provided for the 2010 NBCC design ground motions, in accordance with the project scope. The analyses associated with the 2015 NBCC were carried out for comparison purposes only.

The following summarizes the permanent ground deformations, stiffness parameters, and the load-resistance curves provided as input to the SSI analyses under the design ground motions consistent with the 2010 NBCC.

Displacement Profiles, Strength and Stiffness Parameters

It was recommended that the mean permanent lateral displacement and vertical settlement profiles be utilized in the soil structure interaction (SSI) analyses of the shafts and tunnels.

The displacements, and post liquefaction stiffness and strength parameters associated with the earthquake (EQ) record "Loma Prieta NS" were recommended as input to SSI analyses for the on-land shafts, and those associated with the EQ record "Chi Chi NS" were recommended for the riser shaft. The EQ records Loma Prieta NS and Chi Chi NS represent the average response of all six EQ records considered in the analyses in terms of the horizontal displacement profiles, as shown on Figure 5-60 for the on-land and riser shafts.

The horizontal displacement profile along the outfall tunnel associated with the EQ record "Chi Chi NS" that generally represents an average response in terms of the horizontal displacements at the tunnel invert as shown on Figure 5-61, was recommended for the SSI analyses of the outfall tunnel. The horizontal displacements in both longitudinal and transverse directions along the effluent tunnel are not considered to be significant.

The post-seismic consolidation settlements estimated at the on-land shafts and riser shaft are shown on Figure 5-60, and the settlements along the tunnel invert are shown on Figure 5-61. It is noted that the post-seismic consolidation settlements along the outfall tunnel and at the on-land and riser shafts were estimated based on Approach 1, as noted in Section 5.1. Approach 2 was also utilized to estimate settlement along the outfall tunnel as shown on Figure 5-61. It is noted that the vertical displacements resulting from the lateral movement of the ground obtained from the FLAC analyses were added to the post-seismic consolidation settlements (referred to as total settlement), especially for the riser shaft and outfall tunnel.

It was recommended that a maximum differential settlement, established by considering two cases (including and excluding the vertical displacements due to lateral deformation), be used for the purpose of the outfall tunnel design, especially in the vicinity of the riser shaft. It was also recommended that the overall total settlement profiles estimated along the tunnel by both approaches be considered for the purpose of the outfall tunnel design.

The post-seismic consolidation settlements along the effluent tunnel based on Approach 1 are shown Figure 5-62.

The horizontal displacements provided herein, along the tunnel and at the shafts, were estimated considering a section perpendicular to the river bank; however, permanent ground displacements transverse to the tunnel alignment should also be considered for the purpose of the shaft and tunnel designs. A guideline on this is previously provided in Section 5.5.1.

Load-Resistance Curves

Non-linear py curves were developed as input to the seismic evaluation of the on-land shafts, as well as for the steel pipe piles to be installed into the marine deposit to support the riser shaft. A set of py curves at the on-land shafts and the piles below the riser shaft were developed for non-liquefied and liquefied ground conditions. The py curves for non-liquefied ground conditions were developed following American Petroleum Institute (API) guidelines. The py curves for liquefied ground conditions were developed using the approach described by Boulanger et al, (2003) using the p-multiplier concept.

Axial and toe responses of the on-land shafts and the piles supporting the riser shaft were represented by tz and Qz non-linear springs. The tz springs were developed following API guidelines for the non-liquefied ground conditions, and the p-multiplier computed based on Boulanger et al. (2003) was also applied to the tz springs to account for the liquefied ground conditions.

The Qz springs were developed based on the elastic settlement at set potential loads to provide the expected reaction of the soil at the base of the on-land shafts under liquefied conditions. The elastic settlements were estimated utilizing the post-seismic stiffness values obtained from the FLAC analyses. The springs were capped at the bearing capacities estimated under the liquefied ground conditions. The bearing capacity was determined based on the estimated residual strength of the soils underlying the shafts. No Qz spring was provided for the piles supporting the riser shaft as the piles are considered to be friction piles. The soil parameters used to develop the springs are summarized in Table 5-4.

Table 5-4: Summary of Soil Parameters – Non-linear Springs

Soil Layer	Effective Unit Weight (kN/m^3)*	Friction Angle (degrees)	Cohesion (kPa)	Subgrade Modulus (kN/m^3)
Unit 1	19	36	-	44,530
Unit 2	7.2	-	40	-
Unit 3	9.2	34-35	-	20,387 – 23,260
Unit 4	9.2	-	0.22-0.26σ'v	-

*The effective unit weight was estimated based on a water level at approximately 3 m below ground surface

Typical py, tz and Qz springs provided as input to the SSI analyses under liquefied ground conditions are shown on Figures 5-63 through 5-65. Figure 5-64 shows the typical py curves for the three relevant layers present at the outfall shaft. Figure 5-65 shows the typical tz curves for the same three relevant soil layers at the outfall shaft. Figure 5-66 shows a typical Qz curve for the base of the outfall shaft.

Post-Seismic Settlements – Marine Deposit (Unit 4)

The potential for the marine deposit (Unit 4) to undergo cyclic softening under the design ground motions consistent with the 2010 NBCC is considered to be low. However, excess pore water pressure in the order of 25 percent can be expected under the design ground motions consistent with the 2010 NBCC. It is also expected that the excess pore water pressure generated within the upper 20 m of Unit 4 is likely to dissipate within the design life of the structure resulting in post-seismic consolidation settlements occurring over a period of time.

A volumetric strain in the order of 0.3 percent is expected under the design ground motions in the on-land area and the resulting post-seismic consolidation settlement is estimated to be in the order of 60 mm. This settlement is expected to be uniform across Annacis Island considering the extent of Unit 4 underlying the site and therefore, it is not considered as part of the post-seismic consolidation settlements. The riser shaft is supported on piles and the post-seismic settlements in Unit 4 are not expected to impact the riser shaft. However, the settlements may have a local impact, especially at the connection between the riser shaft and the tunnel.

It is noted that Unit 4, especially in the offshore area is expected to undergo cyclic softening under the 2015 design ground motions. The excess pore water pressure in the order of 85 percent or higher can be expected under this scenario, resulting in about 3 percent volumetric strain following dissipation of the excess pore water pressure. The post-seismic consolidation settlement in the order of 0.5 m is expected and this may have a significant impact at the connection between the riser shaft and tunnel.

Diffuser System

The permanent lateral displacement and vertical settlement profiles, as well as soil parameters to establish springs to represent the pipe-soil interaction were provided as input to the SSI analyses of the diffuser manifold system.

The lateral displacement profile along the transverse direction of the diffuser manifold was established based on the results of the analyses completed in the subsequent phase together with the results from the initial phase as shown on Figure 5-66.

The lateral displacement at the riser shaft location was obtained from the subsequent analyses (i.e., Section A-A' with the 50th percentile design $(N_1)_{60cs}$ profiles). The variation in the displacement along the diffuser manifold was established utilizing the results of the previous analyses carried out in the initial phase, which included the same section used in the subsequent phase through the riser shaft (i.e., Section A-A') and another section through the edge of Mungo's hole near the end of the manifold system (i.e., Section C-C'). The ratio between the lateral displacements obtained from the sections was applied to the predicted displacements at the riser shaft in the subsequent phase to develop the lateral displacement profile along the diffuser manifold.

The ground displacement along the diffuser manifold (i.e., longitudinal direction) was recommended to be 5 percent of the displacement computed along the transverse direction.

The settlement profiles along the diffuser manifold were established as shown on Figure 5-67 based on the results of the 2D ground response analyses (i.e., Approach 1 as noted in Section 5.1), together with the CPT data obtained at the riser shaft location and in the vicinity of the eastern end of the manifold.

It was also recommended that an angle of friction of 33 degrees under the pre-liquefied conditions, and a residual shear strength of 0.1 times the effective overburden stress under the post-liquefied conditions be utilized for the non-compacted native backfill sand surrounding the diffuser pipe to establish the spring constants to represent the soil in the SSI analyses. Also, a submerged unit weight of 9.2 kN/m^3 was recommended.

It is noted that the horizontal ground displacement profiles were estimated for "free-field" conditions and the horizontal movement of the diffuser pipe would be dependent on the lateral fixity at the riser shaft. Similarly, the settlement along the diffuser pipe was also estimated based on "free-field" conditions, considering the extent of liquefaction below the invert of the diffuser pipe, and the settlement of the diffuser pipe would be dependent on the vertical fixity of the riser shaft and the ground improvement footprint around the riser shaft.

Riser Shaft

Limiting earth pressures were provided in lieu of the free-field displacement to model the lateral load imposed on the riser shaft due to lateral spreading as input to the preliminary design, and it is understood that the same approach was also utilized in the detailed design of the riser shaft.

It was recommended that the limiting pressure be considered as an all-around load for the riser shaft comprising a concrete block due to lateral spreading and that no passive resistance be considered. The guidelines developed by Japan Road Association (JRA 2002) based on pile performance in liquefied soils following the 1995 Kobe earthquake and centrifuge tests and analyses carried out by Boulanger for pile foundations in liquefied and lateral spreading ground (September 2003) were used to establish the kinematic loading (i.e., limiting pressure) on the riser shaft.

In accordance with the JRA guidelines, the liquefied soil may be represented as imposing a lateral pressure of 30 percent of the total vertical stress. However, there are case studies indicating that the pressure could be two to three times higher than that estimated based on the JRA guidelines. Hence, it was recommended that a sensitivity analysis be carried out considering a limiting earth pressure based on a limiting pressure of up to 60 percent of the total vertical stress.

A free body diagram illustrating the compliance springs to model the axial and lateral resistance of the piles supporting the riser shaft, a compliance spring to model the base shear resistance, and the limiting earth pressure that were provided for the SSI analyses is shown on Figure 5-68. This is applicable due to liquefaction and the resulting lateral spreading during seismic shaking.

In terms of base shear at the bottom of the concrete block, there may be two scenarios as noted below:

- The concrete block moves more than the liquefied sand underlying the block, resulting in the development of base shear resistance; however, the liquefaction and the upward water migration during liquefaction may form a water barrier at the concrete base resulting in very minimal base shear resistance. This is called a water film effect and the phenomenon has been proven in shake table tests.
- Alternatively, the liquefied sand moves more than the concrete block (i.e., flows around the block) resulting in drag force on the block as opposed to base shear resistance.

Considering the uncertainty with respect to the development of base shear resistance following liquefaction and resulting lateral spreading, it was recommended that the SSI analyses be carried out excluding the base shear resistance.

In terms of bearing resistance at the base of the concrete block, minimal bearing resistance is expected directly from the underlying liquefied sand. Since the concrete block is not keyed into the clay and the sand underlying the block may liquefy resulting in weak soil underlying the block, the stiffer piles would be engaged to provide the axial resistance under seismic loading and the resistance directly from the underlying soils is expected to be minimal. Hence, no normal compliance springs were provided to model the bearing resistance directly from the underlying soils at the concrete block base.

6.0 CLOSURE

We trust that the contents of this 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.

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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 cannot 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.

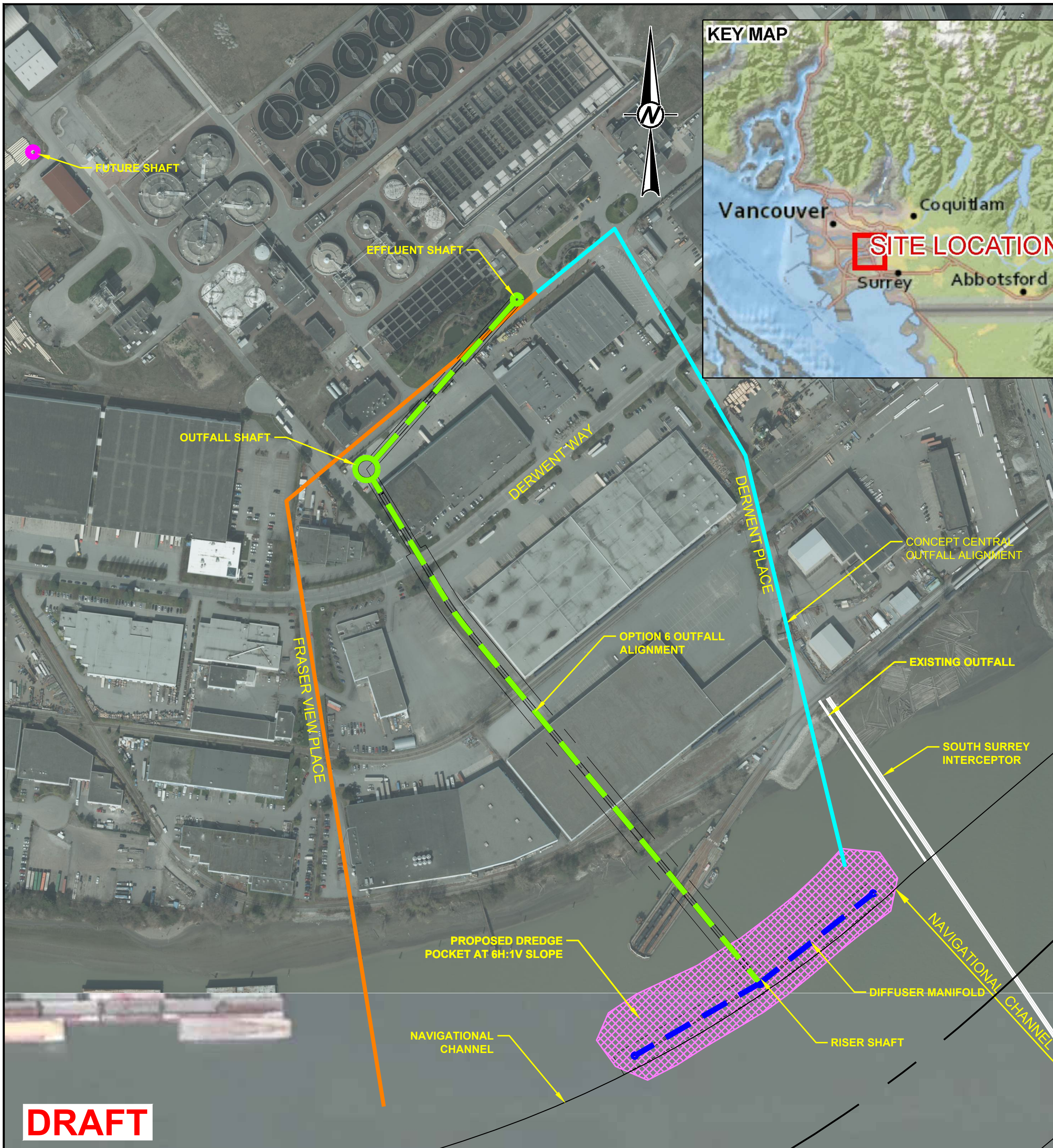
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater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



FRASER RIVER

0 100 200
1:5,000 METRES

NOTE(S)

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

REFERENCE(S)

1. NAVIGATIONAL CHANNEL AND ORTHOPHOTO PROVIDED BY BLACK & VEATCH FILE: ANNACISNAD27Z10GRND.TIF, ANNACIS-RIVER.TIF, C000A_XXXXX-1.DWG
2. 60% DETAIL DESIGN DRAWING OBTAINED FROM CDM SMITH.

CLIENT
CDM SMITH CANADA ULC

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

CONSULTANT
Golder Associates

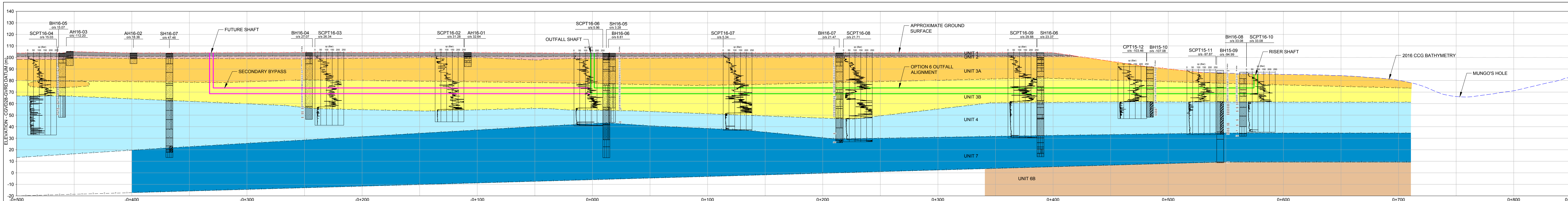
YYYY-MM-DD	2017-11-29
DESIGNED	Y. WITTWER
PREPARED	G. BETTOSCHI
REVIEWED	
APPROVED	

TITLE
PROJECT LOCATION PLAN

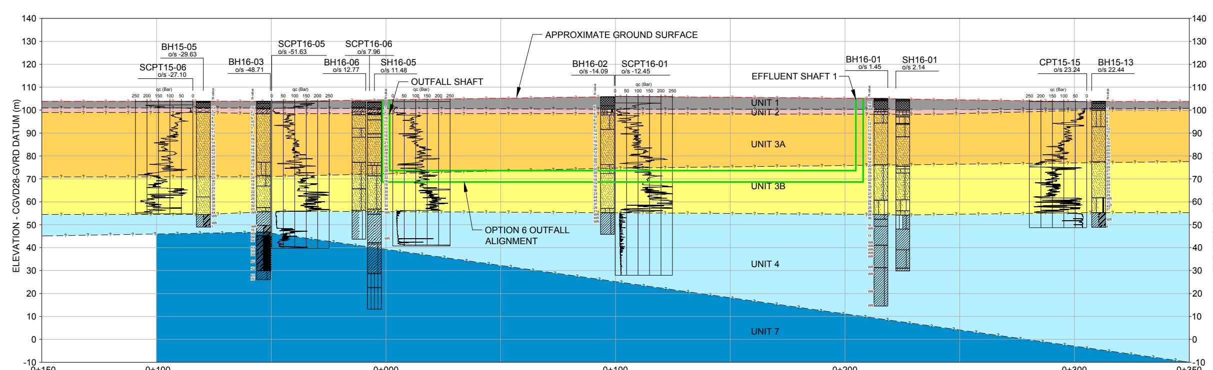
PROJECT NO.	PHASE/TASK	REV.	FIGURE
1525010	2100	A	2-1

Path: \\golder.gds\gal\burnaby\CAD-GIS\Client\CDM_Smith\Annacis_Island\99_PROJECTS\1525010_WWTP\02_PRODUCTION\2000\ | File Name: 1525010-2100-001.dwg

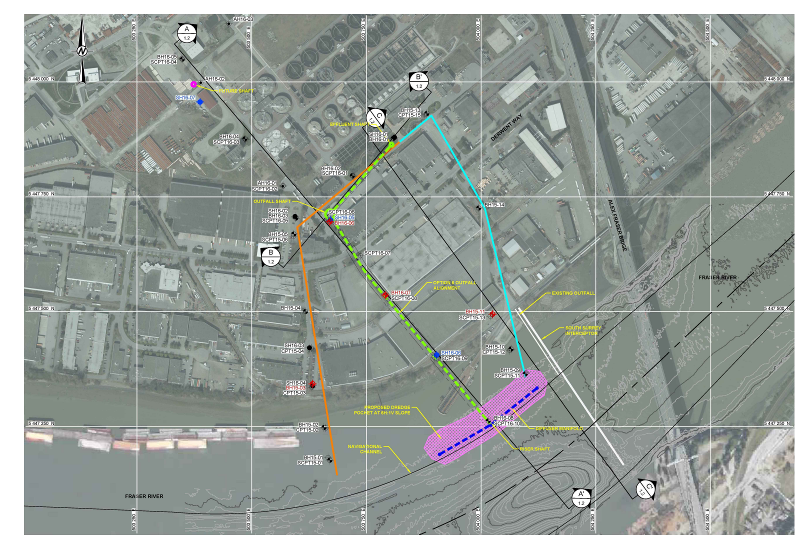
25 mm
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A



SCALE 1:1,000 m A-A' 1.1 STRATIGRAPHIC PROFILE



SCALE 1:1,000 m B-B' 1.1 STRATIGRAPHIC PROFILE



PLAN

APPROXIMATE SCALE: 1:5,000m

DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT TEST HOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN TEST HOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND MAY VARY FROM THAT SHOWN.

LEGEND

TEST HOLE RECORD SHOWING SOIL CONDITIONS
(FOR DETAILED GEOTECHNICAL INFORMATION REFER TO RECORD OF BOREHOLE)

BH15-03 o/s 21.14
- DENOTES TEST HOLE TYPE, INVESTIGATION YEAR AND ID NUMBER
- DENOTES PERPENDICULAR DISTANCE FROM TEST HOLE TO SECTION LINE IN METRES
- SPT - DENOTES STANDARD PENETRATION TEST BLOW COUNT (BLOWS/0.3m)

SONIC HOLE LOCATION
BOREHOLE LOCATION
AUGERHOLE LOCATION
(SEISMIC) CONE PENETRATION TEST LOCATION

SOIL LITHOLOGY

PEAT (PT) · ORGANIC SILT (OL) · ORGANIC SILT (OH) · SILTY CLAY (CL) · SILTY CLAY (CI) · CLAY (CH) · CLAYEY SILT (MH) · CLAYEY SILT OR SILT (ML) · CLAYEY SAND (SC) · SAND - WELL GRADED (SW) · SILTY SAND (SM) · SAND - POORLY GRADED (SP) · CLAYEY GRAVEL (GC) · SILTY GRAVEL (GM) · GRAVEL - POORLY GRADED (GP) · GRAVEL - WELL GRADED (GW)

qc (Bar) 0 50 100 150 200 250 — DENOTES CONE PENETRATION TEST TIP RESISTANCE (Bar)

STRATIGRAPHY LEGEND

- UNIT 1 - FILL
- UNIT 2 - OVERBANK DEPOSITS
- UNIT 3A - FRASER RIVER SAND (LOOSE TO COMPACT)
- UNIT 3B - FRASER RIVER SAND (COMPACT TO DENSE)
- UNIT 4 - MARINE DEPOSIT
- UNIT 6B - GLACIAL DEPOSITS
- UNIT 7 - GLACIOMARINE DEPOSIT

NOTES

1. COORDINATES ARE IN UTM NAD83 ZONE 10 AND CONVERTED TO GROUND LEVEL USING A COMBINED SCALE FACTOR OF 10397558.
2. ELEVATIONS ARE IN METRES AND REFER TO CVD28GVRD - 100 METRES PLUS GEODETIC DATUM.
3. STRATIGRAPHIC PROFILE ALONG THE OPTION 6 OUTFALL ALIGNMENT WAS DEVELOPED BASED ON ENGINEERING JUDGEMENT CONSIDERING THE AVAILABLE DATA ALONG THE WESTERN AND CENTRAL ALIGNMENTS
4. SECTION C-C' IS SIMILAR TO SECTION A-A' WITH A DIFFERENCE IN TOPOGRAPHY, SPECIFICALLY IN THE OFFSHORE AREA.

REFERENCES

1. NAVIGATIONAL CHANNEL, ORTHOPHOTO AND GROUND SURVEY DATA 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.

DRAFT

0 50 100
1:1,000 METRES

CLIENT	CDM SMITH CANADA ULC	PROJECT	ANNACIS ISLAND WWTP TRANSIENT MITIGATION AND OUTFALL DELTA, B.C.
CONSULTANT	Golder Associates	TITLE	**STRATIGRAPHIC PROFILES - OPTION 6 OUTFALL ALIGNMENT**
YYYY-MM-DD	2017-11-29		
DESIGNED	Y. WITTWER		
PREPARED	G. BETTOSCHI		
REVIEWED			
APPROVED			

PROJECT NO.	PHASE/TASK	REV.	FIGURE
1525010	2100	A	3-1

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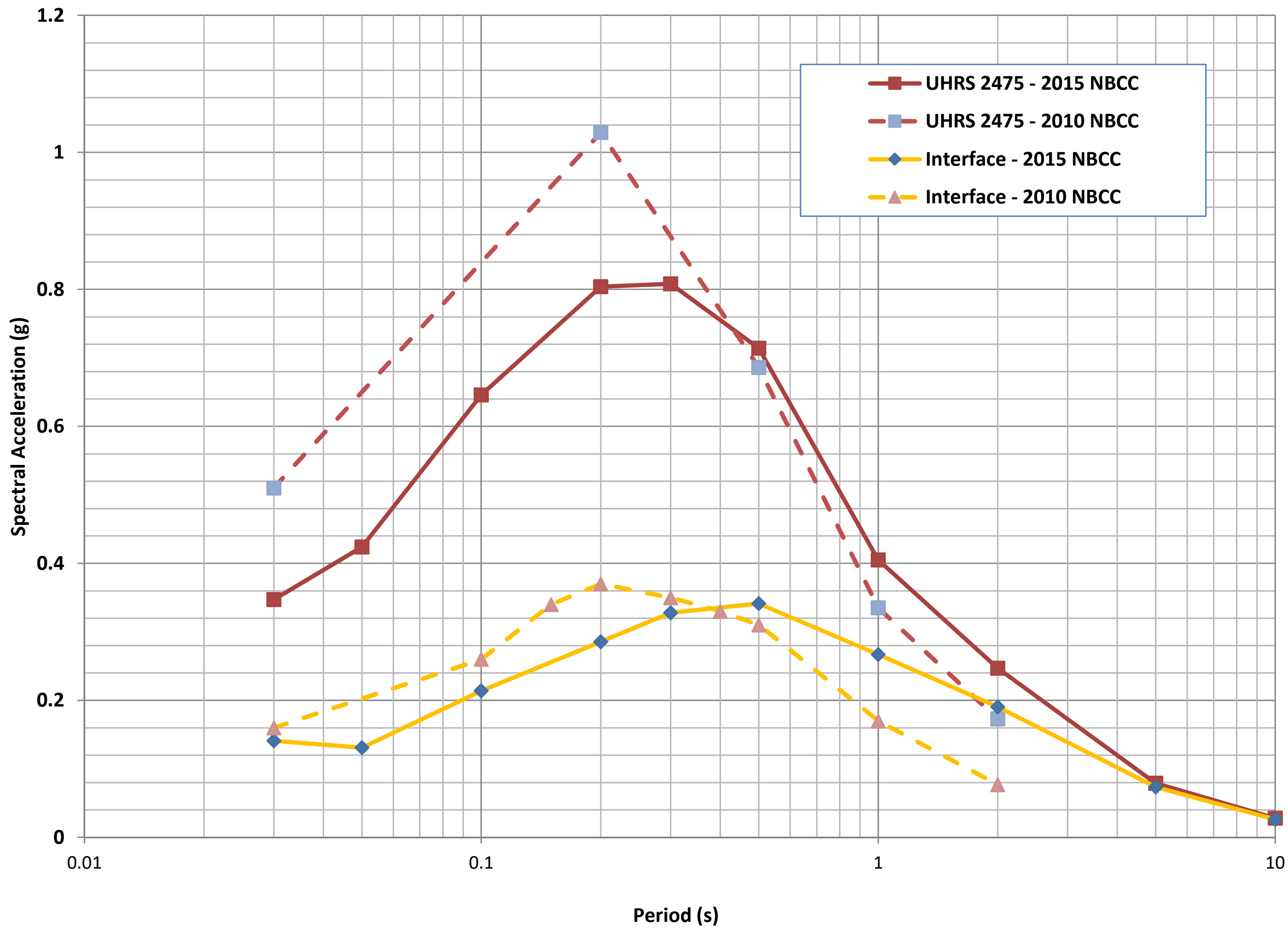

Note:
Acceleration response spectra for 5% damping

CLIENT	CDM SMITH CANADA ULC
CONSULTANT	Golder Associates
YYYY-MM-DD	03 Nov.. 2017
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	XXX
APPROVED	V.F
PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
TITLE	Acceleration Response Spectra 2010 NBCC and 2015 NBCC

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	4-1

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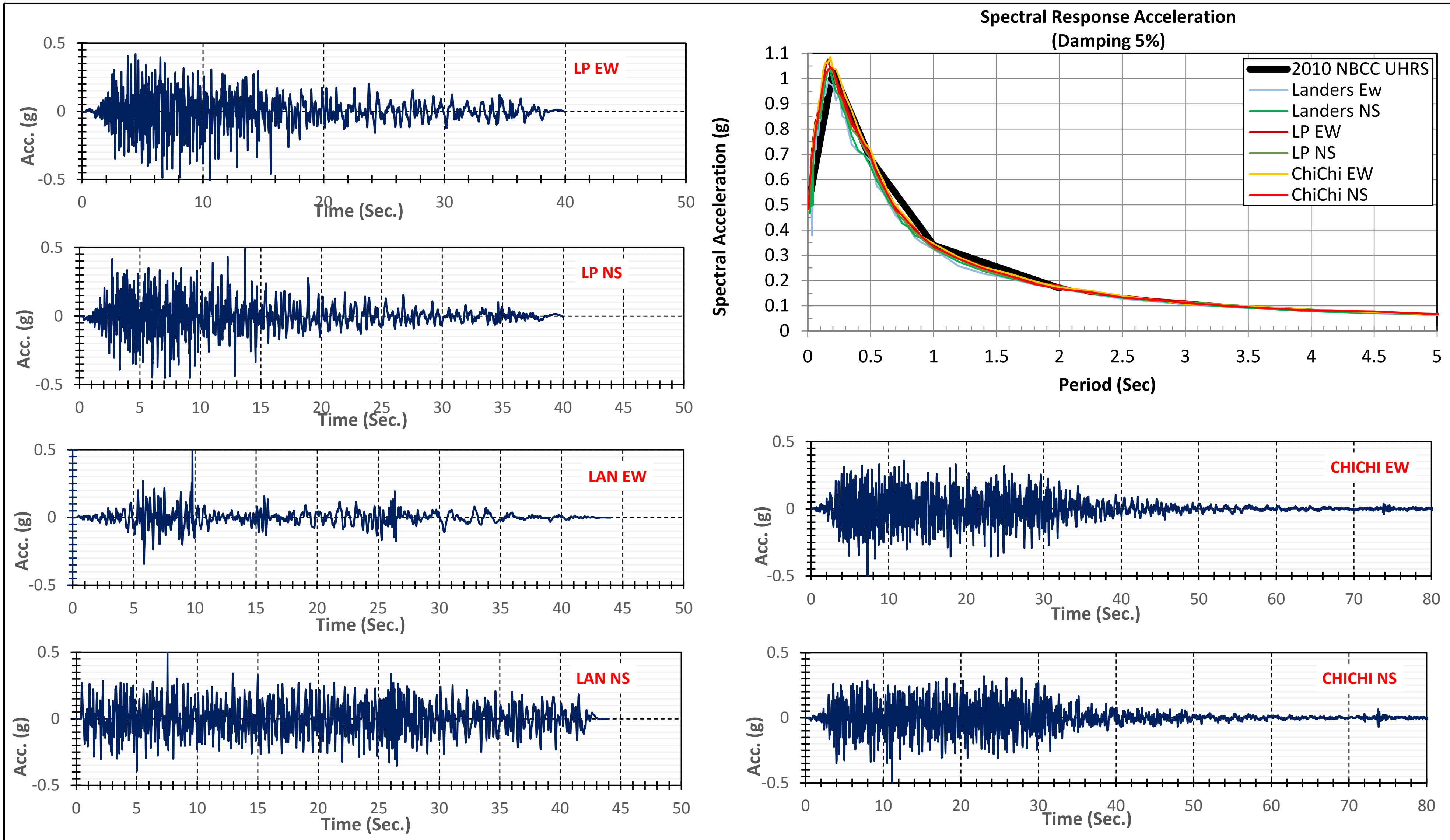
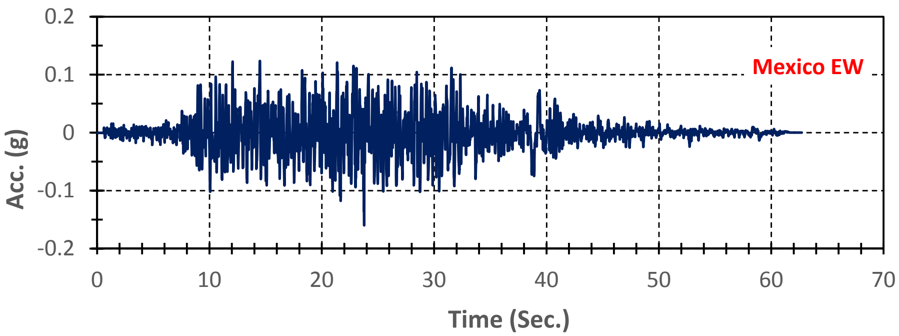
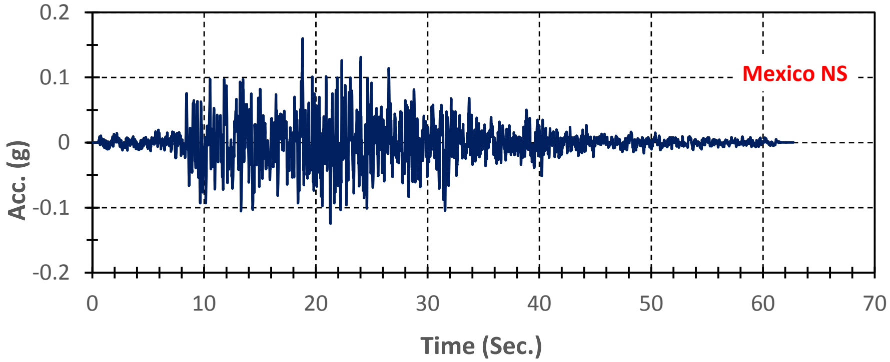
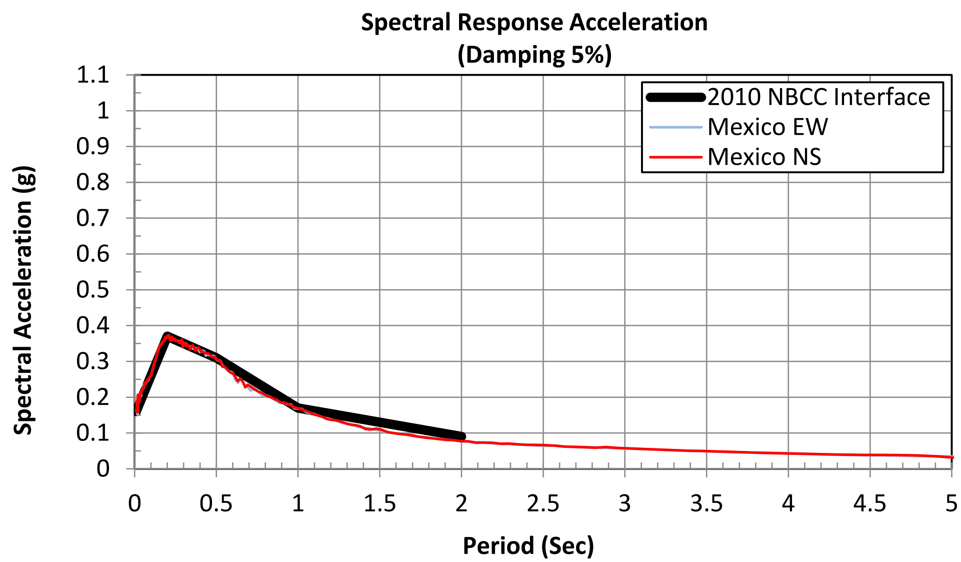


CLIENT **CDM SMITH CANADA ULC**		PROJECT **AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC**	
CONSULTANT Golder Associates	YYYY-MM-DD: 2017-11-30 PREPARED: M.S.K DESIGN: M.S.K REVIEW: Y.B APPROVED: V.F	TITLE **Acceleration Time Histories 2010 NBCC – Crustal/Inslab**	
		PROJECT No.: 1525010	Phase: 2200
		Rev: A	FIGURE: 4-2A



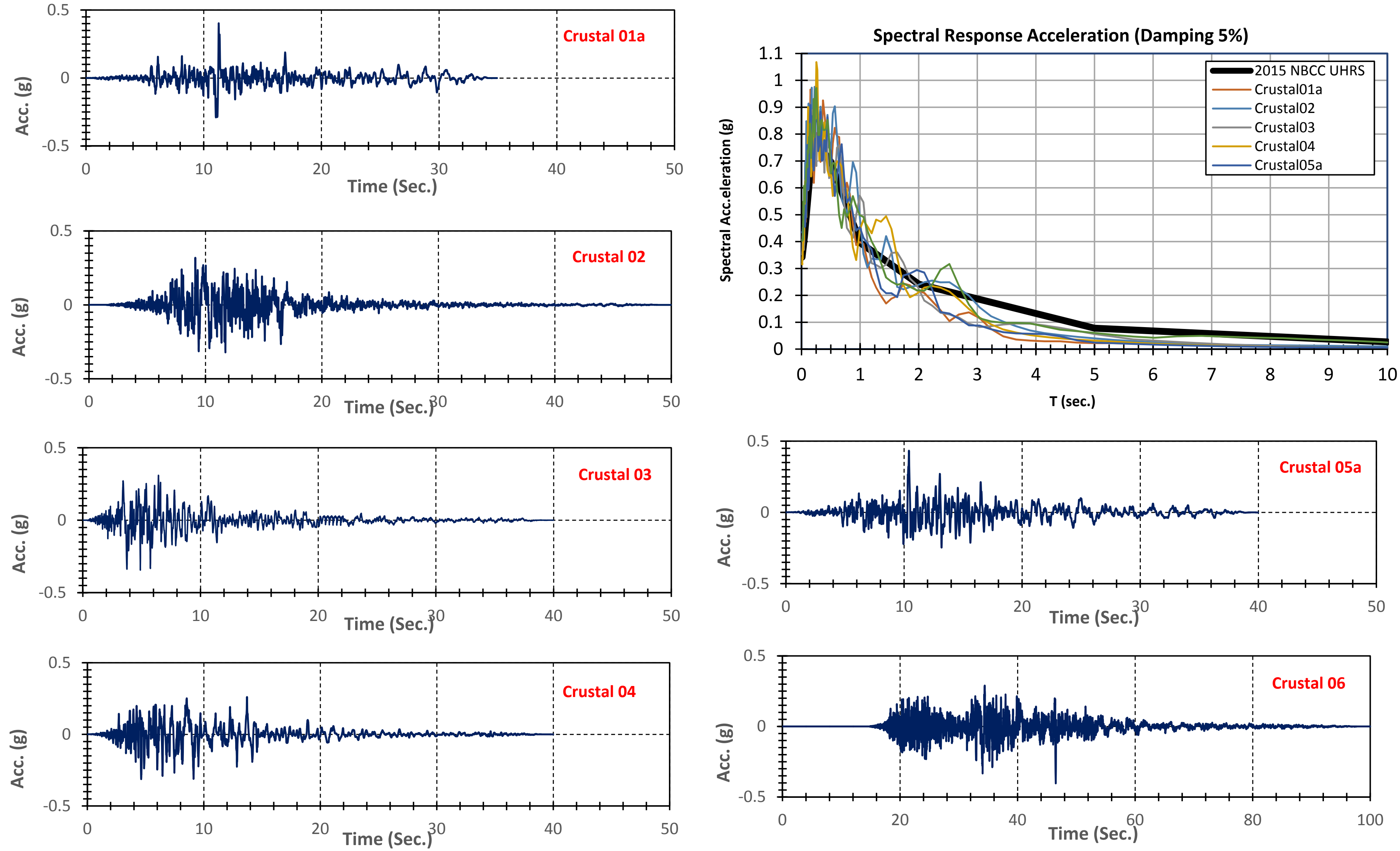




CLIENT	CDM SMITH CANADA ULC	PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates	TITLE	Acceleration Time Histories 2010 NBCC - Interface

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT No.	Phase	Rev	FIGURE
1525010	2200	A	4-2B



CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

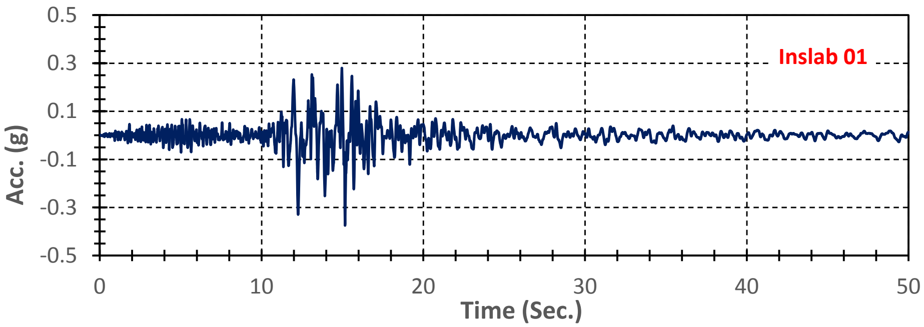
CONSULTANT
Golder Associates

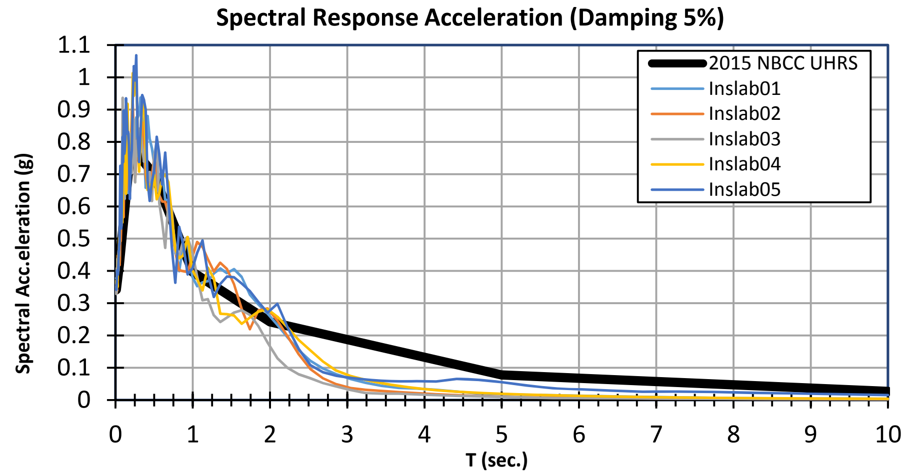
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

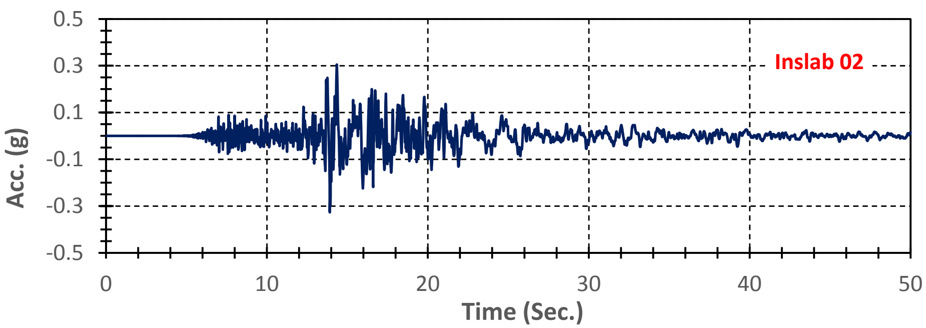
TITLE
Acceleration Time Histories
2015 NBCC – Crustal

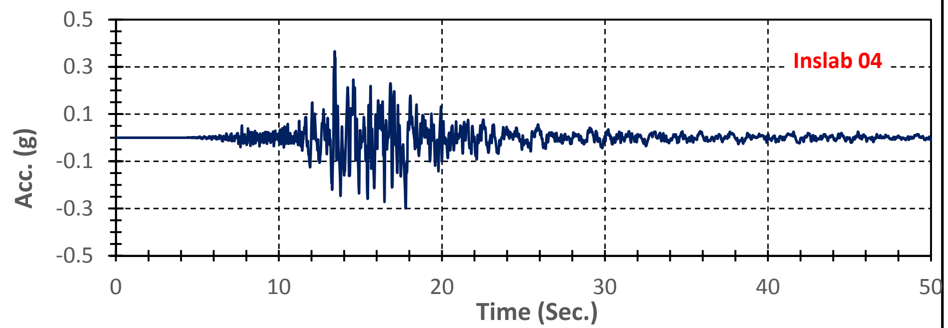
PROJECT No.	Phase	Rev	FIGURE
1525010	2200	A	4-3A

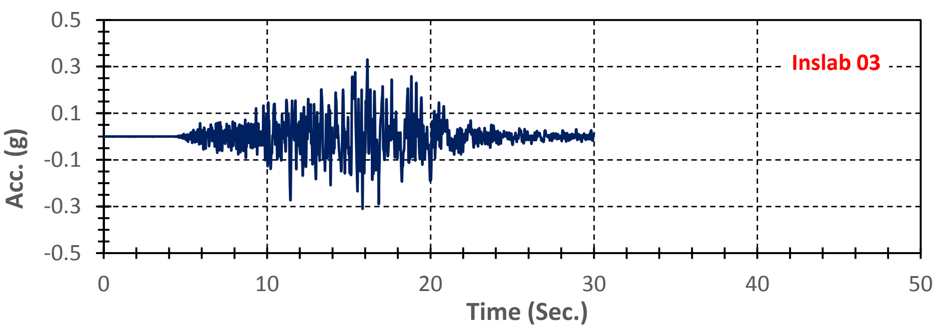
Path: document1 | FileName: DOCUMENT1

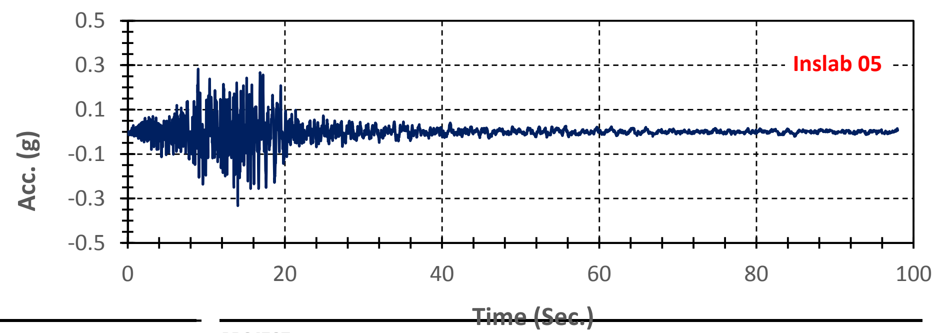












CLIENT

CDM SMITH CANADA ULC

PROJECT

AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

CONSULTANT

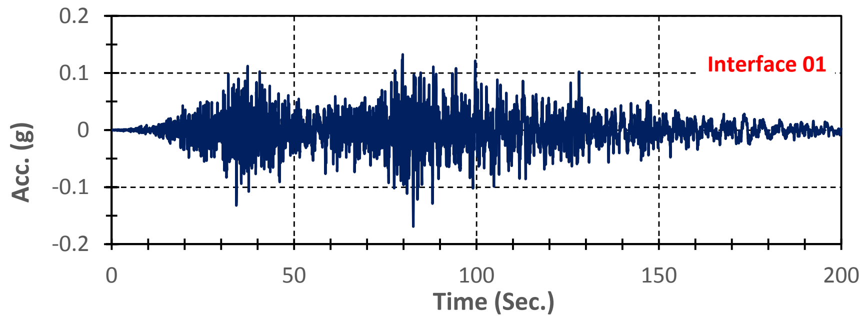
Golder Associates

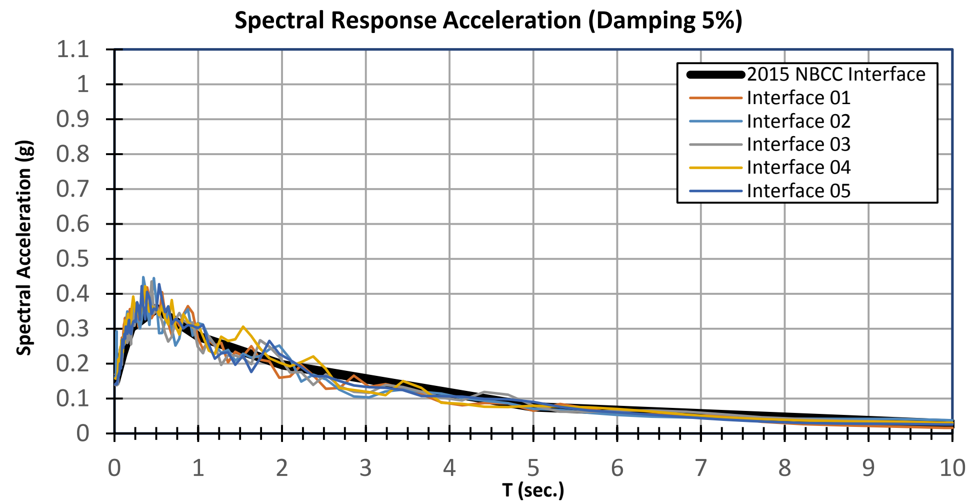
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

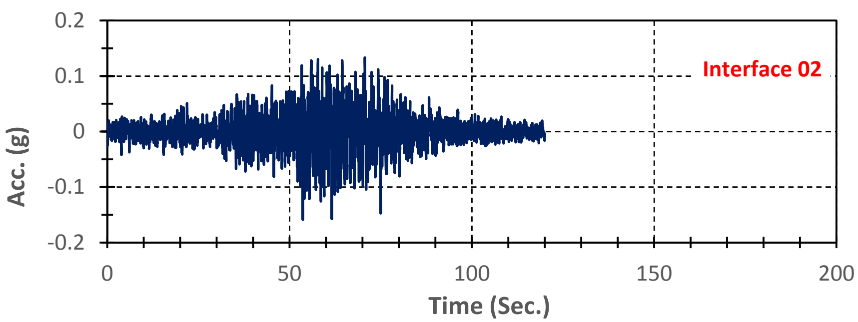
TITLE

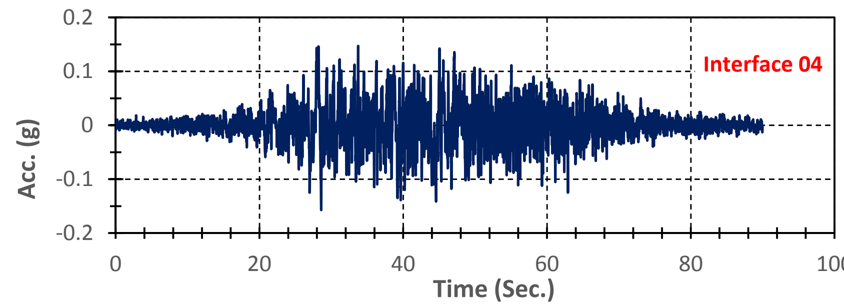
Acceleration Time Histories 2015 NBCC - Inslab

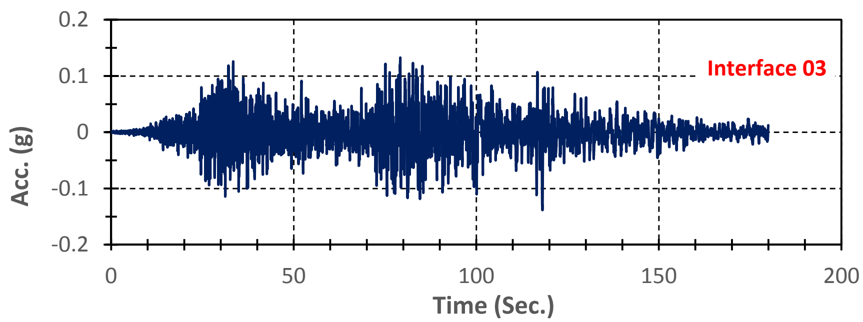
PROJECT No.	Phase	Rev	FIGURE
1525010	2200	A	4-3B

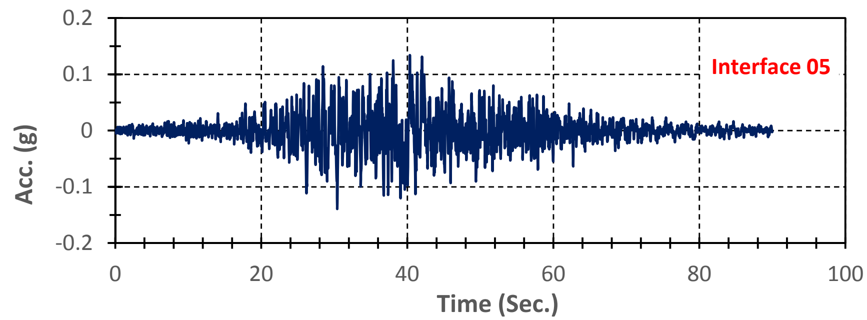












CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

CONSULTANT

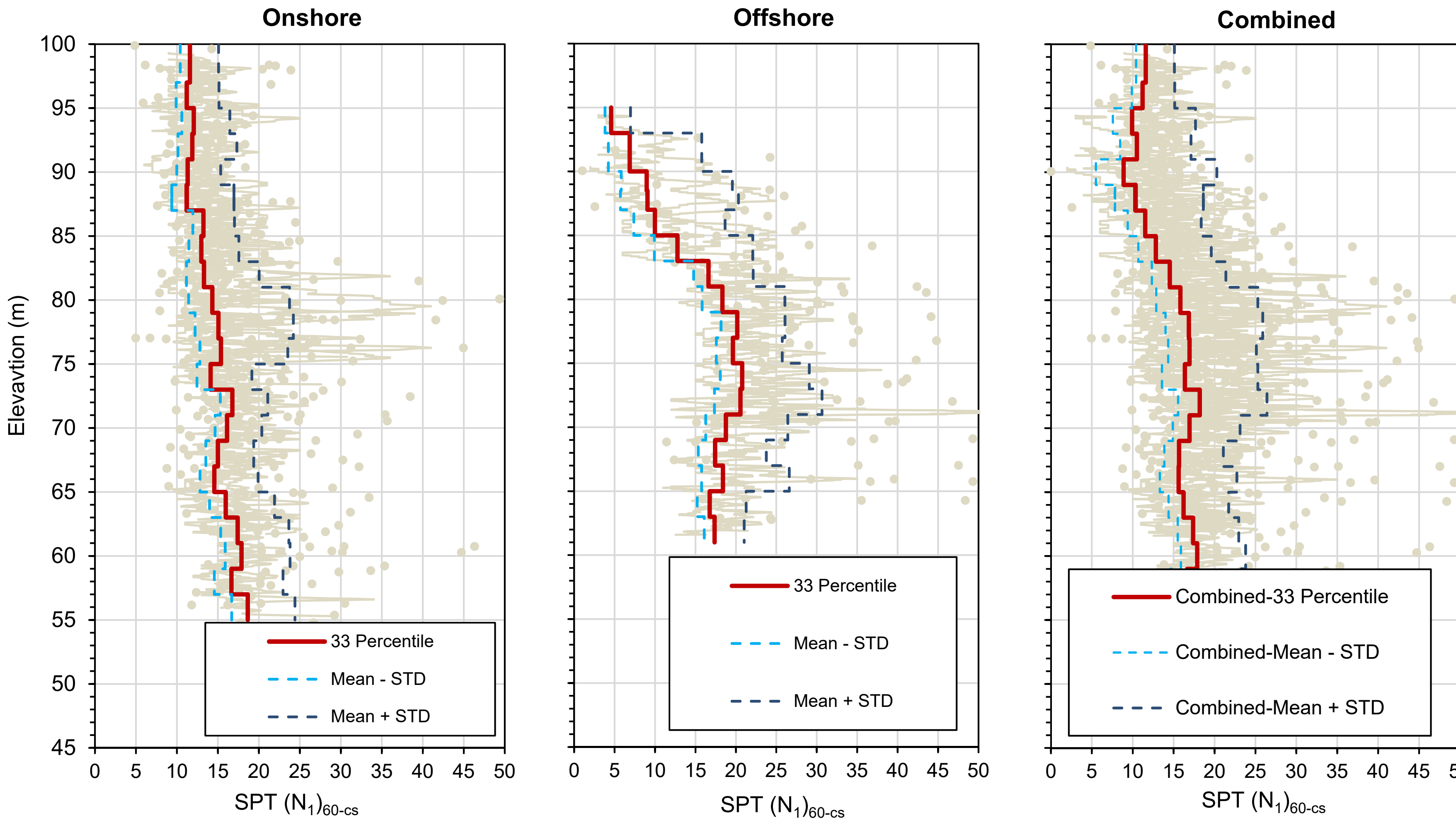
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Acceleration Time Histories
2015 NBCC - Interface

PROJECT No.	Phase	Rev	FIGURE
1525010	2200	A	4-3C

Path: document1 | FileName: DOCUMENT1



CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

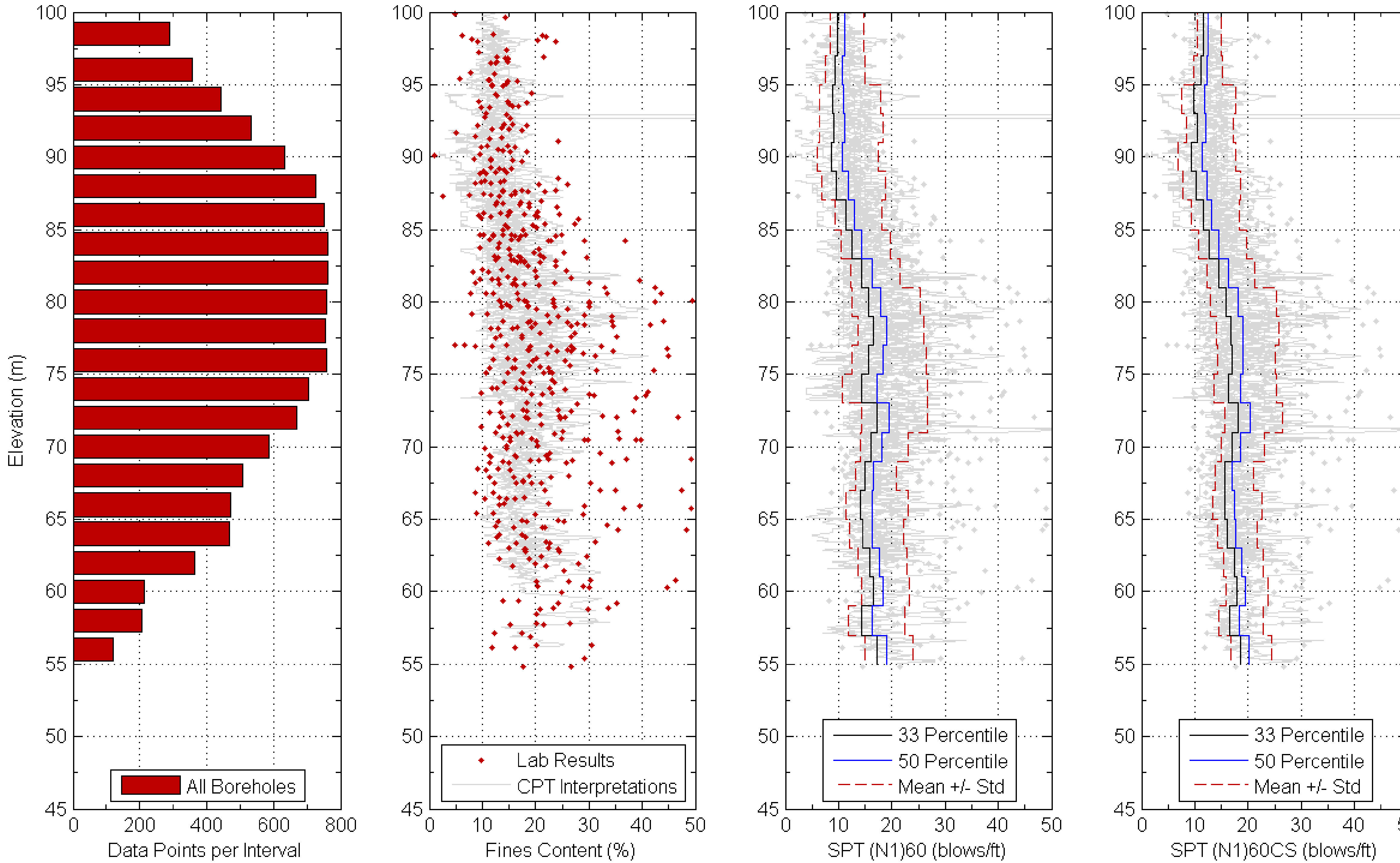
CONSULTANT

Golder Associates

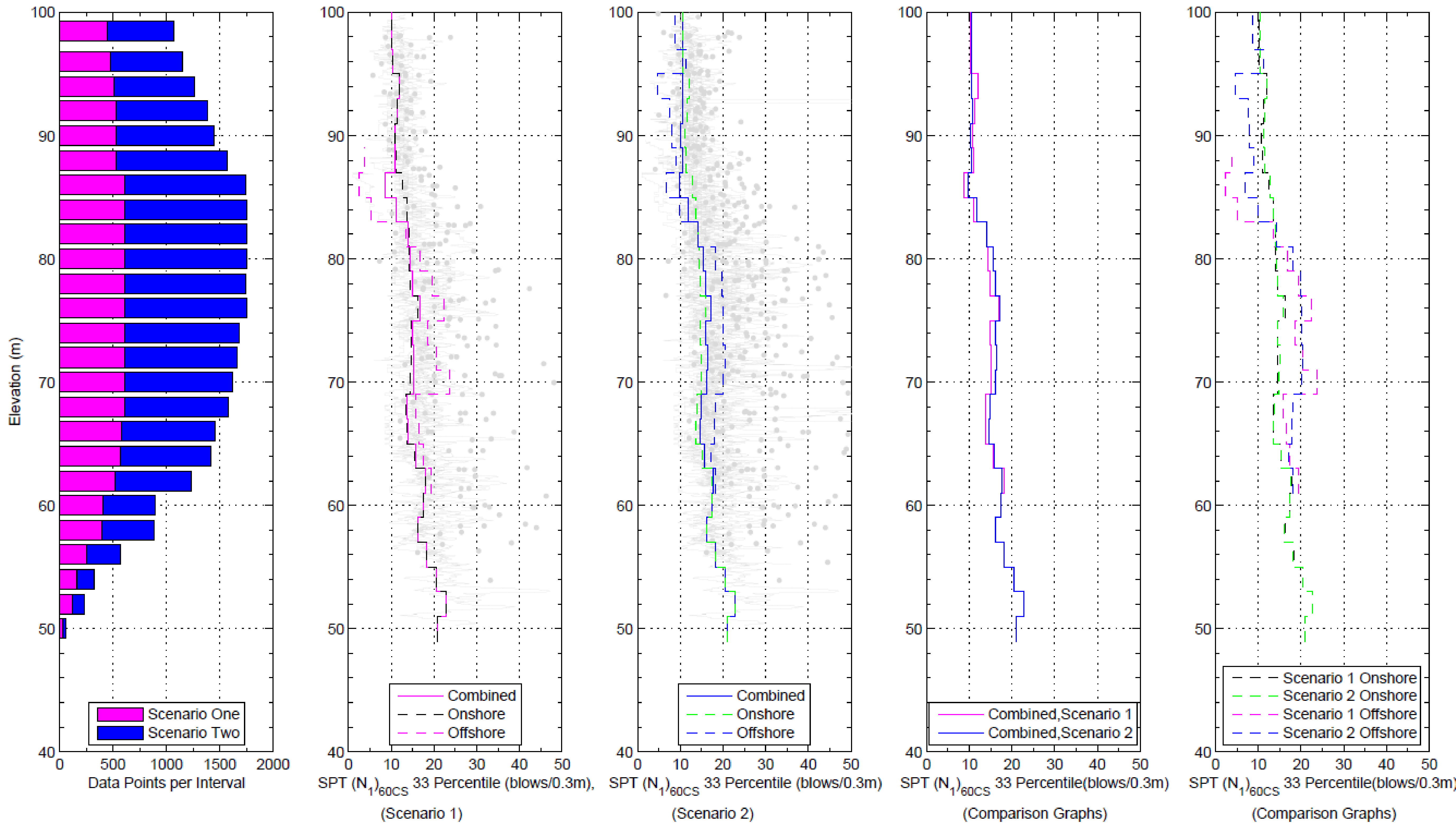
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
SPT $(N_1)_{60CS}$ Profiles
Initial Phase

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-1



CLIENT **CDM SMITH CANADA ULC**	PROJECT **AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC**
CONSULTANT YYYY-MM-DD 2017-11-30 PREPARED M.S.K DESIGN M.S.K REVIEW Y.B APPROVED V.F	TITLE **SPT $(N_1)_{60CS}$ Profiles** **Subsequent Phase**
	PROJECT No. 1525010 · Phase 2100 · A · FIGURE **5-2**



CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

CONSULTANT

Golder Associates

DD-MMM-YYYY	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
SPT $(N_1)_{60CS}$ Profiles – 33rd Percentile Values
Scenario 1 vs. Scenario 2

PROJECT No.	PHASE	REV	FIGURE
1525010	2000	B	5-3

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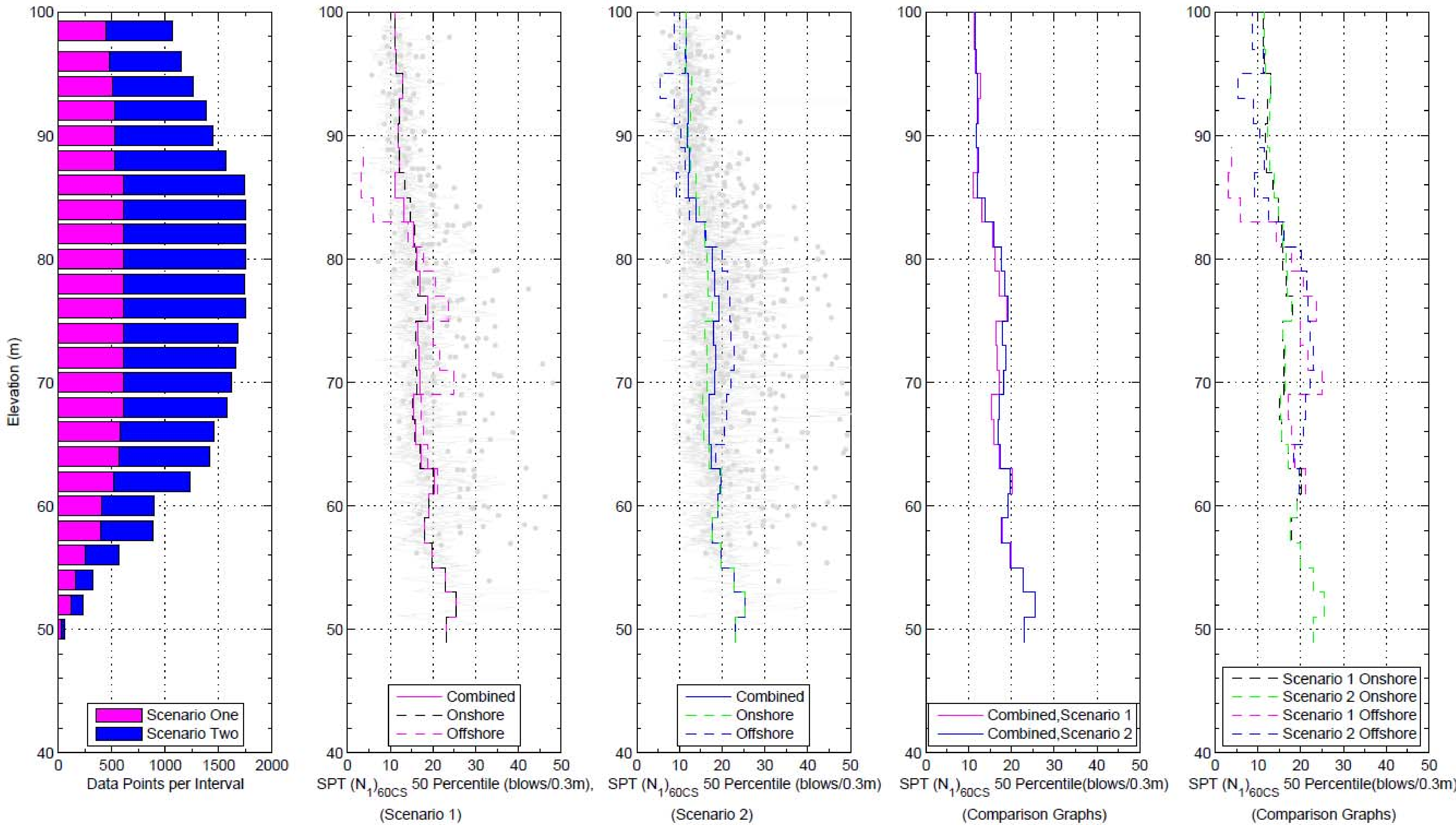


CLIENT
CDM SMITH CANADA ULC

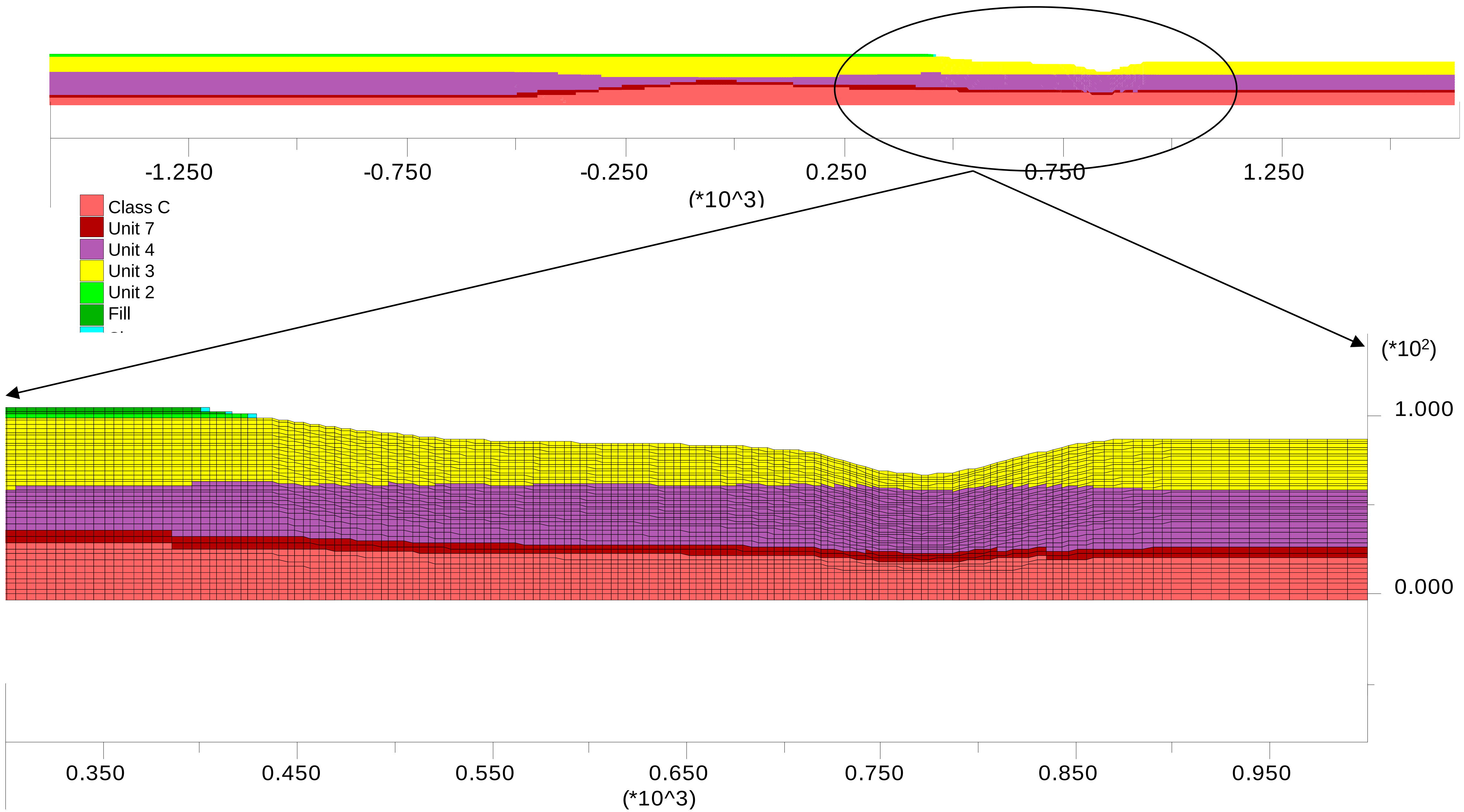
PROJECT
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

CONSULTANT
Golder Associates

DD-MMM-YYYY	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
SPT $(N_1)_{60CS}$ Profiles – 50th Percentile Values
Scenario 1 vs. Scenario 2

PROJECT No.	PHASE	REV	FIGURE
1525010	2000	B	5-4



CLIENT	PROJECT
CDM SMITH CANADA ULC	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

CONSULTANT: Golder Associates

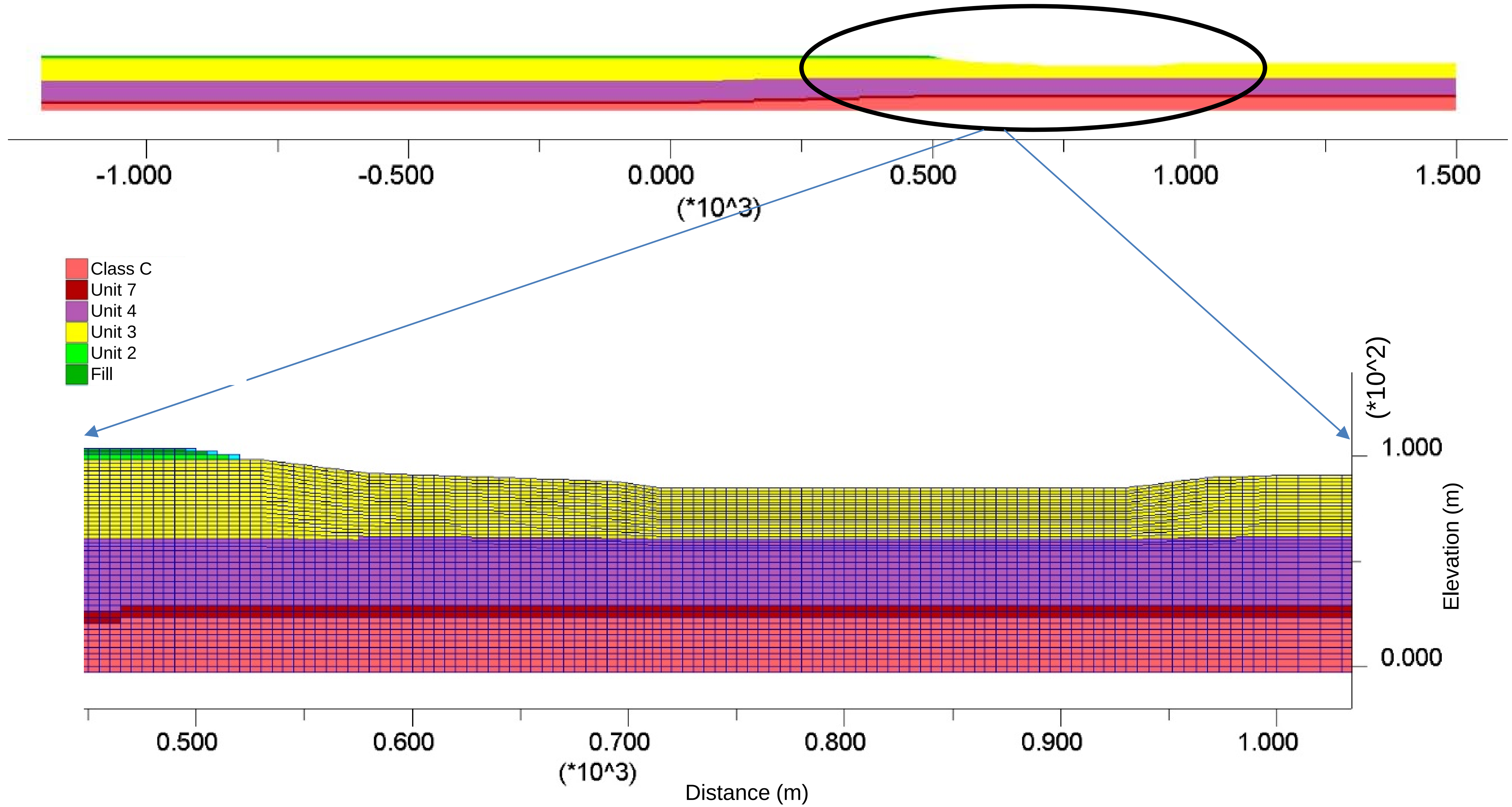
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE

Finite Difference Grid and Stratigraphic Profile Section A-A' – Initial Phase

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-5

Path: document1 | FileName: DOCUMENT1



CLIENT **CDM SMITH CANADA ULC**	PROJECT **AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC**

CONSULTANT

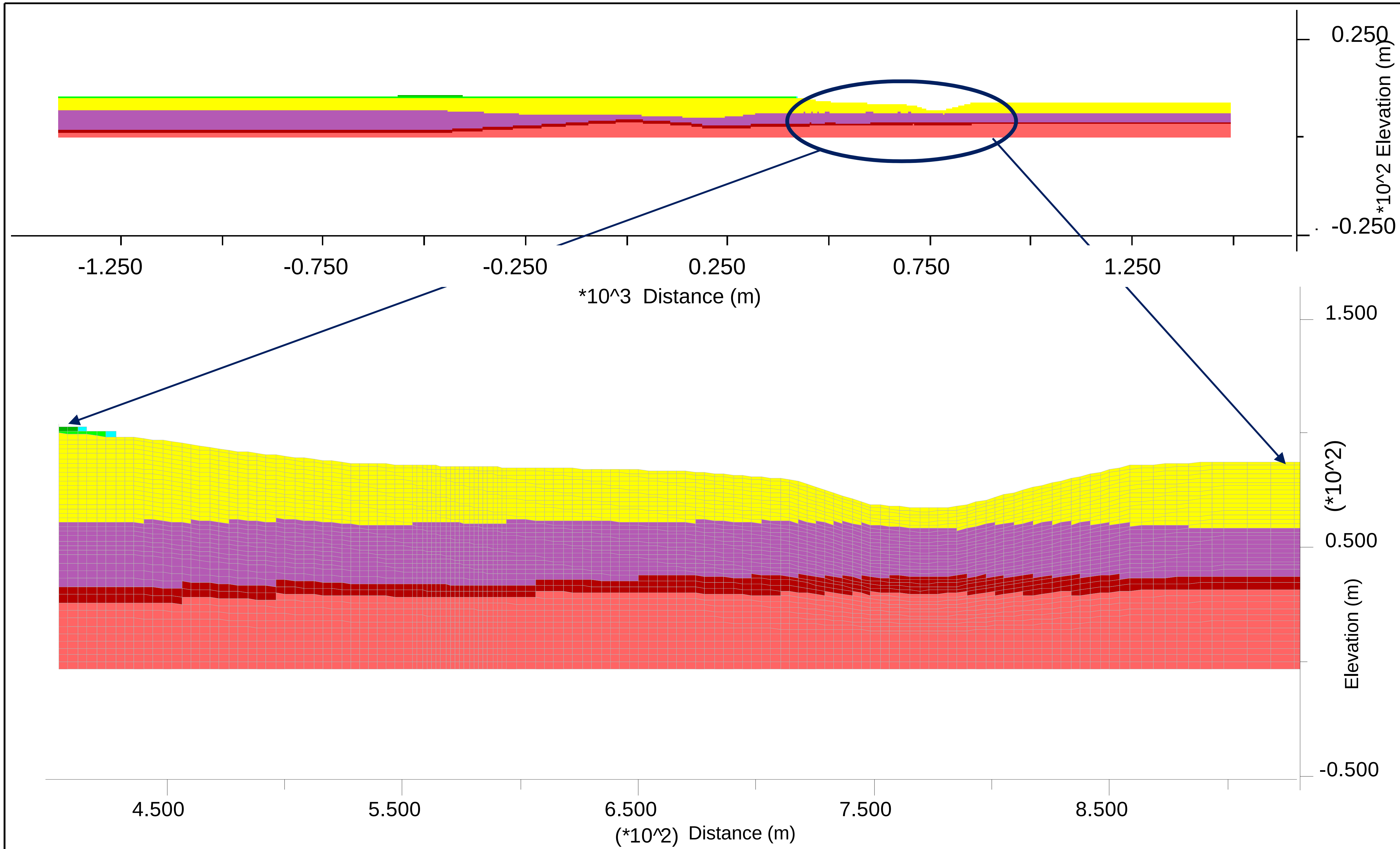
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE

Finite Difference Grid and Stratigraphic Profile Section B-B' – Initial Phase

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-6



User-defined Groups
- Unit7_base
- Unit7
- Unit4
- Unit3
- Unit2
- Fill

CLIENT
CDM SMITH CANADA ULC

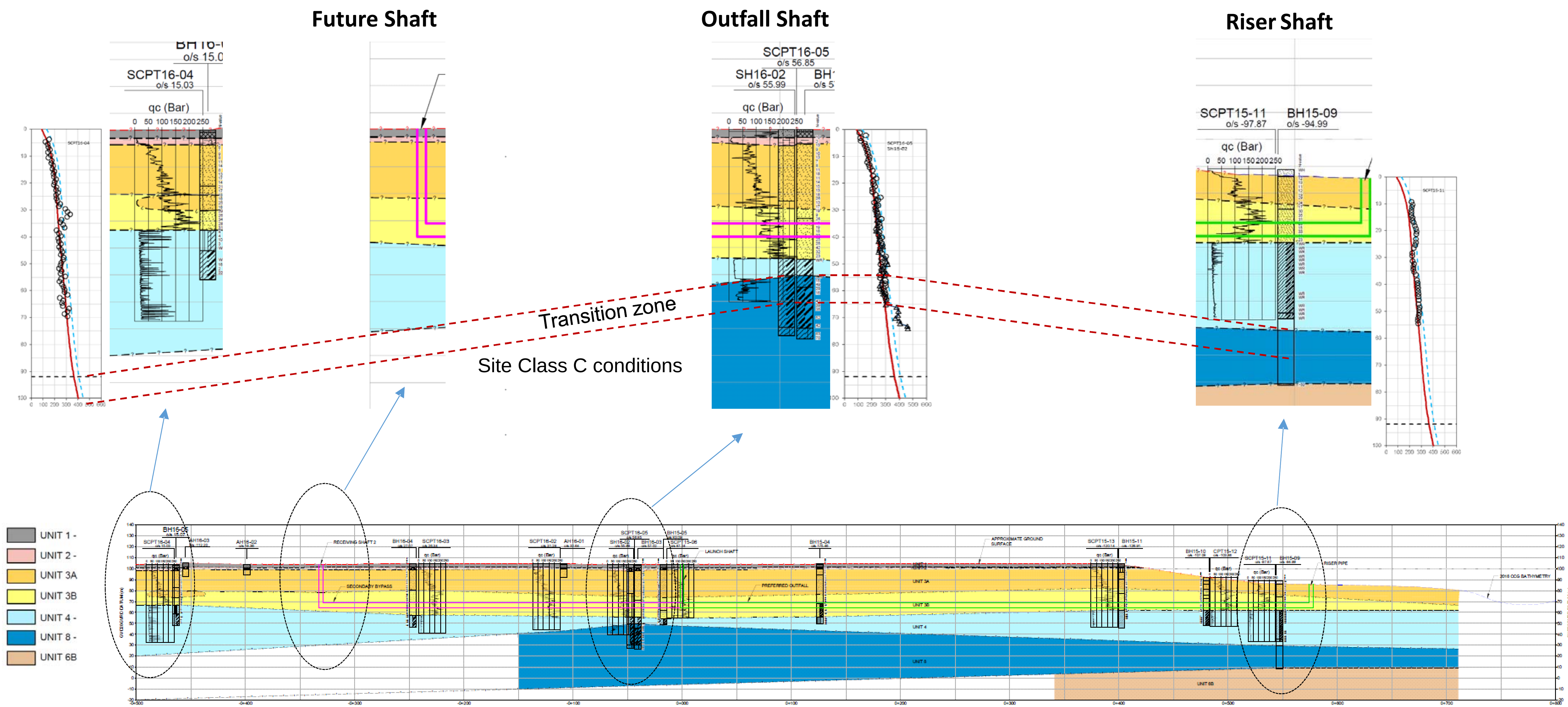
CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
**AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC**

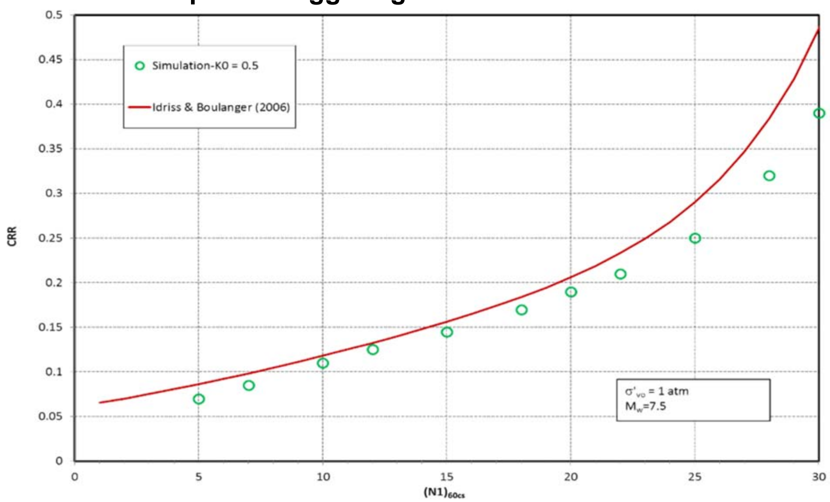
TITLE
**Finite Difference Grid and Stratigraphic Profile
Section A-A' – Final Phase**

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-7

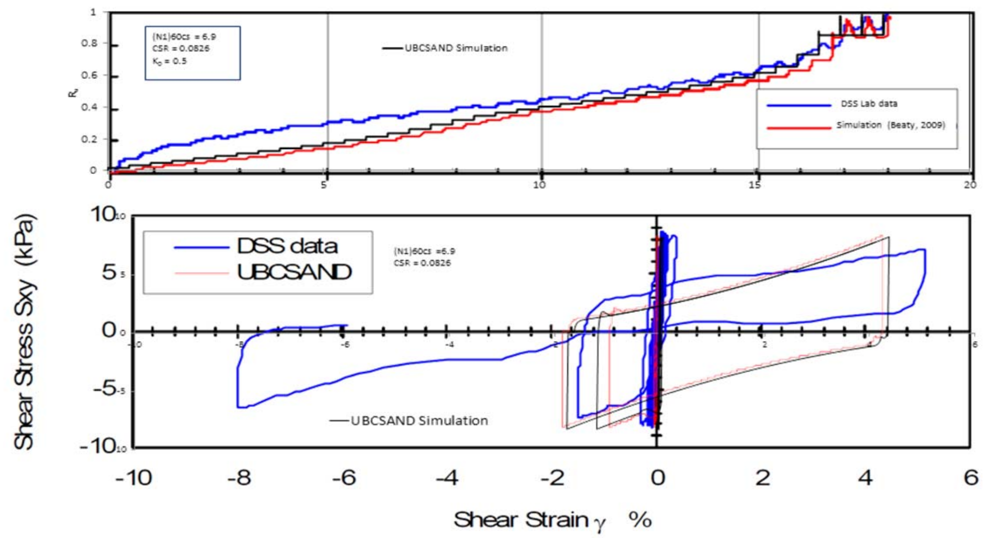


CLIENT	CDM SMITH CANADA ULC
PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F
TITLE	Class C Ground Condition Section A-A' – Initial Phase
PROJECT No.	1525010
Phase	2100
Rev	A
FIGURE	5-8

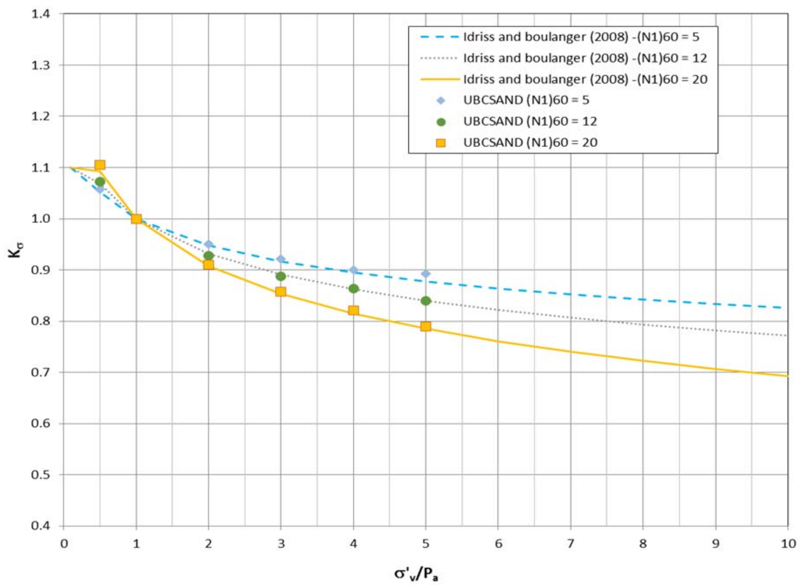
Semi-Empirical Triggering Chart and UBCSAND Prediction



UBCSAND Model Prediction , r_u and stress-strain vs. Laboratory Data



Confining Stress Effect – UBCSAND vs. Laboratory Data



CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

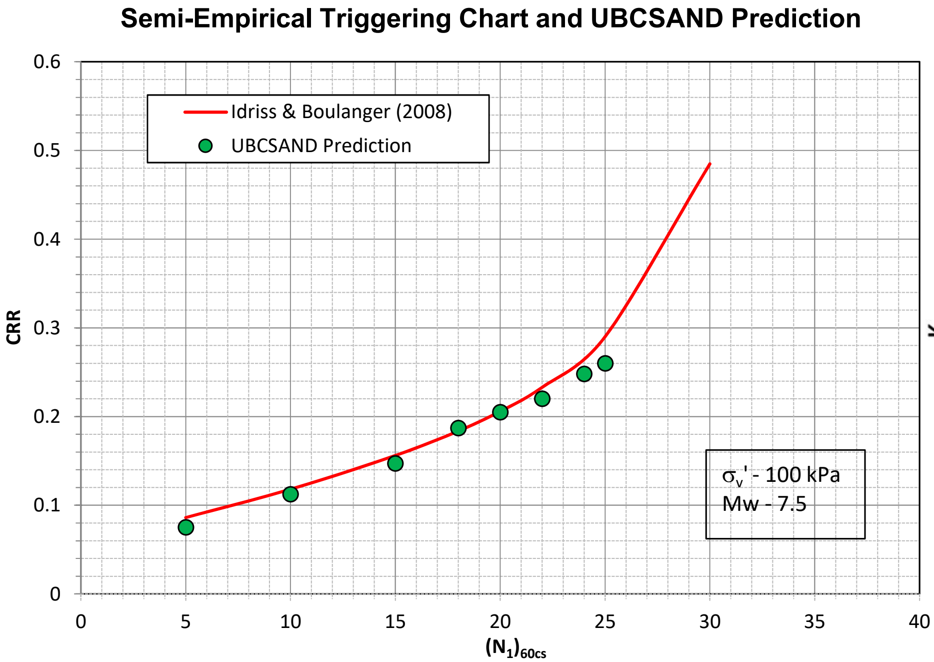
CONSULTANT
Golder Associates

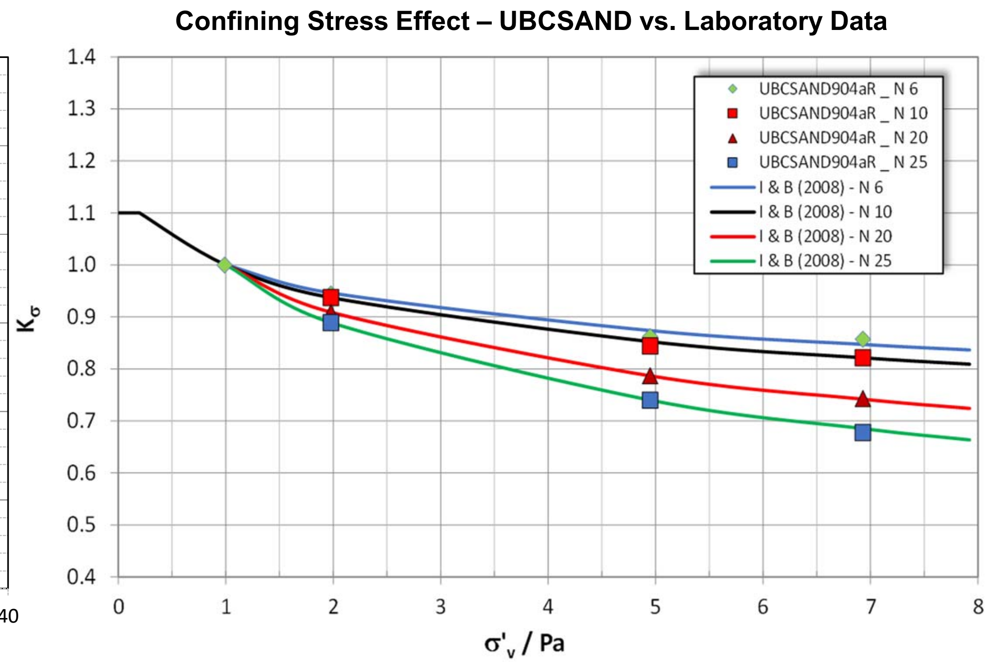
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
UBCSAND Model Calibration Initial Phase

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-9

Path: document1 | FileName: DOCUMENT1





CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

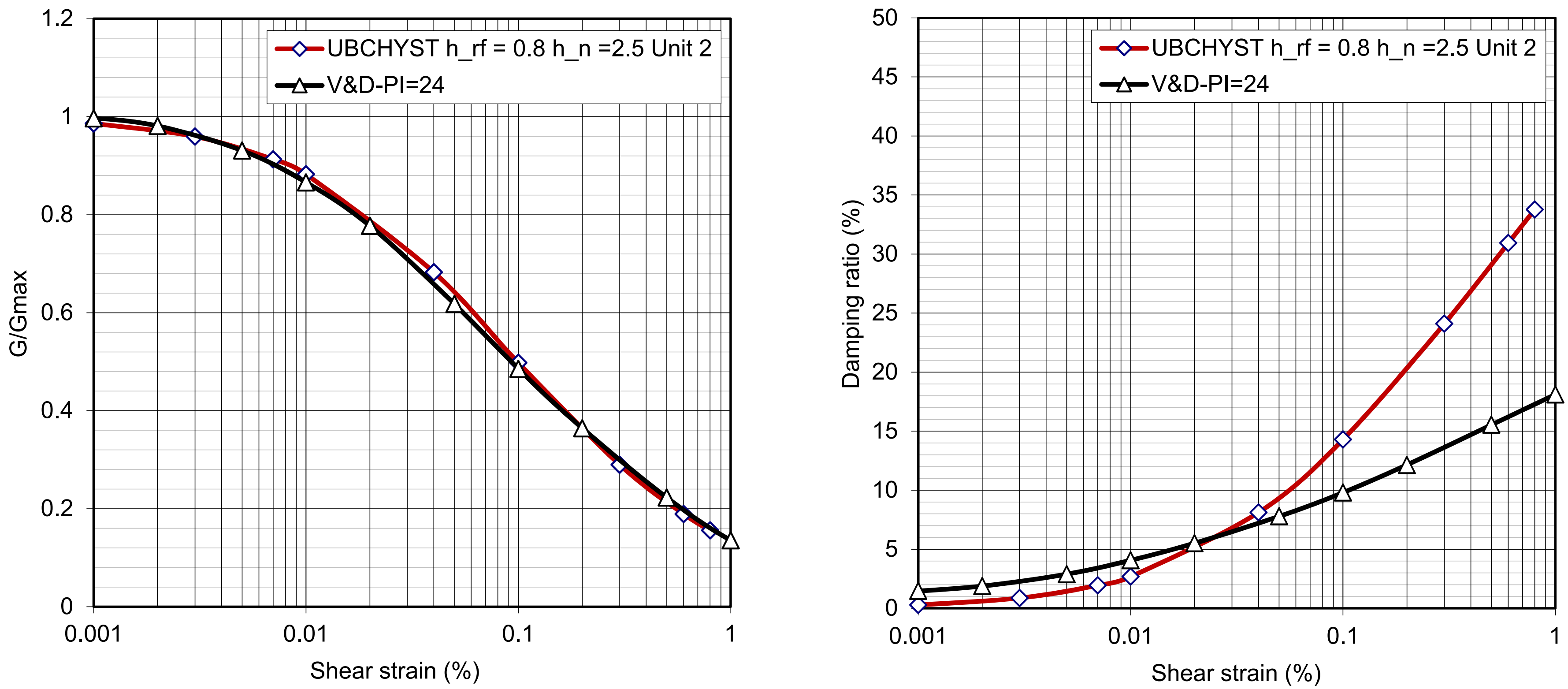
CONSULTANT

Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
UBCSAND Model Calibration Final Phase

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-10



CLIENT
CDM SMITH CANADA ULC

CONSULTANT

Golder Associates

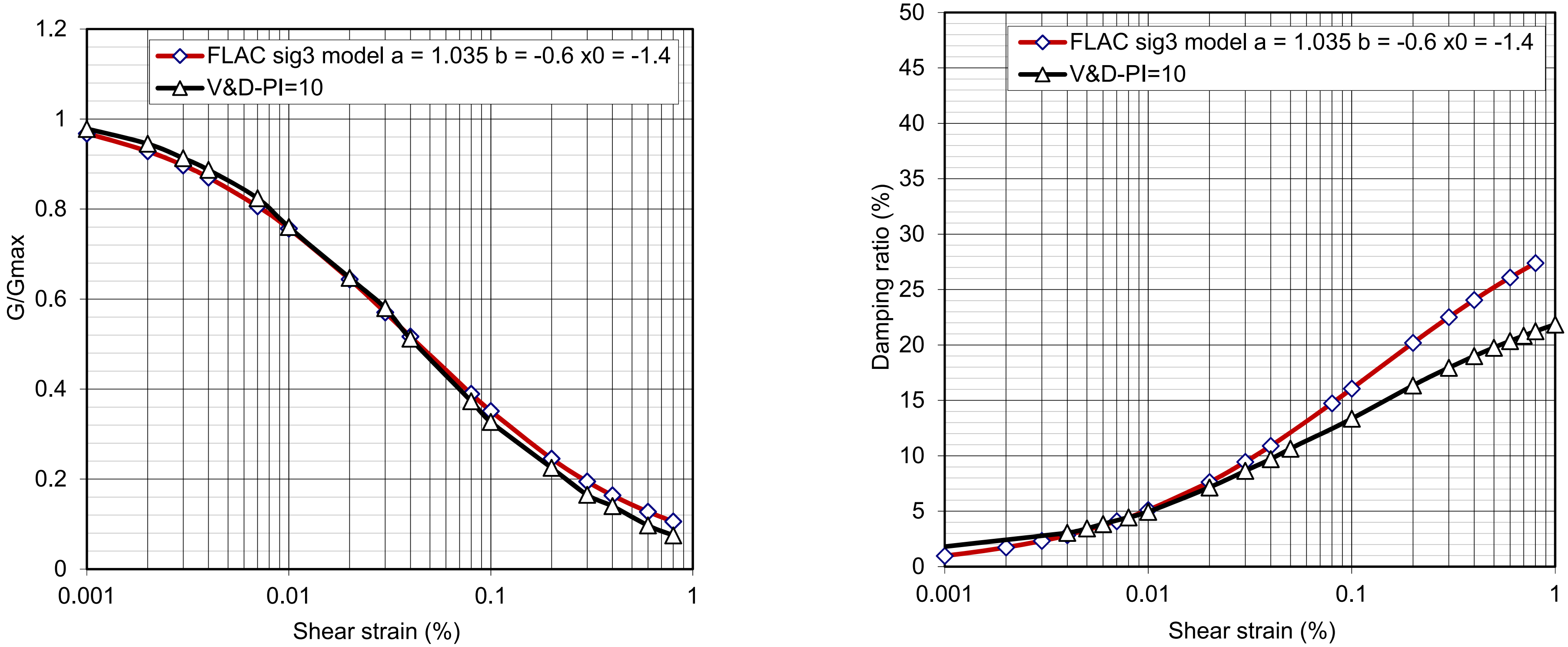
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
UBCHYST Model Calibration
(Unit 2)

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-11

Path: document1 | FileName: DOCUMENT1



CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

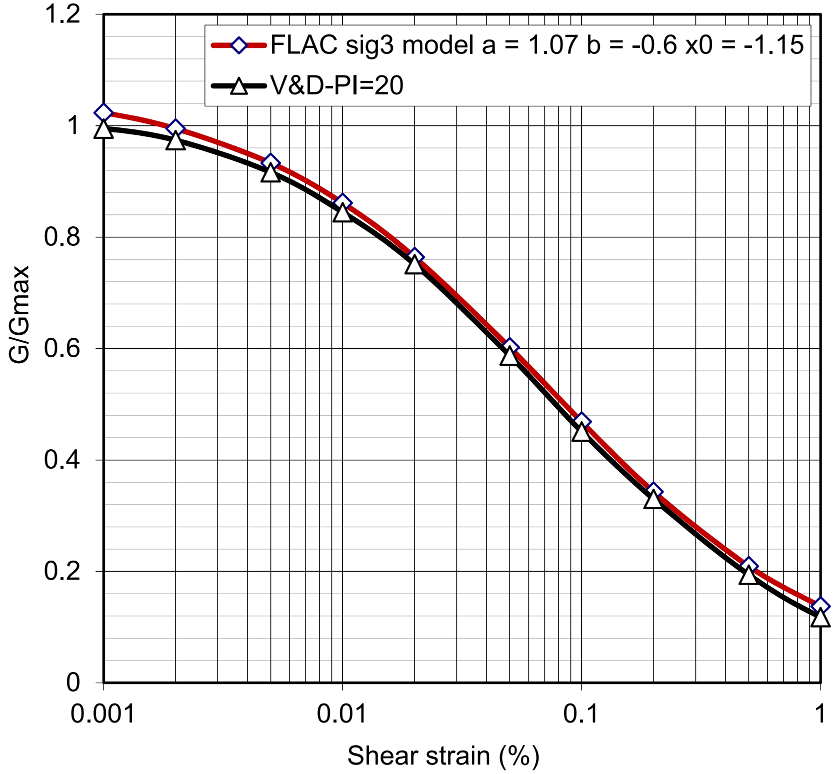
CONSULTANT

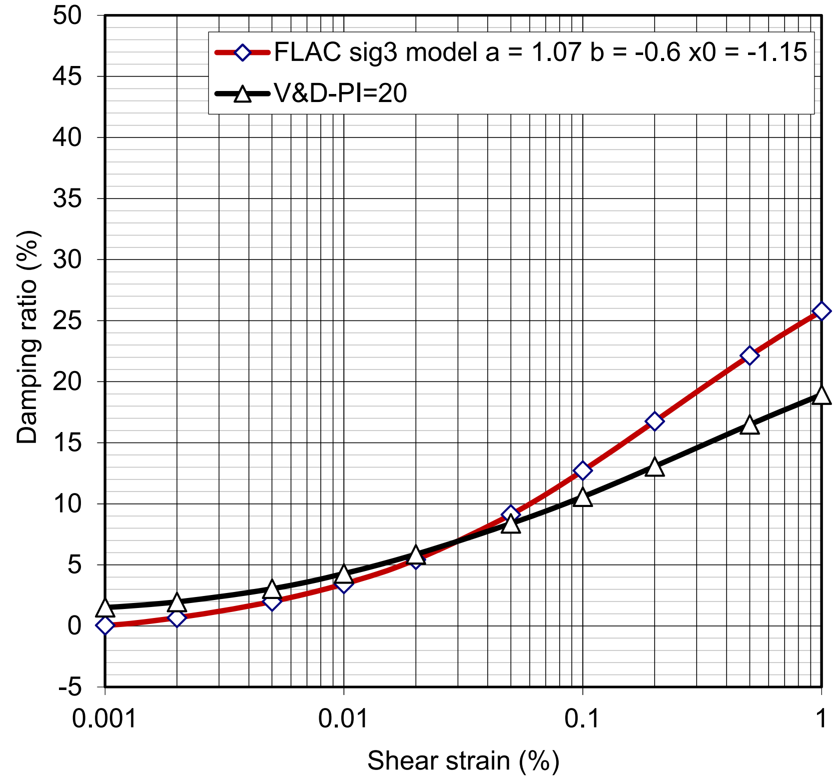
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Hyperbolic Sig3 Model Calibration
(Unit 4)

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-12a





CLIENT
CDM SMITH CANADA ULC

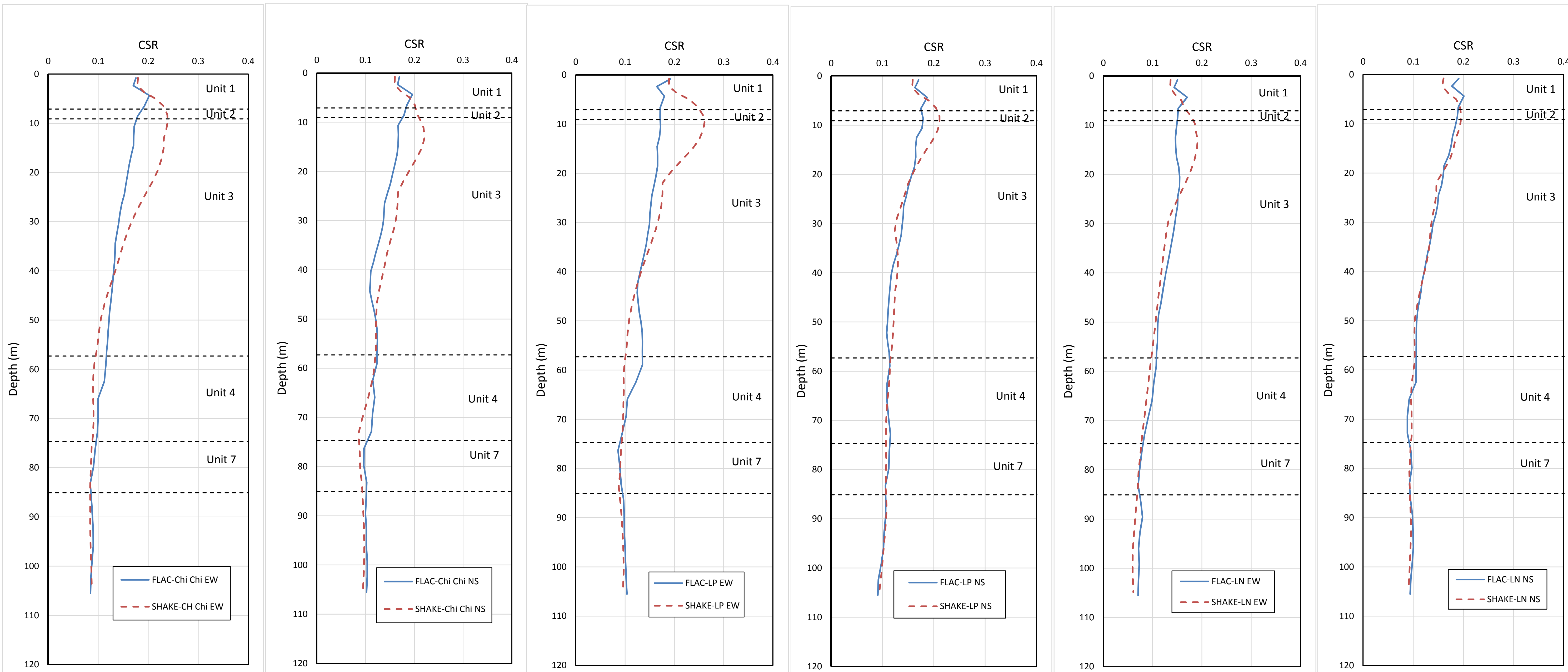
PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Hyperbolic Sig3 Model Calibration (Unit 7)

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-12b



<u>Note</u>:
CSR: Cyclic Resistance Ratio

CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

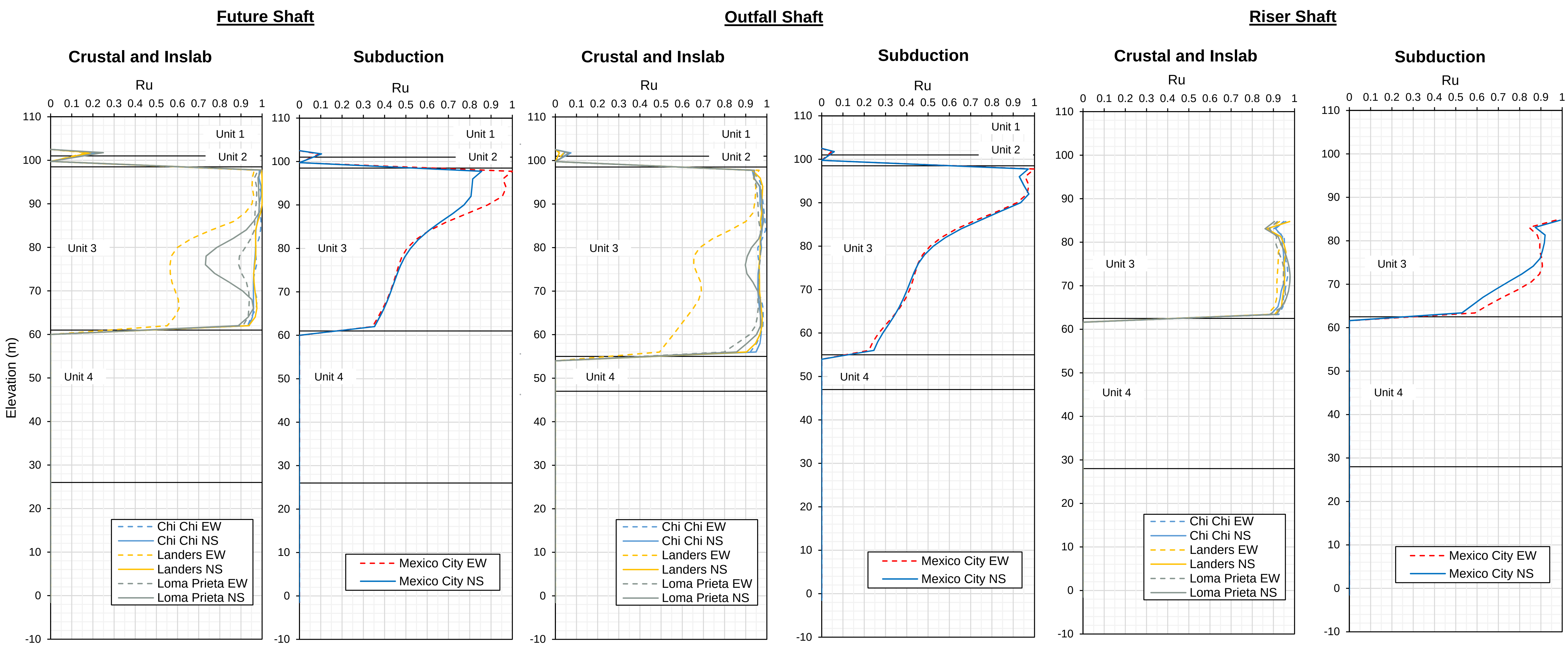
CONSULTANT

Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

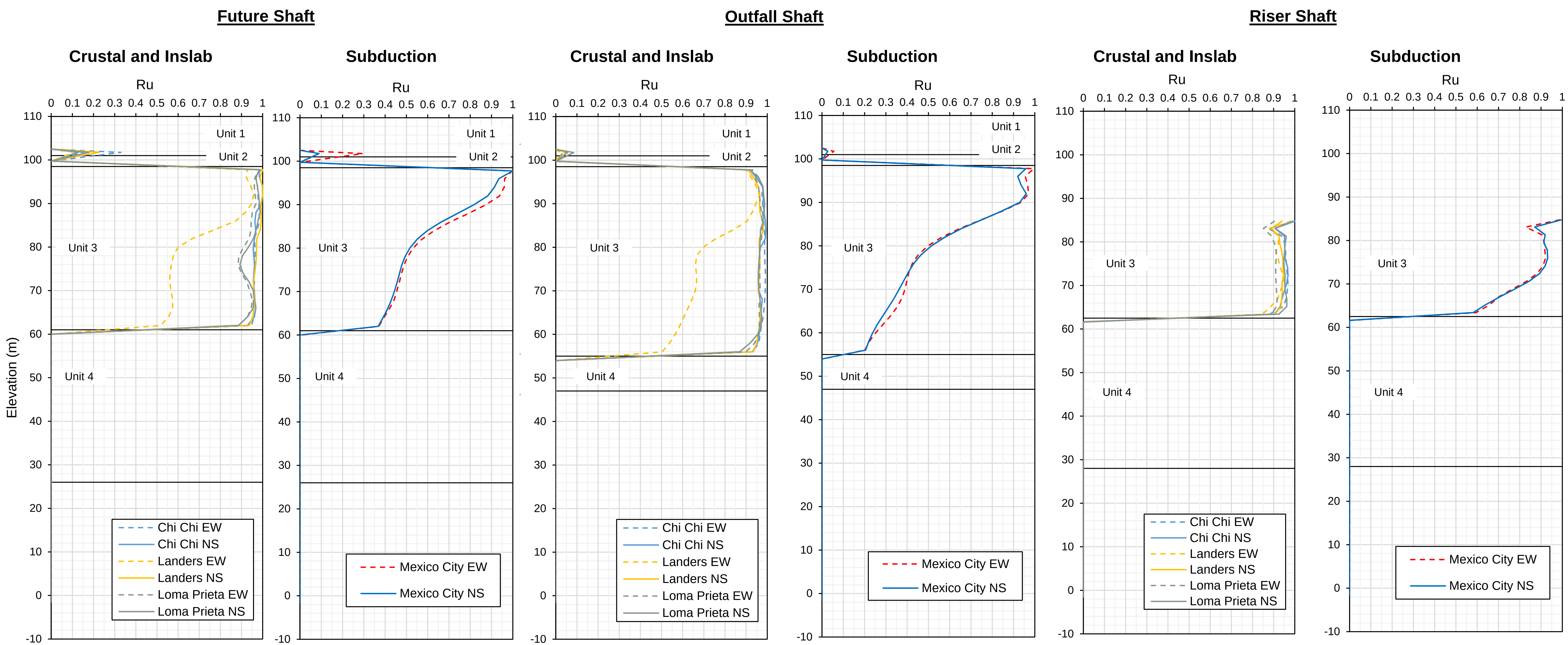
TITLE
1D FLAC vs SHAKE Calibration
Typical Location – Future Shaft

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	**5-13**



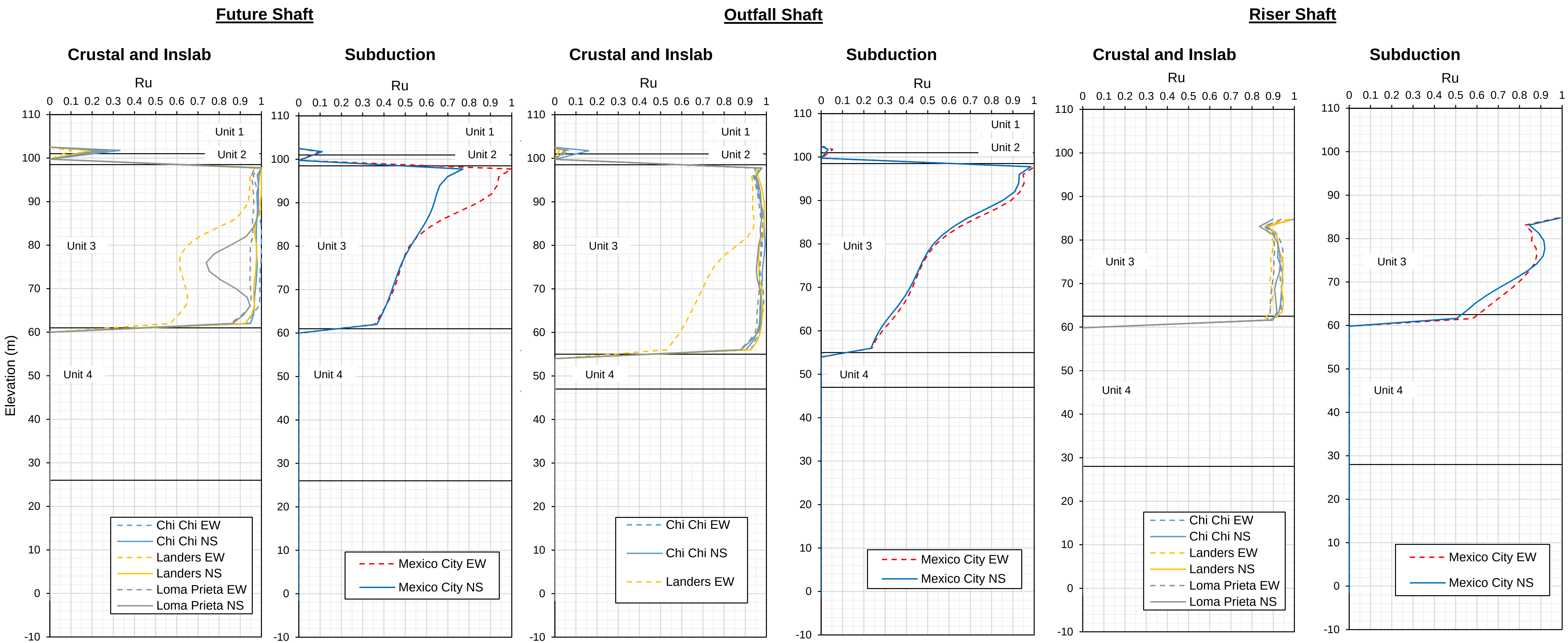
Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT	CDM SMITH CANADA ULC
PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F
TITLE	Extent of Liquefaction – Case 1 Shaft Locations
PROJECT No.	1525010
Phase	2100
Rev	A
FIGURE	5-14



Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT	CDM SMITH CANADA ULC
PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F
TITLE	Extent of Liquefaction – Case 2 Shaft Locations
PROJECT No.	1525010
Phase	2100
Rev	A
FIGURE	5-15



Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT
CDM SMITH CANADA ULC

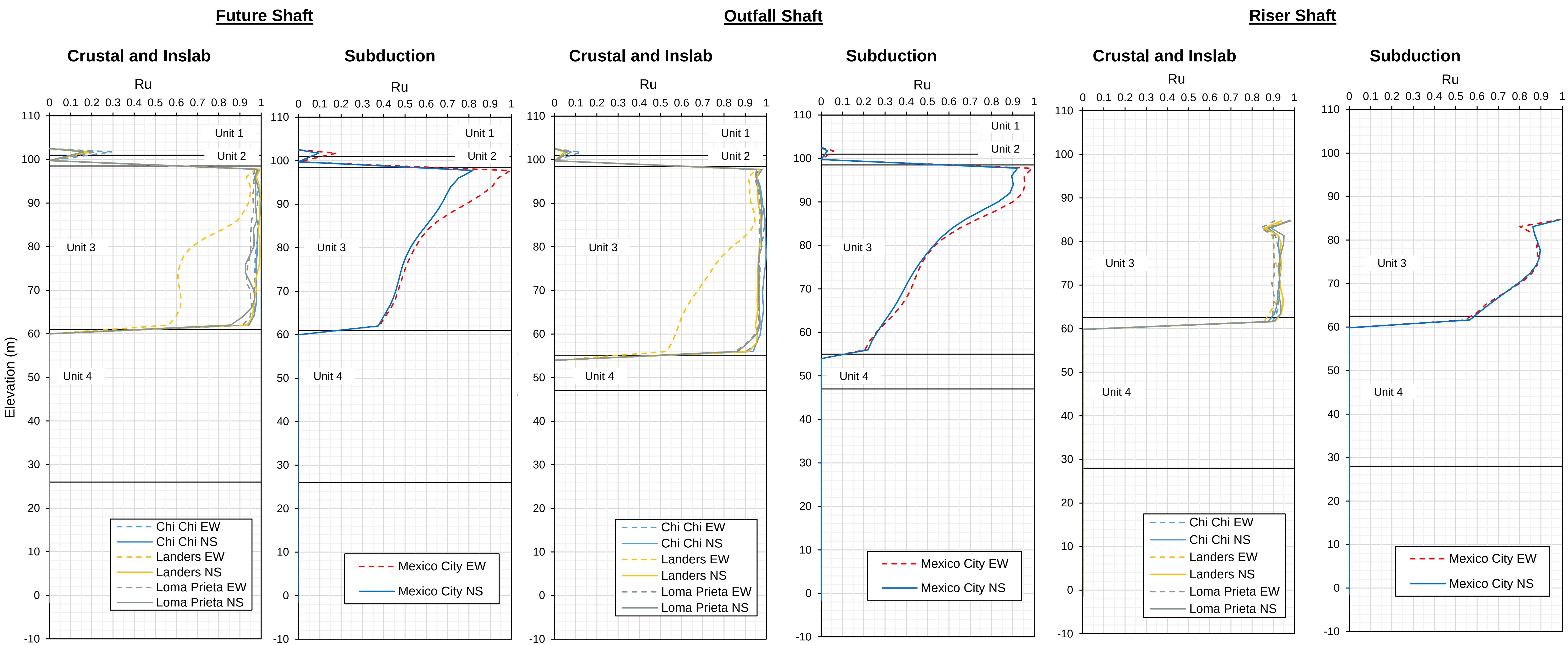
PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Extent of Liquefaction – Case 3
Shaft Locations

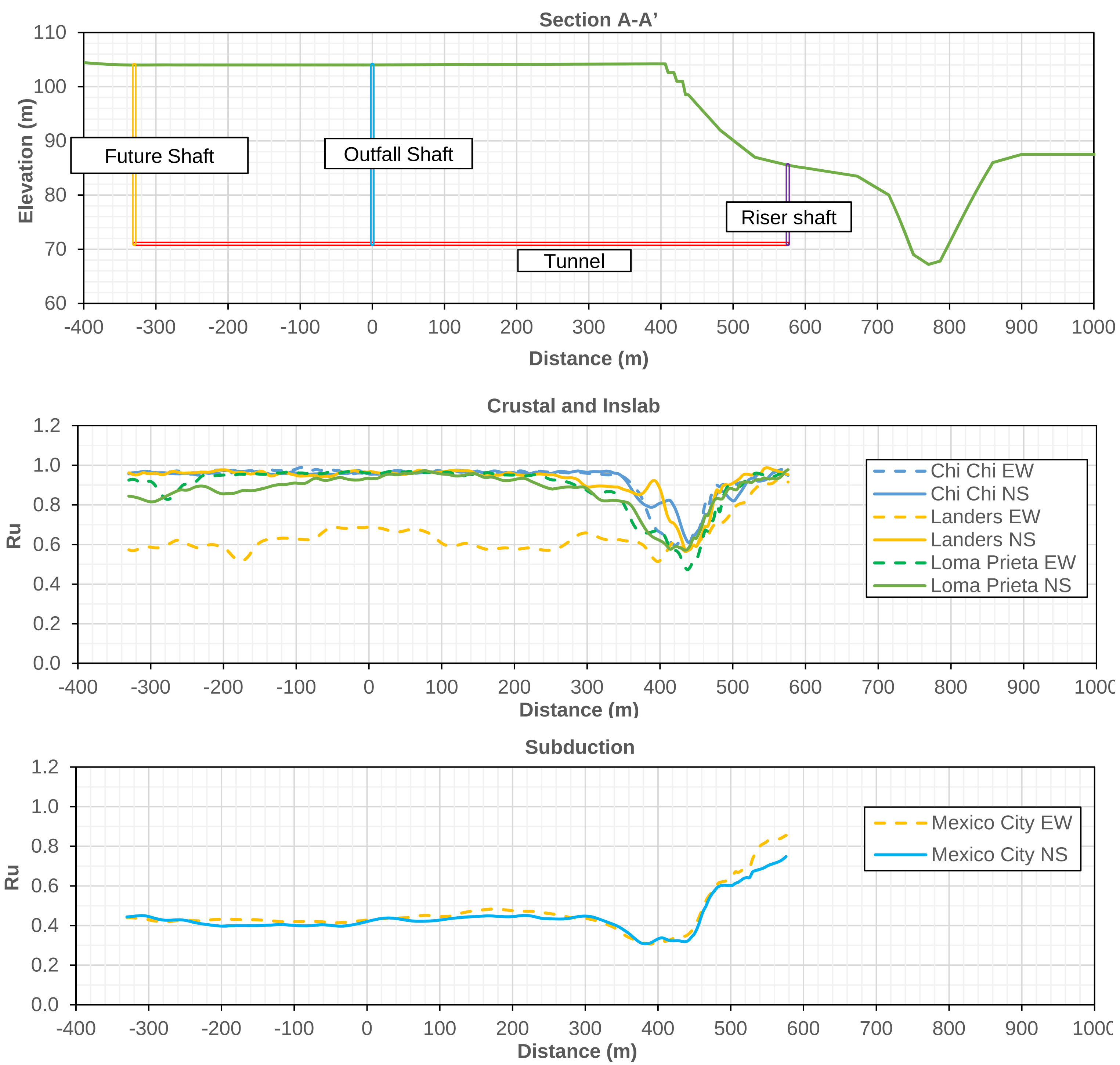
PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-16



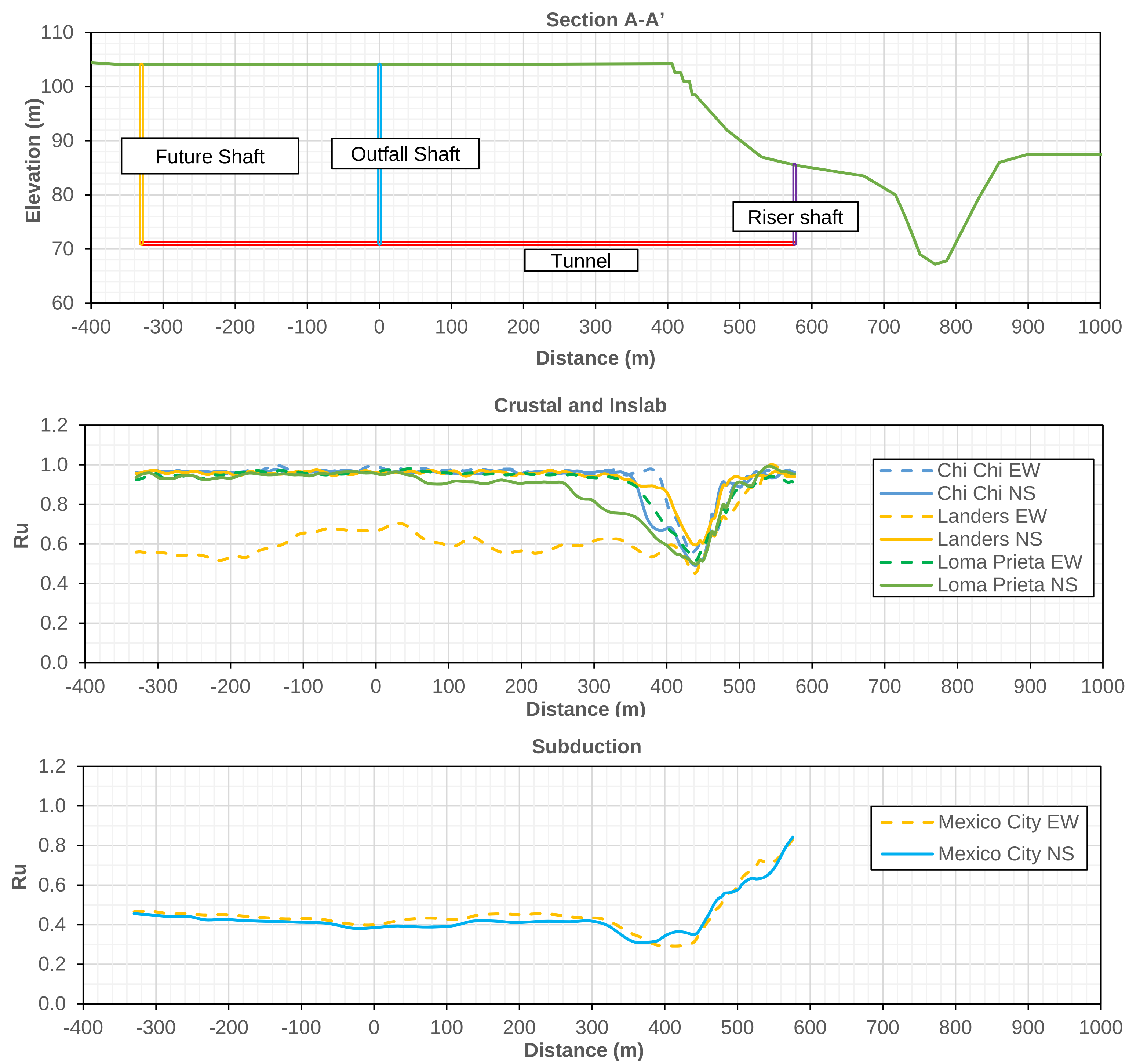
Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT	CDM SMITH CANADA ULC	PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	YYYY-MM-DD: 2017-11-30	TITLE	Extent of Liquefaction – Case 4 Shaft Locations
Golder Associates	PREPARED: M.S.K		
	DESIGN: M.S.K		
	REVIEW: Y.B	PROJECT No.: 1525010	Phase: 2100
	APPROVED: V.F	Rev: A	FIGURE: 5-17

Case 1



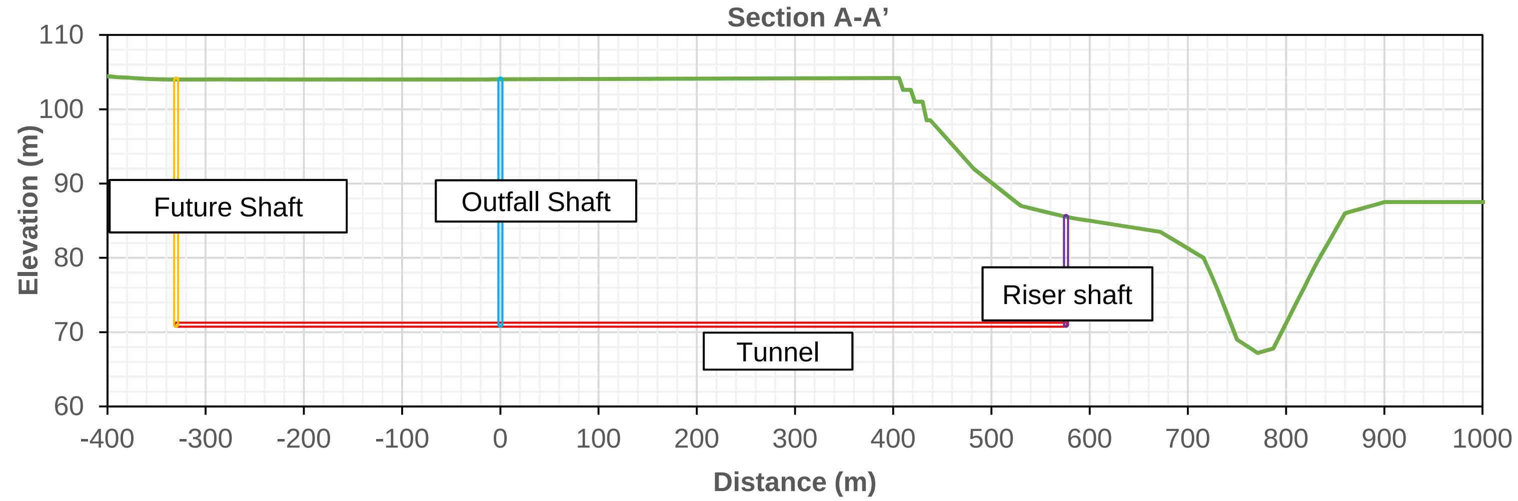
Case 2

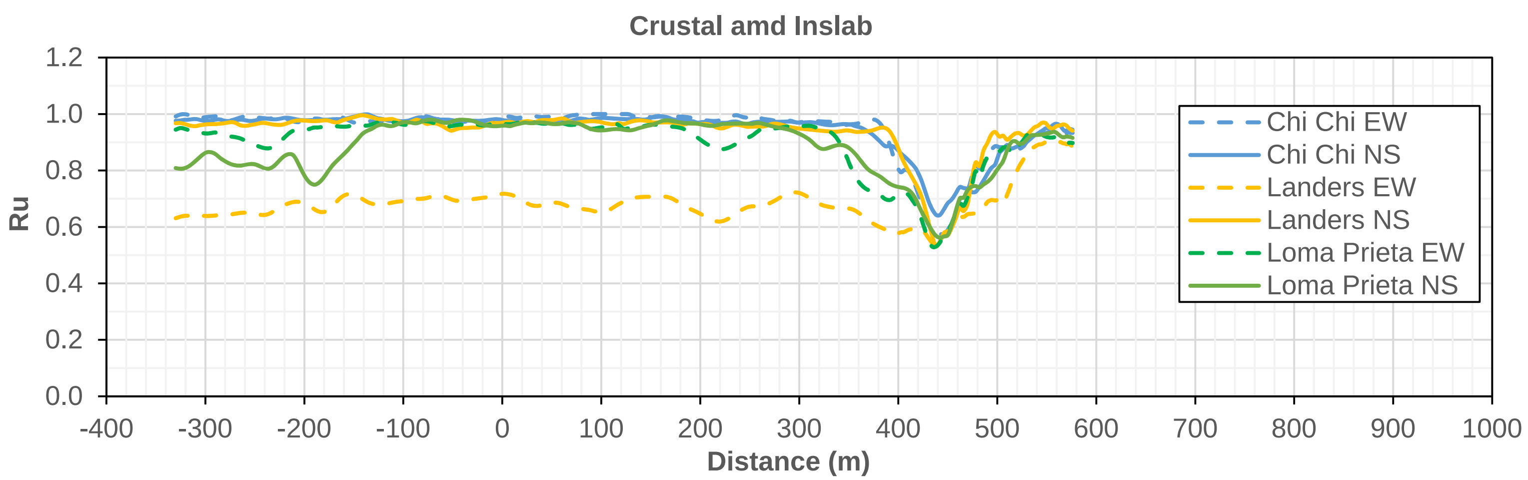


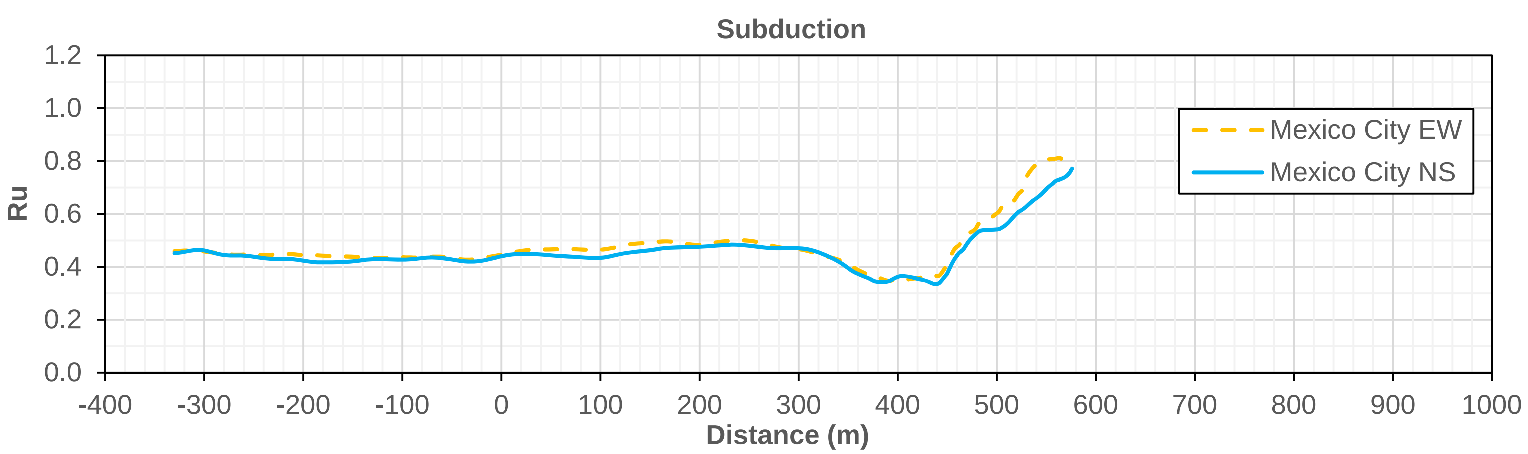
Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT	CDM SMITH CANADA ULC	PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates	TITLE	Extent of Liquefaction Tunnel Level – Cases 1 and 2
YYYY-MM-DD	2017-11-30	PROJECT No.	1525010
PREPARED	M.S.K	Phase	2100
DESIGN	M.S.K	Rev	A
REVIEW	Y.B	FIGURE	5-18
APPROVED	V.F		

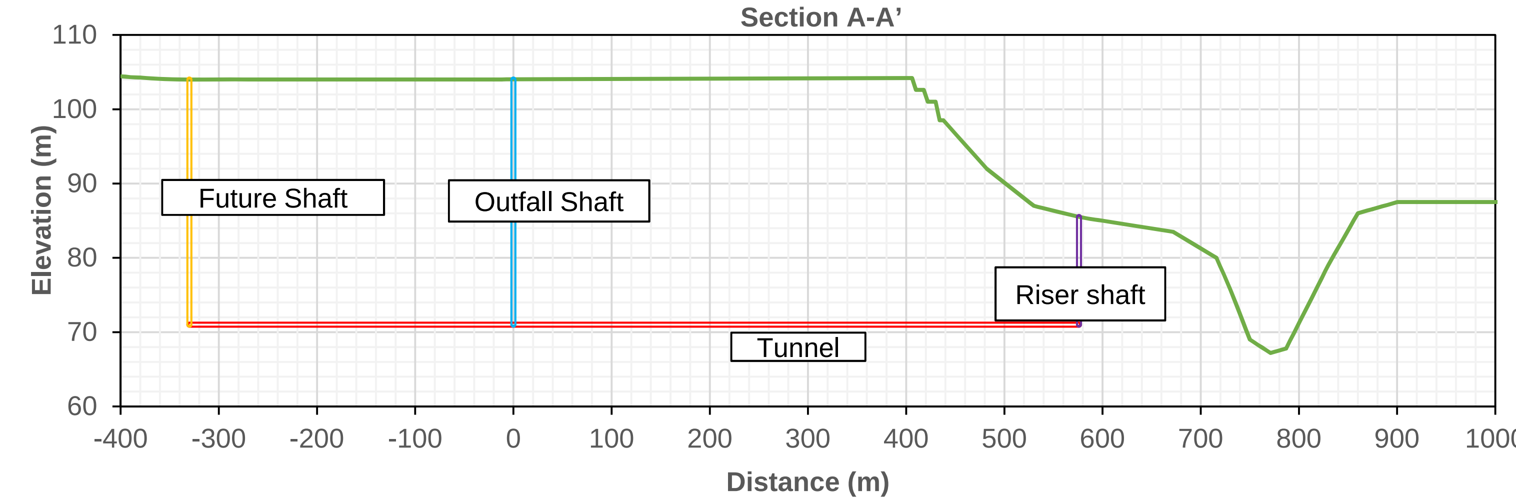
Case 3

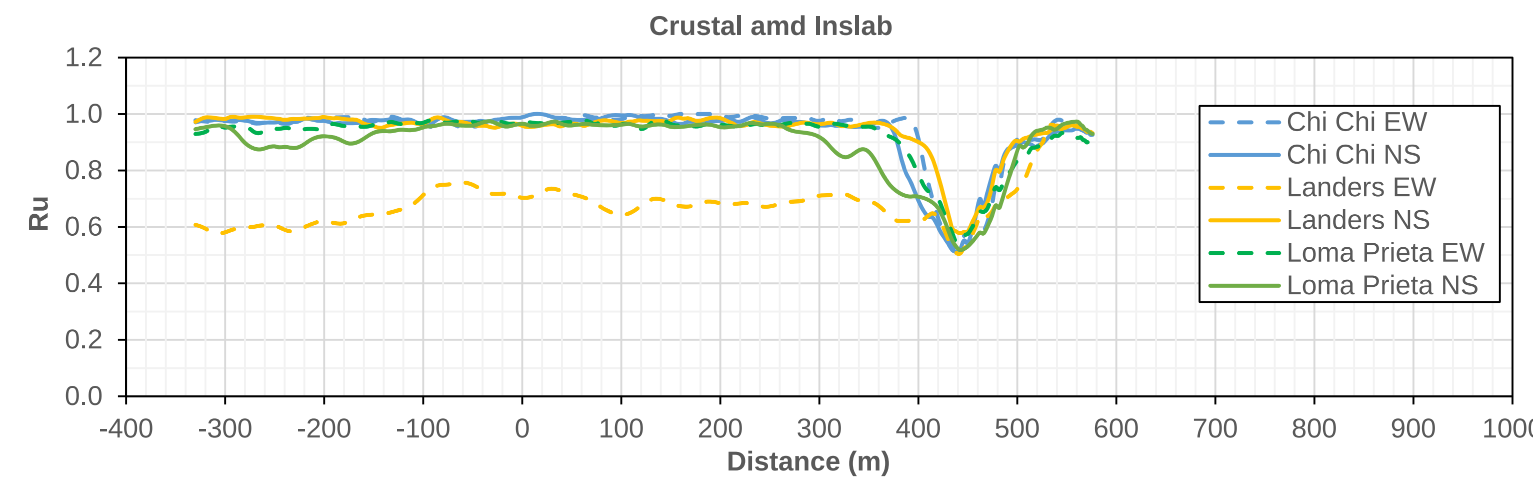


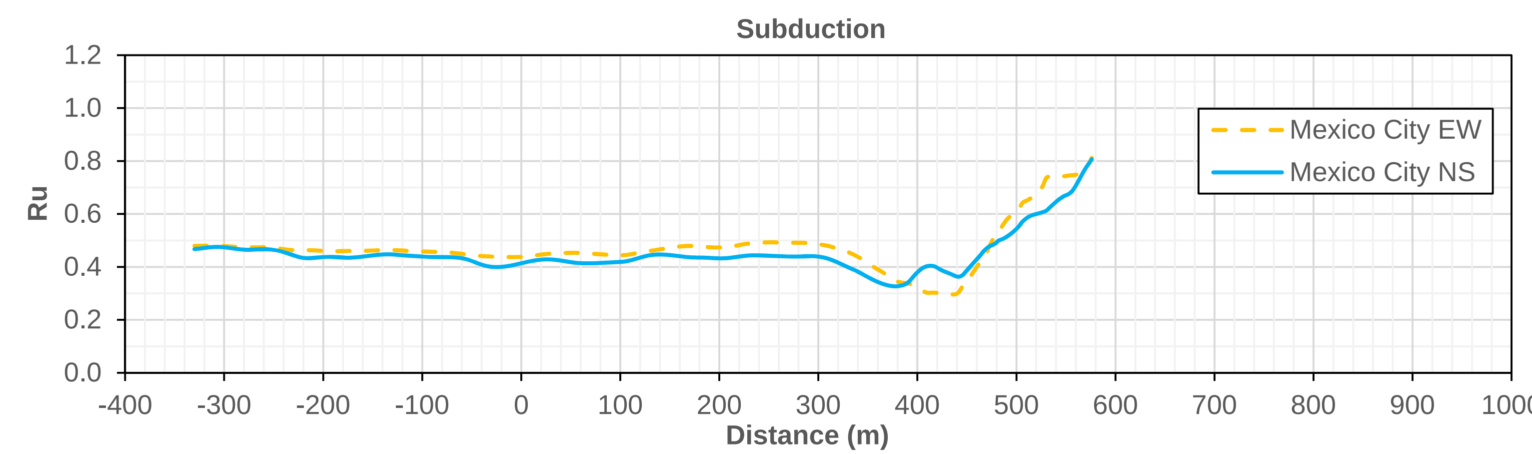




Case 4







Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT
CDM SMITH CANADA ULC

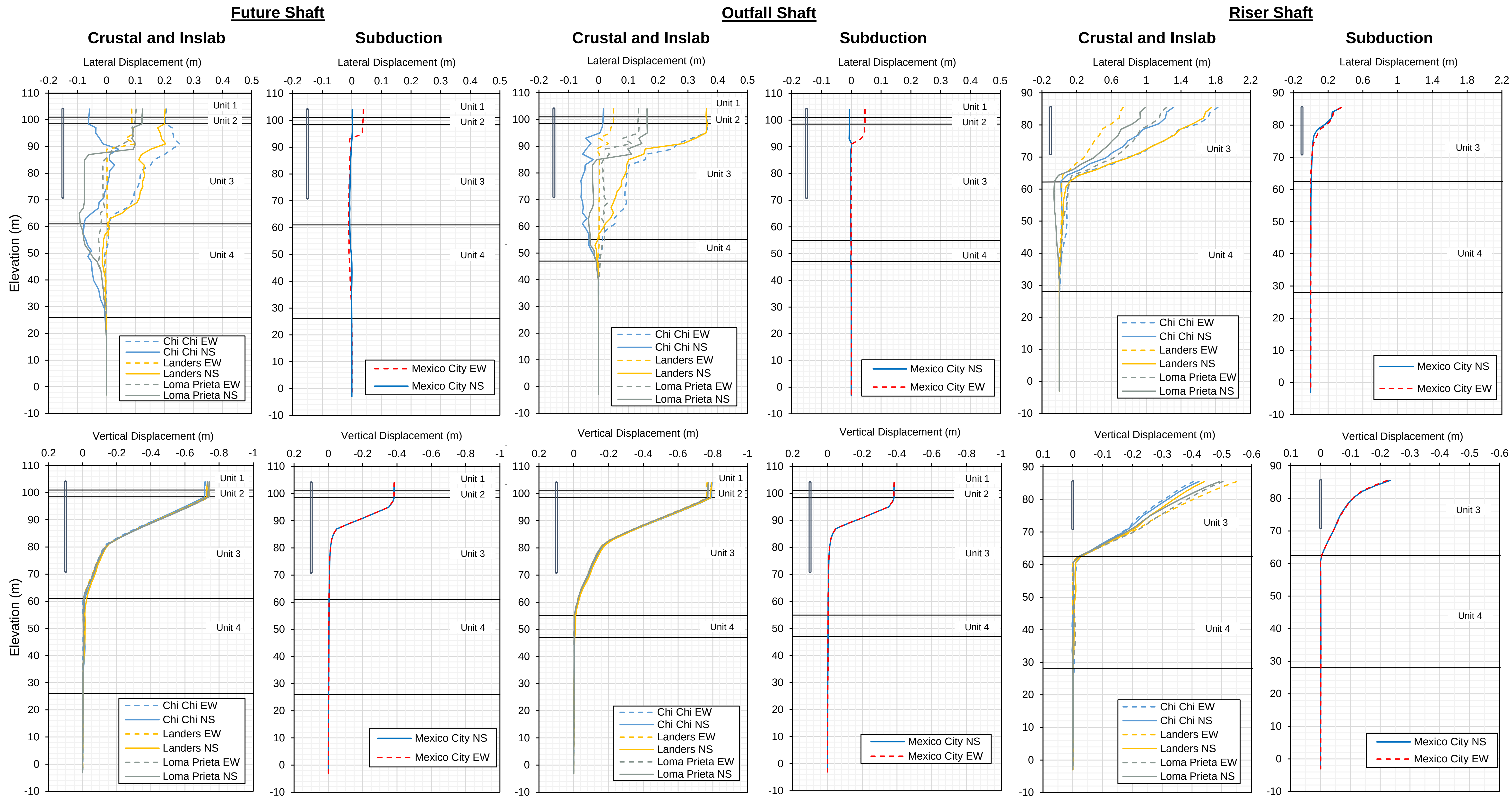
CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Extent of Liquefaction
Tunnel Level – Cases 3 and 4

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-19

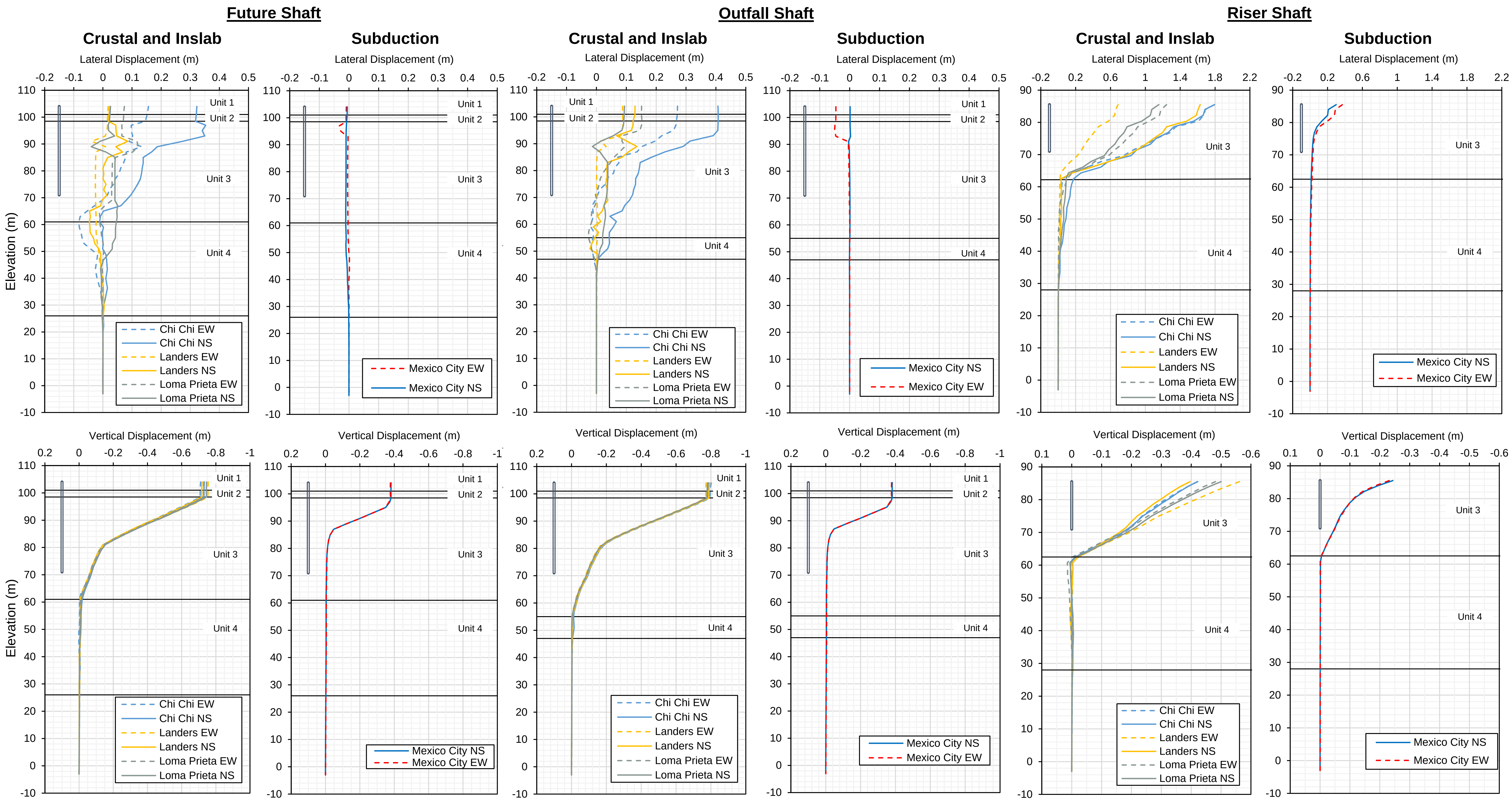


Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT	CDM SMITH CANADA ULC
PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F
TITLE	Horizontal and Vertical Displacement Profiles Shaft Locations - Case 1

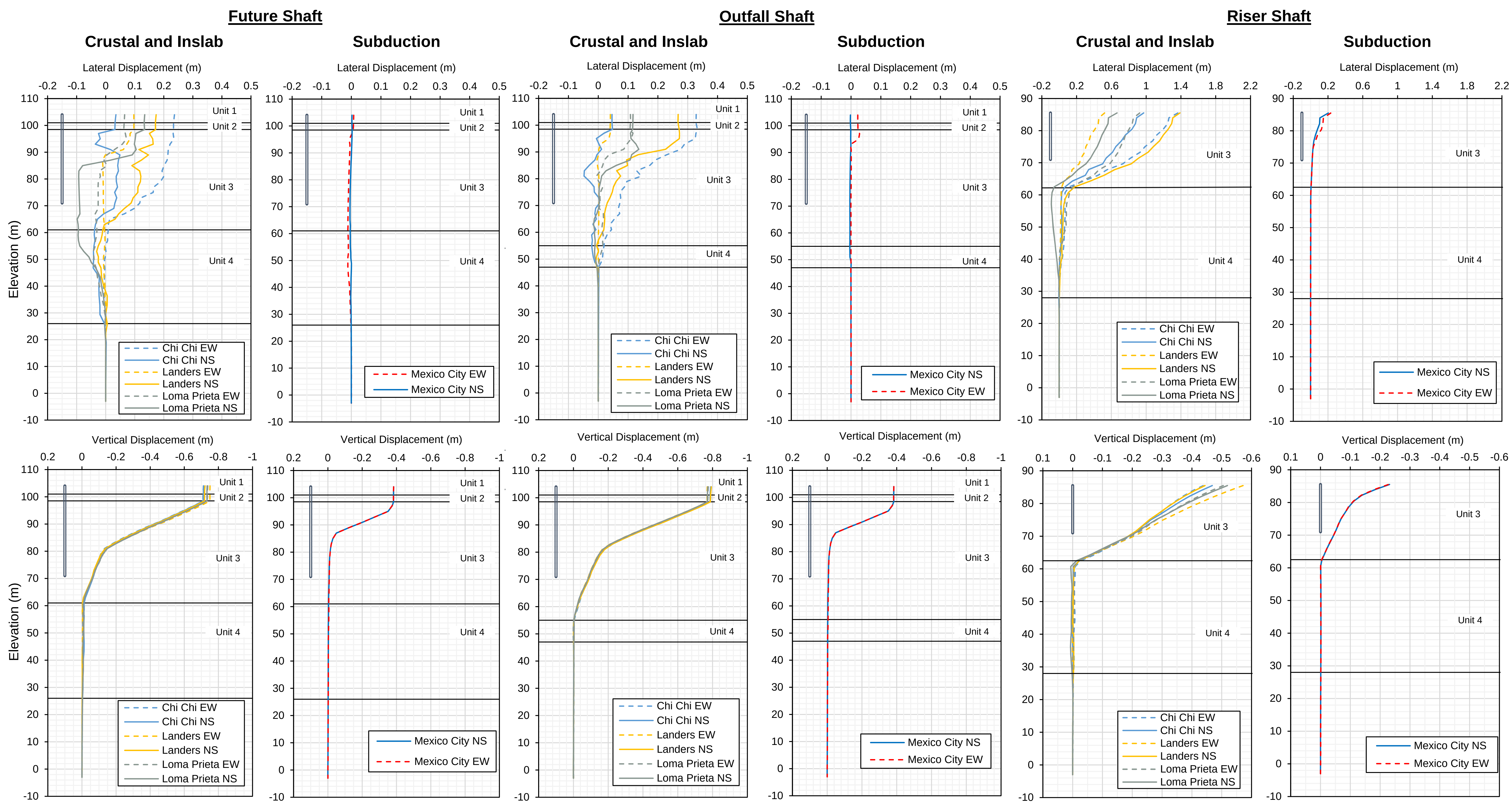
PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-20



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT	CDM SMITH CANADA ULC
PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F
TITLE	Horizontal and Vertical Displacement Profiles Shaft Locations - Case 2
PROJECT No.	1525010
Phase	2100
Rev	A
FIGURE	5-21



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

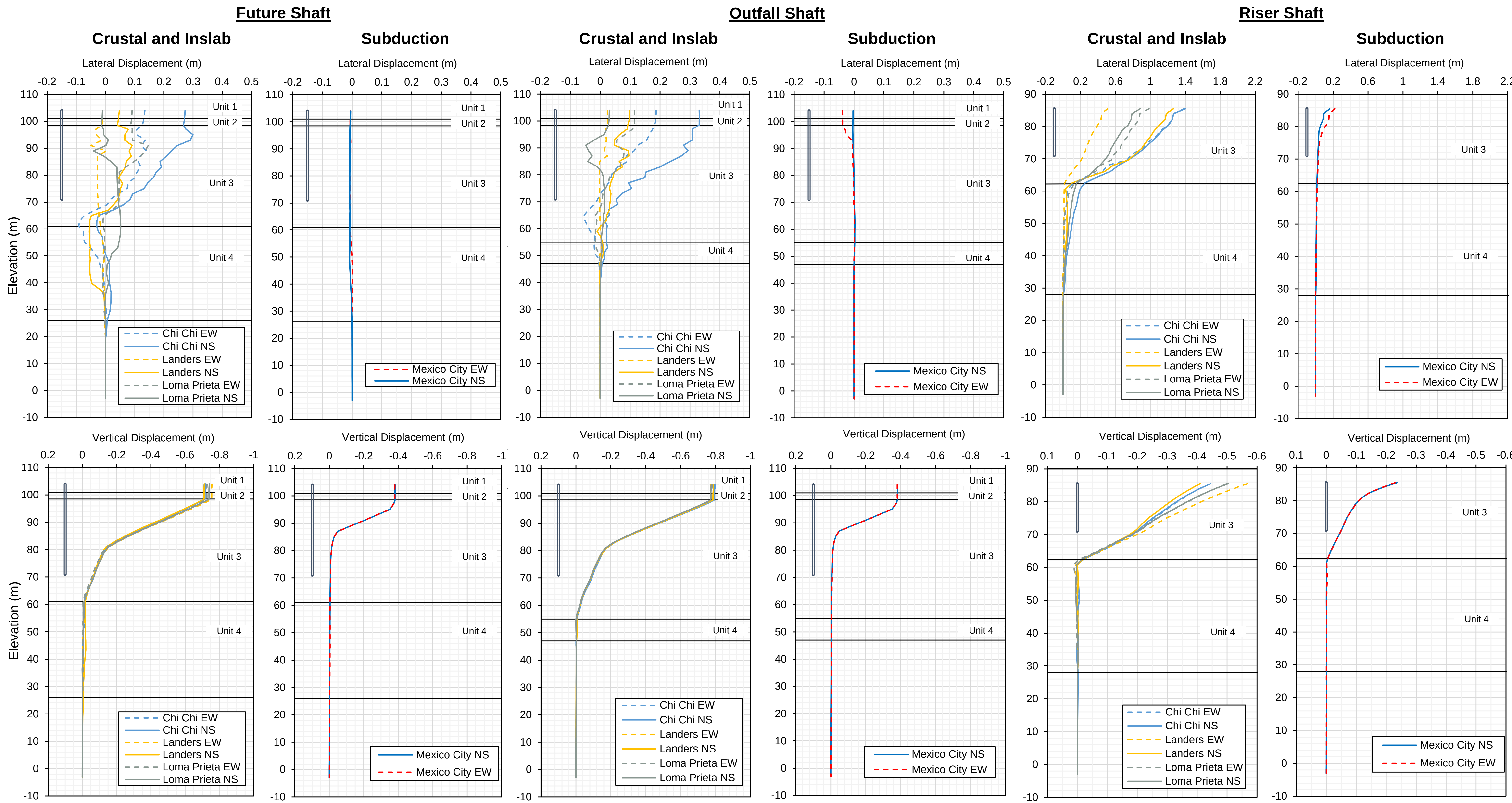
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Horizontal and Vertical Displacement Profiles
Shaft Locations - Case 3

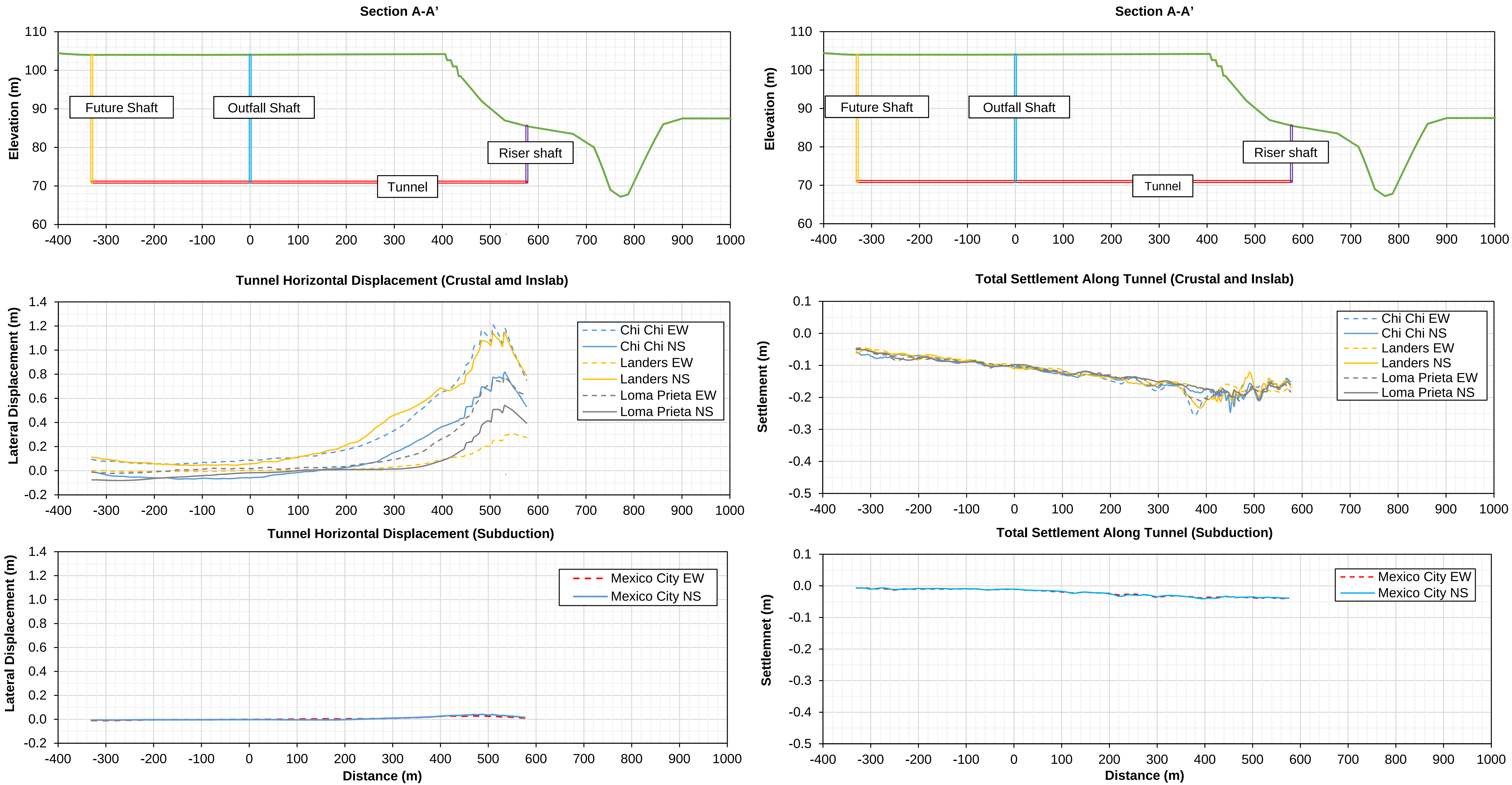
PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-22



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT **CDM SMITH CANADA ULC**	PROJECT **AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC**
CONSULTANT Golder Associates YYYY-MM-DD: 2017-11-30 PREPARED: M.S.K DESIGN: M.S.K REVIEW: Y.B APPROVED: V.F	TITLE **Horizontal and Vertical Displacement Profiles Shaft Locations - Case 4**
	PROJECT No.: 1525010 Phase: 2100 Rev: A FIGURE: **5-23**



CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

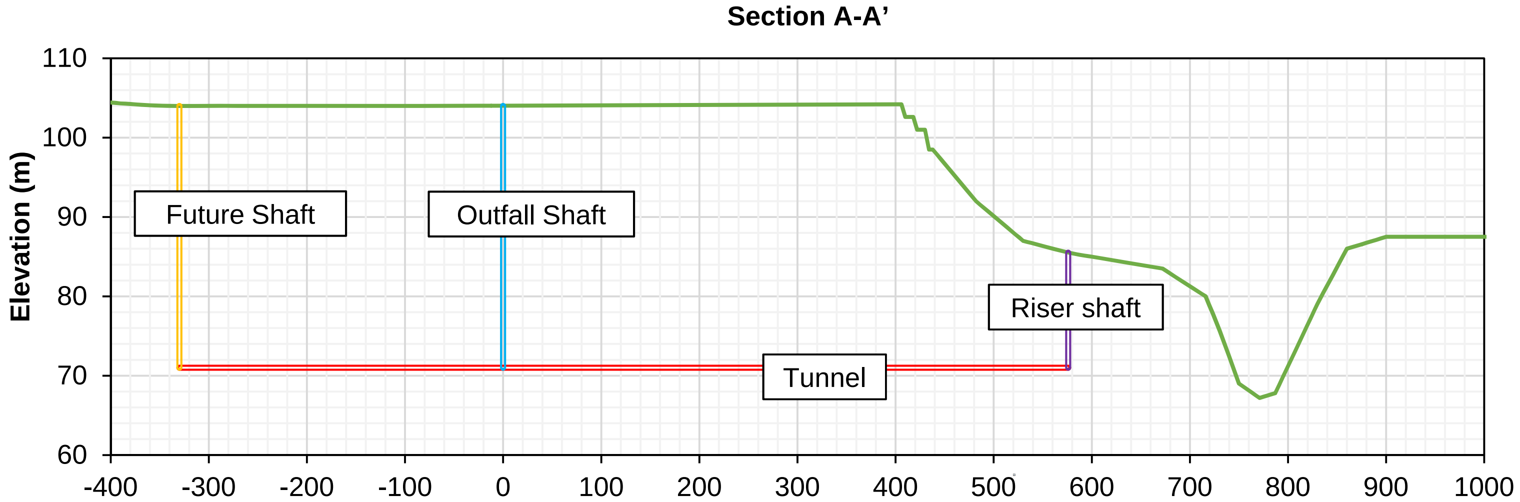
CONSULTANT

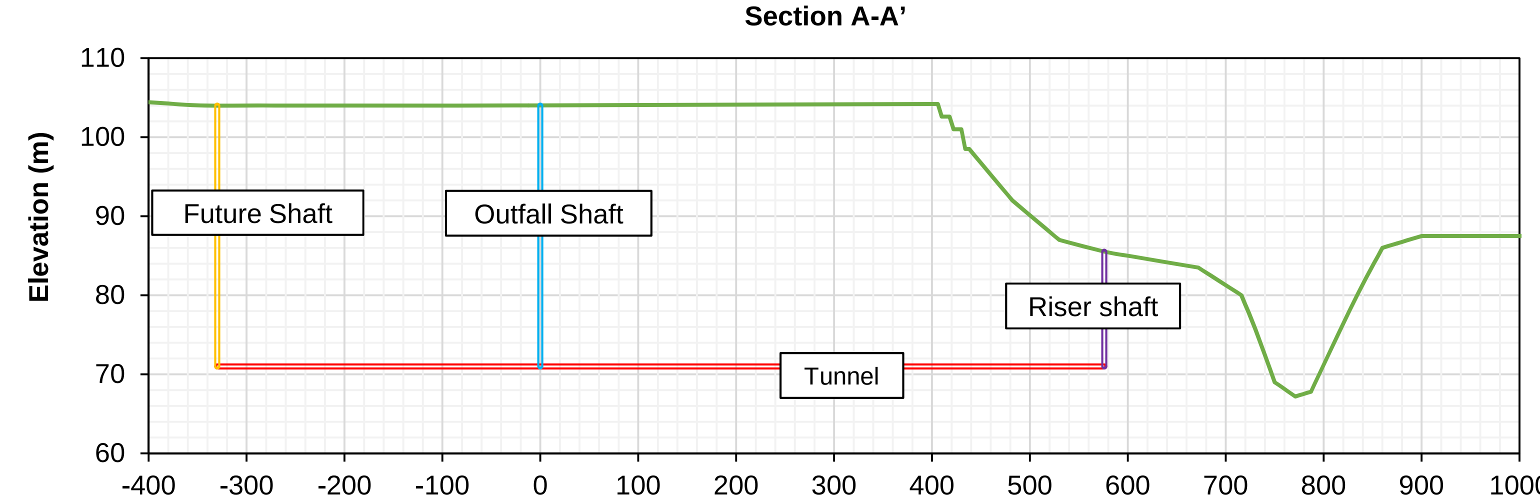
Golder Associates

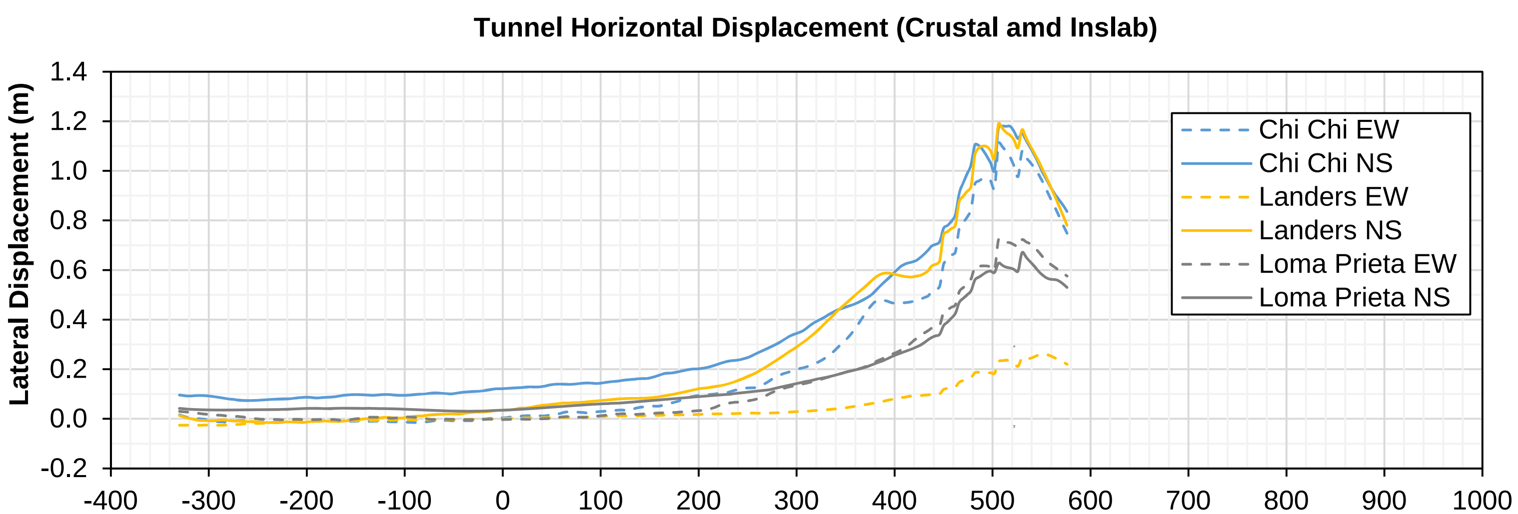
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

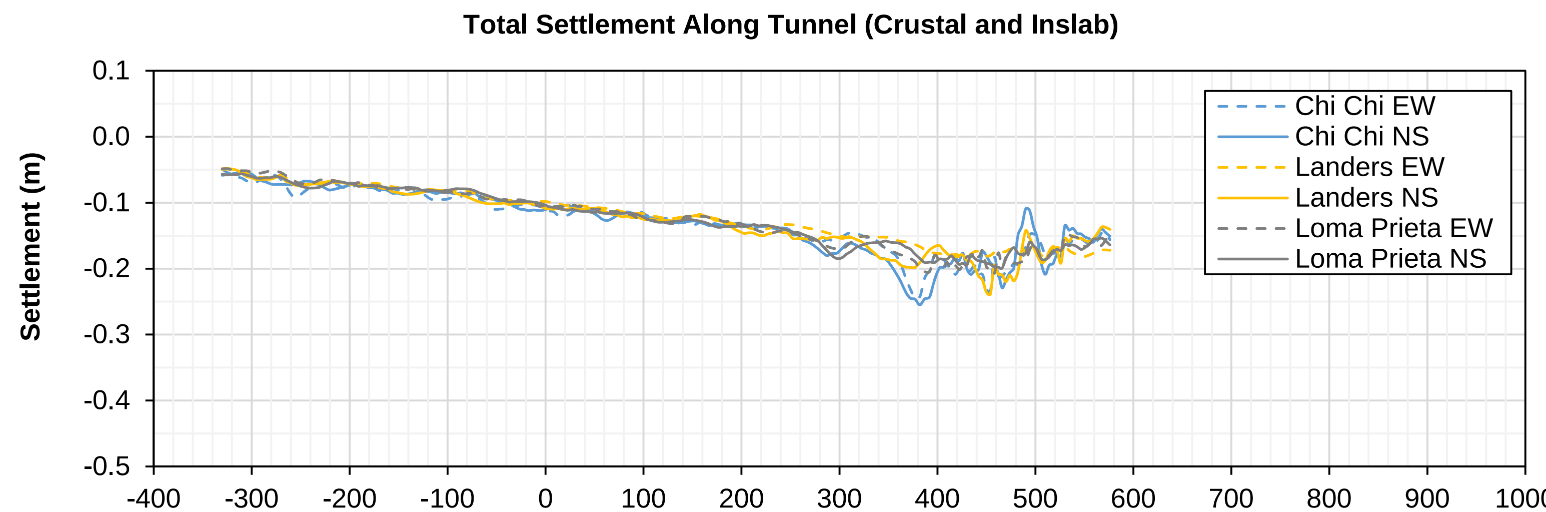
TITLE
Horizontal and Vertical Displacement Profiles
Tunnel Level- Case 1

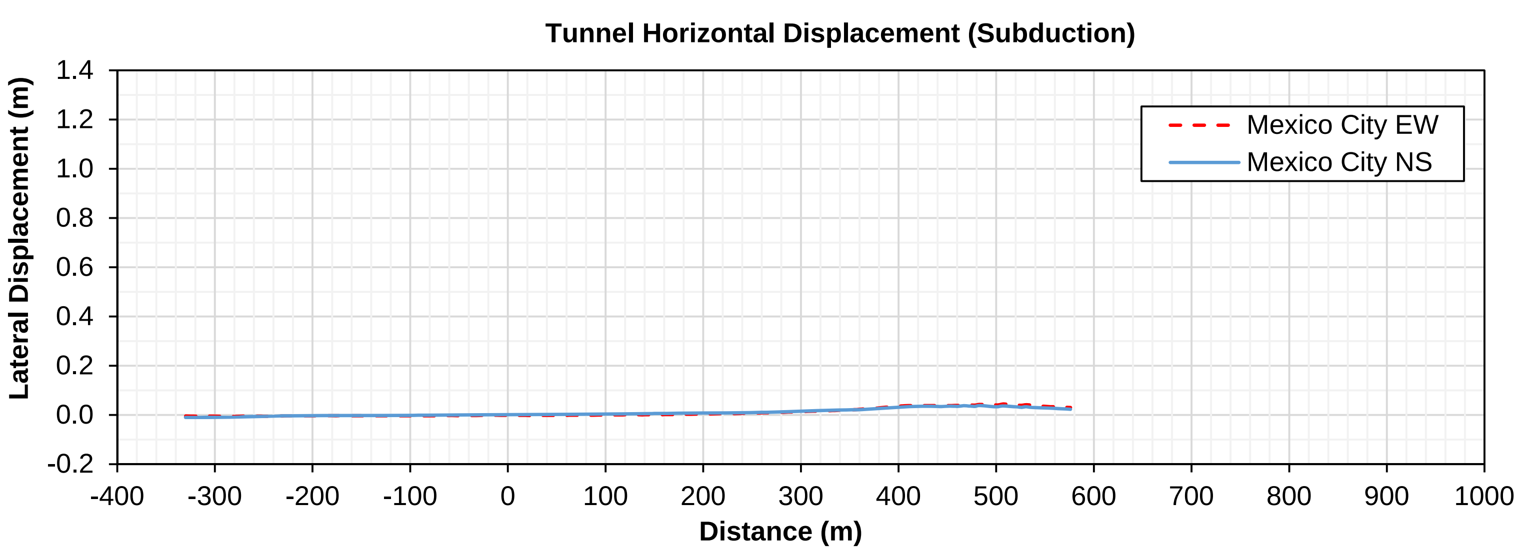
PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-24

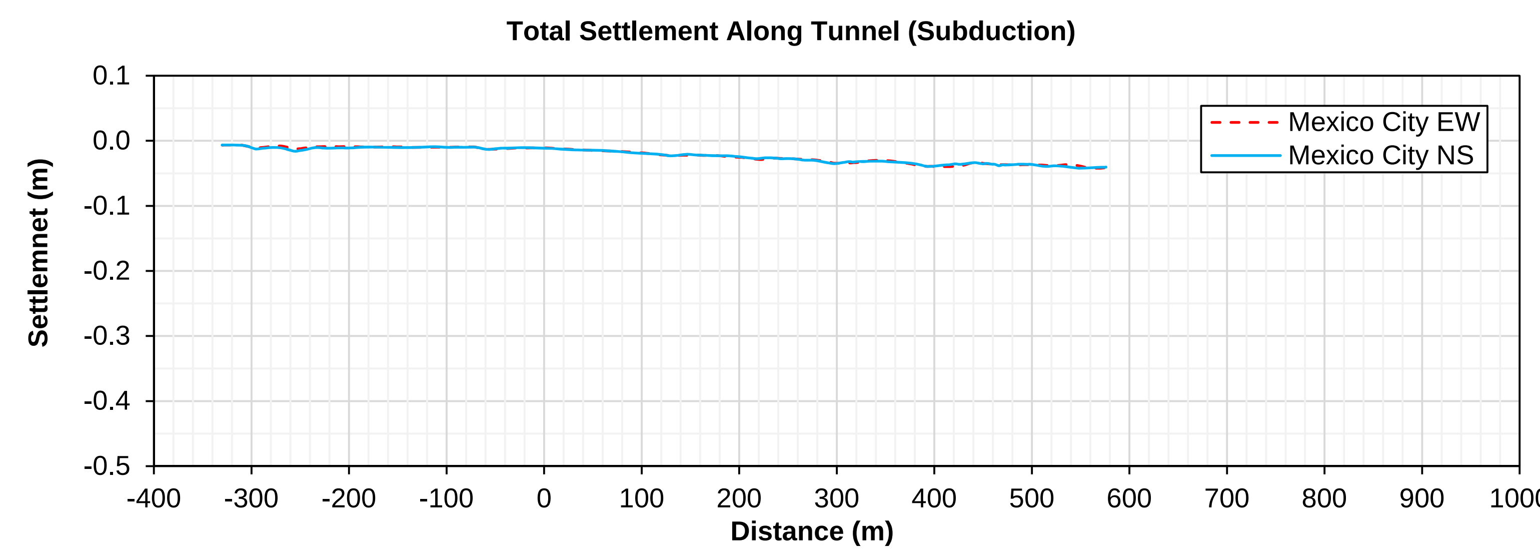












CLIENT
CDM SMITH CANADA ULC

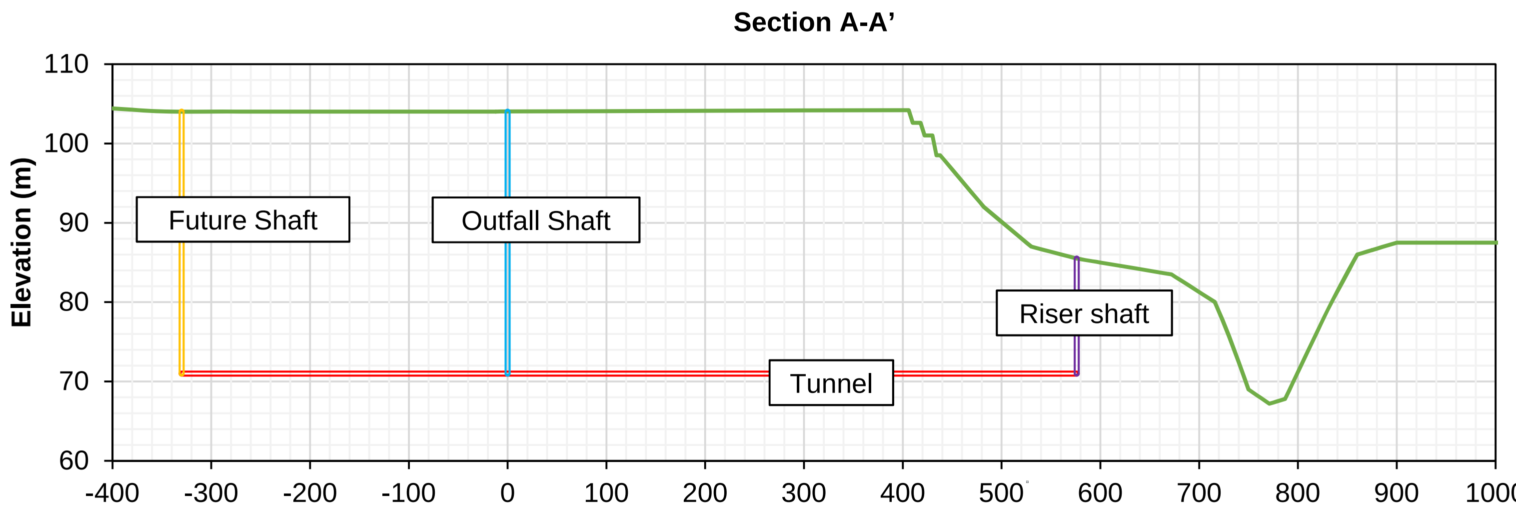
PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

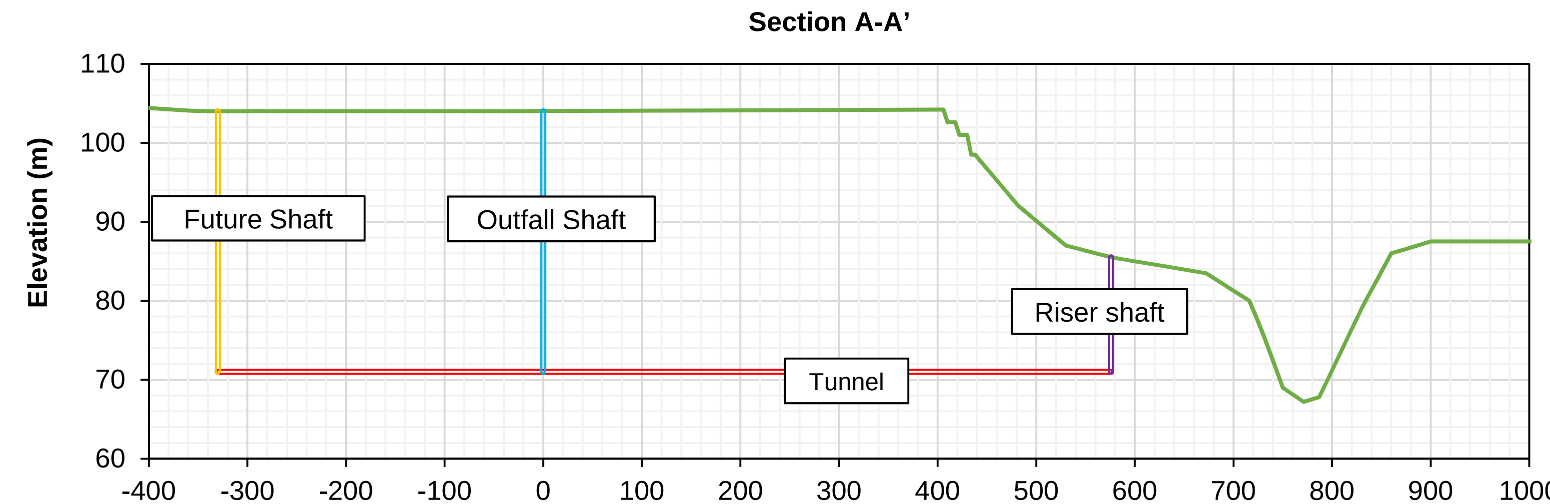
CONSULTANT
Golder Associates

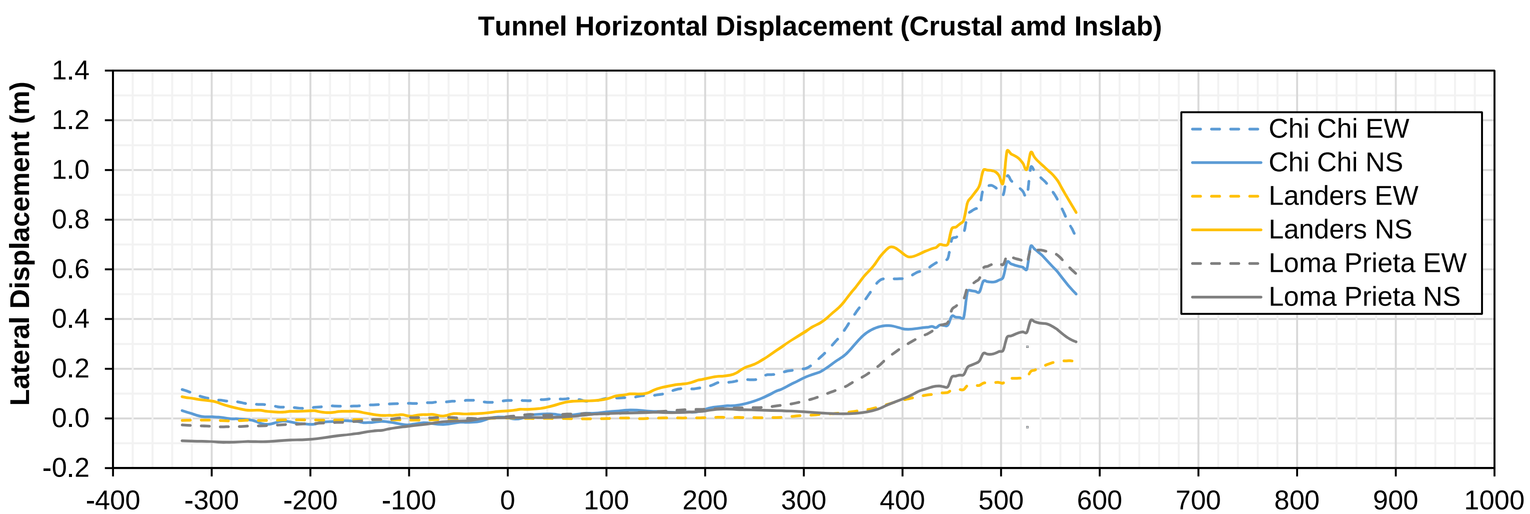
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

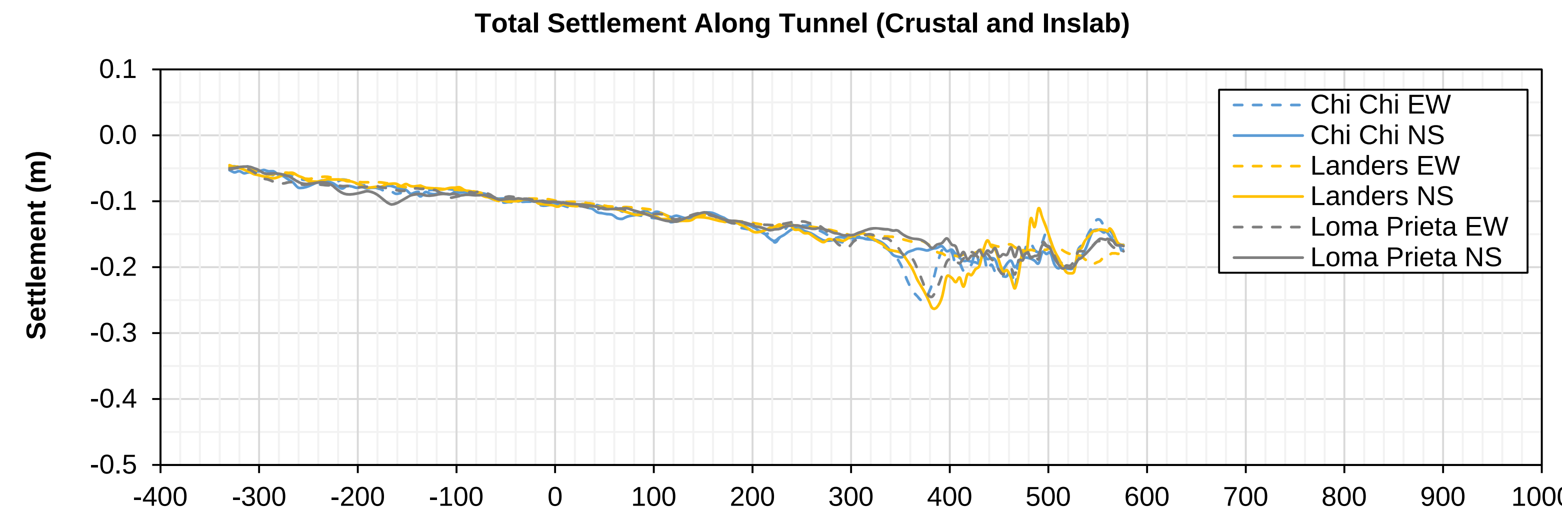
TITLE
Horizontal and Vertical Displacement Profiles
Tunnel Level- Case 2

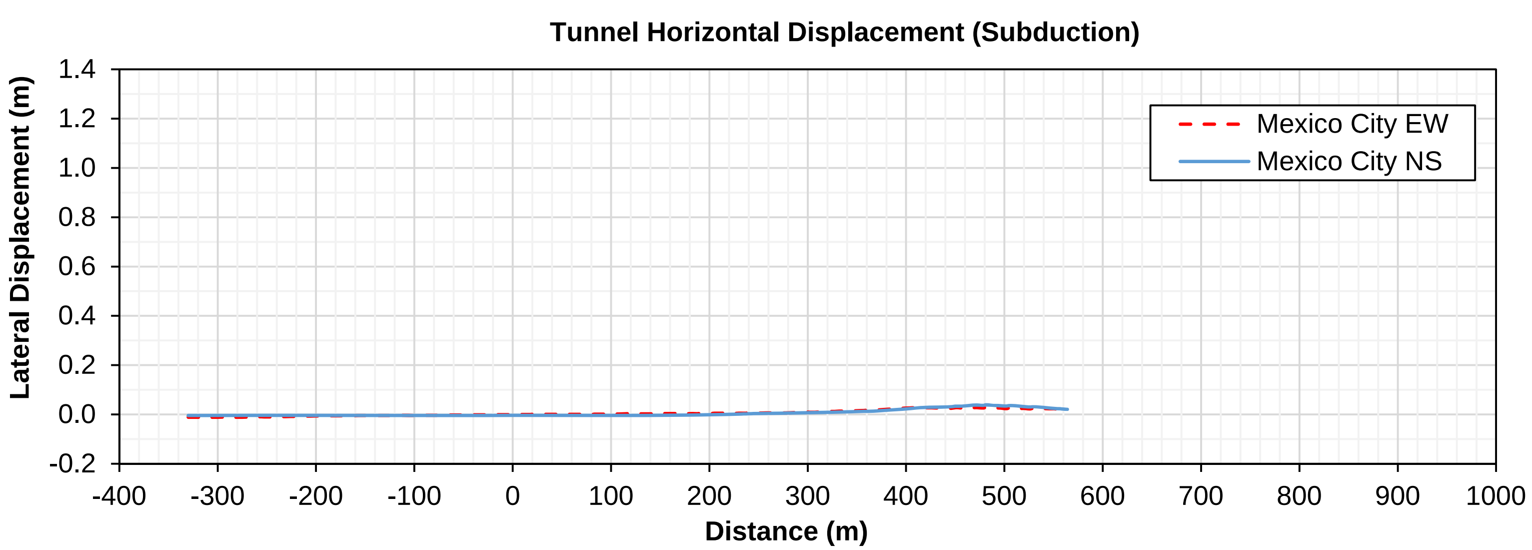
PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-25

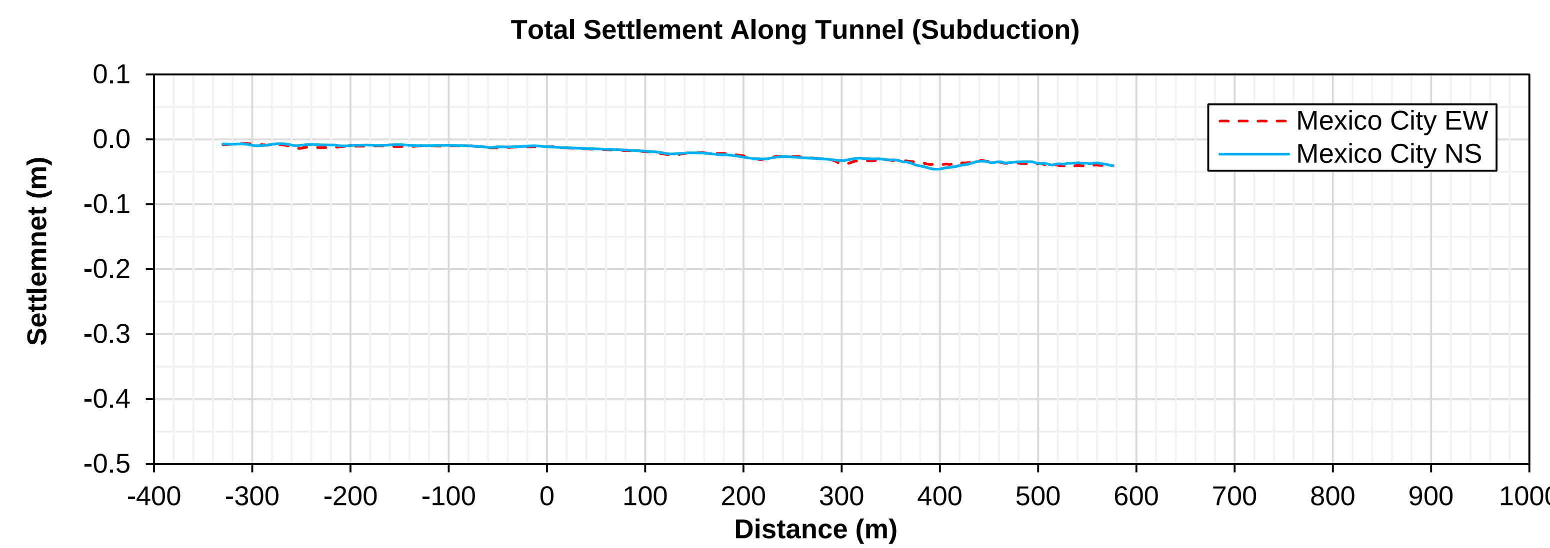




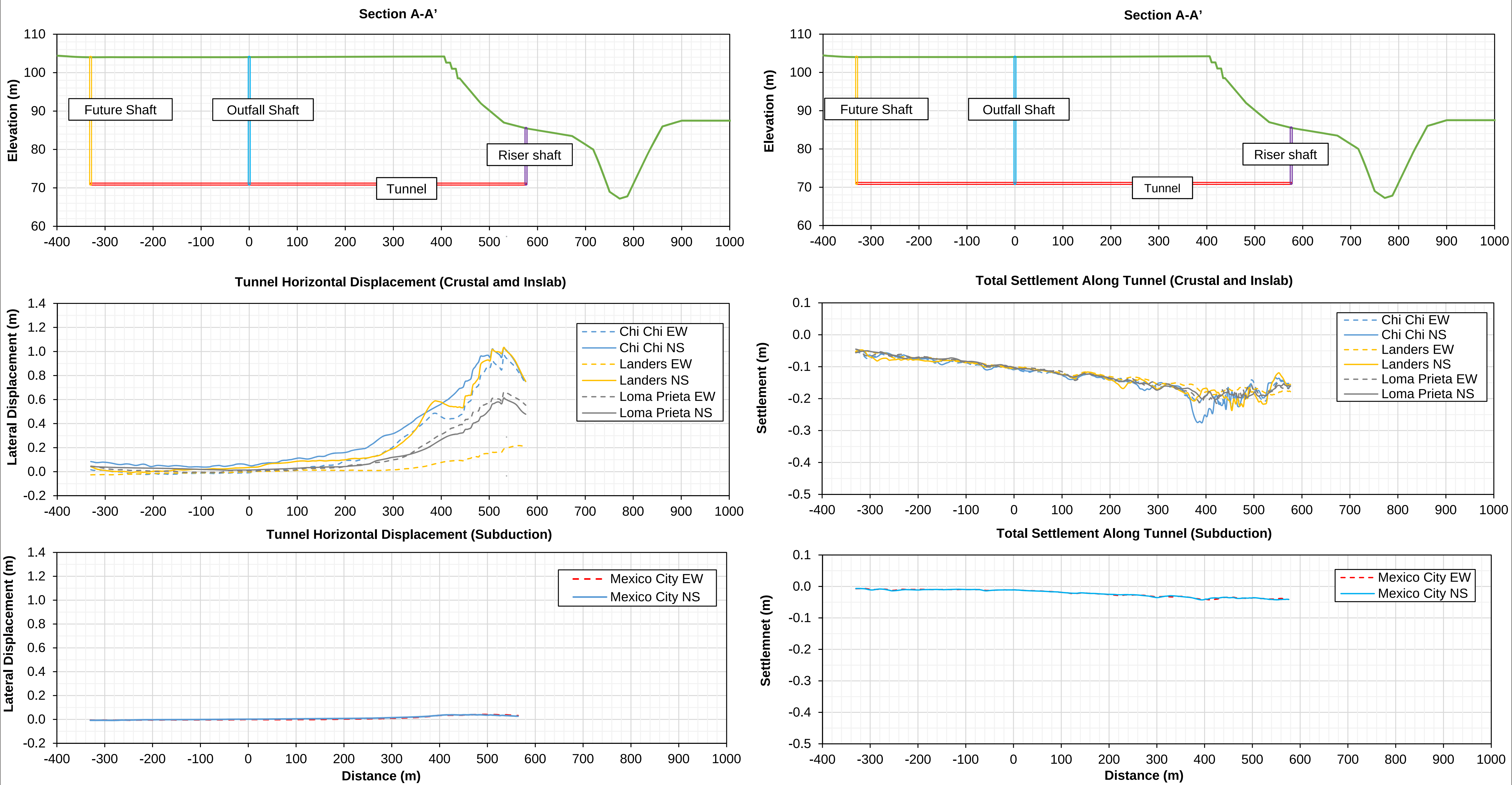






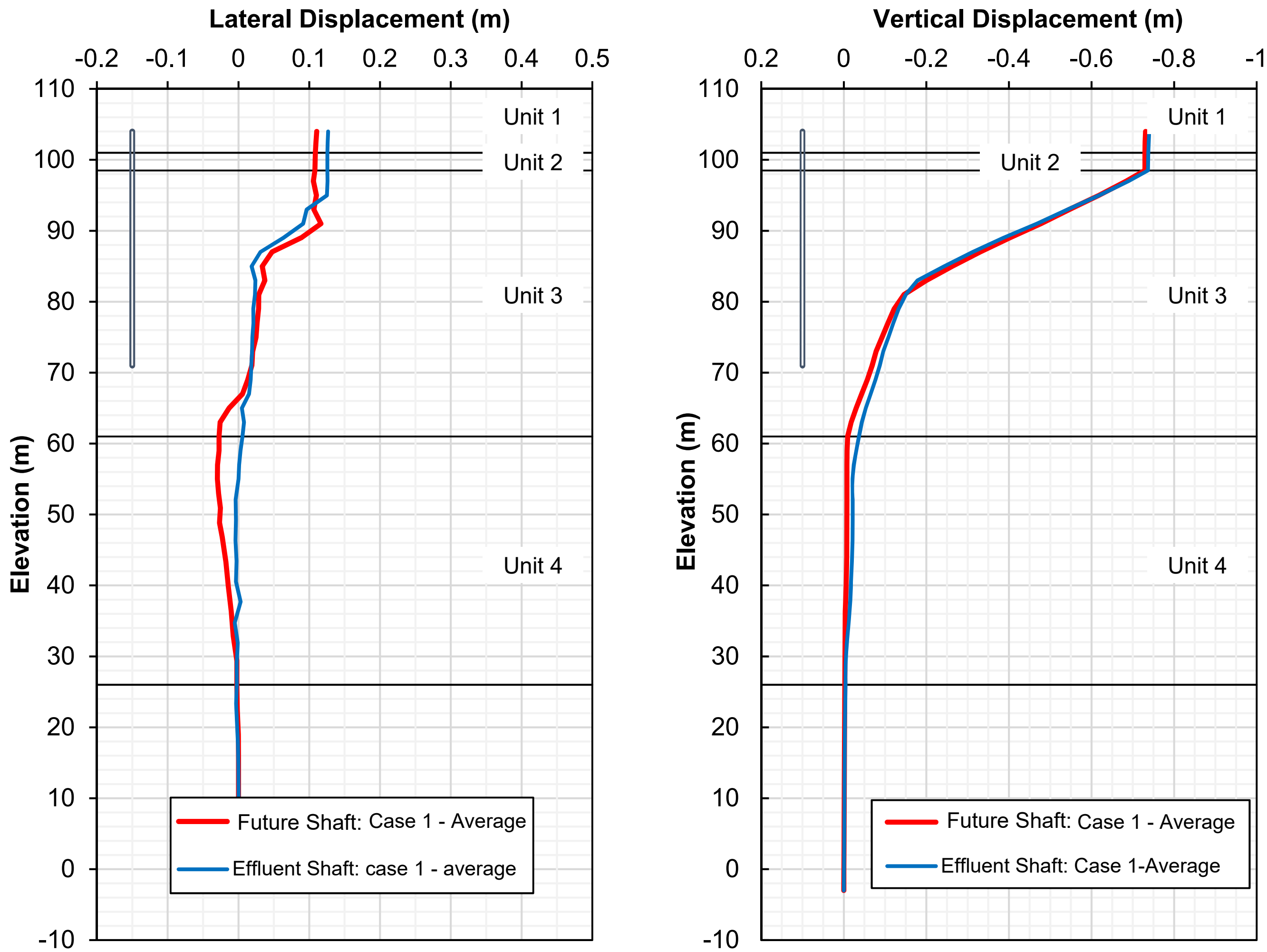


CLIENT CDM SMITH CANADA ULC	PROJECT AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT Golder Associates YYYY-MM-DD 2017-11-30 PREPARED M.S.K DESIGN M.S.K REVIEW Y.B APPROVED V.F	TITLE **Horizontal and Vertical Displacement Profiles** **Tunnel Level- Case 3** PROJECT No. 1525010 · Phase 2100 · Rev A · FIGURE 5-26



CLIENT **CDM SMITH CANADA ULC**	PROJECT **AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC**
CONSULTANT Golder Associates YYYY-MM-DD 2017-11-30 PREPARED M.S.K DESIGN M.S.K REVIEW Y.B APPROVED V.F	TITLE **Horizontal and Vertical Displacement Profiles Tunnel Level- Case 4** PROJECT No. 1525010 · Phase 2100 · Rev A · FIGURE **5-27**

Path: document1 | FileName: DOCUMENT1



CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

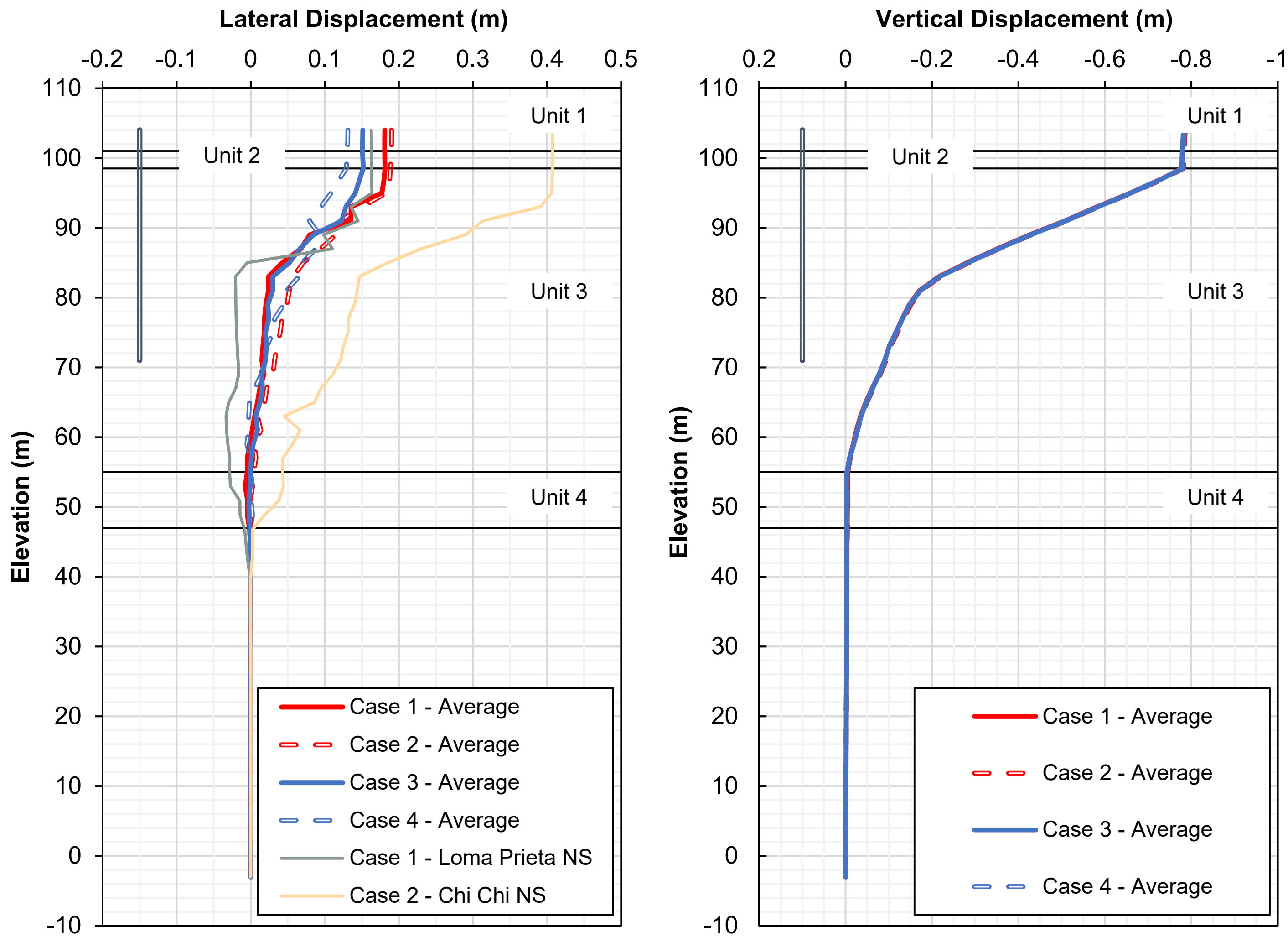
CONSULTANT

Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Average Displacement Profiles – Case 1
Future and Effluent Shaft Locations

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	**5-28**



Note:

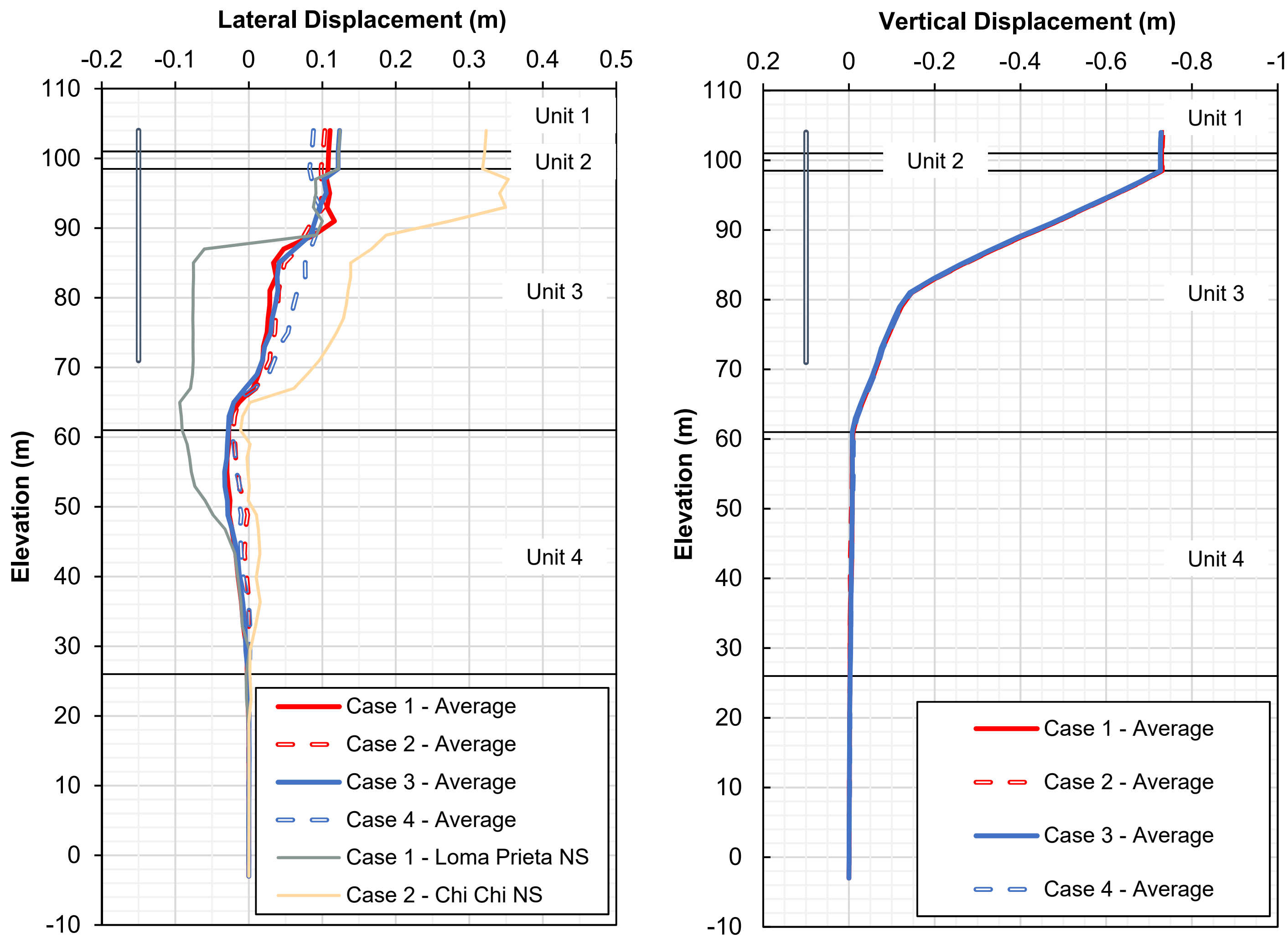
Shown in the plots illustrates the embedment of the shaft

CLIENT	CDM SMITH CANADA ULC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

TITLE
Lateral and Vertical Displacement Profiles Outfall Shaft Location

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-29



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT	CDM SMITH CANADA ULC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

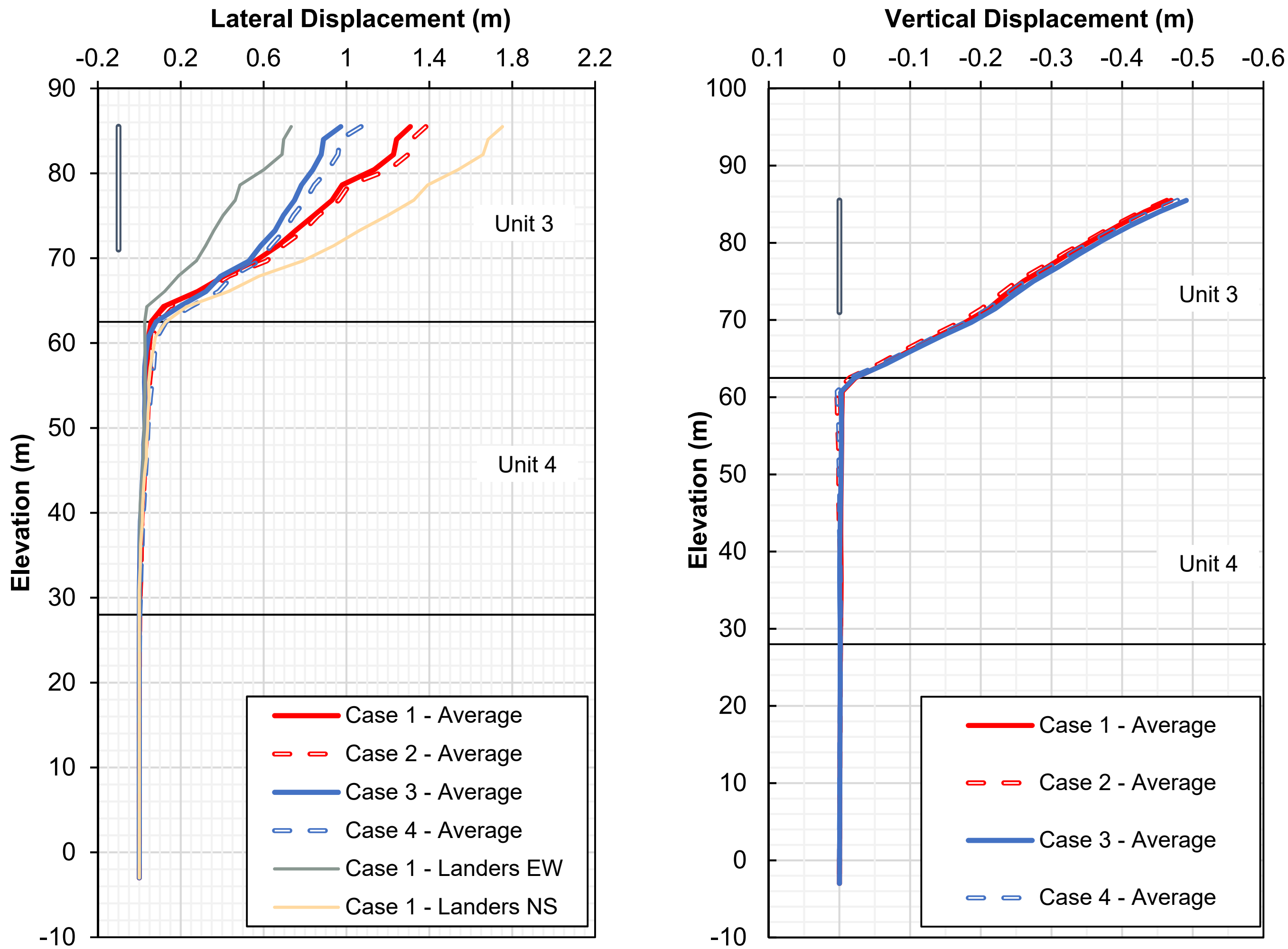
PROJECT

AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE

Lateral and Vertical Displacement Profiles
Future/Effluent Shaft Locations

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-30



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

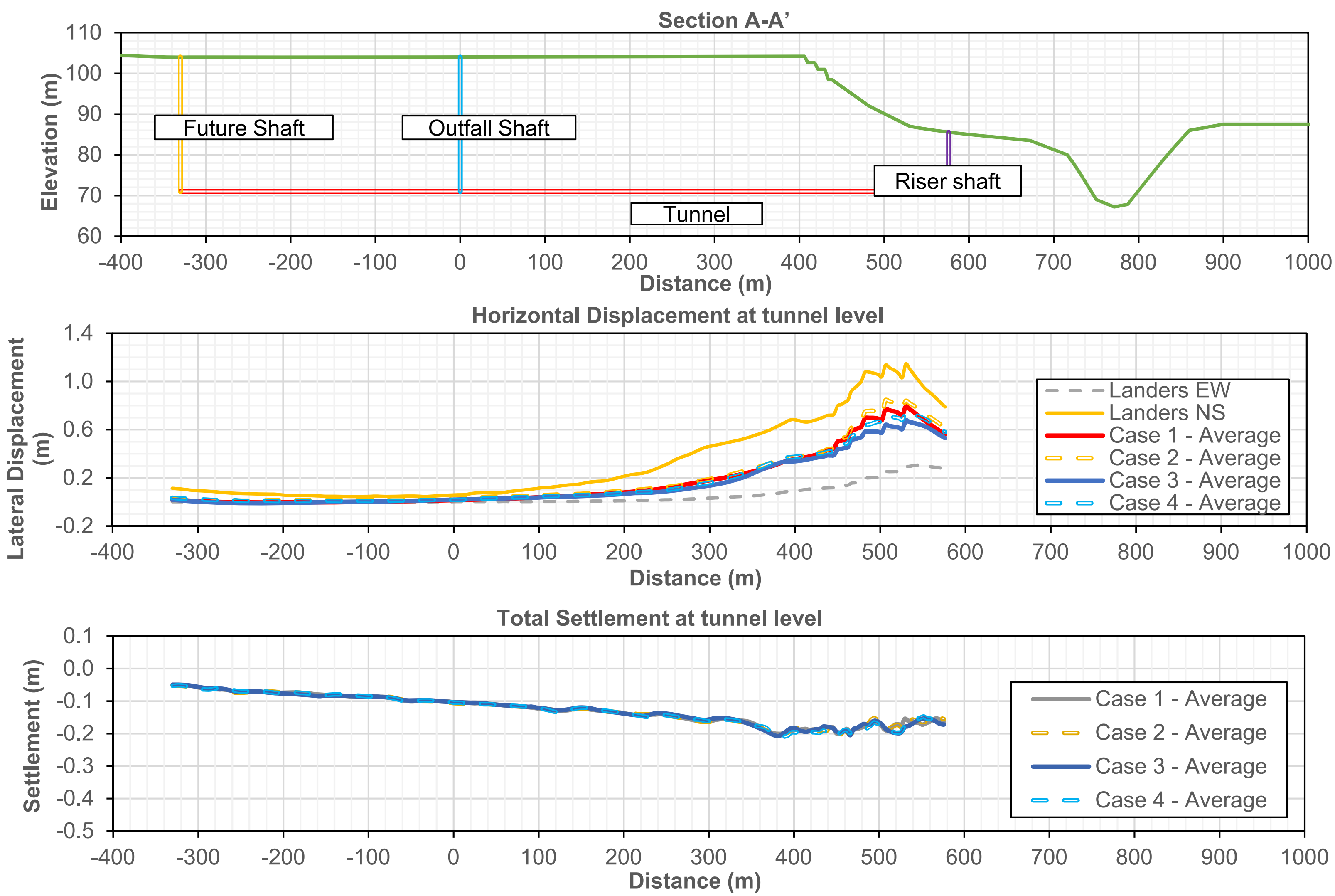
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

TITLE
Lateral and Vertical Displacement Profiles Riser Shaft Location

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	**5-31**

Path: document1 | FileName: DOCUMENT1



CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

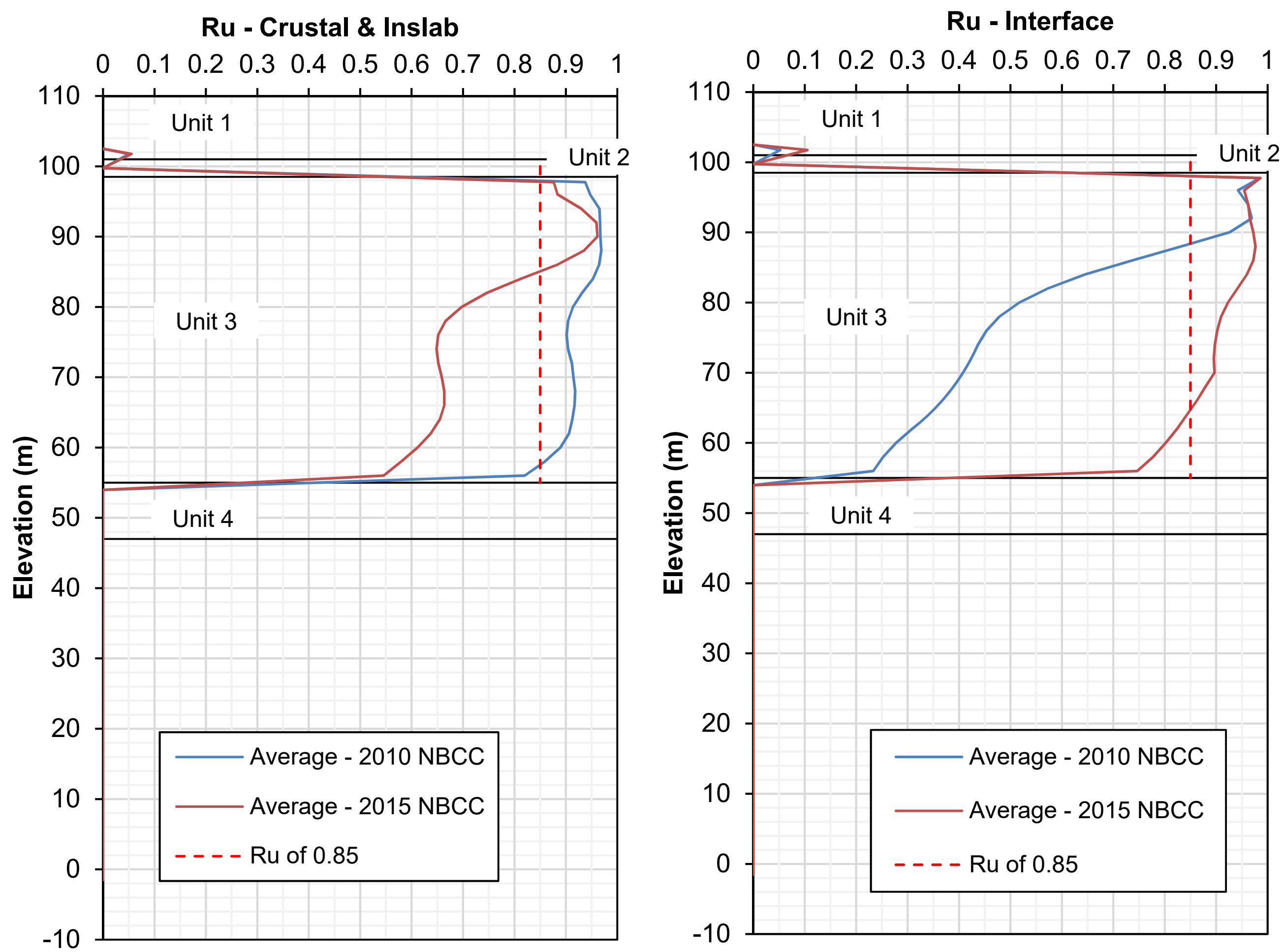
CONSULTANT

Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Lateral and Vertical Displacement Profiles
Tunnel Level

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	**5-32**



Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT
CDM SMITH CANADA ULC

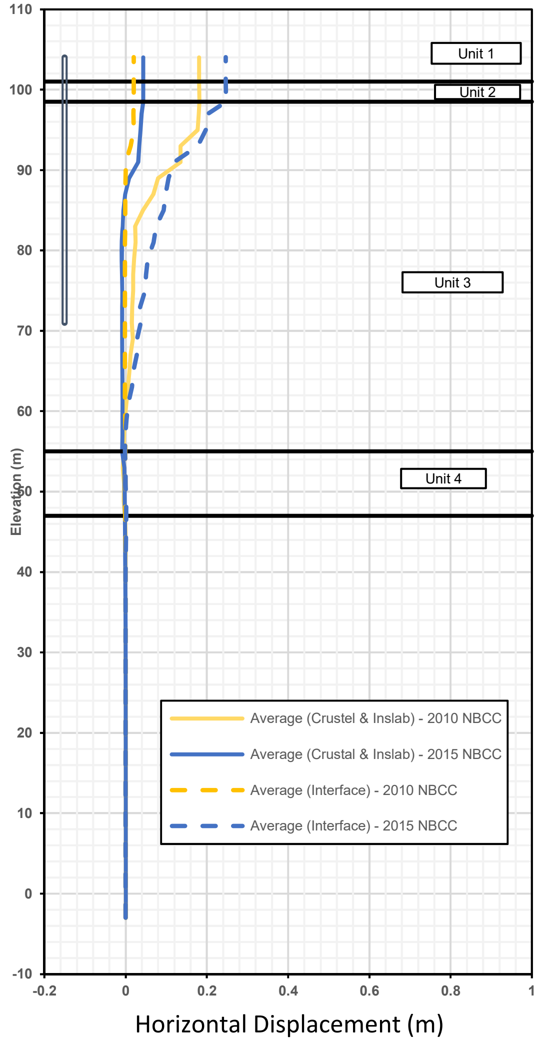
CONSULTANT
Golder Associates

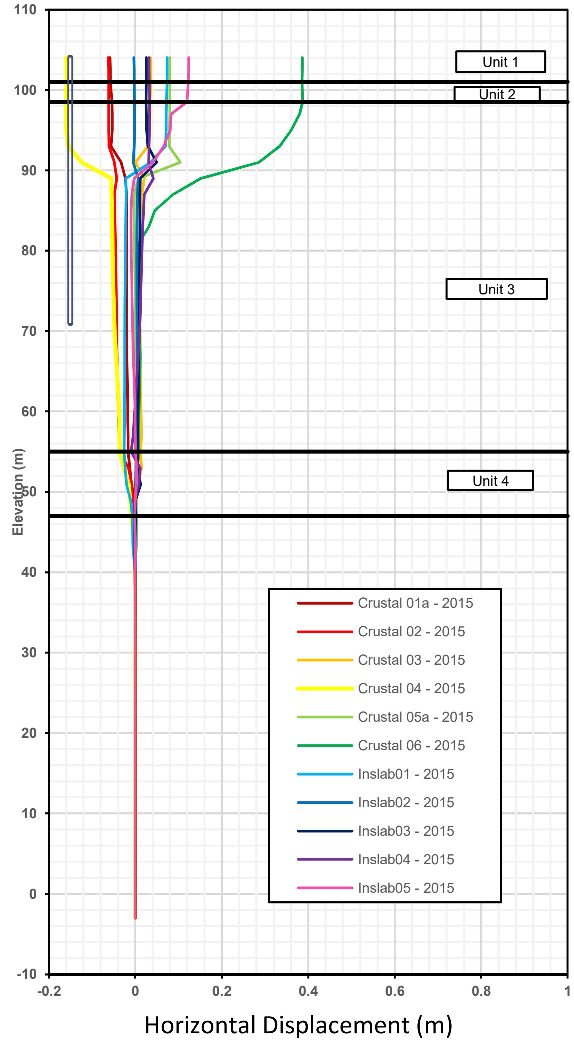
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

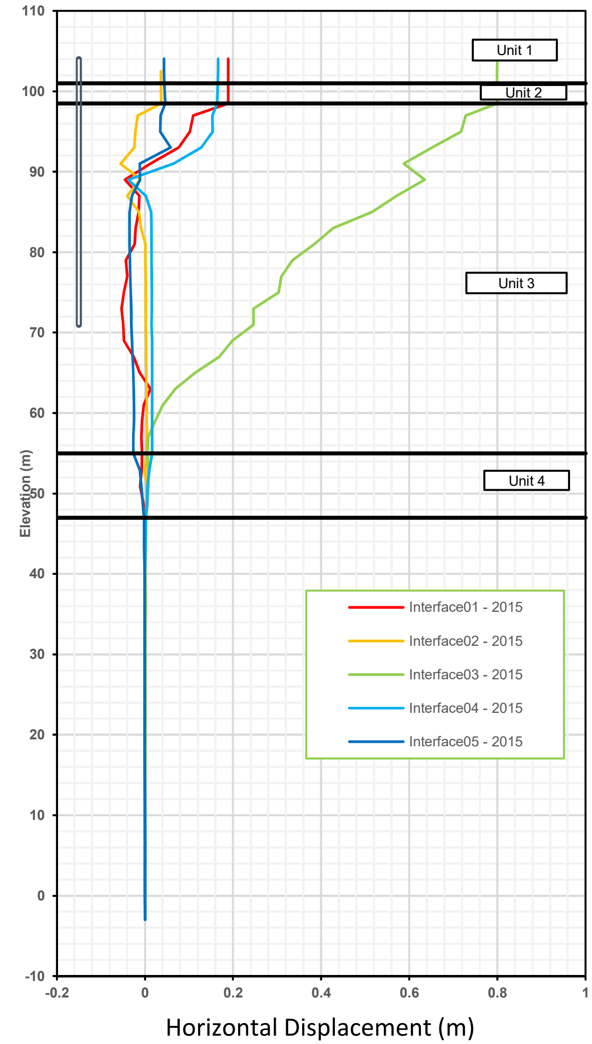
PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Extent of Liquefaction
Outfall Shaft Location

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-33







Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

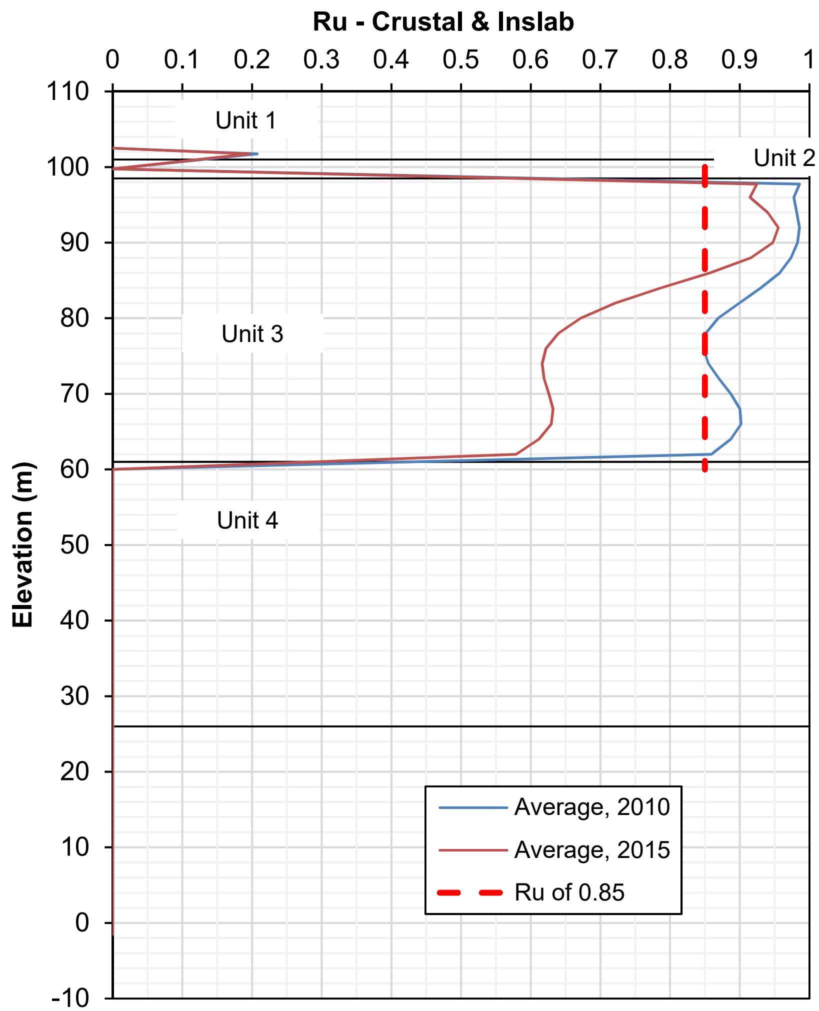
Golder Associates

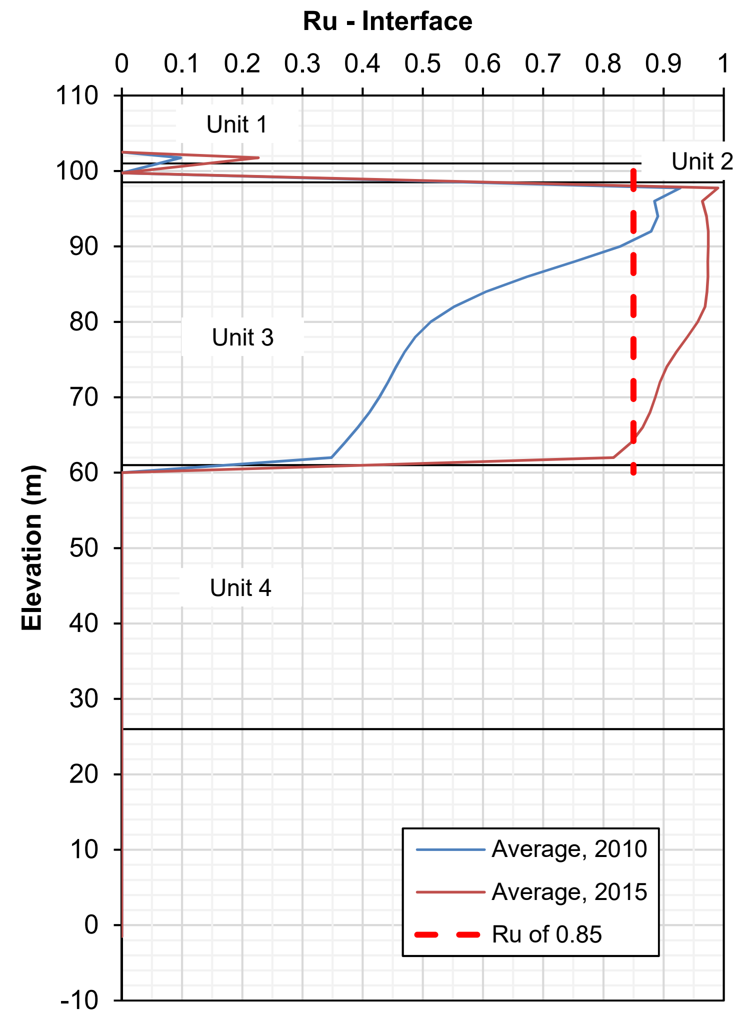
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

TITLE
Horizontal Displacement Profiles Outfall Shaft Location

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-34





Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT
CDM SMITH CANADA ULC

CONSULTANT
Golder Associates

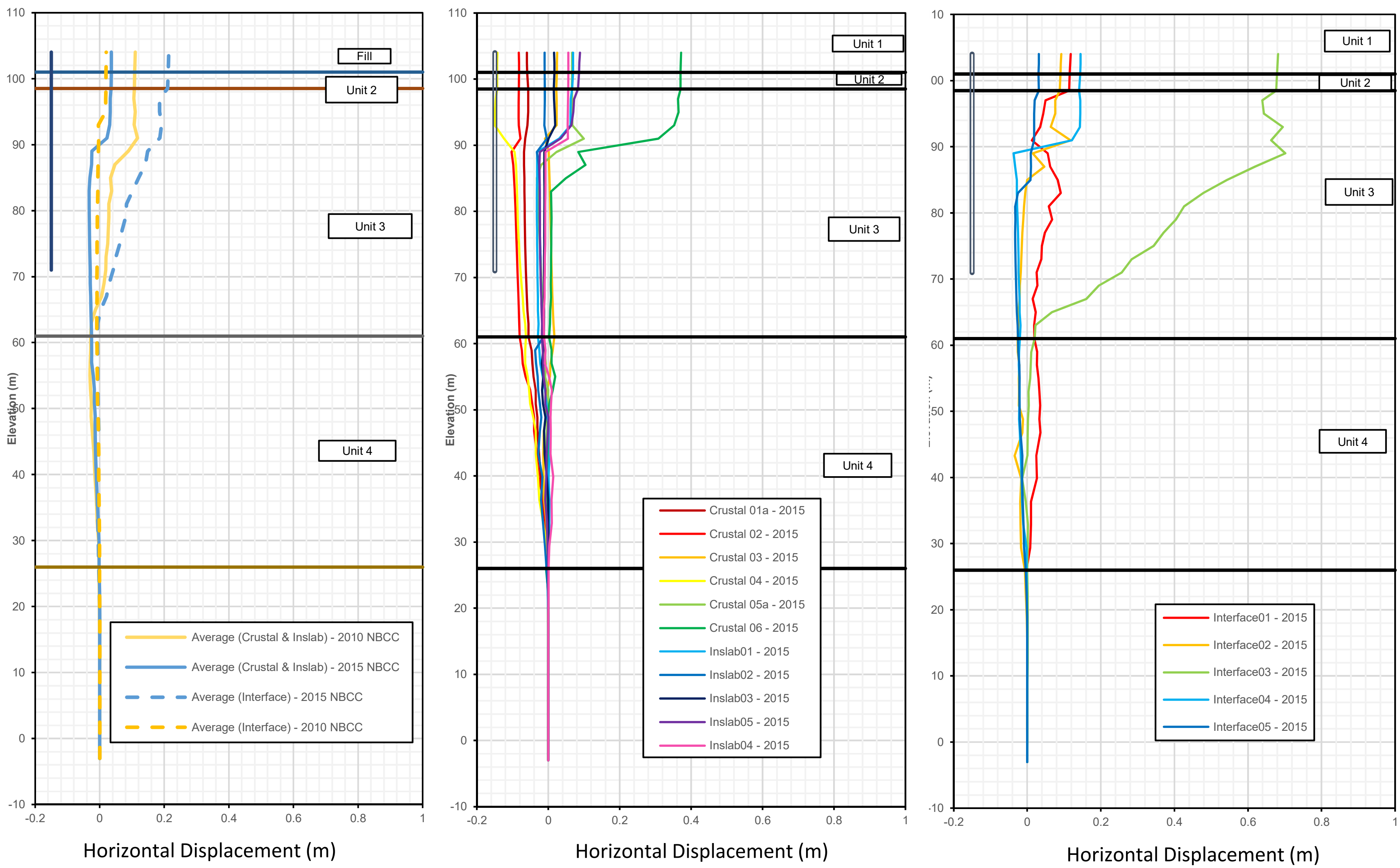
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

TITLE
Extent of Liquefaction Future/Effluent Shaft Locations

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-35

Path: document1 | FileName: DOCUMENT1



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

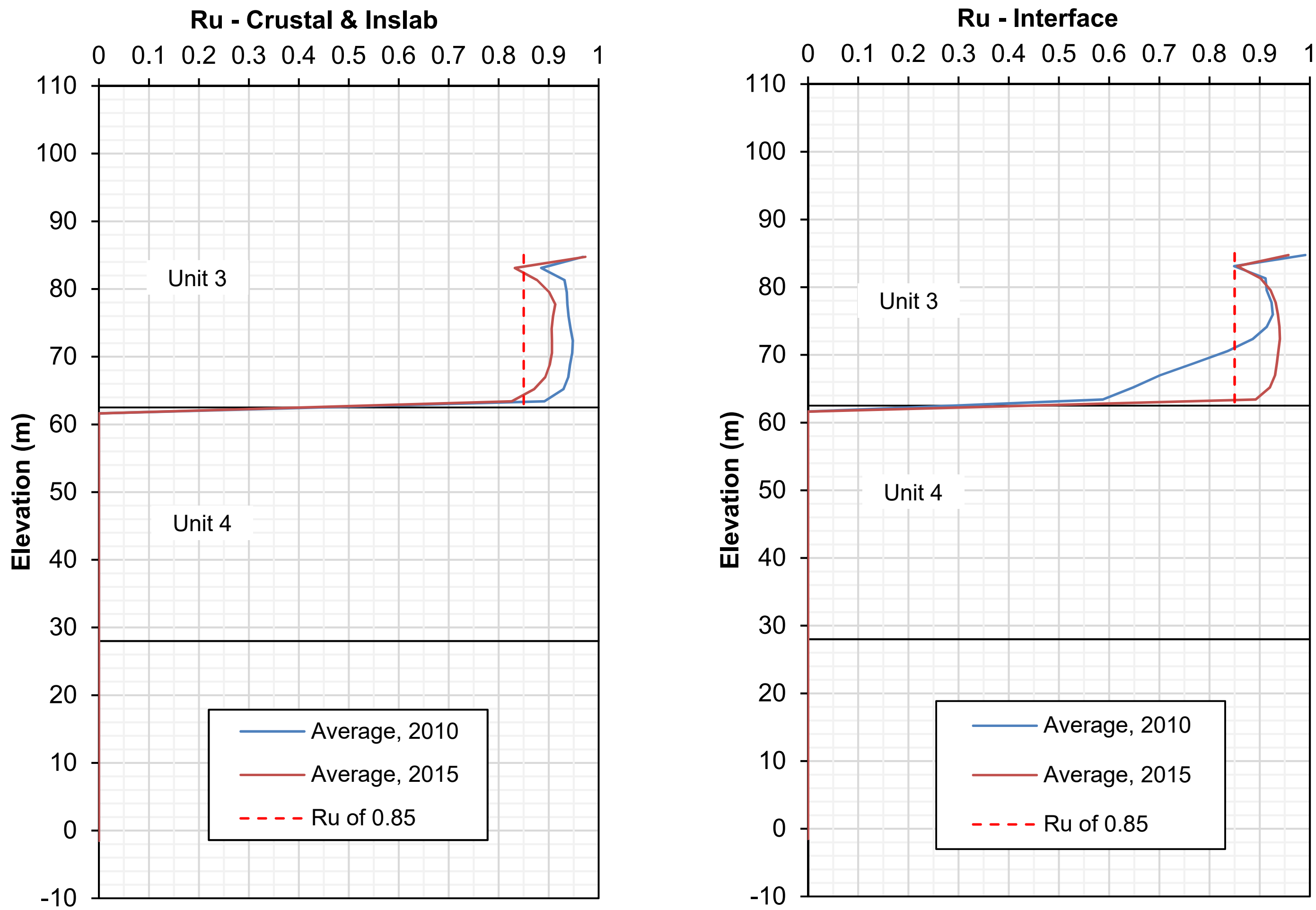
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

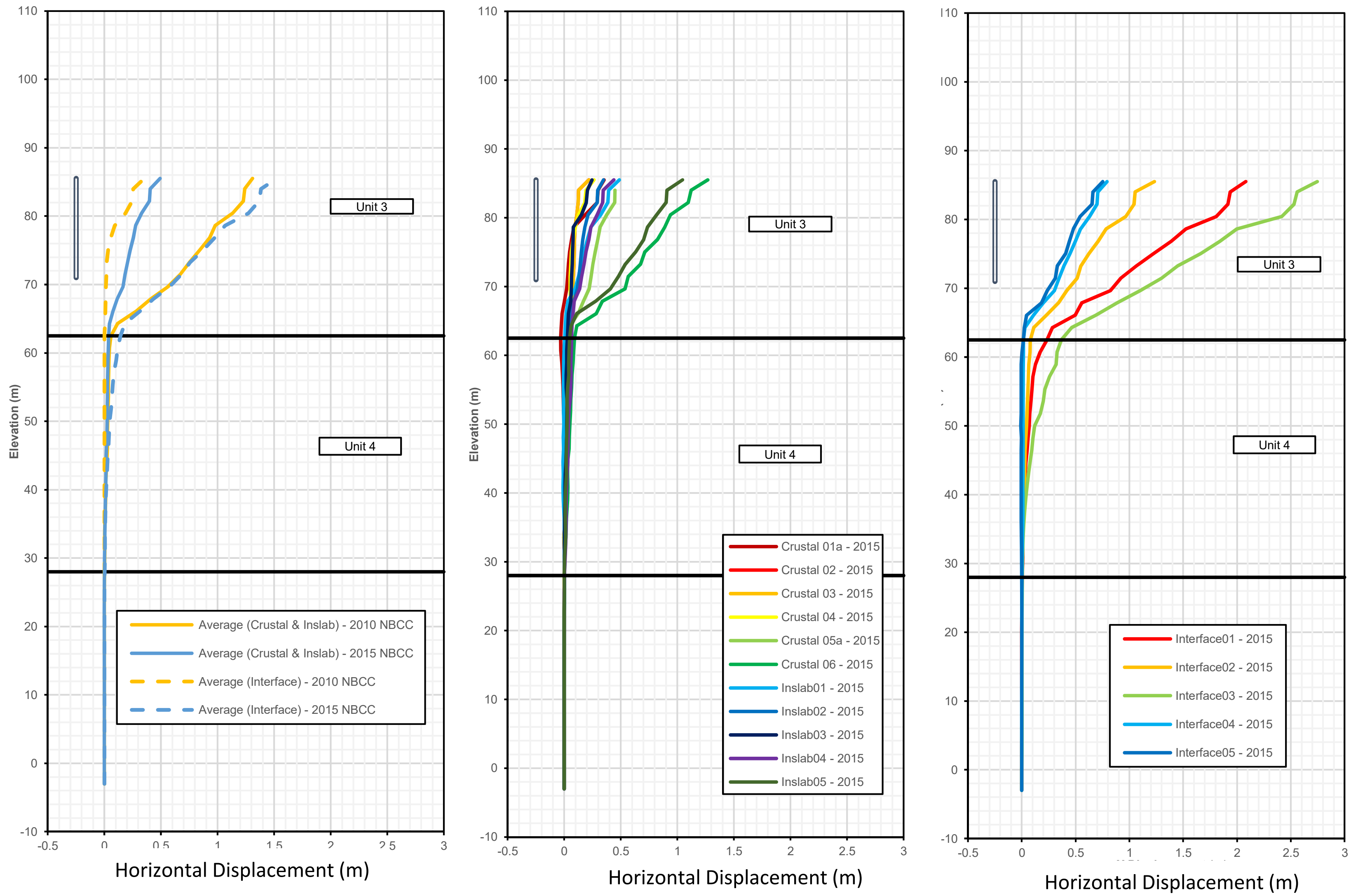
TITLE
Horizontal Displacement Profiles Future/Effluent Shaft Locations

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-36



Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT CDM SMITH CANADA ULC		PROJECT AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC	
CONSULTANT Golder Associates	YYYY-MM-DD: 2017-11-30 PREPARED: M.S.K DESIGN: M.S.K REVIEW: Y.B APPROVED: V.F	TITLE Extent of Liquefaction Riser Shaft Location	
		PROJECT No. 1525010	Phase 2100 / Rev A / FIGURE 5-37



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

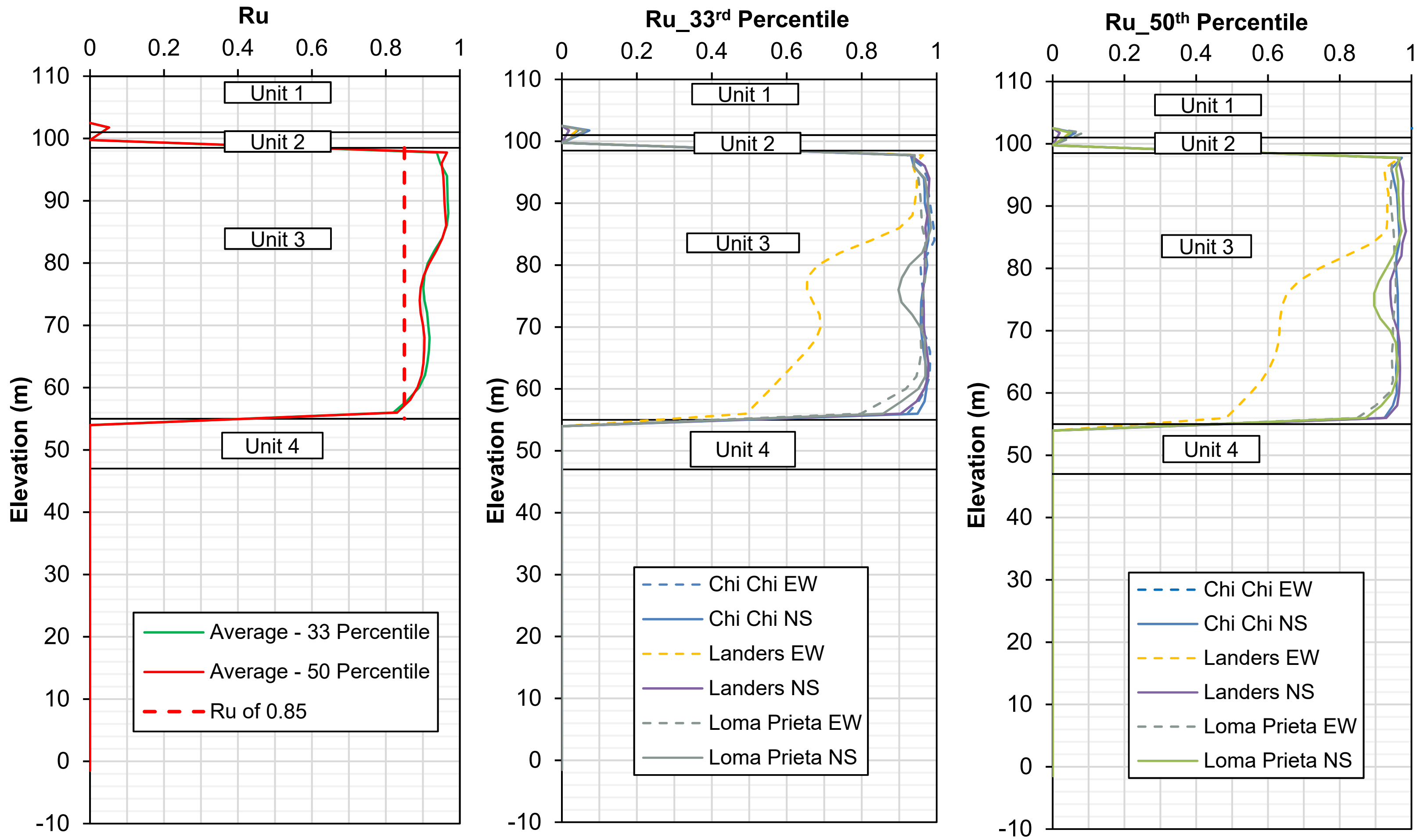
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

TITLE
Horizontal Displacement Profiles Riser Shaft Location

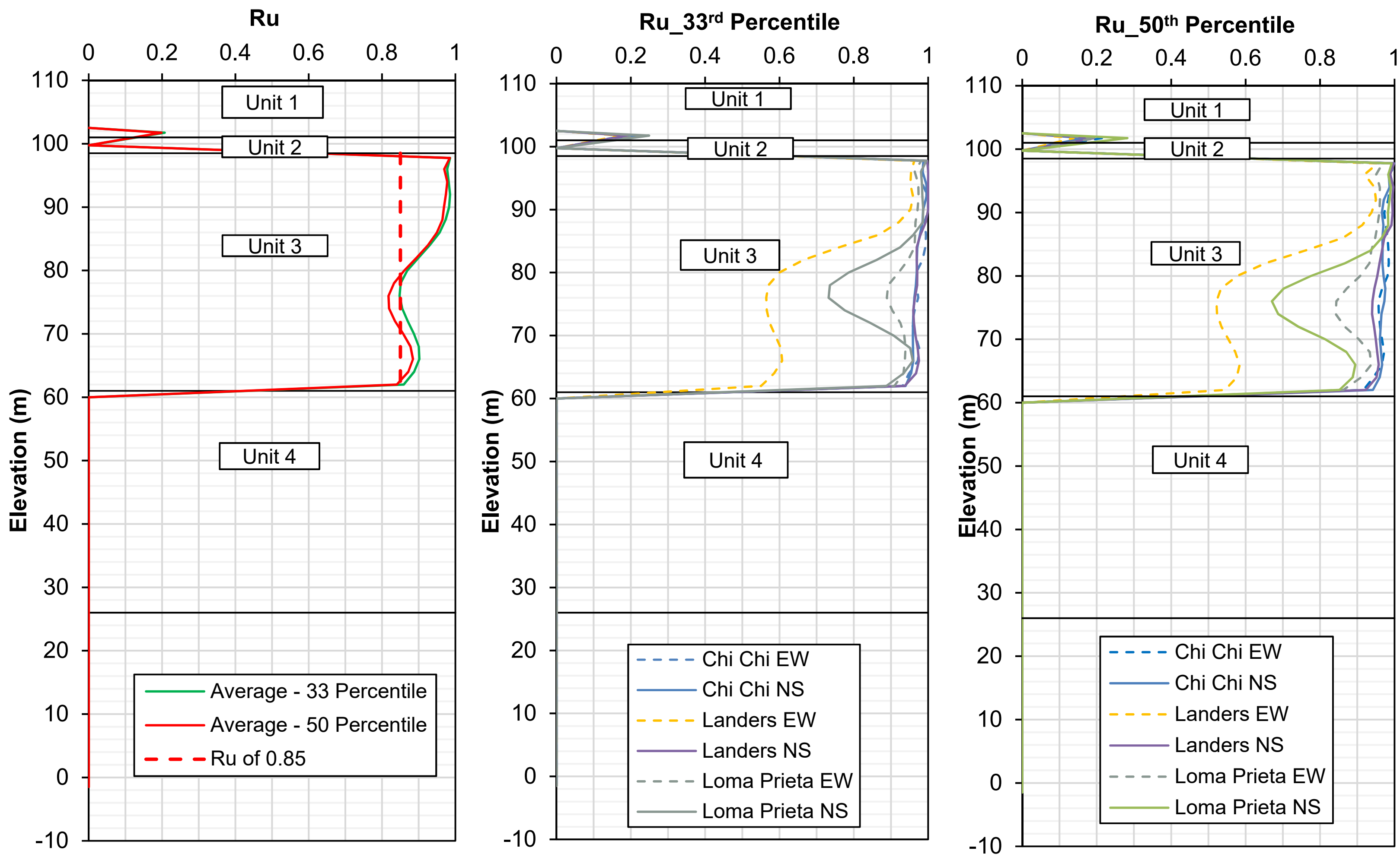
PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-38



<u>Note</u>:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT	CDM SMITH CANADA ULC	PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates	TITLE	Extent of Liquefaction Outfall Shaft Location
YYYY-MM-DD	2017-11-30		
PREPARED	M.S.K		
DESIGN	M.S.K		
REVIEW	Y.B		
APPROVED	V.F		

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-39



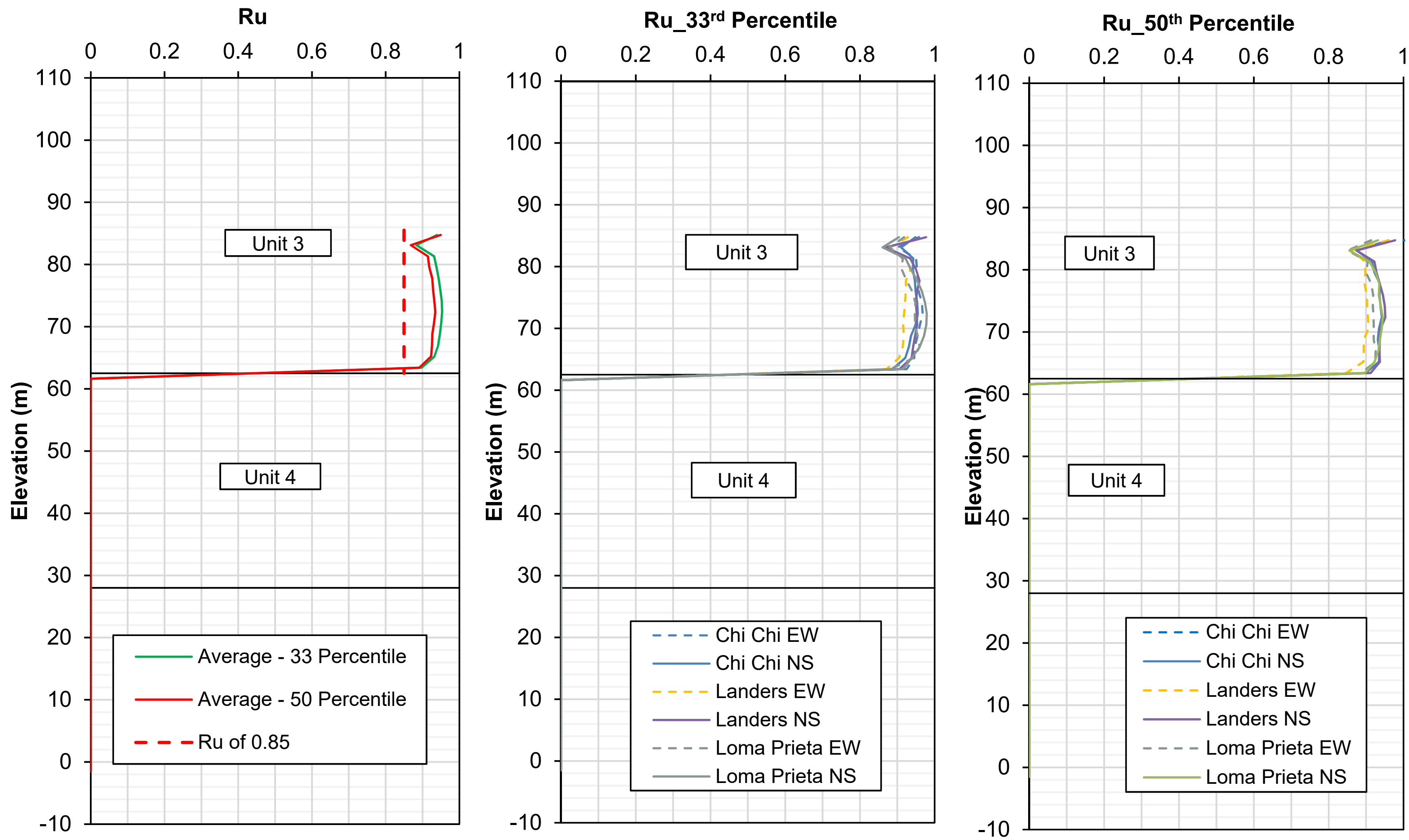
Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT	CDM SMITH CANADA ULC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

TITLE
Extent of Liquefaction Future/Effluent Shaft Locations

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-40



Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

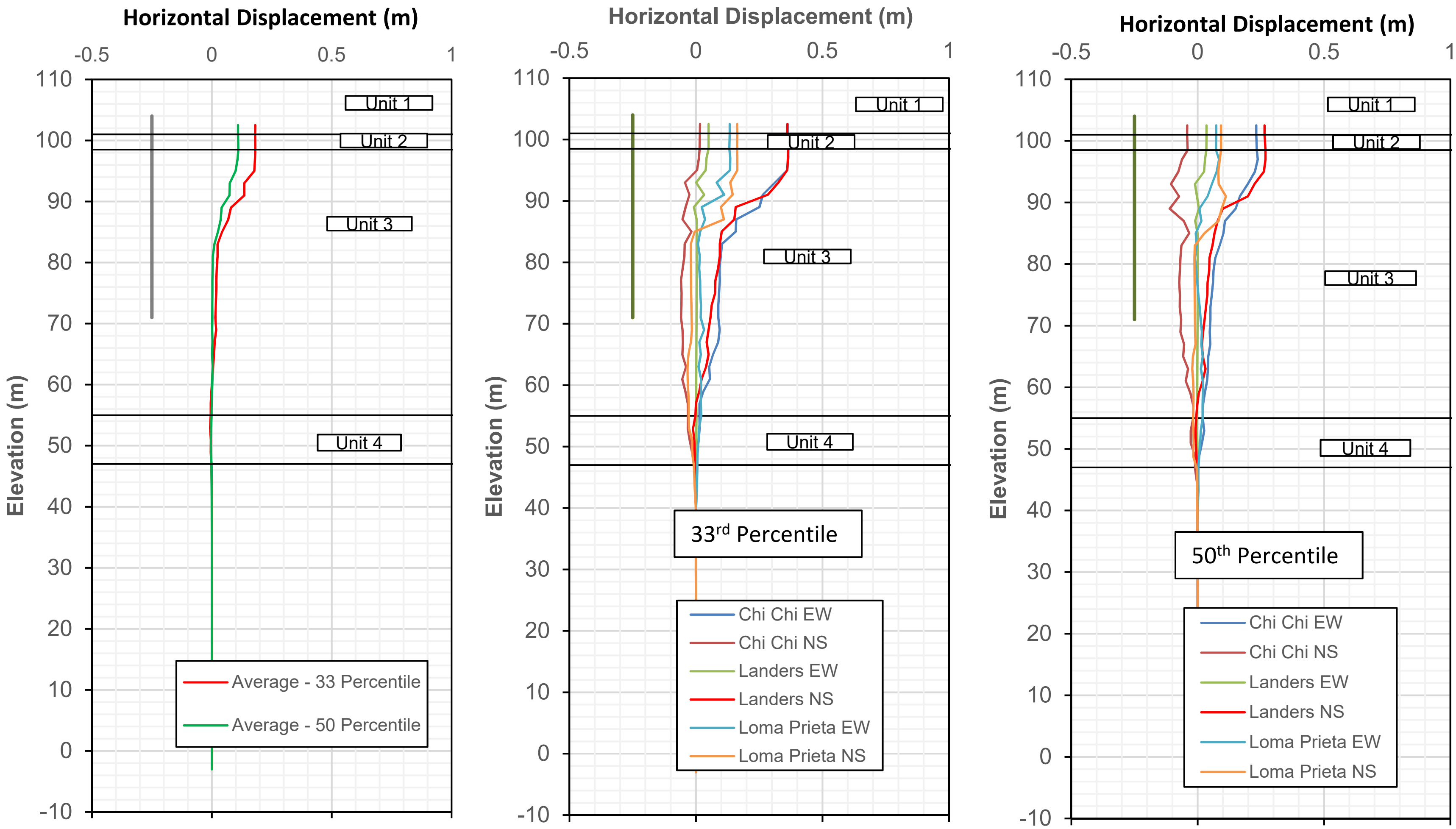
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

TITLE
Extent of Liquefaction
Riser Shaft Location

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-41

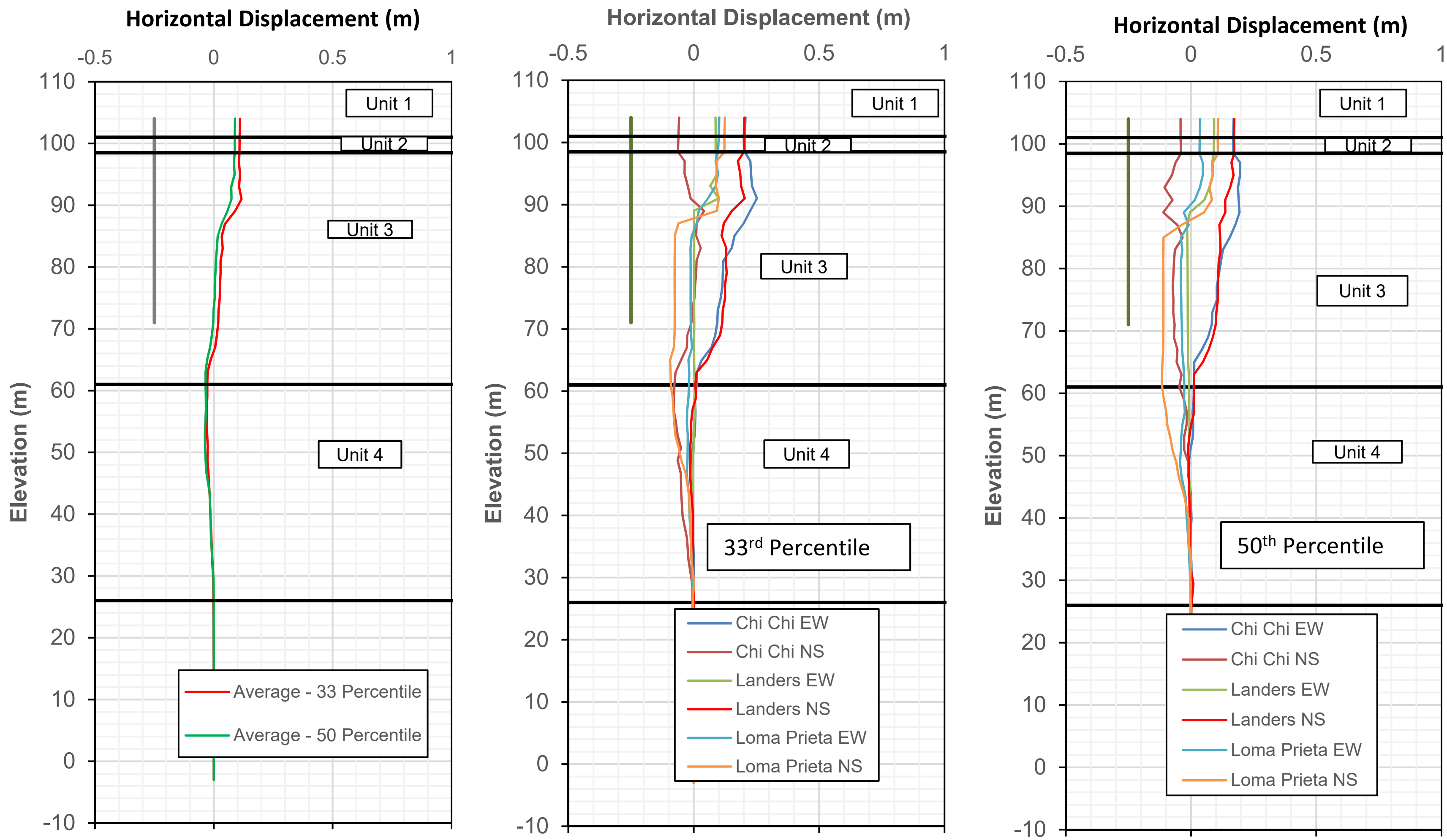


Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT	CDM SMITH CANADA ULC	PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates	TITLE	Horizontal Displacement Profiles Outfall Shaft Location
YYYY-MM-DD	2017-11-30		
PREPARED	M.S.K		
DESIGN	M.S.K		
REVIEW	Y.B		
APPROVED	V.F		

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-42



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

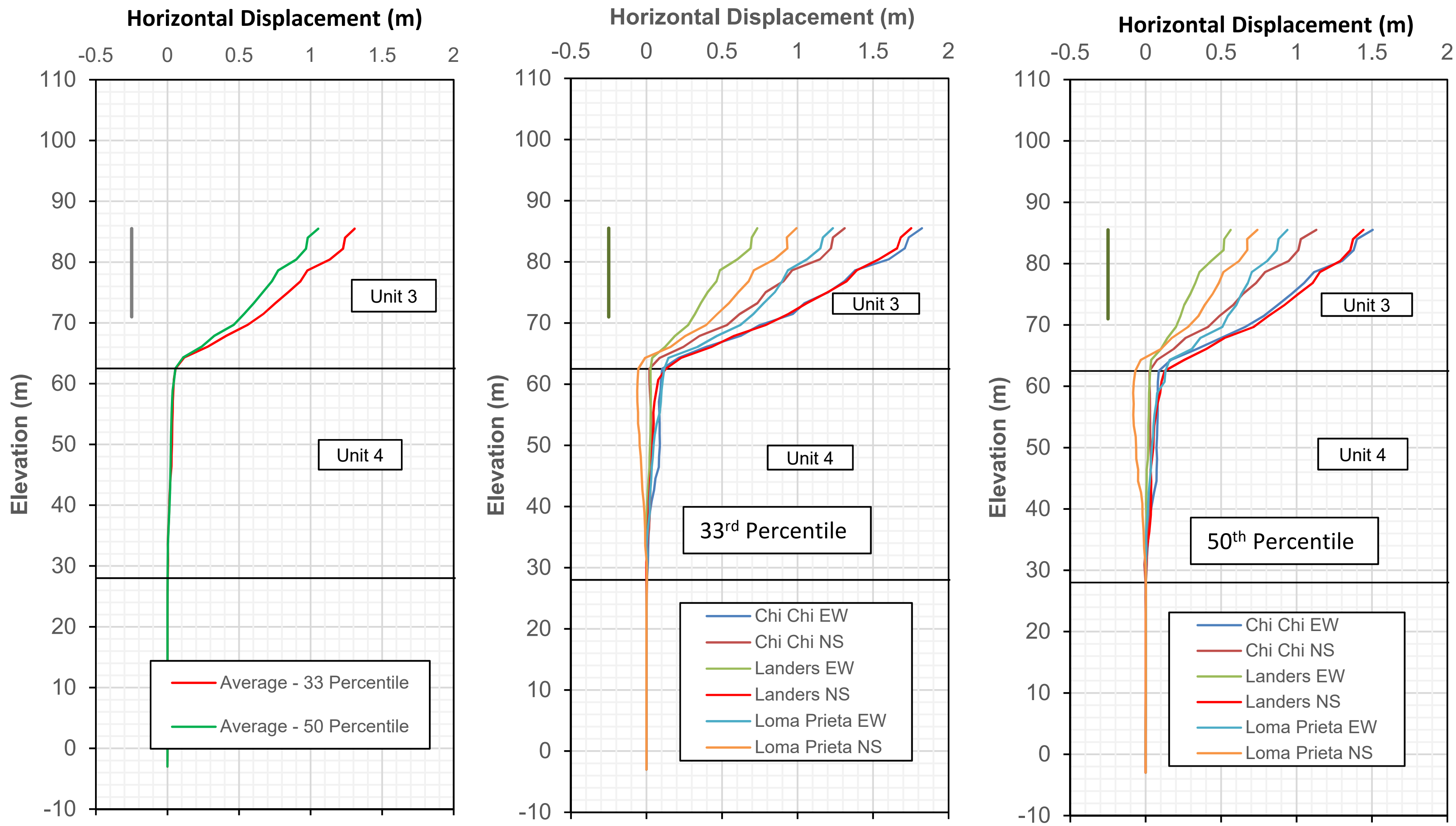
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

TITLE
Horizontal Displacement Profiles Future/Effluent Shaft Locations

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-43



Note:

Shown in the plots illustrates the embedment of the shafts

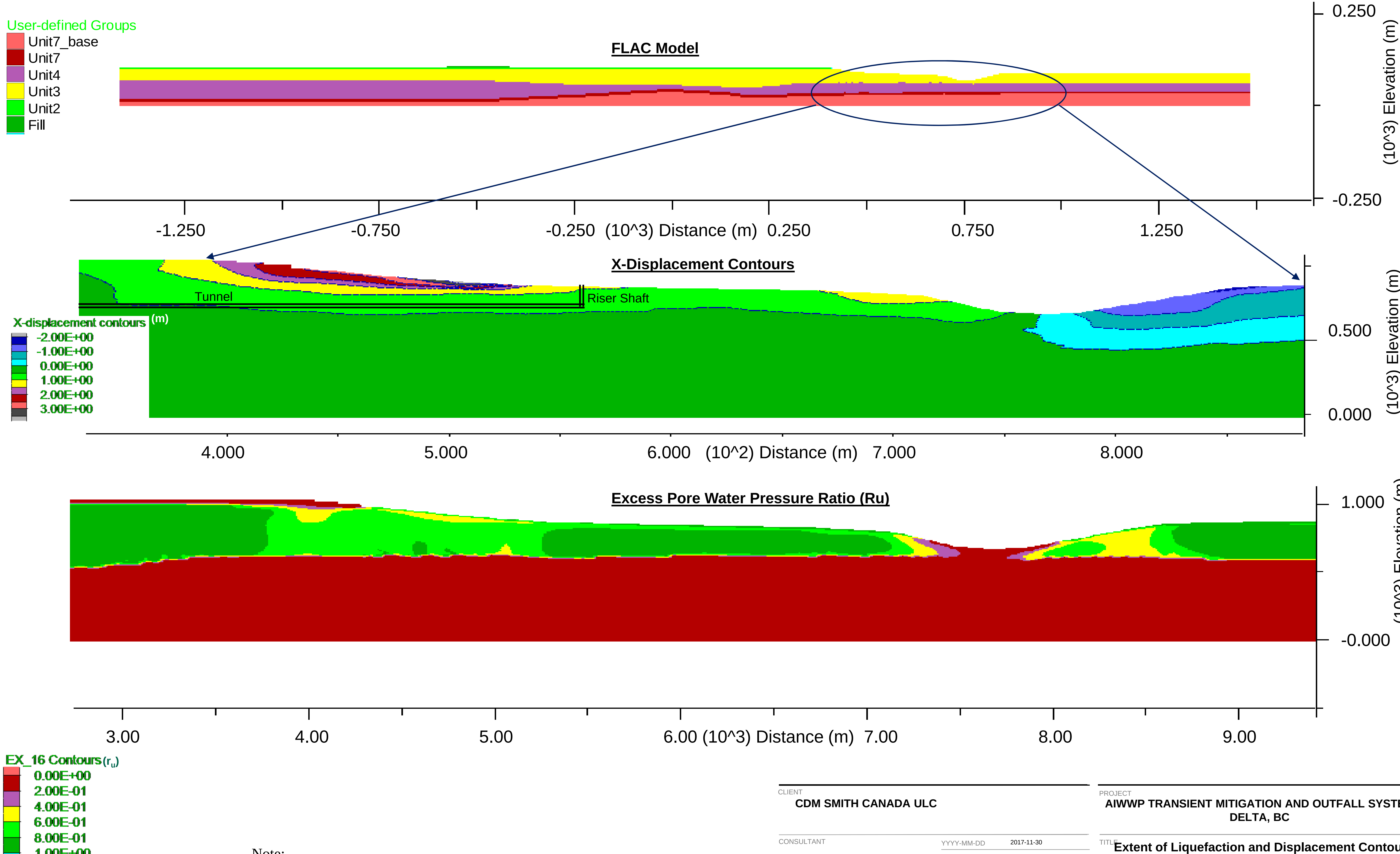
CLIENT	CDM SMITH CANADA ULC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE

Horizontal Displacement Profiles
Riser Shaft Location

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-44



Note:

Input Motion: 2010NBCC - ChiChiNS

CLIENT
CDM SMITH CANADA ULC

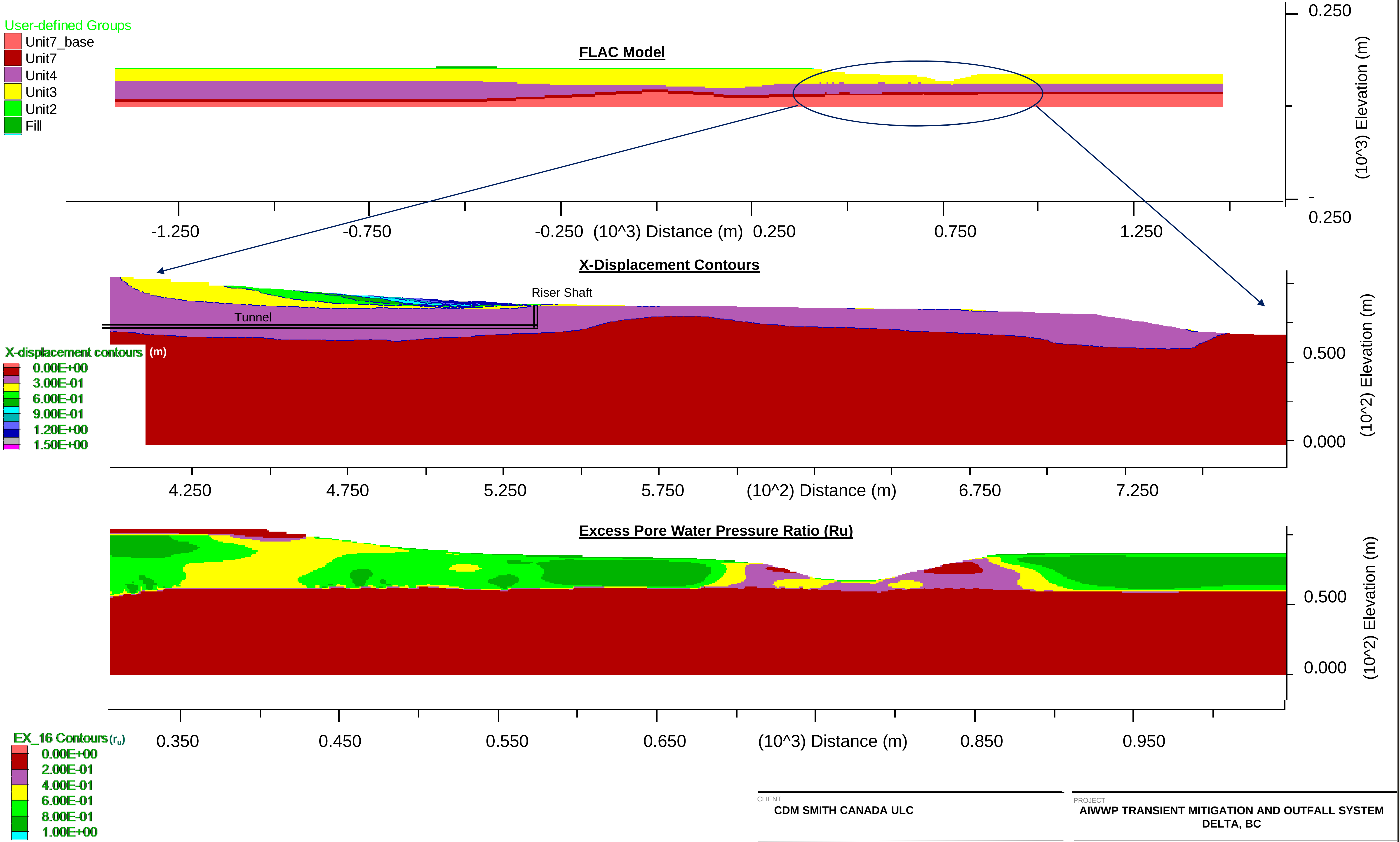
PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Extent of Liquefaction and Displacement Contours
Typical Earthquake Record – Chi Chi NS

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-45



Note:
Input Motion: 2015NBCC – Inslab 02

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

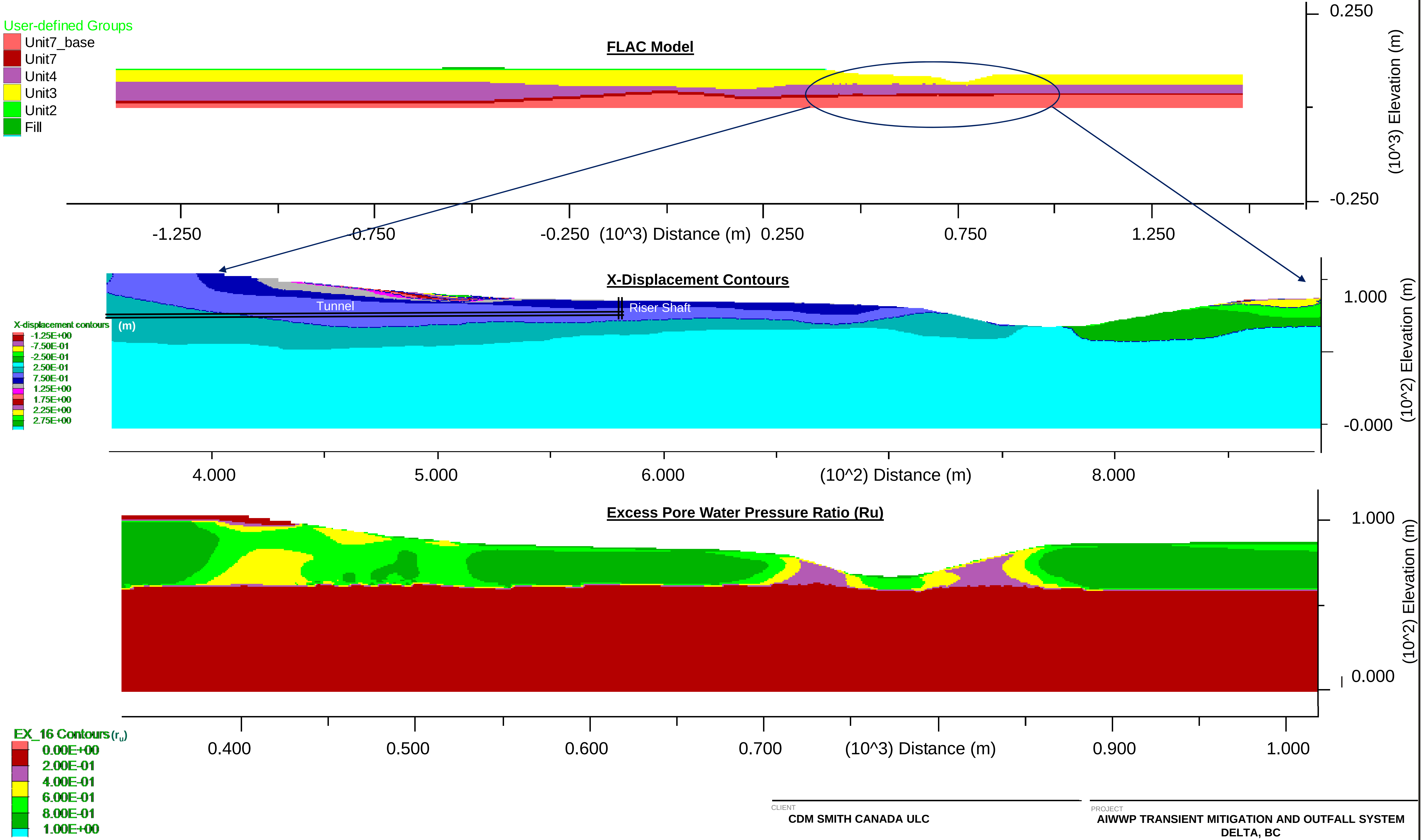
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Extent of Liquefaction and Displacement Contours
Typical Earthquake Record – Inslab 02

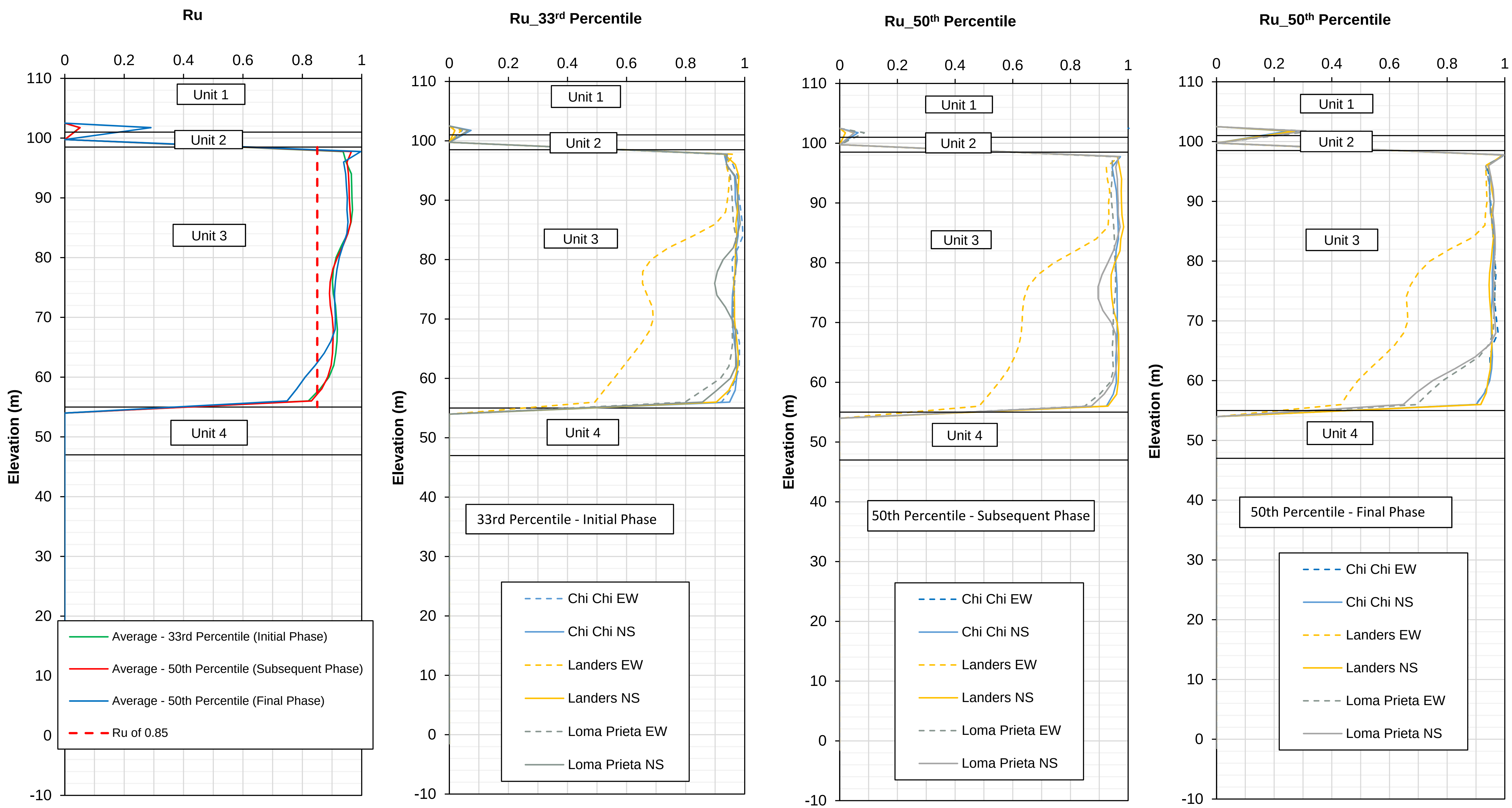
PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-46



Note:
Input Motion: 2015NBCC – Interface 02

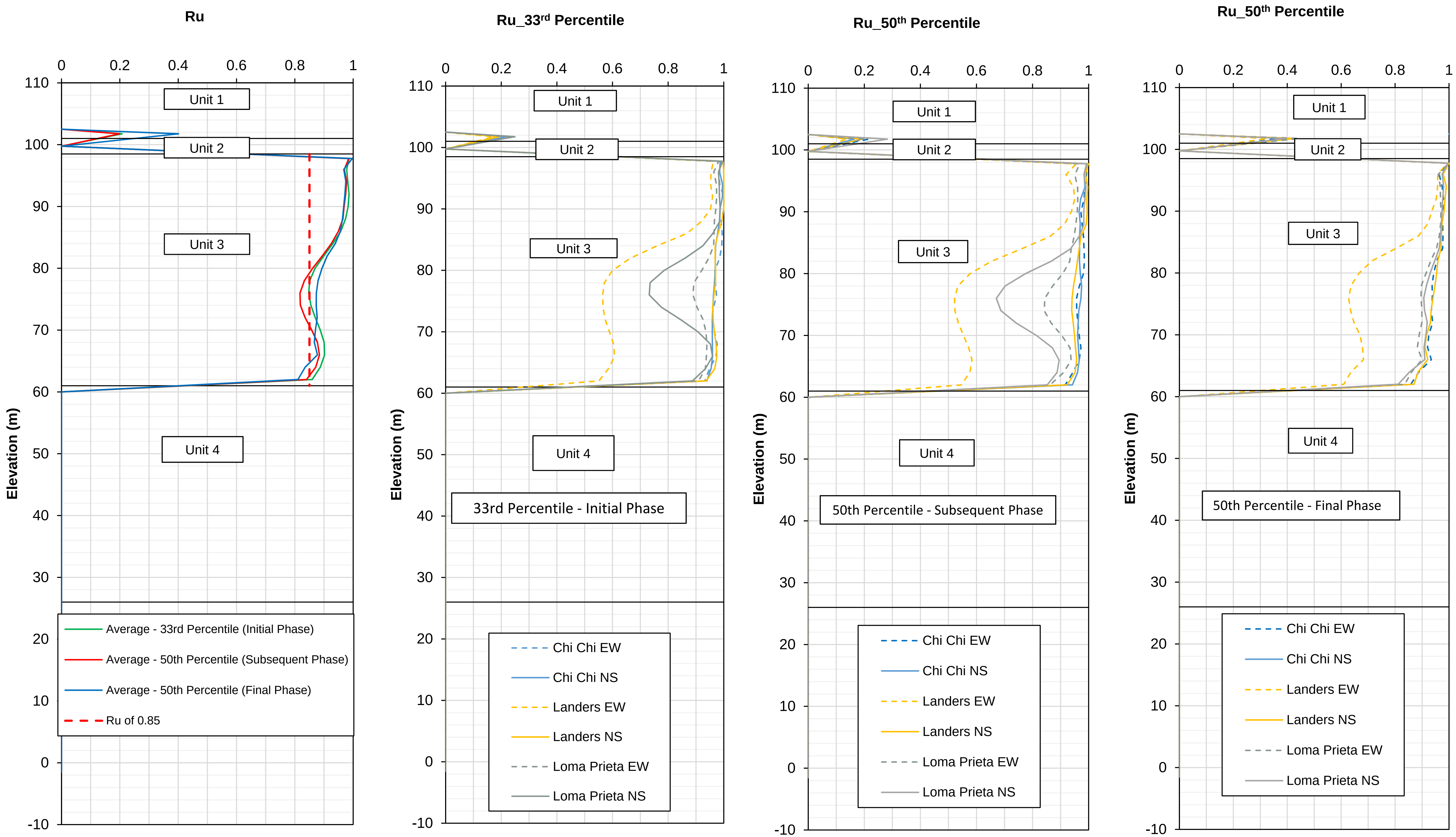
CLIENT CDM SMITH CANADA ULC	PROJECT AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT Golder Associates YYYY-MM-DD 2017-11-30 PREPARED M.S.K DESIGN M.S.K REVIEW Y.B APPROVED V.F	TITLE Extent of Liquefaction and Displacement Contours Typical Earthquake Record – Interface 02

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-47



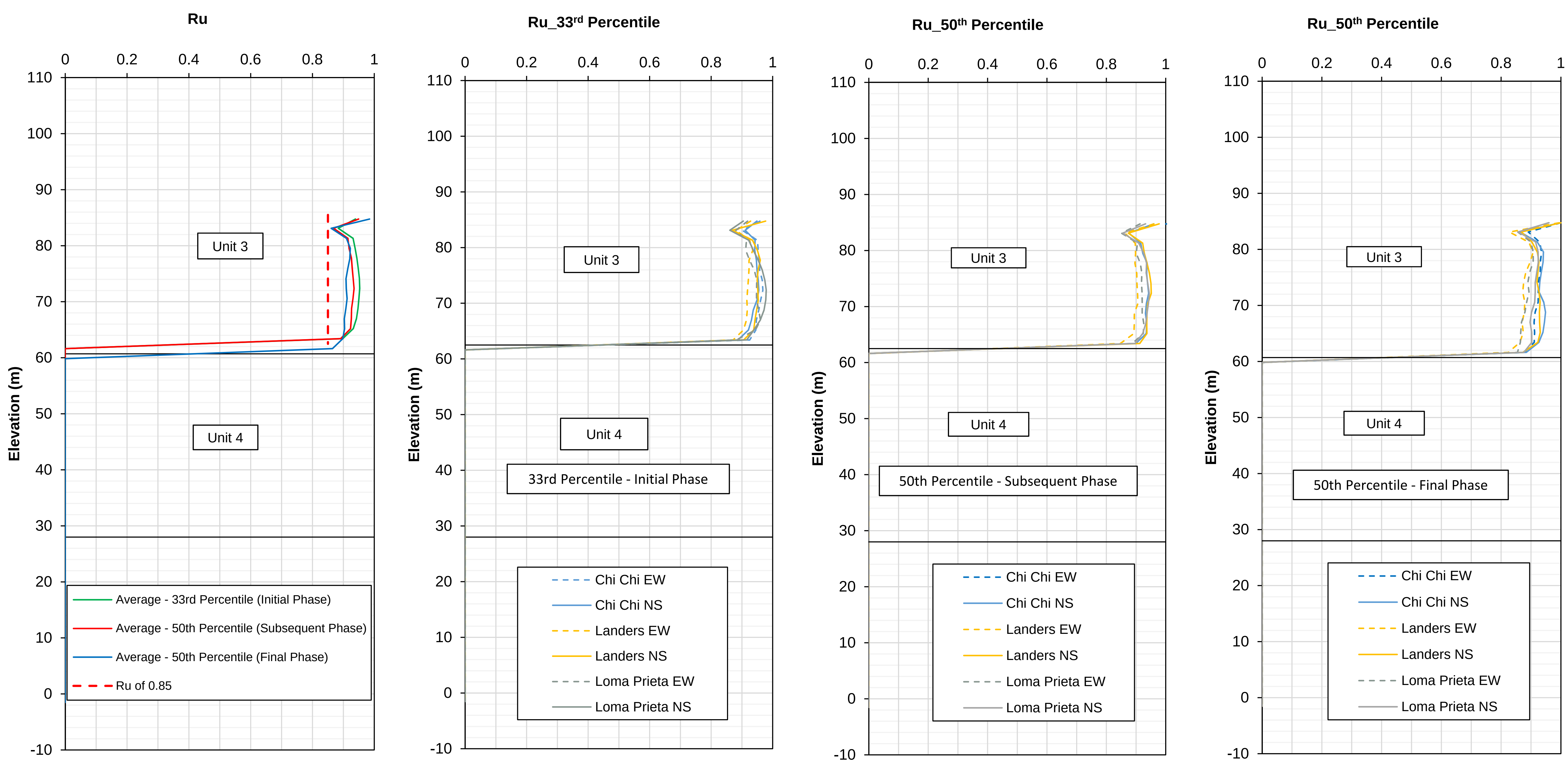
Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT CDM SMITH CANADA ULC	PROJECT AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT Golder Associates YYYY-MM-DD 2017-11-30 PREPARED M.S.K DESIGN M.S.K REVIEW Y.B APPROVED V.F	TITLE **Extent of Liquefaction** **Outfall Shaft Location** PROJECT No. 1525010 · Phase 2100 · Rev A · FIGURE **5-48**



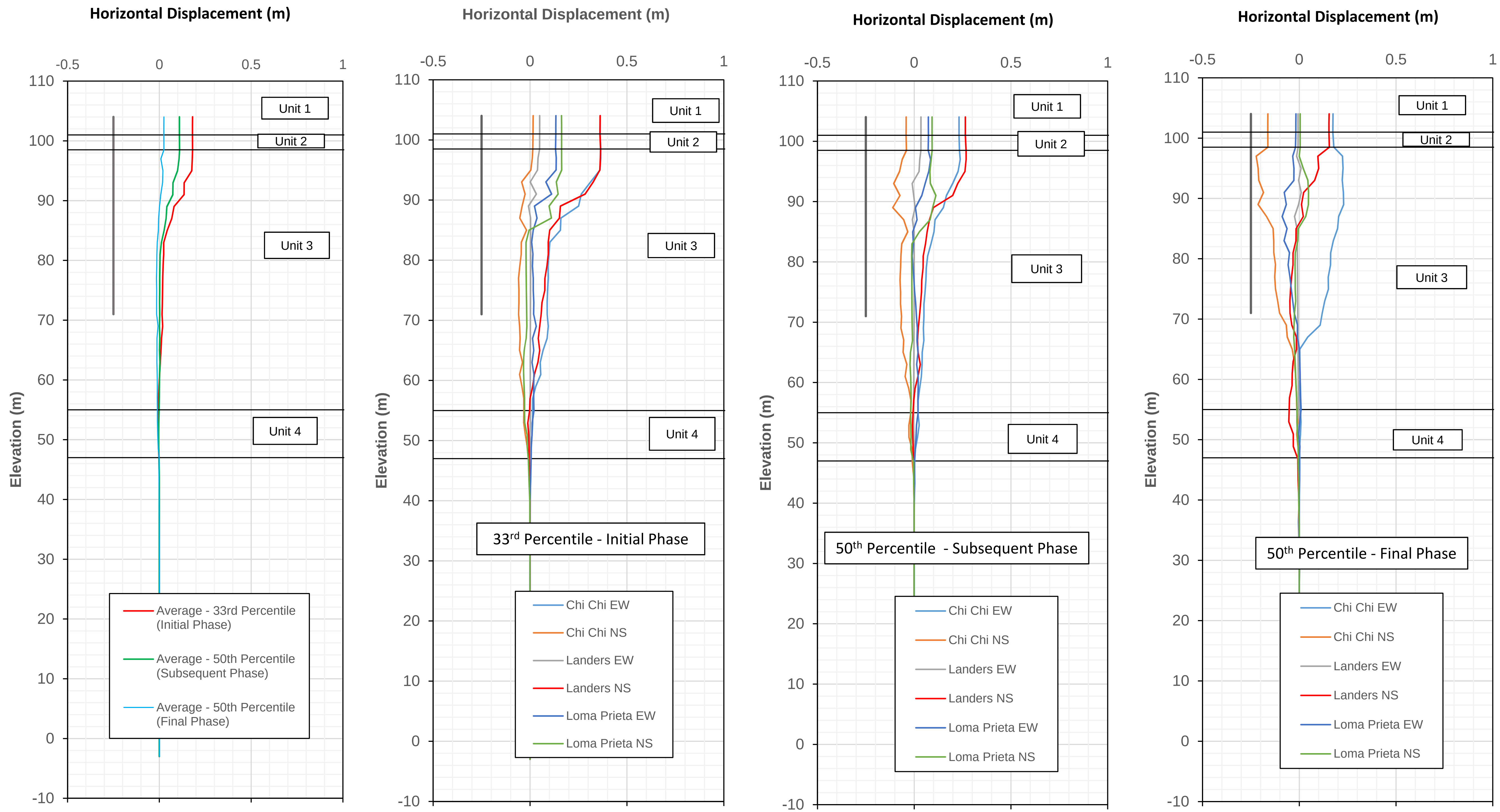
Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT	CDM SMITH CANADA ULC
PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F
TITLE	Extent of Liquefaction Future/Effluent Shaft Locations
PROJECT No.	1525010
Phase	2100
Rev	A
FIGURE	5-49



Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

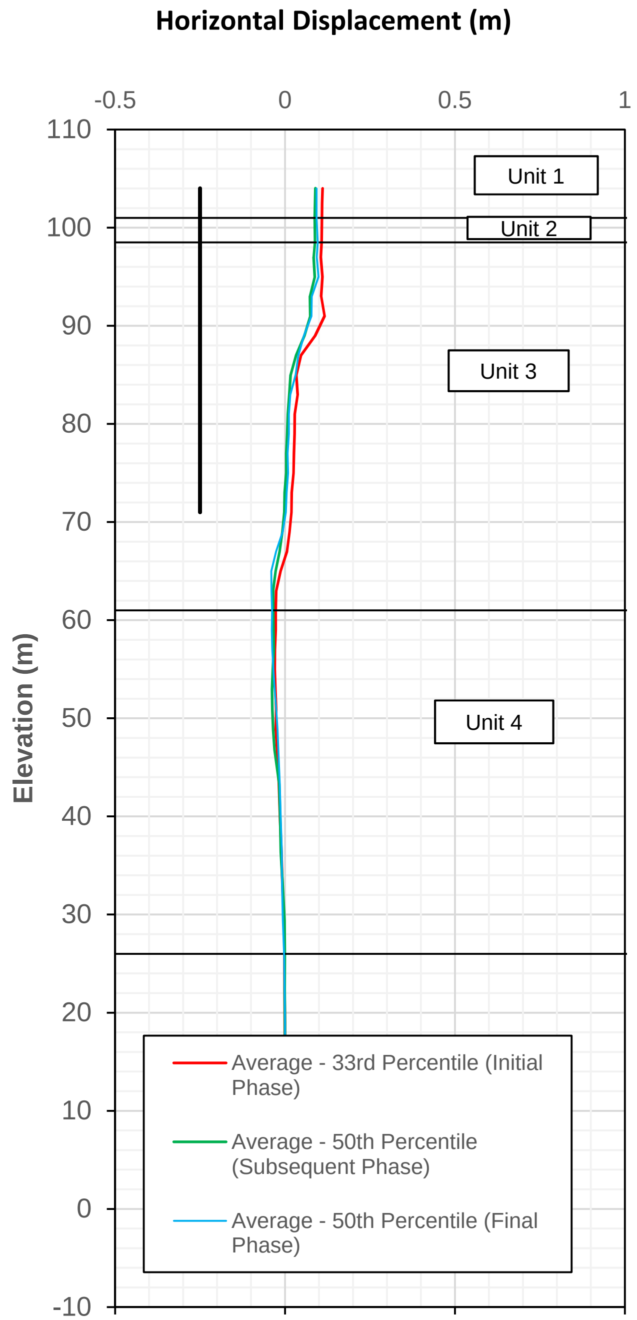
CLIENT	CDM SMITH CANADA ULC	PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates	TITLE	Extent of Liquefaction Riser Shaft Location
YYYY-MM-DD	2017-11-30	PROJECT No.	1525010
PREPARED	M.S.K	Phase	2100
DESIGN	M.S.K	Rev	A
REVIEW	Y.B	FIGURE	5-50
APPROVED	V.F		

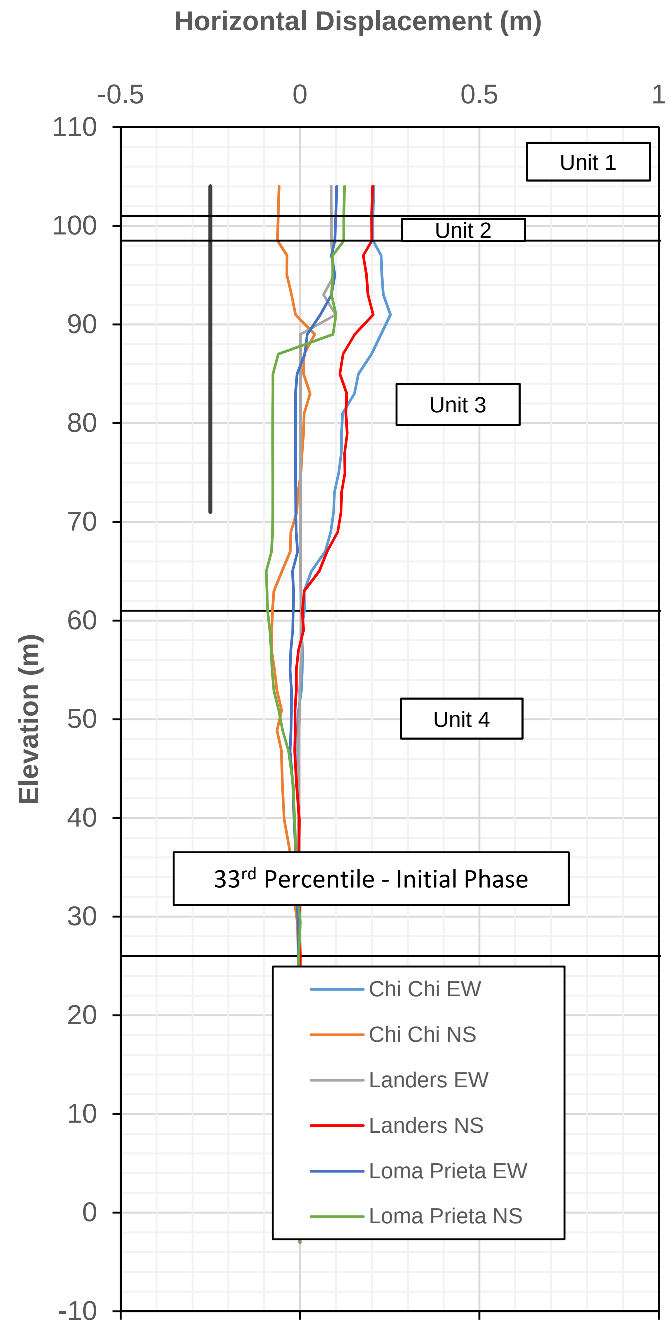


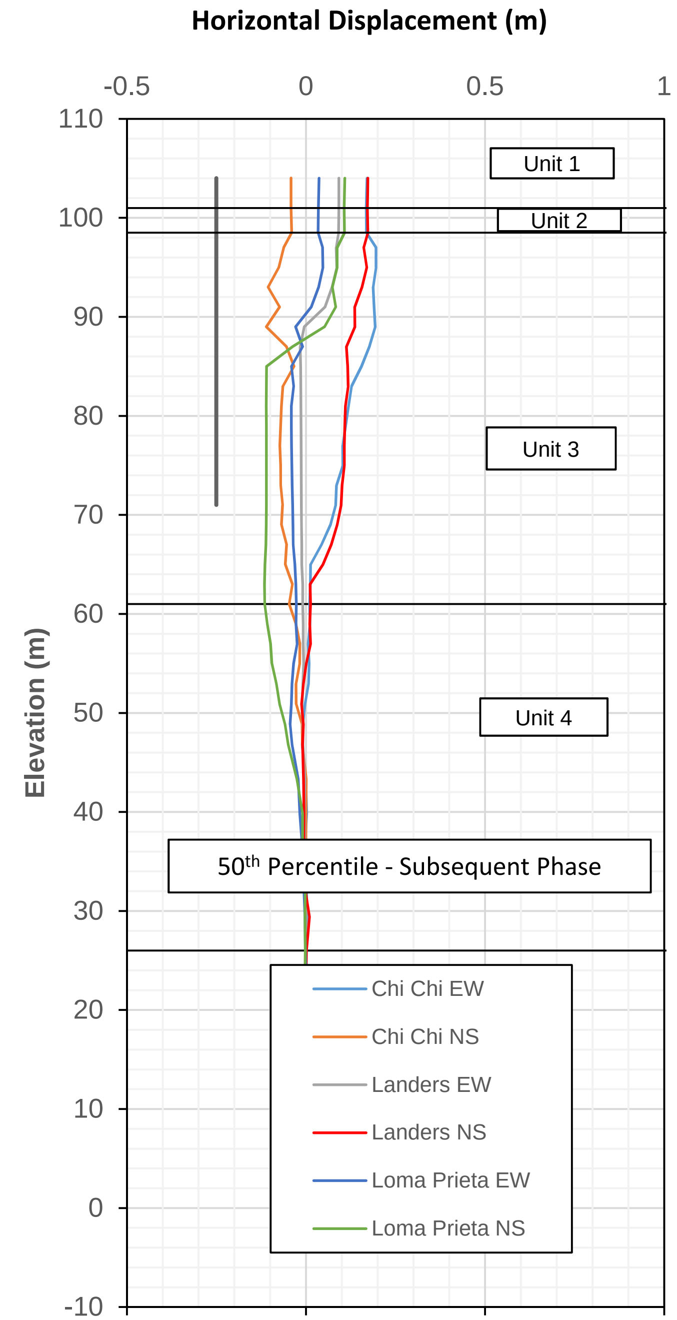
Note:

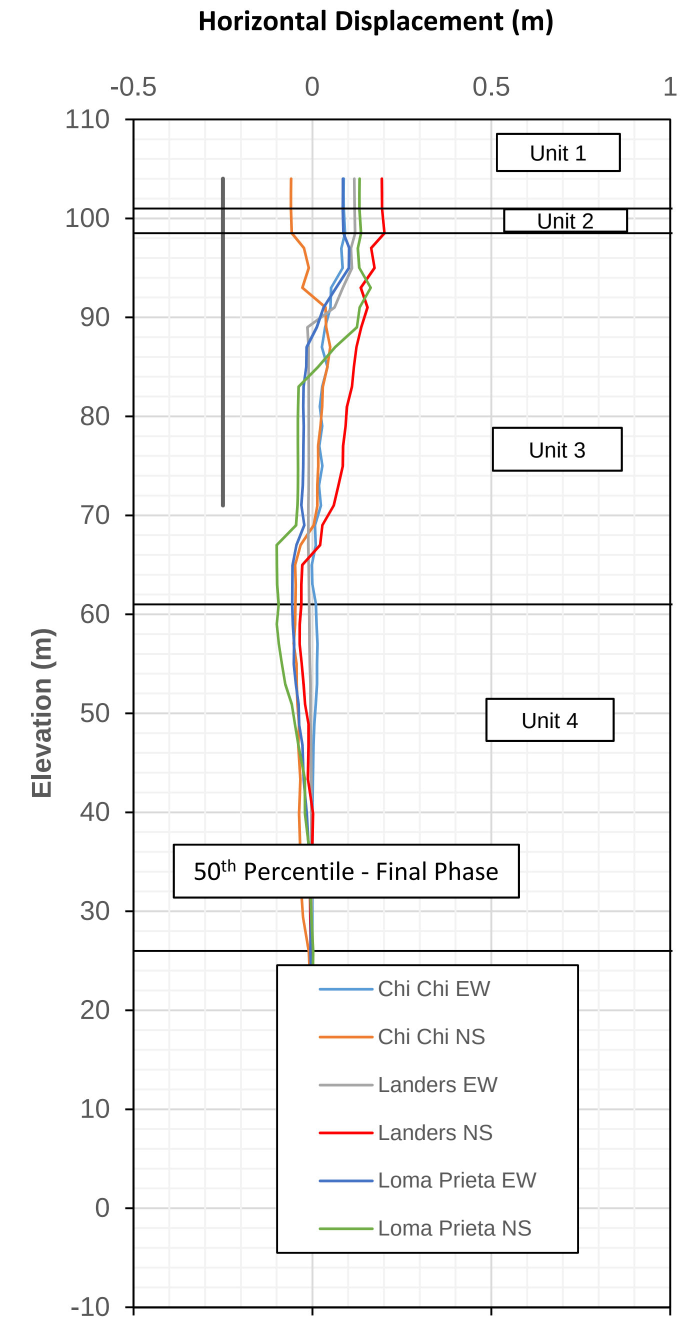
Shown in the plots illustrates the embedment of the shafts

CLIENT	CDM SMITH CANADA ULC
PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F
TITLE	Horizontal Displacement Profiles Outfall Shaft Location
PROJECT No.	1525010
Phase	2100
Rev	A
FIGURE	5-51





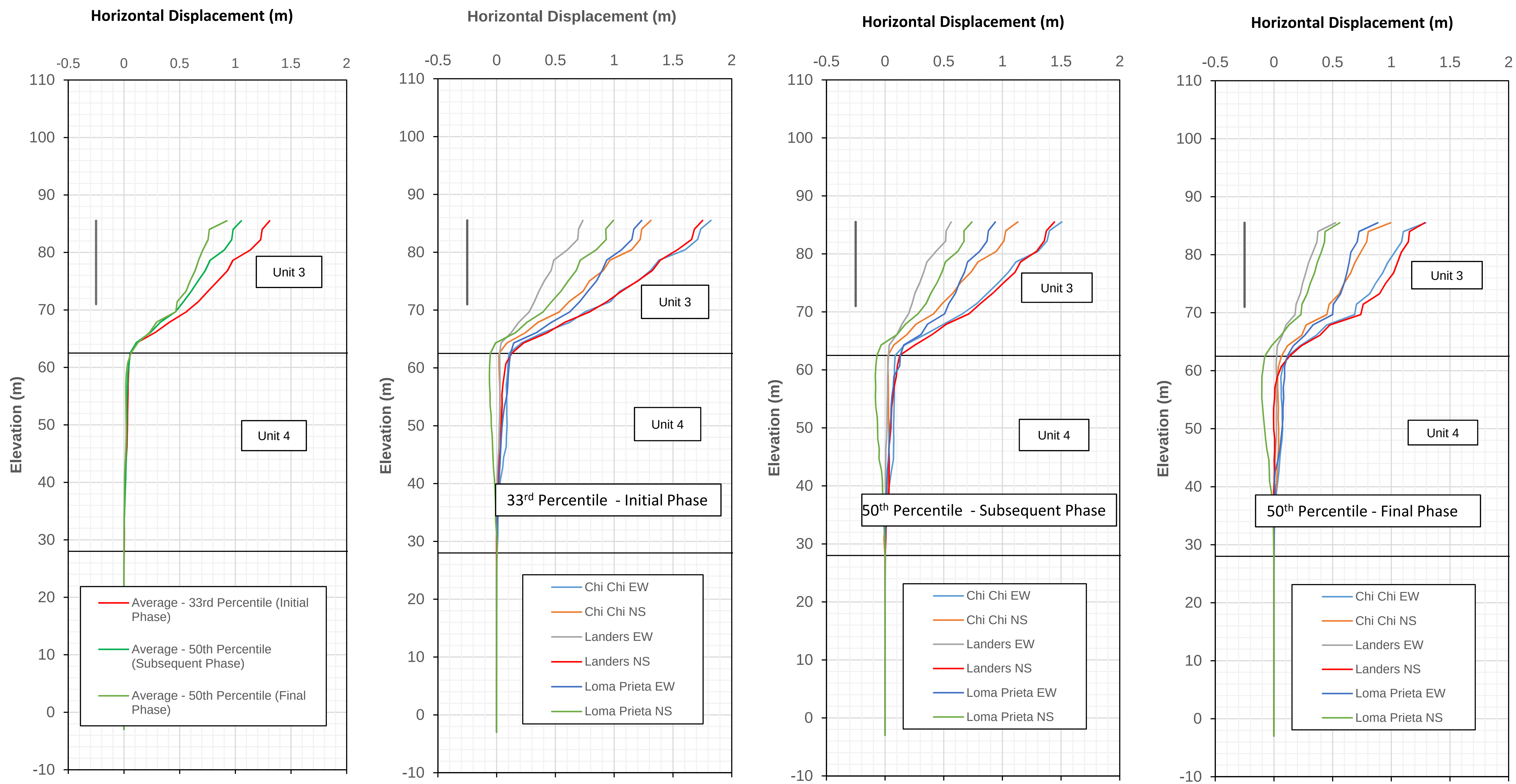




Note:

Shown in the plots illustrates the embedment of the shafts

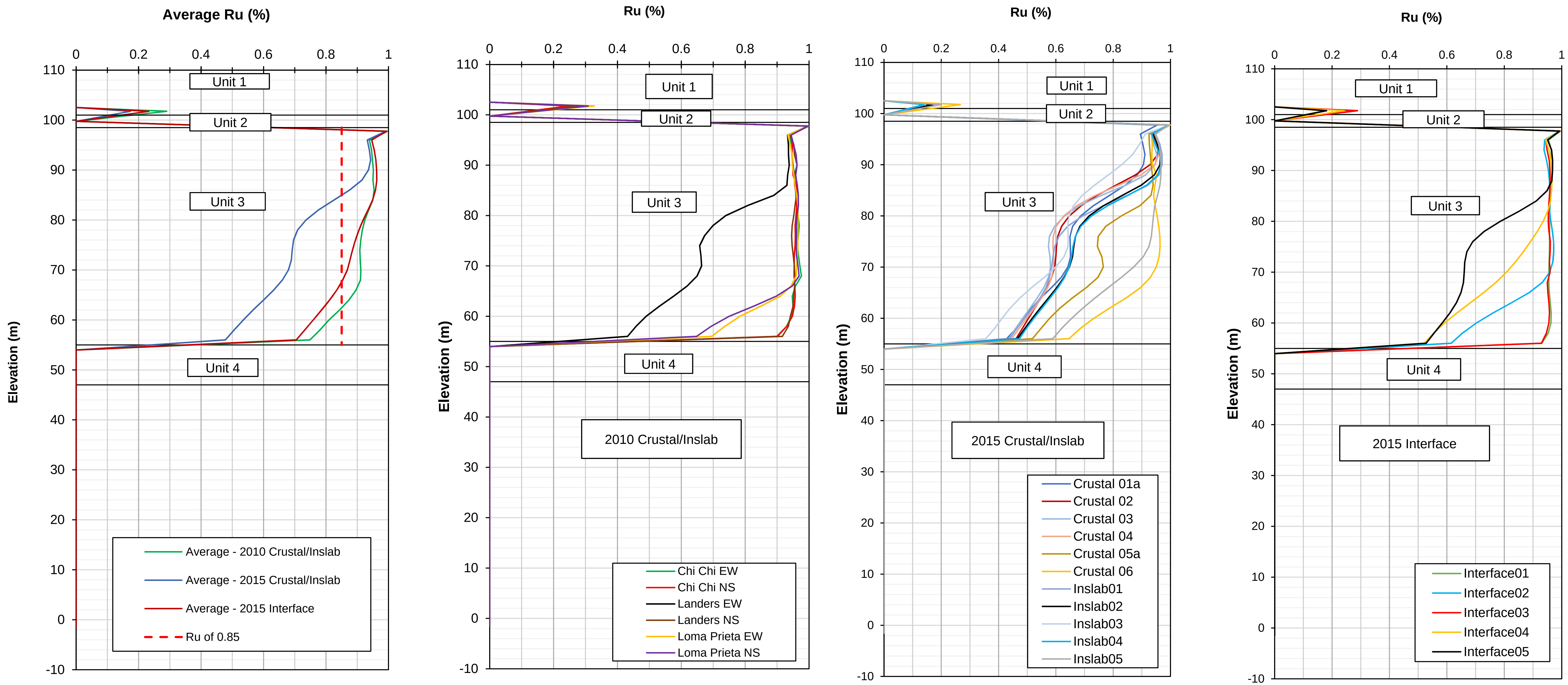
CLIENT	CDM SMITH CANADA ULC
PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F
TITLE	Horizontal Displacement Profiles Future/Effluent Shaft Locations
PROJECT No.	1525010
Phase	2100
Rev	A
FIGURE	5-52



Note:

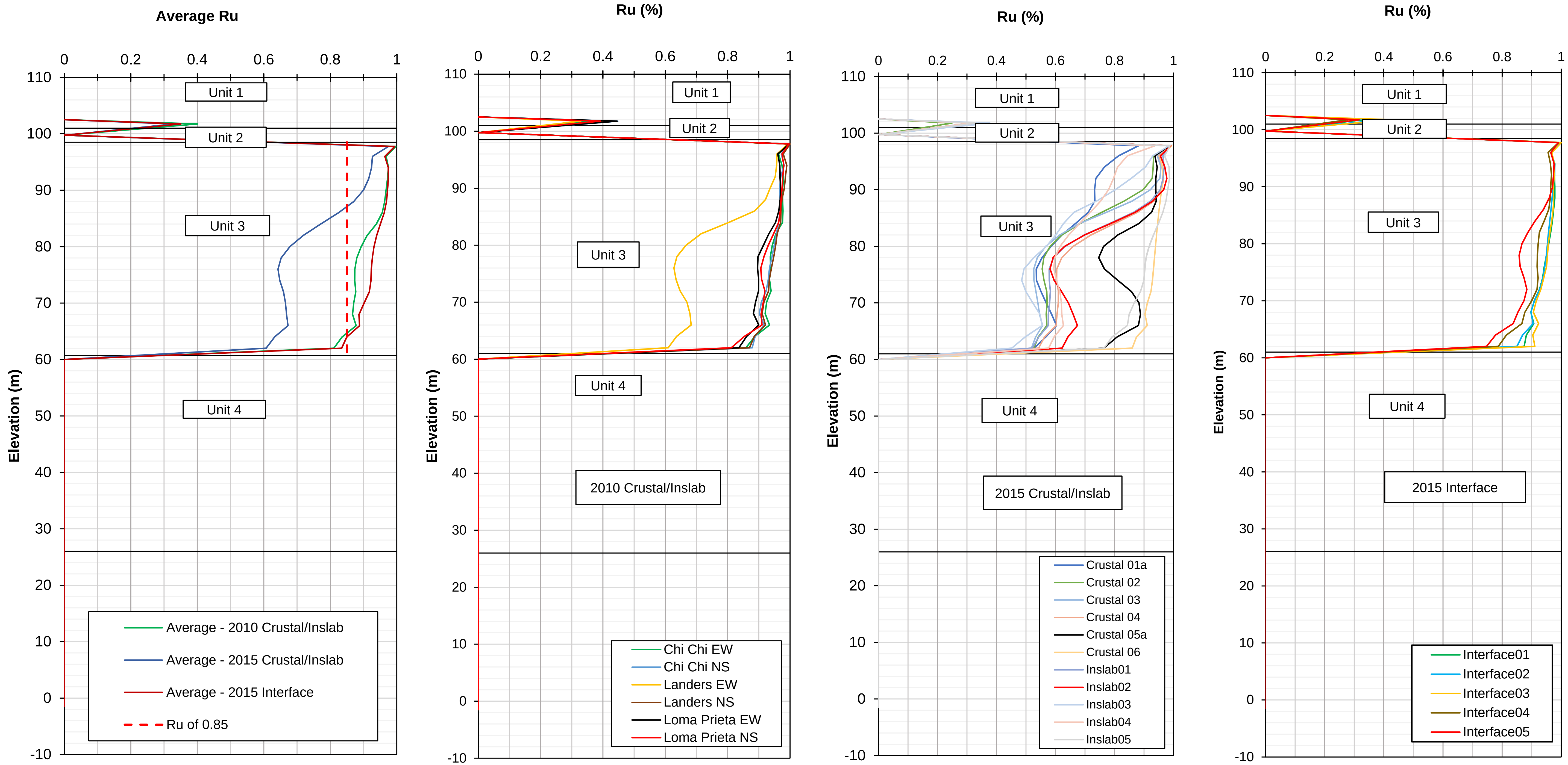
Shown in the plots illustrates the embedment of the shafts

CLIENT	CDM SMITH CANADA ULC	PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates	TITLE	Horizontal Displacement Profiles Riser Shaft Location
YYYY-MM-DD	2017-11-30	PROJECT No.	1525010
PREPARED	M.S.K	Phase	2100
DESIGN	M.S.K	Rev	A
REVIEW	Y.B	FIGURE	5-53
APPROVED	V.F		



Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT	CDM SMITH CANADA ULC
PROJECT	AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F
TITLE	Extent of Liquefaction Outfall Shaft Location
PROJECT No.	1525010
Phase	2100
Rev	A
FIGURE	5-54



<u>Note</u>:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

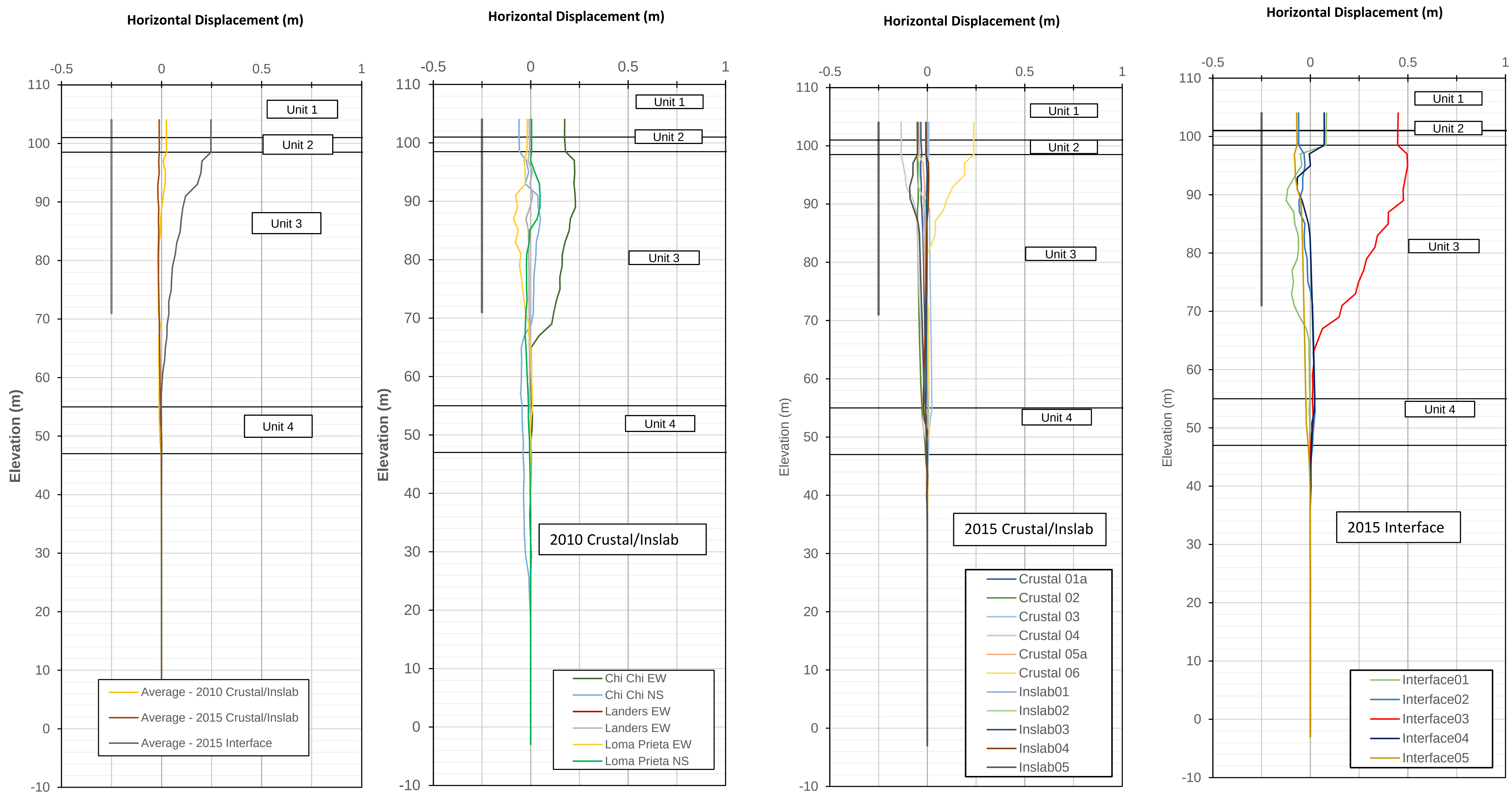
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Extent of Liquefaction
Future/Effluent Shaft Locations

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	**5-55**



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT
CDM SMITH CANADA ULC

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

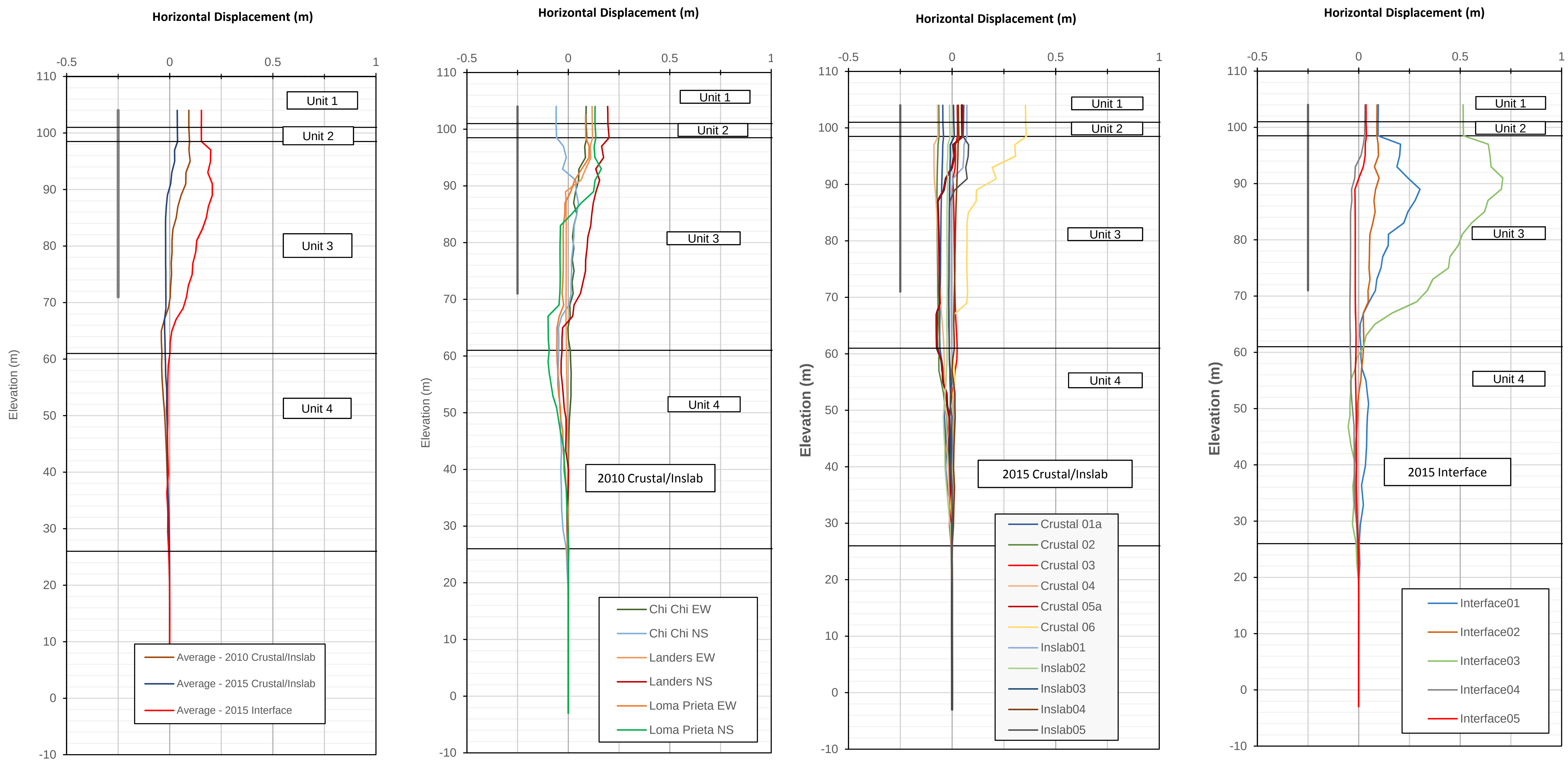
CONSULTANT

Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Horizontal Displacement Profiles Outfall Shaft Location

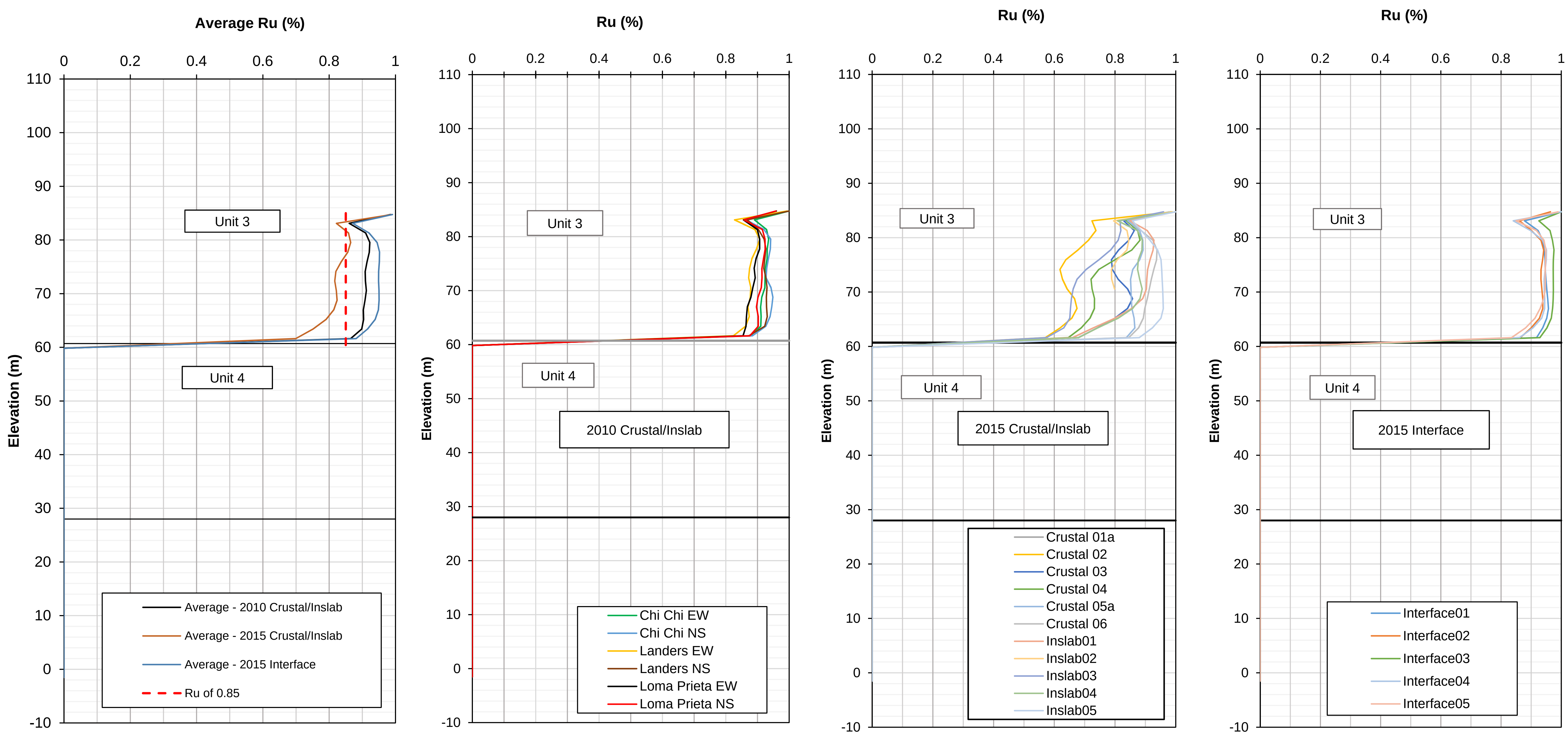
PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-56



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT CDM SMITH CANADA ULC	PROJECT AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
CONSULTANT Golder Associates YYYY-MM-DD 2017-11-30 PREPARED M.S.K DESIGN M.S.K REVIEW Y.B APPROVED V.F	TITLE Horizontal Displacement Profiles Future/Effluent Shaft Locations
	PROJECT No. 1525010 · Phase 2100 · Rev A · FIGURE 5-57



Note:
Ru: Excess Pore Water Pressure Ratio
Ru > 0.85 is considered liquefied

CLIENT
CDM SMITH CANADA ULC

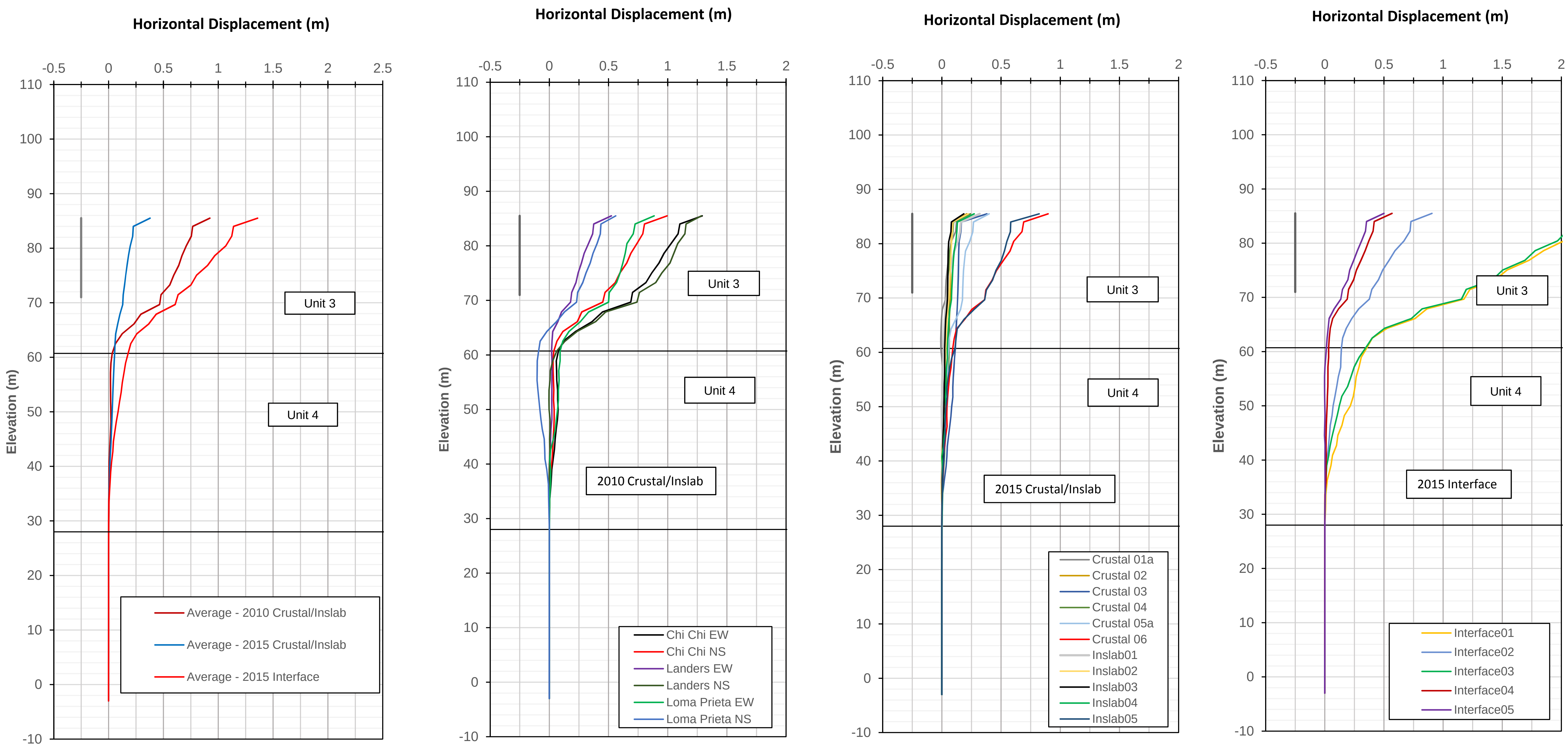
CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Extent of Liquefaction
Riser Shaft Location

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-58



Note:

Shown in the plots illustrates the embedment of the shafts

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

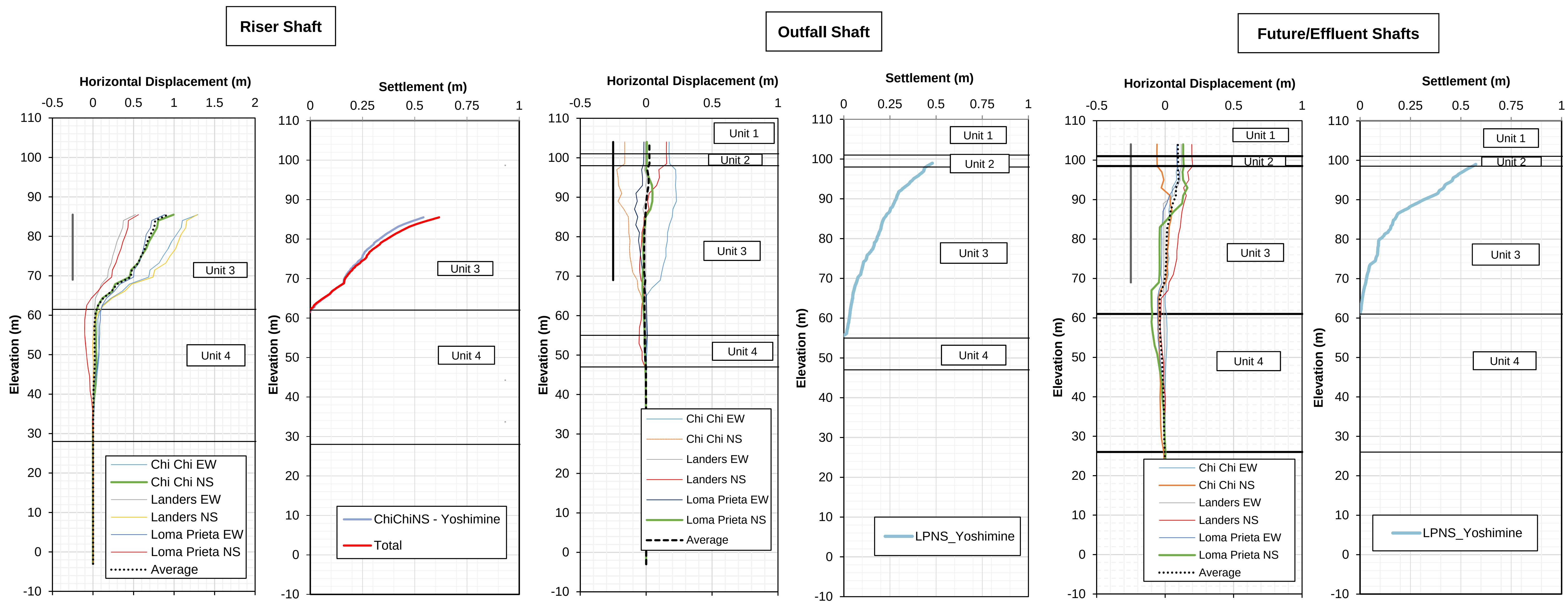
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC

TITLE
Horizontal Displacement Profiles Riser Shaft Location

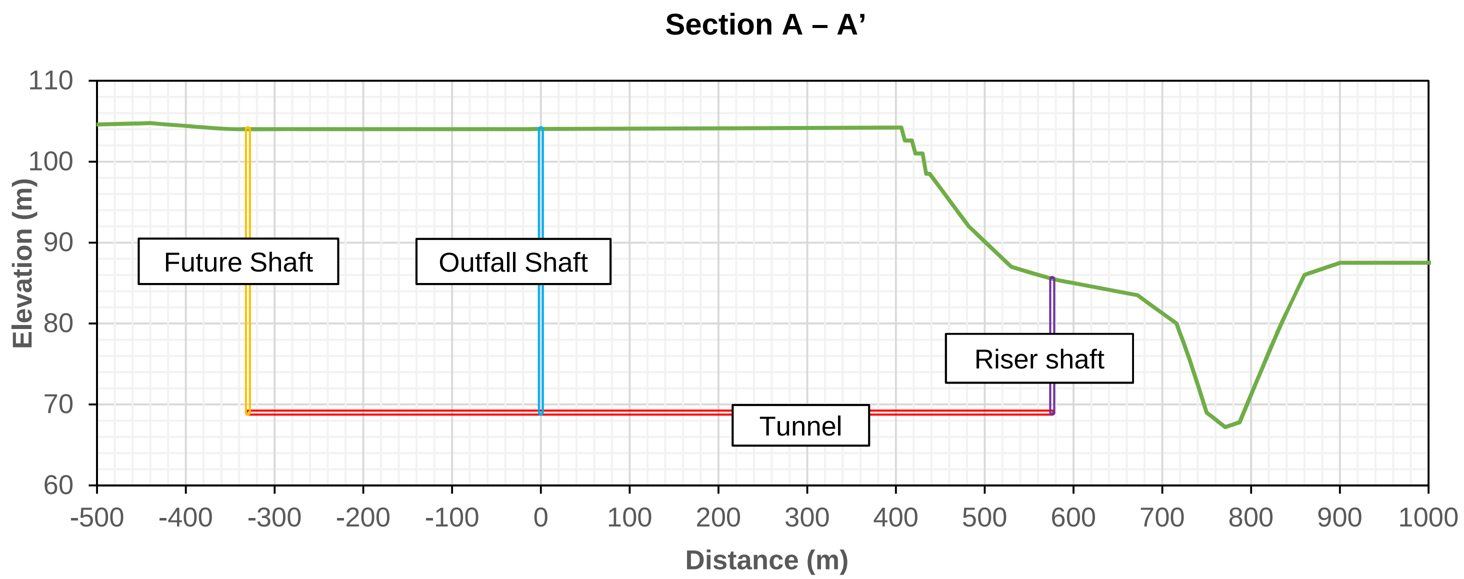
PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-59

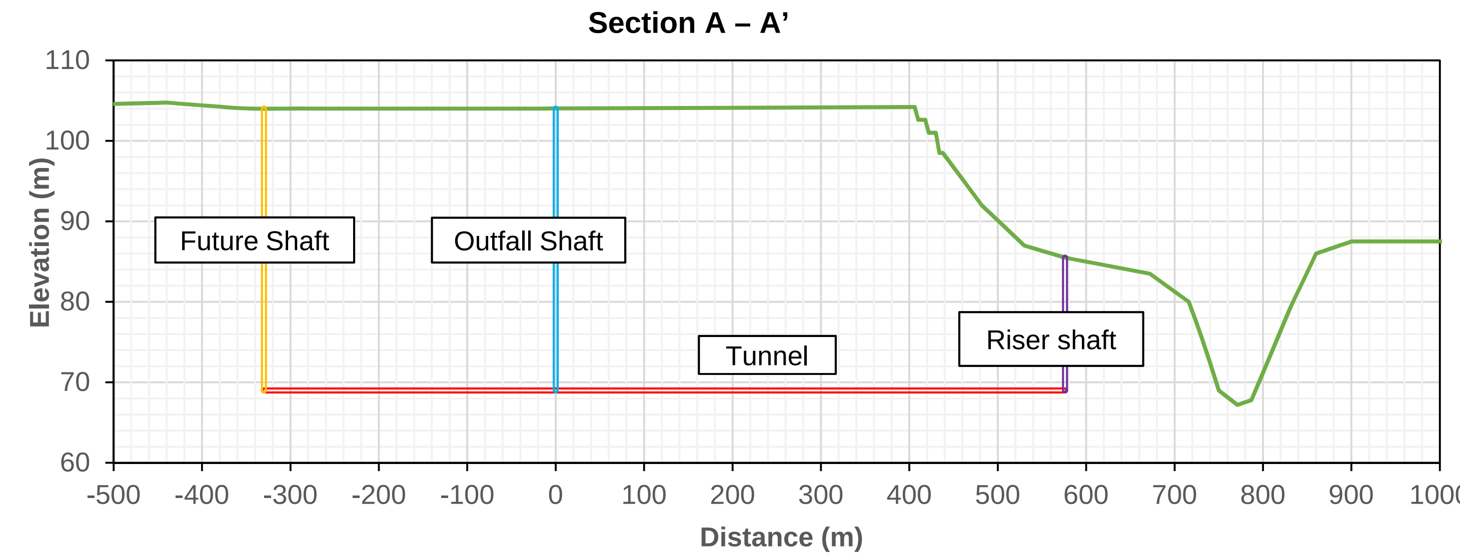


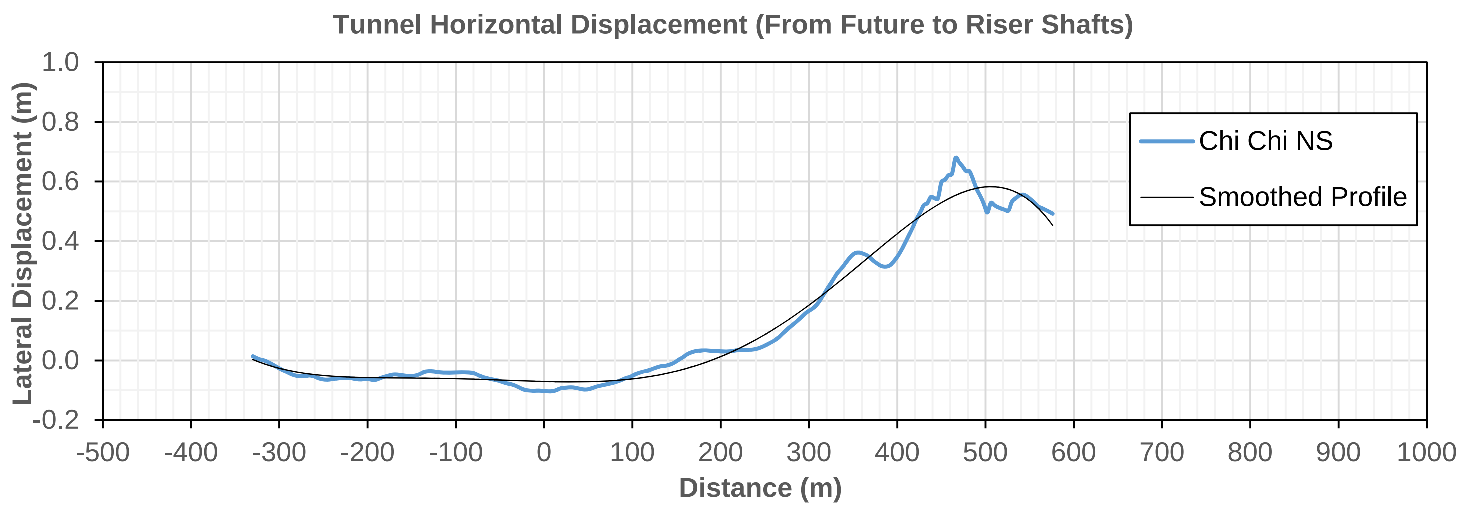
Note:

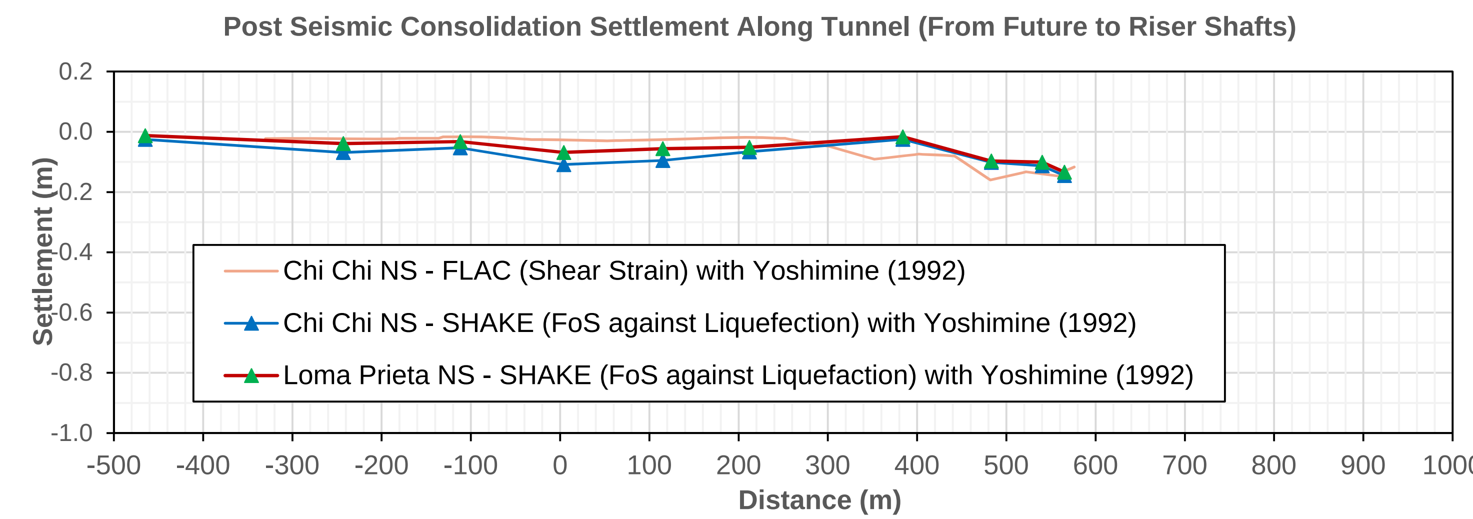
Shown in the plots illustrates the embedment of the shafts

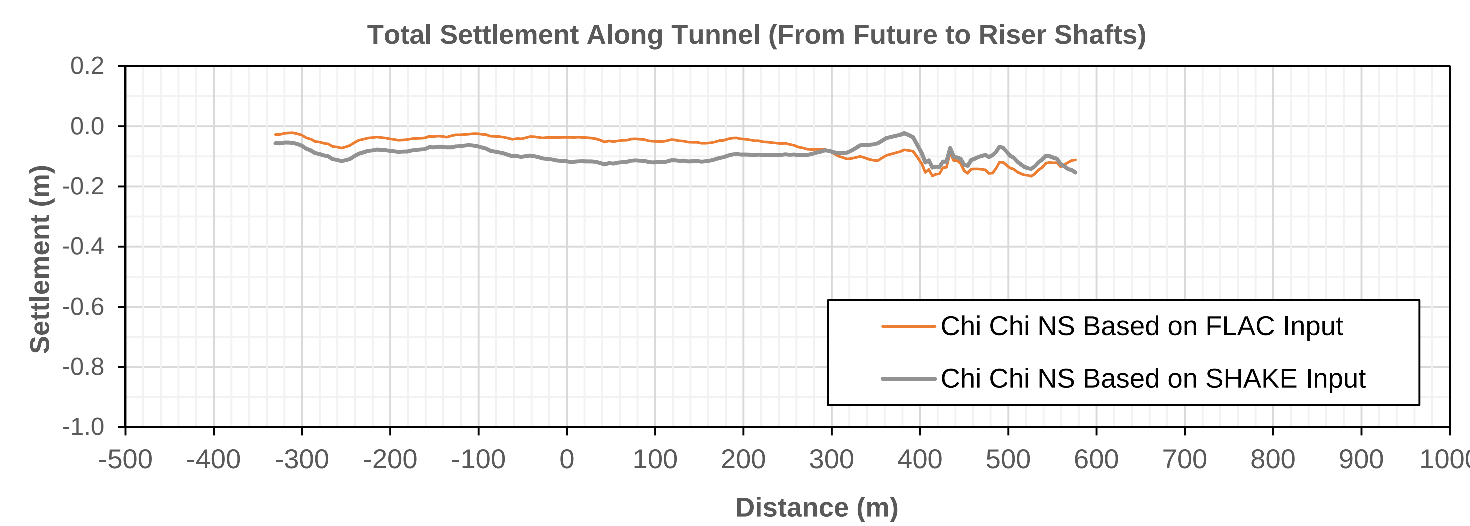
CLIENT **CDM SMITH CANADA ULC**	PROJECT **AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC**
CONSULTANT Golder Associates YYYY-MM-DD 2017-11-30 PREPARED M.S.K DESIGN M.S.K REVIEW Y.B APPROVED V.F	TITLE **Horizontal and Vertical Displacement Profiles SSI Analyses - Shaft Locations**
	PROJECT No. 1525010 · Phase 2100 · Rev A · FIGURE **5-60**











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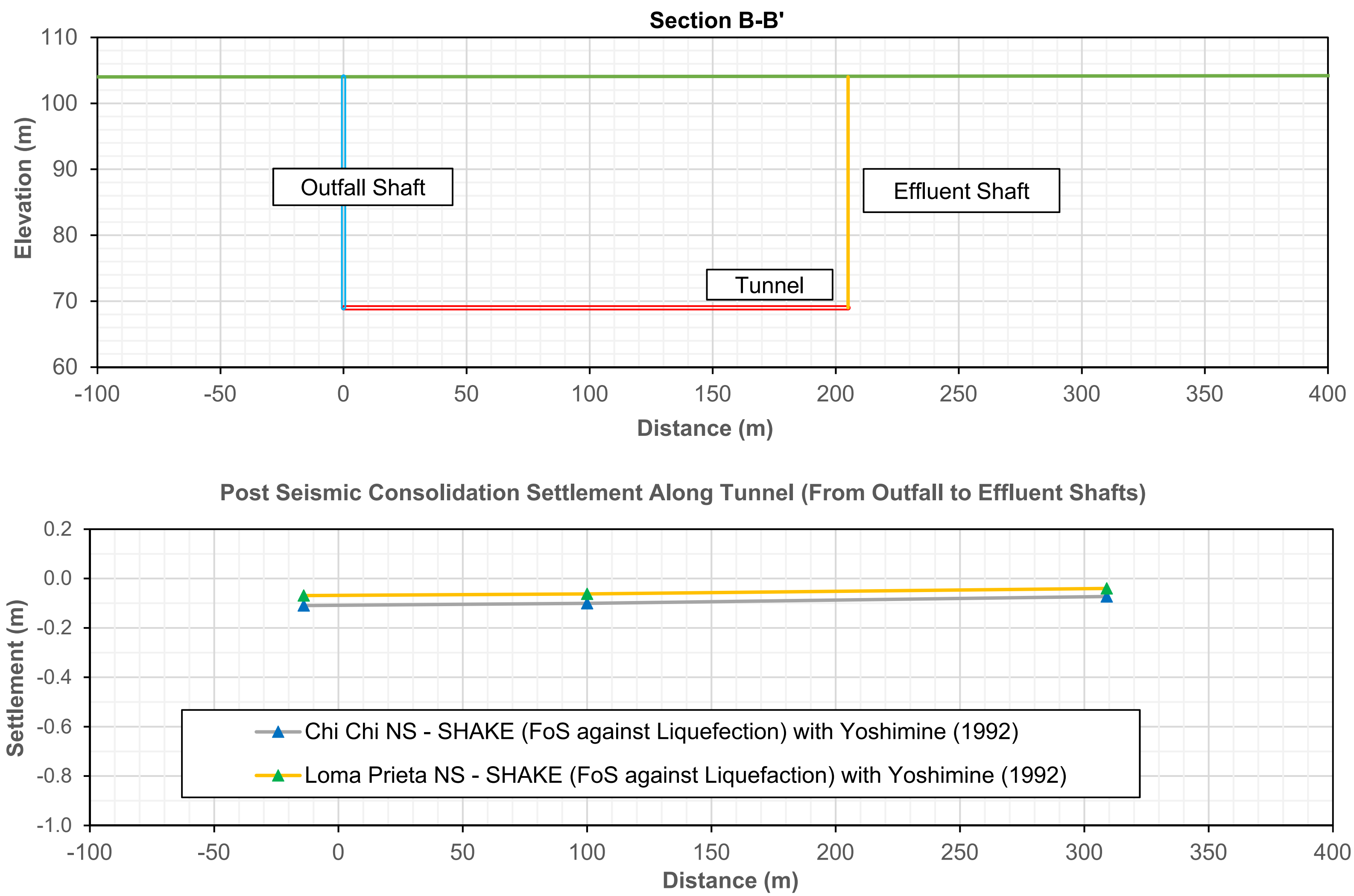
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Horizontal and Vertical Displacement Profiles Outfall Tunnel

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-61



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DELTA, BC

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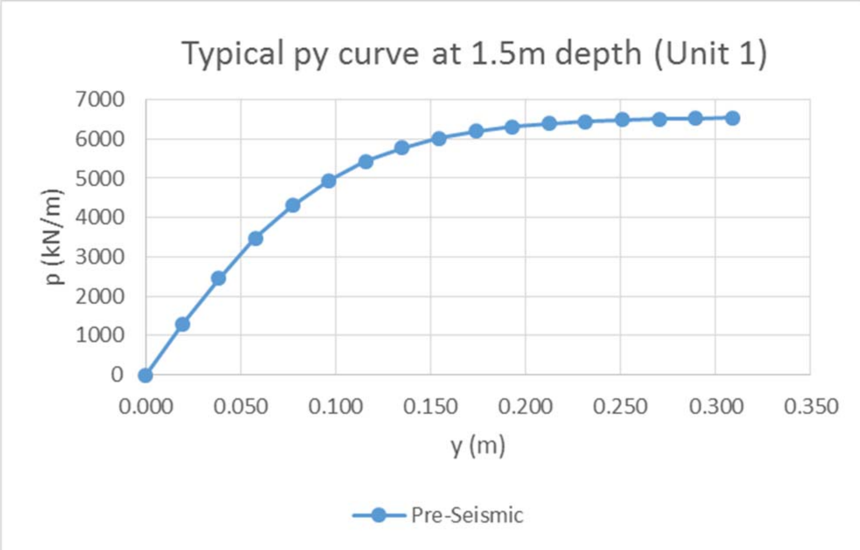
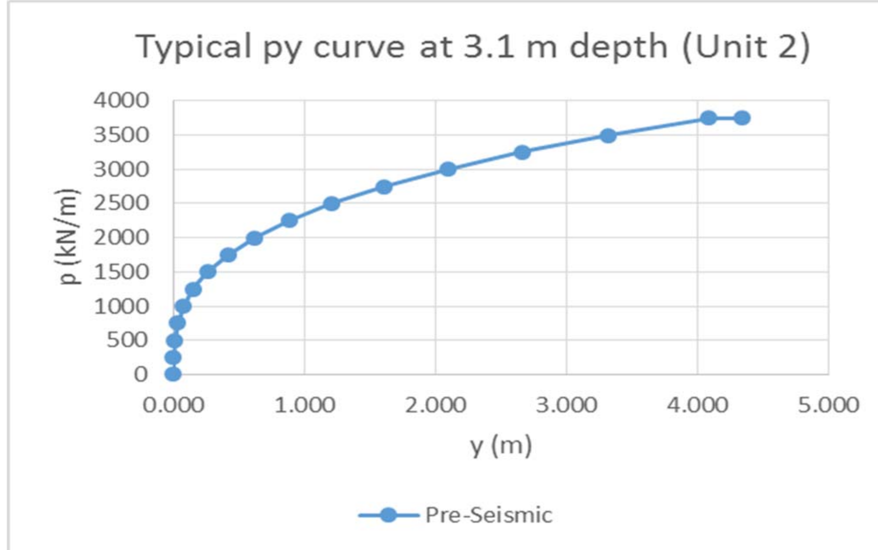
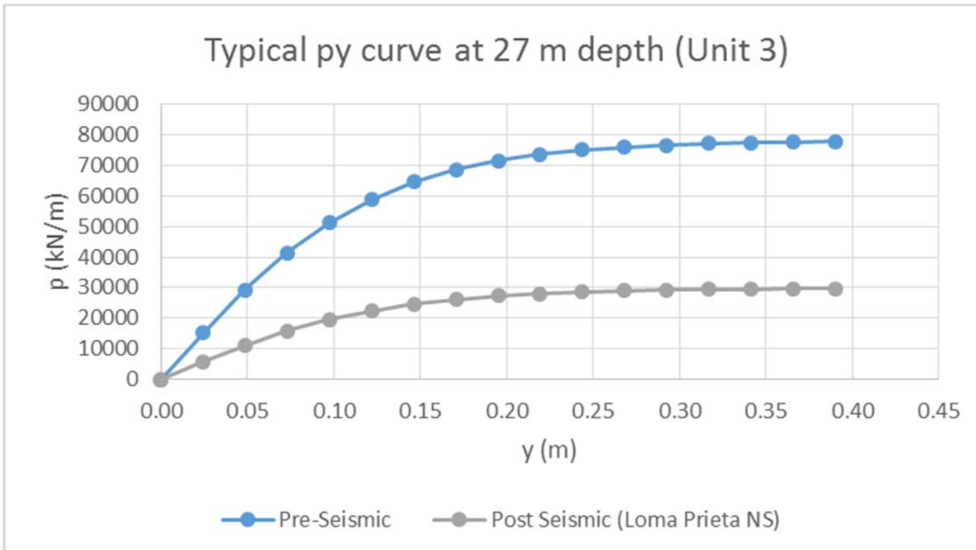
YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Vertical Displacement Profiles
Effluent Tunnel

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	**5-62**







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DELTA, BC**

CONSULTANT

Golder Associates

YYYY-MM-DD	29 NOV 2017
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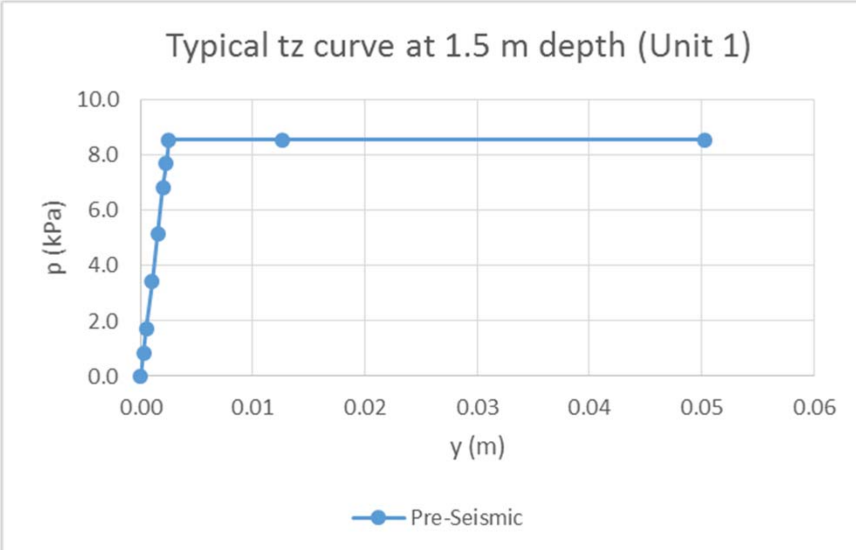
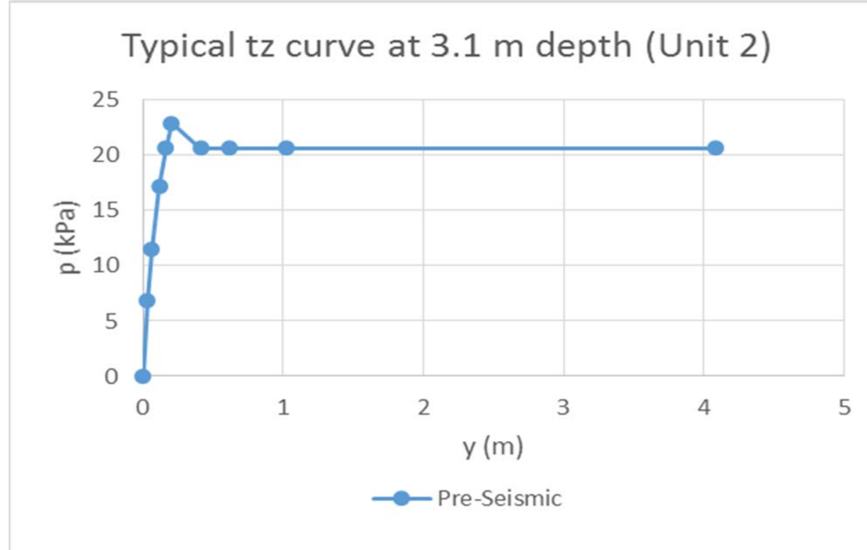
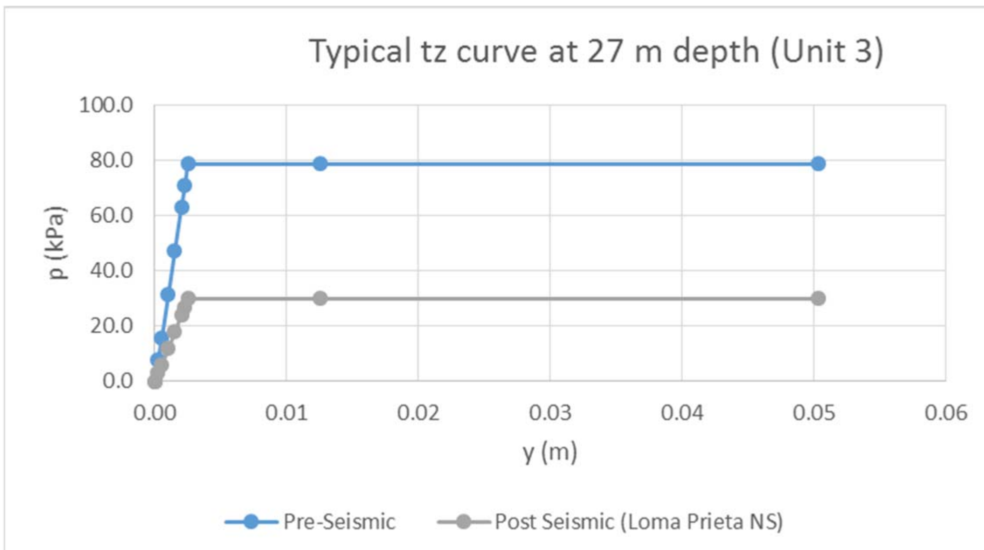
DESIGN	YEW
REVIEW	XXX
APPROVED	XXX

TITLE
Typical py Curves

PROJECT No.	PHASE	Rev	FIGURE
1525010	2100	A	**5-63**







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PROJECT
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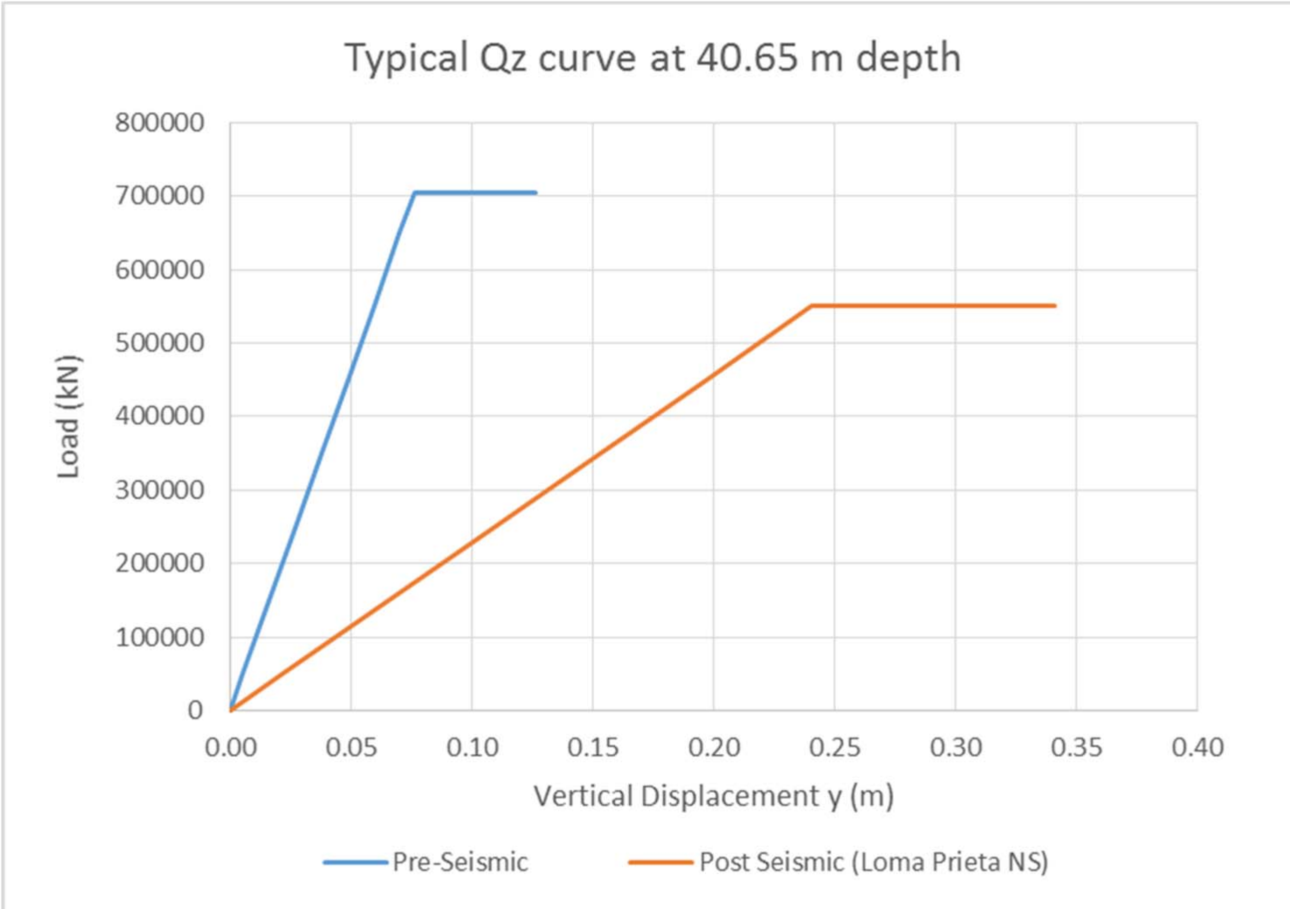
DELTA, BC

CONSULTANT
Golder Associates

YYYY-MM-DD	29 NOV 2017
PREPARED	YEW
DESIGN	YEW
REVIEW	XXX
APPROVED	XXX

TITLE
Typical tz Curves

PROJECT No.	PHASE	Rev	FIGURE
1525010	2100	A	**5-64**



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AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

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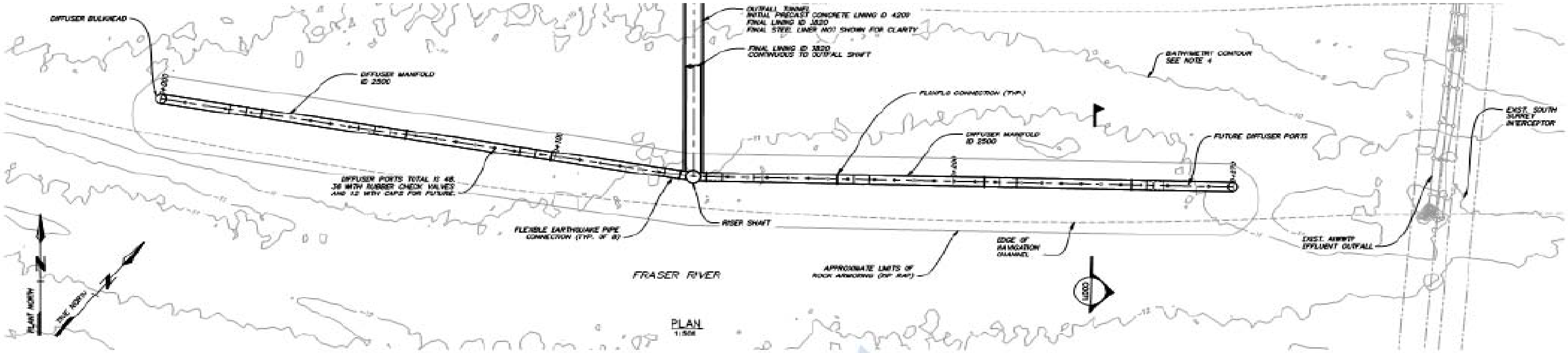
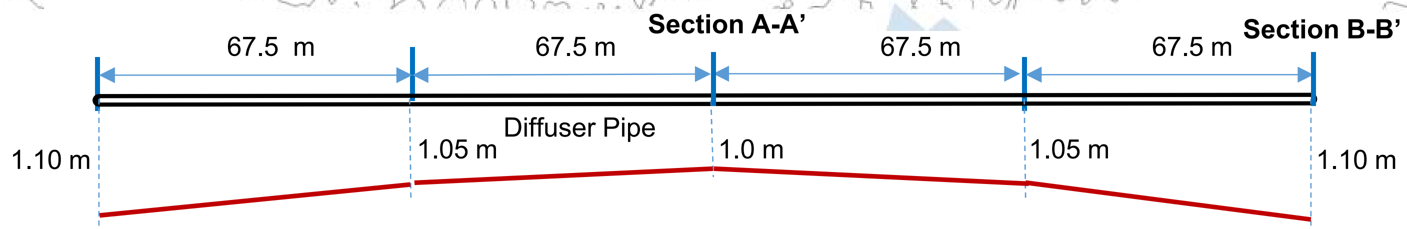
YYYY-MM-DD	29 NOV 2017
PREPARED	YEW
DESIGN	YEW
REVIEW	XXX
APPROVED	XXX

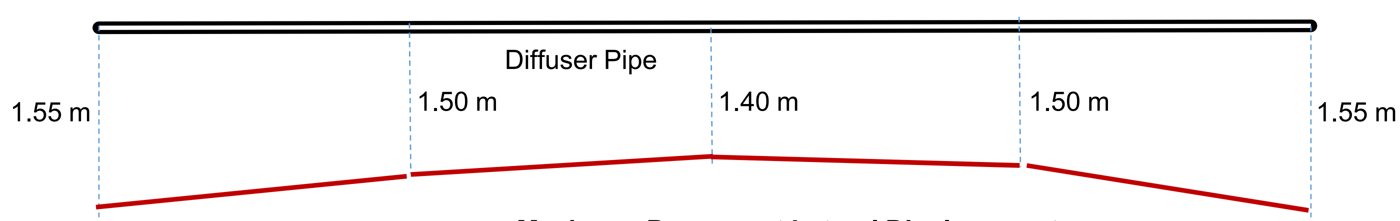
TITLE
Typical Qz Curve

PROJECT No.	PHASE	Rev	FIGURE
1525010	2100	A	**5-65**





Average Permanent Lateral Displacements



Maximum Permanent Lateral Displacements

Notes:
The Diffuser Profile was obtained from Drawing No. A61 X-C-014 provided by CDM Smith Canada ULC

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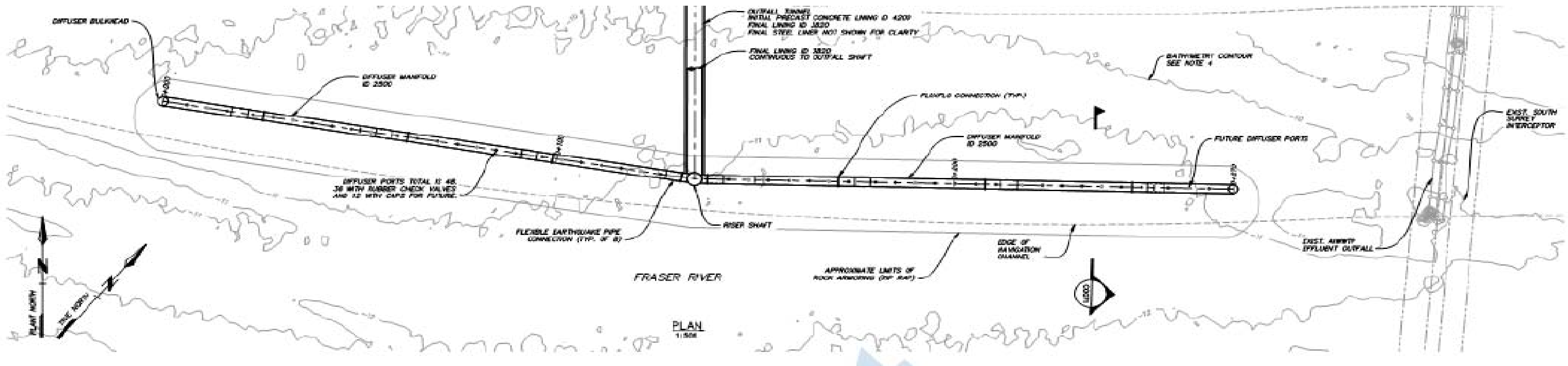
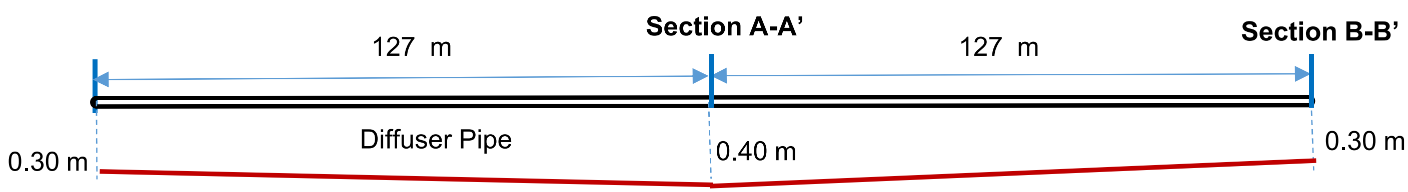
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Horizontal Displacement Profiles Diffuser Manifold

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	**5-66**





Average Post-seismic Vertical Displacements

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PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

Notes:
The Diffuser Profile was obtained from Drawing No. A61 X-C-014 provided by CDM Smith Canada ULC

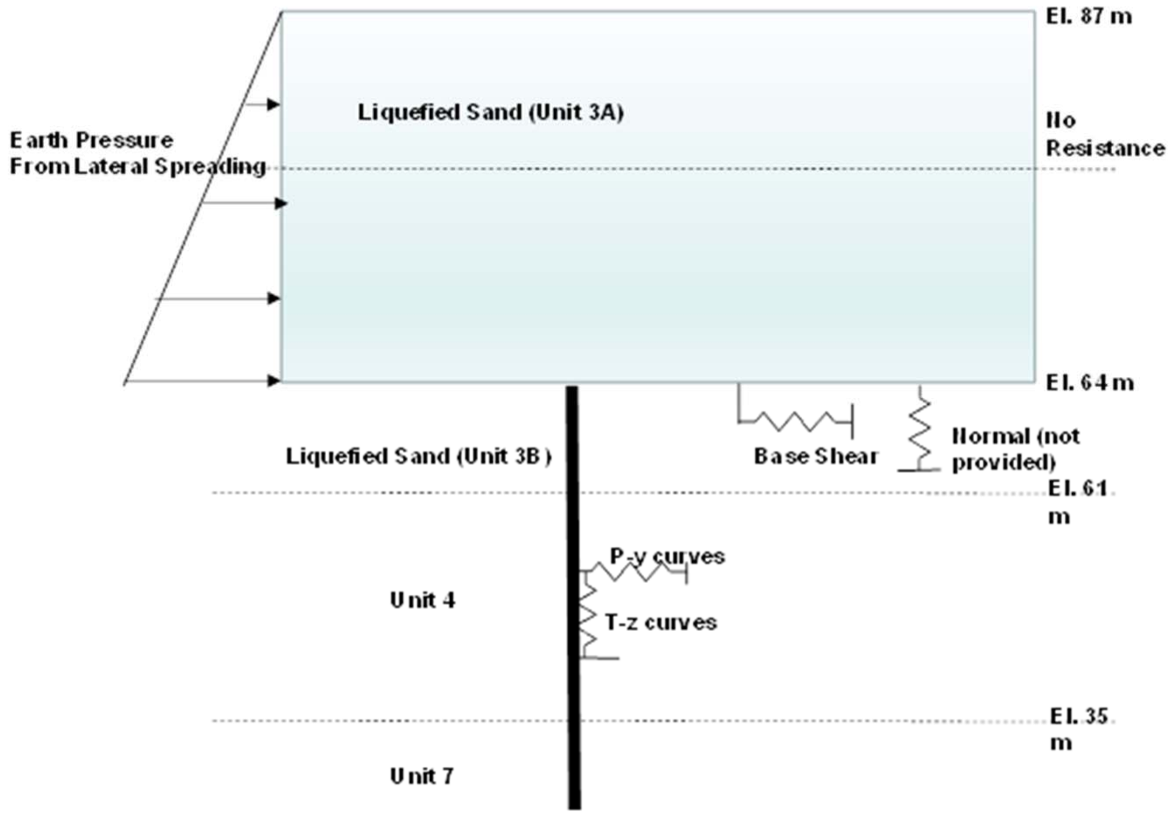
CONSULTANT

Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Post-seismic Settlement Profiles
Diffuser Manifold

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	**5-67**



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PROJECT
AIWWP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-30
PREPARED	M.S.K
DESIGN	M.S.K
REVIEW	Y.B
APPROVED	V.F

TITLE
Free Body Diagram – Riser Shaft

PROJECT No.	Phase	Rev	FIGURE
1525010	2100	A	5-68

APPENDIX A

Interim Technical Memorandum – 2D Ground Deformation Analyses – 2010 NBCC and 2015 NBCC

TECHNICAL MEMORANDUM

DATE 15 September 2016

REFERENCE 1525010-046-TM-RevA

TO John Newby
Project Manager
CDM Smith Canada ULC

FROM Viet Tran/Mahmood Seid-Karbasi
Viji Fernando/Upul Atukorala

EMAIL Viji_Fernando@golder.com

RESULTS OF TWO DIMENSIONAL GROUND RESPONSE ANALYSES
ANNACIS ISLAND WWTP TRANSIENT MITIGATION AND OUTFALL, DELTA, BC

This is an interim Technical Memorandum presenting a summary of the results of the two-dimensional (2D) ground response analyses carried out to assess the liquefaction potential of site soils and the resulting permanent ground deformations along the proposed Option 6 outfall alignment for "as-is" ground conditions. The analyses were carried out for the design ground motions corresponding to a return period of 2,475 years consistent with the 2010 National Building Code of Canada (NBCC). The geotechnical parameters, as requested by McMillen Jacobs Associates (MJA), as input to the structural evaluation of the shafts and tunnel are also presented herein.

The analyses were carried out considering a section perpendicular to the river bank extending through Mungo's hole and a section perpendicular to the river bank through the edge of Mungo's hole to exclude the effect of the Mungo's hole. The locations of the sections are shown on Figure 1-1. The lateral ground deformations are expected to be influenced by the variations in the soil parameters as well as the input ground motions with respect to their polarity. The ground deformation analyses were carried out for the following cases:

- Case 1: Design SPT$(N_1)_{60}$ profile based on combined penetration resistance data from both onshore and offshore areas;
- Case 2: Design SPT$(N_1)_{60}$ profile same as Case 1 with the input ground motions applied with reversed polarity;
- Case 3: Design SPT$(N_1)_{60}$ profiles considering the penetration resistance data from the onshore and offshore areas separately; and
- Case 4: Design SPT$(N_1)_{60}$ profile same as Case 3 with the input ground motions applied with reversed polarity.

1.0 GROUND RESPONSE ANALYSES

Detailed ground response analyses were carried out to evaluate the permanent ground displacements under the 2,475 year design ground motions using the 2D finite difference computer code FLAC^{2D} (Version 7.0) developed by Itasca Consulting Ltd. FLAC allows the domain of interest to be modeled by elements or zones. Each element or zone behaves according to a prescribed stress-strain law in response to the applied forces or boundary conditions. In addition, via user-defined subroutines, the program allows the implementation of specific constitutive relations to appropriately model phenomena such as liquefaction and the associated softening and strength reductions in soils.

The site soils were modeled as Mohr-Coulomb materials to numerically simulate the characteristic behavior of the different materials under static loading conditions.

The user-defined constitutive model UBCSAND that is capable of capturing the liquefaction potential of granular soils was used to numerically simulate the characteristic behaviour of the existing fill and sand (Units 1 & 3) under seismic loading conditions. The primary input parameters for the UBCSAND model are as follows:

- Normalized standard penetration test (SPT) blow counts $(N_1)_{60}$ corrected for fines content;
- Internal friction angle (ϕ) and unit weights; and
- Hydraulic conductivity.

The organic silt/clayey silt layer (Unit 2) that is considered not liquefiable under the design ground motions was modeled as a non-linear Mohr-Coulomb material using a hyperbolic stress-strain relation and modified Masing's rule incorporated in the user-defined routine UBCHYST. The following input parameters were utilized to capture the dynamic behaviour of fine-grained soils using the UBCHYST model under dynamic loading:

- Unit weights and undrained shear strength;
- Small strain shear modulus;
- Modulus reduction and damping curves; and
- Hydraulic conductivity.

Both UBCSAND and UBCHYST models were developed by Prof. Dr. Peter M. Byrne and his colleagues at the University of British Columbia (UBC).

The marine deposits (Units 4 & 7) underlying the site, which are considered non-liquefiable under the design ground motions were modeled as Mohr-Coulomb materials with an equivalent linear approach (ELA) to simulate the non-linear cyclic behavior. The model includes shear modulus degradation with shear strain and shear strain-dependent damping using a three parameter hysteretic model built into FLAC.

The input parameters required for the ground response analyses have been derived using the data collected from the cone penetration tests (CPTs) and boreholes put down in the vicinity of the proposed Option 6 outfall alignment and are summarized in the Golder's Geotechnical Interpretive Report dated June 25, 2016. The input parameters used in the analyses are summarized in Table 1-1 below. The design SPT$(N_1)_{60}$ profiles used in the analyses are shown on Figure 1-2.

Table 1-1: Summary of Engineering Parameters

Soil Unit	Unit Description	SPT $(N_1)_{60cs}$ (blows/0.3 m)	Unit Wt. (kN/m³)	ϕ' (deg.)	Su (kPa)	K_h (m/s)	K_h/K_v (m/s)
Unit 1	Fill	20	19	35	-	1.3E-4	10
Unit 2	Organic Silt to Clayey Silt	-	17	--	40	3.0E-7	5
Unit 3	Fraser River Sand – Offshore	10 – 17	19	34 - 35	-	1.3E-4	10
	Fraser River Sand - Onshore	4 – 18			-	1.3E-4	10
Units 4 & 7	Clayey Silt to Silty Clay - Offshore	-	19	--	$0.26\sigma'_v$	4.0E-7	5
	Clayey Silt to Silty Clay - Onshore	-			$0.22\sigma'_v$	4.0E-7	5

Note: K_h is horizontal hydraulic conductivity and K_v is vertical hydraulic conductivity.

1.1 Model Calibration

The cyclic resistance ratio (CRR) vs. (N1)60-cs (clean sand) equation recommended by Idriss & Boulanger (2008) was used to calibrate the element behavior. In addition, the overburden correction factors recommended by Idriss & Boulanger (2014) were used to calibrate the element behavior for stress levels other than 100 kPa. When following this methodology, a correction factor larger than unity is applied for confining stress levels less than 100 kPa.

The parameters of the UBCSAND model were obtained by carrying out a series of single element simple shear test simulations using the computer code FLAC and the user-defined UBCSAND model. The characteristic CRR value of each soil unit was used as benchmark in the single element tests. The parameters of the UBCSAND constitutive model were adjusted in order to predict triggering of liquefaction in approximately 15 cycles of loading at a cyclic stress ratio equal to the characteristic CRR value and a good agreement between the Idriss & Boulanger (2014) CRR vs. $(N_1)_{60}$ chart data and model predictions was achieved as shown on Figure 1-3.

A comparison of the stress path and excess pore pressure response of an element test with results from a laboratory cyclic simple shear test (Byrne et al, 2004) is shown on Figure 1-3. The results indicate that UBCSAND can adequately capture the load deformation response observed in the laboratory both prior to and following triggering of liquefaction. The overburden correction factors recommended by Idriss & Boulanger (2014) and those predicted by the UBCSAND model are also shown on Figure 1-3.

Typical modulus reduction and damping curves available from literature for Unit 2 and those computed from FLAC using the UBCHYST model are shown on Figure 1-4 while the modulus reduction and damping curves from literature and computed using the built-in model utilizing ELA for Units 4 and 7 are shown on Figure 1-5.

1.2 Input Ground Motions

3 sets of ground motions with each set comprising two orthogonal time-histories to represent the crustal and inslab earthquakes and 1 set ground motions comprising two orthogonal time-histories to represent the interface (subduction) earthquakes were used in the ground response analyses for the design ground motions consistent

with the 2010 NBCC. Further details on the ground motions are provided in Golder's Technical Memorandum "Site Seismicity, Seismic Performance Expectations and Input Ground Motions, Annacis Outfall WWTP Transient Mitigation and Outfall System" dated July 6, 2016. Table 1-2 provides a summary of the seed motions selected for development of acceleration-time histories for analyses.

Table 1-2: Details of the Ground Motions - Stage V Expansion [2,475-Year Return Period]

Earthquake	Date of Earthquake	Magnitude	Seismic Source
Loma Prieta	October 18, 1989	M7.0	Crustal and Inslab
Landers	June 28, 1992	M7.3	
Chi Chi	September 20, 1999	M7.6	
Mexico	September 19, 2985	M8.1	Subduction

1.3 FLAC^{2D} Models and Dynamic Analyses

Following completion of the single element calibration process discussed above, 2D models were generated in the computer code FLAC as a collection of single elements along the soil profile. The ground response was simulated in the different soil zones as earthquake loading progressed while complying simultaneously with equilibrium, strain compatibility, boundary conditions, and the prescribed stress-strain laws.

The finite difference models developed for the 2D ground response analyses are shown on Figures 1-6 and 1-7 for the sections extending south through Mungo's hole and at the edge of Mungo's hole, respectively. The lower model boundary extends to the top of the Class C ground conditions. Competent ground comprising Pleistocene deposits was not encountered along the proposed alignment except at one location west of the proposed outfall shaft location. In addition, no site-specific shear wave velocity measurements are available at depth across the alignment to confirm the Class C conditions; therefore, correlations developed by Hunter (1995) for the Fraser River Delta were used to establish the Class C ground conditions across the site as shown on Figure 1-8.

The analyses were conducted using the effective-stress approach where generation, distribution and dissipation of excess pore water pressures of the sand-like layers during earthquake shaking were accounted for.

The 2D ground response analyses were carried out for a series of six spectrum-compatible earthquake records representing the crustal and inslab seismic sources, and two earthquake records representing the subduction event as input motions. These spectrum compatible ground motions correspond to Class C ground conditions excluding the effects of overburden soils, and for that reason a compliant base boundary condition was used to apply the input motions at the base. An average shear wave velocity of 450 m/s was used for the Class C ground condition. For analysis purposes, the water level was considered at CVD28GVRD elevation 101 m.

The coupled stress-flow response was simulated by taking into account the dissipation and/or redistribution of pore pressures that occur with time and the strains caused by such changes in pore pressure. Two mechanical effects are considered in this case: (i) changes in pore pressure induced by volume changes, and (ii) changes in effective stresses caused by pore pressure changes. The effects induced by the pore pressure dissipation/redistribution process, seepage forces, and the loading conditions were accounted for at every step of calculation, capturing the coupled stress-flow response. The groundwater flow formulation in FLAC is based on Darcy's Law for an anisotropic porous medium and the Continuity Equations. The characteristic hydraulic conductivity values were established for each soil unit based on comparing grain size data and correlating to the results of the hydrogeological tests. The hydraulic conductivity values assigned to the different soil units are also summarized in Table 1-1.

1.4 Results of 2D Ground Response Analyses

The ground response analyses associated with the crustal and inslab motions predict soil liquefaction (defined herein as zones developing an excess pore pressure ratio, a ratio between the excess pore water pressure and the overburden stress, of 85% or more at the end of shaking) through the entire sand deposit onshore and at riser shaft locations for all the cases considered in the analyses. The extent of liquefaction is generally limited to the upper 15 m within the sand deposit under the subduction event. The extent of liquefaction predicted for Cases 1 through 4 at the shaft locations and along the tunnel are shown in Figures 1-9 through 1-14 for all eight input ground motion time-histories.

It is noted that the lateral displacement and settlement profiles for the effluent shaft were established from the second FLAC model extending through the edge of the Mungo's hole and the analyses were carried out only for Case 1. The average displacement profiles associated with Case 1 at the PDBCO and effluent shafts are shown on Figure 1-15 and there is no significant difference in the displacement and settlement profiles and therefore, the displacement and settlement profiles established for the PDBCO shaft can also be used for the effluent shaft.

The computed permanent lateral ground displacement and vertical settlement profiles established at the shaft locations for Cases 1 through 4 are presented in Figures 1-16 through 1-19 while the displacement and settlement profiles along the tunnel for the same cases are shown on Figures 1-20 through 1-23.

The results of the analyses indicate the following:

- The maximum permanent lateral displacement resulting from the crustal and inslab ground motions is computed to be in the order of 0.3 m at the post-disaster bypass conduit (PDBCO) and effluent shafts while the lateral displacement is computed to be in the order of 0.4 m at the outfall shaft. The lateral displacement due to subduction event is computed to be in the order of 0.05 m at the on-land shafts;
- The maximum permanent vertical settlements at the on-land shafts are computed to be in the order of 0.8 m due to the crustal and inslab motions and they are computed to be in the order of 0.4 m due to the subduction event;
- The maximum permanent lateral displacement resulting from the crustal and inslab ground motions is computed to be in the order of 1.8 m at the riser shaft while that is computed to be in the order of 0.4 m due to the subduction event;
- The maximum permanent vertical settlement at the riser shaft is computed to be in the order of 0.6 m due to the crustal and inslab motions and it is computed to be in the order of 0.2 m due to the subduction event;
- The maximum permanent lateral displacements resulting from the crustal and inslab ground motions vary up to 1.2 m along the tunnel alignment and generally insignificant further away in land from the foreshore area while those from the subduction event along the tunnel alignment are computed to be insignificant; and
- The permanent vertical settlements along the tunnel vary from 0.05 m to 0.2 m with the maximum settlement occurring at the river bank and within the river channel.

2.0 SOIL-STRUCTURE INTERACTION MODELLING

The following sections provide our input to the soil-structure interaction (SSI) analyses, which we understand will be undertaken by MJA. The displacement profiles and the stiffness parameters as requested by MJA are summarized in spreadsheets and the electronic files associated with the spreadsheets are included as part of this technical memorandum.

2.1 Input to SSI Analyses

Lateral displacement profiles induced by the different earthquake input motions vary along the shafts and at the tunnel alignment. The extent of soil liquefaction also varies with each input motion. Consequently, the kinematic loads that are imposed on the shafts will be dependent on the non-linear soil reactions that develops along the soil-shaft interface (which, in turn, is dependent on the extent of soil liquefaction) and the computed profile of lateral displacement. In our experience, the specific displacement profile that will result in the largest bending moments and shear forces in the shaft cannot be pre-determined without some screening-level simplified analyses.

Consistent with similar projects completed by Golder in the past, we recommend that the soil-structure interaction analyses be carried out in stages:

Stage-1: Using simplified models where the shafts and tunnel are modelled using "stick elements" in combination with applicable non-linear p-y curves. In this case, the structure response may be evaluated for all different ground deformation profiles and the sensitivity of the structure to different ground displacements profiles can be established.

Stage-2: Using rigorous analysis of the soil-structure system with 2D/3D soil-structure interaction models utilizing the stiffness parameters established at the end of shaking provided in the electronic format. In this case, consideration may be given to the critical displacement profiles identified in Stage-1 above relative to the structure response. Alternatively, consideration may be given to the deformation profiles described below:

a) **On-land Shafts**: Utilize the mean permanent lateral displacement and vertical settlement profiles computed from Case 1 associated with crustal and inslab ground motions. In addition, use the displacement profiles associated with Case 1 - Loma Prieta NS and Case 2 - Chi Chi NS together with the mean settlement profiles to assess the sensitivity of the forces with respect to the functional requirements under the design ground motions. The computed mean as well as the displacement profiles associated with Case 1 - Loma Prieta NS and Case 2 - Chi Chi NS are shown in Figures 2-1 and 2-2, for the PDBCO and effluent shafts and the outfall shafts, respectively.

b) **Riser Shaft**: Utilize the mean permanent lateral displacement and vertical settlement profiles computed from Case 1. In addition, use the displacement profiles associated with Case 1 - Landers NS and Case 1 – Landers EW together with the mean settlement profiles to assess the sensitivity of the forces with respect to the functional requirements under the design earthquake. The computed mean as well as the displacement profiles associated with Case 1 - Landers EW and NS are shown in Figure 2-3.

c) **Tunnel**: Utilize the mean permanent lateral displacement and vertical settlement profiles computed from Case 1. In addition, use the displacement profiles associated with Case 1 - Landers EW and Case 1 – Landers NS together with the mean settlement profiles to assess the sensitivity of the forces with respect to the functional requirements under the design earthquake. The computed mean as well as the displacement profiles associated with Case 1 - Landers EW and NS are shown in Figure 2-4.

2.2 Direction of Movements and Resultant Movements

Evidence from past earthquakes for sites underlain by liquefiable soils indicates that liquefaction-induced ground movements primarily occur in a direction perpendicular to the river banks. These studies focus on visible, shallow or surficial movements. We are unaware of studies on displacement patterns in soils where liquefaction has occurred at depths in the order of 20 m to 30 m below ground surface.

Considering that the design earthquake scenario for the proposed outfall is a rare event and that the cross section analyzed is approximately perpendicular to the river bank, we recommend that the permanent displacements transverse to the tunnel alignment be taken as 50% of the computed mean permanent displacements in the longitudinal direction. The transverse movements could occur in the upstream as well as the downstream direction. The same recommendation would be applicable for the riser shaft. The lateral displacements in the transverse direction for the on-land shafts are expected to be in the same order of the displacement in the longitudinal direction.

2.3 Vertical Input Ground Motions

It is noted that the vertical input ground motions can be provided as input to the SSI analyses if they are considered necessary.

3.0 CLOSURE

We trust that the information presented in this Technical Memorandum is sufficient for your immediate requirements. Please do not hesitate to contact us if you have questions or require clarification of contents.

GOLDER ASSOCIATES LTD

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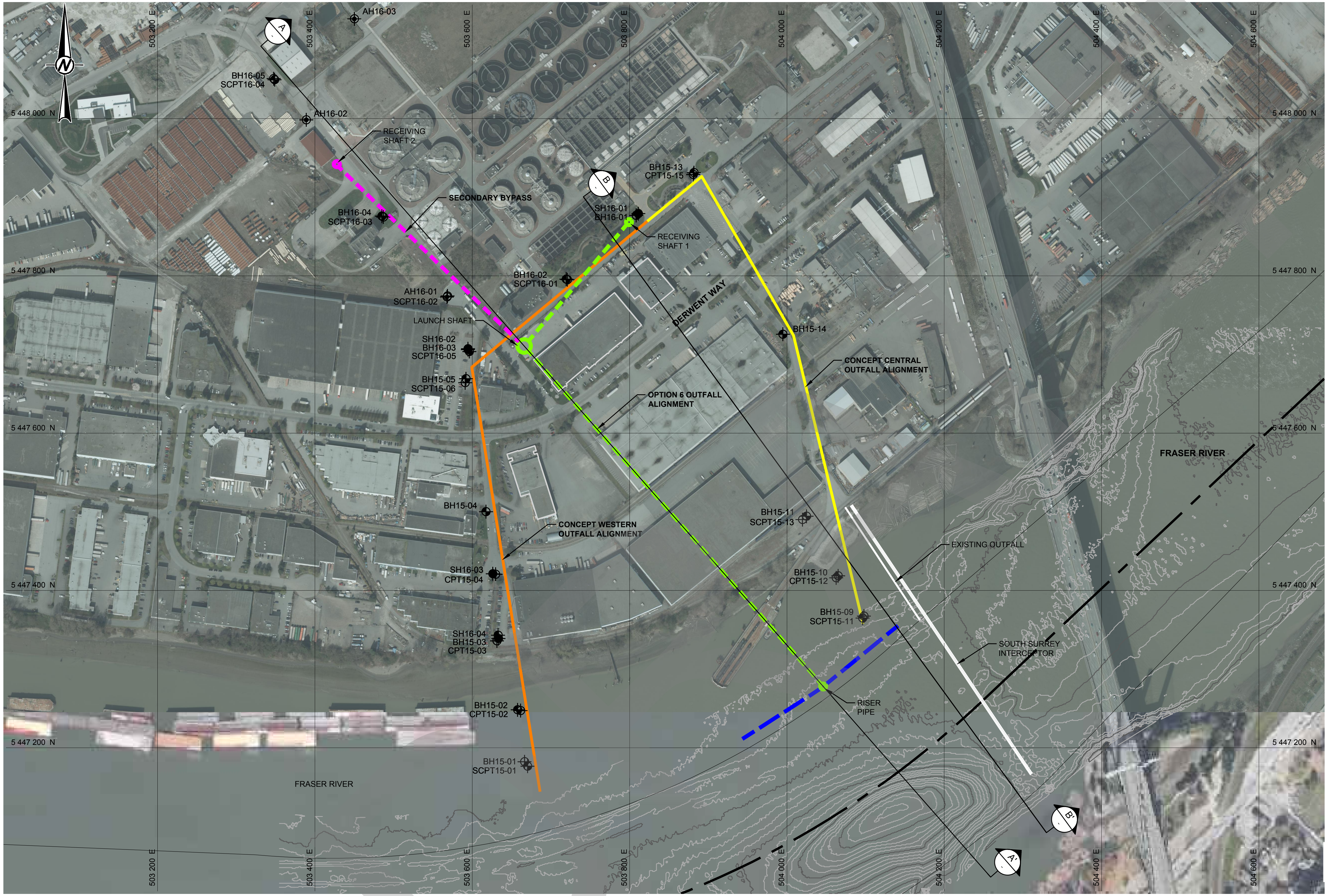
VT/MSK/VF/UA/sq

Attachments: Figures 1-1 through 1-23
Figures 2-1 through 2-4
Excel Files: Displacement Profiles Cases 1 & 2.xlsx
Displacement Profiles Cases 3 & 4.xlsx
Recommended Disp Profiles for SSI Analyses.xlsx
Strength and Stiffness Case 1.xlsx
Strength and Stiffness Case 2.xlsx
Strength and Stiffness Case 3.xlsx
Strength and Stiffness Case 4.xlsx

o:\final\2015\3 proj\1525010 cdm_annacis island wwtp\1525010-046-tm-reva\1525010-046-tm-reva 2d deformation results 15sep_16.docx



DRAFT

0 100 200
1:2,500 METRES

NOTE(S)

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

REFERENCE(S)

1. BASE DRAWING AND ORTHOPHOTO PROVIDED BY BLACK & VEATCH
FILE: ANNACISNAD27Z10GRND.TIF, ANNACIS-RIVER.TIF, C000A_XXXXX-1.DWG

CLIENT
CDM SMITH CANADA ULC

CONSULTANT

Golder Associates

YYYY-MM-DD	2016-09-14
DESIGNED	V. FERNANDO
PREPARED	S. REDDY
REVIEWED	
APPROVED	

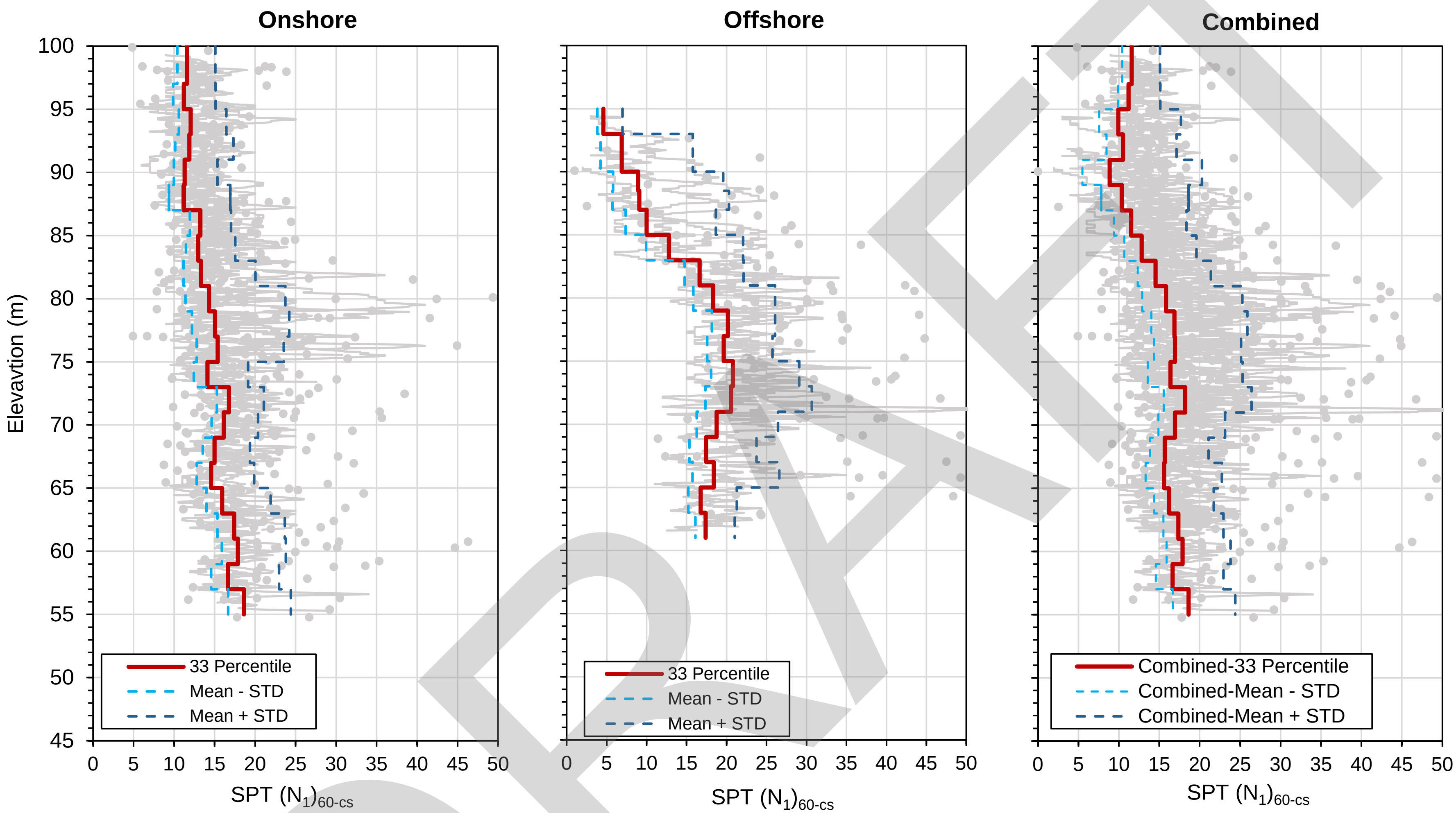
PROJECT
ANNACIS ISLAND WWTP TRANSIENT MITIGATION AND OUTFALL
DELTA, B.C.

TITLE
LOCATION OF CROSS SECTIONS

PROJECT NO.	PHASE	REV.	FIGURE
1525010	2000	A	1-1

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

Path: \\golder.gds\gal\burnaby\CAD-GIS\Client\CDM_Smith\Annacis_Island\99_PROJECTS\1525010_WWTP\02_PRODUCTION\2000\ | File Name: 1525010-2000-002.dwg



PROJECT	
	CDM SMITH CANADA ULC **AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM** **DELTA, BC**
TITLE	**SPT $(N_1)_{60\text{-}CS}$ Profiles**

Golder Associates

PROJECT No.	1525010		PHASE No. 2100
DESIGN	VT	15SEP16	SCALE NTS / REV.
CADD	----	----	**FIGURE 1-2**
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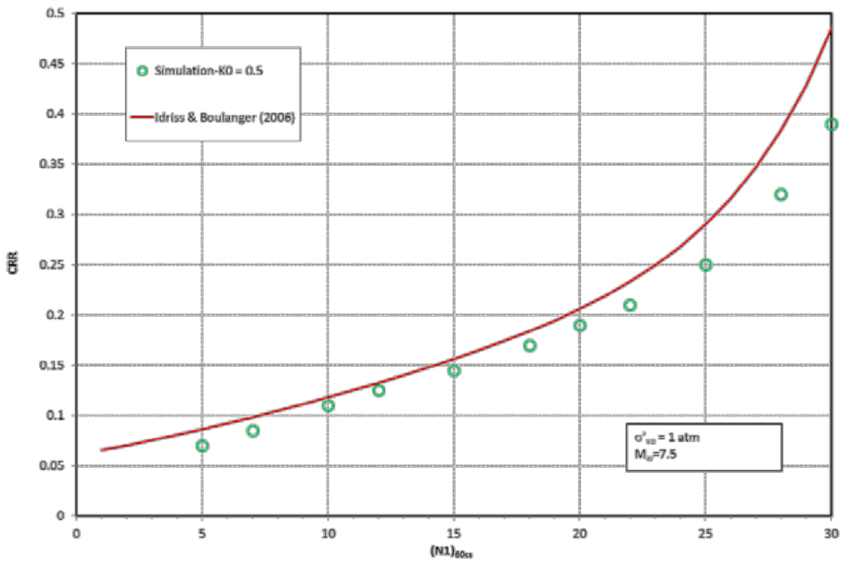
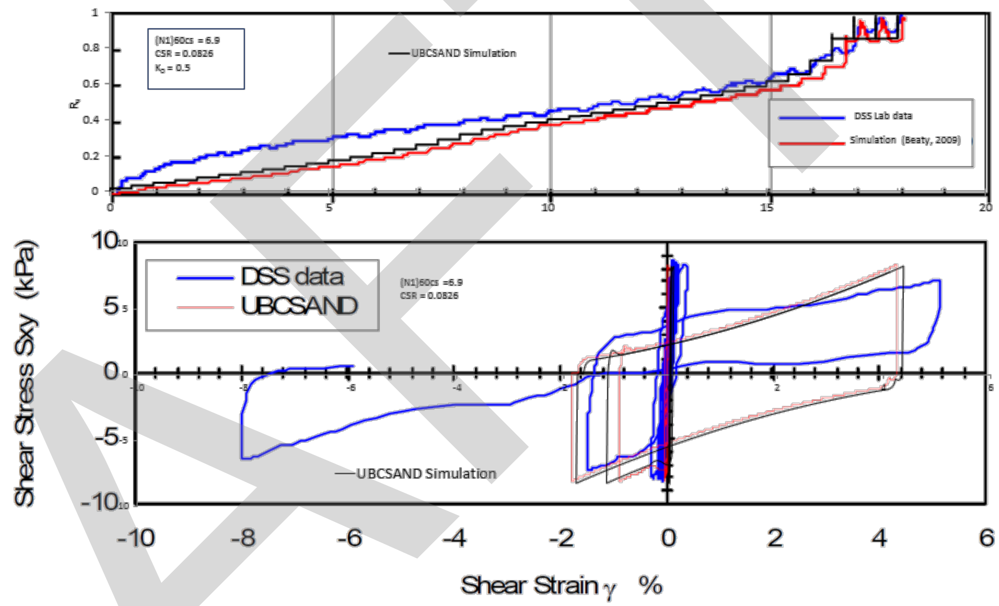
REVIEW	UA	15SEP16	

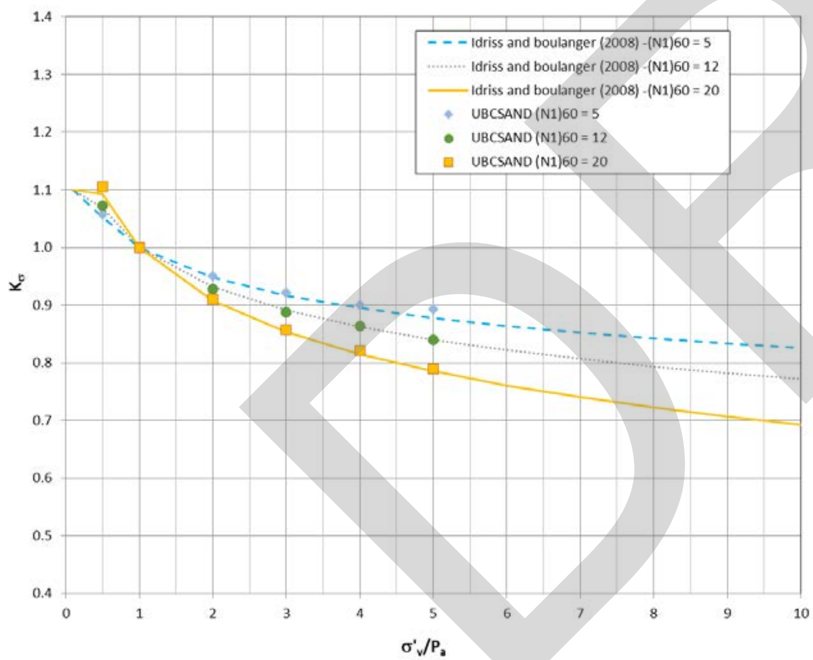
Semi-Empirical Triggering Chart vs. UBCSAND



Laboratory vs. UBCSAND



Confining Stress Effect – Empirical vs. UBCSAND



PROJECT	CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC				
TITLE	Model Calibration – UBCSAND Model				
Golder Associates	PROJECT No.	1525010		PHASE No. 2100	
	DESIGN	VT	15SEP16	SCALE NTS	REV.
	CADD	----	----	FIGURE 1-3	
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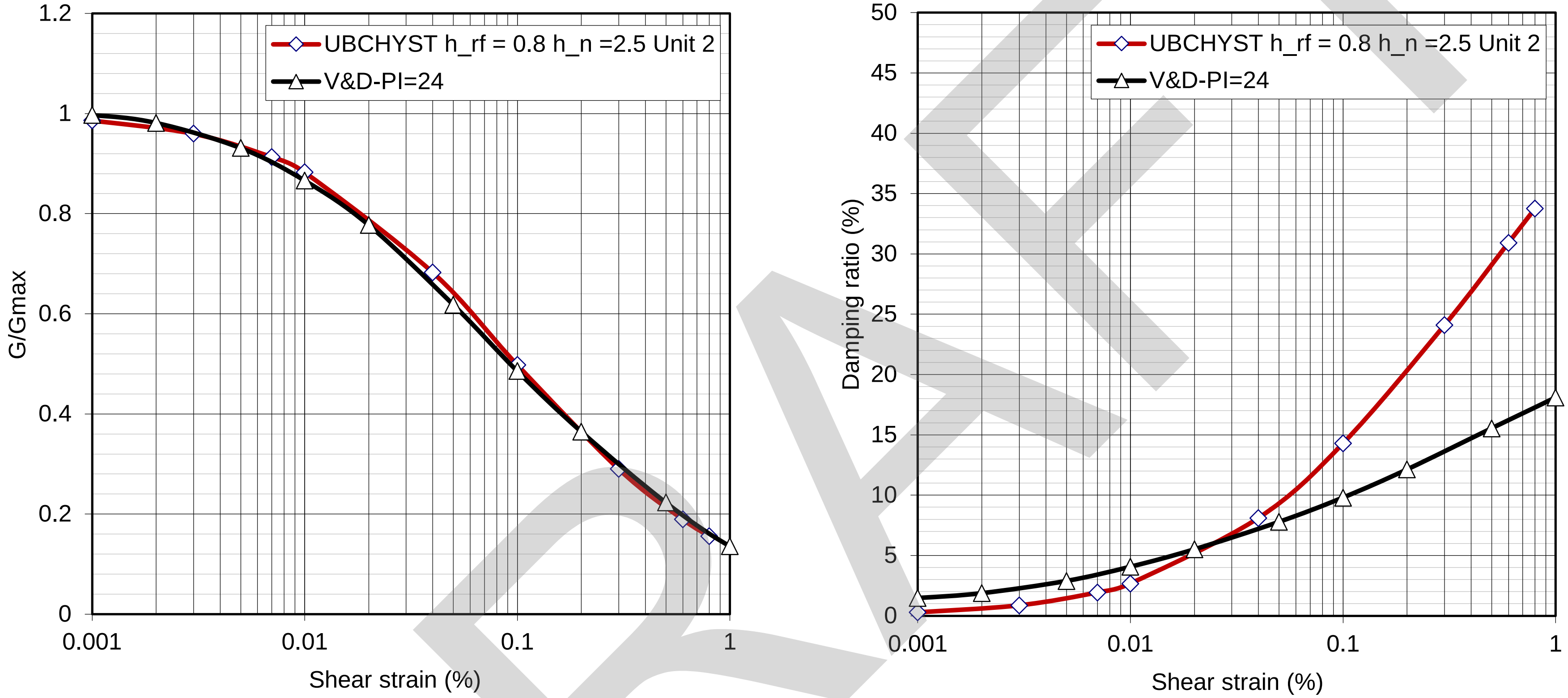

PROJECT

CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE

Model Calibration – UBCHYST Model (Unit 2)

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
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CADD	----	----	**FIGURE 1-4**	
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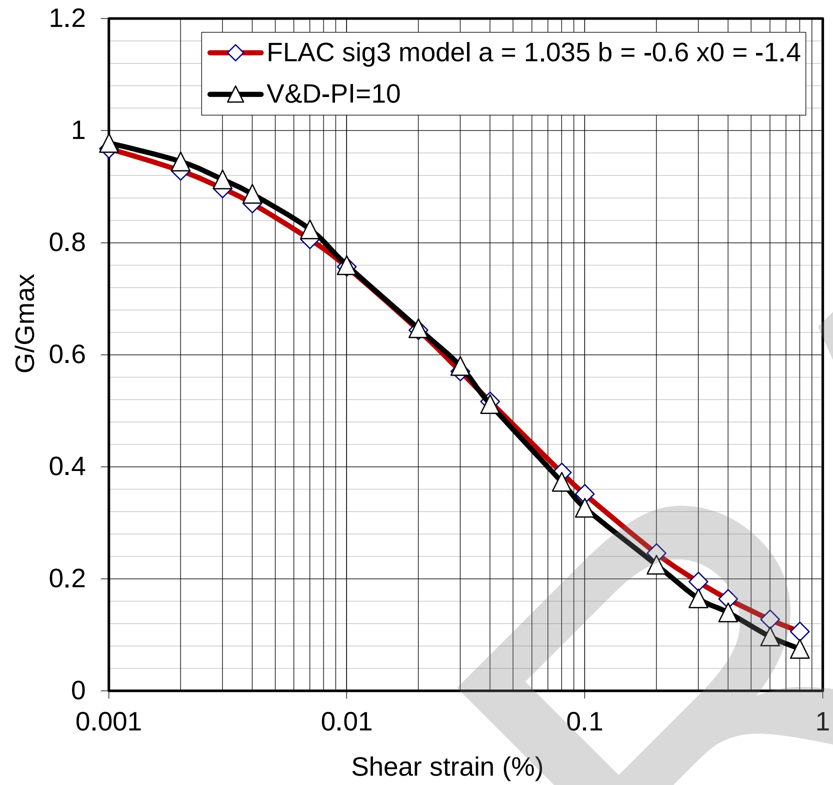

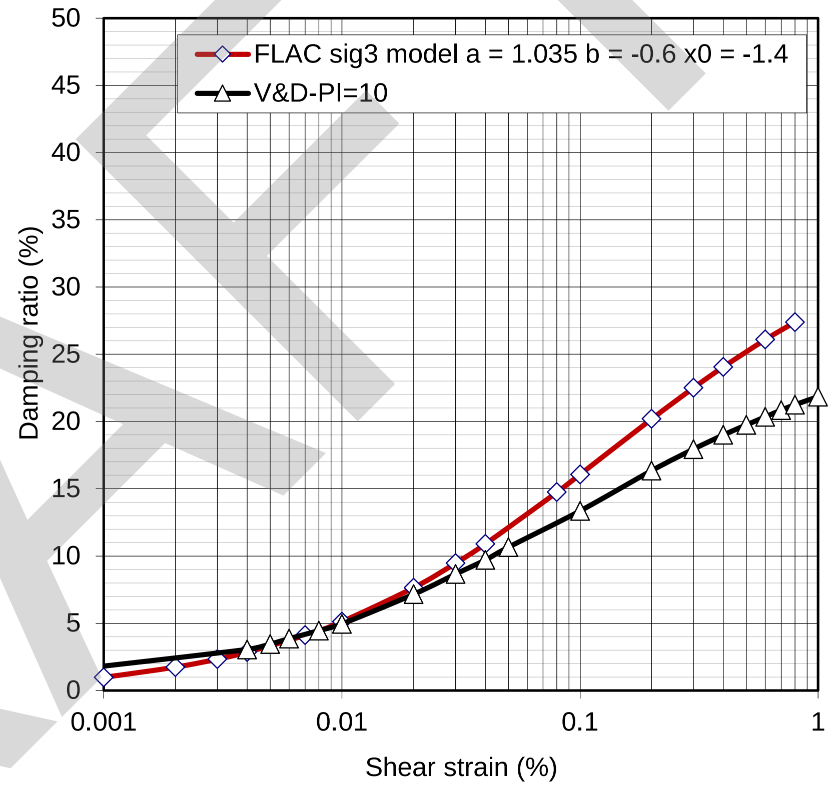

PROJECT
CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Model Calibration – FLAC SIG3 Model (Unit 4)

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
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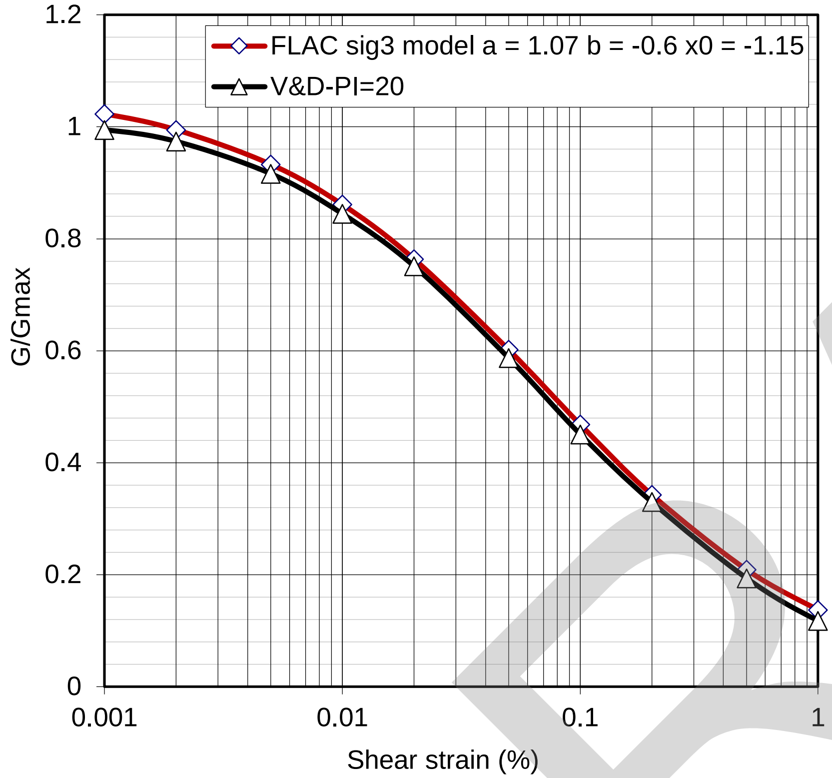
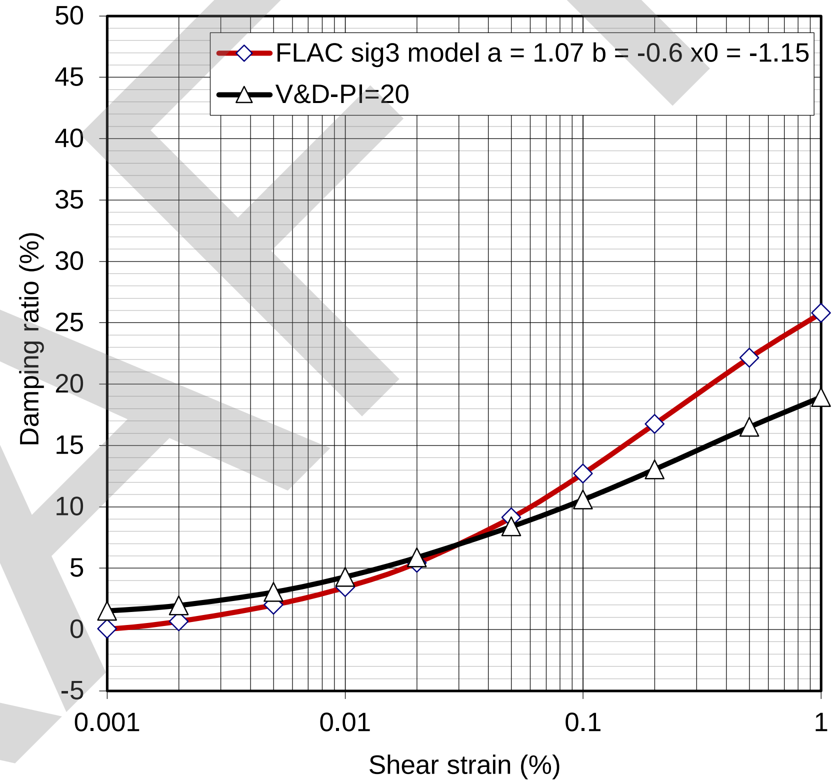
REVIEW	UA	15SEP16		





DRAFT

PROJECT
CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Model Calibration – FLAC SIG3 Model (Unit 7)

Golder Associates

PROJECT No.	1525010	
DESIGN	VT	15SEP16
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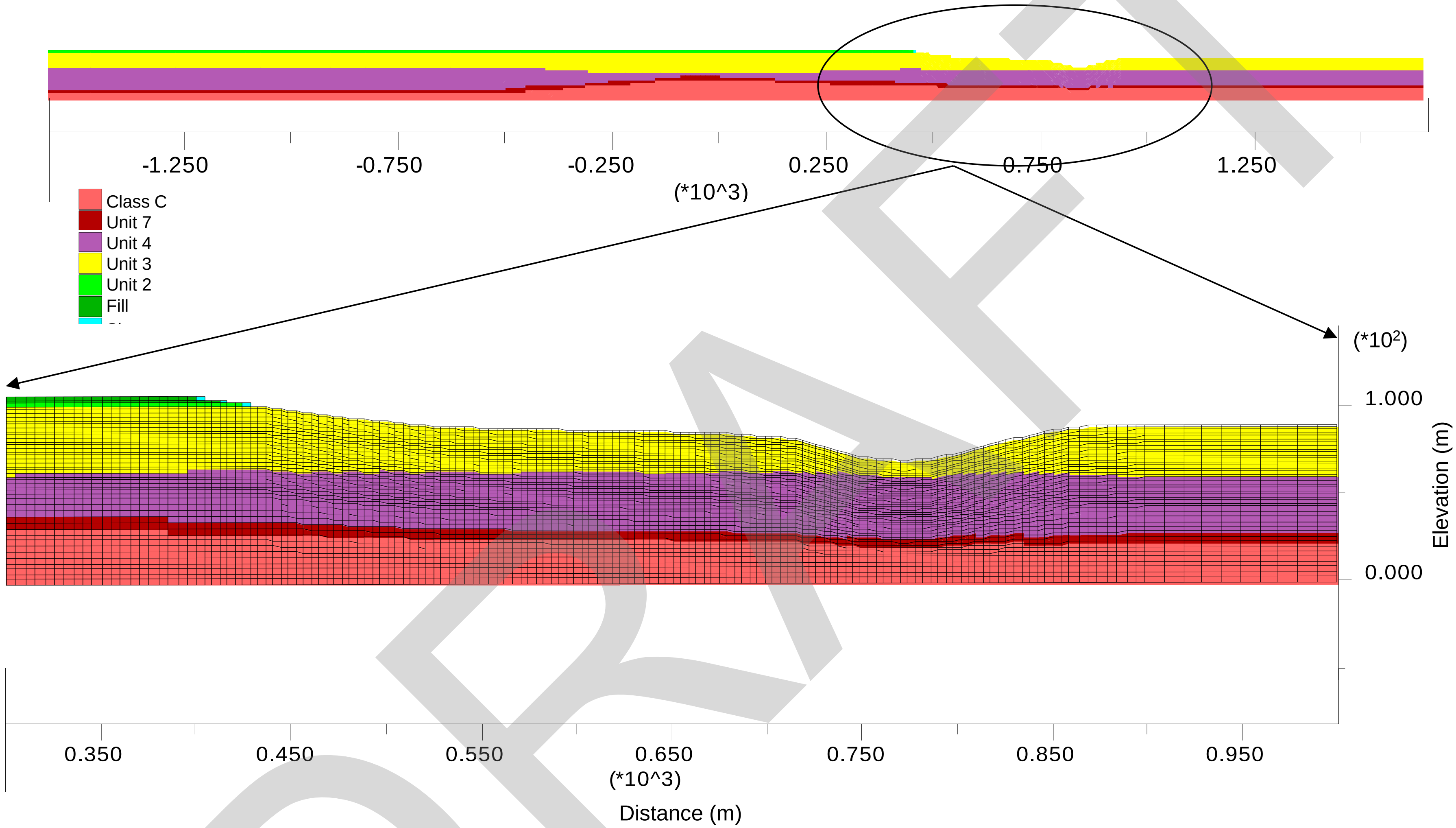
REVIEW	UA	15SEP16

PHASE No. 2100
SCALE NTS
REV.

FIGURE 1-5b



PROJECT	**CDM SMITH CANADA ULC** **AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM** **DELTA, BC**
TITLE	**Finite Difference Grid and Stratigraphic Profile (Section A-A')**

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
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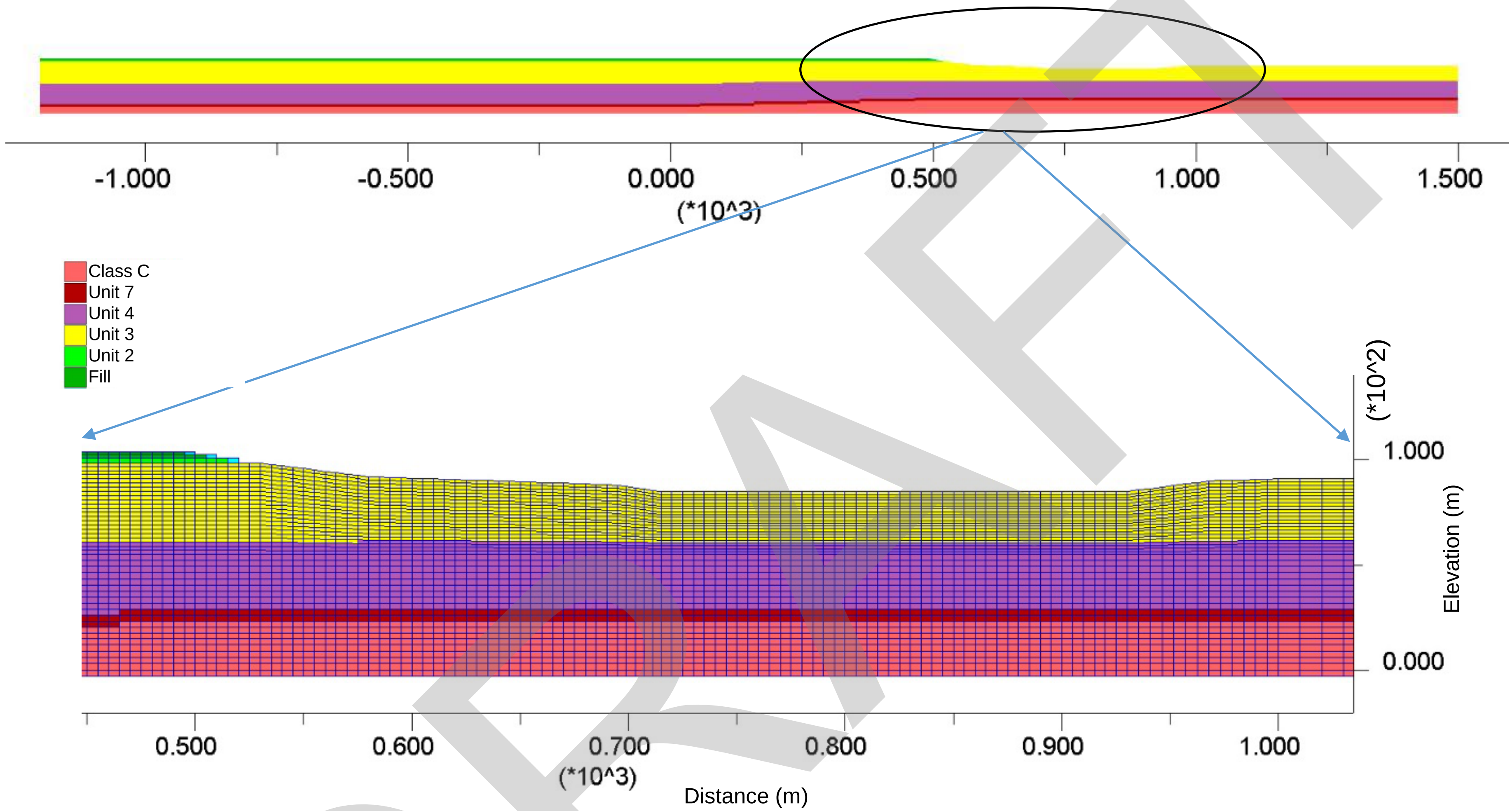

PROJECT

CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE

Finite Difference Grid and Stratigraphic Profile (Section B-B')

Golder Associates

PROJECT No.	1525010		PHASE No. 2100	
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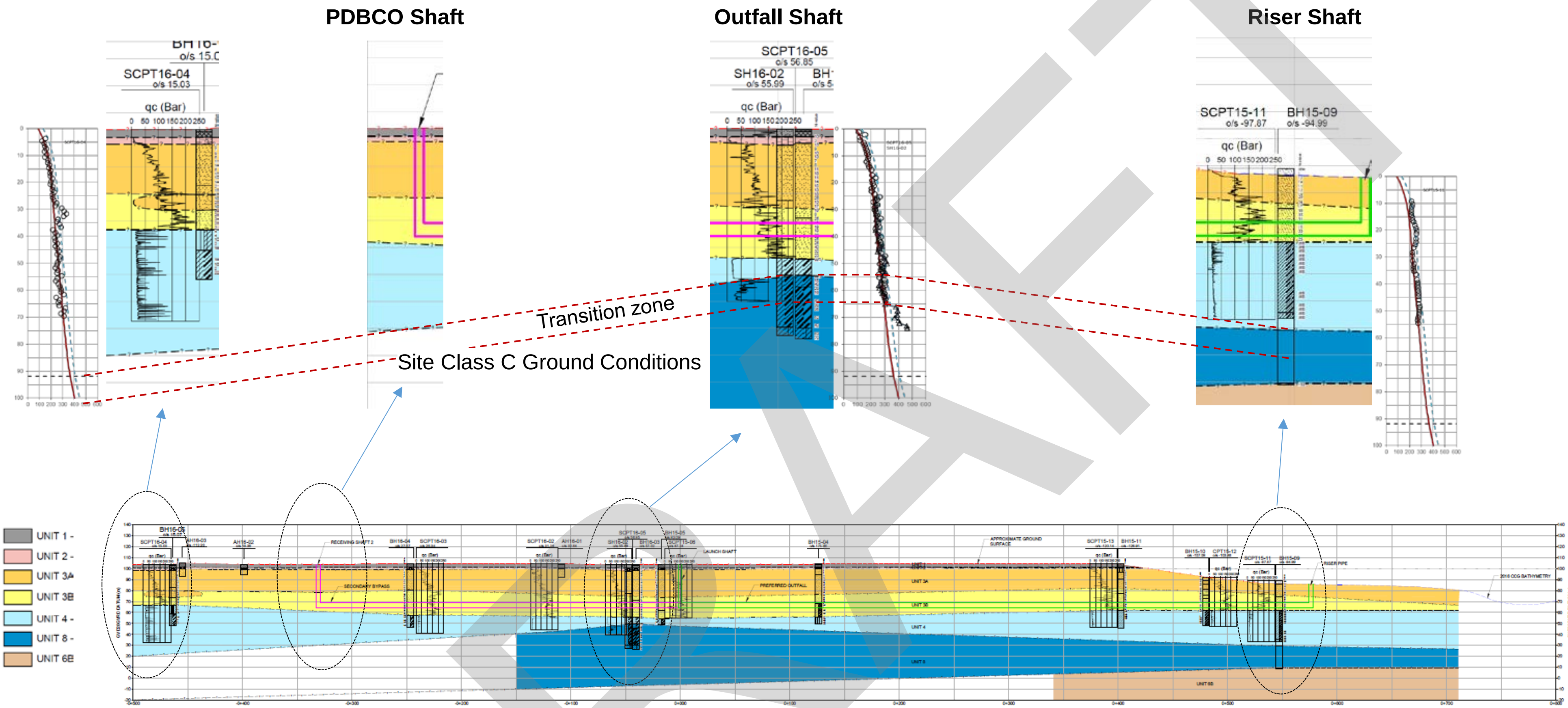

PROJECT

CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE

Class C Ground Conditions Along Outfall Alignment

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
DESIGN	VT	15SEP16	SCALE NTS	REV.
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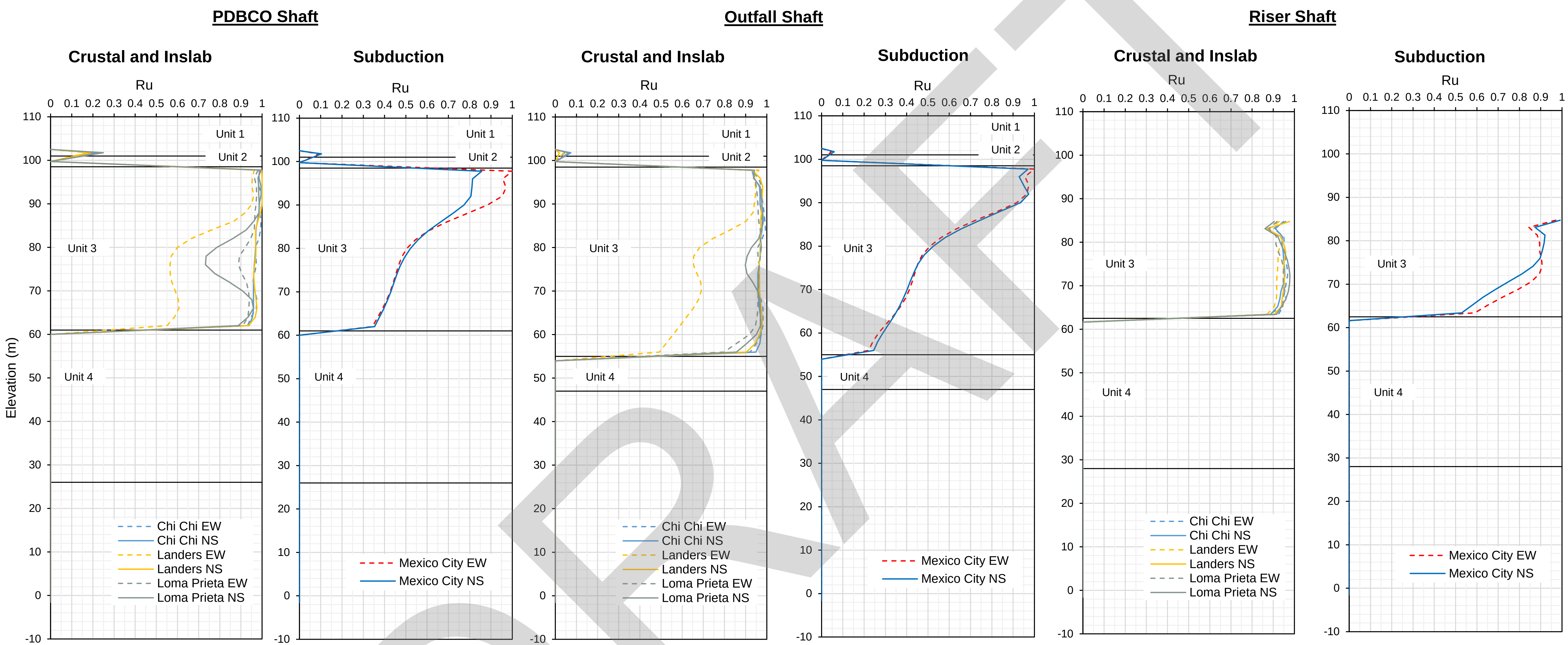

Note:
Ru is the excess pore water pressure ratio

DRAFT

PROJECT	CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC					
TITLE	**Predicted Extent of Liquefaction – Case 1 Shaft Locations**					
Golder Associates	PROJECT No.		1525010	PHASE No. 2100		
	DESIGN	VT	15SEP16	SCALE NTS	REV.	
	CADD	----	----	**FIGURE 1-9**		
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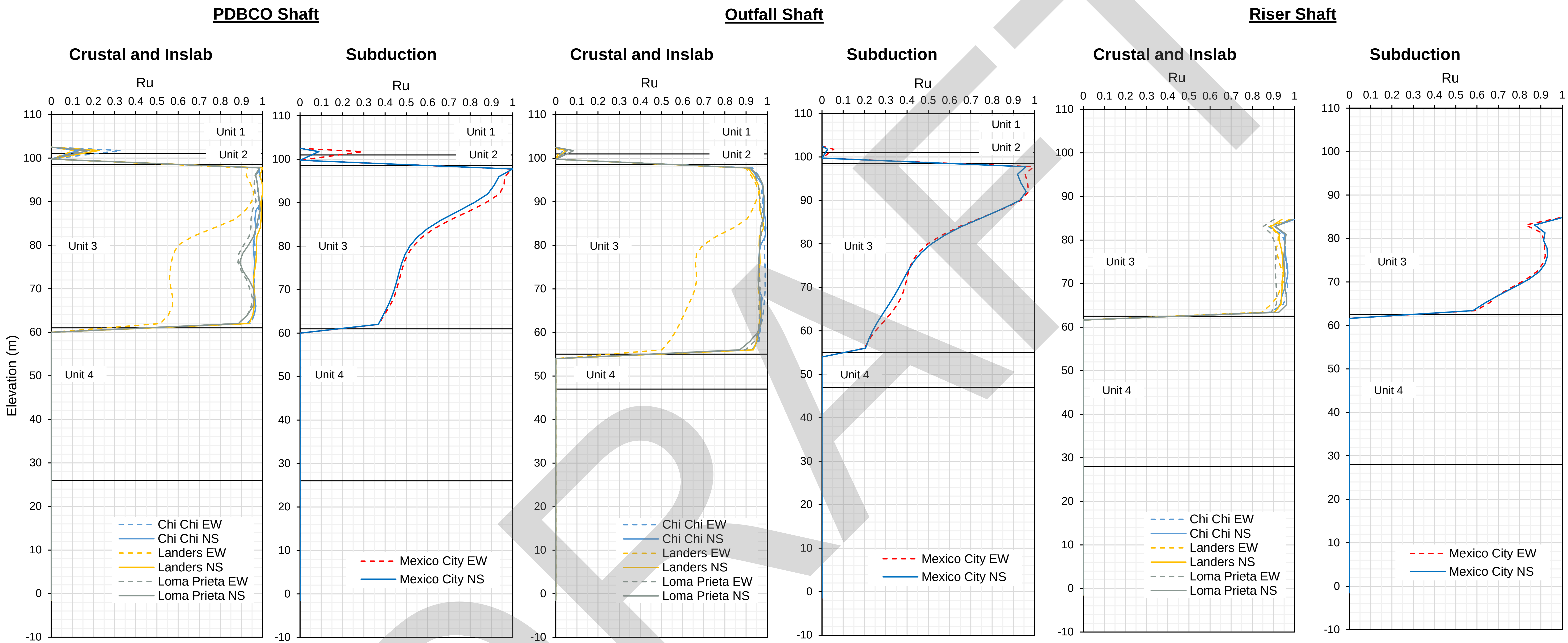
	REVIEW	UA	15SEP16			



Note:
Ru is the excess pore water pressure ratio

PROJECT	CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
TITLE	**Predicted Extent of Liquefaction – Case 2** **Shaft Locations**

Golder Associates				
PROJECT No.	1525010		PHASE No.	2100
DESIGN	VT	15SEP16	SCALE NTS	REV.
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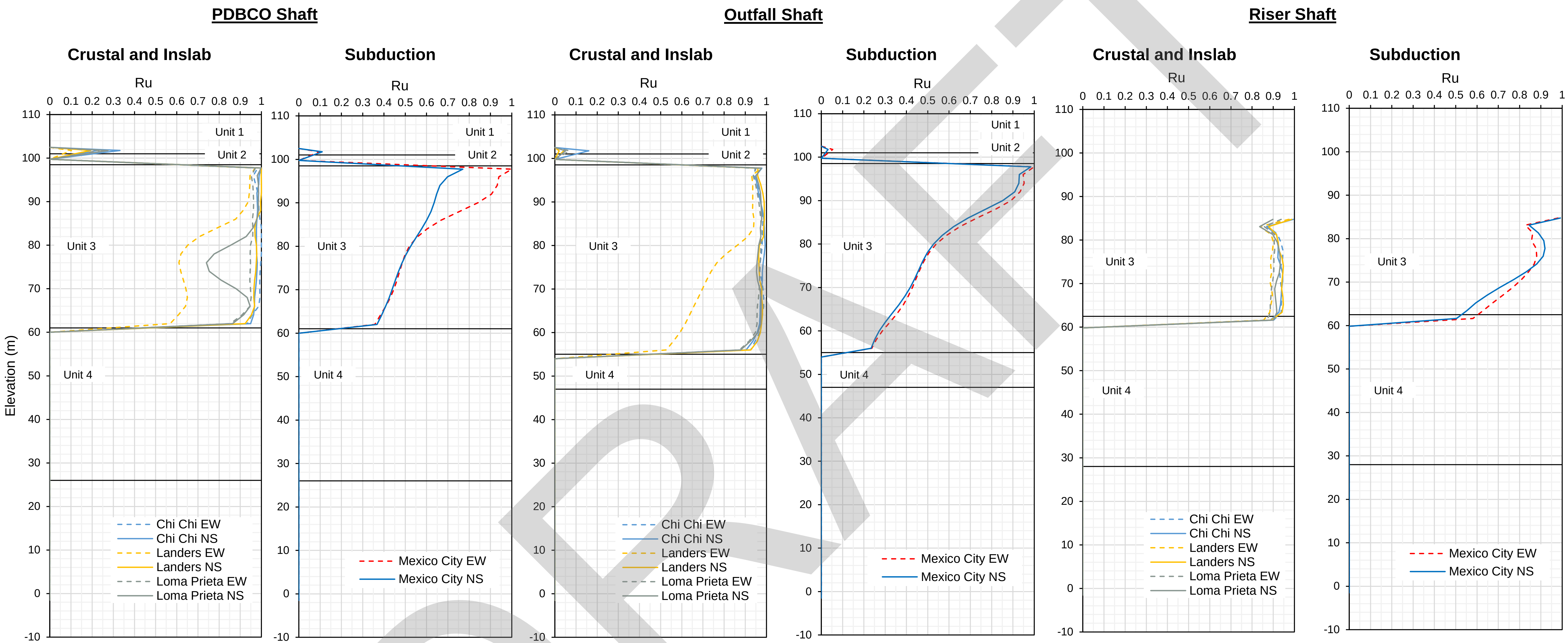

<u>Note</u>:
Ru is the excess pore water pressure ratio

DRAFT

PROJECT
CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Predicted Extent of Liquefaction – Case 3 Shaft Locations

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
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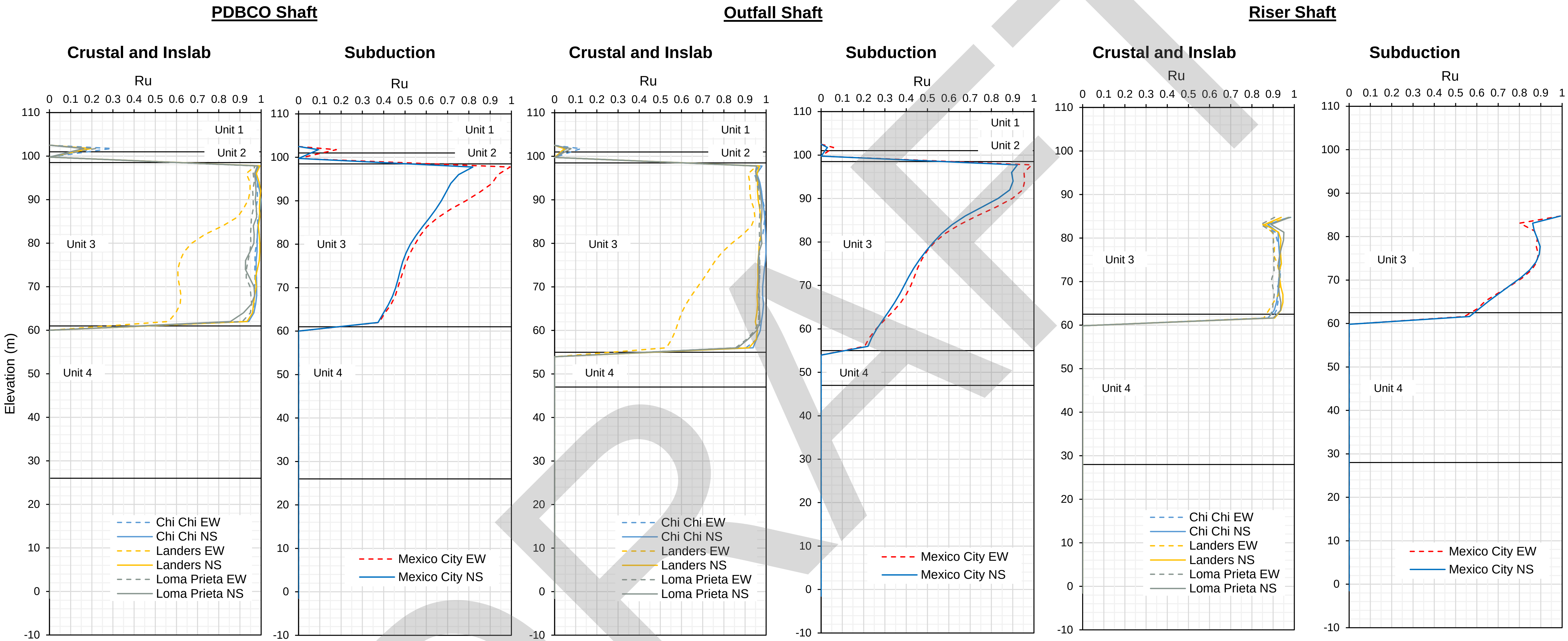
REVIEW	UA	15SEP16		



Note:
Ru is the excess pore water pressure ratio

PROJECT	CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
TITLE	**Predicted Extent of Liquefaction – Case 4 Shaft Locations**

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
DESIGN	VT	15SEP16	SCALE NTS	REV.
CADD	----	----		
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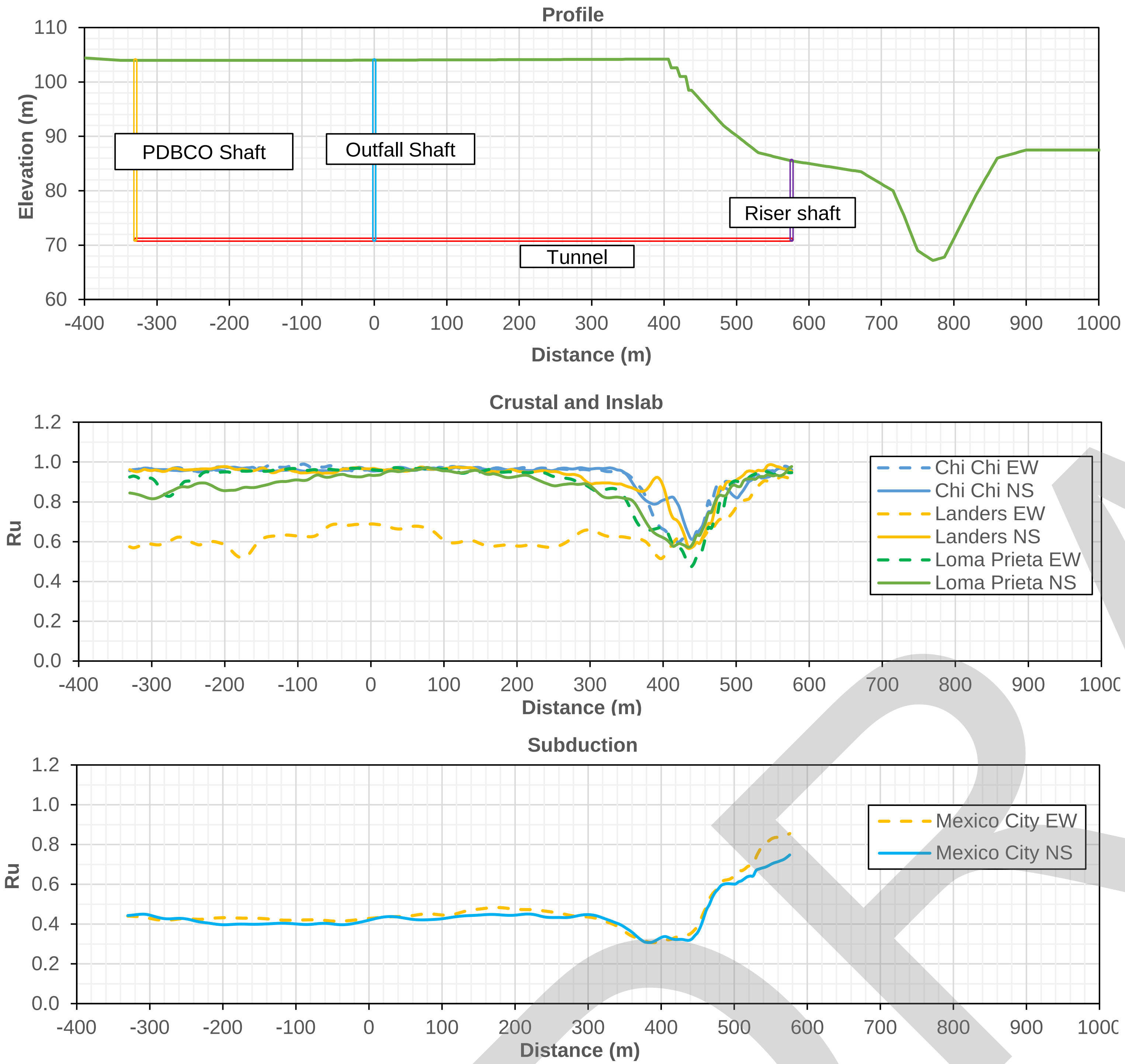
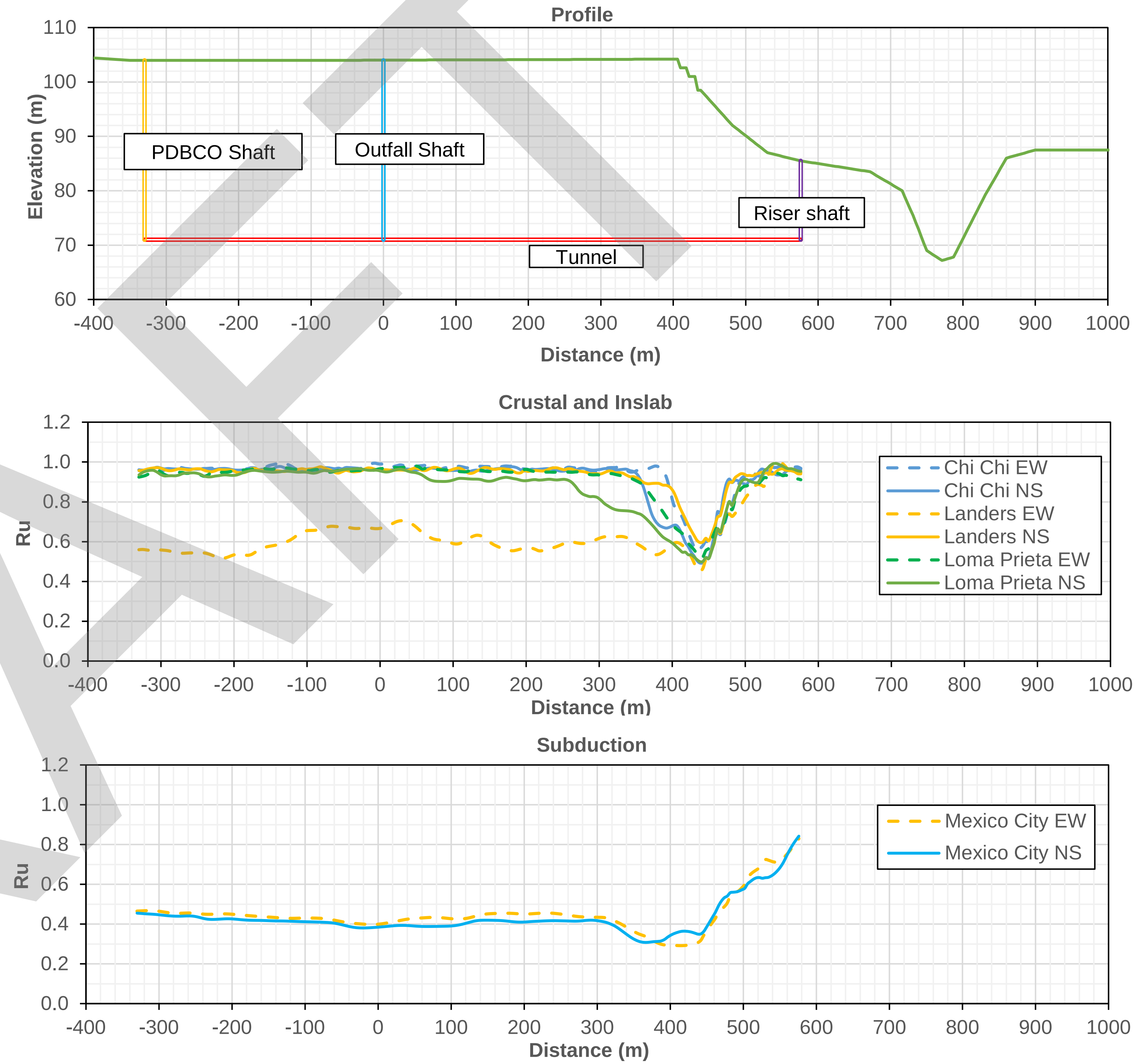
REVIEW	UA	15SEP16		

Case 1



Case 2



Note:
Ru is the excess pore water pressure ratio

PROJECT
CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Predicted Extent of Liquefaction
At Tunnel Level – Cases 1 and 2

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
DESIGN	VT	15SEP16	SCALE NTS	REV.
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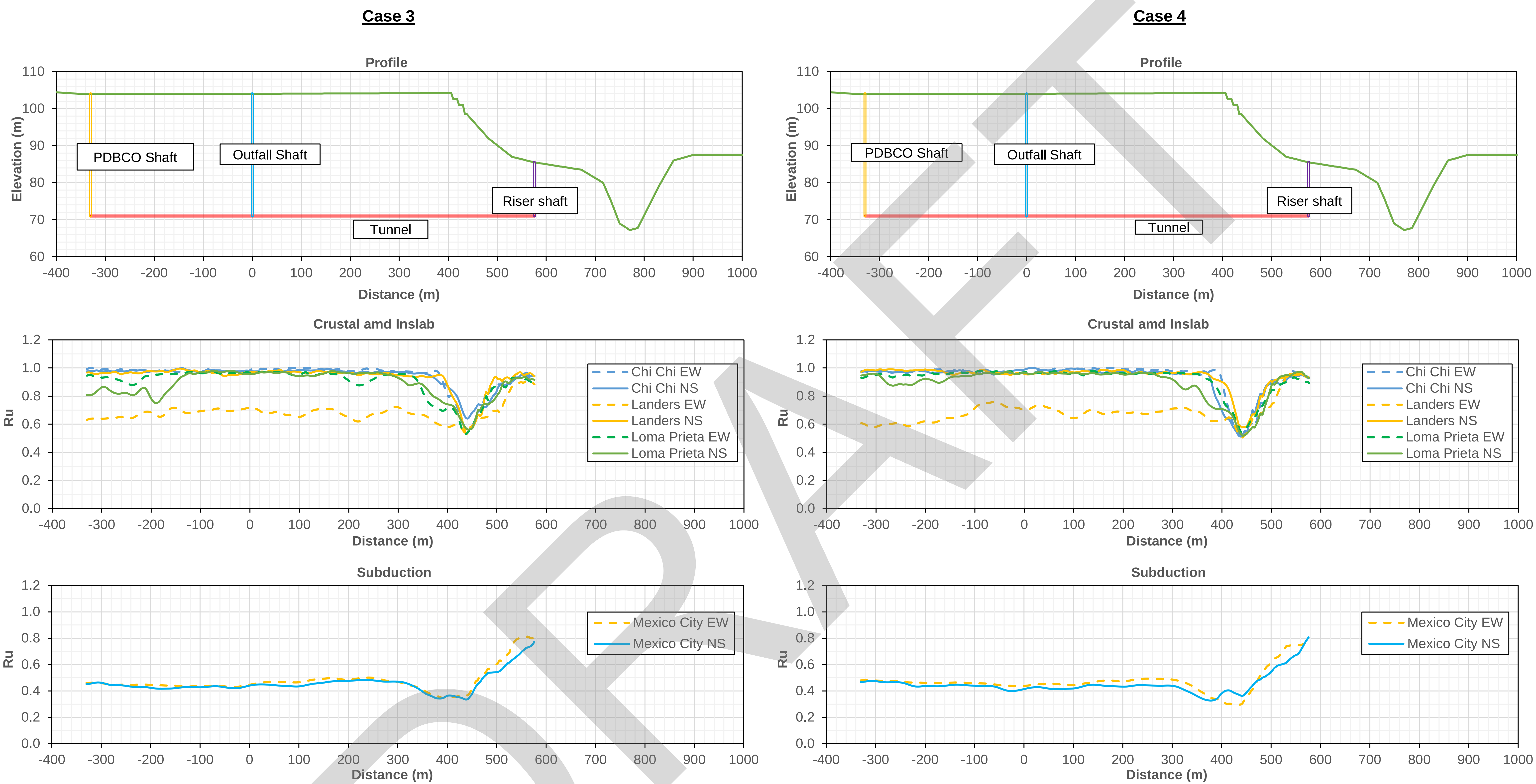

Note:
Ru is the excess pore water pressure ratio

DRAFT

PROJECT
CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Predicted Extent of Liquefaction
At Tunnel Level – Cases 3 and 4

Golder Associates

PROJECT No.	1525010	PHASE No.	2100
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CADD	----	----	
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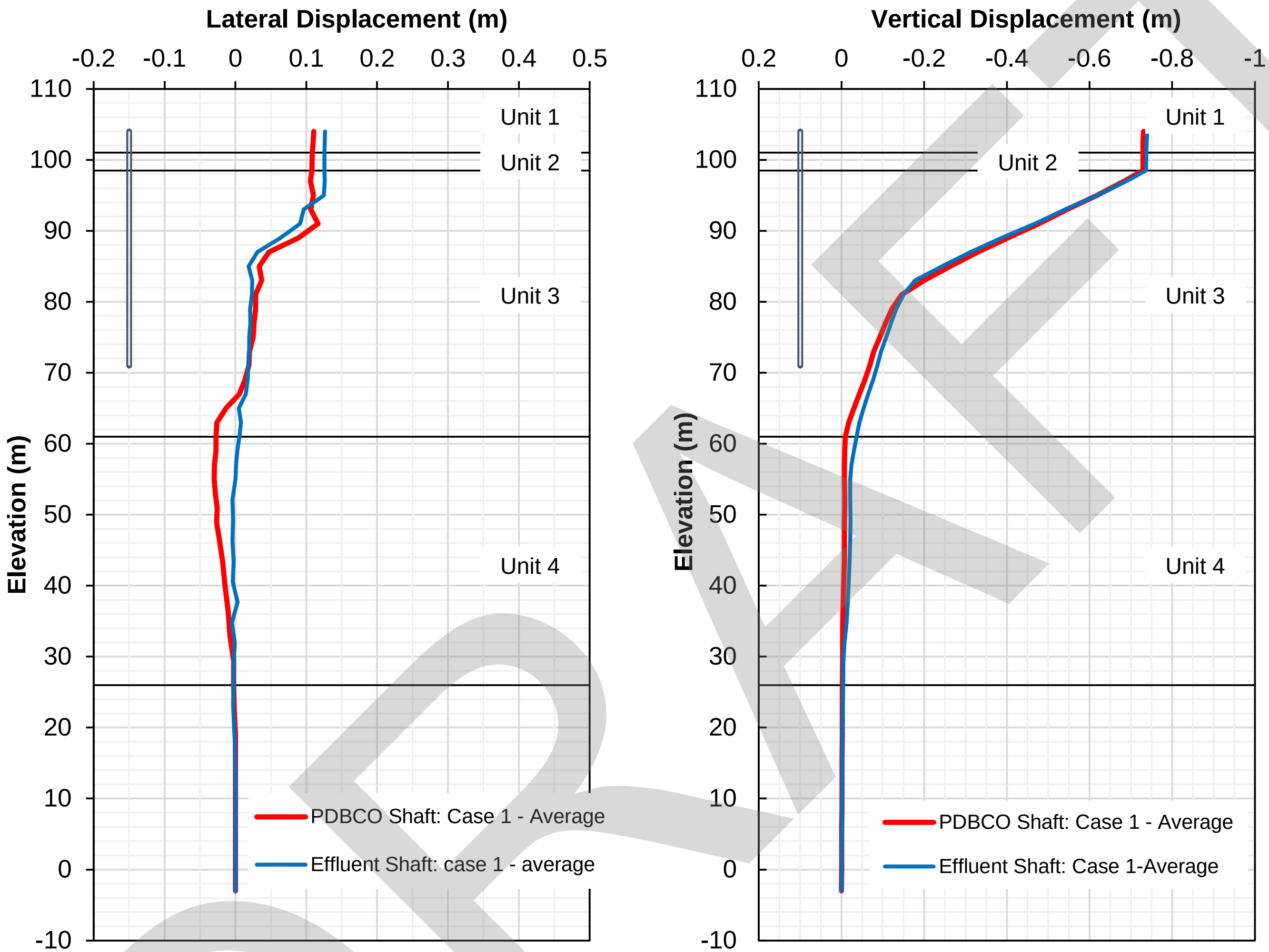
REVIEW	UA	15SEP16	



DRAFT

PROJECT
CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Average Displacement Profiles – Case 1 PDBCO and Effluent Shaft Locations

Golder Associates

PROJECT No.	1525010		PHASE No. 2100
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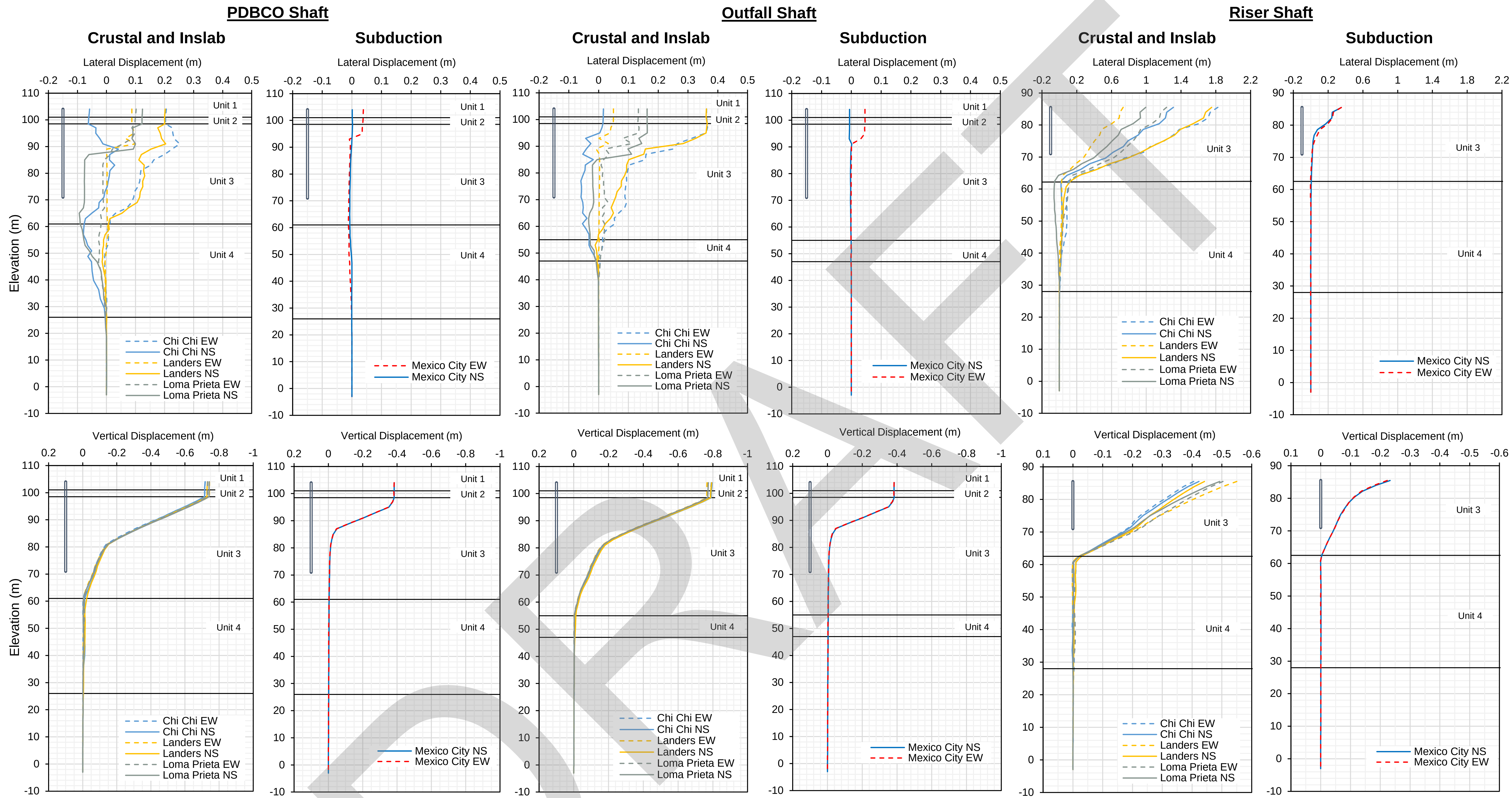
REVIEW	UA	15SEP16	



Note:

Shown in the plots illustrates the embedment of the shafts

DRAFT

PROJECT	CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
TITLE	Permanent Lateral and Vertical Displacement Profiles Shaft Locations - Case 1

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
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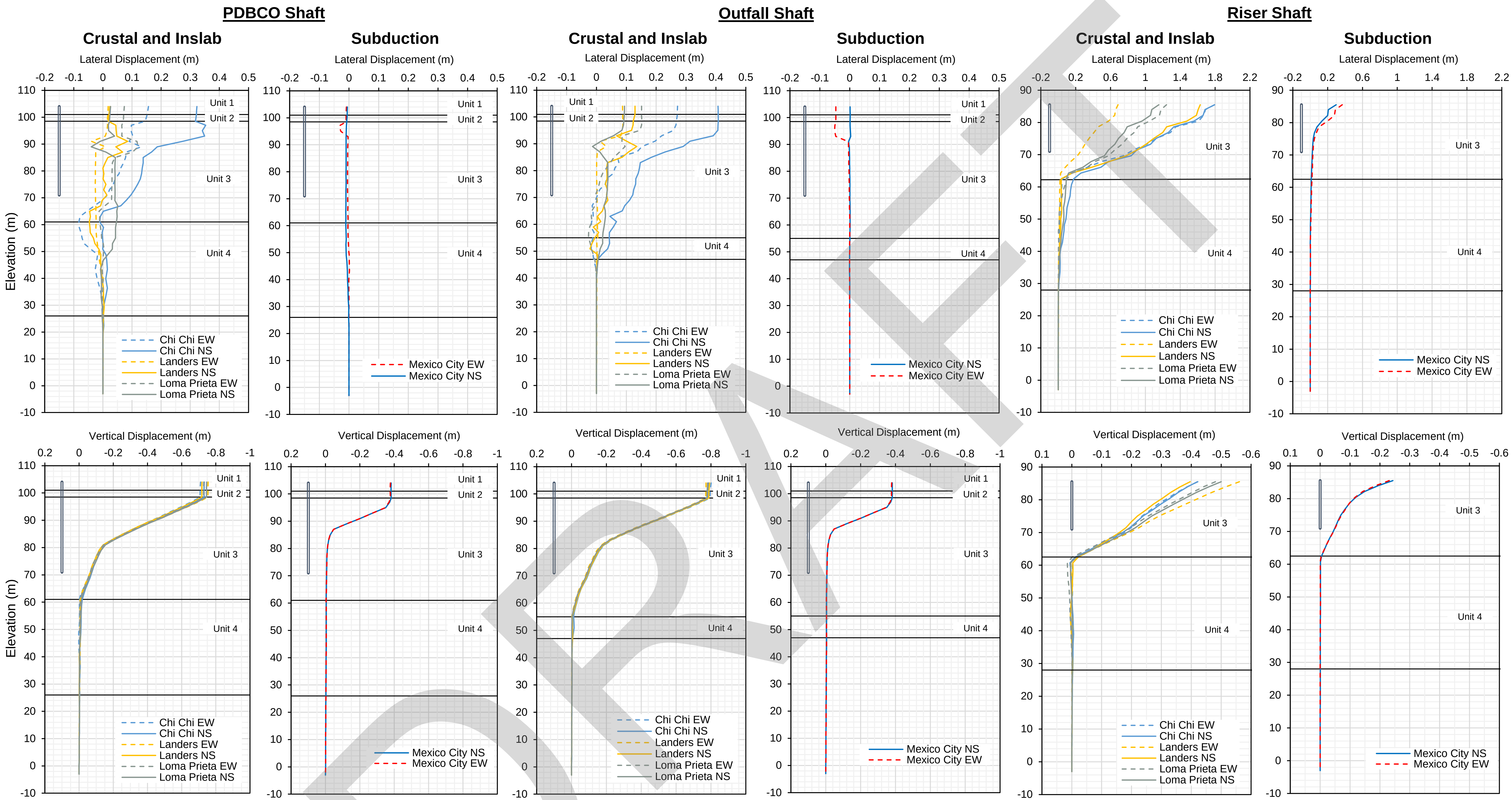
CHECK	VF	15SEP16	FIGURE 1-16	
REVIEW	UA	15SEP16		



Note:

Shown in the plots illustrates the embedment of the shafts

DRAFT

PROJECT	
CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC	
TITLE	
Permanent Lateral and Vertical Displacement Profiles Shaft Locations - Case 2	

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
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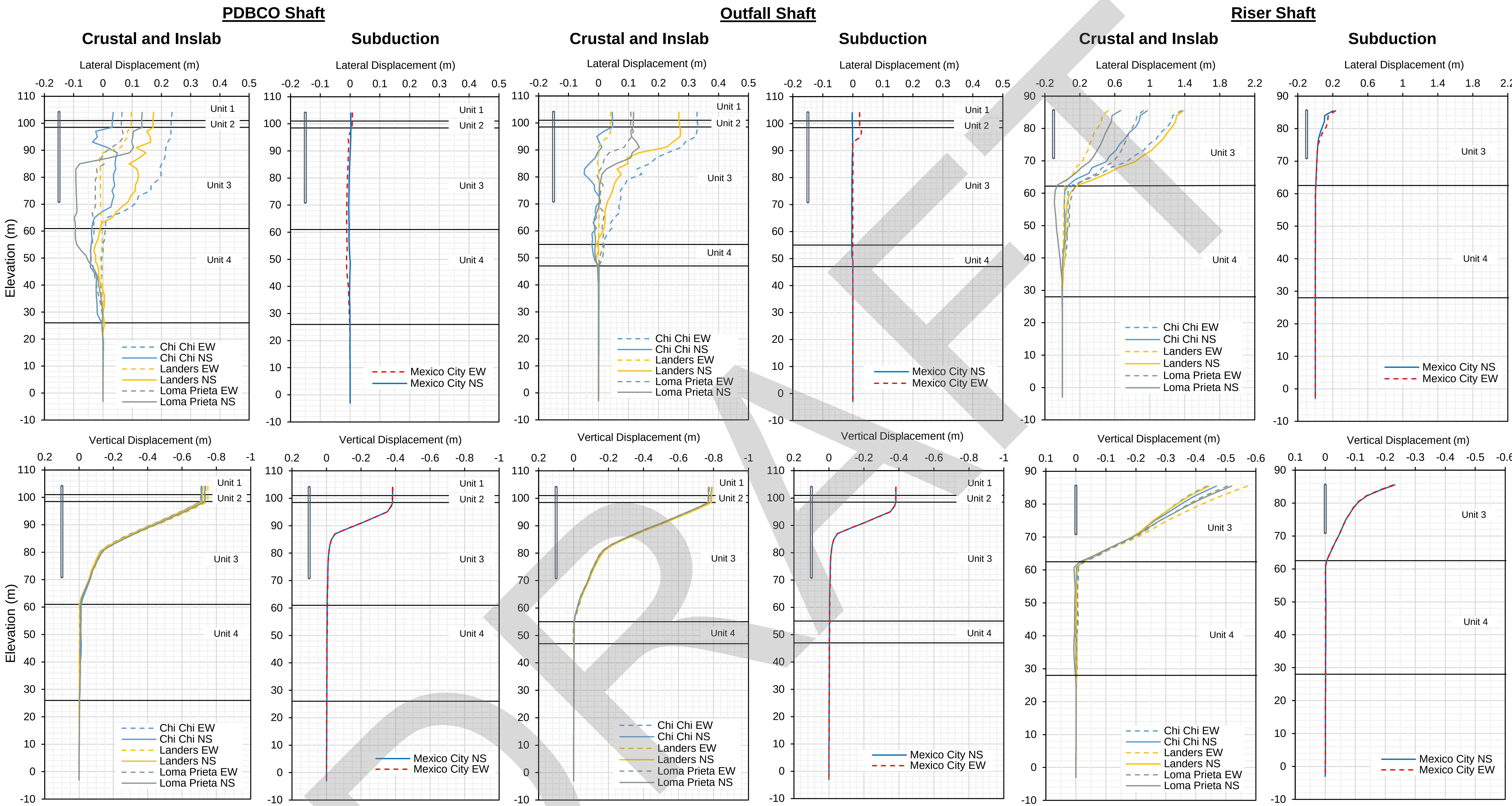

Note:

Shown in the plots illustrates the embedment of the shafts

DRAFT

PROJECT	CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
TITLE	Permanent Lateral and Vertical Displacement Profiles Shaft Locations - Case 3

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
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CADD	----	----	FIGURE 1-18	
CHECK	VF	15SEP16		
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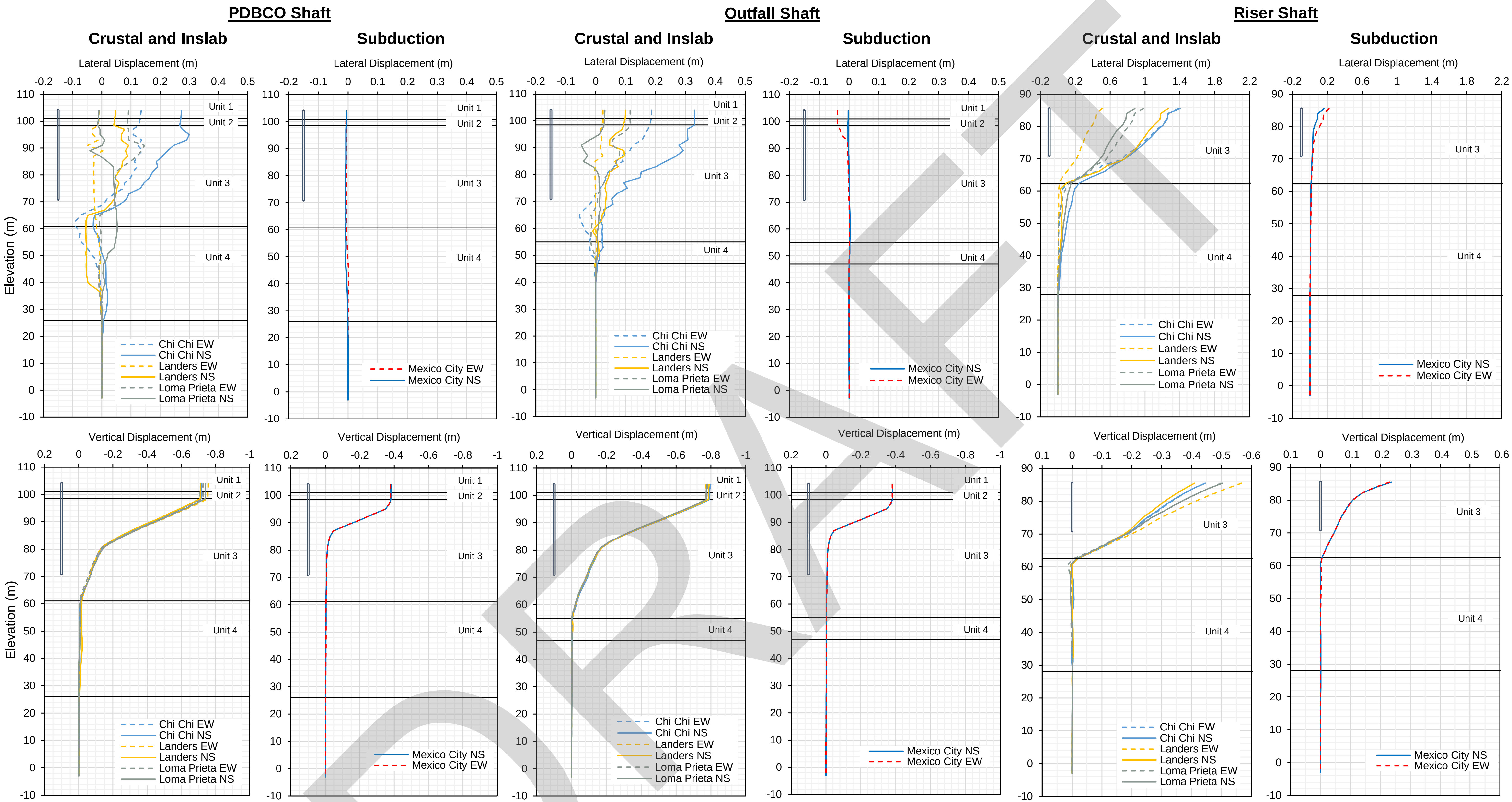

Note:

Shown in the plots illustrates the embedment of the shafts

PROJECT	CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
TITLE	Permanent Lateral and Vertical Displacement Profiles Shaft Locations - Case 4

PROJECT No.	1525010		PHASE No.	2100
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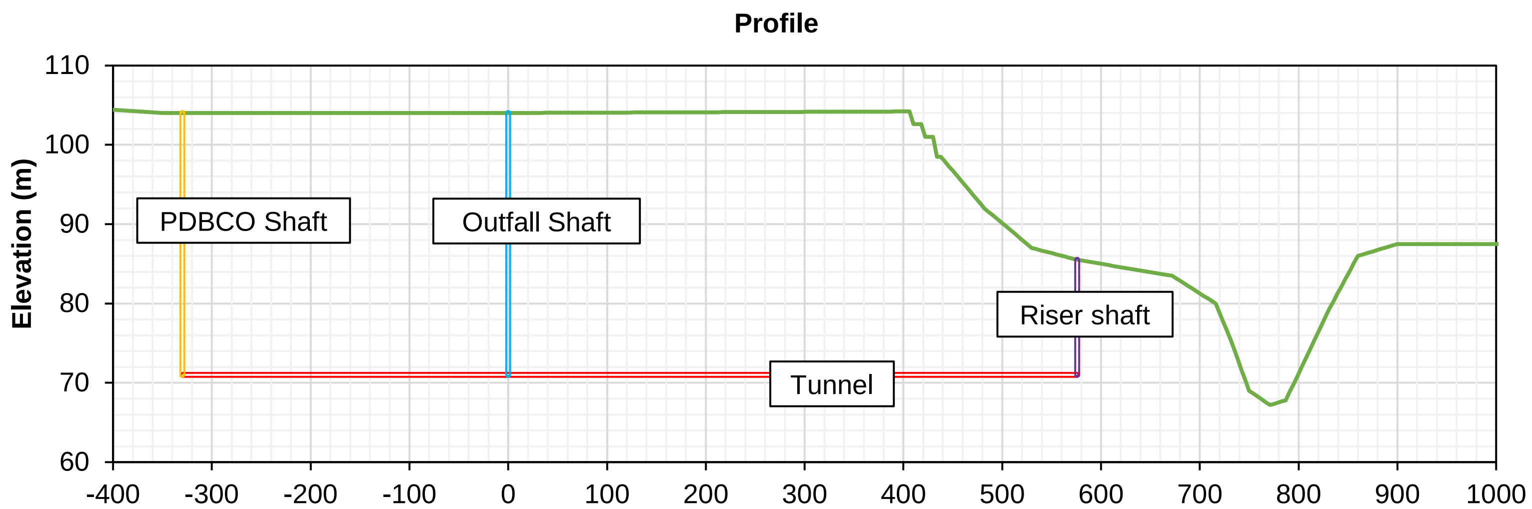
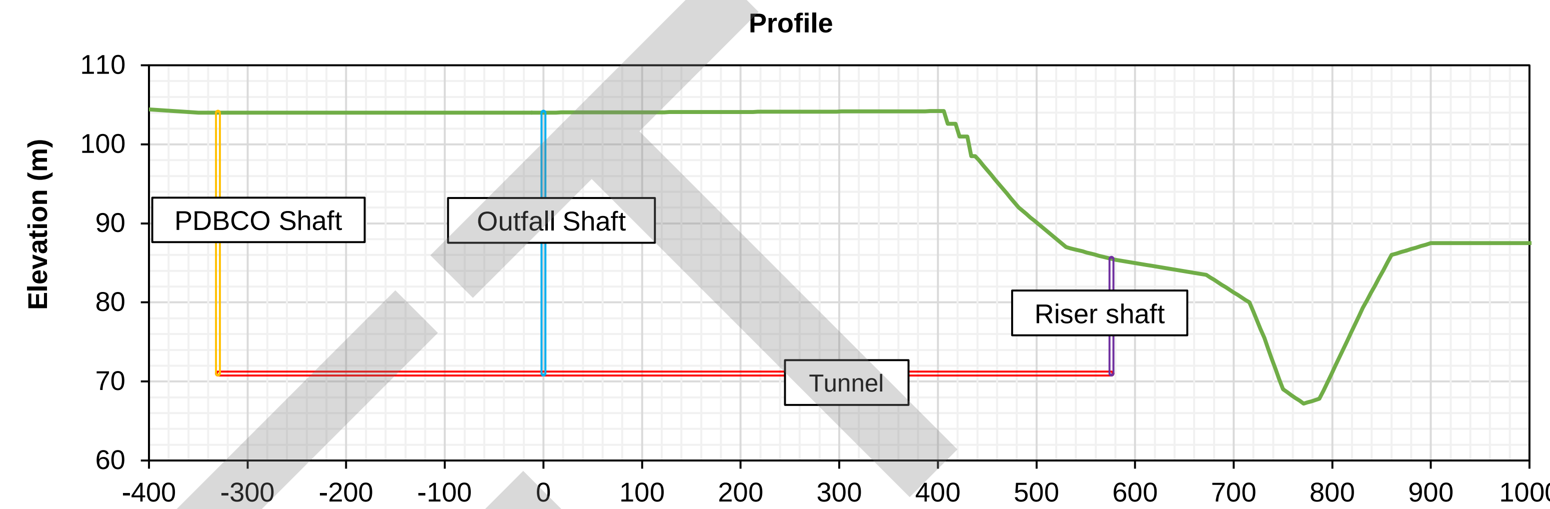
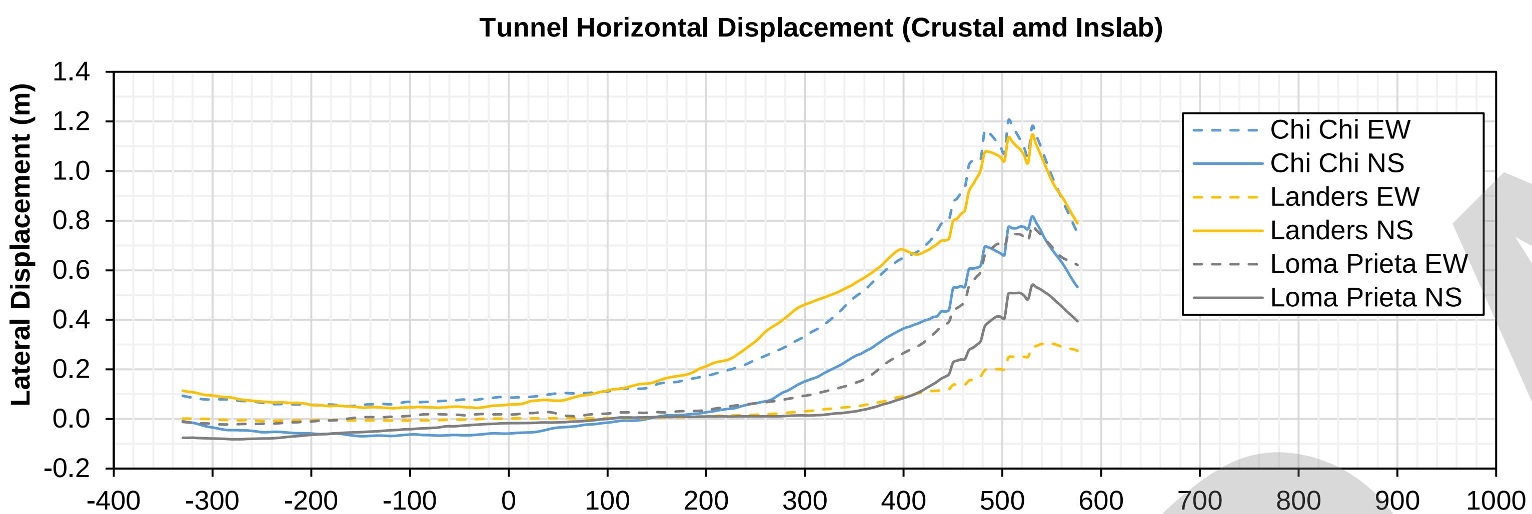
CADD	----	----		
CHECK	VF	15SEP16	FIGURE 1-19	
REVIEW	UA	15SEP16		

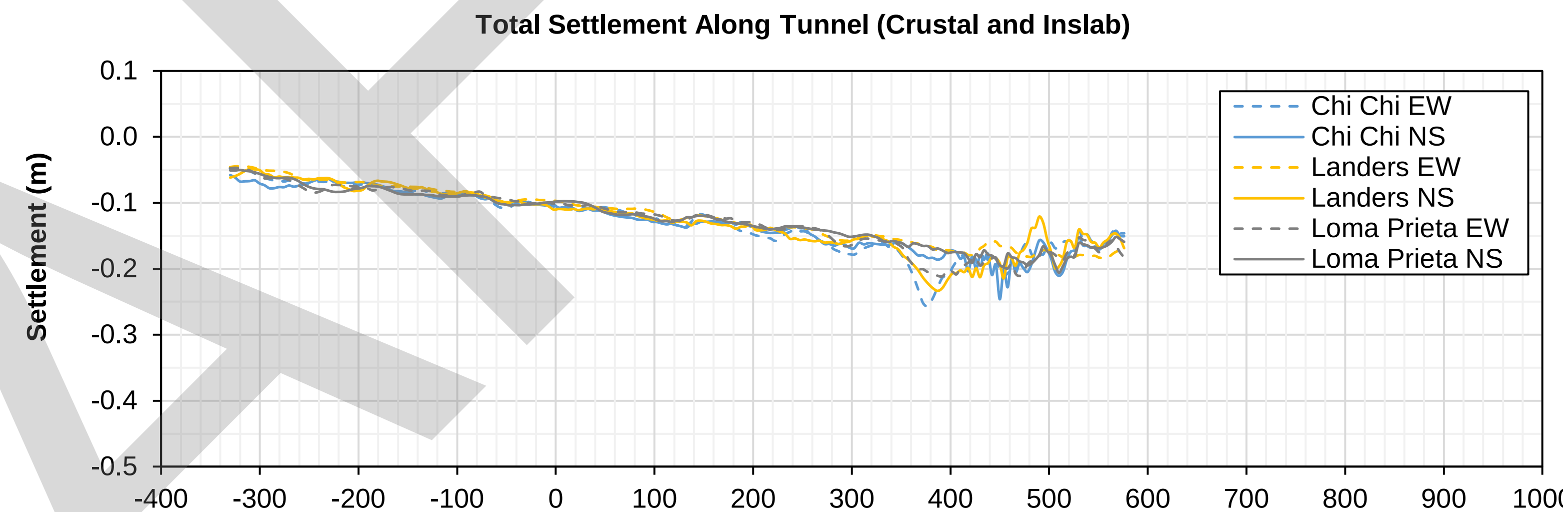
Golder Associates

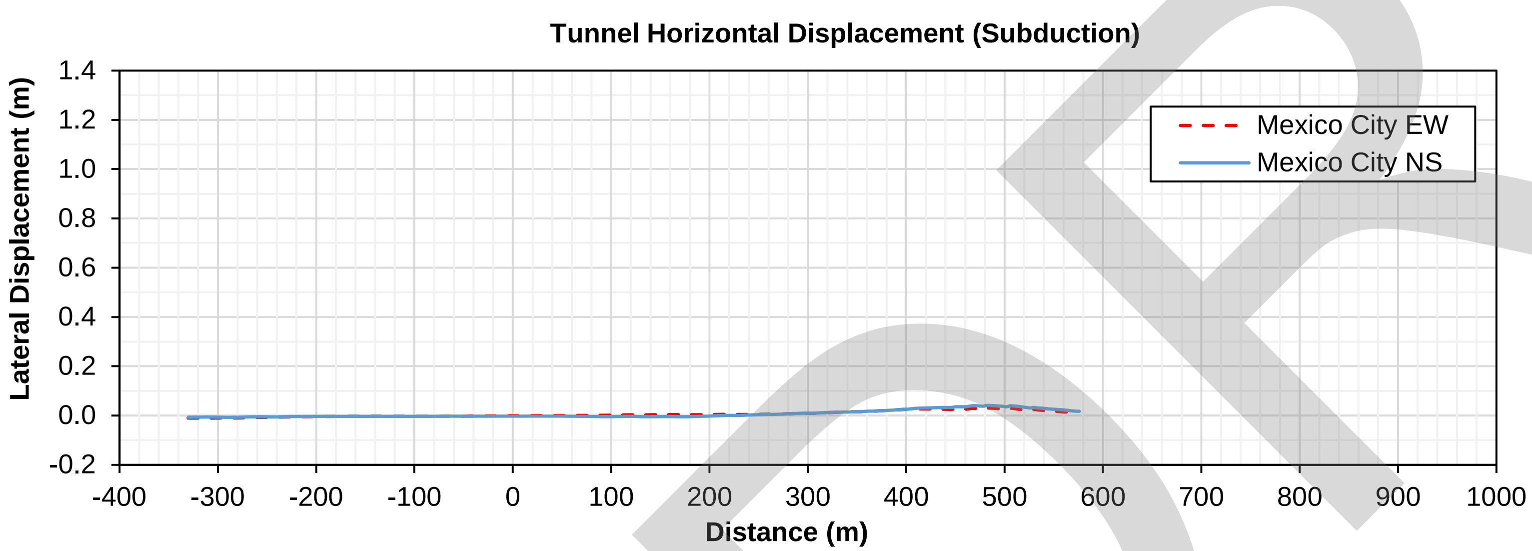
DRAFT

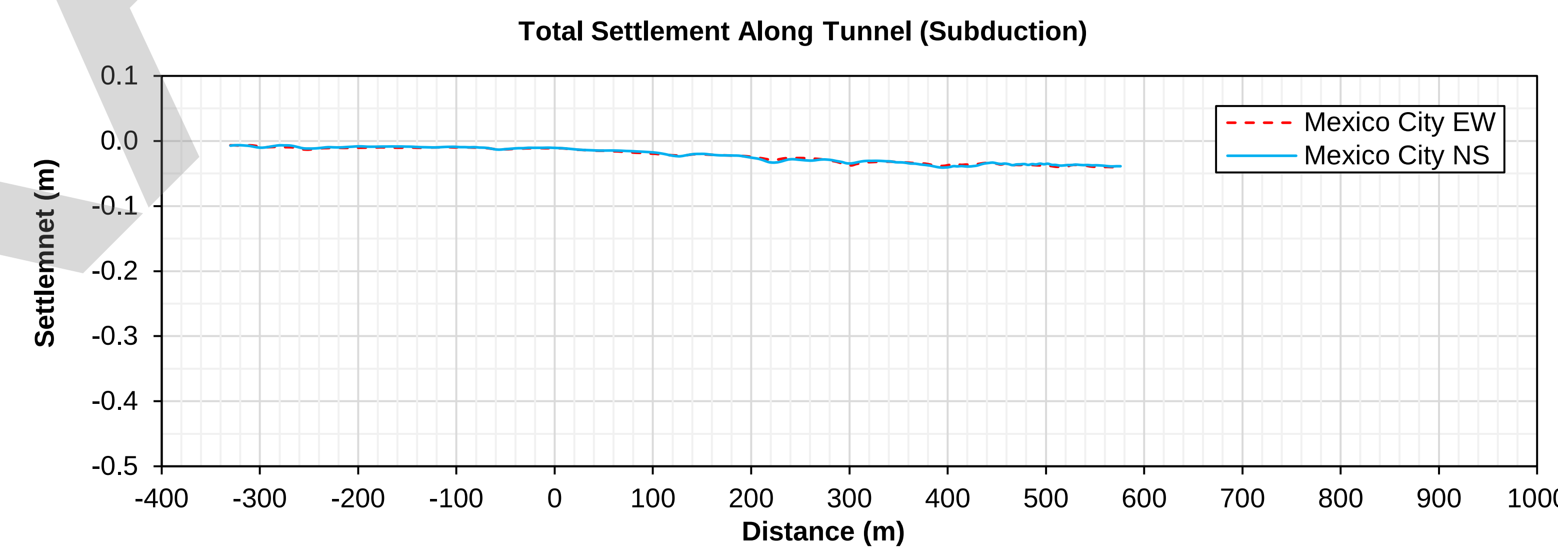












DRAFT

PROJECT

CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE

Permanent Lateral and Vertical Displacement Profiles At Tunnel Level- Case 1

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
DESIGN	VT	15SEP16	SCALE NTS	REV.
CADD	----	----		
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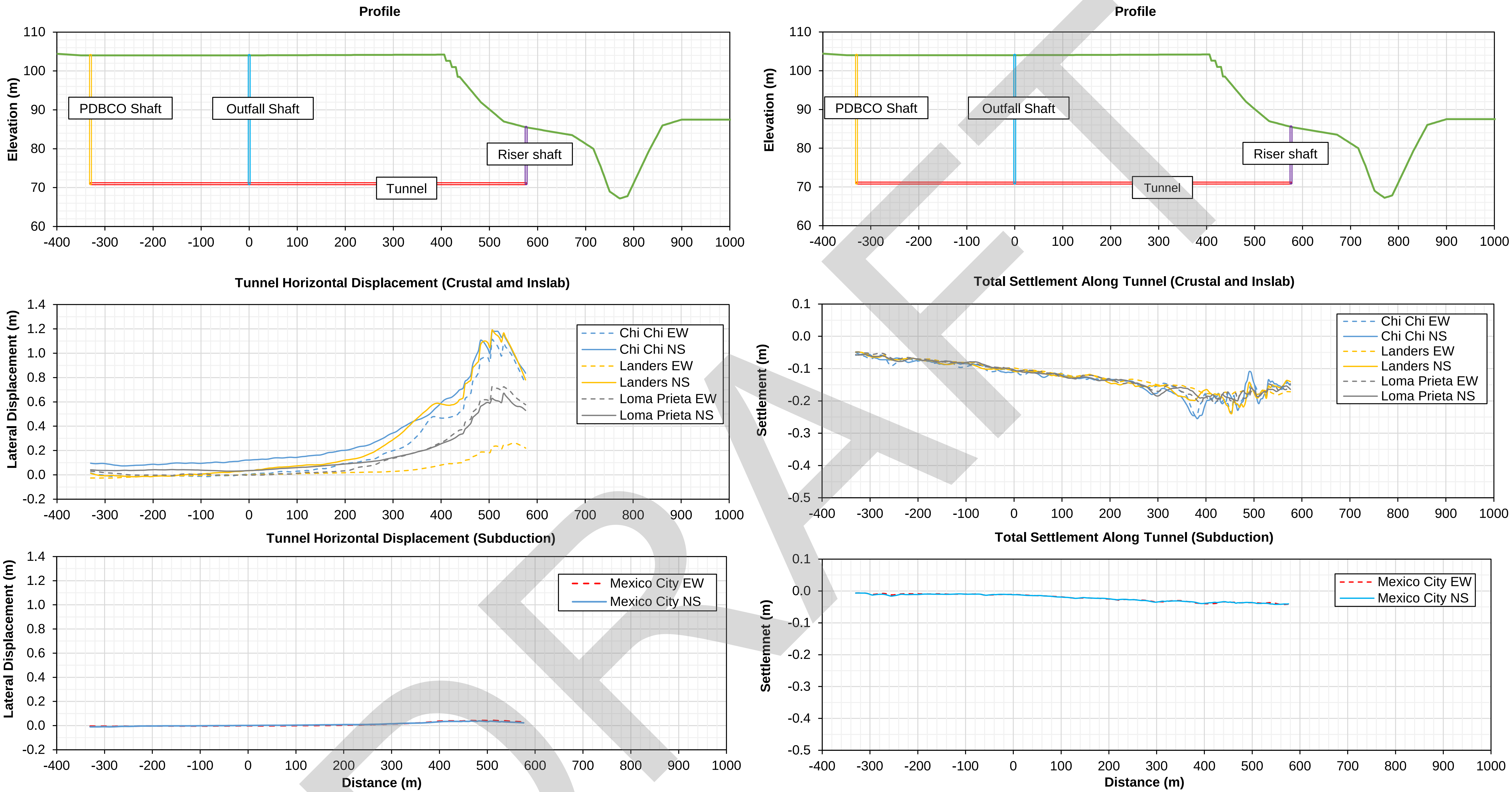
REVIEW	UA	15SEP16		



PROJECT
CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Permanent Lateral and Vertical Displacement Profiles
At Tunnel Level- Case 2

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
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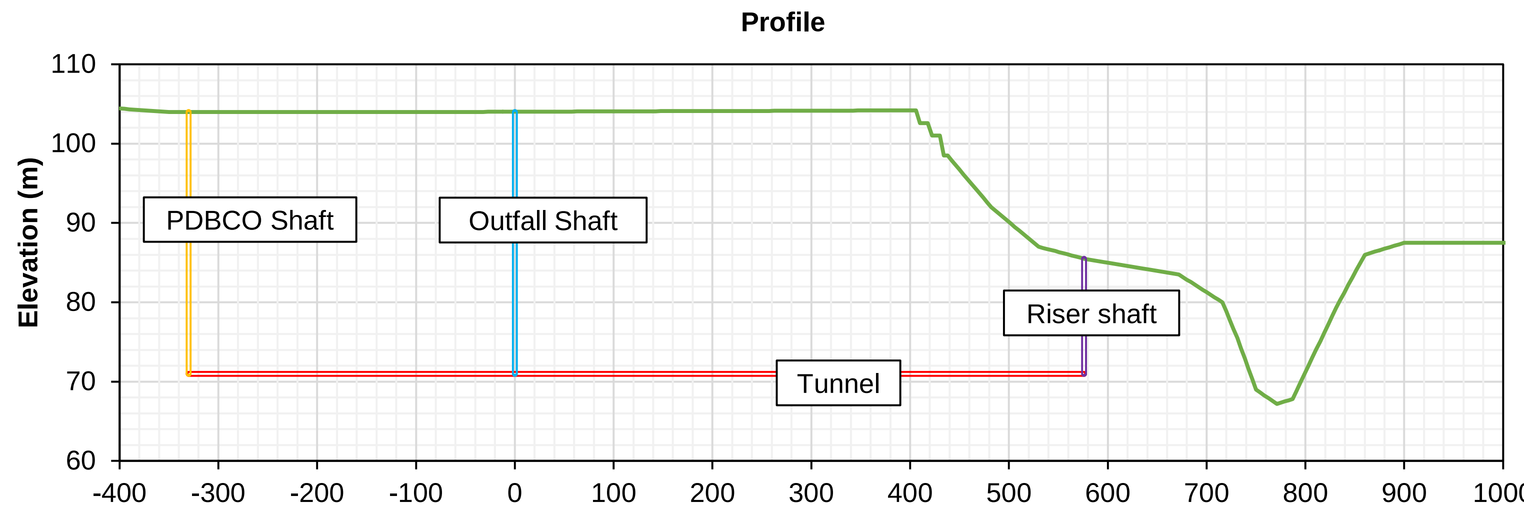
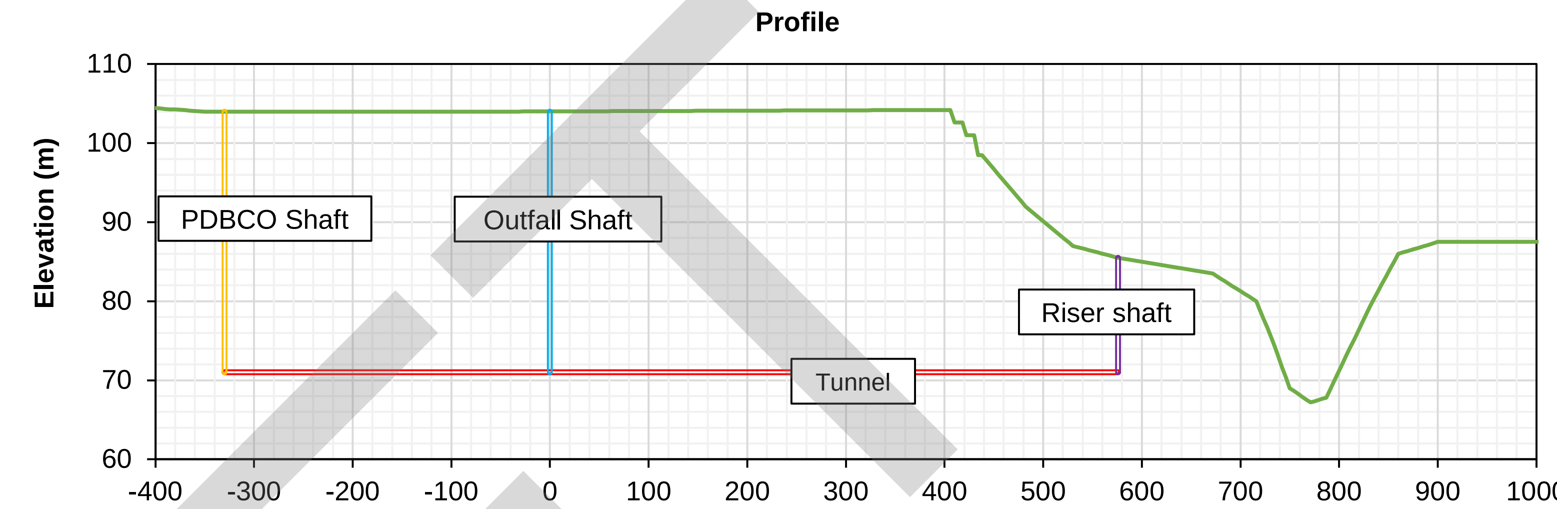
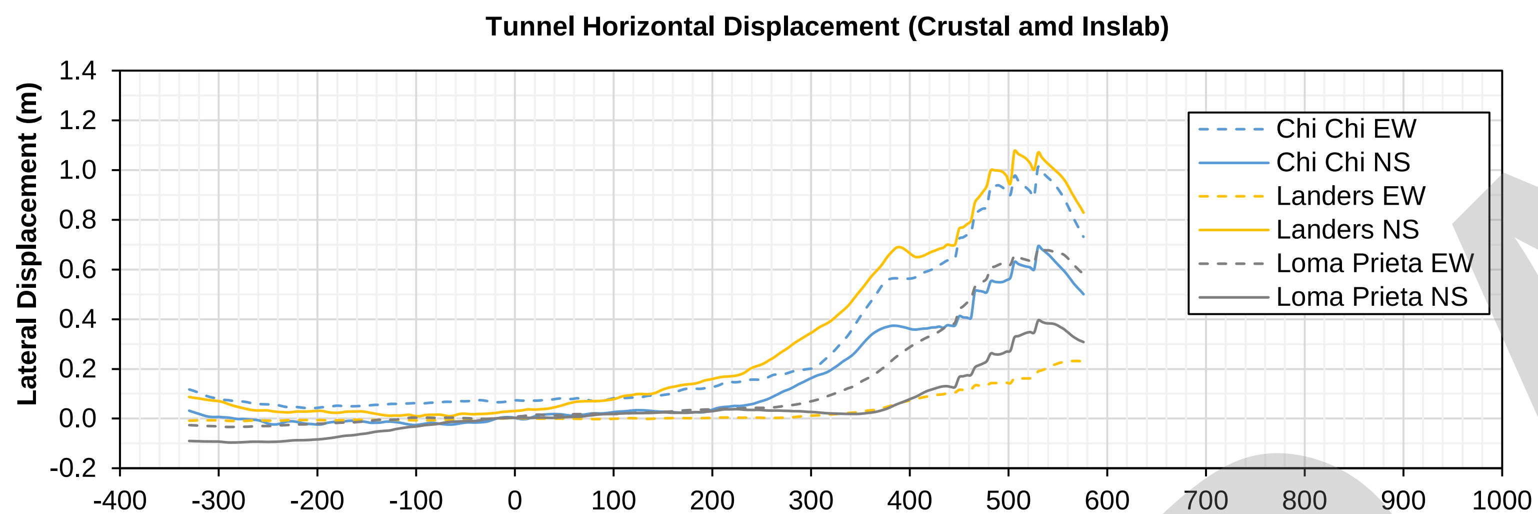
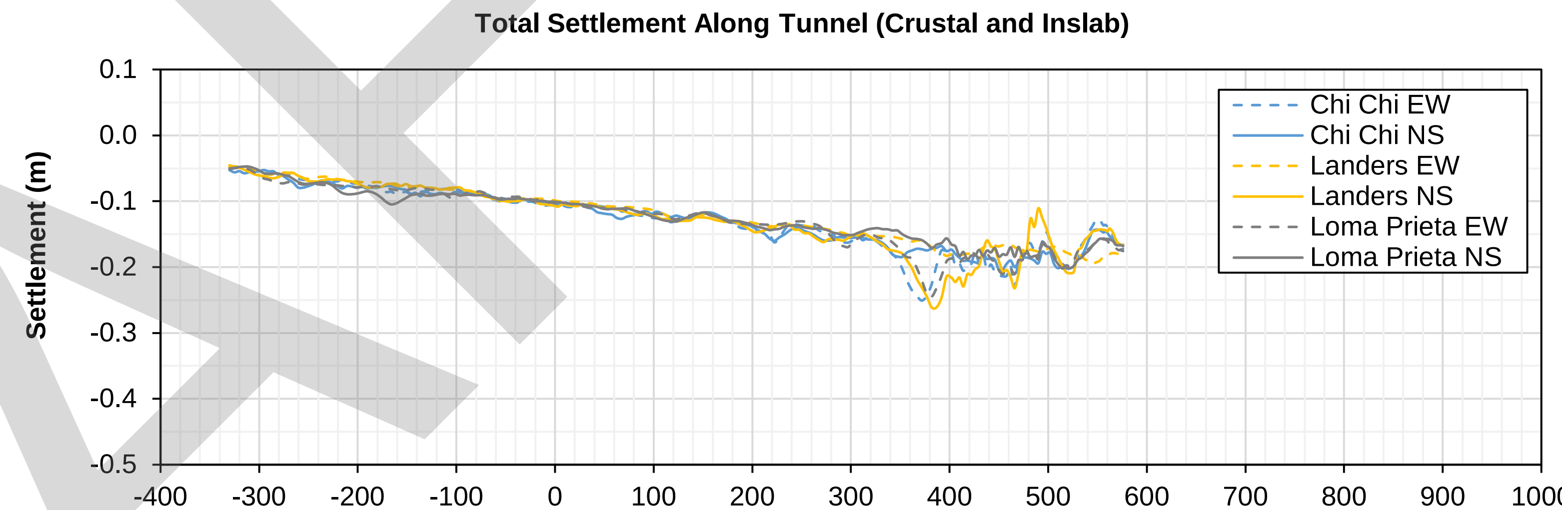
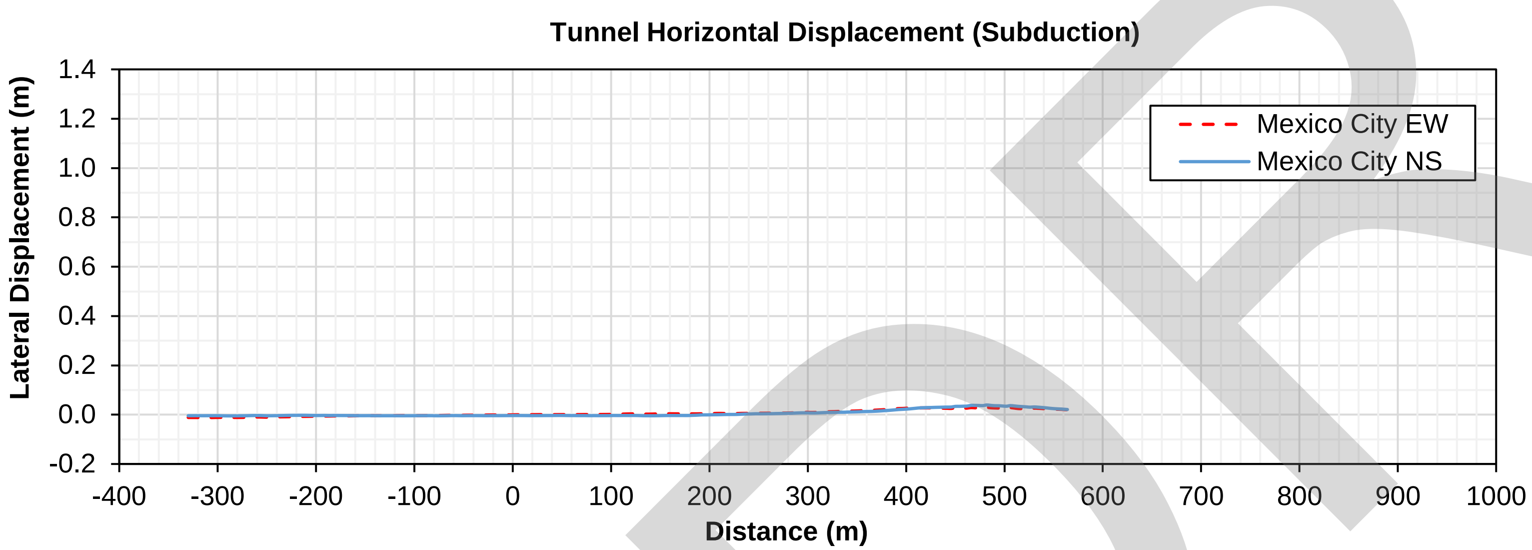
REVIEW	UA	15SEP16		

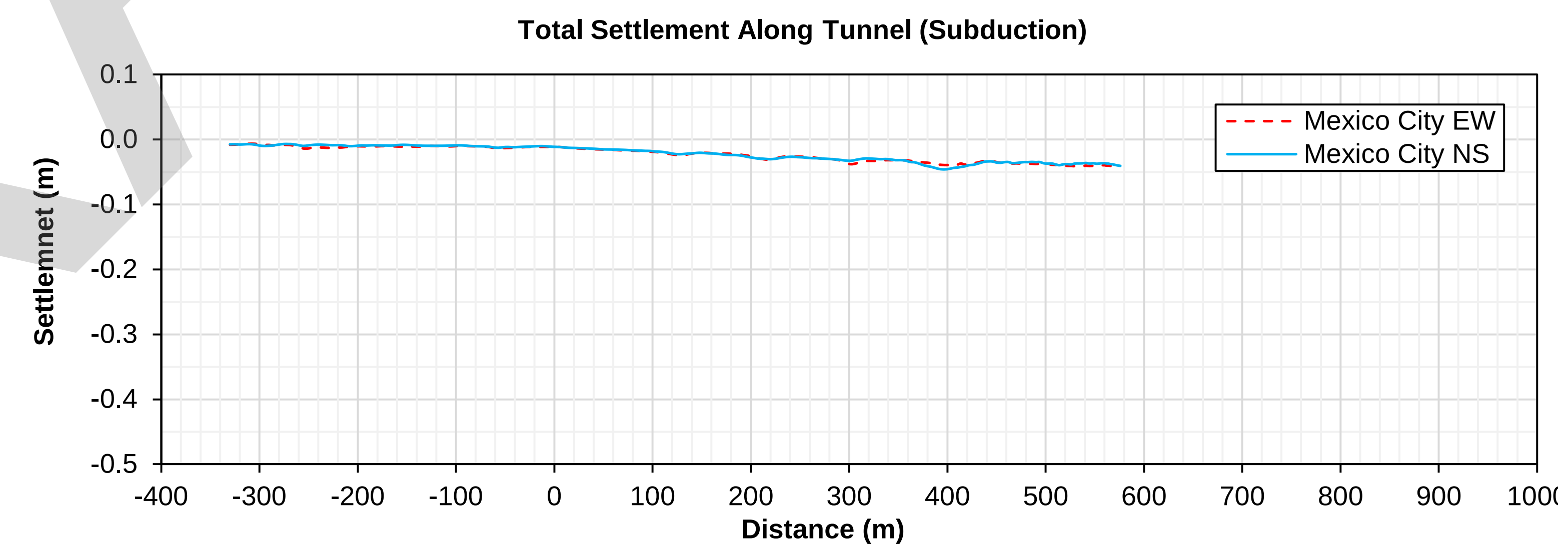












PROJECT
CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
Permanent Lateral and Vertical Displacement Profiles At Tunnel Level- Case 3

Golder Associates

PROJECT No.	1525010		PHASE No.	2100
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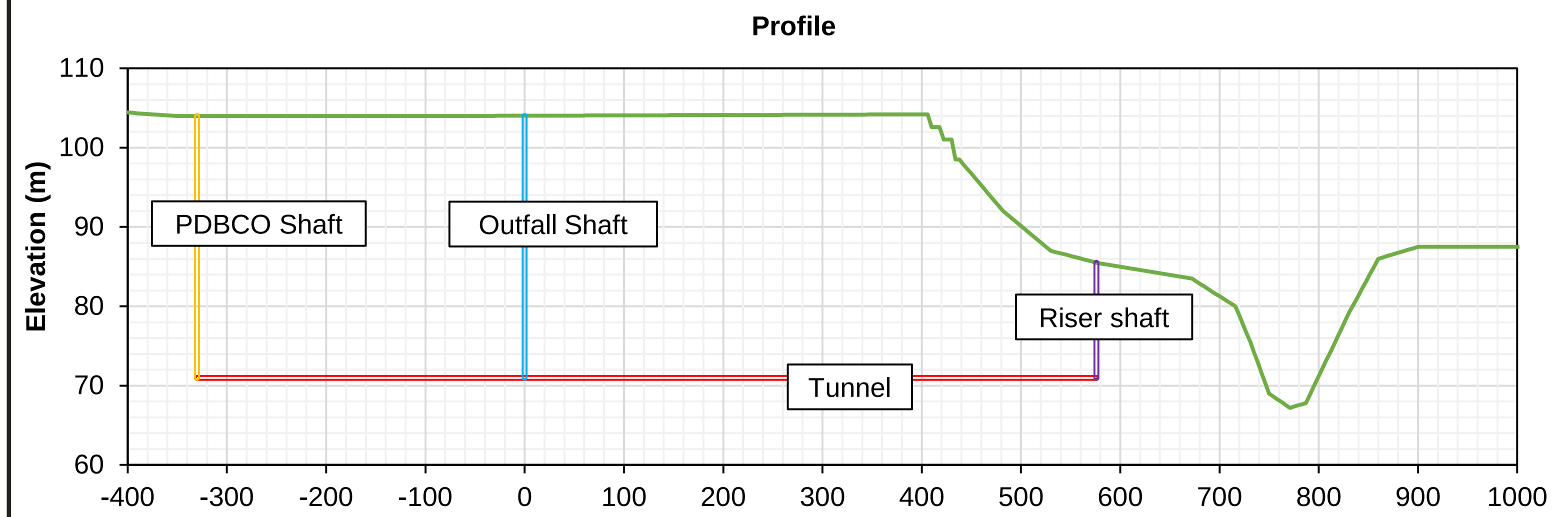
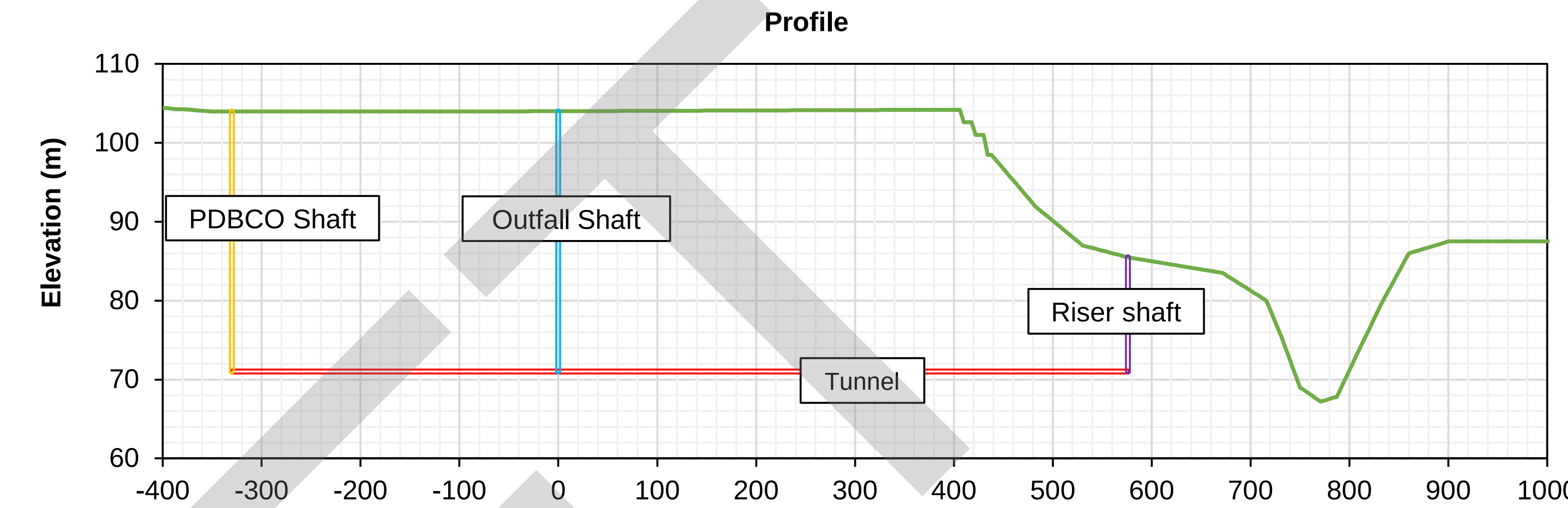
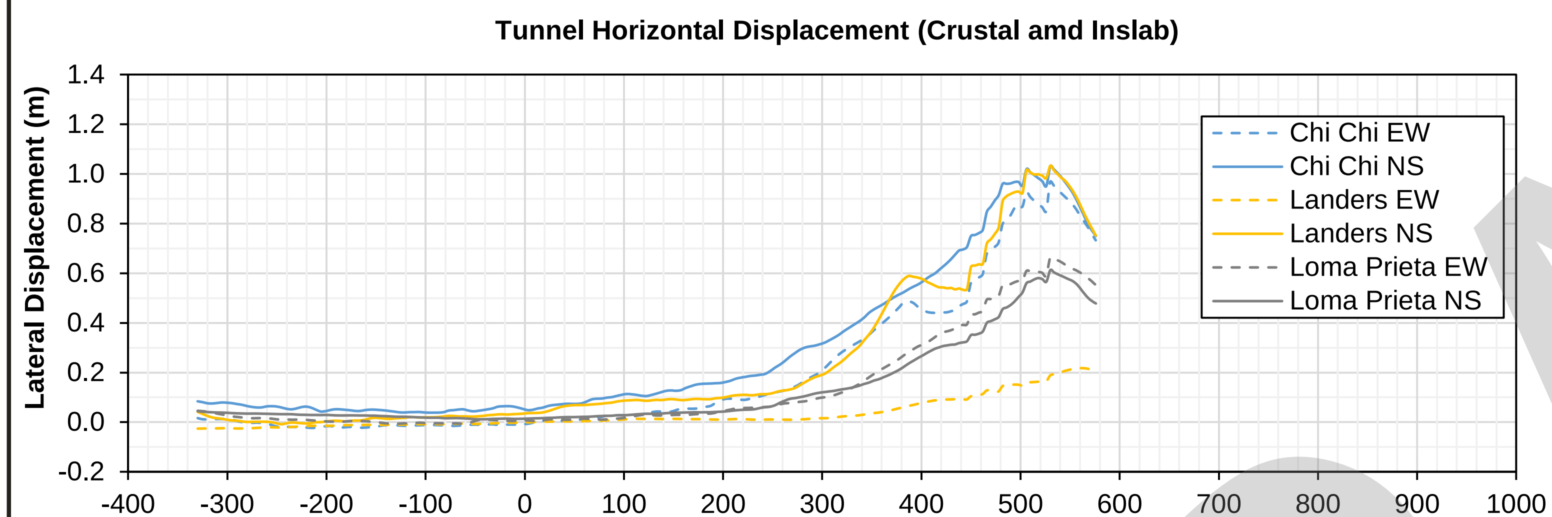
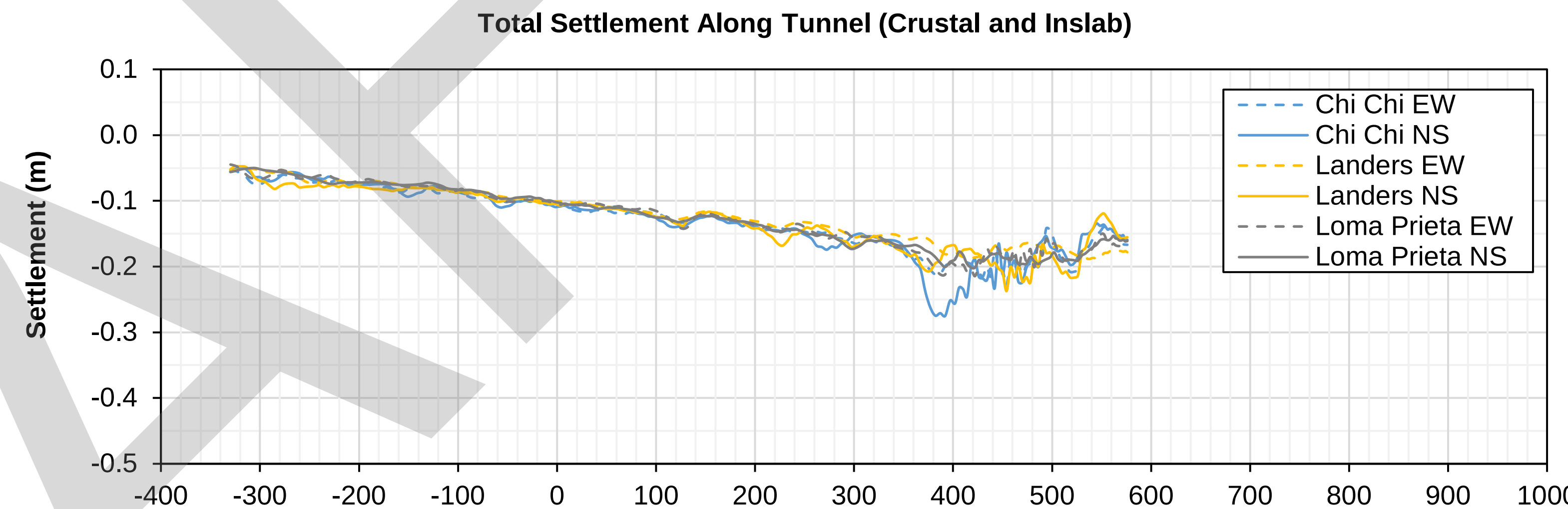
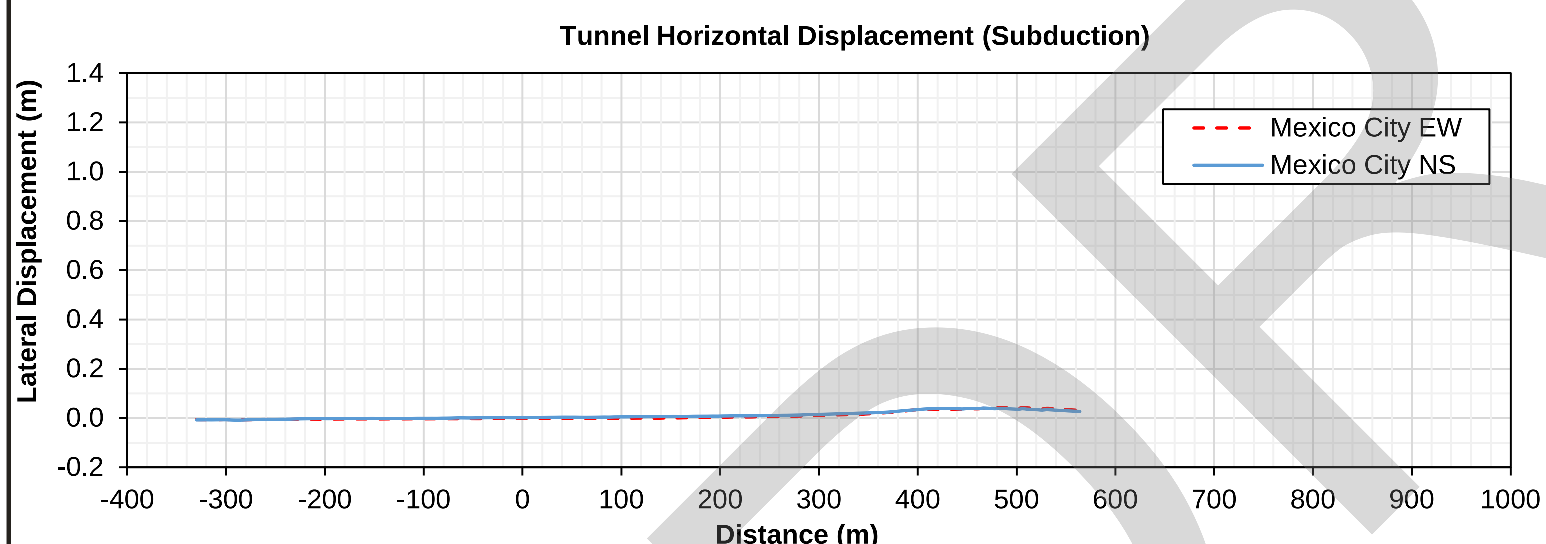
REVIEW	UA	15SEP16		

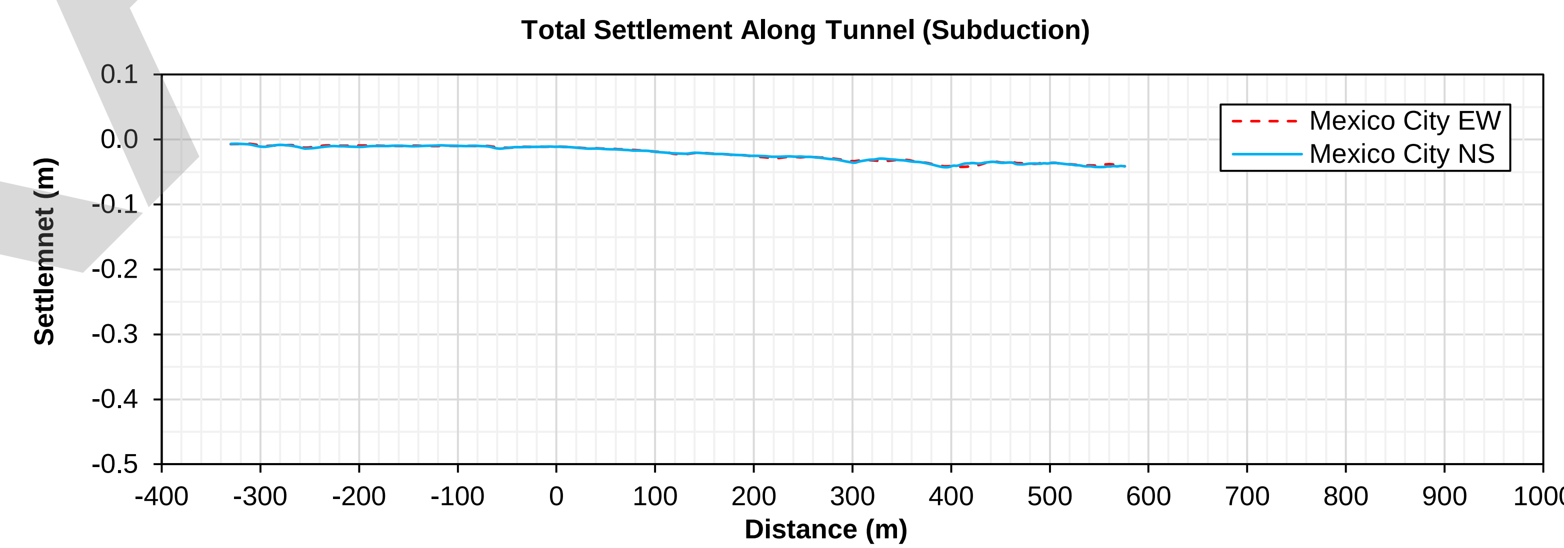










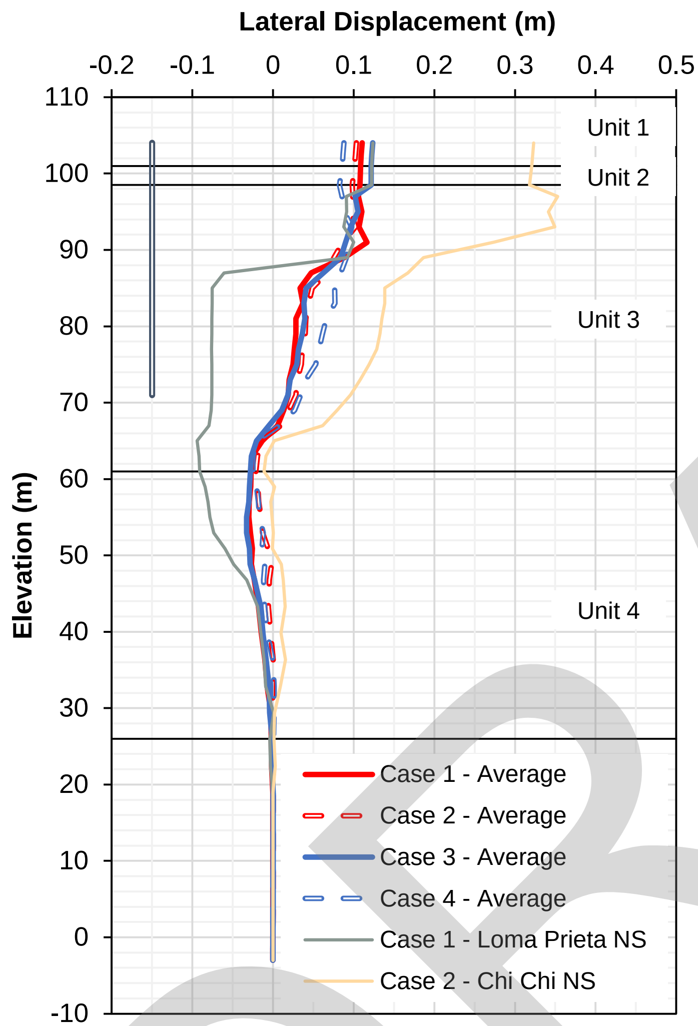


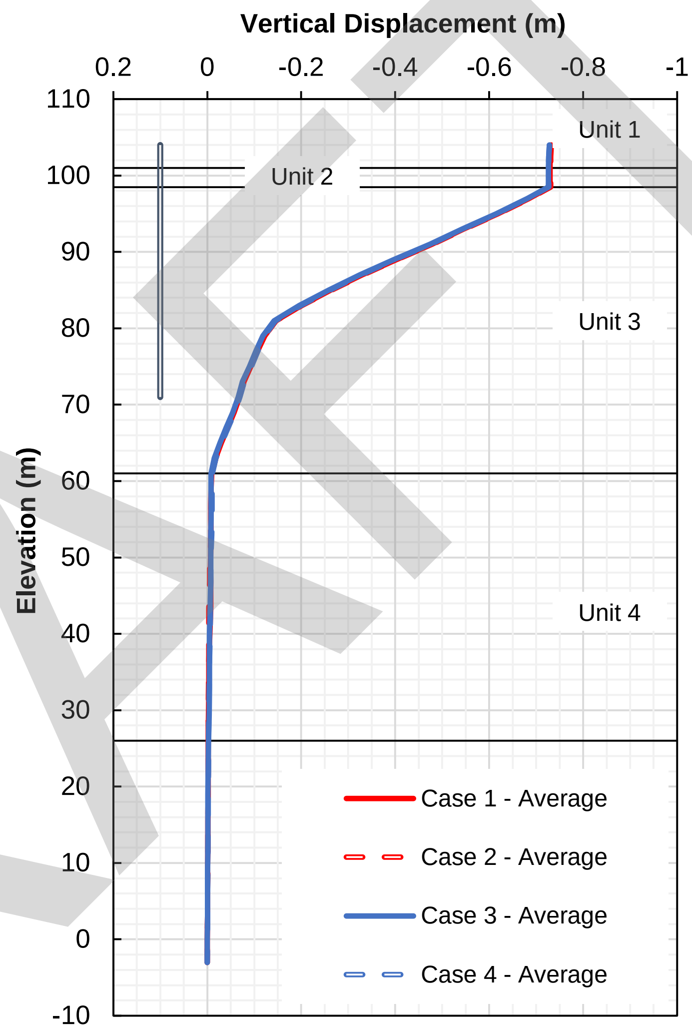
DRAFT

PROJECT	CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
TITLE	Permanent Lateral and Vertical Displacement Profiles At Tunnel Level- Case 4

PROJECT No.	1525010		PHASE No.	2100
DESIGN	VT	15SEP16	SCALE NTS	REV.
CADD	----	----	FIGURE 1-23	
CHECK	VF	15SEP16		
REVIEW	UA	15SEP16		

Golder Associates





<u>Note</u>:

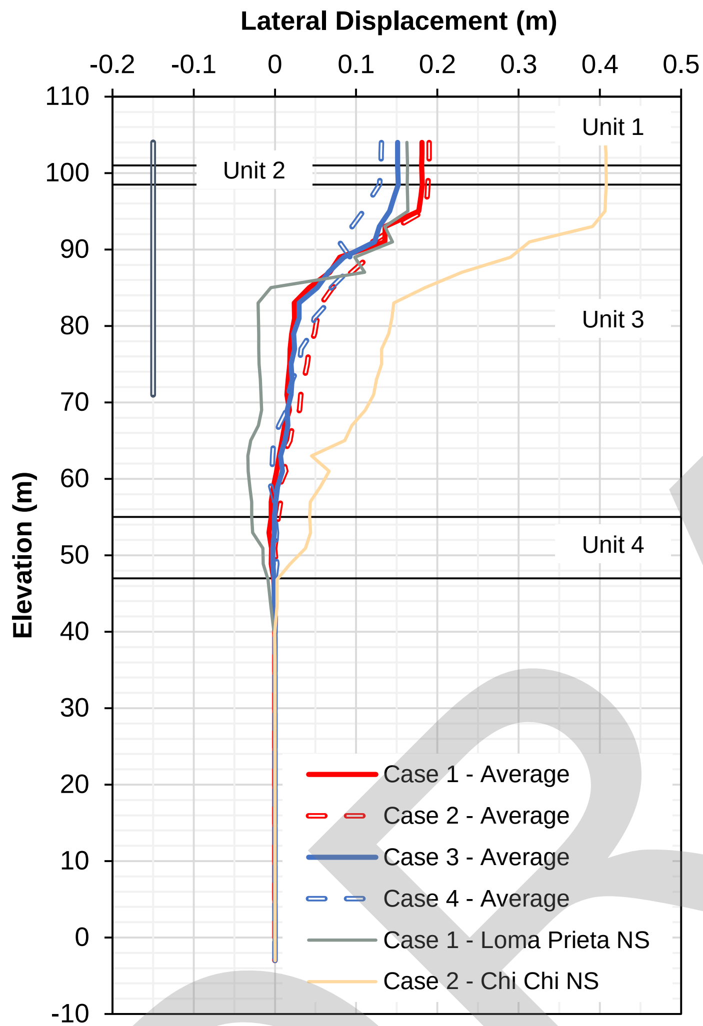
Shown in the plots illustrates the embedment of the shafts

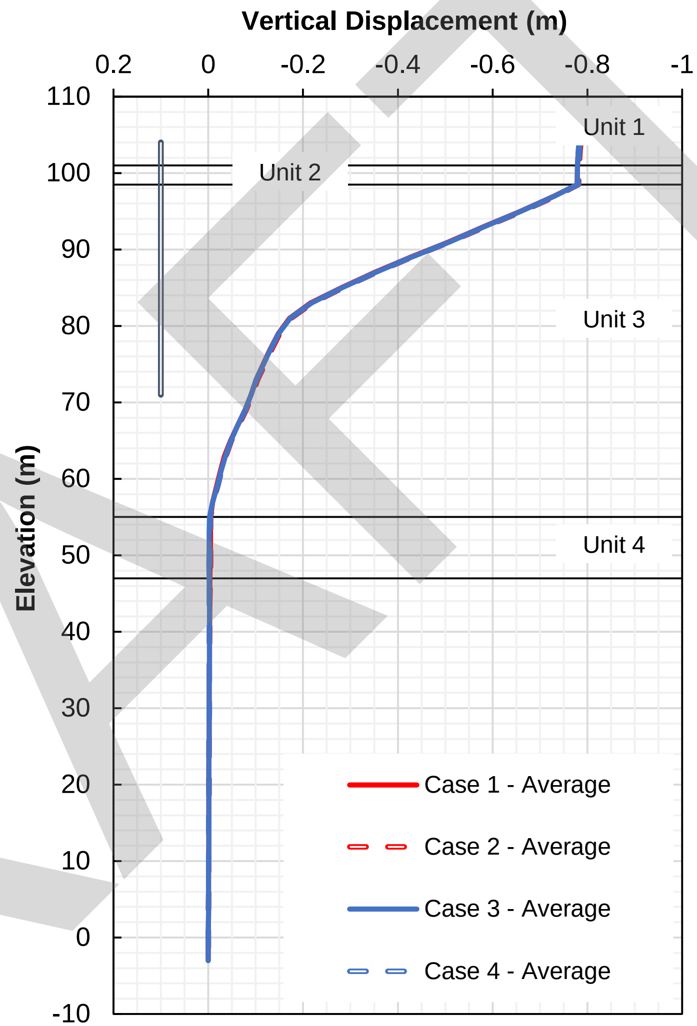
DRAFT

PROJECT	CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC
TITLE	**Lateral and Vertical Displacement Profiles for SSI Analyses PBDCO and Effluent Shaft Locations**

Golder Associates

PROJECT No.	1525010		PHASE No. 2100
DESIGN	VT	15SEP16	SCALE NTS / REV.
CADD	----	----	**FIGURE 2-1**
CHECK	VF	15SEP16	
REVIEW	UA	15SEP16	





Note:

Shown in the plots illustrates the embedment of the shaft

DRAFT

PROJECT	CDM SMITH CANADA ULC AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM DELTA, BC				
TITLE	Lateral and Vertical Displacement Profiles for SSI Analyses - Outfall Shaft Location				
Golder Associates	PROJECT No.	1525010		PHASE No. 2100	
	DESIGN	VT	15SEP16	SCALE NTS	REV.
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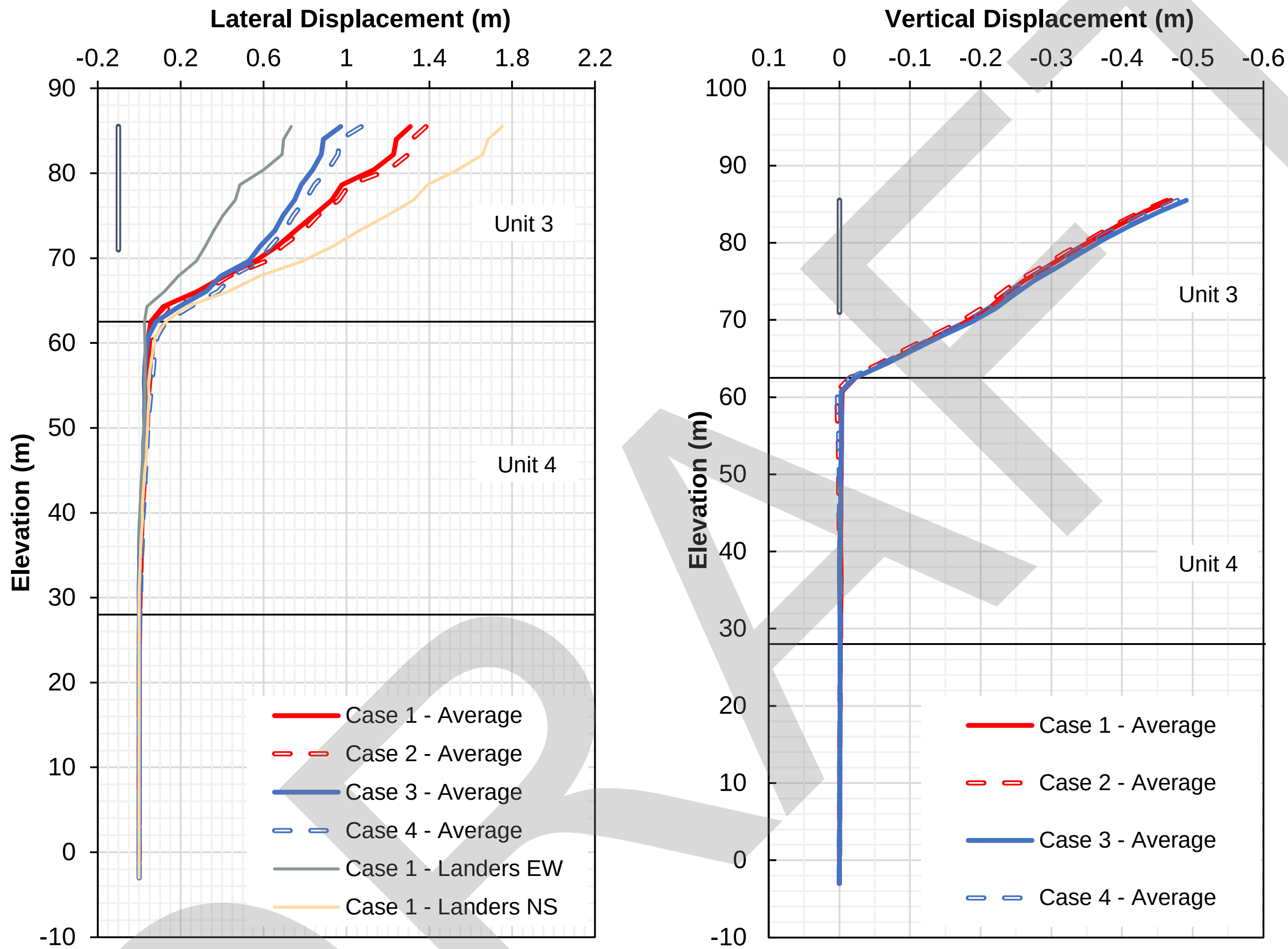
	REVIEW	UA	15SEP16		



Note:

Shown in the plots illustrates the embedment of the shafts

DRAFT

PROJECT

CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE

Lateral and Vertical Displacement Profiles for SSI Analyses - Riser Shaft Location

Golder Associates

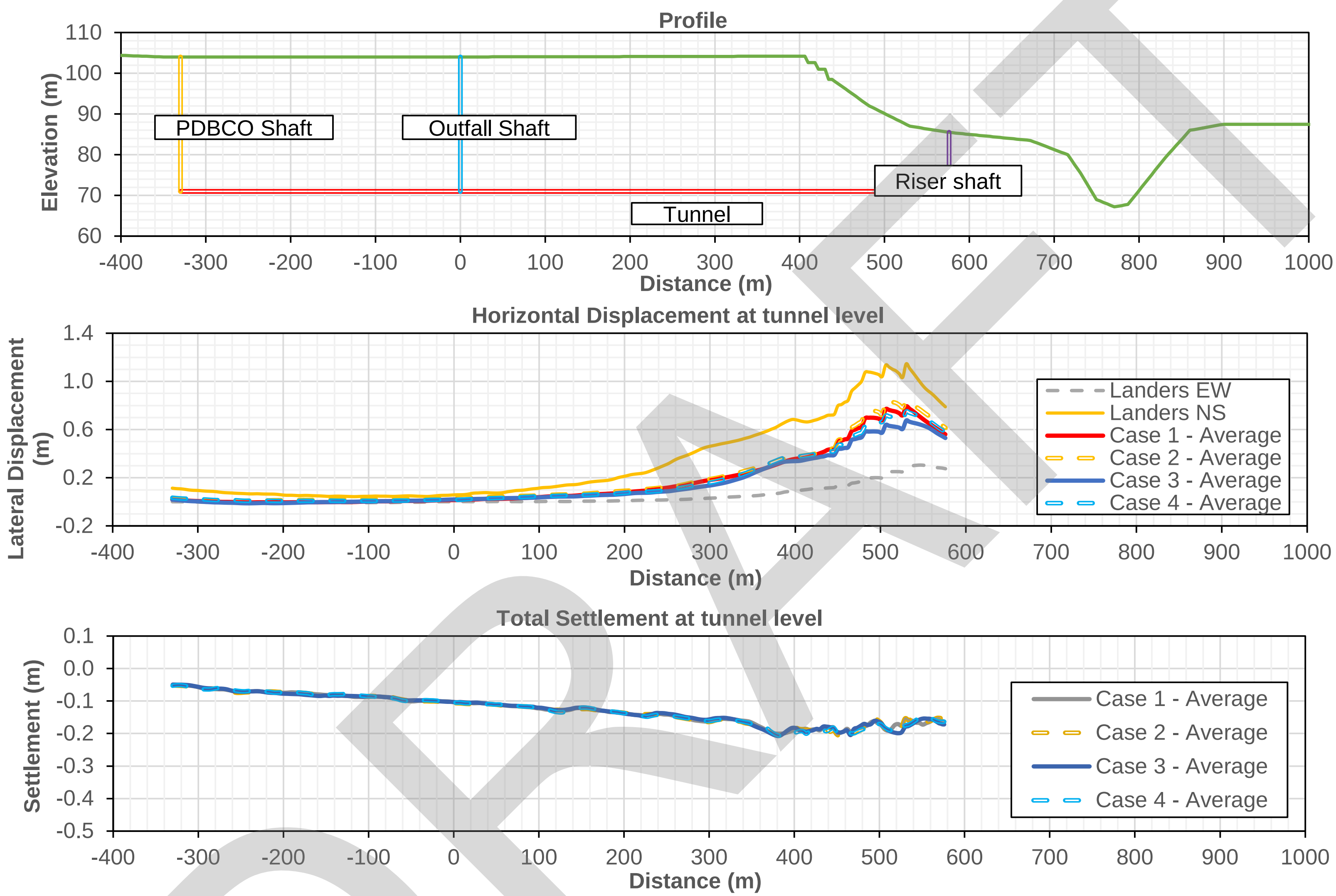
PROJECT No.	1525010	
DESIGN	VT	15SEP16
CADD	----	----
CHECK	VF	15SEP16
REVIEW	UA	15SEP16

PHASE No. 2100

SCALE NTS

REV.

FIGURE 2-3



PROJECT	**CDM SMITH CANADA ULC** **AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM** **DELTA, BC**
TITLE	**Lateral and Vertical Displacement Profiles for SSI Analyses - Tunnel Level**

Golder Associates				
PROJECT No.	1525010		PHASE No.	2100
DESIGN	VT	15SEP16	SCALE NTS	REV.
CADD	----	----	**FIGURE 2-4**	
CHECK	VF	15SEP16		
REVIEW	UA	15SEP16		

TECHNICAL MEMORANDUM

DATE 6 January 2017 **REFERENCE No.** 1525010-062-TM-RevA/2000

TO John Newby
Project Manager, CDM Smith Canada ULC

FROM Yannick Wittwer/Mahmood Seid-Karbasi/Viji Fernando/Trevor Fitzell **EMAIL** Viji_Fernando@golder.com

RESULTS OF TWO DIMENSIONAL GROUND RESPONSE ANALYSES BASED ON SEISMIC HAZARD PARAMETERS CONSISTENT WITH THE 2015 NBCC
ANNACIS ISLAND WWTP TRANSIENT MITIGATION AND OUTFALL, DELTA, BC

This is an interim Technical Memorandum presenting a summary of the results of the two-dimensional (2D) ground response analyses carried out using design ground motion parameters consistent with the 2015 National Building Code of Canada (NBCC). An interim Technical Memorandum dated September 15, 2016 was issued previously summarizing the results of the 2D ground response analyses using design ground motions consistent with the 2010 NBCC. The ground deformation analyses were previously carried out for the following cases:

- Case 1: Design SPT$(N_1)_{60}$ profile based on combined penetration resistance data from both onshore and offshore areas.
- Case 2: Design SPT$(N_1)_{60}$ profile same as Case 1 with the input ground motions applied with reversed polarity.
- Case 3: Design SPT$(N_1)_{60}$ profiles considering the penetration resistance data from the onshore and offshore areas separately.
- Case 4: Design SPT$(N_1)_{60}$ profile same as Case 3 with the input ground motions applied with reversed polarity.

The analyses for the seismic hazard parameters associated with the 2015 NBCC were carried out to assess the sensitivity of the site response to the updated 2015 seismic hazard parameters and were carried out only considering the SPT$(N_1)_{60}$ profile consistent with Case 1 for comparison purposes at this time.

1.0 INPUT GROUND MOTIONS – 2015 NBCC

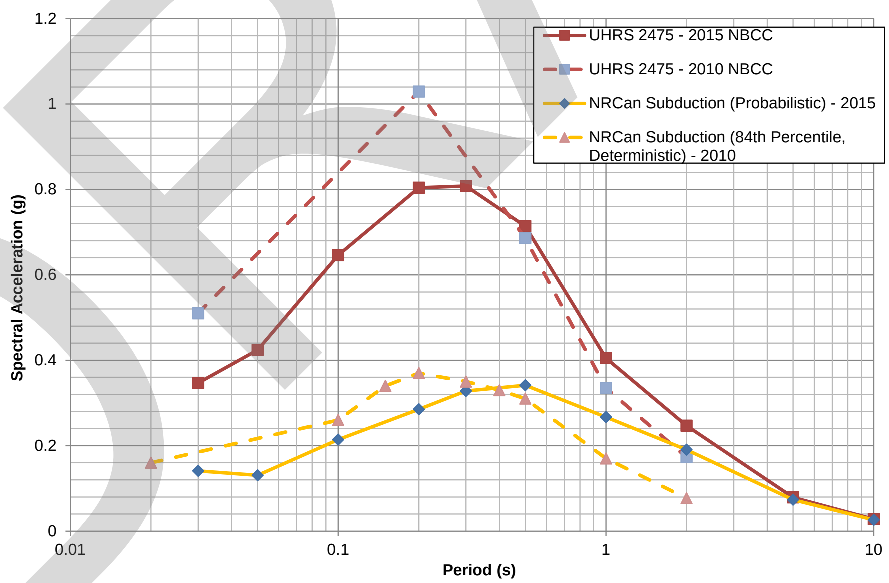
A total of eleven single-component acceleration time-histories to represent the crustal and inslab earthquakes and a total of five single-component acceleration time-histories to represent the interface (subduction) earthquakes were used in the ground response analyses for the design ground motions consistent with the 2015 NBCC. Further details on the ground motions are provided in Golder's Technical Memorandum "Site Seismicity, Seismic Performance Expectations and Input Ground Motions, Annacis Outfall WWTP Transient Mitigation and Outfall System" dated July 6, 2016. Table 1-1 provides a summary of the seed motions selected for development of acceleration-time histories for analyses.

Table 1-1: Details of the Ground Motions - Stage V Expansion [2,475-Year Return Period]

Earthquake	Year of the Earthquake	Magnitude	Station ID	Type of Earthquake
Northridge, CA	1994	M6.7	MAN	Crustal
Loma Prieta, CA	1989	M6.9	SJTE	Crustal
Loma Prieta, CA	1989	M6.9	G06	Crustal
Loma Prieta, CA	1989	M6.9	CAP	Crustal
Northridge, CA	1994	M6.7	W15	Crustal
El Mayor-Cucapah, MX	2010	M7.2	MDO	Crustal
Nisqually, WA	2001	M6.8	USGS 7032	Inslab
Nisqually, WA	2001	M6.8	USGS 2101	Inslab
Geiyo, Japan	2001	M6.8	EHM007	Inslab
Geiyo, Japan	2001	M6.8	EHM016	Inslab
Nisqually, WA	2001	M6.8	USGS 7008	Inslab
Tohoku, Japan	2011	M9.0	YMT009	Interface
Tohoku, Japan	2011	M9.0	TCGH09	Interface
Tohoku, Japan	2011	M9.0	YMTH01	Interface
Maule, Chile	2010	M8.8	USC ME	Interface
Maule, Chile	2010	M8.8	USC LACH	Interface

The 2,475-yr Uniform Hazard Response Spectra (UHRS) and the interface earthquake spectra provided by Natural Resources Canada (NRCan) for the 2010 NBCC and 2015 NBCC are shown in Figure 1-1.

Figure 1-1: Comparison of 2475-yr UHRS and Interface Response Spectra



Acceleration response spectra of the input motions along with the design spectra corresponding to the 2015 NBCC are shown on Figures 1-2a and 1-2b.

Figure 1-2a: Acceleration Response Spectra – Crustal and Inslab Earthquakes.

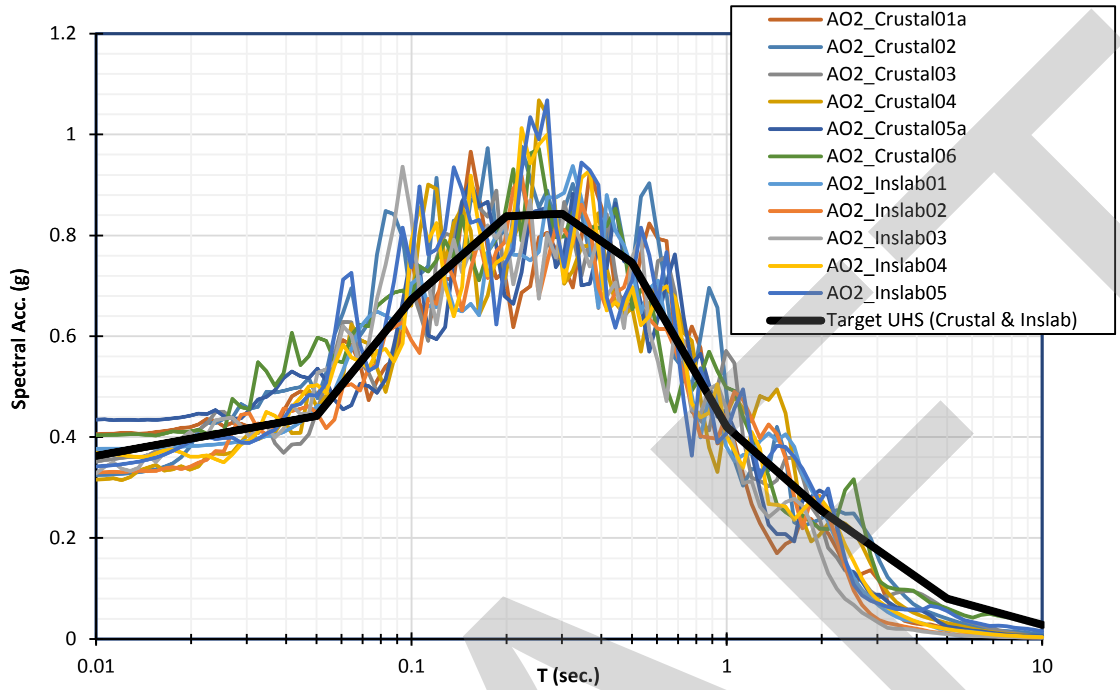
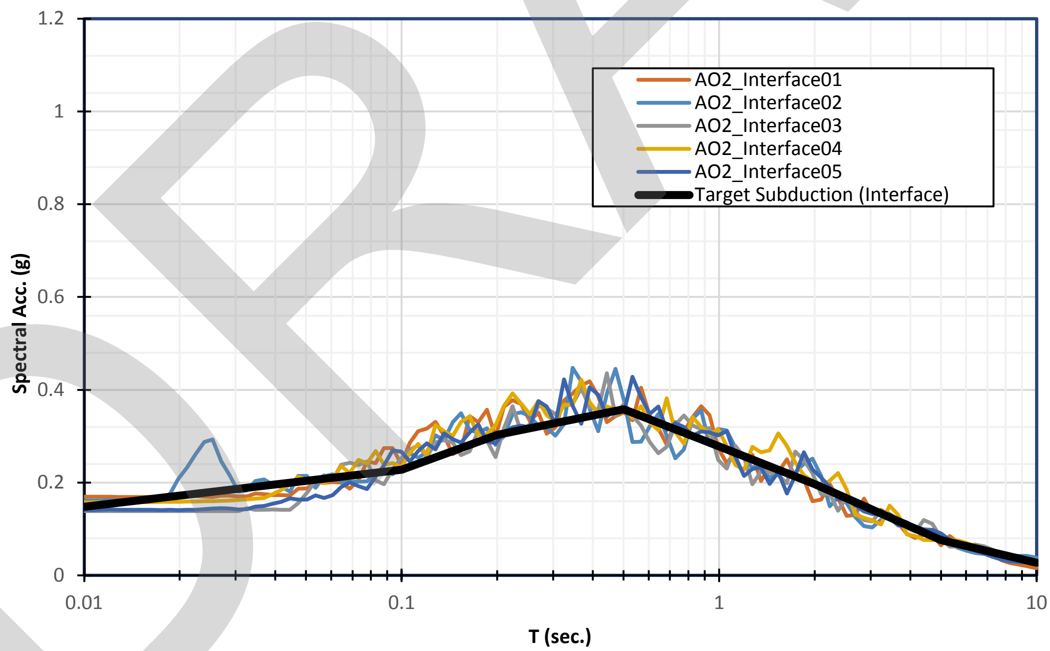


Figure 1-2b: Acceleration Response Spectra – Interface Earthquakes.



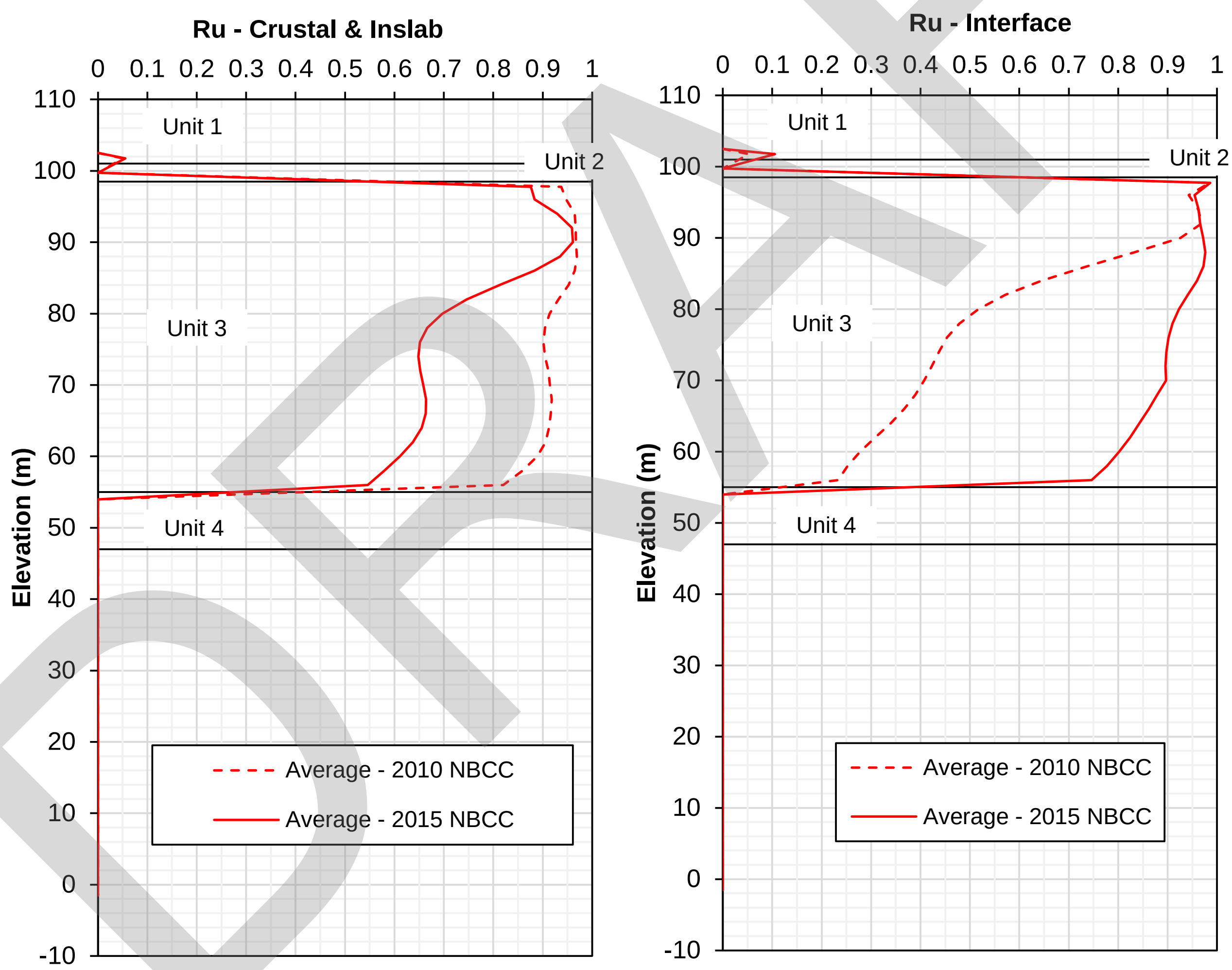
2.0 RESULTS OF 2D GROUND RESPONSE ANALYSES

This section presents the results of the 2D ground response analyses completed for the design ground motions consistent with the 2015 NBCC. A comparison was also made between the excess pore pressure ratios (Ru) and the lateral displacements predicted previously at the shaft locations consistent with the 2010 NBCC and those predicted for the motions consistent with the 2015 NBCC. The results of the analyses are as noted below.

Outfall Shaft

The previous analysis suggested that the entire sand deposit at the on-land shaft locations may liquefy under the crustal and inslab design ground motions consistent with the 2010 NBCC, while in the current analysis the predicted liquefaction depth is limited to the upper 20 m under the crustal and inslab motions consistent with the 2015 NBCC motions (see Figure 2-1). In contrast, the upper 12 m was predicted to liquefy under the interface ground motions consistent with the 2010 NBCC, while the upper 25 m of the sand deposit is predicted to liquefy under the 2015 NBCC interface ground motions (see Figure 2-1).

Figure 2-1: Expected Excess Pore Pressure Ratios for Outfall Shaft.

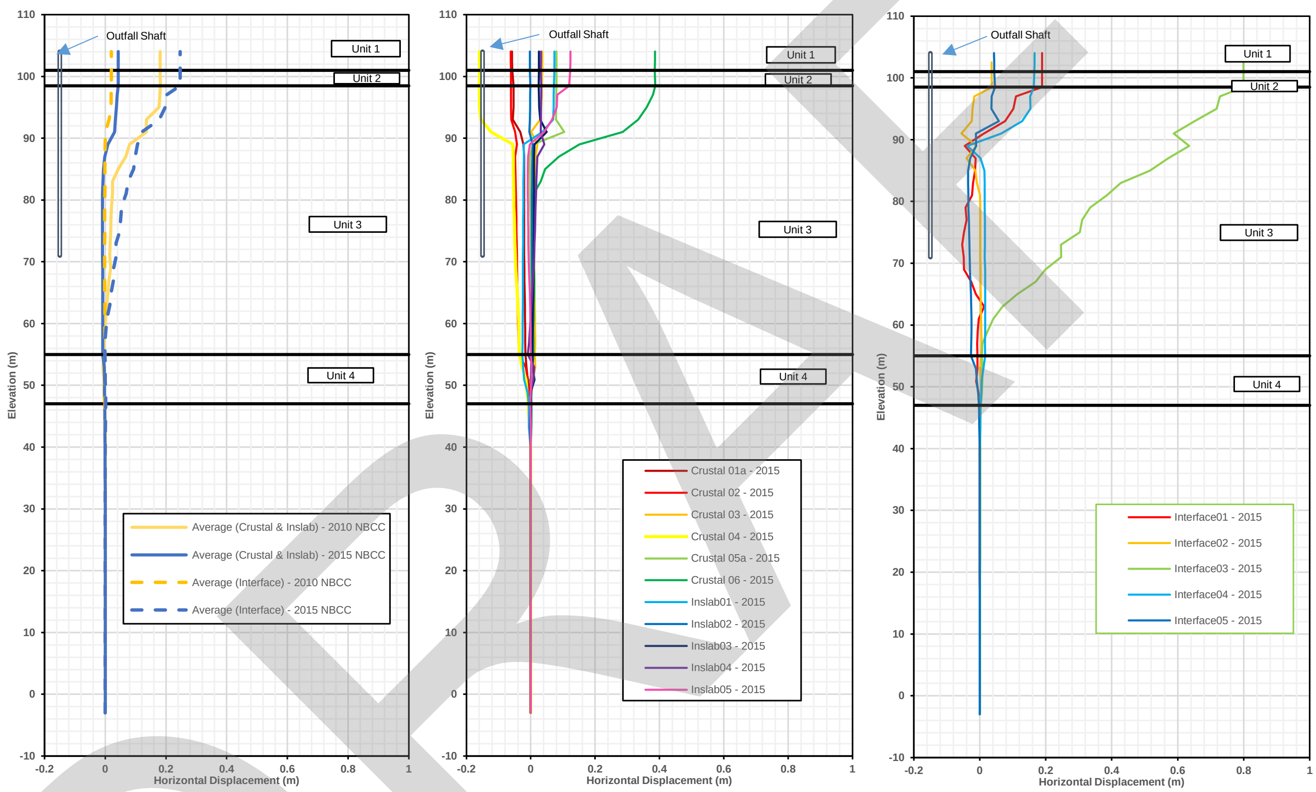


Note: An Ru of 85% and higher is considered indicative of liquefaction

The average lateral displacement profiles at the outfall shaft location under the crustal and inslab design ground motions consistent with the 2010 NBCC are generally higher than those predicted under the crustal and inslab design ground motions consistent with the 2015 NBCC (see Figure 2-2). On the contrary, the lateral displacement profiles are generally higher under the interface ground motions consistent with the 2015 NBCC compared to the motions consistent with the 2010 NBCC (see Figure 2-2). In addition, the lateral displacements predicted under the interface ground motions consistent with the 2015 NBCC are generally similar or higher (i.e. up to 20%) compared to those predicted under the crustal and inslab motions consistent with the 2010 NBCC.

The computed lateral displacements, considering the 16 earthquake records comprising both crustal and inslab, and the interface sources, associated with the 2015 NBCC are also shown on Figure 2-2.

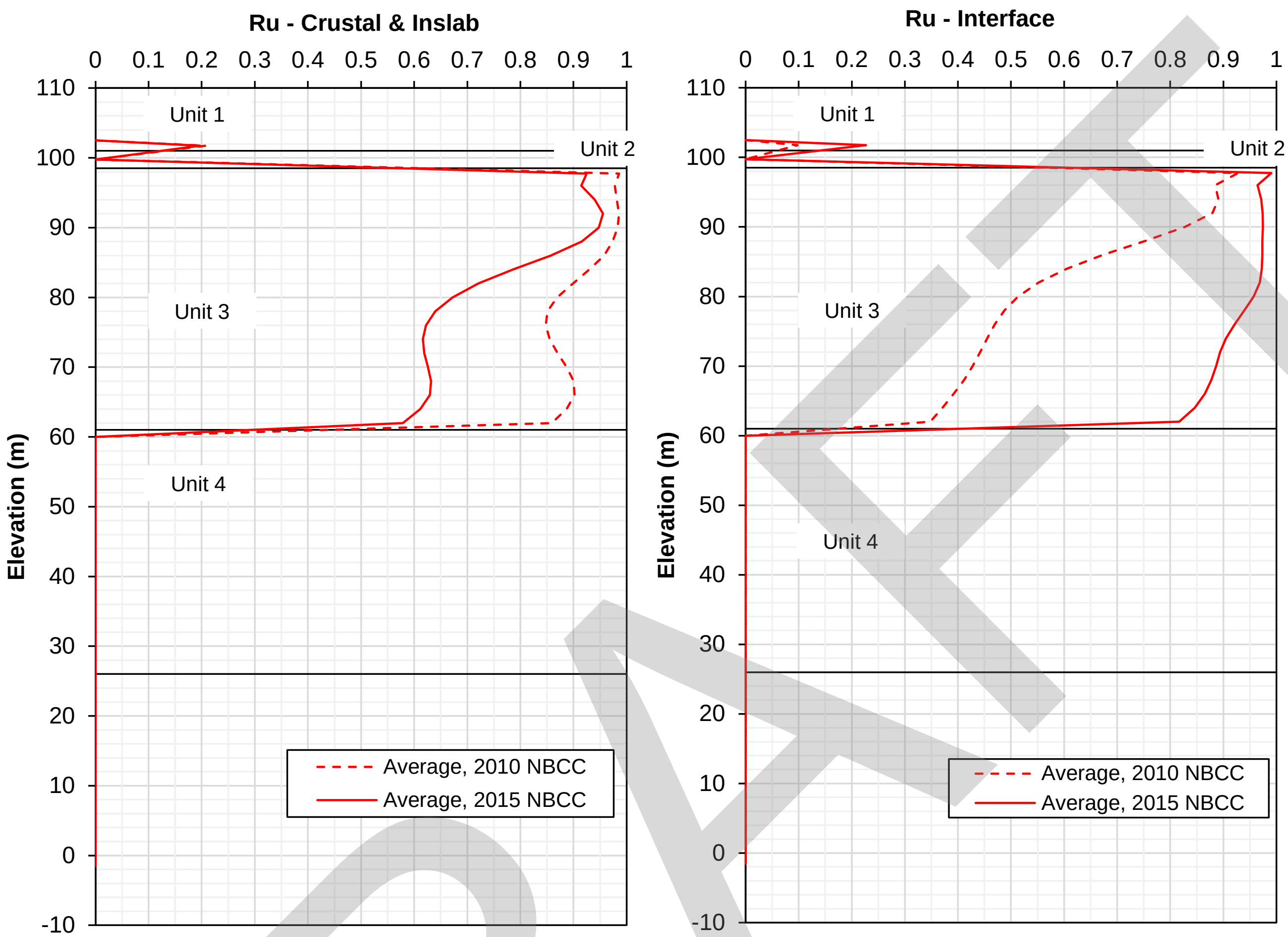
Figure 2-2: Computed Lateral Displacement Profiles (Outfall Shaft).



PDBCO Shaft

The previous analysis indicated that the entire sand deposit may liquefy under the crustal and inslab design ground motions consistent with the 2010 NBCC, while the predicted extent of liquefaction is limited to the upper 15 m under the crustal and inslab motions consistent with the 2015 NBCC (see Figure 2-3). On the contrary, only the upper 10 m is predicted to liquefy under the interface ground motions consistent with the 2010 NBCC, while the entire sand deposit may liquefy when subjected to the 2015 NBCC interface ground motions (see Figure 2-3).

Figure 2-3: Expected Excess Pore Pressure Ratios for PDBCO Shaft.

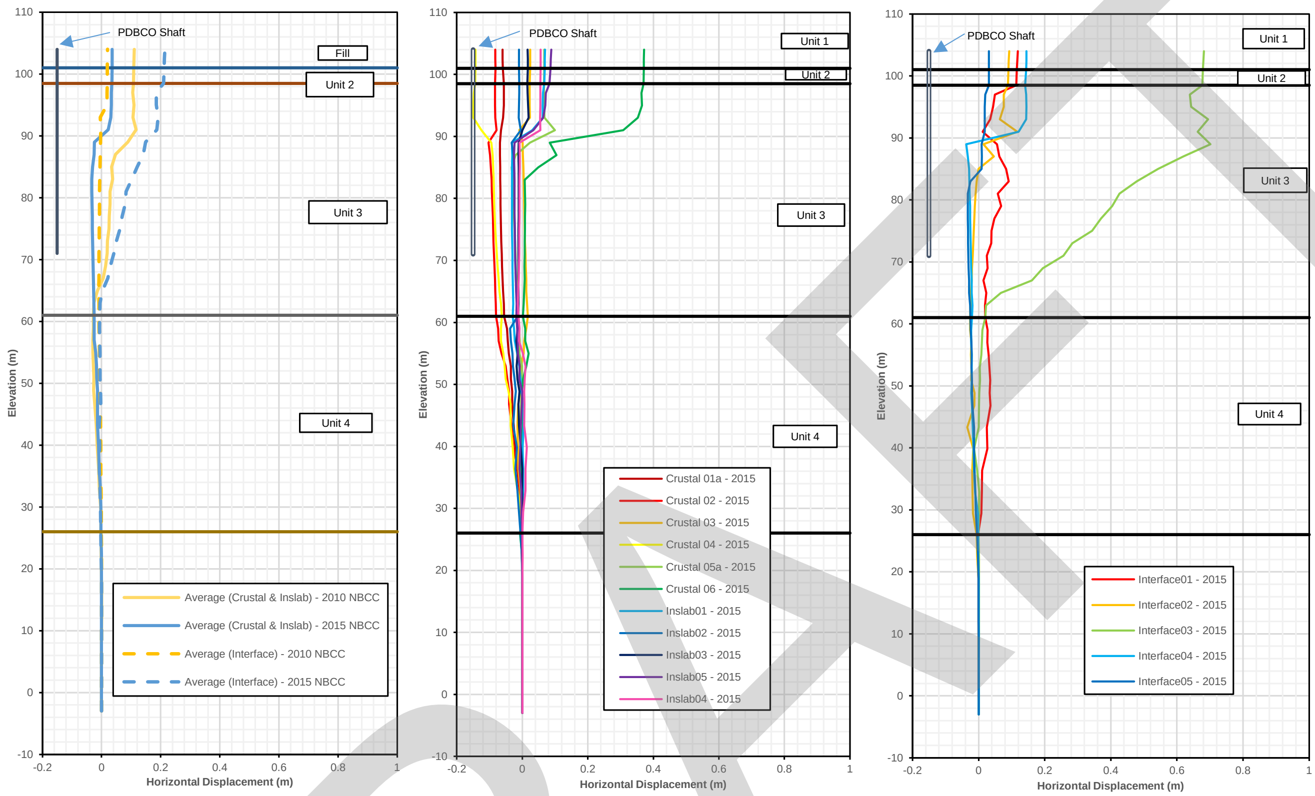


Note: An Ru of 85% and higher is considered indicative of liquefaction

Similar to at the outfall shaft location, the predicted average lateral displacement profiles at the PDBCO shaft location under the crustal and inslab design ground motions consistent with the 2010 NBCC are generally higher than those predicted under the crustal and inslab design ground motions consistent with the 2015 NBCC (see Figure 2-4). Also, similarly, the lateral displacement profiles are generally higher under the interface ground motions consistent with the 2015 NBCC compared to the interface motions associated with the 2010 NBCC (see Figure 2-4). Furthermore, the lateral displacements predicted under the interface ground motions consistent with the 2015 NBCC are generally higher (i.e., up to 50%) compared to those predicted under the crustal and inslab motions consistent with the 2010 NBCC.

The computed lateral displacements, considering the 16 earthquake records comprising crustal and inslab, and the interface sources, associated with the 2015 NBCC are also shown on Figure 2-4.

Figure 2-4: Computed Lateral Displacement Profiles (PDBCO Shaft).



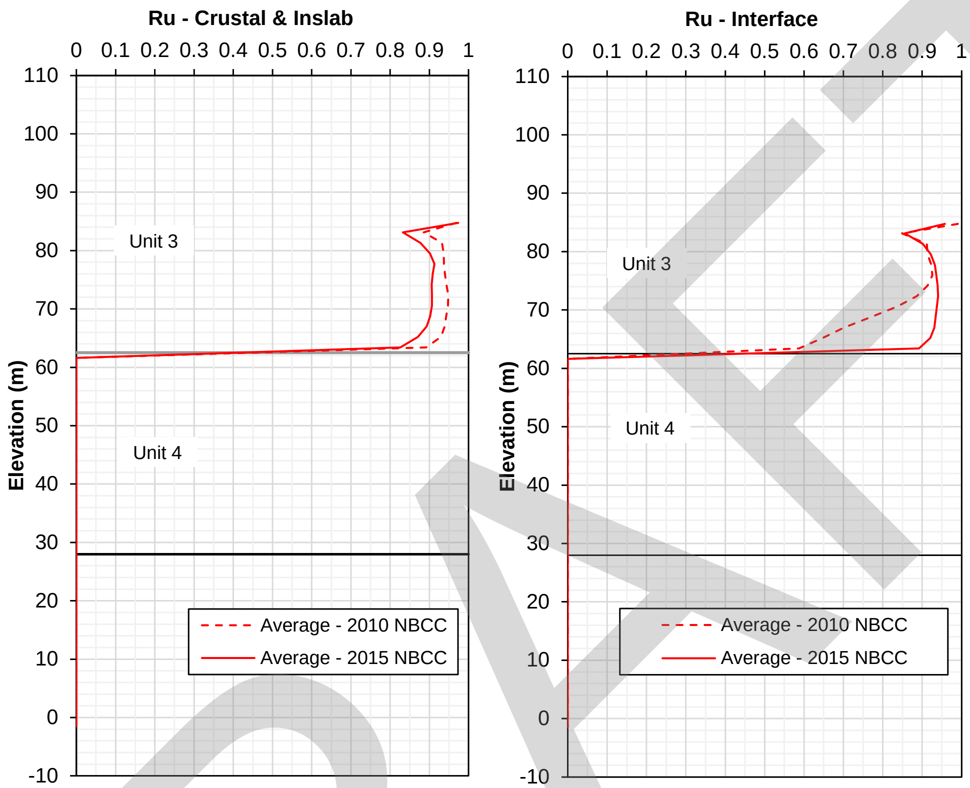
Riser Shaft

The analyses indicate that the entire sand deposit may liquefy under the crustal and inslab design ground motions consistent with both the 2010 NBCC and 2015 NBCC. The liquefaction is limited to the upper 15 m under the interface motions consistent with the 2010 NBCC, while it extends through the entire deposit under the 2015 interface motions (See Figure 2-5).

The average lateral displacement profiles at the riser shaft location under the crustal and inslab design ground motions consistent with the 2010 NBCC are generally higher than those predicted under the crustal and inslab design ground motions consistent with the 2015 NBCC. The lateral displacement profiles are generally higher under the interface ground motions consistent with the 2015 NBCC compared to the 2010 NBCC interface ground motions (see Figure 2-6). In addition, the displacements predicted under the interface motions consistent with the 2015 NBCC are generally similar to those predicted under the crustal and inslab motions consistent with the 2010 NBCC (See Figure 2-6).

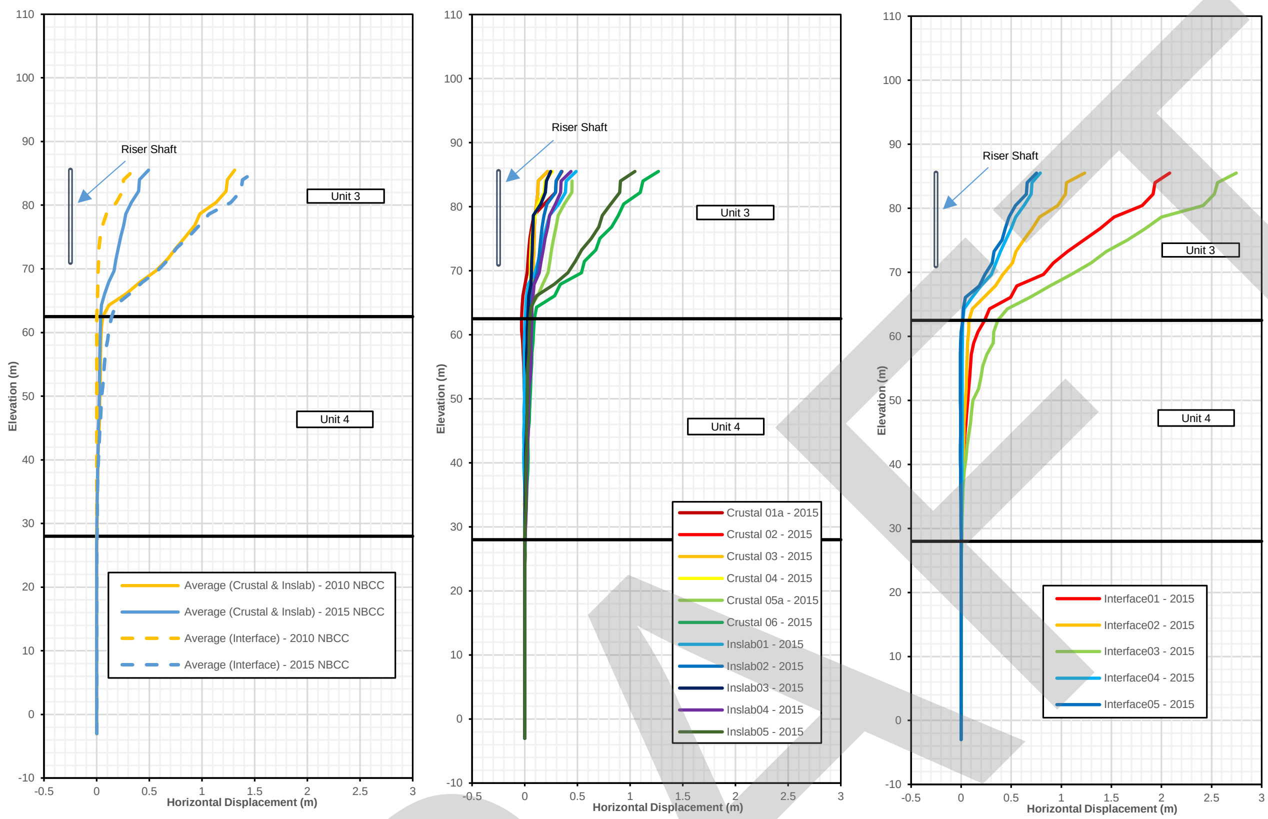
The computed lateral displacements, considering the 16 earthquake records comprising crustal and inslab, and the interface sources, associated with the 2015 NBCC are also shown on Figure 2-6.

Figure 2-5: Expected Excess Pore Pressure Ratios for the Riser Shaft.



Note: An Ru of 85% and higher is considered indicative of liquefaction

Figure 2-6: Computed Lateral Displacement Profiles (Riser Shaft).



3.0 CONCLUSION

The lateral displacements associated with the design input for the soil-structure interaction analyses are estimated to be similar for the riser shaft under both 2010 NBCC and 2015 NBCC and they are estimated to be higher (up to 50%) for the 2015 NBCC compared to the 2010 NBCC for the on land shafts. For the design displacement profiles, the governing earthquake source is found to be different under the 2010 NBCC and 2015 NBCC; the crustal and inslab earthquake motions govern the extent of liquefaction and lateral displacements for the 2010 NBCC, while the interface earthquake motions govern under the 2015 NBCC.

4.0 CLOSURE

We trust that the information presented in this interim Technical Memorandum is sufficient for your immediate requirements. Please do not hesitate to contact us if you have questions or require clarification of contents.

Yours truly,

GOLDER ASSOCIATES LTD.

Yannick Wittwer, BASc, P.Eng.
Junior Geotechnical Engineer

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Associate, Senior Geotechnical Engineer

Reviewed by:

Trevor Fitzell, P.Eng.
Principal, Senior Geotechnical Engineer

VT/MSK/VF/kn

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APPENDIX B

Interim Technical Memorandum – 2D Ground Deformation Analyses – 2010 NBCC

TECHNICAL MEMORANDUM

DATE 3 February 2017 **REFERENCE No.** 1525010-068-TM-RevA-2000

TO John Newby
Project Manager, CDM Smith Canada ULC

FROM Yannick Wittwer / Mahmood Seid Karbasi / Viji Fernando / Trevor Fitzell **EMAIL** Viji_Fernando@golder.com

RESULTS OF TWO DIMENSIONAL GROUND RESPONSE ANALYSES BASED ON 50TH PERCENTILE SPT$(N_1)_{60CS}$ VALUES, ANNACIS ISLAND WWTP TRANSIENT MITIGATION AND OUTFALL, DELTA, BC

This is an interim Technical Memorandum presenting the results of additional two-dimensional (2D) ground deformation analyses for the above project. The analyses were carried out to assess the sensitivity of the liquefaction potential of site soils, and the resulting permanent ground deformations, to variations in the design SPT$(N_1)_{60cs}$ values along the proposed Option 6 outfall alignment for "as-is" ground conditions. The analyses were carried out for the design ground motions corresponding to a return period of 2,475 years consistent with the 2010 National Building Code of Canada (NBCC).

It is noted that the ground deformation analyses were previously carried out using a uniform model with respect to the design profiles of SPT$(N_1)_{60cs}$ corresponding to the 33rd percentile values, as per the Task Force guidelines (2007)[1]. However, the recent studies carried out by Boulanger et al. (2016)[2] indicated that the representative $(N_1)_{60cs}$ for use in uniform models can range from the 30th to the 70th percentile of the stochastic $(N_1)_{60cs}$ distributions, depending on the intensity of shaking and the variations in the soil parameters, topography, etc. Considering the potential impact of the $(N_1)_{60cs}$ profiles adopted on the predicted ground deformations, further analyses were carried out in order to evaluate their sensitivity to 50th percentile design profiles of $(N_1)_{60cs}$ rather than the 33rd percentile values used previously.

The analyses were previously carried out for four cases considering variations in the soil parameters on the land and offshore areas, and the results of the sensitivity analyses presented herein are associated with the $(N_1)_{60cs}$ design profiles corresponding to Case 1, considering the 33rd and 50th percentile values.

1.0 DESIGN SPT$(N_1)_{60CS}$ PROFILES

The statistical analyses previously carried out to establish the design $(N_1)_{60cs}$ profile corresponding to 33rd percentile values were re-evaluated to establish the corresponding 50th percentile values. Figure 1-1 shows that the design $(N_1)_{60cs}$ profiles associated with the 33rd and 50th percentile values are about one to three blow counts different.

[1] Geotechnical Guidelines For Buildings On Liquefiable Sites in Greater Vancouver, dated 8 May 2007

[2] Nonlinear deformation analyses of an embankment dam of a spatially variable liquefiable deposit. Ross W Boulanger, Jack Montgomery "Soil Dynamics and Earthquake Engineering", July 2016

2.0 RESULTS OF 2D GROUND RESPONSE ANALYSES

This section presents the results of the 2D ground response analyses completed for the design $(N_1)_{60cs}$ profile corresponding to the 50th percentile values. A comparison was also made between the excess pore pressure ratios (Ru) and lateral displacements predicted previously at the shaft locations using the 33rd percentile values and those predicted for the 50th percentile values. The results of the analyses are presented in the following sections.

2.1 On-land and Riser Shafts

The results of the previous analyses considering the design $(N_1)_{60cs}$ profile corresponding to the 33rd percentile values indicated that the entire sand deposit at the shaft locations could potentially liquefy under the 2,475-year design ground motions consistent with the 2010 NBCC. The extent of liquefaction for the design $(N_1)_{60cs}$ profile corresponding to the 50th percentile values is also found to be similar to that for the 33rd percentile values, as shown in Figures 2-1 through 2-3 below.

The computed excess pore pressure ratios considering the six earthquake records comprising crustal and in-slab sources associated with the 2010 NBCC are also shown on Figures 2-1 through 2-3 for the design $(N_1)_{60cs}$ profiles corresponding to both the 33rd and 50th percentile values.

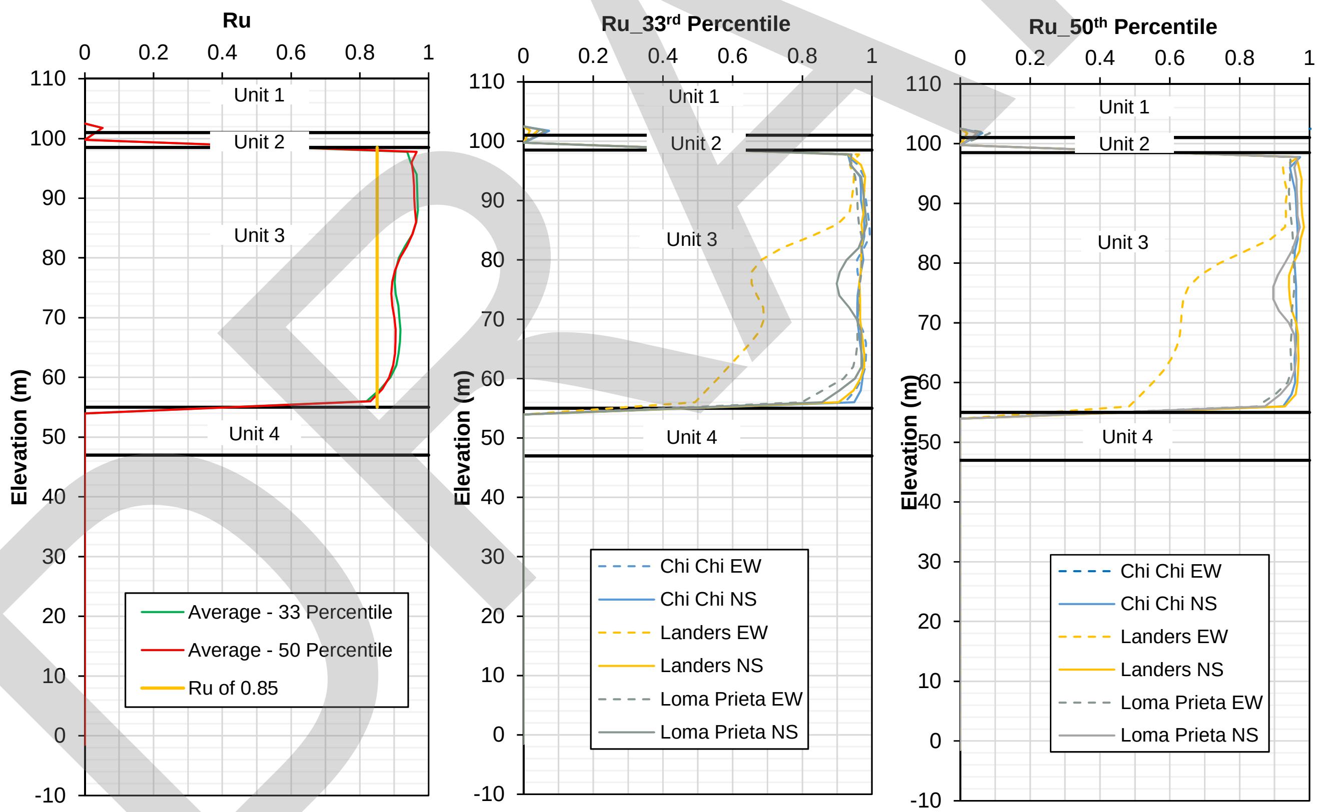

Figure 2-1: Computed Excess Pore Pressure Ratios for Outfall Shaft

Note: Ru of 85% and higher is considered indicative of liquefaction

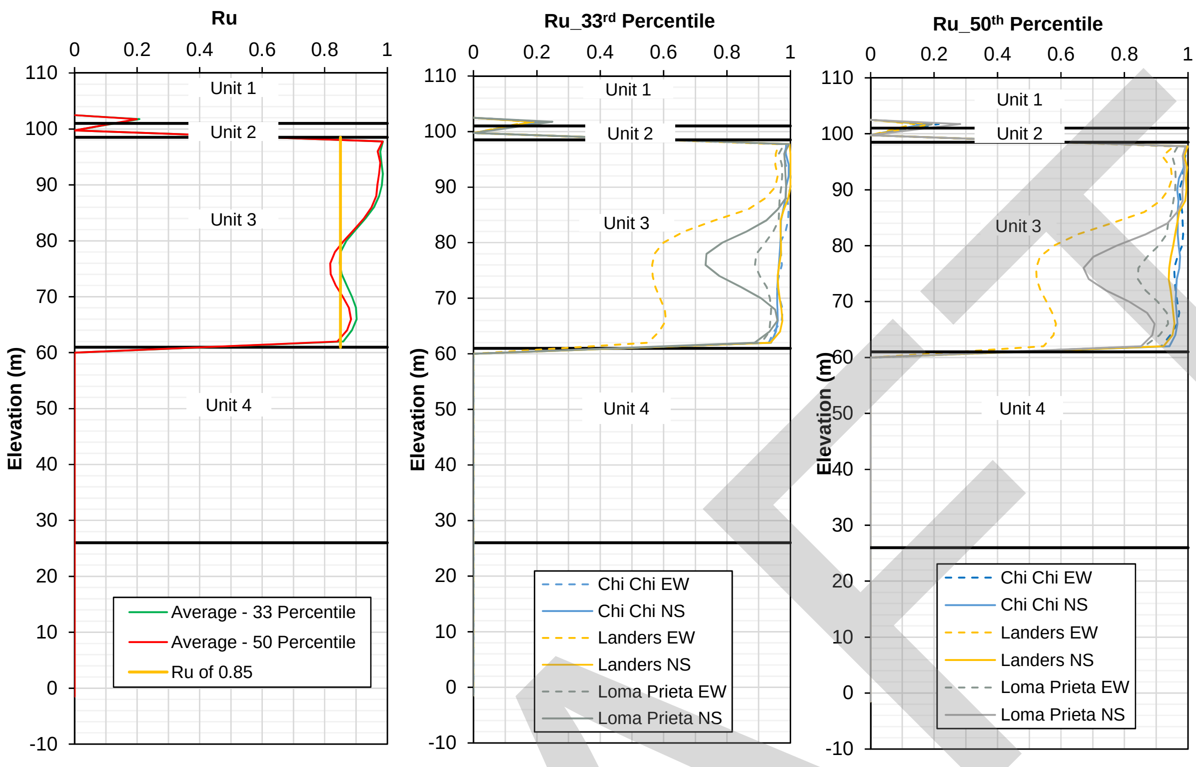

Figure 2-2: Computed Excess Pore Pressure Ratios for PDBCO Shaft

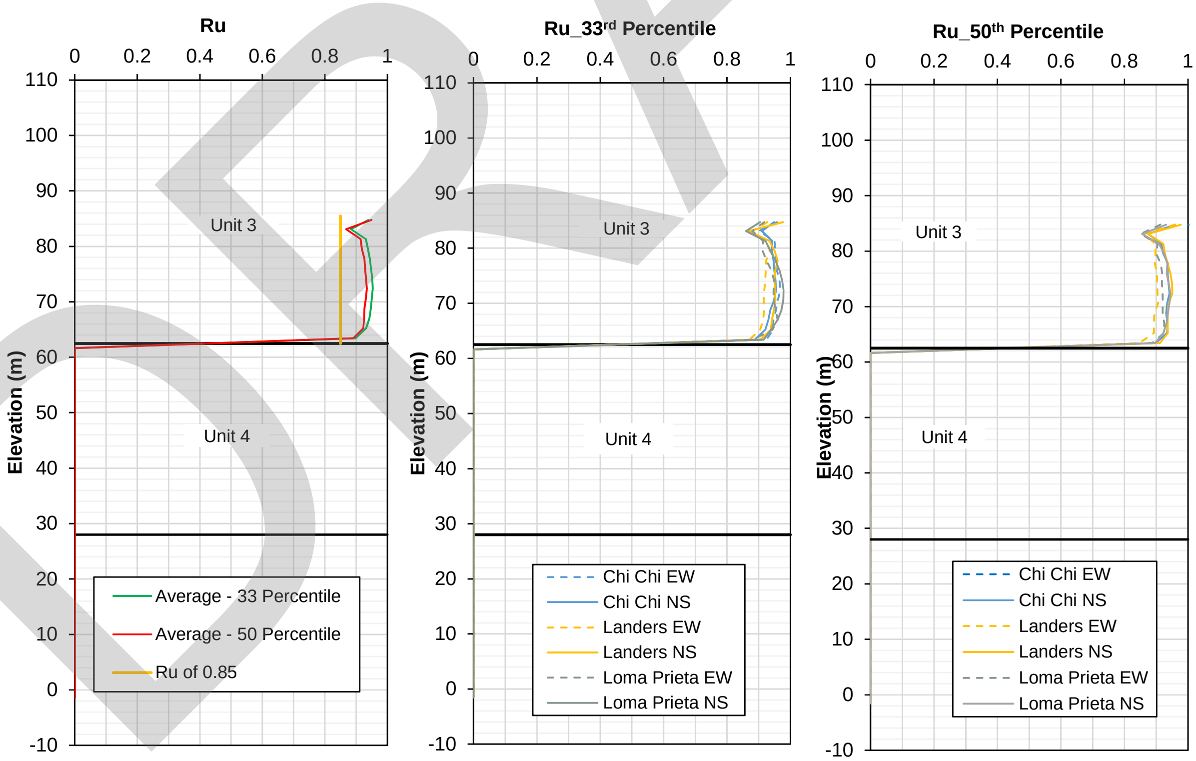

Figure 2-3: Computed Excess Pore Pressure Ratios for Riser Shaft

The average lateral displacement profiles considering the design $(N_1)_{60cs}$ profile corresponding to the 33rd percentile values are slightly higher than those for the 50th percentile values, as sown on Figures 2-4 and 2-5 at the Outfall and PDBCO shaft locations. However, the average lateral displacements at the riser shaft location are generally about thirty percent lower for the design profile corresponding to the 50th percentile values compared to those for the 33rd percentile values, as shown on Figure 2-6.

The computed lateral displacements, considering the six earthquake records comprising crustal and in-slab sources, associated with the 2010 NBCC, are also shown on Figures 2-4 through 2-6 for the design $(N_1)_{60cs}$ profiles corresponding to both the 33rd and 50th percentile values.

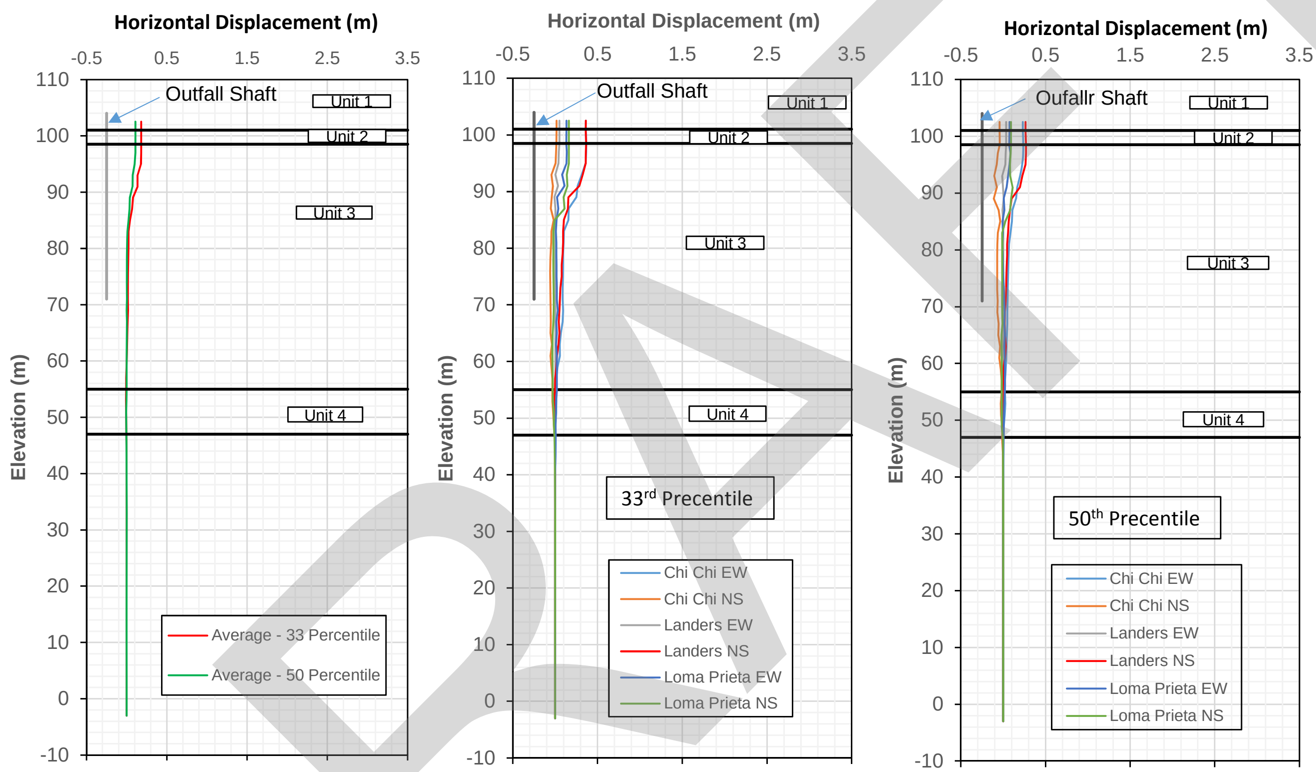

Figure 2-4: Computed Lateral Displacement Profiles (Outfall Shaft)

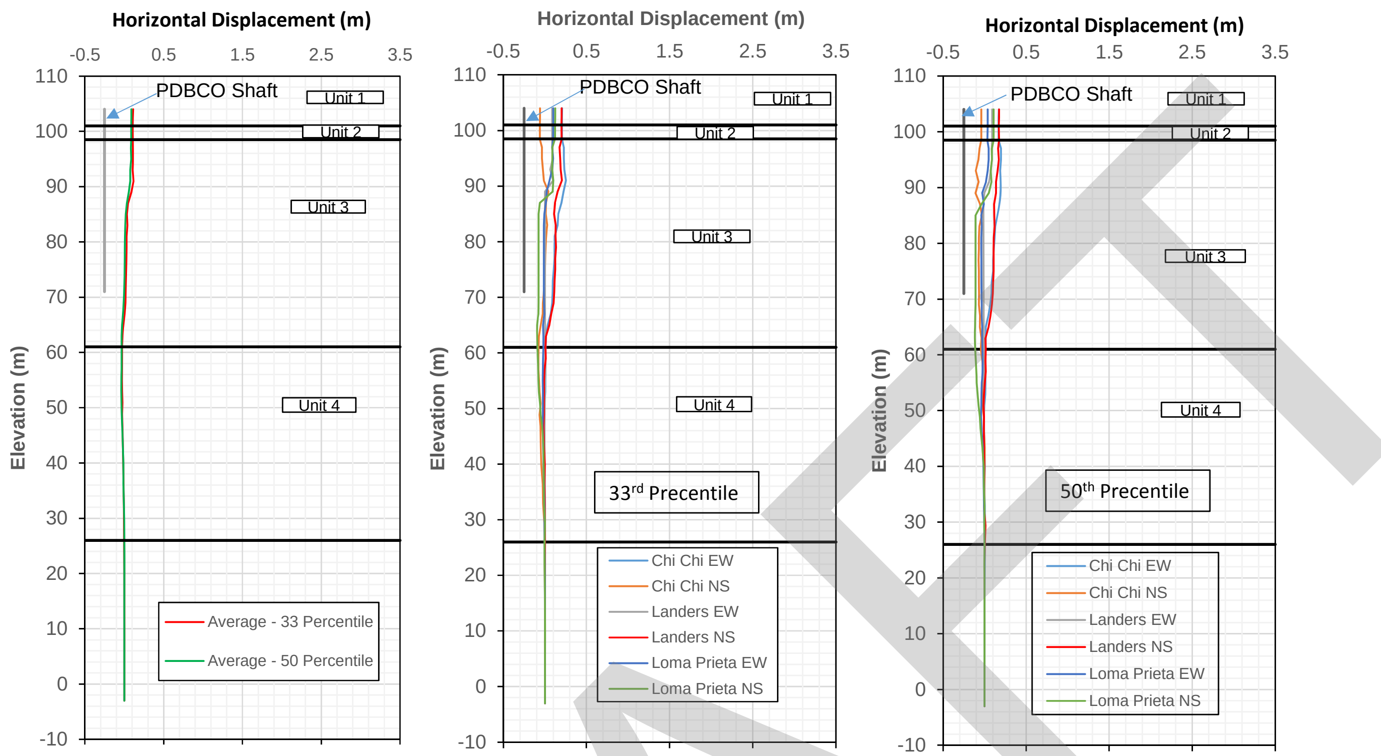

Figure 2-5: Computed Lateral Displacement Profiles (PDBCO Shaft)

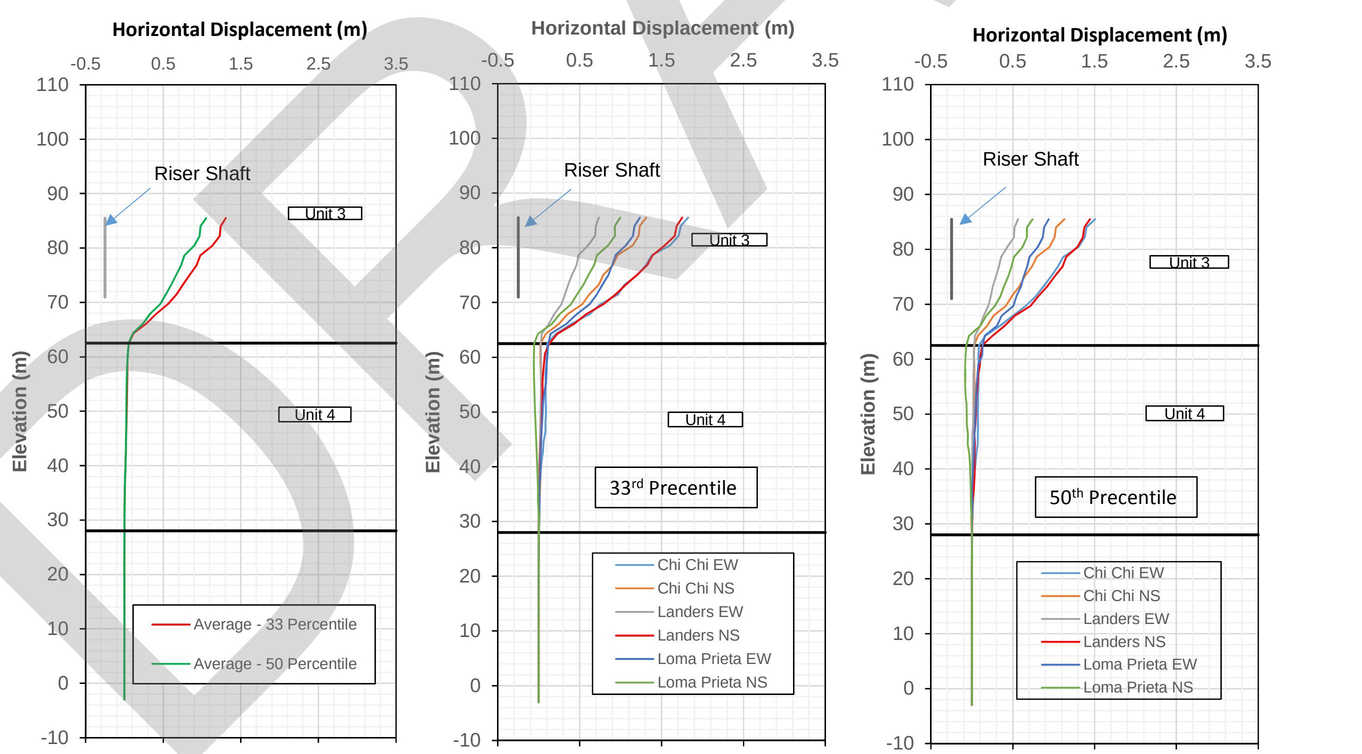

Figure 2-6: Computed Lateral Displacement Profiles (Riser Shaft)

3.0 CONCLUSION

The previous 2D ground deformation analyses were carried out for the design $(N_1)_{60cs}$ profiles corresponding to the 33rd percentile values as per the Task Force guidelines. The use of the 33rd percentile values has been the local practice for assessment of the potential liquefaction of granular soils and the resulting lateral ground displacements. Considering the recent studies based on the stochastic $(N_1)_{60cs}$ distributions, further analyses were carried out to assess the impact on the predicted lateral spreading considering the 50th percentile $(N_1)_{60cs}$ values.

The results of the analyses indicate that the differences in the predicted lateral displacements between the two design $(N_1)_{60}$ profiles at the on-land shaft locations are insignificant. However, the predicted lateral displacements were generally thirty percent lower at the riser shaft location for the 50th percentile $(N_1)_{60cs}$ values compared to that of the 33rd percentile $(N_1)_{60cs}$ values. It is noted that the predicted lateral displacements at the on-land shaft locations are generally smaller; hence, an appreciable difference as seen at the riser shaft location could not be realized with the increase in the penetration values at the on land shaft location. It is also noted that the relatively lower predicted lateral displacements at the riser shaft for the 50th percentile $(N_1)_{60cs}$ values may not actually result in a comparable reduction in the predicted lateral deflection of the shaft; this is because the higher $(N_1)_{60cs}$ values result in increased strength of the sand, which would likely increase the lateral load imposed on the shaft.

4.0 CLOSURE

We trust that the information presented in this interim Technical Memorandum is sufficient for your immediate requirements. Please do not hesitate to contact us if you have questions or require clarification of contents.

Yours truly,

GOLDER ASSOCIATES LTD.

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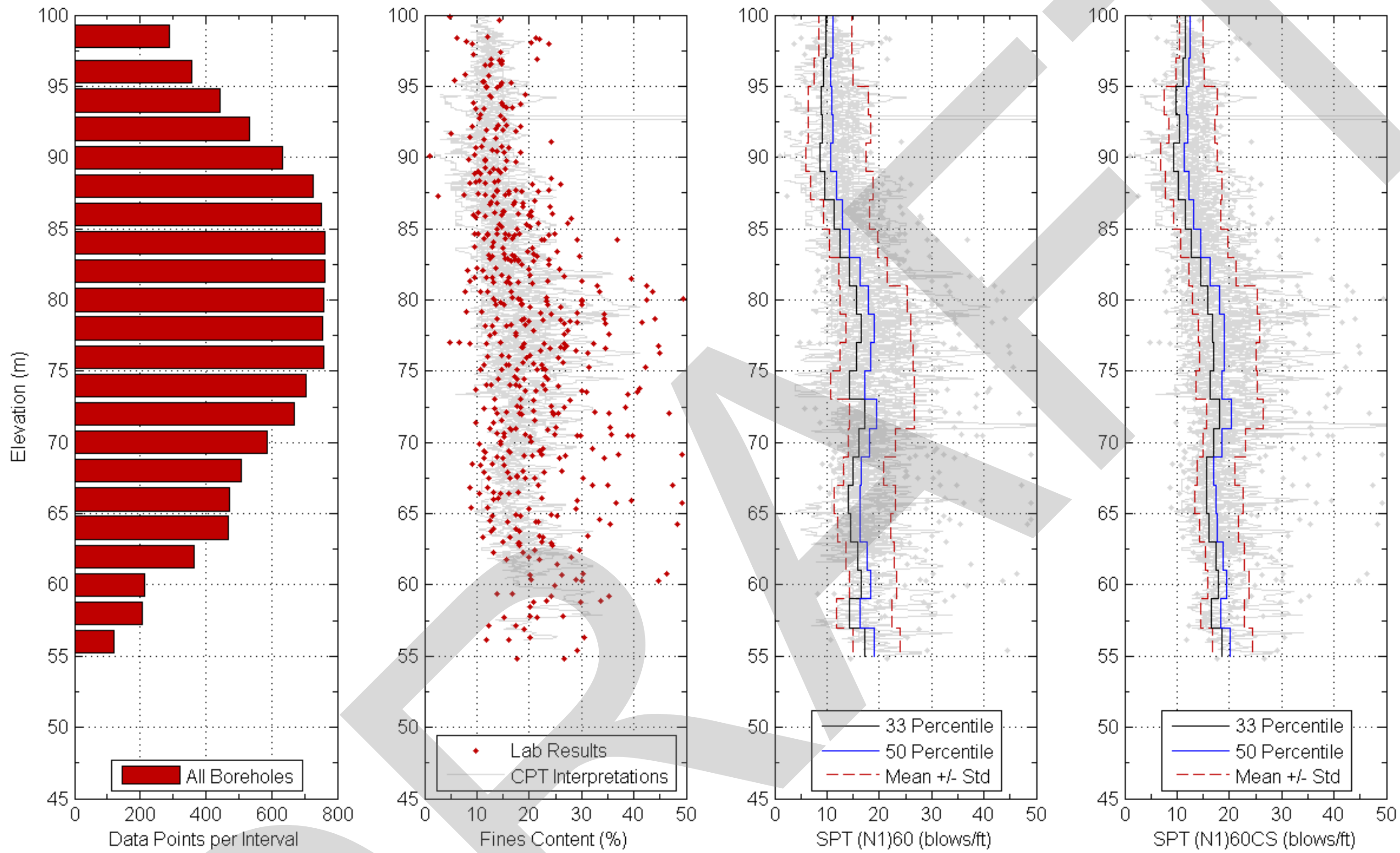

PROJECT
CDM SMITH CANADA ULC
AIWWTP TRANSIENT MITIGATION AND OUTFALL SYSTEM
DELTA, BC

TITLE
SPT $(N_1)_{60}$ Profiles

Golder Associates

PROJECT No. 1525010			PHASE No.	
DESIGN	M.S.K	30JAN17	SCALE NTS	REV.
CADD	--		**FIGURE 1-1**	
CHECK				
REVIEW				

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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