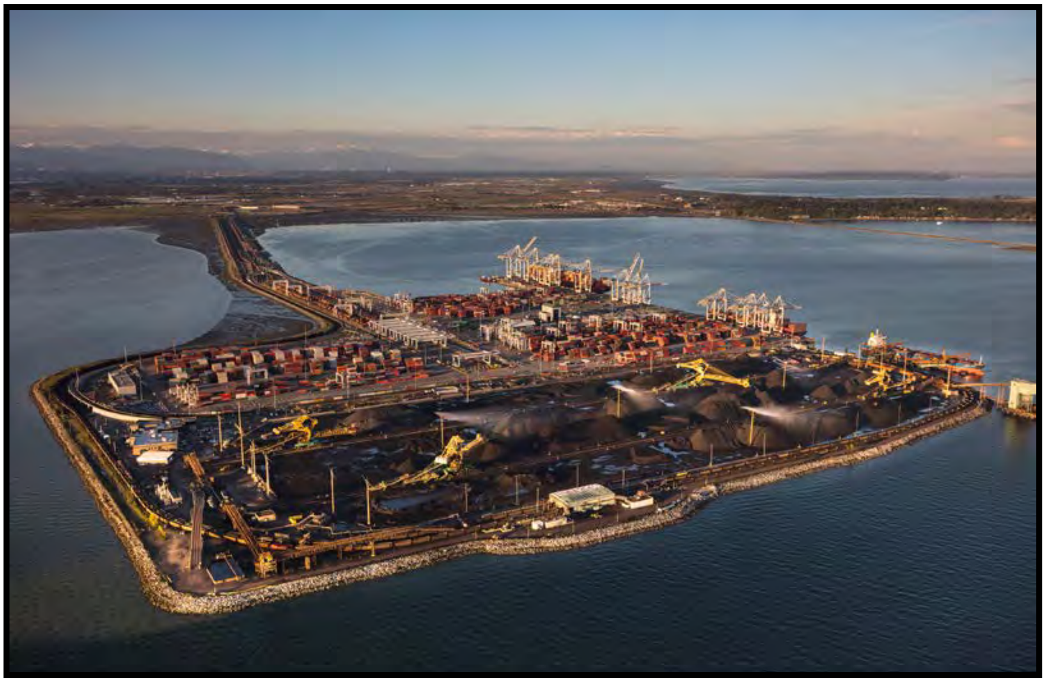
# GEOTECHNICAL ENGINEERING ASSESSMENT REPORT

# New Cargo Project
# Westshore Terminals
# 1 Roberts Bank, Delta, BC



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**Changes:** Updated Storage Building settlement and preload assessment section.
Added bridging scope geotechnical input for structural assessment

## Executive Summary

Braun Geotechnical Ltd. (Braun) and Naesgaard Amini Geotechnical Ltd. (NAGL) have been retained by Westshore Terminals to provide geotechnical engineering services for the proposed New Cargo project.

The existing site is reclaimed land connected to Delta, BC by an approximately 4km long causeway to the north. The land reclamation covers an area approximately 1300m north-south by 1200m east-west. The Westshore facility is situated on the southern portion of the site and has overall dimensions of about 1100m by 500m in the east-west and north-south directions, respectively. The Delta Port Terminals facility is located on the northern portion of the reclaimed area. The site is relatively flat at approximate El. 8m and generally level with the Delta Port Terminal site. The east, south and west sides slope down to the ocean floor. The east side is adjacent to an area dredged for Berth 2 and extends down about 30m. There is a relatively steep submarine slope located approximately 400m off the south side of the site.

The site is currently occupied by an operating coal handling facility which includes existing train tracks around the site perimeter, access roads, a dumper pit located on the south side of the site, conveyors, transfer towers, and a ship loader within the Berth 2 area (near/within the crest of the east slope). The coal is stored in piles with a maximum height of 26m (85ft) but are typically in about 15m (50ft) high within the stockyard area.

The main components of the project include:

- An 11m deep dumper pit and tunnel on the south side of the site.
- A 660m long by 70m wide storage building on the north side of the site which is proposed to be constructed in two phases.
- Approximately 1800m of conveyors supported on bents up to 40m high. The conveyors extend from the dumper pit on the south side of the site, and traverse the west side of the site to the storage building, and east of the storage building to Berth 2.
- Transfer towers up to approximately 43m high located where the conveyors change direction.

The majority of the conveyors, transfer towers and the dumper pit are located in relatively close proximity to the site perimeter slope where they are more likely to be impacted by seismic displacements.

The objective of this assessment is to provide geotechnical design recommendations to support the current phase of structural design of the proposed new facility. Review of the performance of existing structures is beyond the scope of this work. Seismic assessment of the Berth 2 area is in progress at the time of this report and will be provided within a separate report (Doc. 20-8543-REPORT-003).

Braun/NAGL carried out a geotechnical exploration program which included Cone Penetration Test (CPTs) and Seismic CPTs (SCPTs), followed by Sonic drilling for soil sampling and laboratory testing. A total of 11 CPTs and 4 SCPTs to 30m to 60m depth and 7 Sonic test holes to depths of 12 to 30m were completed. Routine laboratory testing was carried out on the collected soil samples. Available subsurface information from previous projects within and near the site was used to supplement the collected data. The collected data and laboratory test results are presented in the Data Report (Doc#: 20-8543-REPORT-001-Rev 5) dated July 9, 2021.

The site is underlain by a near surface zone of granular fill approximately 8m thick which was placed on the seabed to raise grades to the current site elevation of approximately El. 8.0m (Chart Datum). The fill typically comprises silty sand to sand with some silt in compact to dense condition. The upper layer, approx. 1.5m, in the stockyard is sand mixed with coal. The fill is

underlain by generally compact sand to silty sand to a depth of 9m to 22m. The sand is underlain by loose to compact silty sand, sandy silt, and low plastic silt to the maximum depths of the recent exploration. Based on deep test hole exploration by Golder (2011), this soil deposit is expected to extend to a depth of 75 to 118m below ground surface. Deep test holes by Golder encountered 4m to 34m thick zones of silty clay to clayey silt below. Very dense till-like soils are expected below depths of approximately 104m to 115m in the vicinity of the site. Groundwater was encountered at an average depth of approximately 3m (El. 5.0m Chart Datum).

The loose to compact soils underlying the site comprising sand to low to non-plastic silts are susceptible to liquefaction and are predicted to undergo extensive liquefaction in the 2475 year return period (A2475) design earthquake. Liquefaction triggering assessment using a conventional CPT-based method indicates that the majority of the subsoils below the dense fill to depths in the range of 40m to 50m have the potential for liquefaction. Limit equilibrium slope stability analyses completed using post-seismic soil strengths indicate that the perimeter slopes will undergo flow slide failure with an influence zone that extends a horizontal distance of about 75m to greater than 100m inland from the slope crest, depending on location. Empirical methods estimate large lateral spreading displacements in the range of up to 5m at the locations of the proposed structures.

Fully coupled effective stress nonlinear dynamic analyses were carried out using the two-dimensional finite difference program FLAC Version 8 and advanced constitutive soil models PM4Sand and PM4Silt. FLAC models the propagation of seismic wave from “firm ground” up through the soil strata to the ground surface, and the associated buildup of excess pore water pressures and soil deformation. The results of the FLAC numerical analyses indicate that the soils underlying the site have the potential for liquefaction to about 30m depth in the A2475 design earthquake, which is less than that predicted by the simplified method. Analyses confirmed the potential for flow slide failure near the site perimeter and large displacements during or after a seismic event. Post-earthquake reconsolidation settlements are estimated to be in the range of 0.5m using simplified methods.

Seismically-induced displacements at the locations of the proposed structures were estimated using FLAC in the North-South and East-West directions. The FLAC models incorporated the existing coal stockpiles. The North-South analysis incorporated the proposed dumper pit, storage building, potash stockpile, tie rods connecting the north and south storage building footings, and the submarine slope south of the site.

North-South post-seismic displacements at the locations of proposed structures typically range from 0.6m northward on the north portion of the site due to the influence of local stockpiles to 4.2m southward near the south slope of the site. Estimated East-West displacement at the location of the structures range from 3.5m to the west near the west slope to 4.8m to the east near the east slope. Additional seismic assessment of the Berth 2 area will be described in Doc.# 20-8543-REPORT-003.

The estimated seismically induced displacements are too large for conventionally designed structures to meet the design performance criteria for life safety and non-collapse during the A2475 design earthquake. Mitigative measures are required to either reduce the ground displacements or increase the tolerance of the structures to accommodate the displacements. Conventional ground improvement is not considered feasible for all structures due to potential negative effects on existing structures, soils not conducive to densification at some locations and depths, and the relatively large area spanned by the structures. Accordingly, the current design concept is to use conventional grade supported foundations with structures detailed to accommodate the estimated total and differential displacements. Ground improvements will be used locally where feasible to improve bearing resistance and provide more uniform subgrade conditions. Where ground improvement is not feasible, foundations will be designed for reduced bearing resistance.

Previous use of the area within which a majority of the storage building is proposed included stockpiling of coal with a maximum height of approximately 26m and an average height of approximately 15m. Silt, silty zones, and deep clay underlying the site are compressible. Storage of coal over portions of the site is expected to have partially "preloaded" the footprint of the coal stockpiles, but the actual loading history is variable across the site. The reduction in settlement due to the preloading effect of the coal is expected to be greater on the south side of the storage building and potash stockpile area compared to the north, and highly variable in the east-west direction.

New loads from the proposed storage building including the 21m high potash stockpile with a unit weight of 11.8kN/m$^3$ will be more than the existing average 15m high coal stockpiles with a unit weight of 8.6kN/m$^3$. The additional loading will result in additional total and differential settlements.

The settlement model for the storage building was calibrated based on available historical settlement information obtained from the Delta Port site to the north. Without preloading, estimated future settlements are approximately 550mm, 775mm, and 350mm below the north footing, potash stockpile, and south footing, respectively. Actual settlements and time for settlements to occur cannot be predicted accurately, and may vary as discussed within this report. It is recommended that a range of 50% to 200% of the estimated settlements be considered in the design. The estimated settlements and potential differential settlements are larger than the tolerance of conventional structures. A preload is recommended to reduce settlements and to address many of the uncertainties including differential settlements, amount and rate of settlements, varying site loading history, variable soil conditions, and potential buried structures and rock berm. Preloading will also reduce requirements / frequency of maintenance of the building.

Settlements of transfer towers and conveyor bents are expected to be in the range of 150mm to 300mm and 100mm to 150mm, respectively, assuming the recommended allowable pressure of 125kPa is present below the entire footing.

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# 1. INTRODUCTION

As requested, Braun Geotechnical Ltd. (Braun) and Naesgaard-Amini Geotechnical Ltd. (NAGL) have carried out a geotechnical assessment for the preliminary stage of the above-referenced project. The geotechnical work has been performed in general accordance with the original scope of work outlined in our proposal dated April 2, 2020 Rev. 2 (our reference No. P20-6784), and subsequent additions to the work scope as the preliminary design phase progressed. The scope of work included geotechnical exploration programs and provision of geotechnical recommendations for the project. No consideration has been given to environmental aspects.

The results of the geotechnical exploration and laboratory testing are presented in the Data Report (Doc#: 20-8543-REPORT-001-Rev5) dated July 9, 2021. Historical test hole information collected by others was also considered for the preparation of this assessment report. A list of historical information along with relevant test hole logs are provided in the above noted Data Report.

The purpose of this report is to provide geotechnical recommendations to support preliminary structural design of the proposed dumper pit, tunnel, storage building, transfer towers and conveyor support towers. The recommendations provided in this report are based on the available information at the time of this report.

The primary geotechnical issues for the current phase of design are as follows:

1. Seismic design including assessment of liquefaction and consequence of liquefaction, i.e. flow slide failure, lateral spreading displacements, and post-earthquake reconsolidation settlement for the A2475 design earthquake.
2. Selection of the foundation systems to support the structures under service and seismic loading conditions, including requirements for ground improvement.
3. Recommendation for bearing capacity and evaluation of foundation settlements under service conditions.
4. Recommendations for horizontal earth pressures for the walls and buried structures.
5. Estimation of settlements and preliminary preloading requirements for the storage building.

Additional geotechnical assessment will be required during the detailed design phase. This work is expected to include, but not limited to, review of potential storage building vendor requirements, evaluation of the settlement of the storage building and interaction with adjacent transfer towers, detailed preload design, review of dumper pit excavation and shoring details, seismic assessment of the dumper pit, seismic soil-structure interaction analyses of a typical transfer tower, seismic assessment for various return period earthquakes if required, additional assessment of spout platform foundation piles, and ground improvement design.

Some information provided within this report may be updated based on seismic assessment of Berth 2 which is currently in progress. The results of the seismic assessment of the Berth 2 area will be provided within a separate report (Doc. 20-8543-REPORT-003).

The National Building Code of Canada, NBCC 2015 is the primary code for the design of the proposed inland structures. Where NBCC does not provide specific guidelines, the Canadian bridge design code, CSA-S6-14 and the 2016 BC Supplement to CSA-S6-14 are used for further guidance.

## 2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The existing site is reclaimed land and is situated approximately 4.7km southwest off the coast of Delta, BC as shown in attached Figure 2-1. The site is located immediately southwest of the Delta Port Terminal facility, and approximately 2.5km northwest of the Tsawwassen ferry terminal. The site is accessed by an approximately 4km long causeway northeast of the site.

For the remainder of this report, and to maintain consistency with site orientation being used by other consultants, project north will be considered to be the general direction of the causeway relative to the site and all elevations are in Chart Datum.

The Westshore site is about 1100m by 500m in east-west and north-south directions, respectively. The site is relatively flat, at approximate El. 8m and generally level with the Delta Port Terminal site to the north. The east, south and west sides slope down to the ocean floor. The slope heights vary between 6 to 7m to the west, 8 to 11m to the south and approximately 30m to the east. The east side is adjacent to a dredged area extending down to about El. -21m. The average gradient of the east, west and south slopes is approximately 2.7H:1V (Horizontal to Vertical).

The site is currently occupied by an operating coal handling facility. The existing coal facility infrastructure includes train tracks around the site perimeter, access roads, a dumper pit located on the south side of the site, conveyors, transfer towers, and a ship loader within the Berth 2 area (near/within the crest of the east slope). The coal is stored in stockpiles up to approximately 26m (85ft) high within the central portion of the site.

Based on the proposed layout drawings by CWA Engineers Inc. (CWA) dated August 31, 2020 (Figure 2-2), it is understood that the proposed new cargo facilities will include construction of a new railcar dumper and tunnel north of the existing dumpers, a potash storage building on Line D on the northwest corner of the site, and associated transfer towers, conveyors and conveyor support bents. The new facilities will be required to handle a new cargo at the site and will consist of approximately 1800m of additional conveyors stretching from the dumper pit west and then north to the northwest corner of the site, then along the northern border to Berth 2.

The initial proposed site layout included a new surge bin and conveyors extending to the Berth 2 area along the east shoreline. Initial efforts focused primarily on geotechnical engineering work to develop potential concepts for foundation design for the proposed structures located near the crest of slope in the Berth 2 area. The geotechnical assessment indicated the potential for large seismically induced displacements and flow slide failure in this area that required significant ground improvement works. As such, on June 26, 2020, it was determined by the project team that the construction of these near bank structures was not practical and alternative arrangements were developed to avoid the necessity to construct these structures.

## 3. AVAILABLE GEOTECHNICAL INFORMATION

The following relevant information by others was made available for the purpose of this study. Where available, relevant test hole information is included in the Braun/NAGL Data Report which is provided under separate cover (Document # 20-8534-2020-12-14).

- Memo of January 15, 2003 by Cook Pickering & Doyle Ltd. for Berth 2 Quadrant Loaders.
- Memo of April 24, 2003 by Cook Pickering & Doyle Ltd. for Berth 2 Quadrant Loaders.
- Roberts Bank Seismic Evaluation Report by Golder Associates dated May 18, 2011 (confidential report).
- Geotechnical Report by Levelton Consultants Ltd. dated April 18, 2013 for the Terminal Facilities Building.

- Geotechnical Report by Levelton Consultants Ltd. dated December 10, 2014 for Reservoir 5 and Pump House.
- Geotechnical Report by Thurber Engineering Ltd. dated October 3, 2016 for the Bailey Bridge Replacement at the southwest side of the site.
- Technical Memo by Reilly Engineering Associates Ltd. of December 18, 2017
- SCPT18-01 completed by Braun Geotechnical in 2018 northwest of Berth 2
- Geotechnical Report by Thurber Engineering Ltd. dated October 24, 2019 for the Berth 2 Dolphin Replacement Project.
- Geotechnical Report by Thurber Engineering Ltd. dated April 1, 2020 for the Berth 2 Dolphin Replacement Project.
- Offshore Test Hole information acquired by Thurber Engineering Ltd. in April 2020 for the Berth 2 Dolphin replacement project.
- Draft Independent Settlement Assessment by Klohn Crippen Berger of June 21, 2021 for the Storage Building.

## 4. GEOTECHNICAL EXPLORATION & LABORATORY TESTING

The geotechnical explorations included Cone Penetration Testing (CPT) and Seismic CPT (SCPT) completed in April 2020 and June 2021. The June 2021 exploration was carried out to collect additional information for soil characterization of the Berth 2 area. Soil sampling was carried out at 3 locations in June 2020 and 4 locations near Berth 2 in June 2021 using Sonic drilling equipment.

The CPTs were advanced to depths of 30 to 60 m below ground surface. Augers installed to a depth of 4.6m to anchor the drill rig for CPT testing were sampled and logged.

Laboratory testing was carried out for determination of soil grain size distribution, fines content, water content, and plasticity indices of select samples. The exploration location plan is presented in Figure 2-1.

Test hole and CPT logs by Braun/NAGL and others are provided in the Data Report.

## 5. SOIL AND GROUNDWATER CONDITIONS

The findings of the geotechnical investigation were generally consistent with the subsurface soil profile encountered by others. A comparison of the CPTs carried out by Braun/NAGL and others in terms of tip resistance variation up to 50m depth along the site alignment is presented in Appendix A.

The site investigation carried out by Braun/NAGL was advanced to a maximum depth of 60m. Below 60m depth (~El. -52m), the soil stratigraphy was characterized based on deep CPTs and boreholes carried out by Golder in 2011 and Thurber 2020 deep CPTs in the Berth 2 area. The Golder (2011) report presents boreholes and CPTs advanced to El. -105m within three land-based test locations and El. -167m within three offshore test locations. A summary table with the soil stratigraphy encountered by Golder (2011) is presented in Table 5-1.

Table 5-1: Approximate thickness of main soil stratigraphic units (modified from Golder 2011, Table 4-1).

| **Test hole** | **Granular soils [3]** | | | | **Fine-grained (cohesive) soils** | | | | **Till-like soils** | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Depth (m) | | Elevation [2] (m) | | Depth (m) | | Elevation [2] (m) | | Depth (m) | Elevation [2] (m) |
| | From | To | From | To | From | To | From | To | From | From |
| CPT10-01 /BH10-01 | 0 | 86 | -70 | -156 | 86 | 103 [1] | -156 | -173 [1] | N/A | N/A |
| CPT10-02 /BH10-02 | 0 | 75 | -46 | -121 | 75 | 89 | -121 | -135 | 89 | -135 |
| CPT10-03 | 0 | 118 | -16 | -134 | 118 | 143 [1] | -134 | -159 [1] | N/A | N/A |
| SCPT10-04 /SH10-04 | 0 | 96 | 6.5 | -89.5 | 96 | 109 | -89.5 | -102.5 | 109 | -102.5 |
| SCPT10-05 /SH10-05 | 0 | 77 | 6.5 | -70.5 | 77 | 104 | -70.5 | -97.5 | 104 | -97.5 |
| SCPT10-06 /SH10-06 | 0 | 75 | 3.5 | -71.5 | 75 | 109 | -71.5 | -105.5 | 109 | -105.5 |

[1]Exploration terminated before reaching the top of till-like soils stratum.
[2] Ground surface/seabed elevations were obtained from Golder 2011, Table 3-1.
[3]Although described as granular by Golder, subsoils in the noted depth intervals include fine-grained low-plastic to non-plastic silts as well as silt/sand mixtures and sands.

A generalized subsoil profile in order of increasing depth based on the soils encountered at the location of test holes and exploration information by others is provided below. Subsurface conditions at other site locations may vary.

*COAL & SAND/COAL MIX*

A layer of coal and sand fill mixed with coal was encountered to approximately 0.3 to 2.0m depth within all test holes. Additional test pits were advanced by R. F. Binnie and Associates (Binnie) for the storage building area (see Section 7).

Generally, coal was encountered to greater depths at the location of the proposed potash storage building. A 1.5m thick layer of coal was encountered near the east slope below the asphalt pavement at CPT20-11.

*SAND FILL*

Sand (FILL) with occasional wood debris and seashells was encountered below the coal to a depth of approximately 8m. The sand is brown-grey near surface, transitioning to grey with depth. The fill is generally dense becoming compact with depth. Near surface fill is generally looser on the northeast side of the site. Occasional seashells were encountered within the fill and may be attributed to the fill having been dredged from the Berth 2 area on the east side of the site.

Grain size distributions for select samples of the fill are presented in Figure 5-1. The fill is generally comprised of poorly graded medium to fine sand.

The fines content of the samples tested typically varies between 5% and 16%. Exceptions included SONIC20-02 at 2.3 m with fines content of 43%, and SONIC20-03 at 2.1 m with fines content of 34%.

The shear wave velocities measured within this layer vary between approximately 145m/s to 260m/s, and generally decrease with depth, as indicated on Figure 5-4.

*SAND*

Sand with occasional interbedded silt layers was encountered below the fill to a depth of approximately 9m to 22m. Based on the measured cone tip resistance, the sand was generally compact with some dense zones. Results of the grain size distribution tests carried out on select samples classified these soils as poorly graded medium sand with trace to some silt (Figure 5-2).

The interbedded layers comprise firm low plastic to non-plastic silt with some sand to sandy silt with thickness varying between 0.3 to 1.2m. The grain size distribution of a sample collected from a silt interlayer is also presented in Figure 5-2 (SONIC20-01, 9.4m depth).

The shear wave velocities measured within this layer vary between approximately 140m/s to 255m/s, and generally increase with depth.

*Silty SAND / Sandy SILT / SILT*

Loose to compact silty sand to firm sandy silt interbedded with thin silt layers and occasional layers with wood fragments was encountered below the sand to the maximum depths of exploration. Based on the deep CPTs and boreholes by Golder (2011) in the vicinity of the site, this soil deposit is expected to extend to approximate depths of 75 to 118m.

Grain size distribution test results on select samples are shown in Figure 5-3. The samples tested were classified as silt with some sand to silty sand, which demonstrates the variable and interbedded nature of this zone.

The plasticity index of this layer varied between PI= 0 to 20%.

Shear wave velocities measured within this layer vary between approximately 140m/s to 360m/s and generally increase with depth.

*SILTY CLAY / CLAYEY SILT*

Based on the Golder (2011) data, medium plastic silty clay to clayey silt (Deep Clay) was encountered below the silty sand/sandy silt layer. The thickness of this layer is expected to vary between 4m and 34m below the site.

The average plasticity index of the tested samples was approximately 26% with minimum and maximum values of 6% and 35%. Consolidation test results completed by others indicated these cohesive soils are normally to lightly consolidated and estimated the coefficient of compression (Cc) between 0.19 and 0.68 and the coefficient of recompression (Cr) between 0.05 and 0.12.

Shear wave velocities measured within this layer vary between approximately 250m/s to 500m/s and generally increase with depth.

*TILL-LIKE SOILS*

Very dense till-like soils are expected below depths of approximately 104m to 115m in the vicinity of the site based on information in the Golder (2011) report.

The shear wave velocity measured within this layer at FD95-S1 was approximately 450m/s.

*GROUNDWATER*

Groundwater interpreted from the CPTs and encountered during drilling was at an average depth of approximately 3 m. Groundwater levels are expected to fluctuate seasonally and with tidal variation. Due to site constraints, it was not possible to install piezometers.

# 6. SEISMIC ASSESSMENT

## *6.1. Seismic Performance Criteria*

The proposed inland structures should be designed to withstand the A2475 design earthquake (2% probability of exceedance in 50 years) with no collapse and no loss of life. The existing structures may have been designed for different seismic criteria and are outside of the current scope of work. Seismic performance criteria of the Berth 2 area will be discussed within a separate report (Doc. 20-8543-REPORT-003).

As indicated in Section 1, NBCC 2015 is the primary code for seismic design of the proposed structures. The focus of the seismic assessment at this stage has been the A2475 design earthquake. Limited assessment has been performed for the A475 design earthquake at this design stage.

## *6.2. Seismic Design Challenges*

The project site is located on reclaimed land constructed above young loose to compact liquefiable soils. The fill slopes down to the seabed on three sides of the site. The seismic design challenges at this site are primarily caused by extensive liquefaction that is predicted to occur during the A2475 design earthquake.

The main design challenges under the A2475 design earthquake are itemized below:

1. The perimeter slopes of the site are underlain by liquefiable soils and will likely become unstable in post-earthquake conditions (flow slide failure).
2. The proposed facility is a long linear structure. Portions of the facility run parallel to the south, west and east slopes and are located marginally outside the predicted zone of flow slide failure.
3. The structures are prone to large total and differential ground displacements induced by the design earthquake.
4. The proposed structures are relatively tall and sensitive to rotation of the foundations.
5. Lateral spreading displacements of more than 10m near the slope crests are predicted. The effects of lateral spreading decrease with distance from the crest resulting in differential displacements in the longitudinal direction of the structures. Displacements are expected to occur in both orthogonal directions.
6. Variable soil conditions and slope geometry cause differential displacements in the transverse direction of the structures.
7. The coal and potash stockpiles cause local lateral spreading type displacements that change the global pattern of lateral spreading of the site. This causes additional differential displacements. Vertical displacement of the potash stockpile in the storage building will result in forces pushing the north and south footings apart from each other.
8. Lateral spreading is expected to result in large displacements of shallow foundations and significant kinematic loading on piles.
9. Liquefaction significantly reduces the bearing resistance of shallow foundations and the axial capacity of piles.
10. Generally, there is potential for buoyancy of underground structures in liquefiable soils. This needs to be assessed for the dumper pit.

11. Mitigation of the consequences of liquefaction using conventional ground improvement techniques at the site may be impractical due to congestion of the site and impacts on existing facilities, inefficient due to presence of high fines content soils, and expensive due to the large footprint of the project.

This section addresses the items above and provides background information to be used as an aid to judgment and to formulate the seismic design recommendations provided in Section 7.

### *6.3. Seismicity of the Site*

The seismic response spectrum for firm ground (Site Class C with an average shear wave velocity of 450m/s) was obtained from the National Building Code 2015 (NBCC, 2015) seismic hazard calculator from the Natural Resources Canada (NRC) website as summarized in Table 6-1. The hazard deaggregation data for the site has been obtained from the NRC. The NRC hazard calculation sheet and the deaggregation data for the A2475 earthquake at the PGA and 1s period are presented in Appendix B.

Table 6-1: Design response spectra for outcropping firm ground (Site Class C)

| **Return Period (years)** | **2475** | **975** | **475** | **100** |
|---|---|---|---|---|
| **Probability of exceedance in 50 years** | **2%** | **5%** | **10%** | **40%** |
| PGA (g) | 0.423 | 0.305 | 0.224 | 0.101 |
| Sa (0.05) | 0.515 | 0.367 | 0.27 | 0.123 |
| Sa (0.1) | 0.785 | 0.561 | 0.413 | 0.188 |
| Sa (0.2) | 0.977 | 0.703 | 0.516 | 0.236 |
| Sa (0.3) | 0.99 | 0.715 | 0.525 | 0.236 |
| Sa (0.5) | 0.876 | 0.629 | 0.457 | 0.195 |
| Sa (1.0) | 0.488 | 0.339 | 0.24 | 0.095 |
| Sa (2.0) | 0.292 | 0.197 | 0.135 | 0.05 |
| Sa (5.0) | 0.09 | 0.053 | 0.031 | 0.011 |
| Sa (10.0) | 0.032 | 0.018 | 0.011 | 0.004 |

Westshore-Response spectrum.xlsx

### *6.4. Seismic Classification of the Site*

The Site Class interpreted based on liquefaction potential of the underlying soils and average shear wave velocity of the upper 30m of the soil stratigraphy ($V_{s30}$) according to NBCC (2015) is summarized in Table 6-2.

Table 6-2: Seismic Site Classification

| Location | SCPT / Test Hole | $V_{S30}$ (m/s) | Design Return Period (yr) | |
|---|---|---|---|---|
| | | | 475 & 2475 | 100 |
| Proposed Dumper Pit | SCPT20-01 | 199 | F | D |
| Existing Dumper Pit | GSC-FD95-S1 | 171 | F | E |
| South West Bridge | SCPT16-05 | 184 | F | D |
| NW of the Storage Building | SCPT20-06 | 197 | F | D |
| SE of the Storage Building | SCPT20-09 | 175 | F | E |
| Near East Slope | SCPT21-02 | 197 | F | D |
| Near East Slope | SCPT18-01 | 202 | F | D |

Vs_inprogress.xlsx

### *6.5. Design Ground Motions*

Golder (2016) developed design ground motions for the Massey Tunnel Replacement Bridge Project based on the NBC2015 5th generation seismic hazard model. The motions were spectrally matched to scenario response spectra. Five sets of ground motions were developed for each earthquake source, crustal, inslab and interface (subduction). Each set included two orthogonal horizontal motions and one vertical motion. The ground motions were developed for three levels of earthquake, A2475, A975 and A475 for outcropping firm ground with a shear wave velocity of 450m/s. The results were provided in a Golder Technical Memorandum dated March 7, 2016 and includes details of the procedures used to develop the design ground motions. The Technical Memorandum is publicly available online (*https://engage.gov.bc.ca/app/uploads/sites/52/2017/04/Technical-Background-Geotech-2.pdf*).

The A2475 and A475 horizontal motions from the above project were scaled by a factor of 1.09, the ratio of the PGA at the Westshore site to the PGA at the Massey Tunnel, and used for the Westshore project. This provides 15 pairs of horizontal motions per earthquake level. All 15 pairs (a total of 30) of horizontal motions were used for site response analysis. One horizontal motion from each pair totaling 15 motions were used in FLAC analyses. Ground motion time histories for the A2475 earthquake are included in Appendix C. The response spectra of the design ground motions after scaling by a factor of 1.09 are shown on Figure C-1 in Appendix C. The crustal and inslab ground motions have been matched to a scenario design response spectrum which is consistent with the uniform hazard response spectrum (UHRS) in the period range of about 3 seconds or shorter and is lower than the UHRS for periods longer than 3 seconds. The interface ground motions were matched to a scenario design response spectrum, which is consistent with the UHRS in the periods longer than about 3 seconds and is lower than the UHRS for periods shorter than 3 seconds. Refer to the Golder 2016 report for more details.

### *6.6. Site Specific Response Analysis*

The objectives of the site specific response analysis are to provide design response spectra for use in structural design and to obtain the cyclic stress ratio profiles for use in the liquefaction triggering assessment for A2475 and A475 earthquakes.

### *6.6.1. Soil Stratigraphy*

The generalized soil stratigraphy within the site used for site response analysis is presented in Table 6-3. The silts encountered at shallow depths are assumed to have a low plasticity index (PI ~ 10%) and the deep silts and clays are assumed to have PI=30%.

The shear wave velocity (Vs) profile for the upper 50m is based on the estimated average of the available Vs data collected at the site in 2020. Below 50m depth, subsurface information from Golder (2011) test hole SCPT10-4R has been used. Firm ground (Vs=450 m/s) is assumed to be at an approximate depth of 115m.

Table 6-3: Assumed generalized soil stratigraphy in site response analysis

| Depth (m) | | Soil description | PI (%) |
|---|---|---|---|
| 0 | 8 | sand | - |
| 8 | 13 | silty sand | - |
| 13 | 16 | sand | - |
| 16 | 21 | silt | 10 |
| 21 | 39 | silt and sand | - |
| 39 | 49 | sand | - |
| 49 | 51 | silt | 10 |
| 51 | 70 | sand | - |
| 70 | 78 | silt and sand | - |
| 78 | 86 | silt | 30 |
| 86 | 91 | sand | - |
| 91 | 94 | silt | 30 |
| 94 | 115 | clay | 30 |

### *6.6.2. Methodology*

The assumed soil column has been modeled using the software package Deepsoil Version 7.0 developed by Hashash et al. (2020) at the University of Illinois. The software performs one dimensional non-linear as well as equivalent-linear elastic total stress analyses.

The modulus reduction and damping curves (MRDC) by Seed and Idriss (1970) (upper bound of $G/G_{max}$ and lower bound of damping curves) and Vucetic and Dobry (1992) have been used for the sand and silt/clay zones, respectively.

The stress dependent MRDC developed by Darandeli (2001), lower and upper range Vs profiles (Figure 6-1), non-linear and linear analysis methods have been considered as part of sensitivity analyses and development of the design response spectra.

### *6.6.3. Design Response Spectra*

Ground surface response spectra for the A2475 and A475 events have been estimated from the average of 10 crustal, 10 inslab and 10 interface motions (Figures 6-2 and 6-3, respectively) using the equivalent linear method. The non-linear analyses resulted in lower responses and are not shown on the figures.

The results indicate that there is de-amplification for periods shorter than 1s and amplification for periods longer than 1s. It is recommended that the design site response spectrum not be less than 80% of the relevant code-based spectrum (similar to that indicated in the 2016 BC Supplement to CSA-S6-14). The recommended design response spectra for the A2475 and A475 events are the envelopes of the calculated average responses and 80% of the average Site Class D and E code-based spectra. As indicated in Table 6-2, the site varies between Site Class D and E based on $V_{S30}$.

The design spectrum for the A100 earthquake was determined based on the average of Site Class D and E code-based spectra (Figure 6-4).

## 6.7. Liquefaction Assessment

Liquefaction assessment typically includes the review of the following items:

1. Are the subsoils susceptible to liquefaction?
2. Will liquefaction be triggered?
3. Does liquefaction cause a bearing failure (e.g. footings, piles) or flow slide failure of slopes?
4. If failure is not likely or the failure does not extend to the location of the structures, are the liquefaction induced displacements tolerable?
5. If the displacements are not tolerable, use either ground improvement to mitigate the consequences of liquefaction, or use structural solutions to improve seismic performance.

Brief answers to the above for the subject site are as follows:

1. Yes, the site is underlain by soils susceptible to liquefaction.
2. Yes, the A2475 and A475 design earthquakes trigger liquefaction.
3. Yes, flow slide failure of the site perimeter slopes is likely. Potential bearing failure of shallow foundations is discussed in Section 7.
4. The estimated ground displacements beyond the flow slide failure zone are large and can not be accommodated by typical structures.
5. Ground improvement and structural solutions are discussed in Section 7.

The following subsections present the site liquefaction assessment in more details.

### 6.7.1. Liquefaction Susceptibility

Saturated cohesionless soils such as gravel, sand and low to non-plastic silt exhibit sand-like behavior and are susceptible to liquefaction and/or cyclic mobility. The classical definition of earthquake induced liquefaction is that cyclic loading from earthquake shaking generates high excess pore pressures such that the effective stress approaches zero resulting in significant loss of strength and stiffness of the soils.

The soil units encountered from ground surface to the maximum depths of the recent exploration (60m) generally comprise sand, silty sand, sandy silt and silts. Two Atterberg tests completed on silt samples indicate a plasticity index, PI, of about 10% indicating relatively low plasticity. Visual examination of the remaining soil samples indicate that the soils were primarily low to non-plastic. A summary of available Atterberg limits plotted over the Bray and Sancio (2006) criteria (Figure 6-5) indicates that the silts are susceptible to liquefaction. The Idriss and Boulanger criterion for susceptibility of fine-grained soils to liquefaction is PI < 7%, which is expected to include the majority of the silts at the site.

Based on the results of the geotechnical investigation by Golder (2011), the deep silty clay to clay layer varies widely, from being susceptible to not susceptible to liquefaction. This layer is more than 80 m deep and is not considered in the liquefaction assessment.

Liquefaction assessment was carried out using the conventional semi-empirical method as presented below, and also as part of the non-linear effective stress dynamic numerical analysis (FLAC).

### *6.7.2. Conventional liquefaction triggering assessment*

Conventional liquefaction triggering assessment was carried out according to the Boulanger & Idriss (2014) CPT-based method using the program CLIQ version 3 (Geologismik, 2006) for three levels of earthquake, A2475, A475 and A100. With this method the Cyclic Stress Ratio (CSR) profile is compared to the Cyclic Resistance Ratio (CRR) profile. Liquefaction triggering is predicted where the factor of safety against liquefaction, as defined below, is less than 1.0.

$$FS_{liq} = \frac{CRR1}{CSR} \cdot MSF \cdot K_\sigma \cdot K_\alpha$$

Where:

$FS_{Liq}$ is the factor of safety against liquefaction triggering

$CSR = 0.65 \cdot \frac{\tau_{max}}{\sigma'_{vo}}$ obtained from site response analysis and shown on Figure 6-6.

$CRR_1$ is the normalized cyclic resistance ratio, i.e. at 100kPa vertical effective stress and for an earthquake moment magnitude of 7.5. It depends on the normalized penetration resistance of the soil after correction for fines content;

MSF is the magnitude scaling factor;

$K_\sigma$ is the overburden correction factor;

$K_\alpha$ is the static shear stress bias (slope) correction factor. This factor was assumed to be 1;

$\tau_{max}$ is the maximum shear stress obtained from the site response analysis;

$\sigma'_{vo}$ is the vertical effective stress before earthquake shaking.

Two combinations of the CSR profile and earthquake magnitude were considered as follows:

1. The envelope of the average CSR profiles from the crustal and inslab ground motions. A moment magnitude M=7.1 was assumed for this CSR profile.
2. The average CSR from the interface ground motions. A moment magnitude M=9 was assumed for this CSR profile.

The above-mentioned design magnitudes were obtained using the deaggregation data at the PGA and 1s period in general accordance with the Finn and Wightman (2007) method. The magnitude scaling factor (Youd et al. 2001 and Idriss 1999) for each bin was weighted based on its contribution and summed up from which a representative magnitude was calculated.

Other assumptions in the liquefaction triggering assessment are as follows:

- Water table depth = 3m.
- Fines content has been estimated from the CPT data using Boulanger and Idriss (2014) correlation assuming a fitting parameter $C_{FC}$ = - 0.1. This parameter was selected based on the comparison of laboratory fines content data with the CPT correlation as shown on Figure 6-7.
- Soil behavior index, Ic < 2.7 has been assumed as the criterion for sand-like behavior.
- The effect of the material stockpiles and the weight of structures have not been considered.

Figure 6-8 shows an example of the liquefaction triggering assessment carried out for SCPT20-06 for the A2475 crustal/inslab design earthquake. The results indicate that the 7m thick upper crust is dense and not liquefiable. The majority of the soil profile in the depth range of 7m to 50m, where the CRR is smaller than the CSR, is predicted to liquefy. Appendix D presents the results of liquefaction triggering assessment for the Braun/NAGL 2018 and 2020 CPTs for the A2475 and A475 design earthquakes. The extent of liquefaction varies from a depth of 3m to approximately 50m for the A2475 and 6m to 26m for the A475 design earthquakes.

The A475 earthquake CSR profile was scaled proportional to the PGA ratio of A100 to A475 to approximately estimate the CSR for the A100 earthquake. Liquefaction triggering analysis using this CSR indicates that the site is not prone to liquefaction under the A100 earthquake.

Appendix D presents the results of the liquefaction triggering assessment for the A2475 and A475 earthquakes for the 2018 and 2020 CPTs by Braun / NAGL.

### *6.7.3. Stability of Slopes in Post-Earthquake Condition*

The stability of site edge slopes and the potash stockpile in post-earthquake conditions has been evaluated for the A2475 earthquake. The two-dimensional limit equilibrium software Slide Version 8 developed by Rocscience was used for this analysis. The lower bound residual shear strength recommended by Idriss and Boulanger (2008) has been assigned to the liquefied soils. The analyses indicate that flow slide failure is likely for the three perimeter slopes.

Figures 6-9 to 6-12 present the results of slope stability analyses for static and post-earthquake conditions.

### *6.7.4. Earthquake-Induced Lateral Displacements from Empirical Methods*

The Zhang et. al (2004) empirical method has been used to estimate the order of magnitude of lateral spreading displacements. This method is based on correlations from past case histories of lateral spreading after earthquakes. Calculations were carried out for the 2018 and 2020 CPTs by Braun / NAGL using the program CLIQ V.3. In order to be consistent with the developed method, the CPT-based method according to Youd et al. (2001) has been used for the liquefaction assessment.

The horizontal displacements towards the south for the A2475 earthquake were estimated at four distances from the south slope crest for comparison with the results obtained from numerical analysis and also for assessment of the scatter of results due to soil variability. The slope height has been assumed to be 10m. The displacements below a depth of twice the slope height have been ignored.

As a typical example, Figure 6-13a presents the calculated horizontal displacements at 90m distance from the crest for different CPTs. Displacements vary in the range of 1.5 to 3.7m. The currently available empirical methods including Zhang have a margin of error in the range of 0.5 to 2 times the mean estimated value. Figure 6-13b presents the trend of the estimated horizontal displacements with distance from the crest of the slope.

### *6.7.5. Post-liquefaction Reconsolidation Settlement*

Post-liquefaction reconsolidation settlement occurs as the seismically induced excess pore pressures dissipate with time after ground shaking. This may take minutes to days for the Westshore site.

The post-liquefaction reconsolidation settlements for the A2475 earthquake have been estimated using the Boulanger and Idriss (2014) method in the program CLIQ. The settlement was estimated based on liquefaction triggering assessment using the mean CSR profile (crustal/inslab) obtained from FLAC numerical analysis (see FLAC profiles shown with dashed lines on Figure 6-6) and using soil conditions local to each structure.

Post-liquefaction reconsolidation settlements below a depth of 30m are expected to have negligible effect on the proposed structures and hence have not been considered. Reconsolidation settlement has been assumed to be negligible in densified soils. Assumed densification depths are 18m for the storage building, 18m below outbound (east of the storage building) transfer towers (except TT#77), and 10m below the outbound conveyor bents and take up structure. As the dumper will extend approximately 11m below surface, consolidation settlements to 11m depth have been neglected. The proposed extent of densification is discussed in Section 7.

Table 6-4 provides a summary of the estimated post-liquefaction reconsolidation settlements and densification depths considered for each structure.

Table 6-4: Estimated post-liquefaction reconsolidation settlements for A2475 earthquake

| Location | Test Hole | Explored Depth (m) | Densification depth (m) | Post-Liquefaction reconsolidation settlement (m) |
|---|---|---|---|---|
| Dumper | CPT20-01 | 50 | - | 0.3 |
| Drive | CPT20-02 | 30 | - | 0.5 |
| P40-B1 | CPT20-03 | 30 | - | 0.3 |
| P40-B2 | CPT20-03 | 30 | - | 0.3 |
| TT #P42 | CPT16-05 | 30 | - | 0.3 |
| P45-B1 | CPT16-05 | 30 | - | 0.3 |
| P45-B2 | CPT16-05 | 30 | - | 0.3 |
| P45-B3 | CPT20-04 | 30 | - | 0.5 |
| P45-B4 | CPT20-04 | 30 | - | 0.5 |
| TT P#47 | CPT20-04 | 30 | - | 0.5 |
| P50-B1 | CPT20-05 | 30 | 10 | 0.2 |
| TT P#52 | CPT20-05 | 30 | 18 | 0.1 |
| TT P#57 | CPT20-05 | 30 | 18 | 0.1 |
| Building Bayline 1 | CPT20-06 | 50 | 18 | 0.2 |
| Building End Phase 1 | CPT20-08 | 30 | 18 | 0.1 |
| Building End Phase 2 | CPT20-10 | 50 | 18 | 0.1 |
| Take up | CPT20-10 | 50 | 10 | 0.3 |
| P65-B1 | CPT20-10 | 50 | 10 | 0.3 |
| P65-B2 | CPT20-11 | 50 | 10 | 0.2 |
| TT P#67 | CPT20-11 | 50 | 18 | 0.1 |
| P70-B1 | CPT18-01 | 48 | 10 | 0.4 |
| P70-B2 | CPT18-01 | 48 | 10 | 0.4 |
| TT#77 | CPT20-13 | 60 | - | 0.3 |
| Sampling Tower | CPT20-13 | 60 | - | 0.3 |

displacement calculation 2020-11-13.xls

### *6.8. Dynamic Numerical Analysis*

Two-dimensional nonlinear effective stress dynamic analyses have been carried out using the finite difference program FLAC version 8.0 (Itasca, 2015) for two sections, an East-West (E-W) section and a North-South (N-S) section.

The primary objectives of numerical analyses were as follows:

1. To estimate the extent of liquefaction.
2. To get insight into the patterns of ground movement and to estimate the magnitude of ground displacements.
3. To evaluate the ground stability in post-earthquake conditions (flow slide failure check).

Due to the project schedule and the requirement for an early estimate of the order of magnitude of displacements in the Berth 2 area, the E-W FLAC Section was analyzed first with the free field conditions and simplified stratigraphy.

Subsequently, the N-S FLAC section was analyzed in more detail with respect to the subsoil stratigraphy and the effect of material stockpiles (coal and potash). The proposed dumper pit structure, storage building footings and potash stockpile retaining walls were added to the model in some permutations of the N-S FLAC analyses. The coal stockpile was subsequently added to the E-W FLAC section. The objectives of these additional details in the model were to evaluate the following specific items:

1. The effect of local lateral spreading caused by the potash stockpile in the storage building i.e. the differential seismically induced displacements between the north and south footings.
2. The response of the tie rods connecting the north and south footings.
3. The stability of the potash stockpile and footings in post-earthquake conditions.
4. Seismically induced earth pressures on the south and north walls of the dumper pit.
5. The potential for buoyancy of the proposed dumper pit as a result of liquefaction.

The following sections provide a brief overview of the methodology of analysis, assumptions and a summary of the main results to address the seismic related issues listed above. More details can be found in Appendix E.

### *6.8.1. Methodology and Assumptions*

Figures 6-14 and 6-15 show the locations of the two FLAC sections and the CPTs/test holes used to develop generalized geotechnical models.

The non-linear effective stress constitutive model PM4Sand (Boulanger & Ziotopoulou, 2017) has been used for soils with sand-like behavior and PM4Silt (Boulanger & Ziotopoulou 2018) has been used for fine-grained soils with clay-like behavior. The generic calibration factors of the two soil models were used with small adjustments for PM4Sand. Analyses were performed in ground water mode and flow and pore pressure redistribution were allowed. The horizontal design ground motions have been applied to the base of the model as shear stress time histories (compliant base) and analyses were solved to the end of earthquake. Subsequently, a post-earthquake flow slide check was carried out by assigning the Idriss and Boulanger (2008) lower bound residual strength to the liquefied soils where the excess pore pressures at the end of shaking were maintained in the non-liquefied soils.

The elevation profile of the seabed and the firm ground (till) were obtained from available bathymetry data and the Golder (2011) report. The soil conditions for the upper 60m depth were interpreted primarily based on the 2018 and 2020 site investigation data by Braun/NAGL. For depths below 60m, Thurber's 2020 deep marine CPT data was used on the east side and Golder's deep test hole/CPTs and The Geological Survey of Canada Deep SCPT were used for the south.

The E-W FLAC section is approximately 3km wide and 120m deep and extends beyond the reclaimed land in both east and west directions to capture the effect of the natural slope on the west side and the deep dredge pocket on the east side (Figure 6-16). The N-S FLAC section is approximately 4.6km wide and 120m deep. It includes the reclaimed land and extends to the south to capture the potential effect of the submarine slope.

The soil strata considered for the FLAC analyses from top to bottom are:

- 5m to 8m of dense sand fill
- 85m to 95m of sand or sand/silt interbeds with variable layering and mixtures of sand/silt
- 10m to 20m of silty clay
- Till (firm ground) at a depth in the range of 105m to 115m below the ground surface

There is considerable variability in the stratigraphy in the sand or sand/silt interbeds soil units across the site. A set of parametric analyses with four scenarios were carried out to evaluate the effects of the inclusion of silt layers or their permeabilities in the N-S FLAC section (Table 6-5). The analyses showed that all 4 cases resulted in comparable displacements with the all-sand case and the all-sand with silt permeability case (WS-N-S-14 & 15) being slightly greater. Based on the above, the simplifying assumption of all-sand (WS-N-S-15) was used for the design sections. Silts generally behave better than loose to compact sands under cyclic loading and post-earthquake conditions. Therefore, the exclusion of silty layers should be a conservative assumption. However, low permeability silt layers act as flow barriers and may cause void redistribution and water filming which may in turn result in an increased potential for flow slide failure. Proper modelling of void redistribution and water filming in the numerical model is difficult. Therefore, it is common to check the post-earthquake stability using a simplified procedure, where lower bound residual shear strength values are assigned to the liquefied sand layers.

An earlier set of parametric analyses for the E-W FLAC model showed that the all-sand assumption with inclusion of one low vertical permeability layer (FLAC case WS-23) to simulate a silt layer near the toe of the east slope resulted in larger displacements and more extensive flow slide failure for the east slope area. This was important to the earlier design revisions, which included some proposed structures near the crest of the east slope in the Berth 2 area.

Figures 6-16 and 6-17 show the generalized soil units in the E-W and N-S sections, respectively. It has been assumed that the phreatic surface profile was at El. 5m sloping down to sea level at El. 2.5m. The actual shape of the phreatic surface near the slope is not known. Two scenarios were assumed for the E-W Section; the phreatic surface starts to slope down to sea elevation at a horizontal distance of 45m or 250m back from the slope crest (see Appendix E for illustration).

Table 6-5: Scenarios considered for inclusion of silts in the N-S Section parametric analysis

| FLAC Filename | Silt Layer | Silt Permeability | Silt Calibration |
|---|---|---|---|
| WS-N-S-12 | Yes | Yes | Note 1 |
| WS-N-S-13 | Yes | Yes | Note 2 |
| WS-N-S-14 | No | Yes | N/A |
| WS-N-S-15 | No | No | N/A |

Westshore- FLAC cases.xlsx

Note 1: Generic PM4Silt calibration
Note 2: Calibration based on Fraser River Silt (Boulanger & Wijewickreme, 2019)

The profiles of normalized cone tip resistance corrected for fines content, $q_{c1N\text{-}cs}$ for select CPTs (see the marked CPTs on Figure 6-15) were obtained from the program CLIQ. These profiles were divided into discrete layers with representative $q_{c1N-cs}$ values. These profiles were used as input parameters for sand-like soils in the FLAC models. Each CPT profile was extended laterally in the model to the tributary area of the CPT. The simplified layers were assumed parallel to the seabed for the N-S Section and horizontal for the E-W Section. The small strain shear modulus, $G_{max}$, of soils was interpreted from the available shear wave velocity data at the site. An undrained shear strength equal to 0.22 to 0.3 times the vertical effective stress was assumed for the deep silty clay. Tables E-3 and E-4 in Appendix E present a summary of the assumed soil parameters and structural properties, respectively.

The effect of the coal stockpiles has been included in some of the analysis cases. Coal is stockpiled in east-west oriented rows within the central portion of the site, and there may be a large stockpile or no stockpile depending on the location of the design section and the configuration of the stockyard at the time of the simulated event. Accordingly, the average volume of coal stored onsite was assumed to be spread evenly over the stockpile area for the E-W FLAC analysis resulting in a uniform 3m high coal stockpile over the entire stockpile yard. The coal and potash stockpiles were considered in the N-S section as distinct embankments (with infinite length in and out of the plane of analysis) with 60% of the maximum volume. 60% of the maximum volume of stockpiles was considered to be a reasonable assumption in combination with a rare earthquake event in discussion with the design team.

The effect of ground improvement proposed beneath some of the transfer towers and conveyor bents has not been considered in the FLAC model due to their limited lateral extent and depth. The proposed ground improvement beneath the storage building footings has been taken into consideration.

Tables E-1 and E-2 in Appendix E present select FLAC analysis cases for E-W and N-S sections, respectively. Some of these cases were selected for design as indicated on Figures 6-24 and 6-25.

### *6.8.2. Results of Dynamic Numerical Analysis*

#### 6.8.2.1. Selection of a Representative Ground Motion

Analysis has been carried out on an E-W FLAC model for 15 horizontal ground motion time histories for comparison. The calculated horizontal displacement profiles of the ground surface within the reclaimed land at the end of earthquake shaking are presented in Figure 6-18a. The analyses indicate a large scatter in the calculated displacements from the suite of design ground motions. For example, at a 60m set back from the crest of the east slope (x-coordinate ~1675m), the calculated horizontal displacements for the 15 motions varied from 1m to 12m. The interface ground motion INT02 (Tohoku 2011) resulted in displacements significantly larger than the rest of the motions primarily due to the long duration of the strong motions (greater number of strong loading cycles).

Based on the commentary of Canadian Highway Bridge Design Code, CSA-S6-14, it is considered reasonable to use the average of the critical subgroup for the design. Commentary J of the NBCC 2015 recommends a more severe demand, which is using the average of the five critical ground motions. Based on engineering judgment and the fact that the proposed facility is largely unoccupied, it is proposed to use the average results from the interface subgroup which is interpreted to be the critical subgroup. Due to long run times for the numerical analyses, the crustal ground motion CRU03 (Landers 1992), whose E-W horizontal displacement profile was found to be close to the average of the interface displacement profile (Figure 6-18b), was selected as the representative motion. The N-S FLAC section was also analyzed for the 5 interface ground motions (Table E-2, Appendix E). It was verified that the CRU03 motion represents the approximate average displacement of interface motions for the N-S direction as well. Therefore, CRU03 was used for parametric analyses and design recommendations (Tables E-1 and E-2, Appendix E). All the results presented hereafter are based on this ground motion unless stated otherwise. It is anticipated that additional analyses using all 15 ground motions will be carried out during the detailed design phase of the project.

#### 6.8.2.2. Extent of Liquefaction

Figures 6-19 shows the patterns of maximum excess pore pressure ratio, Ru (ratio of maximum excess pore pressure during earthquake shaking to the initial vertical effective stress) for the two FLAC sections. Zones with maximum Ru values greater than 0.7 were interpreted as being liquefied. The analyses predict liquefaction extending to depths of about 30m on the land side and about 40m to 45m on the marine side.

Figure 6-20 compares the approximate depth of liquefaction for the A2475, A975, A475 and A200 earthquakes using ground motion CRU03. The A475 and A200 earthquake generated considerably less pore pressures than the A2475 and A975 earthquakes. The A200 ground motion has been obtained by scaling the A475 motions with a scaling factor proportional to the PGA values (~0.65). The A475 and A975 ground motions were obtained by scaling the Massey Tunnel Replacement project motions by a factor of 1.09 (also see Section 6.5).

#### 6.8.2.3. Post-Earthquake Stability Check

Post-earthquake stability checks have been performed for the A2475 earthquake using CRU03 for both FLAC sections. The results indicate that the A2475 earthquake will likely cause flow slide failure at all three perimeter slopes (Figure 6-21). The failure zones extend a horizontal distance of about 55m, 50m and 55m inland from the crest of the east, west and south slopes, respectively.

The analysis on the E-W Section was repeated for lower levels of earthquakes for comparison. The results are summarized below:

- For the A2475 and A975 earthquakes, flow slide failure is predicted for the east and west slopes. The extent of the failure zone for the A2475 and A975 events were similar.
- For the A475 earthquake, flow slide is predicted only on the east side but confined within the slope itself and does not extend inland beyond the slope crest.
- For the A200 earthquake, flow slide is not predicted.

#### 6.8.2.4. Patterns and Magnitude of Ground Displacements

Ground shaking causes sloped ground or ground in the vicinity of a free face slope to "march" in the downhill direction resulting in permanent horizontal and vertical displacements. Liquefaction significantly increases the earthquake-induced displacements, also known as lateral spreading. These displacements are greatest near the slope crest and decrease with distance from the slope crest. Post-earthquake flow slide failure causes the slope to go through further displacements until it reaches a flatter geometry which can be sustainable with the residual shear strength of the soils. The horizontal displacements behind a slope free face can extend to long distances upslope from the crest. The vertical displacements typically attenuate faster than the horizontal displacements.

Figures 6-22 and 6-23 present the patterns of post-earthquake horizontal and vertical displacements for the E-W and N-S sections, respectively. The analyses indicate that lateral spreading and flow slide failure occur in both the east and west slopes resulting in displacements in excess of 10m near the crests decreasing to zero displacements near the middle of the site. Likewise, lateral spreading and flow slide failure occur in the submarine slope located about 400m south of the site and the slope located at the south edge of the site. The N-S displacements within the site are primarily affected by the south perimeter slope and the material stockpiles. The impact of the submarine slope on the displacements of the island is relatively small.

Figures 6-24 and 6-25 present a summary of the calculated earthquake-induced horizontal and vertical displacement profiles for the ground surface (with original ground surface at El. 8m) in the E-W and N-S directions. The displacements are for the post-earthquake condition (after flow slide failure). The proposed locations of the main structures are annotated on the figures. For the E-W direction, the results are shown for the analysis cases with and without the material stockpile and for two phreatic surface assumptions. For the N-S direction the results are shown for the analysis cases with and without the coal stockpile and for the governing phreatic surface profile. The displacements in the E-W section at the locations of the structures vary from 0 to 4m in the horizontal direction and from 0.1m to 1.8m in the vertical direction. The displacements in the N-S section at the locations of the structures vary from 0.2m to 4.2m in the horizontal direction and from 0.1m to 1.0m in the vertical direction. Post-earthquake reconsolidation settlements are not included in the above-mentioned vertical displacement and would be additional.

The analysis of the N-S section was repeated with the addition of vertical ground motion to the base of the model. It was concluded that addition of the vertical ground motion had a small effect on the ground displacements at the location of the proposed structures.

Inclusion of the material stockpile increases the displacements near the slope crests in the E-W section. The material stockpile also increases the displacements near the south slope. Local lateral spreading type displacements are caused by the tendency of the stockpiles to spread and slump over liquefiable ground. This effect increases the horizontal displacements on the south side of the stockpiles and decreases or reverses the horizontal displacement on the north sides of the stockpiles as shown on Figure 6-25 (compare cases N-S-15 and N-S-19R). Dynamic analysis of an earlier version of the E-W section was repeated for the ground motion CRU03 with A975, A475 and A200 earthquakes for comparison. The A475 and A200 earthquakes resulted in significantly smaller displacements as compared with the A975 and A2475 earthquakes (Figure 6-26). Typical displacement and acceleration time histories near transfer tower P77 are also presented on Figure 6-26 for the ground motion CRU03-2475.

Its should be noted that ongoing seismic assessment of the Berth 2 area with additional information from the 2021 geotechnical exploration may result in updated displacements near the Berth 2 area.

#### 6.8.2.5. Local effects in the Storage Building

The local effects of the potash stockpile on the footings and the connecting tie rods were evaluated in the N-S section. The coal stockpile on the south side of the building was removed from the model to get the maximum lateral spreading effect of the potash stockpile on the footings. The model included the potash stockpile at 60% of maximum volume, 10m wide north and south footings (with unfactored vertical and horizontal deadload, 50% of live load and 50% of snow load), about 4m high wall structures retaining the potash stockpile, 80m long tie rods at 6.1m spacing center to center and a 25m wide by 18m deep ground improvement block under each footing (see Figure 6-27). The footings, retaining walls and tie rods were simulated using elastic beam elements. Frictional interface elements were included at the contact surface of soil with footings and retaining walls. Some parametric analyses with and without ground improvement and tie rods with different axial stiffness were carried out. Figure 6-28 illustrates the patterns of displacements in the building. Liquefaction causes a bearing failure where the liquefied soil gets squeezed out laterally which in turn causes the potash stockpile to slump. Figure 6-29a shows the exaggerated deformed shape of the footings and retaining walls. Figure

6-29b and c show the time history of horizontal displacements of the north and south footings and the axial force in the tie rod.

Table 6-6 presents a summary of the results from a series of parametric analyses for the storage building area. The model with no ground improvement and no tie rods resulted in the highest differential horizontal displacements of 1.42m between the footings. It should be noted that the parametric analysis is for comparative purposes only, and densification below the storage building is required for post-earthquake bearing resistance. Addition of ground improvement reduced the differential horizontal displacements to 1.04m. Further, addition of the 2" tie rod reduced the different displacements to 0.92m. Increasing the axial stiffness of the tie rods further reduced the differential horizontal displacements of the footings.

Flow slide failure check using the residual strength for the liquefied soils indicates that the system is stable in post-earthquake condition with inclusion of ground improvement and tie rods.

Table 6-6: A summary of the results of parametric analyses for the storage building area

| | | | Tie Rod Structural Properties | | | | | Results | |
|---|---|---|---|---|---|---|---|---|---|
| FLAC Analysis Filename | Ground Improvement (GI) | Tie Rod | Dia. | Weight | Area (A) | Elastic Modulus (E) | E.A | Axial Force | Differential Horizontal Displacement |
| | | | (inch) | (lb/ft) | (mm2) | GPa | GigaN | (MN) | (m) |
| WS-N-S-25 | yes | yes | 2" | 8.4 | 1550 | 167 | 0.26 | 2.5 | 0.8 |
| WS-N-S-24 | yes | yes | 2.3" | 11 | 1900 | 189 | 0.36 | 3.1 | 0.71 |
| WS-N-S-20R | yes | yes | 2" | | 1200 | 100 | 0.12 | 1.3 | 0.92 |
| WS-N-S-23 | yes | No | - | - | - | - | - | - | 1.04 |
| WS-N-S-26 | yes | yes | - | - | - | - | 0.72 | 4.5 | 0.52 |
| WS-N-S-27 | yes | yes | - | - | - | - | 162 | 9 | 0.01 |
| WS-N-S-28 | No | yes | 2" | | 1200 | 100 | 0.12 | 1.6 | 1.07 |
| WS-N-S-29 | No | No | - | - | - | - | 0.00 | - | 1.42 |

Tie-Rod.xlsx

Note: Case WS-N-S-27 with a large E.A of 162 GigaN was analyzed to assess the upper bound effect of nearly rigid tie rods.

#### 6.8.2.6. Dumper pit

The proposed dumper pit has been simulated as a concrete box using beam elements with a frictional interface at the contact surfaces with the surrounding soil. The top, sides and base of the box are assumed to be 0.8m, 1.0m and 3m thick, respectively. Excavation works may use permanent sheet piles for shoring. These sheet piles have not been included in the FLAC model. The existing dumper pit is offset in the east-west direction from the proposed dumper pit and therefore was not included in the north-south FLAC section. The influence of the existing coal dumper pit on the proposed structures may be evaluated in the detailed design phase.

Figure 6-30 shows the pattern of ground displacements and excess pore pressure ratio in the vicinity of the dumper pit. The soils on the sides of the dumper pit are compact to dense and do not liquefy, however the soils beneath it are expected to liquefy. The dumper pit primarily moves horizontally towards the south with slight rotation and heave. The analysis indicates that flotation does not occur due to a combination of the shear strength of the surrounding soils, the weight of the dumper and the frictional resistance between the soils and the side walls.

The time histories of total horizontal stress (soil and hydrostatic pressures) on the south wall are shown on Figure 6-31a. The initial, maximum and post-earthquake horizontal pressure profiles are extracted and plotted on Figure 6-31b. It may be observed that the pre-earthquake (initial on the figure) and post-earthquake stresses are close to the at-rest pressure profile. The profile of maximum dynamic stresses (maxima on the figure) is somewhere between the at-rest and the passive pressure profiles. The reason for a trough in the maximum dynamic stress profile may be due to the low resolution of the FLAC model (coarse mesh) or potentially arching effect in the soil due to inward bending of the middle of the wall. The side walls are expected to be stiffer than what has been assumed in the model due to presence of the interior cross walls and therefore the arching effect may not be reliable. Therefore, the dynamic pressure profile has been visually adjusted and indicated to be the design profile (Figure 6-31b). This needs to be verified with more detailed analysis (e.g. with a finer mesh and more realistic structural properties for the concrete box).

Detailed seismic assessment of the dumper pit will be required during the detailed design phase of the project.

#### 6.8.2.7. Combination of Kinematic and Inertial Loading

Understanding the concurrent shaking intensity and displacements at foundation level helps establish reasonable load factors to combine the effects of inertial loading and kinematic loading.

At sites with lateral spreading, the displacements typically increase with the duration of shaking and are at their maximum near the end of the earthquake. On the other hand, the maximum inertial loading in the structure typically occurs during the period of strong shaking. It is expected that the predicted liquefaction will result in some level of base-isolation. This will result in a considerable reduction in the intensity of ground shaking and therefore the inertial loading. Figure 6-32 demonstrates this concept for the ground motion 2475-CRU03. It may be observed in Figure 6-32b that the maximum displacement occurs at the end of shaking. The maximum acceleration near ground surface (Point 1) however occurs at about 10s (Figure 6-32d). After the onset of liquefaction at about 15s (Figure 6-32c), the ground surface shaking significantly drops at about 22s (Figure 6-32d) despite the ongoing strong shaking at firm ground (Figure 6-32e). Figure 6-32f shows that liquefaction drops the shaking level at Point 4 (further inland) after 22s of shaking. However, liquefaction did not drop the response spectrum at Point 4 (Figure 6-32g) because a portion of the strong shaking occurred early on, before the onset of liquefaction.

Section 7.2.4 provides additional discussion on kinematic and inertial load combinations.

### *6.8.3. Discussion*

Seismic assessment was conducted using both simplified/conventional and more rigorous dynamic non-linear effective stress numerical modelling. Both methods include considerable uncertainties.

This project has adopted a performance-based design methodology, which requires estimation of ground displacements for kinematic loading of the structures. Non-linear effective stress dynamic numerical modelling (numerical modelling) is the primary method to get insight into the patterns and magnitudes of displacements. The advantages of numerical modelling are that it can consider the site specific conditions and capture the main mechanism of movements. On the other hand, simplified/empirical methods have the advantage that they are based on real observations from historical cases. Therefore, it is a good practice to use the empirical methods as a high-level check against the results of numerical modelling.

Numerical modelling and simplified methods both predict liquefaction for A2475 earthquakes. The depth of liquefaction from the numerical modelling is about 30m as compared to about 50m for the conventional method. This difference is expected as non-linear effective stress analysis typically result in lower seismic demands.

Numerical modelling and the limit equilibrium method both predict flow slide failure along the perimeter of the site. The limit equilibrium method, however, indicates greater failure zones behind the crest of slopes than the numerical model. Both methods evaluate the post-earthquake condition of the potash stockpile as stable.

Numerical modelling and the Zhang et al. (2004) empirical method both estimated displacements in the range of meters. Figure 6-33 compares the horizontal displacement profile of the ground surface for the free field conditions with the Zhang et al. method at select distances from the crest of the slope. The magnitude of displacements and the variation with distance are comparable in both methods.

There are significant uncertainties in seismic analyses and design due to the nature of the earthquake hazard, soil variability, simplified assumptions to develop the geotechnical model and dynamic behavior of soils at the element level and the ground as a dynamic system. Numerical modelling carried out for this project provides insight into the response of the system and patterns of ground movement. The quantitative results are considered best estimate values based on the available data.

## 7. DISCUSSION AND RECOMMENDATION

### *7.1. General*

Section 6 presented the seismic design challenges and provided insight into liquefaction potential, patterns of flow slide failure, estimates of horizontal and vertical displacements in the E-W and N-S directions, post-earthquake reconsolidation settlements, seismic response of the storage building foundations and the seismic response of the dumper pit. The information provided in Section 6 will be used as an aid to engineering judgment for the seismic design of various components of the project for the A2475 earthquake.

The design issues related to service conditions are bearing capacity of foundations, settlements under service loading, and lateral earth pressures on the underground structures and retaining walls. The estimated settlements presented in this section do not include potential ongoing global settlements of the entire site as a result of placement of reclamation fill. These would be additional to those presented below and are not expected to affect the performance of the proposed structures.

The objective of this section is to provide recommendations for geotechnical design of the proposed structures for service and seismic conditions. The recommendations provided may require revision as design progresses.

### *7.2. Foundations*

Shallow foundation support is proposed for conveyors, transfer towers and storage building structures. Ground improvement can improve the seismic performance of shallow foundations by mitigating the consequences of liquefaction. Ground improvement is proposed for the storage building and outbound transfer towers and conveyor bents located on the northeast side of the site. Conventional ground improvement (e.g. vibro stone columns) within the area of the dumper pit, tunnel and inbound (west and south of the storage building) transfer towers and conveyors bents, Tower P77, and the sampling tower was considered not practical by the design team due to site restrictions and the potential to damage existing nearby infrastructure. As such it is proposed to design these shallow foundations for reduced post-earthquake bearing pressures.

All structures will be detailed to tolerate the total displacements and differential displacements relative to adjacent connected structures.

### *7.2.1. Foundation Systems Assessed For New Inland Structures In The Berth 2 Area*

Flow slide failure and large seismic displacements are expected near the crest of slope along the east side of the site. Analyses were carried out to assess potential foundation systems that could be feasible in this area. Foundation systems that were reviewed included:

- Large diameter pile groups in various configurations that could resist the forces from the near surface soil movements.
- Removal of a portion of the near surface dense non-liquefiable crust to reduce kinematic loading on the piles.
- Timber pile densification with structures supported on disconnected raft foundations that are anchored to a deadman with a large set back from the slope to reduce horizontal movements.
- Raft foundations supported on grade above densified soils with the foundations anchored to a deadman with a large set back from the slope to reduce horizontal movements.
- Raft foundations supported on a jet grout berm in the form of shear walls in the ground to reduce movements.

The first four foundation systems were determined to not be feasible due to their inability to resist lateral movements or forces, lack of space to allow implementation, potential for causing ground movements and damage to the existing structures and/or cost implications. The last foundation system was deemed to be a viable solution from a technical perspective but was found to be cost prohibitive. As a result of the above analyses, the early design concept of constructing new structures near the slope was revised. It was determined by the design team and Client that the existing material handling and ship loading infrastructure in the Berth 2 area would have to be modified for use with the new cargo project.

### *7.2.2. Estimated Total Displacements*

Figure 7-1 provide best estimate values of horizontal and vertical ground displacements at the locations of the proposed structures. The displacements near the Berth 2 area may be updated based on seismic assessment of Berth 2 which is currently in progress.

The horizontal displacements are based on the results of the E-W and N-S FLAC analyses presented in Figures 6-24 and 6-25, respectively, assuming that the displacements in both directions occur concurrently. The effect of the stockpiles was considered for the structures that were deemed to be located within the zone of influence of the stockpiles. The effect of proposed ground improvement on reduction in horizontal displacements is expected to be minimal and was omitted in the FLAC models, with the exception of the storage building.

The vertical displacements at ground surface were assumed to be the sum of the following components.

a) Vertical component of lateral spreading movement in the east-west direction (obtained from E-W FLAC analysis)

b) Vertical component of lateral spreading movement in north-south direction (obtained from N-S FLAC analysis)

c) Post-earthquake reconsolidation settlement (obtained from simplified method, Table 6-4)

The greatest lateral spreading type displacements typically occur at the end of the earthquake. Post-earthquake reconsolidation settlement is expected to occur over a period of minutes to days after the end of the earthquake.

Shear induced settlement of the shallow foundations has been omitted and is expected to be relatively small due to densification below some structures, and the use of a low design bearing pressure below structures which will not have subgrade densification. This settlement occurs when the ground supporting the foundation softens/weakens due to liquefaction. Additional shear strains are required to mobilize additional soil strength sufficient to support the foundation. Settlements due to sand boils (ejecta) have also been neglected due to the proposed densification and/or considerable thickness of the non-liquefiable crust.

It is noted that actual displacements could vary due to differences in soil conditions, geometry, foundation loading, boundary conditions (e.g. presence of the existing dumper, densification of the existing office building), and randomness in the response of the ground to earthquake shaking and most importantly the uncertainties in the earthquake ground motions. This will result in uncertainty in the estimated total and differential displacements, which should be considered in structural design. For preliminary considerations, it is recommended that adjacent structures be designed for differential displacements in 3 orthogonal directions. There is no established method to compute differential displacements. The Commentary to the Canadian Bridge Code (CSA-S6-14, Section C4.6.6) recommends a minimum post-earthquake differential settlement and differential horizontal displacements equal to one half of the total estimated values. A lower design differential displacement of about 30% of the total estimated displacement is suggested for the design of the New Cargo Structures based on engineering judgment, consultation with the design team and considering the following:

a) It is recognized that the bridge code recommendation is for bridges which have high occupancy and importance. The New Cargo Structures will have low to no occupancy and are not considered to be important structures.
b) The code recommendation is heavily based on Martin et al. 1999 (Guidelines for analyzing and mitigating liquefaction in California) which also discusses the importance of the degree of understanding of the soil conditions qualitatively. The degree of understanding of the soil conditions at the Westshore site is considered to be "typical to high" due to the recent and historical geotechnical site investigations.

Based on the above, the recommended variation in seismic displacements for both horizontal and vertical directions can be estimated as ± 15% of the total estimated displacements. As such, the design differential displacement between adjacent structures could be as high as approximately 30% of the total displacement of a given structure.

For foundations with ground improvement, differential settlement across individual footings may be taken as 30% of the total estimated settlements for foundations with small eccentricity (i.e. relatively uniform bearing pressure below the footing). This value is 50% of the total settlement for foundations without densification.

### *7.2.3. Ground Improvement*

Vibro-stone column densification is proposed for ground improvement below some of the proposed structures. The specific size of the densification block for each structure is provided in the following sections. The objectives of densification are to provide a non-liquefiable crust that can support the shallow foundation loading before and after the earthquake, reduce post-earthquake reconsolidation settlements, and reduce the site variability within the influence zone of each footing and between adjacent footings. The level of ground improvement considered in this report for various structures will not significantly reduce the earthquake-induced horizontal and vertical displacements.

Typically, stone columns should be installed to a performance specification within the upper sand soils with low fines content (less than about 20%) to mitigate liquefaction, and to a method specification within the underlying high fines content soils (silty sands and sandy silts) and silt soils. The performance specification will be developed as part of the bid documents. For preliminary considerations and cost estimate, it is recommended that the method specification be comprised of stone columns installed at a 2.7 m triangular spacing with the additional requirement for a minimum 0.9 diameter within high fines content soils and silts. Pre-drilling may be required at some locations to allow installation of stone columns through the dense fill. A densification field trial program is recommended before the production work to verify the densification methodology and stone column spacing. It is recommended that field trials be carried out at two locations with different soil conditions. Pre- and post-densification CPTs will be required for the field trial and quality control of the production densification works. Installation of stone columns using the bottom-feed method is recommended to minimize spoil and wastewater at the site.

Ground improvement using vibro-stone columns may cause ground movements, settlements, vibrations and consequently damage to adjacent existing facilities. Accordingly, the design team has chosen to eliminate densification near existing structures that may be impacted, or in areas with physical conflict with the proposed densification. Effects of installation of stone columns on adjacent structures must be monitored during construction at any critical locations.

#### *7.2.4. Inertial – Kinematic Load Combinations*

As discussed in Section 6.8.2.7, the combination of inertial and kinematic loading depends on the ground motions and the ground response. It is unlikely that the peaks of kinematic and inertial loadings occur at the same time. Where a simplified design approach is to be used, the 2016 BC Supplement to CSA-S6-14 recommends the following load combinations be considered in design:

- 100% Kinematic demands
- 100% Inertial demands
- 50% Inertial Demands + 100% Kinematic demands

The supplement also states that “in cases where soil softening does not reduce the inertial effect, then a special assessment shall be undertaken to develop an appropriate combination of inertial plus the applicable kinematic effects.” Figure 6-32 indicates that liquefaction reduced the shaking level at the ground surface and therefore the above simplified load combinations are considered satisfactory for ground motion CRU03.

### *7.3. Dumper Pit and Tunnel*

#### *7.3.1. Discussion*

The proposed dumper pit will extend about 11m below ground surface and will be approximately 8m below groundwater level. In addition, it will be located relatively close to a slope to the south, and the existing dumper pit to the southwest (see Figure 2-2).

It is understood that a design-build contractor will design and execute the excavation and construction of the dumper pit and tunnel. Therefore, analysis and design of the temporary works are outside the scope of this report.
General geotechnical related issues include, but not limited to, the following.

1. Construction
    - Excavation
        i. Failure/heave of the bottom during excavation

ii. Buoyancy forces and uplift
iii. Settlements in the vicinity if the ground water level is lowered
iv. Seepage rates into the excavation
v. The magnitude and distribution of the horizontal earth pressures is dependant on the construction and shoring methodology

- Impact on the existing facilities

  The excavation for the dumper pit is expected to be supported using strutted sheet pile shoring designed by others. Vibration from installation of sheet piles and relaxation of horizontal earth pressures from excavation work will cause some level of ground deformation that may affect the nearby existing facilities. The displacement tolerance for the existing nearby structures should be determined and considered in the design and construction of the dumper pit and the tunnel. Other considerations include:

  i. Existing dumper pit which may need some level of monitoring.
  ii. Nearby underground utilities may require monitoring.

2. Service conditions (static)
   - Earth and hydro-static pressures on the perimeter walls and the base
   - Buoyancy/uplift forces
3. Seismic conditions
   - Lateral and vertical displacements
   - Distortion of the structure (racking)
   - Seismically induced earth pressure on the walls
   - Liquefaction-induced buoyancy and uplift

*7.3.2. Seismic Considerations*

Estimated free field ground displacements at the location of the dumper pit and tunnel are expected to be in the range of 0.3 m west, 3.5 m south, and 0.9 m of vertical displacement including post-earthquake reconsolidation settlement. Some rotation and distortion (racking) are also likely to occur as a result of seismic loading.

Liquefaction of soils surrounding and below the dumper pit could result in increased buoyancy effects. Theoretically the buoyancy force below the dumper pit could be as high as 19H kPa where H is the depth of the underside of the dumper or tunnel in metres below surrounding grade.

As discussed in Section 6, the dumper pit was modelled within the north-south FLAC analysis. The analysis indicates potential for some buoyancy of the dumper pit. However, the vertical (upward) displacements are expected to be negligible based on the FLAC analysis due to the presence of non-liquefiable soils surrounding the near surface portion of the dumper pit (~8m soil crust), frictional resistance of the walls, self-weight and residual shear strength of the underlying liquefied soils. A more detailed assessment of the response of the dumper pit under seismic loading conditions will be undertaken in a future scope of work.

The connection between the dumper pit and tunnel should be detailed to tolerate potential differential movements.

*7.3.3. Subgrade Preparation and Foundation Design*

Excavation for the dumper pit is expected to extend into the silty zone below the near surface granular soils. Based on discussions with CWA's structural group, the excavation will be carried out using strutted sheet pile shoring. The interior will be excavated using a clam shell bucket, or similar method, while the excavated hole remains filled with water. Excavation will extend several metres below the underside of the proposed dumper pit. Concrete will be poured up to the underside of the dumper pit using a tremie concrete placing method. Subsequently, water inside the excavation will be pumped out. The concrete plug at the base of the excavation will be sized to resist uplift from hydrostatic pressures. The dumper pit will then be founded directly on the concrete plug. The dumper pit walls will be cast against the shoring sheet piles.

Excavation for the dumper pit and tunnel are expected to result in a net unload of the subgrade. As such, static settlements are expected to be negligible. Additional recommendations regarding foundation design can be provided after details of the excavation and tremie plug are finalized.

### *7.3.4. Horizontal Wall Pressures*

#### 7.3.4.1. Static Lateral Earth Pressures

Under static loading conditions, it is expected that both the dumper pit and tunnel are sufficiently stiff and restrained from rotation or lateral movement such that they can be considered non-yielding structures. For non-yielding walls, a triangular earth pressure distribution using an at-rest earth pressure coefficient ($K_o$) is considered appropriate.

Lateral earth pressures provided for the dumper pit and tunnel assume that soils adjacent to the structures will be comprised of granular material. Groundwater is expected to be at a depth of approximately 3m, and the portion of the structures below 3m will also be subject to hydrostatic pressures. The effects of surcharge and hydrostatic pressures should be considered as discussed below.

Recommended static horizontal wall pressures on the north and south sides of the dumper pit are provided in Figure 7-2. The north side of the dumper pit will be influenced by surcharge loading of the adjacent road, and the nearby coal stockpiles (Figure 7-4). The estimated horizontal pressures from the maximum coal stockpile height of 23m and a uniform roadway surcharge of 24 kPa located at a 3m horizontal offset are provided in Figure 7-2.

Recommended static horizontal wall pressures on the east and west sides of the dumper pit are provided in Figure 7-3. The east and west sides of the dumper pit will include a 6m long by 2.6m wide approach slab to support the tracks in order to reduce potential impact of differential settlement between the dumper pit building and the adjacent ground surface. One end of the approach slab will be supported on the dumper pit wall, and the other side on ground surface. Vertical loading on the ground from the approach slab will result in additional horizontal pressures on the dumper pit walls as indicated in Figure 7-3.

The triangular earth pressure distributions provided in Figures 7-2 and 7-3 include a 20 kPa component for near surface “locked in” loading from compaction of backfill. The compaction induced component may be omitted where backfill is not placed and compacted adjacent to the structures, such as foundation walls constructed using sheet pile shoring as one side of the formwork.

Recommended horizontal wall pressures at the location of Section C, D and E along the tunnel are provided in Figure 7-6. The figures include the effect of the coal stockpile and roadway along the north side of the tunnel, and the train tracks on the south side.

#### 7.3.4.2. Seismic Lateral Earth Pressure

Recommended seismic horizontal wall pressures on the sides of the dumper pit are provided in Figure 7-5.

Horizontal wall pressures on the north side of the dumper pit have been estimated from the N-S FLAC output. FLAC allows the horizontal pressures to be modelled more accurately than conventional methods as it considers displacements, movement of the non-liquefiable crust above the underlying liquefiable soils, and the impact of the coal stockpiles to the north. For seismic conditions, it has been assumed that the coal stockpile is at 60% of maximum service capacity. More detailed assessment of the seismic response of the dumper pit will be carried out in a future phase of work.

Horizontal wall pressures on the east and west sides have been estimated using the method developed by Wood (1973) which provides an estimate of dynamic earth pressures for rigid walls.

### *7.4. Storage Building Settlement Assessment*

#### *7.4.1. Introduction*

This section provides the results of the settlement assessment carried out for the proposed storage building. The objective is to provide estimates of post-construction settlements and assess if preloading is required. Field settlement data collected from previous projects adjacent to the site, provided in a report by Klohn Crippen Berger (KCB) dated June 21, 2021, was used to calibrate and update the settlement model. This section provides preliminary estimates of post-construction settlements with and without preloading based on current design loads.

This assessment is subject to revision based on construction sequencing and review of settlement monitoring results during the preloading period. The focus of this section is Phase 1 of the storage building.

#### *7.4.2. Background*

##### 7.4.2.1. Proposed Building

Phase 1 of the storage building will have plan dimensions of approximately 408 m east-west and 70 m north-south. Phase 2 would extend the building to the east by approximately 256 m.

Based on preliminary drawings by CWA Engineers Ltd. (CWA), the building will be founded on 10 m wide grade supported strip footings located along the north and south sides of the building. Densification using vibro-stone columns to improve seismic performance will be carried out below the footings. Potash would be stored within the building with a maximum stockpile height of approximately 21 m.

##### 7.4.2.2. Available Information

The following information relevant to the settlement assessment was made available to Braun/NAGL:

- Draft Independent Settlement Assessment by KCB dated June 21, 2021.
- Roberts Bank Seismic Evaluation by Golder Associates dated May 18, 2011 (Confidential Report).
- Select 1994 CPT information at the location of Pods 4 and 5 provided by KCB (attached).
- Preliminary storage building drawings provided by CWA Dwgs. 85400-D0010-0110, -0115, -0116, -0117 (Appendix F).
- Storage Building existing coal / sand interface elevation plan and profile drawings by R.F. Binnie & Associates Ltd. dated December 18, 2020 (attached in Appendix 2).

- Storage Building Area – Phase 1 Preload General Arrangement by R.F. Binnie & Associates Ltd. dated June 1, 2021 (attached in Appendix 2).

#### 7.4.2.3. Site Development

The overall Westshore/Deltaport site was constructed over a period of approximately 40 years with the development of five pods. Construction of Pod 1 was completed in 1970. It was used for coal storage and included a ship loader on the east side. Pods 2 to 4 were constructed between 1981 and 1984. Westshore's operations were expanded to Pod 2 in 1984. Pods 3 and 4 remained vacant until the mid-1990s, when Deltaport was developed at Pod 4. Deltaport was expanded to Pod 3 in 2000. Deltaport was expanded again from 2005 to 2007 with the construction of Pod 5. Figure 7-7 shows the general layout of the site and proposed potash storage building plan. Historical air photos are attached in Appendix G.

#### 7.4.2.4. Historical Site Use

Based on available information, the previous use of the area within which a majority of the building is proposed included stockpiling of coal commencing in 1970 within the Pod 1 area. Pod 2 was constructed in the early 1980s, but based on review of historical air photos, coal storage in the area of the proposed storage building does not appear to have commenced until the 1990s. The Westshore office building and other facilities previously occupied a portion of the proposed storage building footprint on the western portion of Pod 1 up to 2017, when these facilities were demolished and coal storage commenced in this area. Due to the relatively short period of time that coal has been stored in the historic office area, the preloading effect of the coal is expected to be considerably less than the surrounding areas. Information provided by Westshore indicates that the coal stockpiles within the footprint of the proposed building were typically about 15 m average height, and 26 m maximum height.

#### 7.4.2.5. Historical Settlement Data

Silt, silty zones, and deep clay underlying the site are compressible. Limited preload settlement monitoring data is available from the development of the site. The available settlement monitoring information is provided on Figures 7-8 to 7-11. The data discussed below was obtained from the KCB report, and the elevations and durations described below are approximate.

Figure 7-9 indicates placement of 8 m of fill at Pod 3 over a period of 2 years. It is understood that fill placement at Pod 3 was not continuous and occurred over a relatively long period of time. A majority of the settlements occurred in the first 700 days. It should be noted that this rate of settlement is likely due to the slow rate of loading. It is expected that settlements would have stabilized faster if fill placement had been completed over a shorter period of time. Total settlements were in the range of 0.6 to 1.2 m after 14 years.

The KCB report indicates that fill was placed from El. 0 to 7 m at Pod 4. Figure 7-10 indicates that settlements of 0.8 to 1.2 m occurred over a period of 8 years. Detailed information regarding rate of fill placement at Pod 4 was not available.

Figure 7-11 indicates fill was placed from approximately El. 2 to 11.65 m at Pod 5 over a period of 120 days in 2 stages. After a surcharge duration of 80 days, the fill was removed to the final site grade of 7 m. Total settlements were in the range of 1.2 to 1.6 m after one year. A majority of the settlements occurred in the first approximately 30 days after placement of the stage 2 fill.

There is considerable scatter in the settlement monitoring data at each of the pods which represents the inherent uncertainty in the prediction of settlements.

### 7.4.3. Historical Coal Loading

Storage of coal over portions of the site is expected to have partially preloaded the footprint of the coal stockpiles, but the actual loading history is highly variable. The loading history of the ground due to coal stockpiles (i.e. magnitude, duration, extent, and location) is expected to be variable in both the east-west and north-south directions. Based on review of historical air photos, the coal stockpiles within Row D were generally placed near the reclaimers, closer to the south footprint of the proposed storage building as shown in Figure 7-12. For this reason, the historical preloading effect of the coal is expected to be greater on the south side of the storage building compared to the north.

Historical coal loading is also expected to vary considerably in the east-west direction. The historical coal storage distribution in the east-west direction was estimated by Westshore and provided for review. The information provided indicated the weight and volume, and estimated stockpile heights for 3 assumed stockpile widths (64 m, 73 m, and 82 m). Figure 7-13 provides the estimated maximum annual average coal stockpile height between 2010 and 2020 for the three assumed stockpile widths. The figure indicates considerable variability in the east-west direction.

The interface elevation of the coal and the underlying sand fill was surveyed by Binnie and the results presented in the attached Binnie Drawings. The survey indicates that the top of the sand surface elevation within the proposed storage building footprint varies by approximately 1.2m in both the east-west and north-south directions. The variability in the soil conditions throughout the building footprint is not significant enough to result in such variability in the settlements. As such, it is expected that the variation in settlements is primarily a result of variable coal loading history.

Without a building preload, the variation in historical coal loading will result in differential settlements below the building and potash stockpile (i.e. a similar magnitude of new vertical loading at different locations could result in different amounts of settlement). Differential settlements are expected to vary considerably in both the north-south and east-west directions. Vertical loading from a building preload with a surcharge will exceed the historical loading and minimize the effects of loading history.

### 7.4.4. Preloading

Vertical loading from the proposed potash stockpile and storage building will exceed that from past coal loading. The compressible soils are expected to be relatively sensitive to new loading. As such, the building and potash stockpile loads will result in additional immediate and long-term settlements of the ground surface and footings.

Even with the benefit of a past preload from an assumed coal stockpile configuration, estimated preliminary post-construction total and differential settlements (provided in Doc.#20-8543-REPORT-002-Rev2) were considered excessive for the proposed storage building. The design team and Owner indicated that the settlement risk was considered to be too great and it was determined that a preload with surcharge would be provided to reduce post-construction settlements. An independent settlement assessment was carried out by KCB and also concluded that preloading should be undertaken.

The preload would typically load the footprint of the storage building to at least the expected long-term sustained loading. A surcharge is typically used to reduce the preload duration and post-construction settlements. A larger surcharge further reduces the risk that the preloading period exceeds the available time.

The duration of the preloading period depends on the soil conditions, amount of surcharge, and the amount of settlement to be eliminated. The current preload configuration for the Phase 1 building is provided on the drawing Storage Building Area-Phase 1 Preload General Arrangement by R.F. Binnie attached in Appendix 2. The drawing illustrates a two-stage preload with an overlap between the two preload stages.

A large volume of sand is required for the preload and it will take several months to import and place it. It is understood that initially the preload will be placed full-height from one end of the building to some distance beyond the middle of the building. When the preload period is complete at the location where preload placement commenced initially, it will be moved gradually to the opposite end over a period of several months. This is expected to result in a more uniform preload surcharge across the middle of the building footprint as opposed to a true two-stage preload which would be affected by edge effects at the overlap.

In order to reduce the preloading duration, the potential for use of wick drains was discussed with an installation contractor (Menard Canada). The contractor indicated that the soils near the surface are relatively dense and would require predrilling to penetrate with the mandrel used to install the wick drains. Furthermore, zones within the underlying silt and sand interbeds are also too dense to penetrate, and refusal would likely be encountered before the target depth is achieved. The contractor also indicated that the proposed densification work would introduce high permeability soils into the subgrade and improve drainage.

Prediction of the magnitude and time of settlement in variable soil conditions, in particular interbedded silty soils, includes uncertainties, especially considering the variable site loading history. The preload surcharge would be used to reduce total settlements and to reduce many of the uncertainties, including differential settlements, the amount and rate of settlements, varying site loading history, variable soil conditions and soil parameters, potential buried structures, and rock berm. Preloading will also reduce, but not eliminate, requirements for and the frequency of maintenance of the building and the equipment.

### *7.4.5. KCB Model Calibration*

Subsequent to the initial settlement assessment provided in the project geotechnical report, KCB carried out an independent settlement assessment which included a review of historical settlement monitoring results from Pods 3 to 5 that were available in their archives. KCB used their historical information from development of the site to calibrate their model. It is understood that the settlement model was developed using soil layering from test holes located near the settlement monitoring data, and the settlement parameters for the various soil units were modified to obtain results that matched the measured values.

KCB initially used the Pod 5 settlement data to calibrate their soil settlement parameters. The calibrated parameters were then input into the Pod 3 soil layering and loading which resulted in over prediction of settlements. KCB chose to use the Pod 3 data due to proximity to the proposed storage building, and revised their calibration and soil parameter to match the Pod 3 settlement data.

It is noted that KCB did not have access to the 2011 Golder consolidation tests on the deep clay layer. The Golder data indicated a compression index (Cc) for the deep clay typically greater than 0.4. KCB used a value of 0.20. This difference may have resulted in over estimation of Cc values for the upper silty soils within their calibrated model.

KCB Tables 3.2 and 3.3 are shown below for comparison with Braun / NAGL soil parameters presented in the following section. The tables provide a summary of the soil parameter selected by KCB for their Pod 5 and Pod 3 calibrations. The Pod 3 calibration values were used in their settlement assessment of the storage building.

*KCB 2021 - Tables 3.2 and 3.3*

**Table 3.2 Soil Types & Recalibrated Parameters for the Pod 5 Area Settlement Analysis**

| Soil Type | Saturated Unit Weight γ | | Moisture Content Mc | Void Ratio $e_0$ | Compression Index Cc | Recompression Index Cr | Secondary Compression Index Cα | Coefficient of Compressibility Cv | |
|---|---|---|---|---|---|---|---|---|---|
| | (kN/m³) | (lb/ft³) | (%) | | | | | (m²/year) | (ft²/year) |
| SAND | 19.5 | 124 | 25 | 0.65 | 0.04 | - | - | 5,000 | 53,790 |
| Silty SAND | 19.5 | 124 | 25 | 0.65 | 0.12 | - | - | 500 | 5,379 |
| Sandy SILT | 19.5 | 124 | 25 | 0.65 | 0.20 | - | - | 120 | 1,291 |
| SILT (upper) | 19.0 | 121 | 30 | 0.85 | 0.40 | - | - | 250 | 2,690 |
| SILT (lower) | 19.0 | 121 | 30 | 0.85 | 0.40 | - | - | 50 | 538 |

Notes:

**Table 3.3 Soil Types & Recalibrated Parameters for Potash Storage Building Settlement Analysis**

| Soil Type | Saturated Unit Weight γ | | Moisture Content Mc | Void Ratio $e_0$ | Compression Index Cc | Recompression Index Cr | Secondary Compression Index Cα | Coefficient of Compressibility Cv | |
|---|---|---|---|---|---|---|---|---|---|
| | (kN/m³) | (lb/ft³) | (%) | | | | | (m²/year) | (ft²/year) |
| SAND | 19.5 | 124 | 25 | 0.65 | - | - | - | - | - |
| Silty SAND | 19.5 | 124 | 25 | 0.65 | 0.12 | 0.012 | 0.006 | 500 | 5,379 |
| Sandy SILT | 19.5 | 124 | 25 | 0.65 | 0.20 | 0.02 | 0.01 | 120 | 1,291 |
| SILT (upper) | 19.0 | 121 | 30 | 0.85 | 0.35 | 0.035 | 0.018 | 250 | 2,690 |
| SILT (lower) | 19.0 | 121 | 30 | 0.85 | 0.20 | 0.02 | 0.01 | 50 | 538 |



### *7.4.6. Model Calibration and Soil Parameters*

The soil parameters used in the settlement model developed in the previous revision of the geotechnical assessment report (Doc.#20-8543-REPORT-002-Rev2) were estimated from empirical correlations based on laboratory test results, general knowledge of the site, past experience on Fraser River sediments, review of the test hole logs, and engineering judgement. Soil settlement parameters for the deep clays were obtained from eight laboratory consolidation tests summarized in the 2011 Golder report. The Cc values ranged from 0.39 to 0.68, with a single outlier of 0.19. The Golder test results indicate a Cc of 0.54 at SH10-04 located closest to the storage building and was selected for the settlement model. The overlying silt and silt/sand mixture zones are also compressible, but retrieval of undisturbed samples is not practical due to thin interbedding with sand and low plasticity.

For the current study, the soil consolidation parameters were refined and calibrated based on available settlement monitoring data. The assumed fill placement height (loading) and duration were based on available information described in the 2021 KCB report.

The settlement parameters were calibrated using the Pod 5 settlement data. Pod 5 was selected because of the availability of higher quality and more detailed information on fill placement and settlement measurements (Figure 7-11). The calibrated model was then checked against settlement monitoring data from Pods 3 and 4.

The Pod 5 soil layering used in the model was based on KCB CPT94-05 to a depth of 30 m, Braun / NAGL CPT20-06 for 30 to 50m depth, and Golder SCPT10-04 below 50 m depth (Figure 7-14). Note that all three test holes are some distance from the settlement gauges at Pod 5, resulting in increased uncertainty in the calibration model.

The soil layering was input into the settlement model and the soil parameters for the silt and silt & sand zones were adjusted to make the model reasonably match the field settlement monitoring data. The soil parameters used in the model are provided in Table 7-1 below, and a comparison of the estimated settlements with the settlement monitoring data is provided in Figure 7-15.

Subsequently, the soil parameters obtained from calibration to the Pod 5 data were input into the Pod 4 soil layering interpreted from CPT94-8. The loading at Pod 4 included placement of fill from El. 0 to El. 7 m. The rate of fill placement is unknown. It was assumed that the fill was placed over a period of 150 days to roughly match the initial rates of field measurements. The assumed soil layering for Pod 4 calibration is shown in Figure 7-16. The calibration parameters from Pod 5 overpredict long-term settlements at Pod 4 by approximately 25% compared to the average measured values, as shown in Figure 7-17.

The soil settlement parameters calibrated from Pod 5 data were then input into Pod 3 soil layering. Test hole information near the settlement gauges in Pod 3 were not available at the time of this report. As such, SCPT14-01 located on the north side of Pod 2 was used for soil layering (Figure 7-18). The predicted total settlement was approximately 50% greater than the average measured settlements (Figure 7-19).

Settlements for Pods 3 and 4 are overpredicted by 50% and 25%, respectively using the calibration from Pod 5. The predictions are considered to be within the range of accuracy for settlement estimates (typically 0.5 to 2 times the actual values). The following reasons may have contributed to the overprediction:

- Pod 3 and 4 settlement monitoring may have missed the initial settlements. As such, actual total settlements may have been greater than measured. It is considered likely that some fill placement may have occurred prior to installation of settlement gauges to allow the gauges to be installed above water and to reduce the potential for gauge damage during placement of fill on the ocean floor.
- There is little information on instrumentation and rate of loading.
- Test hole data immediately adjacent to historical settlement gauges was not available at the time of report preparation. This increases the uncertainty of the calibration model.
- Depth to till-like soils may vary between the pod locations, and the thickness of compressible soils could also vary.

If test hole information becomes available, consideration should be given to re-evaluating the calibration using settlement data with nearby soil conditions.

Based on the above, the soil parameters estimated from calibration with settlement monitoring data from Pod 5 were used for the settlement assessment for the storage building. The selected soil parameters are provided in Table 1 below.

Table 7-1: Soil Parameters Calibrated from Pod 5 settlement data

| Legend | Material | γ | Cc | Cr | Cα | Cv | $e_0$ | Es | Esr |
|---|---|---|---|---|---|---|---|---|---|
| S | Sand | 19 | - | - | - | - | - | 3 x $q_t$ | 3 x Es |
| T | Silt & Sand | 18.5 | 0.1 | 0.01 | 0.005 | 100 | 1 | - | - |
| M | Silt | 18 | 0.25 | 0.025 | 0.0125 | 50 | 1 | - | - |
| C | Clay | 18.5 | 0.54 | 0.12 | 0.027 | 50 | 0.8 | - | - |

Settlement analysis results - sensitivity analysis - 2021-08-24.xlsx

Notes:
γ = Soil unit weight (kN/m$^3$)
Cc = Compression index
Cr = Recompression index = 0.1 x Cc
Cα = Secondary compression index, assumed to be 5% of Cc (Ladd and DeGroot, 2003)
Cv = Coefficient of consolidation (m$^2$/yr)
eo = Intial void ratio
Es = Young's modulus of soil assumed to be equal to 3.0 x qt (Schmertmann Method from USACE 1990)
Esr = Unload/reload elastic modulus = 3 Es
$q_t$ = Cone tip resistance
Cc, Cr, and Cv for the deep clay were obtained from the 2011 Golder consolidation tests

The KCB parameters are presented in Section 5 for comparison. KCB did not carry out an analysis with 100% potash loading and the current preload configuration for comparison of results.

### *7.4.7. Methodology to Estimate Settlements*

Settlement analyses were completed using the software package Settle 3 Version 5 by Rocscience Inc. There are six CPTs within the footprint of the proposed building. Each of the six profiles were used to estimate settlements without preloading using the following steps:

1. Placement of a coal stockpile above the existing site fills within the southern portion of the proposed building to a maximum height of 12 m. The assumed coal cross section is provided in Figure 7-12. It is assumed that the crest of the coal extends from the west end of the proposed building towards the east a horizontal distance of 740 m. The side slopes of the coal stockpile are assumed to be inclined at the angle of repose of 40 degrees.
2. Removal of the coal stockpile.
3. Addition of sustained loads including site grading fill, footing loads, additional loading from stone columns, and ramp fill below the north portion of the potash stockpile.
4. Placement of the potash stockpile within the building to the full height of 21 m.

Following the above analyses, a representative profile (CPT20-09) was used for a series of parametric analyses. Settlements were estimated for a preload with a surcharge of 5% and 20% of the sustained loads and for preloading durations of 3, 6 and 9 months. The building preload was added to the model after removal of the coal stockpile (after Step 2 above).

As discussed in Section 4, it is expected that the preload placement method results in a relatively uniform preload treatment across the entire footprint of the building, i,e, the interior parts of the building will get the benefit of a uniform preload of about 250 m long. Accordingly, the preload is modelled as a single stage preload that is placed across the entire building footprint, i.e. about 400 m long. It is acknowledged that this assumption is optimistic but it is deemed that the amount of error is small. For comparison to KCB results, additional analyses were carried out to estimate the settlement profile in the east-west direction resulting from a two-stage preload with an immediate move between stages. The actual behavior of the ground is expected to be bracketed by the above assumptions but closer to the uniform one-stage loading.

### *7.4.8. Assumptions to Estimate Settlements*

The following list provides assumptions that have been made for the development of the settlement model for the potash storage building.

- The soil stratigraphy used in the analysis is shown in Figure 7-20. CPTs 20-05 to 10 located within the footprint of the proposed building were used to develop simplified soil layering. Deeper information (30 to 50 m depth) at these locations is extrapolated from nearby test holes. The stratigraphy of the ground below a depth of 50 m was interpreted from the Golder (2011) SCPT10-04.
- Soil settlement parameters from the Pod 5 calibration were used.
- Silt and silt & sand mixture soil layers were assumed to have two-way drainage (top and bottom).
- Secondary settlements were estimated using the Mesri option in Settle 3.
- A 12 m high coal stockpile preload has been assumed, as shown in Figure 7-12.
- The potash stockpile has been assumed to be 21 m high and to extend nearly the full length of the Phase 1 building as shown in Figures 7-7 and 7-12.
- Future coal stockpiles are located beyond a zone of influence.
- The drainage effects of densification on the rate and magnitude of post-construction settlement below the footings has not been considered.
- Subgrade loading used in Settle 3 is as summarized in the following section.

### *7.4.9. Loads*

The following loads were considered for the construction stage of the model:

- Weight of stone columns.
- Weight of 0.6 m of site grading fill within the building footprint.
- Weight of ramp fill below the north portion of the proposed potash stockpile.
- Building footing loads and footing weight.
- Weight of 100% of the potash stockpile within the building.

The difference in the unit weight of the excavated coal/sand mixture and replacement sand fill was considered negligible and not included in the model.

It is assumed that the preload will be placed to the required thickness, and additional fill will not be placed to compensate for settlement during preload placement. The sacrificial sand (due to settlement under the preload) will remain in place and will add to the sustained loads. The weight of the sacrificial sand has not been considered in the current settlement model. It is estimated that the thickness of sacrificial sand will an average of about 0.3m and will vary across the building footprint.

Details of the above loads are provided below.

*Stone Columns*

Vertical loading from the addition of gravel for the construction of stone column densification was estimated based on the following:

- 2.7 m triangular spacing for stone columns.
- A stone column diameter of 0.9 m and a depth of 18 m.
- A total gravel unit weight of 20 kN/m$^3$ above the water table and a submerged gravel unit weight of 10 kN/m$^3$ below the water table.
- Groundwater at a depth of 3 m.

The above results in an estimated uniform equivalent load of 21.5 kN/m$^2$. This load was modelled over a width of 25 m centred below each of the north and south footings. Portions of this load were applied at three discrete depths.

*Grading Fill*

It is understood that 0.6 m of fill will be placed to raise the grade within the building footprint. It is assumed that this fill will comprise fine to medium sand (river sand) compacted to 95% Modified Proctor maximum dry density resulting in a moist unit weight of 18 kN/m$^3$. Based on the above, the additional loading from the site grading fill is estimated to be 11 kN/m$^2$ applied over the entire building footprint.

*Ramp Fill*

It is assumed that the ramp fill below the northern approximately half of the proposed potash stockpile will also comprise compacted river sand with a unit weight of 18 kN/m$^3$. The maximum thickness of the fill will be 3.8 m at the north end of the stockpile, gradually decreasing to no fill at approximately the middle of the stockpile. Based on the above, the loading from the ramp fill is estimated to be 69 kN/m$^2$ at the north end of the stockpile, decreasing to 0 kN/m$^2$ below the middle of the stockpile.

*Building Footings*

It is understood that the building footings will be primarily above surrounding grade. As such, the total unit weight of concrete was used to estimate the footing weight. Vertical loading from the footings has been assumed to include self weight (1.2 m thick footings with a unit weight of 24 kN/m$^3$), plus structural loads on the footing. The total pressure at the underside of footings has been assumed to be 42 kN/m$^2$ based on information provided by CWA.

*Potash Loading*

The settlement model has assumed that the building will be fully loaded with potash. It has been conservatively assumed that the full height potash stockpile is a sustained load. The maximum thickness of potash has been assumed to be 3.7 m at the south, 21 m at the middle, and 0 m at the north end of the stockpile. The potash stockpile has been modelled as a embankment with a unit weight of 11.8 kN/m$^3$.

The superimposed pressures from the above loads are shown in Figure 7-22. Figure 7-23 shows the equivalent sand preload shape, assuming a sand unit weight of 16.5 kN/m$^3$ overlain on the current preload configuration. Based the above, the current preload provides an overall surcharge of approximately 5% based on the cross section area of the two.

### *7.4.10. Settlement Estimate Results*

#### 7.4.10.1. General

Total settlement typically includes immediate settlement, primary consolidation settlement, and secondary compression settlement. Immediate settlement occurs in granular soils that can drain the load-induced excess pore pressure relatively quickly. It could also occur in fine-grained soils due to undrained ground movement. Primary consolidation settlement is the compression of the soils as excess pore water pressure due to loading is expelled from the soil voids. Secondary compression is the ongoing creep-type settlements resulting from the rearrangement of the soil particles under constant effective stress that continues after consolidation is complete. Immediate settlement has been considered for the sand zones. Primary consolidation and secondary compression have been considered for silts, clays, and silt & sand mixtures.

The design concerns regarding the settlements are total settlements, differential settlements between the north and south footings, differential settlement of the footings in the east-west direction, and differential settlement of the potash reclaimer supports in both the north-south and east-west directions. Information regarding the settlement tolerance of the building was not available at the time of report preparation.

Foundation and building settlements typically result from foundation loads. However, for the subject building, a large proportion of the foundation settlements are expected to result from loading imposed by the potash stockpile within the building.

Densification is typically expected to reduce total and differential settlements by improving the in-situ soils and by stone columns acting as stiffer vertical elements (similar to weak piles). Installation of stone columns is expected to accelerate settlement in soil layers to installation depth due to the introduction of highly permeable soils into the ground. This effect is typically neglected due to potential for silt contamination of the stone columns. It is understood that the stone columns will be installed after preloading, and preloading will not benefit from the drainage provided by the stone columns.

The effects of variable historical coal loading are expected to be significantly reduced by the preload.

The following sections provide the results of the settlement analyses based on the soil parameters and loading discussed above, without and with preload treatment. Except where noted, the presented results are for the base-case consisting of a single stage preload with 5% surcharge, 6 months duration and CPT20-09 soil conditions.

The settlement estimates should be considered approximate. Settlements are difficult to predict accurately and could vary by 50% to 200% of the predicted values at different site locations. The rate of settlement with time is even more uncertain.

#### 7.4.10.2. Settlement Estimate Without Preloading

The estimated settlements along a north-south section using soil profiles estimated from CPTs 20-05 to 20-10 located within the footprint of the proposed building for 50 years post-construction are provided in Figure 7-24. The total settlement of the ground surface under the potash stockpile is estimated to be approximately 700 and 850 mm. The total settlements at the centre of the north and south footings are estimated to be approximately 500 to 600 mm and 330 to 360 mm, respectively.

The variation in settlements under the building footings due to soil variability estimated using CPTs 20-05 to 20-10 is approximately 120 mm on the north side and 50 mm on the south side (see Figure 7-24). Actual variation could be greater due to the variation of loading history of the site, future building and stockpile loads, and the potential for variation in soil conditions away from the test hole locations.

#### 7.4.10.3. Settlement Estimate With Preloading

The preload presented in Figure 7-21, which provides a 5% surcharge, was used in the initial analysis assuming a 6 month preload duration. A single stage preload was used.

The estimated settlements along a north-south section using CPTs 20-05 to 20-10 for 50 years post-construction are provided in Figure 7-25. The total settlement of the ground surface under the potash stockpile is estimated to be approximately 500 to 580 mm. The total settlements at the centre of the north and south footings are estimated to be approximately 310 mm and 330 mm, respectively. The variation in settlements at the location of the available CPTs under the building footings due to soil variability is estimated to be less than 25 mm (see Figure 7-25).

It is noted that the preload reduces total post-construction settlements substantially, as well as differential settlements between the north and south footings.

The results also indicate that some rotation of the ground surface below the footings may occur due to greater settlements on the interior side of the building. The results indicate that preloading reduces the magnitude of rotation.

Figure 7-26 presents the variation in post-construction settlement with depth. The figure indicates that 280 mm of the total 340 mm post-construction settlement under the footings occurs at depths greater than 40 m. About 200 mm of the total settlement below the footings occur in the deep clay layer below a depth of 80 m.

#### 7.4.10.4. Parametric Analysis of Preload Surcharge and Duration

Based on review of the long-term estimated building settlements with and without preloading, CPT20-09 was chosen as a representative soil profile to allow a set of parametric analyses to be completed for preload durations of 3, 6, and 9 months and surcharges of 5% and 20%. The estimated 50 year post-construction settlements are presented in Figure 7-27 and Table 7-2. The results of the settlement estimate without preloading are also included for comparison.

Table 7-2 – Estimated 50 Year Post-Construction Total Settlements (mm)

| Preload surcharge | Preload duration (months) | North Footing | Middle of building | South Footing |
|---|---|---|---|---|
| no preload | - | 576 | 792 | 350 |
| 5% | 3 | 319 | 564 | 340 |
| 5% | 6 | 313 | 557 | 336 |
| 5% | 9 | 310 | 552 | 333 |
| 20% | 3 | 234 | 373 | 247 |
| 20% | 6 | 199 | 335 | 218 |
| 20% | 9 | 195 | 330 | 213 |

2021-08-21-settlement-preload-sensitivity-analysis 400m long building.xlsx

The results indicate that a greater surcharge has a significant effect on reduction of post-construction settlements. The larger surcharge (20% vs 5%) reduces the estimated post-construction settlements below footings by approximately 27% to 37% depending on the preloading duration. The settlement below the middle of the stockpile is reduced by approximately 40%.

Typically, a longer preload duration will result in less post-construction settlements. However, due to the relatively high drainage rate (high Cv) assumed in the settlement model, the rate of settlement is relatively fast, and the results appear to have a low dependence on preloading duration.

Figure 7-28 provides an estimate of loading versus time and the associated settlements during historical coal loading, building preload (with 5% surcharge and 6 month duration), and up to 50 years after construction. The figure indicates that although the subgrade below the north and south footings have experienced different levels of historical loading, the post-construction settlement rates of are similar as a result of preloading as shown in Figure 7-29.

The post-construction settlement response after 5% surcharge (Figure 7-29a) includes rapid settlement of 50 mm below the footings and 180 mm below the potash stockpile. Long term settlements continue at a decreasing rate to a total settlement of 330 mm below the footings, and 560 mm below the stockpile at 50 years. For a 20% surcharge (Figure 7-29b) the post-construction response includes rapid settlements of 50 mm below the footings and 140 mm below the potash stockpile. Long term settlements continue to a total settlement of 200 to 220 mm below the footings, and 340 mm below the stockpile at 50 years.

#### 7.4.10.5. East-West Analysis with Two-Stage Preload

Figure 7-30 presents the estimated post-construction settlements in the east-west direction for two surcharges (5% and 20%), for a 6 month preload duration, and for both a single stage preload and a two-stage preload with an overlap in the middle.

Estimated settlements near the middle of the building are similar to those described above for the north-south alignment for a single stage preload. For a two-stage preload, there are larger settlements near the preload overlap due to the edge effects.

The settlements near the ends of the building are similar for both a single stage and two-stage preload. The settlements decrease to near zero over a horizontal distance of approximately 100 m. Larger curvatures occur at several locations due to edge effects of the two-stage preload and the ends of the potash stockpile. The location of the end of the potash stockpile could vary during the service life of the building. As such, the large curvatures could occur at various locations along the length of the building.

### *7.4.11. Discussions*

#### 7.4.11.1. Differential Settlements

Differential settlements and changes in settlement gradients are expected to be the primary concerns for the proposed potash storage building. For the east-west direction of the strip footings and building, it is the curvature of the ground surface (change in the differential settlement gradient) that is the primary concern and not the magnitude of settlements or differential settlements. At the time of report preparation, building settlement tolerances were not available.

Settlements are expected to vary with variable ground conditions, loading history, future unbalanced potash stockpile loading in the east-west direction, and densification. Differential settlements between two points may be calculated based on test hole information. This approach requires detailed geotechnical information (i.e. closely spaced test holes advanced to the depth of the till). This is impractical and cost prohibitive. As such, multiple approximate methods for estimating differential settlements/curvatures are discussed below and recommended design values are provided based on review of the results and engineering judgement.

#### 7.4.11.2. North-South Differential Settlements

The calculated post-construction differential settlement between the north and south building footings is less than 30 mm at the centres of the footings and varies depending on the preload surcharge and duration.

It is common to use ½ to ⅔ of the total settlements as the design differential settlement. However, it is expected that a majority of the total settlements will occur at depth due to preloading (Figure 7-26). Ground improvement is expected to further reduce the ground variability and hence shallow differential settlements. Deep differential settlements are less damaging to the structures as they manifest at the ground surface with reduced magnitude and curvature. This is due to the shear stiffness and shear strength of the shallow ground which flattens out the deep differential settlement curvatures. Accordingly, it is considered that the above rule of thumb for differential settlements of ½ to ⅔ of the total settlements can be reduced. It is recommended to allow 1/3 of total settlement as differential, i.e. 340 x 1/3 ~ 120 mm. The same amount of differential settlement may be considered between the north and south stacker/reclaimer tracks.

It is expected that footing settlements may be greater on the interior side of the footings relative to the exterior side. This can cause rotation to the footing that may affect the structural design. The calculated free field rotation of the ground below the footings is about 1 in 200 and 1 in 250 for preload with 5% and 20% surcharge, respectively. These rotations do not include any factor of safety. To knowledge of the authors, there is no established method to determine a design rotation value for shallow footings.

#### 7.4.11.3. East-West Differential Settlements

Two methods are used below to estimate differential settlements in the east-west direction. Method 1 addresses the differential settlements due to the shape of potash loading. Method 2 addresses the differential settlements that could be caused by different soil conditions particularly at depth where little geotechnical information is available. Both methods should be evaluated separately and used for design.

Method 1- Estimated Post-Construction Settlement Profile

The first method is to use the calculated post-construction settlement profile shown in Figure 7-30 to evaluate the resulting kinematic loading on the footings and the building. The kinematic loading can be evaluated in a soil-structure interaction analysis where the building strip footing is supported on a series of soil springs. The settlements can be applied to the ends of the springs. The settlement profile shown in Figure 7-30 is primarily a result of the shape of the potash loading and preloading. As mentioned before, the location of the maximum curvature could vary depending on the length of the potash stockpile within the building.

Method 2: Parametric Numerical Modelling

The effect of differential settlements due to the potential ground variation can be bracketed using a parametric numerical modelling method as described below.

The proposed preloading and installation of stone columns should reduce site variability resulting in a more uniform manifestation of settlements at ground surface. The reduction in post-construction settlement is expected to be greatest within the near-surface soils (e.g. from 0 to 20 m) as the preload will be more efficient due to highly permeable sandy layers and also due to densification. At a mid-range of depth (e.g. from 20 to 40m) the preloading should still be fairly effective in reducing differential settlements caused by soil variability. The deeper soils, including the thick silty clay/clayey silt layer, will achieve a lower degree of consolidation under the preload and will be the primary contributor to long-term settlements (Figure 7-26).

Differential settlements at depth manifest at surface with a reduced intensity (magnitude and curvature). It is understood that there are no guidelines available to estimate ground surface manifestation of deep differential settlements.

To allow structural assessment of potential differential settlements in the east-west direction, the surface manifestation resulting from 300 mm of differential settlement at a depth of 40 m over varying horizontal distances was modelled. The parametric analyses were carried out using the program FLAC to bracket the probable pattern of settlement at the ground surface. The objective is to evaluate the ground surface undulations resulting from an undulation at depth.

Figure 7-31 illustrates the concept of this approach. A differential settlement of 300 mm with a square wave shape with a select wavelength (e.g. $\lambda$=30 m in Figure 7-31) is induced at 40m depth. The analysis considered only the displacement at depth (i.e. the loading induced by the building was not included in the analysis). The objective is to estimate the settlement pattern of the ground surface (e.g. the amplitude and wavelength of the settlements at surface). Figure 7-31 shows the computed shape at the ground surface. The analyses were repeated for a range of induced wavelengths from 16 to 400 m. The results of select analyses are summarized in Figure 7-32. As an example, the differential settlement for $\lambda$=30 m has reduced from 300 mm at 40 m depth to 70 mm at the ground surface. It may be observed that a longer wavelength tends to generate a larger amplitude at the surface but with a milder curvature. Conversely, a shorter wavelength tends to generate a smaller amplitude but a sharper curvature at the ground surface.

The results can be used as input into soil-structure interaction analyses where the building strip footing is supported on a series of soil springs. The provided ground surface settlements (for 5 scenarios) should be applied to the ends of the springs. The deformations, shear forces and bending moments in the footing can be assessed using this method. A modulus of subgrade reaction (scaled to the footing width, Kvb) of 5700 kN/m$^3$ is recommended for computation of the stiffness of the elastic soil springs. During detailed design, consideration could be given to assess the sensitivity of the results to Kvb by using a range of values (e.g. half to double Kvb).

It should be noted that the FLAC analyses were carried out before calibration of the settlement model. The analyses assume 300 mm differential settlement at 40 m depth. The current total post-construction settlement at 40 m depth is 280 mm (Figure 7-26). Assuming differential settlement at depth is ⅔ of the total, it is suggested that the ground surface shapes in Figure 7-32 be scaled by a factor of 0.65.

#### 7.4.11.4. Preload

##### 7.4.11.4.1. Shape of The Preload

The preload configuration used in the above analyses is shown in Figure 7-21. It was developed with input from the design team during an earlier phase of the project. It is estimated that this configuration provides an approximately 5% surcharge relative to sustained loads.

The shape of the preload in the north-south direction does not mimic the shape of the surface pressure from the sustained loads (Figure 7-23). Construction of a peaked preload would be impractical. As such, a flat top preload was proposed with the volume in the apex moved out towards the footings. As such, the loading is proportionately greater above the footings, and lighter in the middle of the building. This is expected to result in reduced footing settlements which is the main objective of preloading.

7.4.11.4.2. Duration of Preload

Figure 7-33 compares the total settlement during preload and post-construction for three preload durations: 3, 6 and 9 months. The results suggest that there is little benefit in increasing the preload duration from 3 months to 6 or 9 months. However, due to uncertainties in the prediction, it is prudent to maintain the currently scheduled duration of 9 months for each preload stage but be prepared to take advantage of a shorter preload duration based on the results of preload settlement monitoring data.

7.4.11.4.3. Preload Length and Overlap

It is expected that construction of the preload will take many months. It is understood that it is proposed to construct the preload full height in the east-west direction, from one end of the storage building to approximately 30 m beyond the middle. When the preload period at the starting end is complete, the preload would be removed from that end and gradually moved to the opposite end. Therefore, all areas of the building footprint will have experienced relatively uniform loading.

The current east-west settlement models do not consider the gradual movement of preload from stage 1 to stage 2 and assume that the entire site is preloaded simultaneously or in two discrete stages with an overlap in the middle. The two-stage preload does not accurately reflect the proposed preloading staging as the intermediate preload moving stage will preload the soils below the overlap to a greater extent than had the entire preload been removed and placed at the stage 2 location immediately. The result is that the larger post-construction settlements indicated below the area of the overlap (Figure 7-30) are likely to be less pronounced, and will depend on the rate at which the preload is moved. A longer preload move time is expected to result in more uniform surcharging of the soils near the middle of the building, and smooth out the settlement spike shown on Figure 7-30.

7.4.11.4.4. Extension of Building Phase 1 Preload into Building Phase 2

The deformation tolerance of the building and structural details of the transition between building phases should be considered in the final design of the extension of the Phase 1 preload into Phase 2 such that the following two criteria are met:

- The overlap should be sufficiently long such that the Phase 2 preload does not adversely affect the completed Phase 1 building.
- The overlap should be sufficiently long such that the Phase 2 preloading can reduce the post-construction settlements under the overlap area to a tolerable range.

If preloading for Phase 2 is to be completed at a later date, it is understood the conveyor would have to remain operational along the north side of Phase 2 during construction. It is understood that the conveyor will be placed within a tunnel to allow preloading. It is recommended that the volume of preload that is offset by the volume of the tunnel be placed at a suitable location to adequately preload the north footing area.

7.4.11.4.5. Preload Surcharge Versus Duration

Based on the results of the parametric analyses on preload duration and surcharge level, consideration could be given to additional analyses to assess potential for use of a larger surcharge for a shorter duration. This could result in a preload length that is shorter than currently proposed. This may reduce the required volume of import preload sand. It is noted there is a minimum preload length required to effectively consolidate the deep soils, that should be assessed during detailed design.

#### 7.4.11.5. Impact on Existing Structures

The large vertical pressures imparted at surface by the preload and potash stockpiles will result in deep settlements that can extend horizontally beyond the perimeter of the loads. This can result in ground surface settlement well beyond the loaded areas.

There is potential for settlement of existing and proposed structures in the vicinity of the loaded areas, with the magnitude of settlement decreasing with distance. It is recommended that the potential for the proposed preload and stockpile to impact adjacent structures and underground services be reviewed during the detailed design stage of the project. Requirement for pre-construction survey and ongoing monitoring during and after construction should be assessed.

#### 7.4.11.6. Initial Stockpile Loading and Building Settlement Monitoring

Placement of the potash stockpile in concentrated areas could increase the potential for differential settlements in the east-west direction. It is recommended that the loading pattern during operation be considered in preload design.

Ongoing settlement monitoring is recommended to assess differential settlements and allow the stockpile location and height to be adjusted if required such that settlements occur uniformly. Monitoring settlements would also allow re-levelling of structures as required until settlement rates are within a tolerable range.

It is recommended that design and construction details be included in the structure to accommodate settlements and allow adjustment of the building components and the stacker/reclaimer tracks. The impact of maintenance and potential downtime on service-stage operations should be considered in preload design.

### *7.4.12. Limitations and Uncertainties*

There is considerable uncertainty in prediction of settlements. The uncertainties in prediction of the rate of settlement with time are even greater. Availability of local settlement data for calibration of the settlement model reduces the uncertainties but does not eliminate them. These uncertainties affect the design of the preload, particularly the prediction of time required for completion of the preloading period. The surcharge level should consider potential variation in the actual preloading period such that the project schedule is not impacted. The uncertainties with regards to post-construction settlements can be reduced by recalibrating the settlement model based on the response of the ground to preloading.

Sources of uncertainty include, but are not limited to, the following items.

1. Across the site, there are variations in the layering and thickness of compressible soil zones.
2. Deep borehole information (below 60 m) was obtained from a single borehole by Golder (2011), which neglects possible site variability below this depth. Note that deep soil variability typically has a lesser impact on surface differential settlements relative to variability within shallow soils.
3. Characterization of compressibility of low plastic silt, interbedded silts, and silt & sand mixtures is challenging due to difficulties in obtaining high quality undisturbed samples.
4. Estimating the time response of the interbedded soils and layered soils under a large loaded area includes uncertainty due to the complex horizontal and vertical drainage paths.

5. There is no established method for estimation of differential settlement and gradient of settlements.
6. Detailed test hole information is not available at the location of historical settlement monitoring gauges which were used to calibrate and verify the settlement model in Pods 3 to 5.
7. Building preload methodology and potash stockpiling patterns can impact post-construction settlements. These details have not yet been finalized.
8. The historical coal stockpile locations, height, duration, and degree of primary consolidation likely vary.

The above-mentioned uncertainties could result in the actual settlements varying from predicted values. The currently planned preloading, surcharging, and densification will reduce the above uncertainties.

### *7.4.13. Future Work*

It is understood that final design of the preload would be completed during the detailed design phase of the project. Items to be addressed during detailed design include, but are not limited to, the following.

1. The currently proposed shape of the preload is preliminary and based on discussions with the design team. This preload shape was also used by KCB for their assessment. The preload configuration may be refined / optimized based on updated subgrade loading and deformation tolerances of the building.
2. If the building is constructed in two phases (i.e. if there is a delay between the construction of Phase 1 and Phase 2), construction and service load interaction between phases should be considered in the design and the construction methodology. This may include impacts from building and stockpile loads, densification, and preloading.
3. The deformation tolerance of the building (which will depend on the structural system, jacking/re-levelling systems, construction joints, etc.) and material handling equipment should be determined by the Structural Engineers and communicated with the Geotechnical Engineers to allow design of an efficient preload. The objective is to keep post-construction deformations within tolerable levels.
4. If additional test hole information near the location of historical settlement gauges in Pods 3, 4 and 5 becomes available, consideration should be given to re-evaluating the calibrations.
5. The construction methodology for the preload should be considered in preload modelling and post-construction settlement estimation.
6. Further assessment of differential settlements in the east-west direction should be carried out. The additional work should also include a review of variable future potash loading along the east-west alignment of the building, interaction with the adjacent transfer towers, the existing waste stockpile located at the northwest corner of the site, and the effects of phased construction of the building.
7. The settlement model can be recalibrated based on the response of the ground to the building preload. This calibrated model can be used to update the settlement estimates of the building under service loading and various configurations of the potash stockpile as necessary.

8. The potential for the proposed preload and stockpile to impact adjacent structures and underground services should be reviewed.

## *7.5. Storage Building Foundation Design*

### *7.5.1. Foundation Design*

It is anticipated that the storage building will be supported on approximately 10m wide strip footings along the north and south sides of the building following densification. Subsequent to densification and removal of unsuitable soils (topsoil, coal and silt which may remain from the densification process), the exposed fill should be compacted to at least 95% Modified Proctor Density (MPD) using a heavy vibratory roller. Where required, structural fill should be provided over the prepared subgrade to reinstate subgrade level and improve bearing capacity. The exposed subgrade should be reviewed and approved by the Geotechnical Engineer prior to placing structural fill and construction of foundations.

For subgrade preparation completed as discussed above, footings may be designed for the bearing resistance values provided in Table 7-3. The bearing resistance values are limited by the punching resistance of the densified zone into the underlying liquefied soils in A2475 earthquake loading. It is understood that the actual design contact pressure below footings is in the range of 86 kPa, which is the value used in the settlement analyses. Higher bearing pressures will increase settlements.

Table 7-3: Factored bearing resistances for static and A2475 earthquake for storage building foundations

<table>
<tr><th rowspan="3">Foundation Subgrade</th><th rowspan="3">Strip Footing Width</th><th>Static</th><th colspan="3">Seismic</th></tr>
<tr><th rowspan="2">Factored Bearing Resistance</th><th rowspan="2">Ultimate Bearing Resistance</th><th colspan="2">Factored Bearing Resistance</th></tr>
<tr><th>During Shaking<br>Resistance Factor =1</th><th>Post Earthquake<br>Resistance Factor = 0.67</th></tr>
<tr><td>Compacted structural fill on densified ground</td><td>10m</td><td>188kPa</td><td>188kPa</td><td>188kPa</td><td>125kPa</td></tr>
</table>

The design pressures assume the following:

- Footings are founded at least 300mm below final finished adjacent grade for frost protection, or are provided with a minimum 300mm thickness of non frost susceptible fill below footings.
- Ground densification is carried out to 18m depth below the current ground surface.
- All load-bearing surfaces are reviewed and accepted by the Geotechnical Engineer.
- Foundation bearing surfaces are no higher than 2H:1V (horizontal to vertical) from the base or toe of adjacent foundation elements and no higher than 1H:1V from the base or toe of sumps, utility structures, or other buried structures.
- The bearing pressure for inclined loading should be reduced in accordance with CSA-S6-14 with the values provided in Table 7-4.

### *7.5.2. Sliding Resistance*

It is understood that the north and south footings will need to resist structural loads inclined at 40 degrees from the vertical, and that these loads will be resisted by sliding resistance below the footings, and tie rods provided between the north and south footings.

An ultimate friction coefficient of 0.5 is recommended between cast in place concrete and the footing subgrade.

### *7.5.3. Seismic Considerations*

Ground improvement (soil densification) is recommended below the building foundations to reduce the potential for punching failure of the footing into the liquefied soils, post-earthquake reconsolidation settlements, and to reduce lateral and vertical movements resulting from the design seismic event. It is recommended that densification for the building comprise vibro-stone columns installed to a depth of 18 m and to a horizontal distance of 8 m beyond the perimeter of the foundations.

A detailed discussion regarding seismic aspects of the storage building is provided in Section 6.

### *7.5.4. Post-Earthquake Slope Stability Analysis Check*

Limit equilibrium slope stability analysis indicates that the potash stockpile is stable in post-earthquake conditions. This is consistent with the numerical modelling results. Figures 7-34 and 7-35 present the results for the static condition at 100% capacity and post-earthquake condition at 60% capacity of the potash stockpile, respectively.

### *7.5.5. Lateral Earth Pressures on Potash Stockpile Retaining Walls*

The potash within the building will be retained with an approximately 4m high wall on the south side as shown on Figure 7-36. The north side wall will be approximately 3.2m high and support an inclined working surface backfilled with granular fill.

The walls should be designed for the lateral earth pressures indicated on Figure 7-36. The lateral earth pressures provided is for fully drained backfill assuming at-rest soil condition for static loading and yielding wall condition for seismic loading. Note that the provided horizontal pressures are total and include seismic and static pressure components.

## ***7.6. Transfer Towers and Conveyor Bents***

### *7.6.1. Discussion*

The conveyors supported on transfer towers and bents near the site perimeter are long linear structures that are expected to be heavily impacted by seismically induced differential displacements across the site. On the south, west and east sides, the conveyors are located near the perimeter slope where seismic displacements are expected to be large. Estimated horizontal and vertical displacements resulting from seismic loading are discussed in Section 6.0 and summarized in Figure 7-1.

The geotechnical related issues can be summarized as follows:

1. Construction
    - Variable subgrade conditions encountered below foundations.
    - Requirement for ground improvement, and/or sub-excavation and replacement of unsuitable soils.

- Impact of construction on the adjacent facilities.

2. Service conditions (static)
   - Settlement (total and differential) and tilting.
3. Seismic conditions
   - Lateral spreading (vertical and horizontal displacements) due to close proximity to the perimeter slopes.
   - Post-seismic reconsolidation settlements and tilting.
   - Differential horizontal and vertical movements relative to adjacent bents, and transfer towers.
   - Post-liquefaction bearing resistance

### *7.6.2. Foundation Design*

It is understood that inbound transfer towers and conveyor bents and outbound Transfer Tower #77 will not include ground improvement due to conflict with existing underground utilities and the presence of nearby infrastructure that may be adversely impacted by densification. These structures will be designed for reduced bearing resistances suitable for the relatively thin zone of dense soils near surface and the underlying liquefiable soils.

Recommended factored bearing resistance values are provided for static and seismic loading conditions in Table 7-4 attached. Total static settlements of transfer towers and conveyor bents are expected to be in the range of 150mm to 300mm and 100mm to 150mm, respectively, and can be reduced with a preload surcharge if required. Immediate and consolidation settlements are estimated to comprise approximately 40% of the total settlements, and are expected to be substantially complete within 2 to 4 months of load application. The secondary compression component of the total settlement would occur over many years at a decreasing rate.

Bearing resistance values for seismic loading have been evaluated based on a simplified two-layered system (non-liquefiable crust over liquefiable soils), local ground conditions and initial estimates of the footing sizes provided by CWA. The two-layered system includes the contribution of the shear resistance of the non-liquefiable near surface soils and the bearing resistance of the underlying liquefiable soils at residual strength. The bearing resistance values should be reduced for inclined loading based on the ratio of the horizontal to vertical loads according to the above noted table.

A geotechnical resistance factor of 1 (consistent with CSA-S6-14) has been used for seismic loading, during earthquake shaking. A resistance factor of 0.67 has been used for the post-earthquake conditions to improve the post-earthquake stability of the system. A resistance factor of 0.67 is equivalent to a factor of safety of 1.5, which is the recommended value by Bray and Macedo (2017) based on past earthquake observations to minimize shear induced settlement, and to reduce tilting.

The bearing resistance values for seismic loading for footings founded on non-densified soils are obtained using simplified methods. It is expected that advanced dynamic soil-structure interaction analyses will be performed during a future stage of the project to provide insight into the behavior of the system and modes of failure, and to verify the footing bearing resistance.

Eliminating densification below footings increases the risk for additional settlements (including shear-induced settlements and loss of material due to ejecta). There are no established methods for estimation of settlement due to ejecta and tilting.

CSA-S6-14 allows eccentricities up to L/3 (where L is the footing size in the direction of the overturning moment). Due to the uncertainties, particularly for tilting of footings on non-densified ground, it is preferable that the magnitude of eccentricity be limited to L/6 (force resultant in the middle third of the footing).

Large surficial cracking is expected in the flow slide failure zone of the edge slopes. The locations of proposed structures are assessed to be beyond the estimated flow slide zone. Therefore, qualitatively, the size of surface cracks within the crust at the locations of the proposed structures are expected to be small relative to the proposed size of the footings. The crack and fissure sizes have not been explicitly estimated.

#### *7.6.3. Seismic Considerations*

Estimated seismically induced displacements are provided in Figure 7-1. Transfer towers, conveyor bents, and the supported conveyers should be designed for the estimated displacements.

Where proposed, densification below transfer towers should comprise vibro-stone columns installed to a depth of 18m below existing grade, and to a horizontal distance of approximately 7 m beyond the perimeter of the foundations. Densification for conveyor bents should be carried out to a depth of 10m below existing grade, and to a horizontal distance of approximately 5 m beyond the perimeter of the foundations. The objectives of ground improvement are to improve bearing capacity of the foundations, reduce the effect of site variability, reduce post-earthquake reconsolidation and shear induced settlement, and prevent post-earthquake punching failure of the footing. The proposed ground improvement will not reduce the earthquake induced lateral spreading displacements significantly.

### *7.7. Assessment Of Spout Tower Piles Subject To Seismically Induced Kinematic Loading*

The current design of the spout tower foundations includes six 1200mm x 44mm (48” x 1.75”) steel pipe piles. The proposed piles are vertical and 75m long, with an embedment depth of approximately 50m.

The spout towers are relatively light structures supported by large piles. The primary design concern is lateral loading due to seismically induced soil movements.

The lateral pile loading analysis was carried out using the software package Group 2019 by Ensoft Inc. The software can model inertial loading on the piles as well as kinematic loading from soil movement.

The analyses were carried out for soil displacements associated with an A2475 earthquake. The analyses did not converge due to excessive movement of the pile group. The analyses were repeated with 85m long 1520mm x 41mm (60”x1.625”) piles which resulted in convergence of the model. The background information, methodology and results of the analyses are provided in Appendix I.

The design return period of the Berth 2 structures is currently under review, and may be smaller than A2475. It is understood that the analyses may be revised based on ground displacements associated with the selected design earthquake during the detailed design phase.

Appendix I provides a summary of the background information and results.

### *7.8. Site preparation*

It is recommended that subgrade preparation below the proposed structures include removal of existing coal and sand/coal mixtures to expose existing sand fill prior to densification. Drainage measures and grades should be incorporated so as to reduce potential for ponding of water, especially during densification if the top feed vibro-stone column method is used for ground improvement.

Stripped surfaces are to be reviewed by the Geotechnical Engineer prior to placing foundations or structural fill. Construction of temporary access measures (granular access pads and roads) suitable for support of equipment may be required.

### *7.9. Structural Fill*

It is recommended that structural fill below the proposed structures be comprised of approved, clean, free draining well graded 75mm minus gravel and sand with less than 5% fines (percent passing the #200 sieve). Alternate, clean, granular fills may also be suitable, and can be reviewed if requested. Structural fill should be placed in maximum 300mm loose lifts and compacted to at least 95% MPD.

It is recommended that structural fill extend beyond the edges of the proposed structures a distance equal to the thickness of confined structural fill. It is recommended that unconfined structural fill extend beyond the edges of the proposed structures by a distance at least twice the thickness of unconfined structural fill.

Density testing during site fill placement is recommended on a regular basis to confirm adequacy of compaction. The results should be forwarded to the Geotechnical Engineer for review. The Geotechnical Engineer should also be contacted to review fill quality, placement, and compaction procedures.

Portions of existing onsite fill soils free of coal, fines and organics may be considered suitable for potential re-use as structural fill, subject to review and approval of the proposed material by the Geotechnical Engineer.

### *7.10. Temporary Cut Slopes*

Excavations up to 1.2m deep can be cut near vertical in accordance with Worksafe BC regulations. Deeper unsupported excavation cuts should be sloped no steeper than 1H:1V (Horizontal to Vertical). These recommended cut slopes should be reviewed by the Geotechnical Engineer during excavation and may require modification based on actual site conditions. Flatter slopes may be required if poor soil conditions, sloughing, or significant seepage is encountered.

Large unsupported excavations may be impractical and/or unachievable where the excavation extends below the water table, and/or where geometric constraints do not permit the proposed excavation cut slopes. In these areas, temporary shoring measures will be required.

### *7.11. Perimeter Drainage*

Requirements for perimeter drains are to be evaluated by the Geotechnical Engineer on a per structure basis when design of the proposed structure(s) has been finalized.

# 8. GEOTECHNICAL INPUT FOR ANALYSIS OF INBOUND STRUCTURES

## *8.1. Introduction*

This section provides a summary of geotechnical information provided for the soil-structure-interaction analyses of the inbound structures carried out by the structural design team during the Bridging Scope phase of the project. The following information was provided to the structural design team for inbound structures:

1. Linear footing springs for the structural spectral analysis.
2. Footing load-displacement curves for structural non-linear dynamic time history analysis.
3. Free field horizontal (east-west and north-south) and vertical time histories of displacements and accelerations at the location of structures for structural non-linear time history analysis.

This phase of the project included numerous iterations and revisions to the inputs. Only the final results are provided in this report.

Geotechnical input for the items listed above was provided in digital format as well. The digital data is available upon request.

The following sections and respective appendices provide brief descriptions, summary background information and results for each item.

## *8.2. Linear Footing Springs*

Shallow footing springs (3 translational and 2 rotational) were estimated to allow soil-structure interaction analysis for inbound structures. Torsional springs (about the vertical axis) were not required by the structural design team. The springs were calculated for liquefied and non-liquefied conditions by push over analyses in 2D FLAC on strip footings (infinite length into the plane of analysis). Incremental rotation, vertical displacement and horizontal displacements were applied in separate analyses to the footing for one cycle of load-unload-reload. The corresponding rocking moment, vertical force and horizontal force were obtained from the analyses. Moment-rotation and load-displacement curves were plotted. Constant spring values were provided as requested by CWA. Spring constant values for each degree of freedom were interpreted visually from the load-displacement curves for the generic strip footings modelled in FLAC (Table J-2 and Figures on Pages J-15 to J-27 in Appendix J). FLAC 2D stiffnesses were corrected approximately to consider the 3D effects and actual sizes of the inbound footings based on elastic solutions by Gazetas 1991 (Table J-3). Table J-1 presents the estimated footing stiffnesses for liquefied and not-liquefied conditions.

Additional details on the methodology for developing the footing springs are presented in Appendix J, Table J-1 notes.

## *8.3. Footing Load-Displacement Curves*

The load-displacement curves for the generic strip footings were obtained from push over analyses in 2D FLAC as described in Section 8.2. These load-displacement curves were corrected approximately to consider the 3D effects and actual sizes of the inbound footings based on elastic solutions by Gazetas 1991. The corrected load-displacement curves for 5 degrees of freedom are presented on pages K-8 to K-18 in Appendix K. Torsional springs were not required by the structural design team. The digital data was provided in a zip file (*20-8543-Westshore-Load-Displacement-DATA-Bridging scope-2021-09-16-Rev0.zip*) that includes 11 Excel files, one for each inbound structure footing. Additional details and background information are provided in Appendix K.

### *8.4. Near-Surface Free Field Ground Motion Time Histories*

Time histories of ground surface free field displacements and accelerations are provided at the location of the inbound structures (Figure L-1) to allow structural time history analyses to be carried out. Time histories are provided for north-south, east-west, and vertical directions for six A2475 design ground motions (one crustal motion, two in-slab motions and three interface motions) selected by the structural design team as listed in Table L-1 in Appendix L.

The near surface ground motions time histories were developed using FLAC 2D by applying the horizontal and vertical design ground motions to firm ground at depth in the east-west and north-south FLAC models (Figures L-2 and L-3). Tables L-2 and L-3 present the list of the near surface ground motion data files for each structure. A total of 1056 text files and 1056 JPG graph files were provided to the structural design team. The digital data was provided in a zip file (*20-8543-Westshore-THA-DATA-Bridging scope-2021-09-16-Rev0.zip)*.

Additional information including background information, the methodology of analysis, method of correction for firm ground movement, an example of a typical set of provided time histories and uncertainties/limitations are presented in Appendix L.

## 9. CLOSURE

This assessment report is based on designs that are under development and is subject to review and modification as design progresses or new information becomes available.

This report is prepared for the exclusive use of Westshore Terminals and their designated representatives and design team, and may not be used by other parties without the written permission of Braun and NAGL. Information from a confidential report has been included within this report, and permission from the Client is required for distribution or disclosure of this report or portions of this report outside the design team.

If the development plans change, or if during construction subsurface conditions are noted to be different from those described in this report, Braun and NAGL should be notified immediately in order that the geotechnical recommendations can be confirmed or modified, if required. Further, this report assumes that field reviews will be completed by Braun and NAGL during construction.

The site contractor should make their own assessment of subsurface conditions and select the construction means and methods most appropriate to the site conditions.

The use of this report is subject to the attached Report Interpretations and Limitations sheet. The reader's attention is drawn specifically to those conditions, as it is considered essential that they be followed for proper use and interpretation of this report.

We hope the above meets with your requirements. Should any questions arise, please do not hesitate to contact the undersigned.

Yours truly,

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# REPORT INTERPRETATION AND LIMITATIONS

## 1. STANDARD OF CARE

Braun Geotechnical Ltd. (Braun) and Naesgaard Amini Geotechnical Ltd. (NAGL) have prepared this report in a manner consistent with generally accepted engineering consulting practices in this area, subject to the time and physical constraints applicable. No other warranty, expressed or implied, is made.

## 2. COMPLETENESS OF THIS REPORT

This Report represents a summary of paper, electronic and other documents, records, data and files and is not intended to stand alone without reference to the instructions given to Braun/NAGL by the Client, communications between Braun/NAGL and the Client, and/or to any other reports, writings, proposals or documents prepared by Braun/NAGL for the Client relating to the specific site described herein.

This report is intended to be used and quoted in its entirety. Any references to this report must include the whole of the report and any appendices or supporting material. Braun/NAGL cannot be responsible for use by any party of portions of this report without reference to the entire report.

## 3. BASIS OF THIS REPORT

This report has been prepared for the specific site, development, design objective, and purpose described to Braun/NAGL by the Client or the Client's Representatives or Consultants. The applicability and reliability of any of the factual data, findings, recommendations or opinions expressed in this document pertain to a specific project as described in this report and are not applicable to any other project or site, and are valid only to the extent that there has been no material alteration to or variation from any of the descriptions provided to Braun/NAGL. Braun/NAGL cannot be responsible for use of this report, or portions thereof, unless we were specifically requested by the Client to review and revise the Report in light of any alterations or variations to the project description provided by the Client.

If the project does not commence within 18 months of the report date, the report may become invalid and further review may be required.

The recommendations of this report should only be used for design. The extent of exploration including number of test pits or test holes necessary to thoroughly investigate the site for conditions that may affect construction costs will generally be greater than that required for design purposes. Contractors should rely upon their own explorations and interpretation of the factual data provided for costing purposes, equipment requirements, construction techniques, or to establish project schedule.

The information provided in this report is based on limited exploration, for a specific project scope. Braun/NAGL cannot accept responsibility for independent conclusions, interpretations, interpolations or decisions by the Client or others based on information contained in this Report. This restriction of liability includes decisions made to purchase or sell land.

## 4. USE OF THIS REPORT

The contents of this report, including plans, data, drawings and all other documents including electronic and hard copies remain the copyright property of Braun/NAGL. However, we will consider any reasonable request by the Client to approve the use of this report by other parties as "Approved Users." With regard to the duplication and distribution of this Report or its contents, we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of this Report by those parties. The Client and "Approved Users" may not give, lend, sell or otherwise make this Report or any portion thereof available to any other party without express written permission from Braun/NAGL. Any use which a third party makes of this Report – in its entirety or portions thereof – is the sole responsibility of such third parties. BRAUN/NAGL ACCEPTS NO RESPONSIBILITY FOR DAMAGES SUFFERED BY ANY PARTY RESULTING FROM THE UNAUTHORIZED USE OF THIS REPORT.

Electronic media is susceptible to unauthorized modification or unintended alteration, and the Client should not rely on electronic versions of reports or other documents. All documents should be obtained directly from Braun/NAGL.

## 5. INTERPRETATION OF THIS REPORT

Classification and identification of soils and rock and other geological units, including groundwater conditions have been based on exploration(s) performed in accordance with the standards set out in Paragraph 1. These tasks are judgemental in nature; despite comprehensive sampling and testing programs properly performed by experienced personnel with the appropriate equipment, some conditions may elude detection. As such, all explorations involve an inherent risk that some conditions will not be detected.

Further, all documents or records summarizing such exploration will be based on assumptions of what exists between the actual points sampled at the time of the site exploration. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of and accept this risk.

The Client and “Approved Users” accept that subsurface conditions may change with time and this report only represents the soil conditions encountered at the time of exploration and/or review. Soil and ground water conditions may change due to construction activity on the site or on adjacent sites, and also from other causes, including climactic conditions.
The exploration and review provided in this report were for geotechnical purposes only. Environmental aspects of soil and groundwater have not been included in the exploration or review, or addressed in any other way.
The exploration and Report is based on information provided by the Client or the Client’s Consultants, and conditions observed at the time of our site reconnaissance or exploration. Braun/NAGL have relied in good faith upon all information provided. Accordingly, Braun/NAGL cannot accept responsibility for inaccuracies, misstatements, omissions, or deficiencies in this Report resulting from misstatements, omissions, misrepresentations or fraudulent acts of persons or sources providing this information.

## 6. DESIGN AND CONSTRUCTION REVIEW

This report assumes that Braun/NAGL will be retained to work and coordinate design and construction with other Design Professionals and the Contractor. Further, it is assumed that Braun/NAGL will be retained to provide field reviews during construction to confirm adherence to building code guidelines and generally accepted engineering practices, and the recommendations provided in this report. Field services recommended for the project represent the minimum necessary to confirm that the work is being carried out in general conformance with Braun/NAGL’s recommendations and generally accepted engineering standards. It is the Client’s or the Client’s Contractor’s responsibility to provide timely notice to Braun/NAGL to carry out site reviews. The Client acknowledges that unsatisfactory or unsafe conditions may be missed by intermittent site reviews by Braun/NAGL. Accordingly, it is the Client’s or Client’s Contractor’s responsibility to inform Braun/NAGL of any such conditions.
Work that is covered prior to review by Braun/NAGL may have to be re-exposed at considerable cost to the Client. Review of all Geotechnical aspects of the project are required for submittal of unconditional Letters of Assurance to regulatory authorities. The site reviews are not carried out for the benefit of the Contractor(s) and therefore do not in any way effect the Contractor(s) obligations to perform under the terms of his/her Contract.

## 7. SAMPLE DISPOSAL

Braun/NAGL will dispose of all samples 3 months after issuance of this report, or after a longer period of time at the Client’s expense if requested by the Client. All contaminated samples remain the property of the Client and it will be the Client’s responsibility to dispose of them properly.

## 8. SUBCONSULTANTS AND CONTRACTORS

Engineering studies frequently require hiring the services of individuals and companies with special expertise and/or services which Braun/NAGL do not provide. These services are arranged as a convenience to our Clients, for the Client’s benefit. Accordingly, the Client agrees to hold the Company harmless and to indemnify and defend Braun/NAGL from and against all claims arising through such Subconsultants or Contractors as though the Client had retained those services directly. This includes responsibility for payment of services rendered and the pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. These conditions apply to specialized subconsultants and the use of drilling, excavation and laboratory testing services, and any other Subconsultant or Contractor.

## 9. SITE SAFETY

Braun/NAGL assumes responsibility for site safety solely for the activities of our employees on the jobsite. The Client or any Contractors on the site will be responsible for their own personnel. The Client or his representatives, Contractors or others retain control of the site. It is the Client’s or the Client’s Contractors responsibility to inform Braun/NAGL of conditions pertaining to the safety and security of the site – hazardous or otherwise – of which the Client or Contractor is aware.
Exploration or construction activities could uncover previously unknown hazardous conditions, materials, or substances that may result in the necessity to undertake emergency procedures to protect workers, the public or the environment. Additional work may be required that is outside of any previously established budget(s). The Client agrees to reimburse Braun/NAGL for fees and expenses resulting from such discoveries. The Client acknowledges that some discoveries require that certain regulatory bodies be informed. The Client agrees that notification to such bodies by Braun/NAGL will not be a cause for either action or dispute.


BASE IMAGE: PROPOSED BOREHOLE LOCATIONS, EXISTING SITE GENERAL ARRANGEMENT PLAN, PROJECT 19899 BY CWA ENGINEERS, DATED: 2020-03-26

BRAUN GEOTECHNICAL LTD.

Naesgaard-Amini Geotechnical Ltd. soils, foundations and earthquake engineering

| Rev. | Description | Date |
|---|---|---|
| 1 | Added Test Holes on East Side | June 25, 2021 |

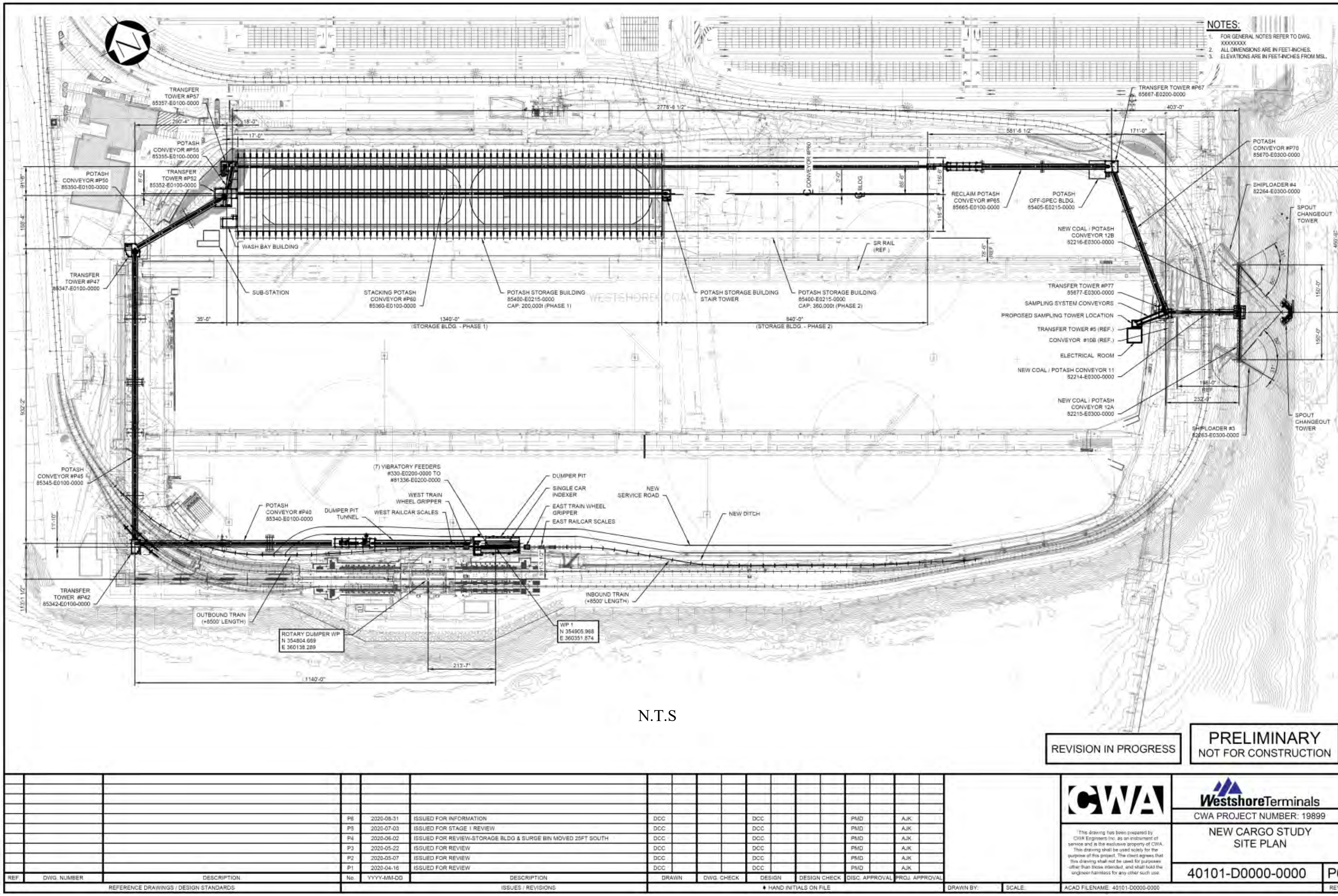
Client: Westshore Terminals

Project: Terminal Upgrades, Delta, BC

Title: LOCATION PLAN

| Project no. | Drawn | Design | Checked | Date | Scale | Drawing no. |
|---|---|---|---|---|---|---|
| 20-8543 | CD | PB | SS | November 5, 2020 | 1:3500 | FIGURE 2-1 |

Figure 2-2: New Cargo Project General Arrangement




N.T.S

Figure 5-1: Sand fill grain size distribution of select samples

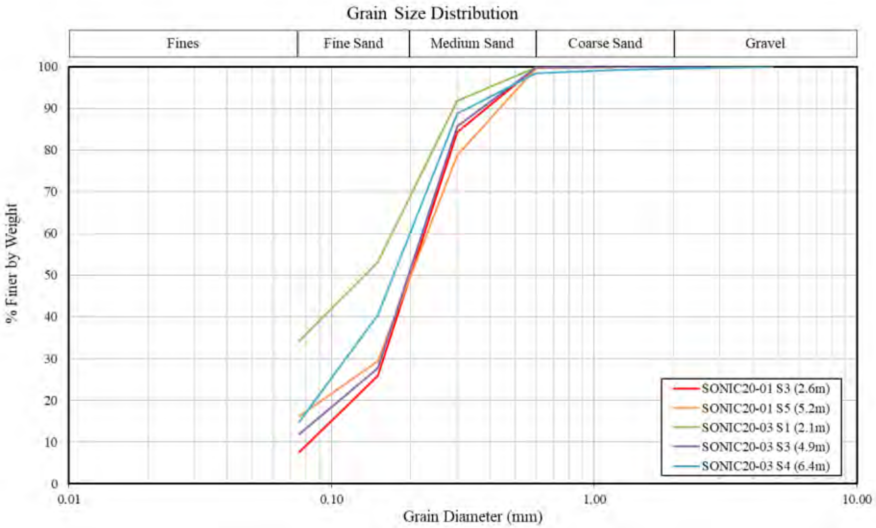



Figure 5-2: Sand grain size distribution of select samples

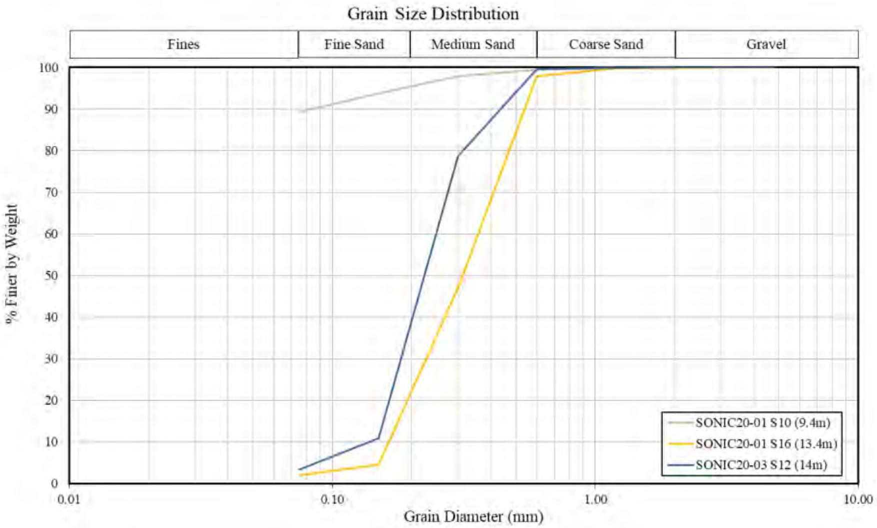



Figure 5-3: Grain size distribution of sand & silt layer of selected samples

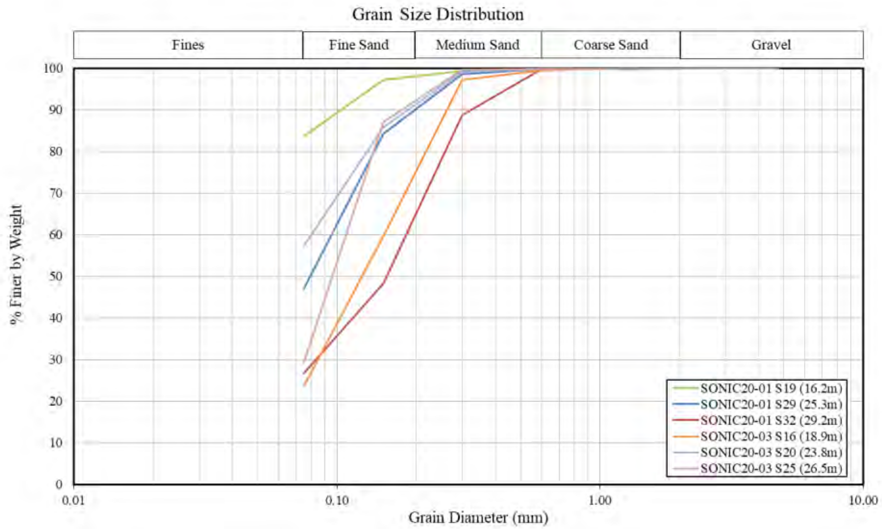


Figure 5-4: Shear wave velocity data

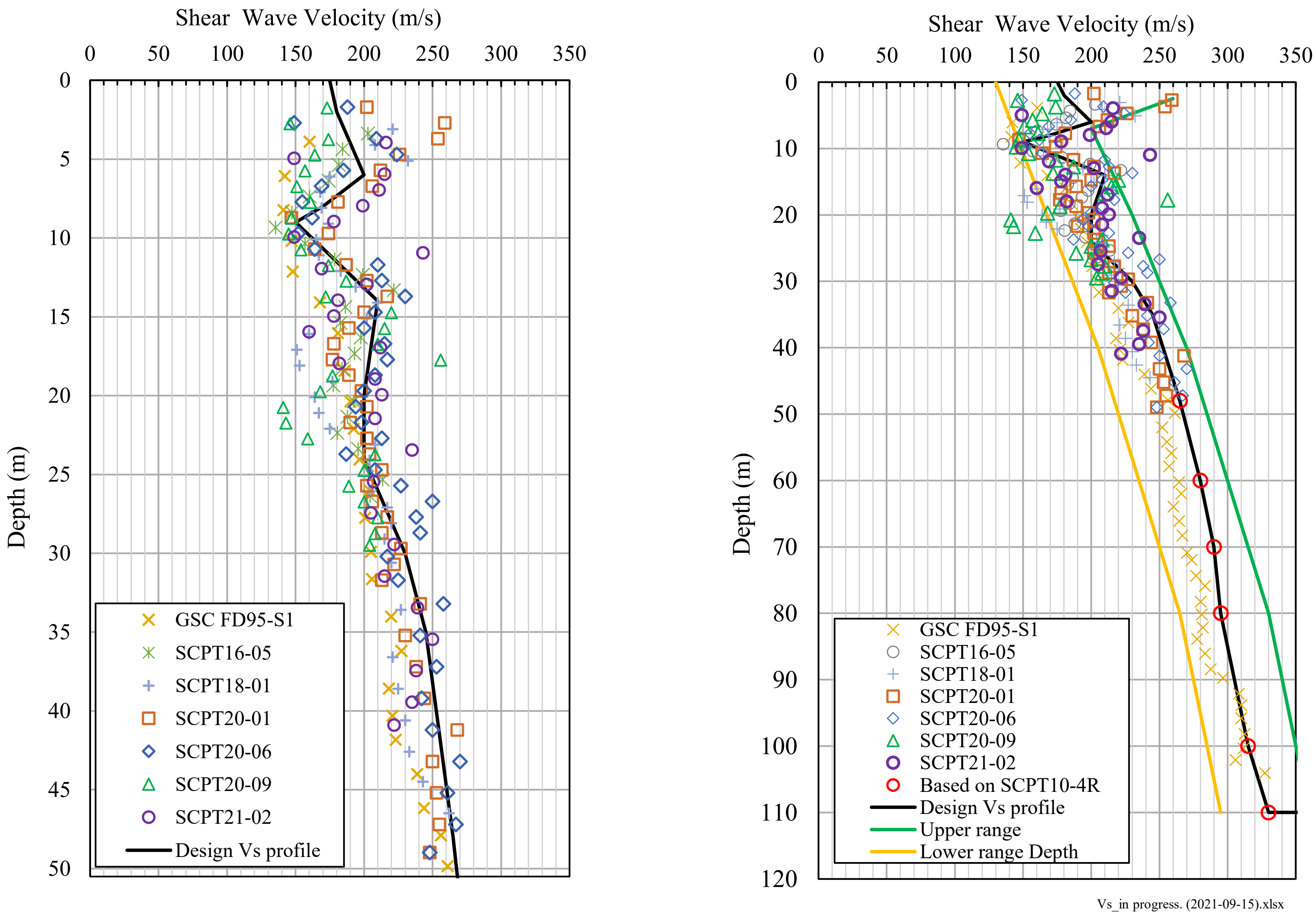


Figure 6-1: Shear wave velocity profile with upper and lower ranges
[Golder (2011) data from confidential report removed from figure]

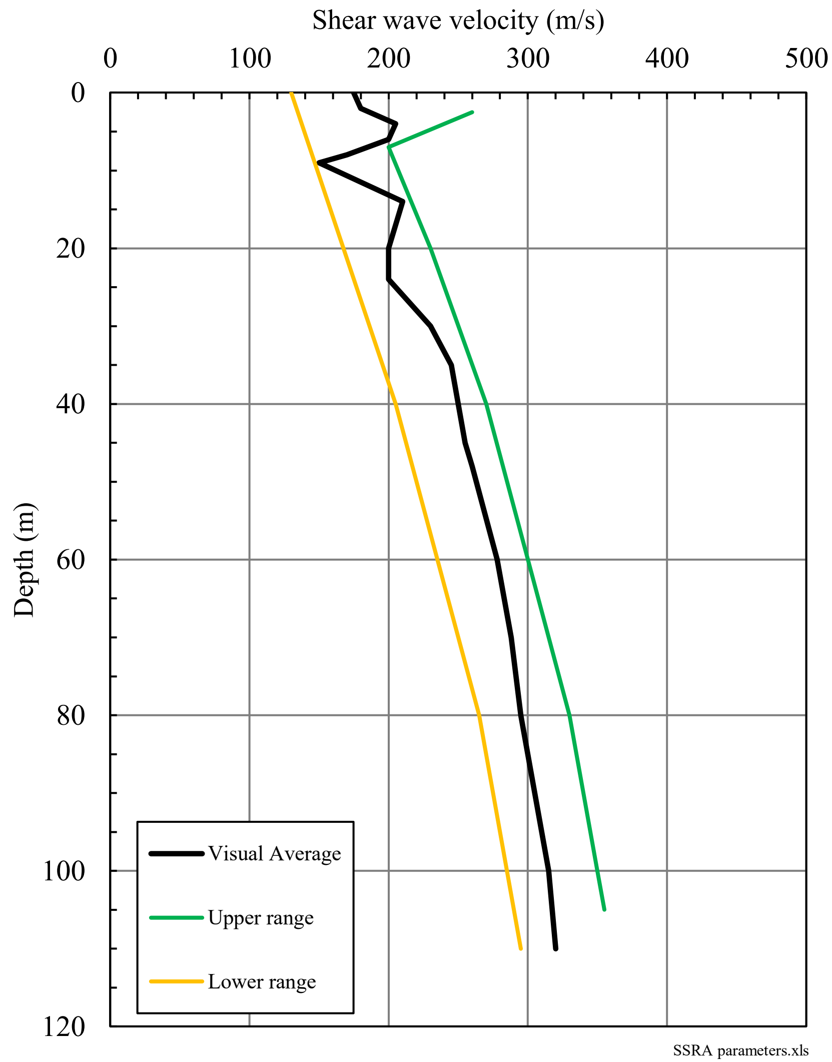
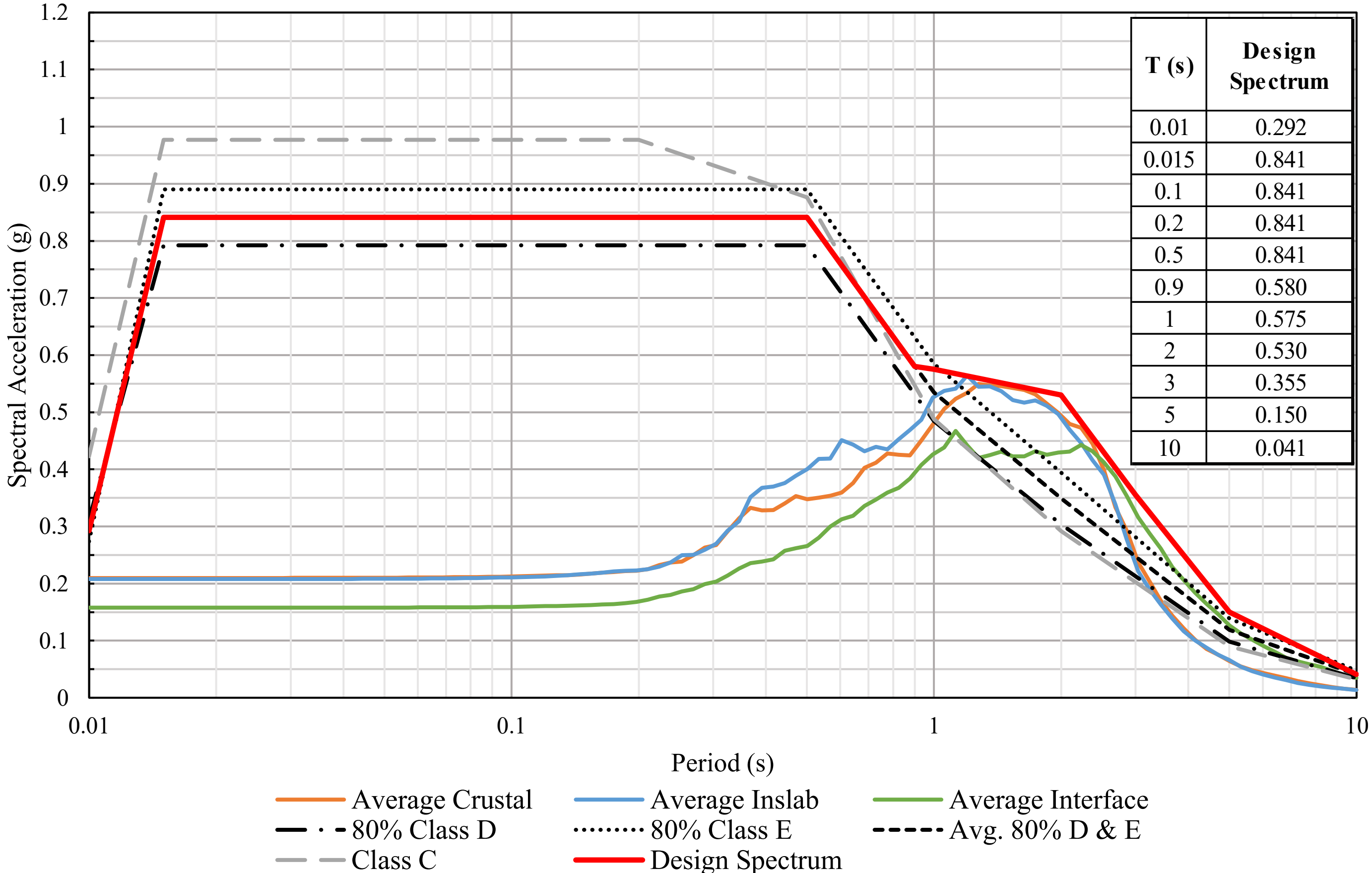


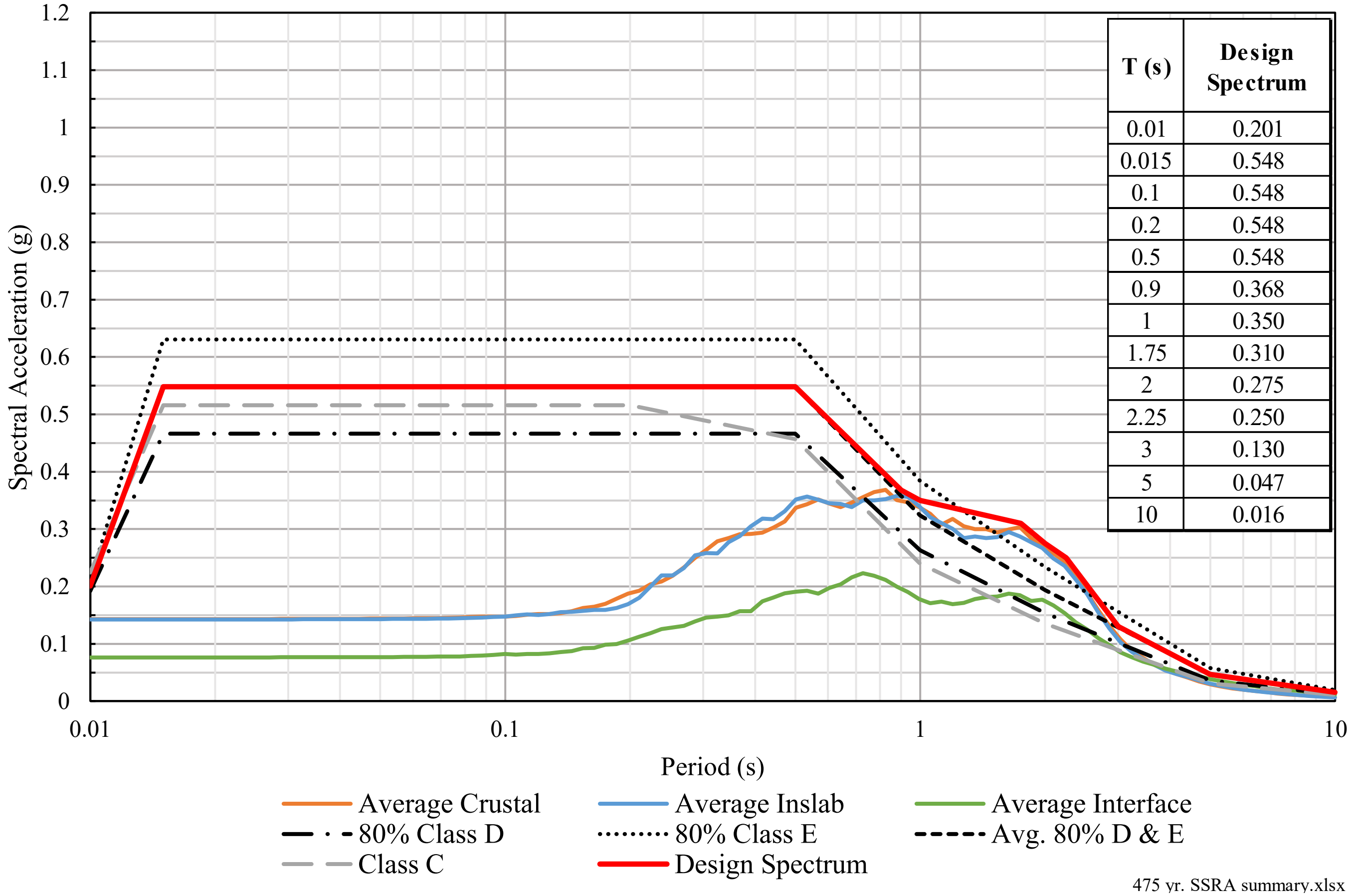
Figure 6-2: Design response spectrum near ground surface - A2475 earthquake - 5% damping




| T (s) | Design Spectrum |
|---|---|
| 0.01 | 0.292 |
| 0.015 | 0.841 |
| 0.1 | 0.841 |
| 0.2 | 0.841 |
| 0.5 | 0.841 |
| 0.9 | 0.580 |
| 1 | 0.575 |
| 2 | 0.530 |
| 3 | 0.355 |
| 5 | 0.150 |
| 10 | 0.041 |

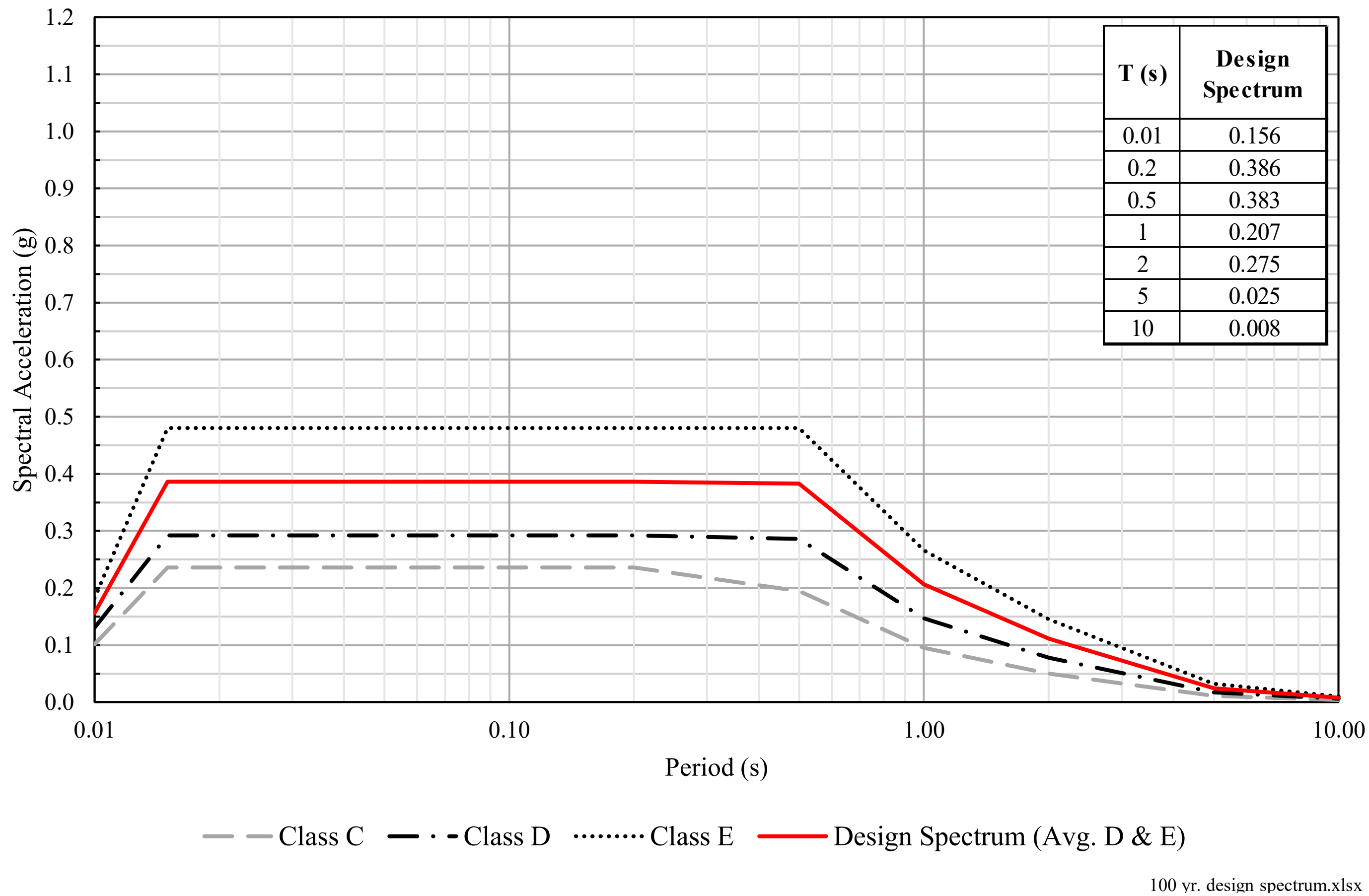
2475 yr. SSRA summary.xlsx

Figure 6-3: Design response spectrum near ground surface - A475 earthquake - 5% damping




| T (s) | Design Spectrum |
|---|---|
| 0.01 | 0.201 |
| 0.015 | 0.548 |
| 0.1 | 0.548 |
| 0.2 | 0.548 |
| 0.5 | 0.548 |
| 0.9 | 0.368 |
| 1 | 0.350 |
| 1.75 | 0.310 |
| 2 | 0.275 |
| 2.25 | 0.250 |
| 3 | 0.130 |
| 5 | 0.047 |
| 10 | 0.016 |

Figure 6-4: Design response spectrum near ground surface – A100 earthquake - 5% damping




| T (s) | Design Spectrum |
|---|---|
| 0.01 | 0.156 |
| 0.2 | 0.386 |
| 0.5 | 0.383 |
| 1 | 0.207 |
| 2 | 0.275 |
| 5 | 0.025 |
| 10 | 0.008 |

Figure 6-5: The Bray and Sancio criteria liquefaction susceptibility of fine-grained soils

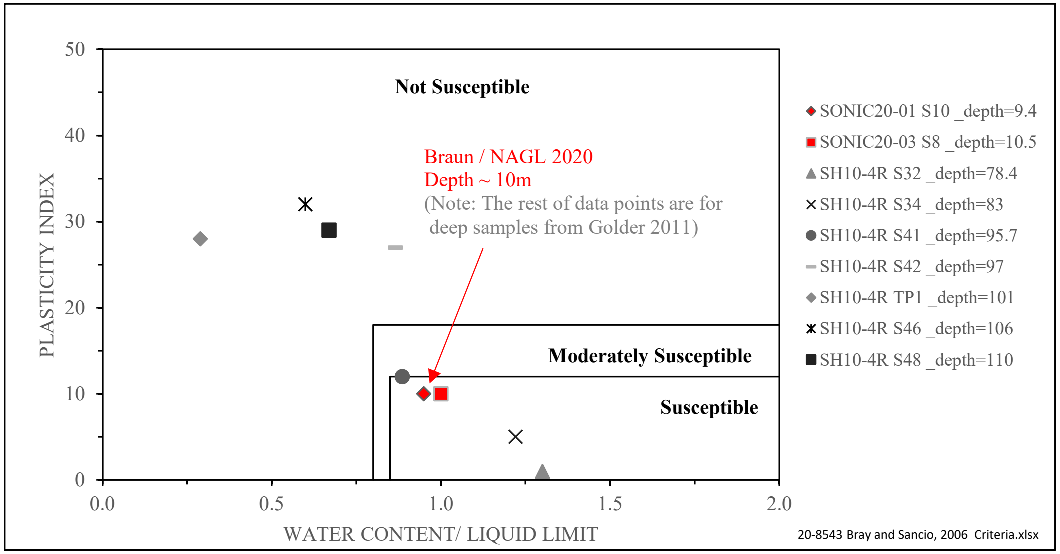


Figure 6-6: CSR profile obtained from site response analysis for A2475 and A475 earthquakes

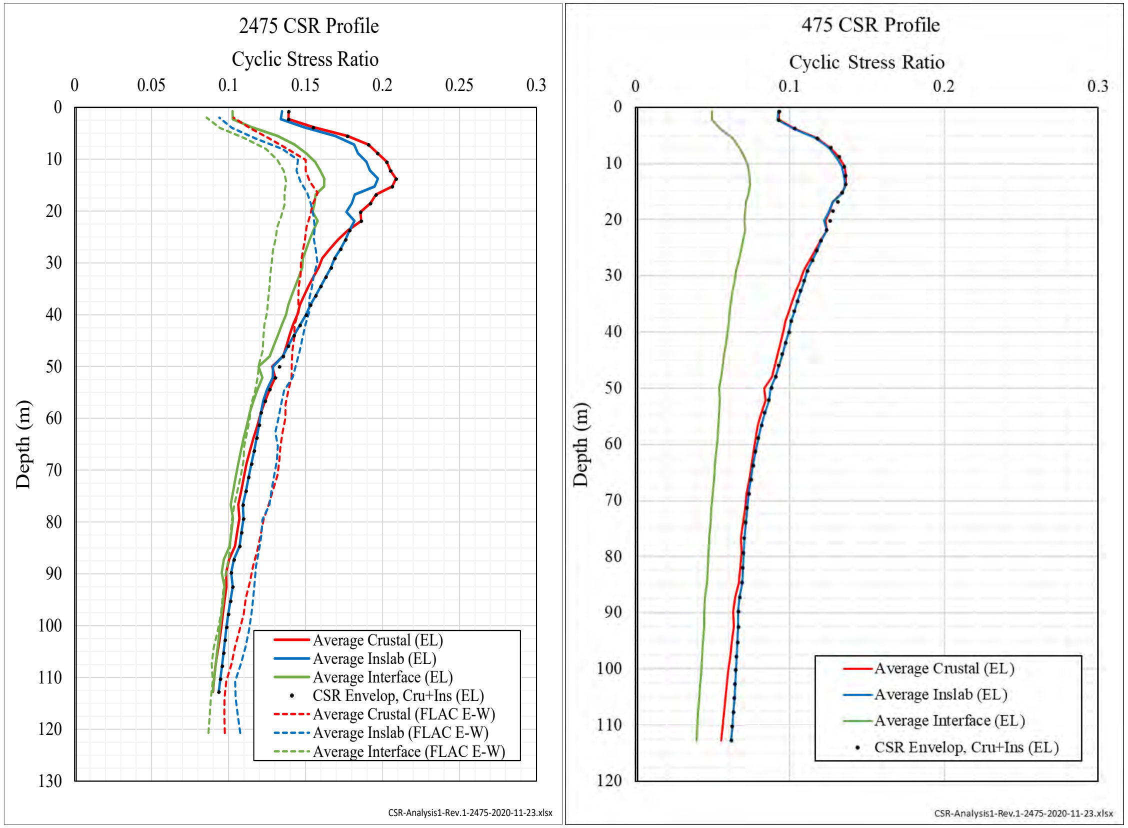
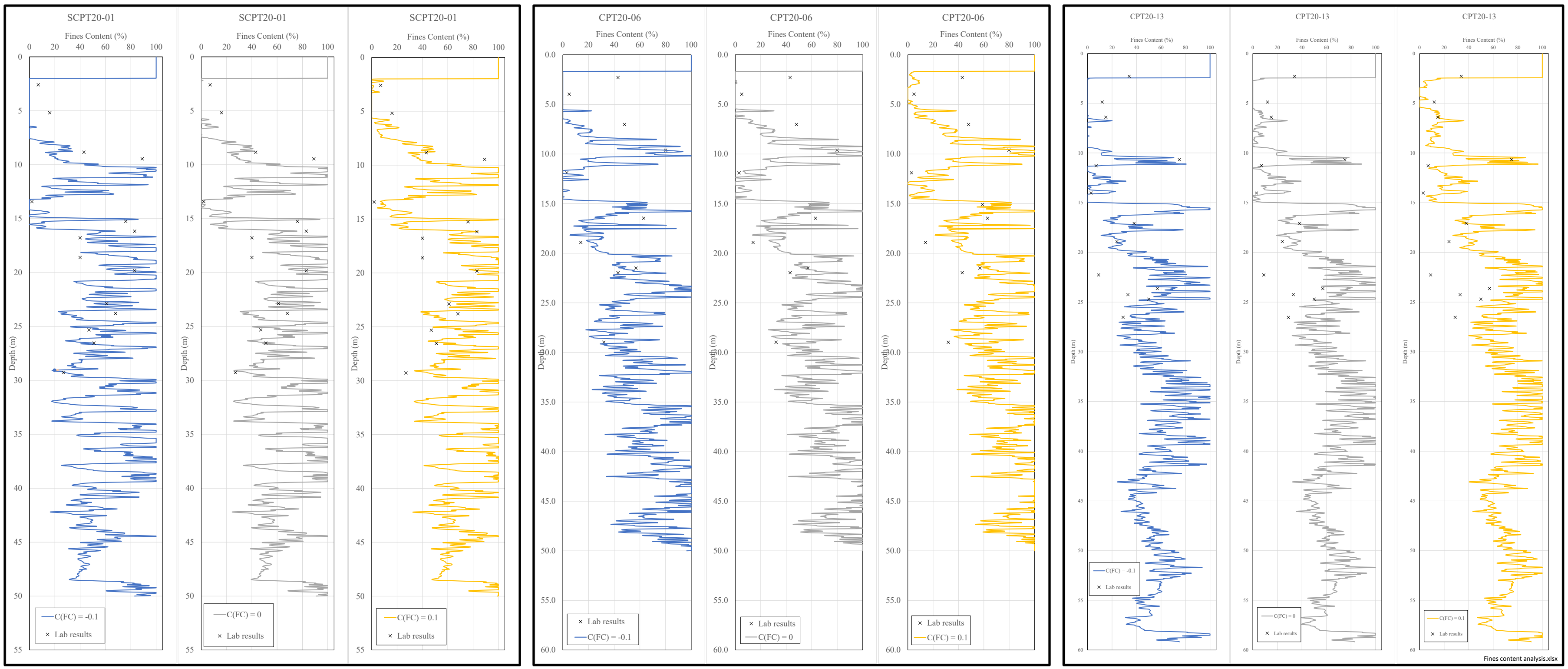


Figure 6-7: Comparison of the laboratory fines content data with the Boulanger and Idriss (2014) CPT correlation

SCPT20-01/SH20-01 SCPT20-06/SH20-02 SCPT20-13/SH20-03




$C_{FC}$= -0.1 $C_{FC}$= 0 $C_{FC}$= 0.1 $C_{FC}$= -0.1 $C_{FC}$= 0 $C_{FC}$= 0.1 $C_{FC}$= -0.1 $C_{FC}$= 0 $C_{FC}$= 0.1

Figure 6-8: Example of liquefaction triggering assessment
SCPT20-06- A2475 Crustal/Inslab

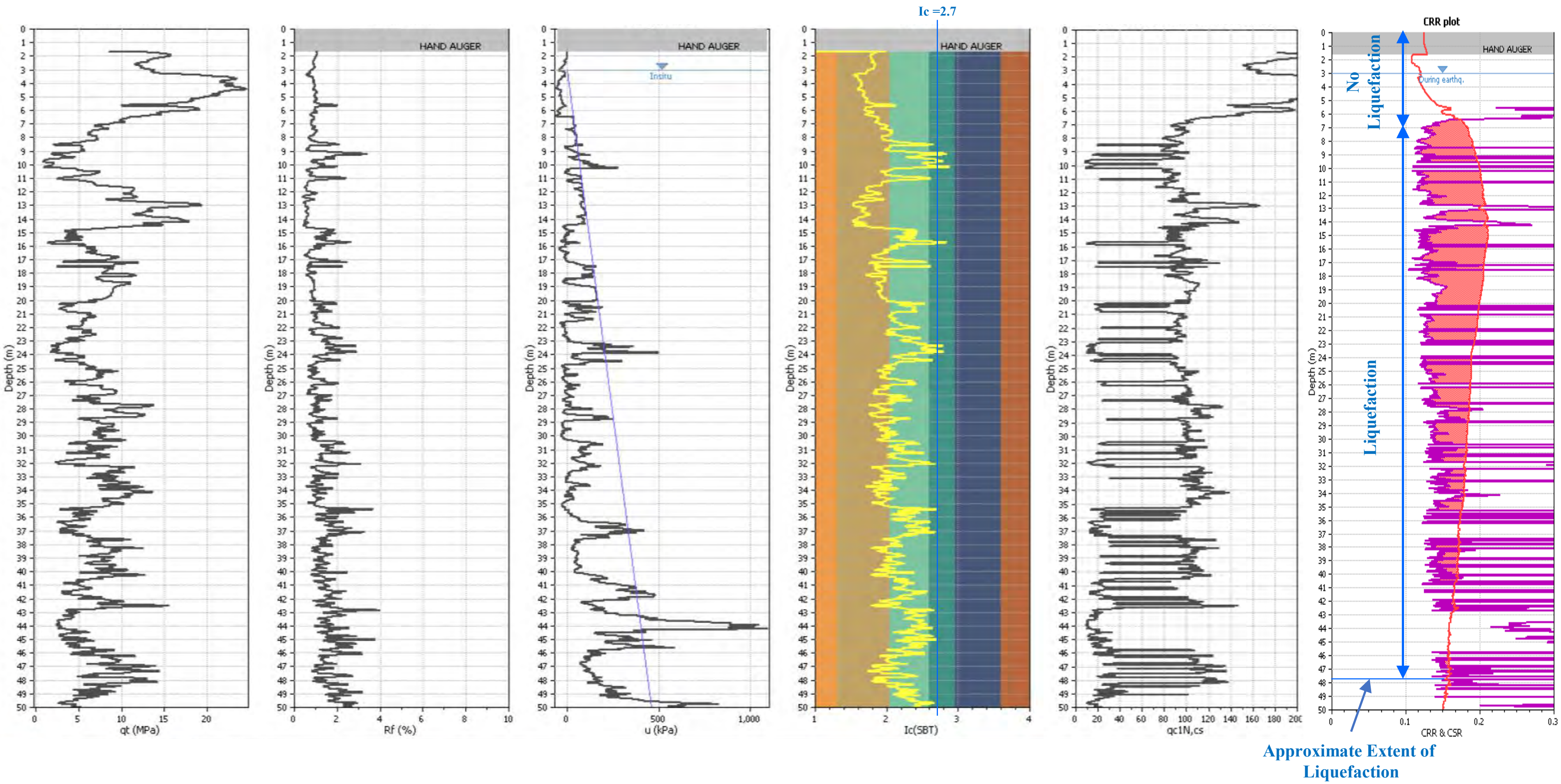
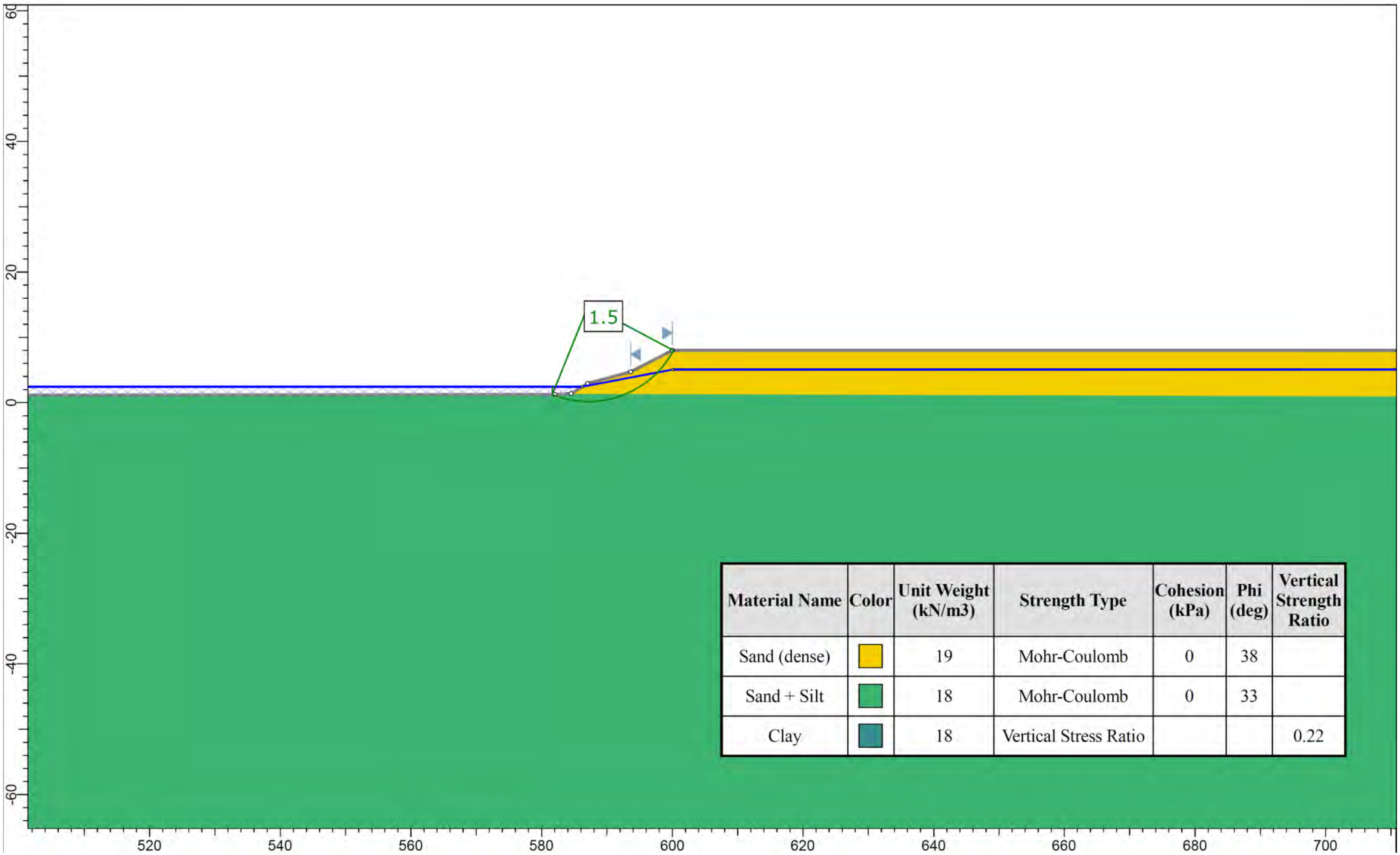
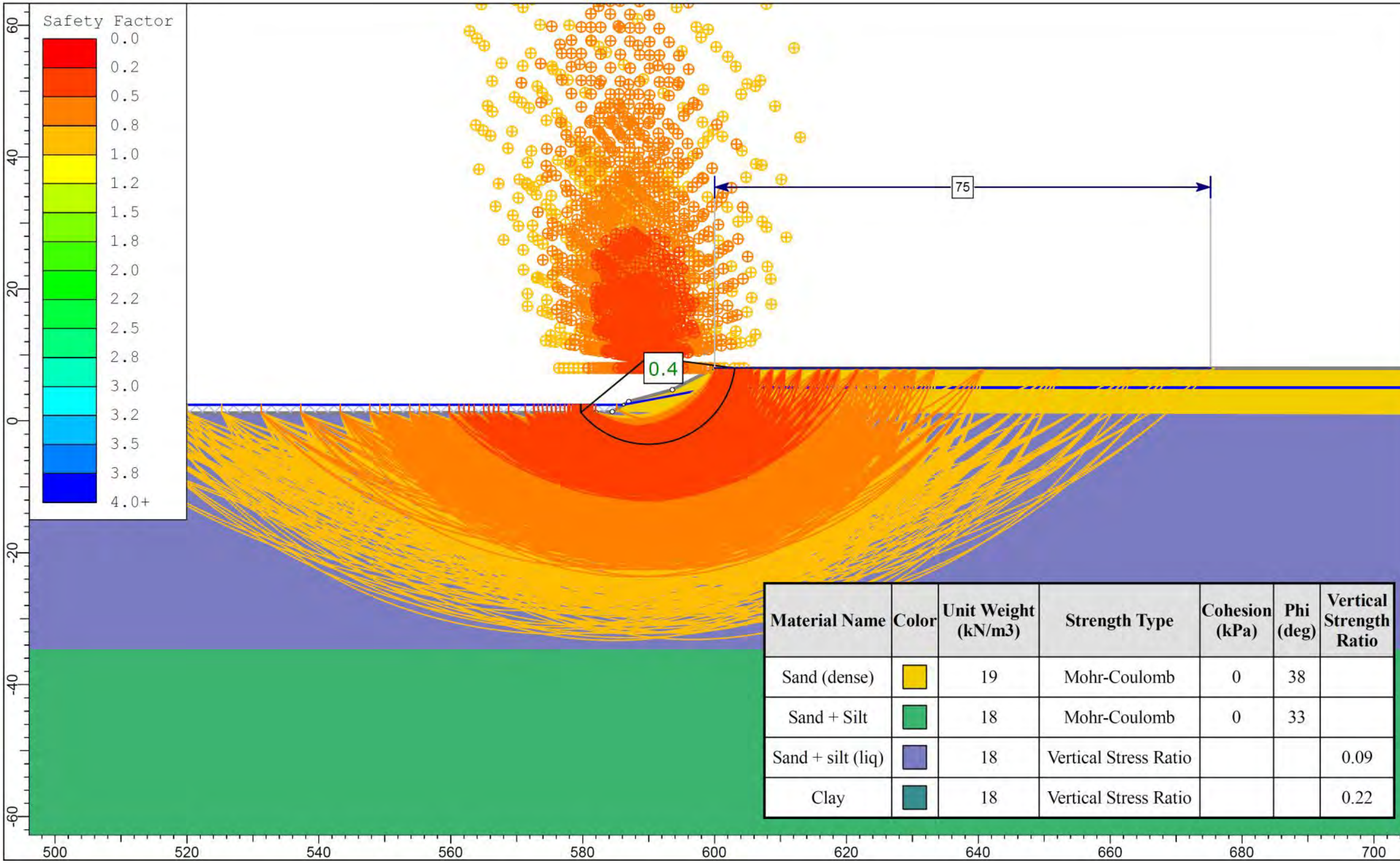


Figure 6-9: West slope stability analysis - Static condition



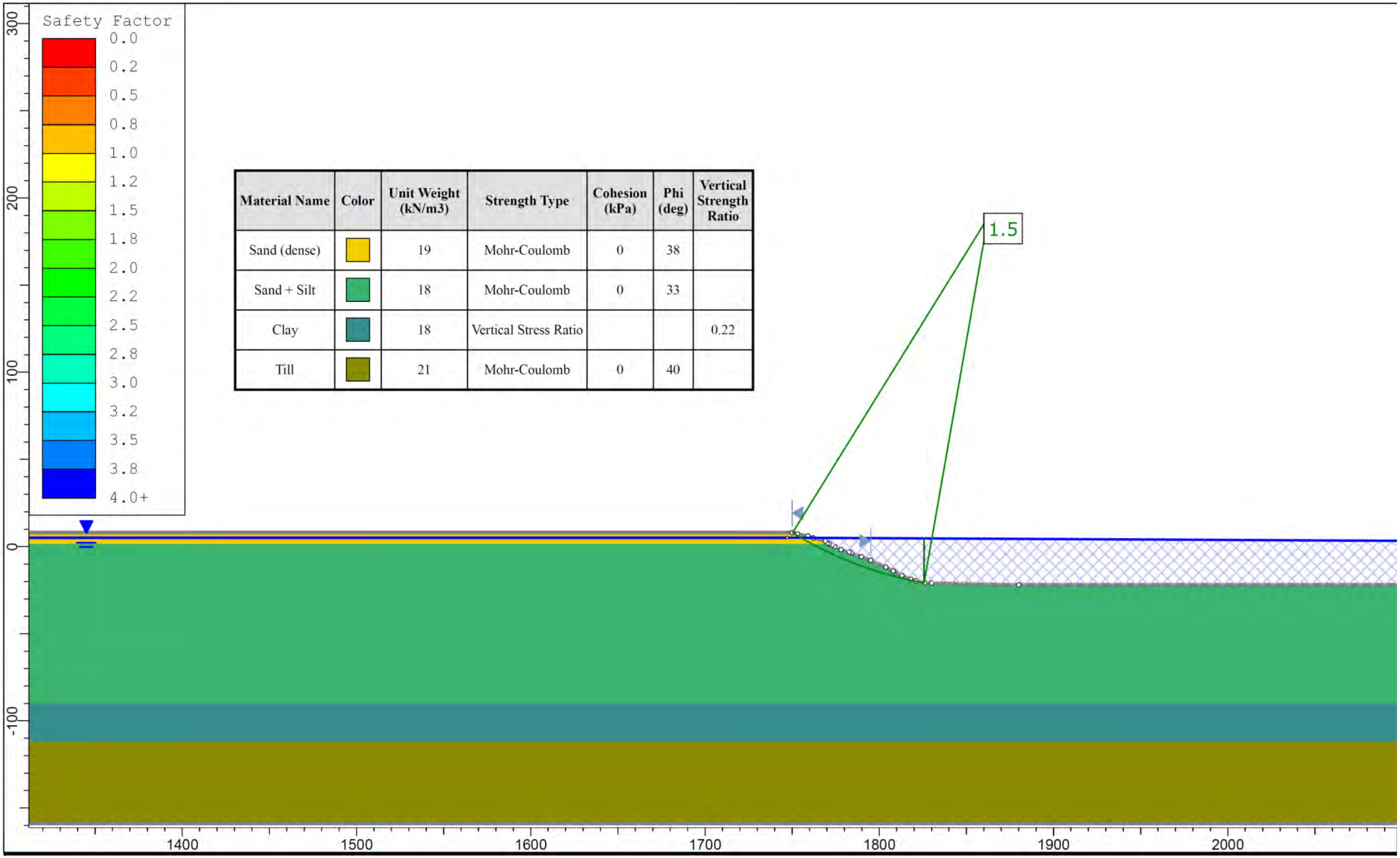

| Material Name | Color | Unit Weight (kN/m3) | Strength Type | Cohesion (kPa) | Phi (deg) | Vertical Strength Ratio |
|---|---|---|---|---|---|---|
| Sand (dense) | | 19 | Mohr-Coulomb | 0 | 38 | |
| Sand + Silt | | 18 | Mohr-Coulomb | 0 | 33 | |
| Clay | | 18 | Vertical Stress Ratio | | | 0.22 |

Figure 6-10: West slope stability analysis - Post-liquefaction (residual) condition- A2475



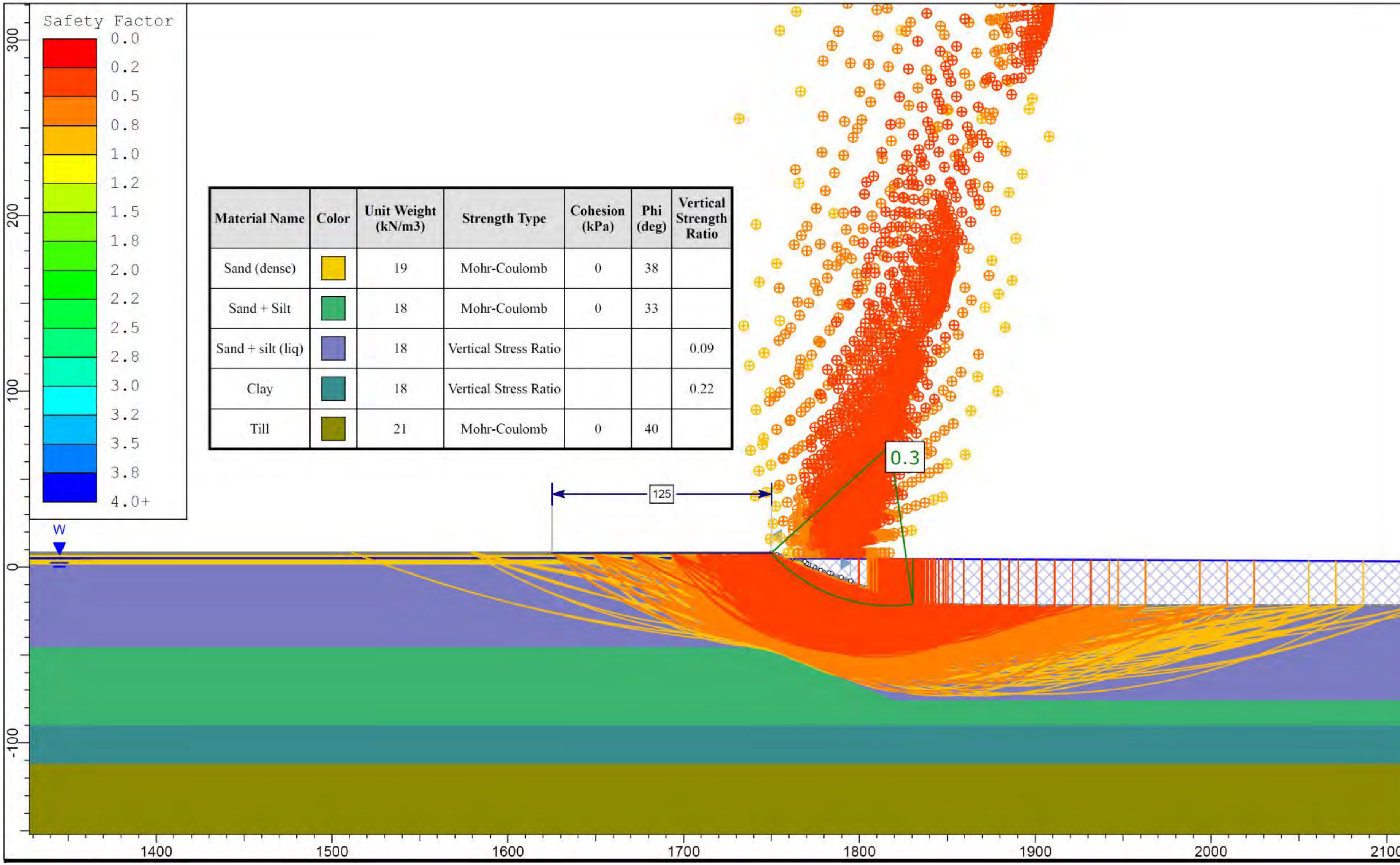

| Material Name | Color | Unit Weight (kN/m3) | Strength Type | Cohesion (kPa) | Phi (deg) | Vertical Strength Ratio |
|---|---|---|---|---|---|---|
| Sand (dense) | | 19 | Mohr-Coulomb | 0 | 38 | |
| Sand + Silt | | 18 | Mohr-Coulomb | 0 | 33 | |
| Sand + silt (liq) | | 18 | Vertical Stress Ratio | | | 0.09 |
| Clay | | 18 | Vertical Stress Ratio | | | 0.22 |

Figure 6-11: East slope stability analysis – Static condition




| Material Name | Color | Unit Weight (kN/m3) | Strength Type | Cohesion (kPa) | Phi (deg) | Vertical Strength Ratio |
|---|---|---|---|---|---|---|
| Sand (dense) | | 19 | Mohr-Coulomb | 0 | 38 | |
| Sand + Silt | | 18 | Mohr-Coulomb | 0 | 33 | |
| Clay | | 18 | Vertical Stress Ratio | | | 0.22 |
| Till | | 21 | Mohr-Coulomb | 0 | 40 | |

Figure 6-12: East slope stability analysis - Post-liquefaction (residual) condition - A2475




| Material Name | Color | Unit Weight (kN/m3) | Strength Type | Cohesion (kPa) | Phi (deg) | Vertical Strength Ratio |
|---|---|---|---|---|---|---|
| Sand (dense) | | 19 | Mohr-Coulomb | 0 | 38 | |
| Sand + Silt | | 18 | Mohr-Coulomb | 0 | 33 | |
| Sand + silt (liq) | | 18 | Vertical Stress Ratio | | | 0.09 |
| Clay | | 18 | Vertical Stress Ratio | | | 0.22 |
| Till | | 21 | Mohr-Coulomb | 0 | 40 | |

Figure 6-13: Zhang et al. (2004) empirical method for estimation of horizontal displacement
Slope Height=10m

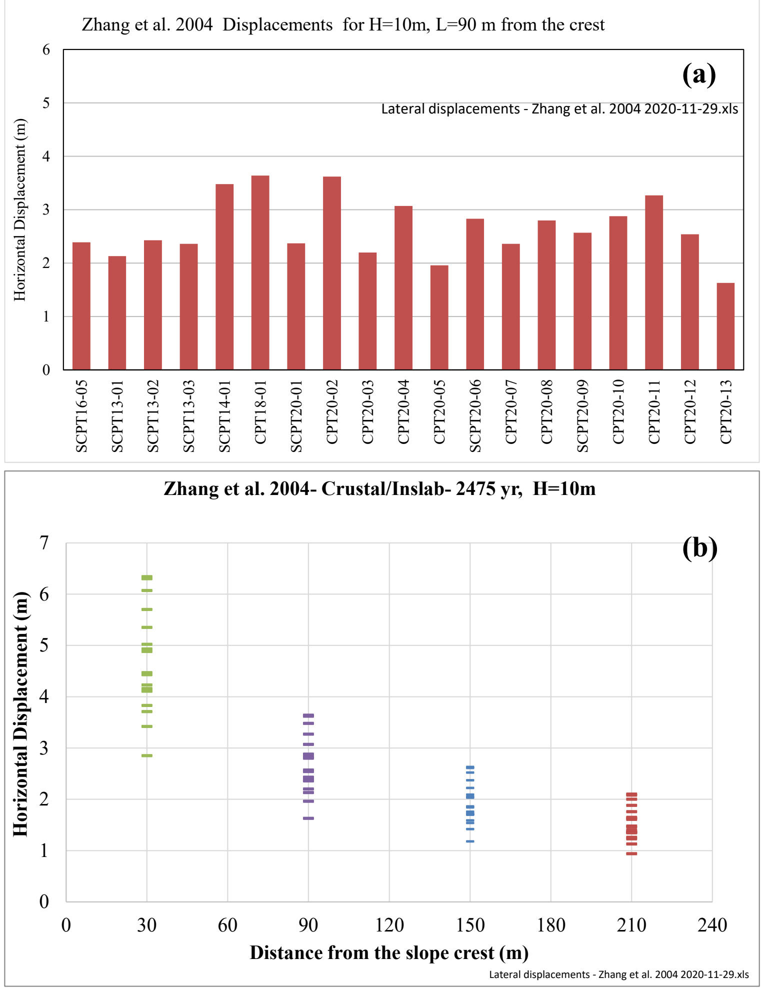


Figure 6-14: Location of the FLAC sections
*(Ref: Base image is modified from Google Earth )*

Figure 6-15: Location of FLAC sections and the CPT/test holes used for each section

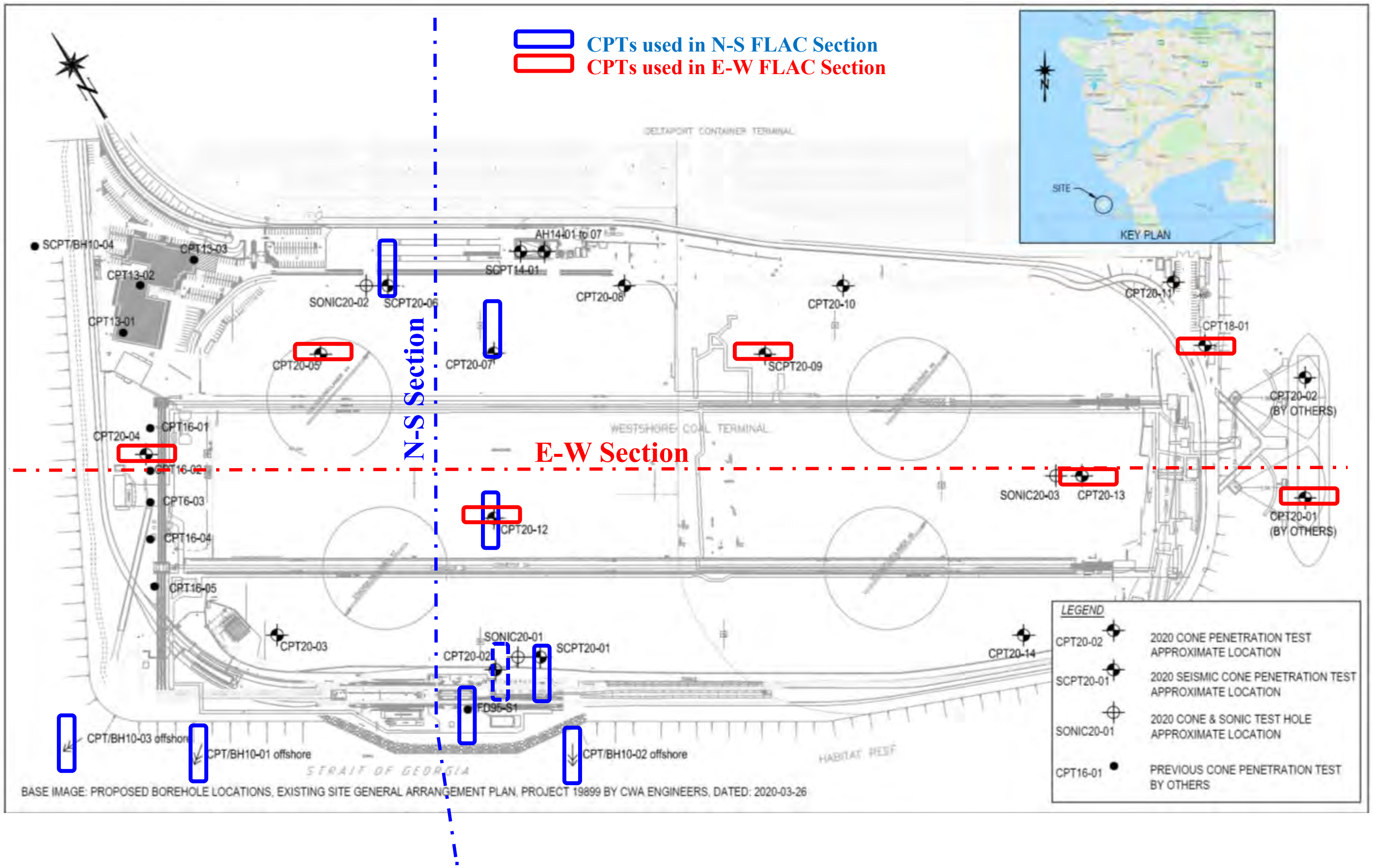


Figure 6-16: East-West FLAC section and soil units

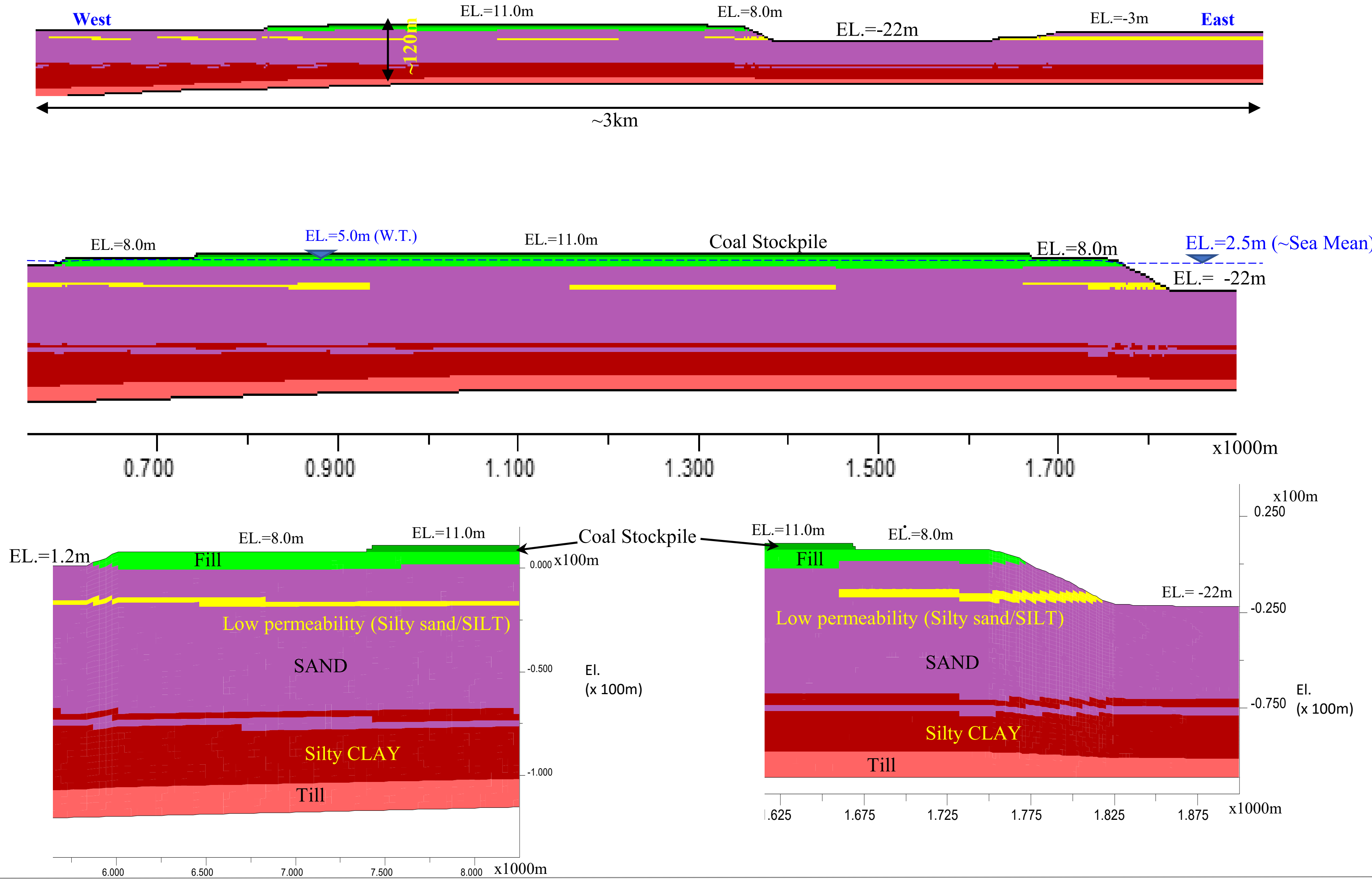


Figure 6-17: North-South FLAC section and soil units
(Note: Silty sand /Silt layers were modeled as sand layers in the design cases)

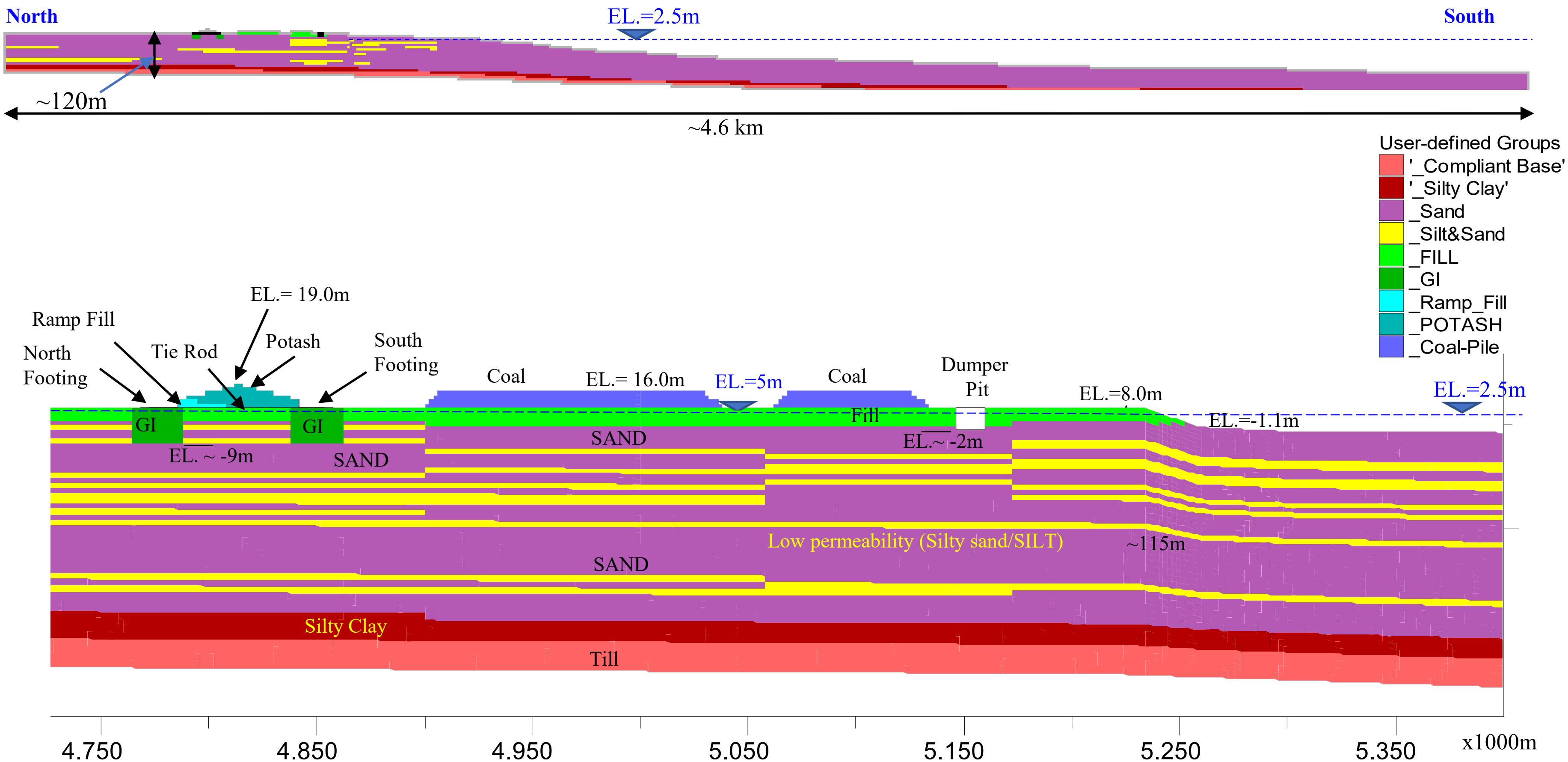


Figure 6-18: Horizontal displacements at the end of earthquake for the E-W Section, all A2475 ground motions (Note: the results are not baseline corrected). Positive displacements indicate movements to east. C=Crustal, I=Inslab, S=Interface or Subduction

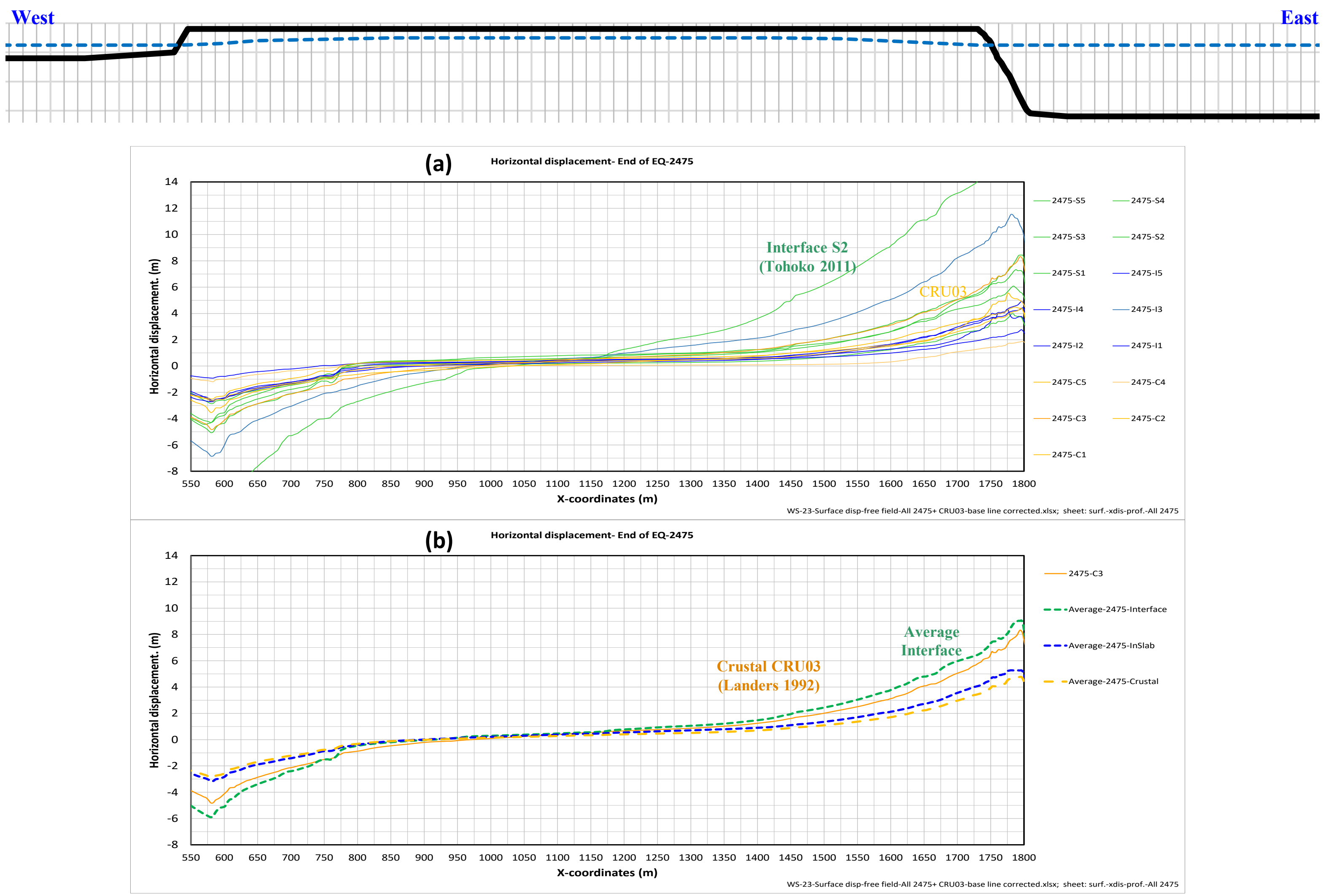


Figure 6-19: Interpreted liquefaction depth from the FLAC model with coal stockpile and structure CRU03-2475

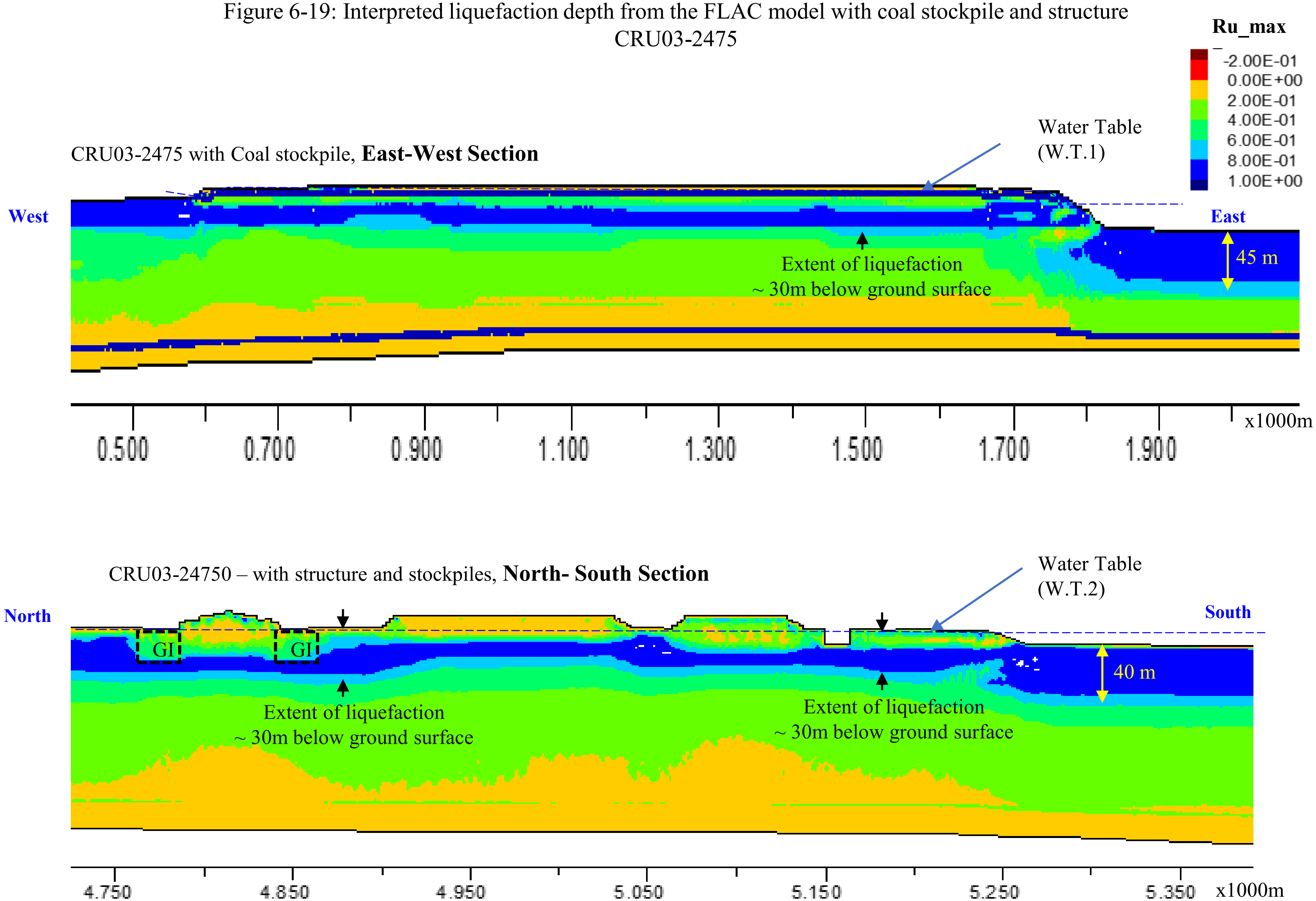


Figure 6-20: Predicted liquefaction depth in FLAC model. East-West Section
No coal stockpile and water table W.T.1
CRU03, A200, A475, A975 & A2475

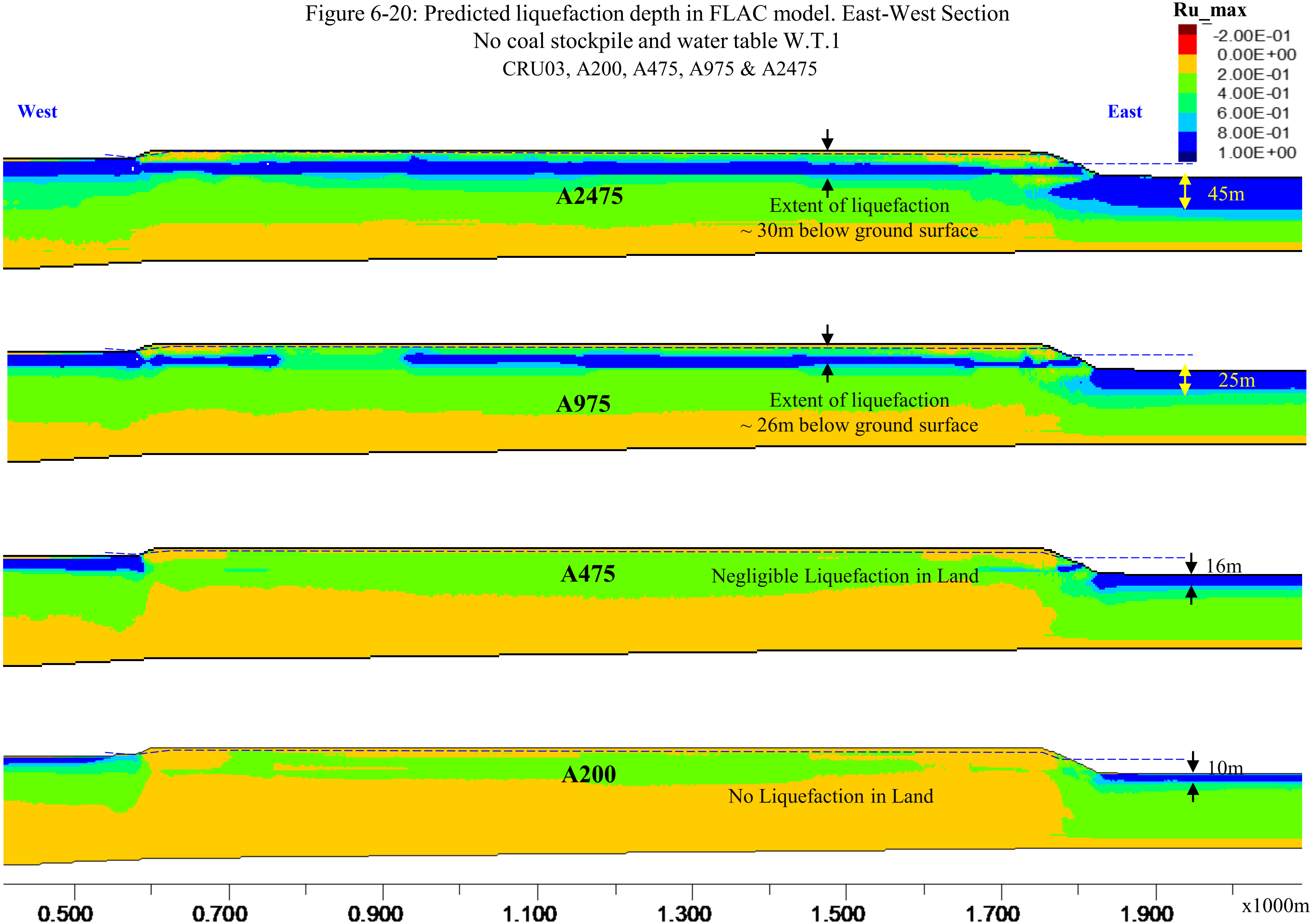
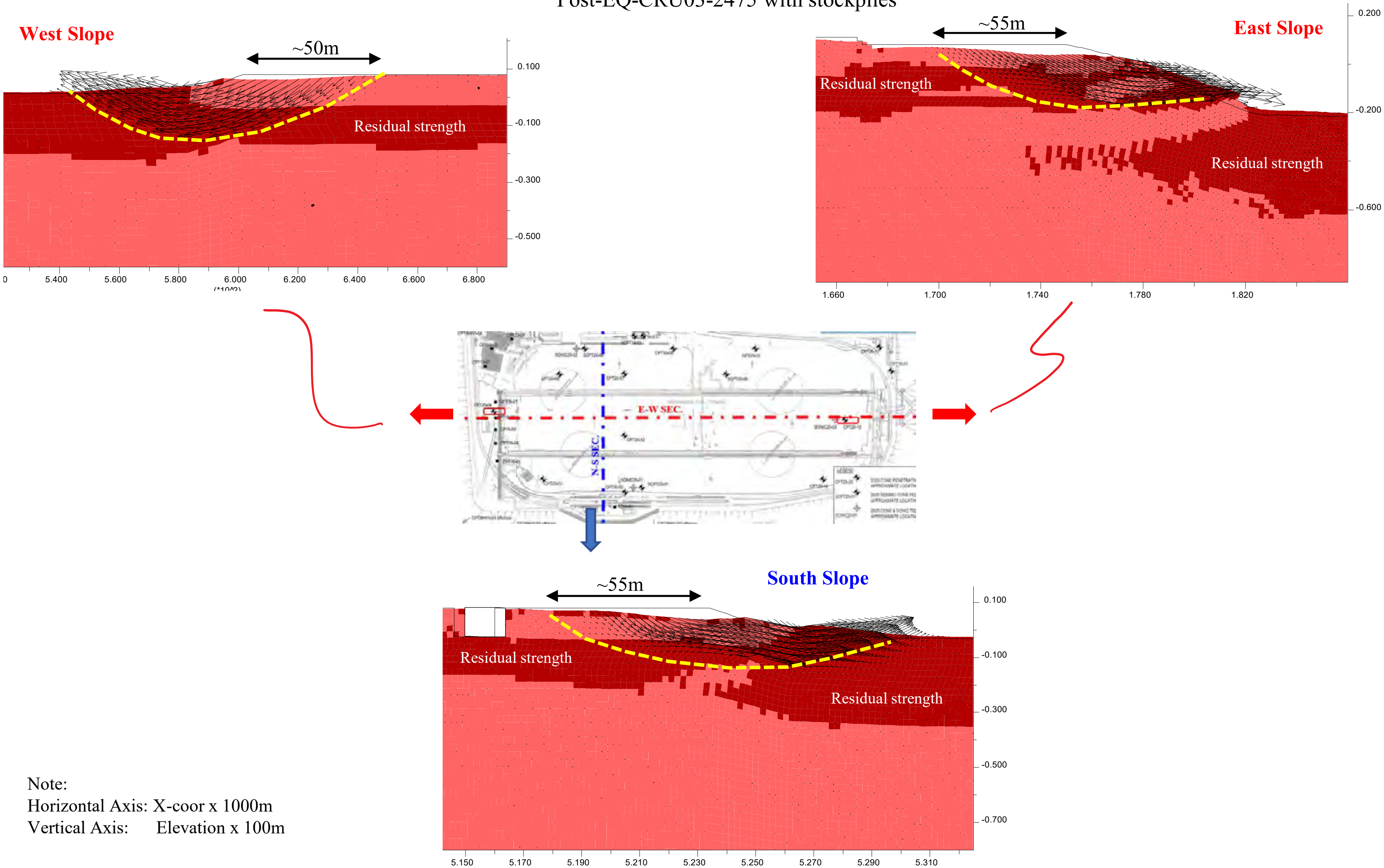


Figure 6-21: Flow slide failure patterns
Post-EQ-CRU03-2475 with stockpiles




Note:
Horizontal Axis: X-coor x 1000m
Vertical Axis: Elevation x 100m

Figure 6-22: Horizontal & vertical displacement contours- East-West Section
CRU03-2475 with coal stockpile

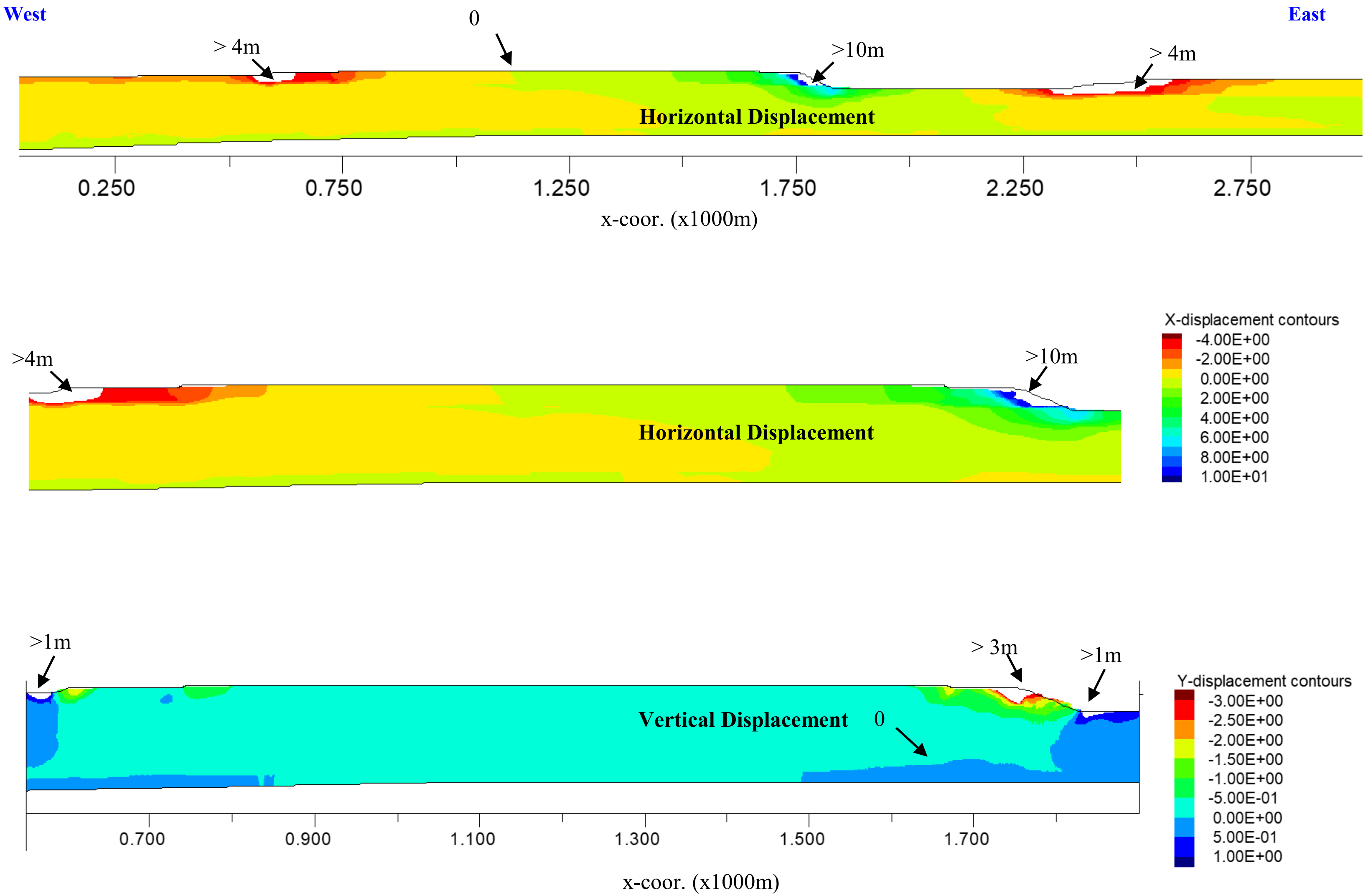


Figure 6-23: Horizontal & vertical displacement contours- North-South Section
CRU03-2475 with structure and coal stockpile

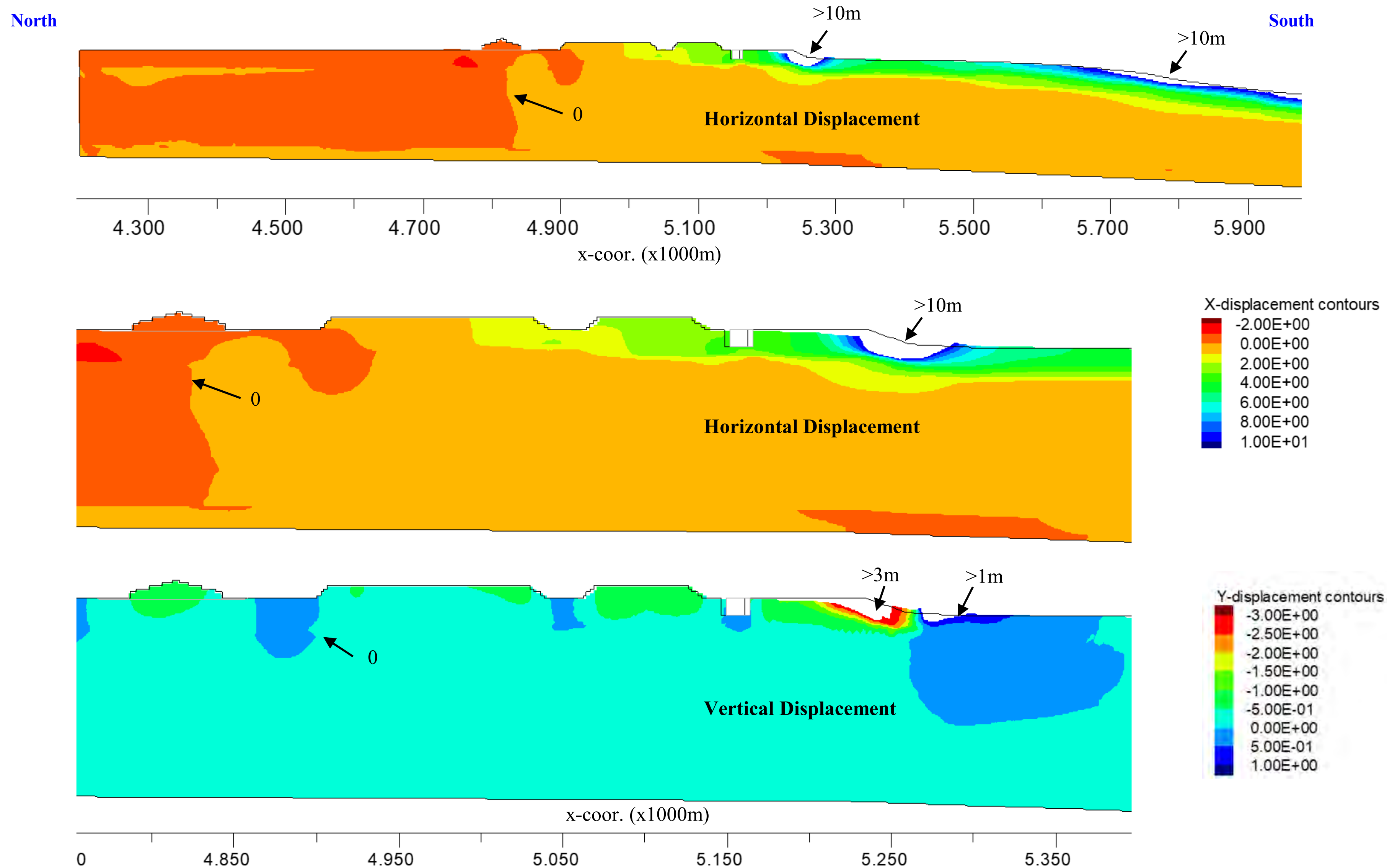


Figure 6-24: Post-earthquake displacements for the E-W Section - A2475-CRU03

Positive values mean eastward horizontal displacement and upward vertical displacement.

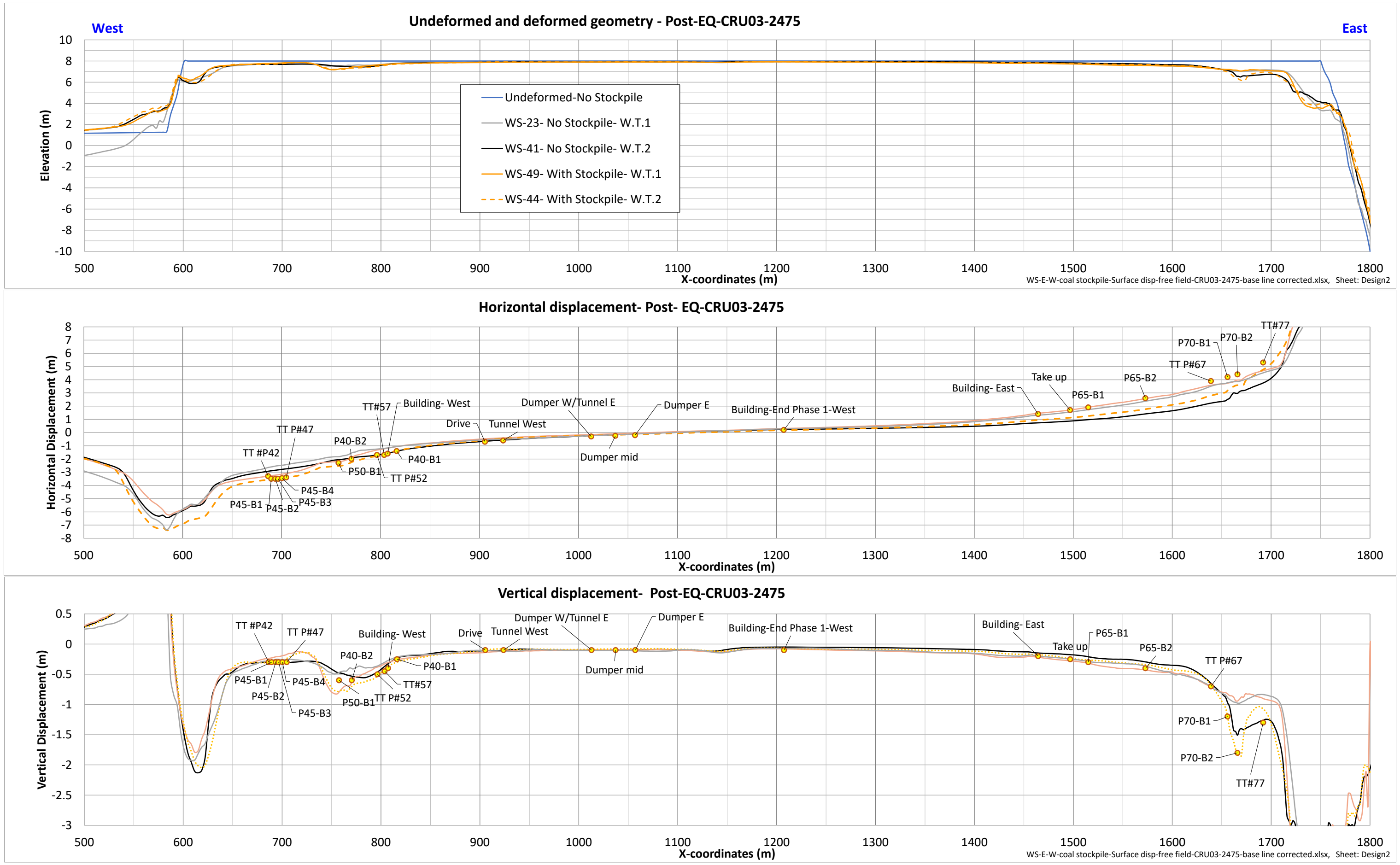


Figure 6-25: Post-earthquake displacements for the N-S Section - A2475-CRU03

Positive values mean southward horizontal displacement and upward vertical displacement.

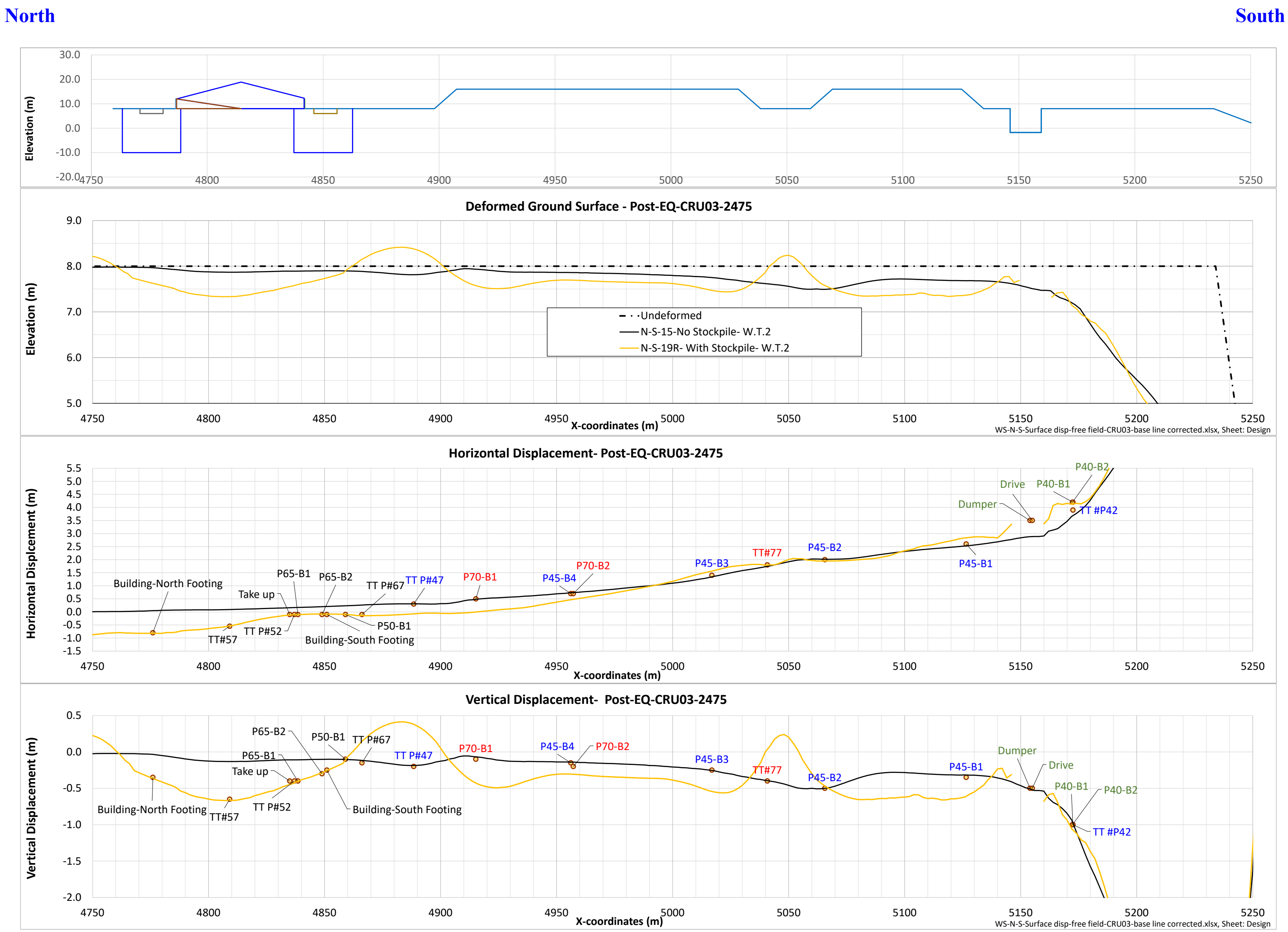


Figure 6-26: Displacements at the end of earthquake and after flow slide failure - East-West Section (CRU03- A200, A475, A975, A2475) and typical displacement time history, (FLAC Case: WS-23-CRU03-2475), reconsolidation settlement is not included in the vertical displacement.

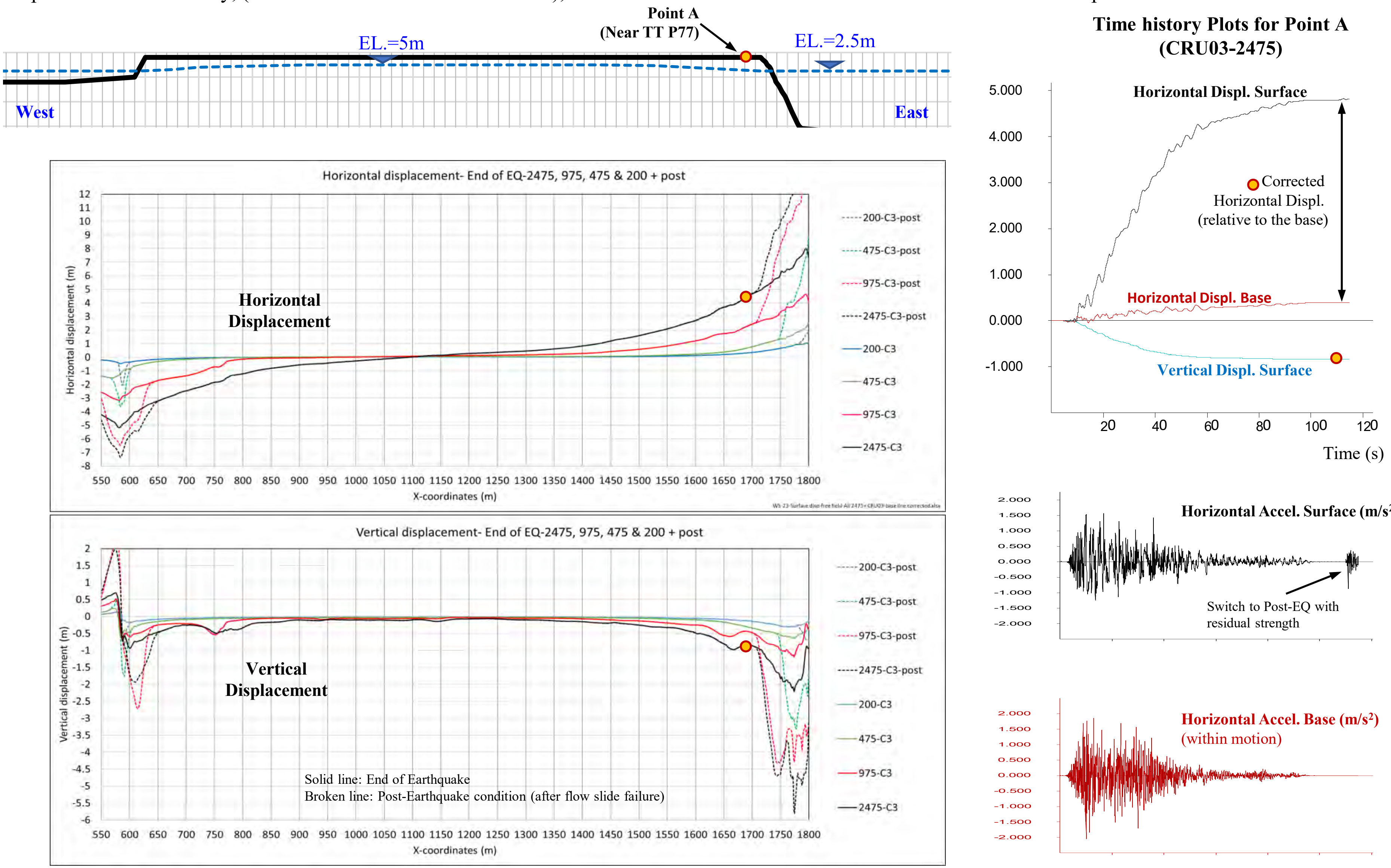


Figure 6-27: North-South FLAC Model - No coal stockpiles

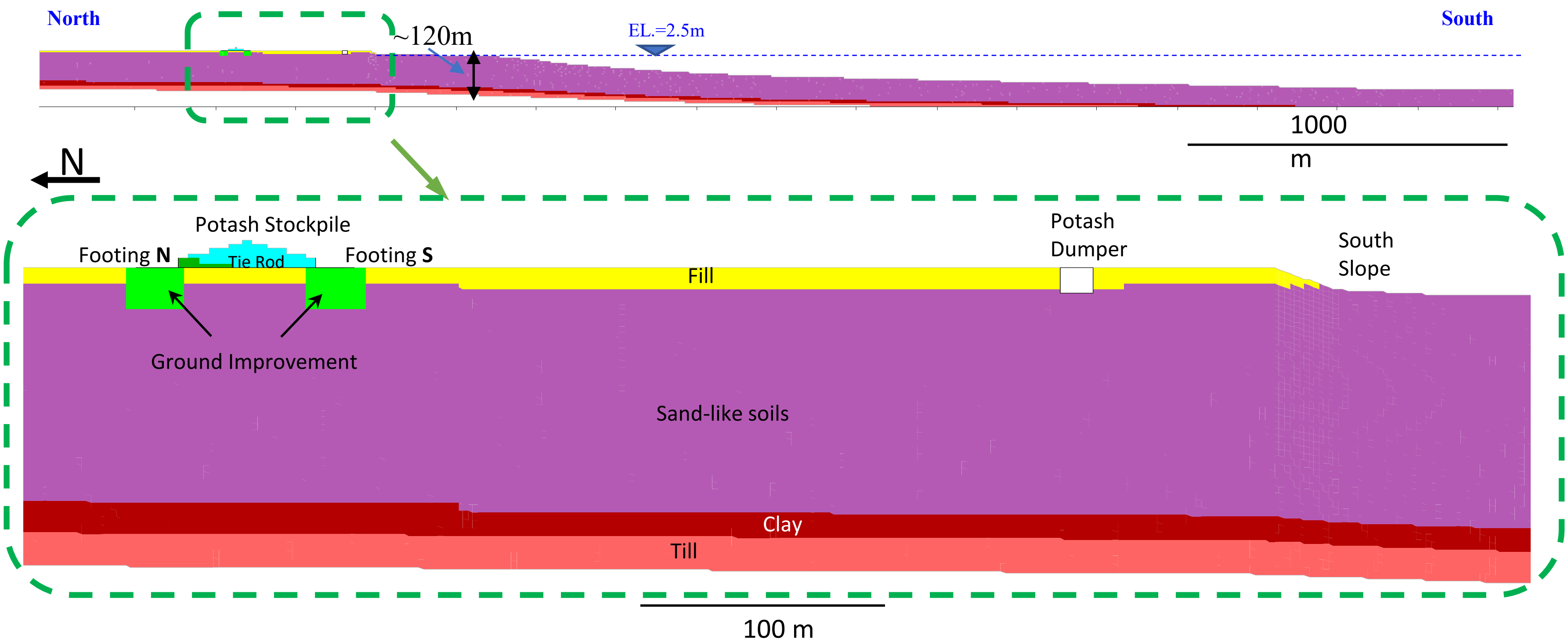


Figure 6-28: Patterns of displacement within the storage building
Case 20R, with Structure & Potash

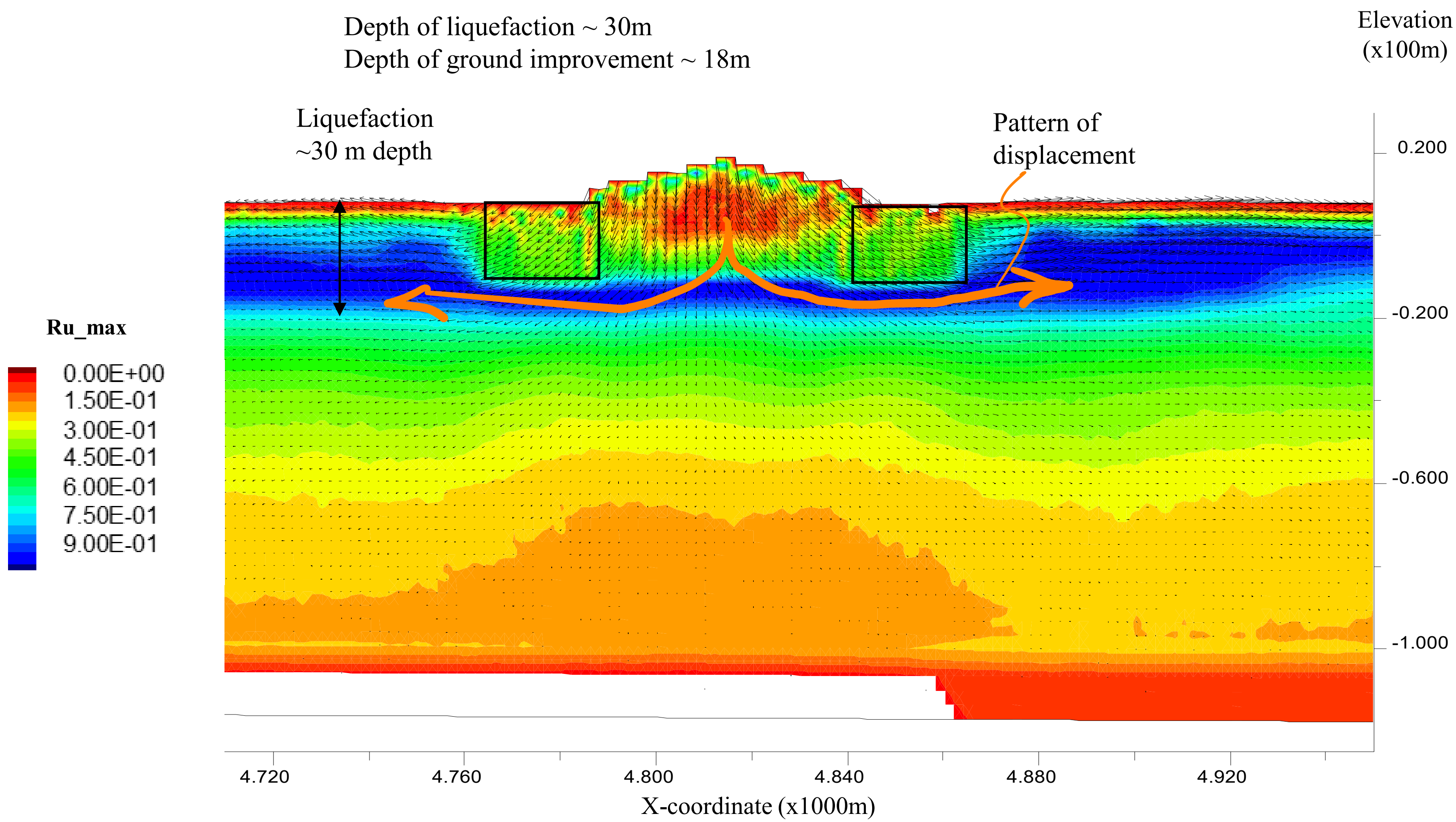
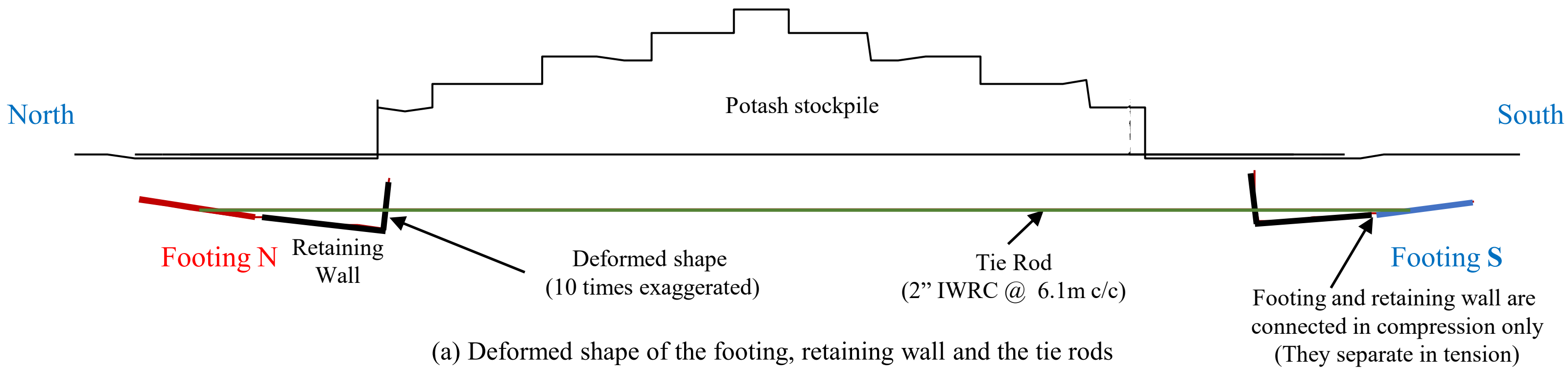
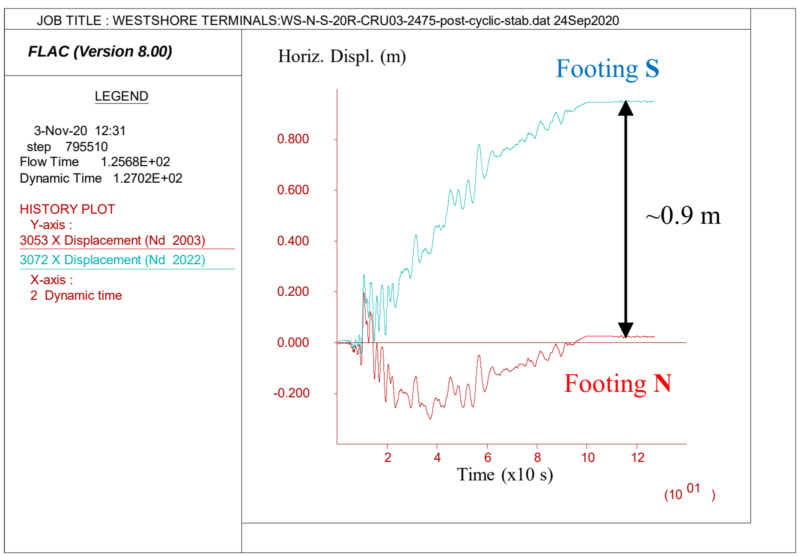


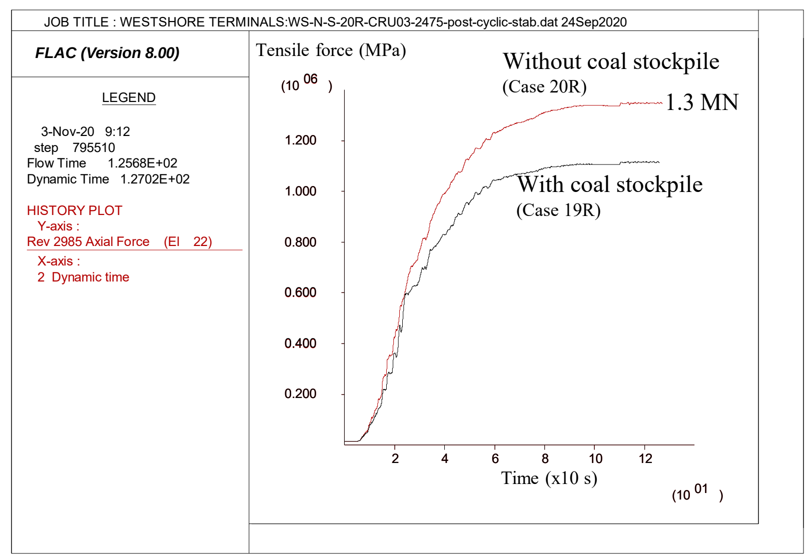
Figure 6-29: Seismic response of the footings and tie rods
2475-CRU03, Post-Earthquake condition (Case 20R)




(a) Deformed shape of the footing, retaining wall and the tie rods




(b) Time history of horizontal displacement of the footings (Case 20R).
(The displacements are not corrected for the base movements)




(c) Time history of axial Force in the tie rod ~ 1.3 MN (Case 20R)
Axial force in tie rod is greater in the absence of coal stock pile

Figure 6-30: Deformation patterns around dumper pit- A2475

Note: Contours present the maximum excess pore pressure ratio, Ru.

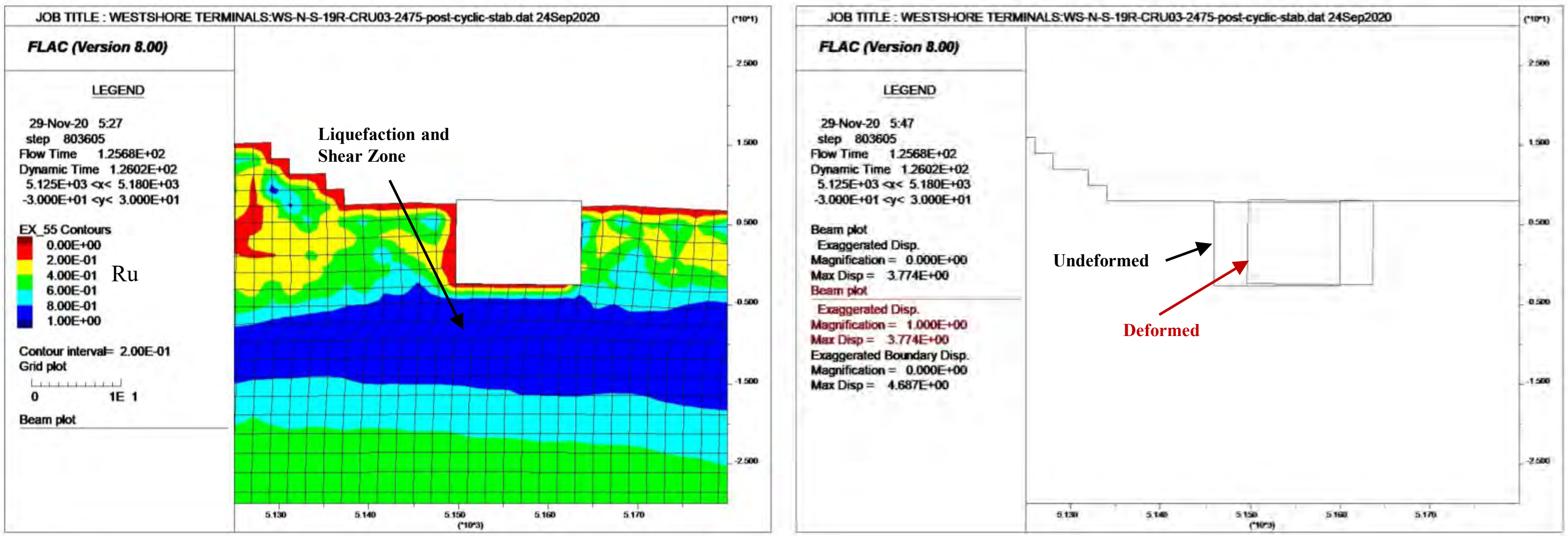
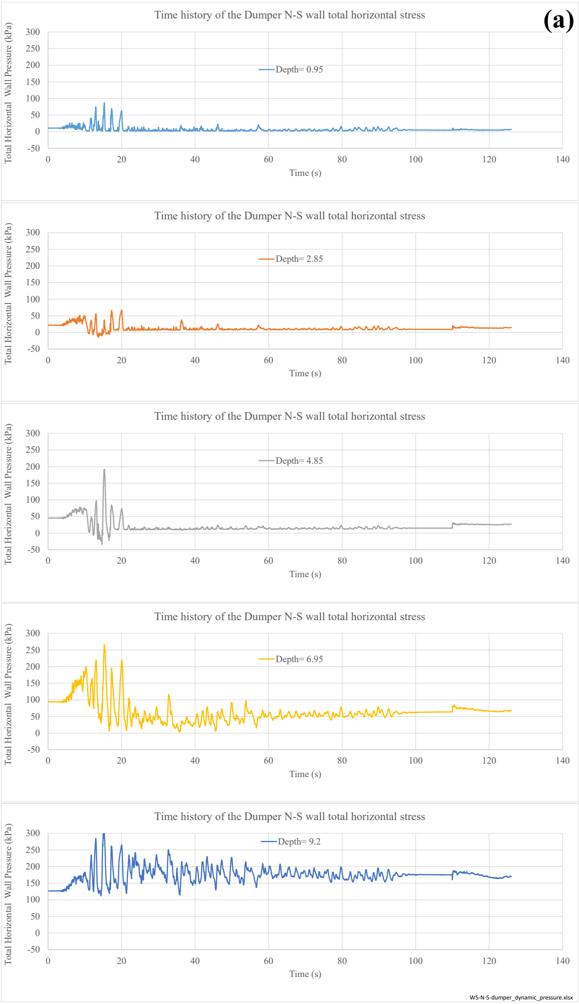
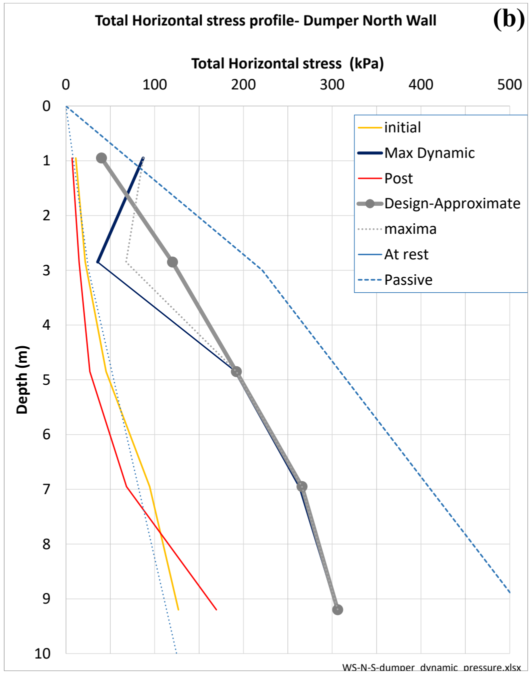


Figure 6-31: Numerical analysis results for the earth pressure on the dumper pit wall from N-S FLAC section- A2475




(a) Time histories of the total horizontal stress on the north side of the dumper pit.




(b) The total horizontal stress profile on the north side of the dumper pit.

Figure 6-32: Examples of time histories of displacement and shaking

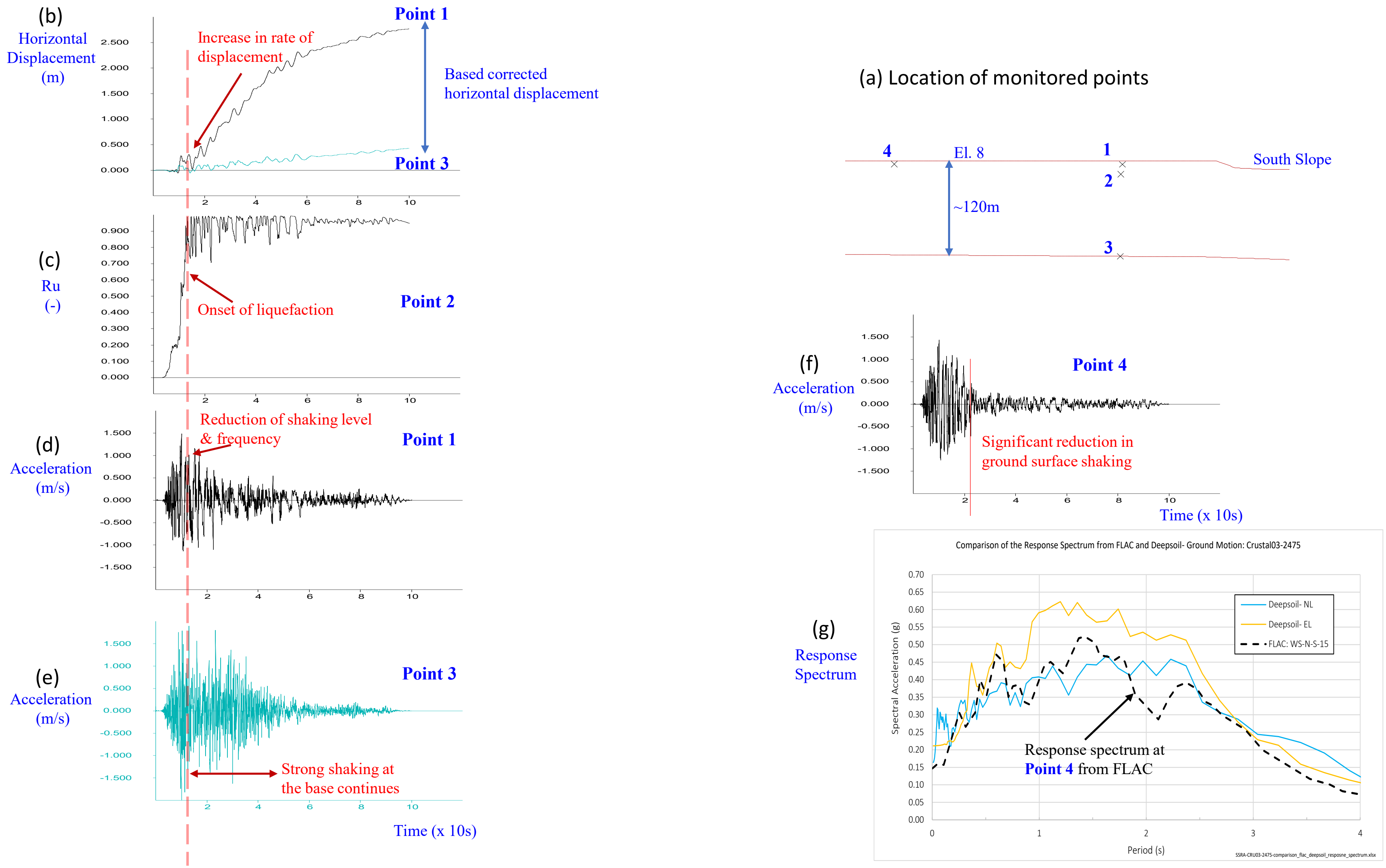


Figure 6-33: Comparison of N-S FLAC results with the Zhang et al. (2004) method
CRU03-2475

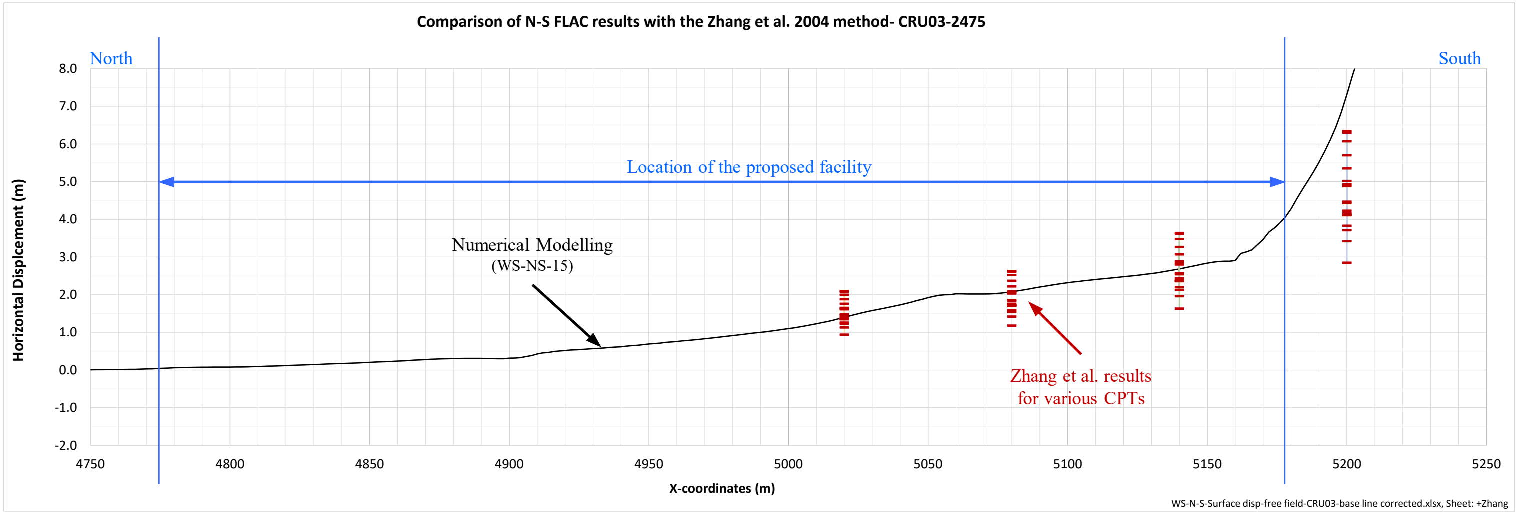


Figure 7-1: Estimated displacements after A2475 earthquake
*(Base drawing 40101-D0000-0000 by CWA)*

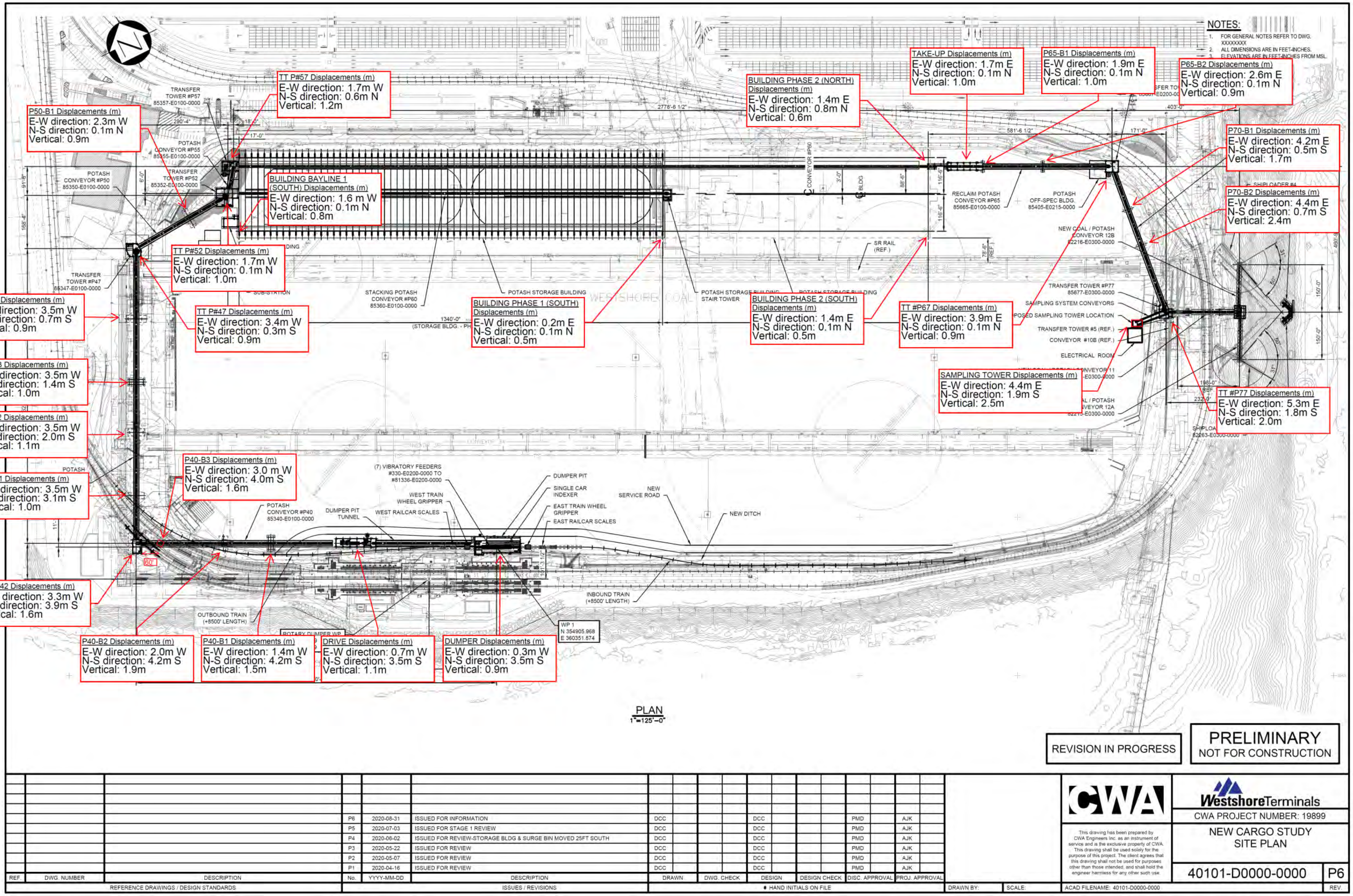


Figure 7-2: Static horizontal wall pressure diagram for north and south sides of dumper pit (at rest)

NOTES:

- Granular soils/backfill adjacent to foundation walls as per the Geotechnical Report.
- Lateral loads indicated are subject to review of actual soil conditions at time of excavation.
- Depending on construction methodology compaction induced pressures may not be applicable.
- Wall pressures are approximate, actual pressures will depend on wall stiffness, groundwater conditions, backfill slope, type of backfill, compaction equipment, and surcharge pressures.

ALL LOADS ARE UNFACTORED

ASSUMPTIONS:

- At rest loading conditions. Other conditions subject to review by Braun Geotechnical.
- All surcharge loads to be reviewed by Braun Geotechnical.
- Groundwater estimated to be at approximately 3.0m depth.
- Assumed road surcharge of 24 kPa, to be reviewed based on actual design vehicle.
- Coal stockpile: Max. height of 23m (75 ft), coal unit weight of 8.6 kN/m$^3$, stockpile toe 11m away from Dumper North wall (horizontal distance), stockpile angle of 40°

Metric Units m & kPa

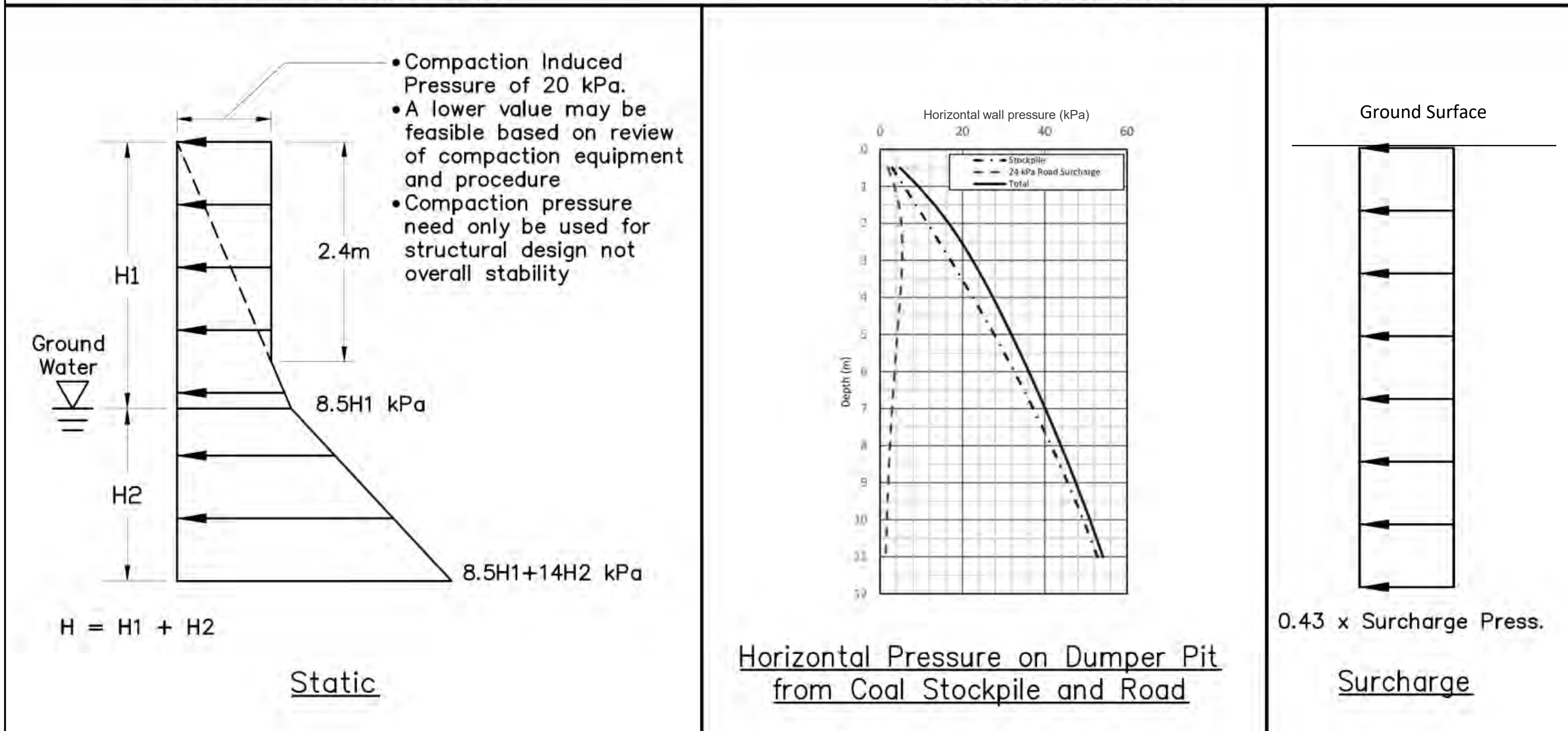


Figure 7-3: Static horizontal wall pressure diagram for east and west sides of dumper pit (at rest)

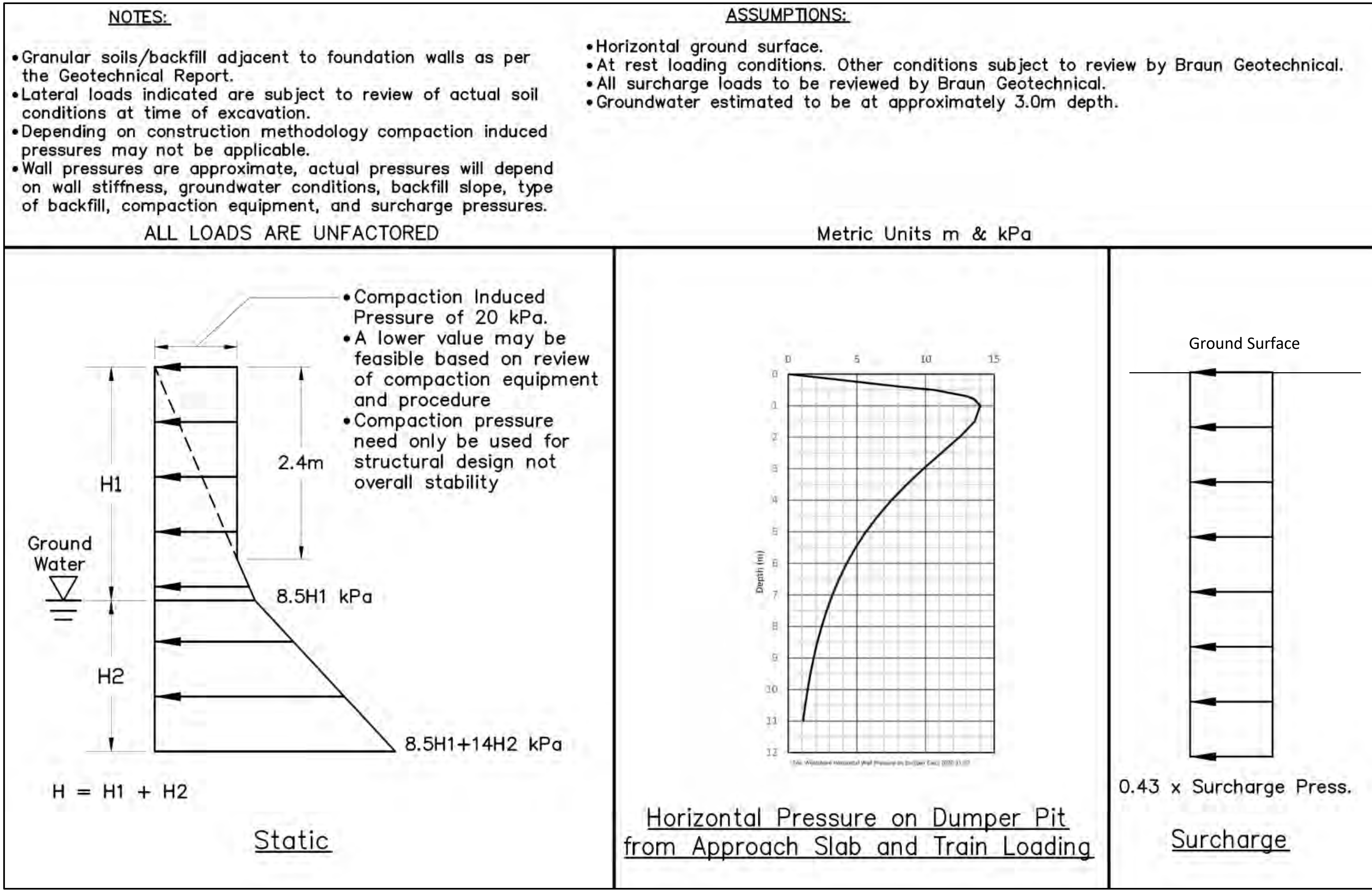


Figure 7-4: Dumper pit north-south cross section showing adjacent road and coal stockpile

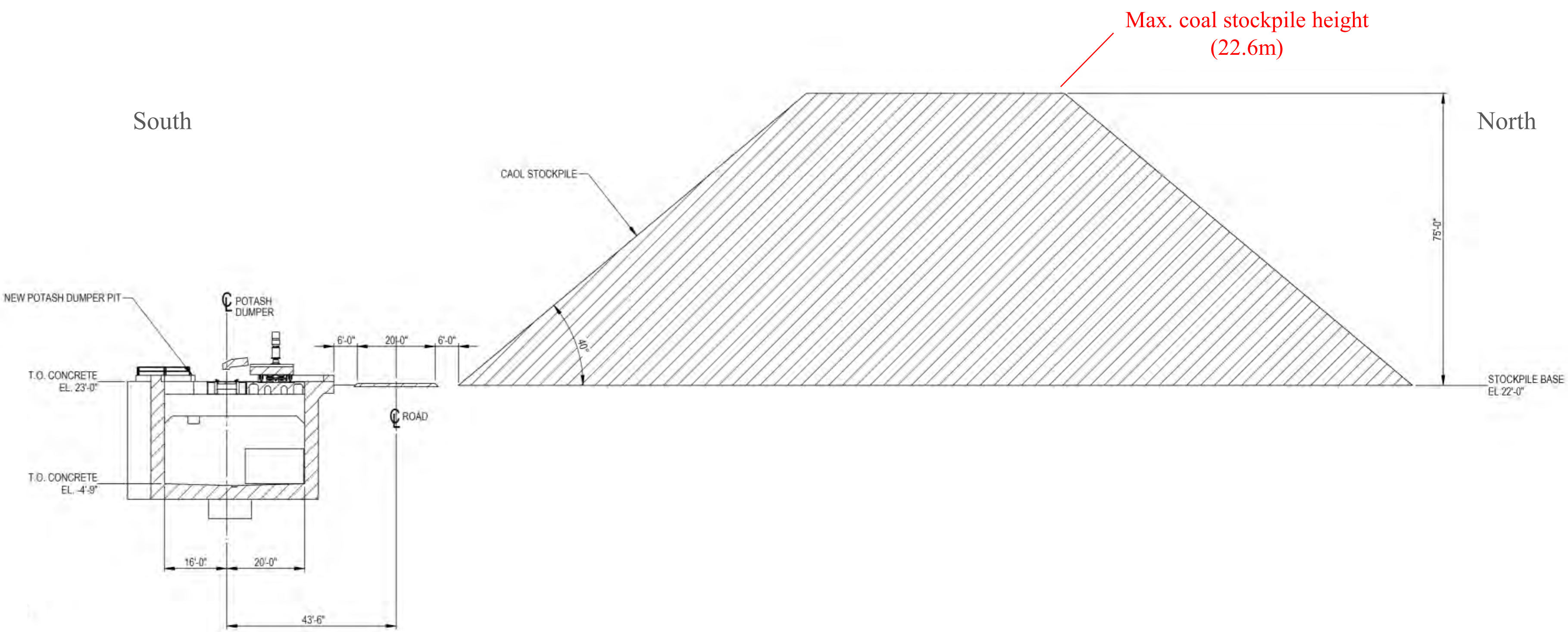


Figure 7-5: Seismic horizontal wall pressure diagram for dumper pit (A2475)

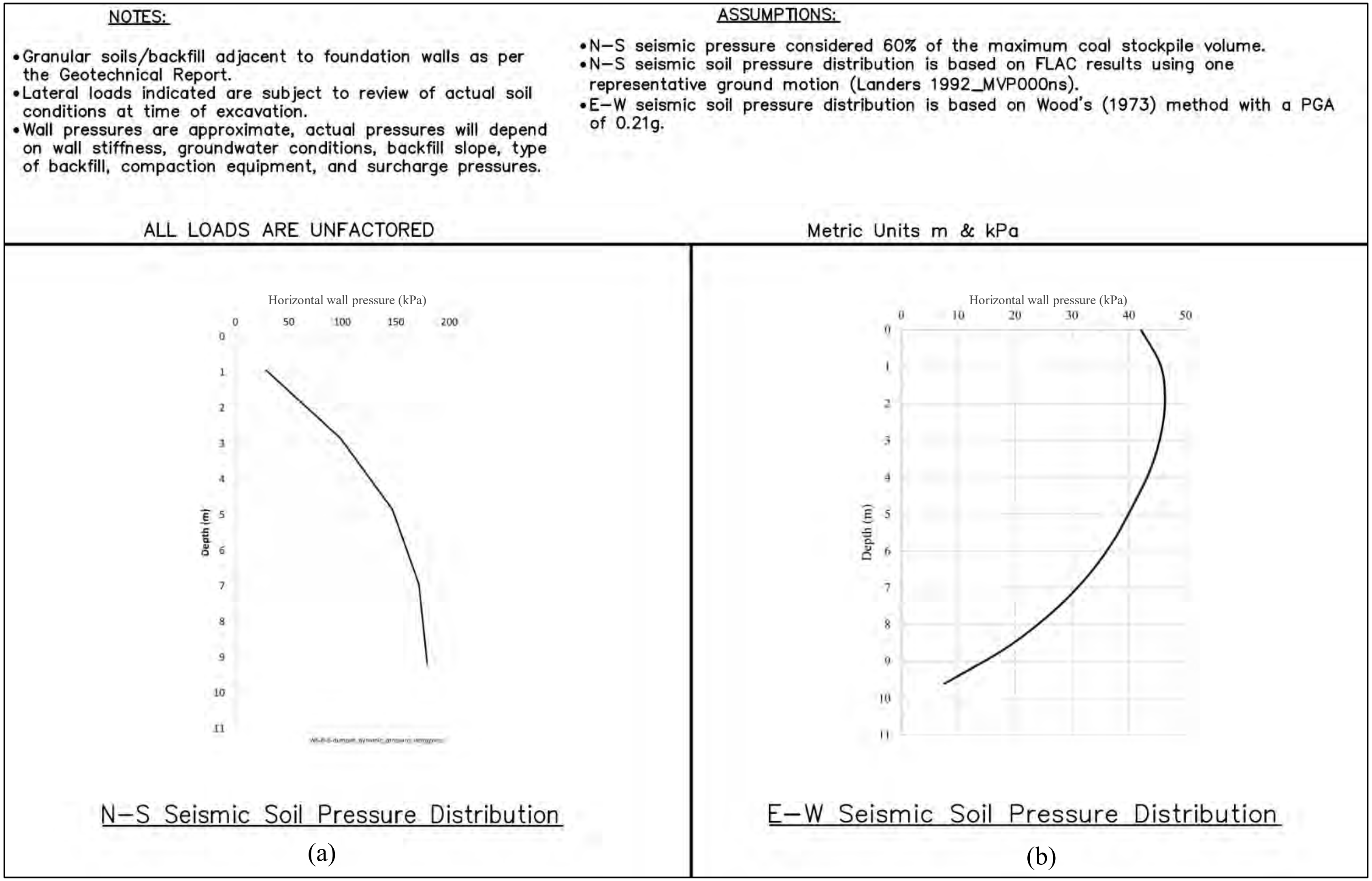
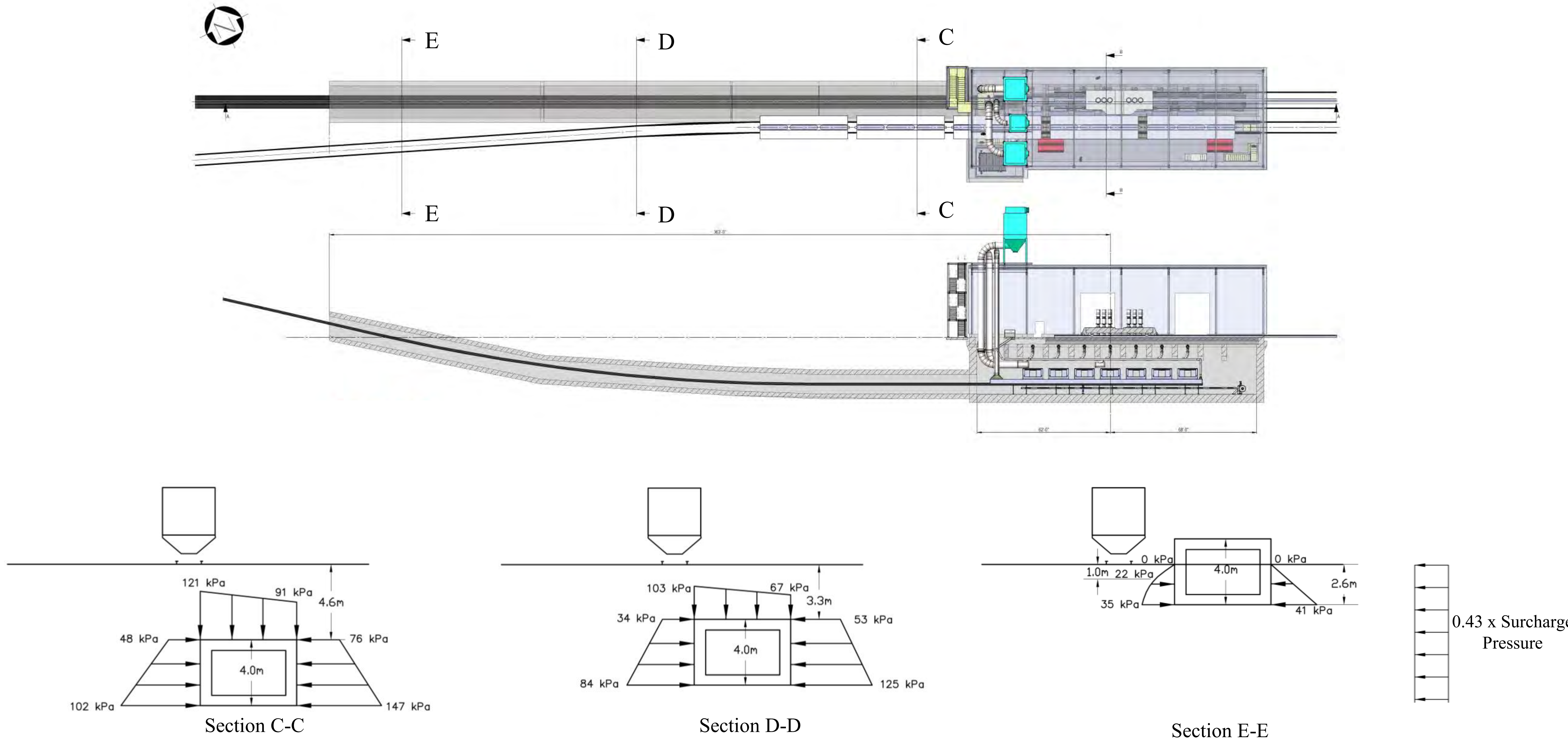


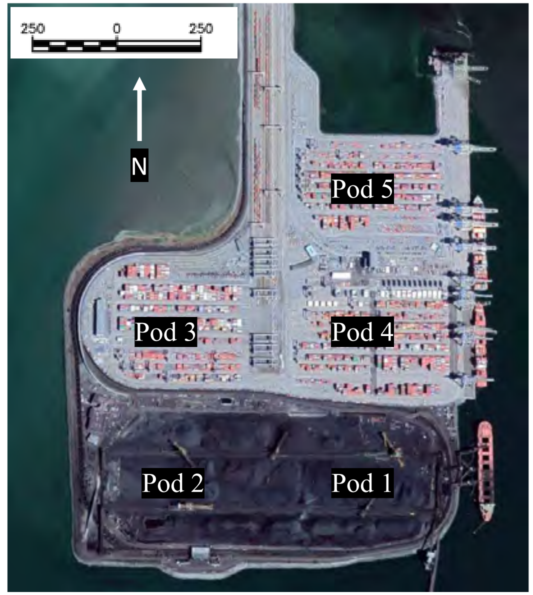
Figure 7-6: Static horizontal wall pressure diagram for north and south sides of tunnel (at rest)

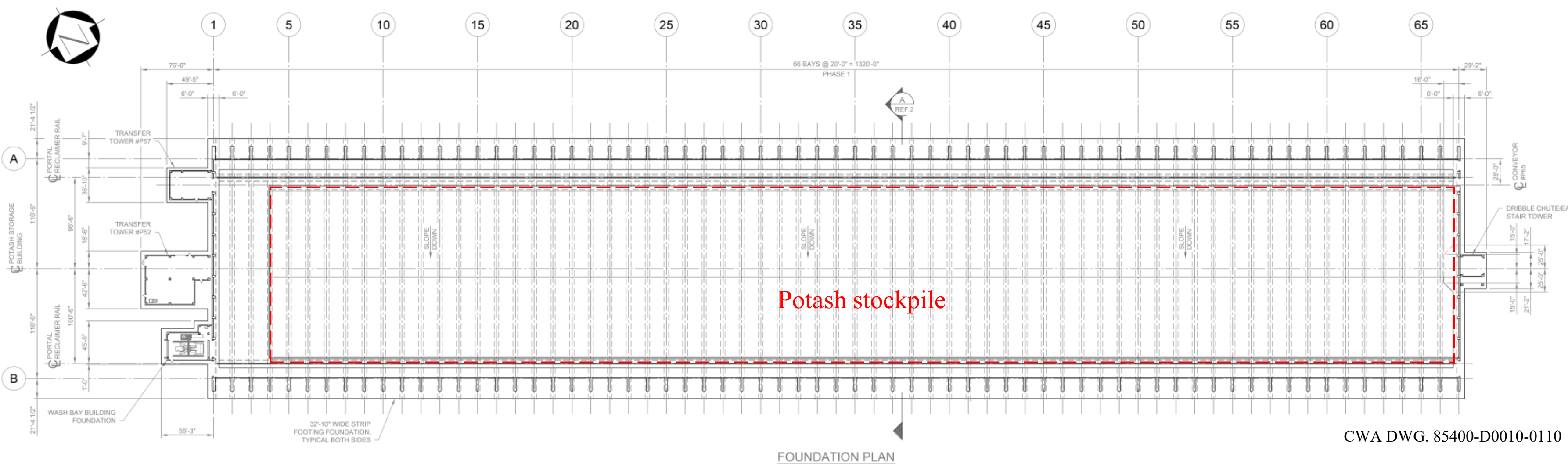



Notes:
- Granular soils/backfill adjacent to foundation walls as per the Geotechnical Report.
- Lateral loads indicated are subject to review of actual soil conditions at time of excavation.
- Depending on construction methodology compaction induced pressures may not be applicable.
- Wall pressures are approximate, actual pressures will depend on wall stiffness, groundwater conditions, backfill slope, type of backfill, compaction equipment, and surcharge pressures.
- All horizontal pressures are unfactored.

Assumptions:
- At rest loading conditions. Other conditions subject to review by Braun Geotechnical.
- All surcharge loads to be reviewed by Braun Geotechnical.
- Groundwater estimated to be at approximately 3.0m depth.
- Assumed road surcharge of 24 kPa, to be reviewed based on actual design vehicle.
- Coal stockpile: Max. height of 23m (75ft), coal unit weight of 8.6kN/m³, stockpile toe 11m away from Dumper North wall (horizontal distance), stockpile angle of 40°
- Presented soil pressure diagram includes: ground water pressure, train surcharge, traffic load on adjacent road, and coal stockpile surcharge.

Figure 7-7: General site layout and the Phase 1 storage building plan




CWA DWG. 85400-D0010-0110

Figure 7-8: General site arrangement plan with location of CPTs and historical settlement gauges (modified from the 2021 KCB report Figure 2.1)

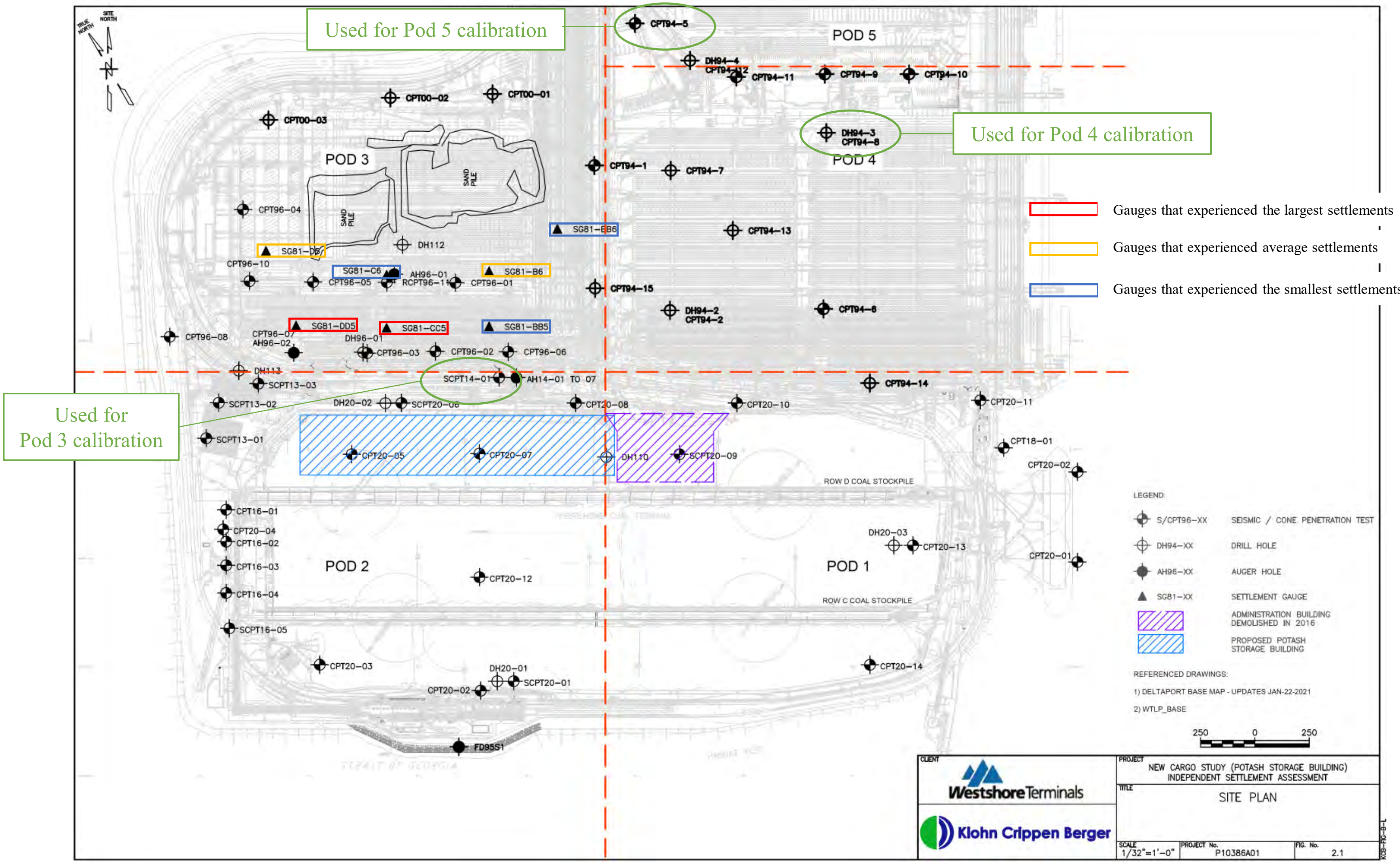
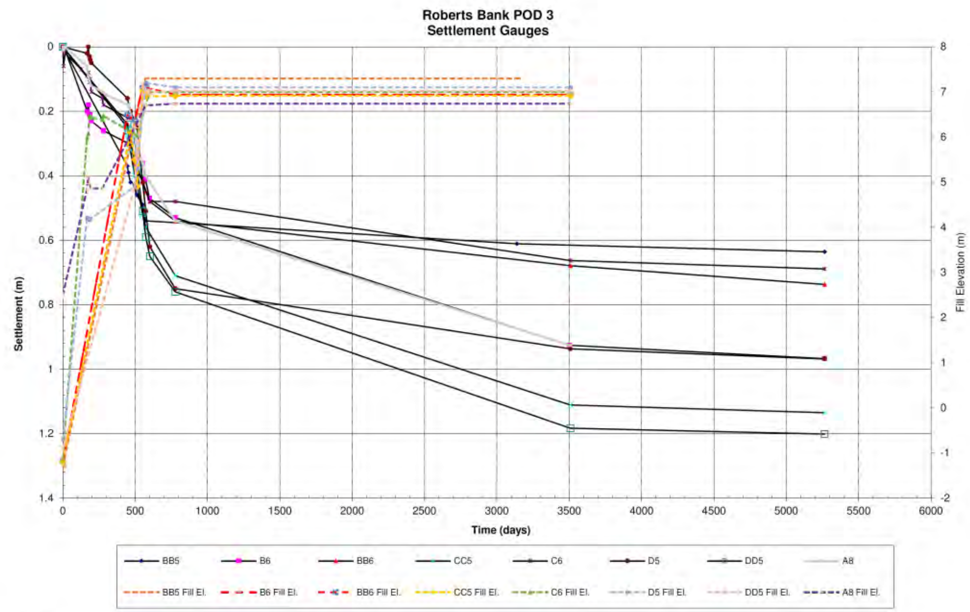


Figure 7-9: Pod 3 settlement gauge monitoring data and fill elevations
(2021 KCB report Figure 2.2)




NOTE:

1. Settlement Data from KCB archive.
2. The monitoring program commenced in September 1981.

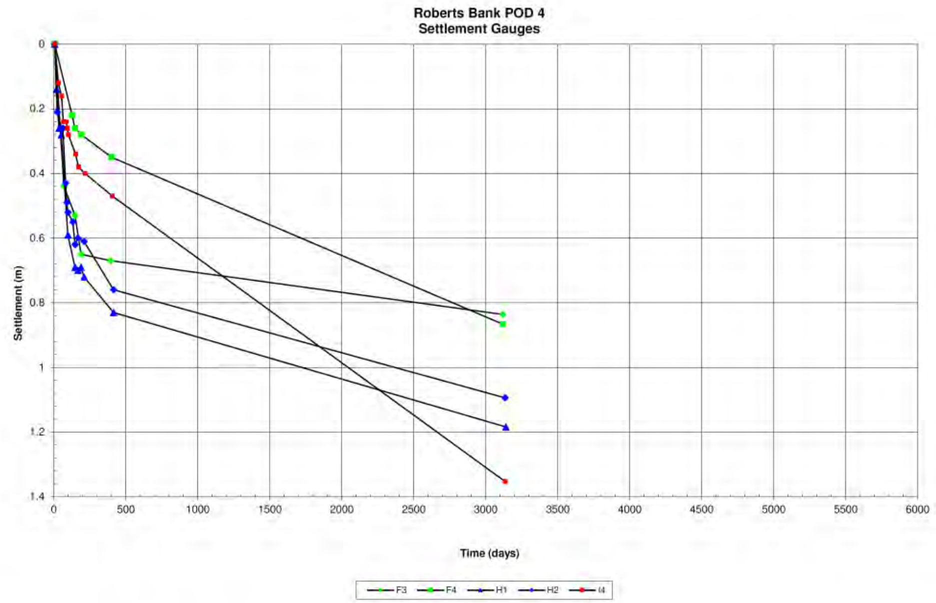
Note: Elevations in Chart Datum

| | |
|---|---|
| PROJECT: | Westshore Terminal<br>New Cargo Study (Potash Storage Building)<br>Independent Settlement Assessment |
| TITLE: | Pod 3 Settlement Gauge Monitoring Data |
| SCALE: - | PROJECT NO: P10386A01 — FIGURE: 2.2 |

DRAFT

Klohn Crippen Berger

Figure 7-10: Pod 4 settlement gauge monitoring data
(2021 KCB report Figure 2.3)




NOTE:

1. Settlement Data from KCB archive.
2. The monitoring program commenced in September 1981.

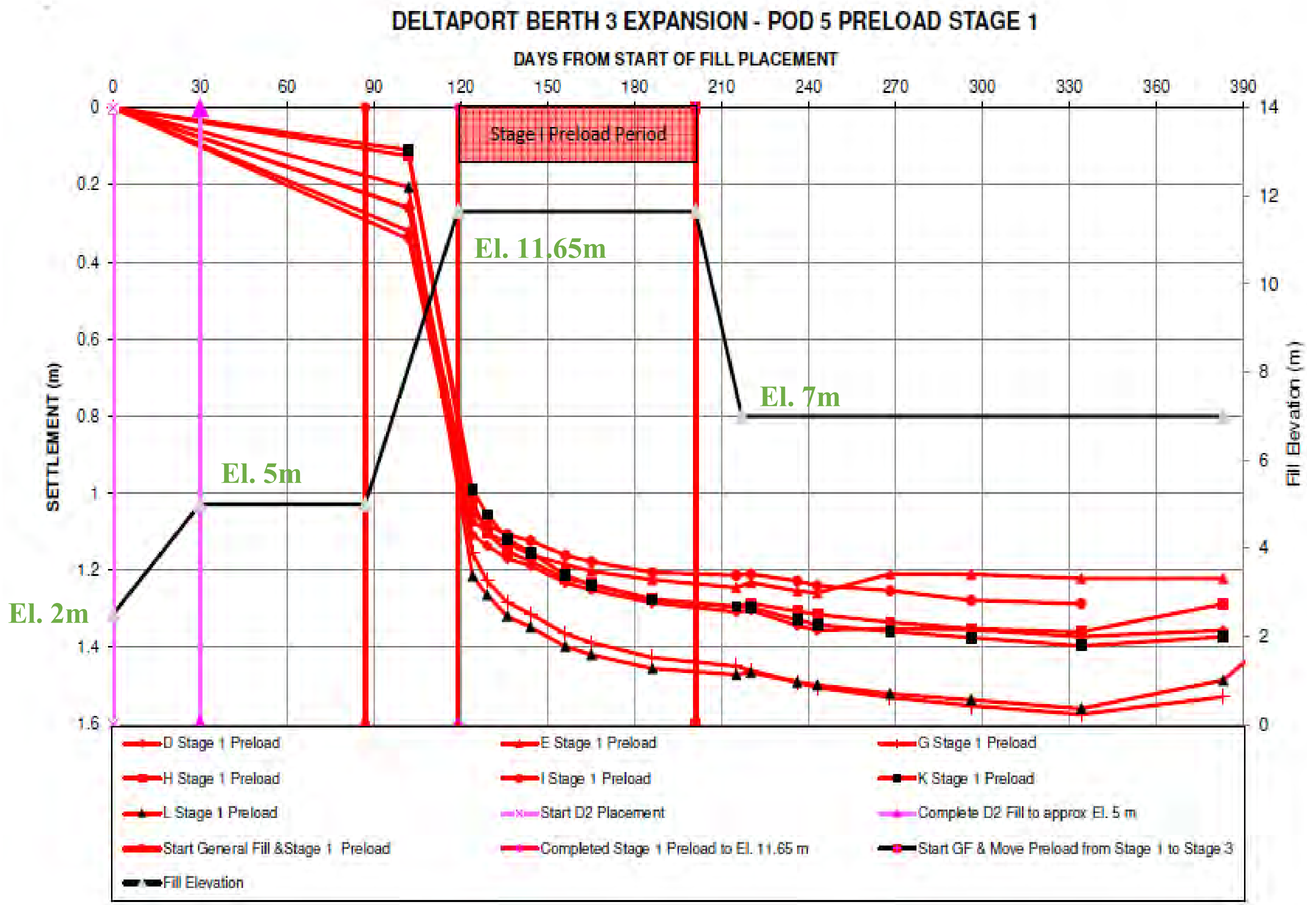
| | |
|---|---|
| PROJECT: | **Westshore Terminal**<br>**New Cargo Study (Potash Storage Building)**<br>**Independent Settlement Assessment** |
| TITLE: | **Pod 4 Settlement Gauge Monitoring Data** |

| SCALE: | PROJECT NO: | FIGURE: |
|---|---|---|
| - | P10386A01 | 2.3 |

**DRAFT**

Klohn Crippen Berger

Figure 7-11: Pod 5 settlement gauge monitoring data and fill elevations
(Modified from the 2021 KCB report Figure 2.6)

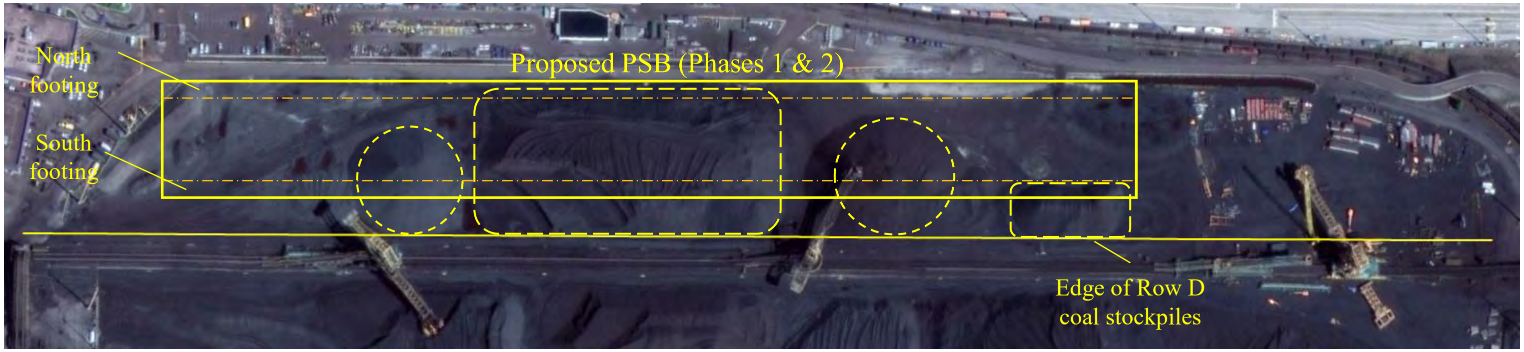



NOTE:
1. Settlement Data from KCB archive.

Note: Elevations in Chart Datum

Figure 7-12: Typical location of coal stockpiles on Row D relative to the approximate location of the proposed potash storage building

a) Typical coal stockpile location




Google Earth, taken on August 2017

b) North-South building cross section with assumed location of historical coal stockpile based on historical air photos and information provided by Westshore

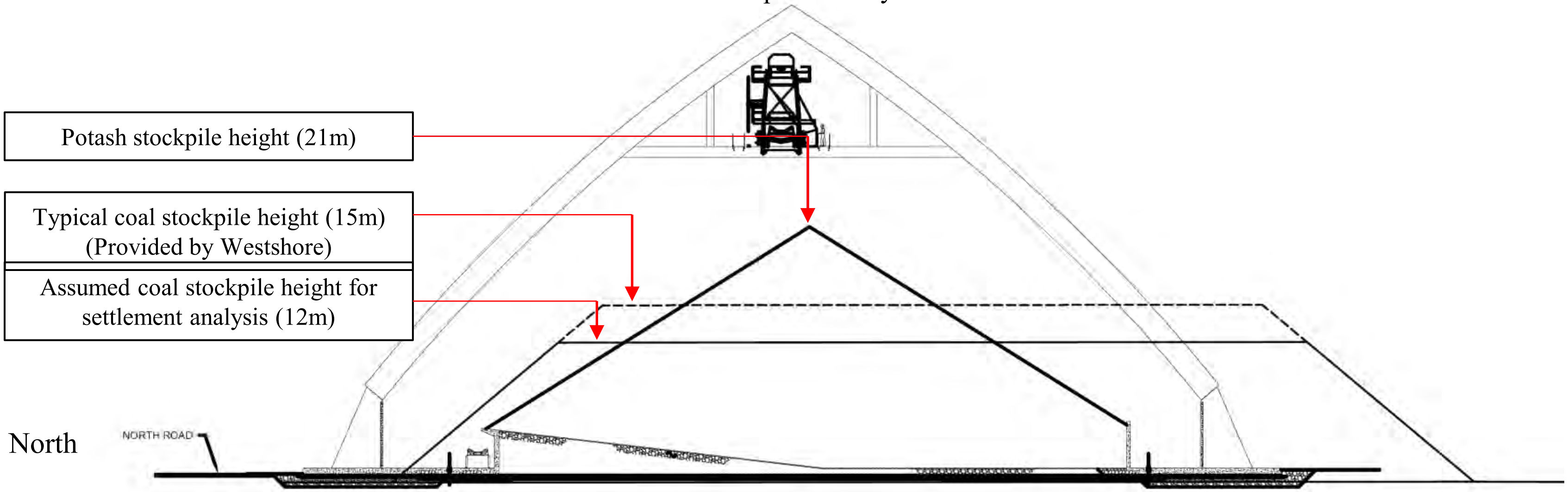
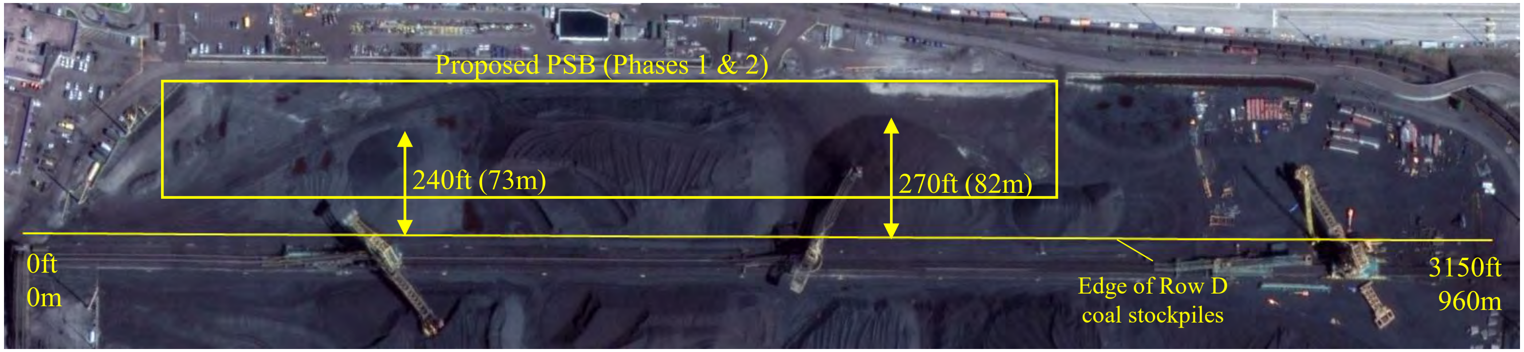


Figure 7-13: Row D historical stockpile height

a) Typical widths of coal stockpiles




b) Maximum annual average coal stockpile height from 2010 to 2020

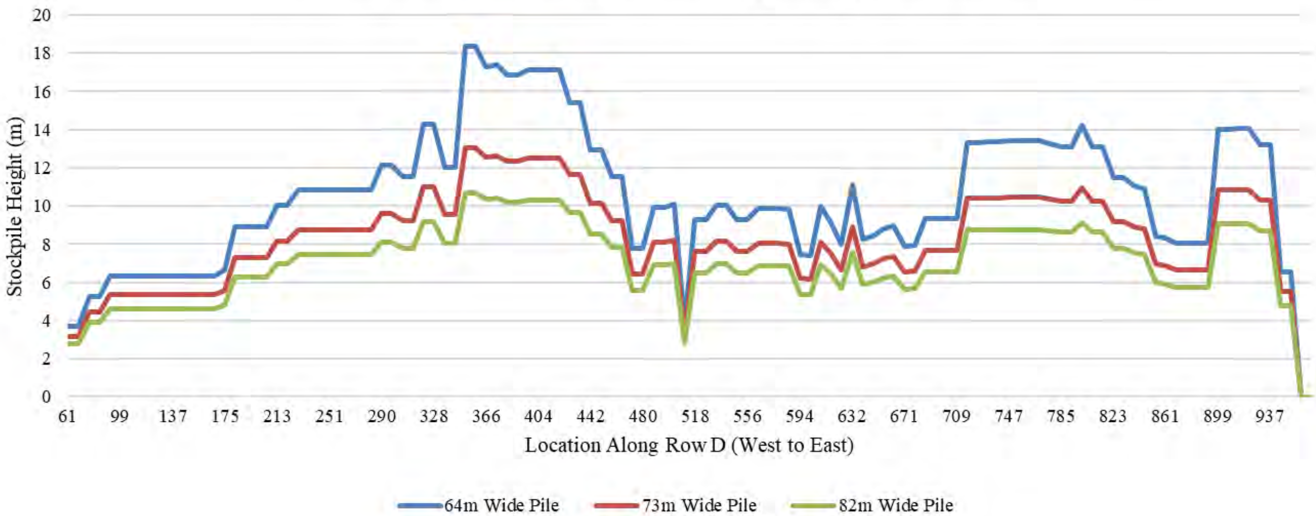
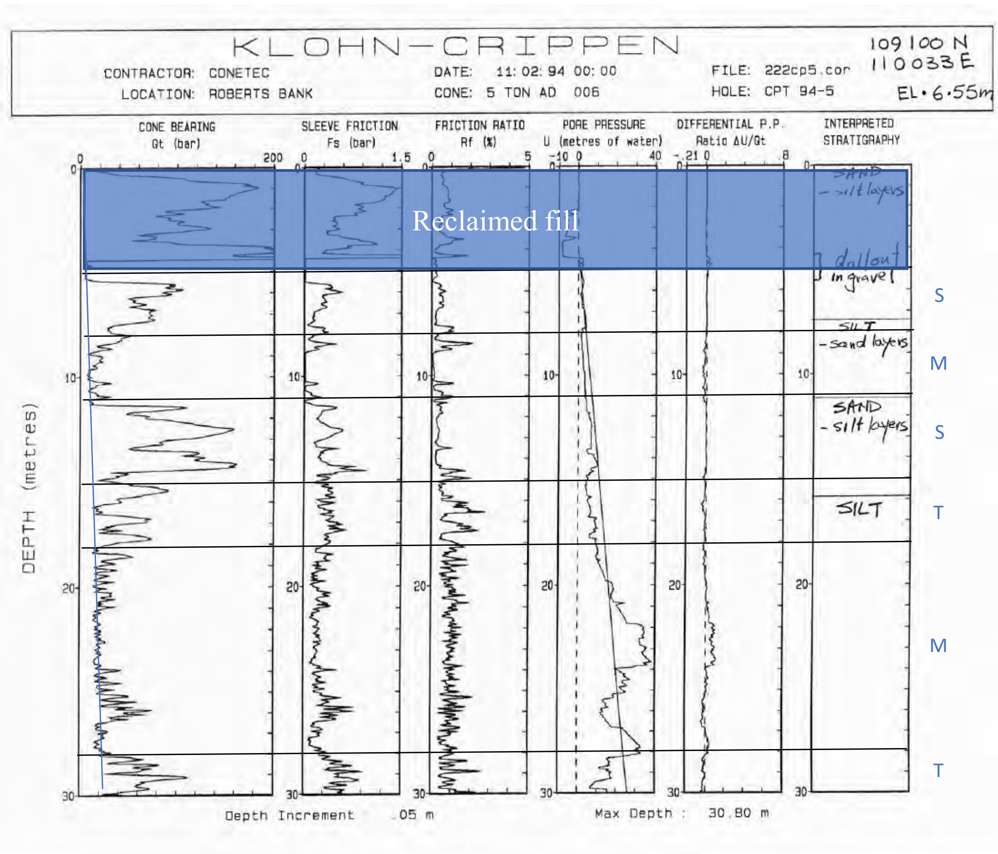
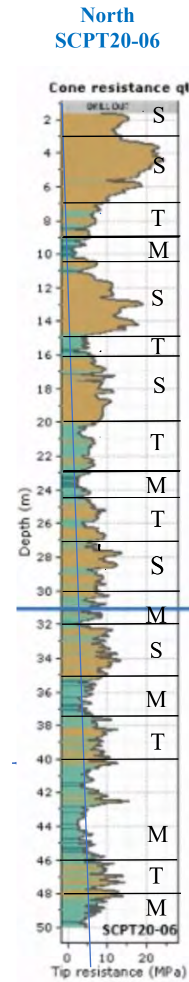


Figure 7-14: Assumed soil profile for Pod 5 calibration




Assumed stratigraphy for depths between 30 to 50m

Assumed stratigraphy for depths greater than 50m
Ref. Golder (2011) SCPT10-04

| Depth below ground surface (m) | | Material |
|---|---|---|
| From | To | |
| 50 | 60 | S |
| 60 | 65 | T |
| 65 | 70 | S |
| 70 | 73 | M |
| 73 | 77 | S |
| 77 | 86 | M |
| 86 | 92 | S |
| 92 | 110 | C |
| 110 | 115 | M |

S=Sand
T= Silt and sand mixture with an intermediate behavior
M=Silt
C=Clay

Figure 7-15: Pod 5 settlement calibration results
(Modified from the 2021 KCB report Figure 2.6)

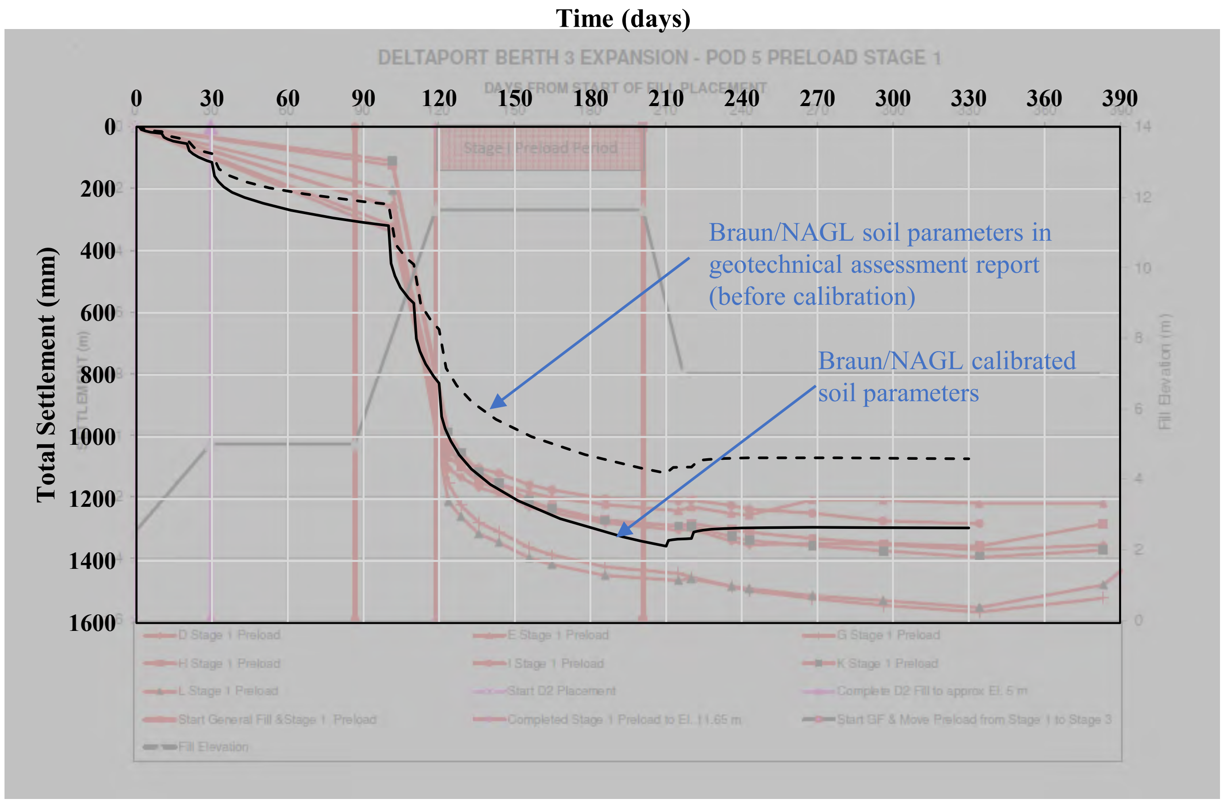
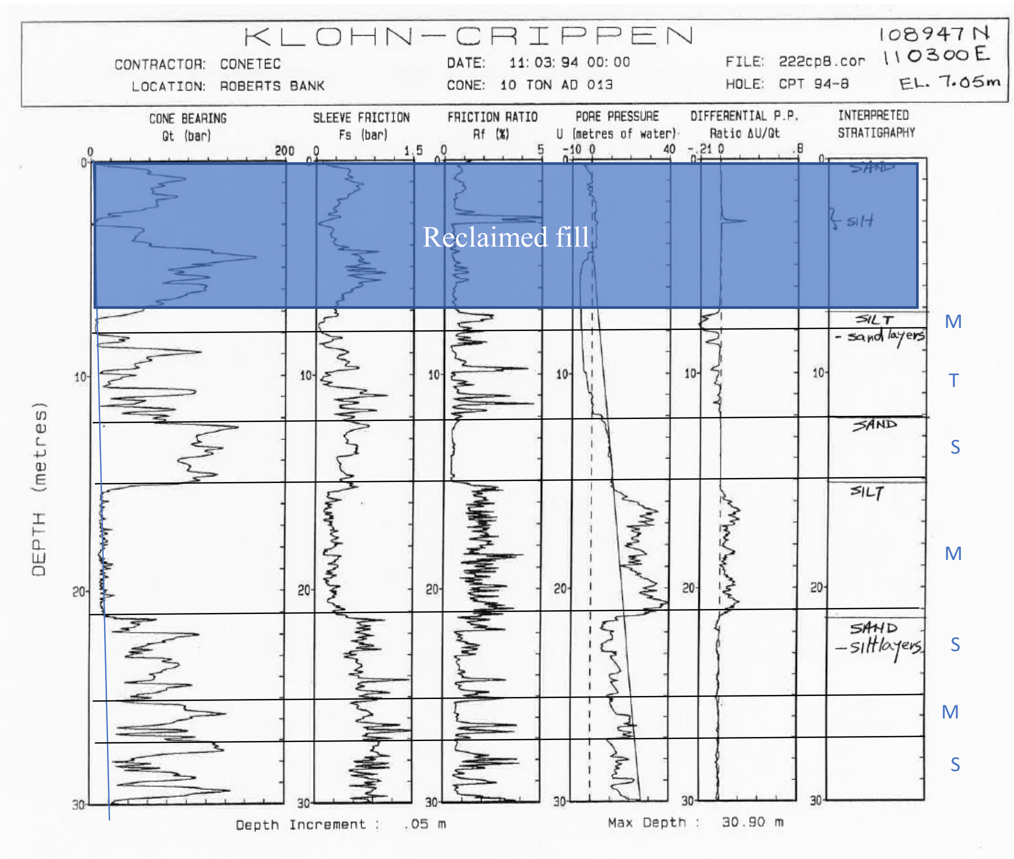
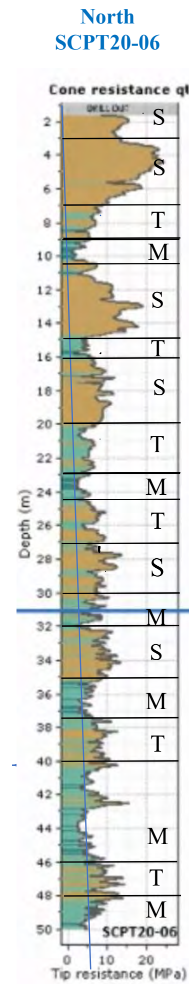


Figure 7-16: Assumed soil profile for Pod 4




Assumed stratigraphy for depths greater than 50m
Ref. Golder (2011) SCPT10-04

| Depth below ground surface (m) | | Material |
|---|---|---|
| From | To | |
| 50 | 60 | S |
| 60 | 65 | T |
| 65 | 70 | S |
| 70 | 73 | M |
| 73 | 77 | S |
| 77 | 86 | M |
| 86 | 92 | S |
| 92 | 110 | C |
| 110 | 115 | M |

S=Sand
T= Silt and sand mixture with an intermediate behavior
M=Silt
C=Clay

Assumed stratigraphy for depths between 30 to 50m

Figure 7-17: Braun/NAGL Pod 4 prediction using Pod 5 calibration
(Modified from 2021 KCB report Figure 2.3)

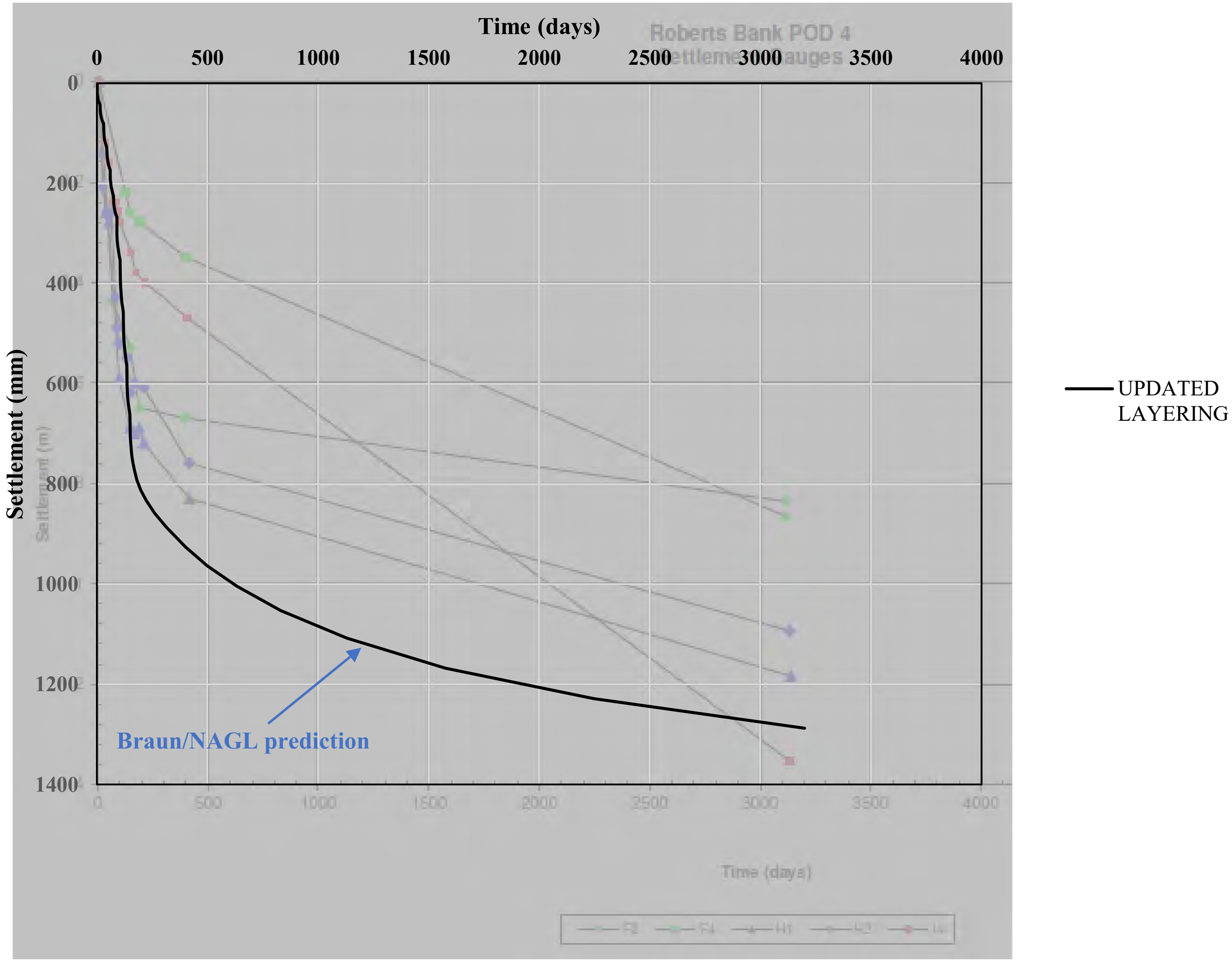
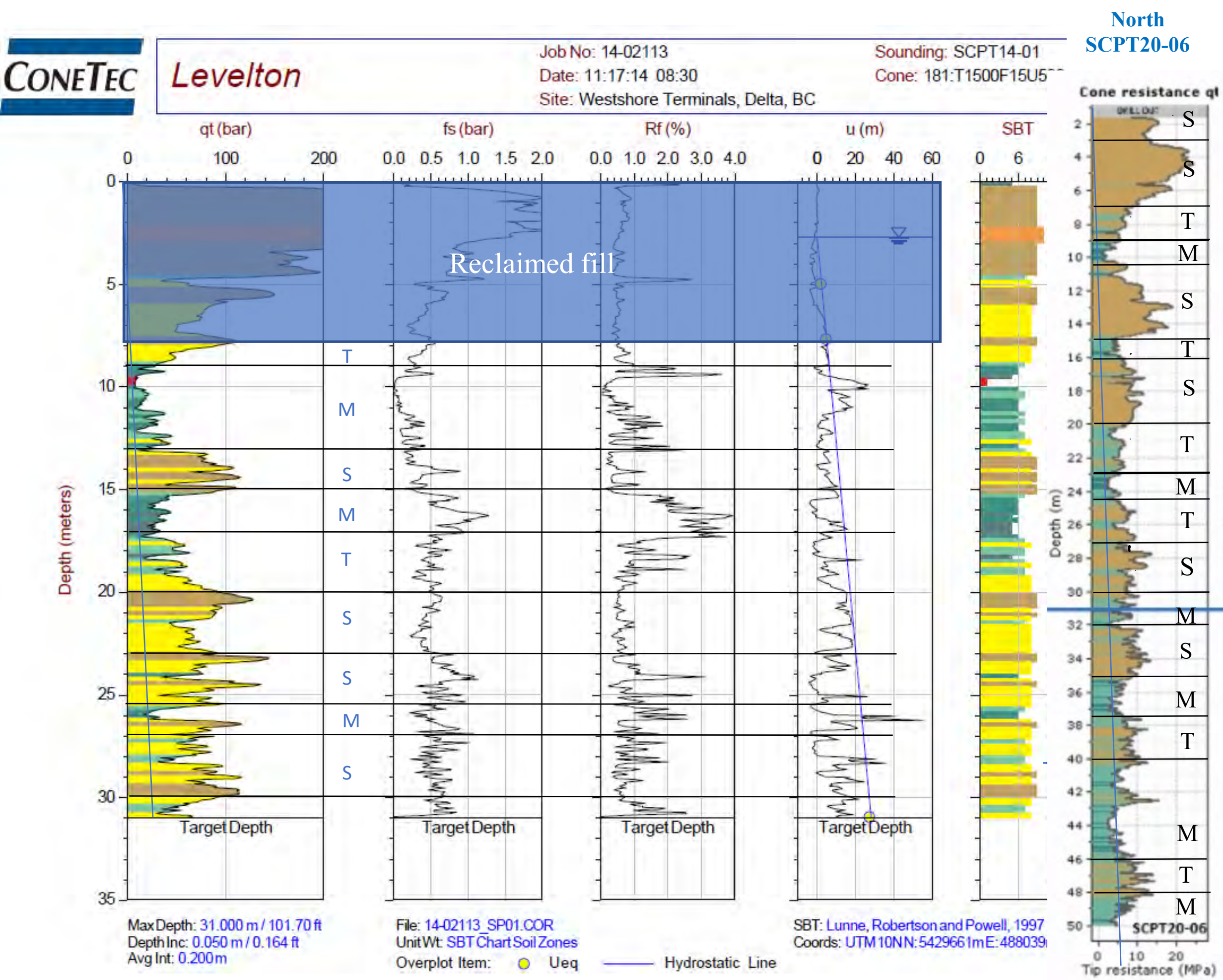


Figure 7-18: Assumed soil profile for Pod 3 settlement prediction




Assumed stratigraphy for depths greater than 50m
Ref. Golder (2011) SCPT10-04

| Depth below ground surface (m) | | Material |
|---|---|---|
| From | To | |
| 50 | 60 | S |
| 60 | 65 | T |
| 65 | 70 | S |
| 70 | 73 | M |
| 73 | 77 | S |
| 77 | 86 | M |
| 86 | 92 | S |
| 92 | 110 | C |
| 110 | 115 | M |

S=Sand
T= Silt and sand mixture with an intermediate behavior
M=Silt
C=Clay

Assumed stratigraphy for depths between 30 to 50m

Figure 7-19: Braun/NAGL Pod 3 prediction using Pod 5 calibration
(modified from 2021 KCB report Figure 2.2)

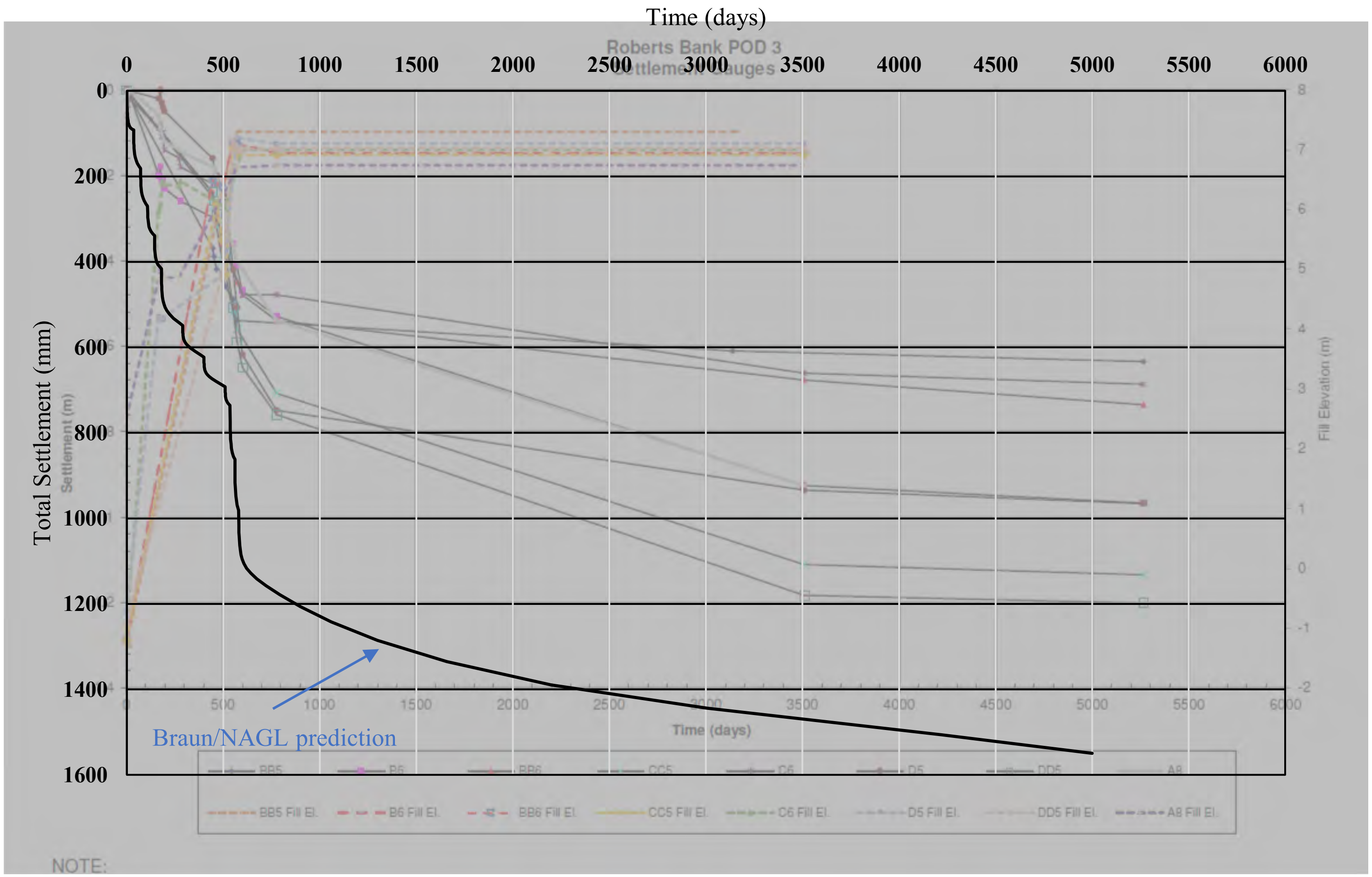
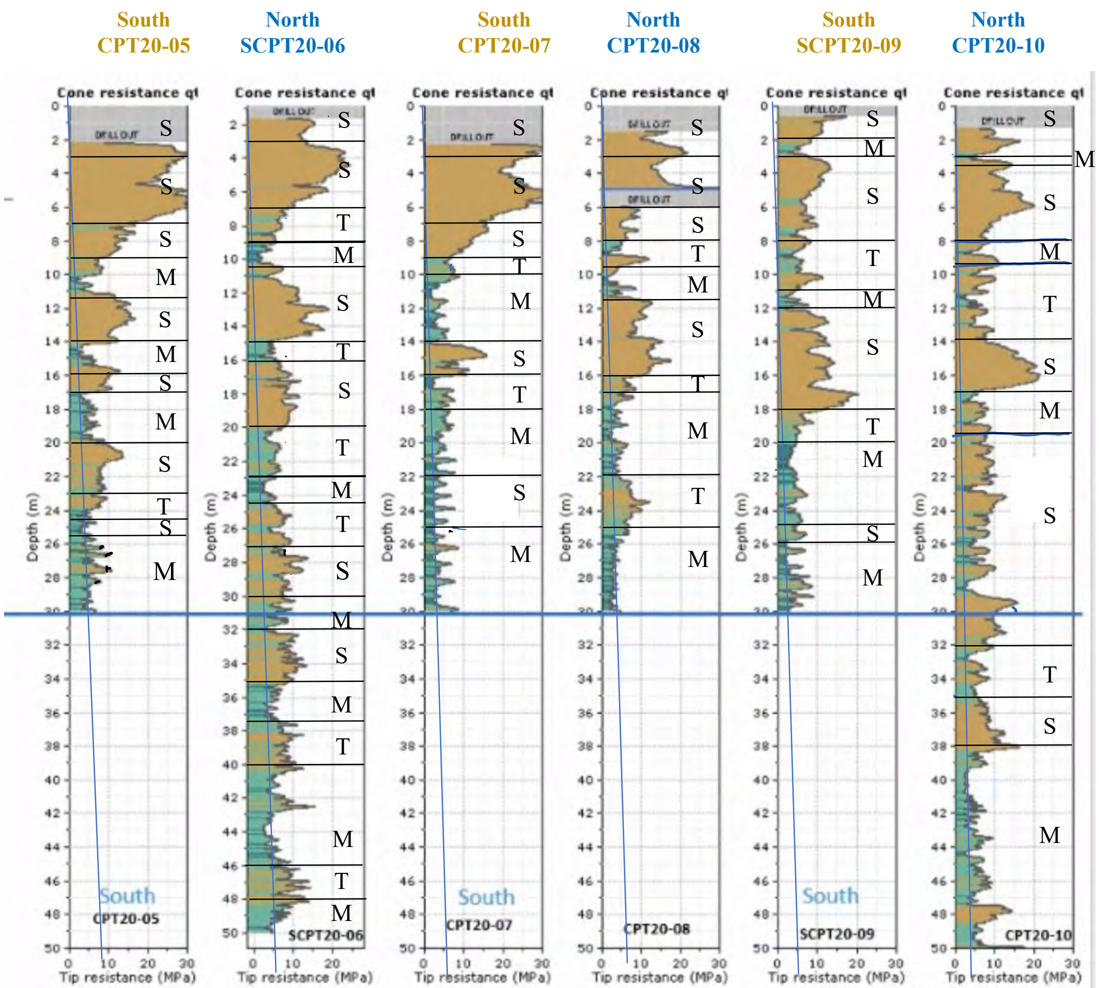


Figure 7-20: Soil profiles within storage building footprint used for settlement assessment

Assumed stratigraphy for depths less than 50m




Assumed stratigraphy for depths greater than 50m
Ref. Golder (2011) SCPT10-04

| Depth below ground surface (m) | | Material |
|---|---|---|
| From | To | |
| 50 | 60 | S |
| 60 | 65 | T |
| 65 | 70 | S |
| 70 | 73 | M |
| 73 | 77 | S |
| 77 | 86 | M |
| 86 | 92 | S |
| 92 | 110 | C |
| 110 | 115 | M |

S=Sand
T= Silt and sand mixture with an intermediate behavior
M=Silt
C=Clay

Figure 7-21: Current preload configuration

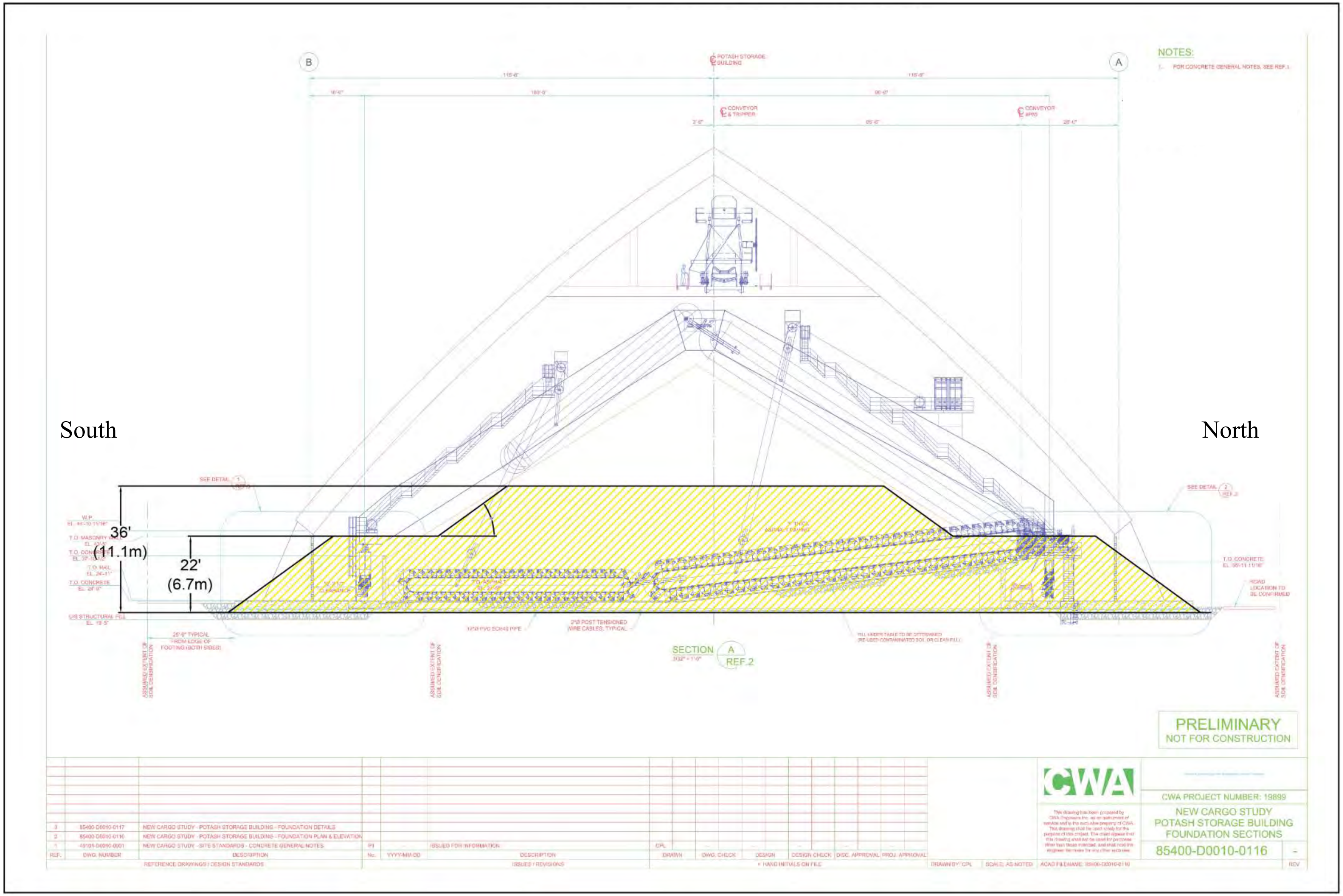


Figure 7-22: Superimposed ground surface pressures from sustained load components

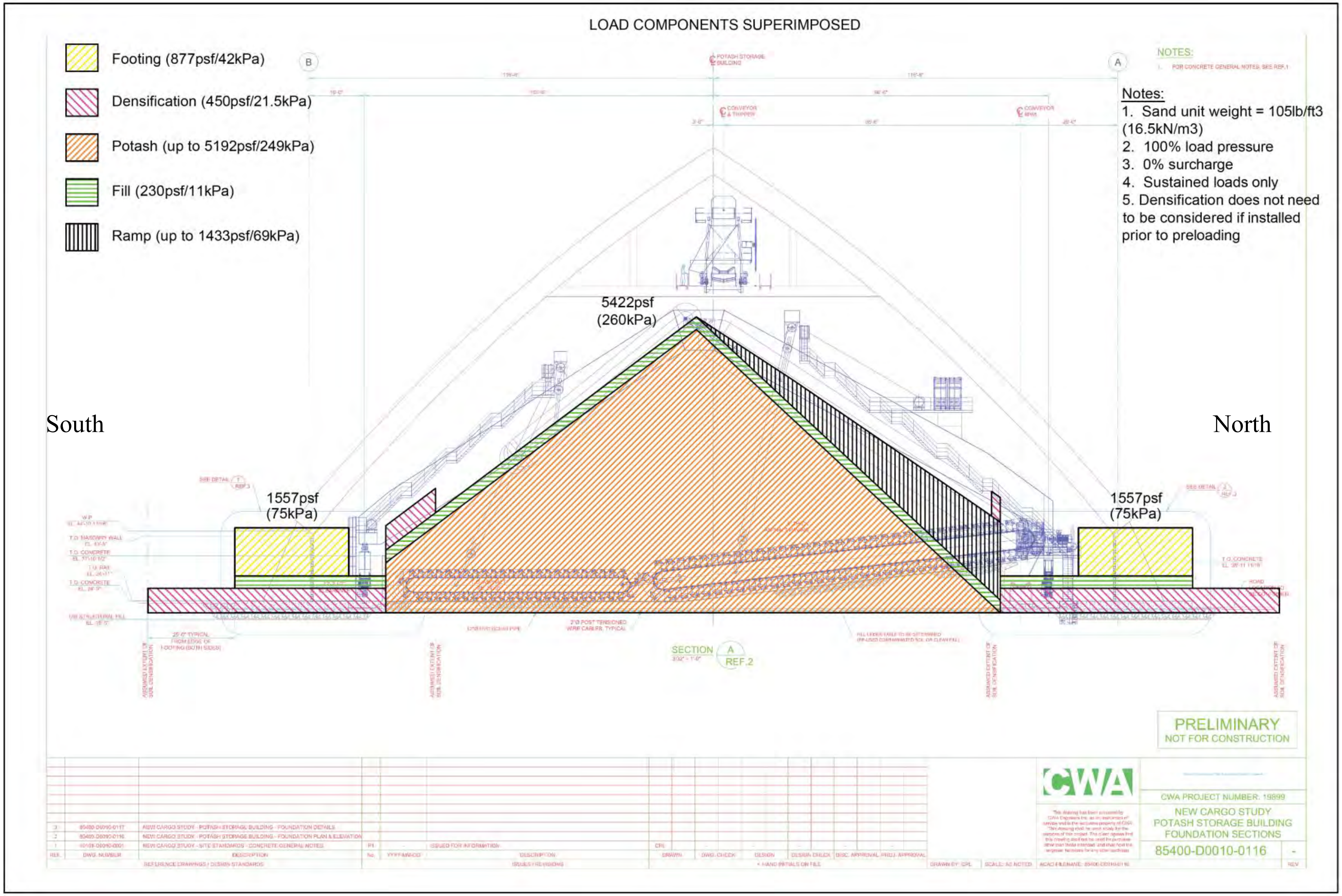


Figure 7-23: Overlay of sand preload height and the current preload configuration

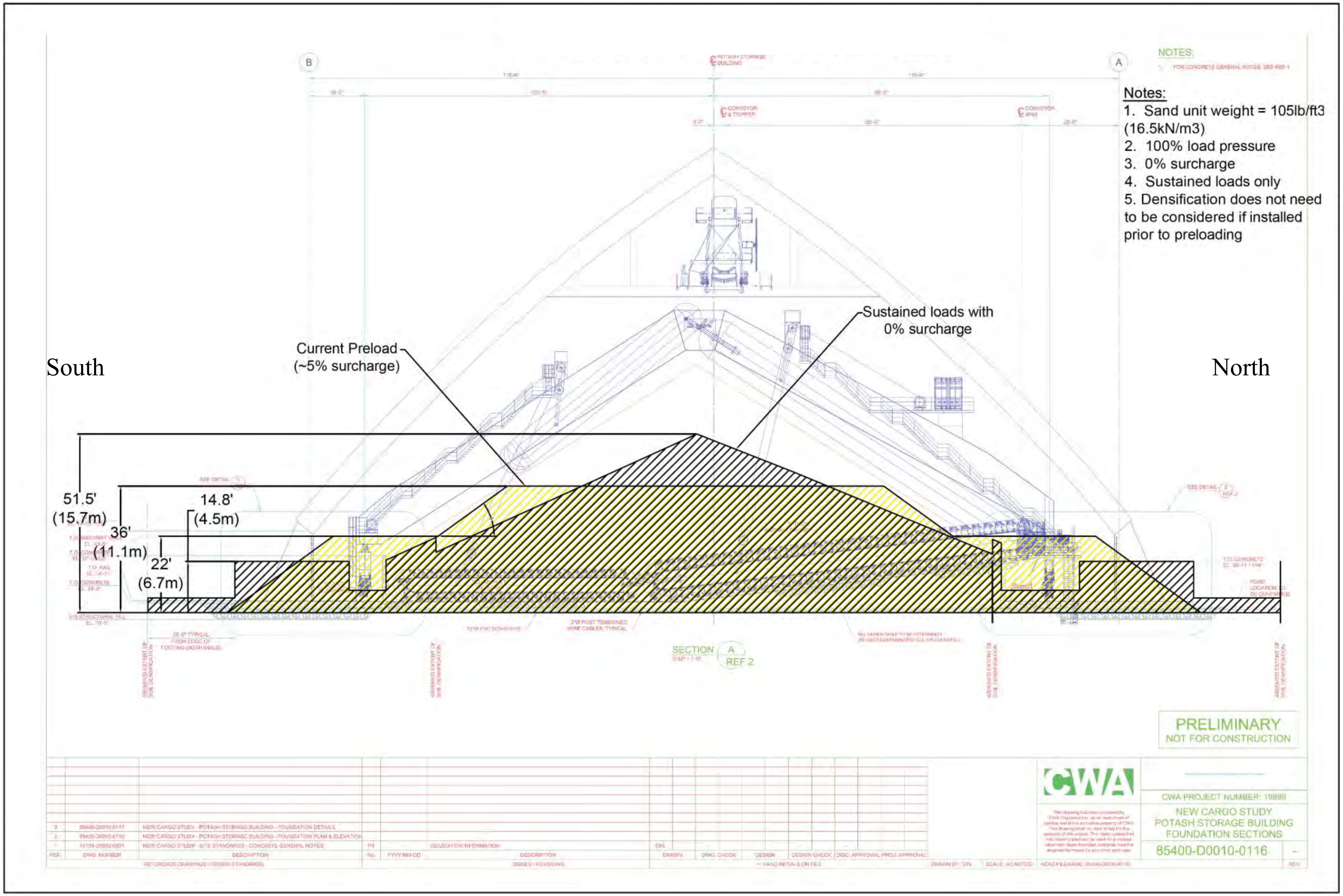


Figure 7-24: Post-construction settlement along a North-South building section
(No preload, 6 soil profiles)

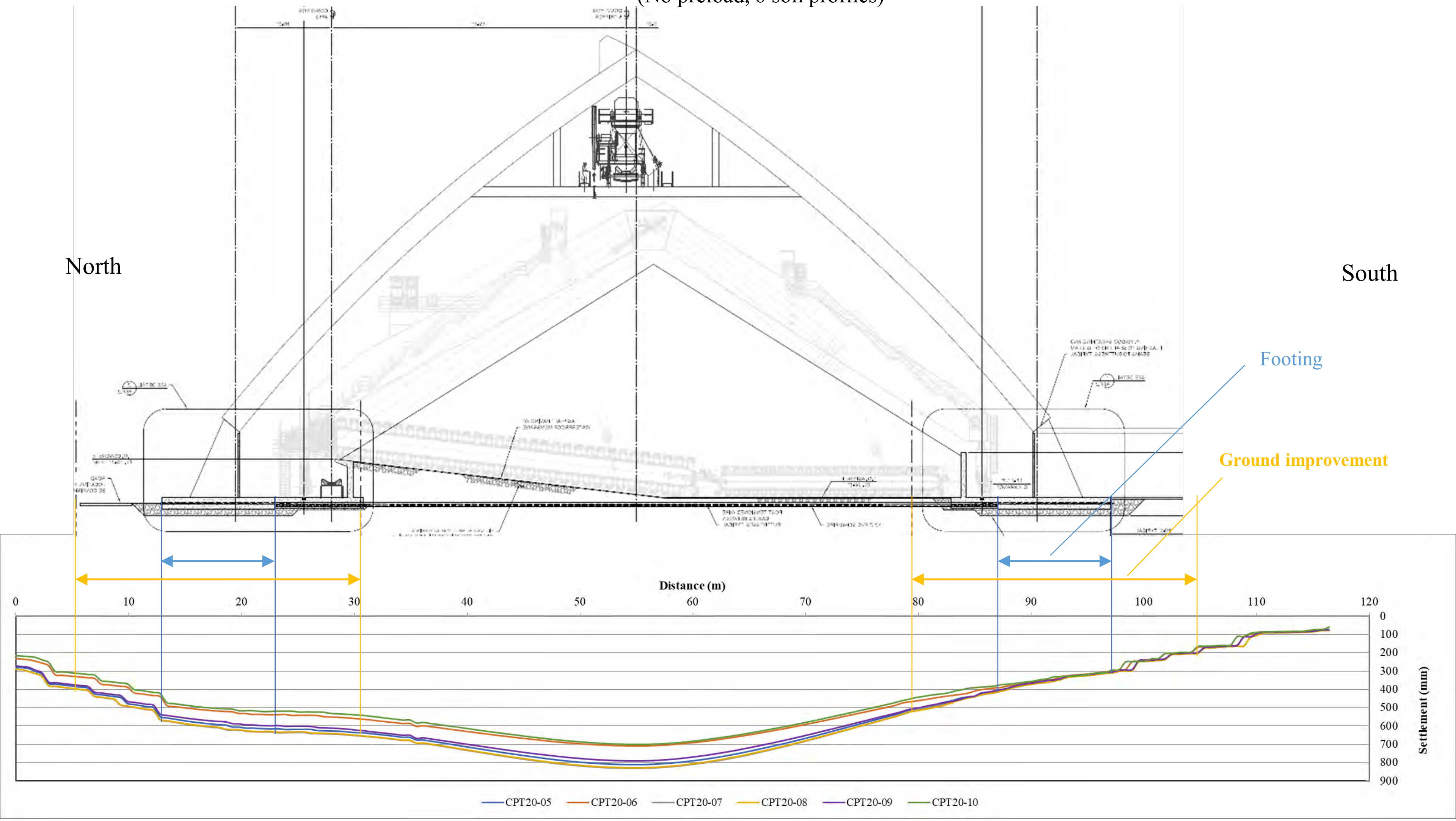


Figure 7-25: Post-construction settlement along a North-South building section
(5% surcharge, 6 month preload, 6 soil profiles)

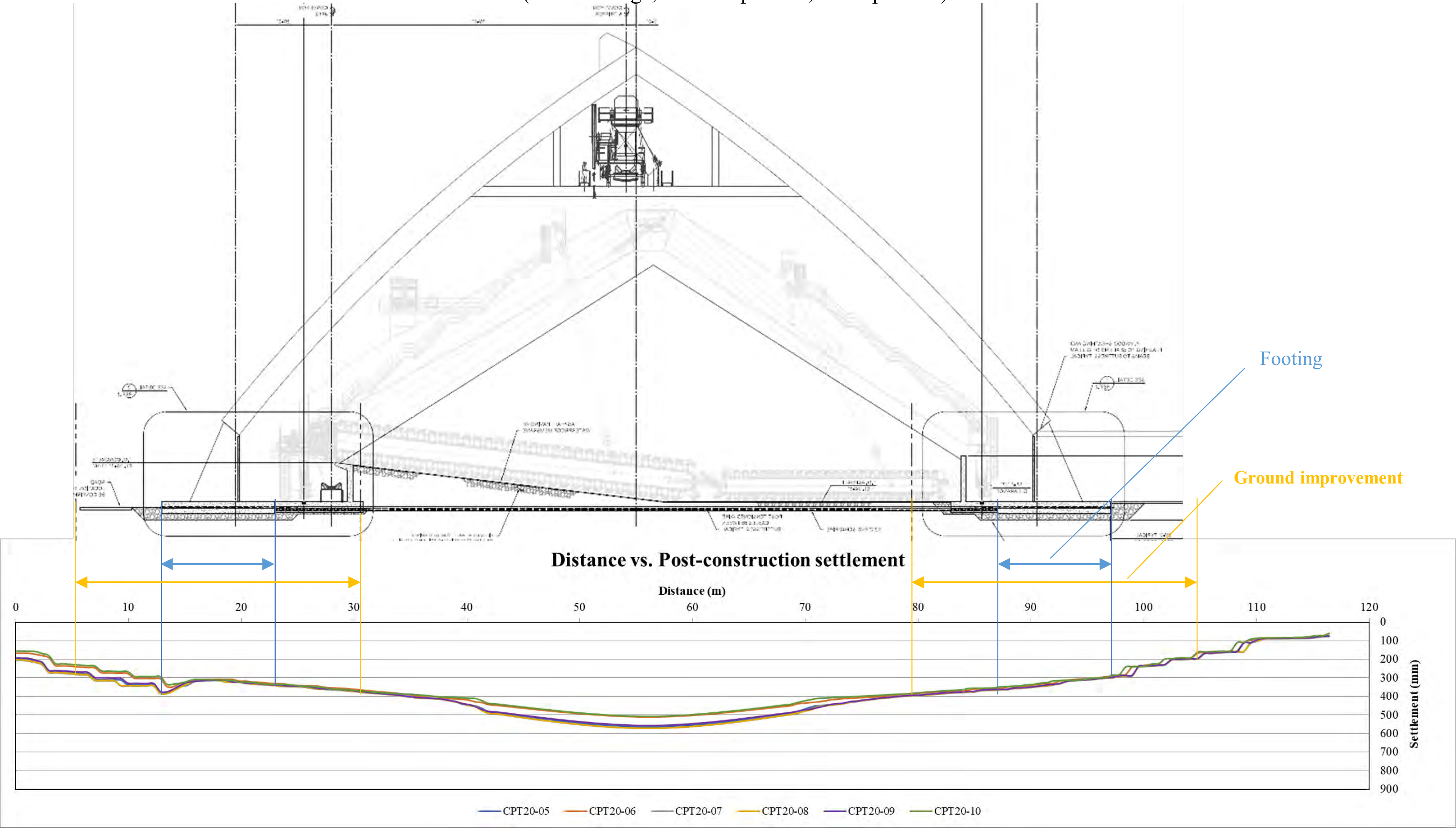


Figure 7-26: Post-construction settlement profile
(5% surcharge, 6 month preload)

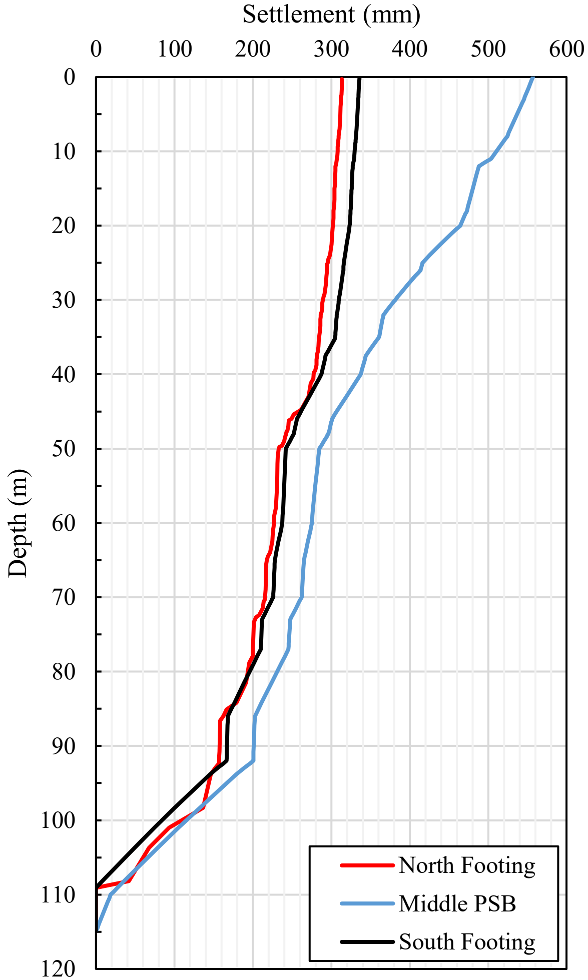


Figure 7-27: Effect of preload surcharge and duration on 50 years post-construction settlements

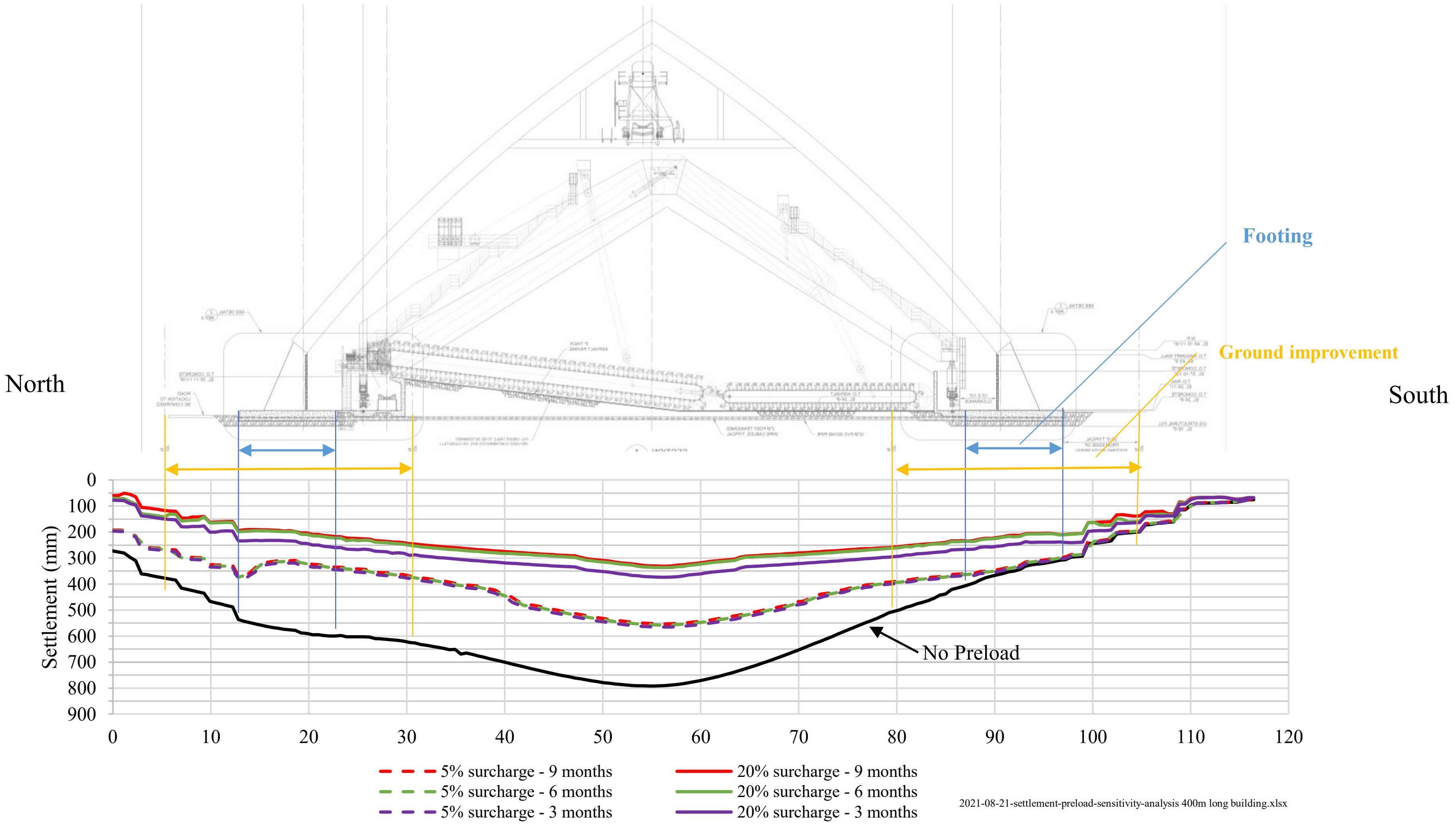
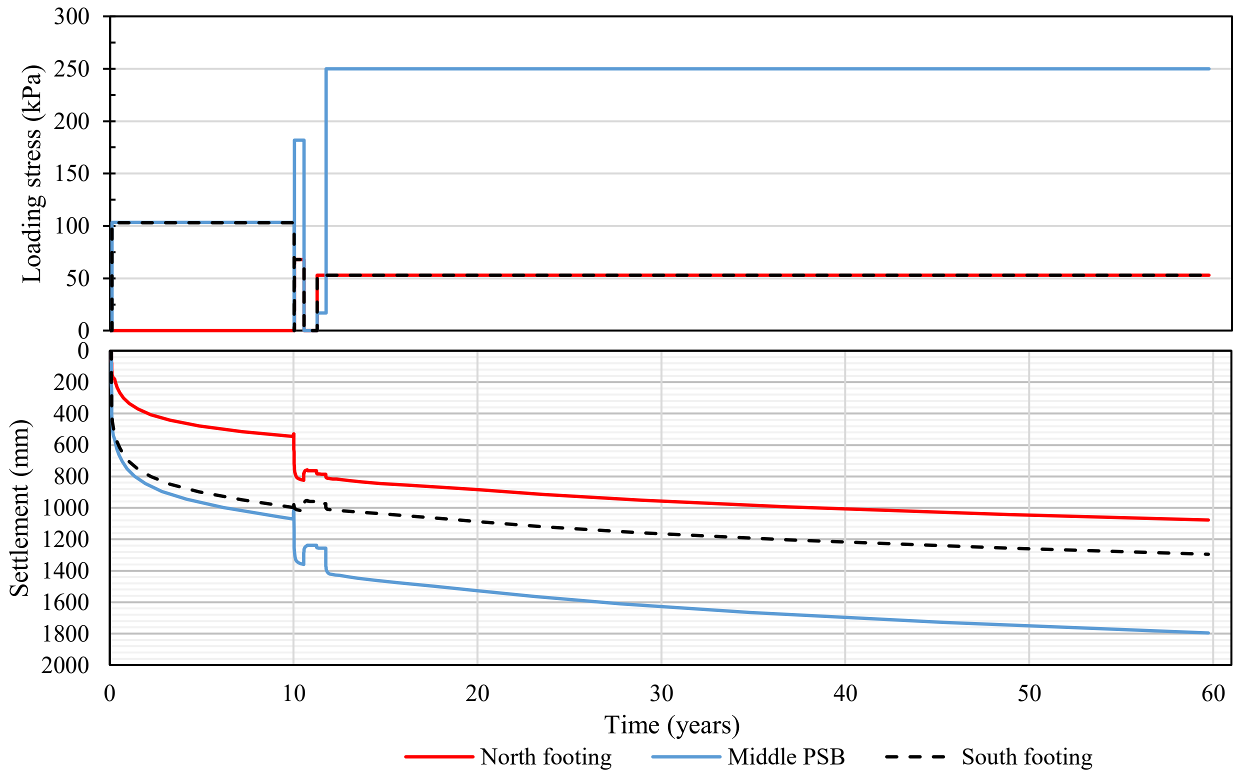


Figure 7-28: Ground surface pressure and total settlements versus time
(5% surcharge, 6 month preload)

Loading stress vs. Time
5% surcharge - 6 months preload

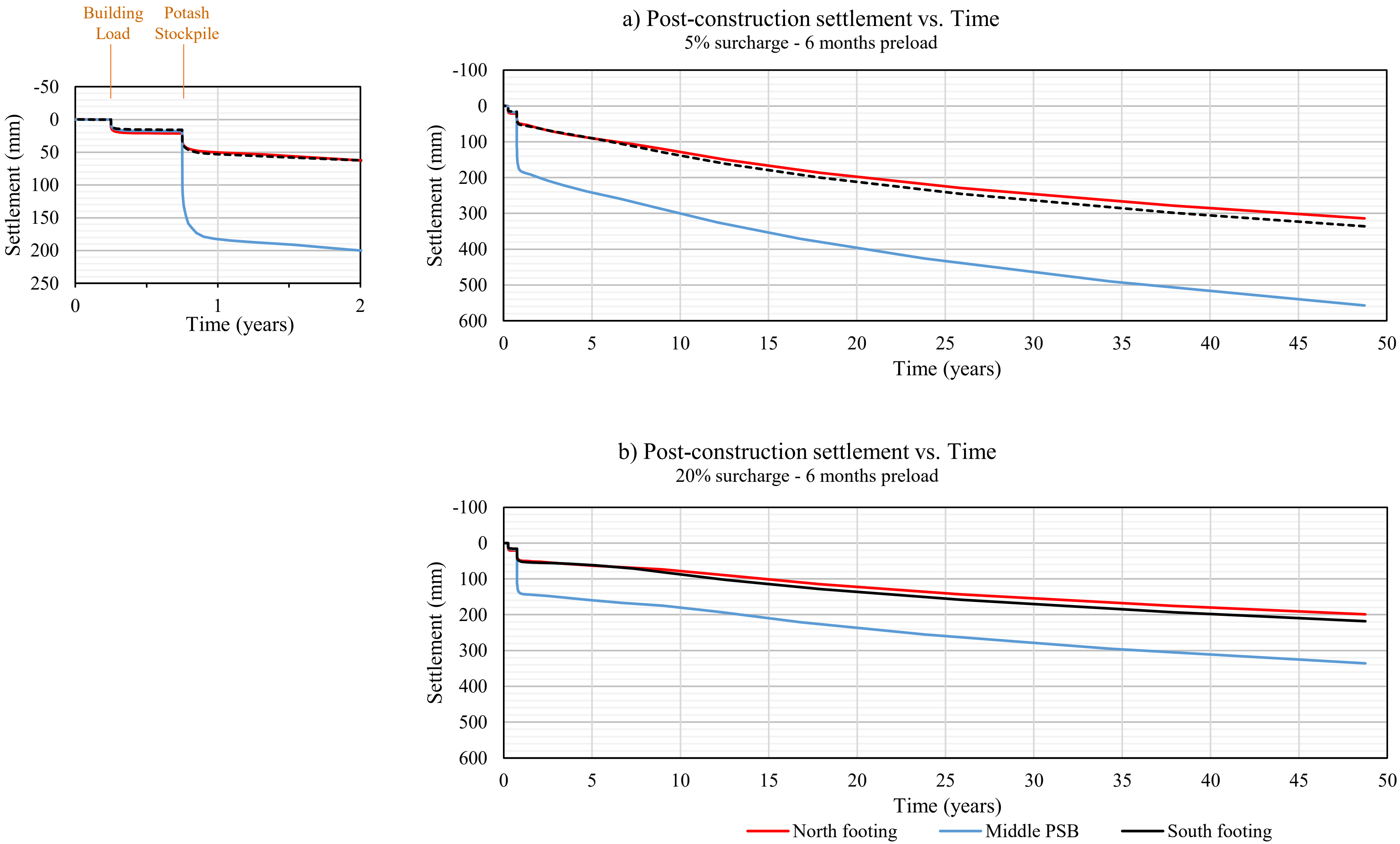
Total settlement vs. Time
5% surcharge - 6 months preload




2021-08-21-settlement-preload-sensitivity-analysis 400m long building.xlsx

Reference time: Commencement of coal loading

Figure 7-29: Post-construction settlement vs. Time




2021-08-21-settlement-preload-sensitivity-analysis 400m long building.xlsx

Figure 7-30: Post-construction settlement in the east-west direction for 2 preload assumptions, (1) one-stage preload, (2) two-stage preload with overlap (6 month preload)

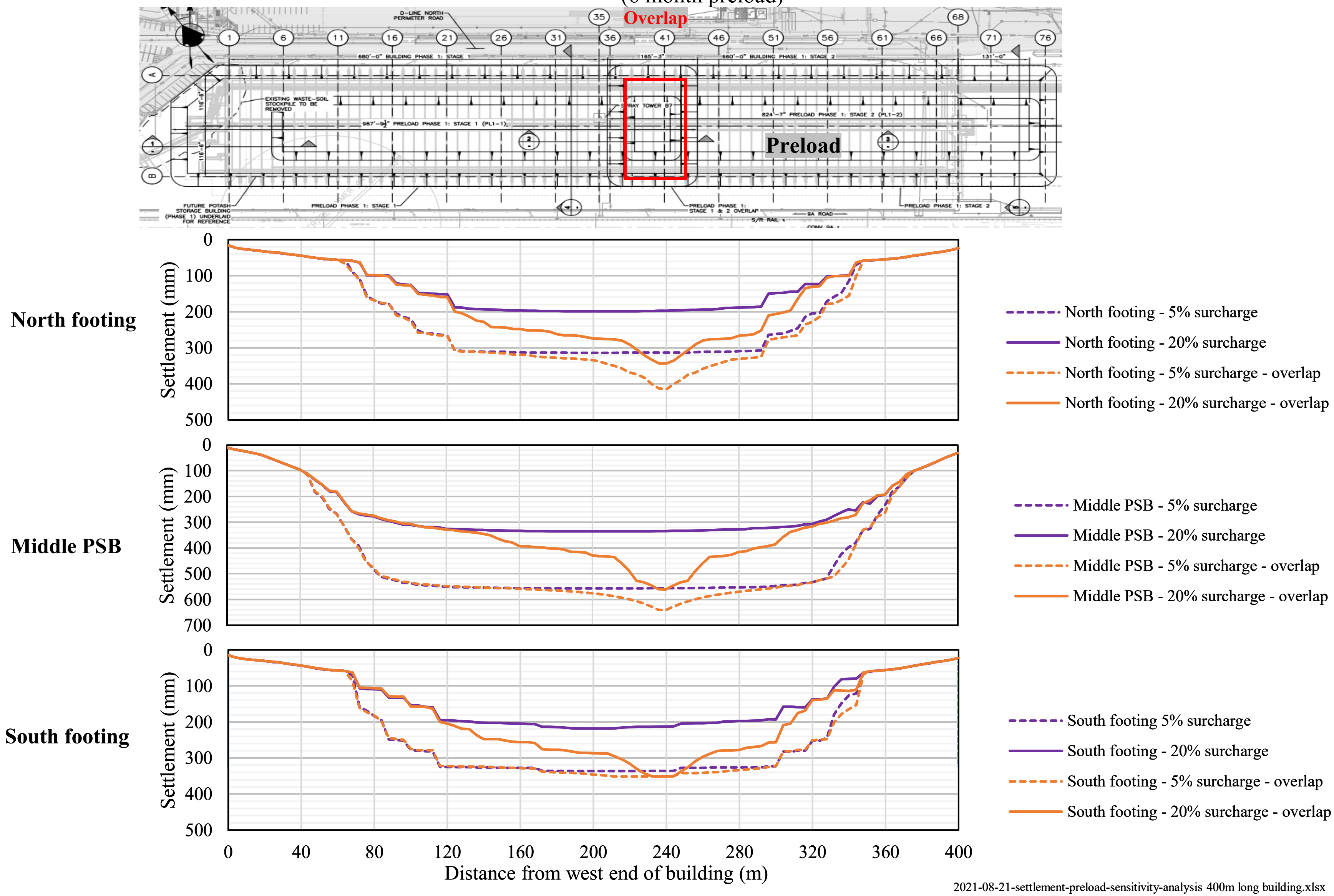


Figure 7-31: Manifestation of ground surface undulations to an induced undulations at depth

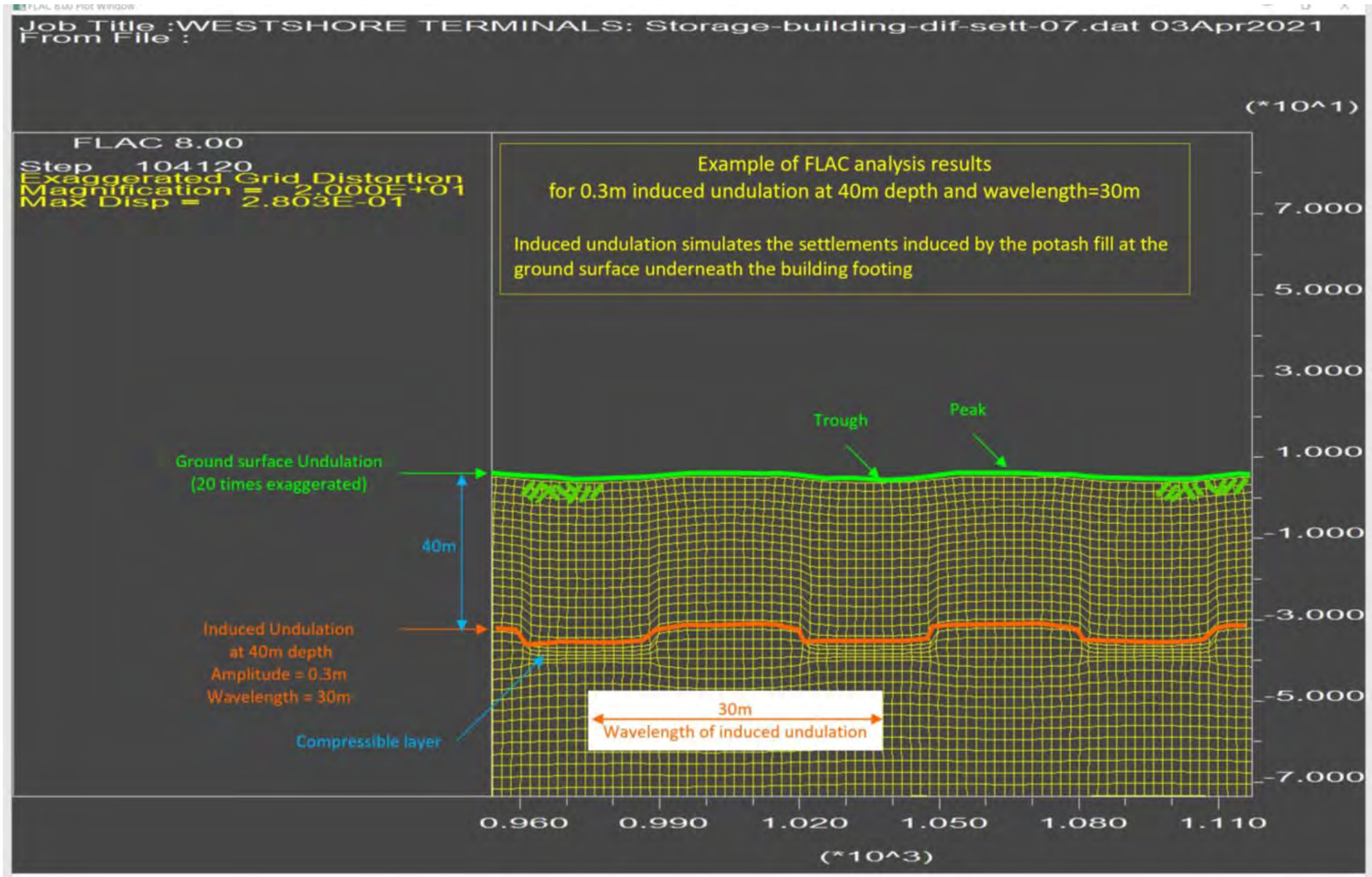


Figure 7-32: Results of parametric analyses for the ground surface undulation as a result of induced undulations at depth

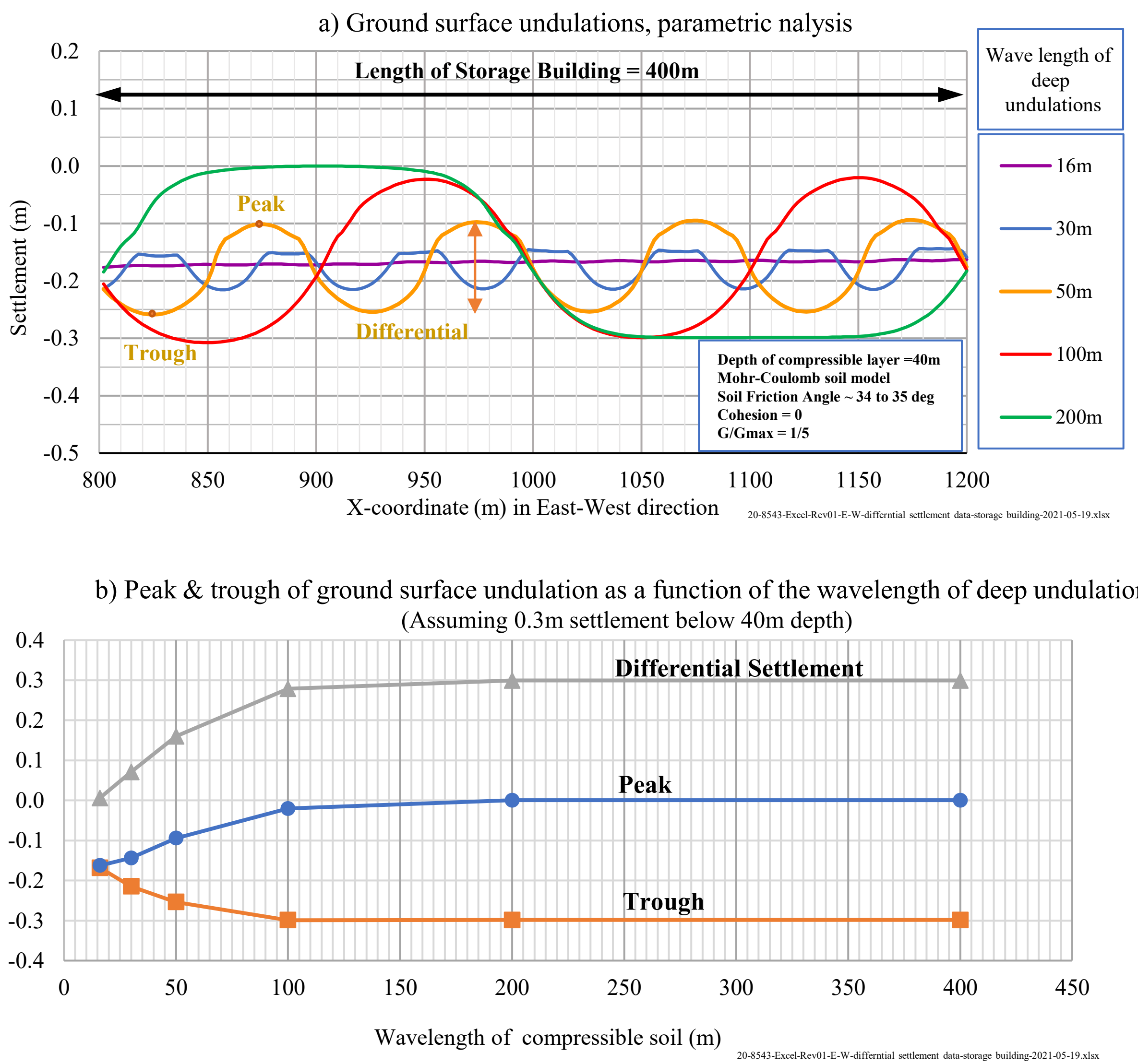
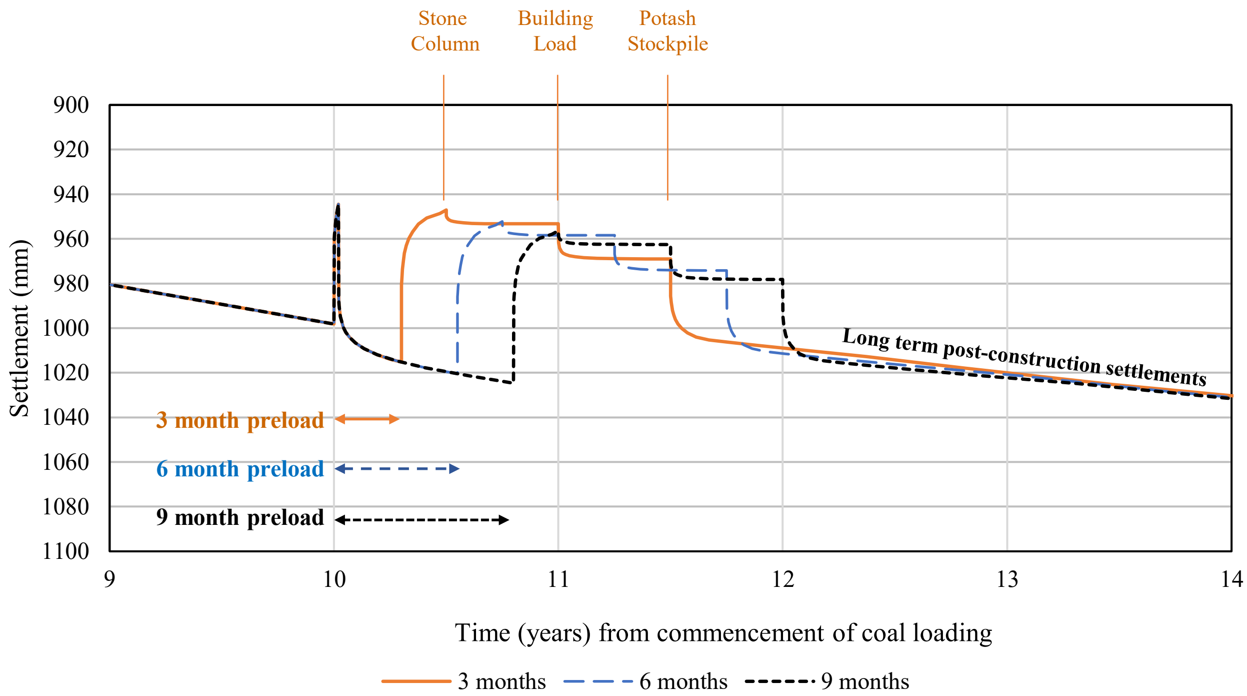
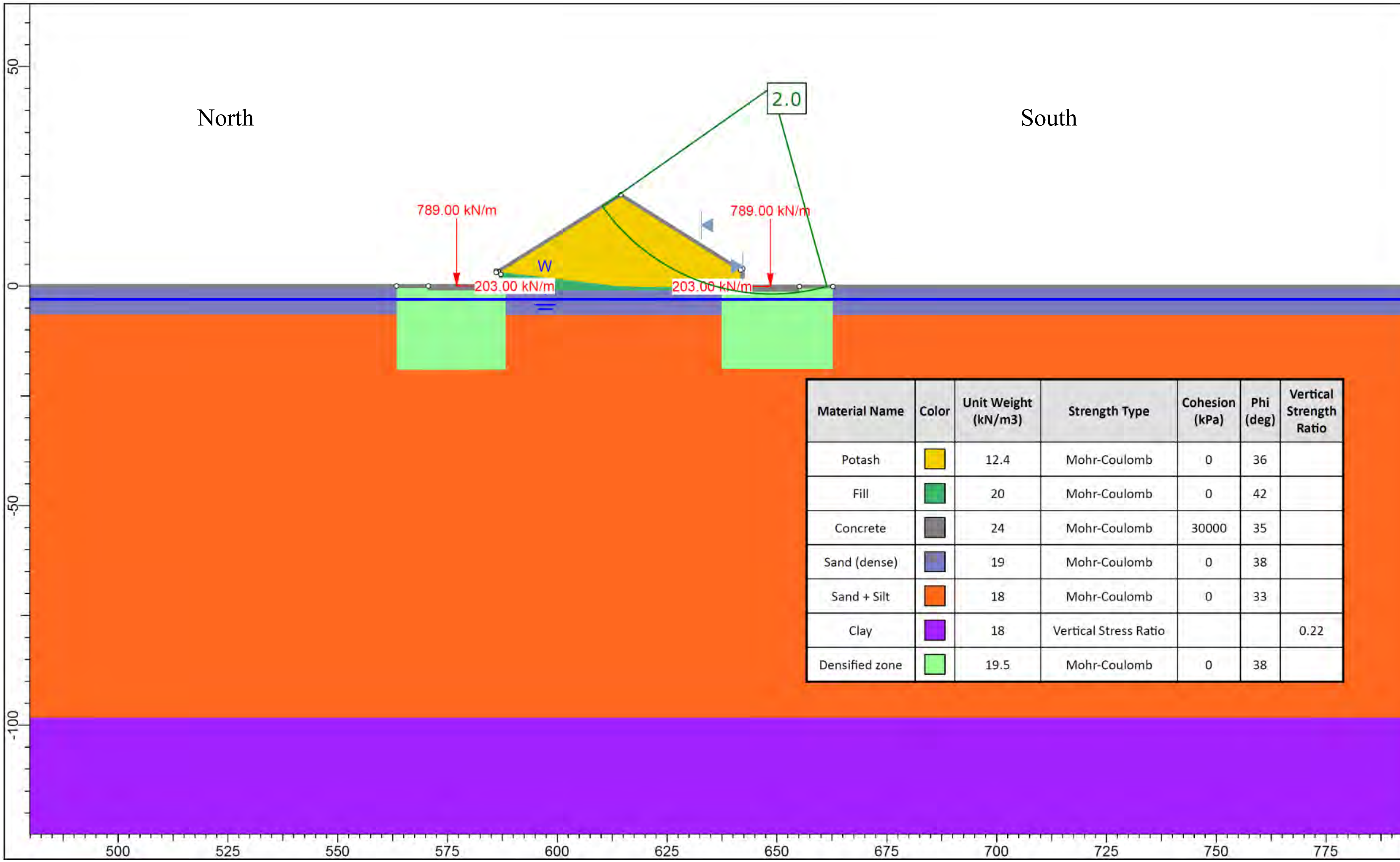


Figure 7-33: Total settlement versus time for 3 preload durations
(5% surcharge - South footing)



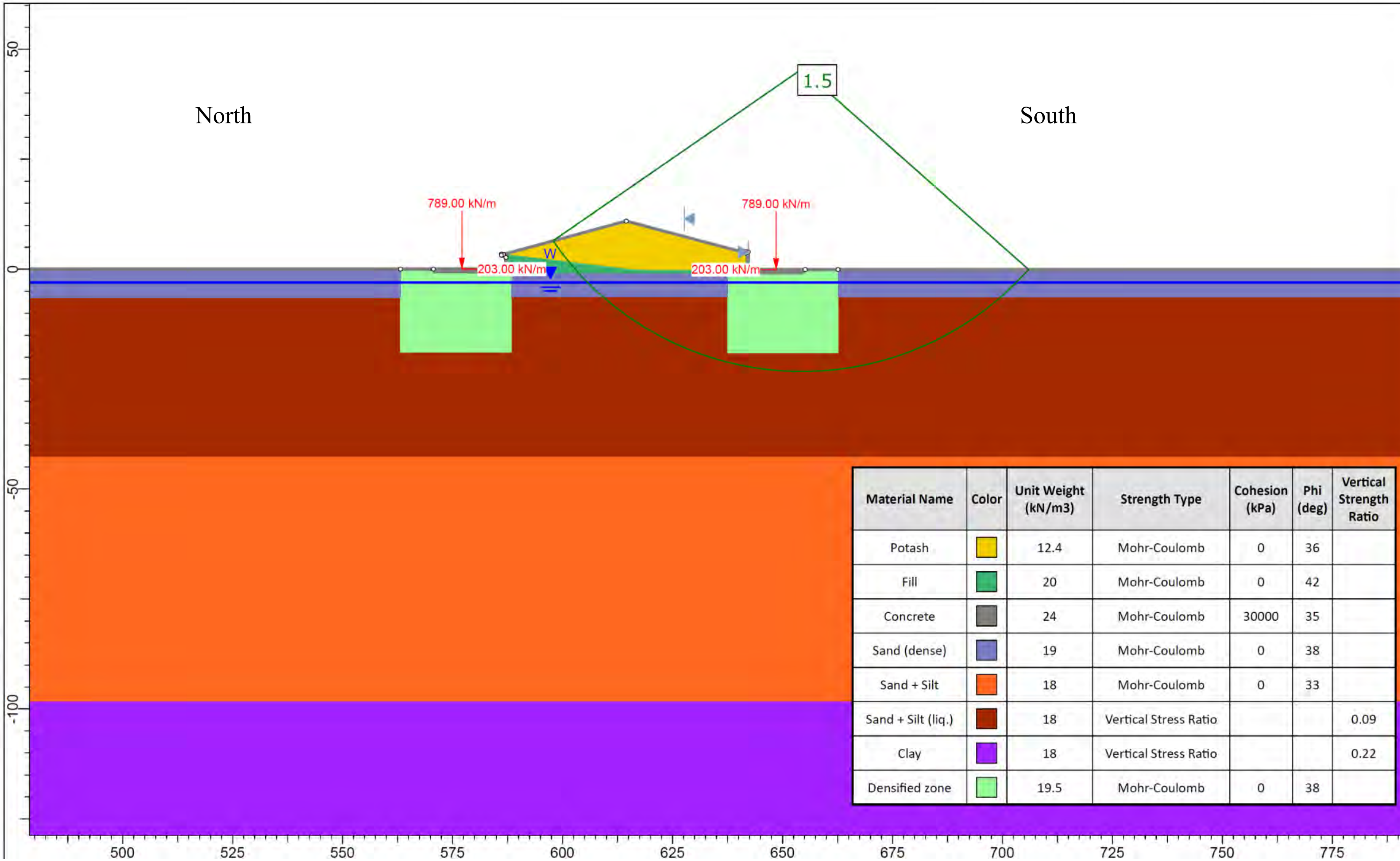

2021-08-21-settlement-preload-sensitivity-analysis 400m long building.xlsx

Figure 7-34: Storage building North-South slope stability analysis - Static condition



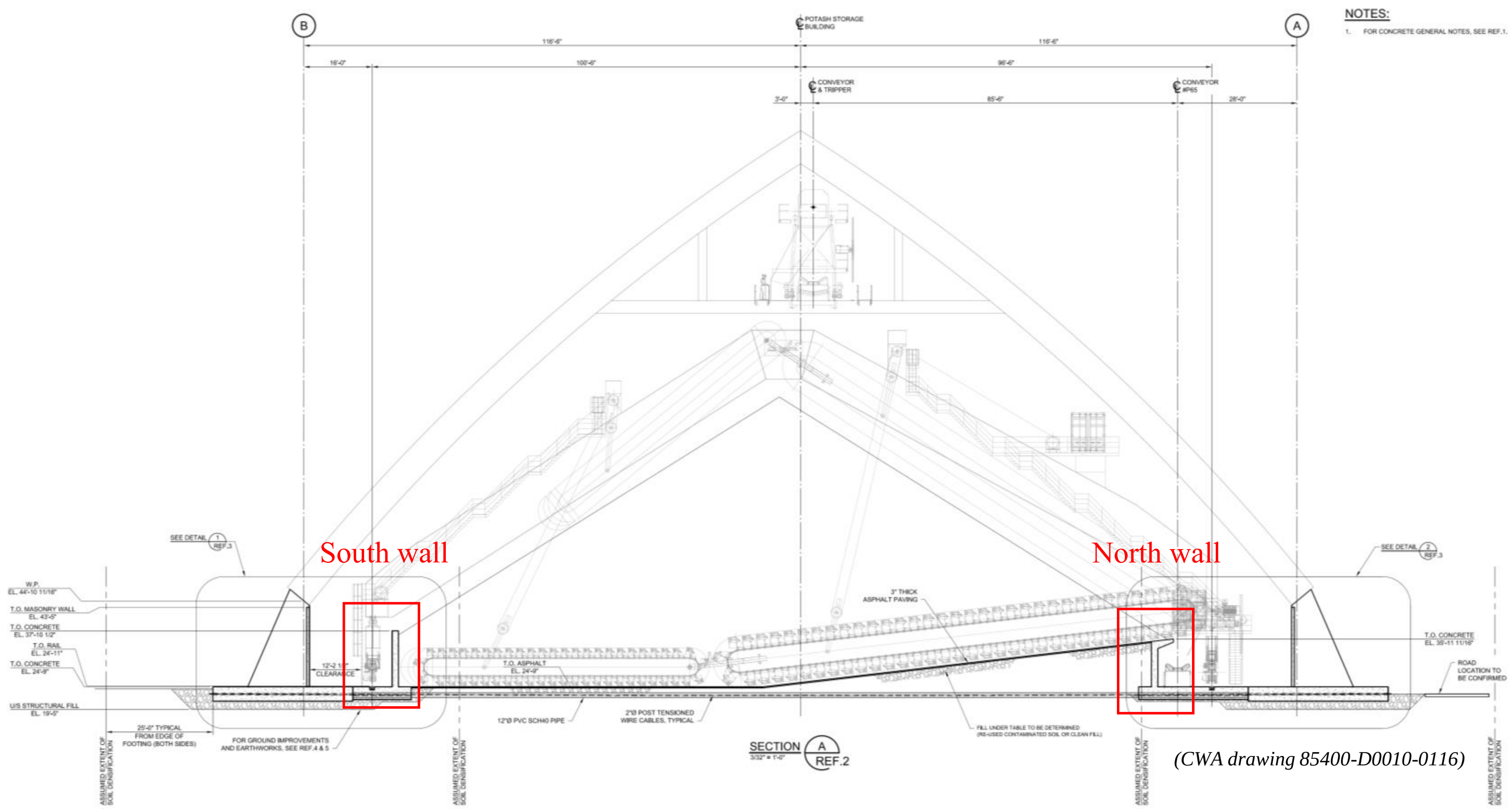

| Material Name | Color | Unit Weight (kN/m3) | Strength Type | Cohesion (kPa) | Phi (deg) | Vertical Strength Ratio |
|---|---|---|---|---|---|---|
| Potash | | 12.4 | Mohr-Coulomb | 0 | 36 | |
| Fill | | 20 | Mohr-Coulomb | 0 | 42 | |
| Concrete | | 24 | Mohr-Coulomb | 30000 | 35 | |
| Sand (dense) | | 19 | Mohr-Coulomb | 0 | 38 | |
| Sand + Silt | | 18 | Mohr-Coulomb | 0 | 33 | |
| Clay | | 18 | Vertical Stress Ratio | | | 0.22 |
| Densified zone | | 19.5 | Mohr-Coulomb | 0 | 38 | |

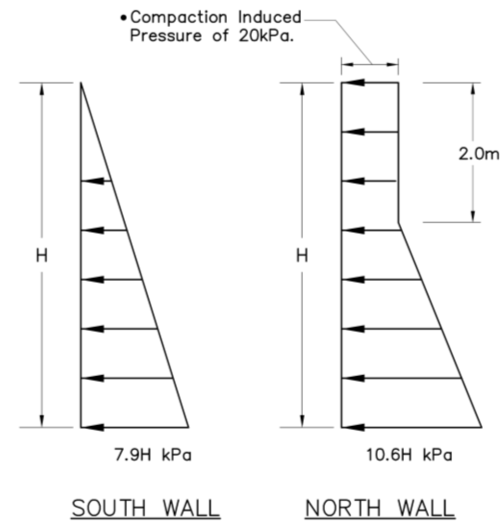
Figure 7-35: Storage building North-South slope stability analysis
Post-liquefaction (residual) condition- A2475



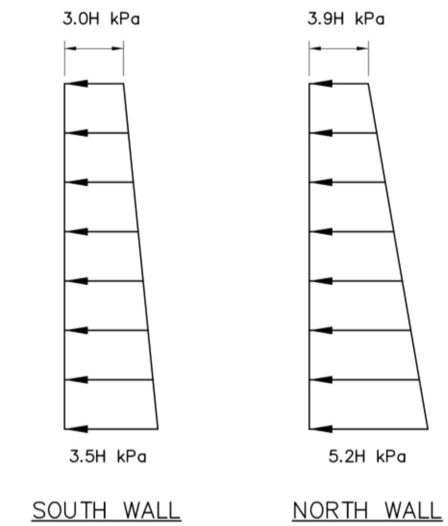

| Material Name | Color | Unit Weight (kN/m3) | Strength Type | Cohesion (kPa) | Phi (deg) | Vertical Strength Ratio |
|---|---|---|---|---|---|---|
| Potash | | 12.4 | Mohr-Coulomb | 0 | 36 | |
| Fill | | 20 | Mohr-Coulomb | 0 | 42 | |
| Concrete | | 24 | Mohr-Coulomb | 30000 | 35 | |
| Sand (dense) | | 19 | Mohr-Coulomb | 0 | 38 | |
| Sand + Silt | | 18 | Mohr-Coulomb | 0 | 33 | |
| Sand + Silt (liq.) | | 18 | Vertical Stress Ratio | | | 0.09 |
| Clay | | 18 | Vertical Stress Ratio | | | 0.22 |
| Densified zone | | 19.5 | Mohr-Coulomb | 0 | 38 | |

Figure 7-36: Horizontal wall pressure diagram for potash retaining walls




(a) Static pressure




(b) Static + Seismic pressure

Assumptions:

· Static Stockpile Slope: 33°
· Seismic Stockpile Slope: 15° (assumes stockpile at 60% capacity).
· Unit Weight: Potash - 12.4 kN/m³, Soil backfill - 20 kN/m³.
· Friction Angle of potash and soil backfill: 36 degrees
· At-rest pressure for static loading conditions.
· Active pressure for seismic loading conditions.
· Seismic peak ground acceleration of 0.21g
. Fully drained loading conditions.

·Seismic wall pressures based on the Mononobe-Okabe procedure.
·Wall pressures are approximate, actual pressures will depend on wall stiffness, stockpile slope, unit weight of stockpile, and any surcharge pressures.
·Surcharge loads or any dynamic loading to be reviewed by Braun/NAGL.
·All pressures are unfactored and in metric Units (m & kPa)
·Stockpile and retaining wall configuration based on CWA drawing 85400-D0010-0116 P1 (2020-10-30).

Table 7-4: Factored bearing resistances for static and A2475 earthquake and inclined loading reduction factors for proposed transfer towers and conveyor bents

| Location | Footing Size (m) | Underside of footing depth (m) | Reference CPTs | Existing Crust thickness (m) | Densification depth (m) | Estimated depth of improvement (m) | STATIC | SEISMIC | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Factored Bearing Resistance (kPa) | Ultimate Bearing Resistance (kPa) | Factored Bearing Resistance (During Shaking) Resistance Factor =1.0 (kPa) | Factored Bearing Resistance (Post Earthquake) - Resistance Factor =0.67 (kPa) |
| TT#P42 | 20x20 | 1.2 | CPT20-03, CPT16-05 | 6 | n/a * | n/a | 188 | 100 | 100 | 67 |
| TT#P47 | 20x20 | 1.2 | CPT16-01, CPT13-01 | 6 | n/a * | n/a | 188 | 100 | 100 | 67 |
| TT#P52 | 25x25 | 1.2 | CPT20-05, CPT20-06, CPT13-03 | 6 | 18 | 14 | 188 | 150 | 150 | 100 |
| TT#P57 | 15x15 | 1.2 | CPT20-05, CPT20-06, CPT13-03 | 6 | 18 | 14 | 188 | 185 | 185 | 125 |
| TT#P67 | 15x15 | 1.2 | CPT20-11 | 3.3 | 18 | 15 | 188 | 200 | 200 | 130 |
| TT#P77 | 20x20 | 1.2 | CPT20-13, CPT18-01 | 5 | n/a * | n/a | 188 | 90 | 90 | 60 |
| Sampling Tower | 12x12 | 1.2 | CPT20-13, CPT18-01 | 5 | n/a * | n/a | 188 | 75 | 75 | 50 |
| Conveyor bents | 20x6 | 1 | varies | 3.3 | 10 | 10 | 188 | 130 | 130 | 85 |
| | | | | 6 | n/a * | n/a | 188 | 75 | 75 | 50 |

* Shear Induced Settlement in non-densified soils: Where densification is eliminated, seismically induced shear settlements additional to previously reported post seismic consolidation and vertical displacements would occur. The amount of shear induced settlement would be a function of footing loads, earthquake intensity, soil conditions, etc., and are estimated to be in the range of 300 to 400mm based on current footing configurations and loading.

Note:
- All bearing resistances values provided are contact pressures at the underside of footings.
- Greater static bearing resistance values may be feasible for specific conditions and settlement tolerances, and can be provided if required.

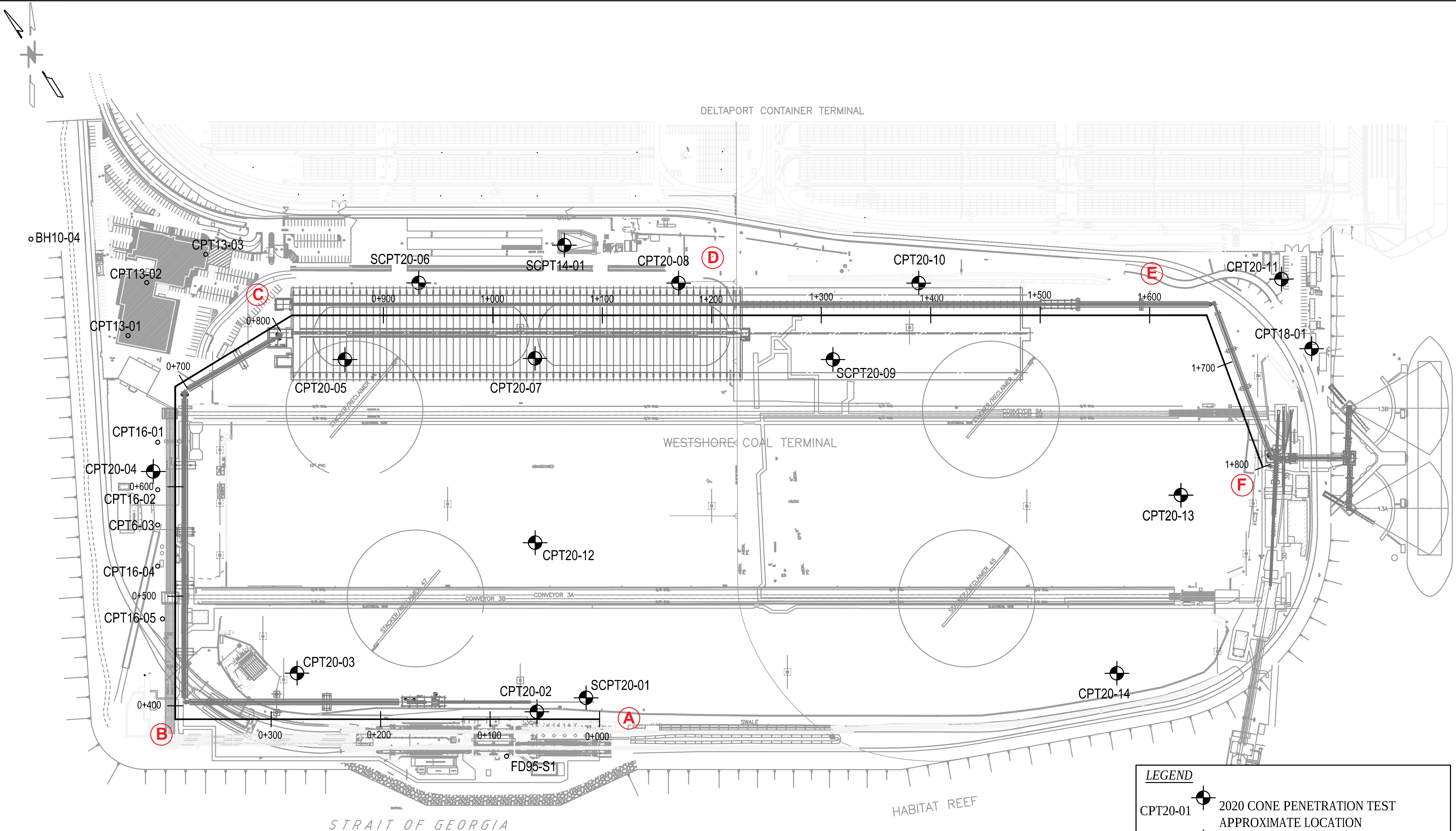
| Inclined Loading Reduction Factors | |
|---|---|
| Load Inclination Ratio (H/V) | Reduction Factor |
| 0.05 | 0.93 |
| 0.10 | 0.85 |
| 0.15 | 0.80 |
| 0.20 | 0.75 |

Note: For seismic loading only

*Punching resistance of footings on non liquefiable crust 2020-12-02 Rev 4.xls*

# Appendix A

# Geotechnical Soil Profile




*LEGEND*

CPT20-01 — 2020 CONE PENETRATION TEST APPROXIMATE LOCATION

SCPT20-01 — 2020 SEISMIC CONE PENETRATION TEST APPROXIMATE LOCATION

PROJECT ALIGNMENT BASE PLAN: DWG. NO. 40101-D0000-0000 PROJECT 19899 BY CWA ENGINEERS, DATED: 2020-08-31

BRAUN GEOTECHNICAL LTD.

Naesgaard-Amini Geotechnical Ltd. soils, foundations and earthquake engineering

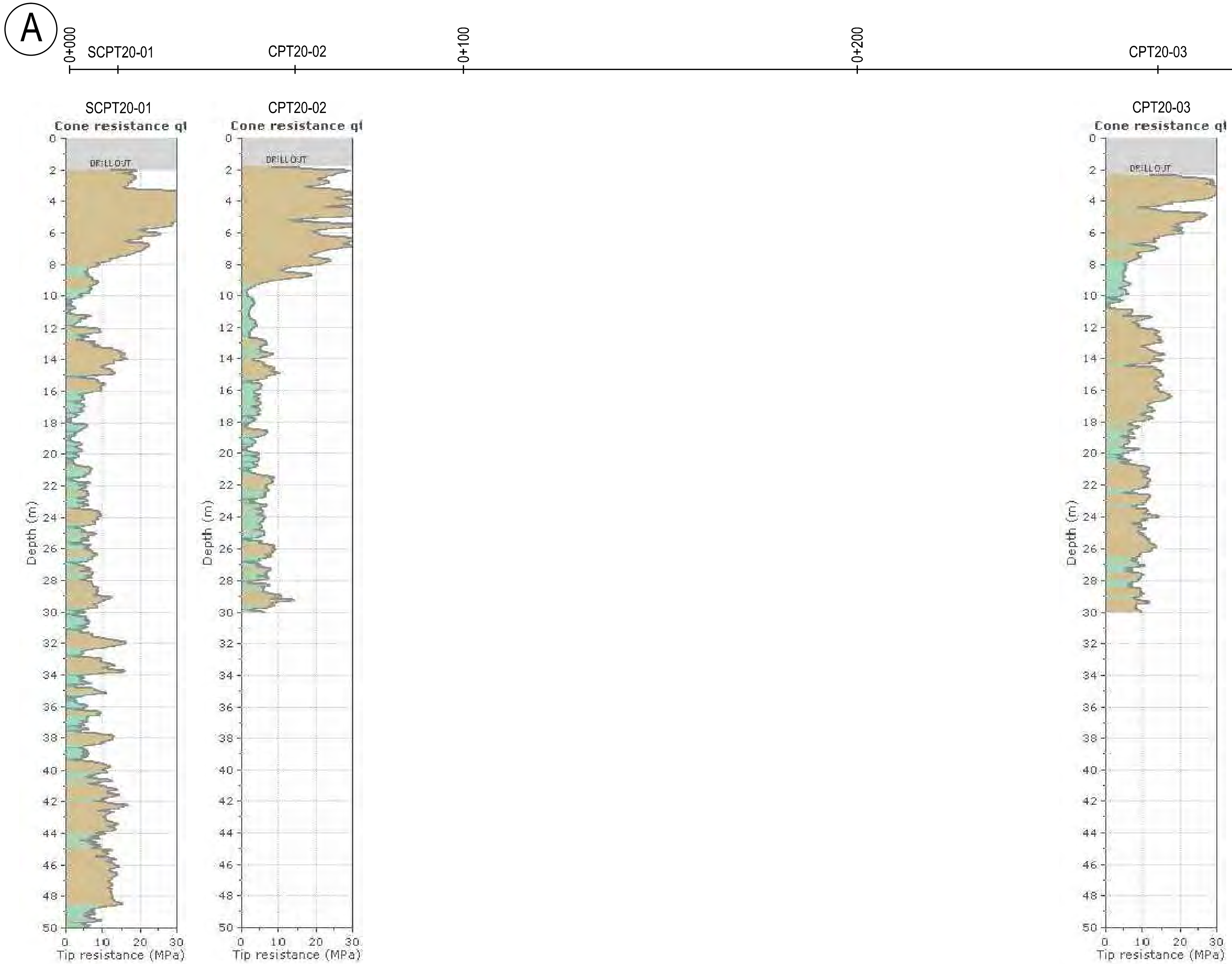
| Rev. | Description | Date |
|---|---|---|
| | | |

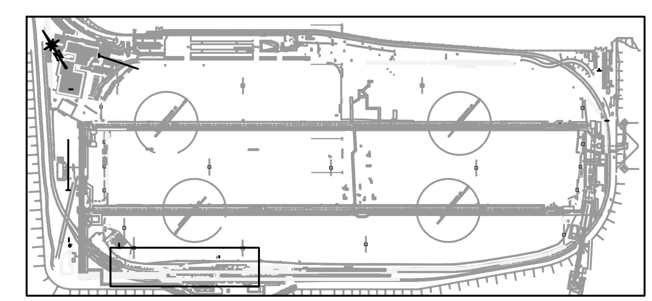
Client: Westshore Terminals

Project: Terminal Upgrades, Delta, BC

Title: GEOTECHNICAL SOIL PROFILE
Site Plan

| Project no. | Drawn | Design | Checked | Date | Scale | Drawing no. |
|---|---|---|---|---|---|---|
| 20-8543 | DD | PB | SS | November 26, 2020 | 1:3500 | APPENDIX A-1 |



KEY PLAN - NTS

| Rev. | Description | Date |
|---|---|---|
| | | |

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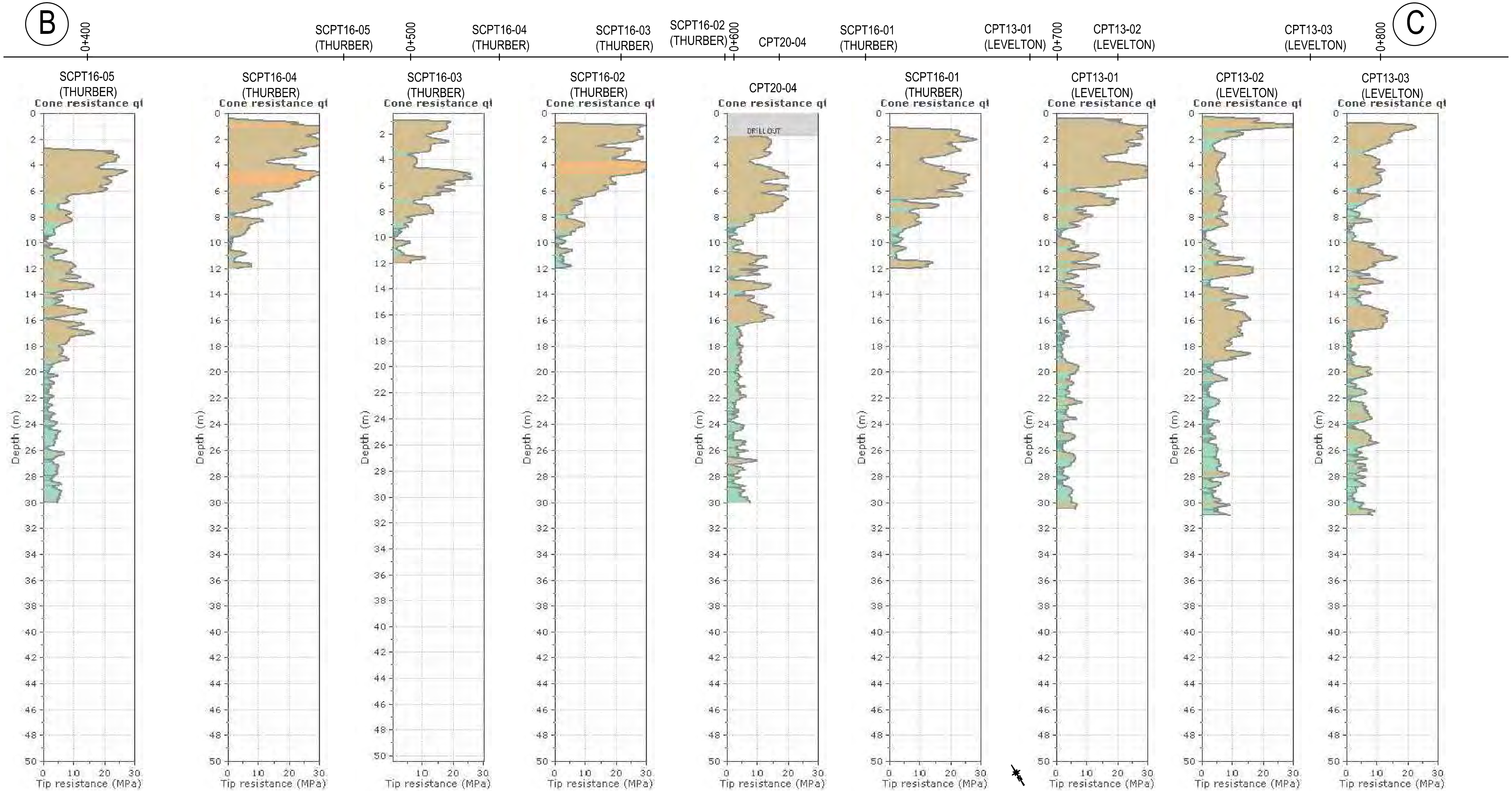
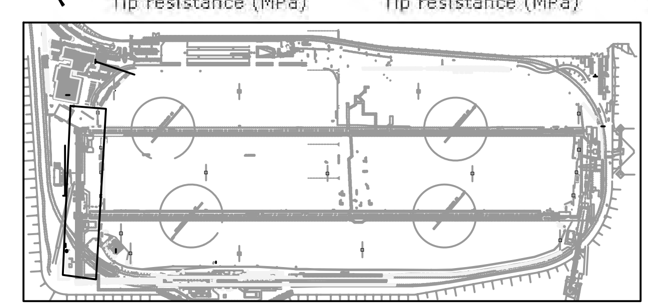
| | |
|---|---|
| Client | Westshore Terminals |
| Project | Terminal Upgrades<br>Delta, BC |
| Title | GEOTECHNICAL SOIL PROFILE<br>Section A-B |

| Project no. | Drawn | Design | Checked | Date | Scale | Drawing no. |
|---|---|---|---|---|---|---|
| 20-8543 | DD | PB | SS | November 26, 2020 | AS NOTED | APPENDIX A-2 |




SCALE: H:1:1250
V:1:300
H AXIS OF LOG: NTS
STATIONING INCREASING CLOCKWISE

KEY PLAN - NTS



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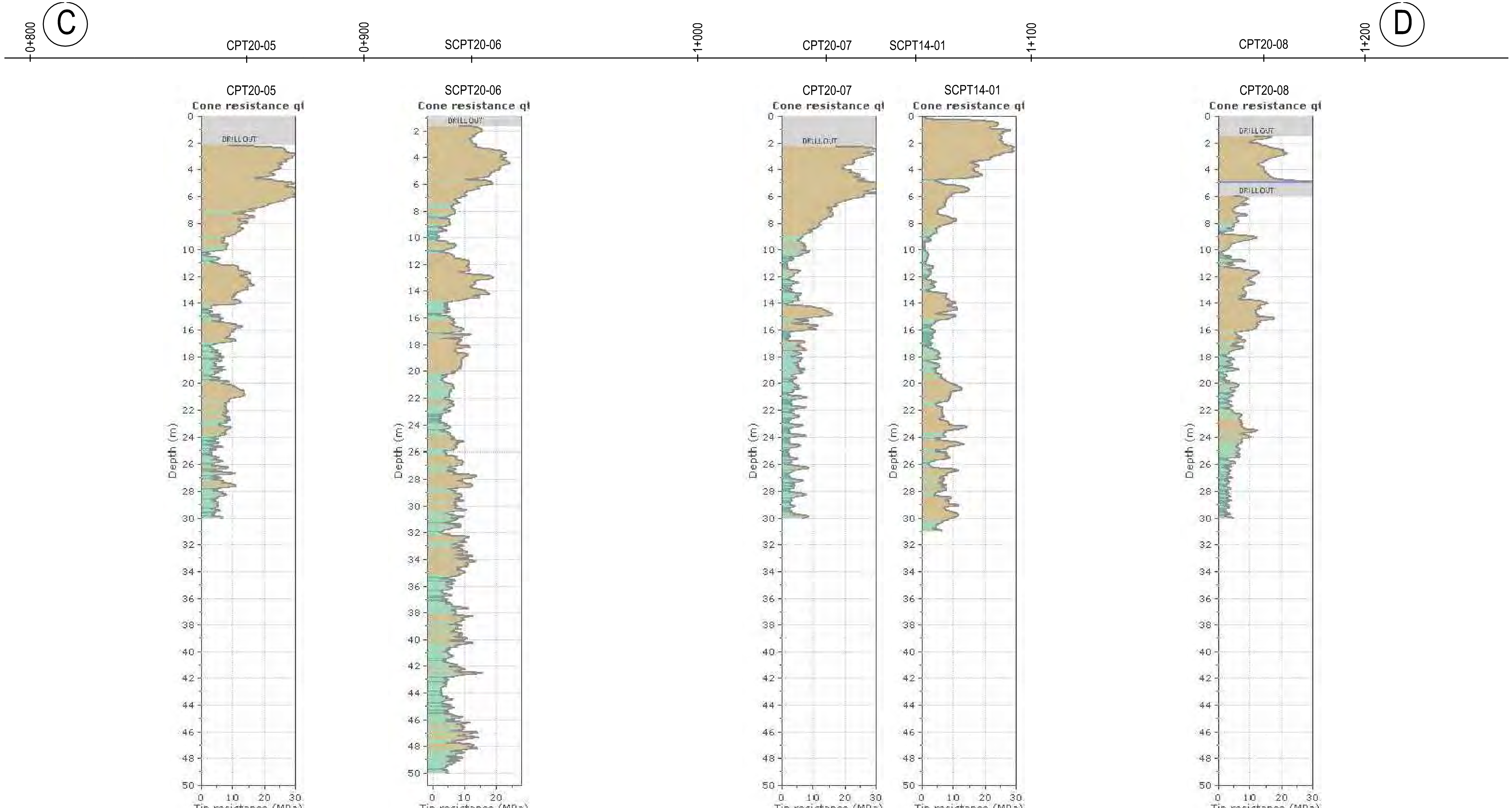
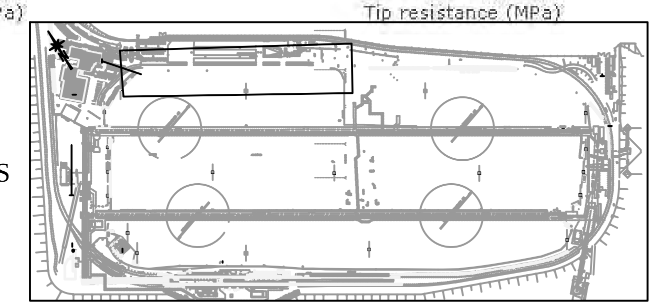
| Rev. | Description | Date |
|---|---|---|
| | | |

| | |
|---|---|
| Client | Westshore Terminals |
| Project | Terminal Upgrades<br>Delta, BC |
| Project no. | 20-8543 |
| Drawn | DD |
| Design | PB |
| Checked | SS |
| Title | GEOTECHNICAL SOIL PROFILE<br>Section B-C |
| Date | November 26, 2020 |
| Scale | AS NOTED |
| Drawing no. | APPENDIX A-3 |




KEY PLAN - NTS



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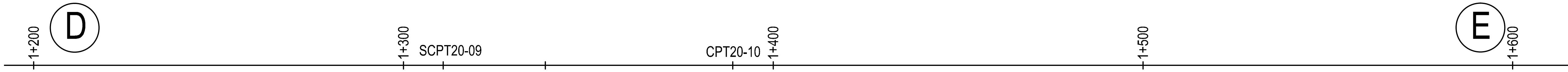
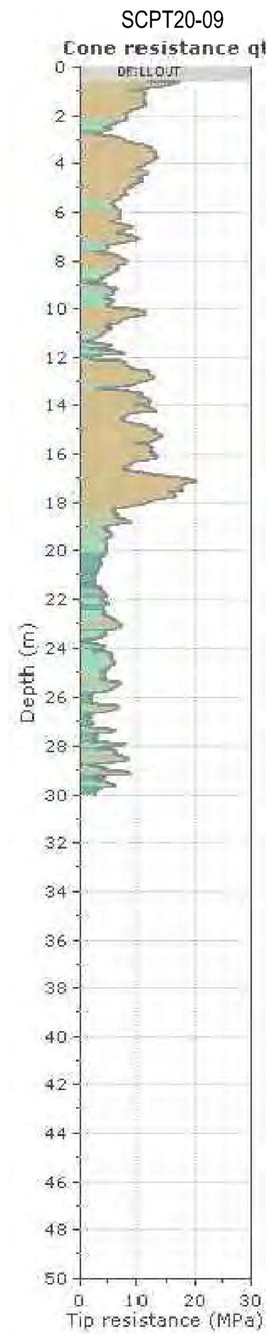
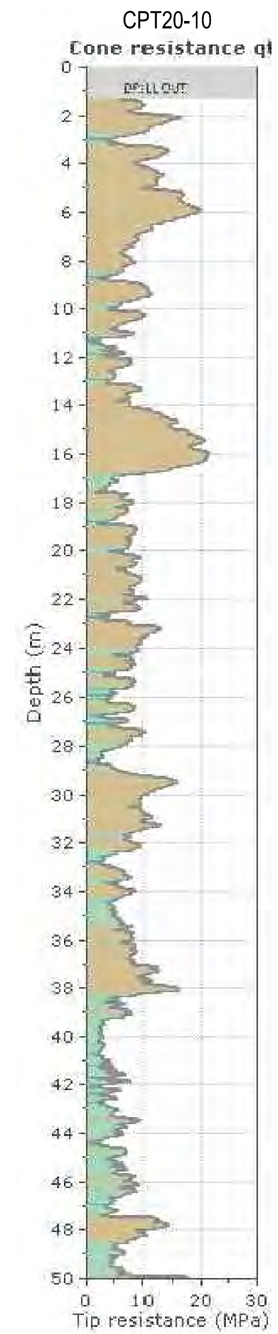
| Rev. | Description | Date |
|---|---|---|
| | | |

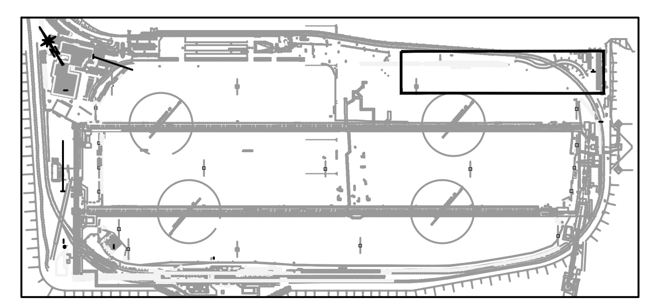
| | |
|---|---|
| Client | Westshore Terminals |
| Project | Terminal Upgrades<br>Delta, BC |
| Title | GEOTECHNICAL SOIL PROFILE<br>Section C-D |

| Project no. | Drawn | Design | Checked | Date | Scale | Drawing no. |
|---|---|---|---|---|---|---|
| 20-8543 | DD | PB | SS | November 26, 2020 | AS NOTED | APPENDIX A-4 |



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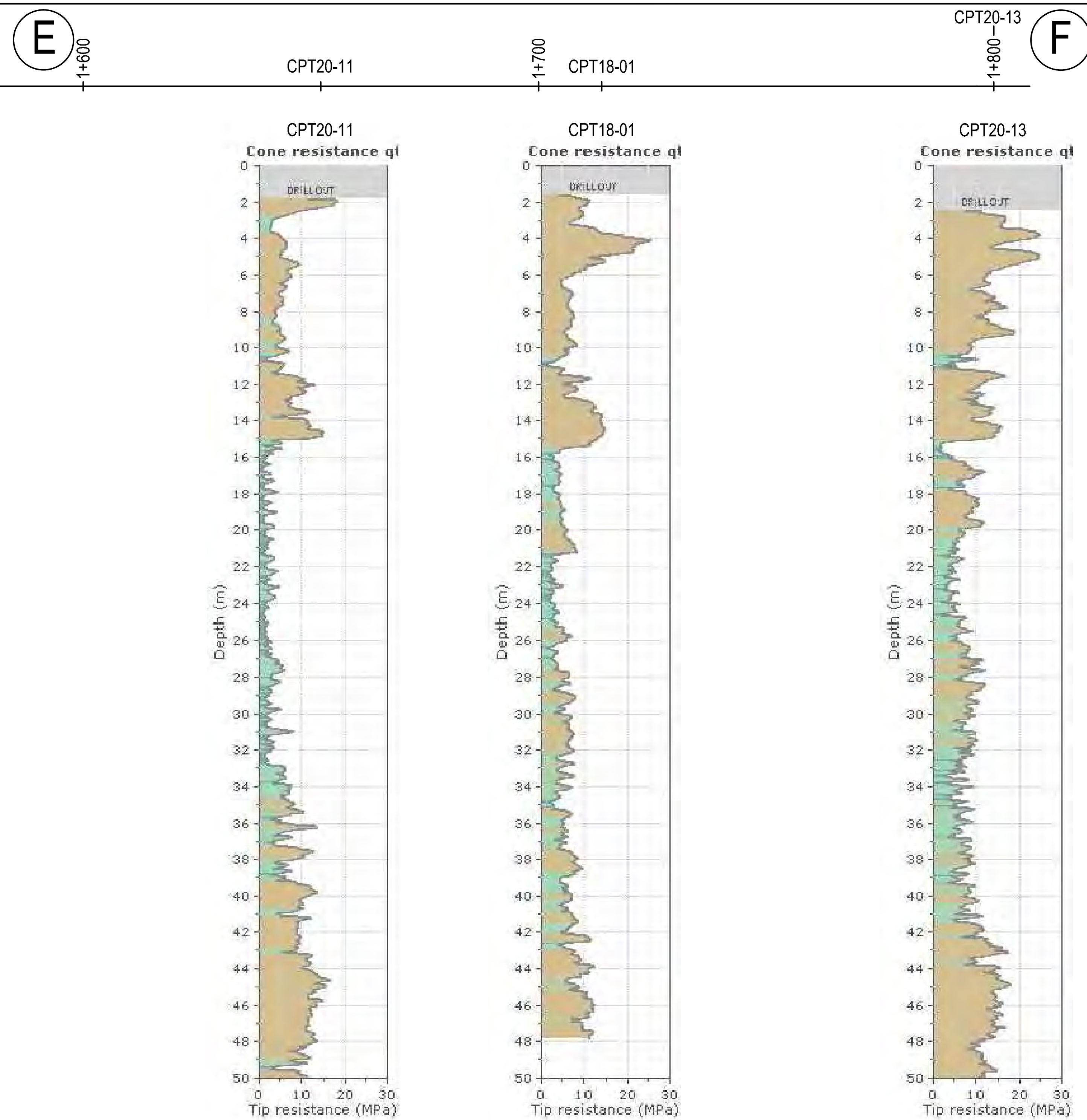
Naesgaard-Amini Geotechnical Ltd. soils, foundations and earthquake engineering

| Rev. | Description | Date |
|---|---|---|
| | | |

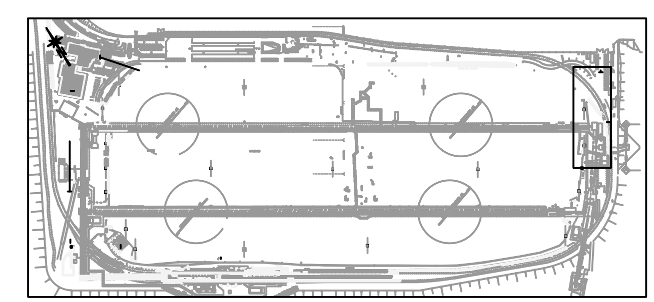
| Client | Westshore Terminals |
|---|---|
| Project | Terminal Upgrades Delta, BC |

| Title |
|---|
| GEOTECHNICAL SOIL PROFILE Section D-E |

| Project no. | Drawn | Design | Checked | Date | Scale | Drawing no. |
|---|---|---|---|---|---|---|
| 20-8543 | DD | PB | SS | November 26, 2020 | AS NOTED | APPENDIX A-5 |




CPT extended to 60m



KEY PLAN - NTS

BRAUN GEOTECHNICAL LTD.

Naesgaard-Amini Geotechnical Ltd. soils, foundations and earthquake engineering

| Rev. | Description | Date |
|---|---|---|
| | | |

| | |
|---|---|
| Client | Westshore Terminals |
| Project | Terminal Upgrades<br>Delta, BC |
| Project no. | 20-8543 |
| Drawn | DD |
| Design | PB |
| Checked | SS |
| Title | GEOTECHNICAL SOIL PROFILE<br>Section E-F |
| Date | November 26, 2020 |
| Scale | AS NOTED |
| Drawing no. | APPENDIX A-6 |

# Appendix B

# NRC Hazard Calculation Sheet and Deaggregation Data

Figure B-1: Seismic Hazard

# 2015 National Building Code Seismic Hazard Calculation

**INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836**
**Western Canada English (250) 363-6500 Facsimile (250) 363-6565**

**Site:** 49.022N 123.173W **User File Reference:** Westshore Terminals 2020-04-13 21:11 UT

**Requested by:** 20-8543, Braun Geotechnical

| Probability of exceedance per annum | 0.000404 | 0.001 | 0.0021 | 0.01 |
|---|---|---|---|---|
| Probability of exceedance in 50 years | 2 % | 5 % | 10 % | 40 % |
| Sa (0.05) | 0.515 | 0.367 | 0.270 | 0.123 |
| Sa (0.1) | 0.785 | 0.561 | 0.413 | 0.188 |
| Sa (0.2) | 0.977 | 0.703 | 0.516 | 0.236 |
| Sa (0.3) | 0.990 | 0.715 | 0.525 | 0.236 |
| Sa (0.5) | 0.876 | 0.629 | 0.457 | 0.195 |
| Sa (1.0) | 0.488 | 0.339 | 0.240 | 0.095 |
| Sa (2.0) | 0.292 | 0.197 | 0.135 | 0.050 |
| Sa (5.0) | 0.090 | 0.053 | 0.031 | 0.011 |
| Sa (10.0) | 0.032 | 0.018 | 0.011 | 0.004 |
| PGA (g) | 0.423 | 0.305 | 0.224 | 0.101 |
| PGV (m/s) | 0.639 | 0.441 | 0.309 | 0.118 |

**Notes:** Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s$^2$). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

## References

**National Building Code of Canada 2015 NRCC no. 56190;** Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

**Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)**
**Commentary J:** Design for Seismic Effects

**Geological Survey of Canada Open File 7893** Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

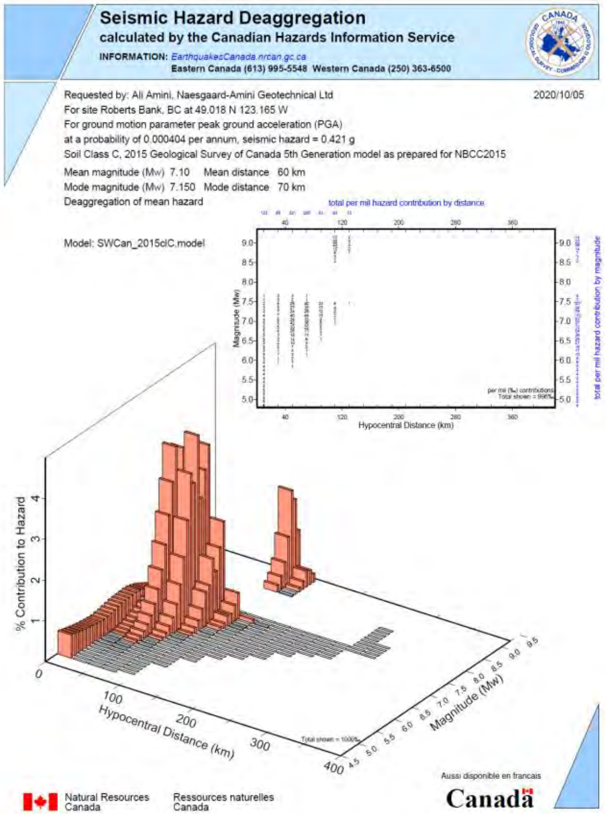
See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Natural Resources Canada Ressources naturelles Canada

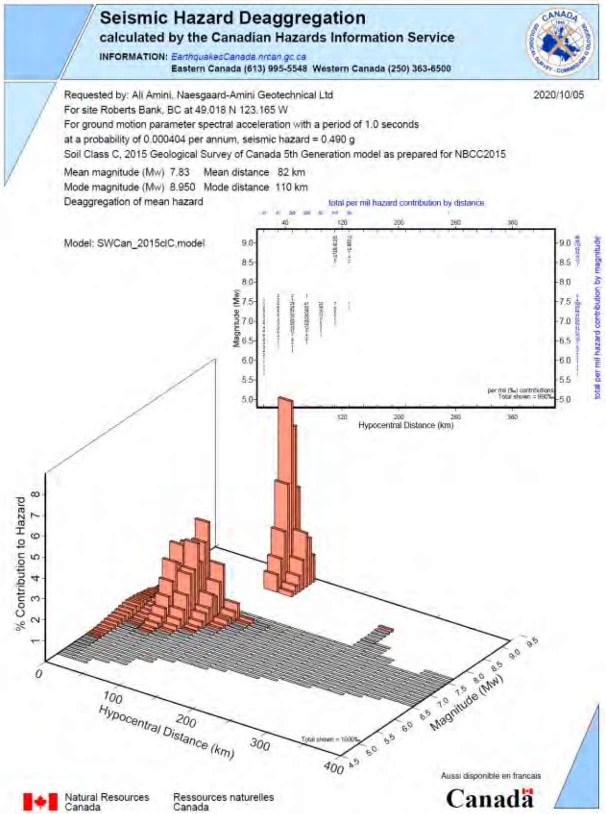
Canada

Figure B-2: Deaggregation data for A2475 (PGA, T=1s and T=2s)

**PGA**




**T=1s**




**T=2s**

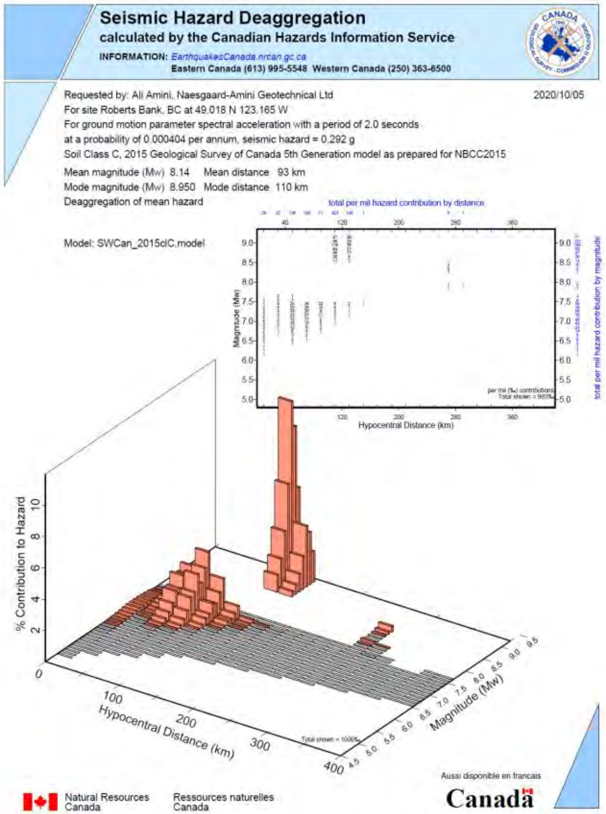
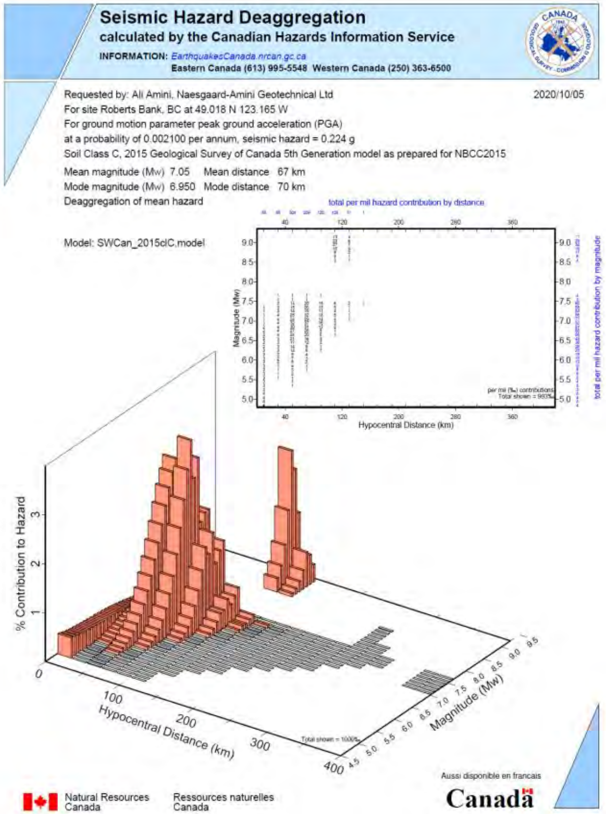
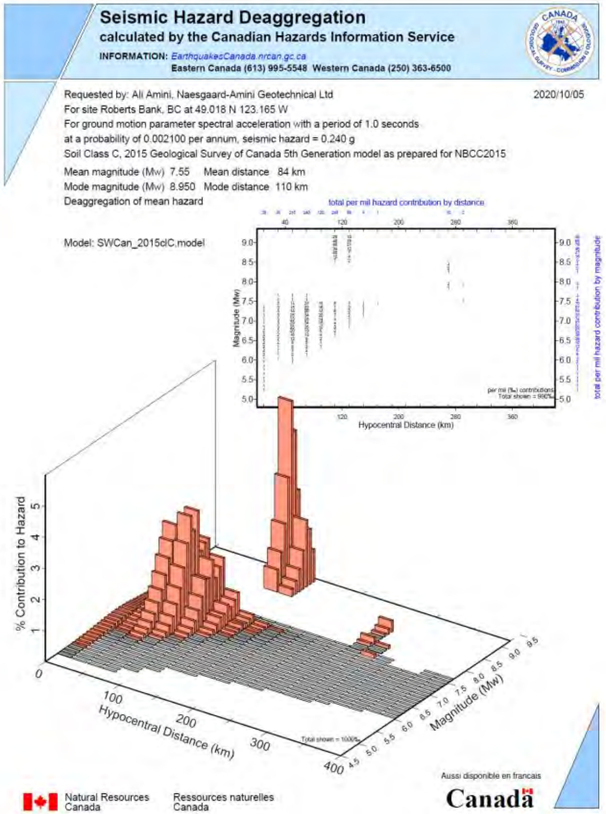


Figure B-3: Deaggregation data for A475 (PGA, T=1s and T=2s)

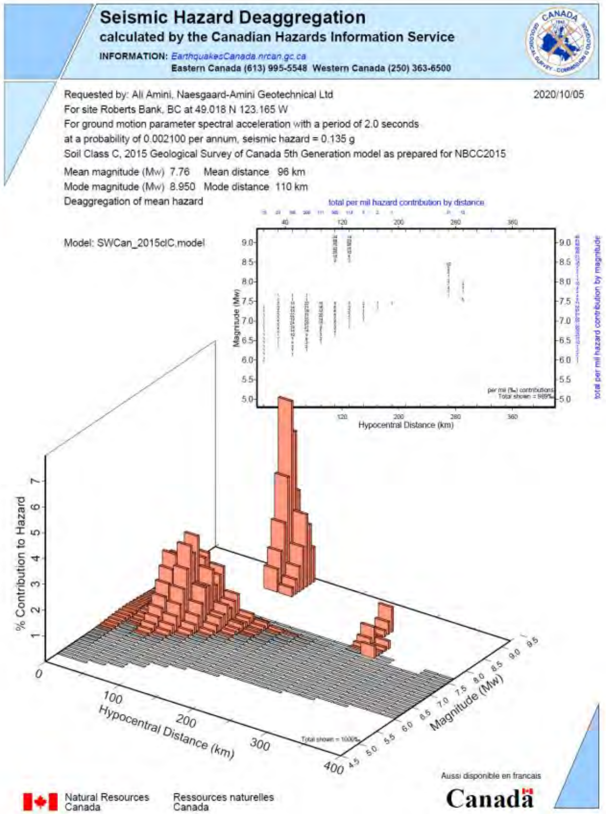
**PGA**




**T=1s**




**T=2s**



# Appendix C

# Design Ground Motions

Table C-1: List of design ground motions

| Type | Abreviation | Horizontal design ground motion | FLAC | SSRA |
|---|---|---|---|---|
| Crustal | CRU01 | HECTOR 1999-JOS090n | x | x |
| | | HECTOR 1999-JOS360n | | x |
| | CRU02 | LANDERS 1992-JOS000 | x | x |
| | | LANDERS 1992-JOS090 | | x |
| | CRU03 | LANDERS 1992-MVP000n | x | x |
| | | LANDERS 1992-MVP090n | | x |
| | | LANDERS 1992-MVPup (vertical) | x | |
| | CRU04 | SFERN 1971_M6.6R36_RSN180_SLABOR | x | x |
| | | SFERN 1971_M6.6R36_RSN270_SLABOR | | x |
| | CRU05 | SMART 1986-45O06NS | x | x |
| | | SMART 1986-45O06EW | | x |
| In-slab | INS01 | ElSalvador 2001-R110_DB-180n | x | x |
| | | ElSalvador 2001-R110_DB-270n | | x |
| | INS02 | ElSalvador 2001-R113_RF-180n | x | x |
| | | ElSalvador 2001-R113_RF-90n | | x |
| | INS03 | Miyagi 2005-MYG006-EWn | x | x |
| | | Miyagi 2005-MYG006-NSn | | x |
| | INS04 | Nisqually 2001-R75-125n | x | x |
| | | Nisqually 2001-R75-215n | | x |
| | INS05 | Tarapaca 2005-IDIEM_C_Ln | x | x |
| | | Tarapaca 2005-IDIEM_C_Tn | | x |
| Interface | INT01 | Michoacan 1985-N00Wn | x | x |
| | | Michoacan 1985-N90Wn | | x |
| | INT02 | Tohoku 2011_M9.0_R209_YMT008_EWn | x | x |
| | | Tohoku 2011_M9.0_R209_YMT008_NSn | | x |
| | INT03 | Tohoku 2001-IWT022-EWn | x | x |
| | | Tohoku 2001-IWT022-NSn | | x |
| | INT04 | Tokachioki 2003_R152-EWn | x | x |
| | | Tokachioki 2003_R152-NSn | | x |
| | INT05 | Tokachioki 2003_R245-Ewn | x | x |
| | | Tokachioki 2003_R245-NSn | | x |

summary of Ground motions.xlsx

Figure C-1: Response Spectra of A2475 design ground motions after being scaled by a factor of 1.09

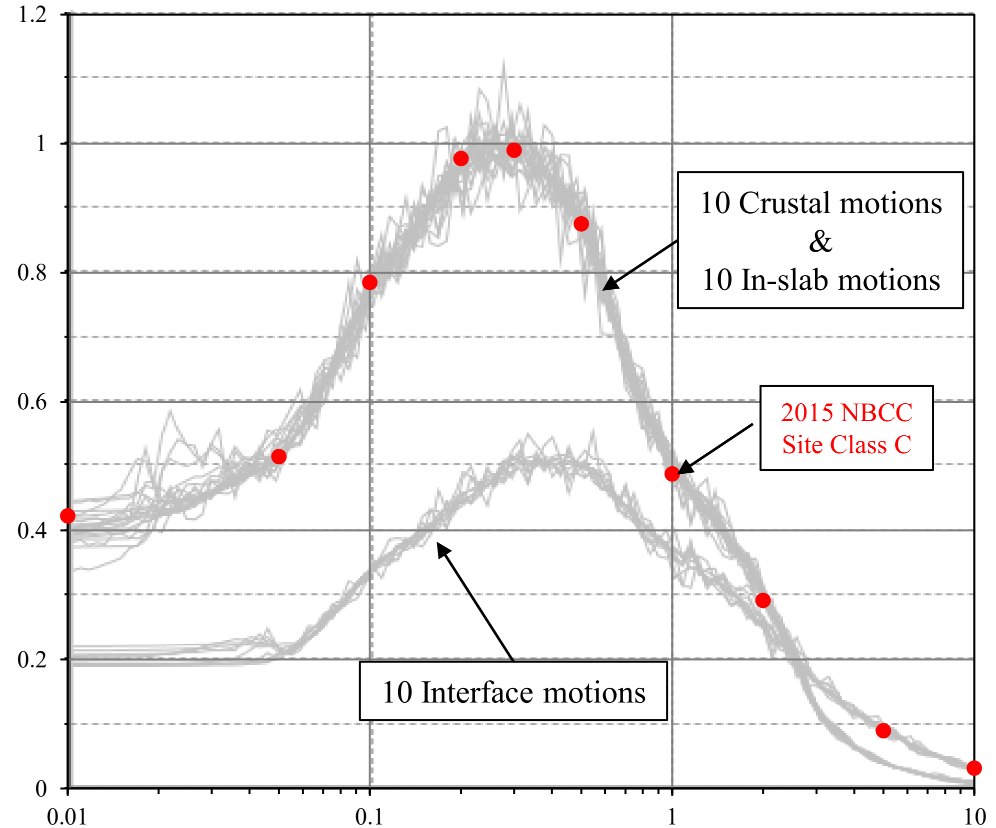
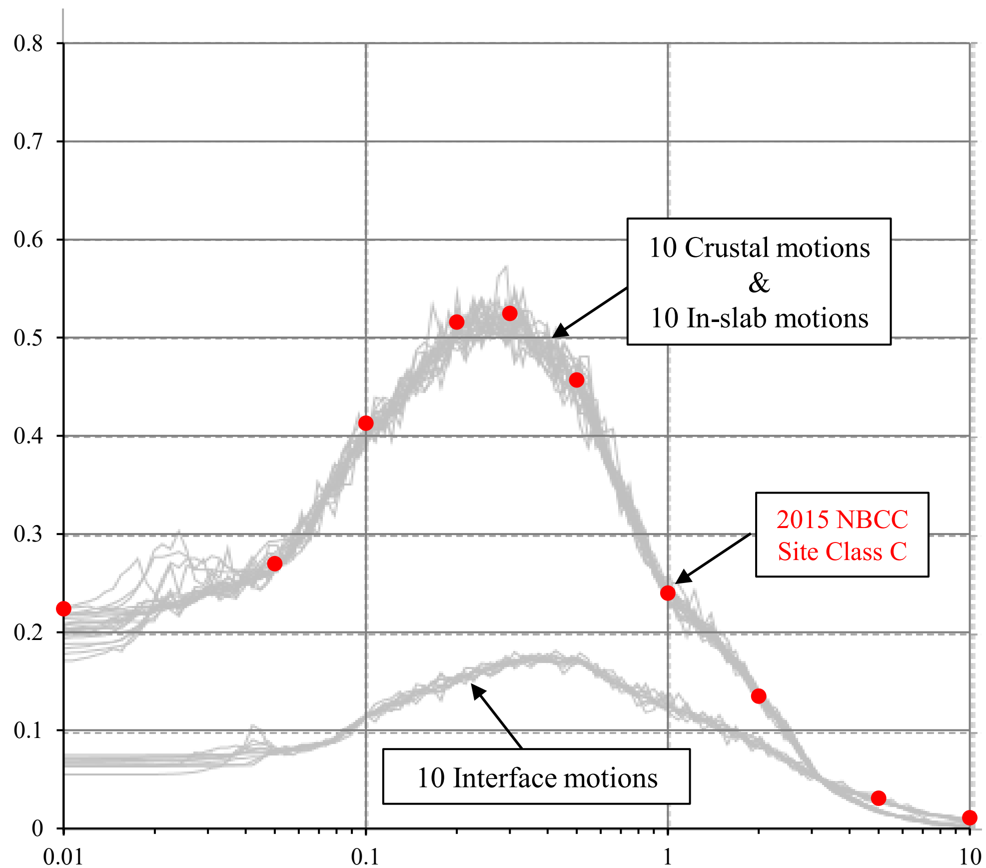
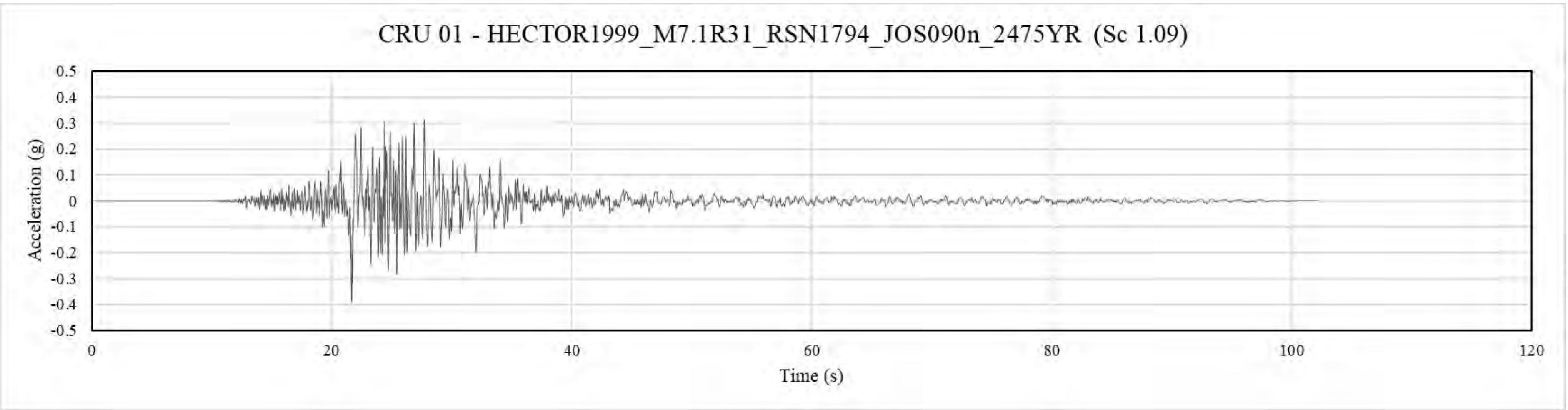
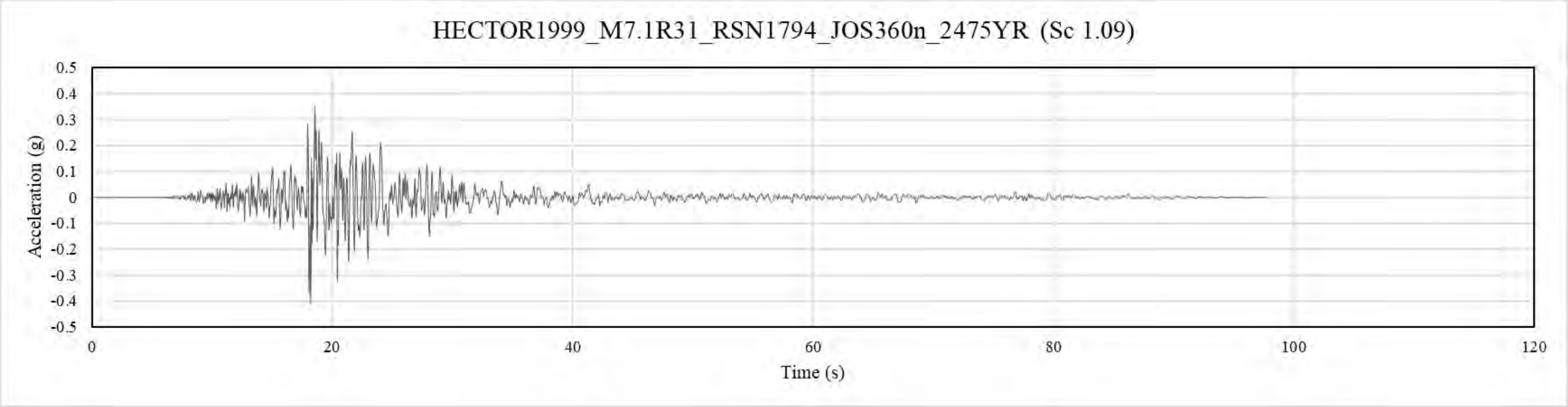


Figure C-2: Response Spectra of A475 design ground motions after being scaled by a factor of 1.09

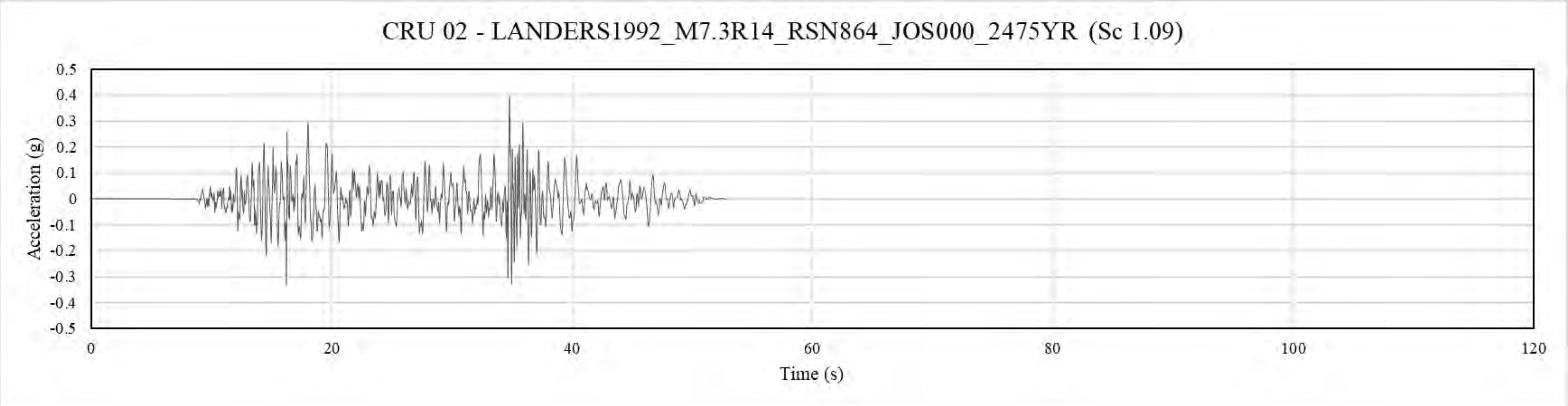


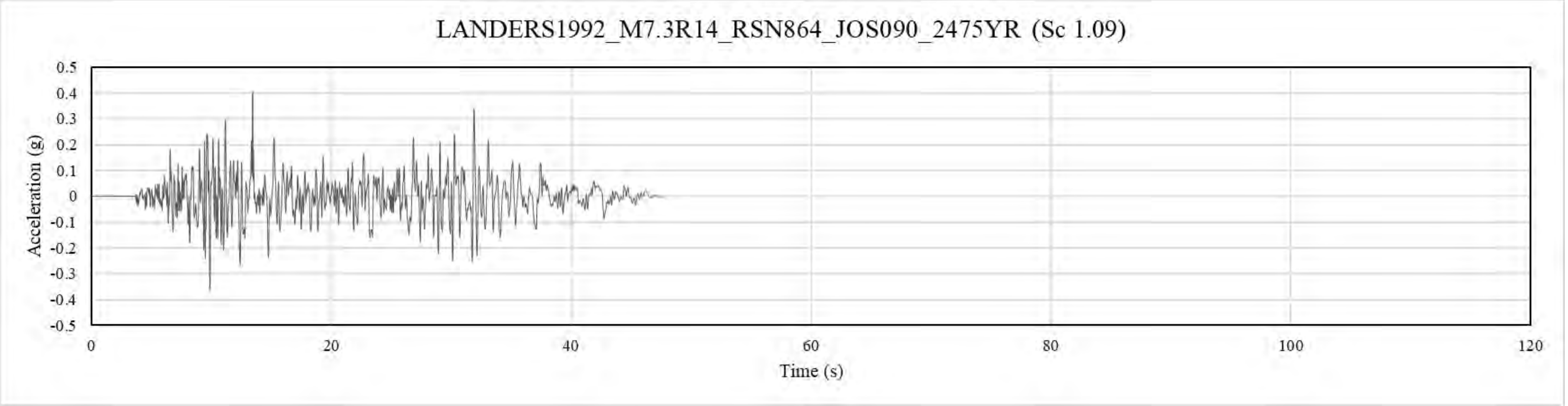
# A2475 – Crustal ground motions



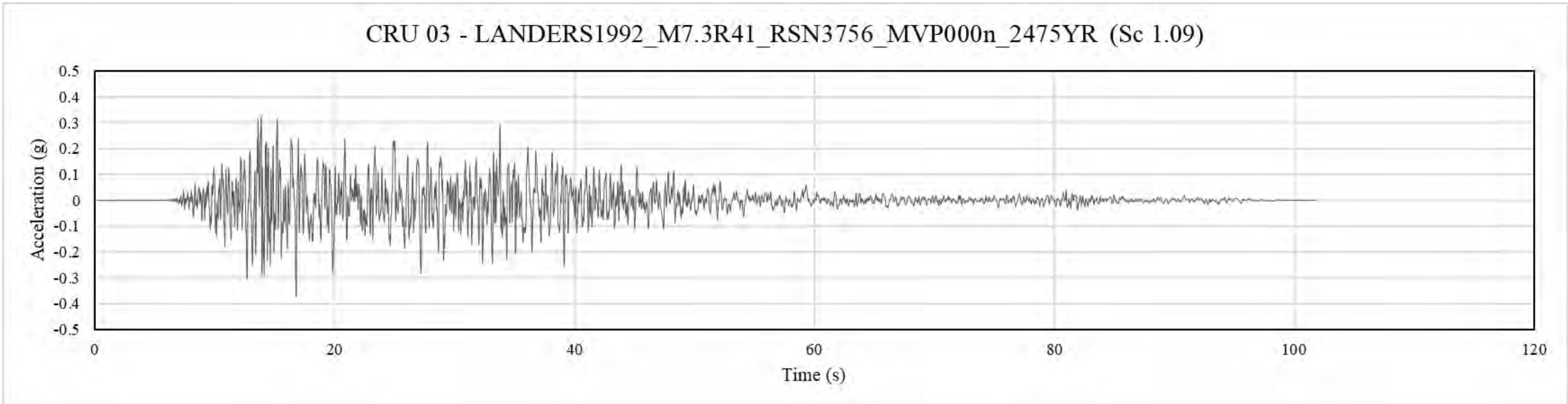



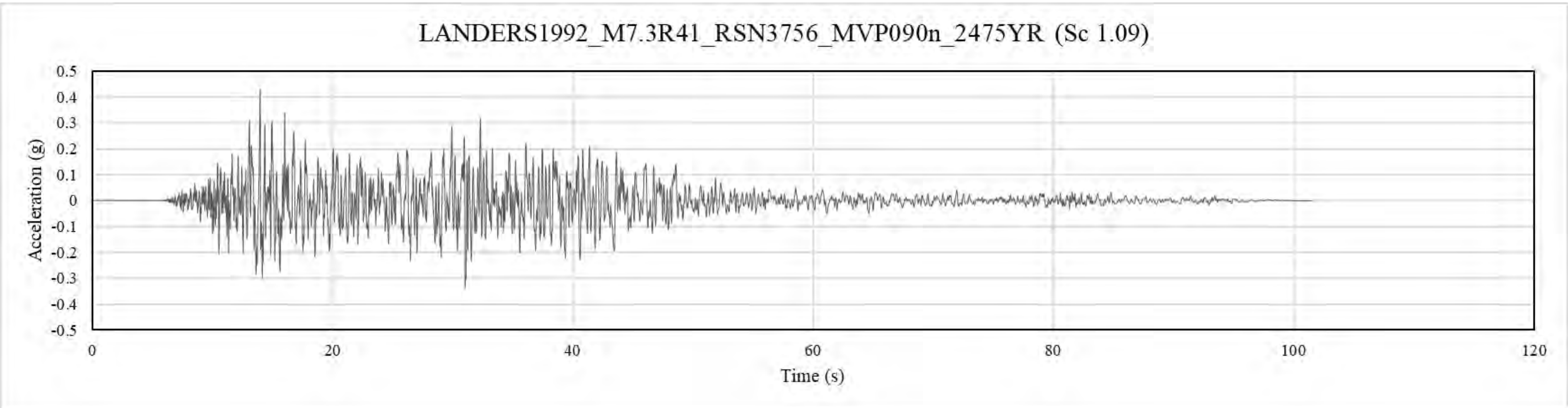
# A2475 – Crustal ground motions



# A2475 – Crustal ground motions



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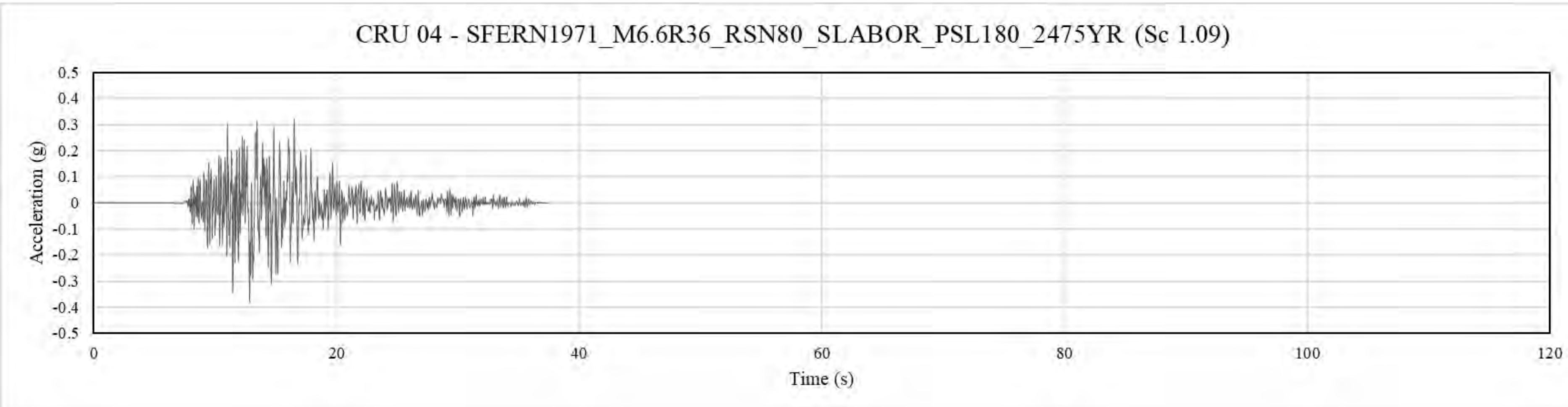
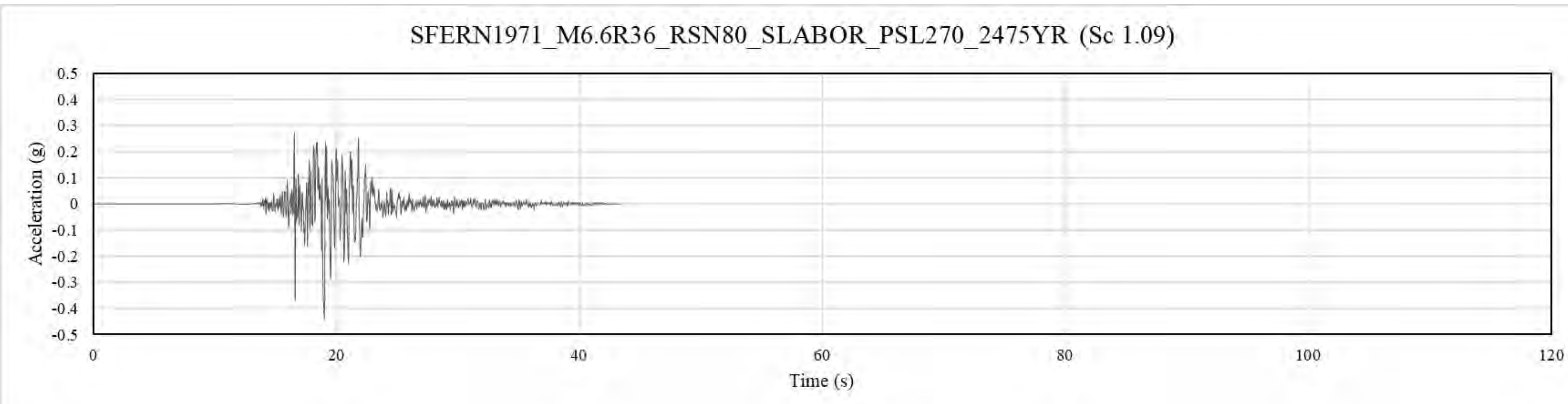
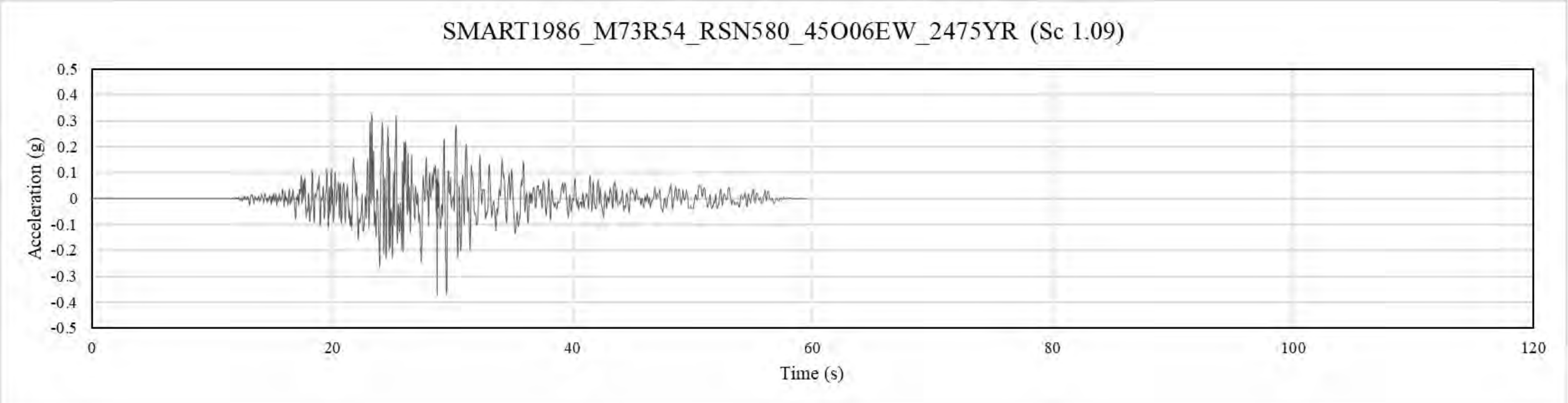
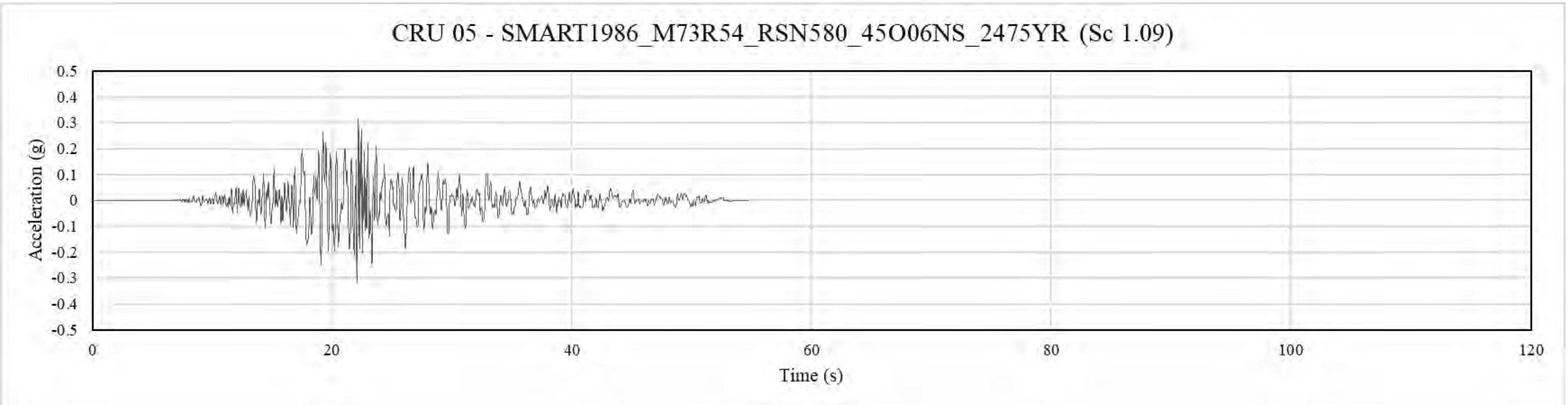


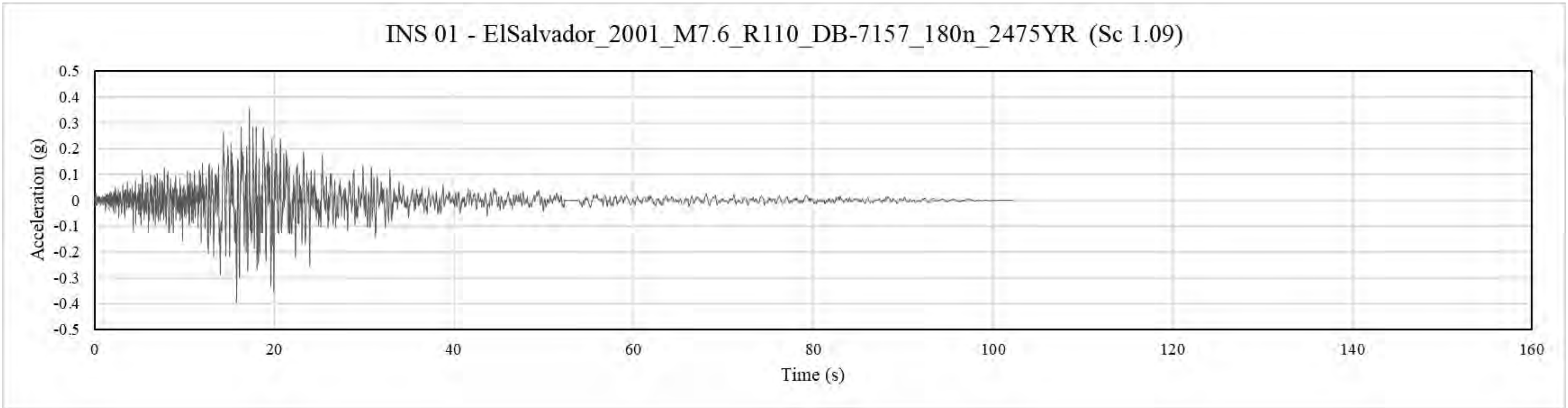



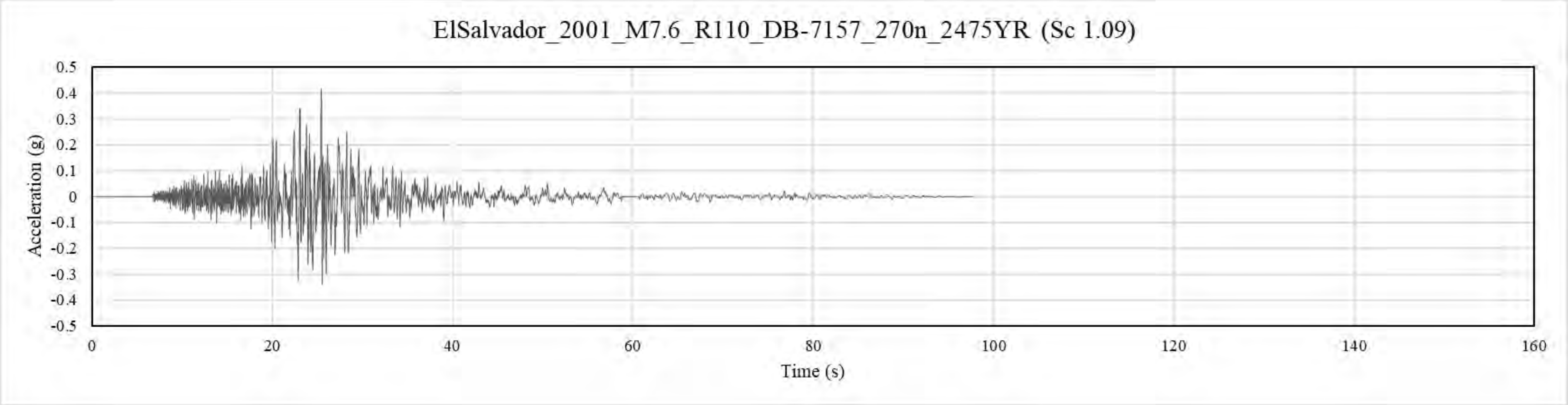
# A2475 – Crustal ground motions



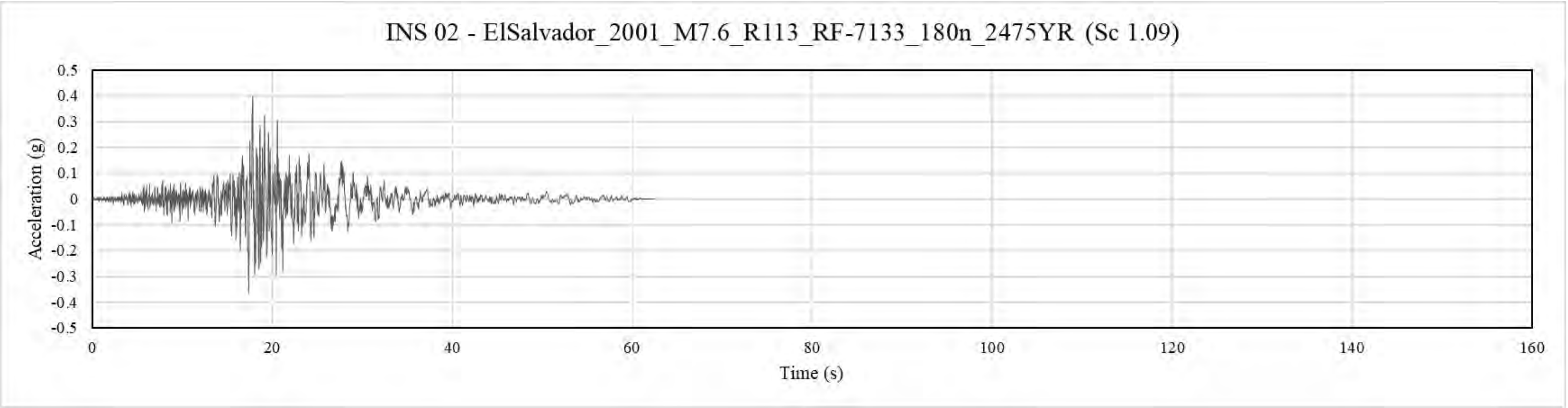



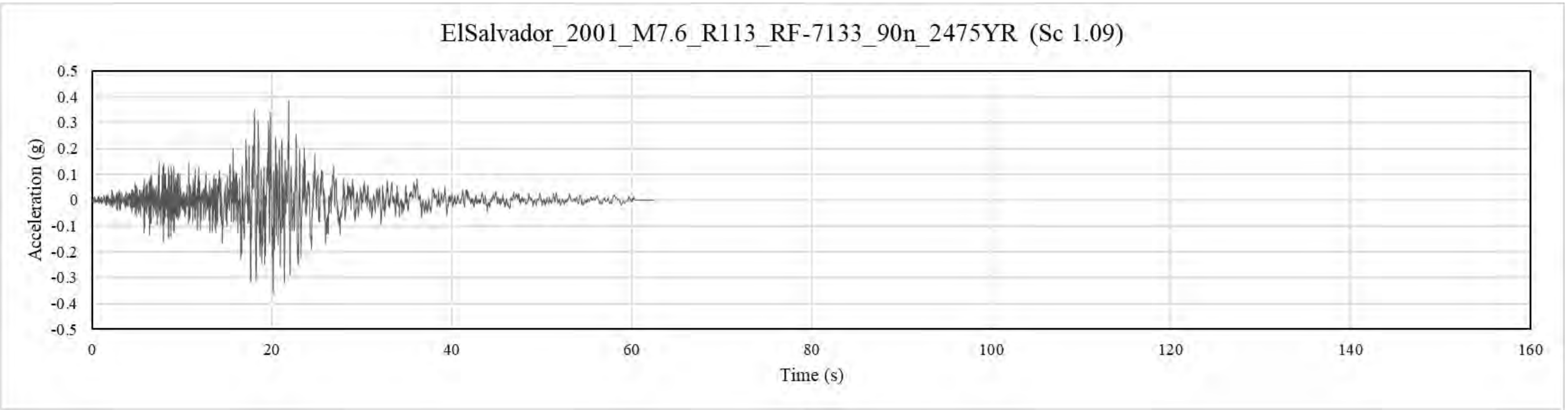
# A2475 – In-slab ground motions



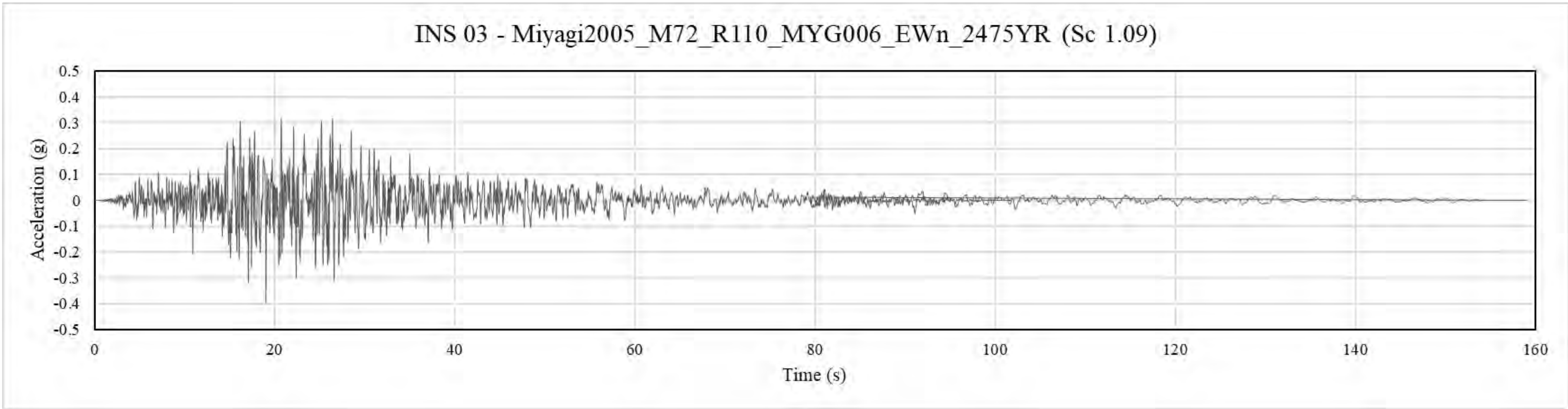



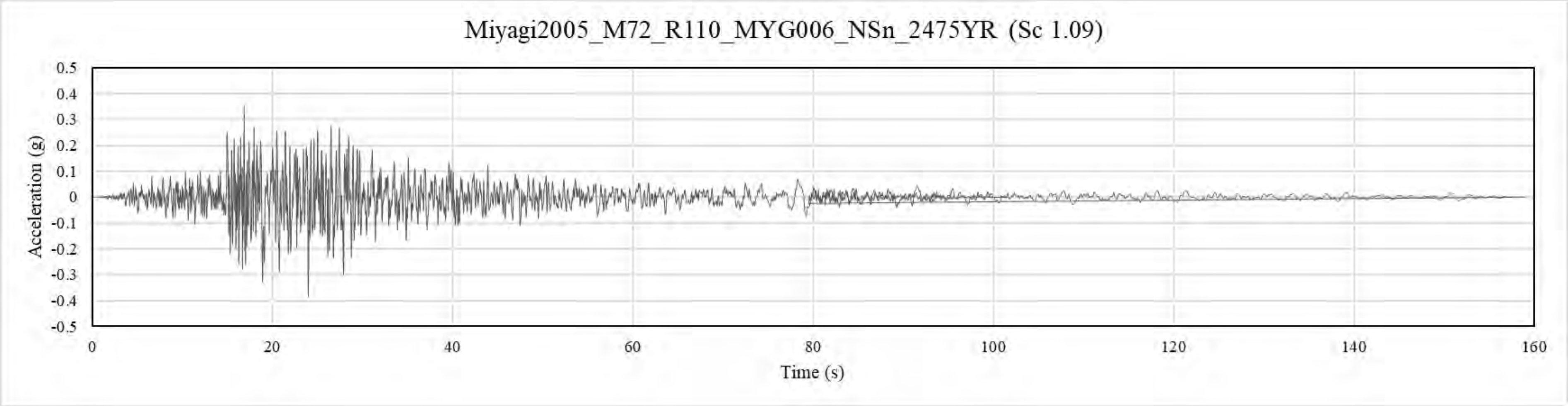
# A2475 – In-slab ground motions



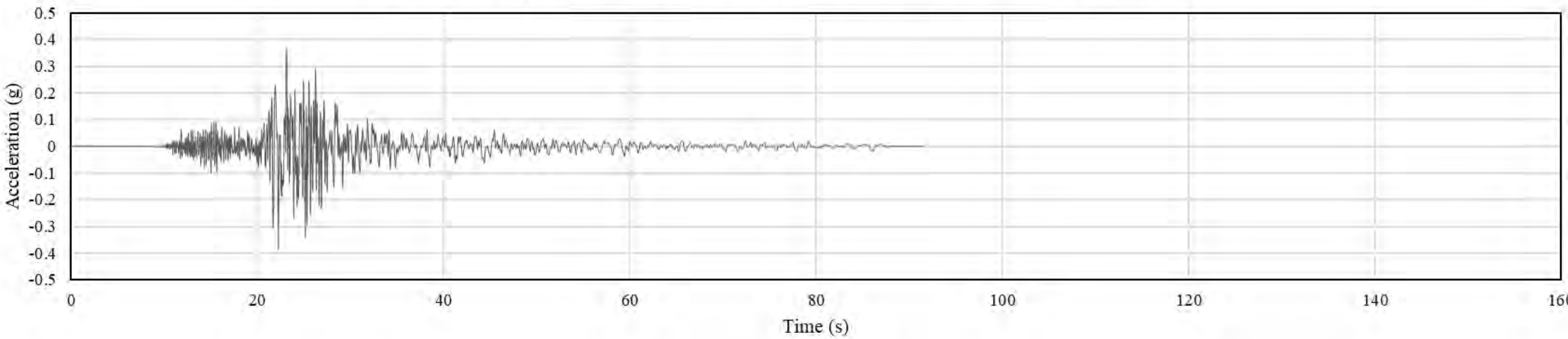
# A2475 – In-slab ground motions



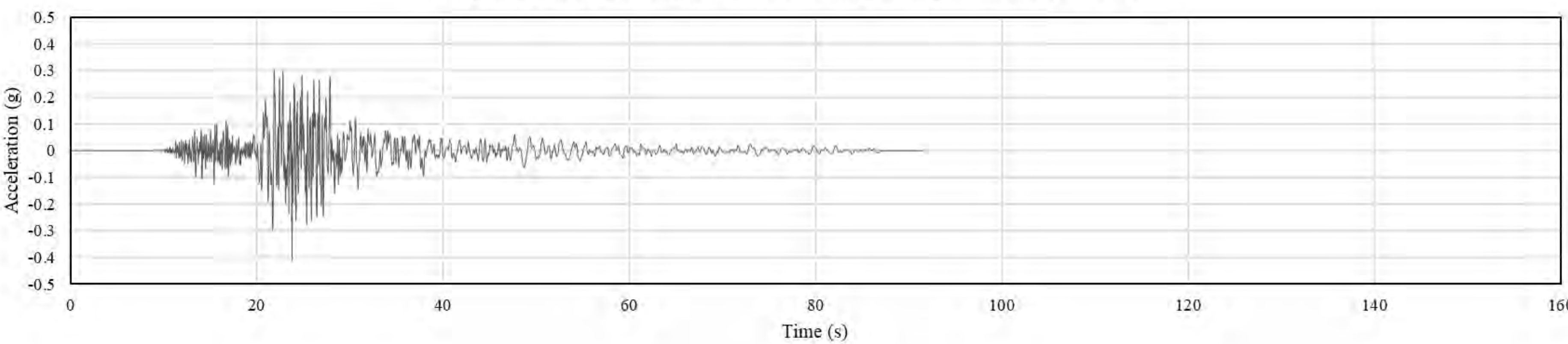



# A2475 – In-slab ground motions

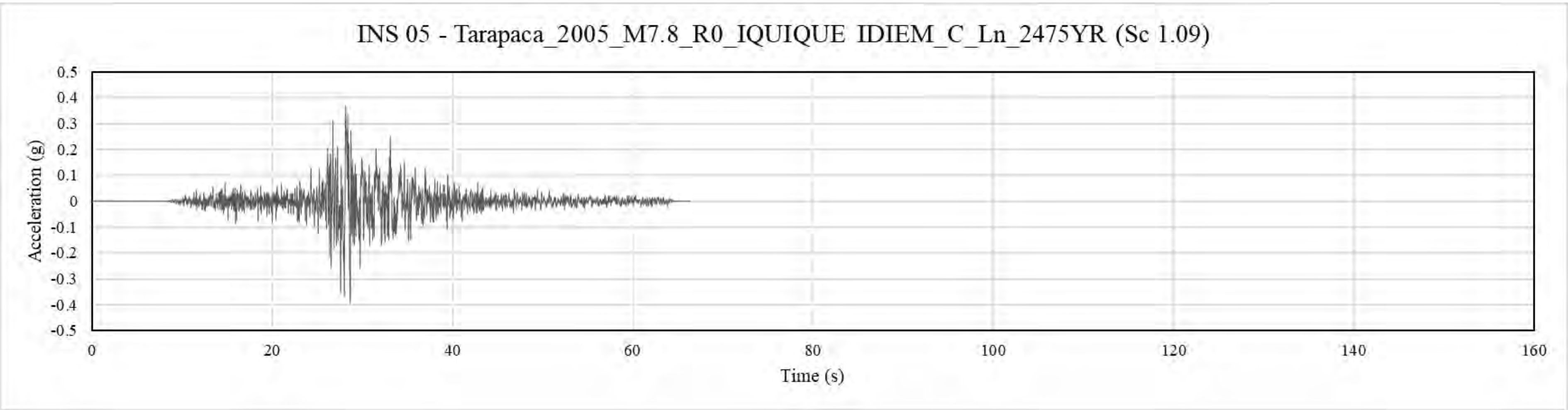
INS 04 - Nisqually_2001_M6.8_R75_7032-1416_125n_2475YR (Sc 1.09)

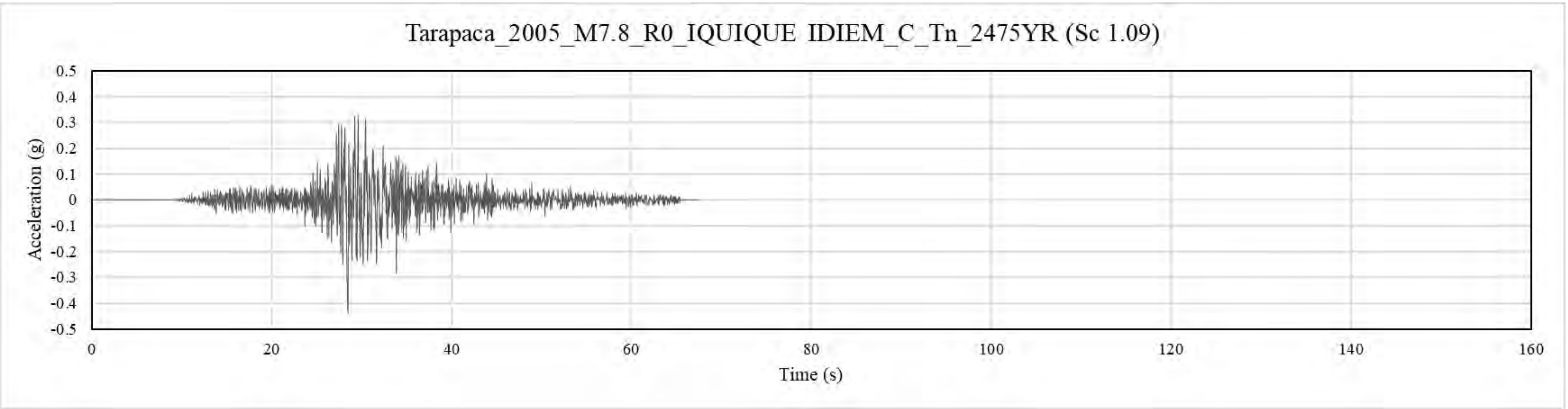



Nisqually_2001_M6.8_R75_7032-1416_215n_2475YR (Sc 1.09)

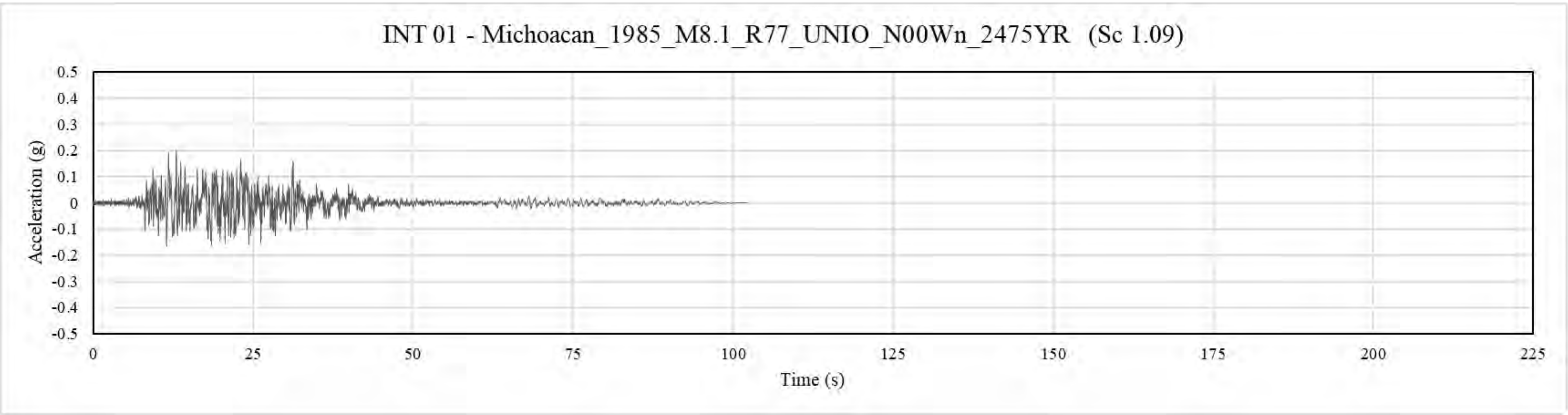


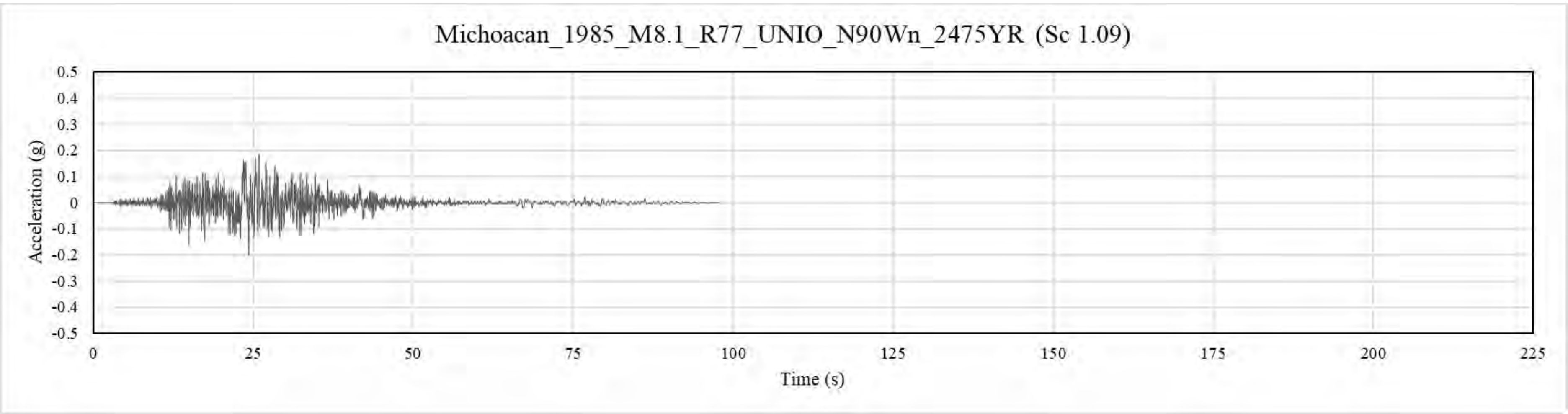
# A2475 – In-slab ground motions



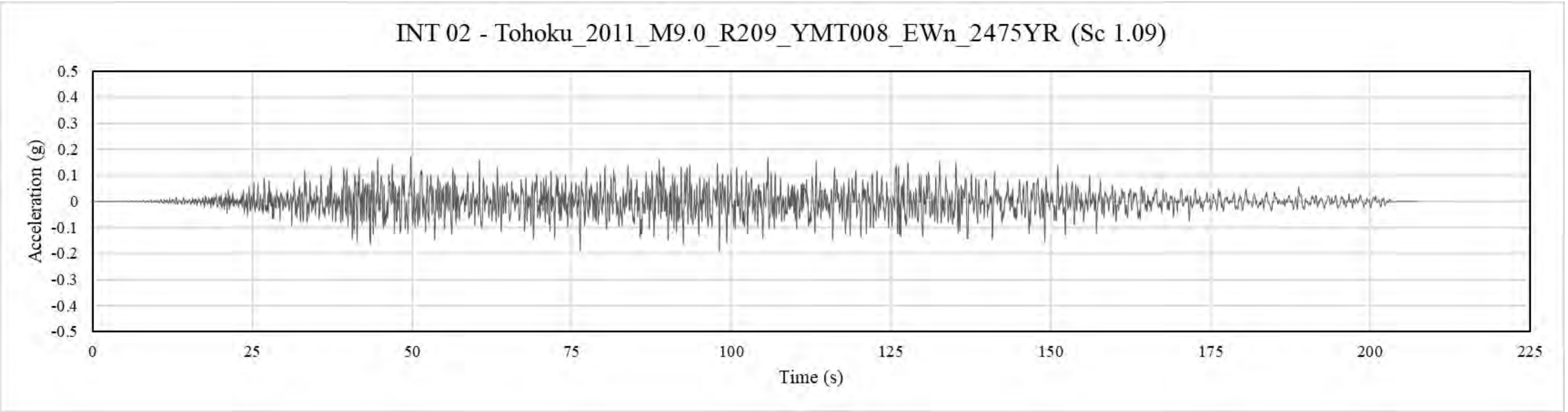



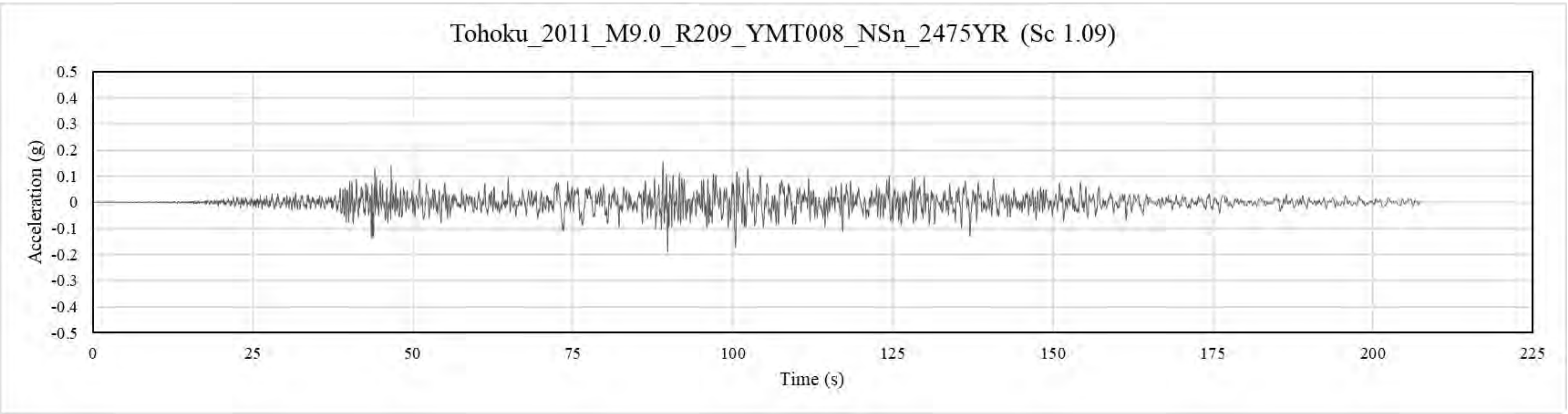
# A2475 – Interface ground motions



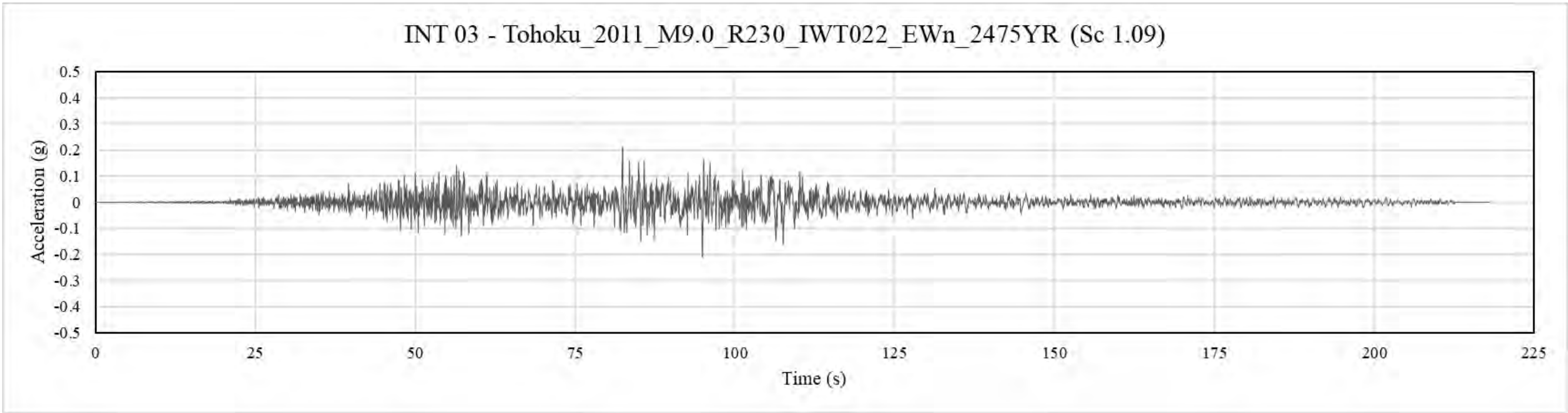



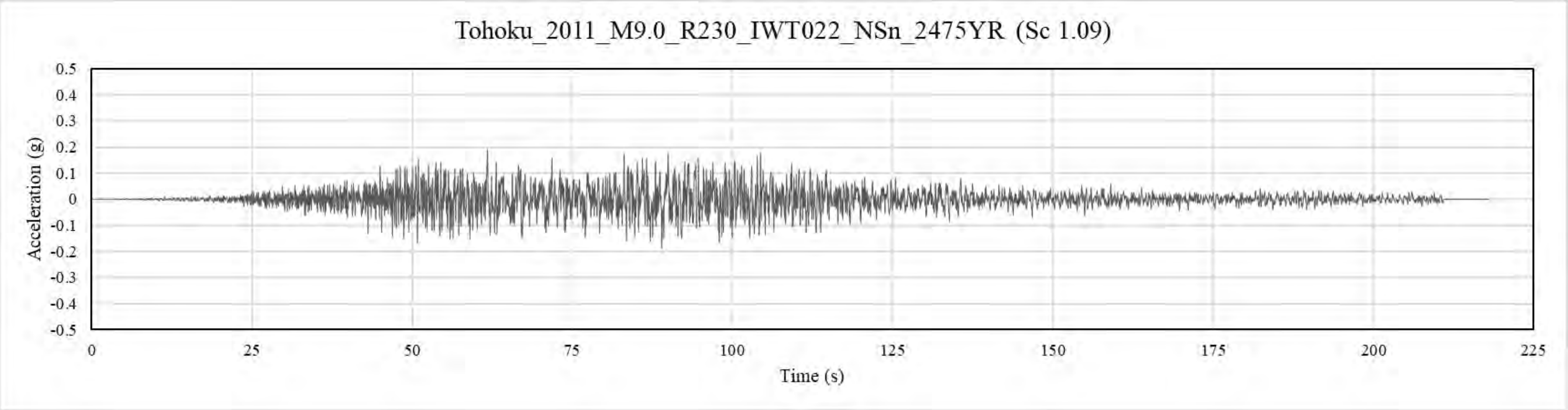
# A2475 – Interface ground motions



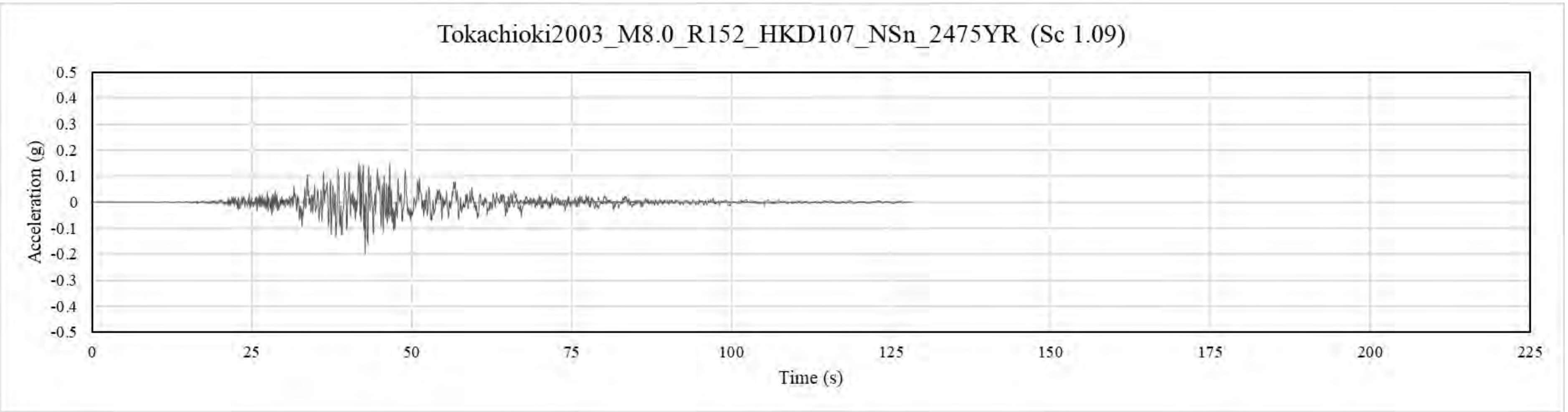



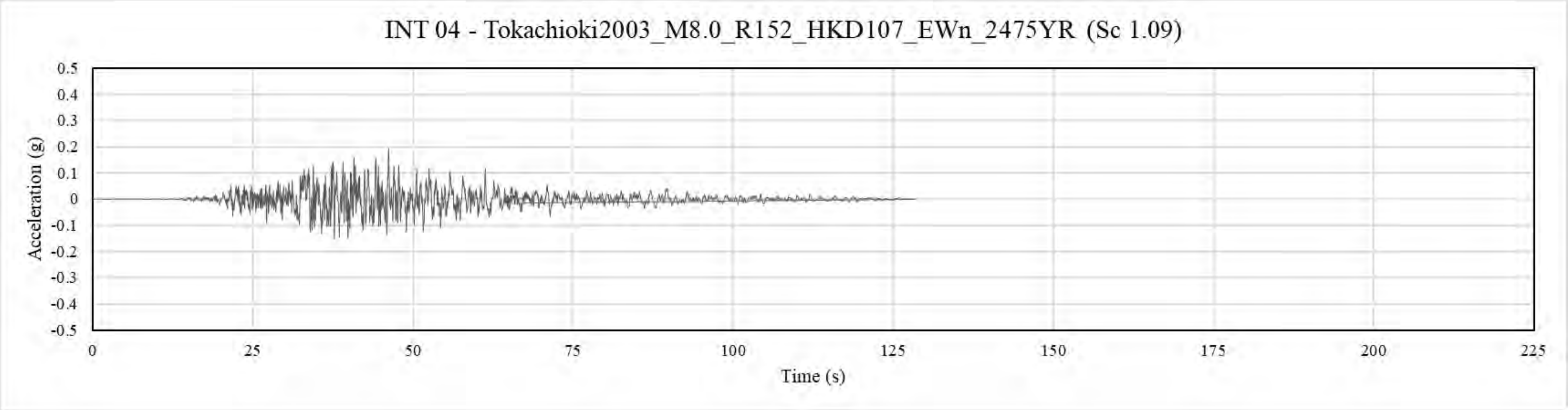
# A2475 – Interface ground motions



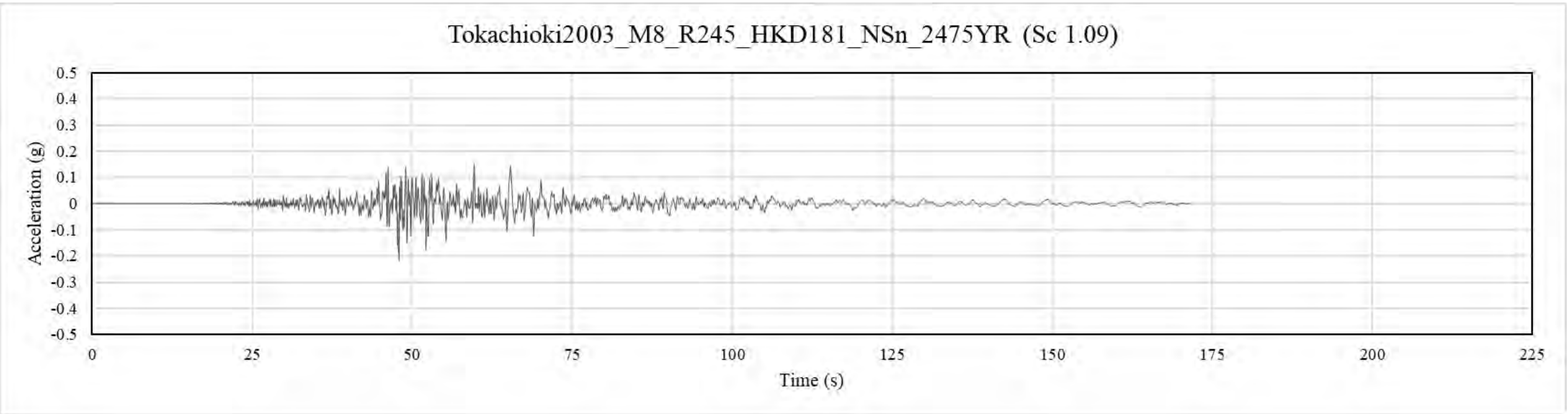



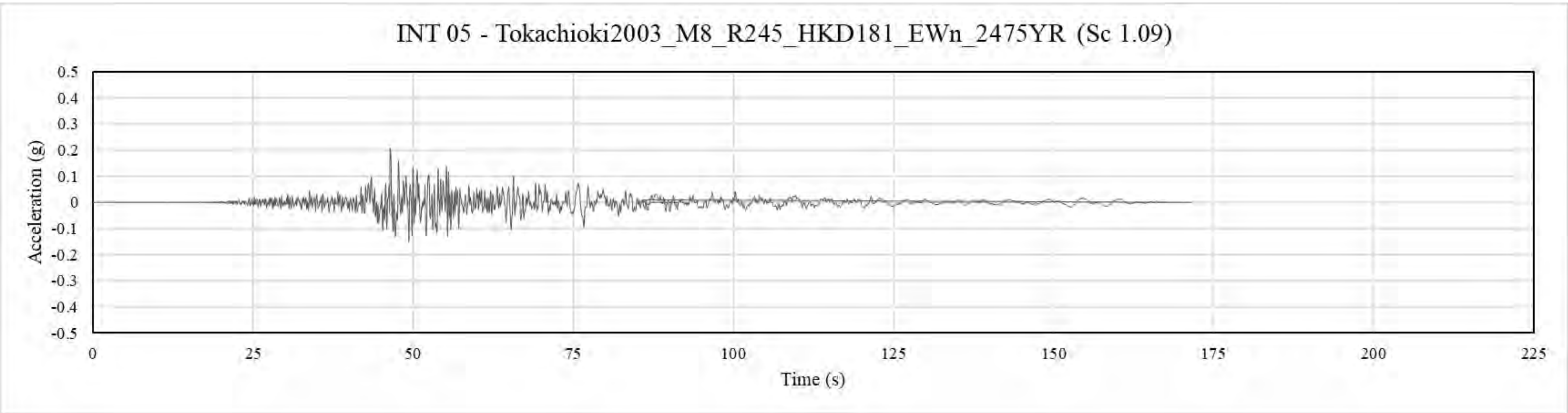
# A2475 – Interface ground motions



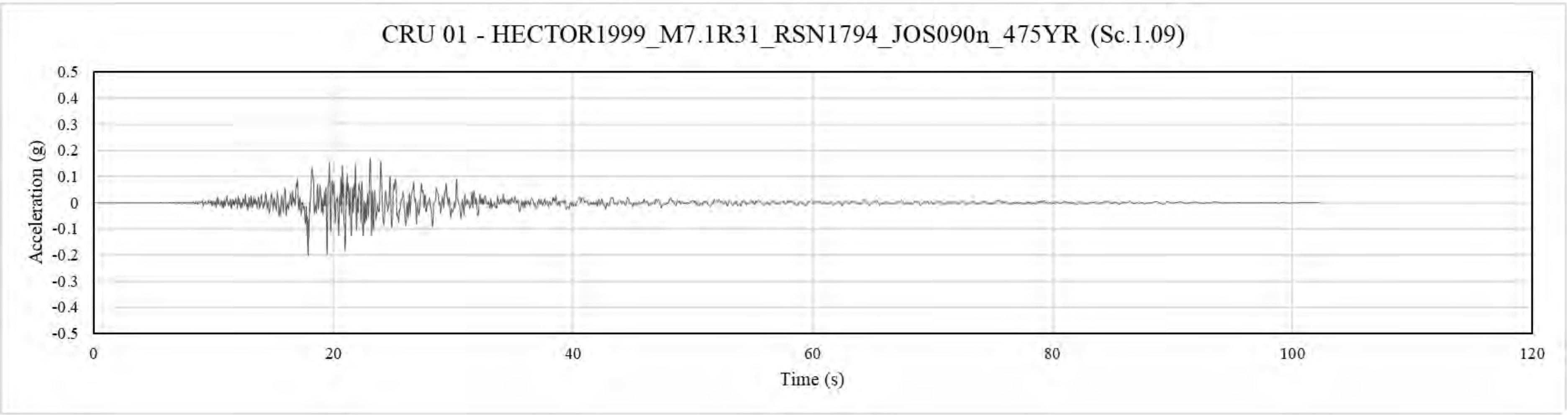



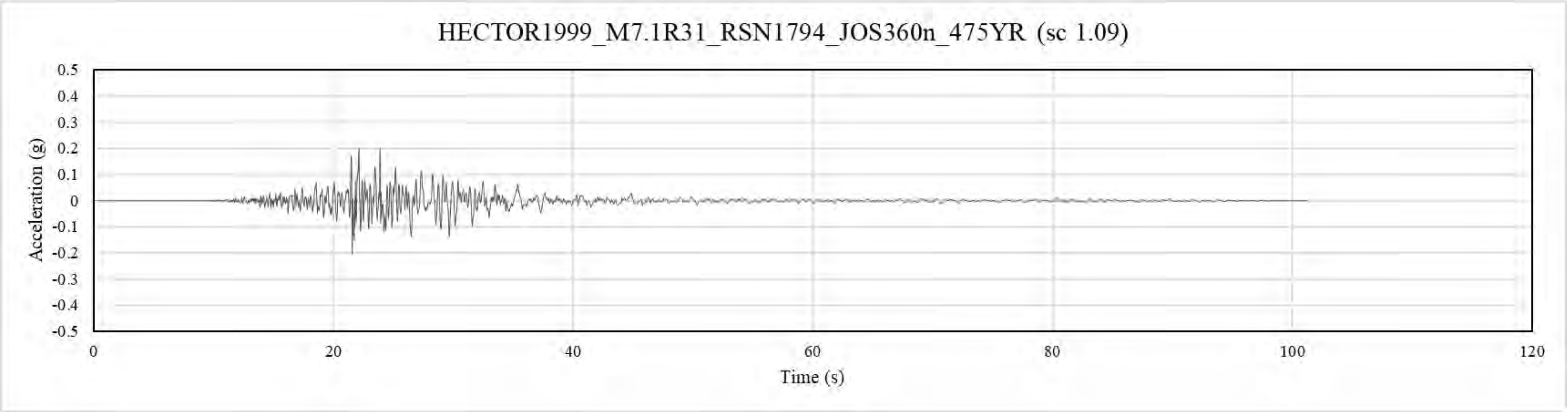
# A2475 – Interface ground motions



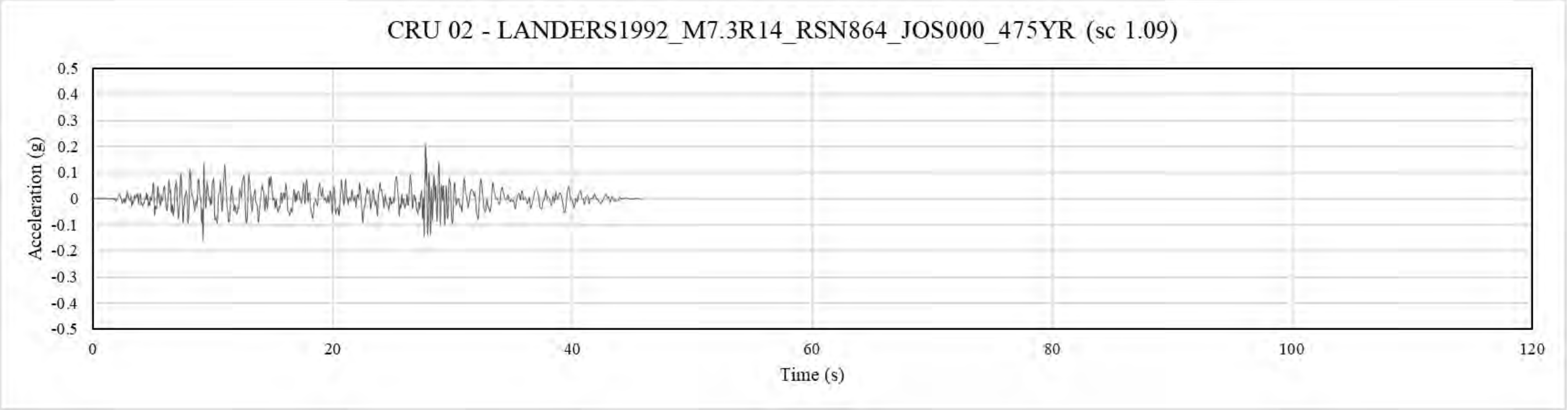



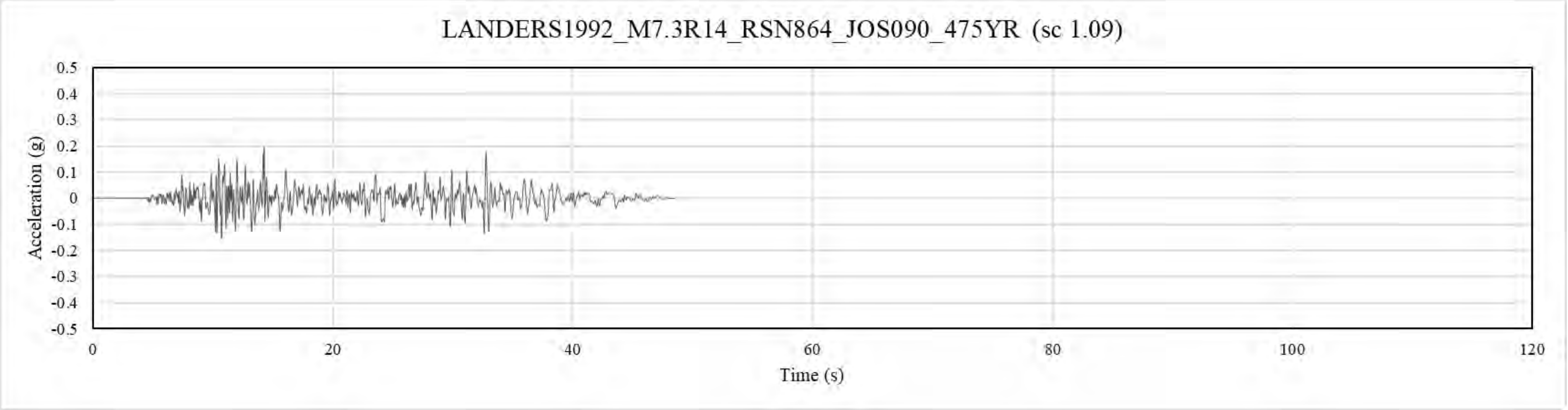
# A475 – Crustal ground motions



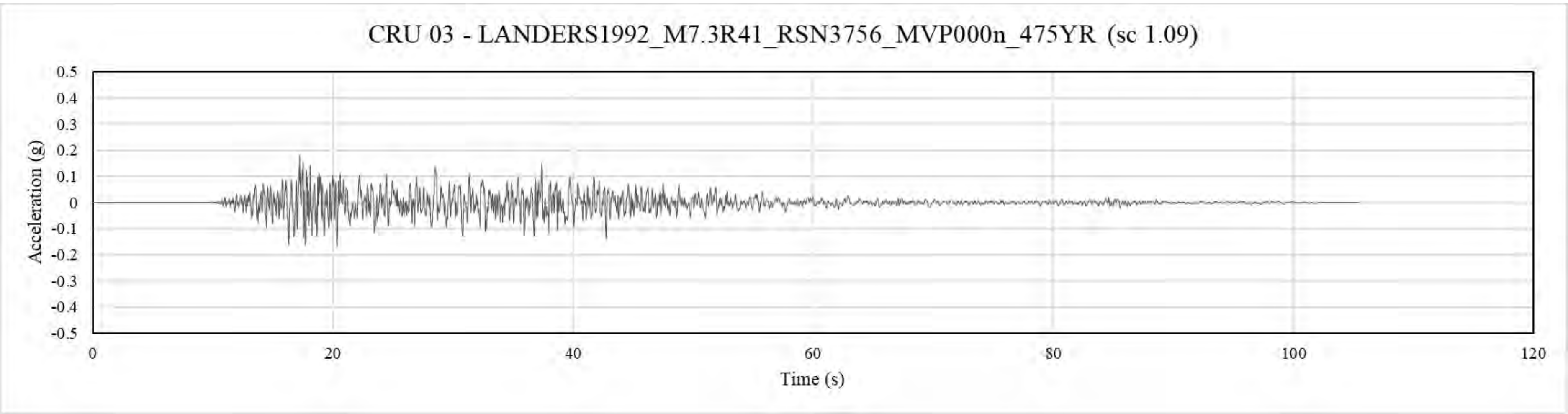



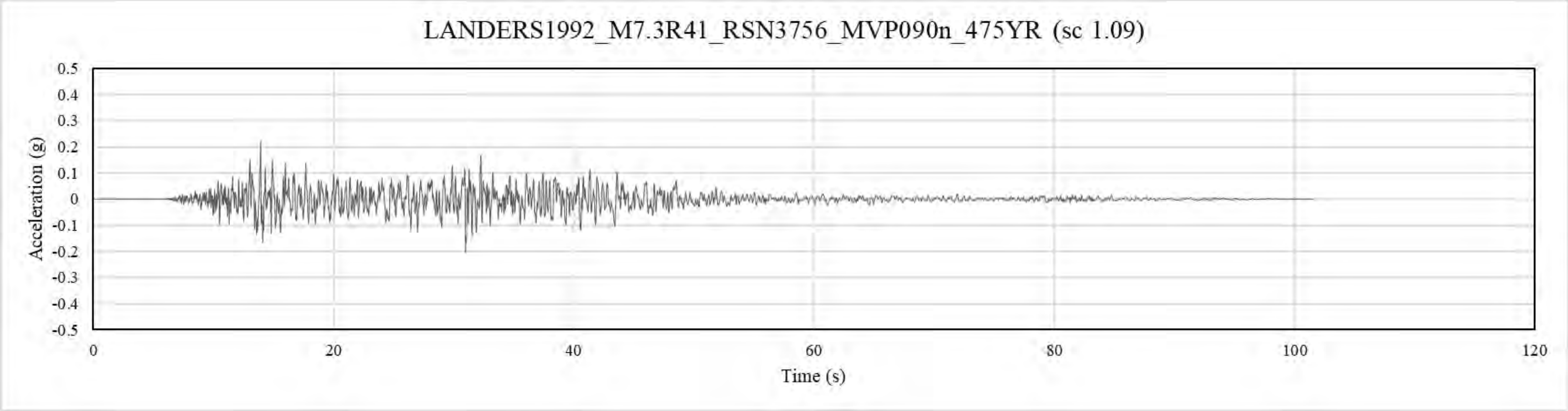
# A475 – Crustal ground motions



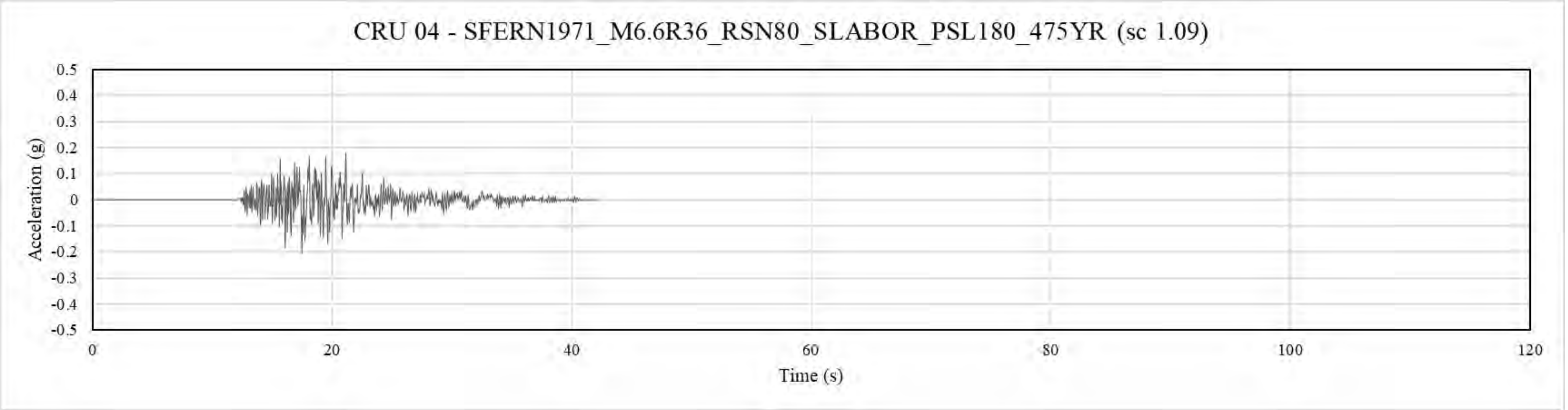



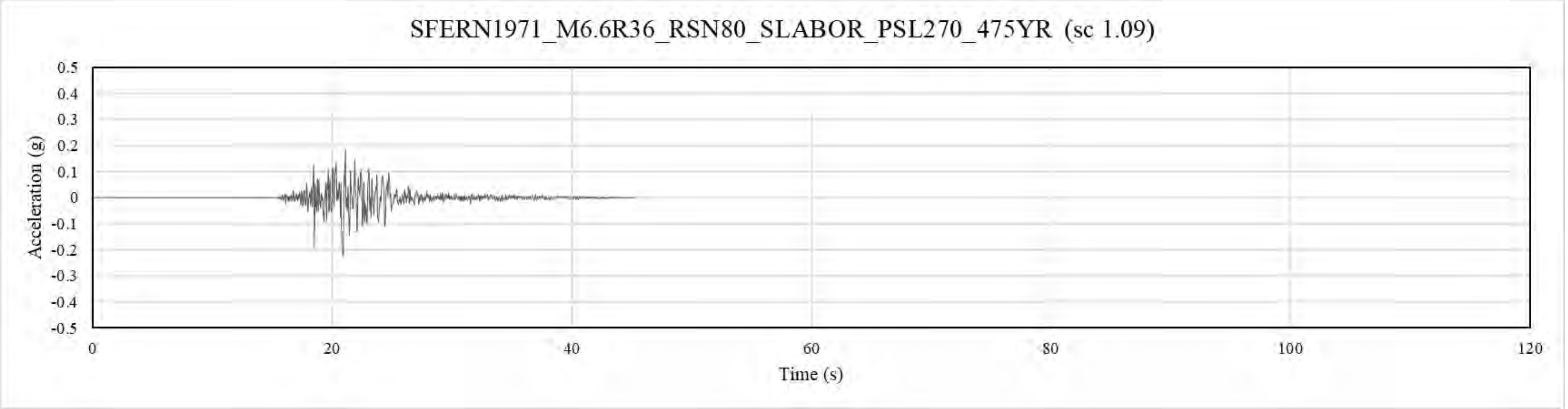
# A475 – Crustal ground motions



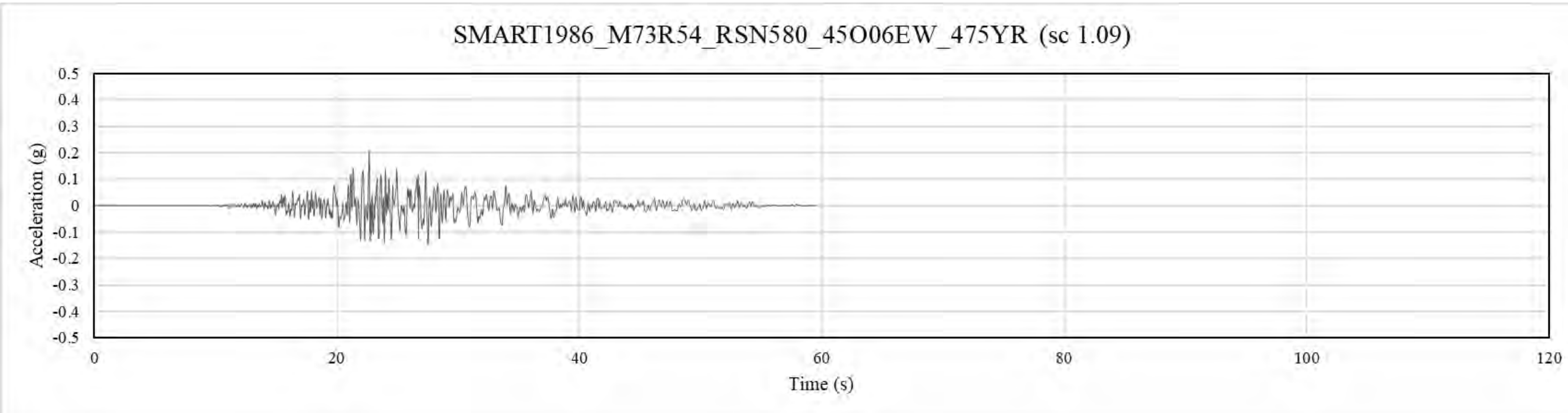



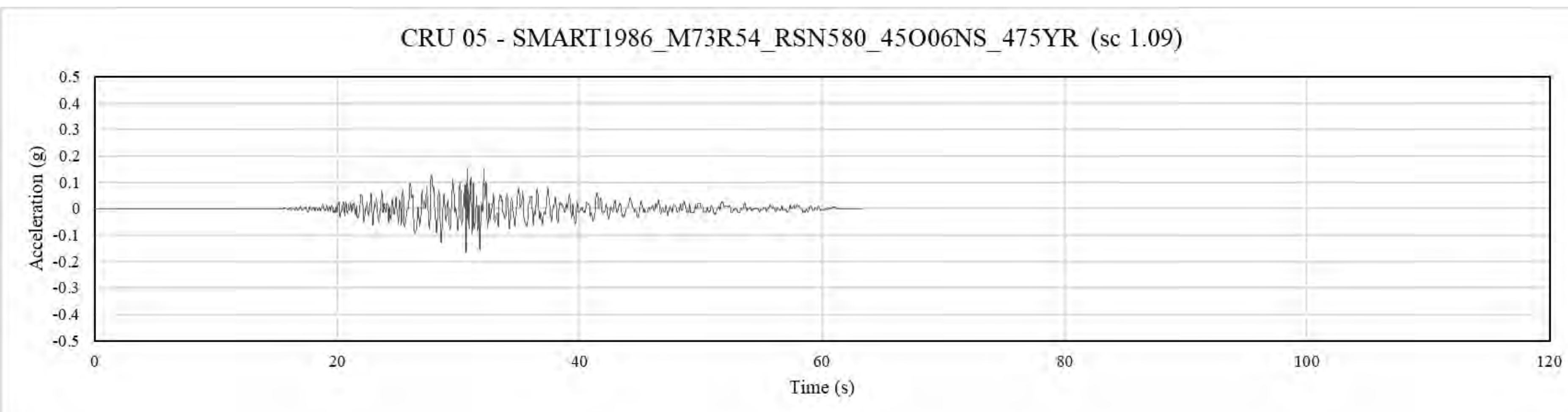
## A475 – Crustal ground motions



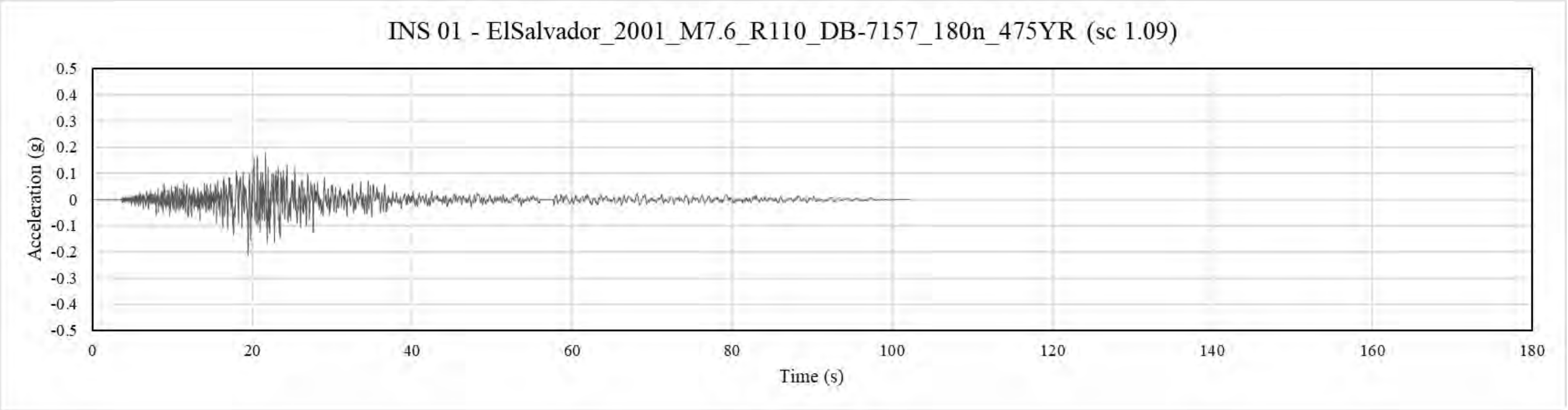



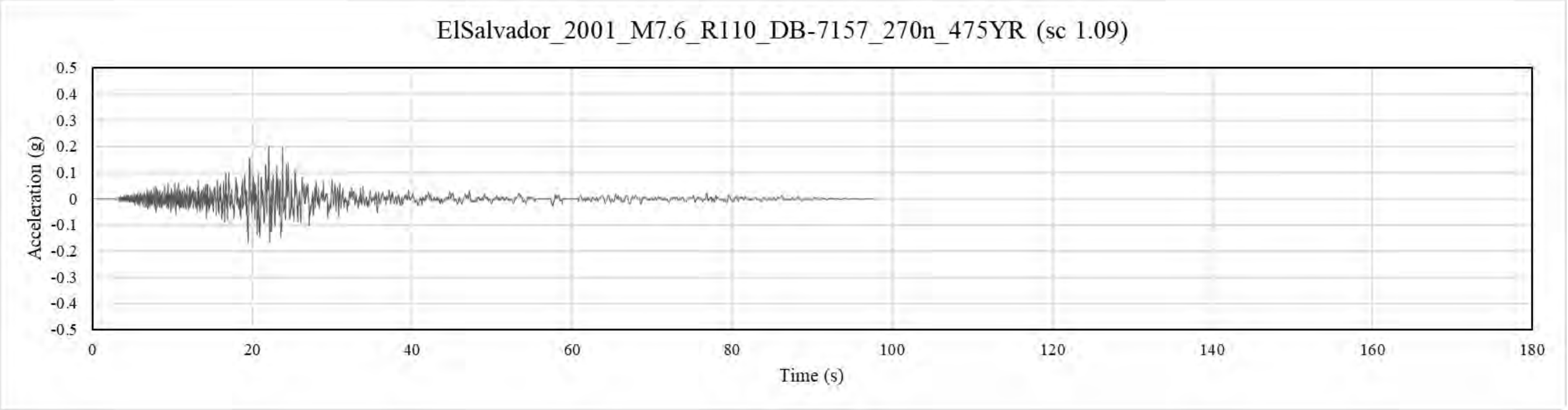
# A475 – Crustal ground motions



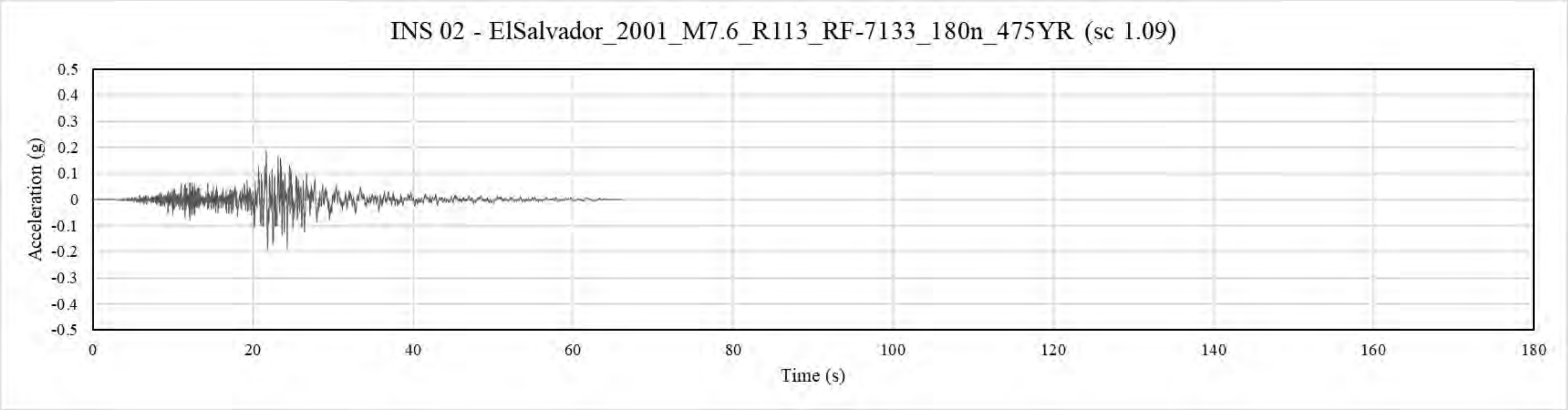



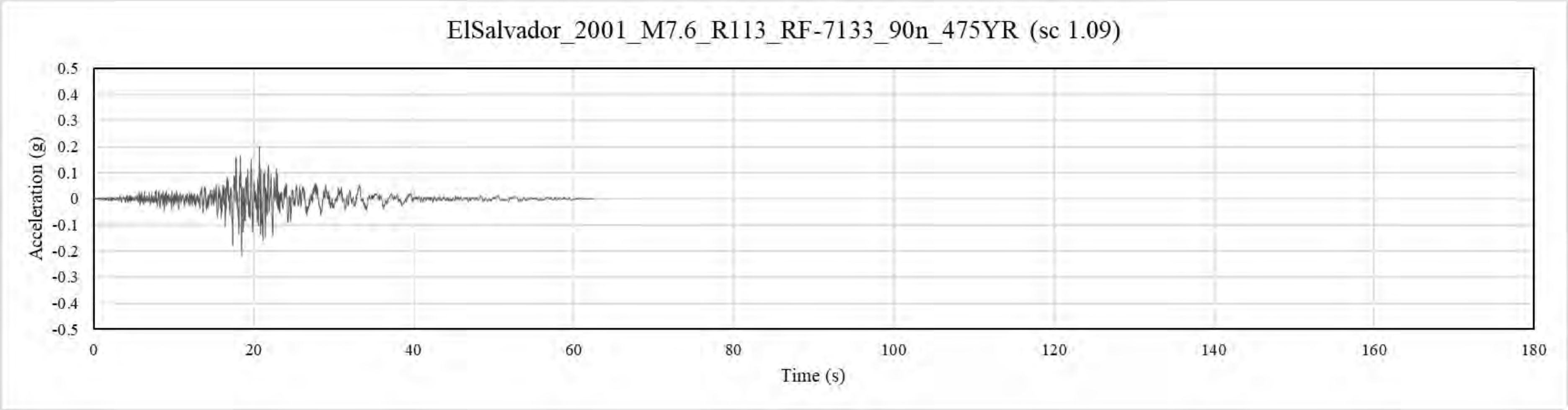
# A475 – In-slab ground motions



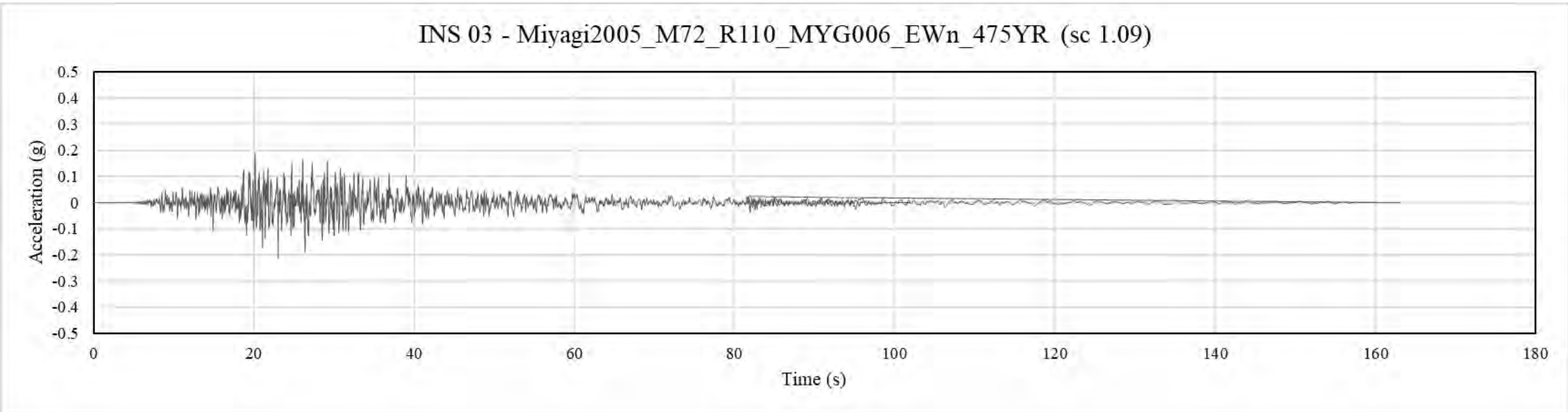



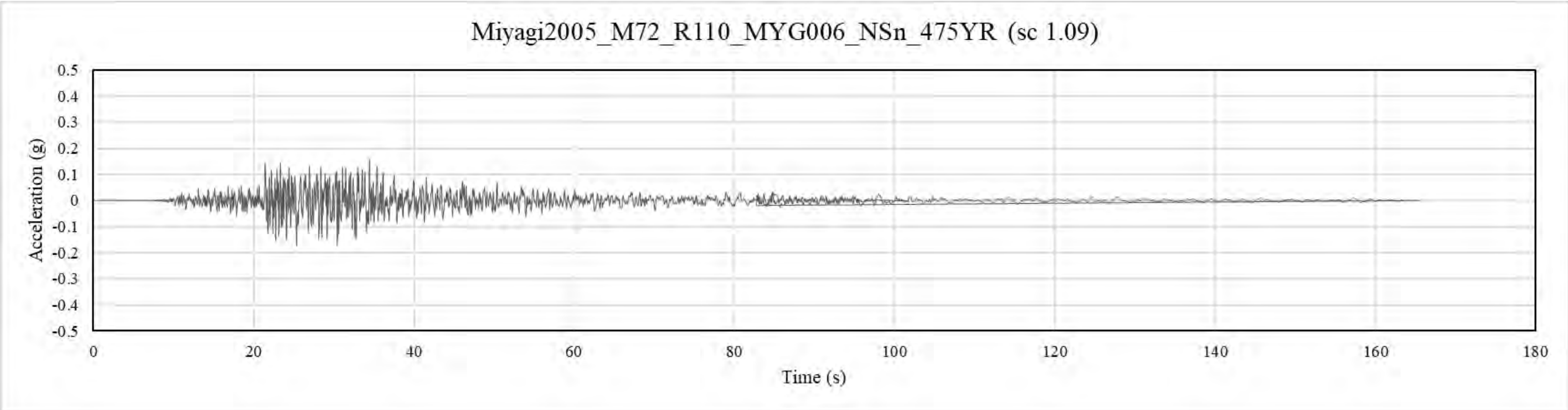
# A475 – In-slab ground motions



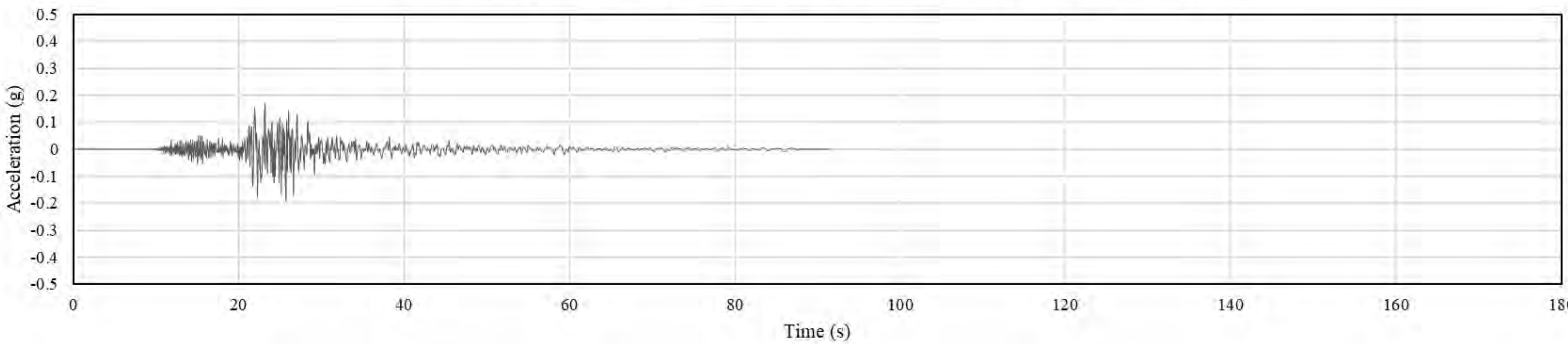
# A475 – In-slab ground motions



# A475 – In-slab ground motions

INS 04 - Nisqually_2001_M6.8_R75_7032-1416_125n_475YR (sc 1.09)

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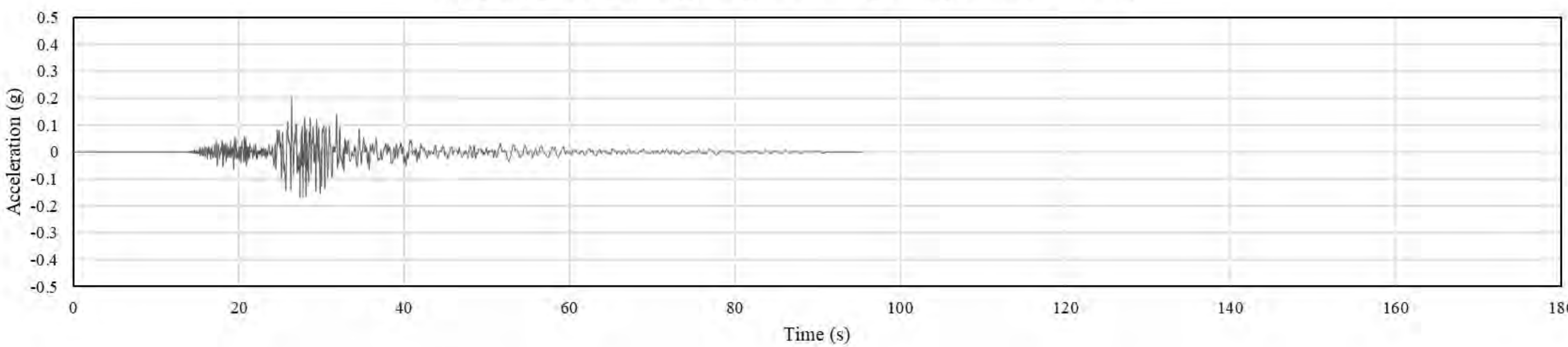
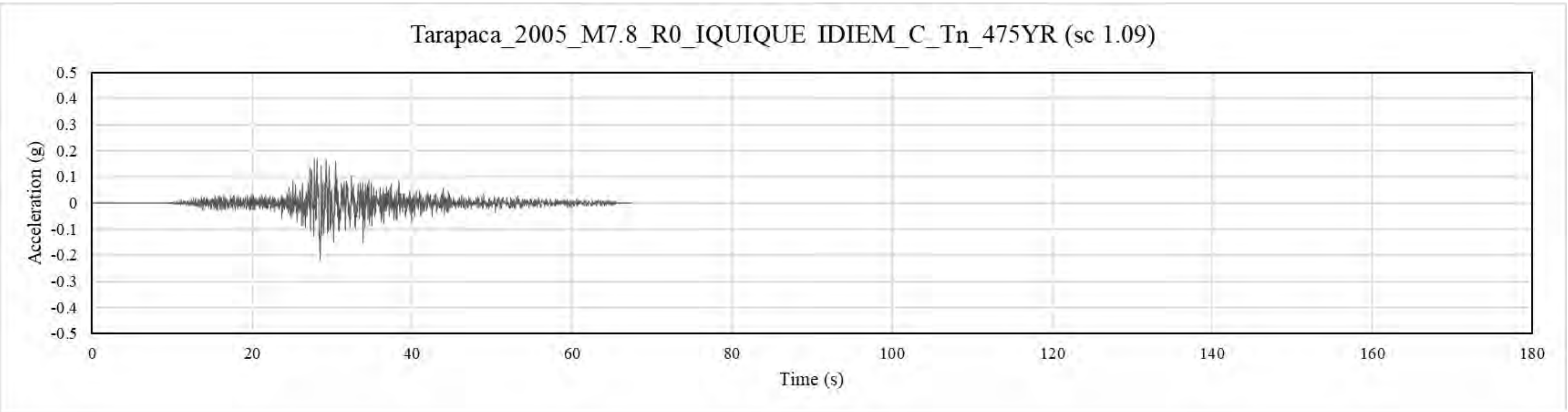
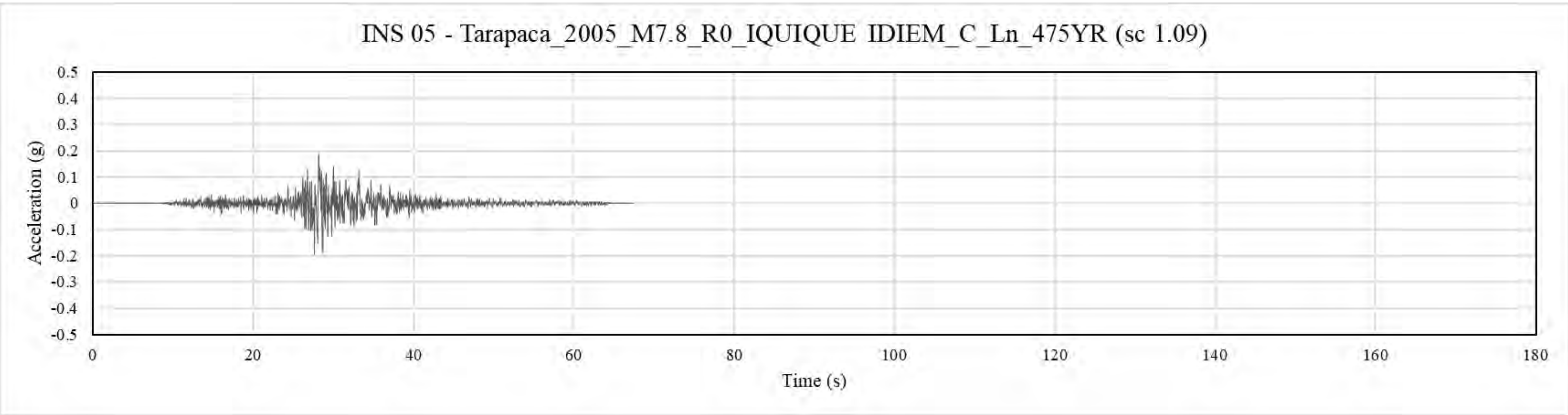
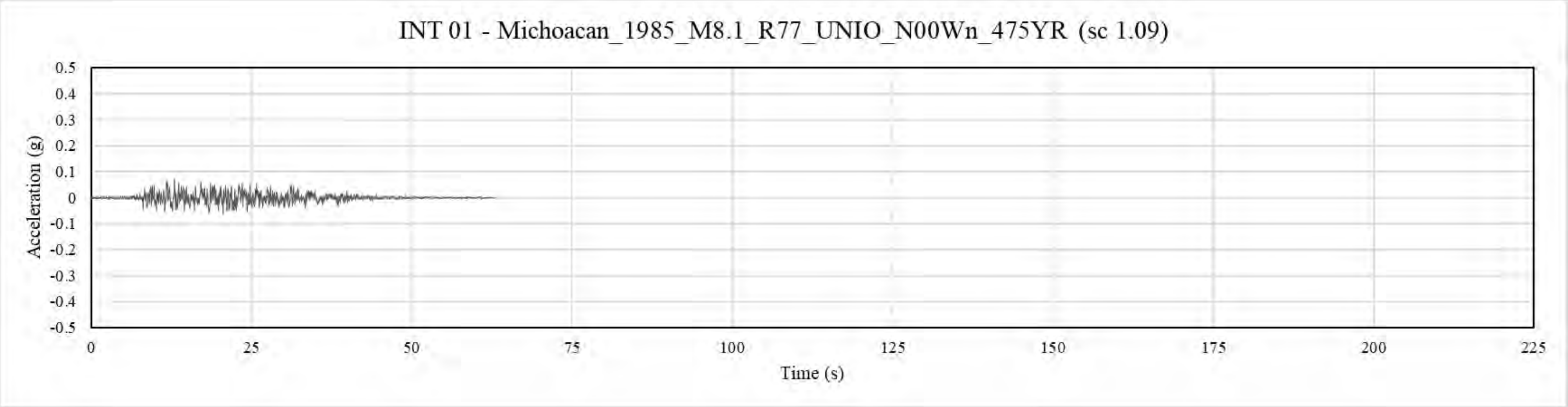


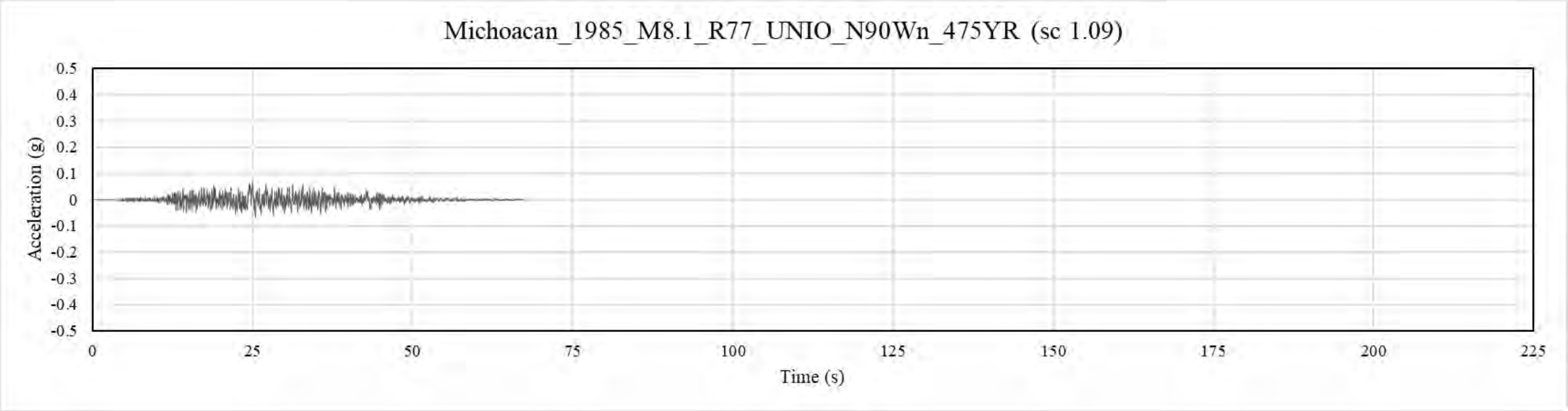
# A475 – In-slab ground motions



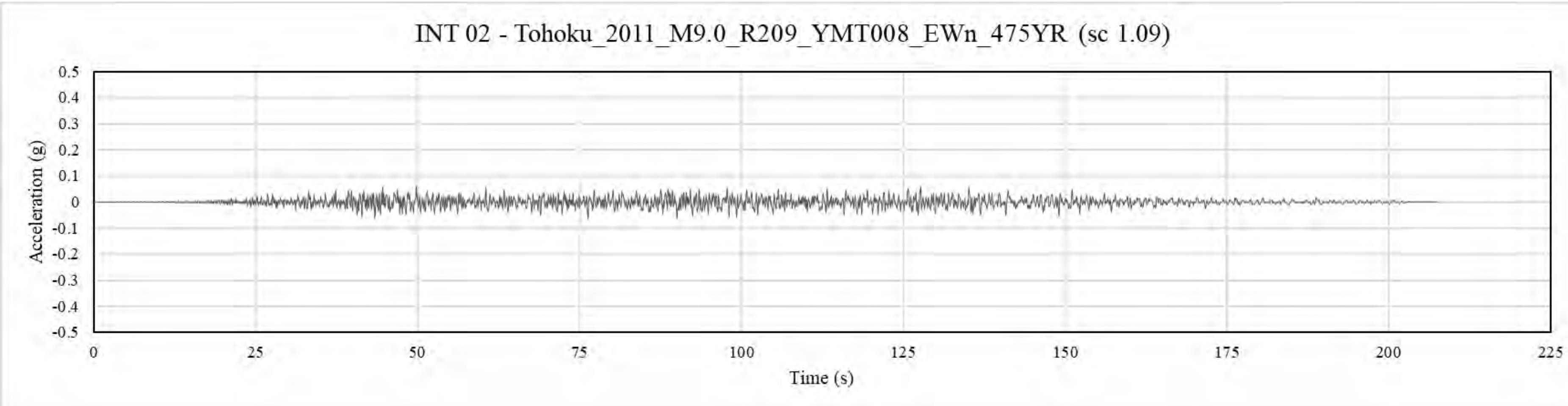



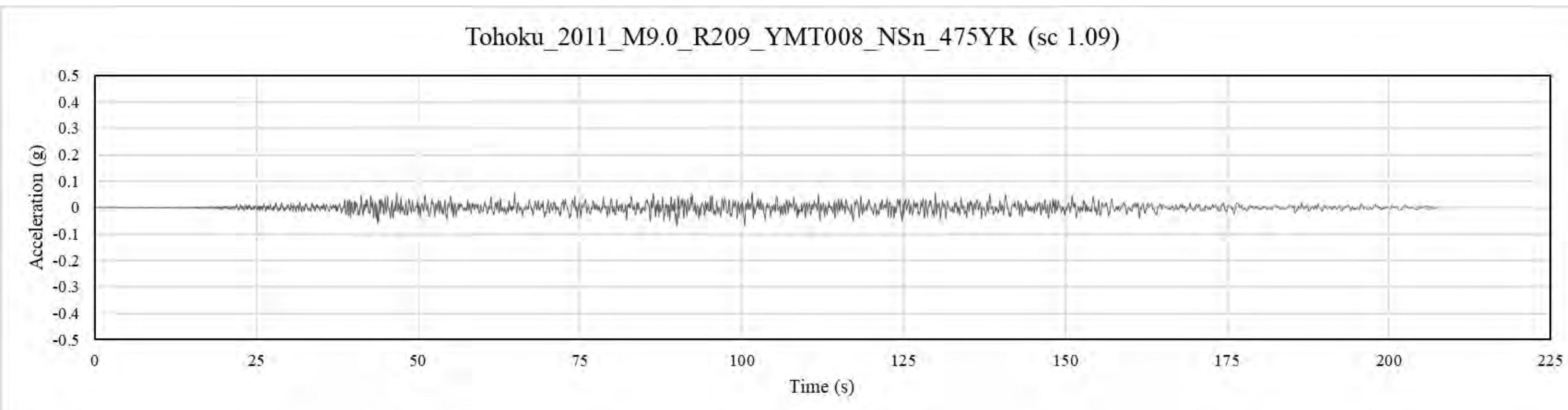
# A475 – Interface ground motions



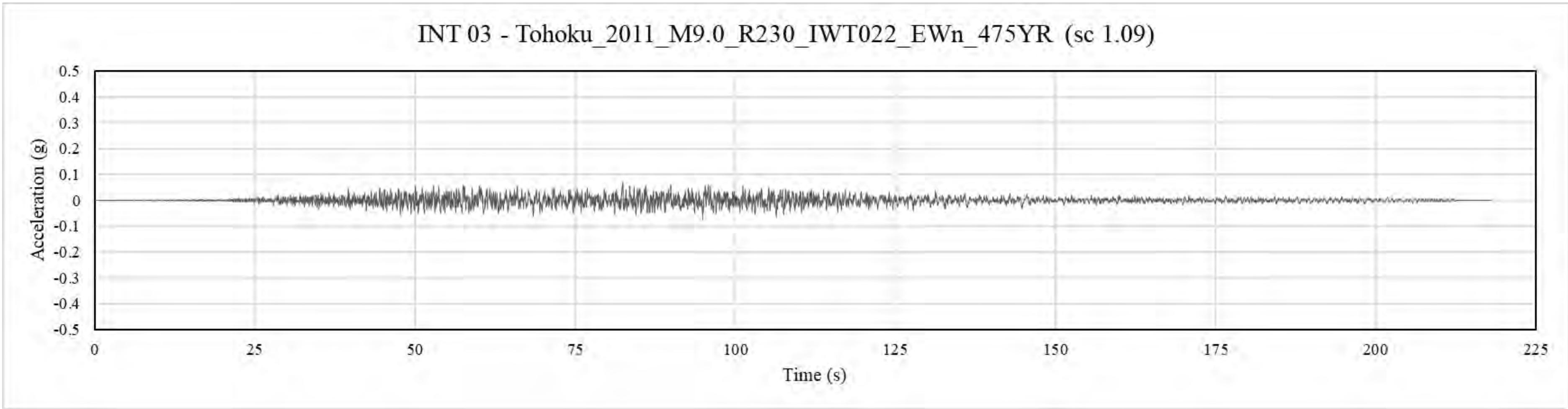



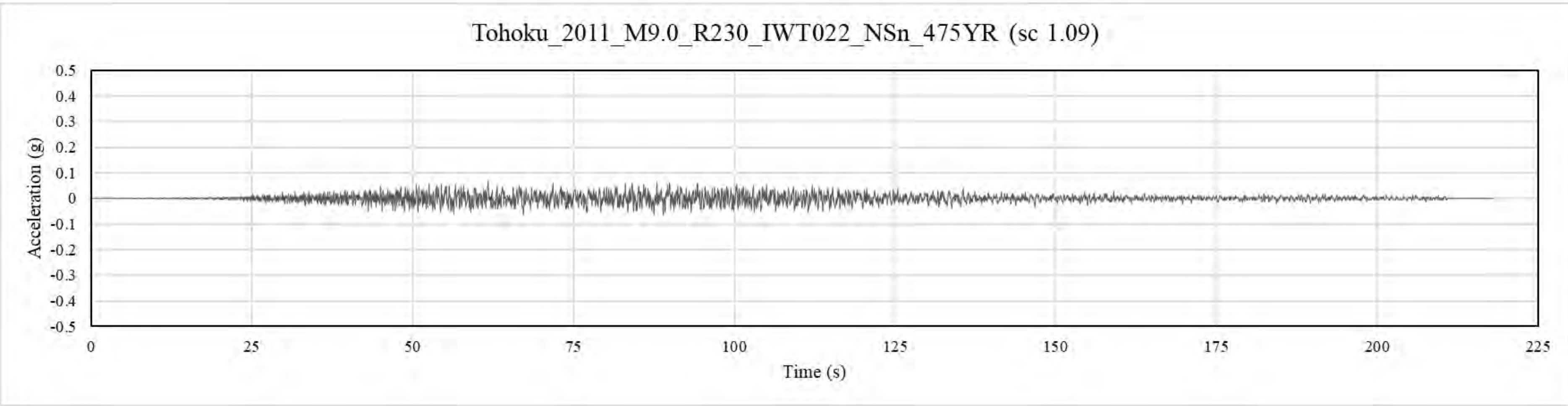
A475 – Interface ground motions



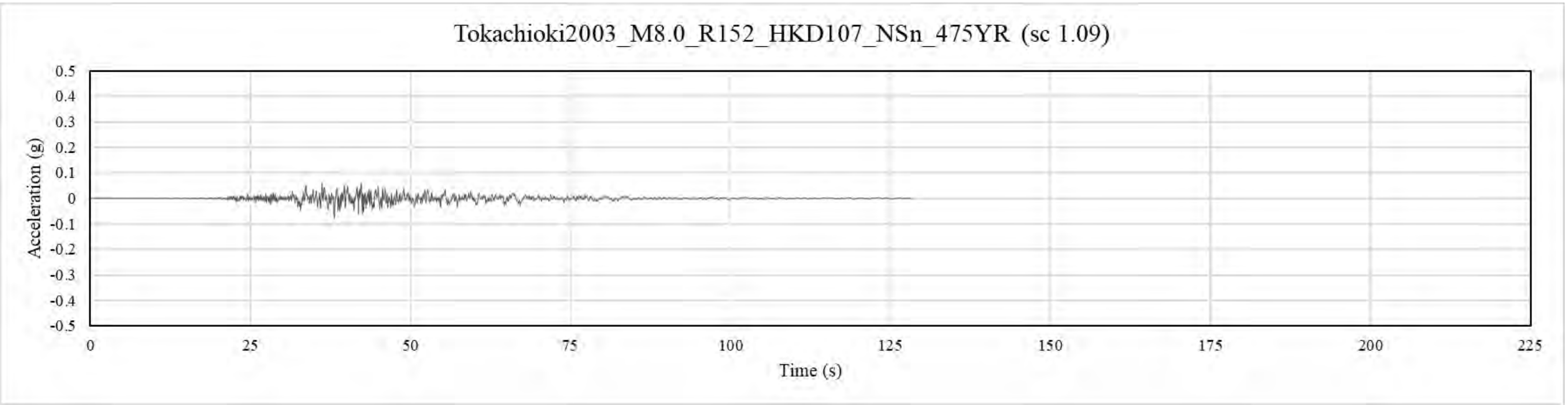



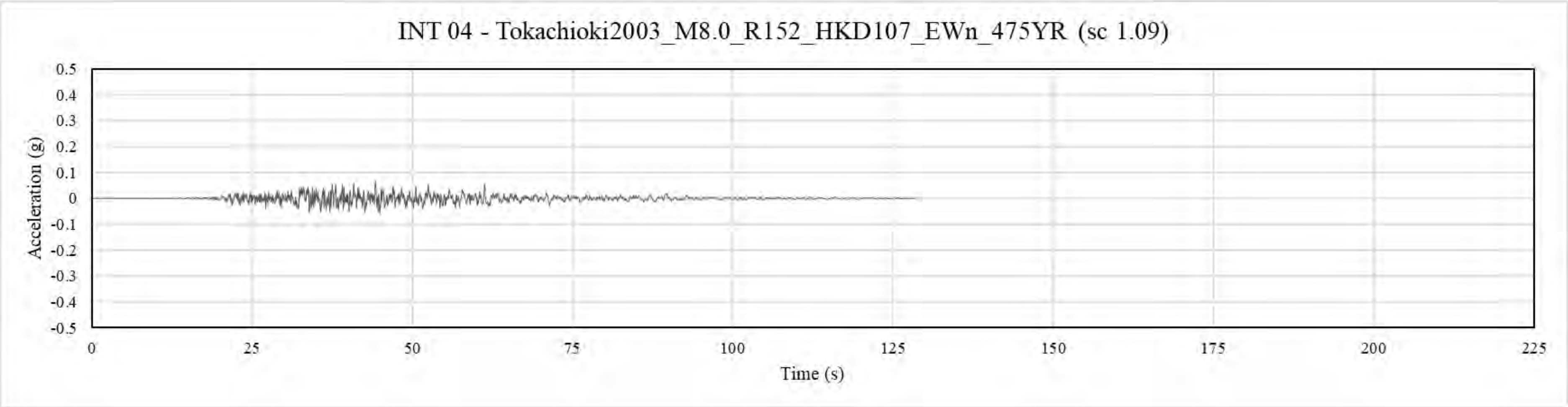
# A475 – Interface ground motions



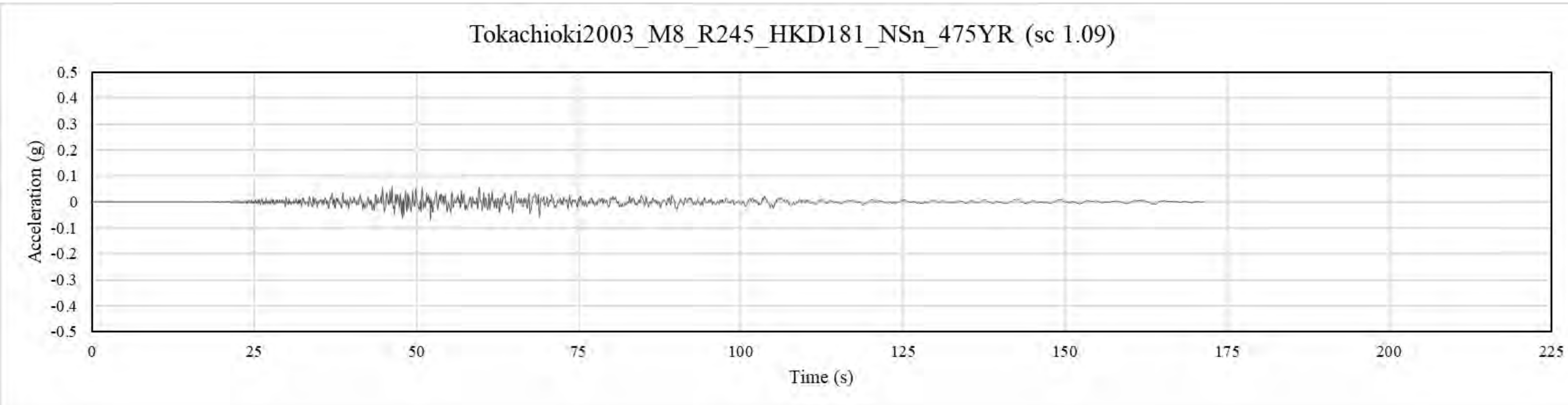



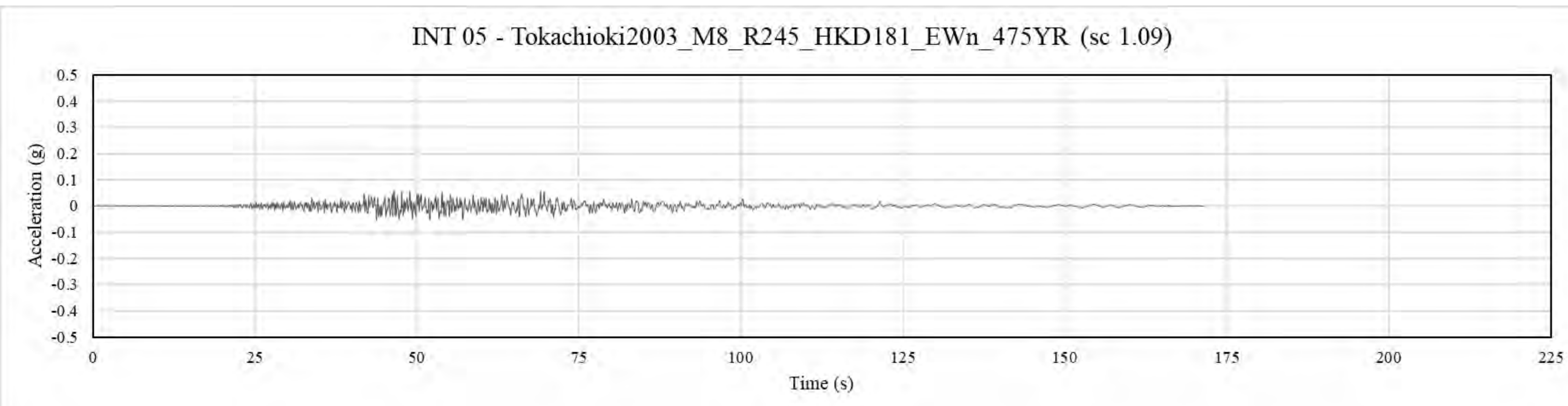
# A475 – Interface ground motions



# A475 – Interface ground motions



# Appendix D

# Liquefaction Triggering Assessment

# Liquefaction Triggering Assessment

# A2475 Crustal and Inslab motions

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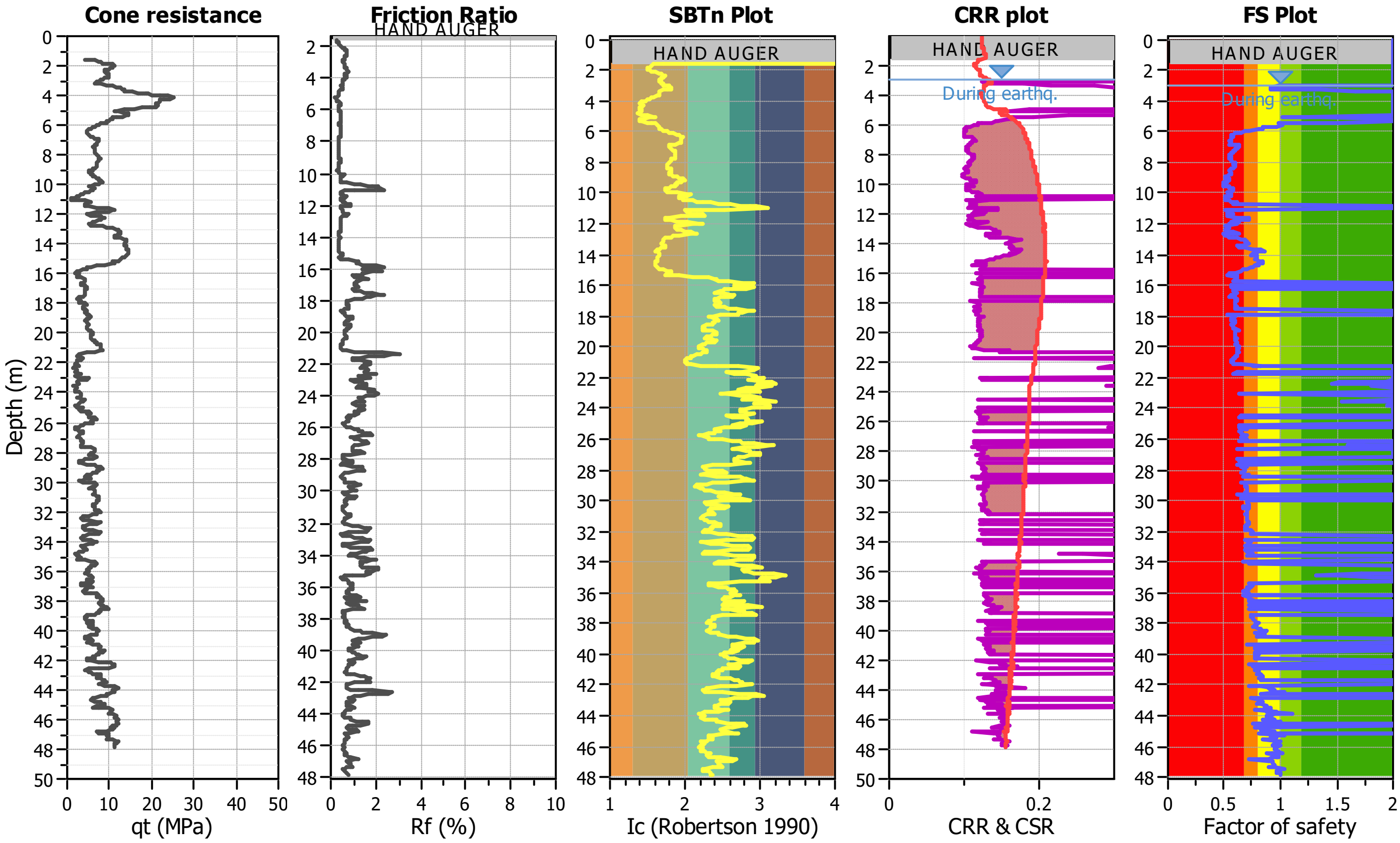
## LIQUEFACTION ANALYSIS REPORT

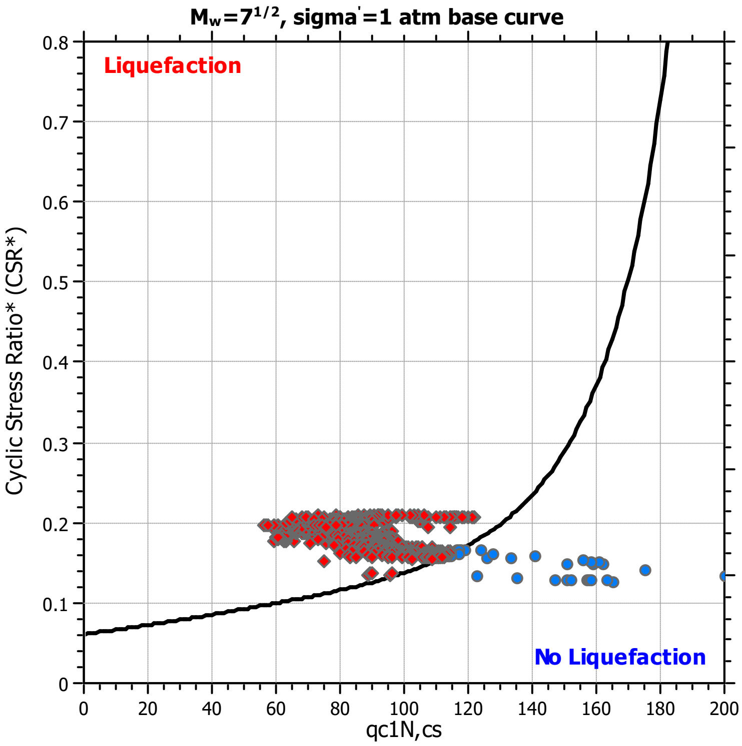
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

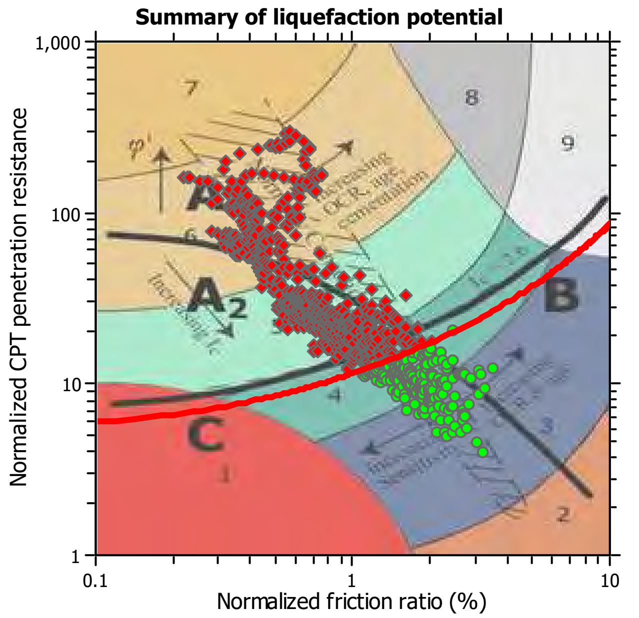
**CPT file : CPT18-01 Berth 2**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Braun Geotechnical Ltd.

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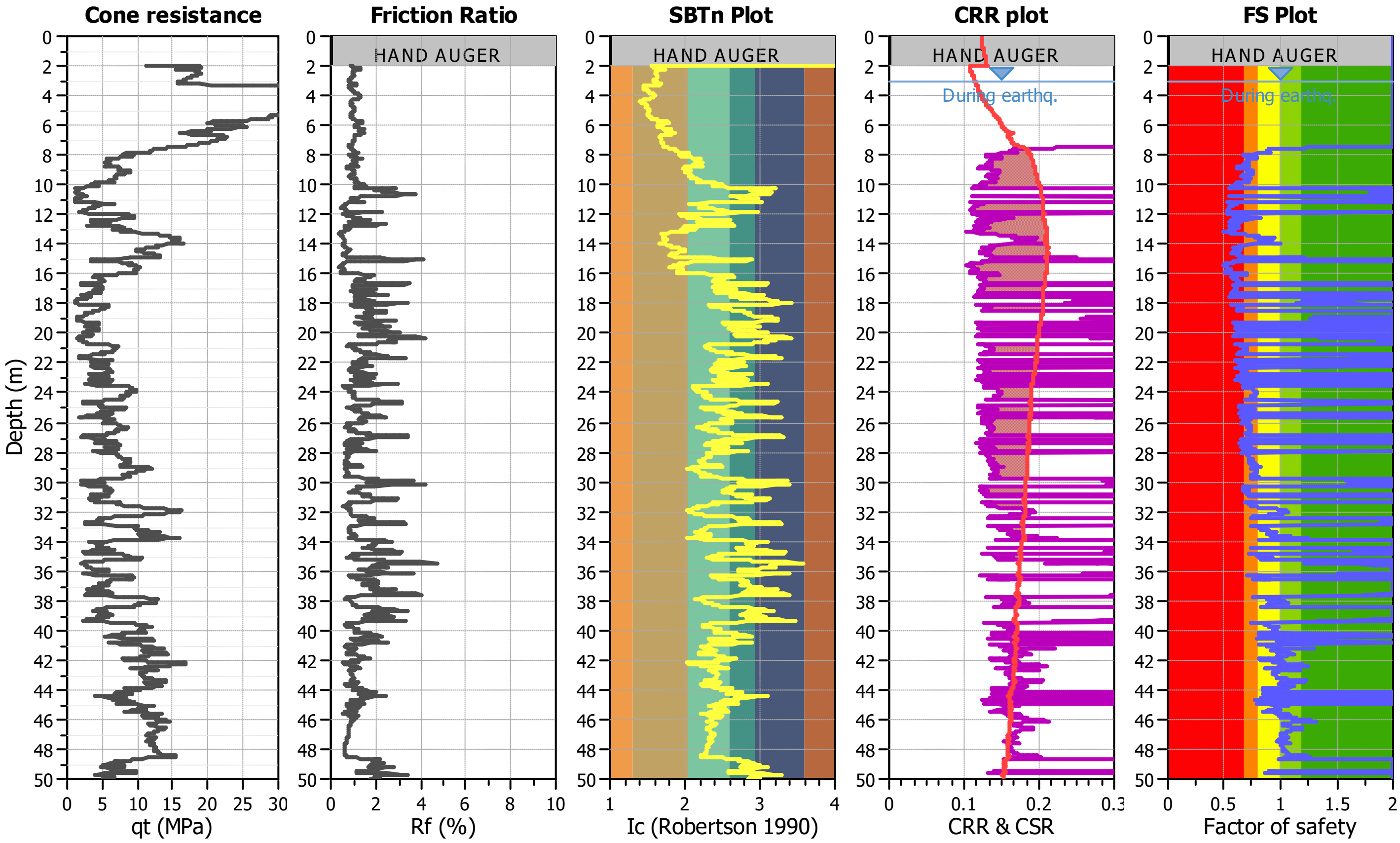
## LIQUEFACTION ANALYSIS REPORT

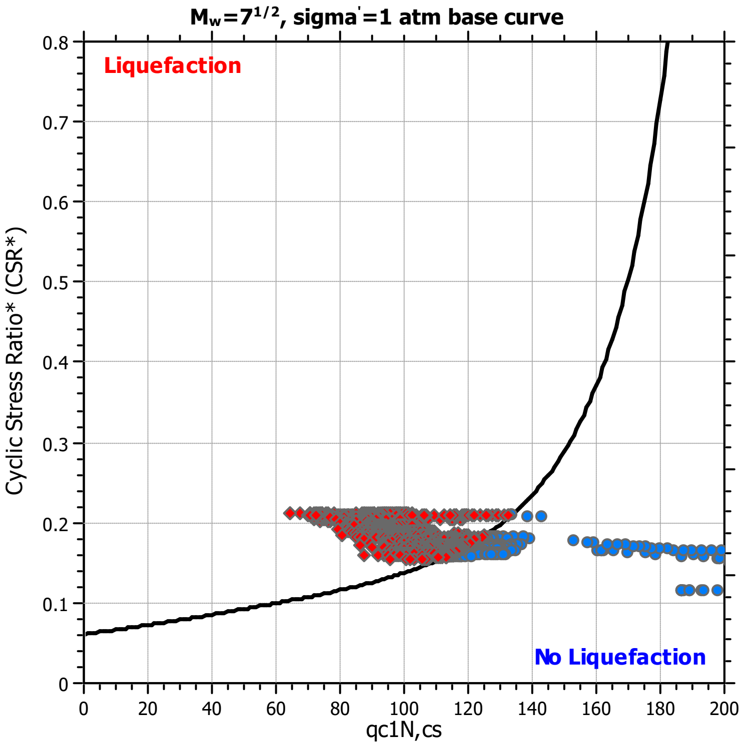
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

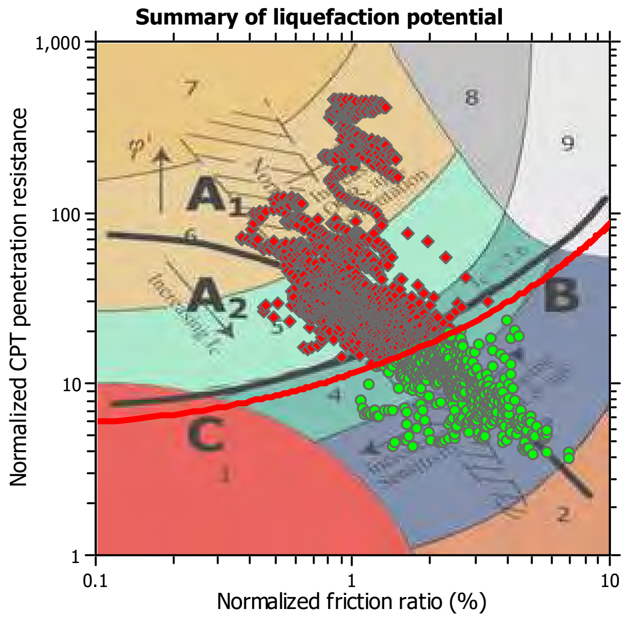
**CPT file : SCPT20-01**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

**Braun Geotechnical Ltd.**

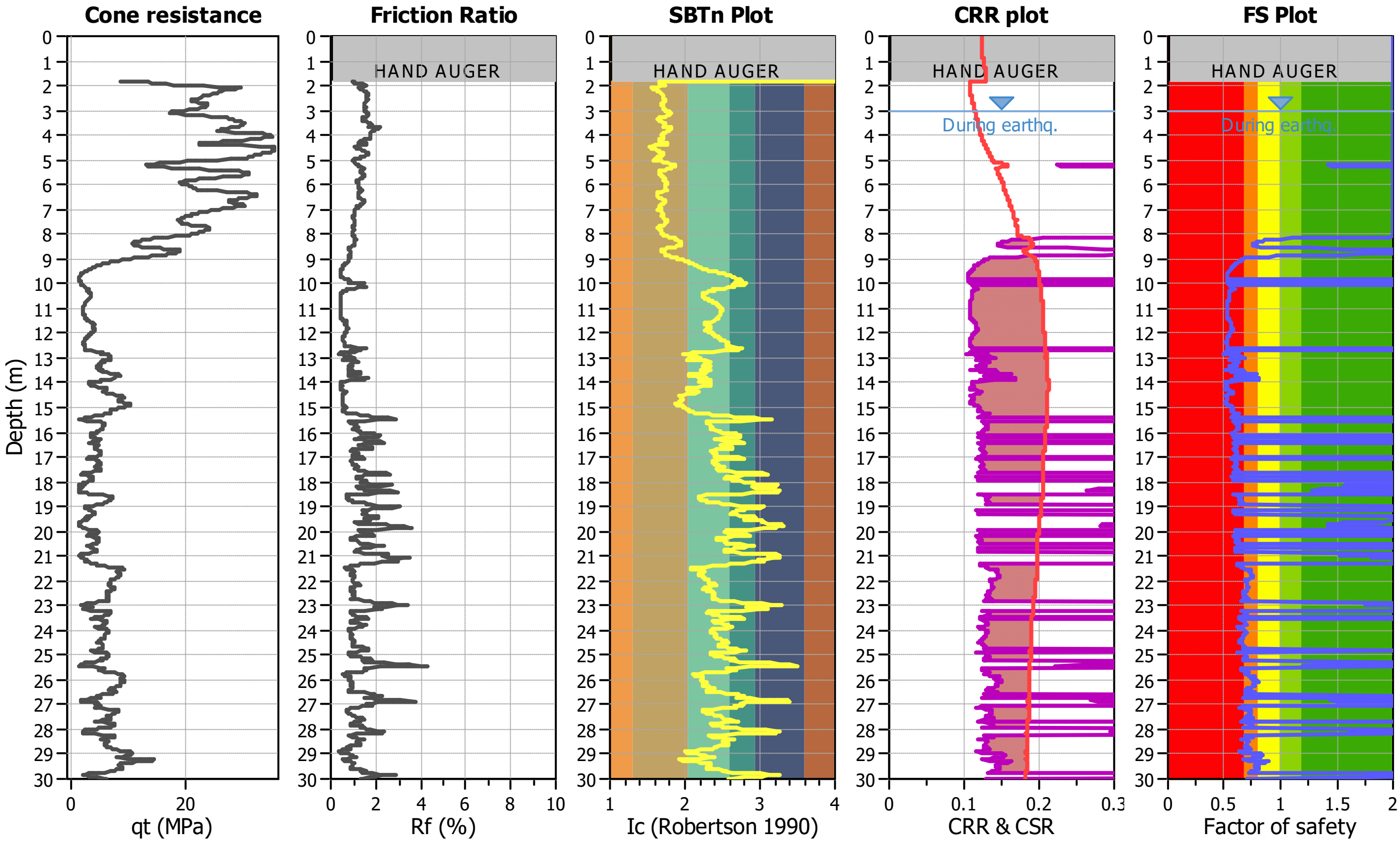
## LIQUEFACTION ANALYSIS REPORT

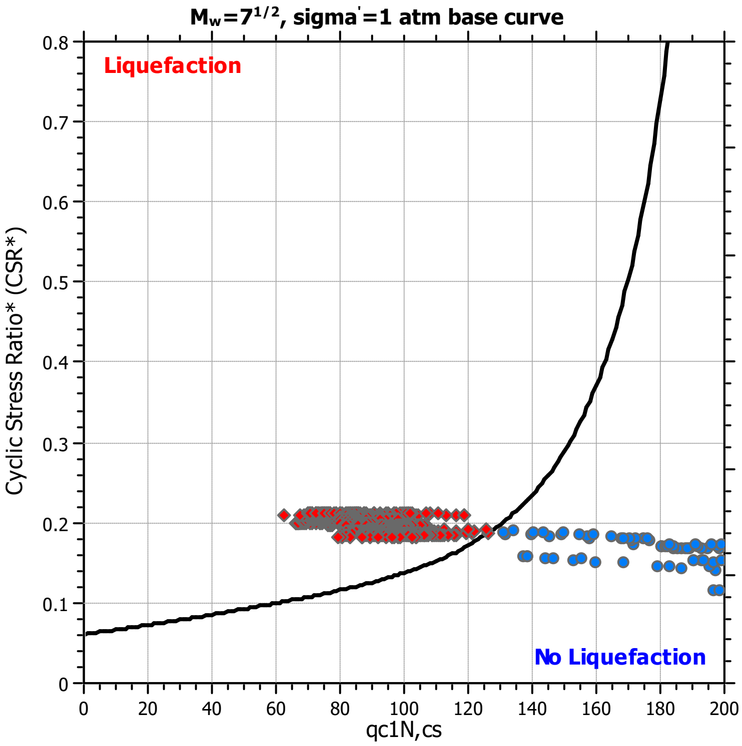
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

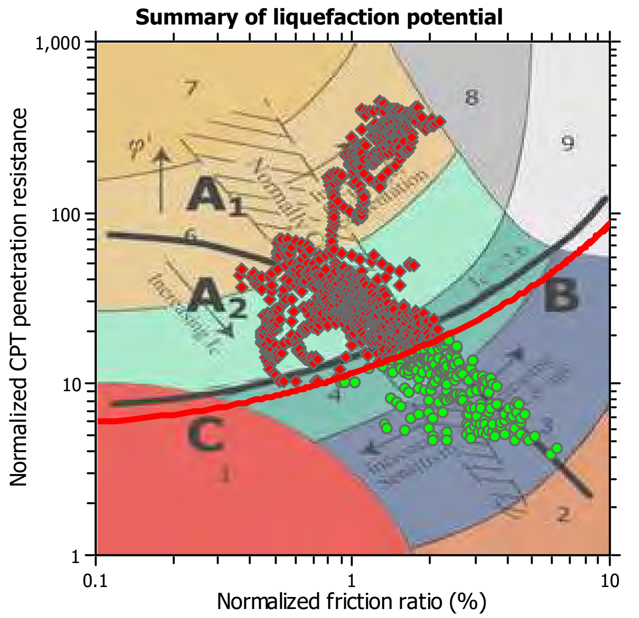
**CPT file : CPT20-02**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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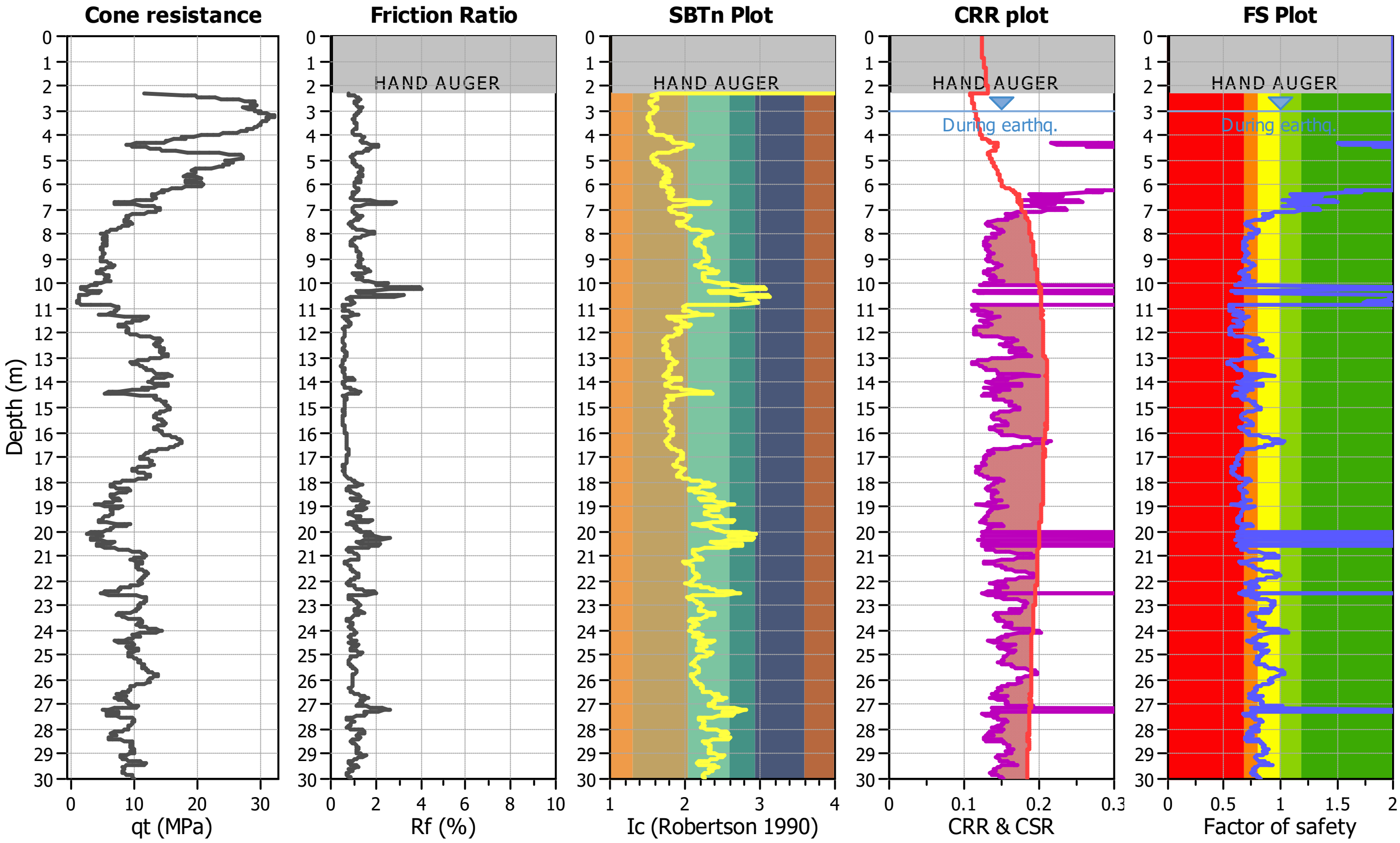
## LIQUEFACTION ANALYSIS REPORT

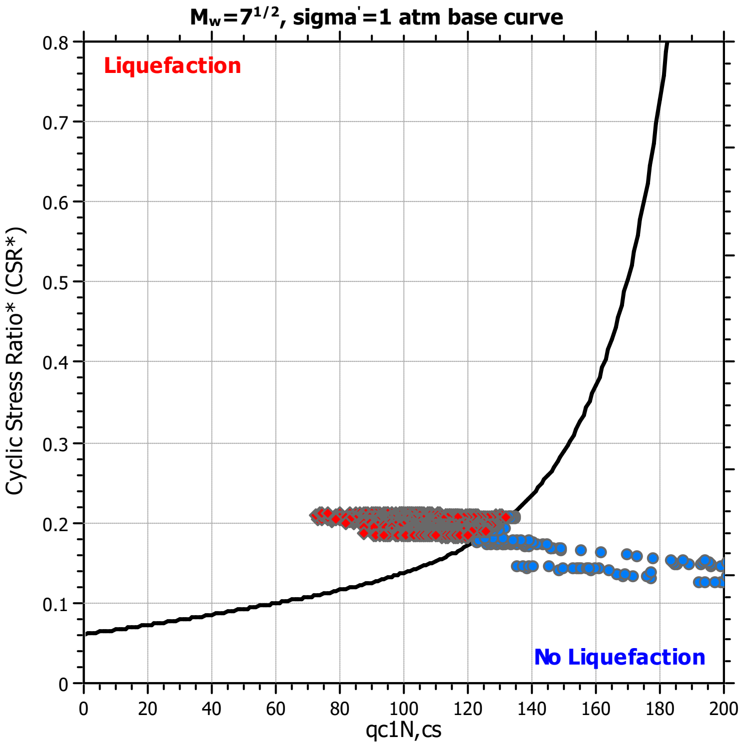
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

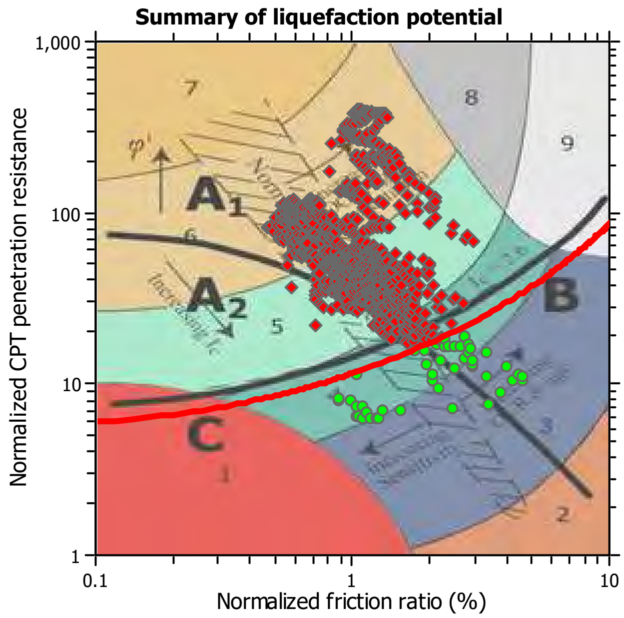
**CPT file : CPT20-03**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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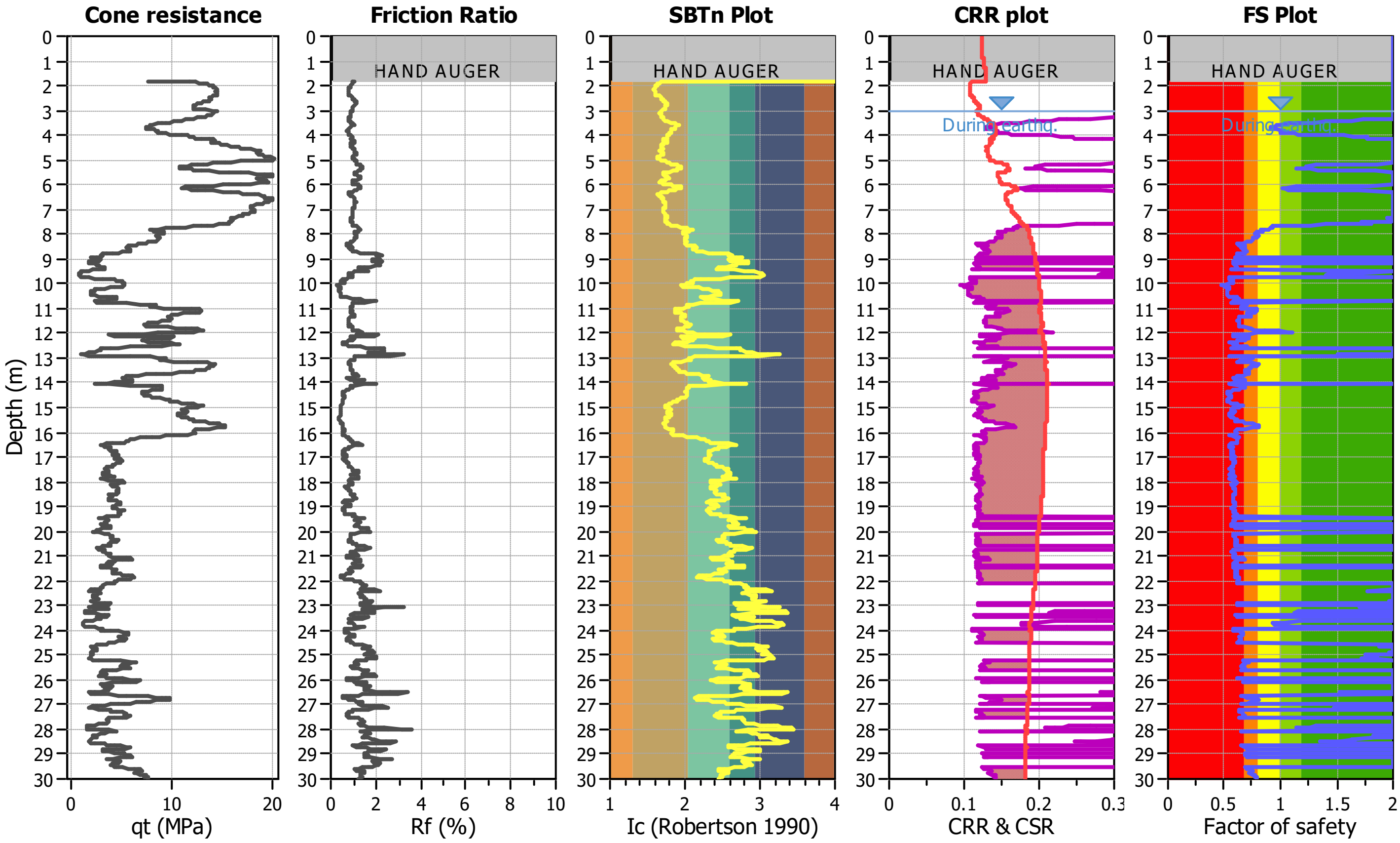
# LIQUEFACTION ANALYSIS REPORT

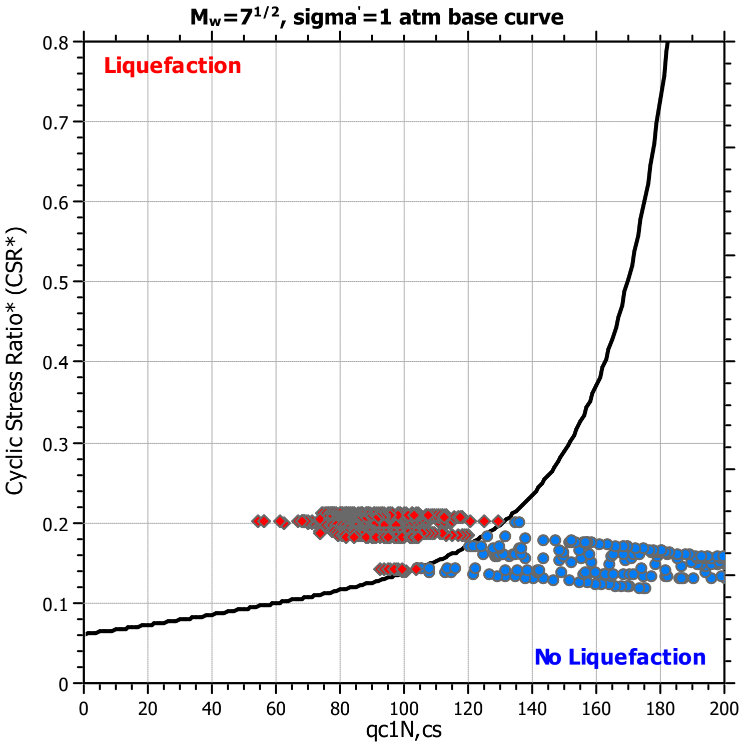
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

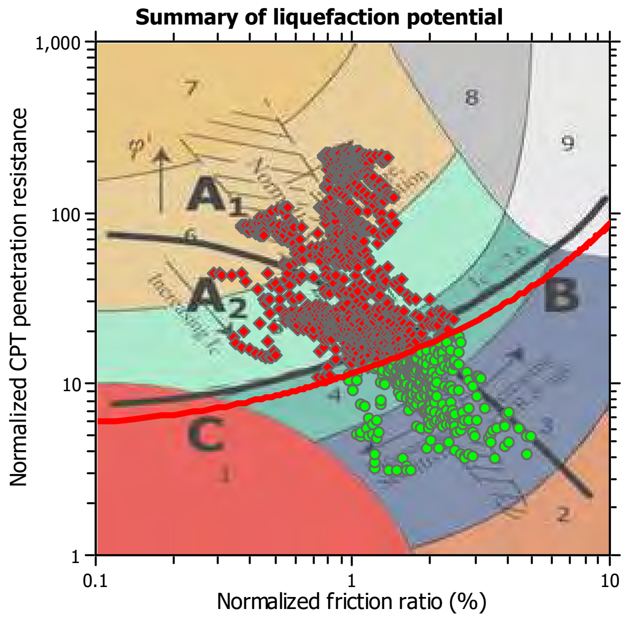
**CPT file : CP20-04**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | | |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | No |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | Limit depth: | N/A |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | MSF method: | Method |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

**Braun Geotechnical Ltd.**

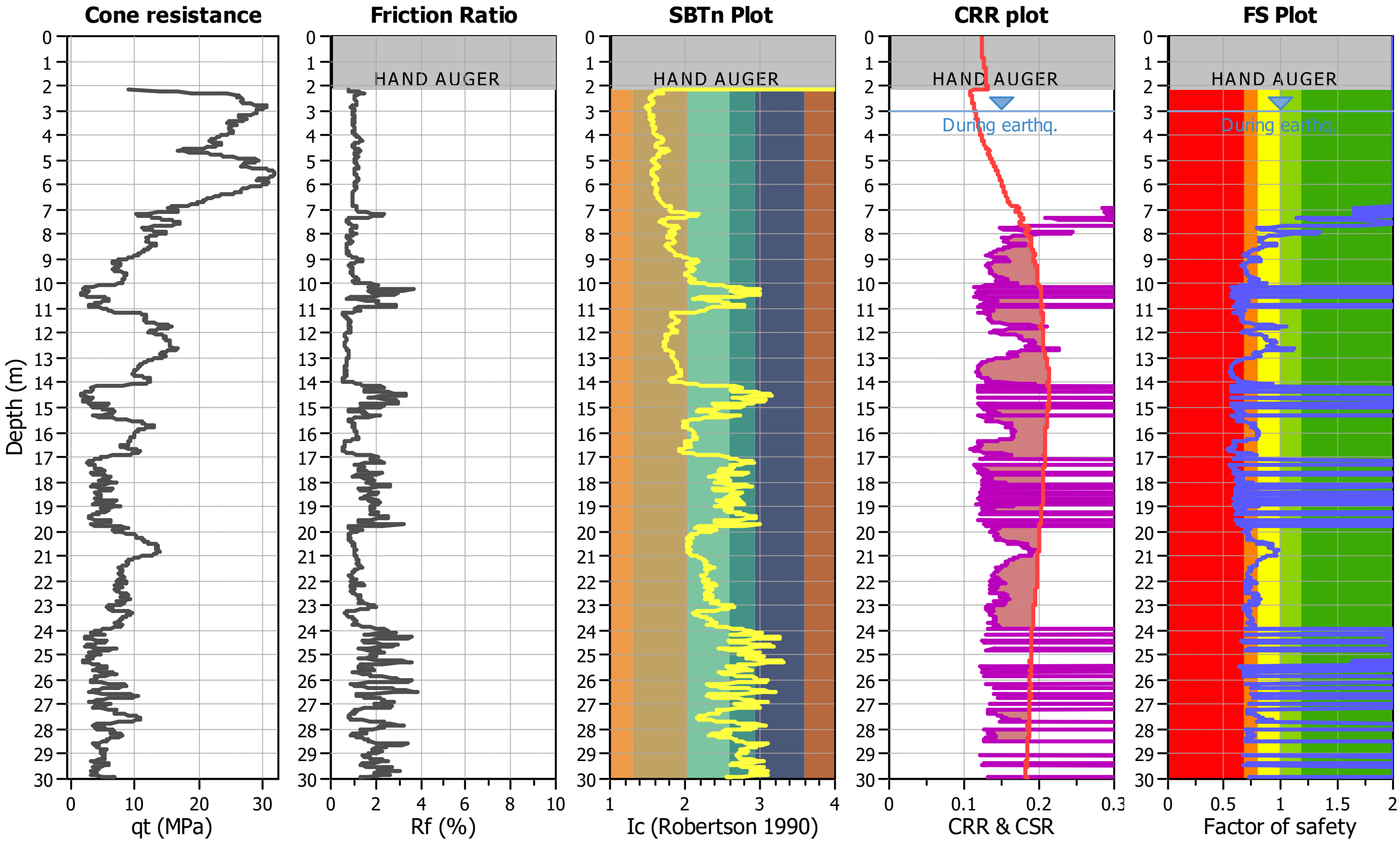
## LIQUEFACTION ANALYSIS REPORT

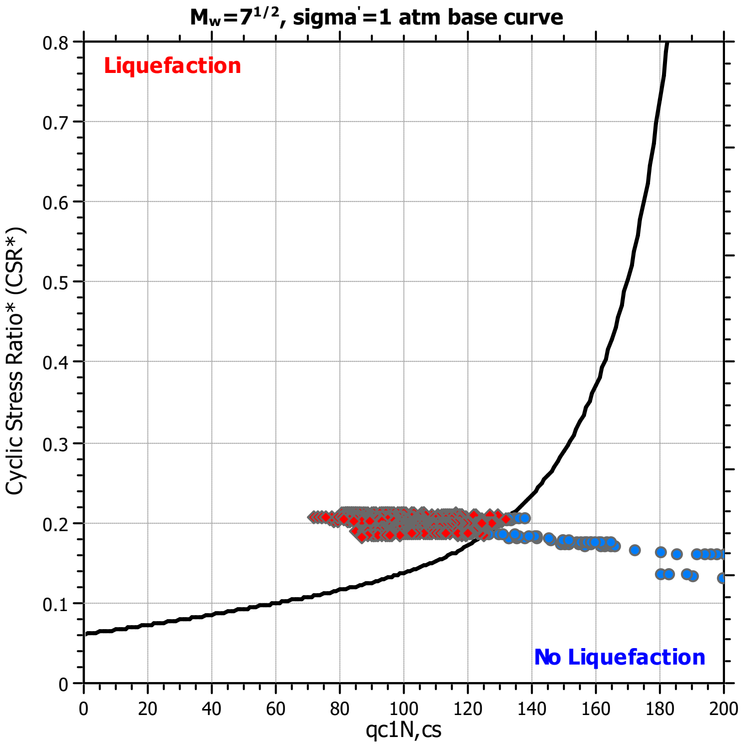
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

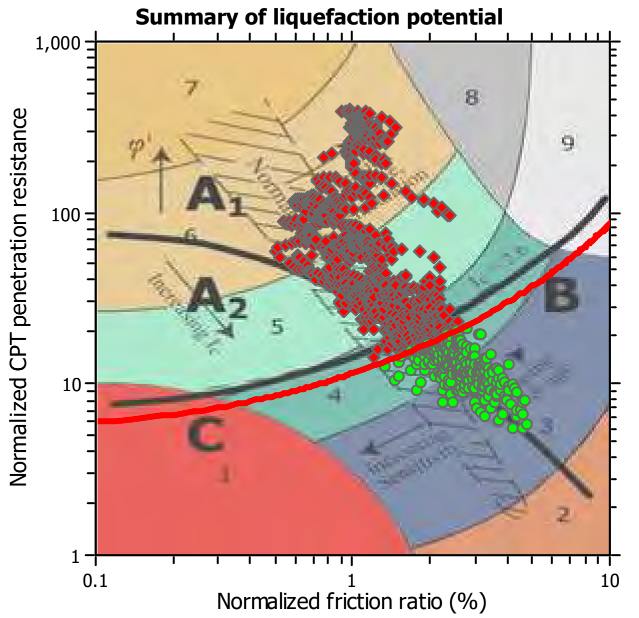
**CPT file : CPT20-05**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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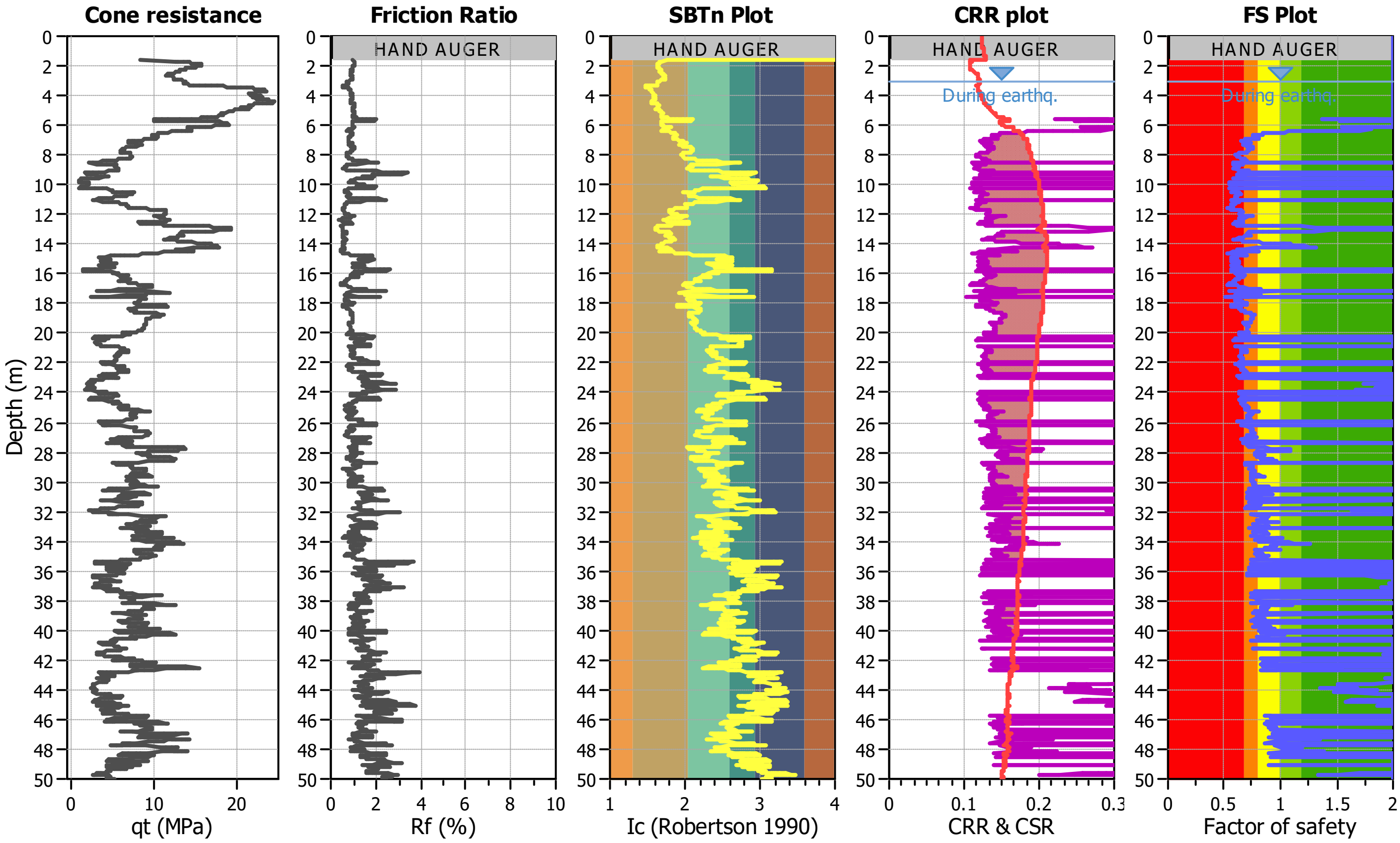
## LIQUEFACTION ANALYSIS REPORT

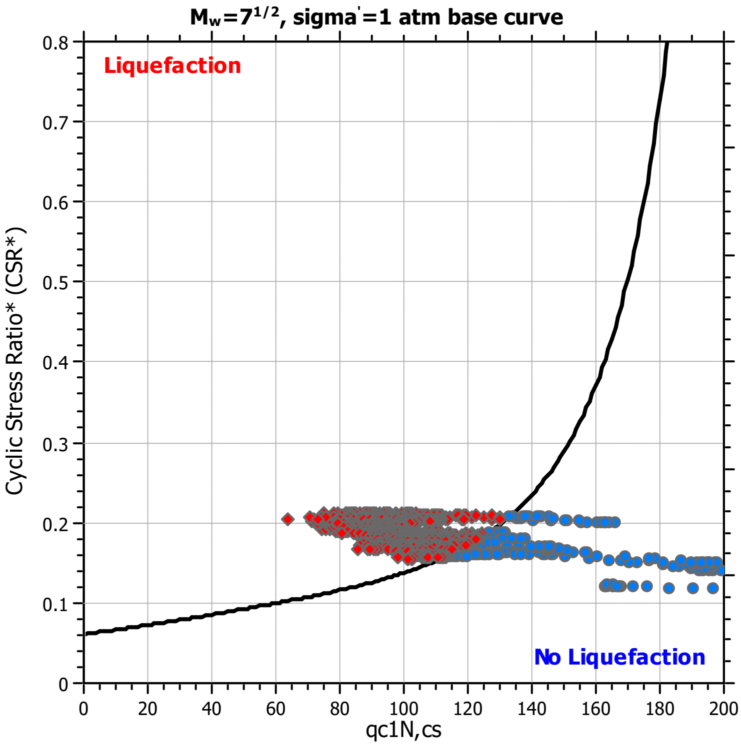
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

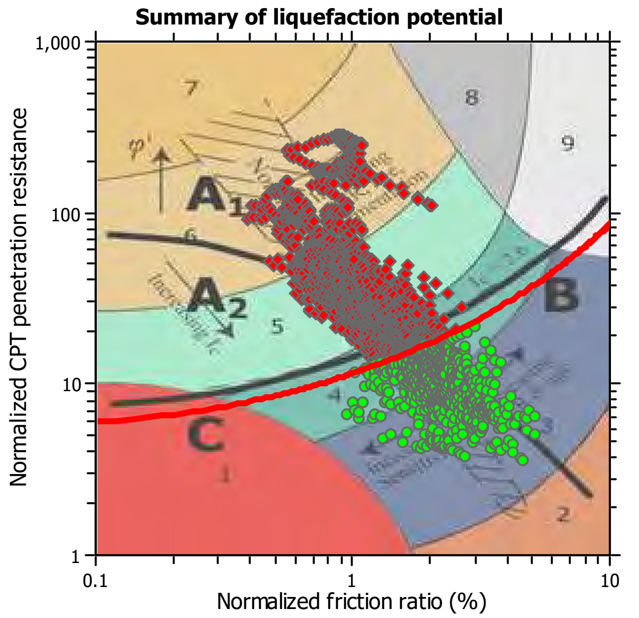
**CPT file : SCPT20-06**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | | |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | No |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | Limit depth: | N/A |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | MSF method: | Method |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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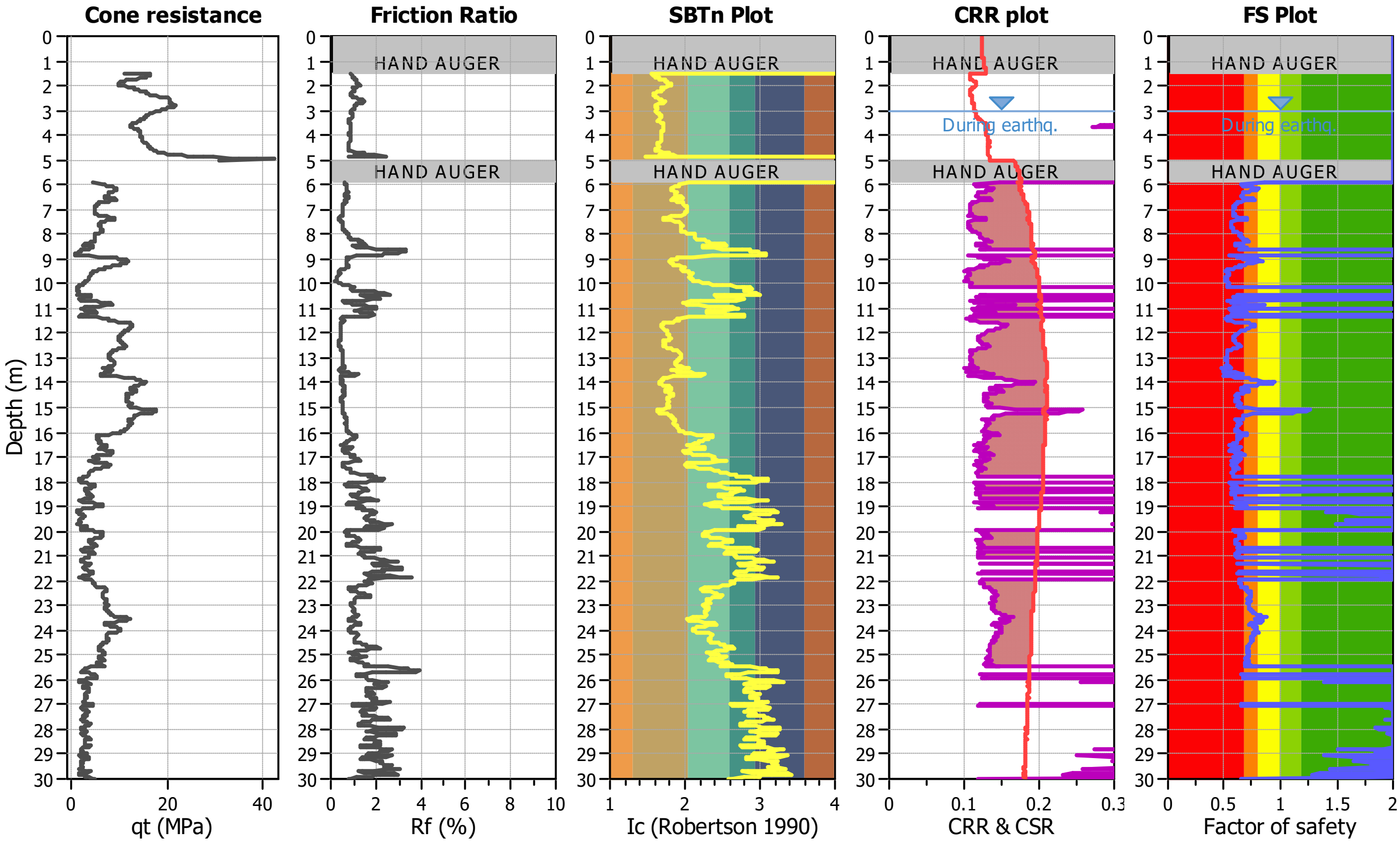
## LIQUEFACTION ANALYSIS REPORT

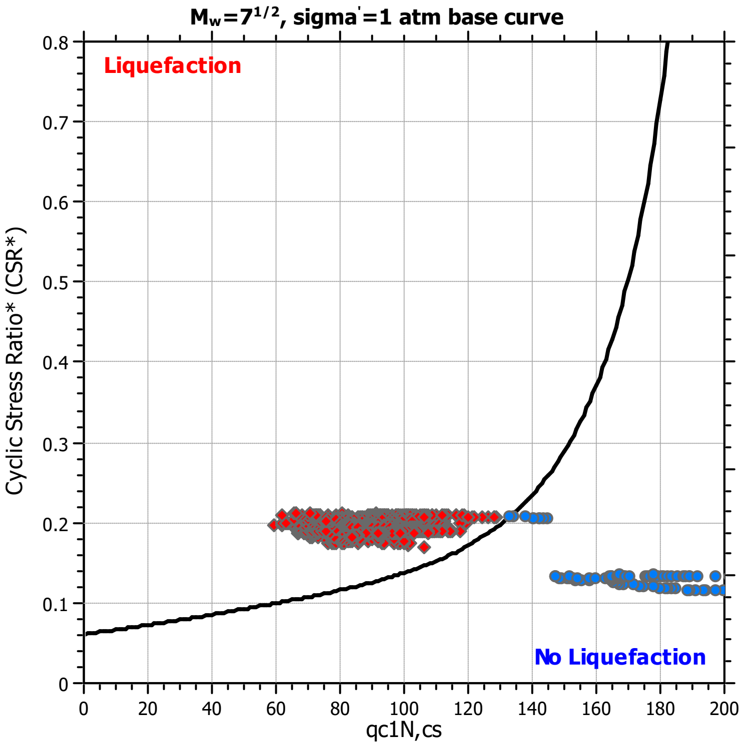
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

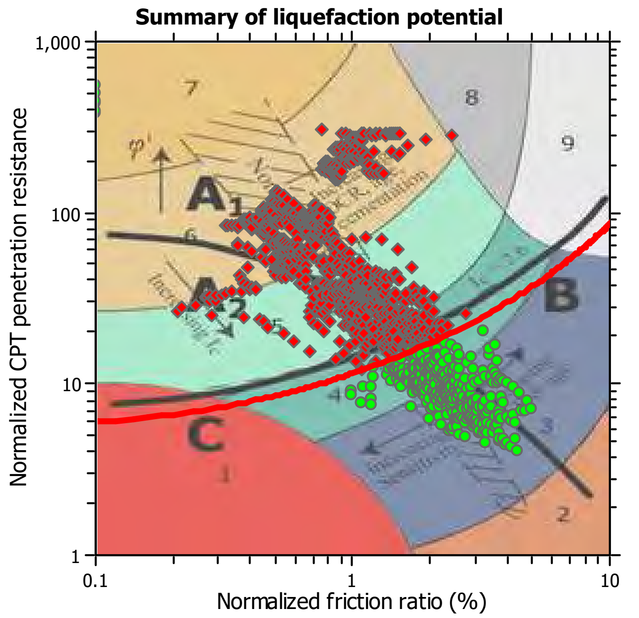
**CPT file : CPT20-08**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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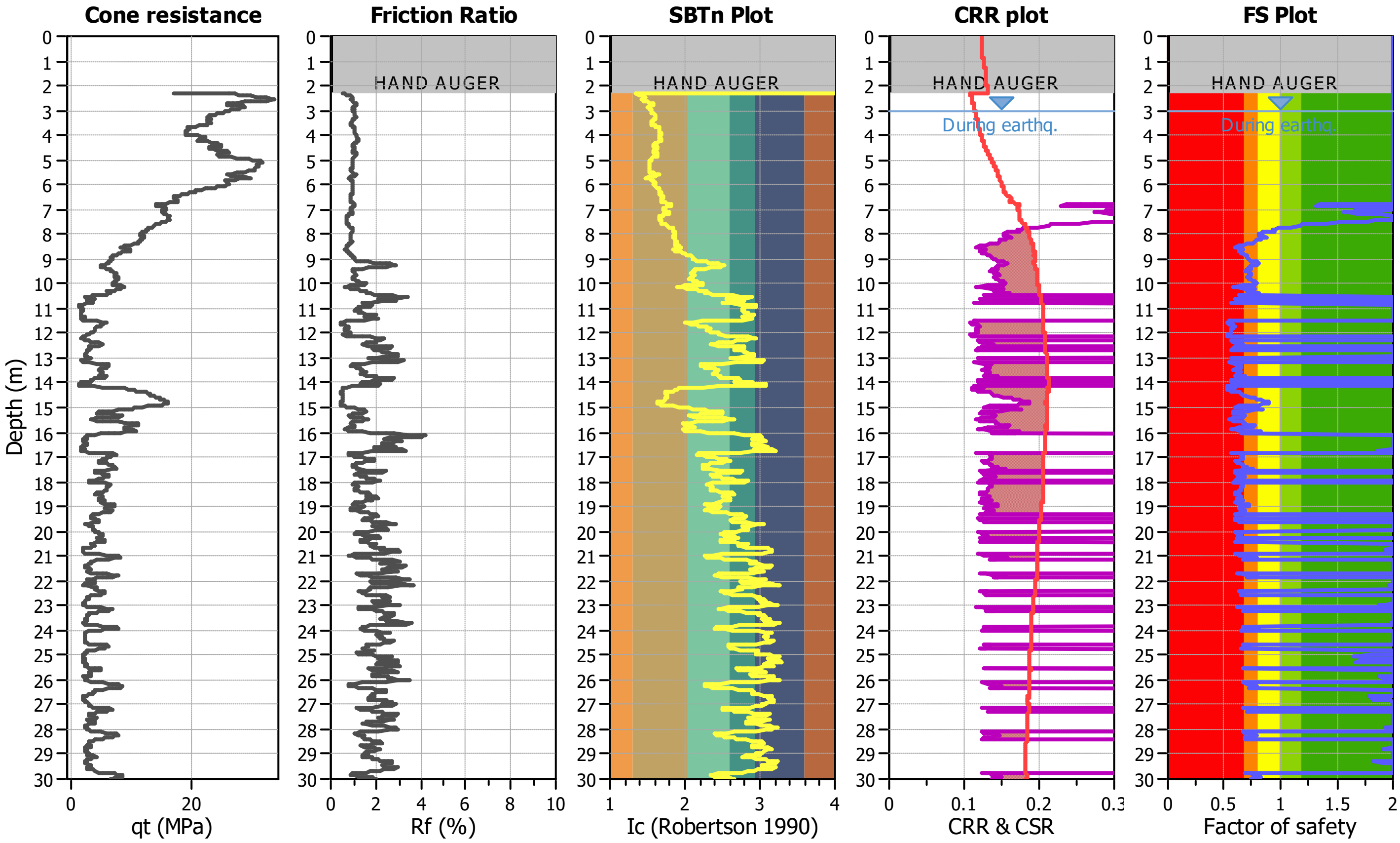
## LIQUEFACTION ANALYSIS REPORT

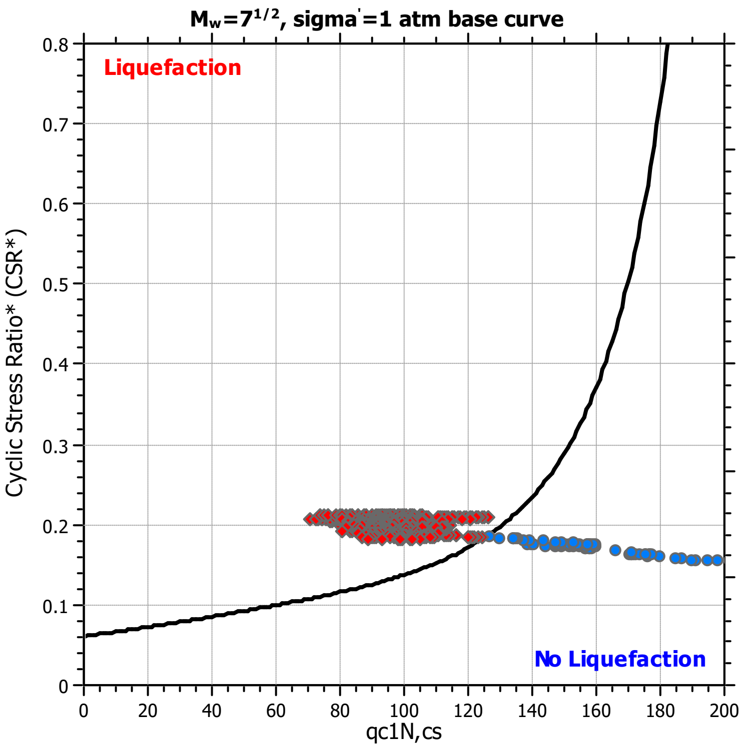
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

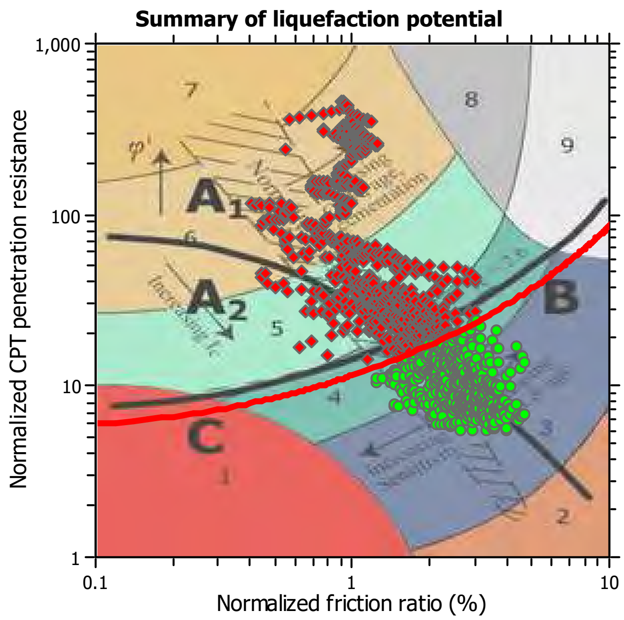
**CPT file : CPT20-07**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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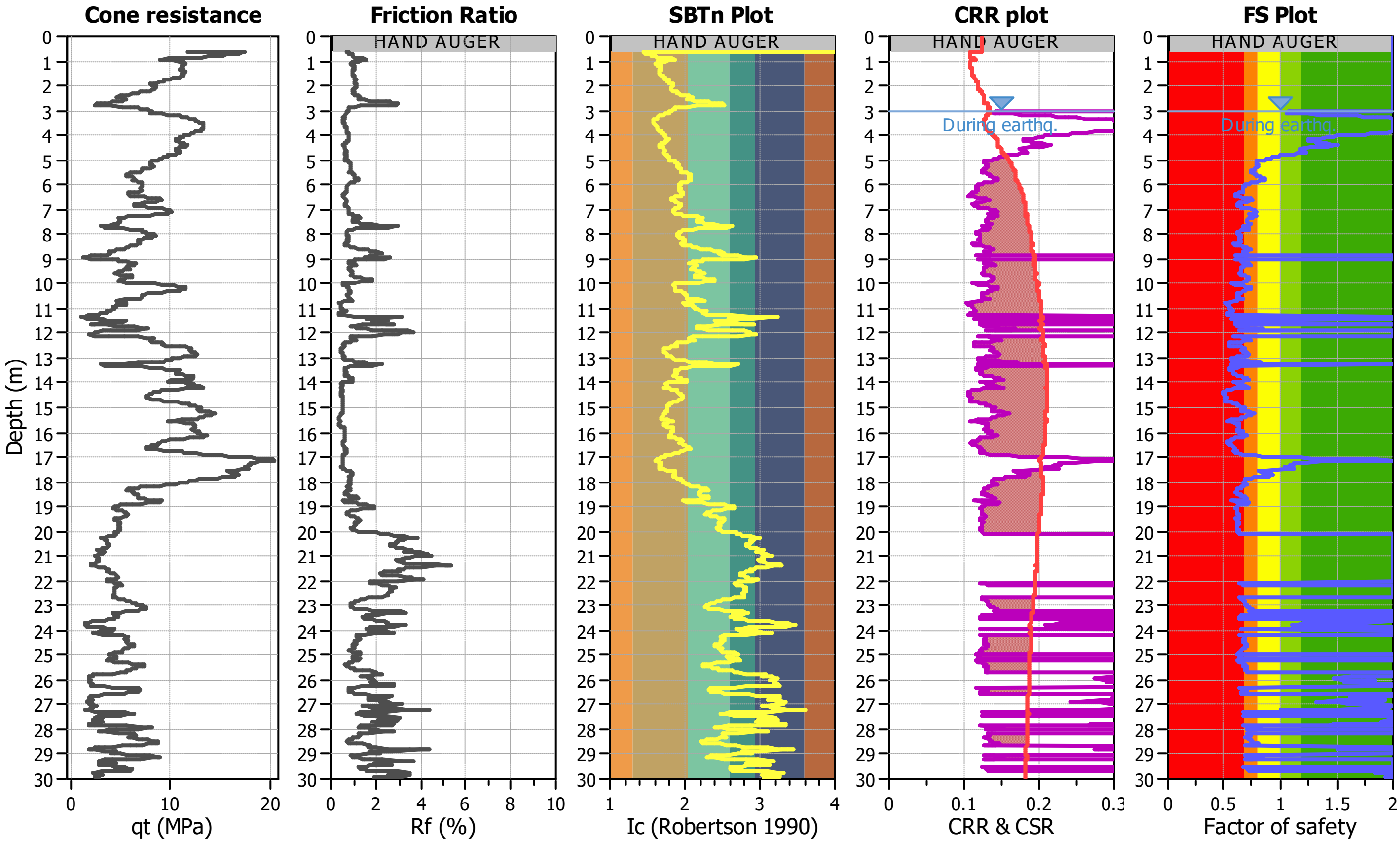
## LIQUEFACTION ANALYSIS REPORT

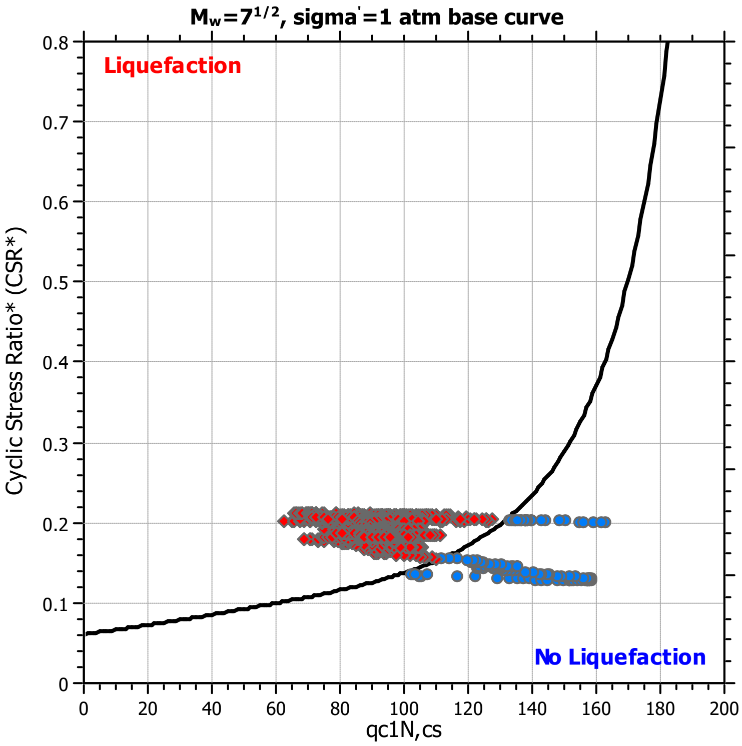
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

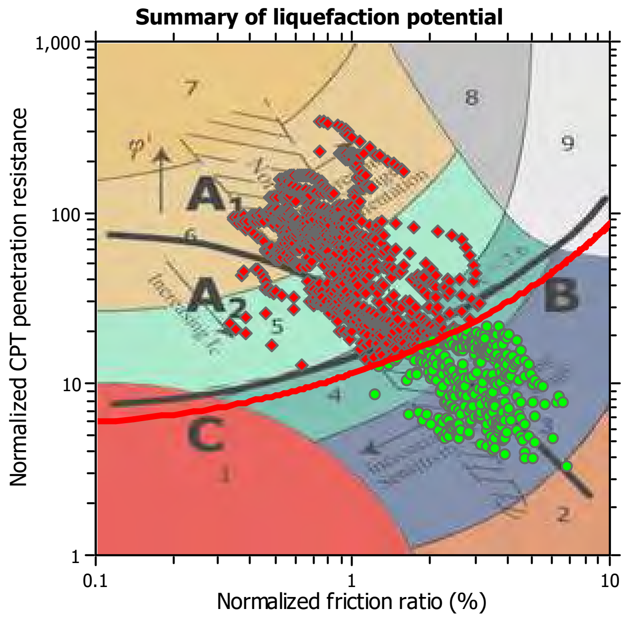
**CPT file : SCPT20-09**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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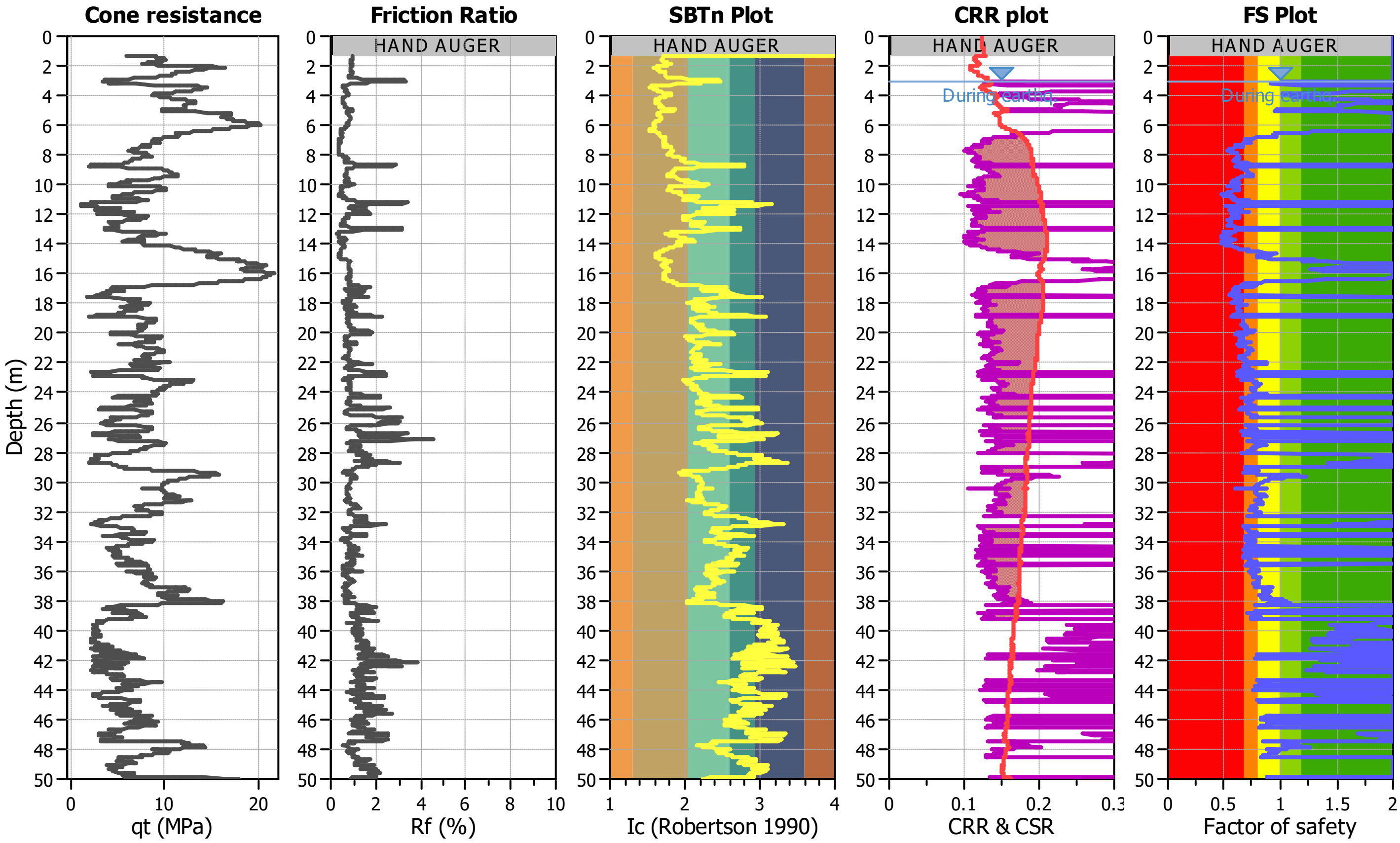
## LIQUEFACTION ANALYSIS REPORT

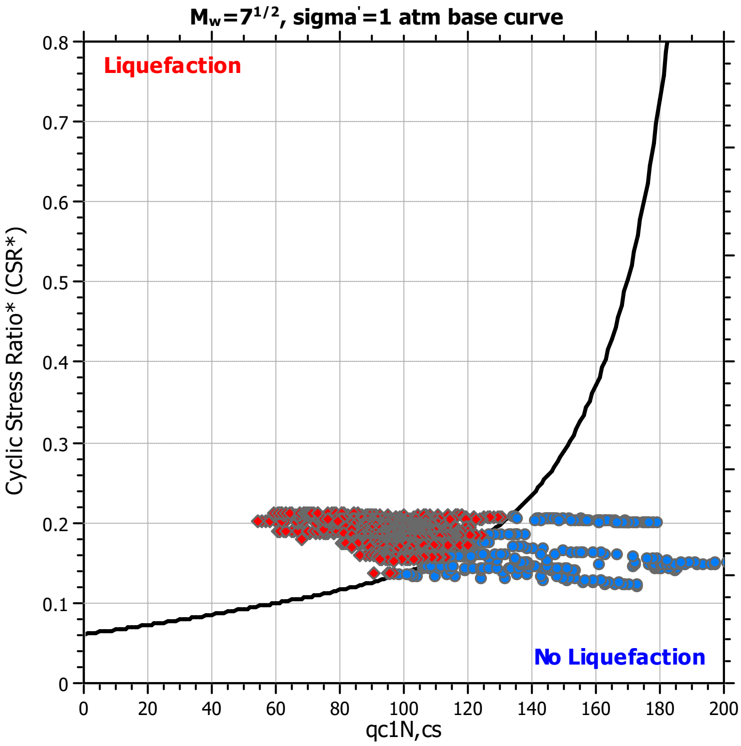
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

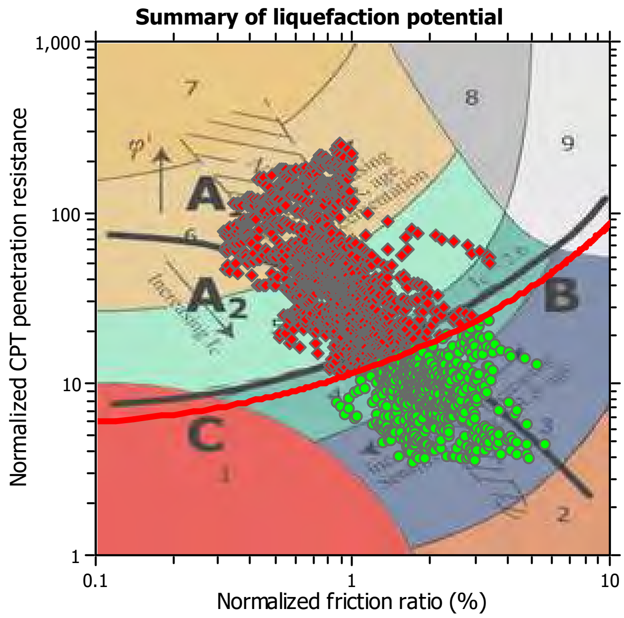
**CPT file : CPT20-10**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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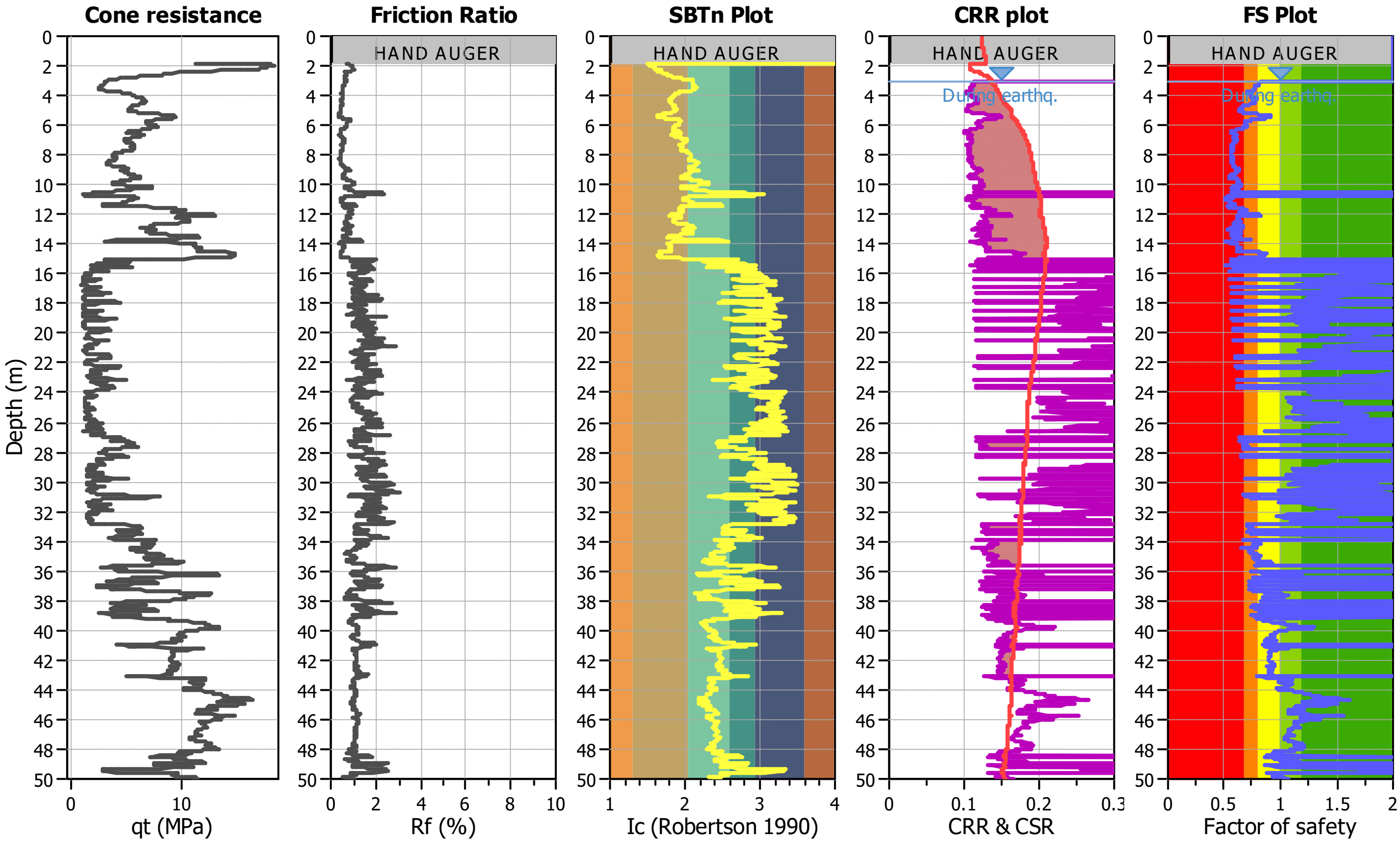
## LIQUEFACTION ANALYSIS REPORT

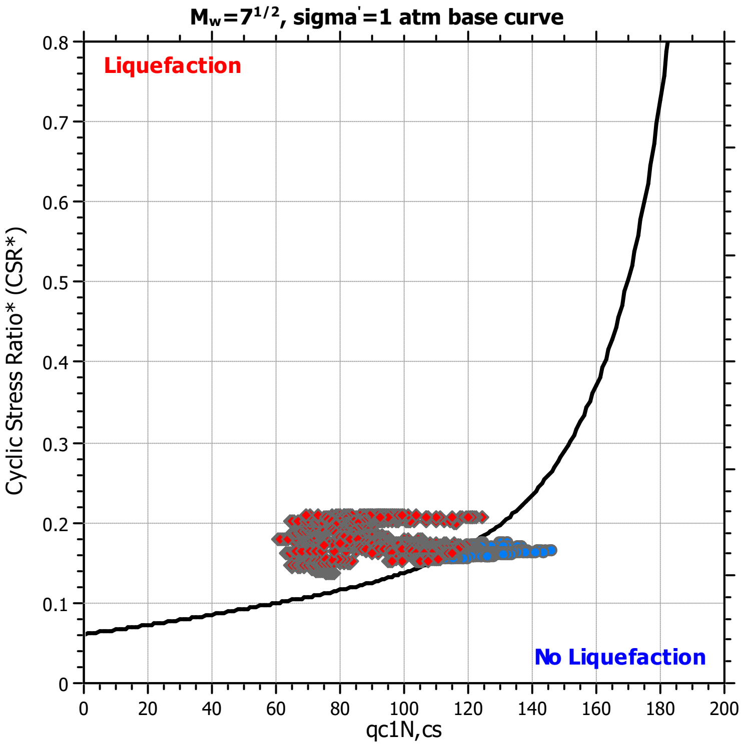
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

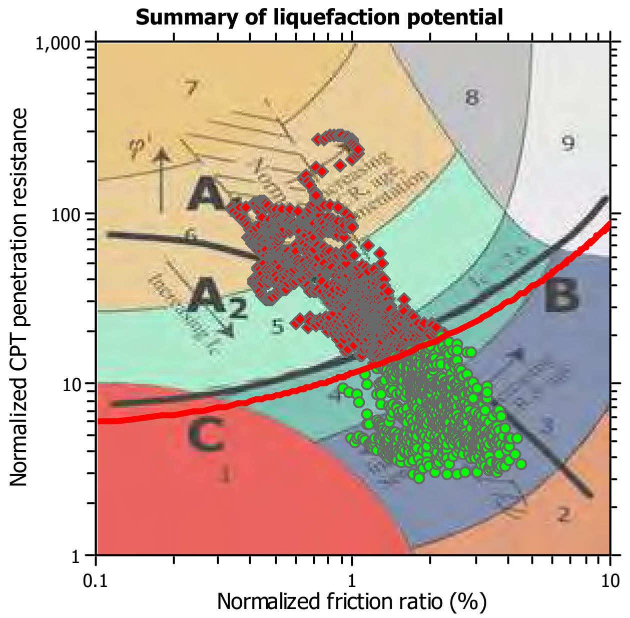
**CPT file : CPT20-11**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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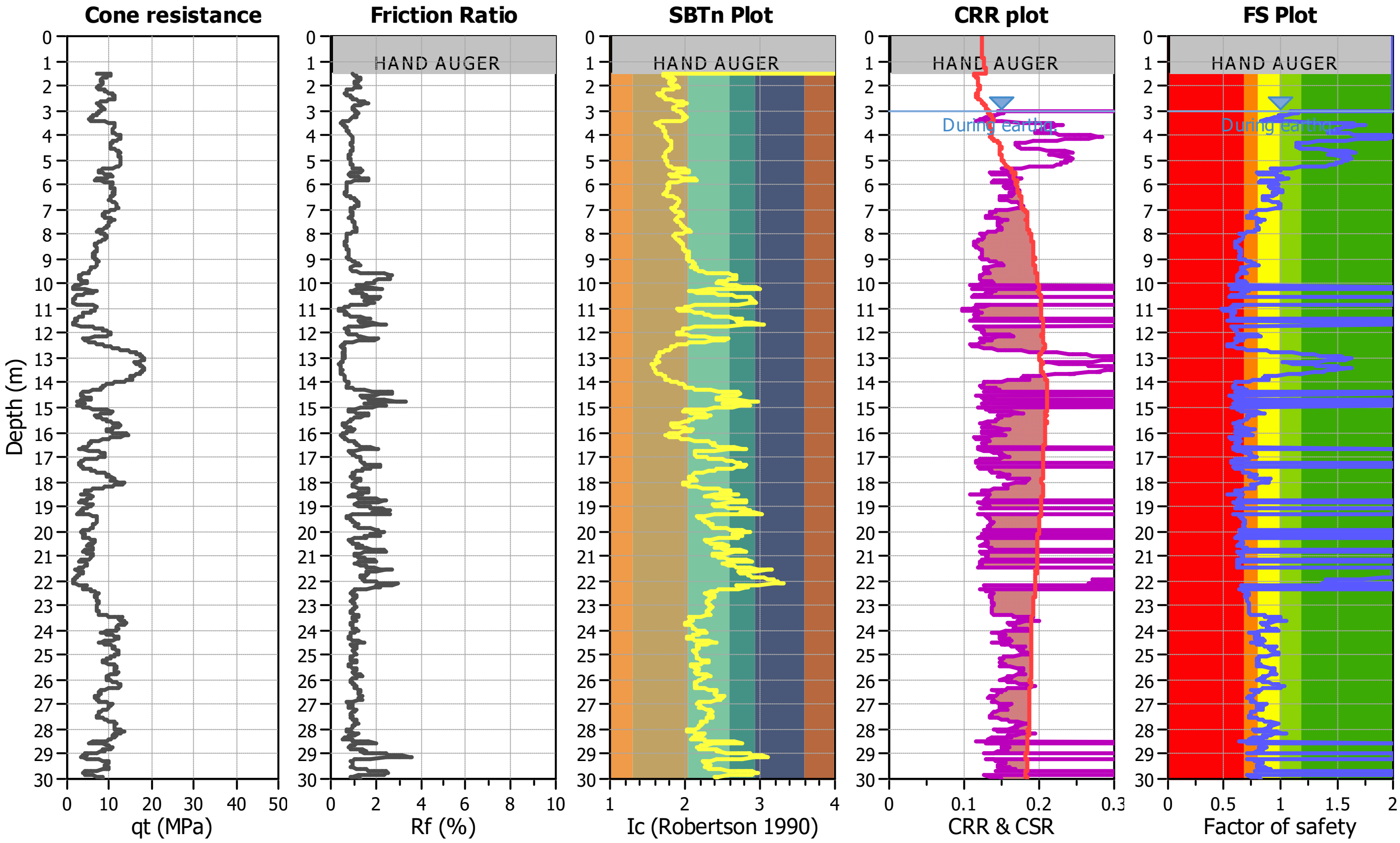
## LIQUEFACTION ANALYSIS REPORT

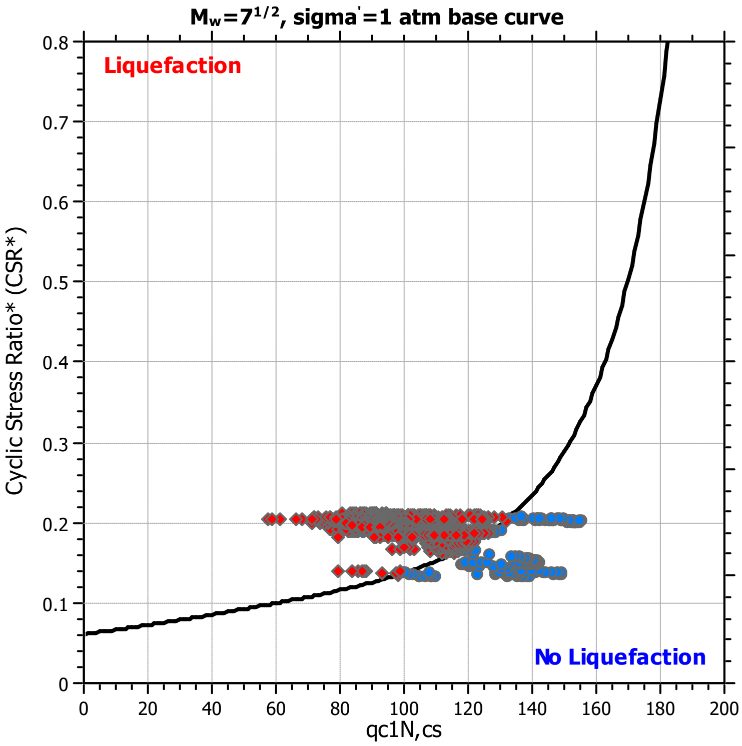
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

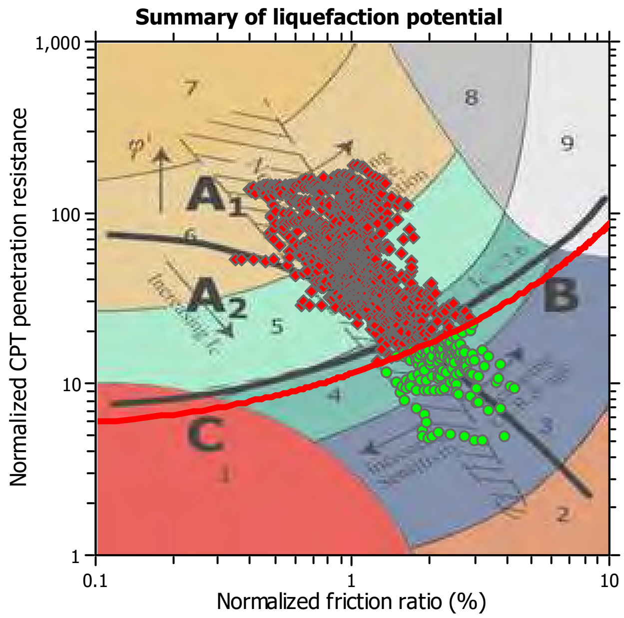
**CPT file : CPT20-12**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | | |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | No |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | Limit depth: | N/A |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | MSF method: | Method |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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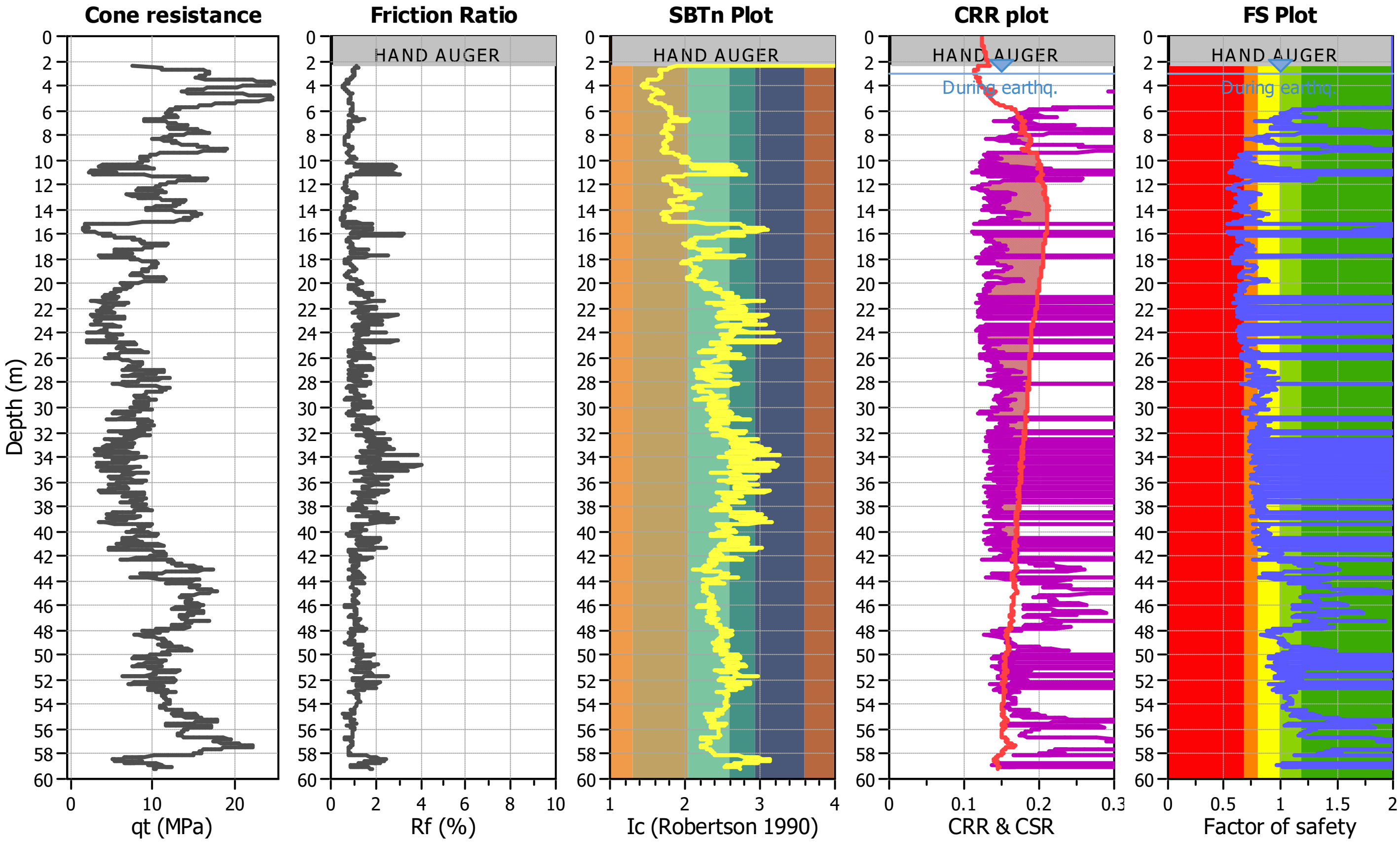
## LIQUEFACTION ANALYSIS REPORT

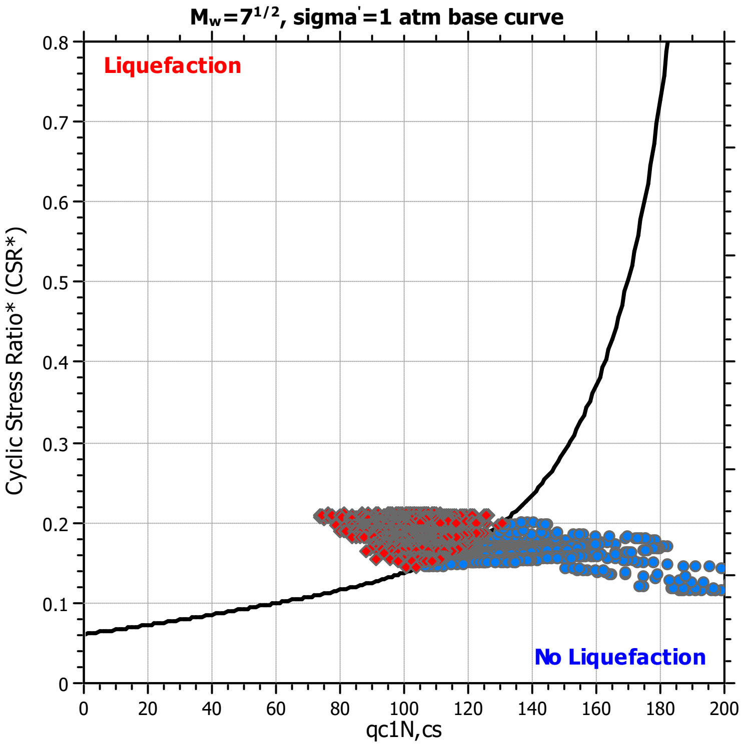
**Project title : Westshore - New Cargo Project** **Location : Delta, BC**

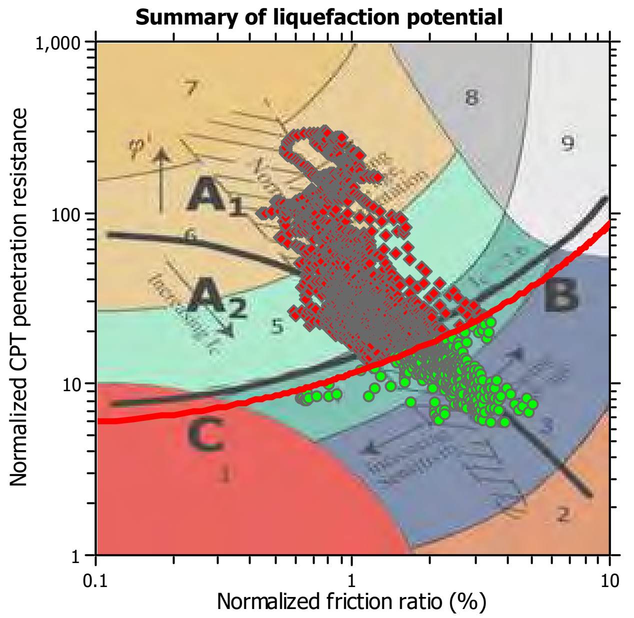
**CPT file : CPT20-13**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 999.00 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

# Liquefaction Triggering Assessment

# A2475 Interface motions

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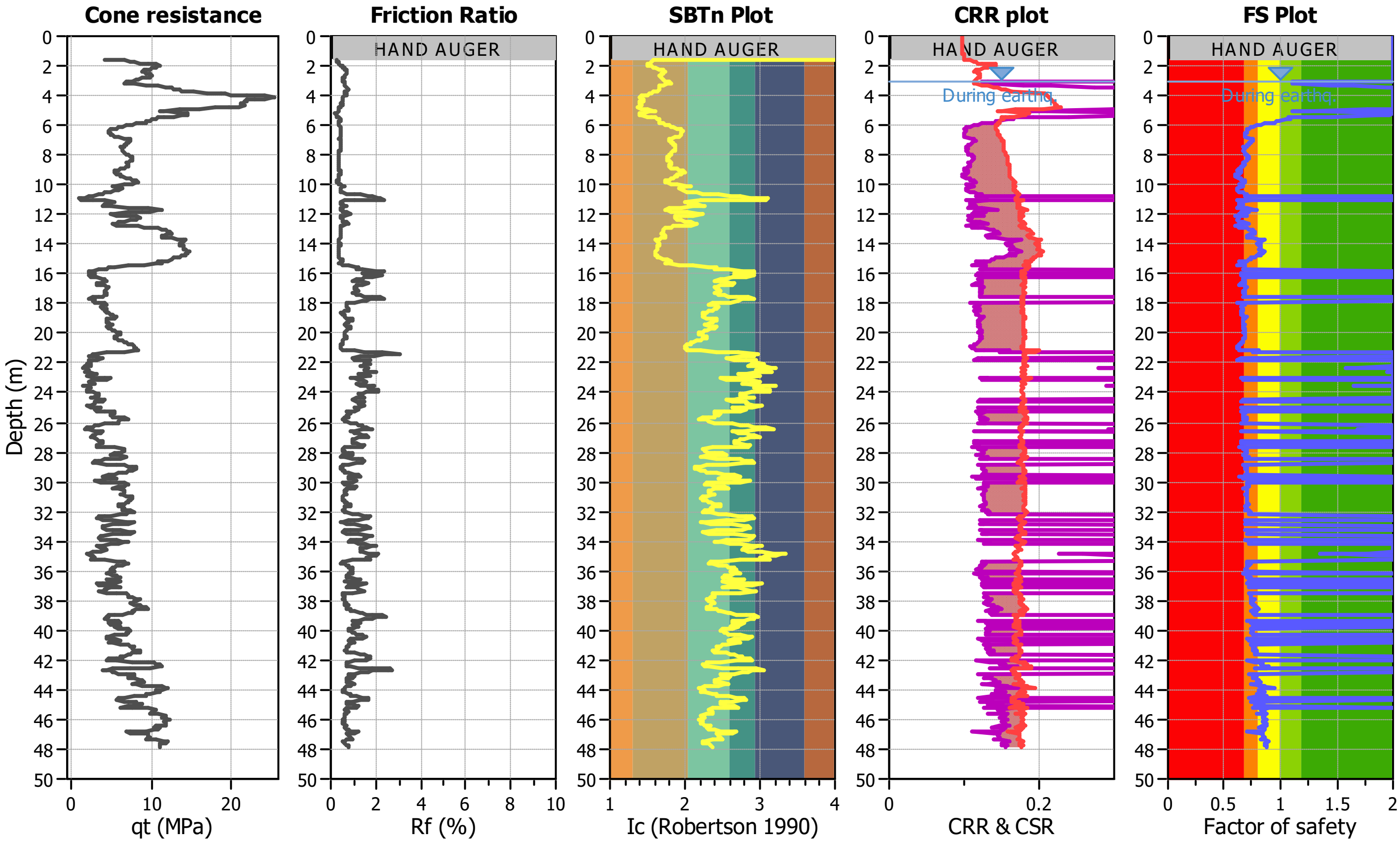
## LIQUEFACTION ANALYSIS REPORT

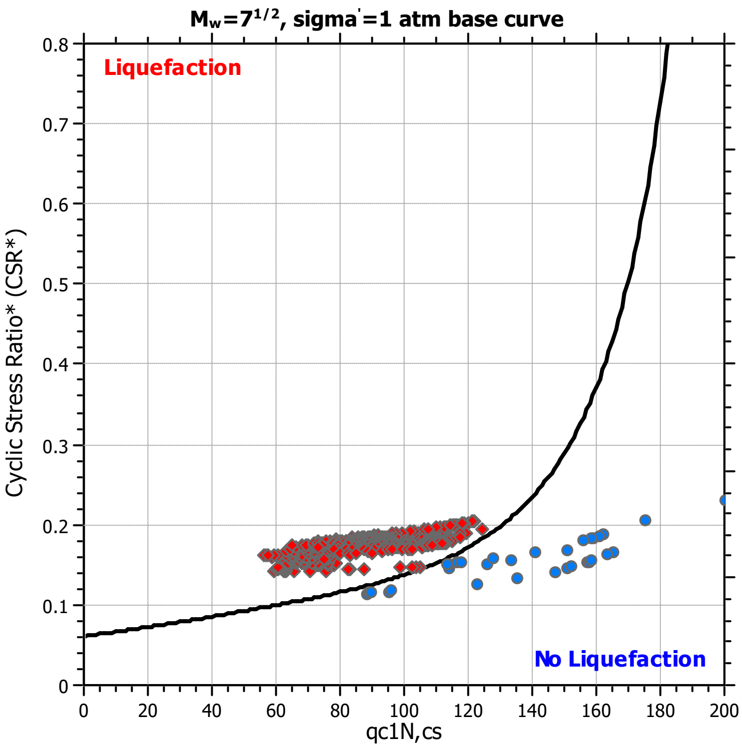
**Project title :** **Location :**

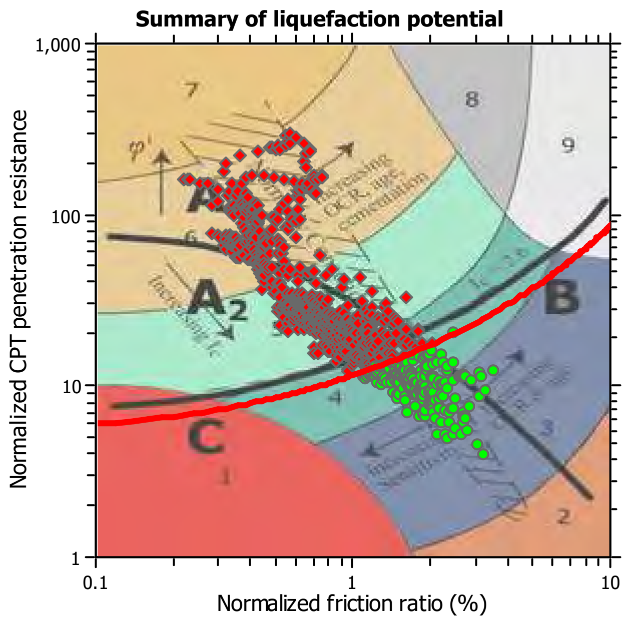
**CPT file : CPT18-01 Berth 2**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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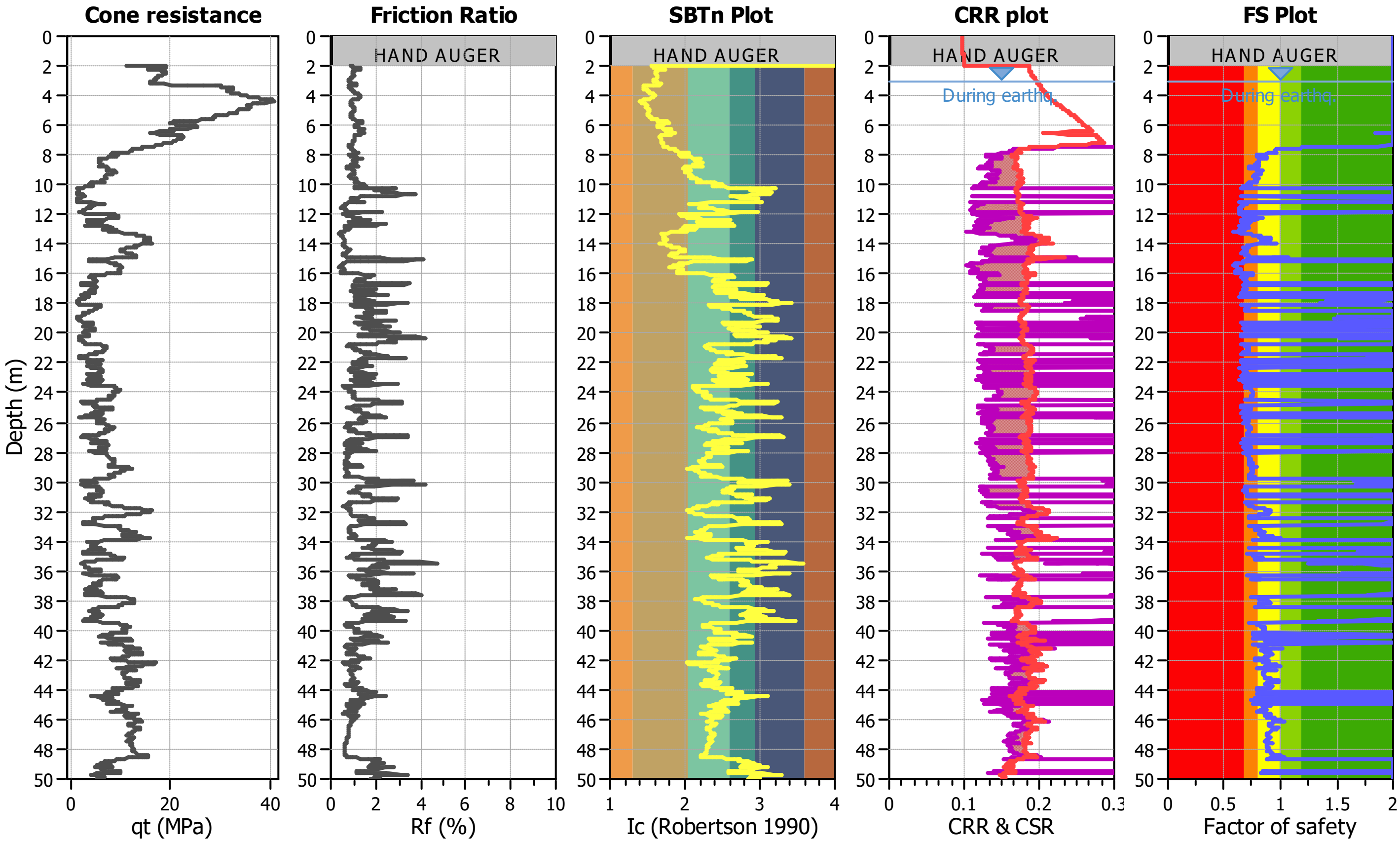
# LIQUEFACTION ANALYSIS REPORT

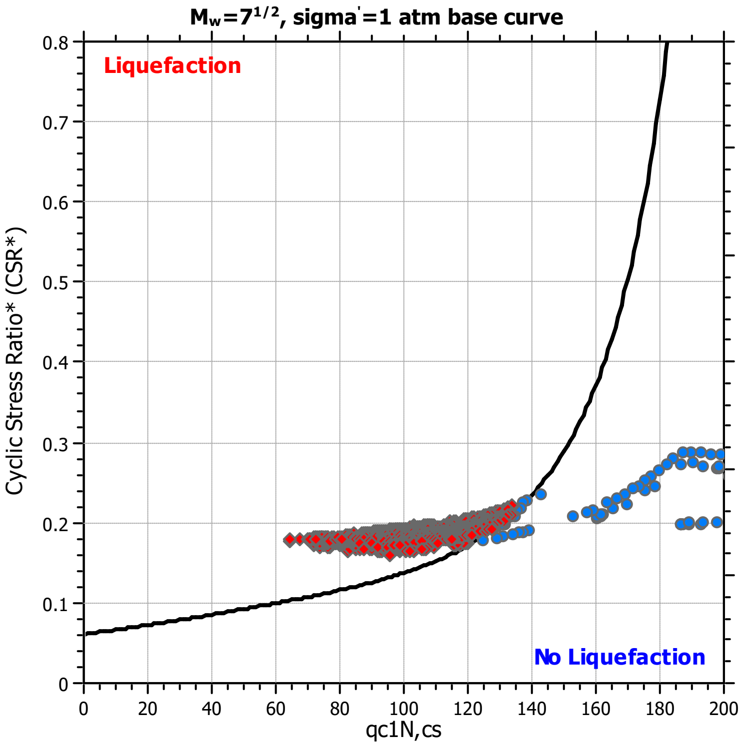
**Project title :** **Location :**

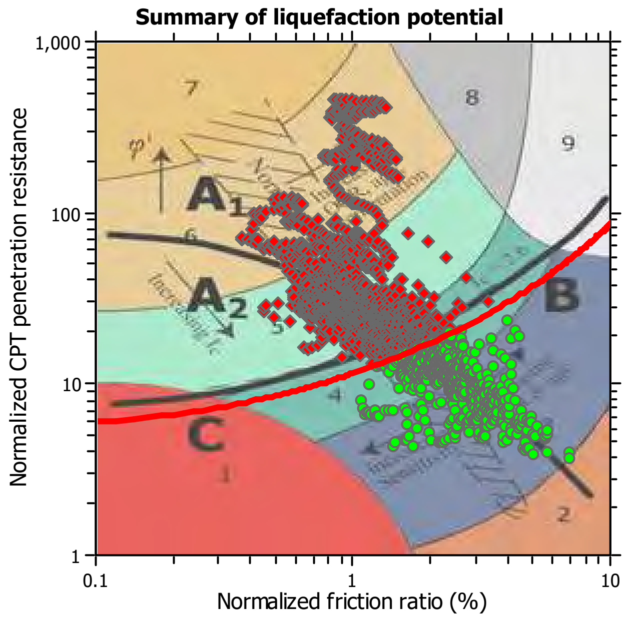
**CPT file : SCPT20-01**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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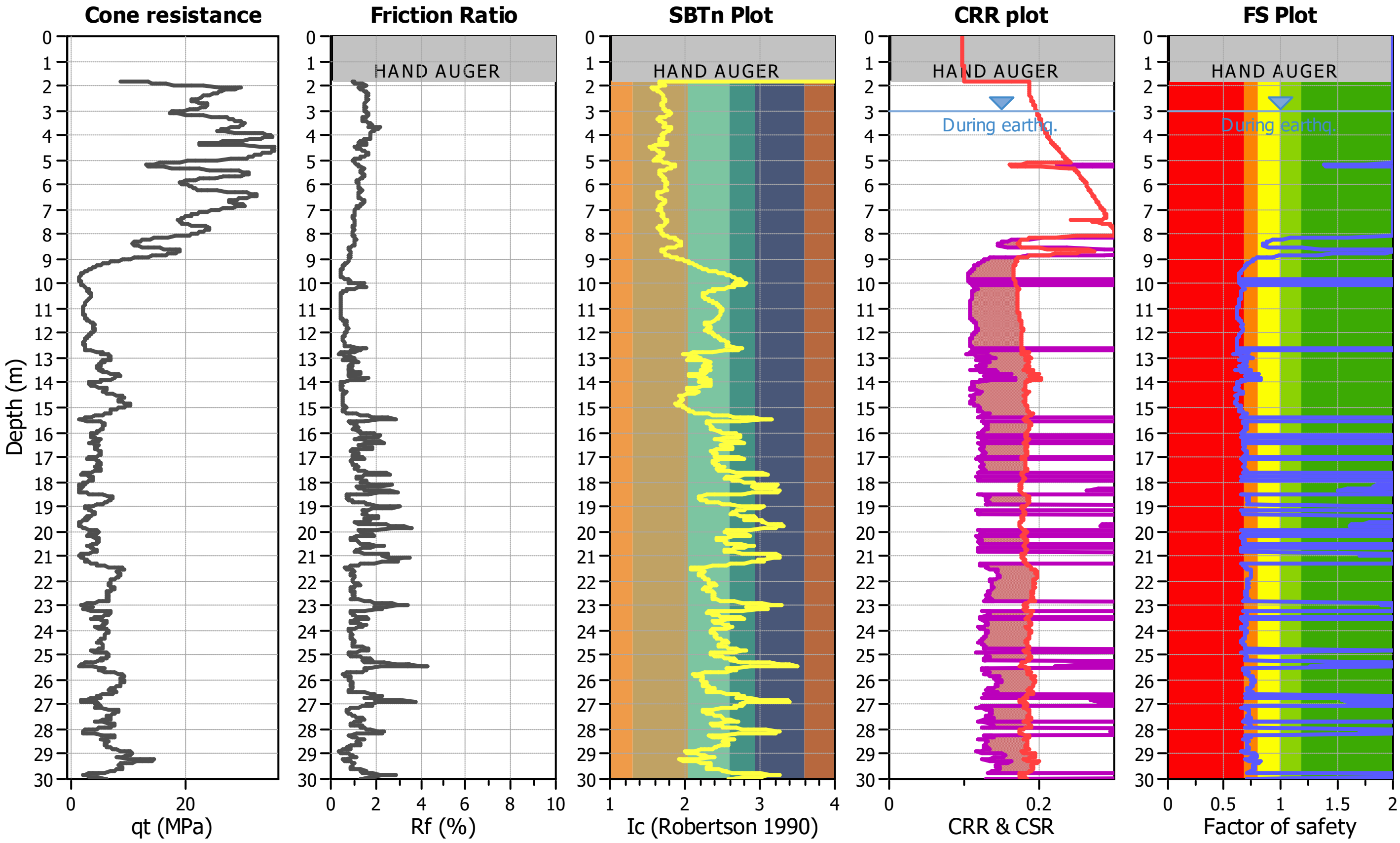
## LIQUEFACTION ANALYSIS REPORT

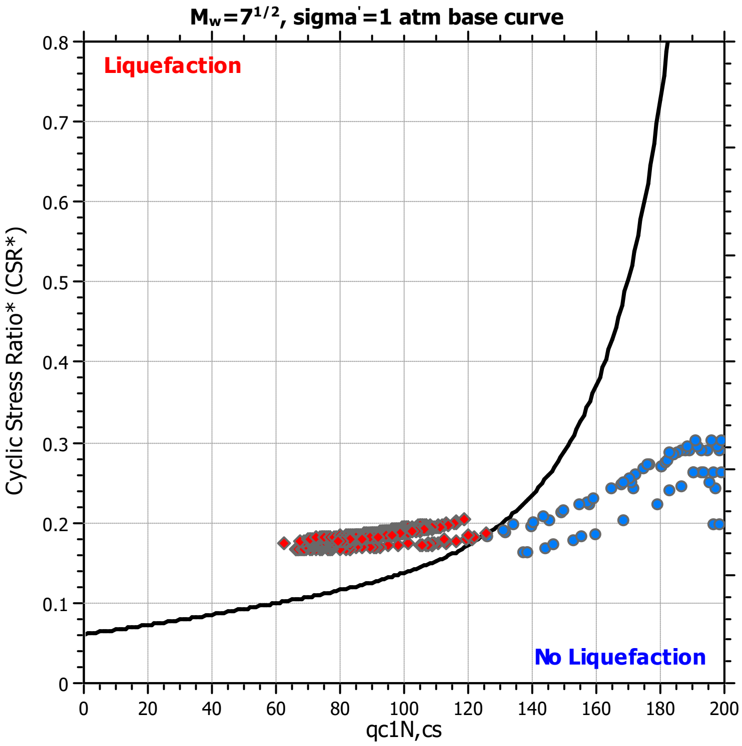
**Project title :** **Location :**

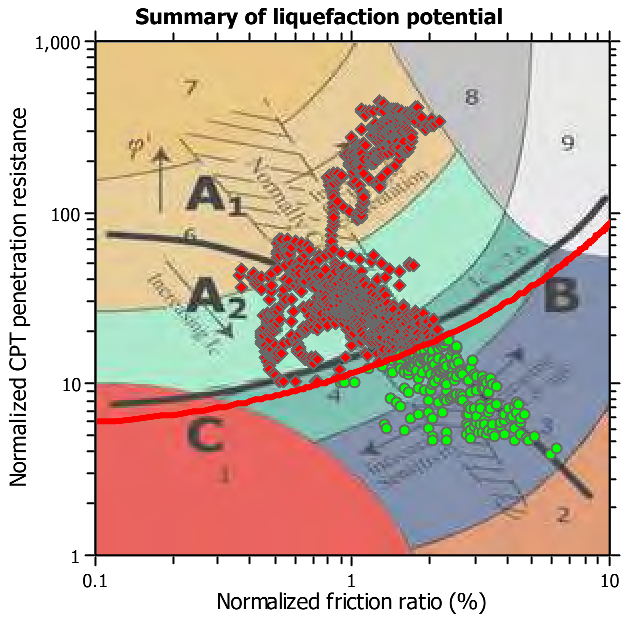
**CPT file : CPT20-02**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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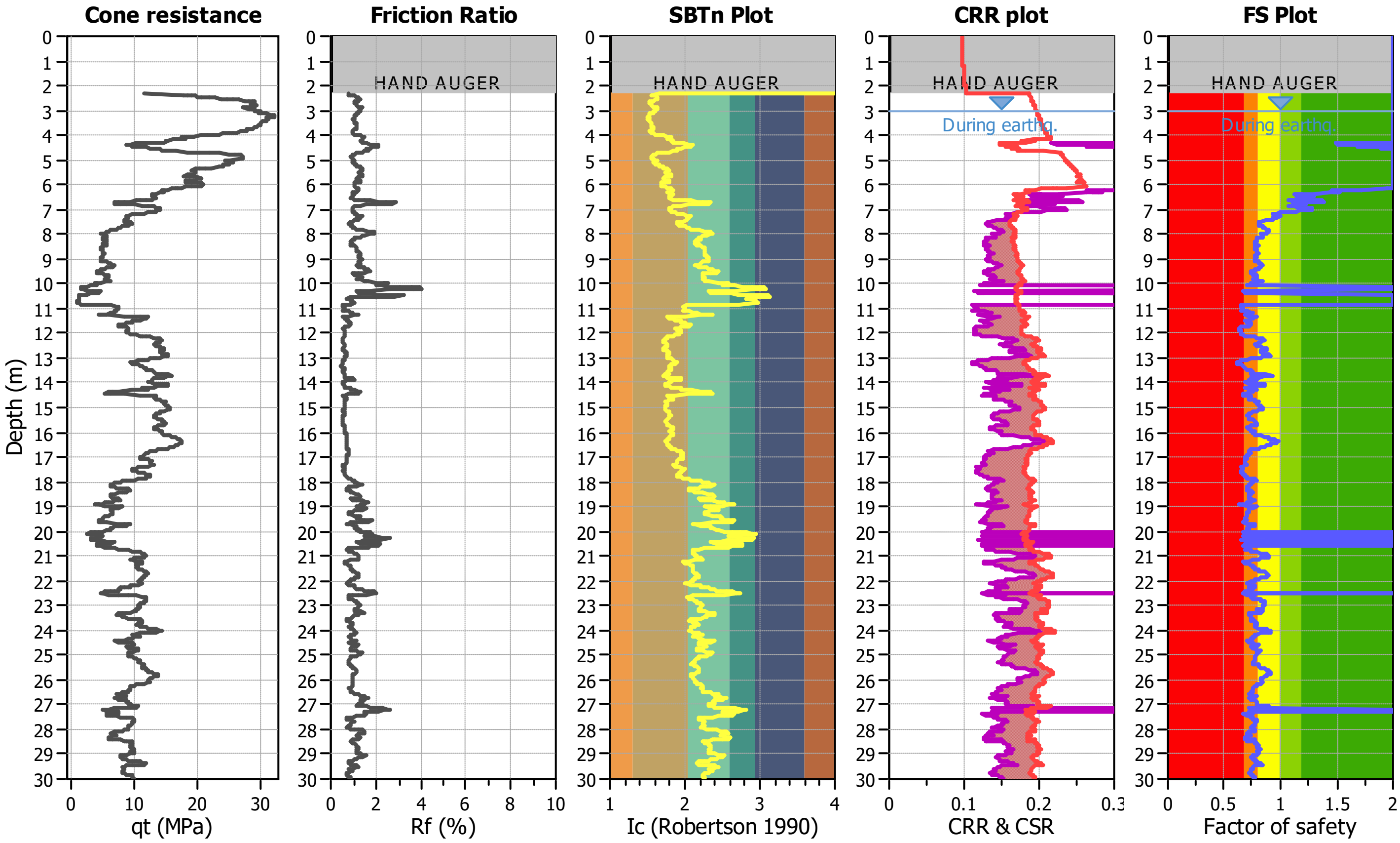
# LIQUEFACTION ANALYSIS REPORT

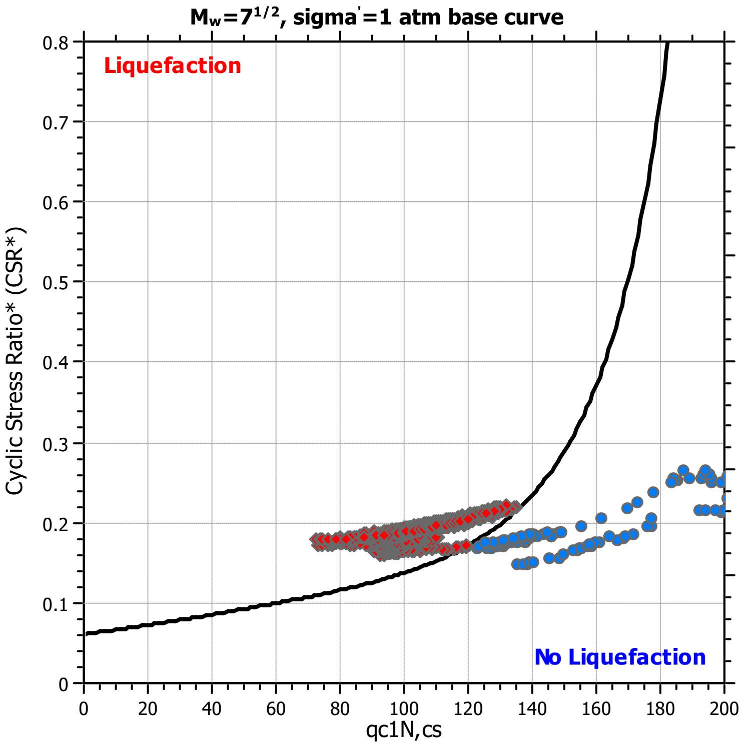
**Project title :** **Location :**

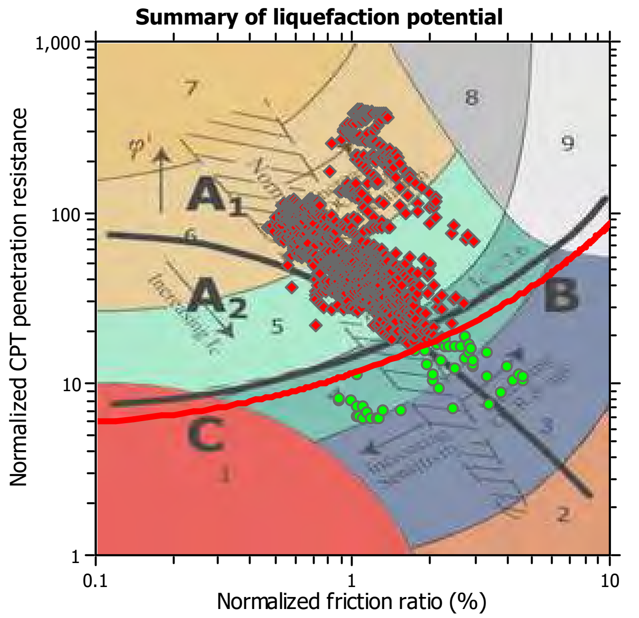
**CPT file : CPT20-03**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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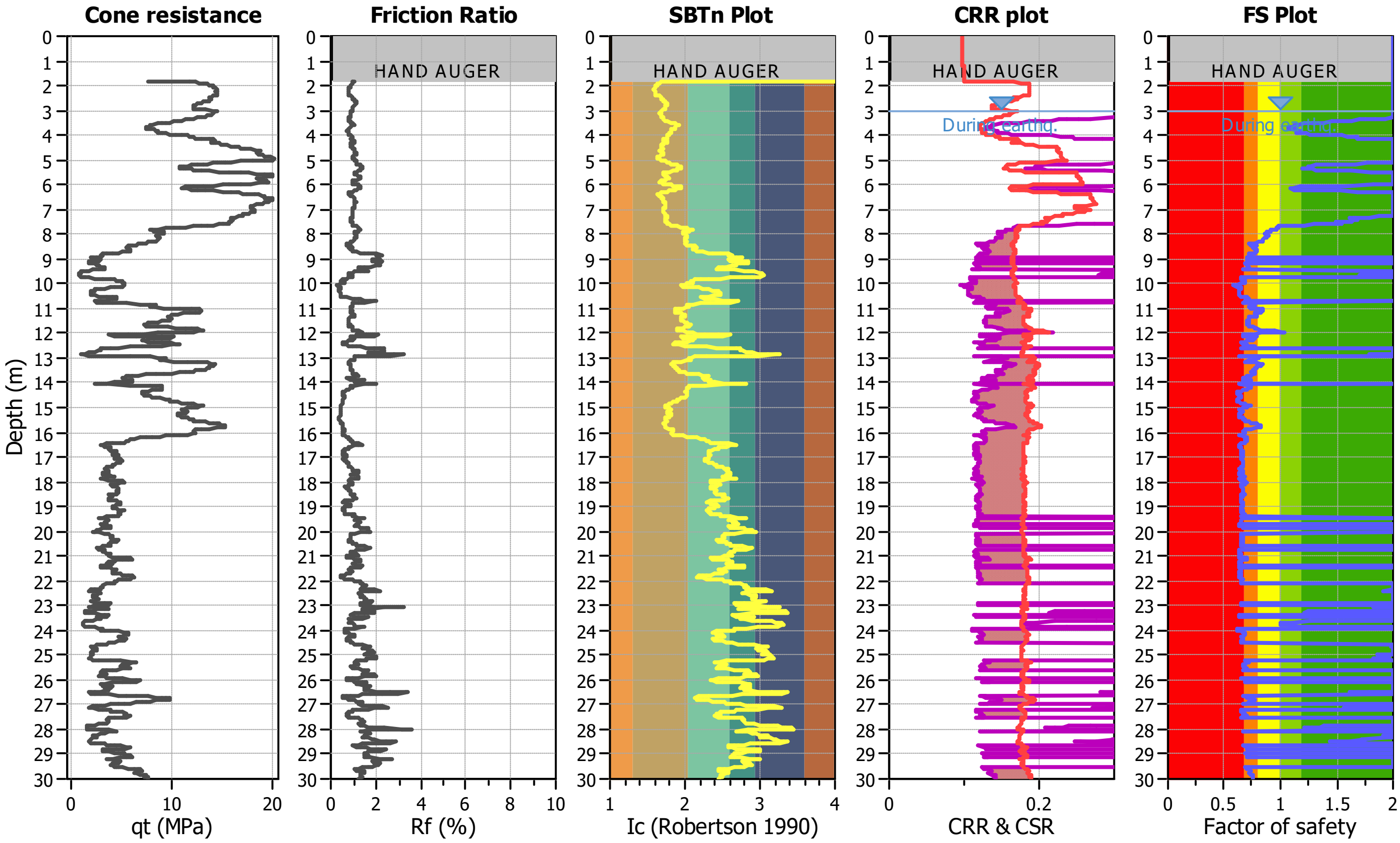
## LIQUEFACTION ANALYSIS REPORT

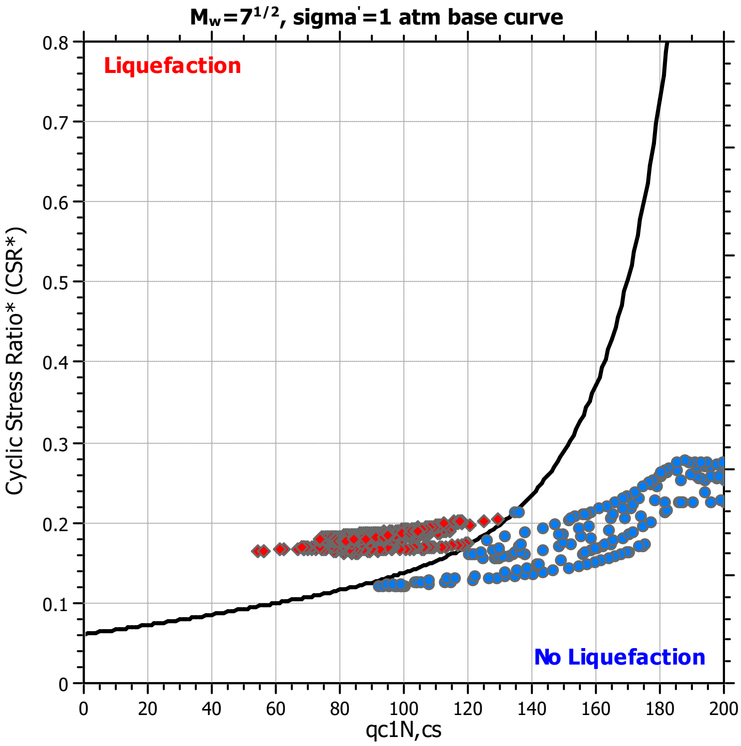
**Project title :** **Location :**

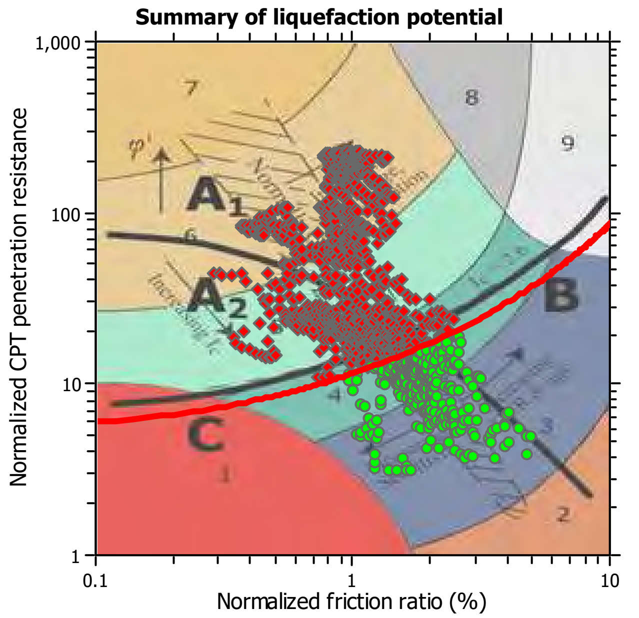
**CPT file : CP20-04**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | | |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | No |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | Limit depth: | N/A |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | MSF method: | Method |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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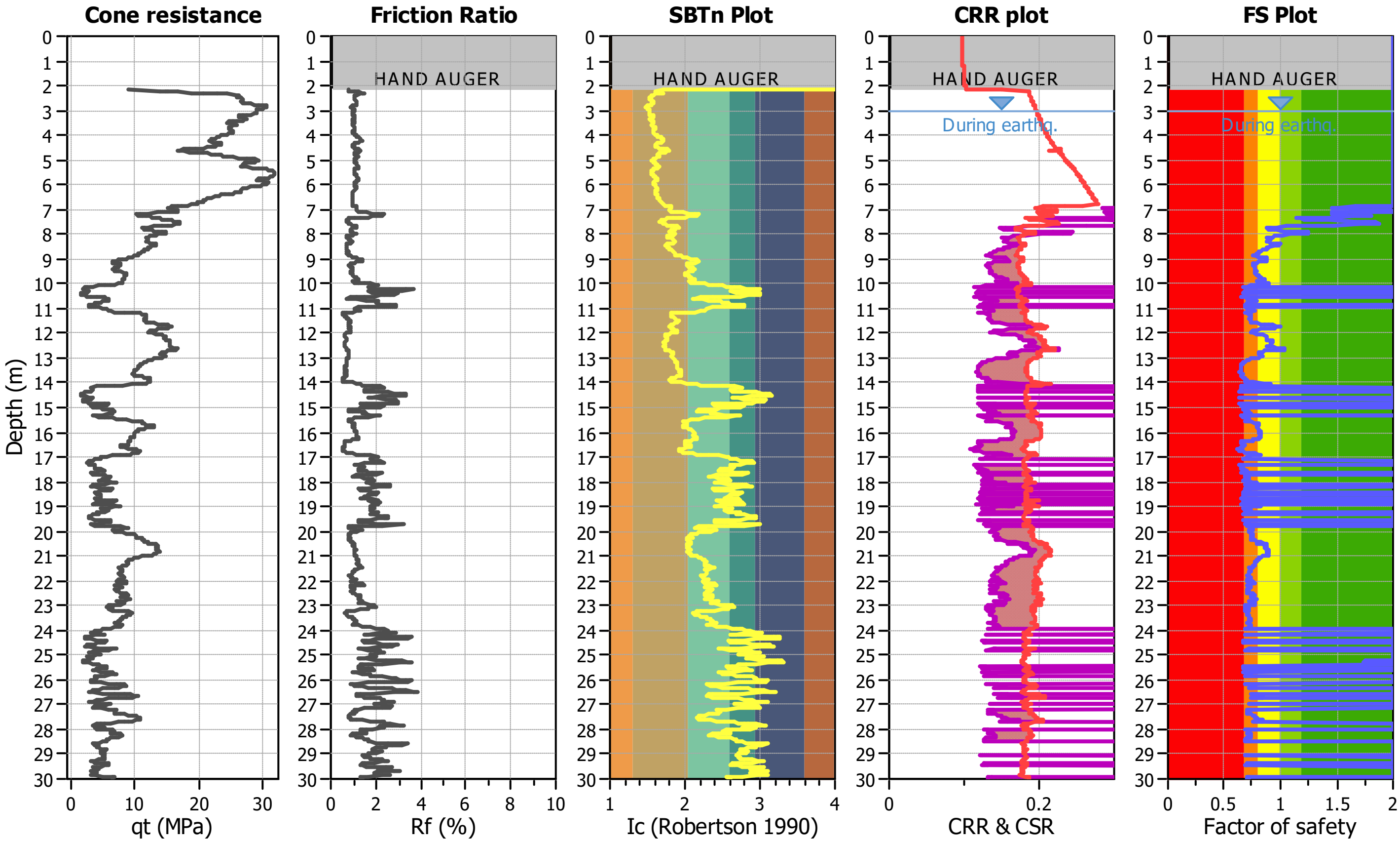
## LIQUEFACTION ANALYSIS REPORT

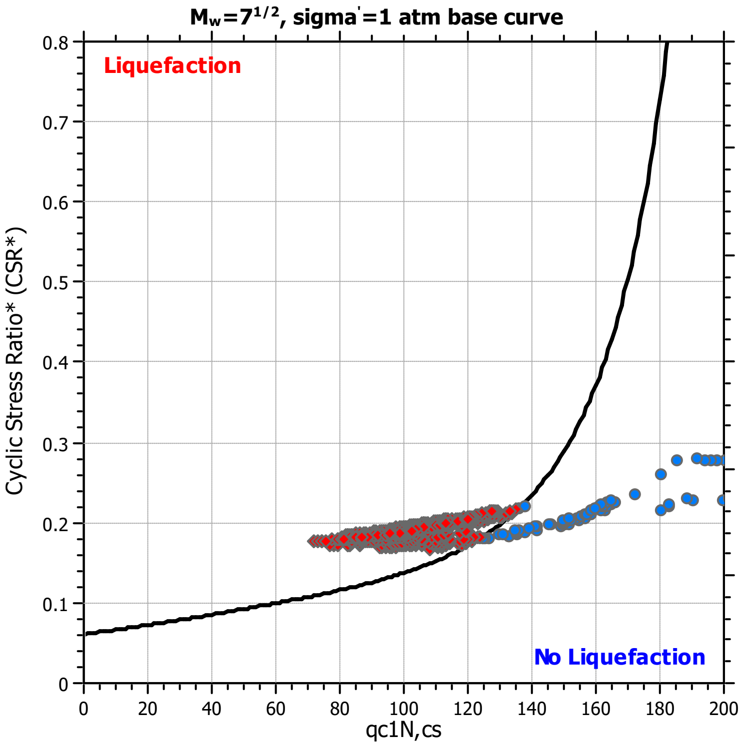
**Project title :** **Location :**

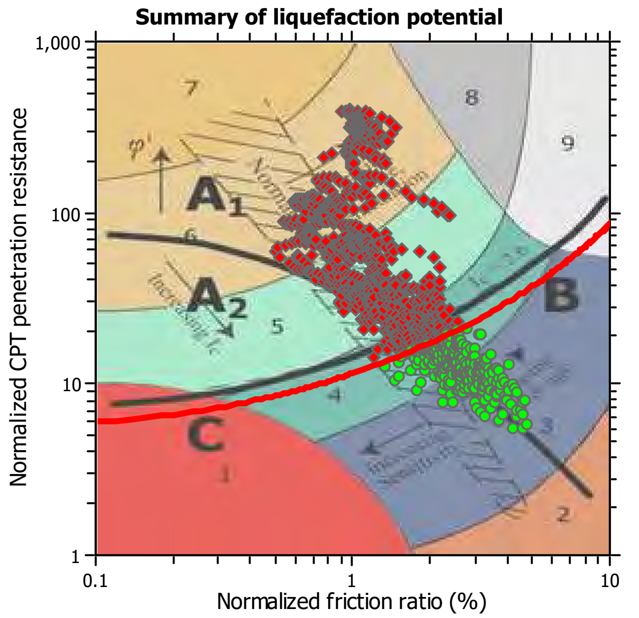
**CPT file : CPT20-05**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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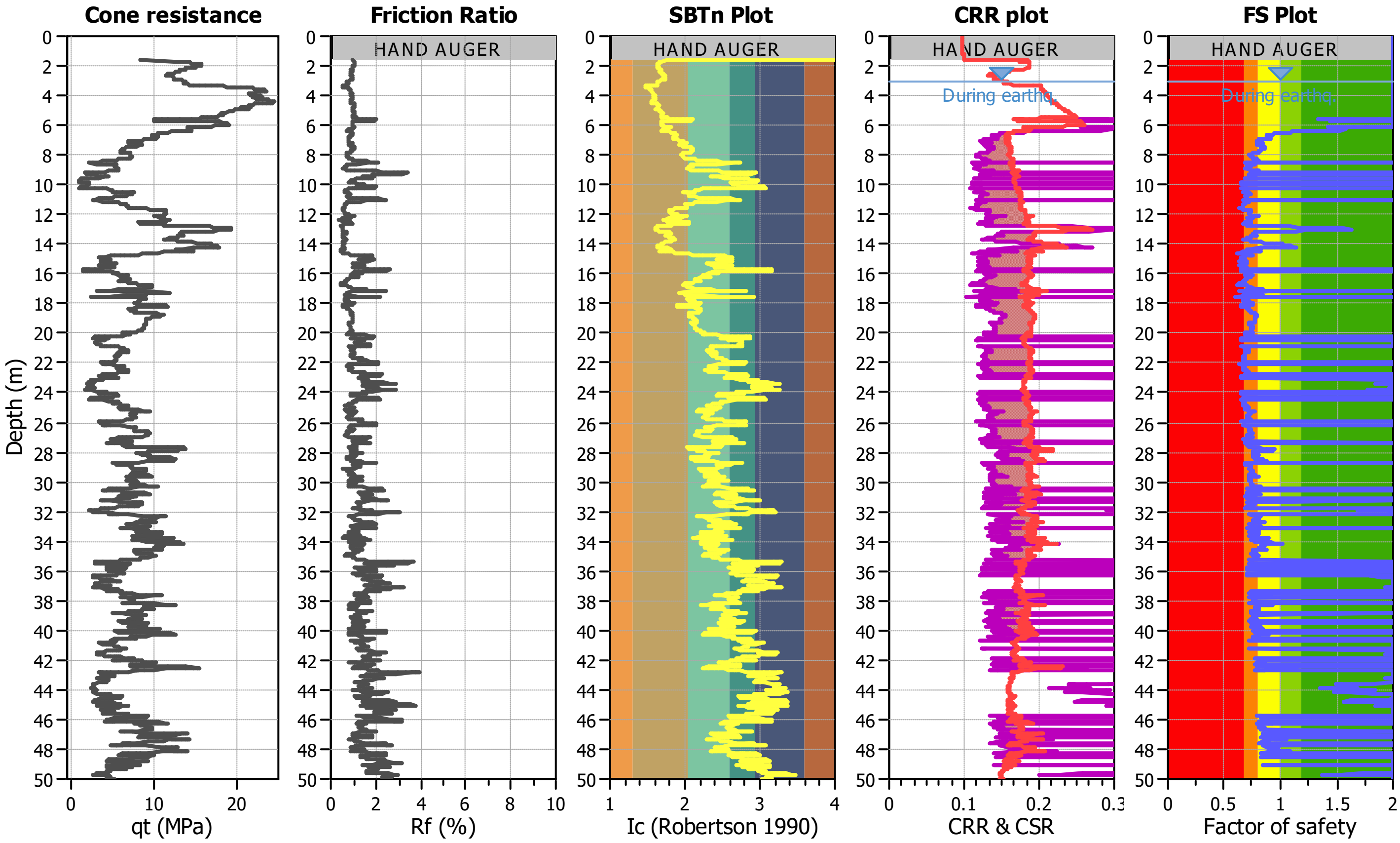
## LIQUEFACTION ANALYSIS REPORT

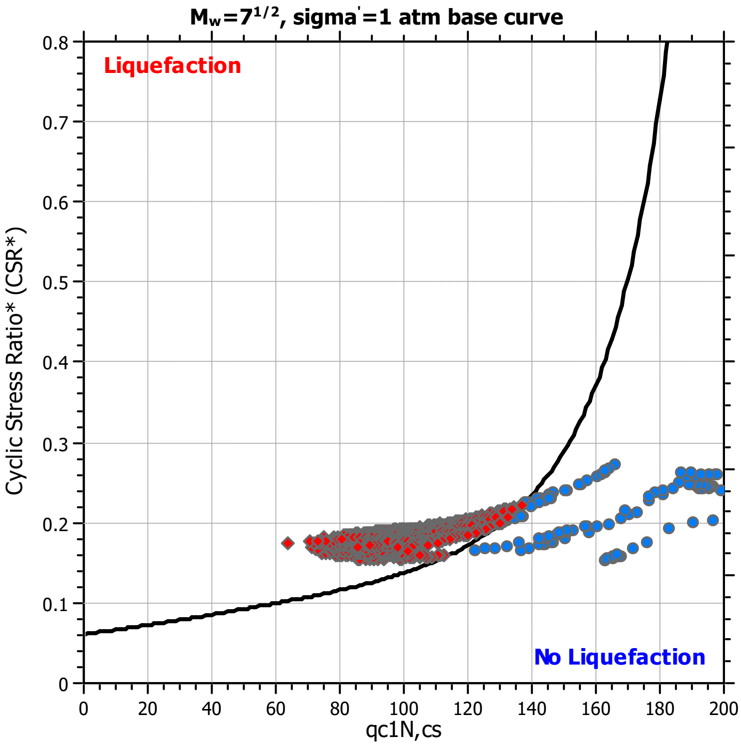
**Project title :** **Location :**

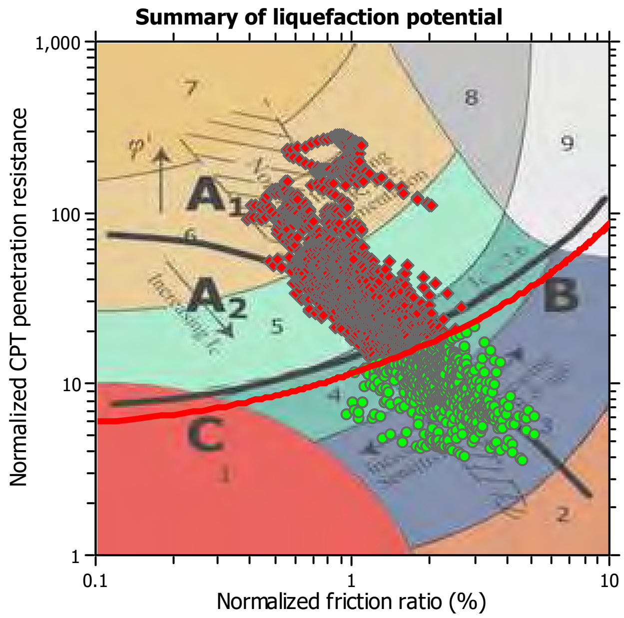
**CPT file : SCPT20-06**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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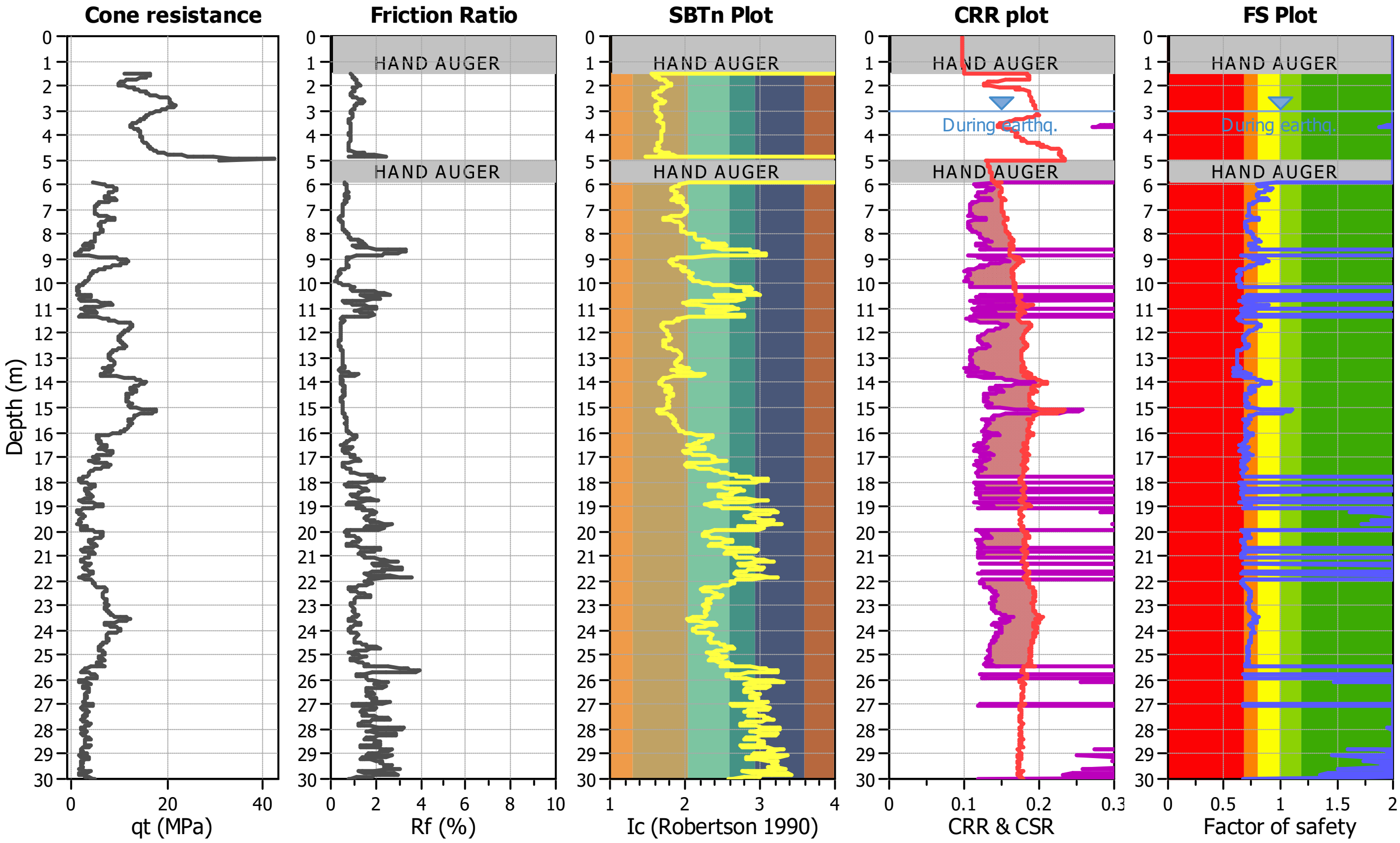
## LIQUEFACTION ANALYSIS REPORT

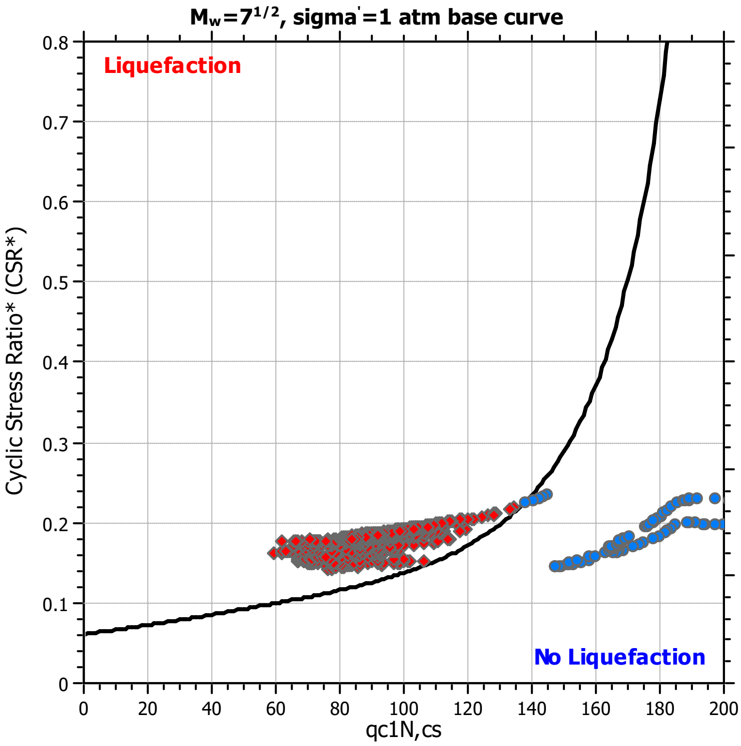
**Project title :** **Location :**

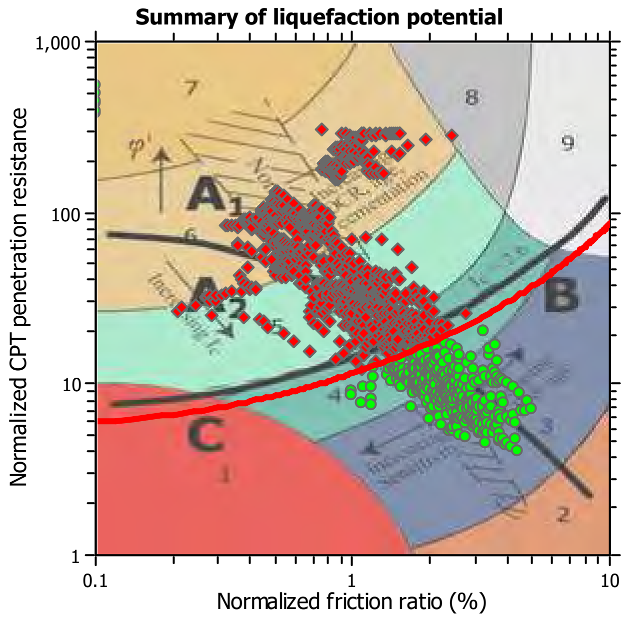
**CPT file : CPT20-08**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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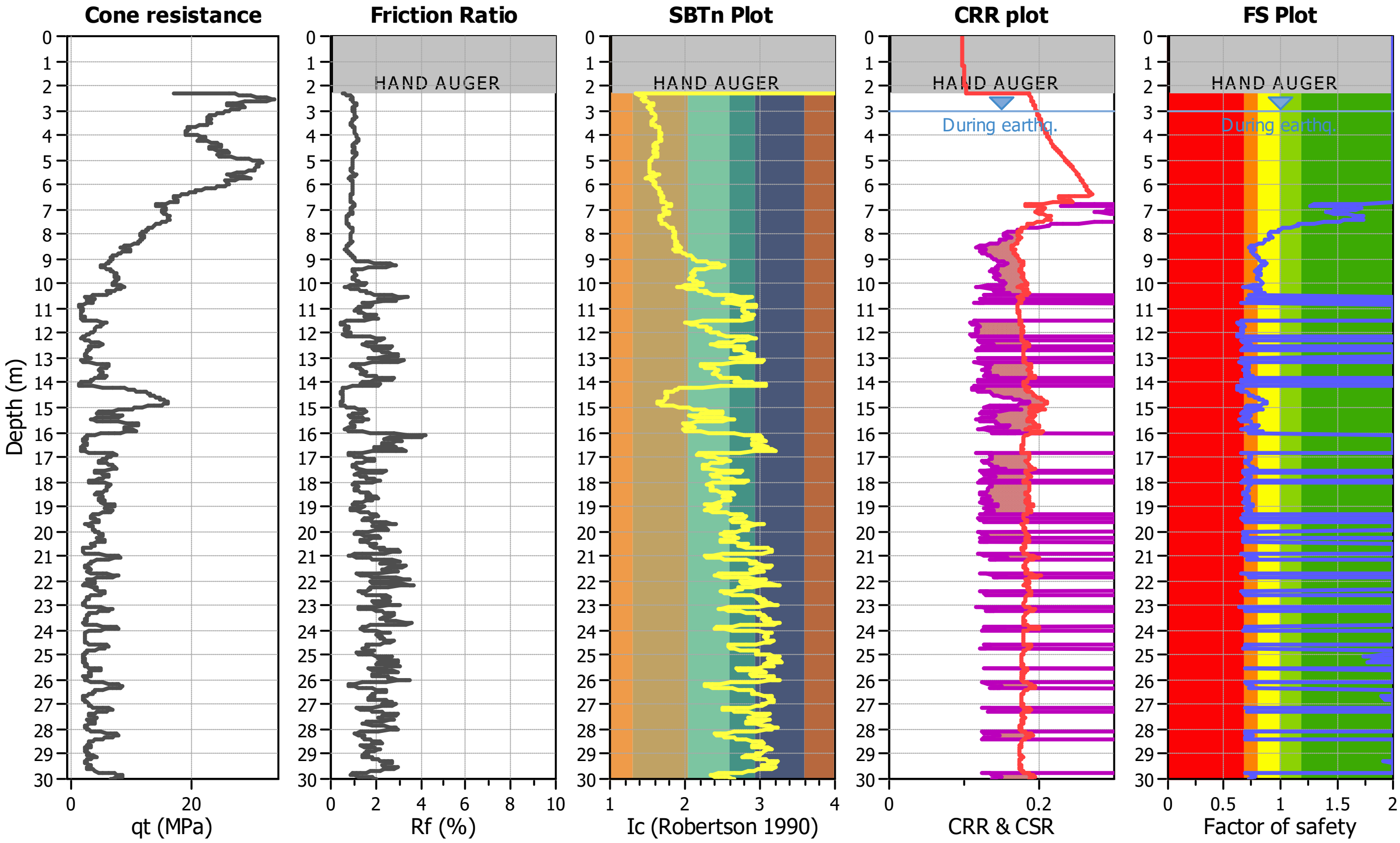
## LIQUEFACTION ANALYSIS REPORT

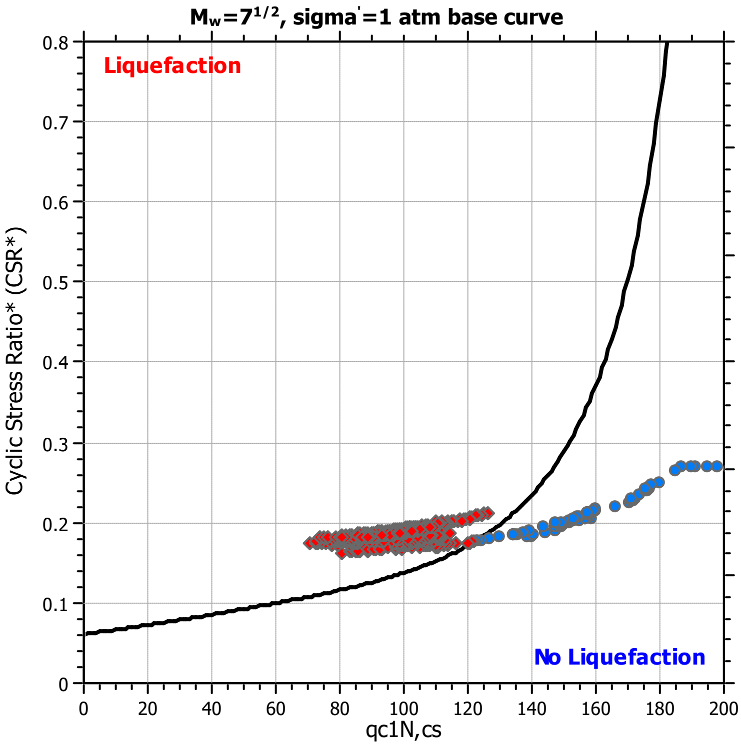
**Project title :** **Location :**

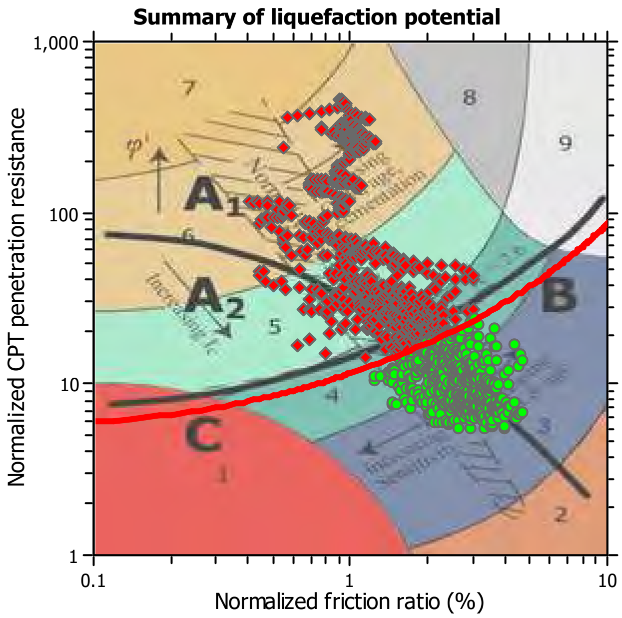
**CPT file : CPT20-07**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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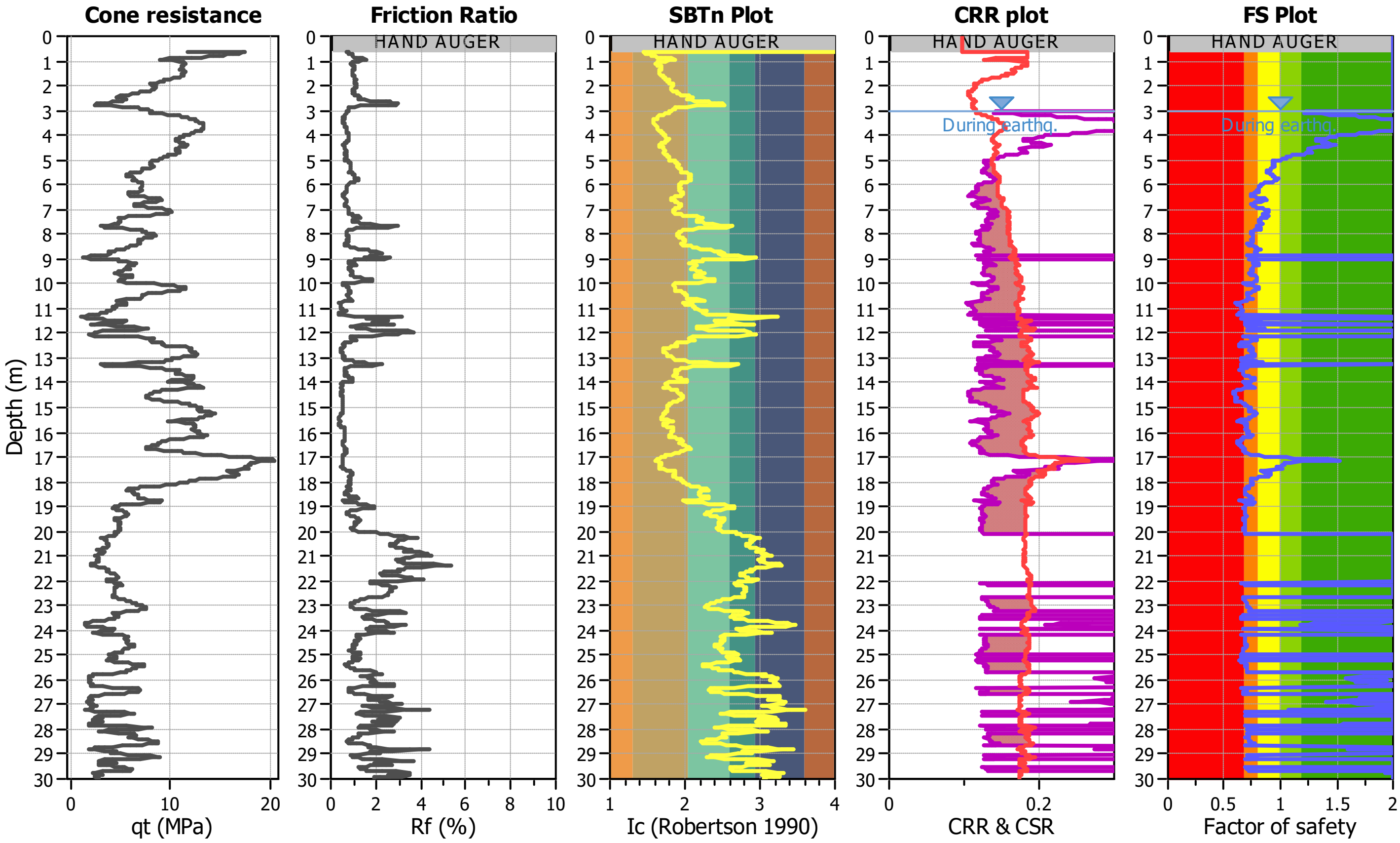
# LIQUEFACTION ANALYSIS REPORT

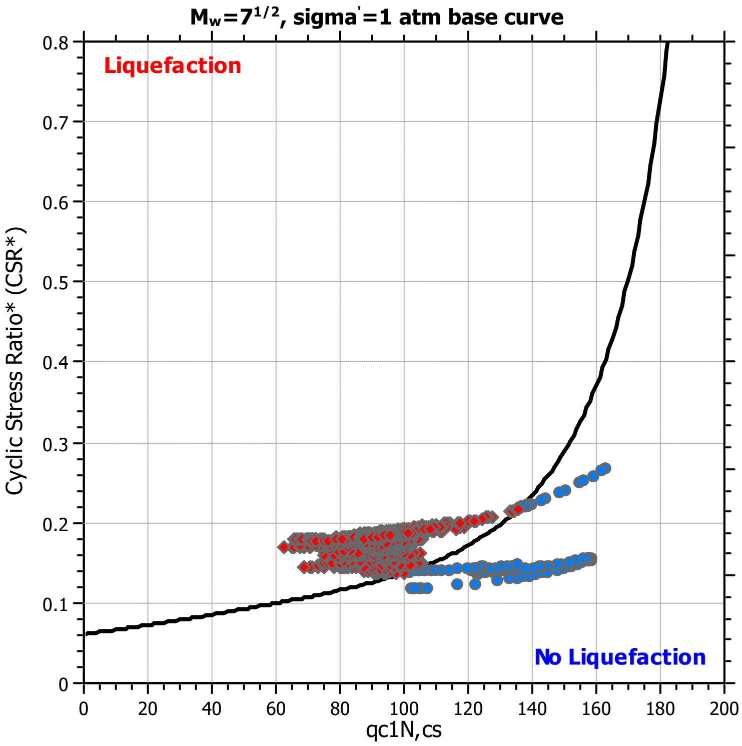
**Project title :** **Location :**

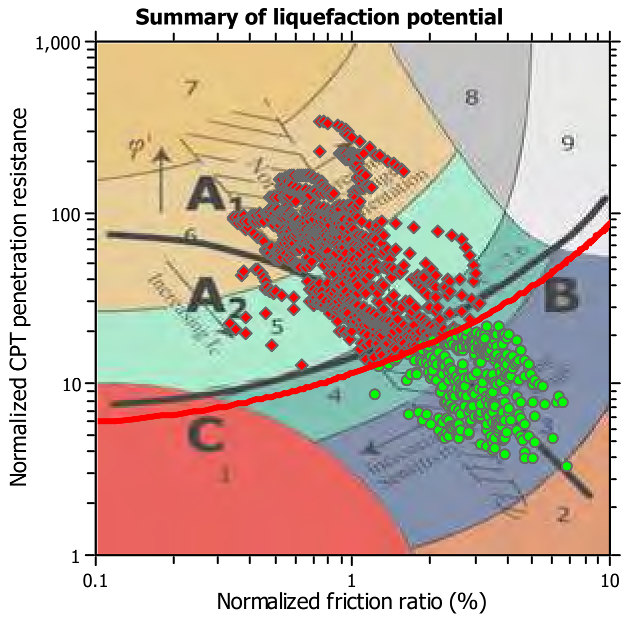
**CPT file : SCPT20-09**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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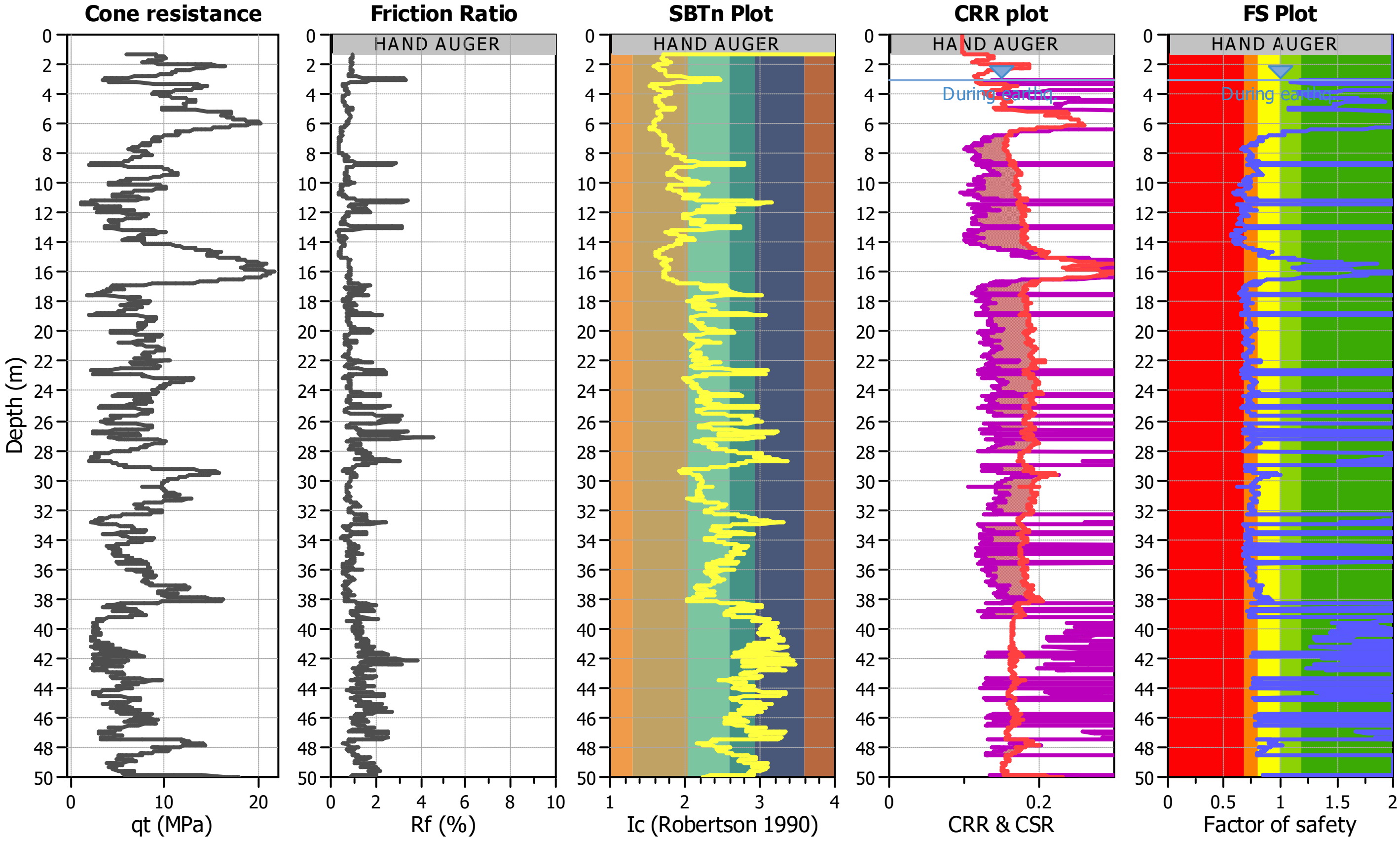
## LIQUEFACTION ANALYSIS REPORT

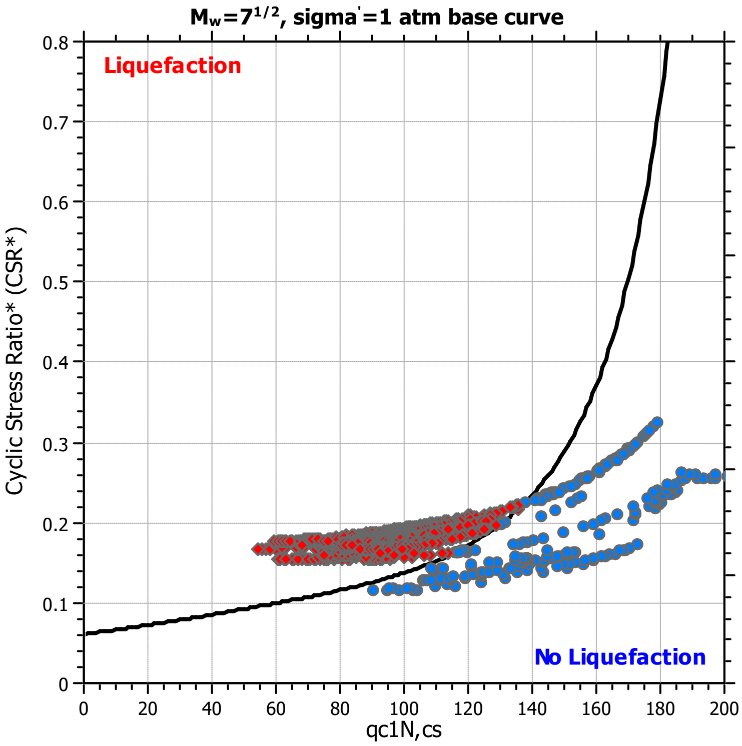
**Project title :** **Location :**

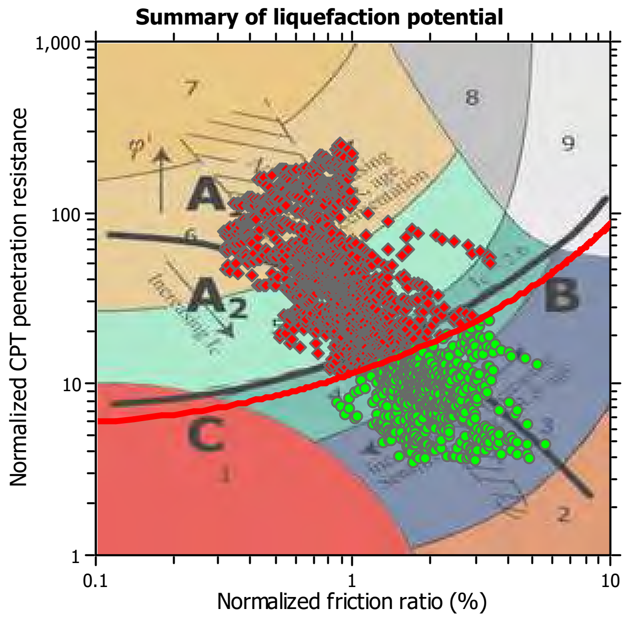
**CPT file : CPT20-10**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

**Braun Geotechnical Ltd.**

## LIQUEFACTION ANALYSIS REPORT

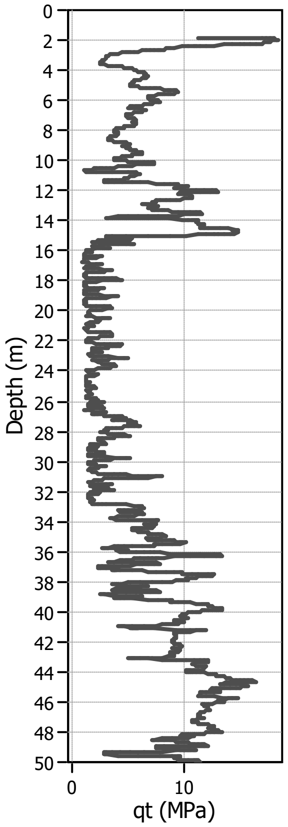
**Project title :** **Location :**

**CPT file : CPT20-11**

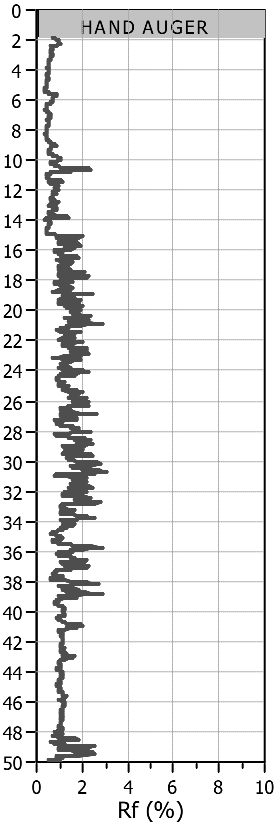
### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

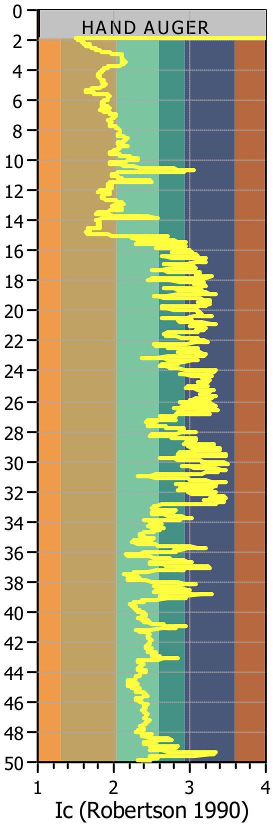
**Cone resistance**



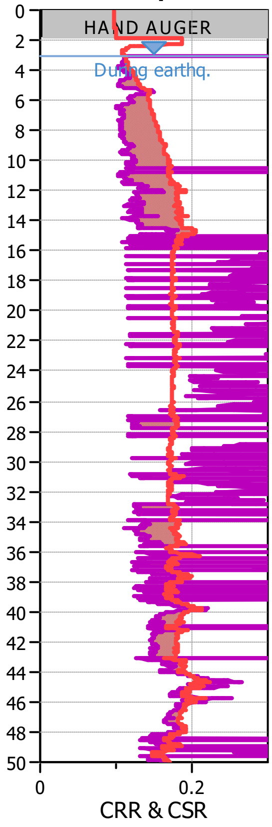

**Friction Ratio**



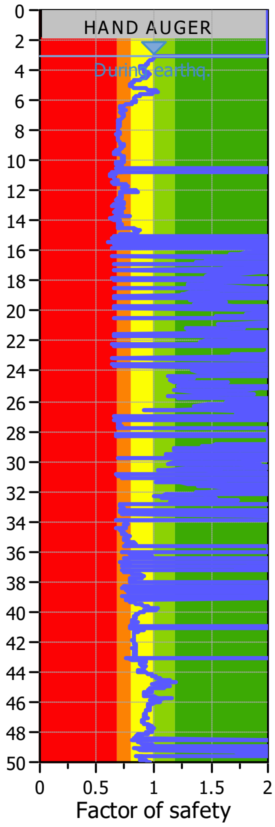

**SBTn Plot**



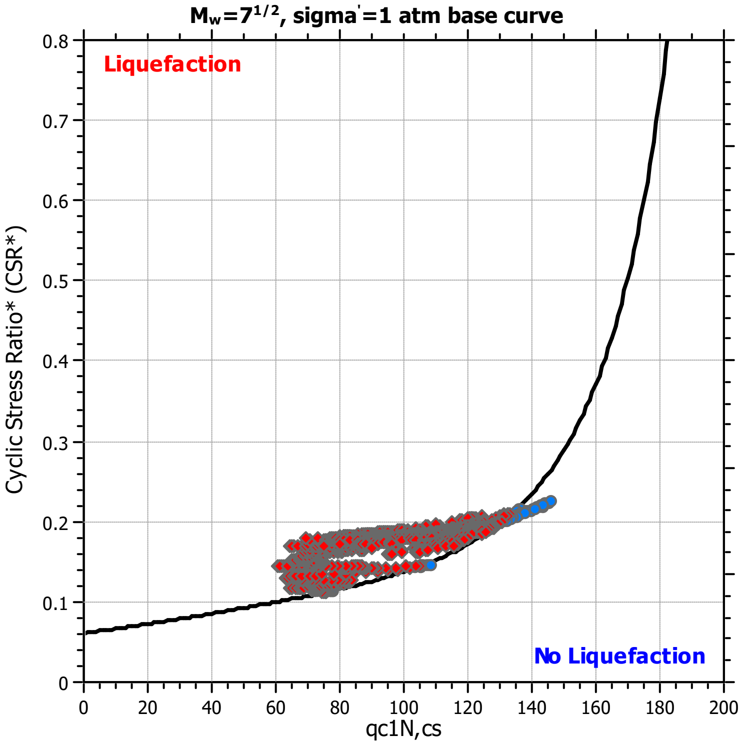

**CRR plot**



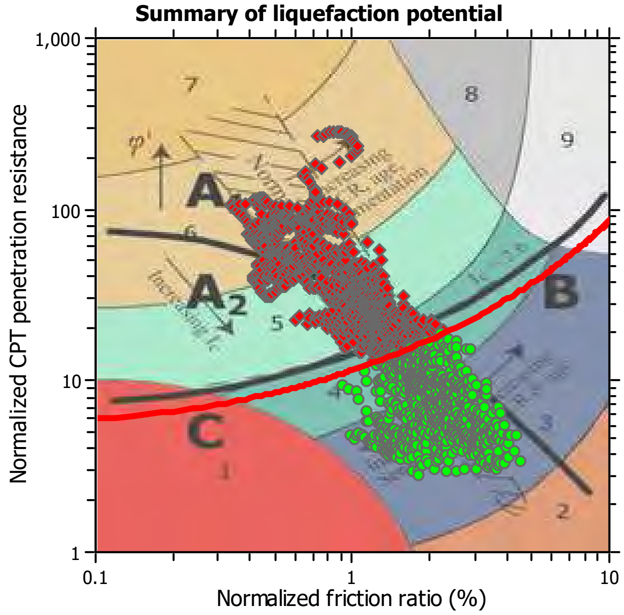

**FS Plot**




**$M_w=7^{1/2}$, sigma'=1 atm base curve**




**Summary of liquefaction potential**




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Braun Geotechnical Ltd.

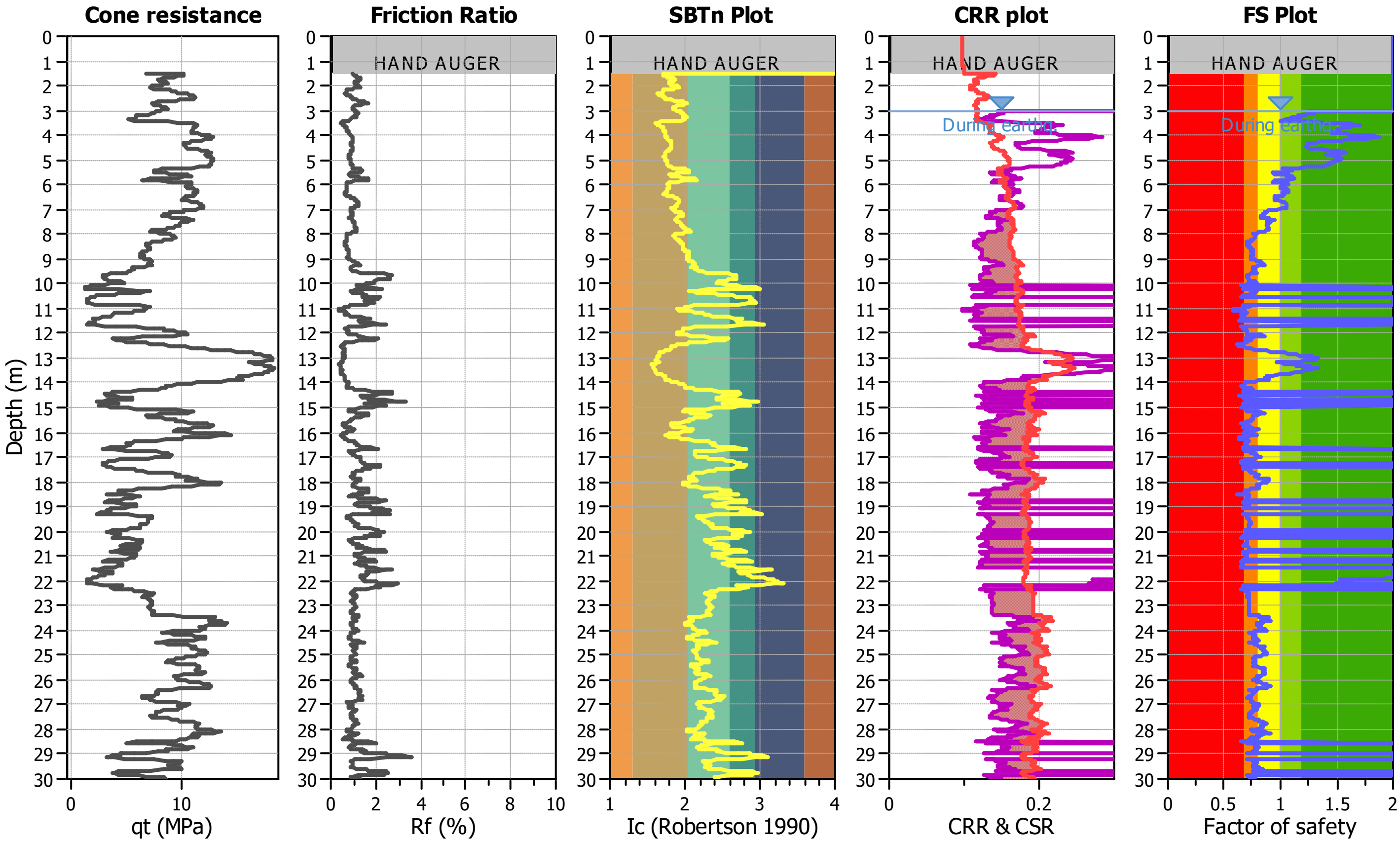
## LIQUEFACTION ANALYSIS REPORT

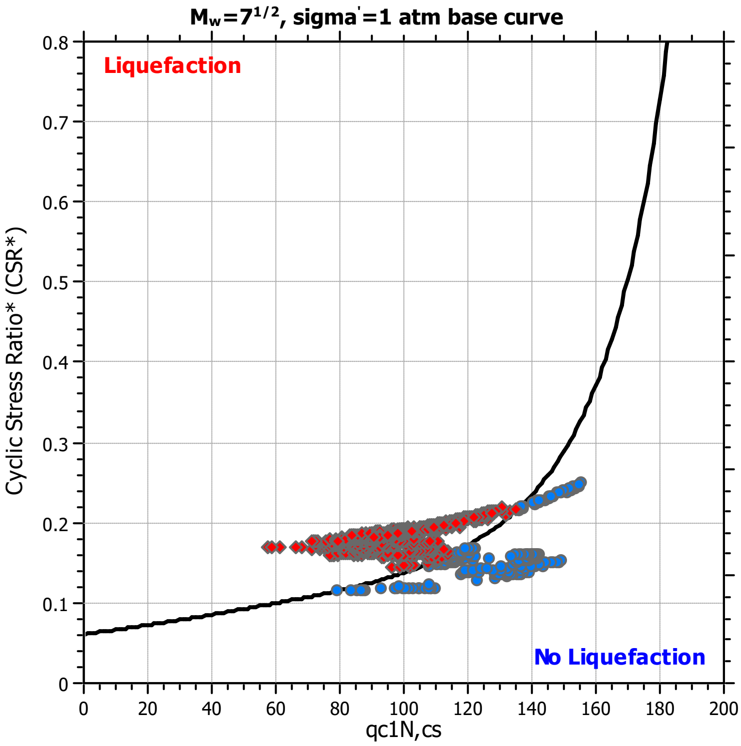
**Project title :** **Location :**

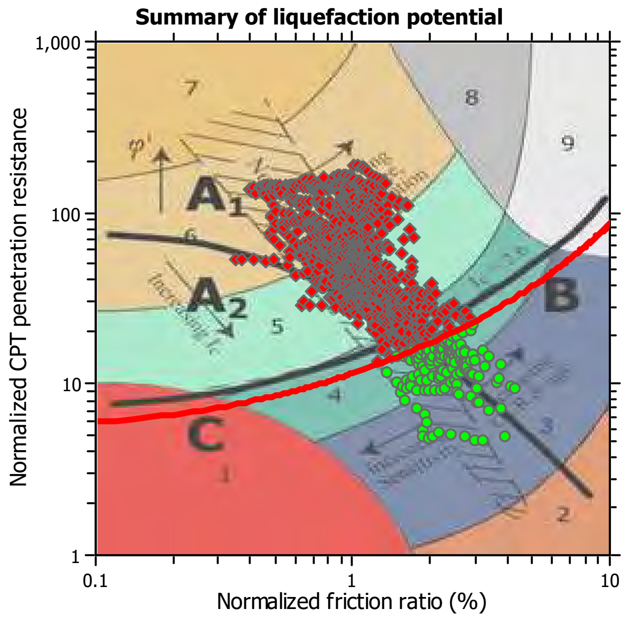
**CPT file : CPT20-12**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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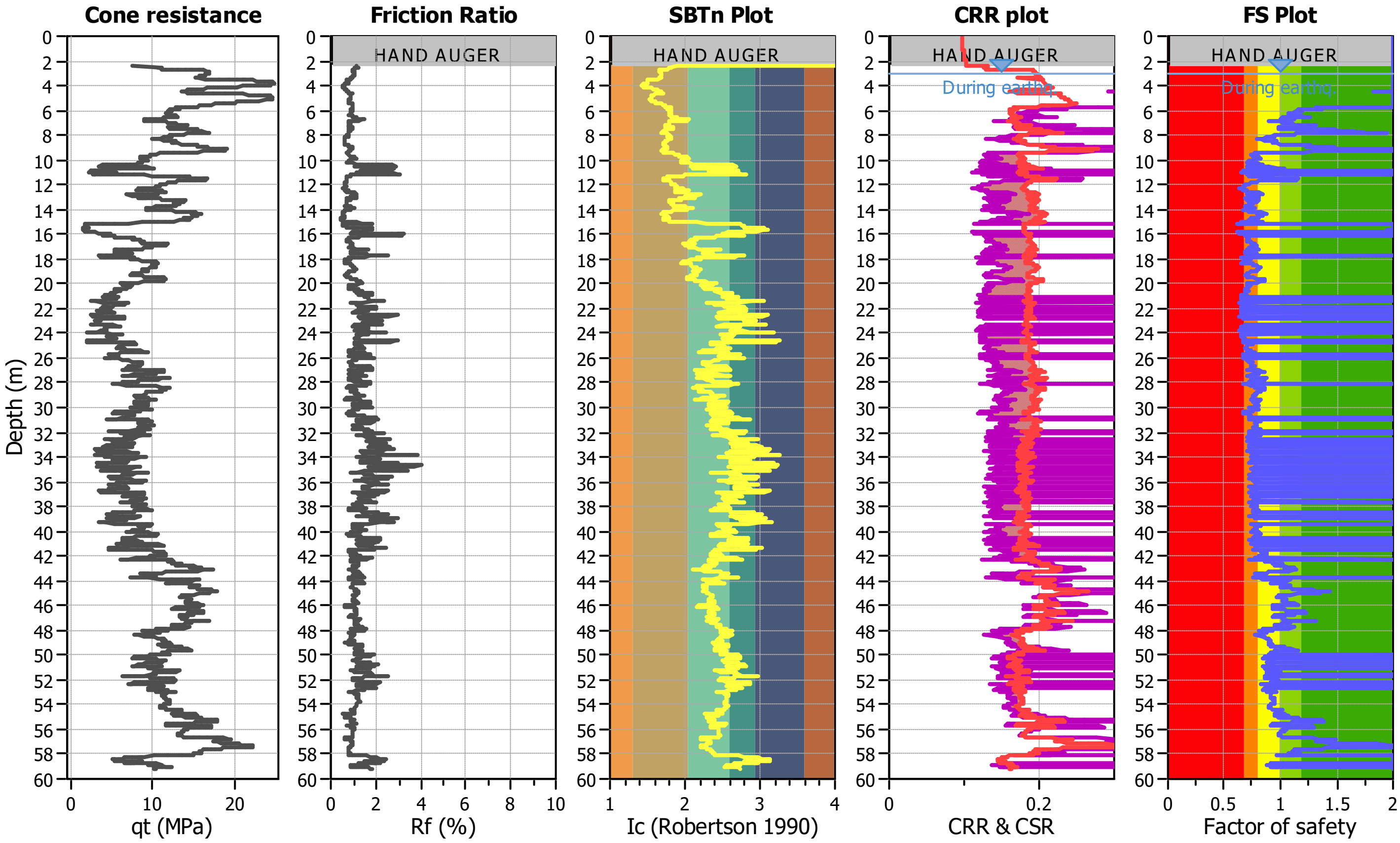
## LIQUEFACTION ANALYSIS REPORT

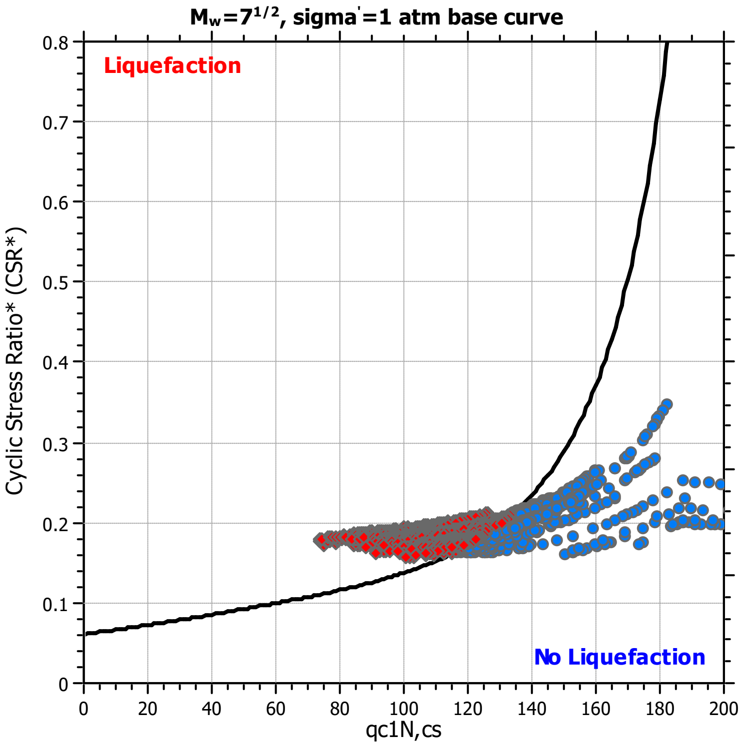
**Project title :** **Location :**

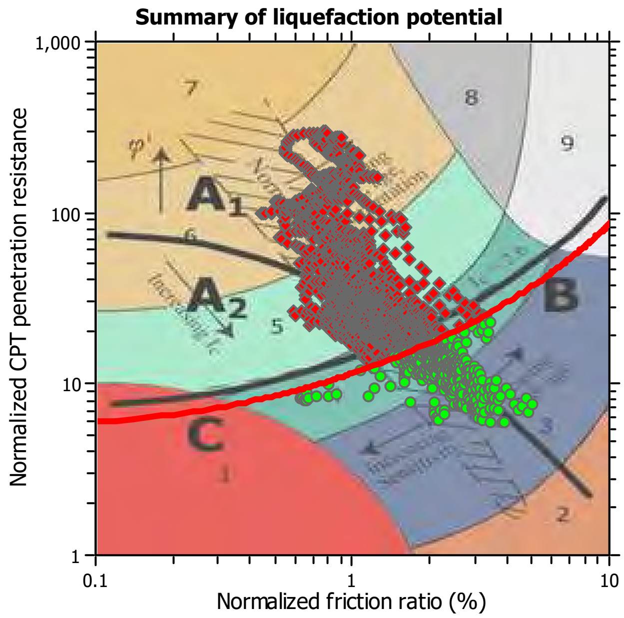
**CPT file : CPT20-13**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

# Liquefaction Triggering Assessment

# A475 Crustal and Inslab motions

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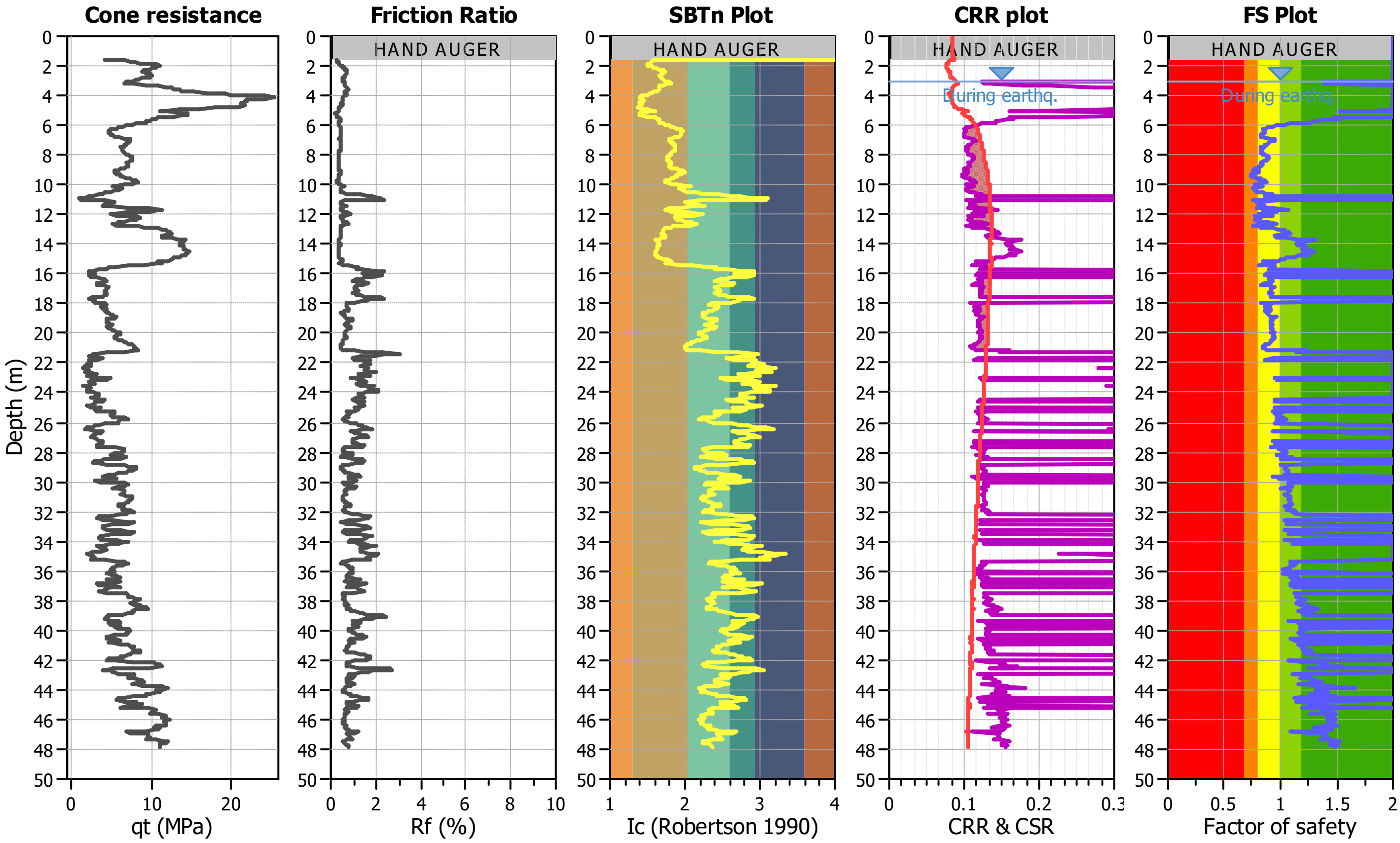
# LIQUEFACTION ANALYSIS REPORT

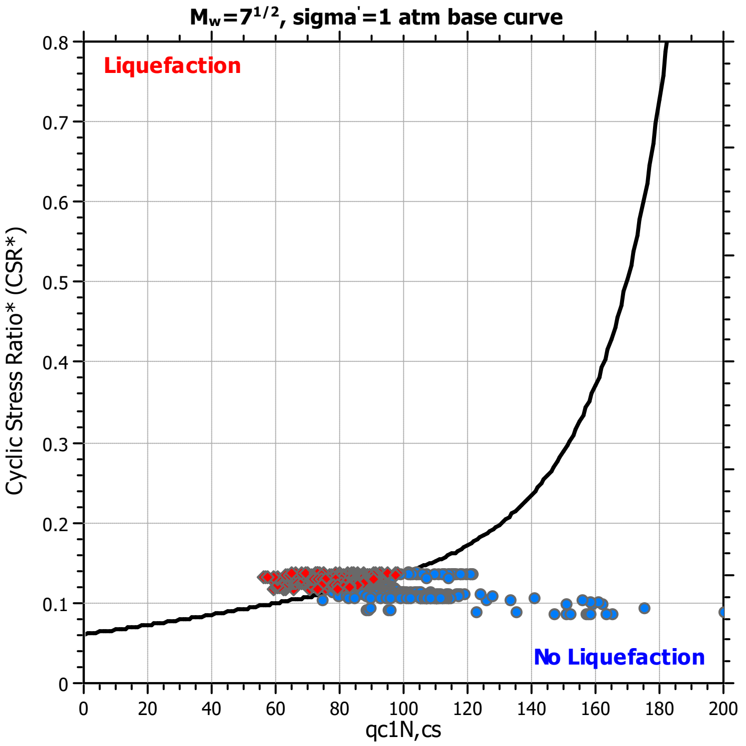
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

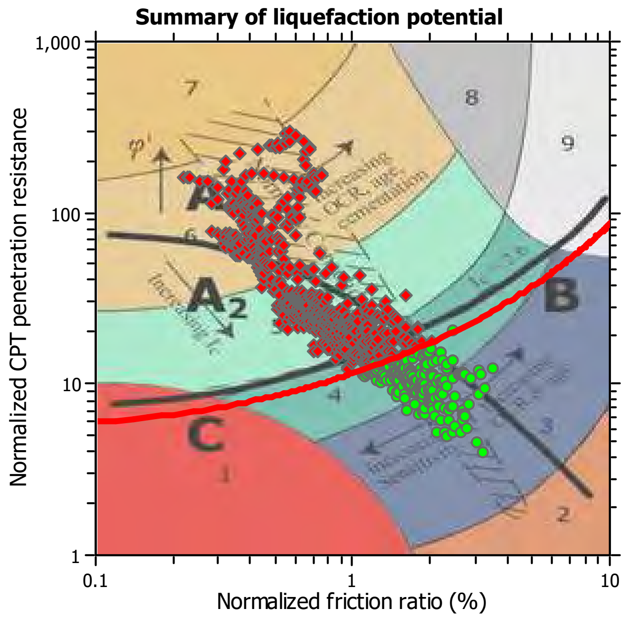
**CPT file : CPT18-01 Berth 2**

## Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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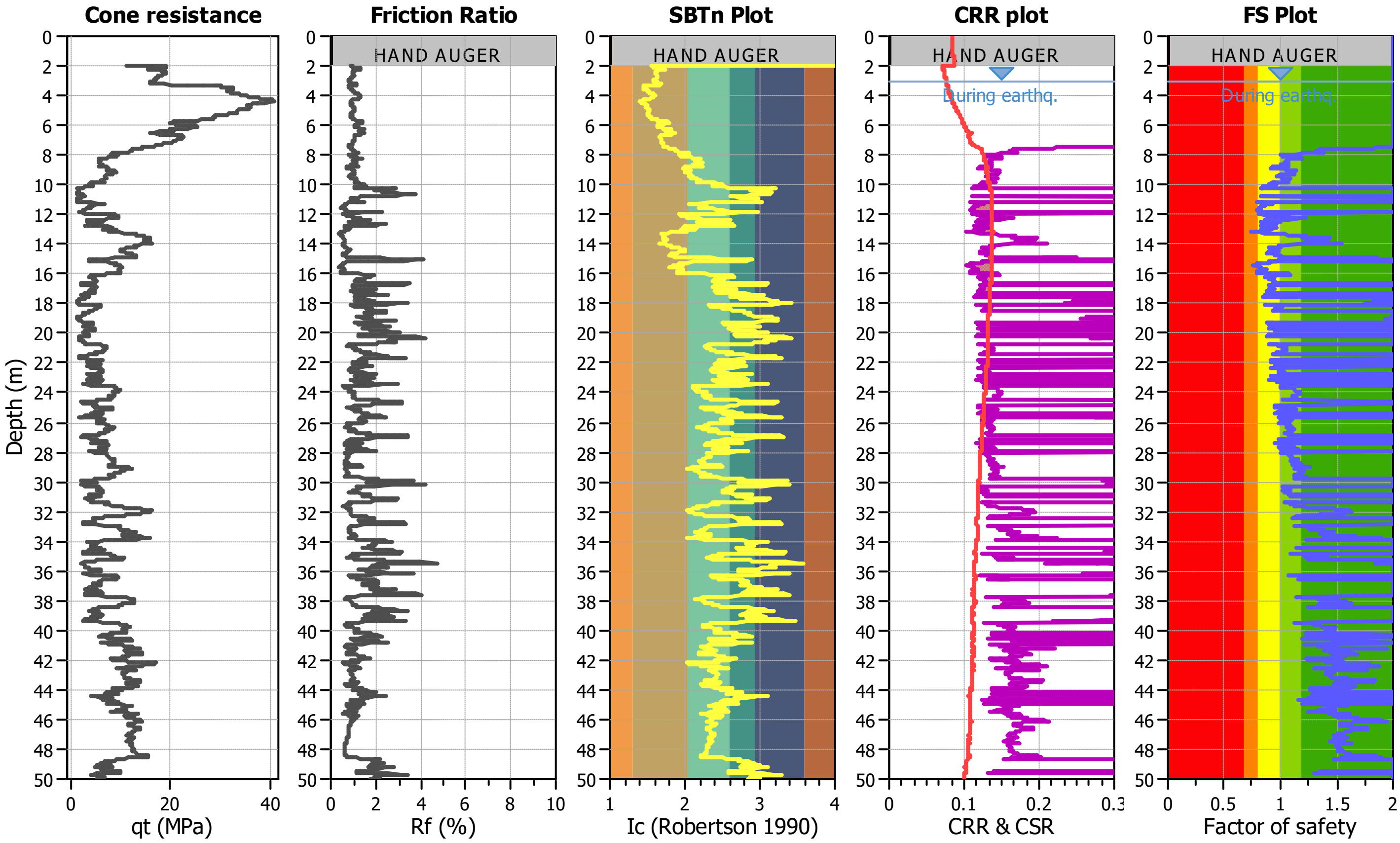
## LIQUEFACTION ANALYSIS REPORT

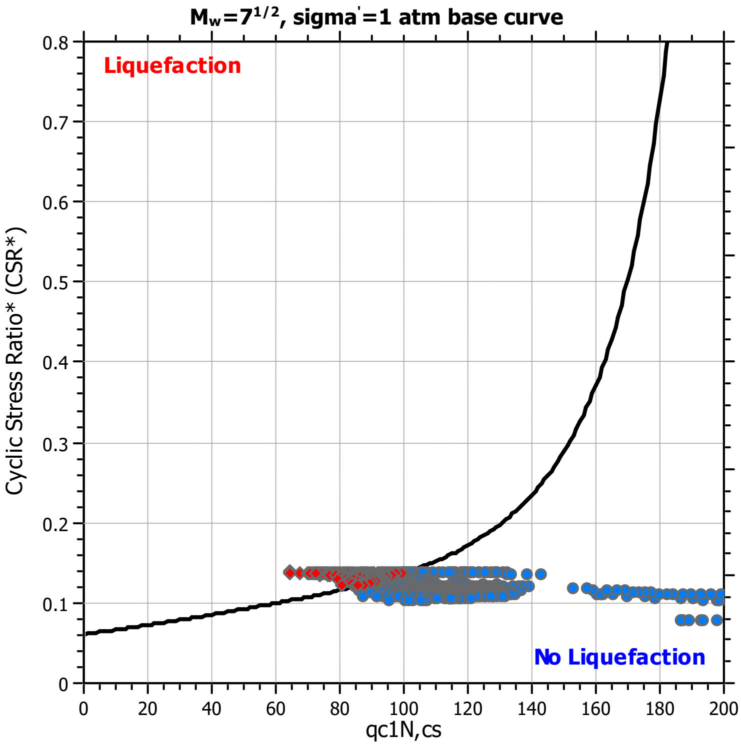
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

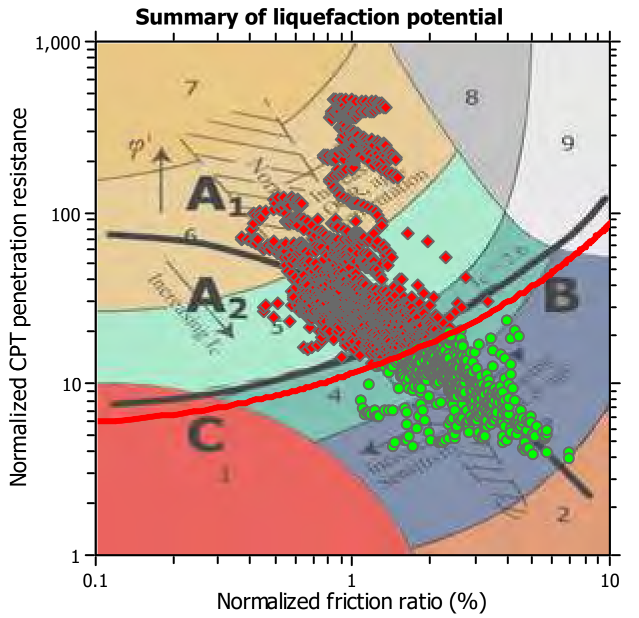
**CPT file : SCPT20-01**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | | |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | No |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | Limit depth: | N/A |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | MSF method: | Method |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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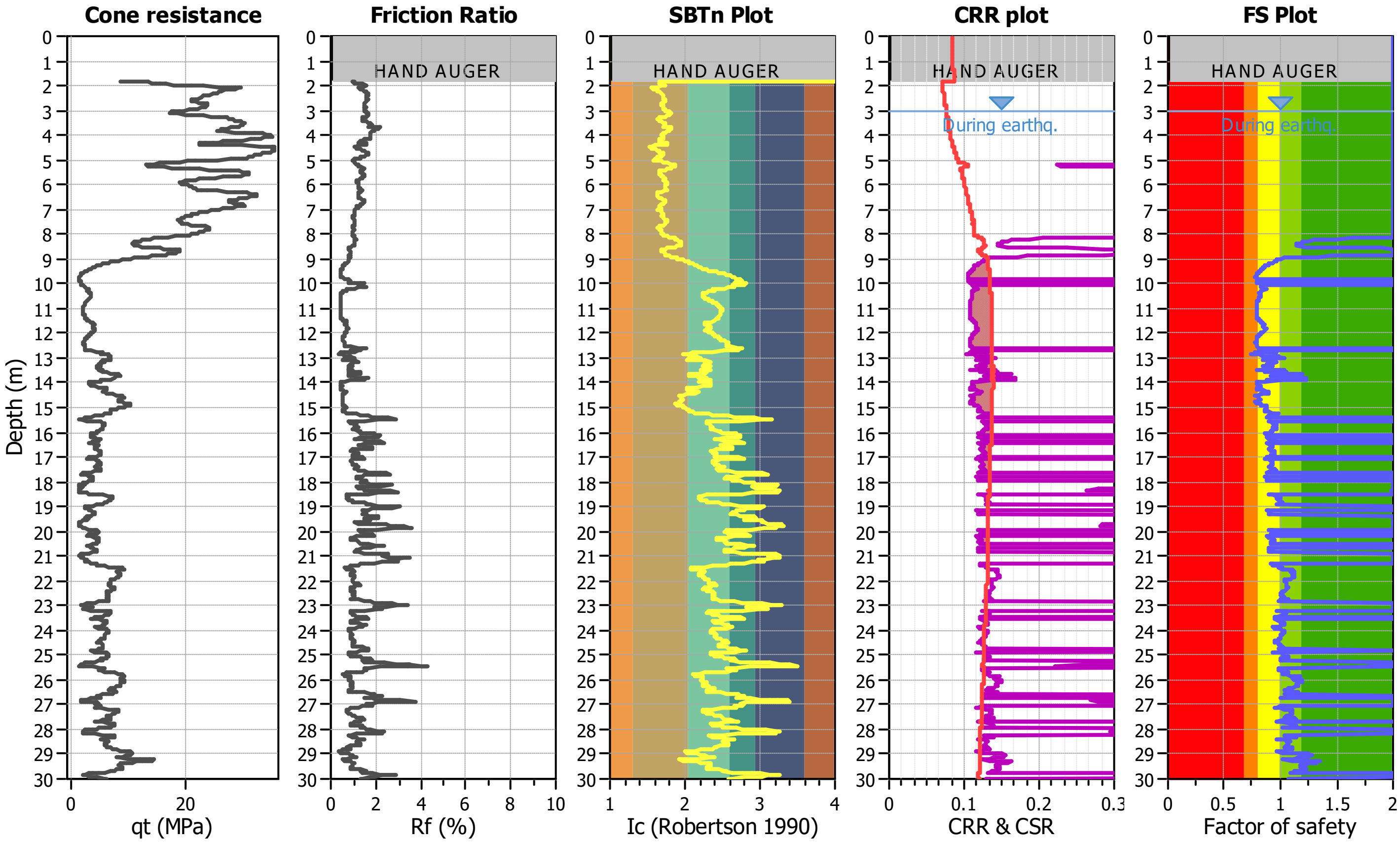
## LIQUEFACTION ANALYSIS REPORT

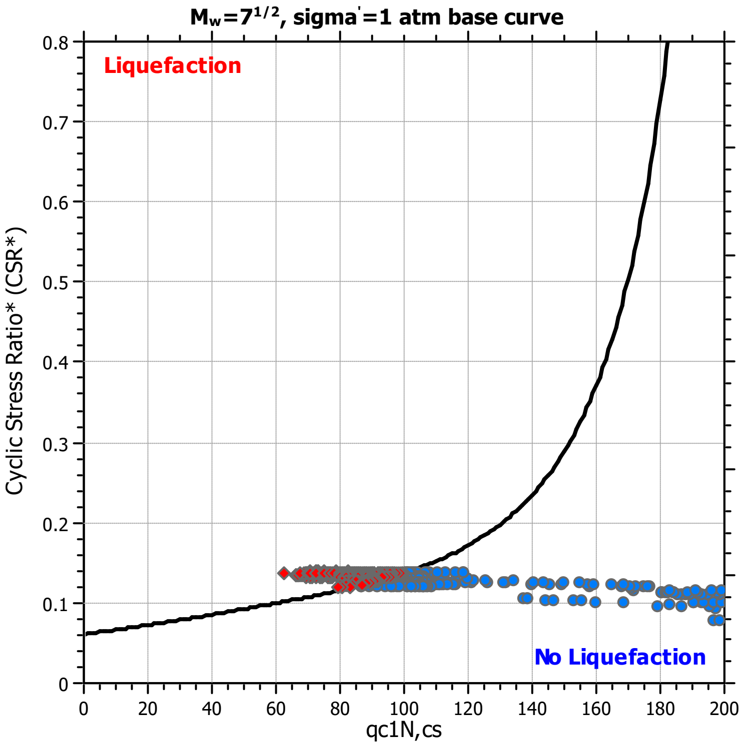
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

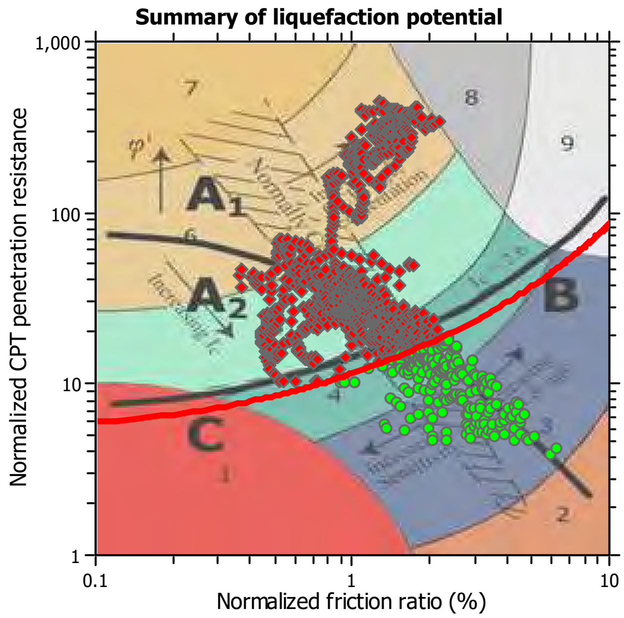
**CPT file : CPT20-02**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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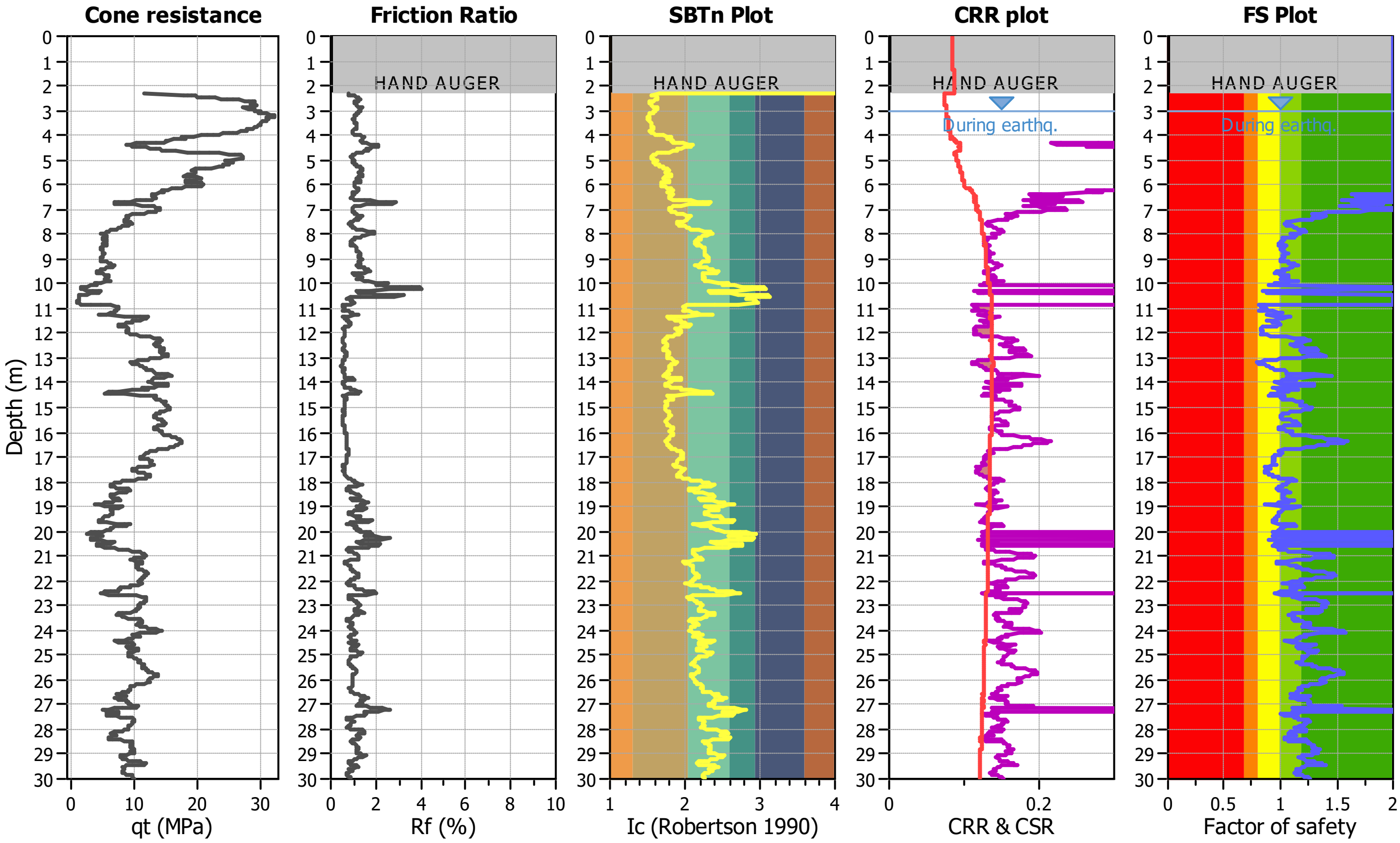
## LIQUEFACTION ANALYSIS REPORT

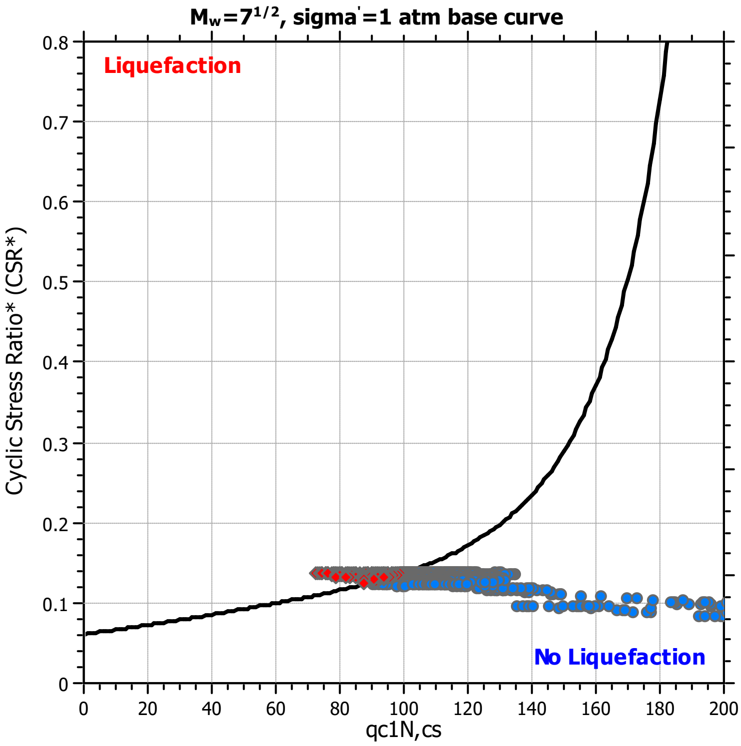
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

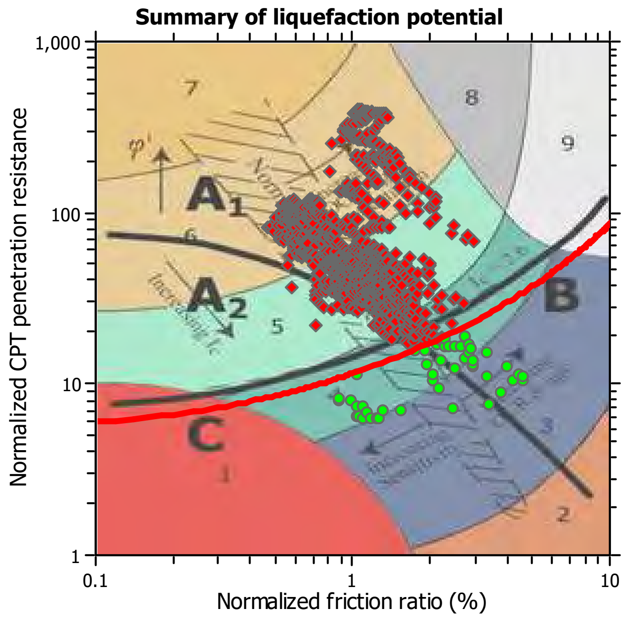
**CPT file : CPT20-03**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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**Braun Geotechnical Ltd.**

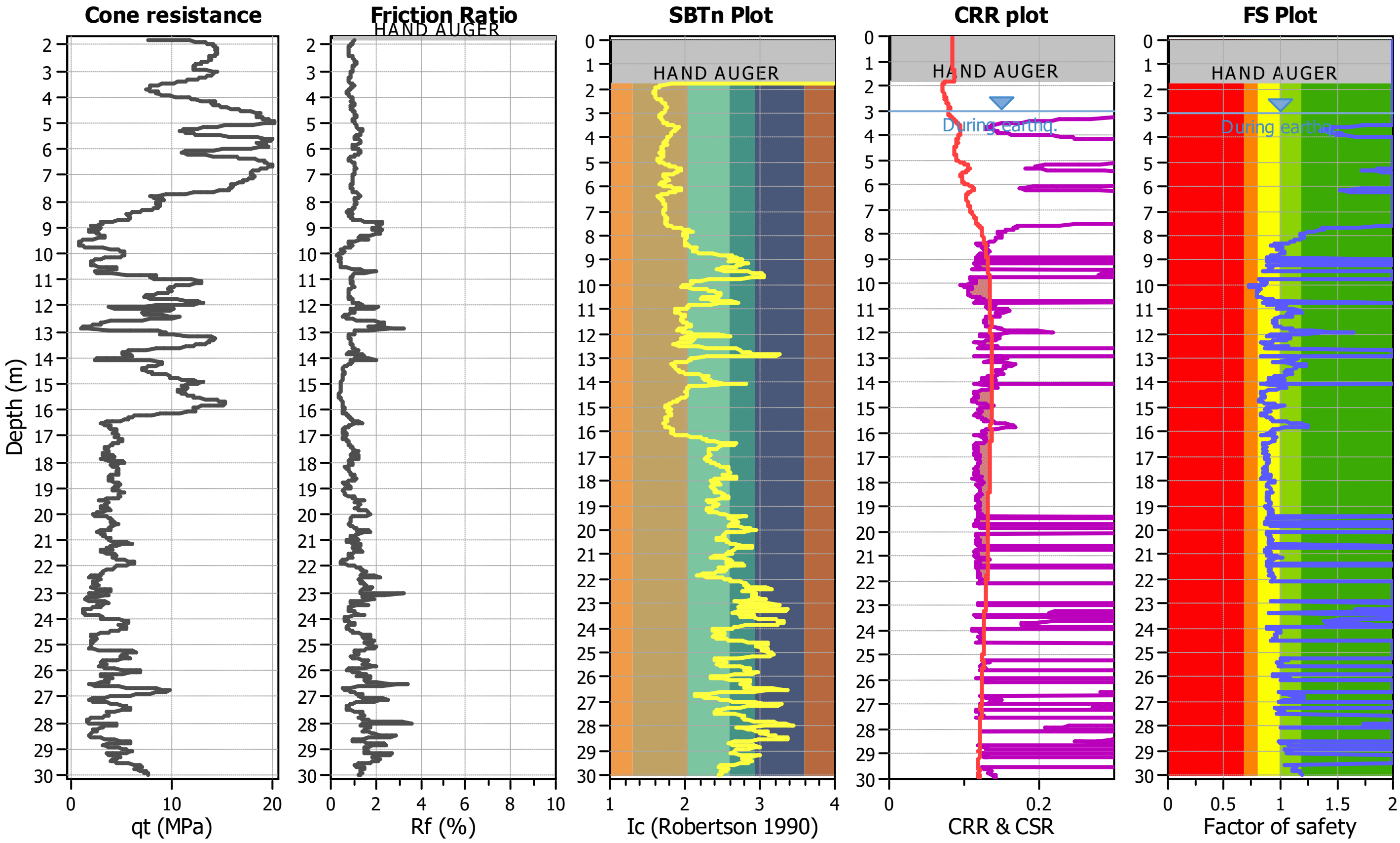
## LIQUEFACTION ANALYSIS REPORT

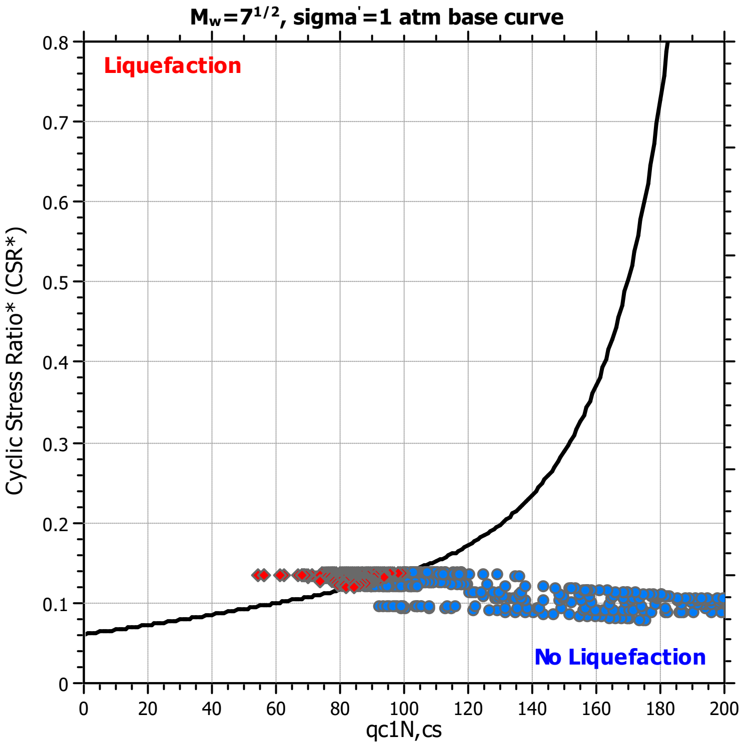
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

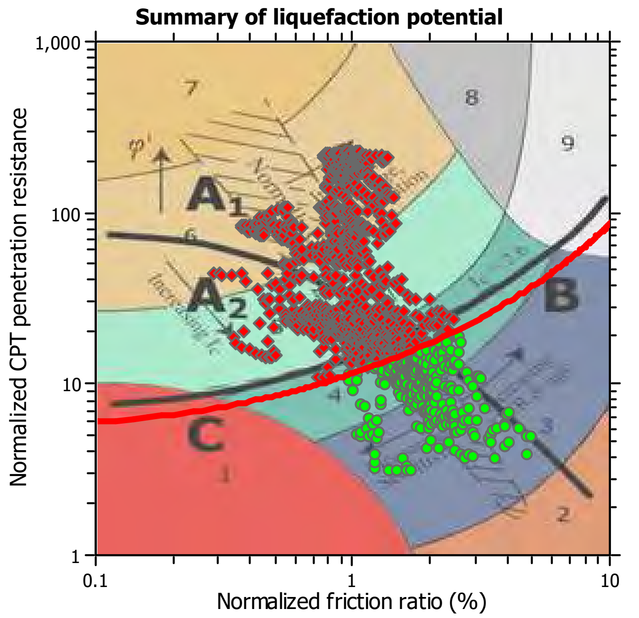
**CPT file : CP20-04**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Braun Geotechnical Ltd.

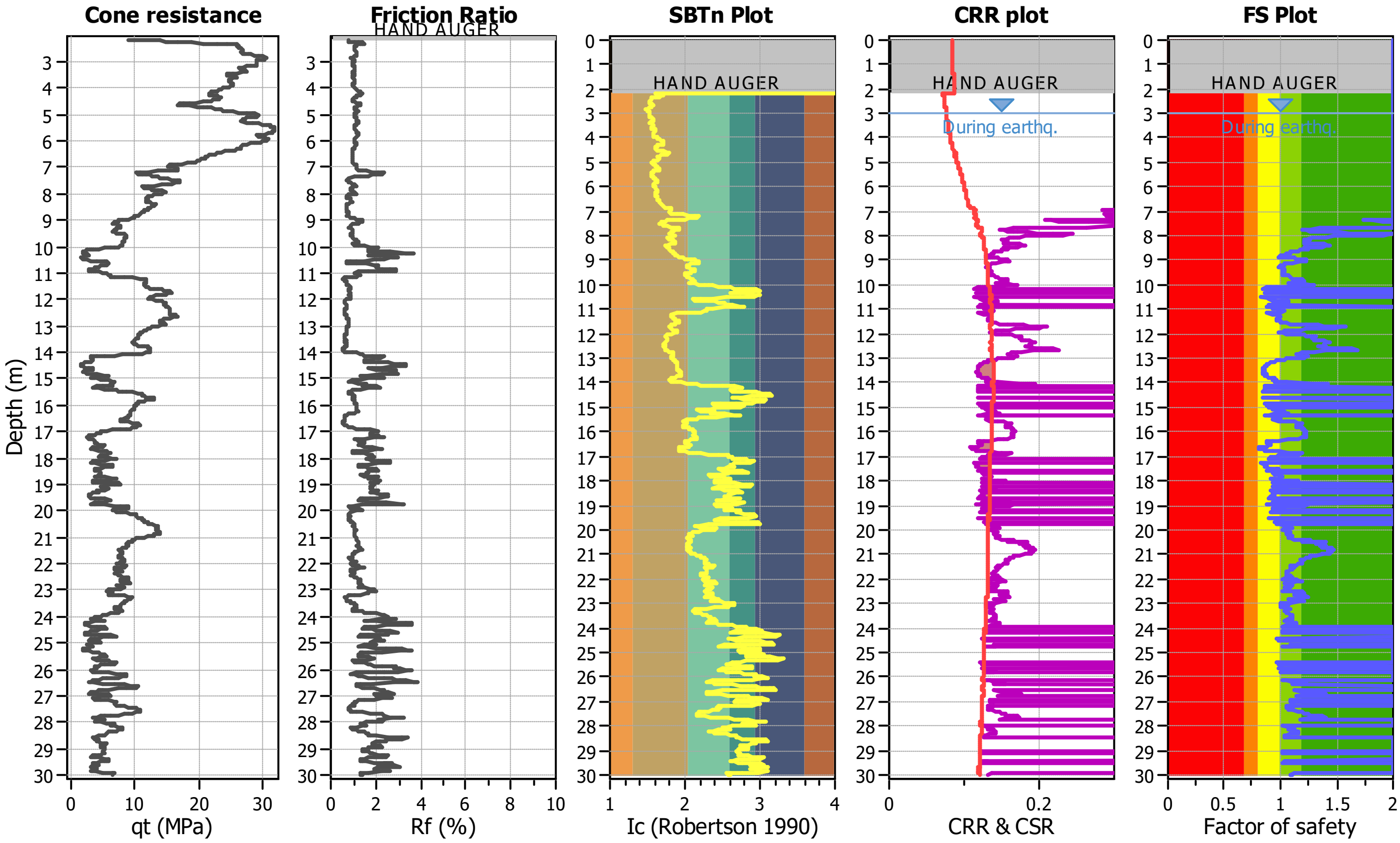
## LIQUEFACTION ANALYSIS REPORT

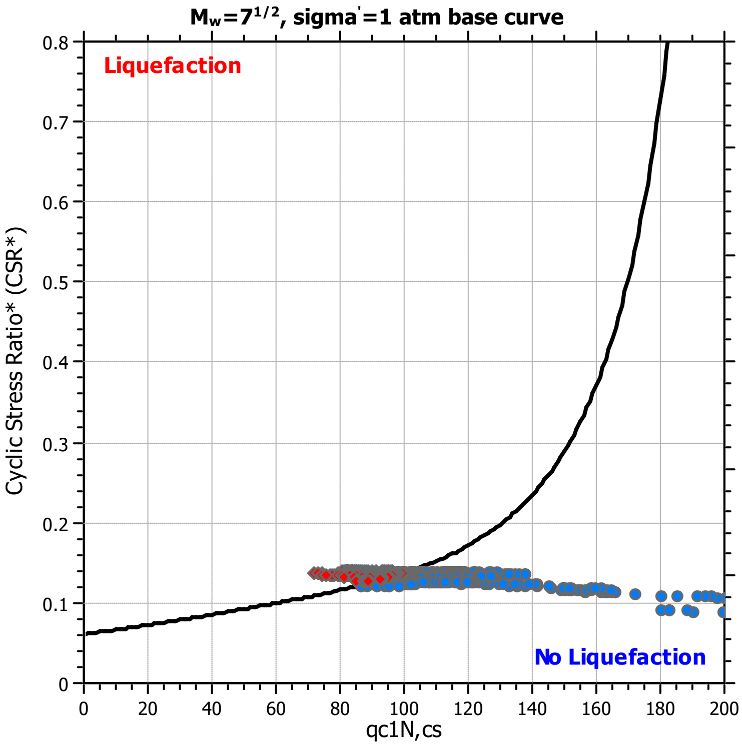
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

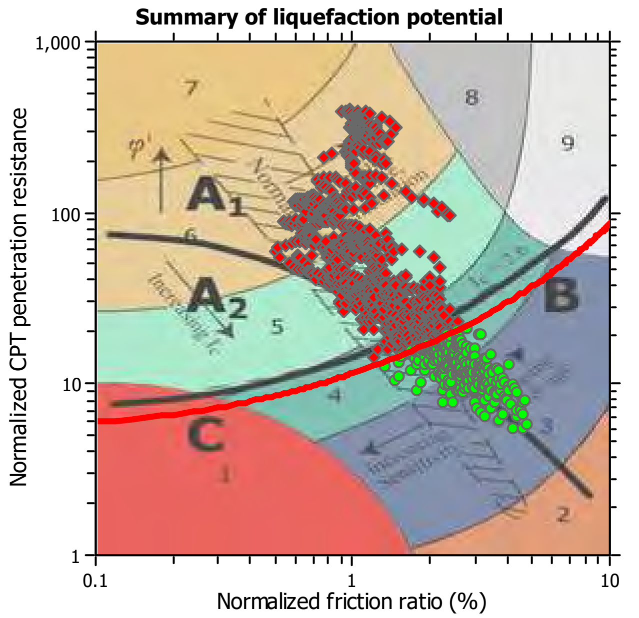
**CPT file : CPT20-05**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Braun Geotechnical Ltd.

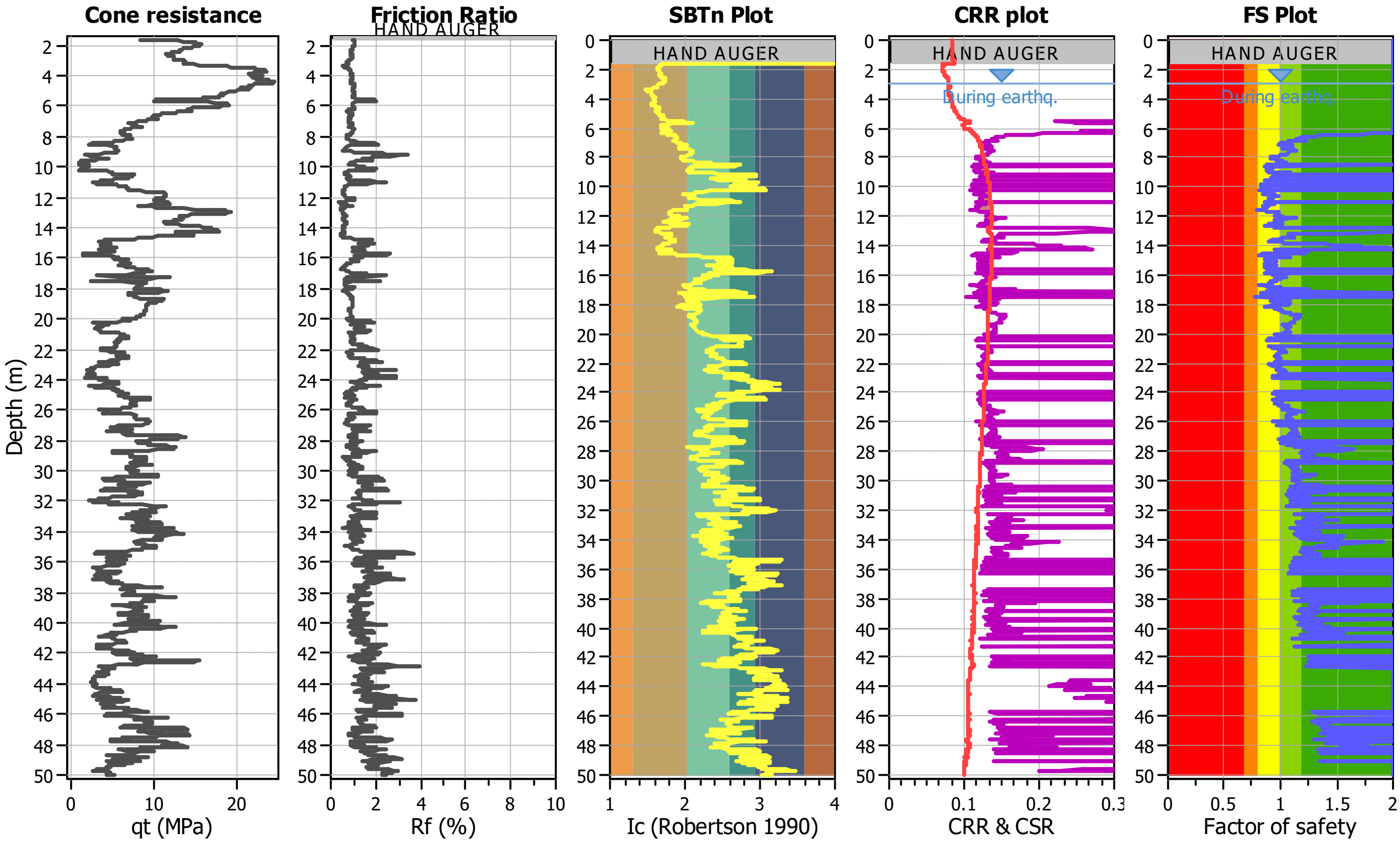
## LIQUEFACTION ANALYSIS REPORT

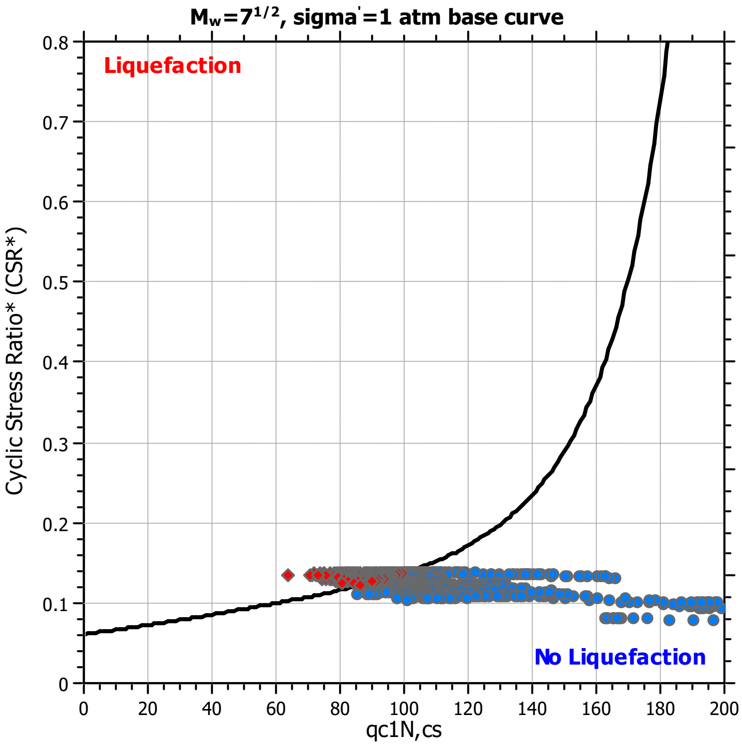
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

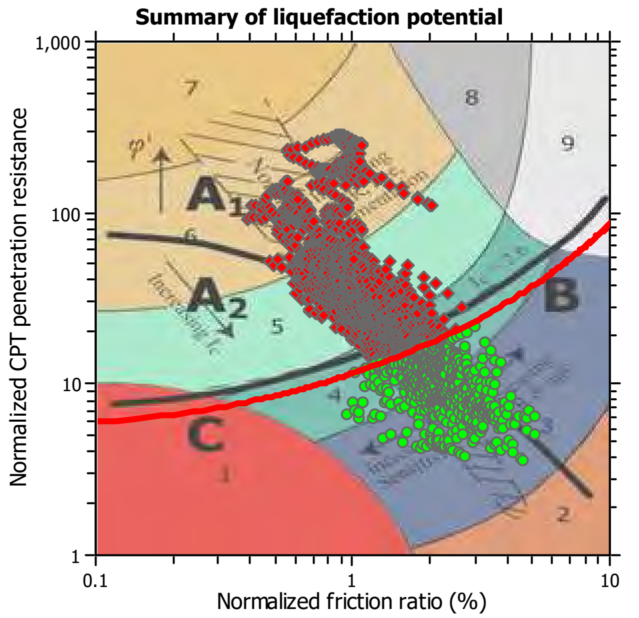
**CPT file : SCPT20-06**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Braun Geotechnical Ltd.

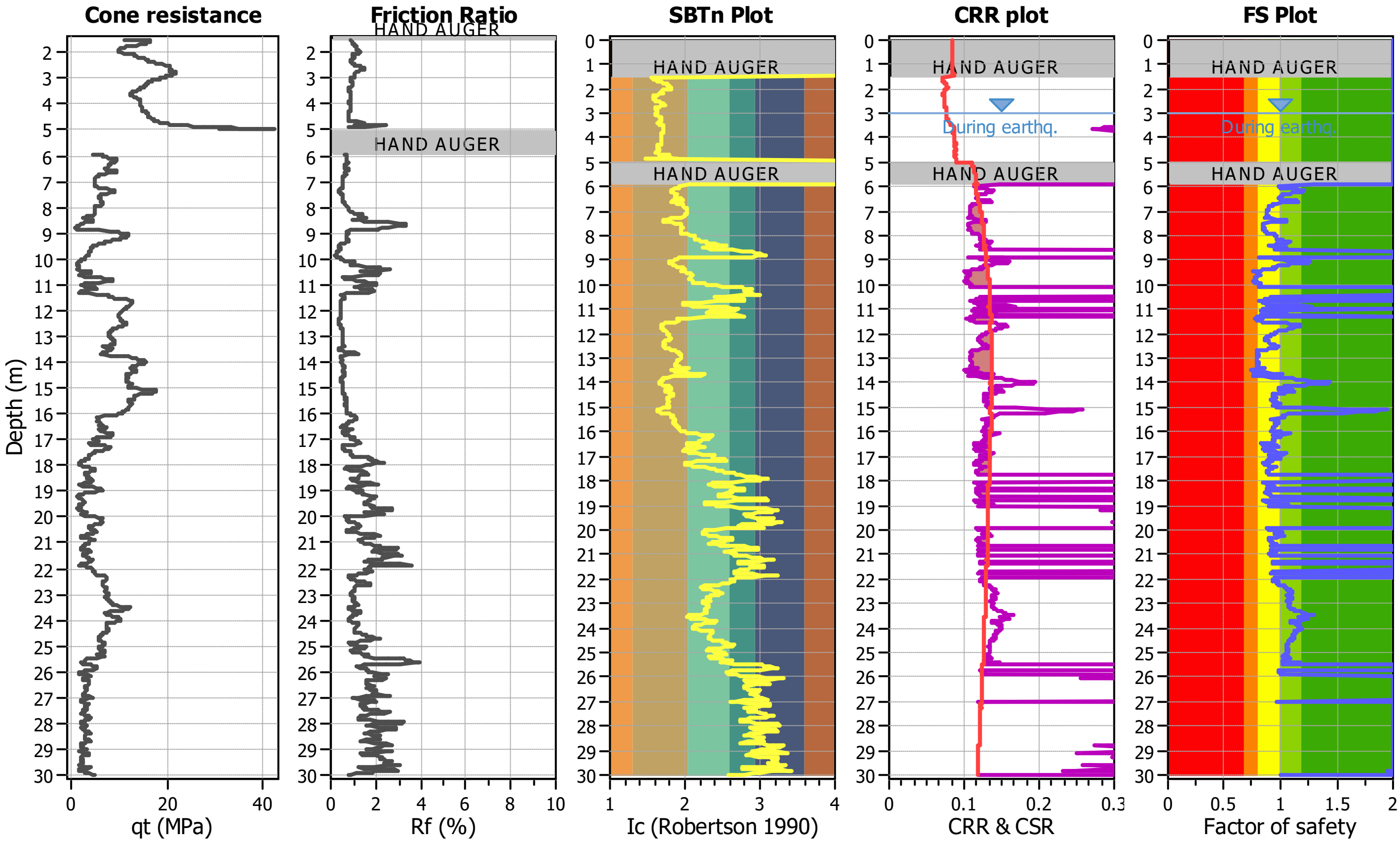
# LIQUEFACTION ANALYSIS REPORT

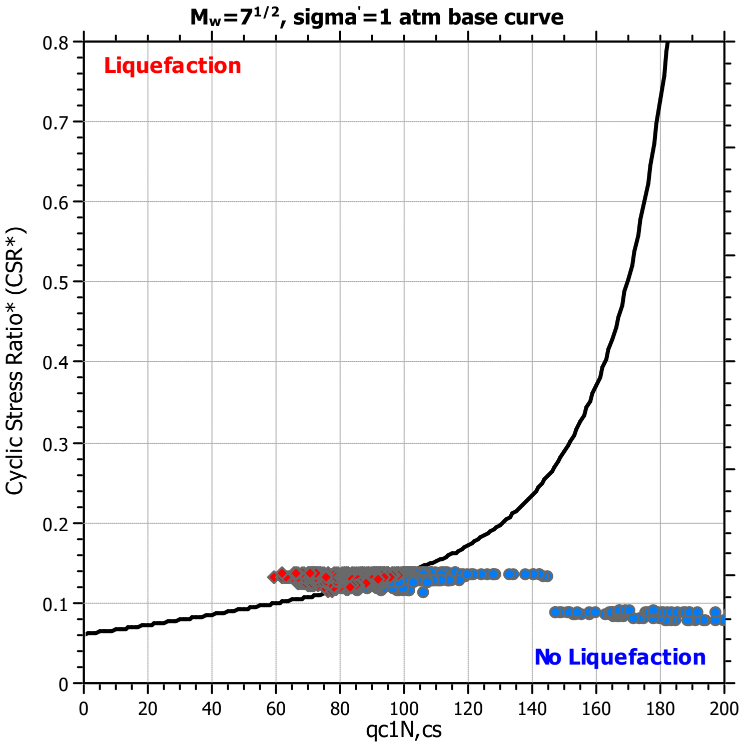
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

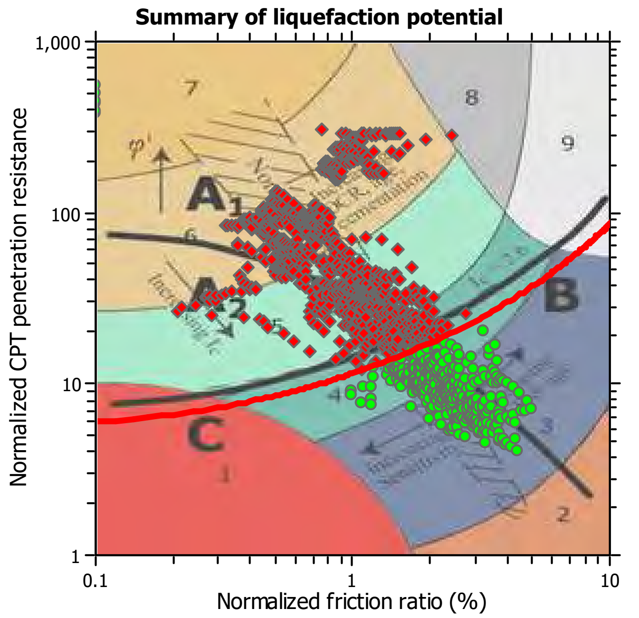
**CPT file : CPT20-08**

## Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

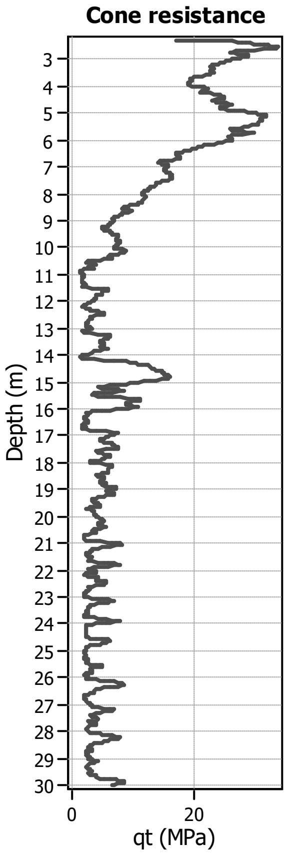
## LIQUEFACTION ANALYSIS REPORT

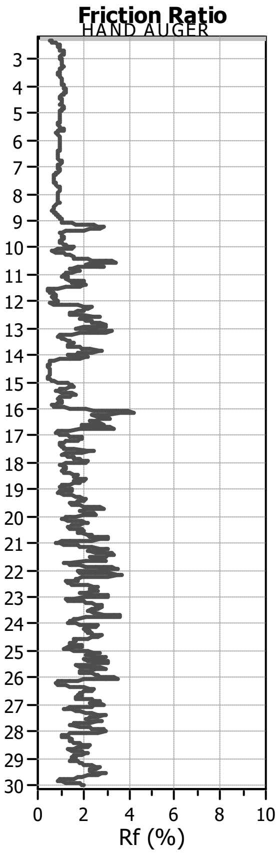
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

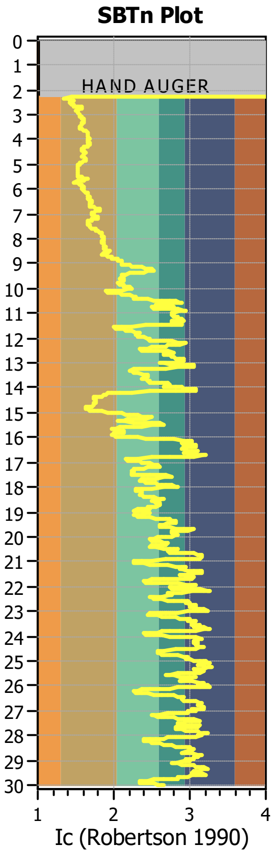
**CPT file : CPT20-07**

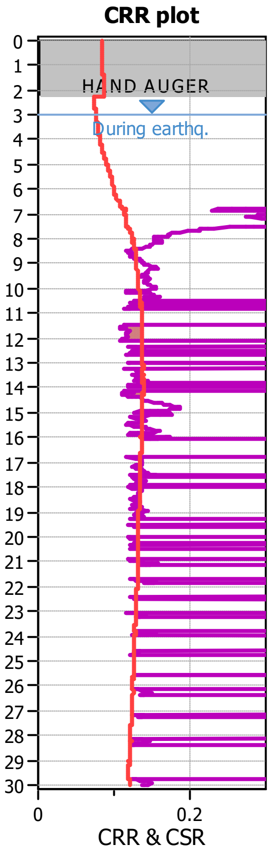
### Input parameters and analysis data

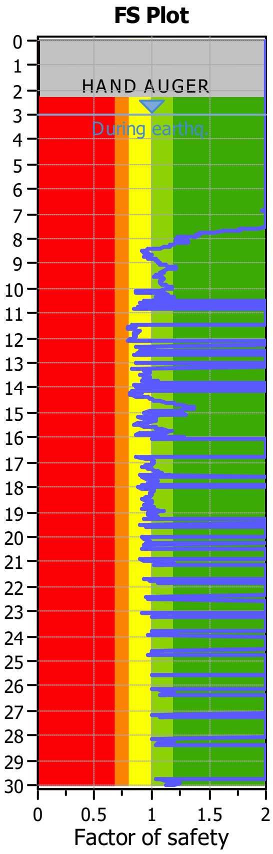
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

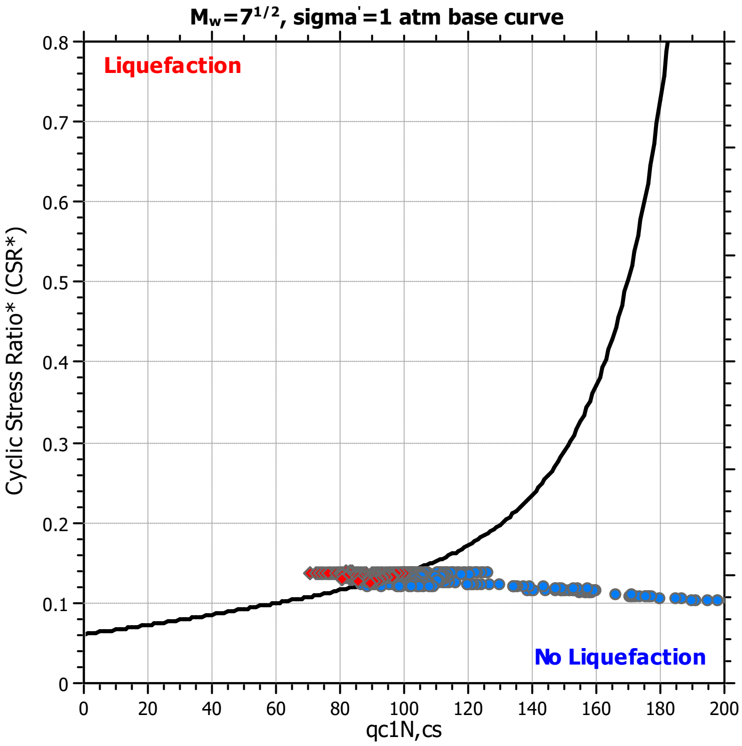


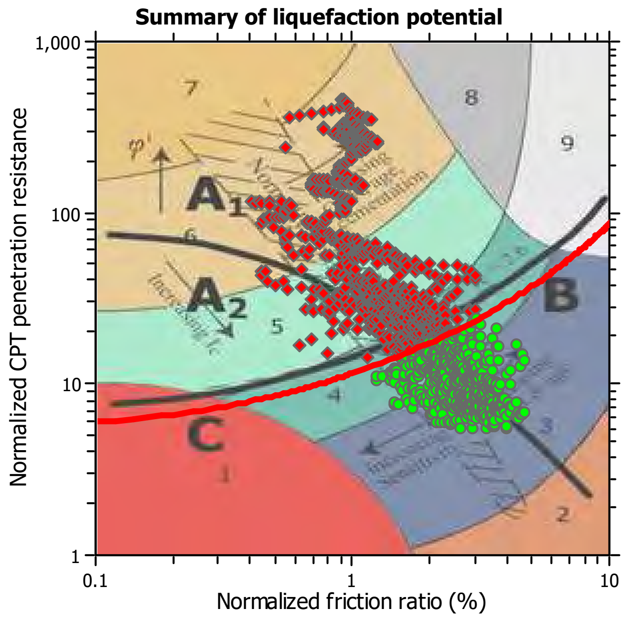




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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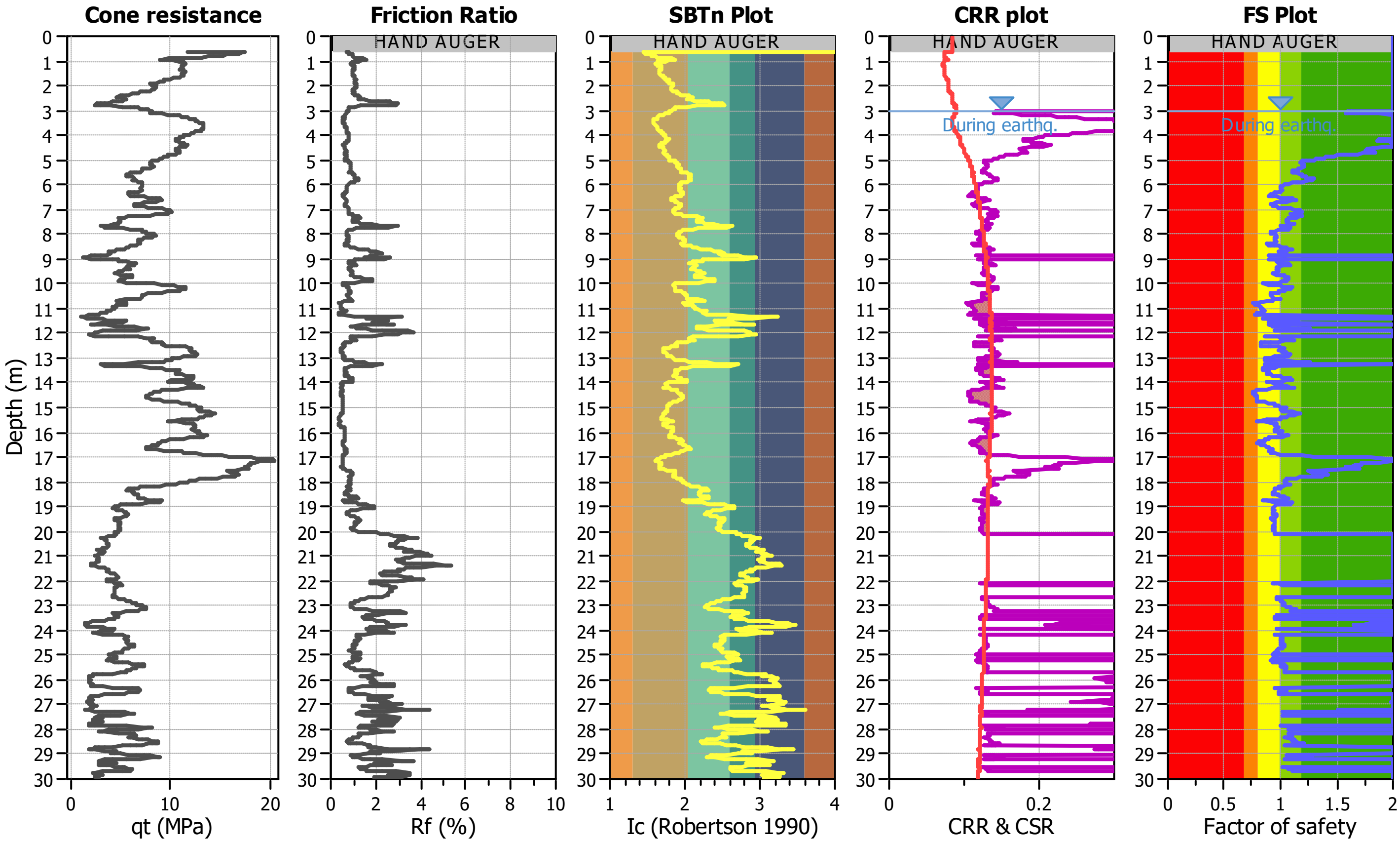
## LIQUEFACTION ANALYSIS REPORT

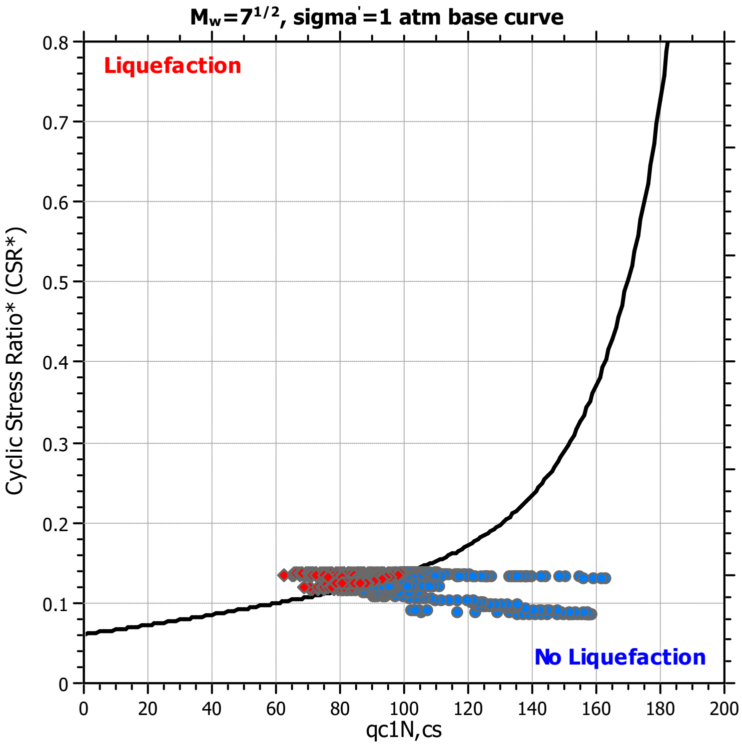
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

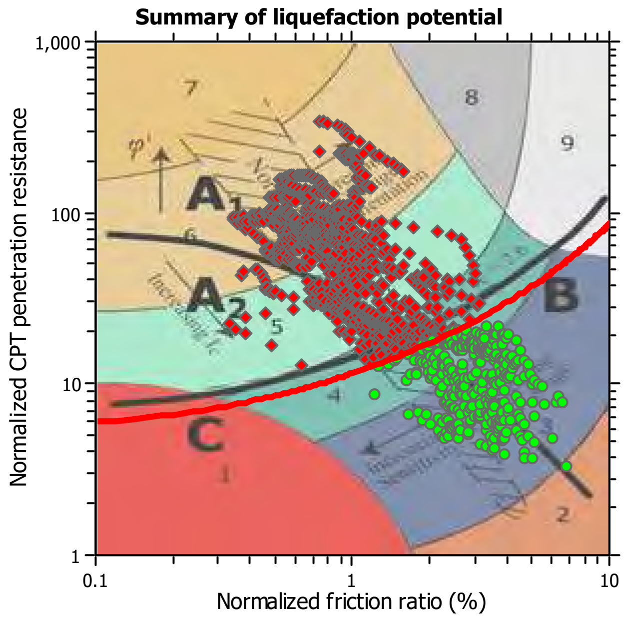
**CPT file : SCPT20-09**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Braun Geotechnical Ltd.

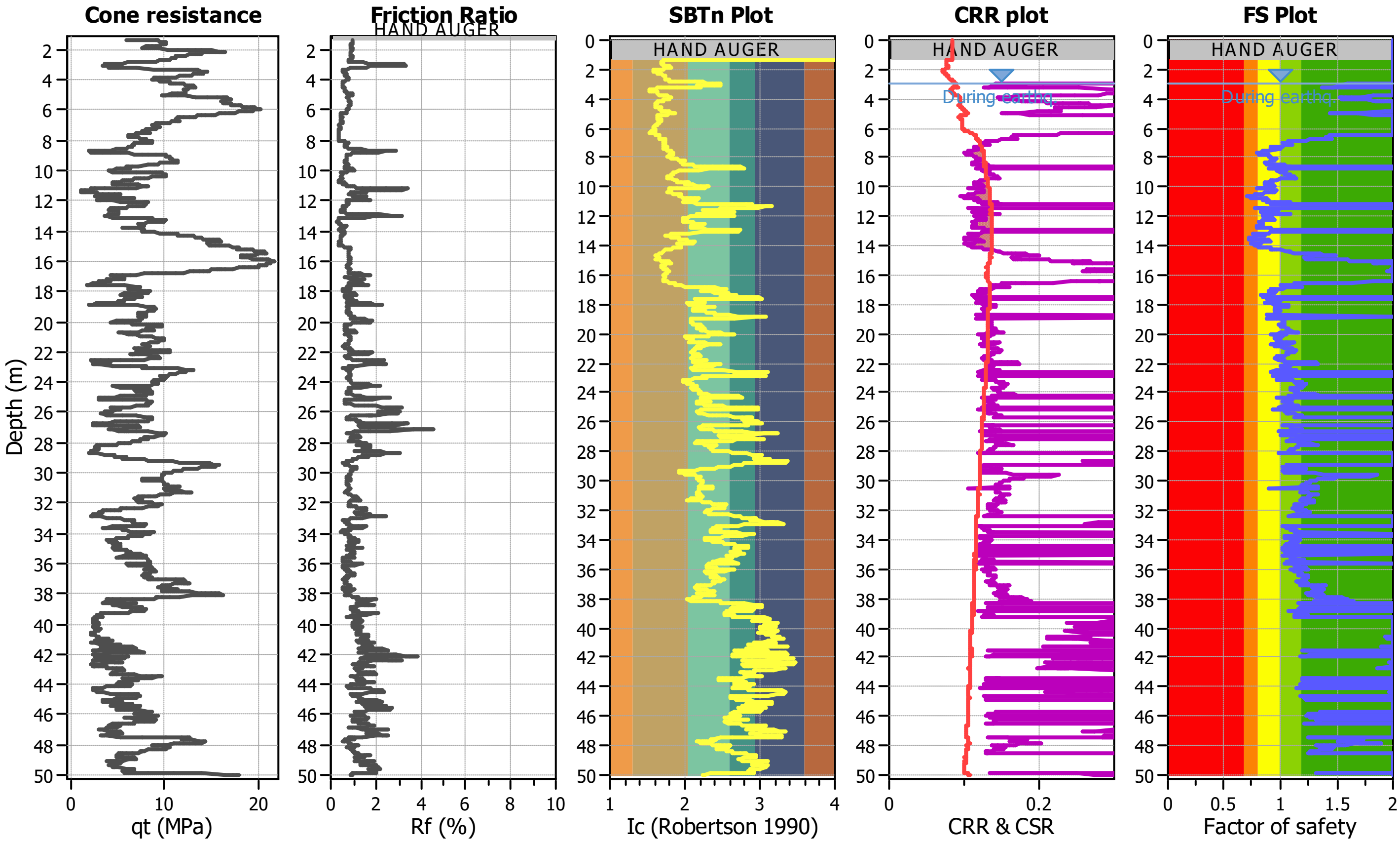
## LIQUEFACTION ANALYSIS REPORT

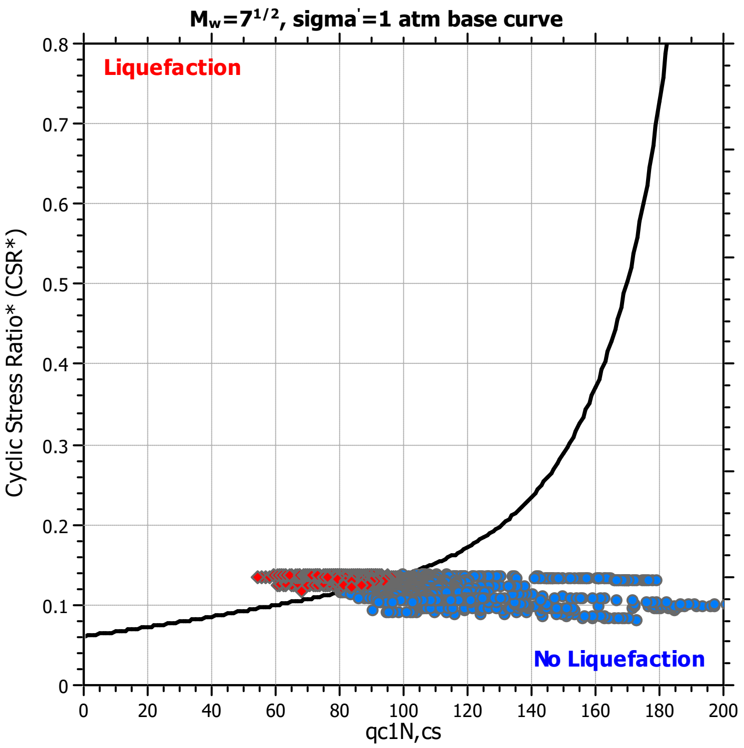
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

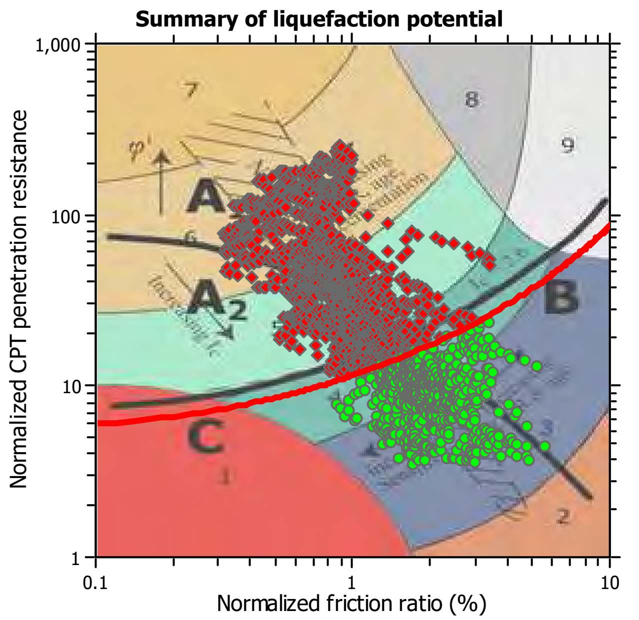
**CPT file : CPT20-10**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Braun Geotechnical Ltd.

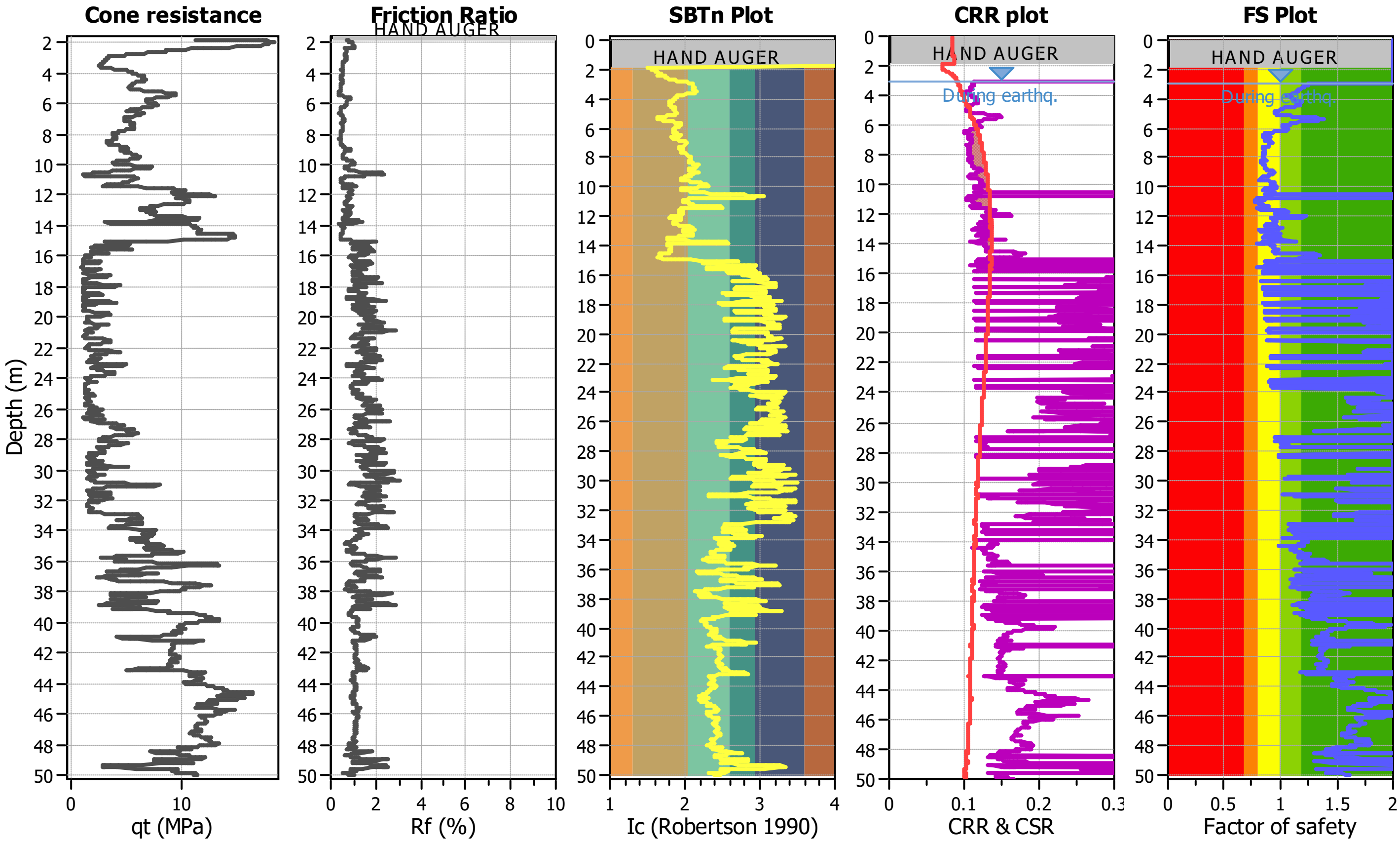
## LIQUEFACTION ANALYSIS REPORT

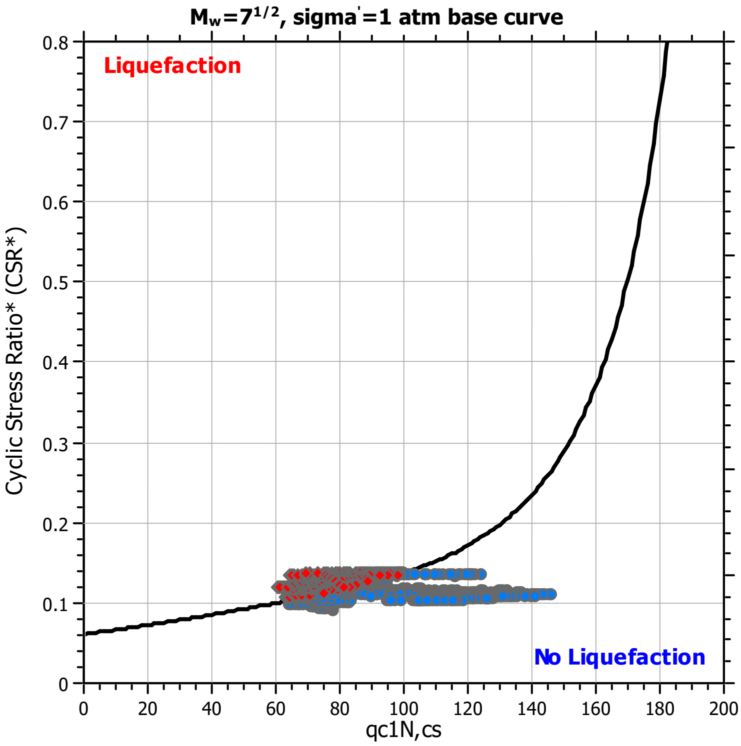
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

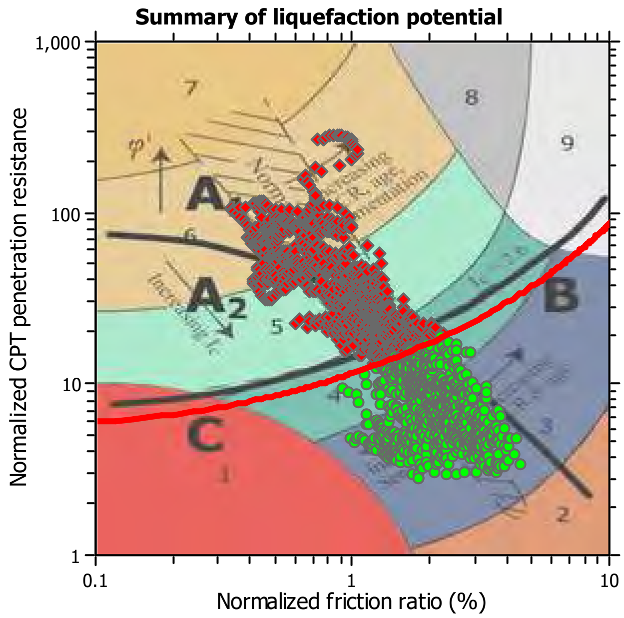
**CPT file : CPT20-11**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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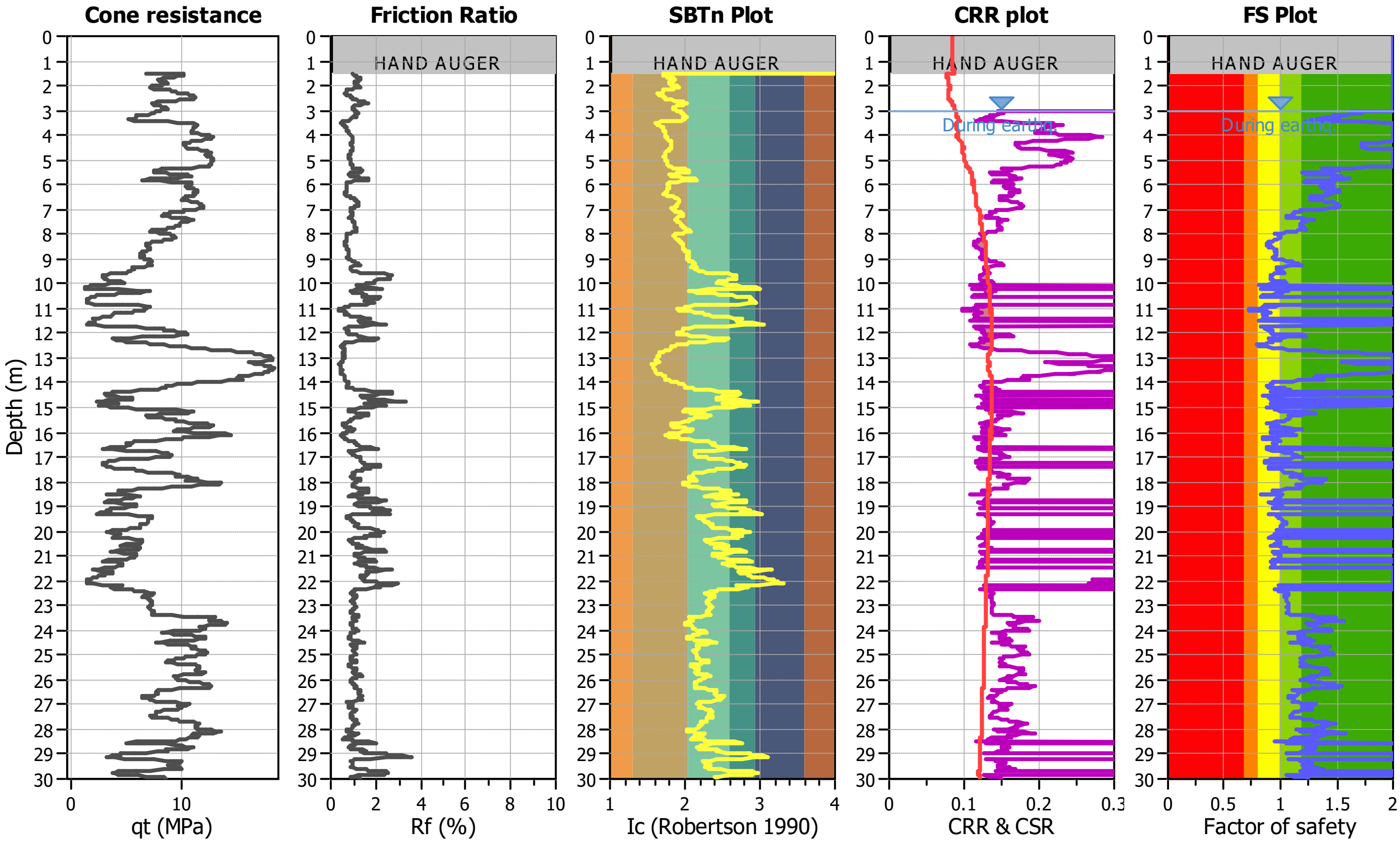
# LIQUEFACTION ANALYSIS REPORT

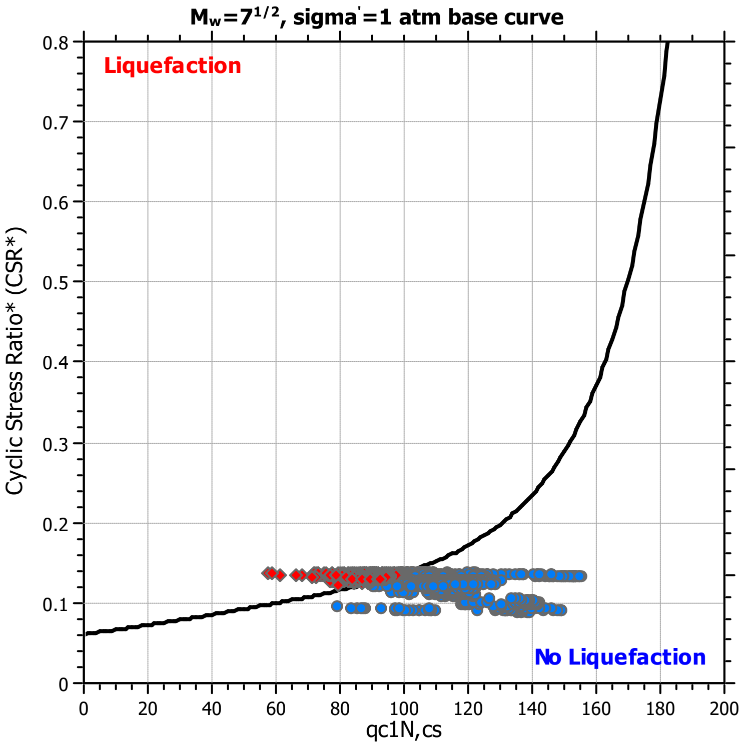
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

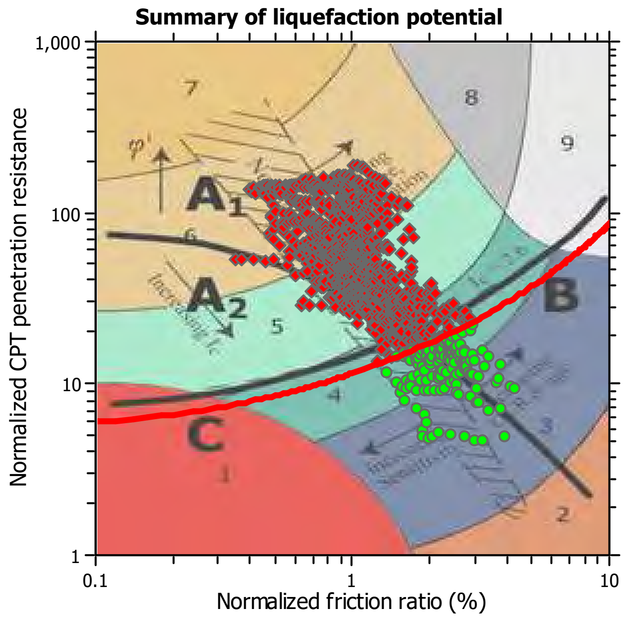
**CPT file : CPT20-12**

## Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | | |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | No |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | Limit depth: | N/A |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | MSF method: | Method |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

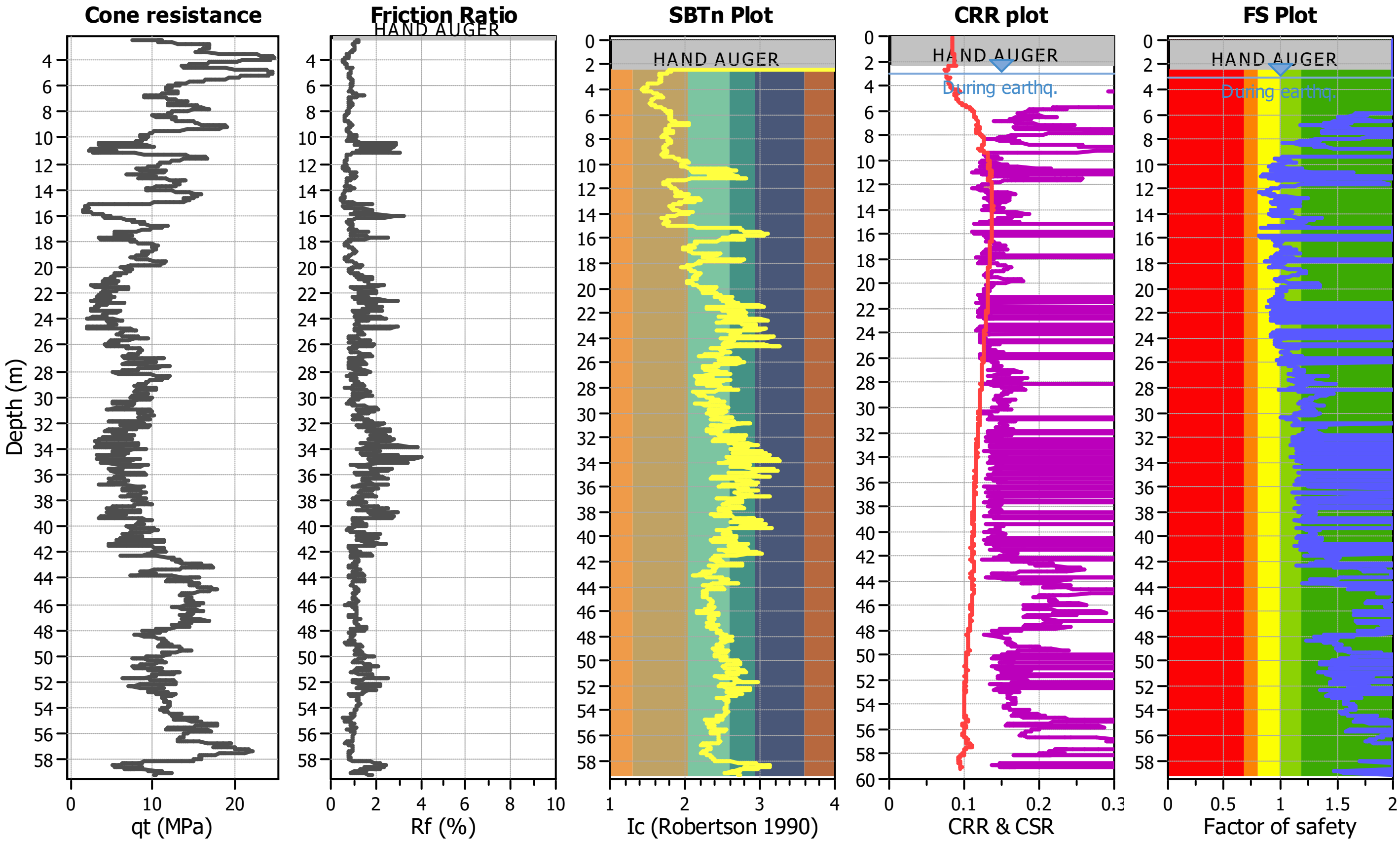
## LIQUEFACTION ANALYSIS REPORT

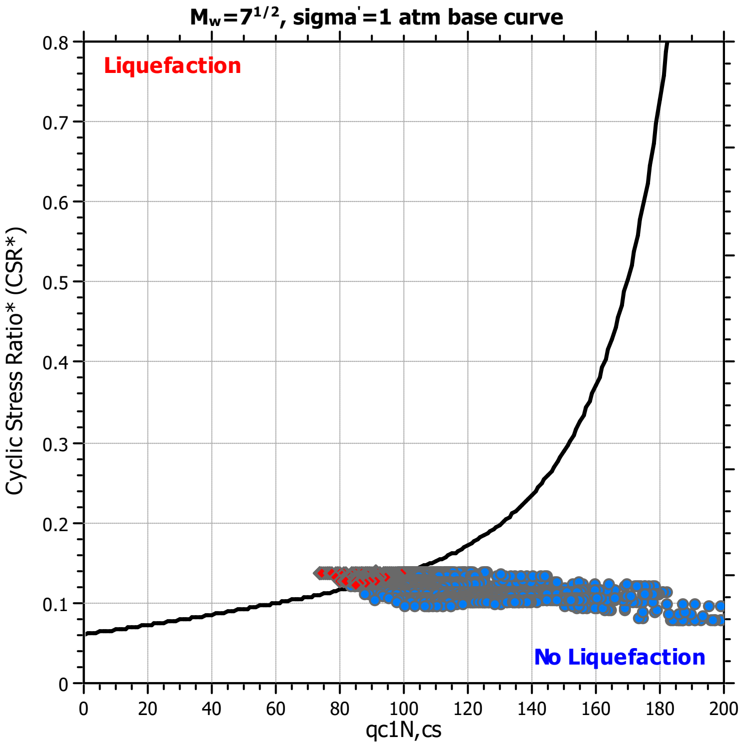
**Project title : Westshore- New Cargo Project** **Location : Delta, BC**

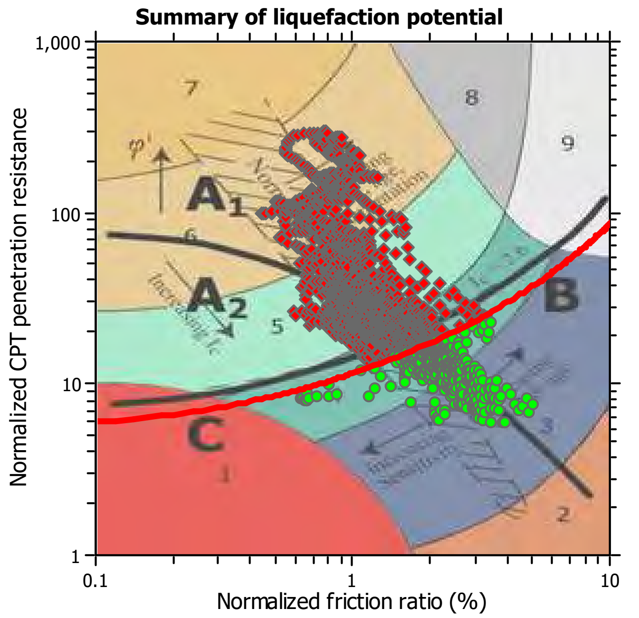
**CPT file : CPT20-13**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.10 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

# Liquefaction Triggering Assessment

# A475 Interface motions

BRAUN GEOTECHNICAL LTD.

Naesgaard-Amini Geotechnical Ltd. soils, foundations and earthquake engineering

**Braun Geotechnical Ltd.**

GEOLOGISMIKI Geotechnical Software

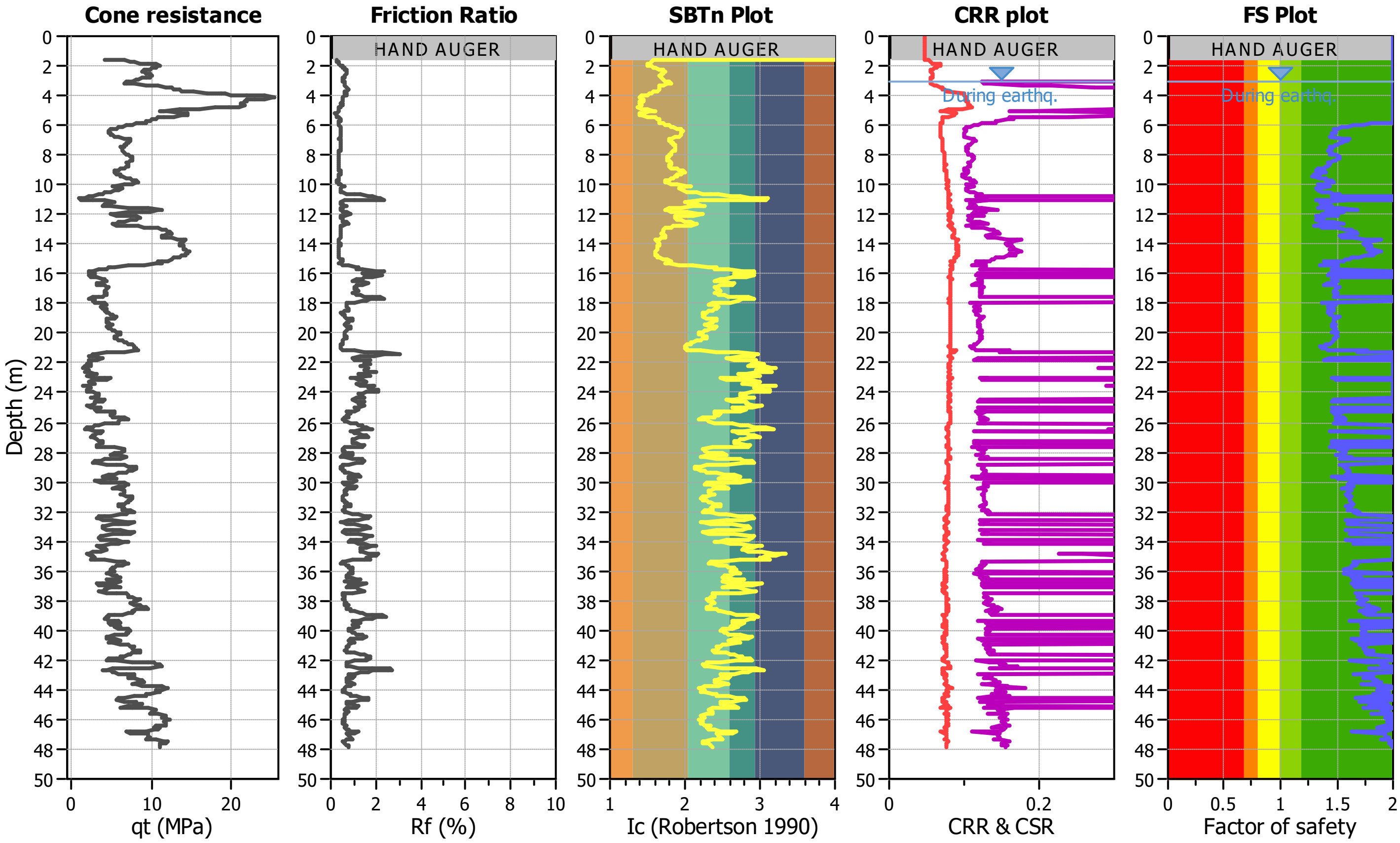
## LIQUEFACTION ANALYSIS REPORT

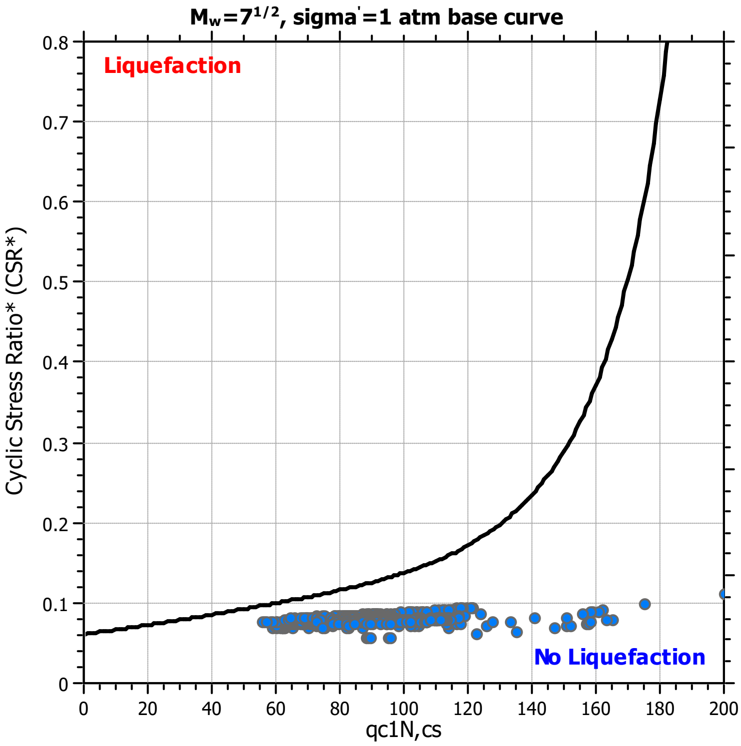
**Project title :** **Location :**

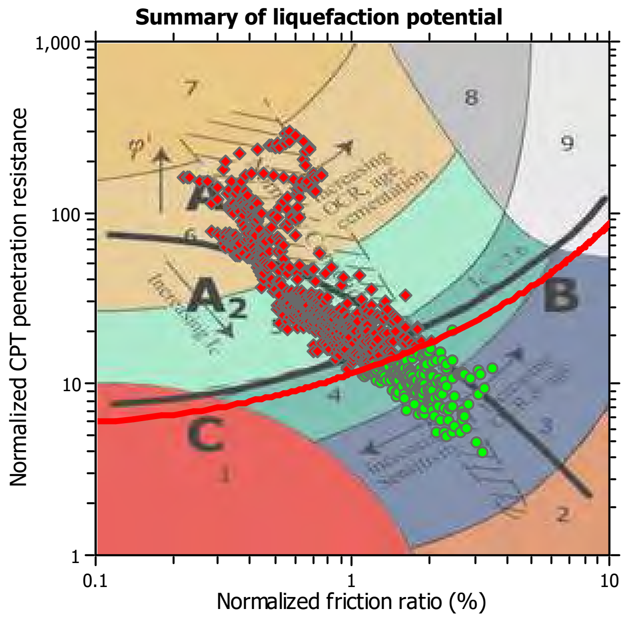
**CPT file : CPT18-01 Berth 2**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Braun Geotechnical Ltd.

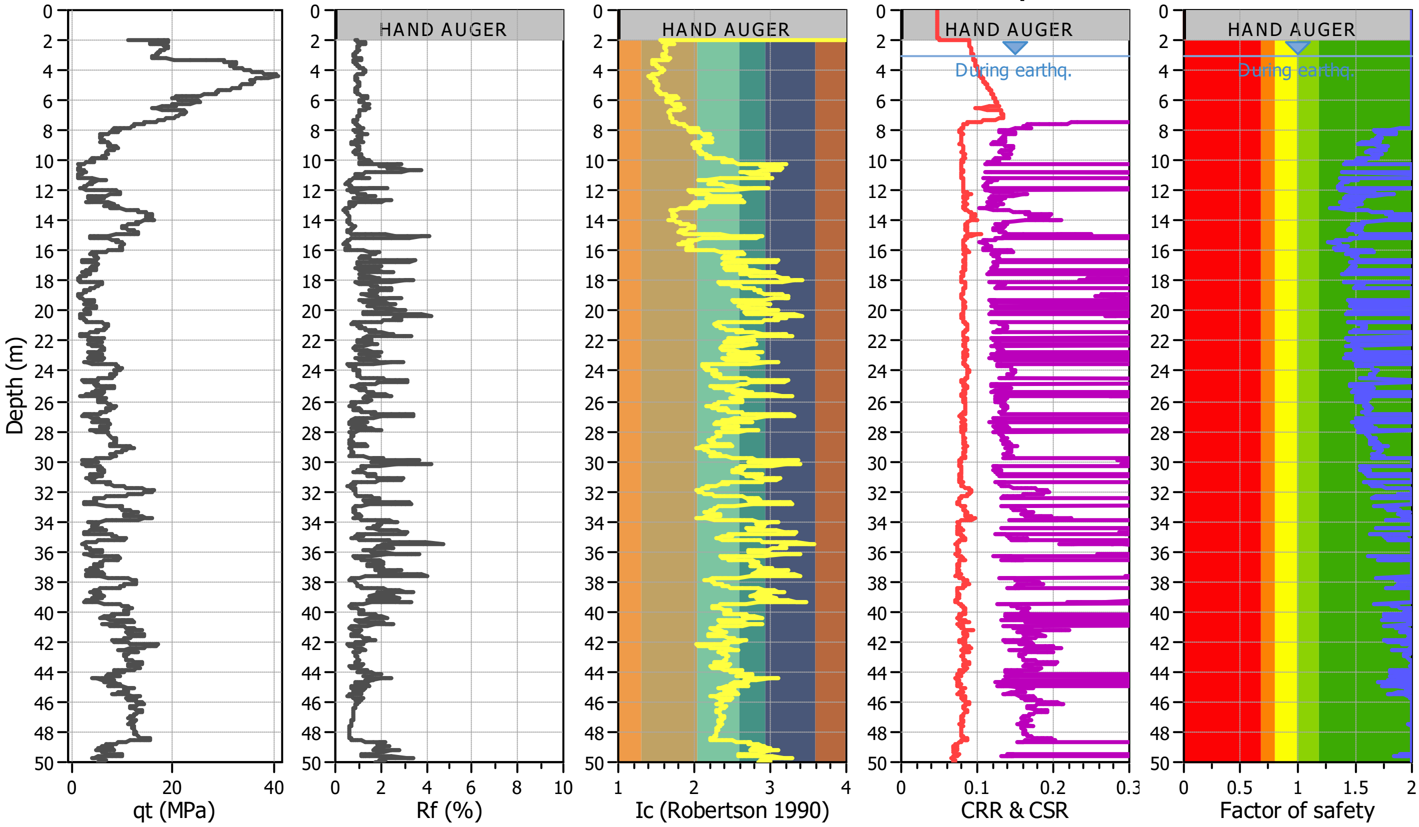
## LIQUEFACTION ANALYSIS REPORT

**Project title :** **Location :**

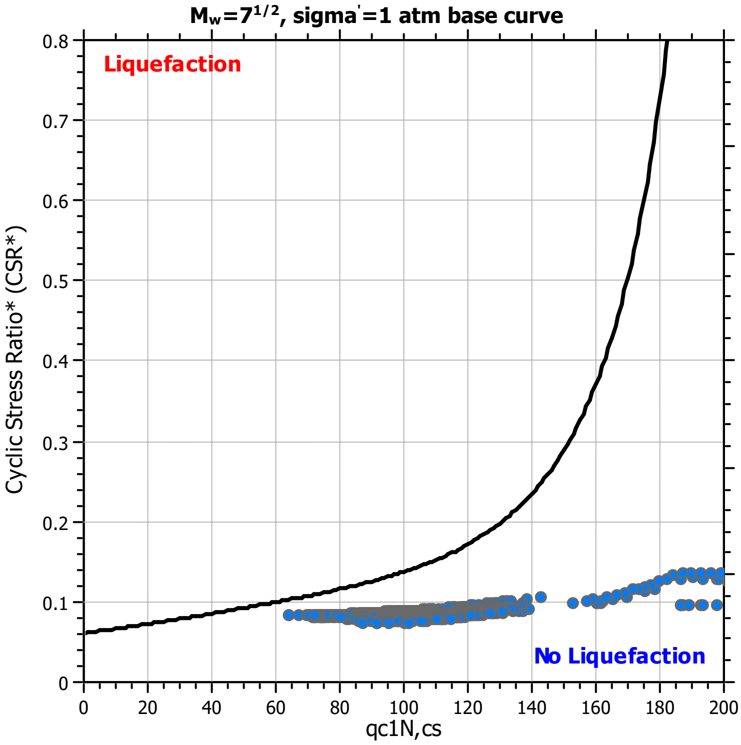
**CPT file : SCPT20-01**

### Input parameters and analysis data

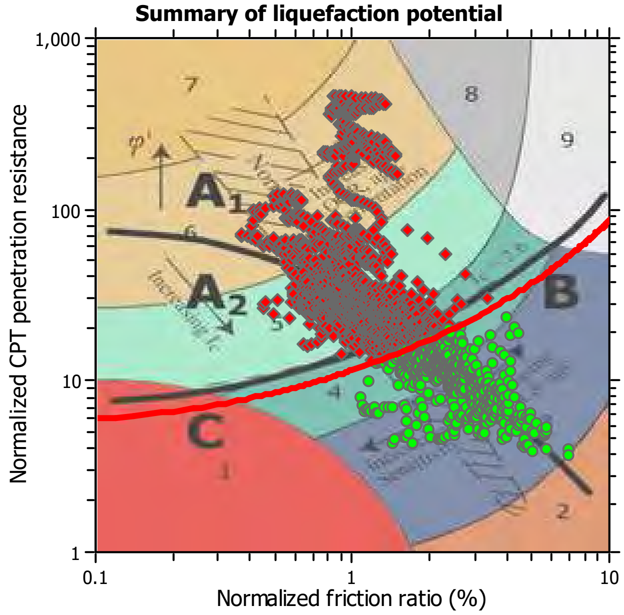
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




$M_w=7^{1/2}$, sigma'=1 atm base curve




**Summary of liquefaction potential**




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Braun Geotechnical Ltd.

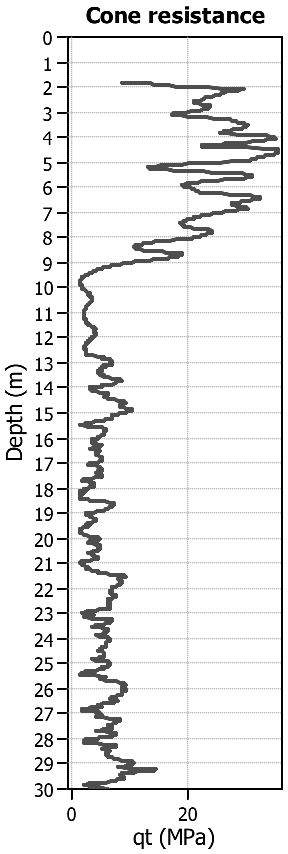
## LIQUEFACTION ANALYSIS REPORT

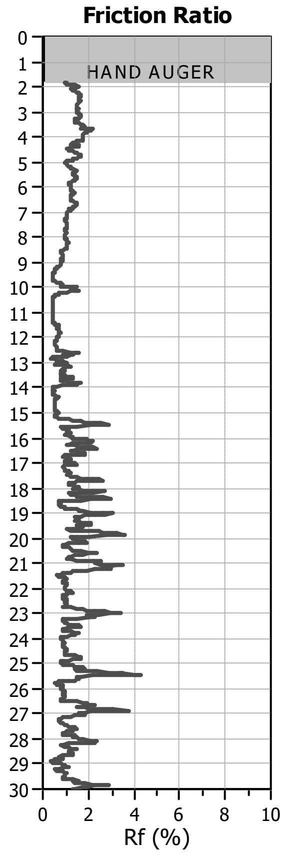
**Project title :** **Location :**

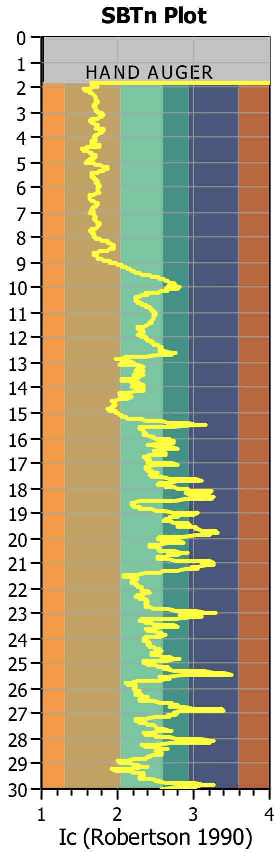
**CPT file : CPT20-02**

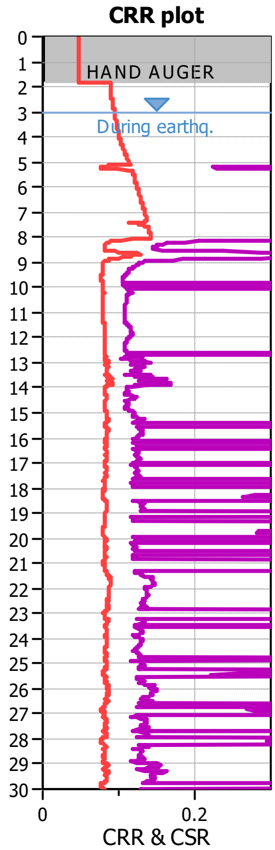
### Input parameters and analysis data

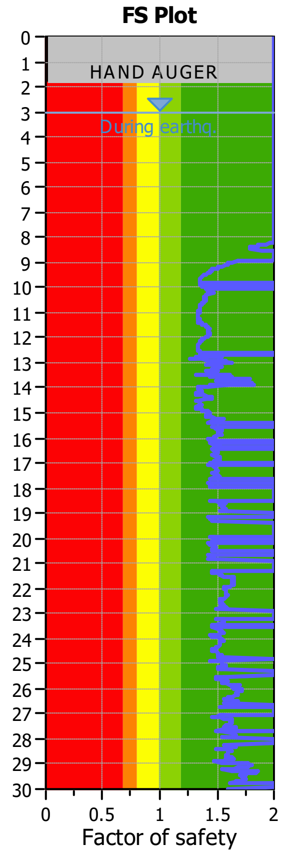
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | Yes |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | 30.00 m |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

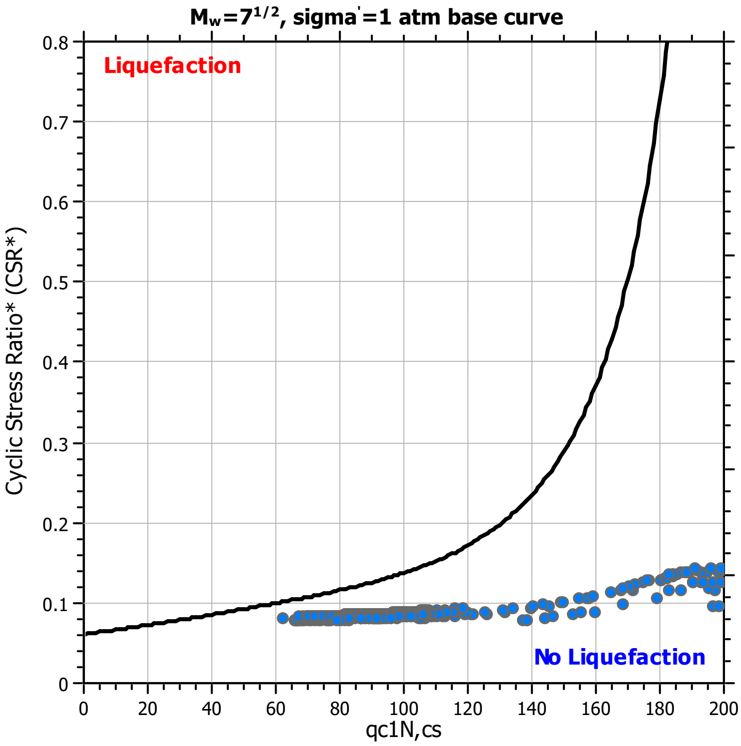


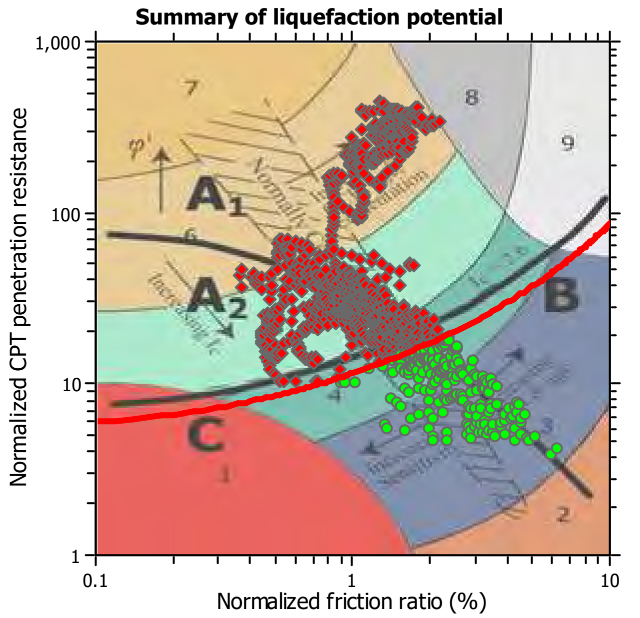




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

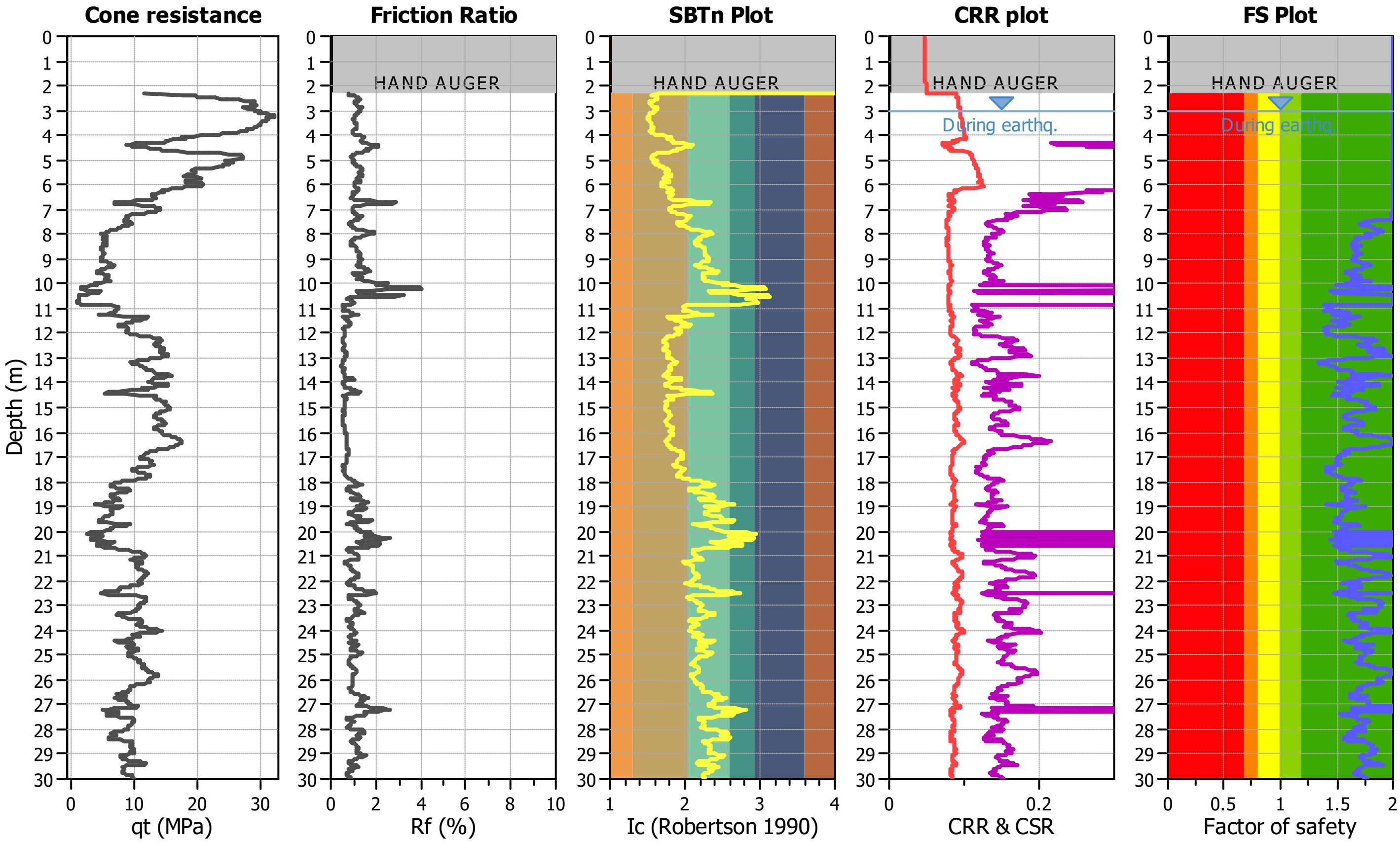
# LIQUEFACTION ANALYSIS REPORT

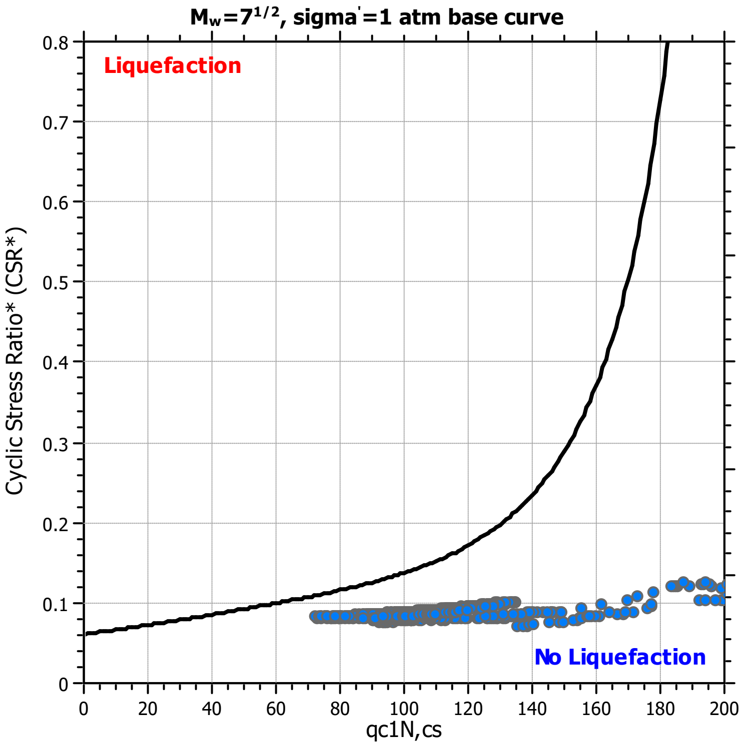
**Project title :** **Location :**

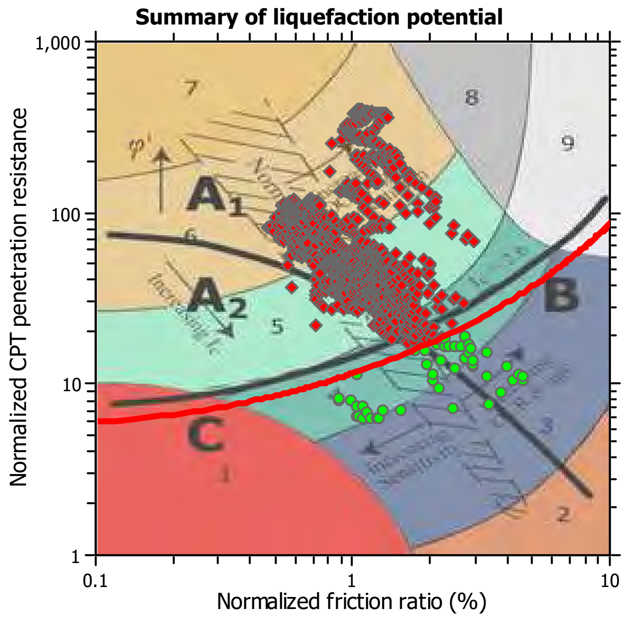
**CPT file : CPT20-03**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | Yes |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | 30.00 m |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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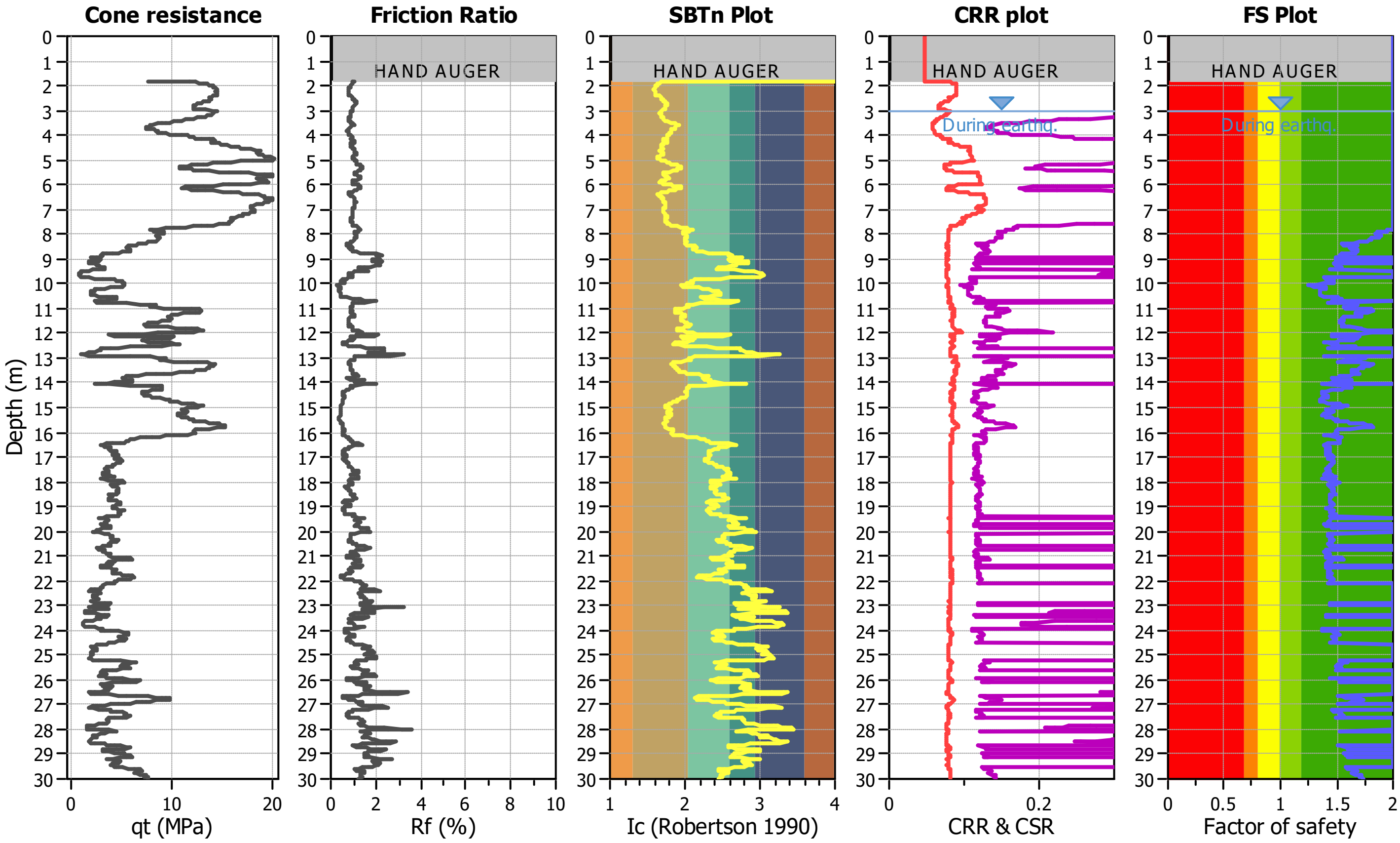
## LIQUEFACTION ANALYSIS REPORT

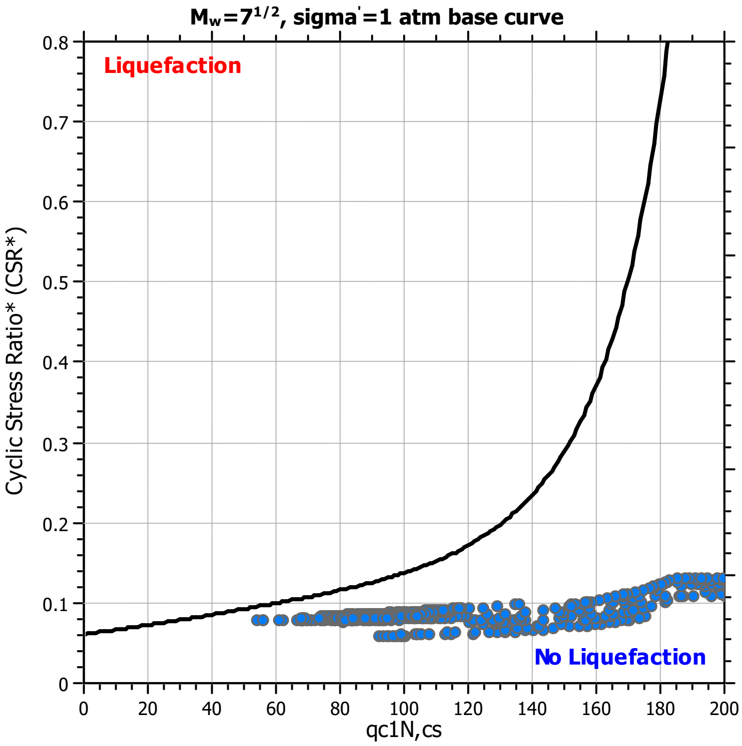
**Project title :** **Location :**

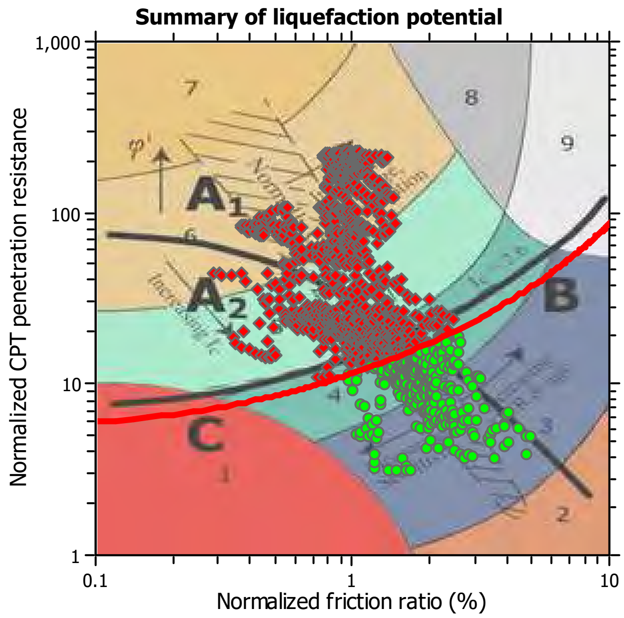
**CPT file : CP20-04**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | | |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | Yes |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | Limit depth: | 30.00 m |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | MSF method: | Method |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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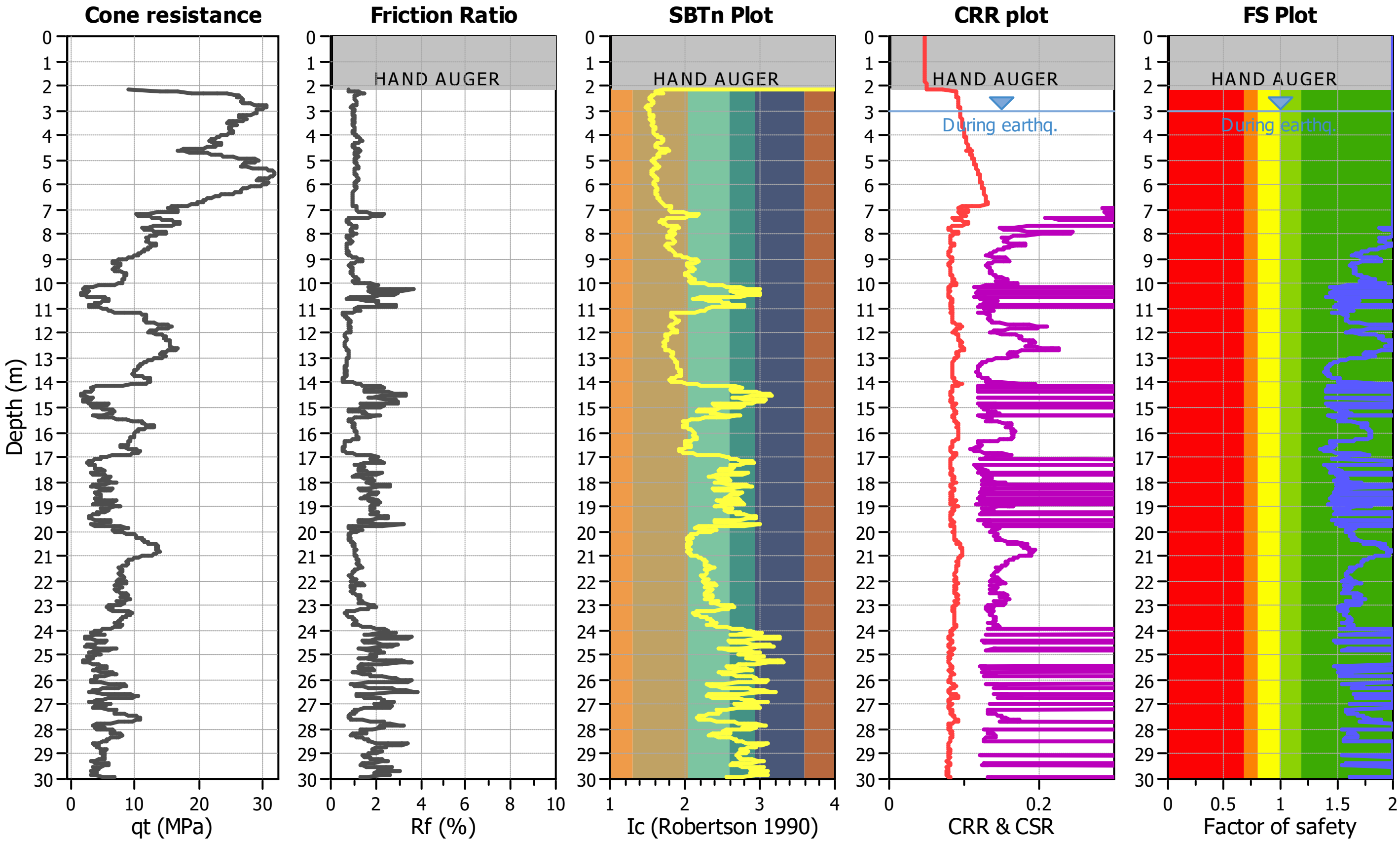
## LIQUEFACTION ANALYSIS REPORT

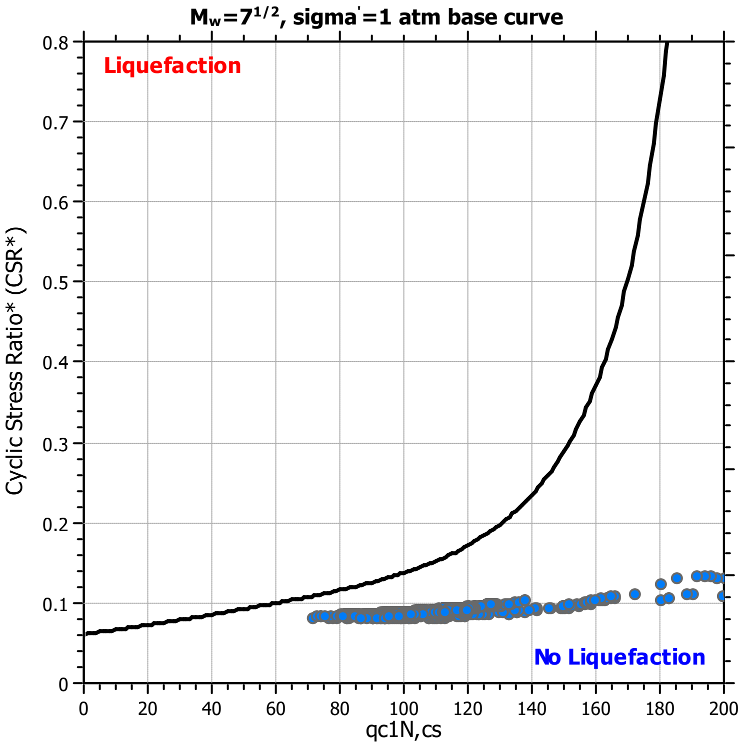
**Project title :** **Location :**

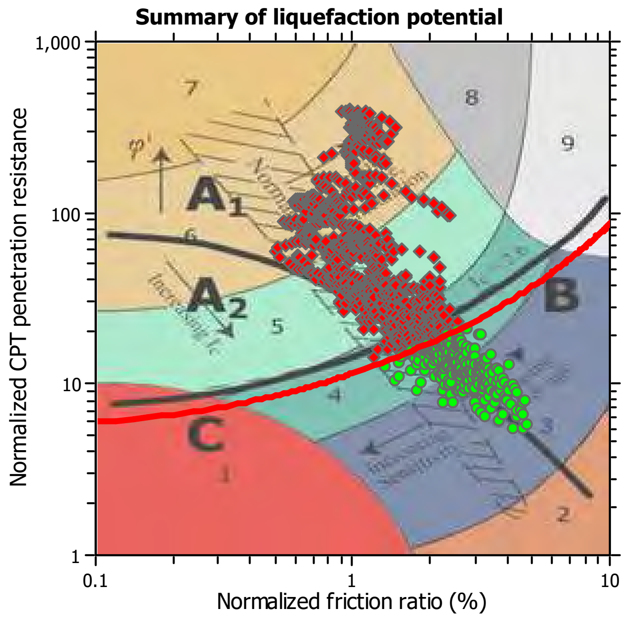
**CPT file : CPT20-05**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | | |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | Yes |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | Limit depth: | 30.00 m |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | MSF method: | Method |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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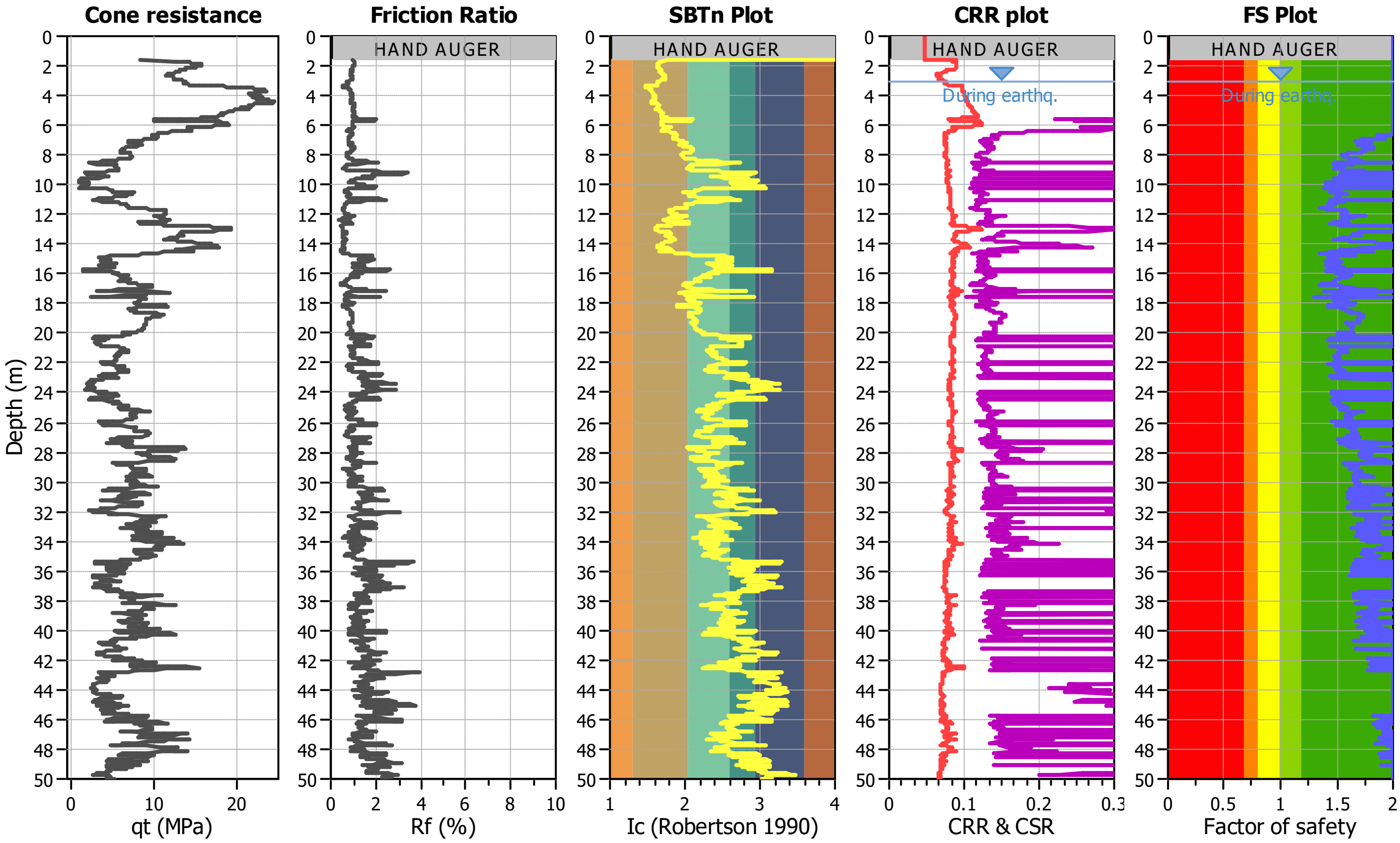
# LIQUEFACTION ANALYSIS REPORT

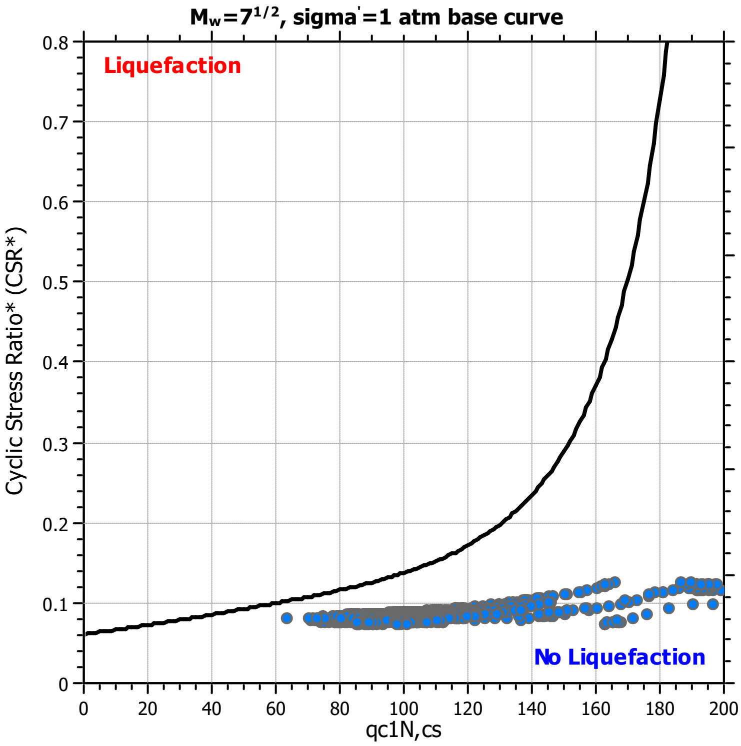
**Project title :** **Location :**

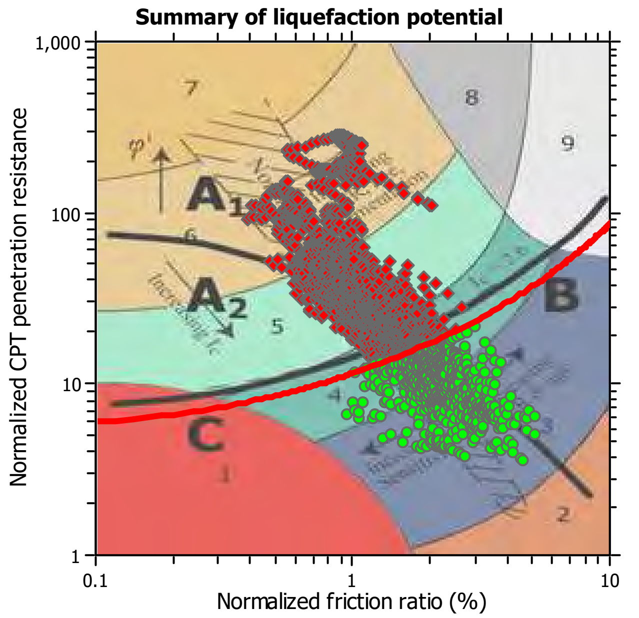
**CPT file : SCPT20-06**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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# LIQUEFACTION ANALYSIS REPORT

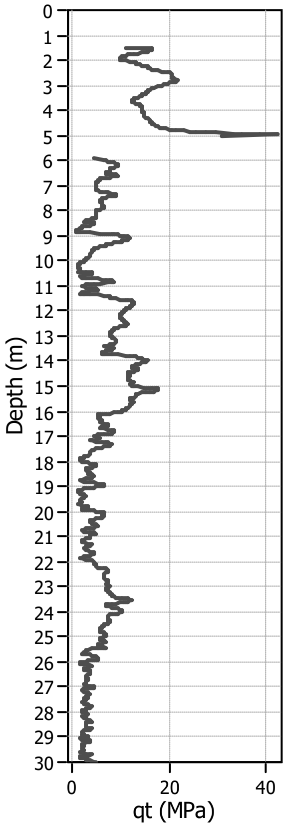
**Project title :** **Location :**

**CPT file : CPT20-08**

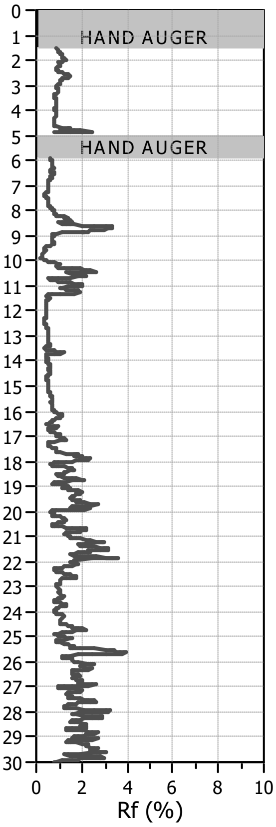
## Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | Yes |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | 30.00 m |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

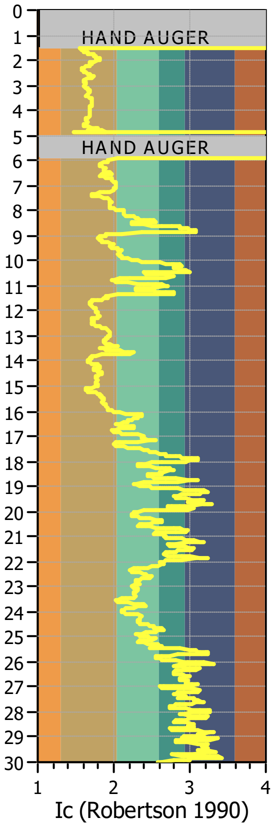
**Cone resistance**



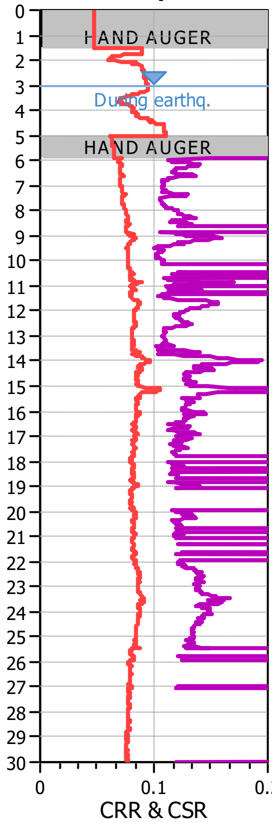

**Friction Ratio**



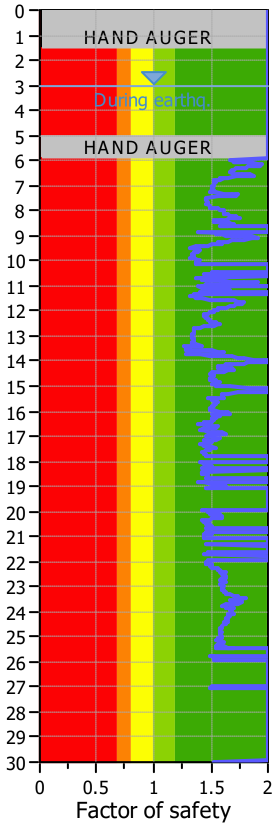

**SBTn Plot**



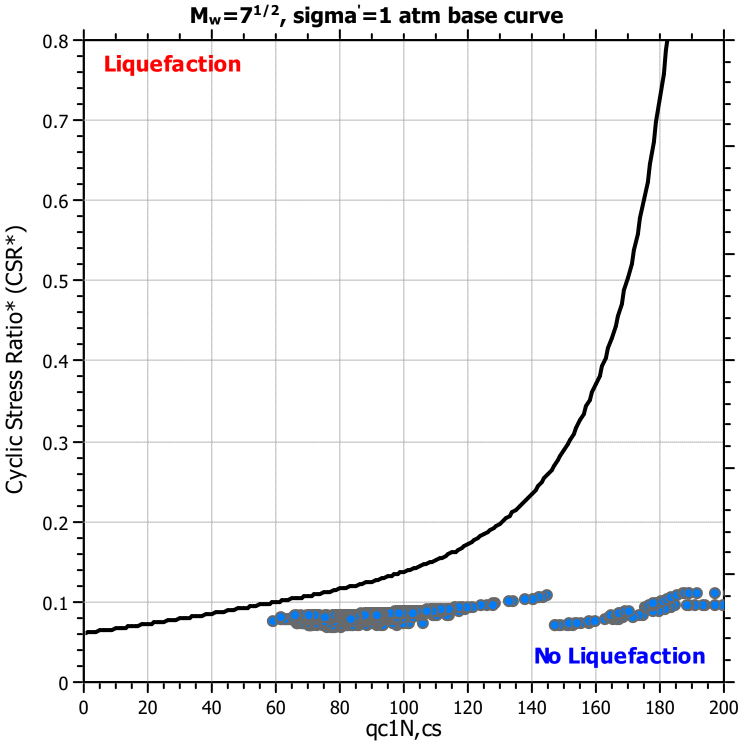

**CRR plot**



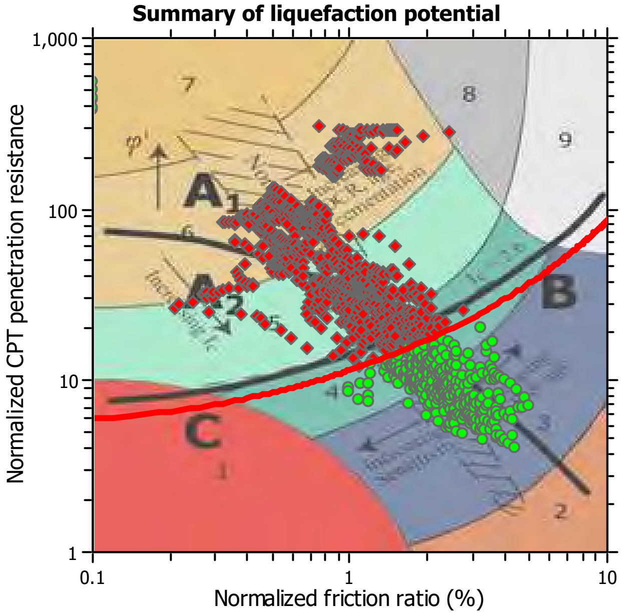

**FS Plot**




**$M_w=7^{1/2}$, sigma'=1 atm base curve**




**Summary of liquefaction potential**




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

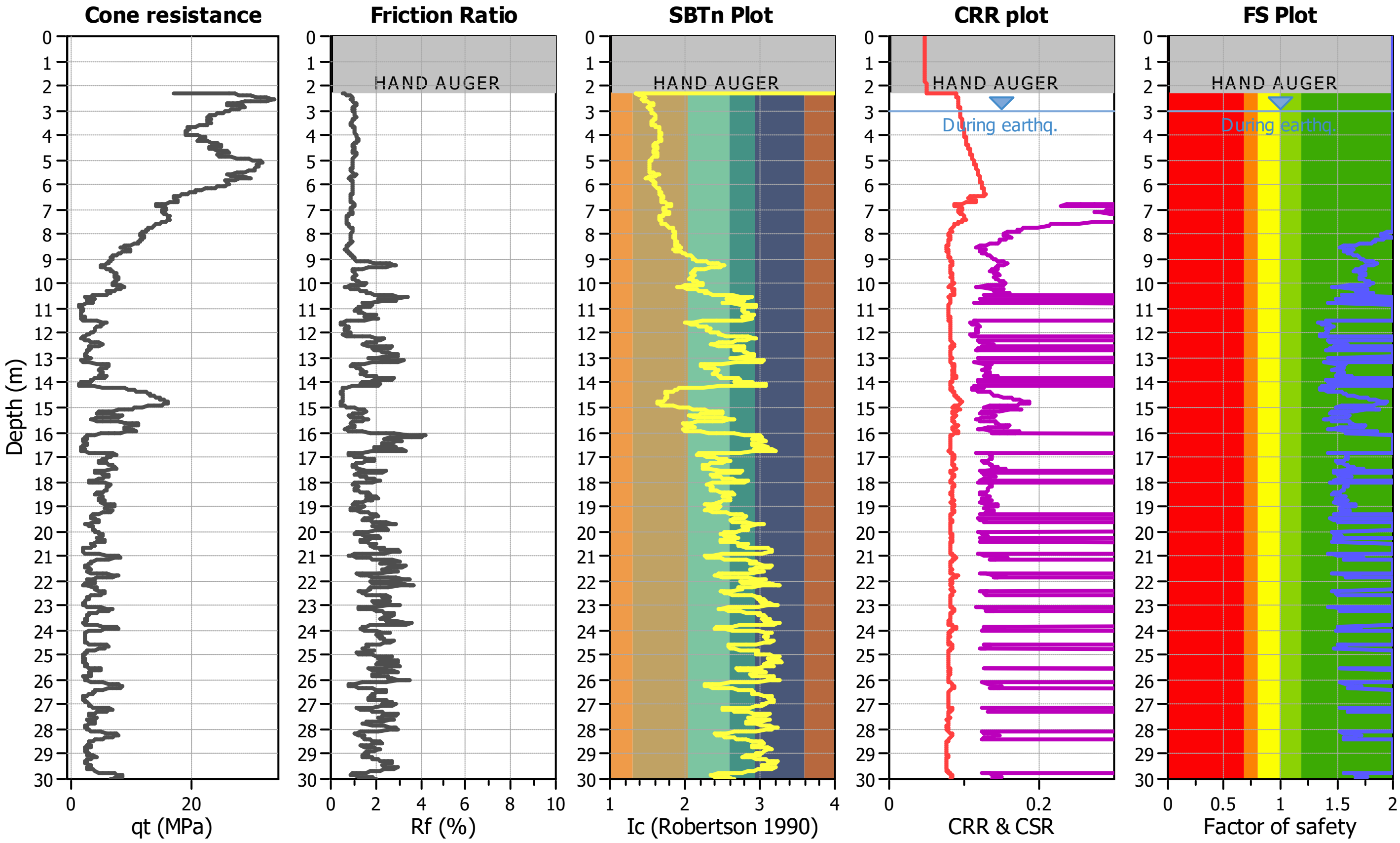
# LIQUEFACTION ANALYSIS REPORT

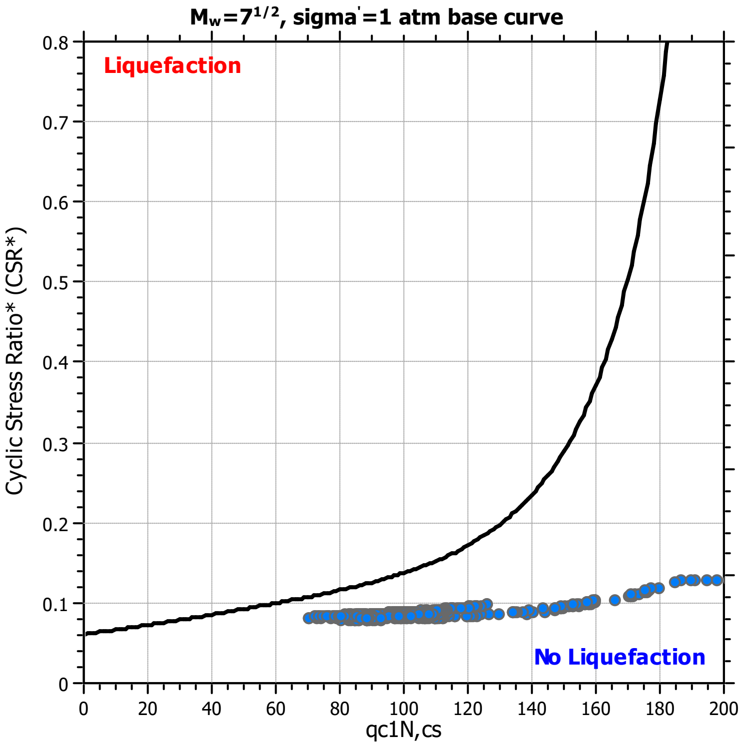
**Project title :** **Location :**

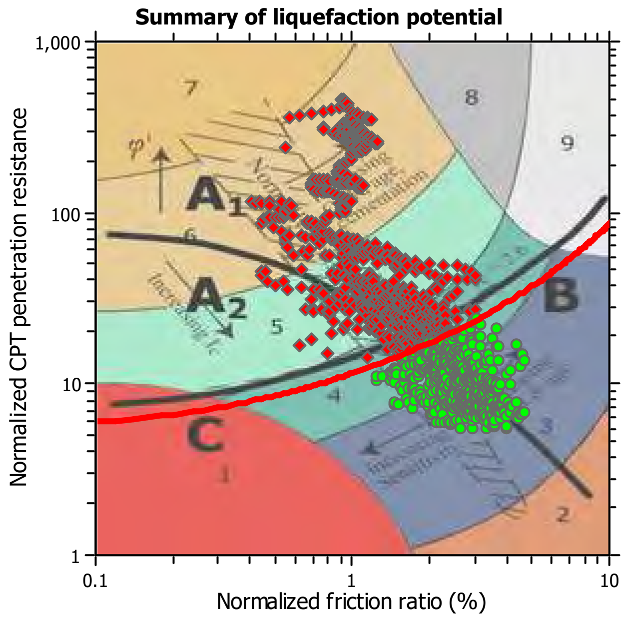
**CPT file : CPT20-07**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | Yes |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | 30.00 m |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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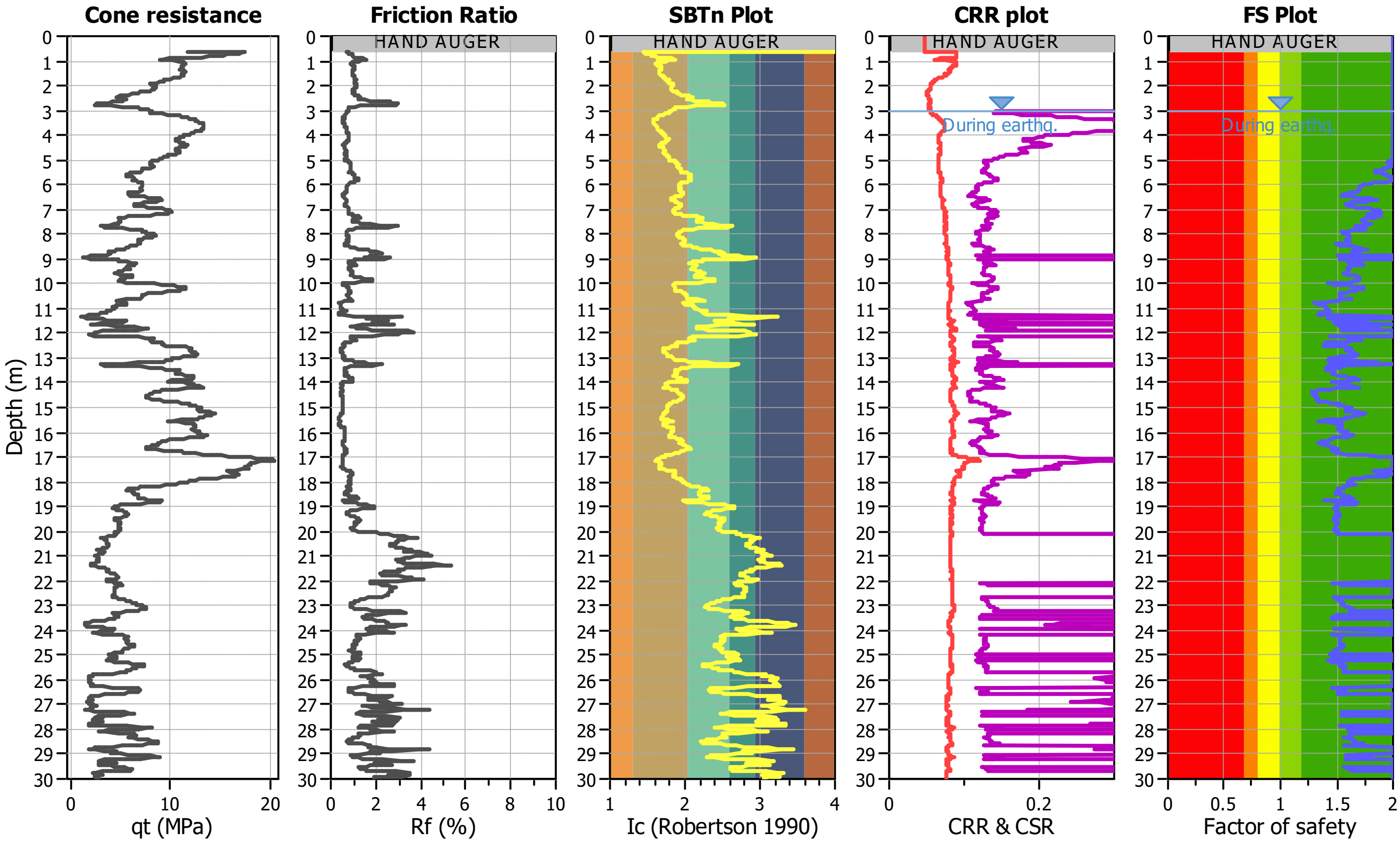
## LIQUEFACTION ANALYSIS REPORT

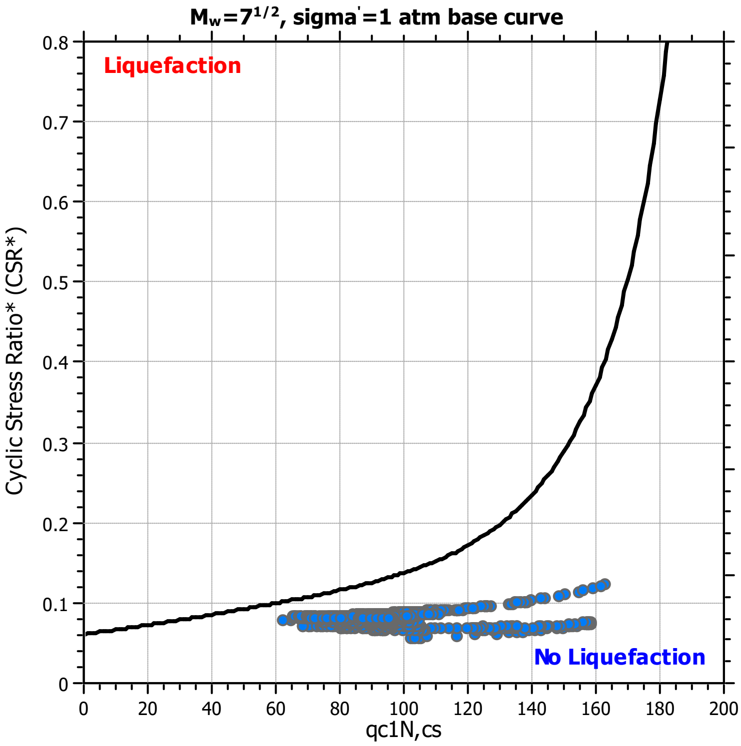
**Project title :** **Location :**

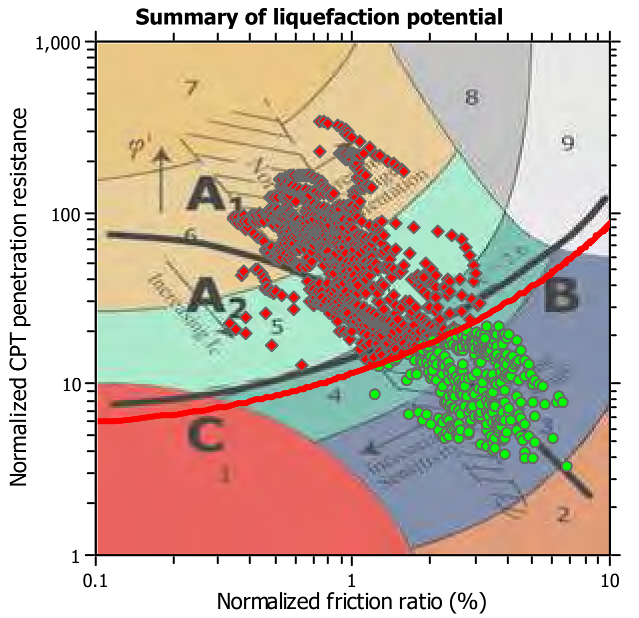
**CPT file : SCPT20-09**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | Yes |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | 30.00 m |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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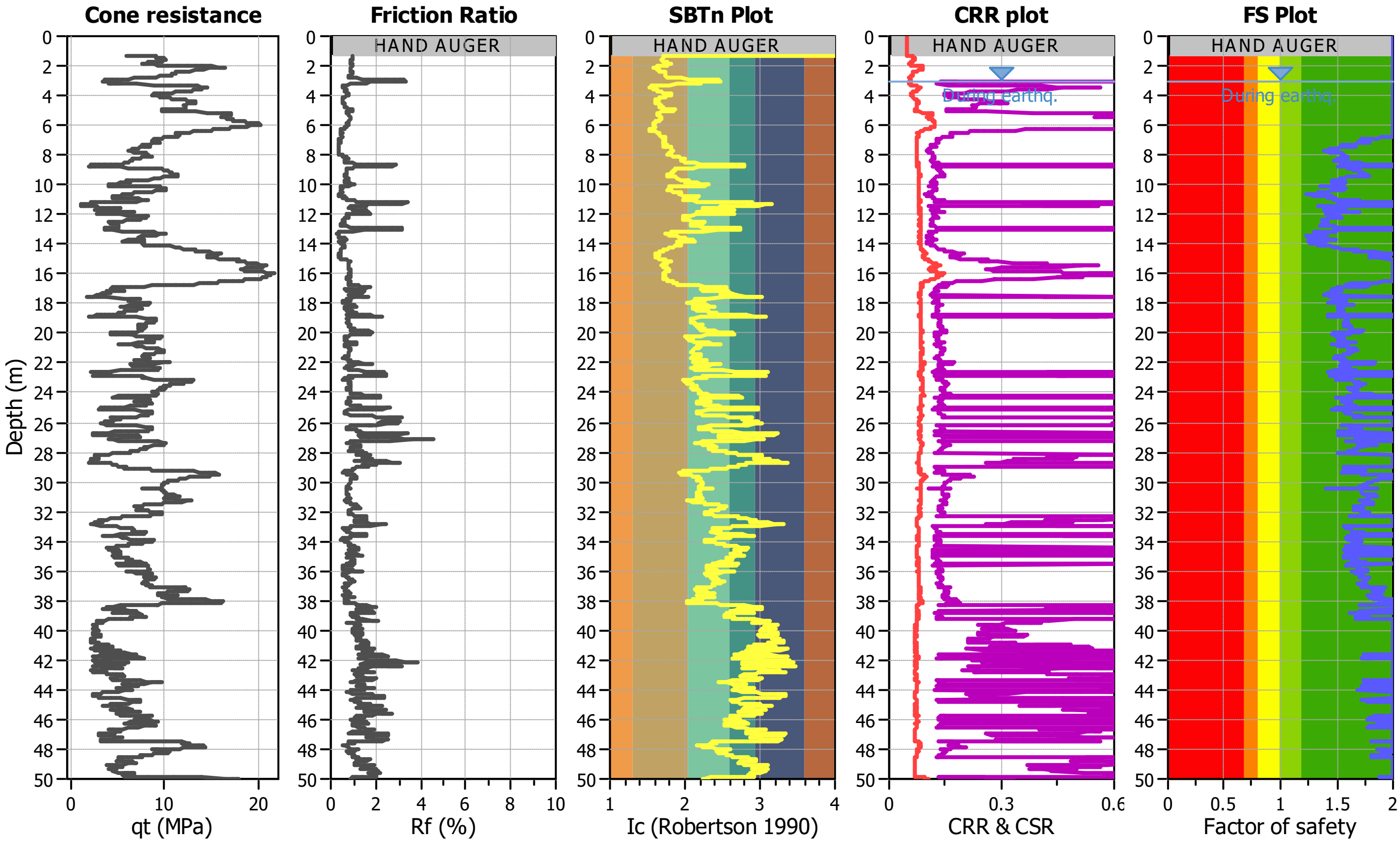
## LIQUEFACTION ANALYSIS REPORT

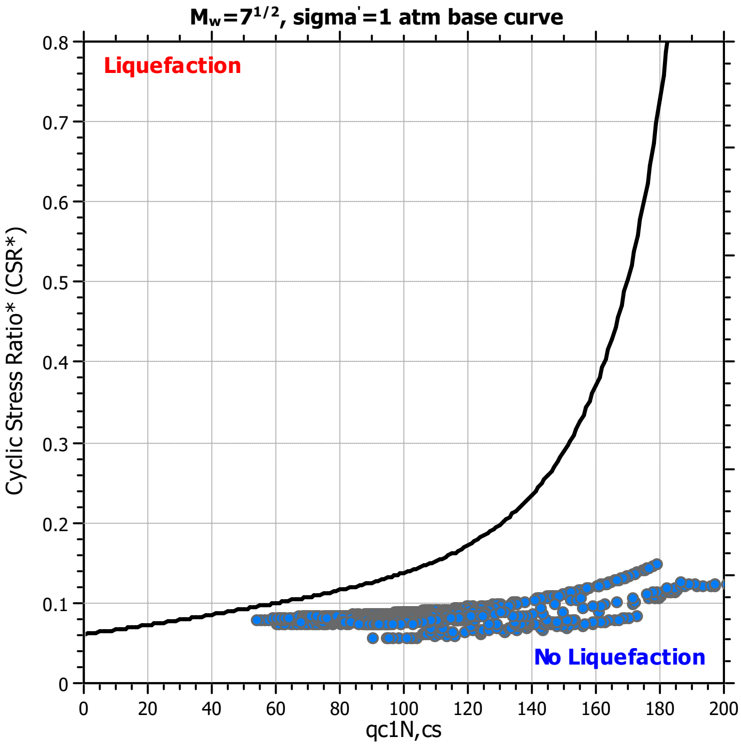
**Project title :** **Location :**

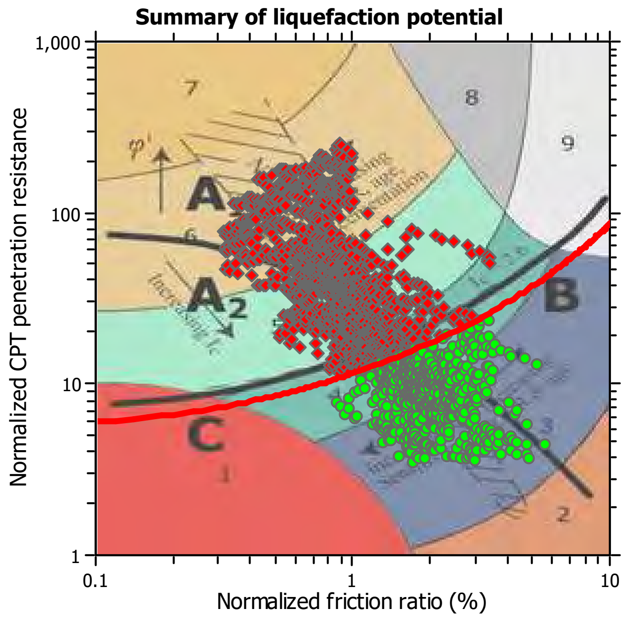
**CPT file : CPT20-10**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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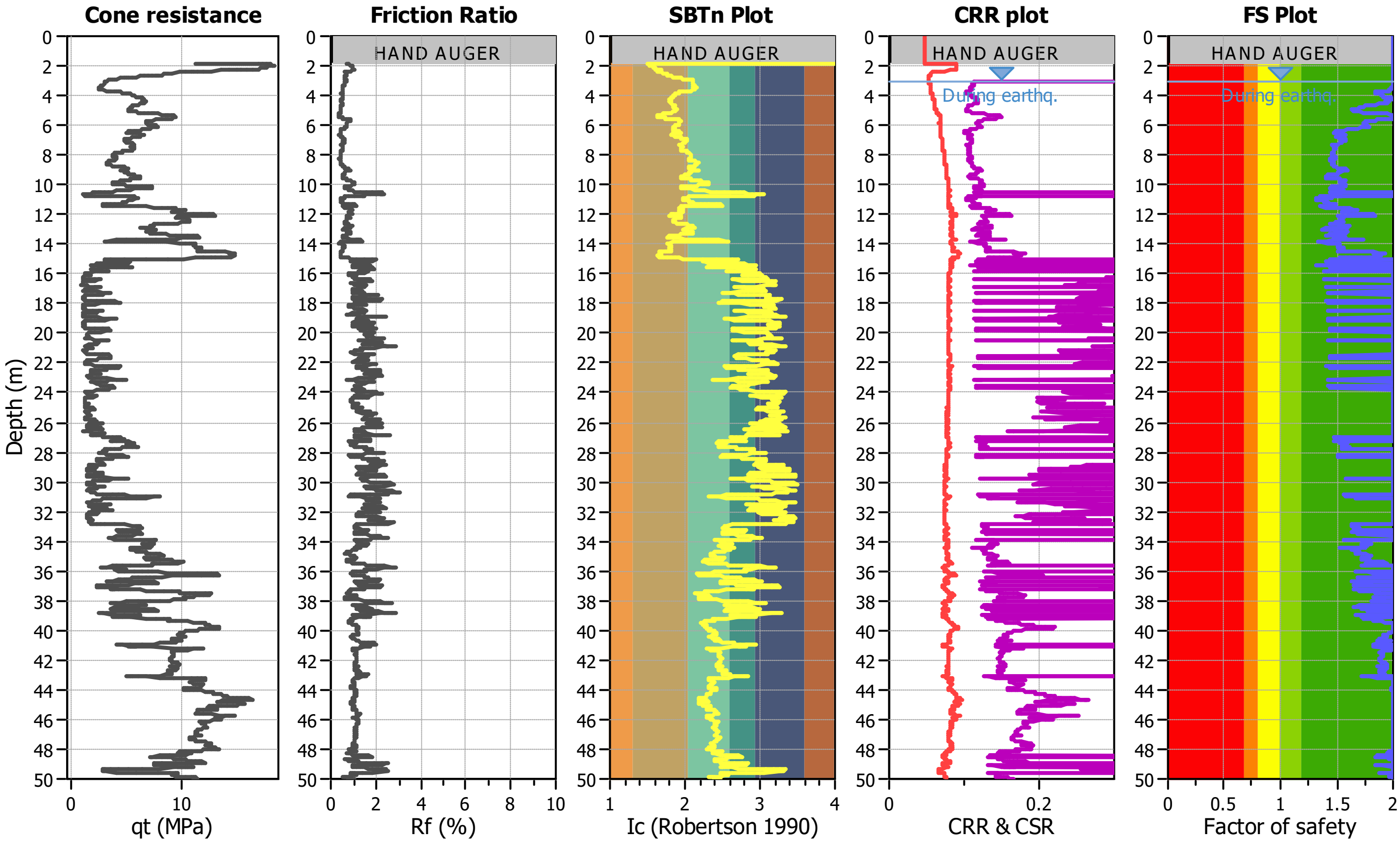
## LIQUEFACTION ANALYSIS REPORT

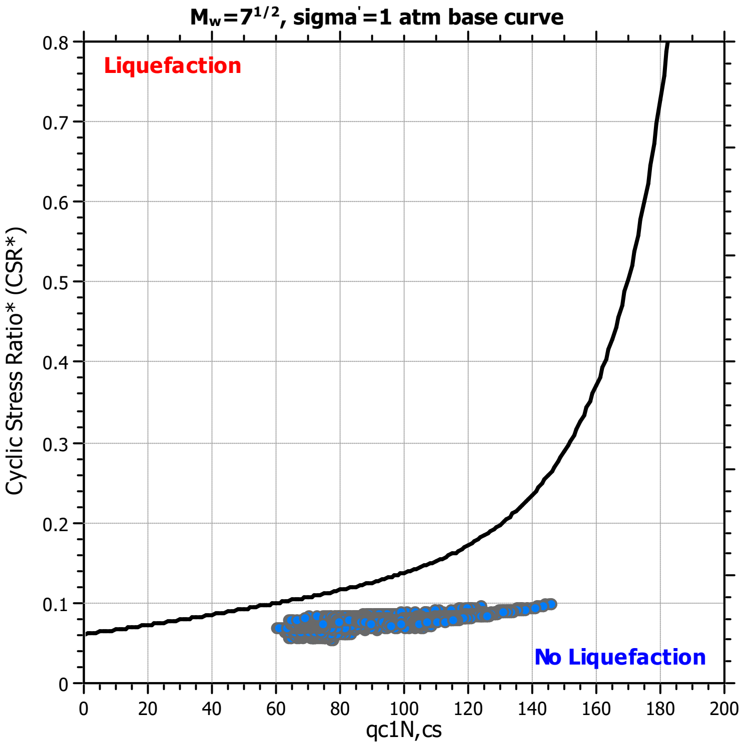
**Project title :** **Location :**

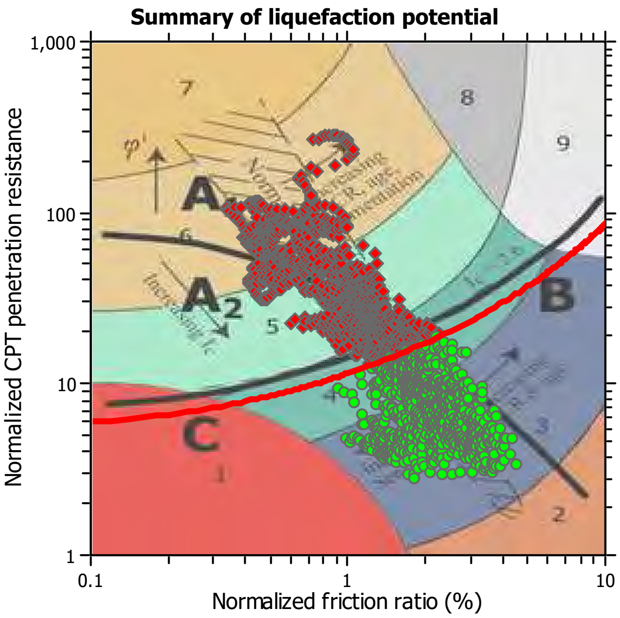
**CPT file : CPT20-11**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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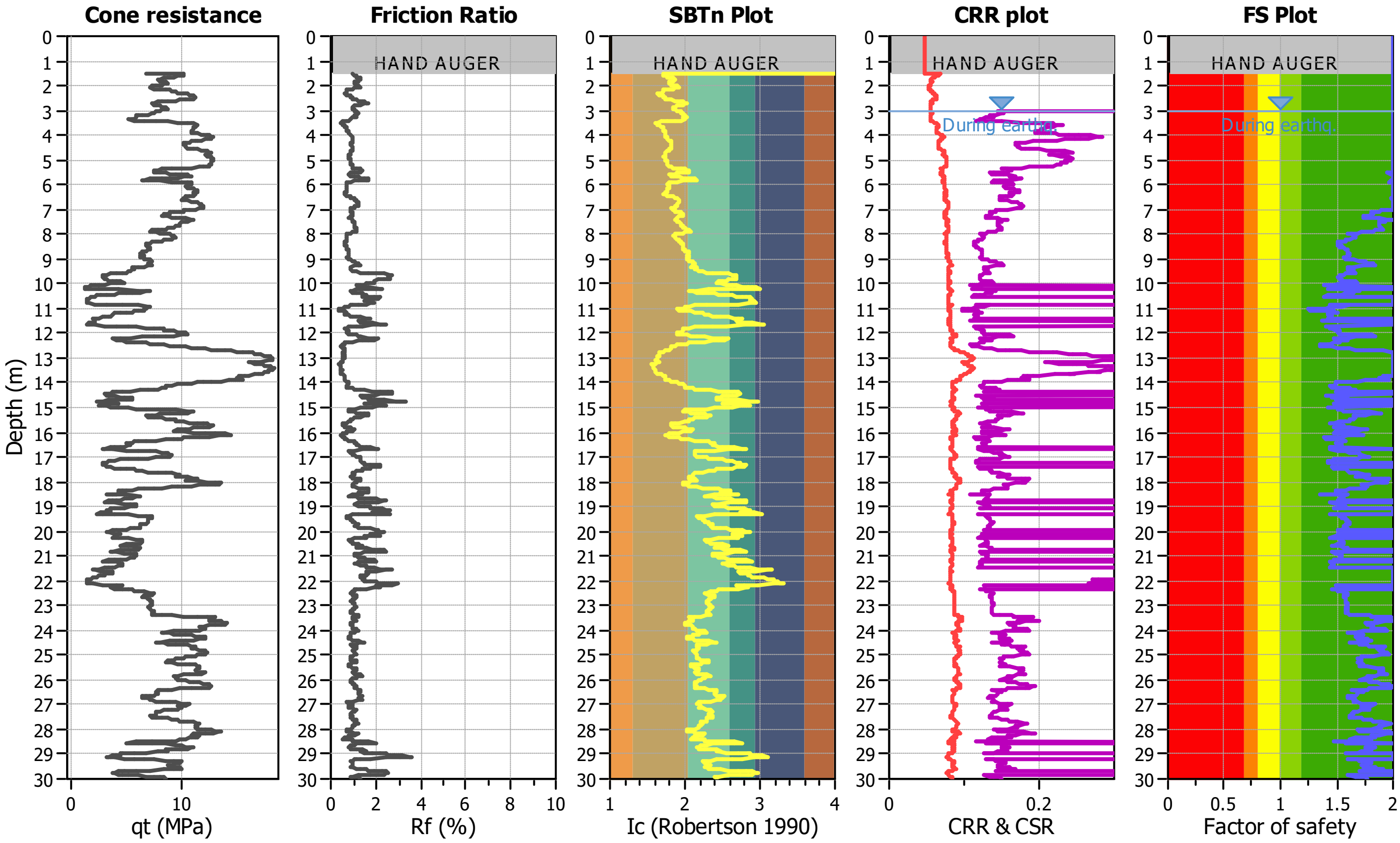
# LIQUEFACTION ANALYSIS REPORT

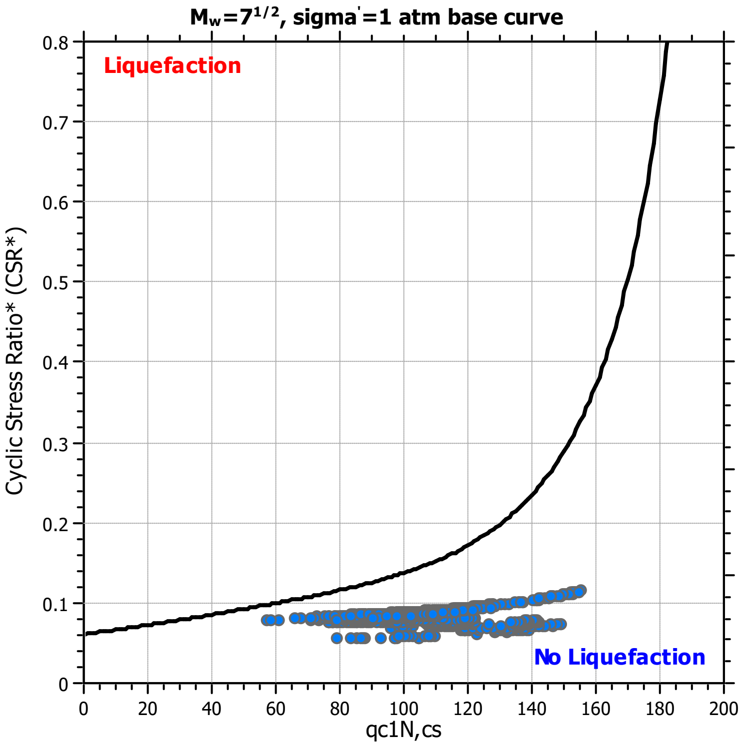
**Project title :** **Location :**

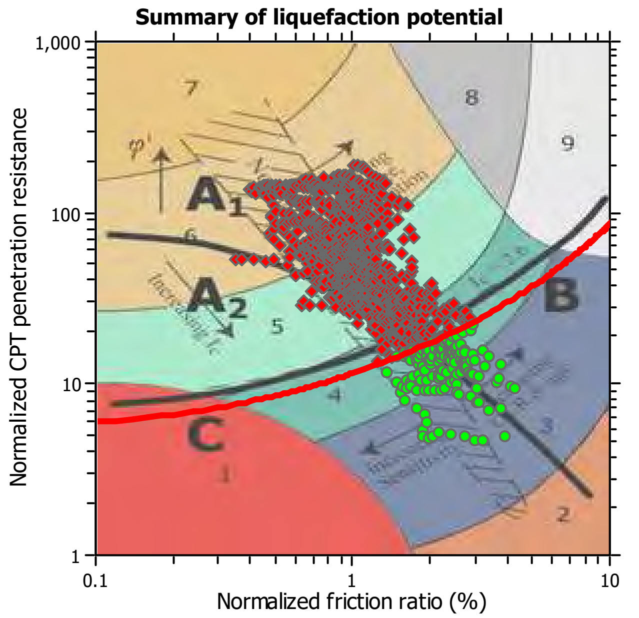
**CPT file : CPT20-12**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | | |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | Yes |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | Limit depth: | 30.00 m |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | MSF method: | Method |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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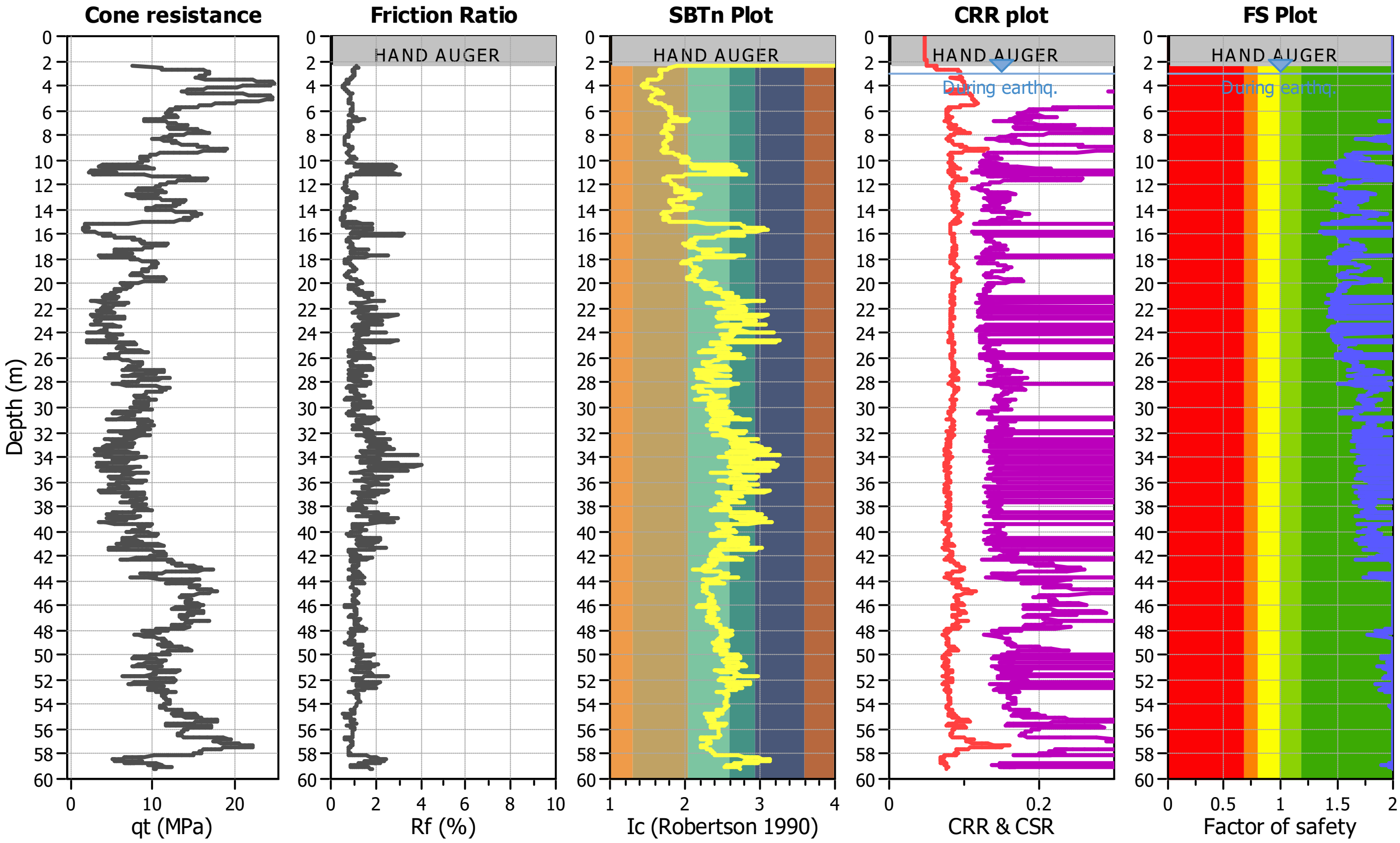
## LIQUEFACTION ANALYSIS REPORT

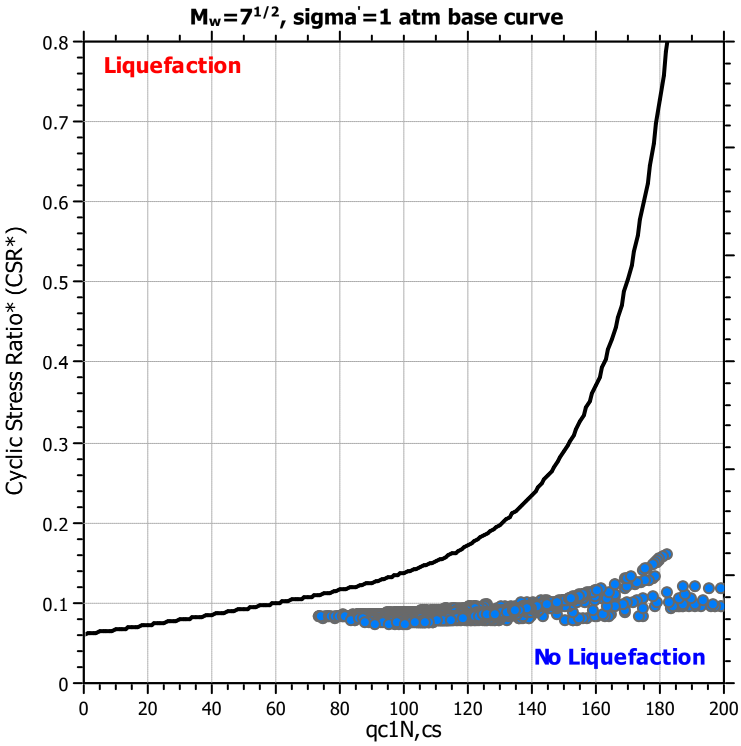
**Project title :** **Location :**

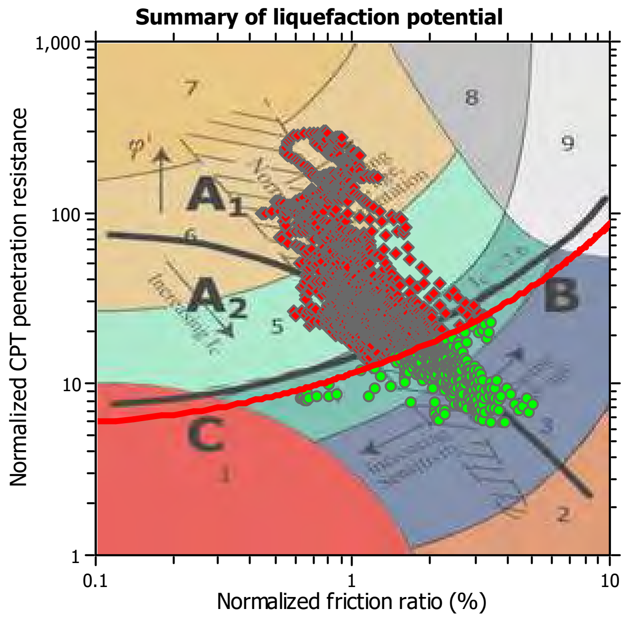
**CPT file : CPT20-13**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 3.00 m | Use fill: | No | Clay like behavior applied: | Sand & Clay |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 3.00 m | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 9.00 | Ic cut-off value: | 2.70 | Trans. detect. applied: | No | MSF method: | Method |
| Peak ground acceleration: | 0.29 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |




Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

# Appendix E

# Numerical Modeling (FLAC)

## APPENDIX E CONTENT:

Appendix E provides select details and information on FLAC analyses as follows:

- Methodology of analysis
- FLAC analysis cases for the E-W and N-S sections
- Assumed parameters for soils, structures and interface elements in tabulated format.
- Some details on the development of simplified layering, soil units and development of the normalized cone tip resistance
- The soil mesh for the E-W and N-S sections
- Assumed ground water table and sea water level
- External hydrostatic pressure on the slope and sea floor
- Stress conditions before earthquake shaking
- Examples of profiles of soil parameters versus elevation
- Examples of cyclic behavior of soil elements during earthquake shaking

* For more information about the constitutive soil models PM4Sand and PM4Silts please refer to the provided referenced manuals

## GENERAL METHODOLOGY OF 2D FLAC DYNAMIC NUMERICAL ANALYSIS

The general procedure used for numerical analyses included the following steps in a chronological manner. After each step the model was brought to equilibrium.

1. Set up model mesh, soil units, material properties, hydrostatic pore water regime, apply water pressure on the offshore, and bring the model to static equilibrium using elastic and then Mohr-Coulomb constitutive models. In some cases, material stockpiles were included in the FLAC model. The storage building footings, tie rods, retaining walls and ground improvement; and a simplified dumper pit concrete box in some of the N-S FLAC models.

2. Switch the soil model to non-linear effective constitutive soil models:
   - Use PM4SAND for sand-like soils with default calibration factors and site specific Gmax (small strain shear modulus) based on measured shear wave velocity data. Slight calibration adjustments were made for using a reduced fluid modulus of 5e8 Pa instead of 2e9 Pa. The reduced fluid modulus was used to reduce analysis run time.
   - Use PM4SILT for clay-like soils with default calibration factors.
   - A set of parametric analyses with four scenarios for inclusion of silts layers or their permeabilities in the FLAC model were performed for N-S FLAC section. In some scenarios silty layers modeled as PM4 SILT model and in some others modeled as PM4SAND model.

3. Turn the flow on and change to dynamic mode with large strain, multi-stepping, nominal 0.5% nominal Rayleigh damping and run for a few seconds with zero excitation.

4. Set the displacements to zero and apply the earthquake horizontal excitation in (and vertical directions in one analyses) using the compliant base method. Solve to the end of earthquake.

5. Check post-earthquake stability by switching the soil strength to the Idriss and Boulanger lower bound residual strength for liquefied granular soils. The pore pressure at the end of earthquake were maintained in the model in the non-liquefied granular soil. Continue the analysis in dynamic mode until the model comes to a stable geometry (if possible). Some elements with bad geometry error (excessive distortion) were changed into elastic model to be able to continue the analysis.

6. Compile and summarize the results at the end of earthquake and post-earthquake conditions.

Table E-1: Selected FLAC analysis cases, E-W Section

| Desciptions | FLAC Analysis Case | EQ Return period | Ground Motion | Ground Improve ment | Coal Stockpile | Su/s'v Deep Clay |
|---|---|---|---|---|---|---|
| **Base Case** | | | | | | |
| Base case- free field- No GI, No stock pile, Water table W.T.1 West slope 1 (Fig. E-9) | WS-23 | 2475 | CRU03 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | CRU01 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | CRU02 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | CRU04 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | CRU05 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | INS01 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | INS02 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | INS03 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | INS04 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | INS05 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | INTF01 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | INTF02 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | INTF03 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | INTF04 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 2475 | INTF05 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 200 | CRU03 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 475 | CRU03 | (-) | (-) | 0.22 |
| As base case but change ground motion | WS-23 | 975 | CRU03 | (-) | (-) | 0.22 |
| **Sensitivity, water table, stockpile, west slope, clay Su** | | | | | | |
| As base case but Water table W.T.2 & West slope 2 (Fig. E-9) | WS-41 | 2475 | CRU03 | (-) | (-) | 0.22 |
| As base case but Water table W.T.2, add Coal stockpile & West slope 2 (Fig. E-9) | WS-44 | 2475 | CRU03 | (-) | Yes | 0.3 |
| As base case but add Coal stockpile & West slope 2 (Fig. E-9) | WS-49 | 2475 | CRU03 | (-) | Yes | 0.3 |
| As base case but increase Su of deep clay | WS-24 | 2475 | CRU03 | (-) | (-) | 0.3 |
| **As base case but add soil mixing for the berth 2 area** | | | | | | |
| As base case but add soil mixing 35m W x 35m H | WS-25 | 2475 | CRU03 | Yes | (-) | 0.22 |
| As above but add soil mixing 35m W x 40m H | WS-26 | 2475 | CRU03 | Yes | (-) | 0.22 |
| As above but add soil mixing 35m W1=51m, W2=7m H=38m | WS-27 | 2475 | CRU03 | Yes | (-) | 0.22 |
| As above but add As but with shear key 7 x 10m below El. -30m | WS-28 | 2475 | CRU03 | Yes | (-) | 0.22 |
| As above bu double soil mix strength | WS-29 | 2475 | CRU03 | Yes | (-) | 0.22 |
| As above but taper soil mix (shape of Christmass Tree) | WS-30 | 2475 | CRU03 | Yes | (-) | 0.22 |

WESTSHOTRE- FLAC cases.xlsx

Table E-2: Selected FLAC analysis cases, N-S Section

| Desciptions | FLAC Analysis Case | Soil Profile Type | Return period | GM | Vertical Motion | Structure | Potash Stock pile | Coal stock pile | Tie Rod | Ground Improvement |
|---|---|---|---|---|---|---|---|---|---|---|
| **Free Field cases with 4 different scenarios of inclusion of silt interlayers** | | | | | | | | | | |
| Silt is included with PM4Silt default calibration factors for Silt, Su/s'v=0.27 | WS-N-S-12 | 1 | 2475 | CRU03 | (-) | (-) | (-) | (-) | (-) | (-) |
| Silt is included with PM4Silt calib. factors for Fraser Delta low PI Silt, Su/s'v=0.27 | WS-N-S-13 | 2 | 2475 | CRU03 | (-) | (-) | (-) | (-) | (-) | (-) |
| Silt is not included but its low permeability is included | WS-N-S-14 | 3 | 2475 | CRU03 | (-) | (-) | (-) | (-) | (-) | (-) |
| All Sand assumption-Silt and its permeability not included- Base Case | WS-N-S-15 | 4 | 2475 | CRU03 | (-) | (-) | (-) | (-) | (-) | (-) |
| **As base case but add stockpiles, building footings & walls + ground motions** | WS-N-S-19R | 4 | 2475 | CRU03 | (-) | Yes | Yes | Yes | Yes | Yes |
| As above but interface motion | WS-N-S-19R | 4 | 2475 | INTF01 | (-) | Yes | Yes | Yes | Yes | Yes |
| As above but interface motion | WS-N-S-19R | 4 | 2475 | INTF02 | (-) | Yes | Yes | Yes | Yes | Yes |
| As above but interface motion | WS-N-S-19R | 4 | 2475 | INTF03 | (-) | Yes | Yes | Yes | Yes | Yes |
| As above but interface motion | WS-N-S-19R | 4 | 2475 | INTF04 | (-) | Yes | Yes | Yes | Yes | Yes |
| As above but interface motion | WS-N-S-19R | 4 | 2475 | INTF05 | (-) | Yes | Yes | Yes | Yes | Yes |
| AS 19R but add vertical motion | WS-N-S-22R | 4 | 2475 | CRU03 | Yes | Yes | Yes | Yes | Yes | Yes |
| **Parametric analysis for tie rods and ground improvement** | | | | | | | | | | |
| AS 19R but no coal stockplie | WS-N-S-20R | 4 | 2475 | CRU03 | (-) | Yes | Yes | (-) | Yes | Yes |
| AS 20R but no tie rod | WS-N-S-23 | 4 | 2475 | CRU03 | (-) | Yes | Yes | Yes | (-) | Yes |
| AS 20R but different tie rod stiffness | WS-N-S-24 | 4 | 2475 | CRU03 | (-) | Yes | Yes | Yes | Yes | Yes |
| AS 20R but different tie rod stiffness | WS-N-S-25 | 4 | 2475 | CRU03 | (-) | Yes | Yes | Yes | Yes | Yes |
| AS 20R but different tie rod stiffness | WS-N-S-26 | 4 | 2475 | CRU03 | (-) | Yes | Yes | Yes | Yes | Yes |
| AS 20R but different tie rod stiffness | WS-N-S-27 | 4 | 2475 | CRU03 | (-) | Yes | Yes | Yes | Yes | Yes |
| AS 20R but no ground improvement | WS-N-S-28 | 4 | 2475 | CRU03 | (-) | Yes | Yes | Yes | Yes | (-) |
| AS 20R but no ground improvement- no tie rod | WS-N-S-29 | 4 | 2475 | CRU03 | (-) | Yes | Yes | Yes | (-) | (-) |

WESTSHOTRE- FLAC cases.xlsx

# Table E-3: Assumed soil parameters

| SOIL UNIT No. | SOIL UNIT | SOIL MODEL | POROSITY | MOIST/SATUR. MASS DENSITY | FRICTION ANGLE | Su | qc1n_cs | Go | PERMEABILITY | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Vertical, $k_v$ | Horizontal, $k_h$ |
| | | | (-) | (kg/m$^3$) | (degrees) | (kPa) | (-) | (-) | (m/s) | (m/s) |
| 1 | Coal | Mohr-Coulomb | 0.5 | 880 | 40 | 0 | - | 400 | - | - |
| 2 | Potash | Mohr-Coulomb | 0.5 | 1264 | 36 | 0 | - | 400 | - | - |
| 3 | Fill | PM4Sand | 0.44-0.47 | 1900 to 1950 | 34-35 | 0 | (2) | (3) | 1E-04 to 5E-04 | 1E-04 to 5E-04 |
| 4 | SAND | PM4Sand | 0.44-0.57 | 1750 to 1950 | 34 | 0 | (2) | (3) | 1.0E-04 | 1.0E-04 |
| 5 | Silt & Sand and Silt | PM4Sand/PM4Silt | 0.44-0.58 | 1750 to 1950 | 33-34 | (1) | (2) | (3) | 1.0E-07 | 1.0E-7 to 1.0E-4 |
| 6 | Silty Clay/Clay | PM4Silt | 0.46 | 1935 | 0 | (1) | (2) | 460 | 1.0E-08 | 1.0E-08 |
| 7 | Till | Elastic | 0.5 | 2100 | - | - | - | (4) | - | - |

WESTSHORE-Soil Parameters.xlsx

**Notes:**

(1) Su is the Undrained shear strength estimated as follows:
Deep Silty Clay /Clay
Lower range Undrained Shear Strength = 0.22 $\sigma'_{vo}$
Upper range Undrained Shear Strength= 0.30 $\sigma'_{vo}$
Silty sand and Silt (in one analysis scenario assumed to have silt behaviour)
Undrained Shear Strength= 0.27 $\sigma'_{vo}$
where $\sigma'_{vo}$ is efective vertical stress

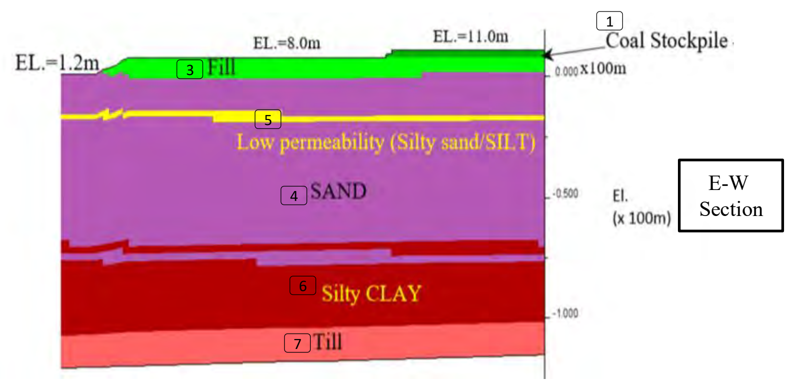
(2) qc1n_cs is normalized tip resistance corrected for fines content and used for obtaining PM4SAND parameters
See the assumed simplified qc1n_cs profiles in figures in this appendix. Qc1n_cs is the figures show zero for silts and 1 clays
For the ground improvement blocks in the storage building, qc1n_cs=150 is assumed (approximate).

(3) Go is shear stiffness coefficient and is a primary input PM4Sand & PM4Silt
For the East-West FLAC Section, Go is back calculated from the measured shear wave velocity, Vs profile
Gmax = ρ.Vs^2
Go =Gmax/( Pa.(σ'm/Pa)^n)
where
ρ = total density (kg/m3)
σ'm=effective mean stress and n=0.5 for sand & n=0.75 for silt and CLAY
Pa is atmospheric pressure ~ 101 kPa
For the North-South FLAC Section, slightly different approach was adopted as follows:
PM4Sand's Go correlation was adjusted to the site conditions in the following relationship
and then was applied to individual layers with assumed qc1n_cs
Go =155*(N160_cs+2.5)^0.5
where
Go is obtained from measured Vs, Go=ρ.Vs^2/( Pa.(σ'm/Pa)^n)
N160_cs=46 (DR)^2
DR=0.465 (qc1n_cs/Cdq)^0.264-1.063
Go=505 for Silty Sand and Silt
Go=460 for deep silty clay to clay based on measured Vs
Go=400 for the material stockpile is an approximate assumption.

(4) The moduli of till (the elastic half space) are assumed as follows:
Shear modulus: G=Gmax= ρ.Vs^2 = 4.3e8 N/m2 ; where ρ = 2100 (kg/m3), Vs=450 m/s
Bulk modulus: = 9.1e8 N/m2 ; Assuming Poisson's ratio ν = 0.3,

**General Notes:**

Density of soils above water table (moist soil density) is 95% of saturated density.
Statics shear modulus of soils = $G_{max}$ /5 and Poisson's ratio of 0.3 for soils.
Friction Angle used in initial Mohr-Coulomb model and post-earthquake conditions are assumed design values.



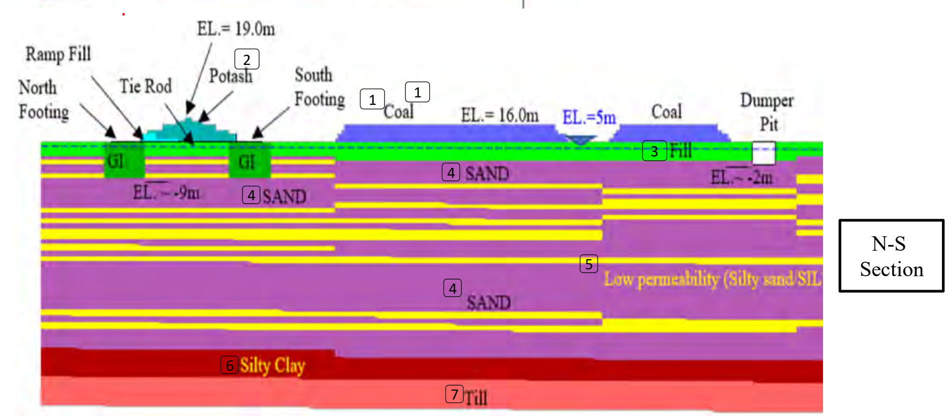



Table E-4: Assumed structural parameters (in N-S FLAC Section) and interface elements

| STRUCTURE No. | DESCRIPTION | MATERIAL | w | h | A | I | E | ρ | Out of plane spacing |
|---|---|---|---|---|---|---|---|---|---|
| | | | (m) | (m) | ($m^2$) | ($m^4$) | ($N/m^2$) | ($kg/m^3$) | (m) |
| | | | Note (1) | Note (2) | Note (3) | Note (4) | Note (5) | Note (6) | Note (7) |
| B1 | Storage building footing | Concrete | 1 | 1 | 1 | 8.33E-02 | 2.0E+10 | 2400 | 1 |
| B2 | Connection b/w building footing & retaining wall footing | Concrete | 1 | 0.6 | 0.6 | 1.80E-02 | 2.0E+10 | 2400 | 1 |
| B3 | Retaining wall footing | Concrete | 1 | 0.6 | 0.6 | 1.80E-02 | 2.0E+10 | 2400 | 1 |
| B4 | Retaining wall | Concrete | 1 | 0.4 | 0.4 | 5.33E-03 | 2.0E+10 | 2400 | 1 |
| B5 | Tie-Rod | Wire-Rope | - | - | 1.20E-03 | 1.60E-06 | 1.0E+11 | 7850 | 6.1 |
| B6 | Dumper pit wall | Concrete | 1 | 1 | 1 | 8.33E-02 | 2.0E+10 | 2400 | 1 |
| B7 | Dumper pit Base | Concrete | 1 | 1.2 | 1.2 | 1.44E-01 | 2.0E+10 | 6000 | 1 |
| B8 | Dumper pit roof | Concrete | 1 | 0.8 | 0.8 | 4.27E-02 | 2.0E+10 | 2400 | 1 |

WESTSHORE-Soil Parameters.xlsx

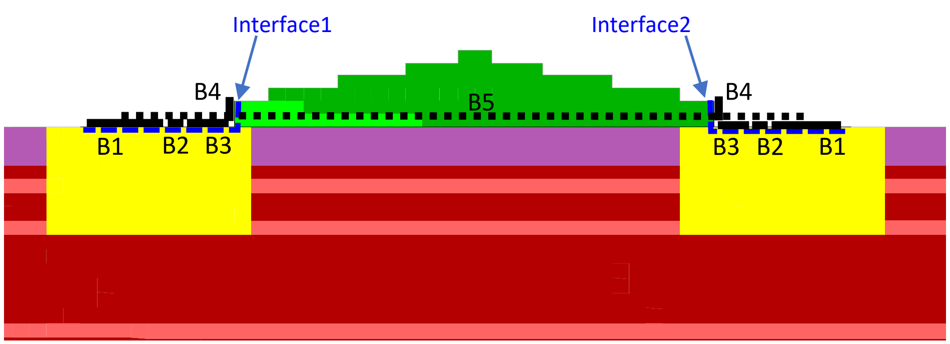
**Soil-concrete interface parameters**:
Friction angle=26.5 deg
Cohesion=0
Shear and Normal stiffness Ks = Kn=2e9 $N/m^2$
Tensile strength=0

**Notes**

(1) Width
(2) Height
(3) Area
(4) Moment of inertia
(5) Young's modulus. Different E and A have been considered for the tie rods in parametric analyses. See the body of report, Table 6-6.
(6) Mass density. The mass density of the base of the dumper pit includes the additional mass from the 2m of tremie concrete below the structural concrete.
(7) Spacing in east-west direction

**General Notes**

All the structural sections have been assumed to be elastic beams.
The building footings and the retaining wall footings were connected in compression but could move apart using a beam with high compressive strength and near zero tensile strength.
Tie rods were assumed elastic in tension but near zero compressive strength.



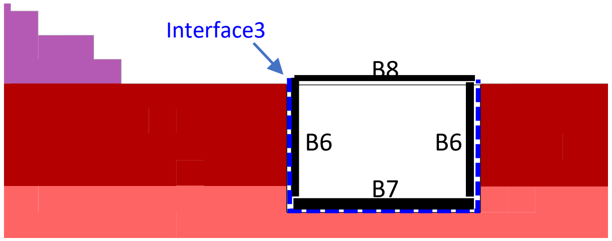



Figure E-1 : Location of FLAC sections and the CPT/Test holes used for each section

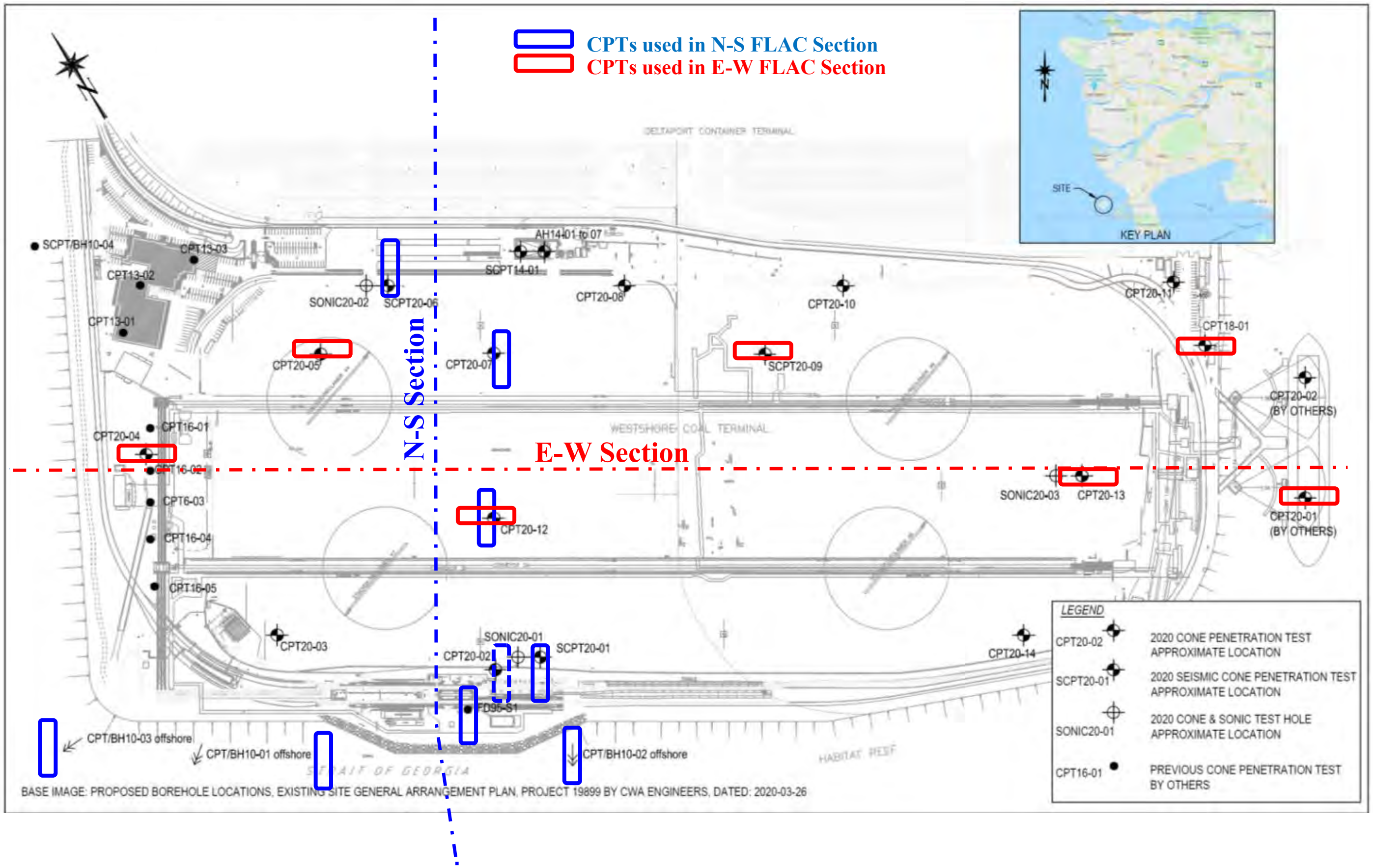


Figure E-2: Soil units and CPTs – East-West Section

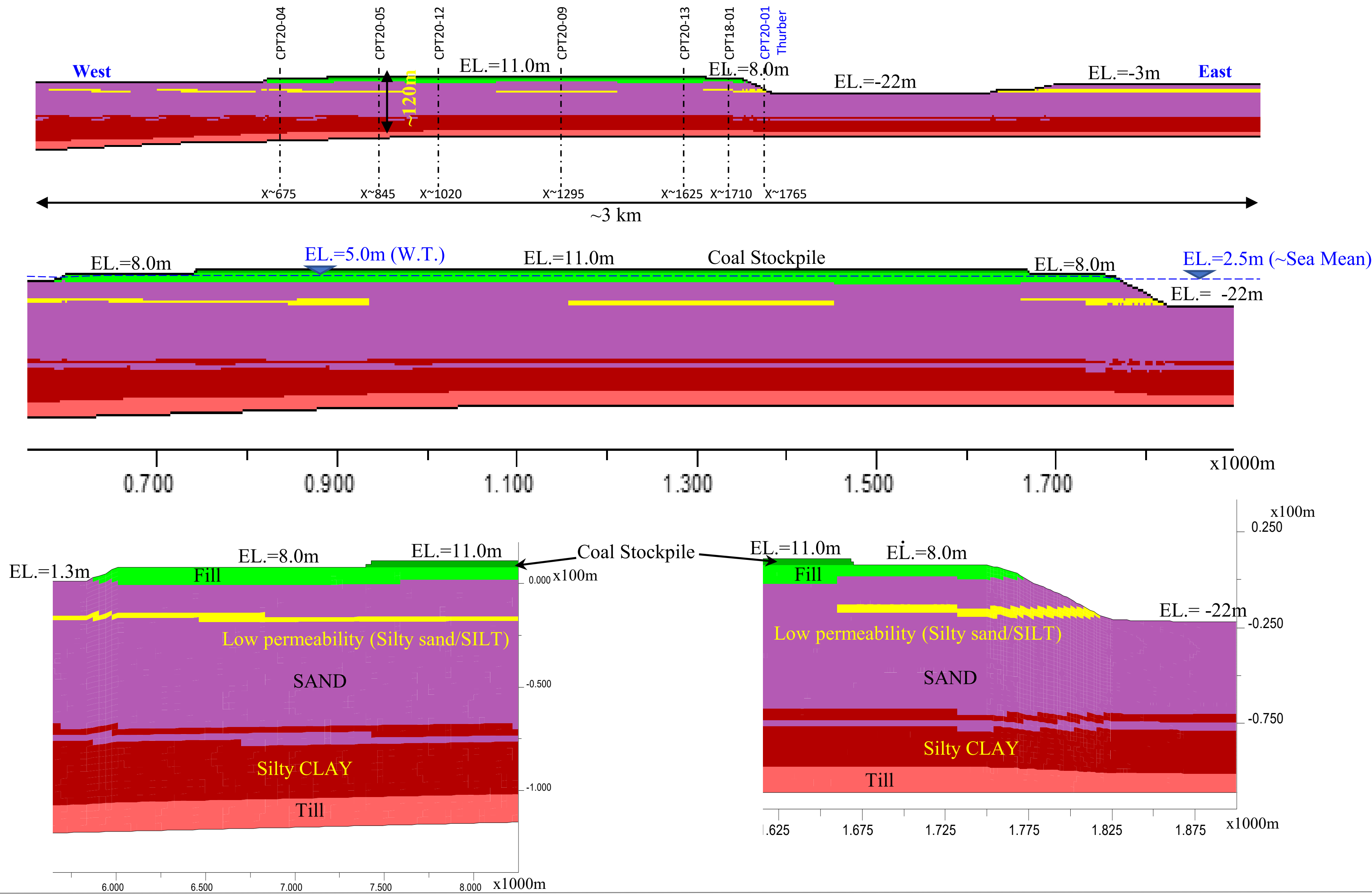


Figure E-3: Soil units and CPTs – North-South Section

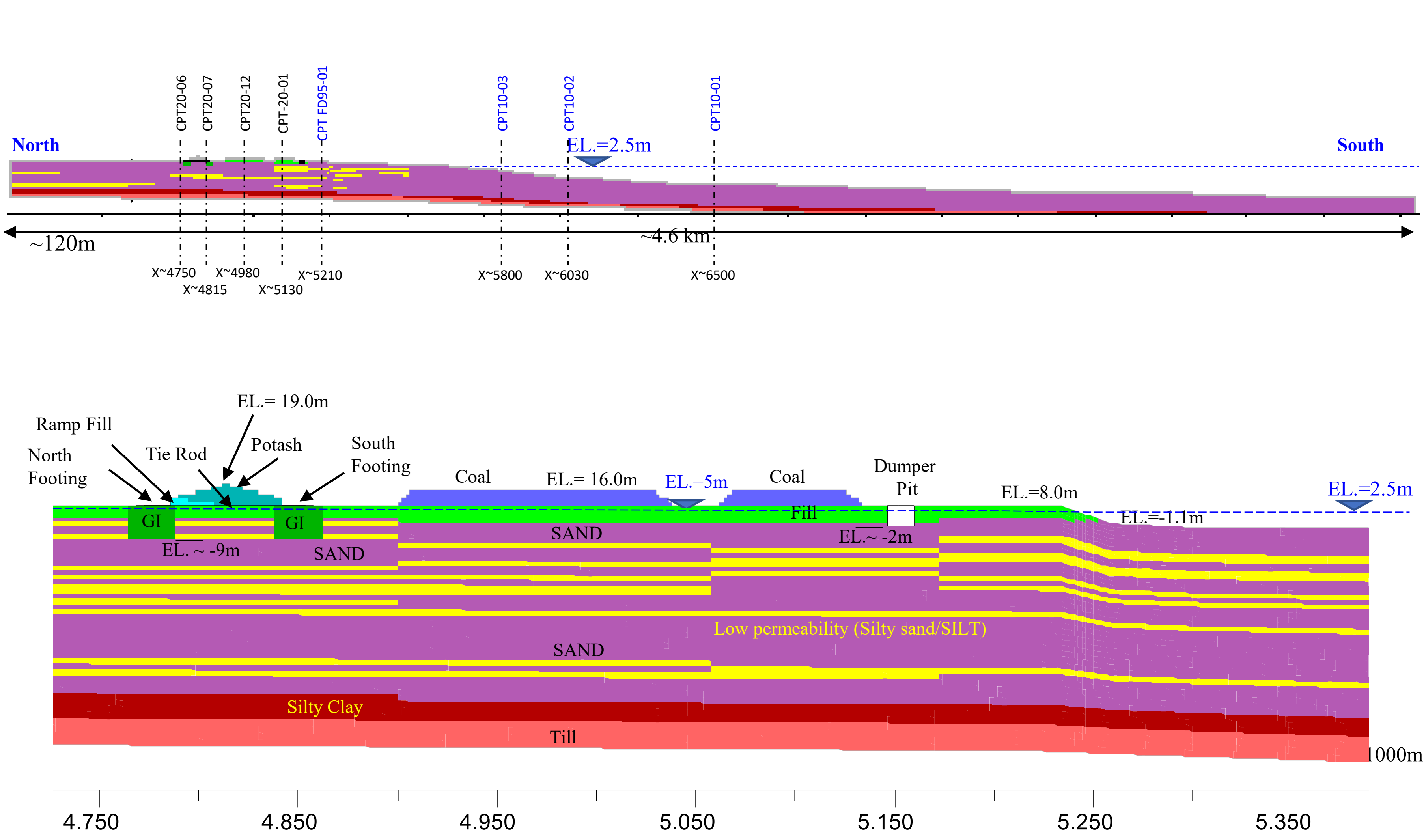
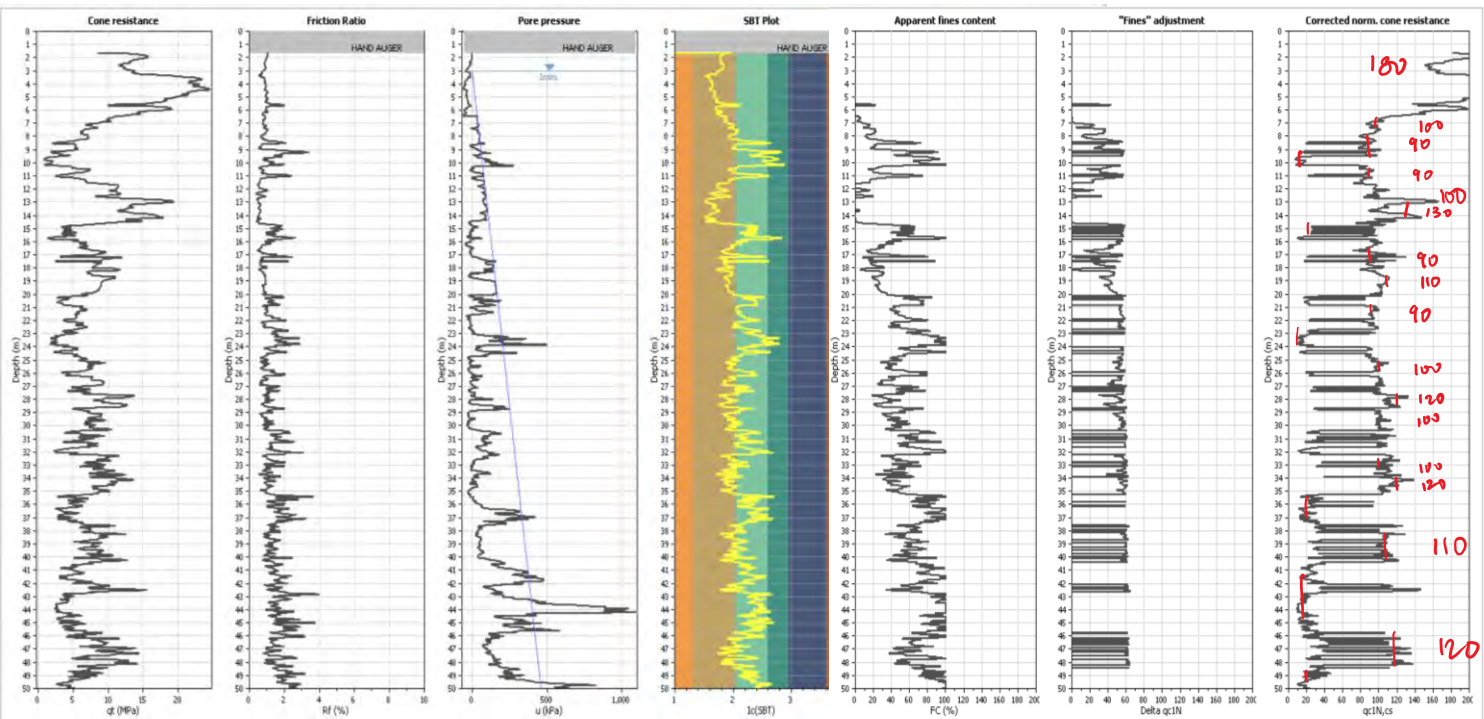


Figure E-4 : Example of simplified layering and qc1n_cs profile used in the FLAC model- CPT20-06

**Interpreted layering and qc1n_cs from the CLIQ plots**




**Assumed simplified qc1n_cs profile used in FLAC**

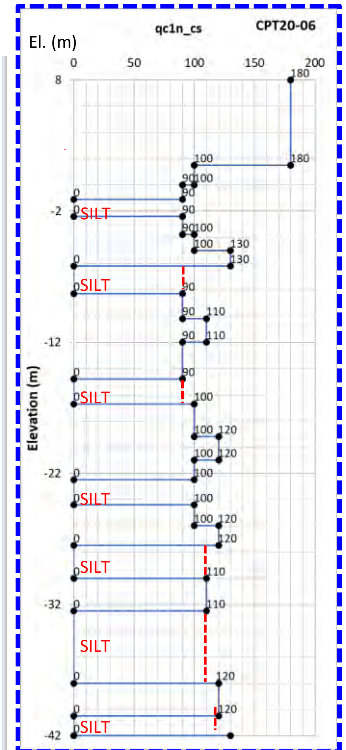


Figure E-5: CPTs used to develop the simplified qc1n_cs profiles for the E-W FLAC Section

| CPT | Depth | x-Coord. | x > | x < | CPT El. |
|---|---|---|---|---|---|
| | m | m | m | m | m |
| CPT20-04 | 30 | 675 | 0 | 760 | 8 |
| CPT20-05 | 30 | 845 | 760 | 933 | 8 |
| CPT20-12 | 30 | 1020 | 933 | 1158 | 8 |
| CPT20-09 | 30 | 1295 | 1158 | 1453 | 8 |
| CPT20-13 | 60 | 1610 | 1453 | 1660 | 8 |
| CPT18-01 | 48 | 1710 | 1660 | 1738 | 8 |
| CPT20-01 (Thurber CPT) | 50 | 1765 | 1738 | 3500 | -22 |

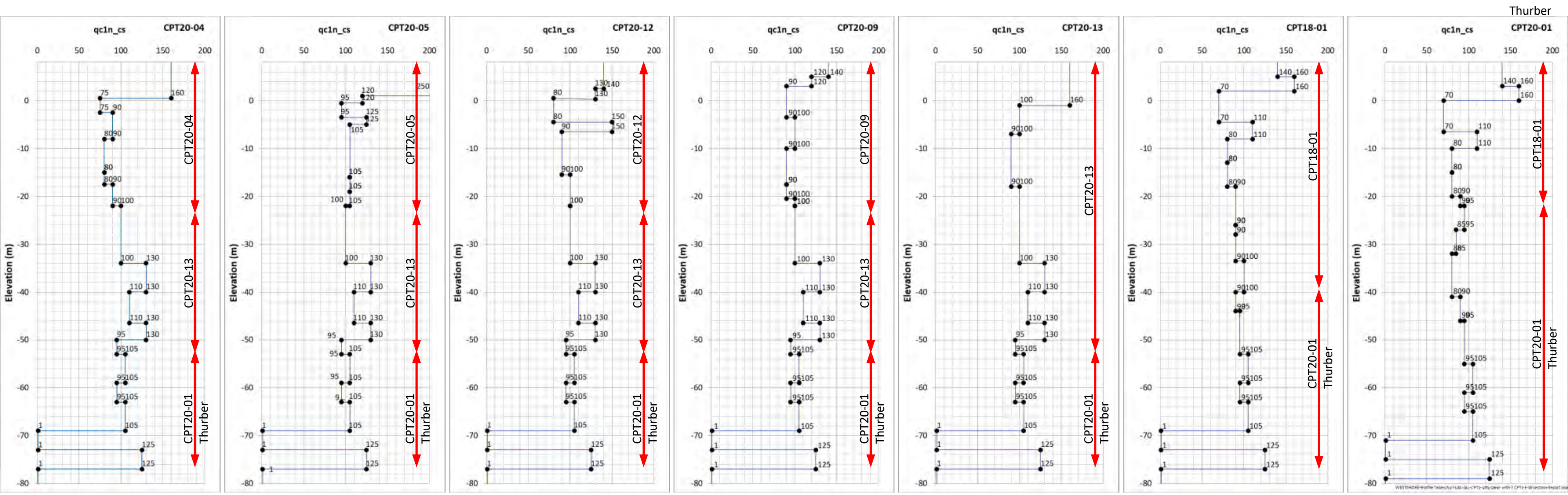


Figure E-6a: CPTs used to develop the simplified qc1n_cs profiles for the N-S FLAC Section

| | CPT | Depth | x-Coord. | i | x > | x < | CPT El. |
|---|---|---|---|---|---|---|---|
| | | m | m | | m | m | m |
| Land CPTs | CPT20-06 | 2006 | 4750 | 200 | 4200 | 4783 | 8 |
| | CPT20-07 | 2007 | 4815 | 233 | 4783 | 4898 | 8 |
| | CPT20-12 | 2012 | 4980 | 315 | 4898 | 5055 | 8 |
| | CPT20-01 | 2001 | 5130 | 390 | 5055 | 5170 | 8 |
| | CPT9501 | 9501 | 5210 | 430 | 5170 | 5259 | 8 |
| Marine CPTs | CPT10-03 | 1003 | 5800 | 725 | 5259 | 5915 | - |
| | CPT10-02 | 1002 | 6030 | 840 | 5915 | 6265 | - |
| | CPT10-01 | 1001 | 6500 | 1085 | 6265 | 8800 | - |

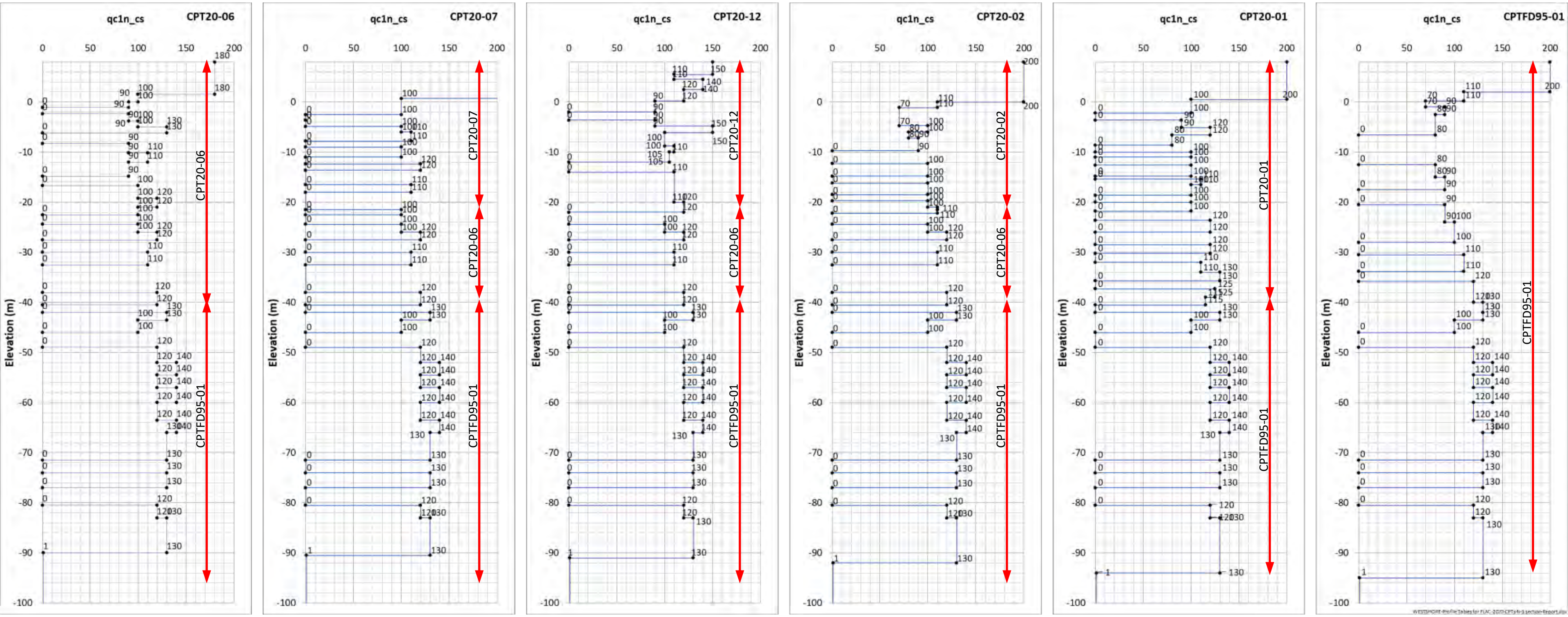
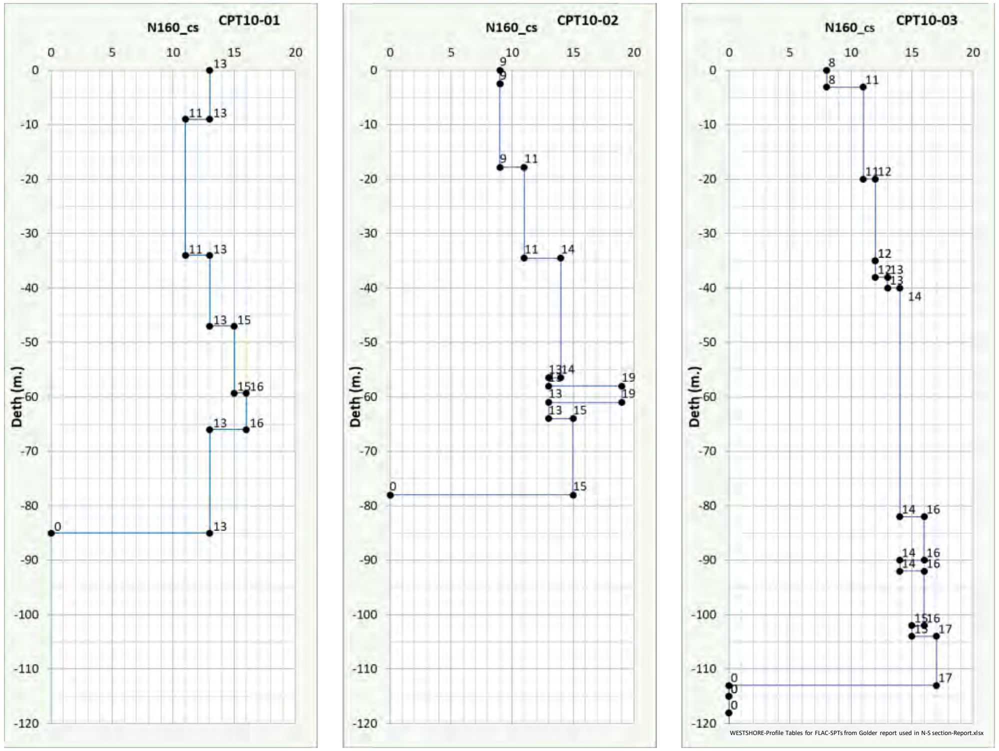


Figure E-6b: Offshore simplified profile for N-S FLAC Section (see the Note below)




Note:
- Golder 2011 used correlations to convert cone tip resistance to N160 _cs profiles ( Ref: Golder 2011-Figure D-3).
  It appears that Golder used 30 percentile of the cloud of the data for design N160 profiles.
- These design N160 values were used in the N-S FLAC Section of Braun/NAGL for the offshore areas where no site specific data was available for Westshore
- The thickness of layers from these profiles were proportionally adjusted to the thickness of the soil deposit between the mudline and top of clay in the FLAC section .

Figure E-7: Soil mesh- E-W FLAC Section

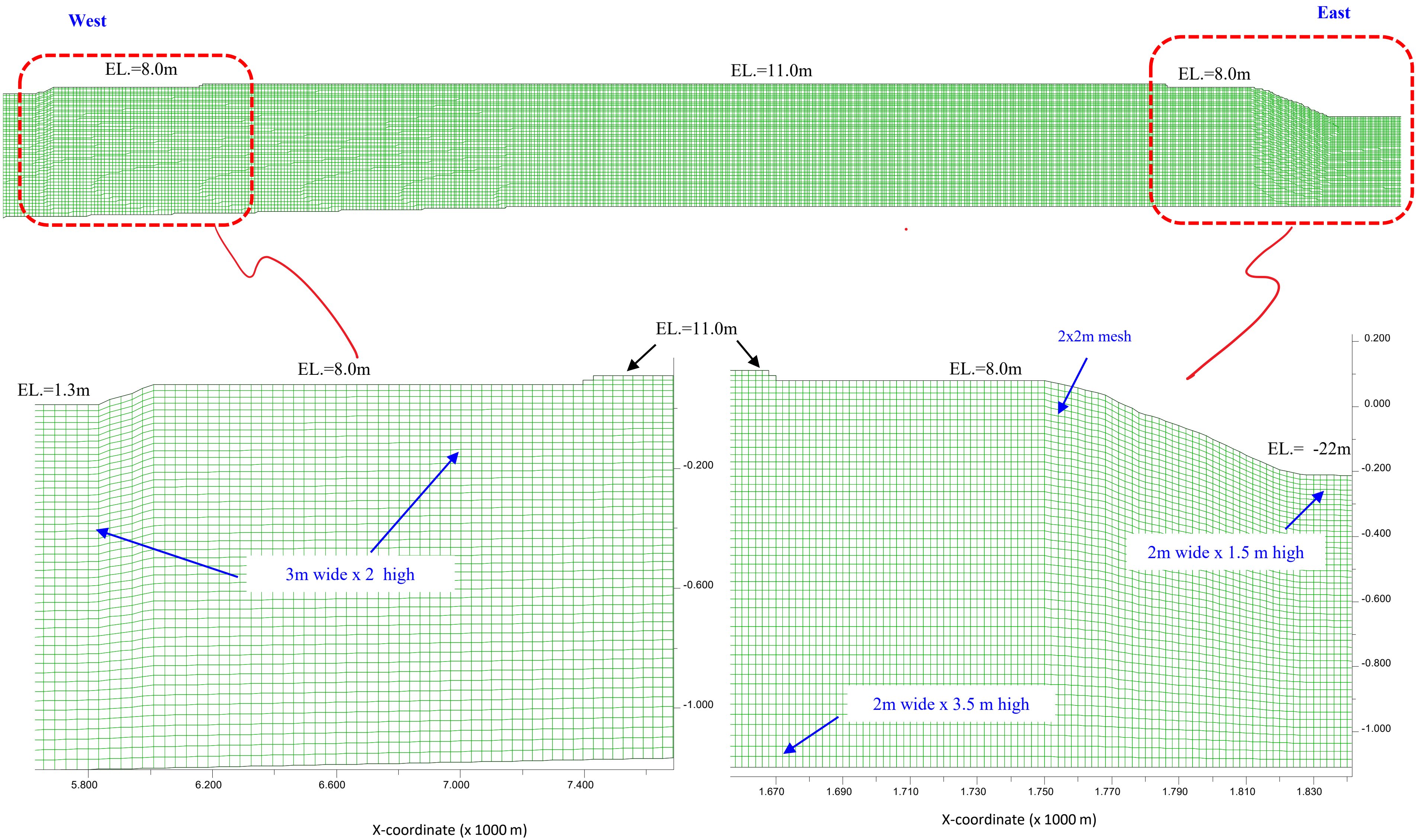


Figure E-8: Soil mesh- N-S FLAC Section

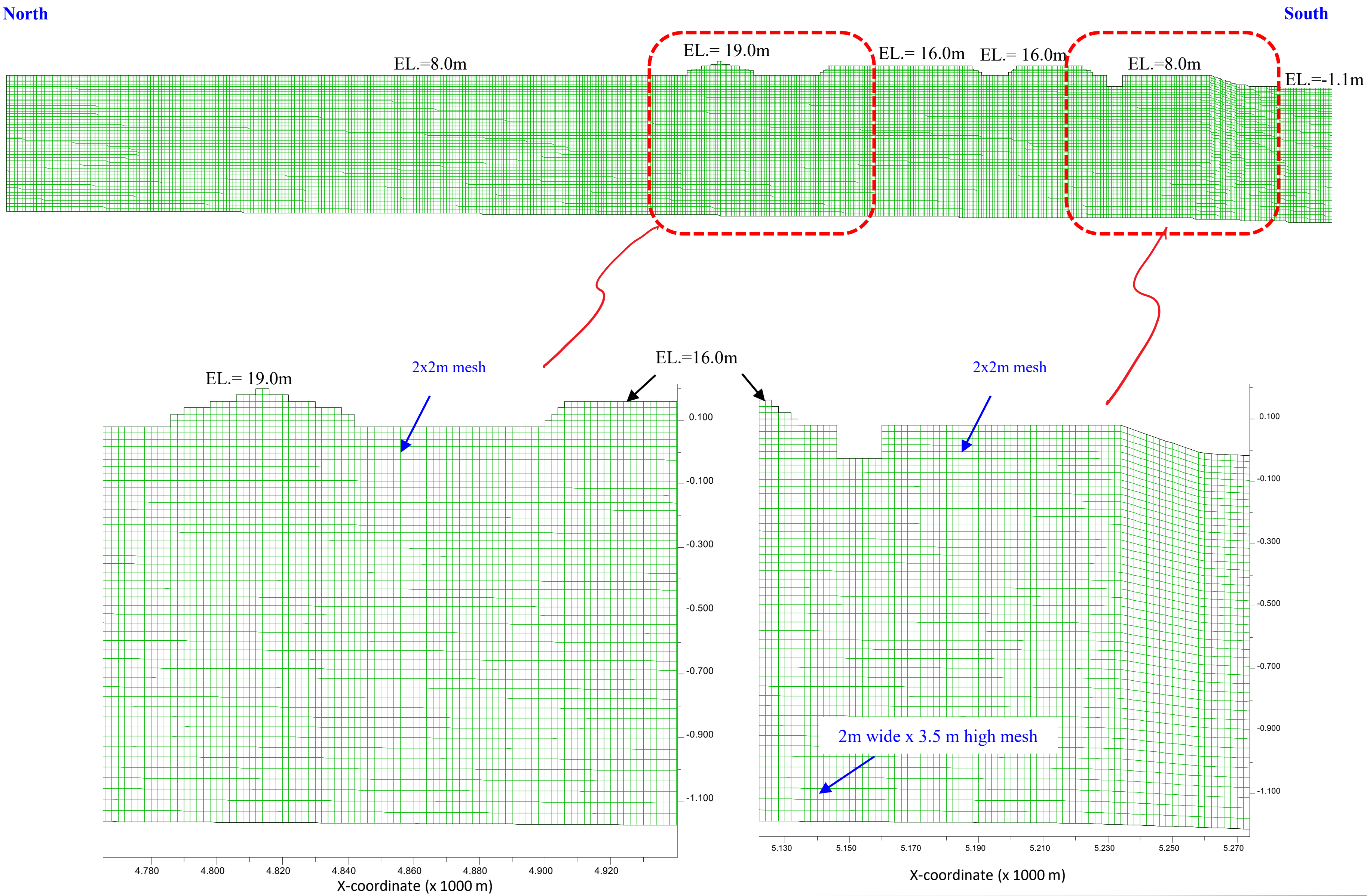
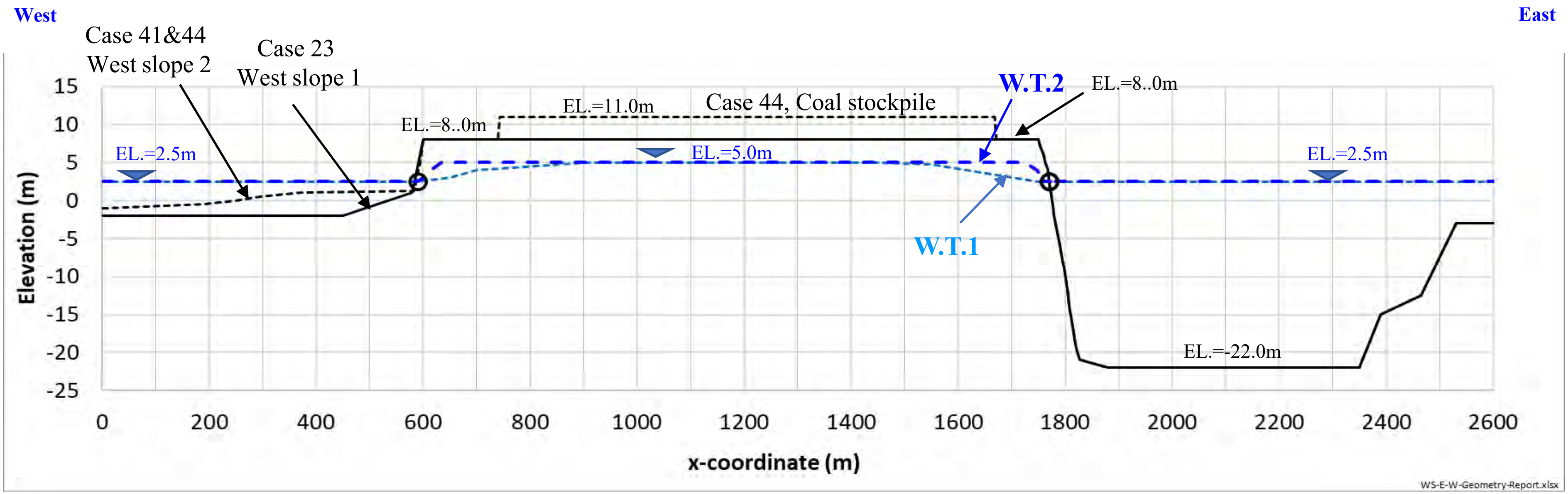


Figure E-9: Assumed geometry and phreatic surface for the E-W and N-S Sections



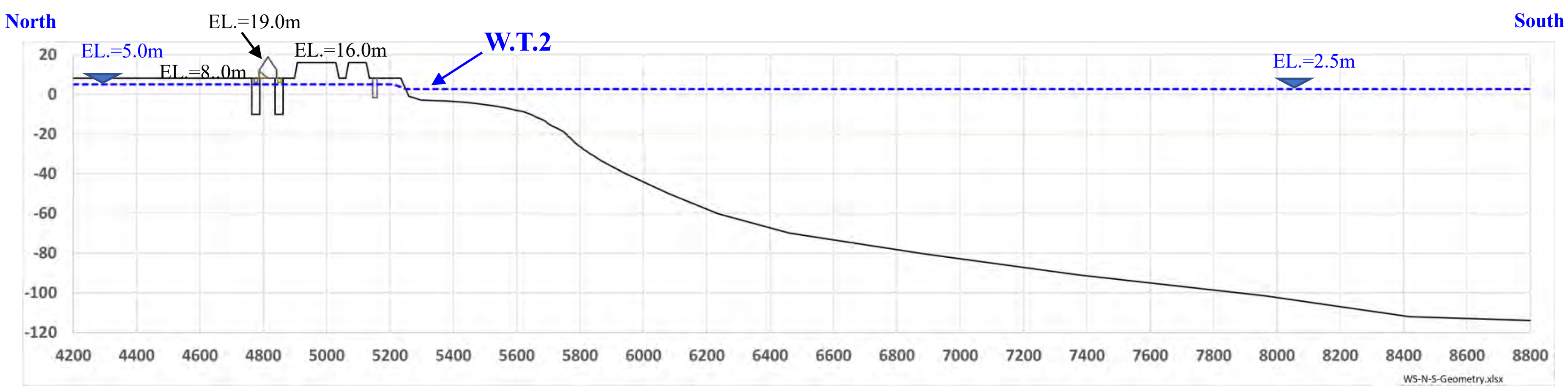



Figure E-10: External water pressure on mudline below sea level
Water pressure = Height of water column x 9800 $N/m^2$

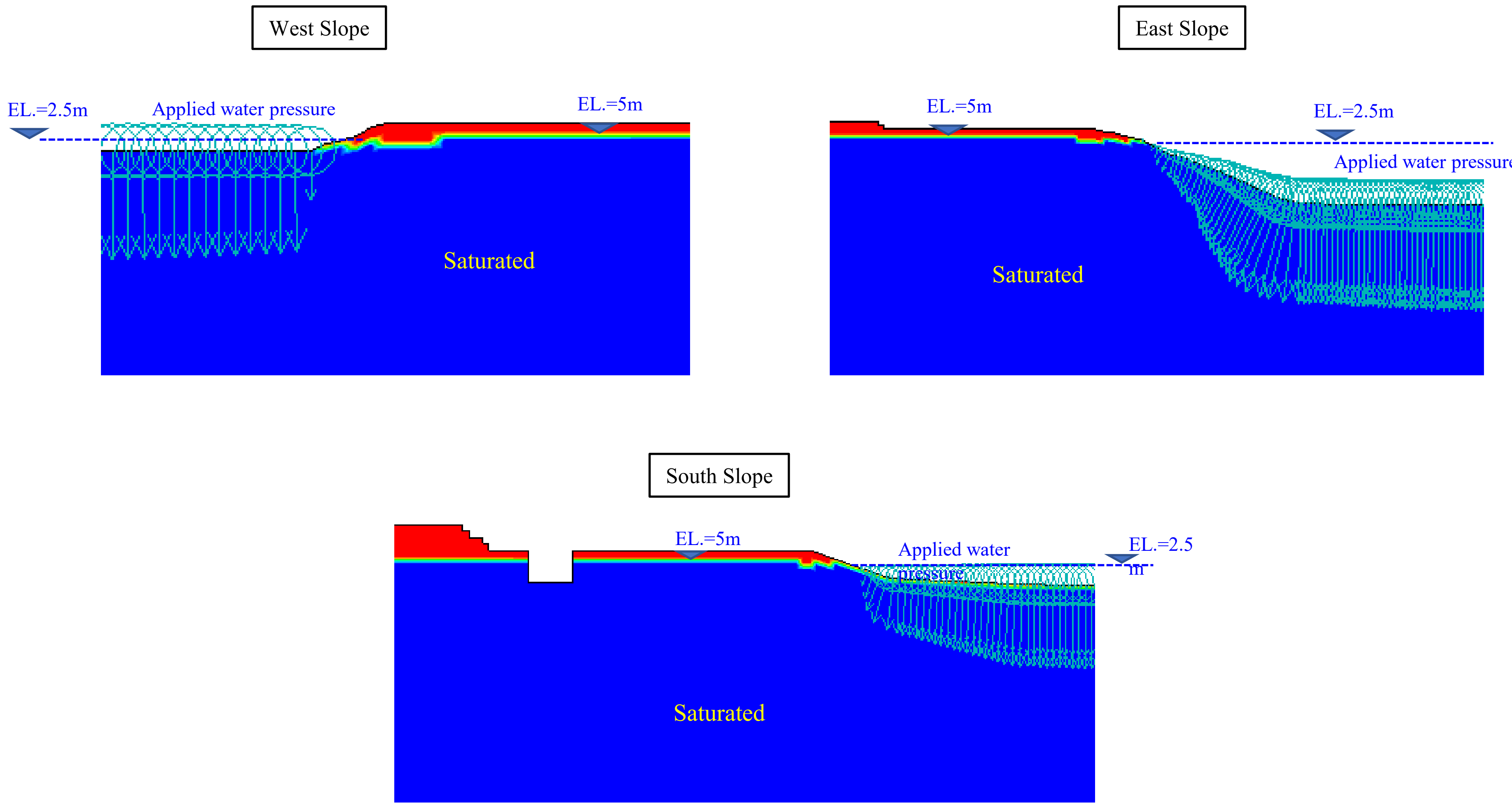


Figure E-11: Stresses in the model before earthquake, E-W FLAC Section

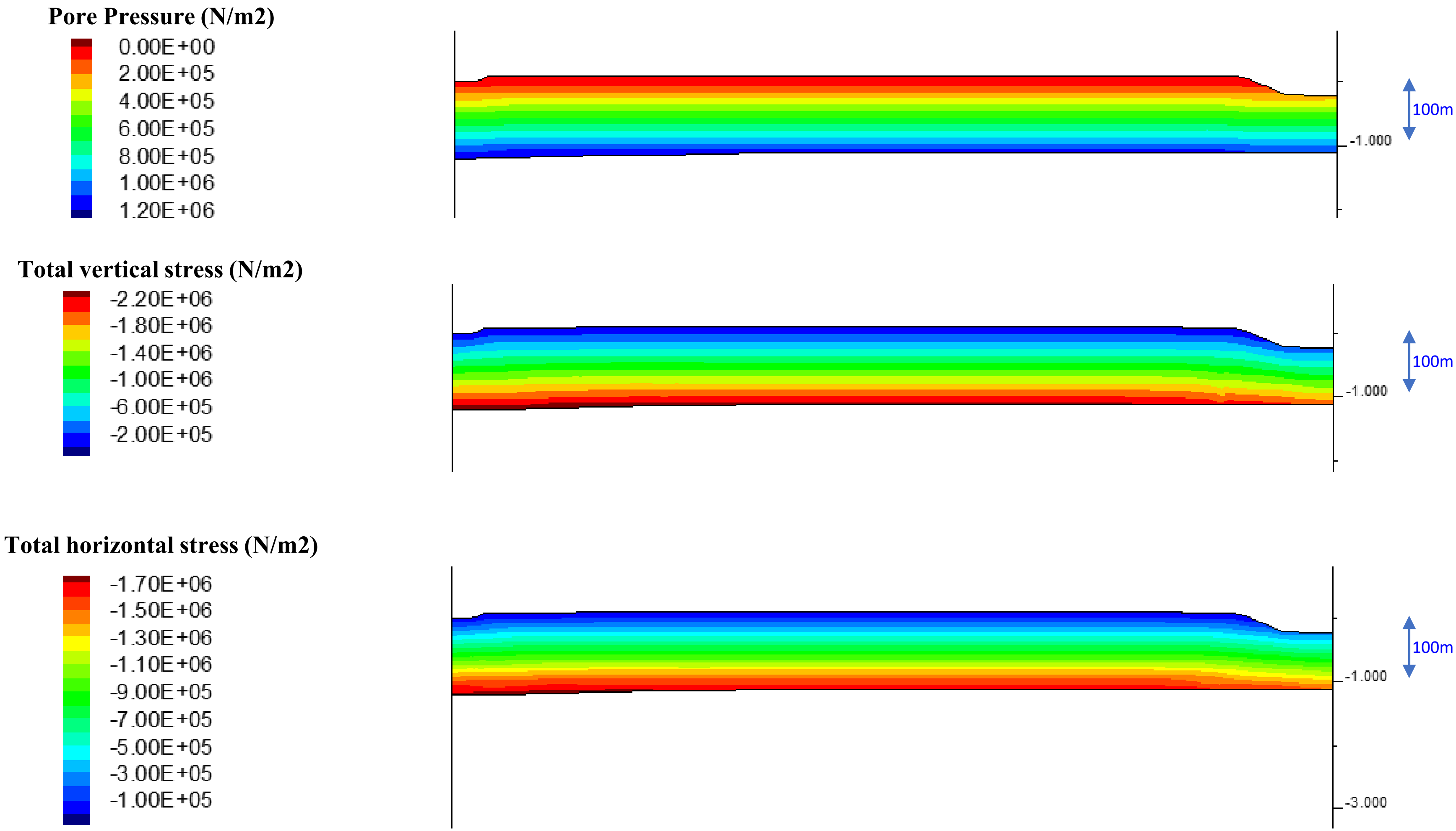


Figure E-12: Stresses in the model before earthquake, N-S FLAC Section

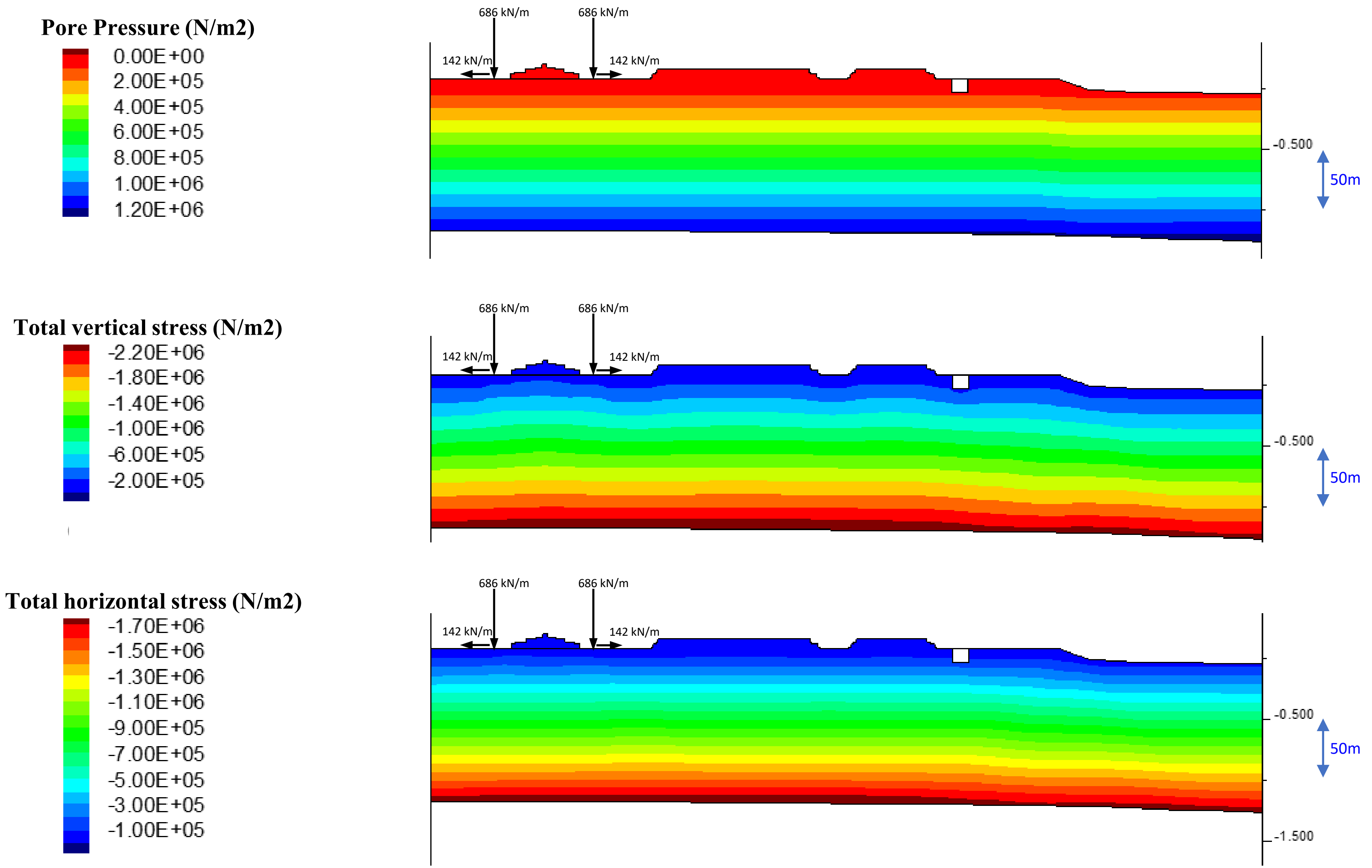


Figure E-13: Example of profiles of select soil parameters near the south slope, E-W FLAC Section

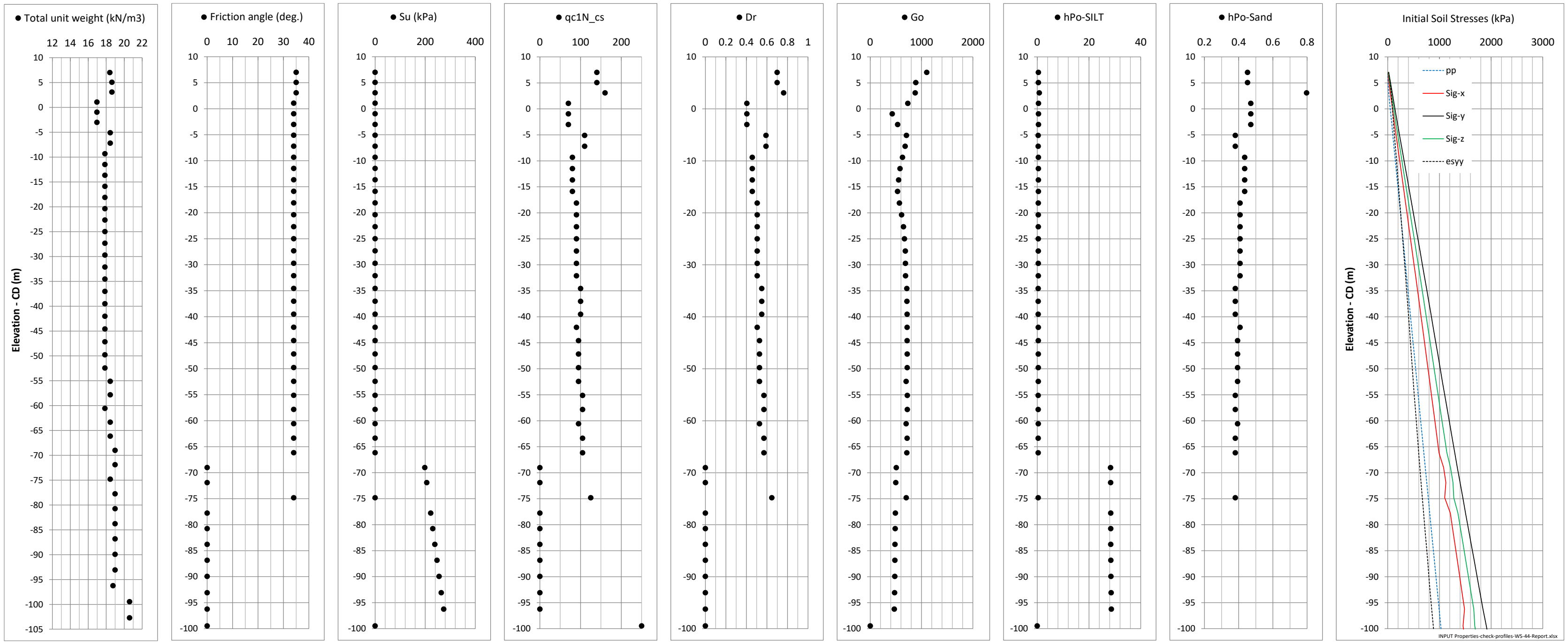


Figure E-14: Example of profiles of select soil parameters near the south slope, N-S FLAC Section

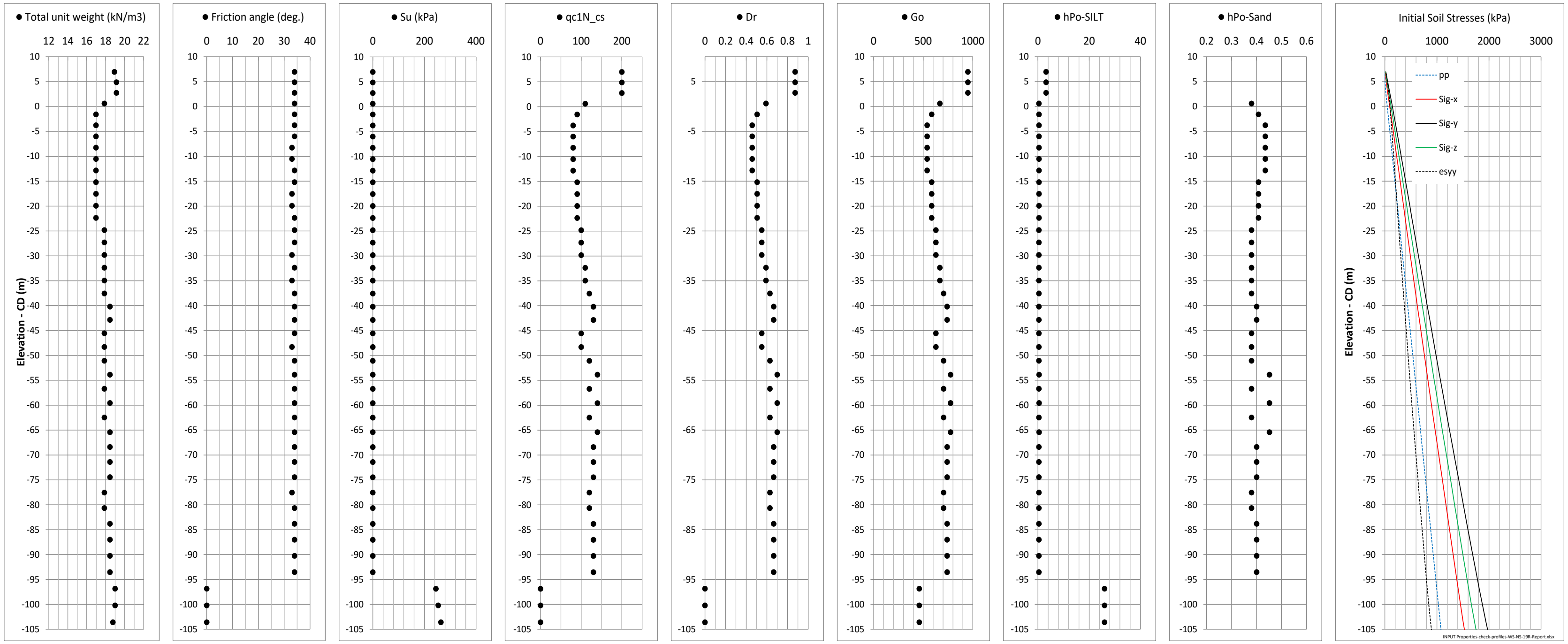


Figure E-15: Example of cyclic behavior in sand-like soils, far from the slope free face, Point A, E-W FLAC Section- 2475-CRU03 ground motion

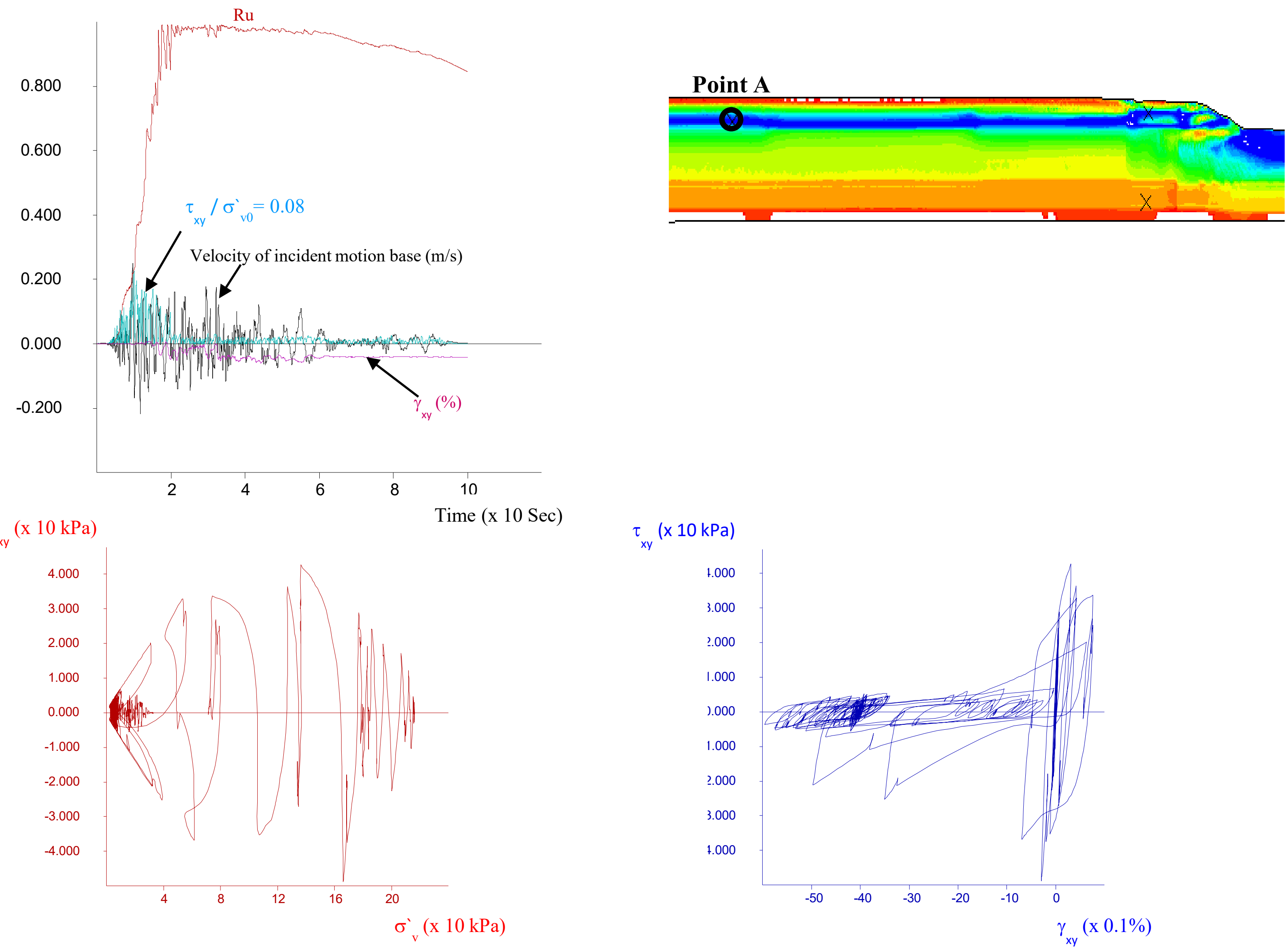


Figure E-16: Example of cyclic behavior in sand-like soils, near the slope free face, Point B, E-W FLAC Section- 2475-CRU03 ground motion

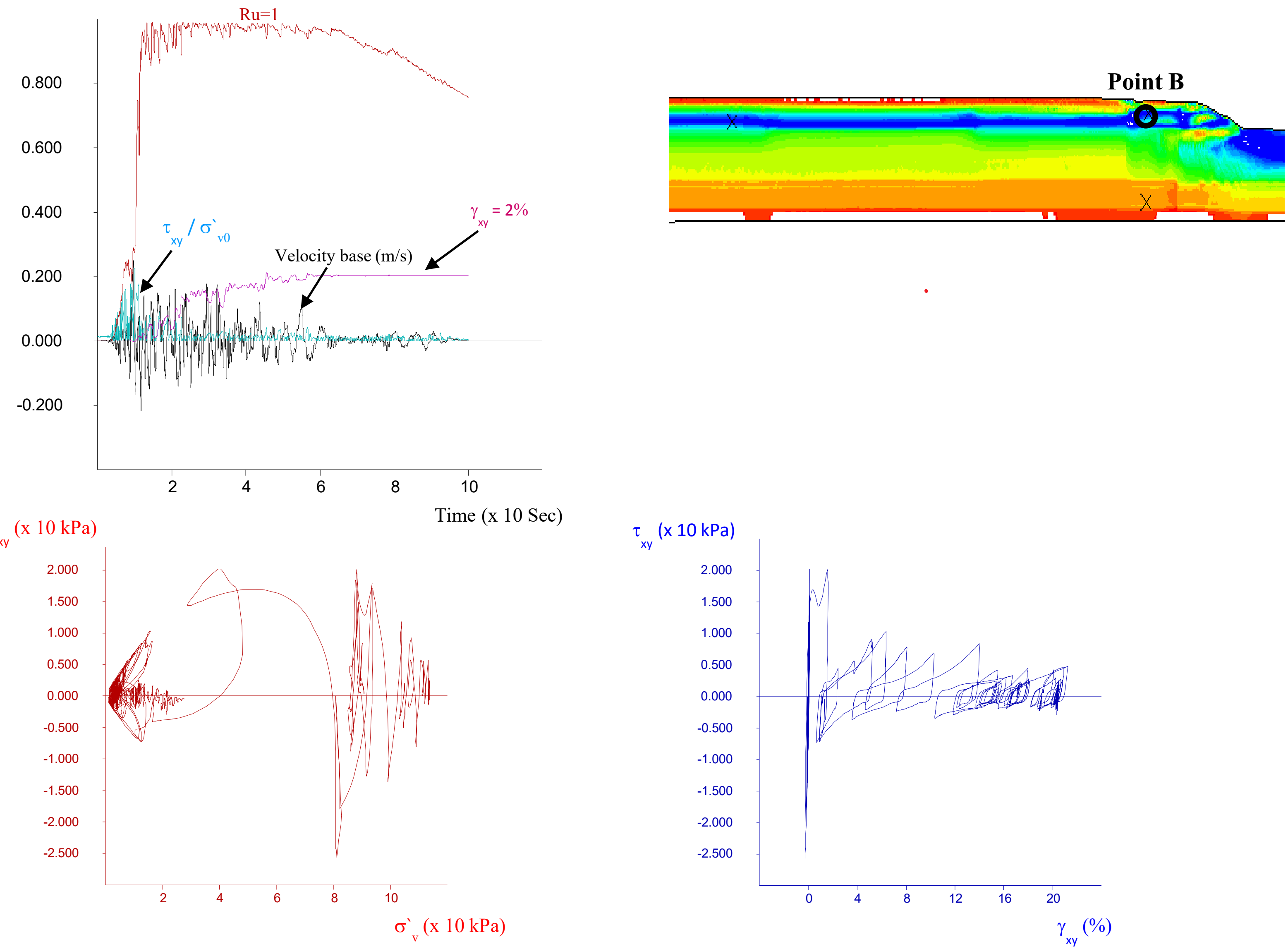
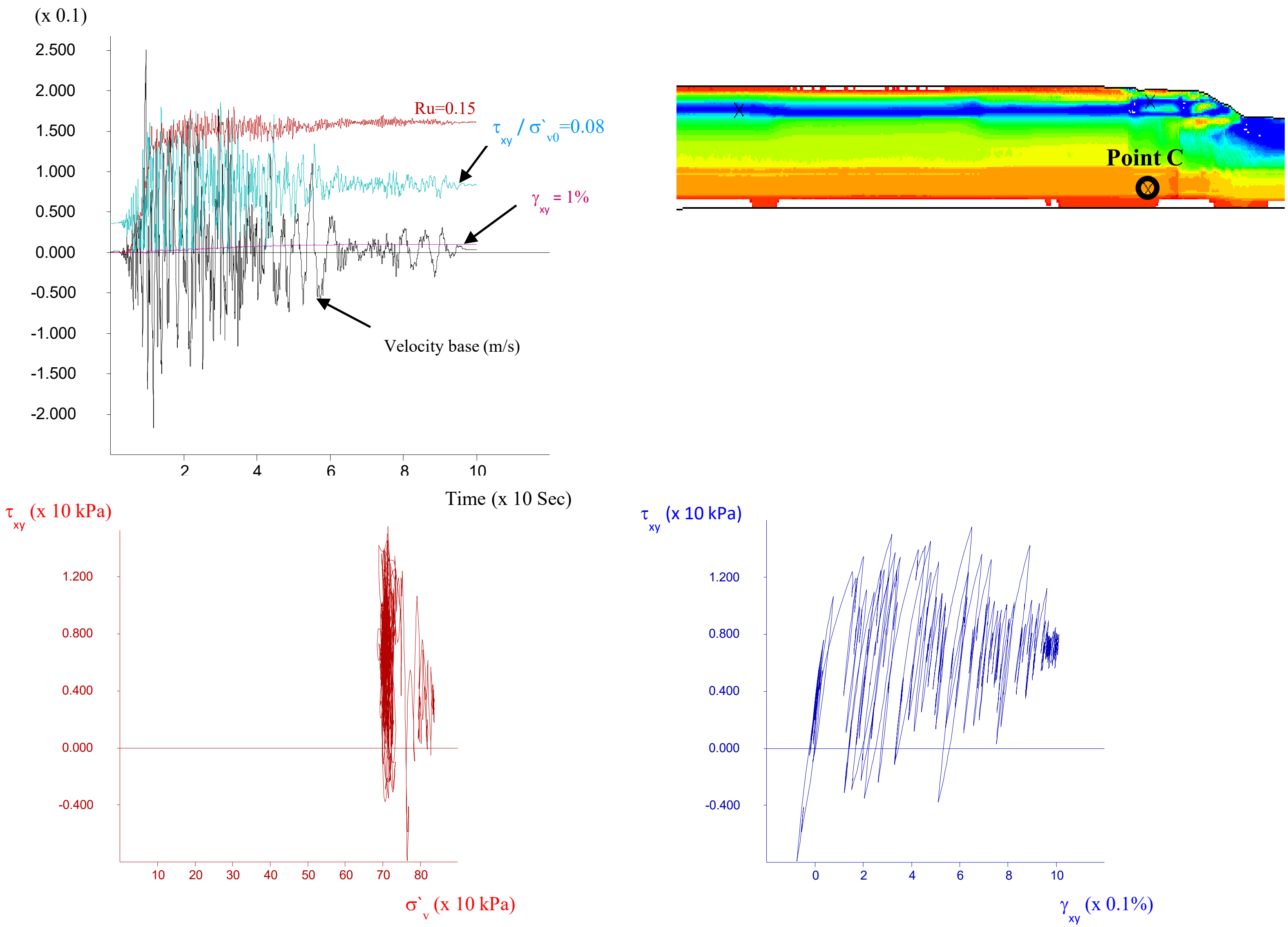


Figure E-17: Example of cyclic behavior in deep silty clay, Point C, E-W FLAC Section- 2475-CRU03 ground motion

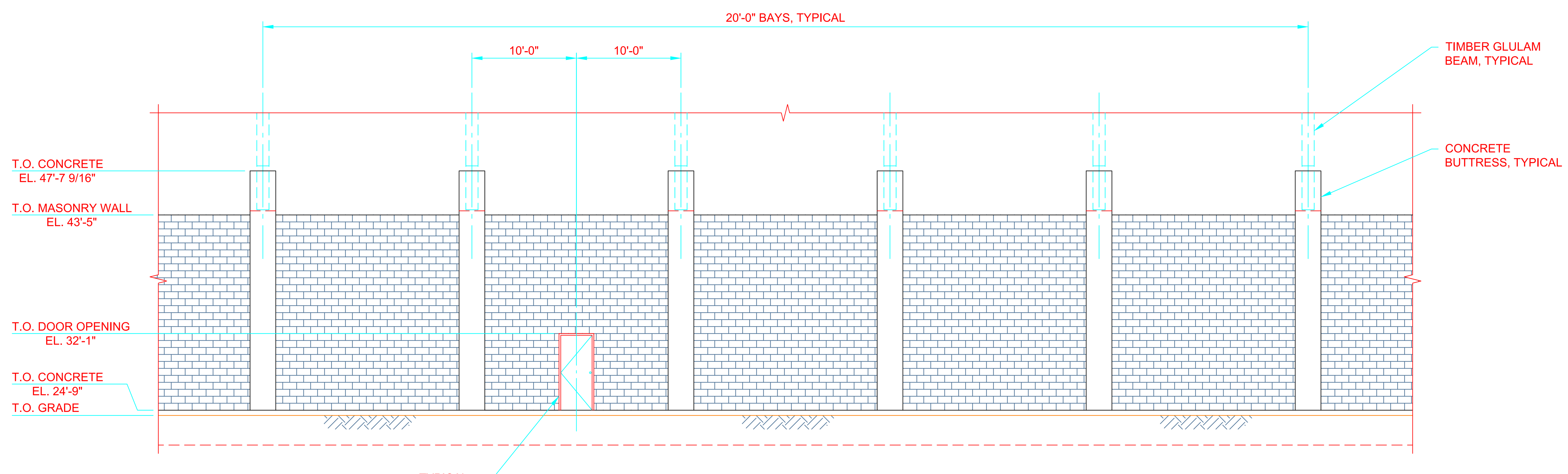


# Appendix F

# CWA Preliminary Storage Building Drawing


FOUNDATION PLAN

1" = 50'-0"




TYPICAL BUILDING FOUNDATION ELEVATION

1/8" = 1'-0"

NOTES:

1. FOR CONCRETE GENERAL NOTES, SEE REF.1.

PRELIMINARY
NOT FOR CONSTRUCTION

| REF. | DWG. NUMBER | DESCRIPTION |
|---|---|---|
| 2 | 85400-D0010-0110 | NEW CARGO STUDY - POTASH STORAGE BUILDING - FOUNDATION SECTIONS |
| 1 | 40101-D0010-0001 | NEW CARGO STUDY - SITE STANDARDS - CONCRETE GENERAL NOTES |

REFERENCE DRAWINGS / DESIGN STANDARDS

| No. | YYYY-MM-DD | DESCRIPTION | DRAWN | DWG. CHECK | DESIGN | DESIGN CHECK | DISC. APPROVAL | PROJ. APPROVAL |
|---|---|---|---|---|---|---|---|---|
| - | | | | | | | | |

ISSUES / REVISIONS

+ HAND INITIALS ON FILE

CWA

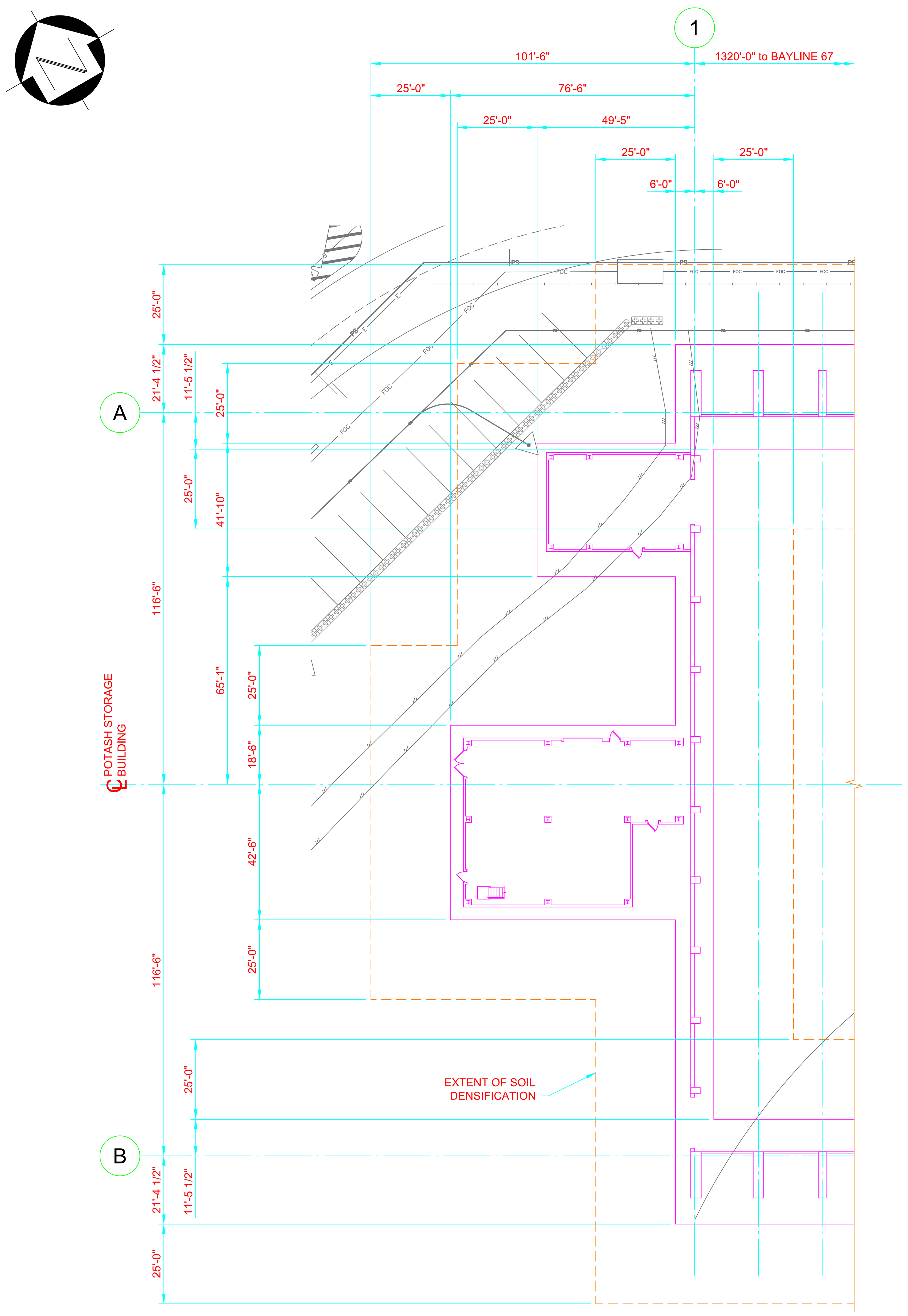
This drawing has been prepared by CWA Engineers Inc. as an instrument of service and is the exclusive property of CWA. This drawing shall be used solely for the purpose of this project. The client agrees that this drawing shall not be used for purposes other than those intended, and shall hold the engineer harmless for any other such use.

CWA PROJECT NUMBER: 19899

NEW CARGO STUDY
POTASH STORAGE BUILDING
FOUNDATION PLAN & ELEVATION

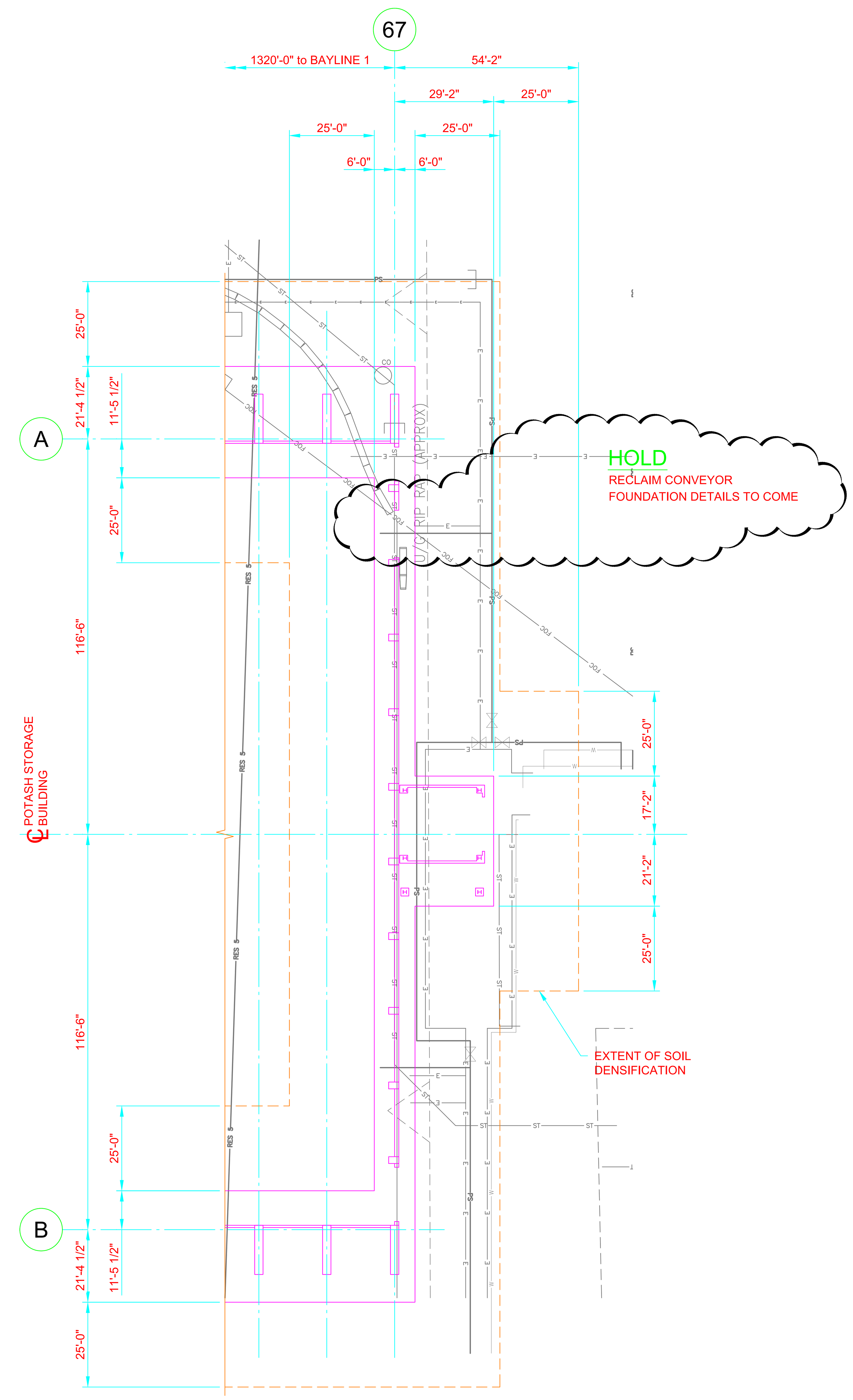
85400-D0010-0110 -

DRAWN BY: CPL | SCALE: AS NOTED | ACAD FILENAME: 85400-D0010-0110 | REV.




## PLAN - POTASH STORAGE BUILDING WEST WALL SOIL DENSIFICATION DETAIL

1" = 25'-0"




## PLAN - POTASH STORAGE BUILDING EAST WALL SOIL DENSIFICATION DETAIL

1" = 25'-0"

NOTES:

1. FOR CONCRETE GENERAL NOTES, SEE REF.1.

REVISION IN PROGRESS

PRELIMINARY
NOT FOR CONSTRUCTION

| REF. | DWG. NUMBER | DESCRIPTION |
|---|---|---|
| 2 | 85400-D0010-0110 | NEW CARGO STUDY - POTASH STORAGE BUILDING - FOUNDATION PLAN & ELEVATION |
| 1 | 40101-D0010-0001 | NEW CARGO STUDY - SITE STANDARDS - CONCRETE GENERAL NOTES |

REFERENCE DRAWINGS / DESIGN STANDARDS

| No. | YYYY-MM-DD | DESCRIPTION | DRAWN | DWG. CHECK | DESIGN | DESIGN CHECK | DISC. APPROVAL | PROJ. APPROVAL |
|---|---|---|---|---|---|---|---|---|
| RIP | - | REVISION IN PROGRESS | - | - | - | - | - | - |

ISSUES / REVISIONS

+ HAND INITIALS ON FILE

CWA



CWA PROJECT NUMBER: 19899

NEW CARGO STUDY
POTASH STORAGE BUILDING
SOIL DENSIFICATION PLAN

85400-D0010-0115 | RIP

DRAWN BY: CPL | SCALE: AS NOTED | ACAD FILENAME: 85400-D0010-0110 | REV.

NOTES:

1. FOR CONCRETE GENERAL NOTES, SEE REF.1.


SECTION A
3/32" = 1'-0"
REF.2

PRELIMINARY
NOT FOR CONSTRUCTION

| REF. | DWG. NUMBER | DESCRIPTION |
|---|---|---|
| 3 | 85400-D0010-0117 | NEW CARGO STUDY - POTASH STORAGE BUILDING - FOUNDATION DETAILS |
| 2 | 85400-D0010-0110 | NEW CARGO STUDY - POTASH STORAGE BUILDING - FOUNDATION PLAN & ELEVATION |
| 1 | 40101-D0010-0001 | NEW CARGO STUDY - SITE STANDARDS - CONCRETE GENERAL NOTES |

REFERENCE DRAWINGS / DESIGN STANDARDS

| No. | YYYY-MM-DD | DESCRIPTION | DRAWN | DWG. CHECK | DESIGN | DESIGN CHECK | DISC. APPROVAL | PROJ. APPROVAL |
|---|---|---|---|---|---|---|---|---|
| P1 | - | ISSUED FOR INFORMATION | CPL | - | - | - | - | - |

ISSUES / REVISIONS

+ HAND INITIALS ON FILE

CWA



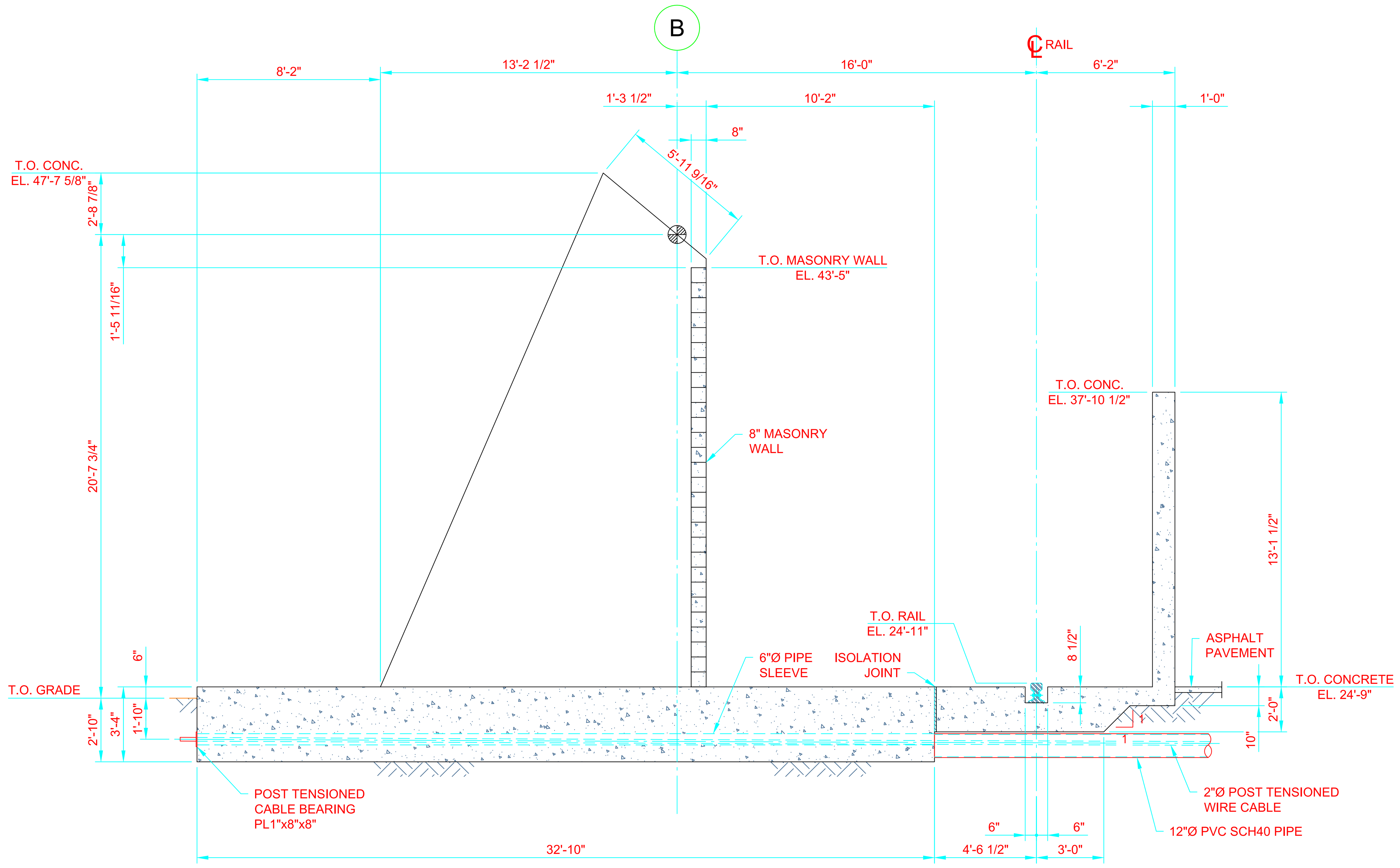
CWA PROJECT NUMBER: 19899

NEW CARGO STUDY
POTASH STORAGE BUILDING
FOUNDATION SECTIONS

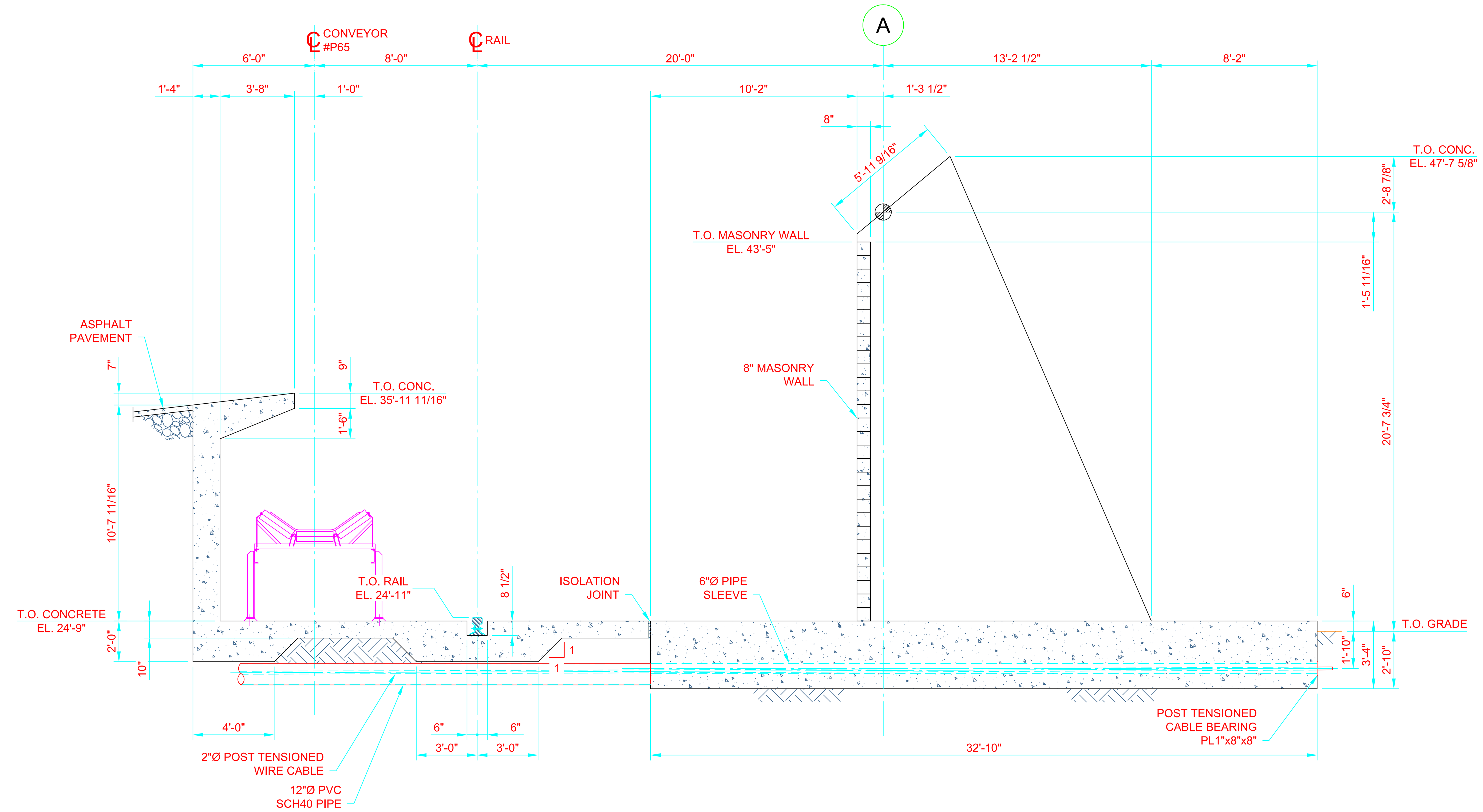
85400-D0010-0116

REV. -

DRAWN BY: CPL | SCALE: AS NOTED | ACAD FILENAME: 85400-D0010-0110




DETAIL 1
1/4" = 1'-0"
REF.2




DETAIL 2
1/4" = 1'-0"
REF.2

NOTES:

1. FOR CONCRETE GENERAL NOTES, SEE REF.1.

PRELIMINARY
NOT FOR CONSTRUCTION

| REF. | DWG. NUMBER | DESCRIPTION |
|---|---|---|
| 2 | 85400-D0010-0116 | NEW CARGO STUDY - POTASH STORAGE BUILDING - FOUNDATION SECTIONS |
| 1 | 40101-D0010-0001 | NEW CARGO STUDY - SITE STANDARDS - CONCRETE GENERAL NOTES |

REFERENCE DRAWINGS / DESIGN STANDARDS

| No. | YYYY-MM-DD | DESCRIPTION | DRAWN | DWG. CHECK | DESIGN | DESIGN CHECK | DISC. APPROVAL | PROJ. APPROVAL |
|---|---|---|---|---|---|---|---|---|
| - | | | | | | | | |

ISSUES / REVISIONS

* HAND INITIALS ON FILE

CWA



CWA PROJECT NUMBER: 19899

NEW CARGO STUDY
POTASH STORAGE BUILDING
FOUNDATION DETAILS

85400-D0010-0117 | REV. -

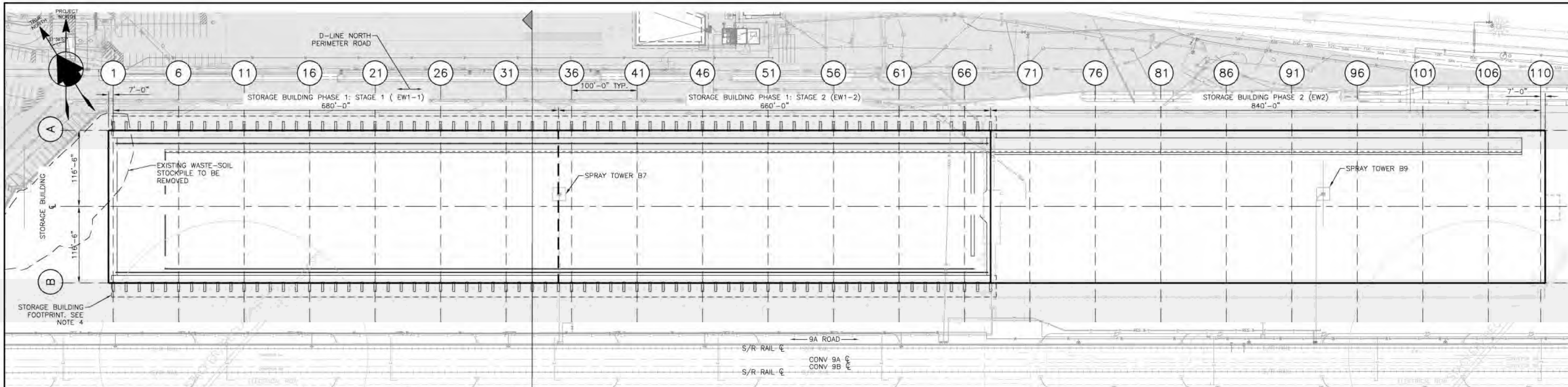
DRAWN BY: CPL | SCALE: AS NOTED | ACAD FILENAME: 85400-D0010-0110

# Appendix G

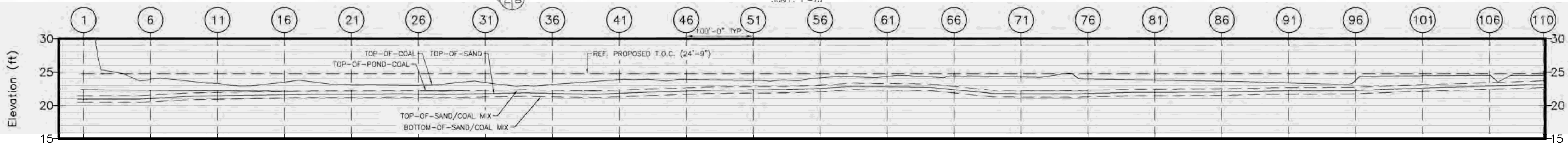
# RF Binnie drawings

Existing coal / sand interface elevation plan and profile dated Dec 18, 2020

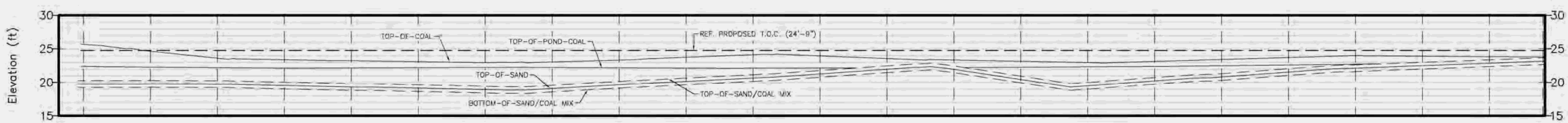
Storage Building Area – Phase 1 Preload General Arrangement dated June 1, 2021.



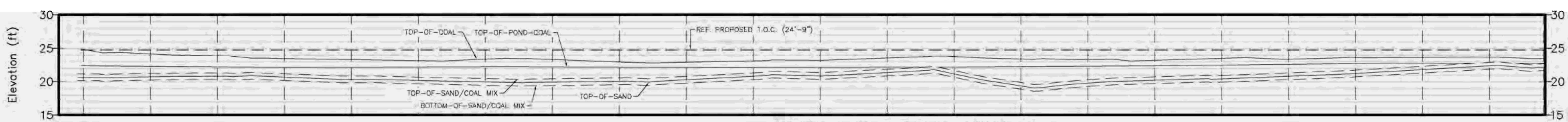

PLAN
SCALE: 1"=75'




PROFILE – EXISTING STRATA – BAYLINE "A"
SCALE: H: 1"=75', V: 1"=7.5'




PROFILE – EXISTING STRATA – BUILDING ℄
SCALE: H: 1"=75', V: 1"=7.5'




PROFILE – EXISTING STRATA – BAYLINE "B"
SCALE: H: 1"=75', V: 1"=7.5'

PRELIMINARY

NOTES:

1) ELEVATIONS OF "TOP-OF-COAL" AND "TOP-OF-SAND" MEASURED BY BINNIE GPS SURVEY DURING TEST EXCAVATIONS (MAY 2020)
2) ELEVATIONS OF "TOP-OF-POND-COAL" CALCULATED BASED ON S/R'S MAXIMUM OPERATING LUFF ANGLE
3) ELEVATIONS OF "TOP-OF-SAND/COAL MIX" AND "BOTTOM-OF-SAND/COAL MIX" WERE VERTICALLY OFFSET FROM "TOP-OF-SAND" MIX BASED ON OBSERVATIONS DURING TEST EXCAVATIONS
4) STORAGE BUILDING LAYOUT AS PER CWA'S "40101-D0000-000D" (UNOFFICIAL) DRAWING, RECEIVED ON 2020/11/20 STORAGE BUILDING CROSS SECTION AS PER CWA'S "85400-D0010-0116 (P2)" DATED 2020/11/13

| No. | By | Date | Revision |
|---|---|---|---|
| P5 | MS | 20/12/18 | ISSUED FOR USE – STAGE 2 STUDY |
| P4 | DY | 20/10/30 | ISSUED FOR REVIEW |
| P3 | DY | 20/07/14 | ISSUED FOR REVIEW |
| P2 | DY | 20/05/27 | ISSUED FOR REVIEW |
| P1 | MS | 20/05/22 | ISSUED FOR REVIEW |

STAMP

Client: Westshore Terminals LP

Consultant: BINNIE
The people behind your infrastructure.
R.F. BINNIE & ASSOCIATES LTD.
300 - 4940 Canada Way,
Burnaby, BC V5G 4K6
TEL 604.420.1721
BINNIE.com

Project Title: NEW CARGO STUDY

Title: EARTHWORKS
STORAGE BUILDING AREA
EXISTING
PLAN & PROFILE

| Design MS | Drawn DY | Checked CV | Plot Scale |
|---|---|---|---|
| Date 20/12/18 | Date 20/12/18 | Date 20/12/18 | AS NOTED |

Drawing Number: 85400-D0010-0010 Revision Number: P5

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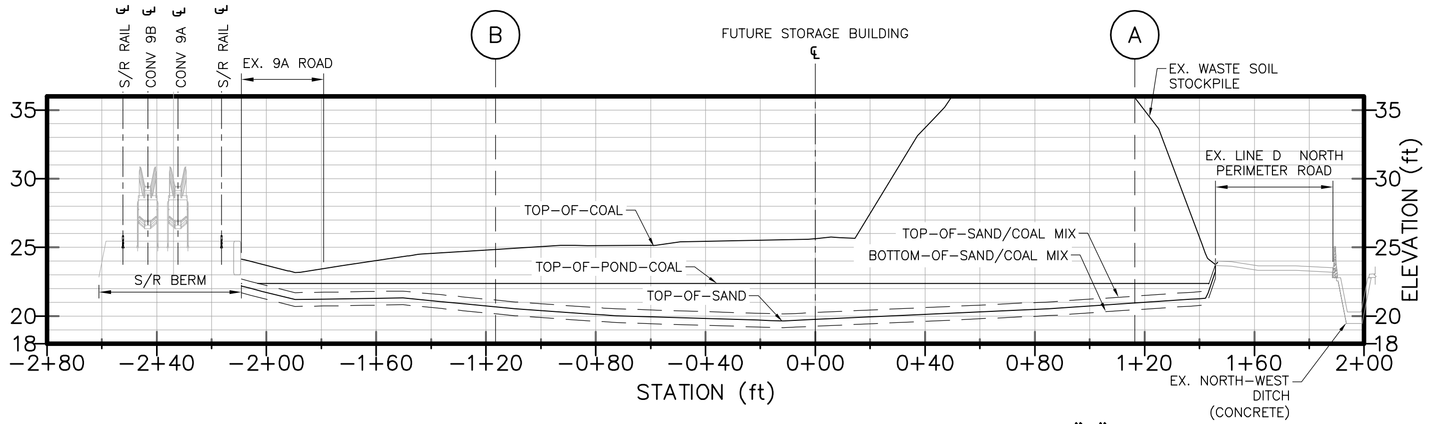
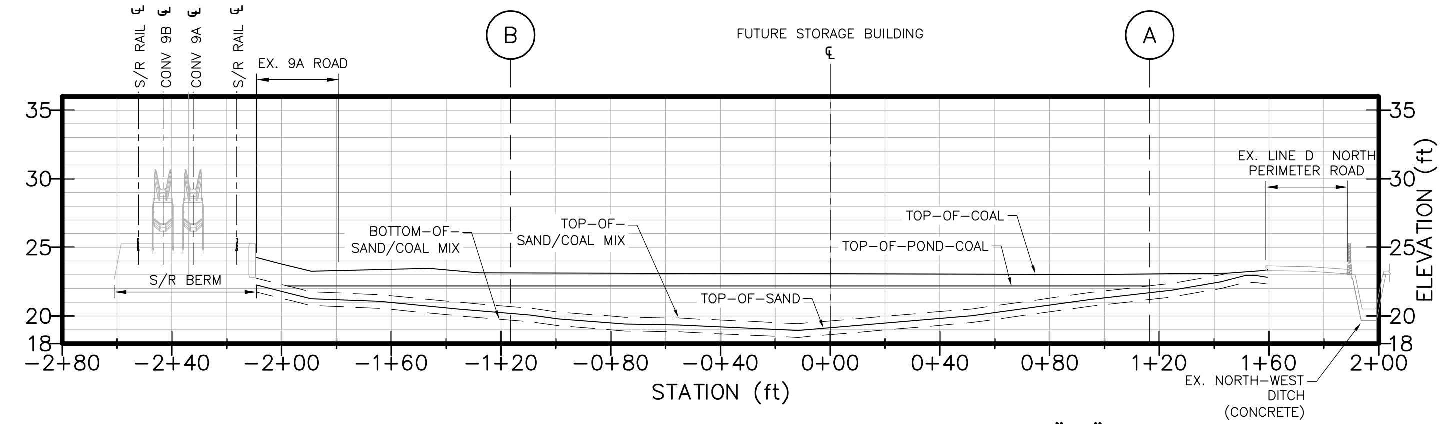



PROFILE – EXISTING STRATA – BAYLINE "1"

SCALE: H: 1"=35', V: 1"=7'

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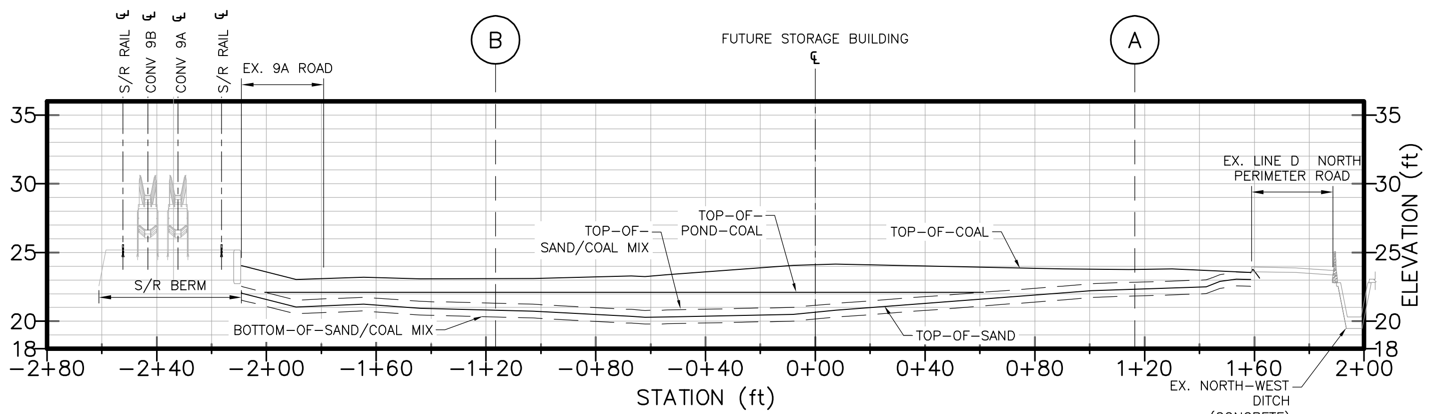


PROFILE – EXISTING STRATA – BAYLINE "51"

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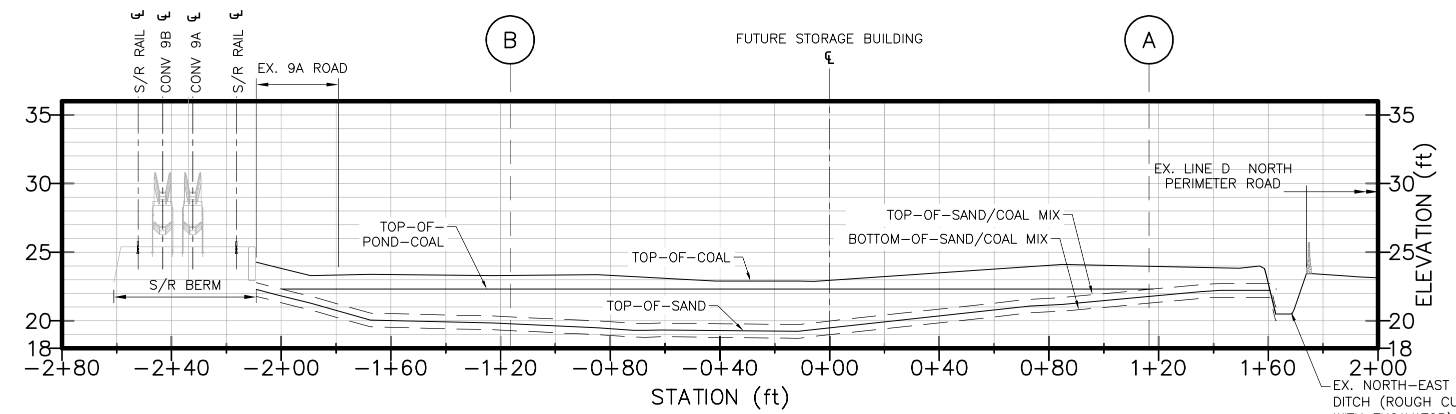


PROFILE – EXISTING STRATA – BAYLINE "76"

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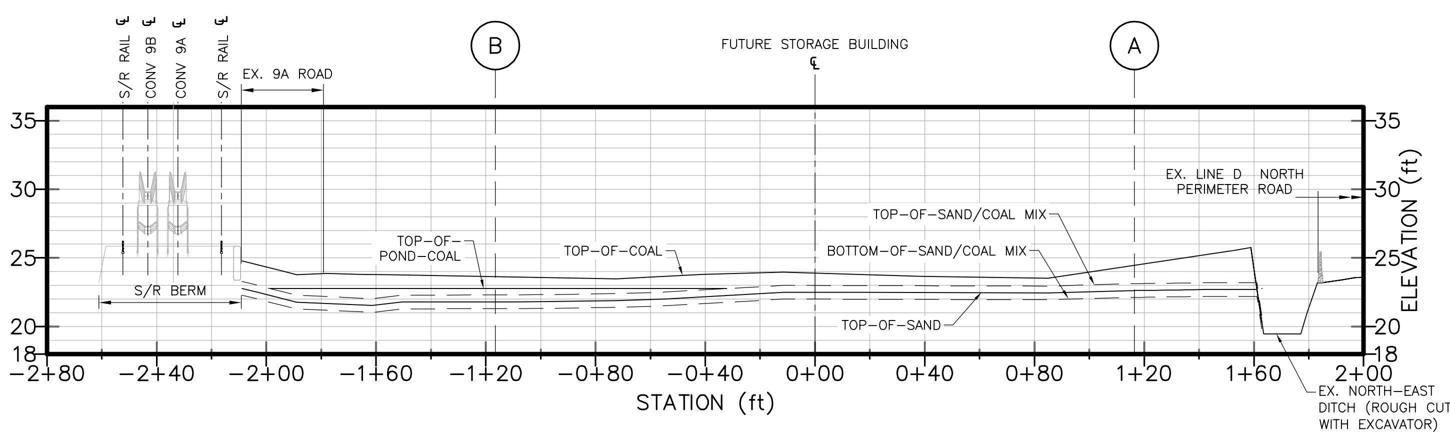
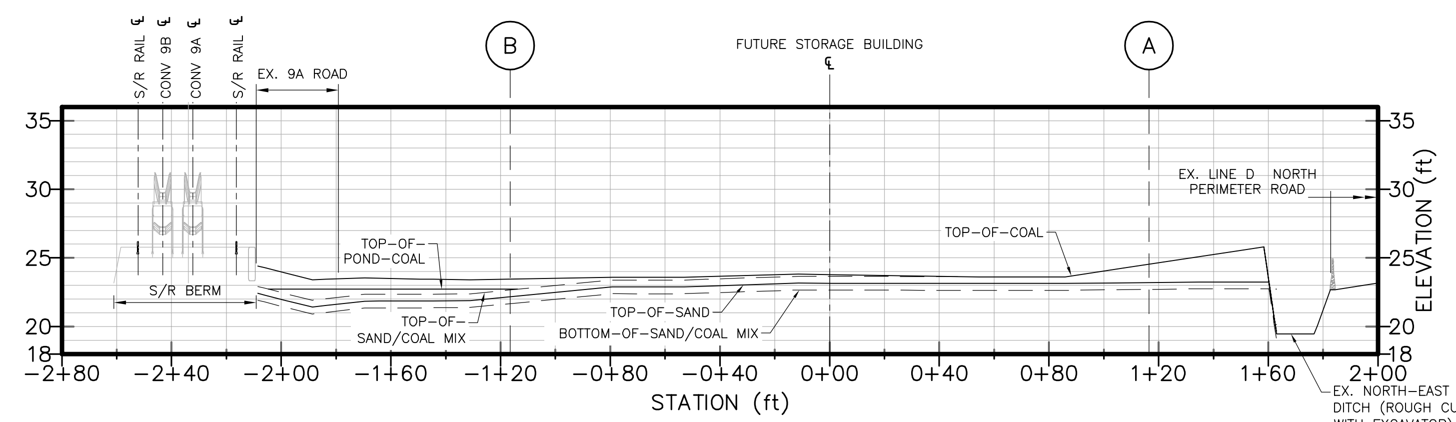



PROFILE – EXISTING STRATA – BAYLINE "101"

SCALE: H: 1"=35', V: 1"=6'




PROFILE – EXISTING STRATA – BAYLINE "110"

SCALE: H: 1"=35', V: 1"=7'

NOTES:

1) ELEVATIONS OF "TOP-OF-COAL" AND "TOP-OF-SAND" MEASURED BY BINNIE GPS SURVEY DURING TEST EXCAVATIONS (MAY 2020)
2) ELEVATIONS OF "TOP-OF-POND-COAL" CALCULATED BASED ON S/R'S MAXIMUM OPERATING LUFF ANGLE
3) ELEVATIONS OF "TOP-OF-SAND/COAL MIX" AND "BOTTOM-OF-SAND/COAL MIX" WERE VERTICALLY OFFSET FROM "TOP-OF-SAND" MIX BASED ON OBSERVATIONS DURING TEST EXCAVATIONS
4) STORAGE BUILDING LAYOUT AS PER CWA'S "40101-D0000-0000" (UNOFFICIAL) DRAWING, RECEIVED ON 2020/11/20. STORAGE BUILDING CROSS SECTION AS PER CWA'S "85400-D0010-0116 (P2)" DATED 2020/11/13

PRELIMINARY

| No | By | Date | Revision |
|---|---|---|---|
| P5 | MS | 20/12/18 | ISSUED FOR USE - STAGE 2 STUDY |
| P4 | DY | 20/10/30 | ISSUED FOR REVIEW |
| P3 | DY | 20/07/14 | ISSUED FOR REVIEW |
| P2 | DY | 20/05/27 | ISSUED FOR REVIEW |
| P1 | MS | 20/05/22 | ISSUED FOR REVIEW |

STAMP

Client: Westshore Terminals LP

Consultant: BINNIE
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300 - 4940 Canada Way,
Burnaby, BC V5G 4K6
TEL 604 420 1721
BINNIE.com

Project Title: NEW CARGO STUDY

Title: EARTHWORKS STORAGE BUILDING AREA EXISTING CROSS-SECTIONS

| Design MS | Drawn DY | Checked CV | Plot Scale |
|---|---|---|---|
| Date 20/12/18 | Date 20/12/18 | Date 20/12/18 | AS NOTED |

Drawing Number 85400-D0010-0011 | Revision Number P5

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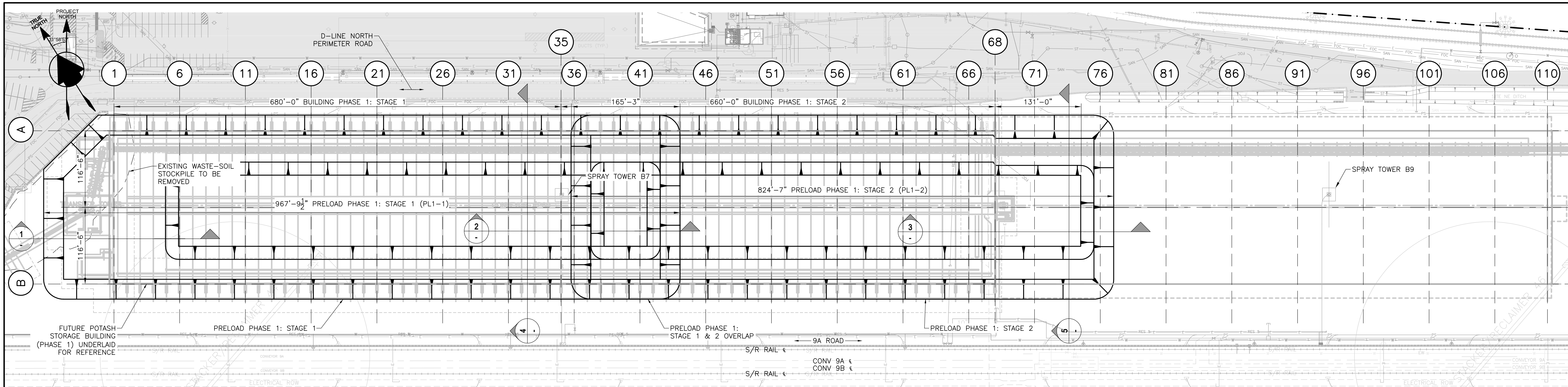
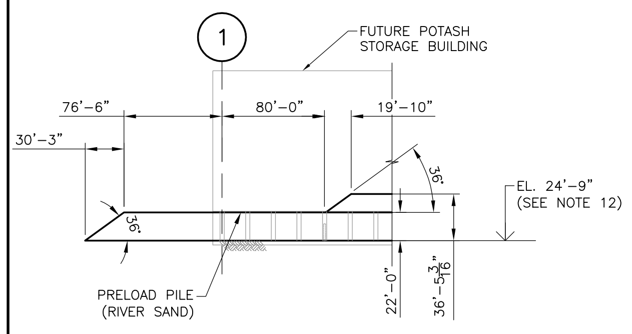


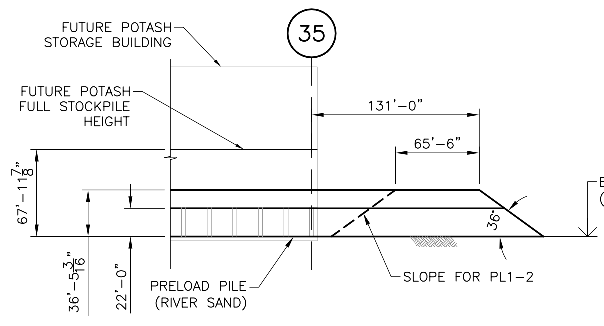
PLAN

SCALE: 1"=75'



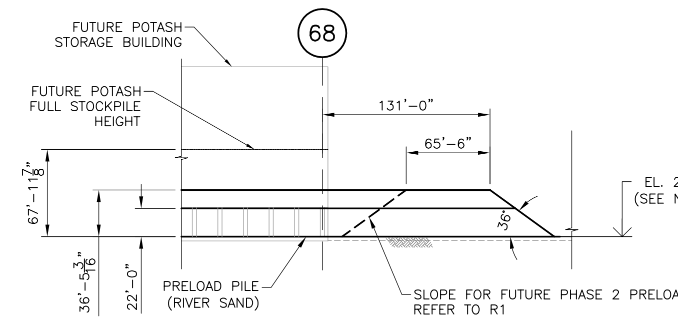
SECTION 1

SCALE: 1"=75'



SECTION 2

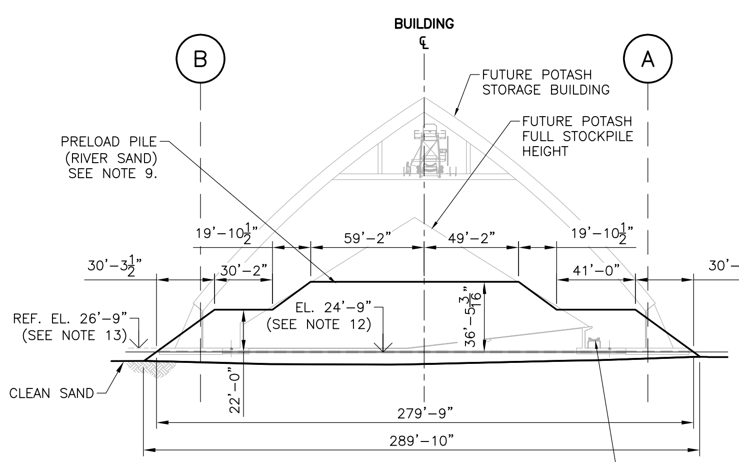
SCALE: 1"=75'



SECTION 3

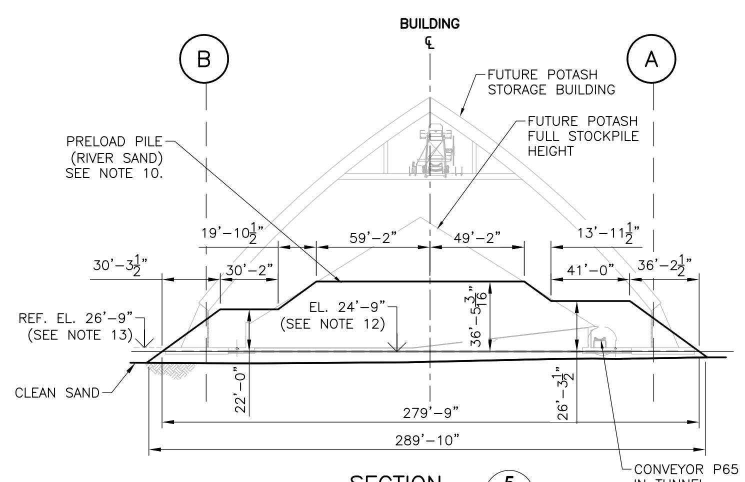
SCALE: 1"=75'

| STORAGE BUILDING – PRELOAD VOLUMES & WEIGHTS – TABLE 1 | | |
|---|---|---|
| PL1 TOTAL SAND IMPORTED | 7,619,000 CU.FT | 377,000 t |
| PL1–1 PRELOAD SAND REQUIRED | 7,619,000 CU.FT | 377,000 t |
| PL1–1 FILL SAND REQUIRED | 1,076,000 CU.FT | 54,000 t |
| PL1–2 PRELOAD SAND REQUIRED | 6,618,000 CU.FT | 328,000 t |
| PL1–2 FILL SAND REQUIRED | 1,046,000 CU.FT | 52,000 t |
| TOTAL SAND REMOVED | 5,497,000 CU.FT | 272,000 t |



SECTION 4

SCALE: 1"=50'



SECTION 5

SCALE: 1"=50'

## NOTES:

1) ALL EXISTING INFORMATION ON STRUCTURES, MONUMENTS, BURIED / UNDERGROUND SERVICES AND UTILITIES ARE BASED ON AVAILABLE RECORDS AND SHOULD NOT BE CONSTRUED AS COMPLETE OR ACCURATE.
2) ALL EXISTING UTILITIES ARE SHOWN SCHEMATICALLY WITH ALIGNMENTS AND LOCATIONS OF UTILITY STRUCTURE POTENTIALLY SHIFTED FOR CLARITY.
3) ALL DIMENSIONS ARE IN IMPERIAL UNITS UNLESS NOTED OTHERWISE.
4) ALL ELEVATIONS ARE SHOWN IN CANADIAN HYDROGRAPHIC (CHS) DATUM AND REFERENCED TO THE DEEP BENCHMARK.
5) COORDINATES SHOWN ARE BASED ON A PROJECT GROUND COORDINATE SYSTEM.
6) FOR CIVIL GENERAL NOTES, REFER TO GENERAL ARRANGEMENT DRAWINGS 88200–D0007–1110 TO 88200–D0007–1116.
7) FOR EARTHWORKS SEQUENCING DRAWINGS, REFER TO DRAWINGS 85400–D0010–0020 TO 85400–D0010–0026
8) FOR EARTHWORKS CUT/FILL DRAWINGS, REFER TO DRAWINGS 85400–D0010–0010 TO 85400–D0010–0014
9) GENERAL PRELOAD CROSS–SECTIONS BASED ON SKETCHES FROM GEOTECHNICAL ENGINEERING MARCH 12, 2021
10) PRELOAD CROSS–SECTION OVER CONVEYOR P65 TUNNEL BASED ON SKETCHES FROM GEOTECHNICAL ENGINEER ON MARCH 25, 2021
11) PRELOAD SHAPES/HEIGHTS BASED ON PLACED DENSITY OF 109LBS / CU.FT
12) PRELOAD WEIGHTS PROVIDED IN METRIC TONNES(t)
13) THE BASE ELEVATION FOR PRELOAD HEIGHTS IS THE FOUNDATIONS' LONG–TERM SETTLEMENT T.O.C. EL. OF 24–9"
14) REFERENCE ELEVATIONS IS THE FOUNDATIONS' POST–CONSTRUCTION T.O.C. EL. OF 26'–9"
15) ABBREVIATIONS
    – EW1–1 = EARTHWORKS PHASE 1: STAGE 1 (CWA STAGE 1A)
    – EW1–2 = EARTHWORKS PHASE 1: STAGE 2 (CWA STAGE 1B)
    – EW2 = EARTHWORKS PHASE 2 (CWA STAGE 2)
16) ABBREVIATIONS
    – PL1–1 = PRELOAD PHASE 1: STAGE 1 (CWA STAGE 1A)
    – PL1–2 = PRELOAD PHASE 1: STAGE 2 (CWA STAGE 1B)
    – PL2 = PRELOAD PHASE 2 (CWA STAGE 2)

PRELIMINARY

| Ref. | Document Number | Description |
|---|---|---|
| 1 | 85400–D0010–0035 | |

| No | Date | Revision Description | Drawn | Drawing Check | Design | Design Check | Disc. Approval | Proj. Approval |
|---|---|---|---|---|---|---|---|---|
| P2 | 21/06/01 | ISSUED FOR USE – NEW CARGO STUDY | DY | MS | MS | CV | | |
| P1 | 21/04/23 | ISSUED FOR REVIEW | JL | CV | MS | CV | | |

STAMP

Client: Westshore Terminals LP

CONSULTANT: BINNIE — The people behind your infrastructure.

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Burnaby, BC V5G 4K6
TEL 604 420 1721
BINNIE.com

Proj Title: NEW CARGO STUDY

Dwg Title: EARTHWORKS STORAGE BUILDING AREA – PHASE 1 PRELOAD GENERAL ARRANGEMENT

Drawing Number: 85400–D0010–0030 | Plot Scale: AS NOTED | Rev: P2

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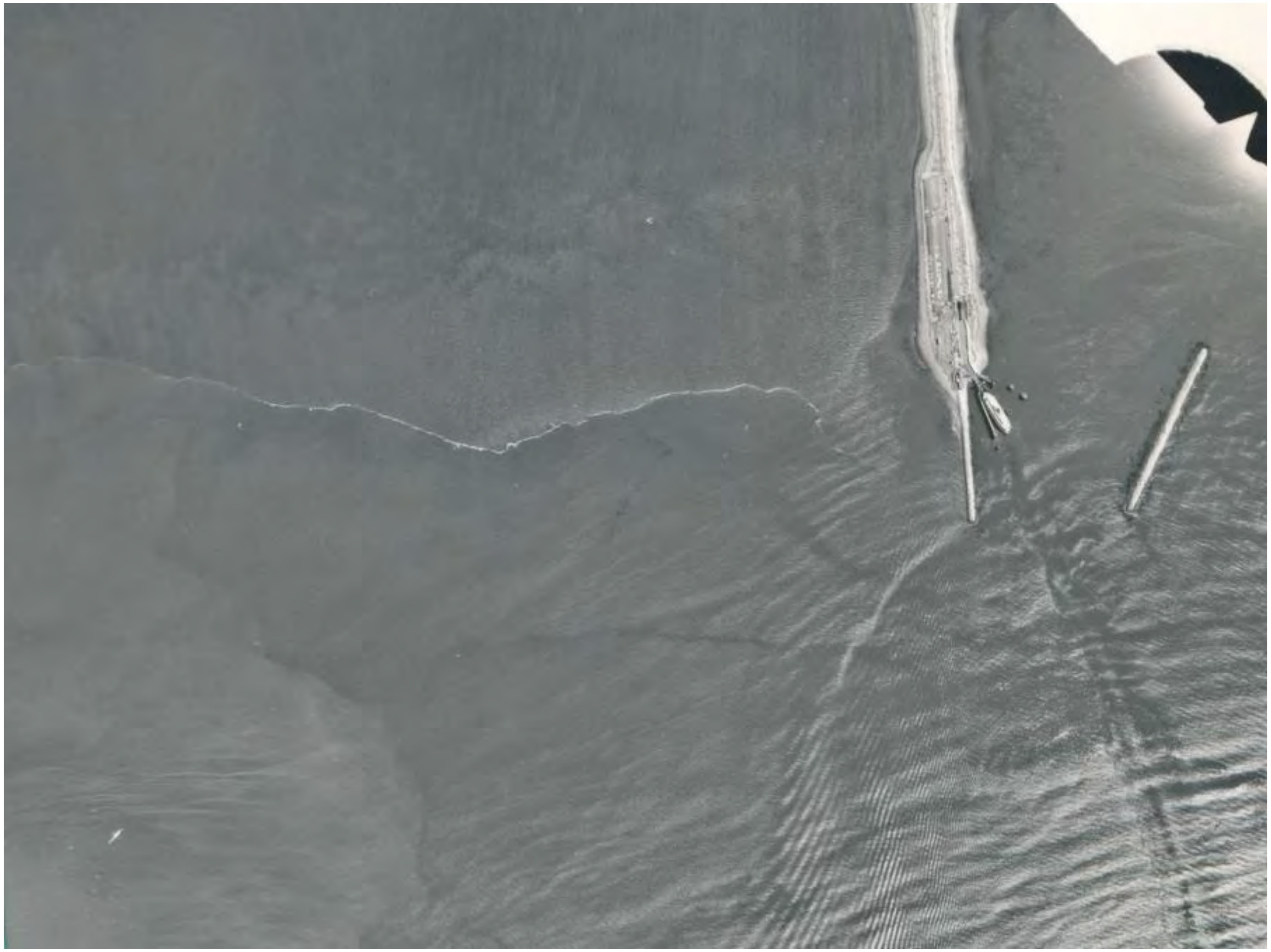
# Appendix H

# Historic Air Photos

Historical Air Photos



1966 – Prior to Construction, Ferry Terminal Visible

Historical Air Photos

1969 – Pod 1Fill Placement

Historical Air Photos

1974 – Pod 1 In Operation

Historical Air Photos

1979 – Similar to 1974

Historical Air Photos

1984 – Pods 2 to 4 constructed, coal storage expanded to Pod 2, Berth 1 constructed

Historical Air Photos

1990 – Similar to 1984

Historical Air Photos

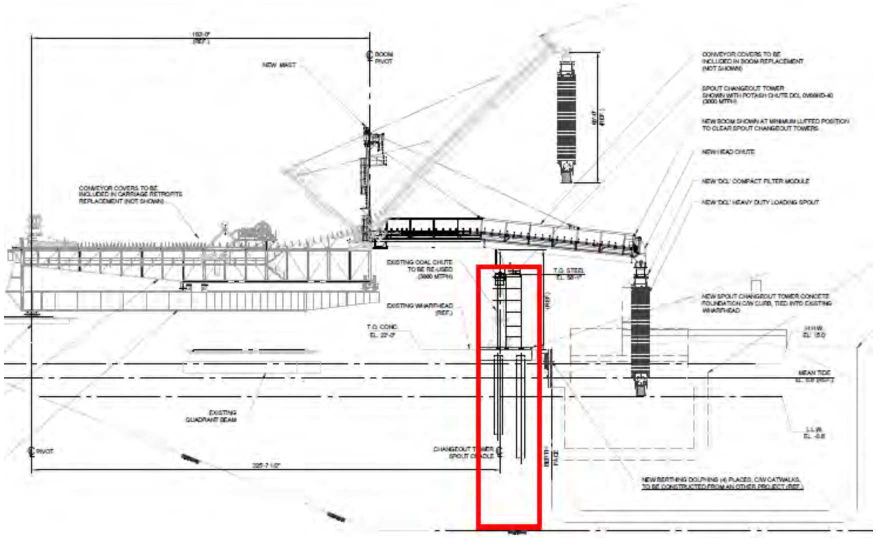
1997 – Coal Storage expanded to Pod 2 Row D, Deltaport constructed in Pod 4, Preloading in Pod 3

Historical Air Photos

2002 –Deltaport expanded to Pod 3

# <u>Appendix I</u>

# Assessment of Spout Tower Foundation Piles Subject to Seismic Kinematic Loading






BRAUN GEOTECHNICAL LTD.

Naesgaard-Amini Geotechnical Ltd.
soils, foundations and earthquake engineering

**Objective:**

Pile design of the proposed spout tower (Figure I-1).
Seismic design performance: No collapse/loss of life for 2475 year return period design earthquake.

**Methodology**

1. Software package used for analysis: Group 2019 by Ensoft
2. Static and seismic inertial loading provided by CWA (Figures I-2 & I-3)
3. Load Combinations used for seismic demand
    - 0% kinematic + 100% inertial + Vertical loads
    - 100% kinematic + 50% inertial + Vertical loads
    - 100% kinematic + 0% inertial + Vertical loads (Not Checked)
4. Connection between pile and pile cap was assumed to be fixed. The pile cap is assumed rigid.
5. Generalized soil stratigraphy and parameters estimated from the Marine CPT20-01 (performed by Thurber Engineering, Figures I-4 and I-5).
6. Slope geometry was obtained from the topographic information shown on Figure I-6.
7. Lateral spreading and flow slide failure cause significant horizontal and vertical soil displacements (Figure I-7).
8. Estimated soil lateral displacement profile (for kinematic loading evaluation) is obtained from the East-West FLAC analysis for a location approximately 70m east from the crest of the east slope (Figure I-8).
9. Initial design section was 48” x 1.75” steel pipe piles 75m long. The length of the piles and moment capacity were insufficient for the seismic loading.
10. The pile size and length was increased iteratively to a 60” x 1.625” 85m long piles.
11. Plastic moment capacity of 60” x 1.625” pile assumed to be 32,600 kN-m.
12. The results of the lateral pile analyses are shown in Figure I-9.

Figure I-1: Pile Numbering and Configuration

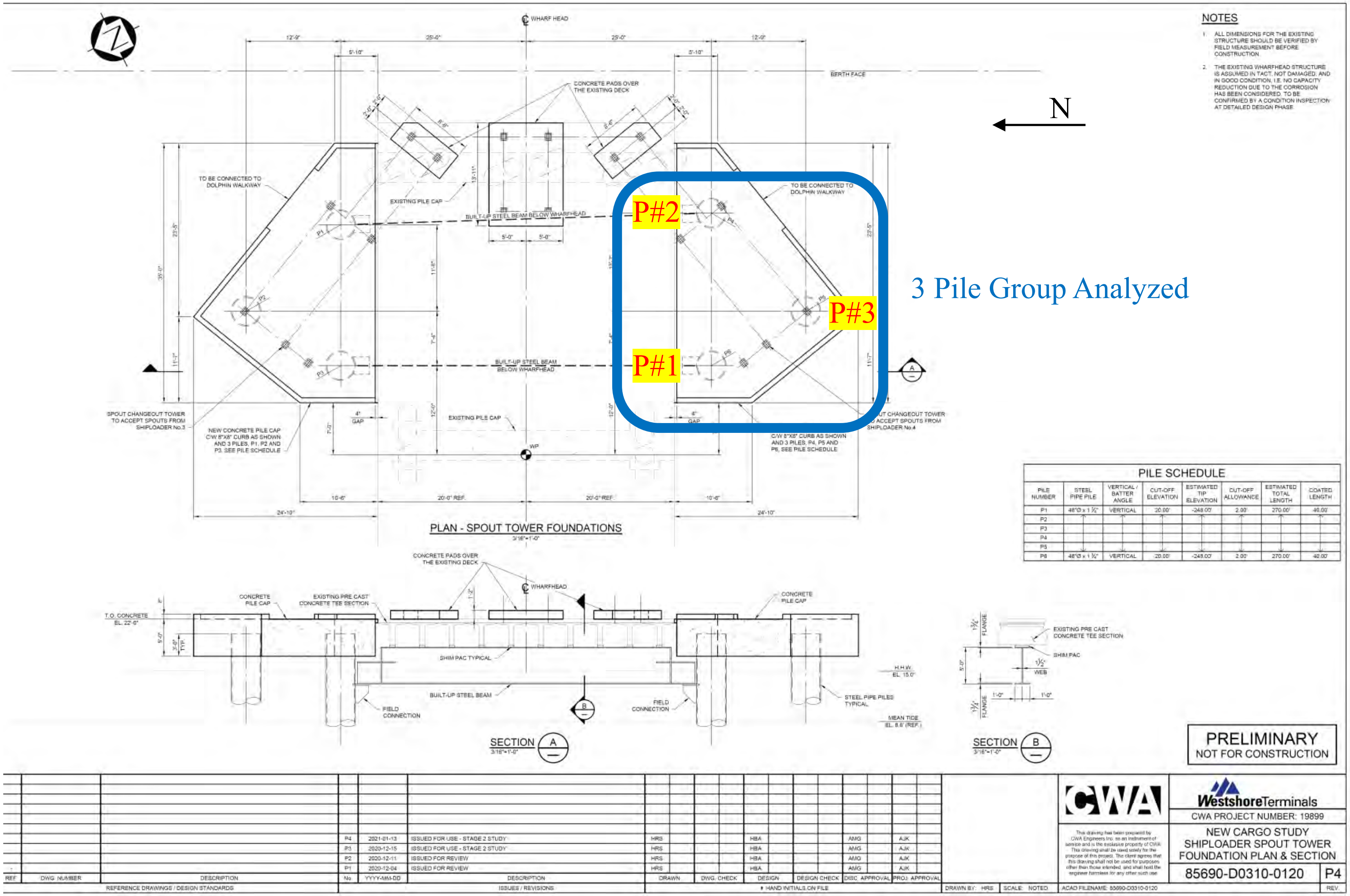


Figure I-2: Loading per Pile Provided by CWA
Note: pile self-weight not included

Unfactored Loading

| | Load Case | | | | |
|---|---|---|---|---|---|
| | Gravity Loads (Fy) | | | Lateral Loads (+/- Fx or Fz) | |
| | Dead | Live | Snow | Wind | Seismic |
| Pile No. | kN | kN | kN | kN | kN |
| P1 | 1640 | 680 | 80 | 50 | 520 |
| P2 | 1160 | 620 | 60 | 50 | 520 |
| P3 | 820 | 180 | 20 | 50 | 520 |
| P4 | 1640 | 680 | 80 | 50 | 520 |
| P5 | 1160 | 620 | 60 | 50 | 520 |
| P6 | 820 | 180 | 20 | 50 | 520 |

Factored Loading

Load Combinations

| | | |
|---|---|---|
| LC1 | 1.25D + 1.5L + 0.5S | |
| LC2 | 1.25D + 1.4W + 0.5L + 0.5S | |
| LC3 | 1.0D + 1.0E +0.25S | *NOTE: This load combination to be used with 50% Kinematic Loading |
| LC4 | 1.0D + 0.5E +0.25S | *NOTE: This load combination to be used with 100% Kinematic Loading |
| | E= Seismic inertial loading | |

LC4 Considered the critical load combination

| | LC1 | LC2 | | LC3 | | LC4 | |
|---|---|---|---|---|---|---|---|
| Pile No. | Gravity Loads (Fy) | Gravity Loads (Fy) | Lateral Loads (+/- Fx or Fz) | Gravity Loads (Fy) | Lateral Loads (+/- Fx or Fz) | Gravity Loads (Fy) | Lateral Loads (+/- Fx or Fz) |
| | kN | kN | kN | kN | kN | kN | kN |
| P1 | 3100 | 2450 | 70 | 1700 | 520 | 1700 | 260 |
| P2 | 2400 | 1800 | 70 | 1200 | 520 | 1200 | 260 |
| P3 | 1300 | 1150 | 70 | 850 | 520 | 850 | 260 |
| P4 | 3100 | 2450 | 70 | 1700 | 520 | 1700 | 260 |
| P5 | 2400 | 1800 | 70 | 1200 | 520 | 1200 | 260 |
| P6 | 1300 | 1150 | 70 | 850 | 520 | 850 | 260 |

Figure I-3: Loading per Pile Group Provided by CWA

**LOADING PER PILE GROUP**

NOTE: Pile Group 1 includes P1, P2 and P3
Pile Group 2 includes P4, P5 and P6
Pile Group 1 and Pile Group 2 assumed to be equal

Unfactored Loading

| | Load Case | | | | |
|---|---|---|---|---|---|
| | Gravity Loads (Fy) | | | Lateral Loads (+/- Fx or Fz) | |
| | Dead | Live | Snow | Wind | Seismic |
| | kN | kN | kN | kN | kN |
| **Pile Group Loading** | 3620 | 1480 | 160 | 150 | 1560 |

Factored Loading

| | LC1 | LC2 | | LC3 | | LC4 | |
|---|---|---|---|---|---|---|---|
| | Gravity Loads (Fy) | Gravity Loads (Fy) | Lateral Loads (+/- Fx or Fz) | Gravity Loads (Fy) | Lateral Loads (+/- Fx or Fz) | Gravity Loads (Fy) | Lateral Loads (+/- Fx or Fz) |
| | kN | kN | kN | kN | kN | kN | kN |
| **Pile Group Loading** | 6800 | 5400 | 210 | 3750 | 1560 | 3750 | 780 |

Load Combination at Pile Cap Used in Analyses

Load Combinations

| | | |
|---|---|---|
| LC1 | 1.25D + 1.5L + 0.5S | |
| LC2 | 1.25D + 1.4W + 0.5L + 0.5S | |
| LC3 | 1.0D + 1.0E +0.25S | *NOTE: This load combination to be used with 50% Kinematic Loading |
| LC4 | 1.0D + 0.5E +0.25S | *NOTE: This load combination to be used with 100% Kinematic Loading |

Figure I-4: Nearby Test Hole Information

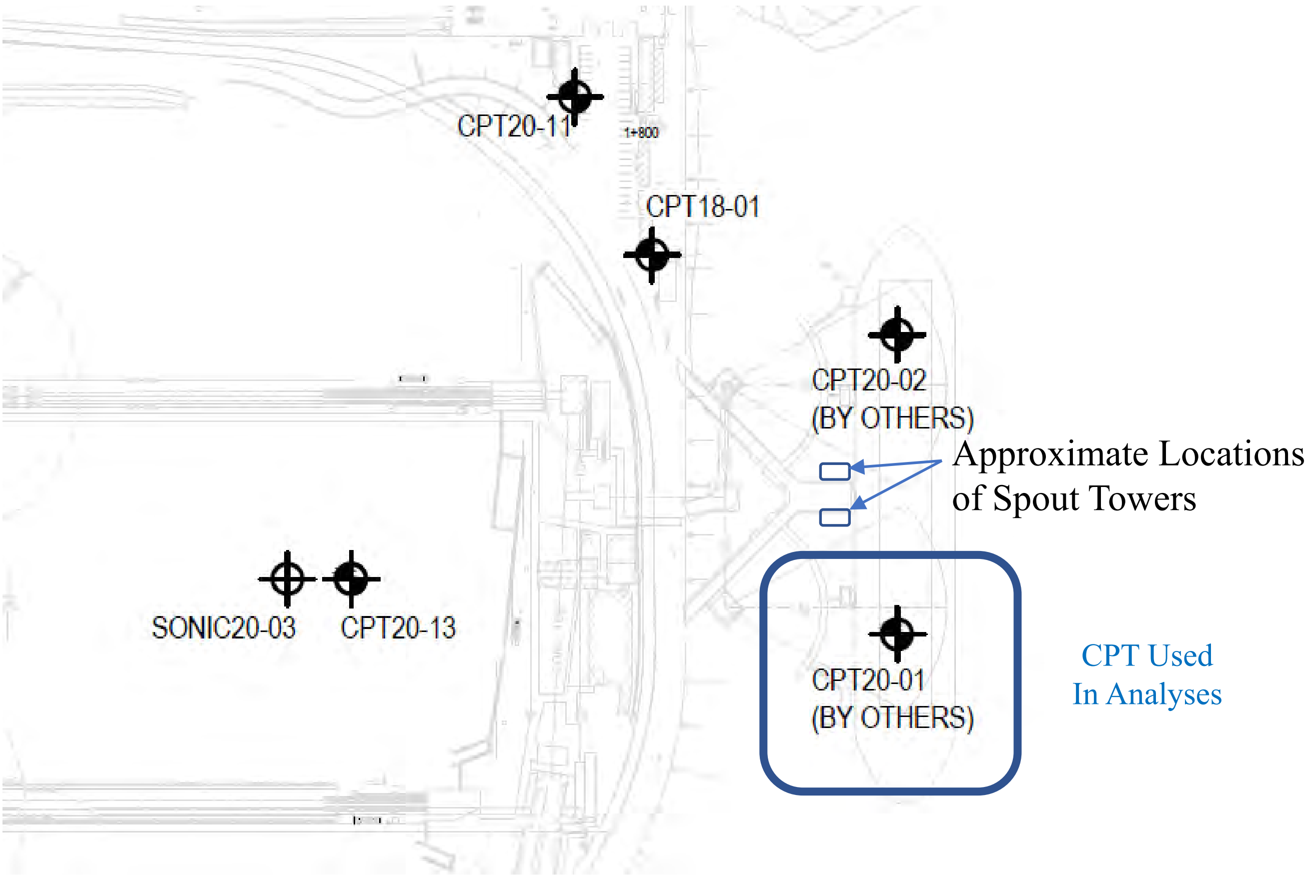
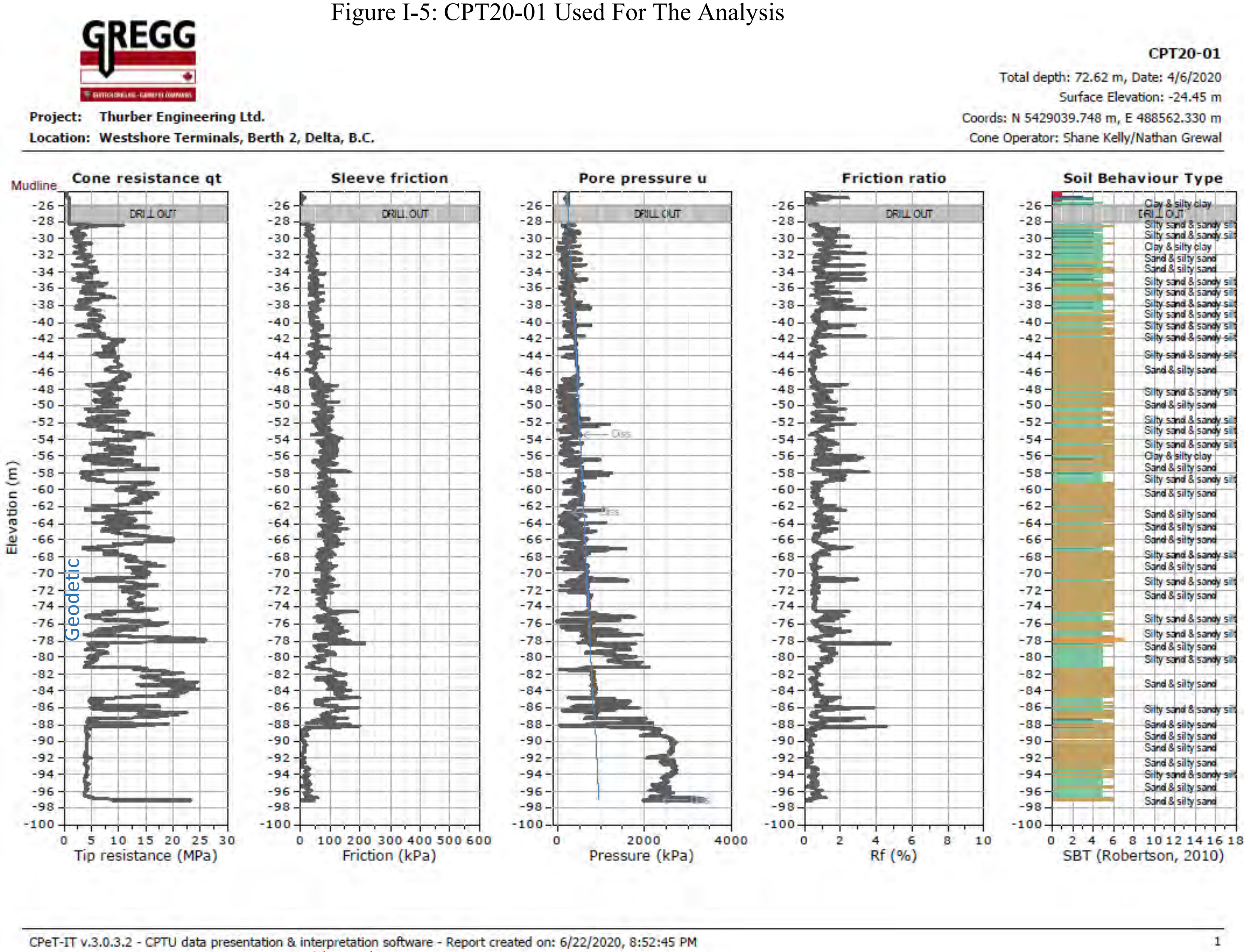


Figure I-5: CPT20-01 Used For The Analysis




| Chart datum | Geodetic datum |
|---|---|
| Mudline el. -21.5m | Mudline el. -24.45m |
| Till el. -94.5m | Till el. -97.55m |

Figure I-6: Slope Configuration

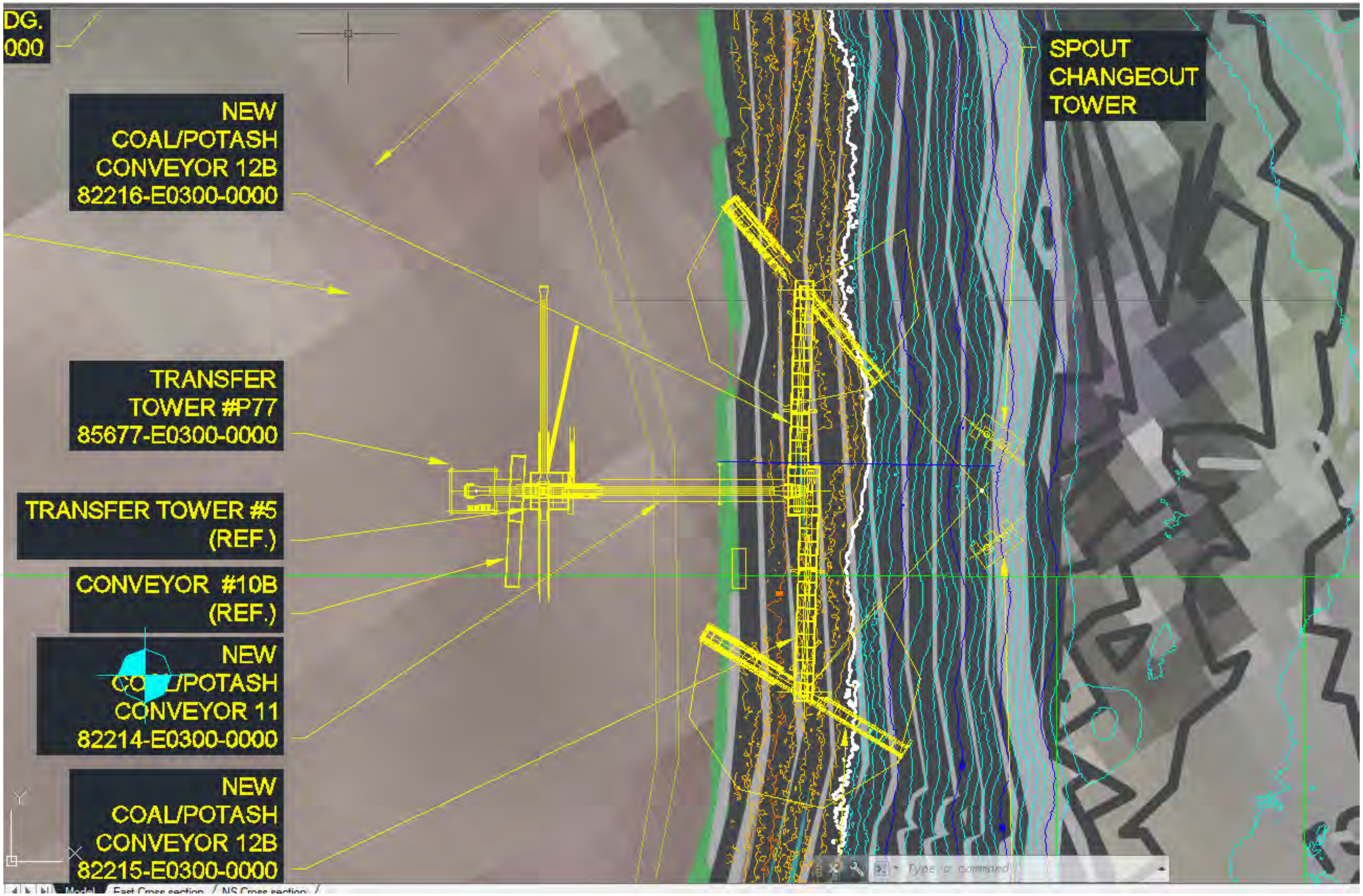


Figure I-7: Flow Slide Pattern
Post-EQ-CRU03-2475 with coal stockpiles

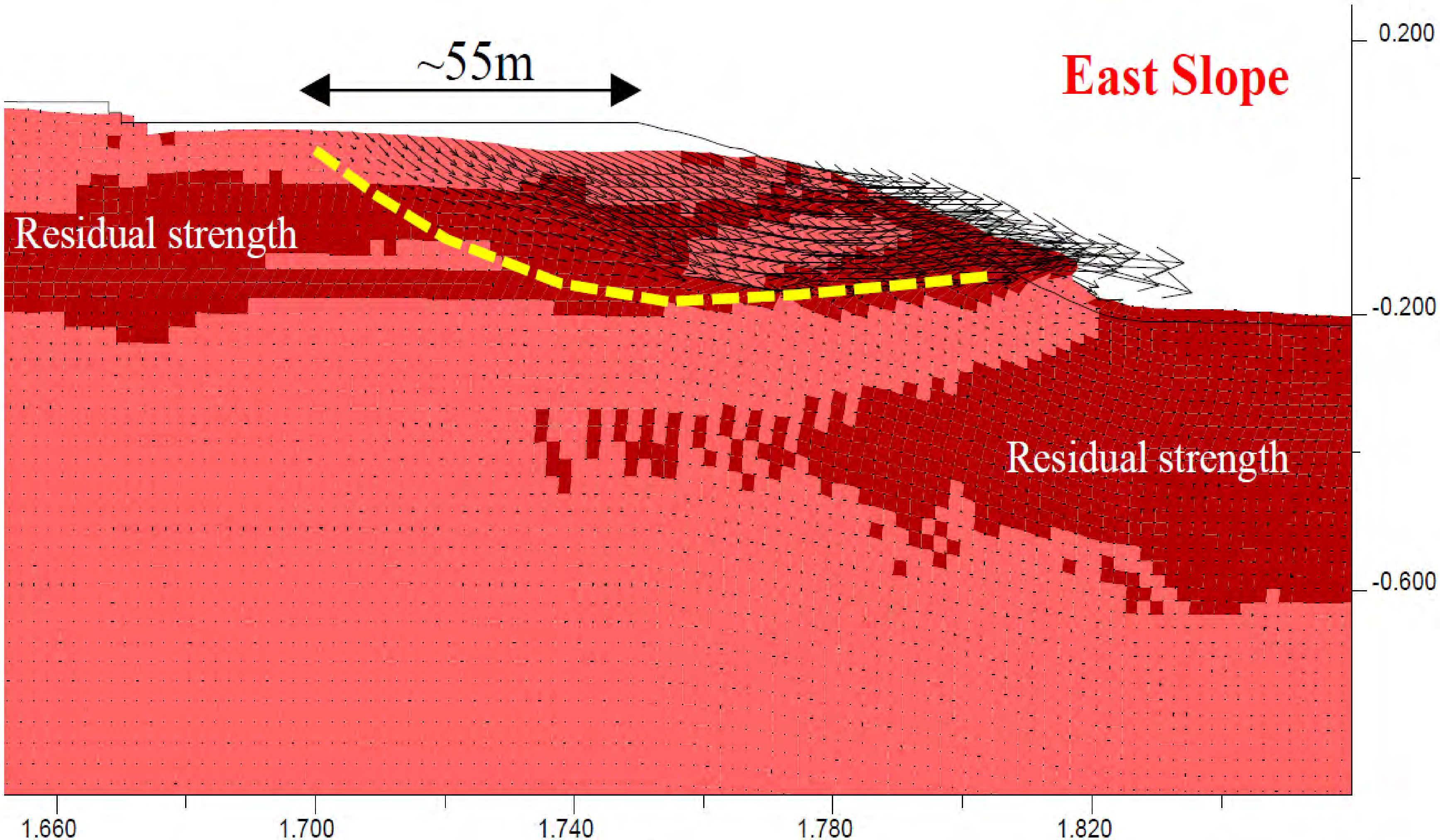


Figure I-8: Slope Deformation Used In Assessment

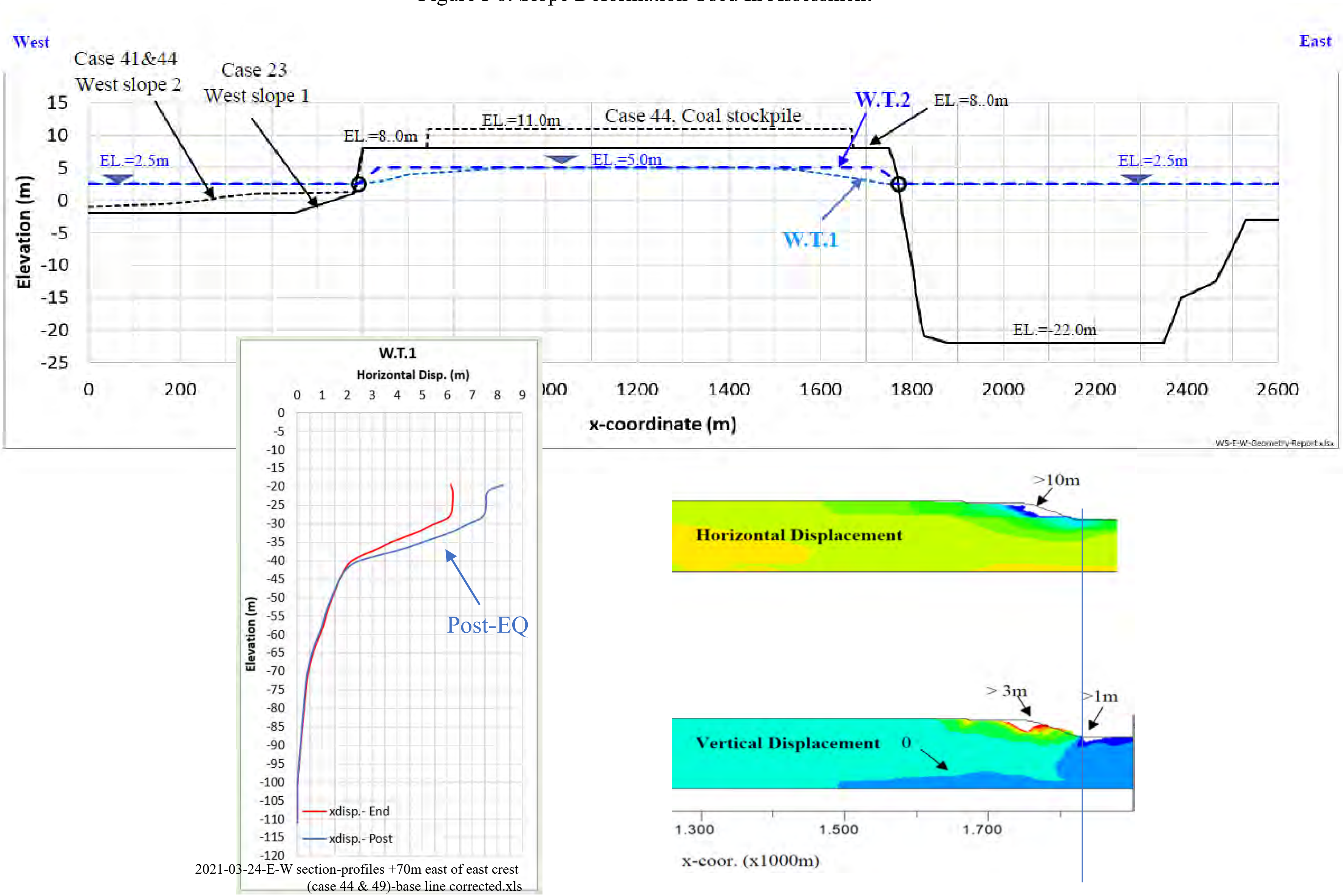
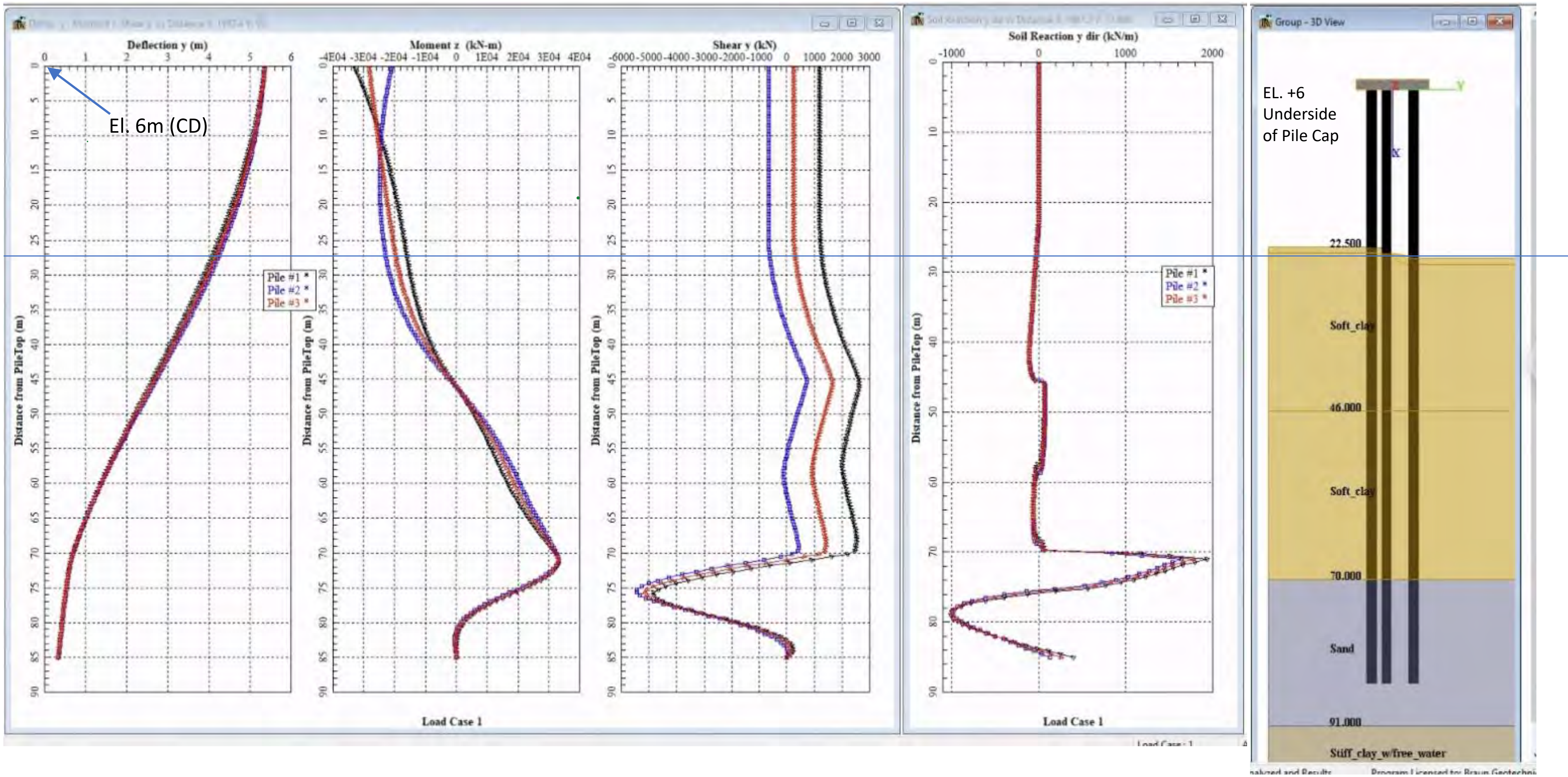


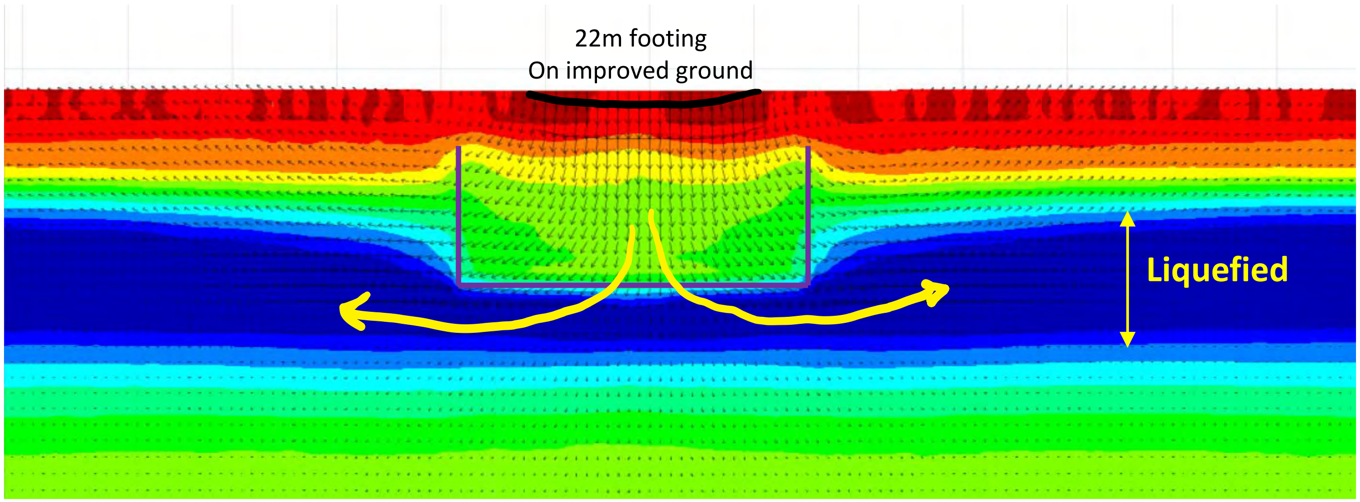
Figure I-9 Results (preliminary- under internal review)
(vertical axis is distance below underside of pile cap)




20-8543 Spout tower_Rev1_2021-07-10_soft clay (sloped ground).gp11d

# Appendix J

## Footing Spring Constants for In-Bound Structures



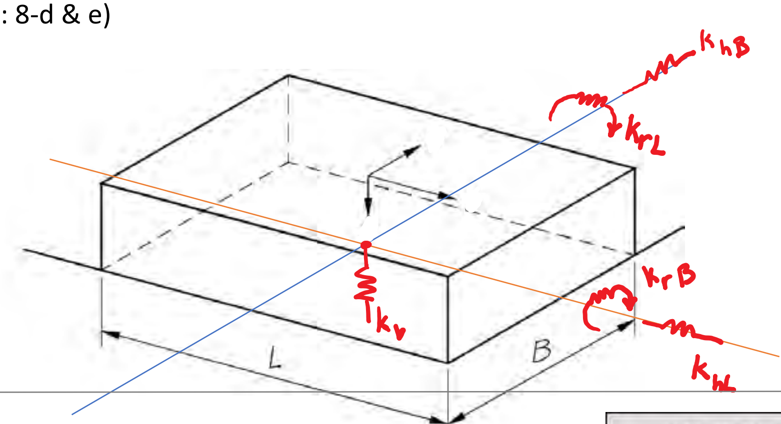
# Summary of Results and Notes

Table J-1- Summary of estimated footing spring stiffness constants for liquefied and non-liquefied cases for the In-Bound Structures

| In-Bound Structures | | Footing Dimensions | | | | Shape Irregularity | Distance b/w loading columns | | Soil Type | Ground Improvement Depth | Footing Dead Load | | Estimated Footing Stiffness (see sketch below) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | | Not- Liquefied | | | | | Liquefied | | | | |
| | | | | | | | | | | | | | Vertical | in Direction B | | in Direction L | | Vertical | in Direction B | | in Direction L | |
| Struc # | Type | B | L | T | L/B | | Dir. B | Dir.L | | | Load | Pressure | Kv | KhB | KrB | KhL | KrL | Kv | KhB | KrB | KhL | KrL |
| | | m | m | m | - | | m | m | | m | kN | kPa | kN/m | kN/m | kN.m/m | kN/m | kN.m/Rad | kN/m | kN/m | kN.m/m | kN/m | kN.m/Rad |
| **P40-B1** | bent | 9 | 41 | 1.2 | 4.6 | Yes | 7.5 | 3.0 | IB1 | - | 1280 | 3 | 2.9.E+06 | 1.4.E+06 | 3.3.E+07 | 3.0.E+06 | 3.2.E+07 | 7.0.E+05 | 5.9.E+05 | 2.0.E+07 | 7.5.E+05 | 1.8.E+07 |
| **P40-B2** | bent | 7.9 | 15.2 | 0.9 | 1.9 | - | 4.8 | 11.6 | IB1 | - | 605 | 5 | 3.2.E+06 | 1.3.E+06 | 2.2.E+07 | 1.5.E+06 | 4.8.E+07 | 7.5.E+05 | 6.3.E+05 | 1.7.E+07 | 4.8.E+05 | 2.2.E+07 |
| **P40-B3** | bent | 8.5 | 13.7 | 0.9 | 1.6 | - | 1.0 | 9.2 | IB1 | - | 1290 | 11 | 3.1.E+06 | 1.4.E+06 | 2.6.E+07 | 1.3.E+06 | 3.5.E+07 | 7.3.E+05 | 6.5.E+05 | 2.0.E+07 | 4.2.E+05 | 1.7.E+07 |
| **P40-B4** | bent | 5.2 | 5.2 | 0.9 | 1.0 | - | 1.0 | 1.0 | IB1 | - | 1550 | 57 | 1.1.E+06 | 5.3.E+05 | 3.9.E+06 | 5.3.E+05 | 3.9.E+06 | 2.2.E+05 | 2.3.E+05 | 3.2.E+06 | 2.3.E+05 | 3.3.E+06 |
| **P42** | TT | 20.5 | 22 | 1.2 | 1.1 | - | 6.7 | 8.0 | IB1 | - | 2115 | 5 | 6.4.E+06 | 3.1.E+06 | 1.3.E+08 | 3.2.E+06 | 1.6.E+08 | 1.0.E+06 | 7.8.E+05 | 8.2.E+07 | 8.2.E+05 | 9.8.E+07 |
| **P45-B1** | bent | 6.1 | 15.3 | 0.9 | 2.5 | Yes | 1.0 | 1.0 | IB1 | - | 1500 | 17 | 2.1.E+06 | 9.3.E+05 | 1.0.E+07 | 1.3.E+06 | 1.3.E+07 | 5.3.E+05 | 4.5.E+05 | 7.8.E+06 | 4.1.E+05 | 9.7.E+06 |
| **P45-B2** | bent | 6.7 | 16.1 | 0.9 | 2.4 | - | 1.0 | 7.6 | IB1 | - | 1400 | 13 | 3.0.E+06 | 1.1.E+06 | 1.4.E+07 | 1.5.E+06 | 5.2.E+07 | 6.7.E+05 | 5.2.E+05 | 1.1.E+07 | 4.8.E+05 | 2.5.E+07 |
| **P45-B3** | bent | 7.3 | 16.7 | 0.9 | 2.3 | - | 4.9 | 9.2 | IB1 | - | 2570 | 21 | 3.3.E+06 | 1.3.E+06 | 1.9.E+07 | 1.7.E+06 | 6.3.E+07 | 7.5.E+05 | 6.0.E+05 | 1.5.E+07 | 5.2.E+05 | 2.9.E+07 |
| **P45-B4** | bent | 6.7 | 19.5 | 0.9 | 2.9 | - | 1.0 | 13.4 | IB1 | - | 1610 | 12 | 3.5.E+06 | 1.2.E+06 | 1.7.E+07 | 1.8.E+06 | 9.5.E+07 | 8.1.E+05 | 5.9.E+05 | 1.3.E+07 | 5.3.E+05 | 3.2.E+07 |
| **P47** | TT | 22.7 | 23.2 | 1.2 | 1.0 | Yes | 9.0 | 12.0 | IB1 | - | 3335 | 7 | 7.6.E+06 | 3.6.E+06 | 3.1.E+08 | 3.6.E+06 | 3.3.E+08 | 1.2.E+06 | 9.3.E+05 | 1.1.E+08 | 9.4.E+05 | 1.2.E+08 |
| **P50-B1** | bent | 5.8 | 18.9 | 0.9 | 3.3 | - | 1.0 | 15.0 | IB1GI3 | 10 | 1230 | 11 | 3.0.E+06 | 1.2.E+06 | 1.4.E+07 | 1.4.E+06 | 5.0.E+07 | 8.2.E+05 | 5.4.E+05 | 1.2.E+07 | 4.1.E+05 | 1.7.E+07 |
| **P52** | TT | 19.8 | 23.3 | 0.9 | 1.2 | Yes | 16.5 | 15.2 | IB1GI1 | 18 | 4730 | 10 | 8.7.E+06 | 3.3.E+06 | 2.6.E+08 | 3.8.E+06 | 3.8.E+08 | 3.7.E+06 | 1.2.E+06 | 1.7.E+08 | 1.4.E+06 | 2.6.E+08 |
| **P57** | TT | 11.3 | 17.2 | 0.9 | 1.5 | Yes | 8.8 | 7.0 | IB1GI2 | 18 | 1575 | 8 | 4.9.E+06 | 1.9.E+06 | 6.7.E+07 | 1.9.E+06 | 9.6.E+07 | 2.2.E+06 | 9.0.E+05 | 4.7.E+07 | 7.0.E+05 | 6.5.E+07 |

Westshore-footing spring.xlsx

See notes on the next page including suggested upper and lower range (Notes: 8-d & e)

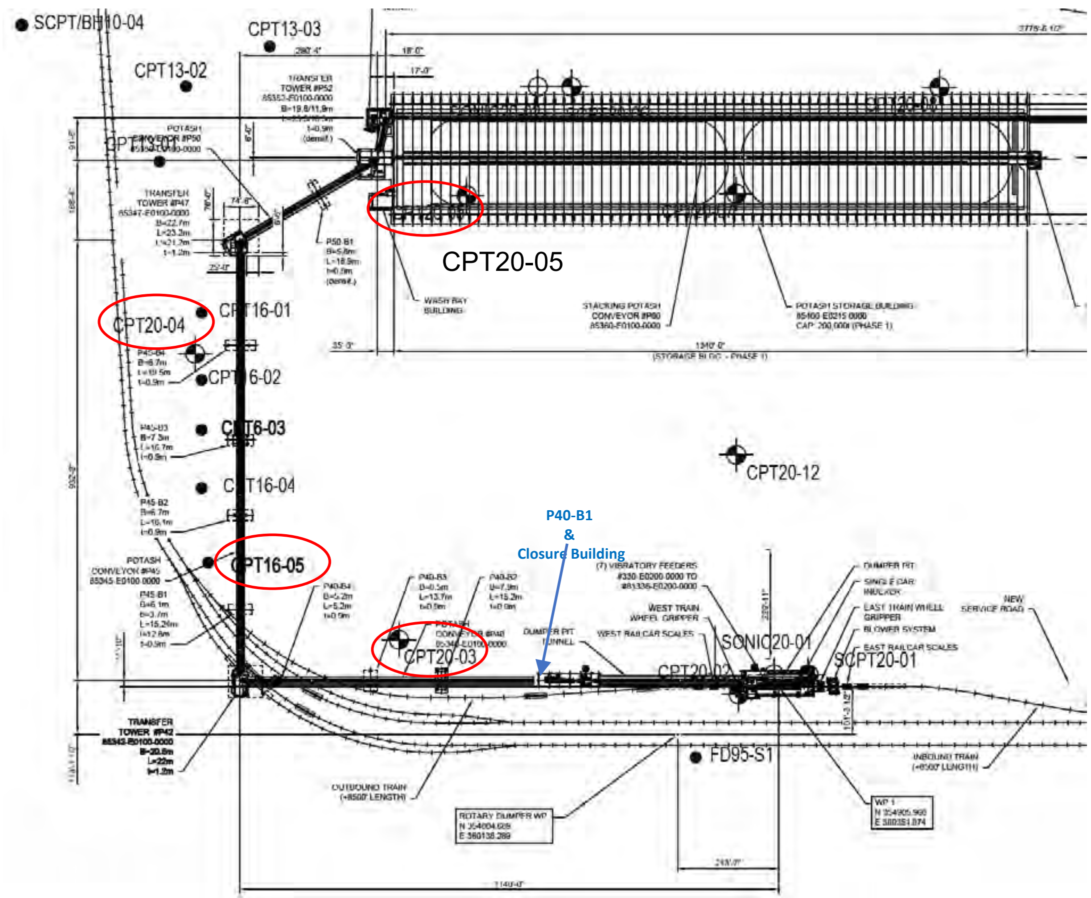


# Notes from Table J-1 and Methodology

1- All footings were assumed to be symmetric with respect to shape and loading conditions except TT P42 for which unsymmetric loading was considered.
2- Some footings have Irregular shapes (e.g. a cut out). These footings were approximated with a rectangle shape.
3- The proposed footings supporting the in-bound structures were categorized based on the following parameters in the order of importance:
 - Footing dimensions, B & L
 - Footing thickness, T
 - Distance between the loading columns
 - Soil type (with or without ground improvement, GI)
 One typical in-situ soil profile was assumed for the in-bound footings (IB1).
 Widths and depths of GI were considered for the zones proposed to be densified (immediately west of the Storage Building).
 In IB1GI1 and IB1GI2, ground improvement extends about 7.5m beyond the footing edges and 18m depth below ground surface.
 In IB1GI3, ground improvement was extends about 5m beyond the footing edges and 10m depth below ground surface.
 Normalized cone tip resistance in the improved ground was assumedqc1ncs=160
 - Dead load
4- Special cases
 Transfer Tower TTP42 was analyzed as a special case due to its relative importance and unsymmetric loading.
 P40-B1 was analyzed as a special case due to its unsymmetric loading and connection of its footing to the closure building footing.
5- Pushover loading in FLAC 2D:
 One cycle of loading was applied for each degree of freedom (h=horizontal, v=vertial, r=rocking)
 Pushover loading in each direction was carried out independently from loading in the other degrees of freedom.
 Deadload, DL + footing self weight, Wf were added before all pushover analyses. This represents the initial condition before earthquake loading.
 Horizontal pushover loading range: +/- 0.45(DL + Wf)
 Vertical pushover loading range: +(DL + Wf) to -0.8(DL + Wf)
 Rocking pushover loading range: Eccentricity in the range of e = ± L/3
6- Push-over analyses described in Bullet #5 were carried out for Liquefied soil conditions (near end of earthquake) and non-liquefied conditions (before earthquake).
7- Load-displacement curves obtained from FLAC 2D analyses represent the behavior of the unit length of a long strip footing.
 As per request of CWA, the load-displacement curves were simplified using a constant stiffness for each degree of freedom.
 FLAC 2D stiffnesses were then corrected approximately to consider the 3D effects based on elastic solutions (Gazetas 1991- Chapter 15- Geotechnical Eng. Handbook by Fang 1991 and WSDOT 2018- Bridge Design Manual LRFD) and out of plane dimension of the footing.
 Limited number of FLAC analyses, a total of 60 analyses, were carried out to approximately cover the range of variation of footing geometry, soil conditions and loading conditions.
 60 = 10 (combination of footing size, soil type, loading distance, etc) x 2 (liquefied and not liquefied) x 3 (degrees of freedom in the plane of analysis)
 More details of the techncial assumptions will be provided in a memorandum.
8- Limitations and uncertainties:
 a- 2D to 3D correction of footing stiffness is approximate, particularly for the liquefied case. The gross section properties were used (the cracked sections were not used).
 b- The effect of footing flexibility has been considered in the plane of analysis by considering the stiffness of the concrete pad and approximate location of the loading columns. The effect of footing flexibility in the out of plane of analysis has not been considered. This results in overestimating the footing stiffness.
 c- The pushover analyses were performed for one cycle of analysis. The stiffnesses reflect the soil behavior in each cycle and do not include the cyclic marching displacements of the footing. In addition, the stiffnesses do not include long term consolidation settlement and post-earthquake reconsolidation settlement.
 d- Generally there are considerable uncertainties in modelling of cyclic soil behavior, soil-structure interaction, soil variabilities and simplifying assumptions. It is prudent to consider a lower and upper range of 0.5 to 2 times the estimated stiffnesses (ASCE/SEI 41-17).
 e- The configuration/interaction of the closure building footing and Conveyor P40-B1 is complicated. An equivalent footing of 9m x 16 m (instead of ~9m x ~41m) was assumed for estimation of footing springs.If the behavior of the structure is sensitive to the stiffness of P40-B1, then a wider range of upper & lower stiffness (1/3 to 3 times estimated values) may be used for this footing (to be discussed with the design team).

Continued from Table J-1

In-Bound structure, foundations and test hole locations



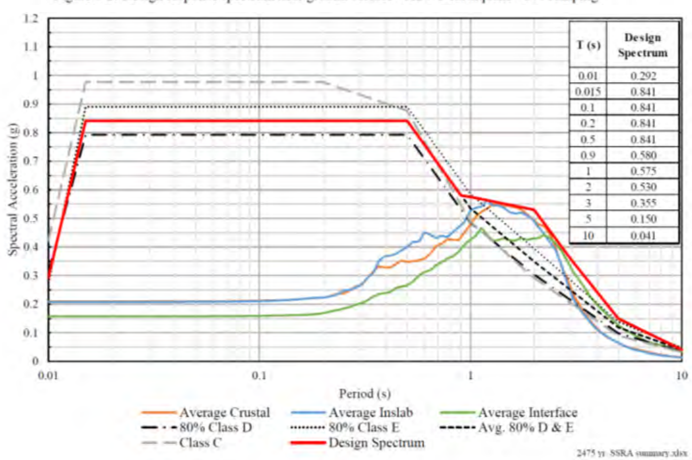
Continued from Table 1

## Comparison of footing stiffness values with ASCE-SEI-41 approximate method

Note: The comparison has been made for non-liquefied conditions only and for overall checking purposes.

# Comparison with ASCE-SEI-41 Method (for non-liquefied only, in B direction)

Figure 6-2: Design response spectrum near ground surface - A2475 earthquake - 5% damping




Sxs/2.5=0.85/2.5=0.34
G/Gmax=0.58~0.6 (interpolation from the table for Site Class D)

**Table 8-2. Effective Shear Modulus Ratio ($G/G_0$)**

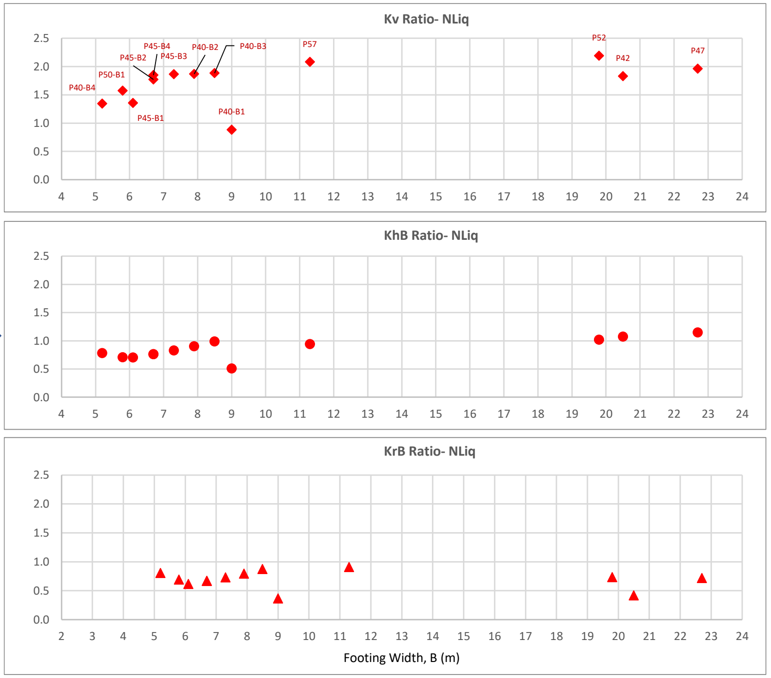
| Site Class | Effective Peak Acceleration, $S_{XS}/2.5^{a}$ $S_{XS}/2.5=0$ | $S_{XS}/2.5=0.1$ | $S_{XS}/2.5=0.4$ | $S_{XS}/2.5=0.8$ |
|---|---|---|---|---|
| A | 1.00 | 1.00 | 1.00 | 1.00 |
| B | 1.00 | 1.00 | 0.95 | 0.90 |
| C | 1.00 | 0.95 | 0.75 | 0.60 |
| D | 1.00 | 0.90 | 0.50 | 0.10 |
| E | 1.00 | 0.60 | 0.05 | b |
| F | b | b | b | b |

[a] Use straight-line interpolation for intermediate values of $S_{XS}/2.5$.
[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

$S_{XS}$ = Spectral response acceleration parameter at short periods for the selected Seismic Hazard Level and damping, adjusted for Site Class

| In-Bound Structures | | Footing Dimensions | | ASCE 41 Simplified Method | | | | Footing Elastic Stiffness (Gazetas)- NLiq in B Direction | | | FLAC: Footing Stiffness + Shape factor- NLiq in B Direction | | | Comparison: Ratio (FLAC/ASCE 41) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Influence depth (2B) | SCPT20-01 | | ASCE41 | | | | | | | | | |
| Struc # | Type | B | L | | Vs_ave | Gmax_ave | G/Gmax | Kv | KhB | KrB | Kv | KrB | KhL | Kv | KhB | KrB |
| | | m | m | m | m/s | kN/m2 | (-) | kN/m | kN/m | kN.m/m | kN/m | kN.m/m | kN/m | kN/m | kN.m/Rad | |
| P40-B1 | bent | 9 | 41 | 18 | 198 | 7.4.E+04 | 0.6 | 3.2.E+06 | 2.8.E+06 | 8.9.E+07 | 2.9.E+06 | 1.4.E+06 | 3.3.E+07 | 0.9 | 0.5 | 0.4 |
| P40-B2 | bent | 7.9 | 15.2 | 16 | 201 | 7.7.E+04 | 0.6 | 1.7.E+06 | 1.4.E+06 | 2.8.E+07 | 3.2.E+06 | 1.3.E+06 | 2.2.E+07 | 1.9 | 0.9 | 0.8 |
| P40-B3 | bent | 8.5 | 13.7 | 17 | 199 | 7.5.E+04 | 0.6 | 1.7.E+06 | 1.4.E+06 | 2.9.E+07 | 3.1.E+06 | 1.4.E+06 | 2.6.E+07 | 1.9 | 1.0 | 0.9 |
| P40-B4 | bent | 5.2 | 5.2 | 10 | 205 | 8.0.E+04 | 0.6 | 8.4.E+05 | 6.7.E+05 | 4.8.E+06 | 1.1.E+06 | 5.3.E+05 | 3.9.E+06 | 1.3 | 0.8 | 0.8 |
| P42 | TT | 20.5 | 22 | 41 | 208 | 8.2.E+04 | 0.6 | 3.5.E+06 | 2.8.E+06 | 3.2.E+08 | 6.4.E+06 | 3.1.E+06 | 1.3.E+08 | 1.8 | 1.1 | 0.4 |
| P45-B1 | bent | 6.1 | 15.3 | 12 | 201 | 7.7.E+04 | 0.6 | 1.6.E+06 | 1.3.E+06 | 1.6.E+07 | 2.1.E+06 | 9.3.E+05 | 1.0.E+07 | 1.4 | 0.7 | 0.6 |
| P45-B2 | bent | 6.7 | 16.1 | 13 | 202 | 7.8.E+04 | 0.6 | 1.7.E+06 | 1.4.E+06 | 2.1.E+07 | 3.0.E+06 | 1.1.E+06 | 1.4.E+07 | 1.8 | 0.8 | 0.7 |
| P45-B3 | bent | 7.3 | 16.7 | 15 | 202 | 7.8.E+04 | 0.6 | 1.8.E+06 | 1.5.E+06 | 2.6.E+07 | 3.3.E+06 | 1.3.E+06 | 1.9.E+07 | 1.9 | 0.8 | 0.7 |
| P45-B4 | bent | 6.7 | 19.5 | 13 | 202 | 7.8.E+04 | 0.6 | 1.9.E+06 | 1.6.E+06 | 2.5.E+07 | 3.5.E+06 | 1.2.E+06 | 1.7.E+07 | 1.8 | 0.8 | 0.7 |
| P47 | TT | 22.7 | 23.2 | 45 | 210 | 8.4.E+04 | 0.6 | 3.9.E+06 | 3.1.E+06 | 4.3.E+08 | 7.6.E+06 | 3.6.E+06 | 3.1.E+08 | 2.0 | 1.1 | 0.7 |
| P50-B1 | bent | 5.8 | 18.9 | 12 | 211 | 8.5.E+04 | 0.6 | 1.9.E+06 | 1.6.E+06 | 2.0.E+07 | 3.0.E+06 | 1.2.E+06 | 1.4.E+07 | 1.6 | 0.7 | 0.7 |
| P52 | TT | 19.8 | 23.3 | 40 | 220 | 9.2.E+04 | 0.6 | 4.0.E+06 | 3.2.E+06 | 3.5.E+08 | 8.7.E+06 | 3.3.E+06 | 2.6.E+08 | 2.2 | 1.0 | 0.7 |
| P57 | TT | 11.3 | 17.2 | 23 | 210 | 8.4.E+04 | 0.6 | 2.4.E+06 | 2.0.E+06 | 7.3.E+07 | 4.9.E+06 | 1.9.E+06 | 6.7.E+07 | 2.1 | 0.9 | 0.9 |

Westshore-footing spring.xlsx




## Conclusion:

The calculated non-liquefied footing stiffnesses in B direction is in the range of 0.5 to 2 times the ASCE recommendations. This is deemed reasonable.

# Background

# An example of loading provided by CWA

**Tower P42** **(Z = South; X = East)**

| | | X (kN) | Y (kN) | Z (kN) | Mx (kN-m) | Mz (kN-m) |
|---|---|---|---|---|---|---|
| Dead Load | D | 30 | -1800 | 0 | 4800 | 500 |
| Snow Load | S | 0 | -235 | 0 | 760 | 160 |
| Live Load | L | 0 | -200 | 0 | 625 | 200 |
| Material Load | LLn | 0 | -100 | 0 | 475 | 115 |
| Belt Tension | Bop | 1000 | 0 | -200 | 2150 | 17400 |
| Wind Load (+X) | W | 500 | 0 | 0 | 0 | 6050 |
| Wind Load (-X) | W | -500 | 0 | 0 | 0 | -6050 |
| Wind Load (+Z) | W | 0 | 0 | 515 | -765 | 0 |
| Wind Load (-Z) | W | 0 | 0 | -515 | 765 | 0 |
| EQ Load (+X) | E | 2050 | 105 | 0 | 360 | 28500 |
| EQ Load (-X) | E | -2050 | -105 | 0 | -360 | -28500 |
| EQ Load (+Z) | E | 0 | -45 | 2450 | -27800 | 50 |
| EQ Load (-Z) | E | 0 | 45 | -2450 | 27800 | -50 |

**Typical Bent** **(Z = Perpendicular to Conveyor; X = Parallel to Conveyor)**

| | | X (kN) | Y (kN) | Z (kN) | Mx (kN-m) |
|---|---|---|---|---|---|
| Dead Load | D | 0 | -1700 | 0 | 0 |
| Snow Load | S | 0 | -365 | 0 | 0 |
| Live Load | L | 0 | -200 | 0 | 0 |
| Material Load | LLn | 0 | -200 | 0 | 0 |
| Belt Tension | Bop | 0 | 0 | 0 | 0 |
| Wind Load (+Z) | W | 0 | 0 | 335 | 6400 |
| Wind Load (-Z) | W | 0 | 0 | -335 | -6400 |
| EQ Load (+Z) | E | 0 | 0 | 600 | 10600 |
| EQ Load (-Z) | E | 0 | 0 | -600 | -10600 |

## Simplified soil layering for In-Bound Structures

- Soil type: IB1- see below- not densified
- Soil type IB1-GI1 and IB1-GI1: as IB1 but the qc1ncs=155 for the upper 18m
- Soil type IB1-GI3 : as IB1 but the qc1ncs=155 for the upper 10m

# Summary of uncorrected load-displacements curves

# For the generic strip footing sizes analyzed in FLAC 2D

(Note: The corrected stiffnesses are presented in Table J-1)

Table J-2: Summary of FLAC pushover analyses- Stiffnesses are for 1m slice of a strip footing (infinite normal to the plane of analysis)

| | Footing | | | | Ground Conditions | | Interpreted Stiffness from FLAC: Not Liquefied | | | Interpreted Stiffness from FLAC: Liquefied | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Width in FLAC | thickness | loading Columns Distance | Equivalent Density including dead load | Ground Improvement Depth | Soil Profile | Kh | Kv | Kr | Kh | Kv | Kr |
| | m | m | m | kg/m3 | m | - | kN/m/mL | kN/m/mL | kN.m/Rad/mL | kN/m/mL | kN/m/mL | kN.m/Rad/mL |
| General Footings | 5 | 0.9 | 1 | 9000 | No | IB1 | 4.5E+04 | 7.0E+04 | 5.3E+05 | 2.0E+04 | 1.4E+04 | 4.4E+05 |
| General Footings | 6 | 0.9 | 1 | 4000 | 10 | IB1GI3 | 4.7E+04 | 8.5E+04 | 7.5E+05 | 2.2E+04 | 3.2E+04 | 6.5E+05 |
| General Footings | 7 | 0.9 | 1 | 4000 | No | IB1 | 4.7E+04 | 8.2E+04 | 9.1E+05 | 2.2E+04 | 2.4E+04 | 7.0E+05 |
| General Footings | 12 | 0.9 | 8 | 4000 | 18 | IB1GI2 | 6.5E+04 | 1.3E+05 | 4.0E+06 | 3.2E+04 | 6.4E+04 | 2.8E+06 |
| General Footings | 16 | 0.9 | 1 | 4000 | No | IB1 | 6.8E+04 | 8.1E+04 | 1.5E+06 | 2.1E+04 | 1.7E+04 | 1.1E+06 |
| General Footings | 16 | 0.9 | 8 | 4000 | No | IB1 | 7.0E+04 | 1.2E+05 | 4.6E+06 | 2.2E+04 | 2.1E+04 | 2.1E+06 |
| General Footings | 19 | 0.9 | 12 | 4000 | No | IB1 | 7.5E+04 | 1.3E+05 | 7.3E+06 | 2.2E+04 | 2.5E+04 | 2.5E+06 |
| General Footings | 22 | 1.2 | 12 | 4000 | No | IB1 | 7.7E+04 | 1.3E+05 | 9.7E+06 | 2.0E+04 | 2.1E+04 | 3.6E+06 |
| General Footings | 22 | 0.9 | 15 | 4000 | 18 | IB1GI1 | 8.5E+04 | 1.7E+05 | 1.3E+07 | 3.2E+04 | 7.3E+04 | 8.5E+06 |
| TT P42 | 22 | 1.2 | 8 | 2700 | No | IB1 | 7.8E+04 | 1.3E+05 | 6.1E+06 | 2.0E+04 | 2.0E+04 | 3.8E+06 |
| P40-B1 | 9 | 1.2 | 7 | 4000 | No | IB1 | 5.3E+04 | 9.0E+04 | 1.8E+06 | 2.2E+04 | 2.0E+04 | 1.1E+06 |
| P40-B1 | 41 | 0.6 to 1.2 | 3 | 2600 to 4100 | No | IB1 | 9.5E+04 | 8.1E+04 | 1.7E+06 | 2.4E+04 | 2.1E+04 | 9.6E+05 |

Westshore-footing spring.xlsx

Table J-3: Correction factors to be applied to FLAC stiffnesses to obtain footing stiffnesses

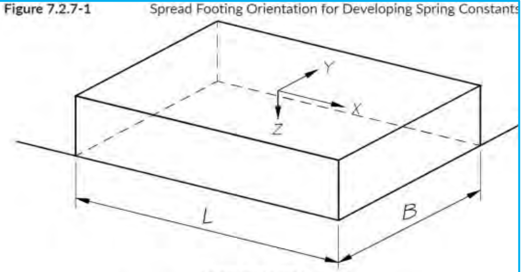
| In-Bound Structures | | Footing Dimensions | | | | Shape Irregularity | Distance b/w loading columns | | Soil Type | Ground Improvement Depth | Footing Dead Load | | FLAC footing Dimensions | | Correction Factor CF=f_size x shape factor x out of plane dimension | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | | | | Dir. B | | | Dir. L | | |
| Struc # | Type | B | L | T | L/B | | Dir. B | Dir.L | | | Load | Pressure | B' | L' | v | hB | rB | v | hL | rL |
| | | m | m | m | - | | m | m | | m | kN | kPa | m | m | - | - | - | - | - | - |
| **P40-B1** | bent | 9 | 41 | 1.2 | 4.6 | Yes | 7.5 | 3.0 | IB1 | - | 1280 | 3 | 9 | 41 | 35 | 27 | 18 | 35 | 31 | 19 |
| **P40-B2** | bent | 7.9 | 15.2 | 0.9 | 1.9 | - | 4.8 | 11.6 | IB1 | - | 605 | 5 | 7 | 16 | 37 | 28 | 25 | 29 | 21 | 10 |
| **P40-B3** | bent | 8.5 | 13.7 | 0.9 | 1.6 | - | 1.0 | 9.2 | IB1 | - | 1290 | 11 | 7 | 16 | 38 | 29 | 28 | 25 | 19 | 8 |
| **P40-B4** | bent | 5.2 | 5.2 | 0.9 | 1.0 | - | 1.0 | 1.0 | IB1 | - | 1550 | 57 | 5 | 5 | 16 | 12 | 7 | 16 | 12 | 7 |
| **P42** | TT | 20.5 | 22 | 1.2 | 1.1 | - | 6.7 | 8.0 | IB1 | - | 2115 | 5 | 22 | 22 | 49 | 40 | 22 | 51 | 42 | 26 |
| **P45-B1** | bent | 6.1 | 15.3 | 0.9 | 2.5 | Yes | 1.0 | 1.0 | IB1 | - | 1500 | 17 | 7 | 16 | 26 | 20 | 11 | 26 | 19 | 9 |
| **P45-B2** | bent | 6.7 | 16.1 | 0.9 | 2.4 | - | 1.0 | 7.6 | IB1 | - | 1400 | 13 | 7 | 16 | 30 | 23 | 16 | 29 | 22 | 11 |
| **P45-B3** | bent | 7.3 | 16.7 | 0.9 | 2.3 | - | 4.9 | 9.2 | IB1 | - | 2570 | 21 | 7 | 16 | 35 | 27 | 21 | 31 | 24 | 14 |
| **P45-B4** | bent | 6.7 | 19.5 | 0.9 | 2.9 | - | 1.0 | 13.4 | IB1 | - | 1610 | 12 | 7 | 19 | 34 | 26 | 18 | 31 | 24 | 13 |
| **P47** | TT | 22.7 | 23.2 | 1.2 | 1.0 | Yes | 9.0 | 12.0 | IB1 | - | 3335 | 7 | 22 | 22 | 57 | 46 | 32 | 57 | 47 | 34 |
| **P50-B1** | bent | 5.8 | 18.9 | 0.9 | 3.3 | - | 1.0 | 15.0 | IB1GI3 | 10 | 1230 | 11 | 6 | 22 | 32 | 25 | 18 | 25 | 19 | 7 |
| **P52** | TT | 19.8 | 23.3 | 0.9 | 1.2 | Yes | 16.5 | 15.2 | IB1GI1 | 18 | 4730 | 10 | 22 | 22 | 49 | 39 | 20 | 54 | 44 | 31 |
| **P57** | TT | 11.3 | 17.2 | 0.9 | 1.5 | Yes | 8.8 | 7.0 | IB1GI2 | 18 | 1575 | 8 | 12 | 22 | 37 | 29 | 17 | 29 | 22 | 8 |

Westshore-footing spring.xlsx

**Table 7.2.7-1** Stiffness of Foundation at Surface

| Degree of Freedom | $K_{sur}$ |
|---|---|
| Translation along x-axis | $\frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65}+1.2\right]$ |
| Translation along y-axis | $\frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65}+0.4\frac{L}{B}+0.8\right]$ |
| Translation along z-axis | $\frac{GB}{1-\nu}\left[1.55\left(\frac{L}{B}\right)^{0.75}+0.8\right]$ |
| Rocking about x-axis | $\frac{GB^3}{1-\nu}\left[0.4\left(\frac{L}{B}\right)+0.1\right]$ |
| Rocking about y-axis | $\frac{GB^3}{1-\nu}\left[0.47\left(\frac{L}{B}\right)^{2.4}+0.034\right]$ |

**Figure 7.2.7-1** Spread Footing Orientation for Developing Spring Constants




Orient axes such that $L > B$.
If $L = B$ use x-axis equations for both x-axis and y-axis.

Ref: WSDOT 2018- Bridge Design Manual (LRFD) M 23-50.18

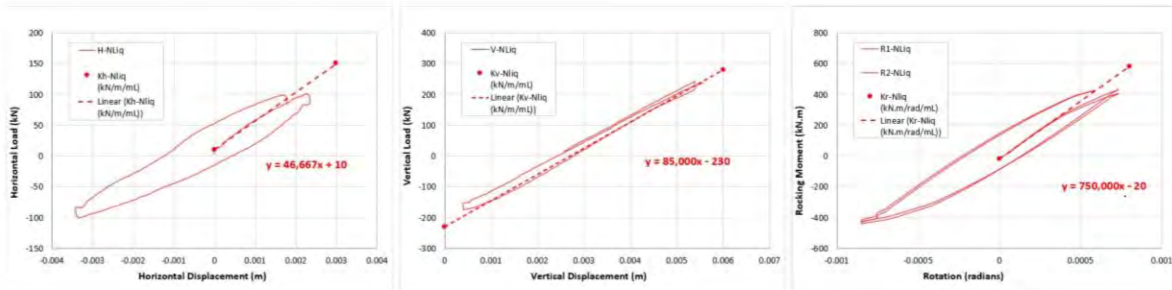
Load-displacement for Generic Footing Sizes from FLAC Push over analysis and interpretation of approximate constant stiffnesses

e name: Spring-06m-LD01-D4000-T90-IB1GI3.xlsx
Date: February 7, 2021

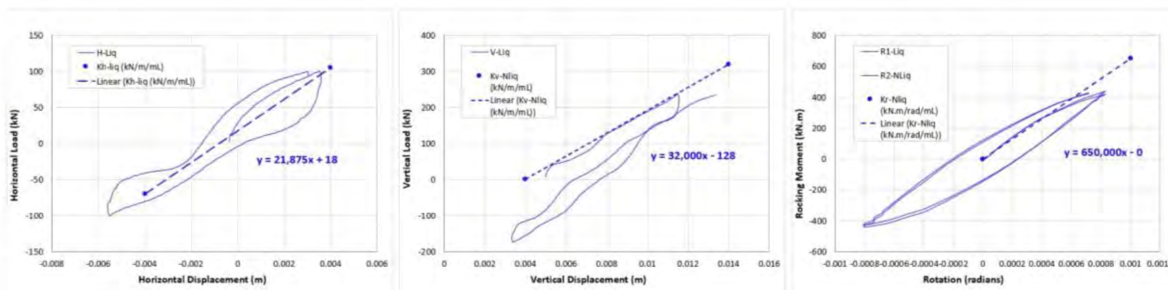
Notes
Load Displacement curves (per meter length of footing in normal to the plane of analysis) from push over analysis obtained from FLAC 2D
These curves should be modified using two multiplying factors as follows
Corrected Load-dis curve= Load-dis (below) x length of footing normal to the plane of analysis x shape factor

Location= Inbound
Soil Type= IB1
Densification(m) = 10
Footing Dimension (m)= 6 In the plane of analysis
Distance between loading columns (m)= 1

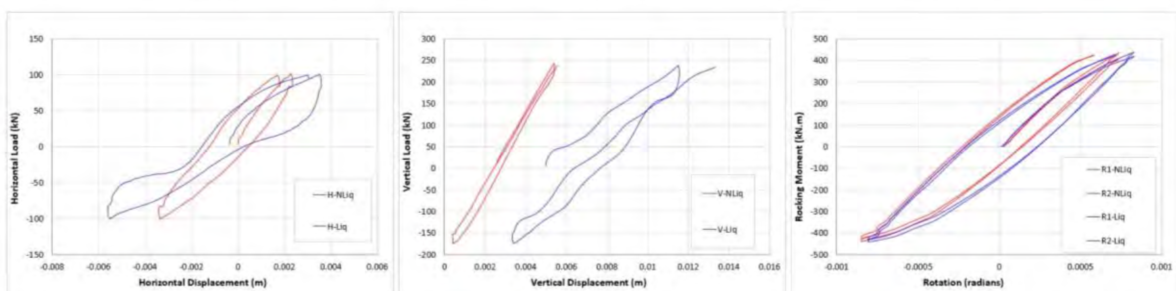
Non-Liq




Liquefied

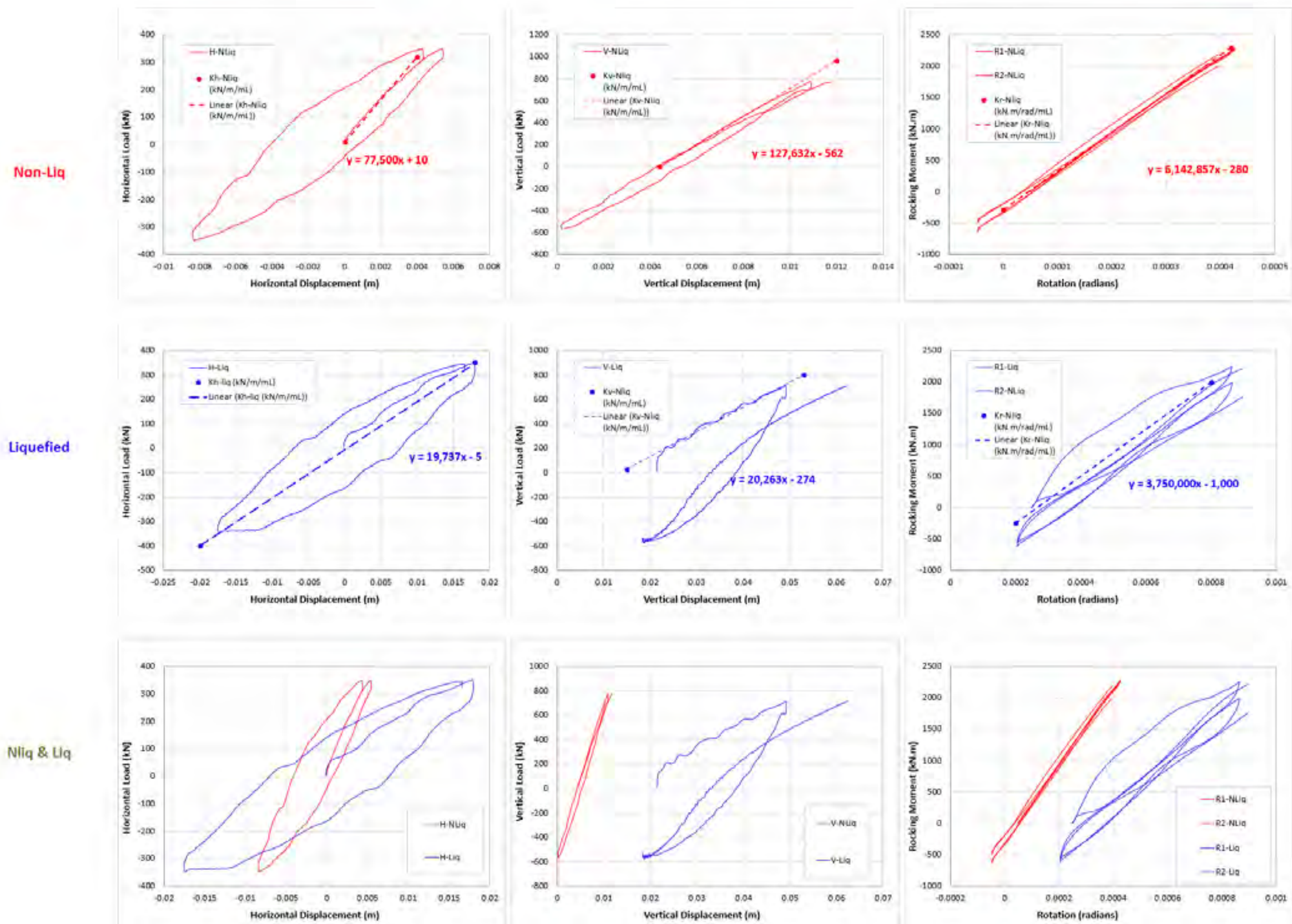



Nliq & Liq

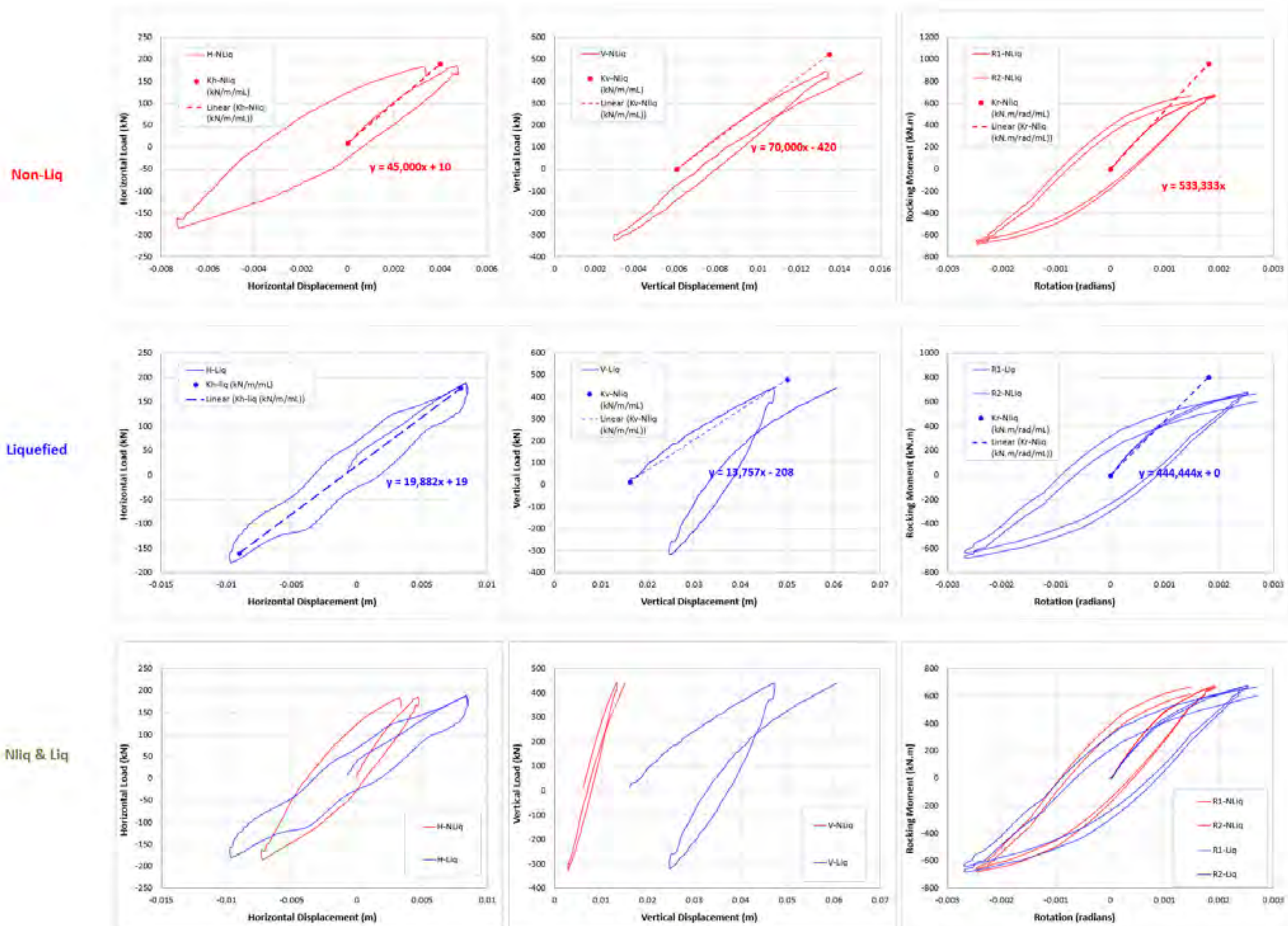


Notes
Load Displacement curves (per meter length of footing in normal to the plane of analysis) from push over analysis obtained from FLAC 2D
These curves should be modified using two multiplying factors as follows
Corrected Load-dis curve= Load-dis (below) x length of footing normal to the plane of analysis x shape factor

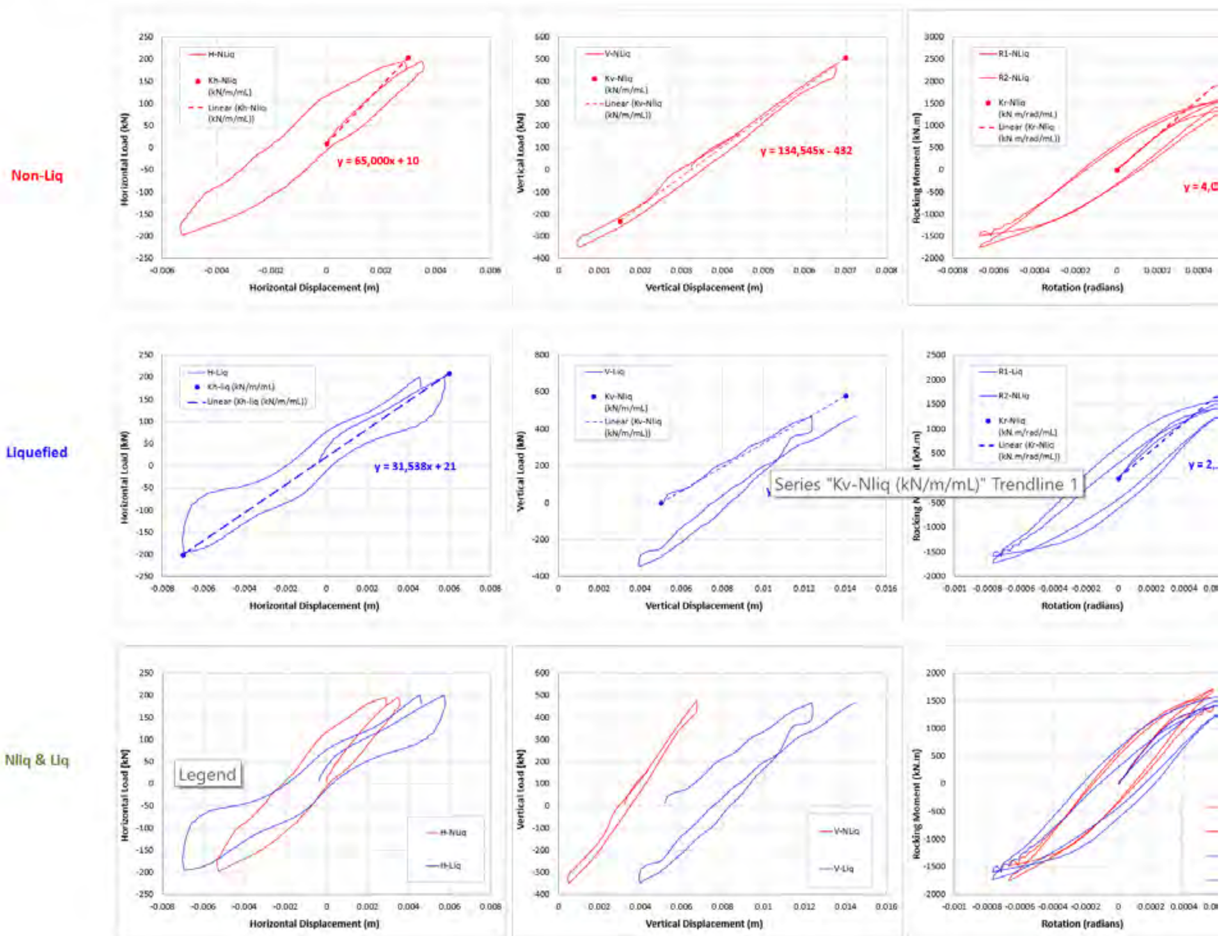
Location= Inbound
Soil Type= IB1
Densification= No
Footing Dimension (m)= 22 In the plane of analysis
Distance between loading columns (m)= 8



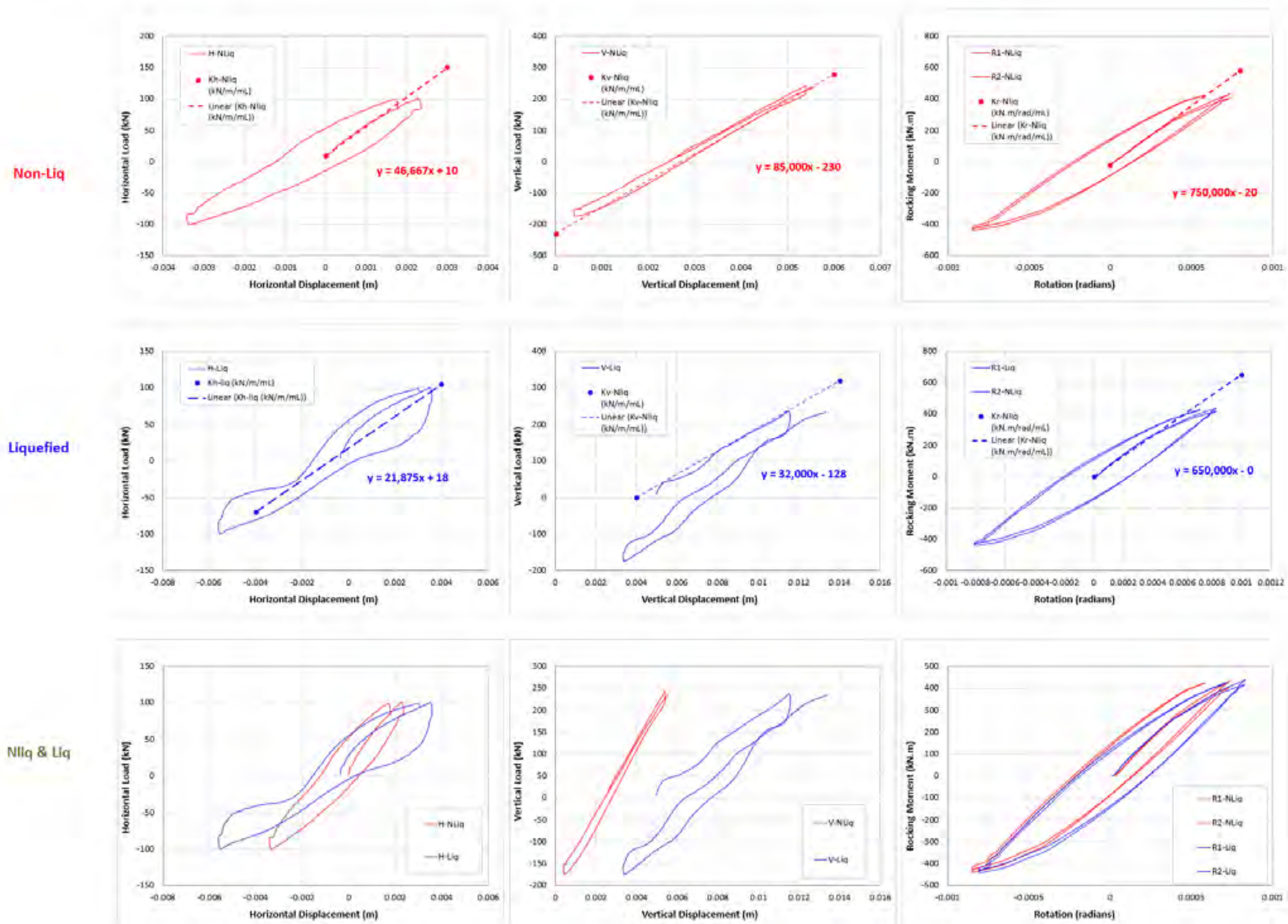
Location= Inbound
Soil Type= IB1
Densification= No
In the (m)= 5
Distance between loading columns (m)= 1



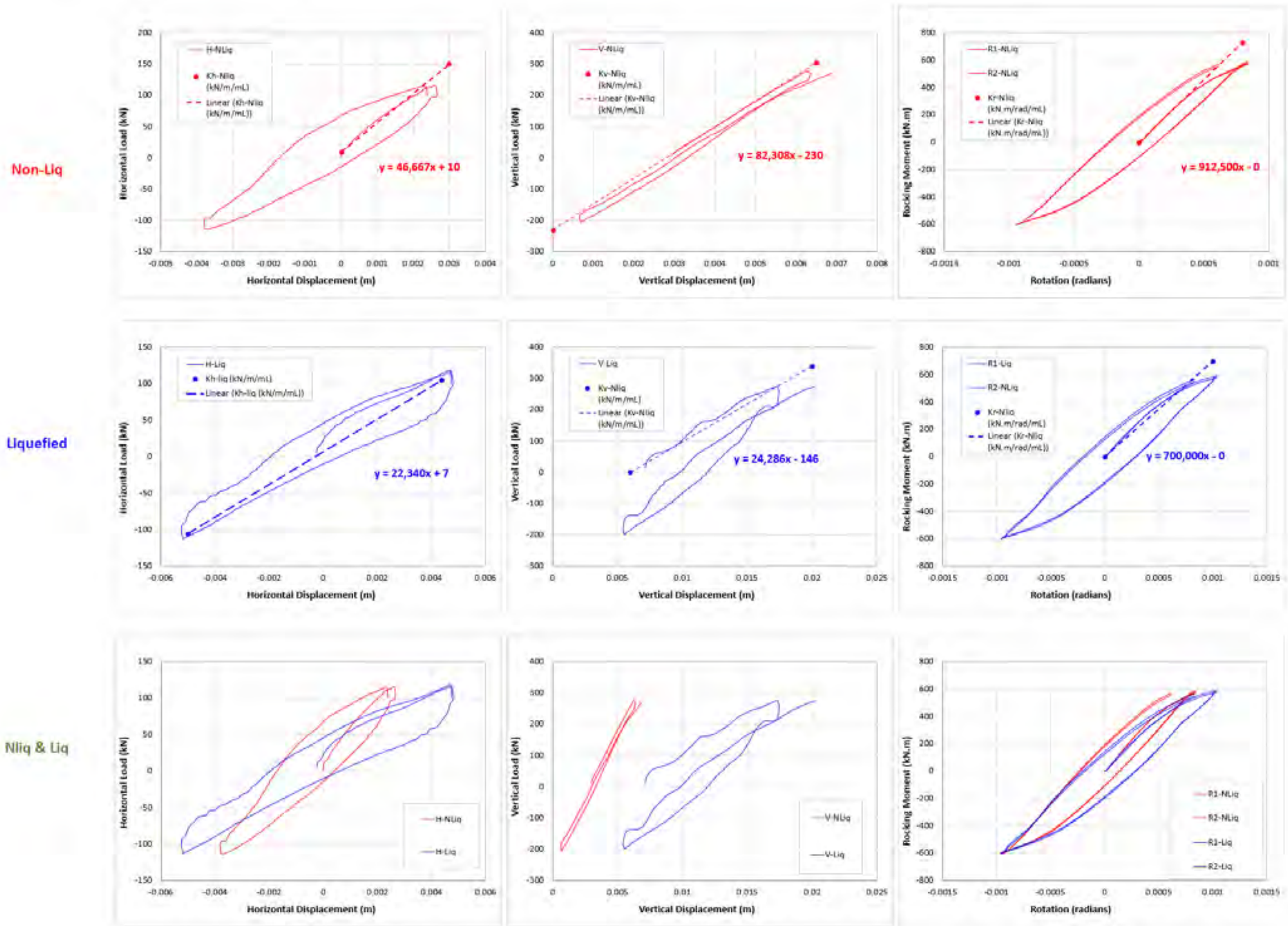
Location= Inbound
Soil Type= IB1
Densification= 18m
Footing Dimension (m)= 12 In the plane of analysis
Distance between loading columns (m)= 8



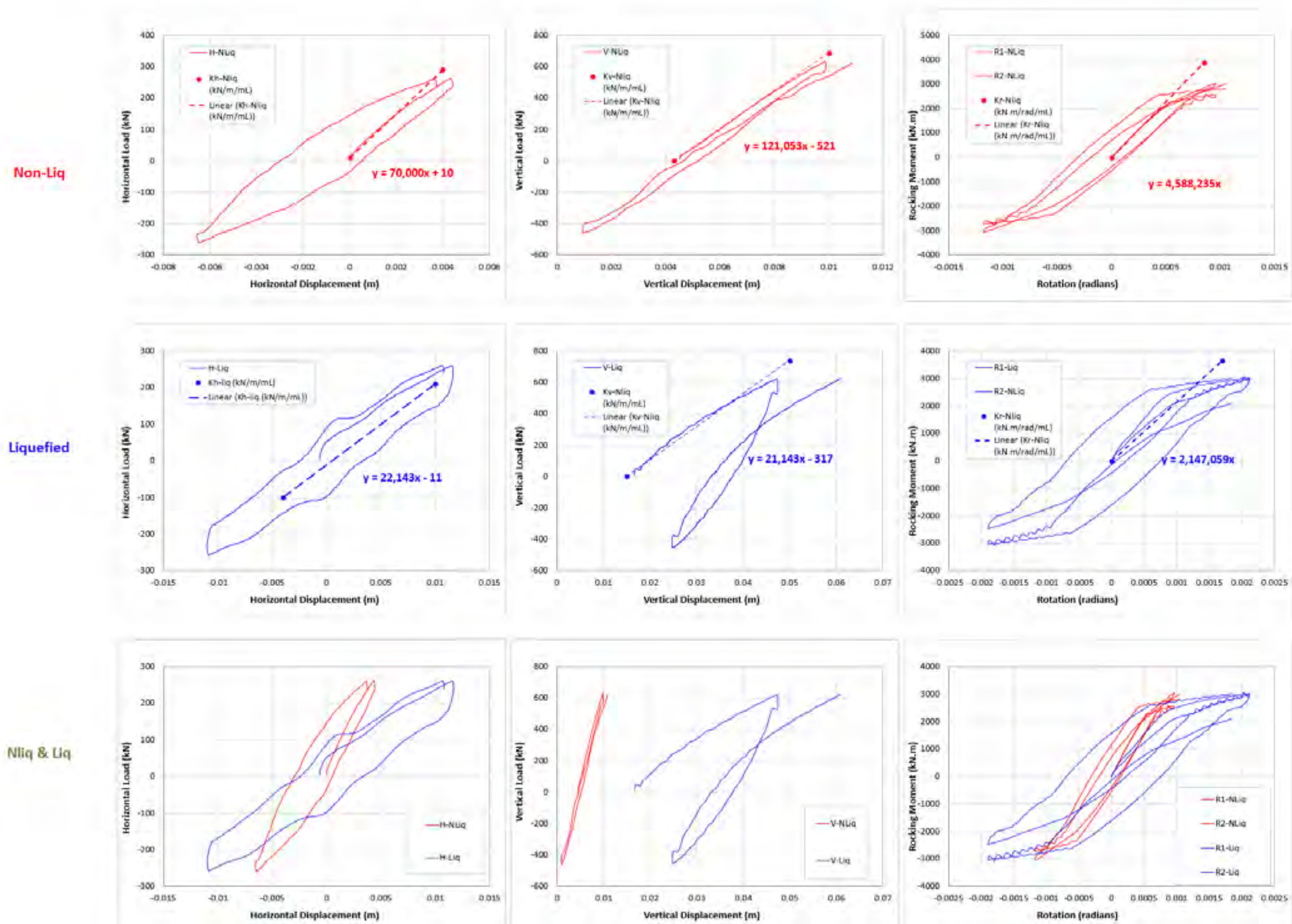
Location= Inbound
Soil Type= IB1
Densification(m) = 18
Footing Dimension (m)= 6 In the plane of analysis
Distance between loading columns (m)= 1



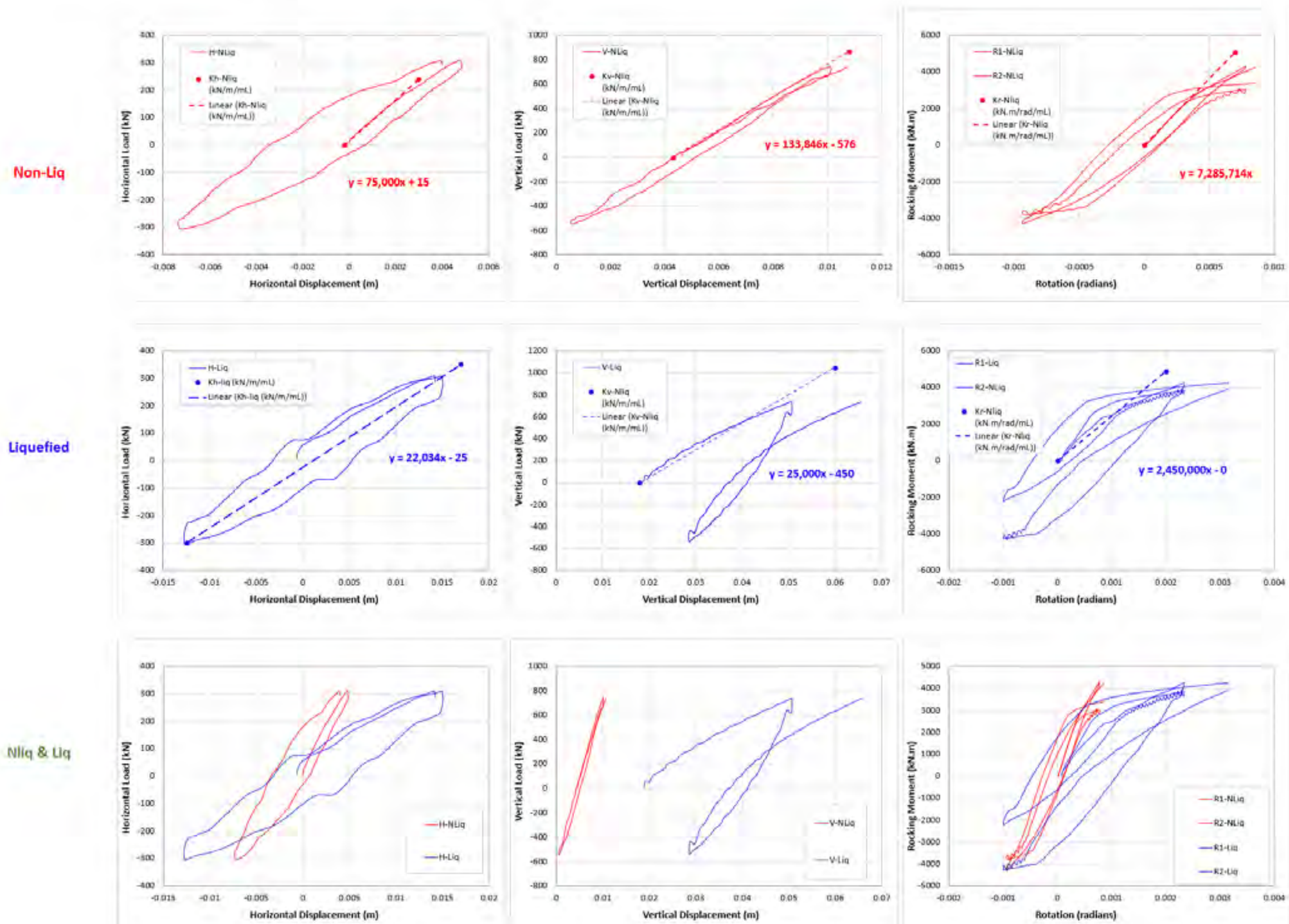
Location= Inbound
Soil Type= IB1
Densification= No
Footing Dimension (m)= 7 In the plane of analysis
Distance between loading columns (m)= 1



Location= Inbound
Soil Type= IB1
Densification= No
Footing Dimension (m)= 16 In the plane of analysis
Distance between loading columns (m)= 8



Location= Inbound
Soil Type= IB1
Densification= No
In the (m)= 19
Distance between loading columns (m)= 12



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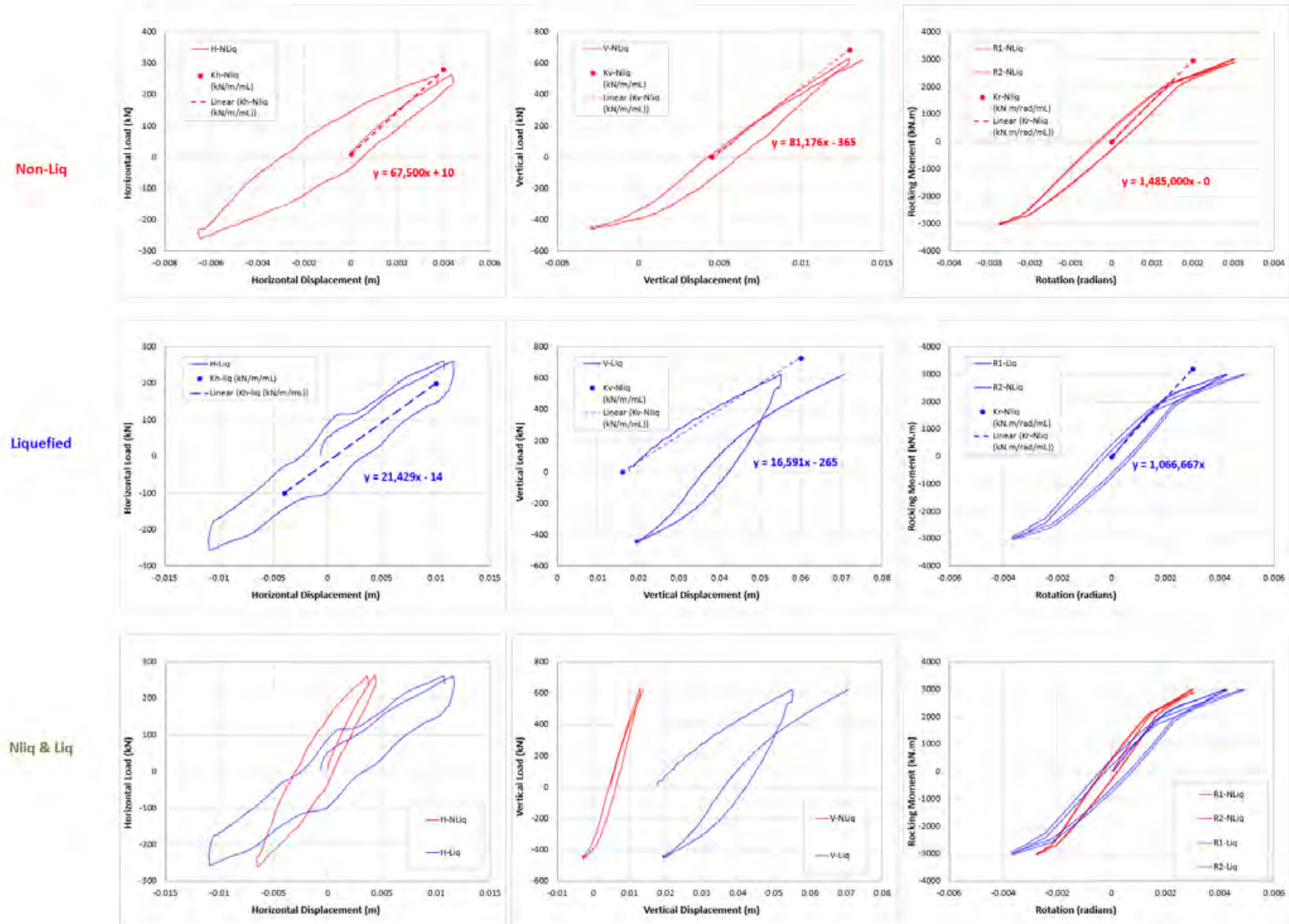
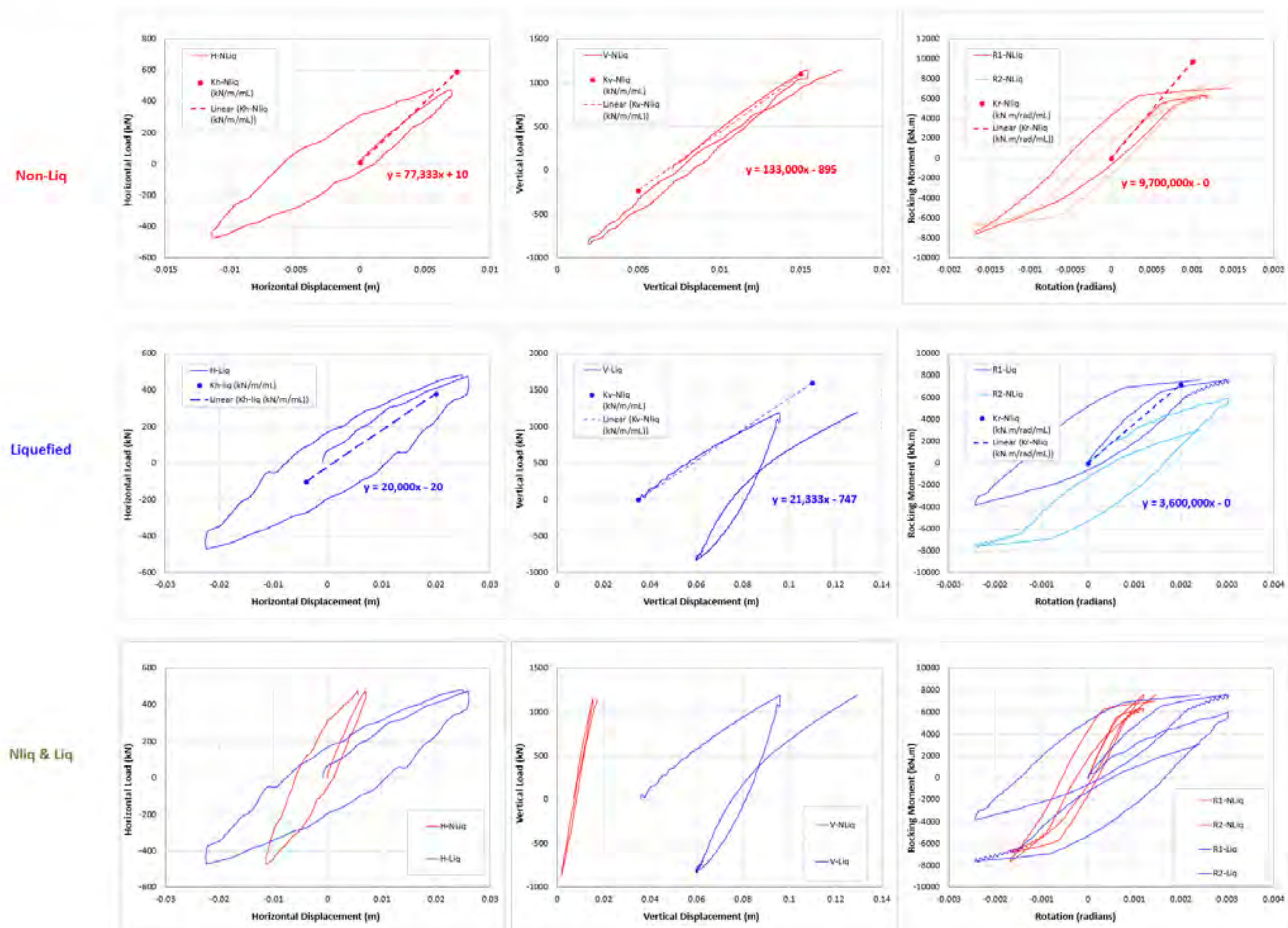
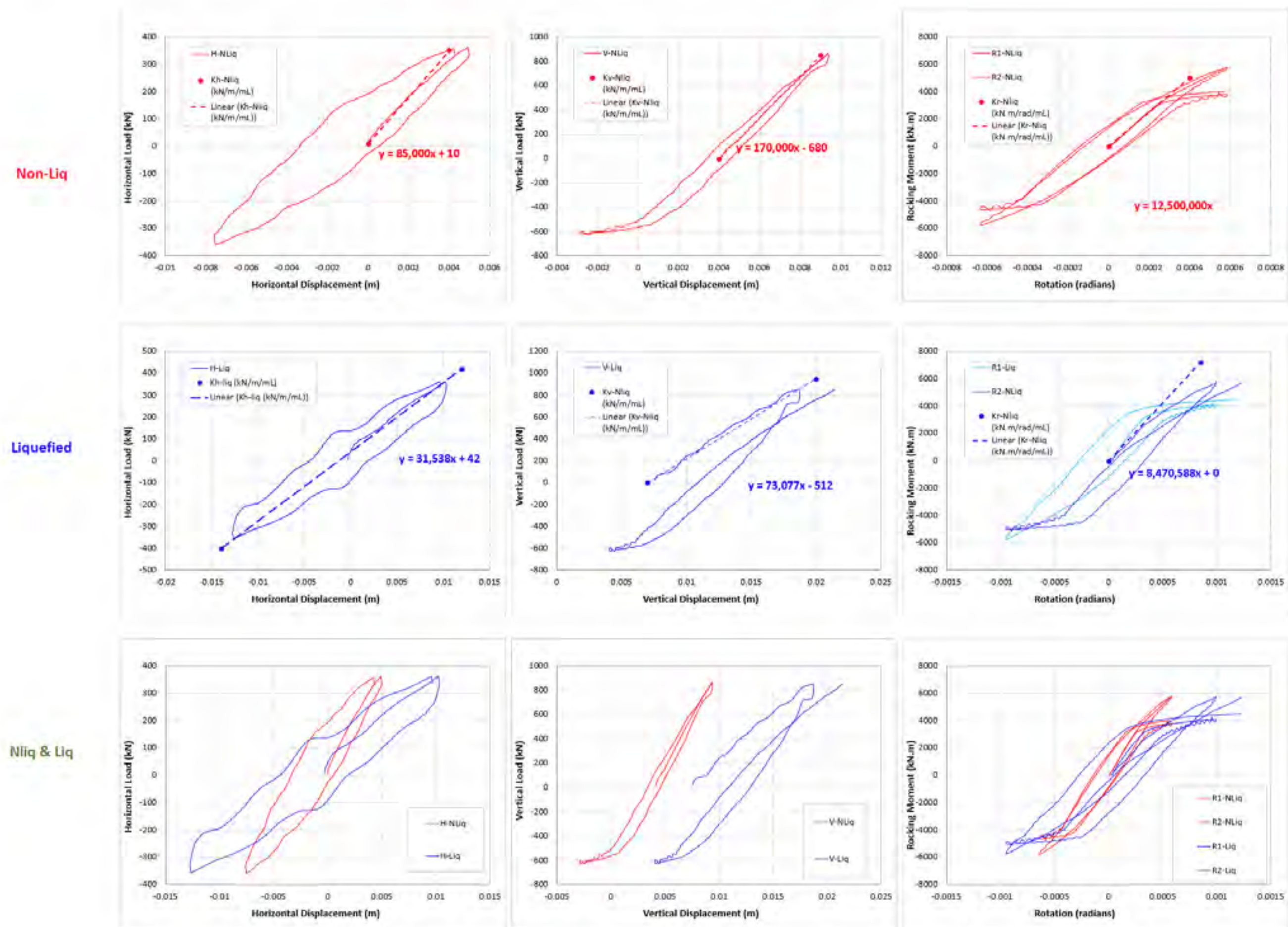
Location= Inbound
Soil Type= IB1
Densification= No
Footing Dimension (m)= 16 In the plane of analysis
e between loading columns (m)= 1



Location= Inbound
Soil Type= IB1
Densification= No
Footing Dimension (m)= 22 In the plane of analysis
e between loading columns (m)= 12

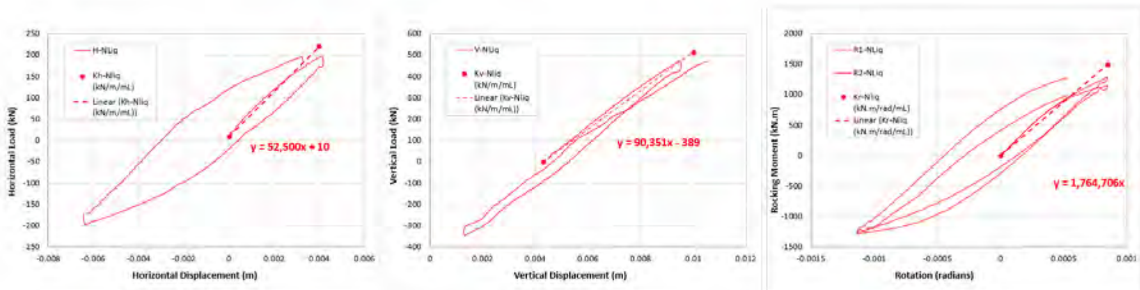


Location= Inbound
Soil Type= IB1
Densification (m)= 18
Footing Dimension (m)= 22 In the plane of analysis
Distance between loading columns (m)= 15

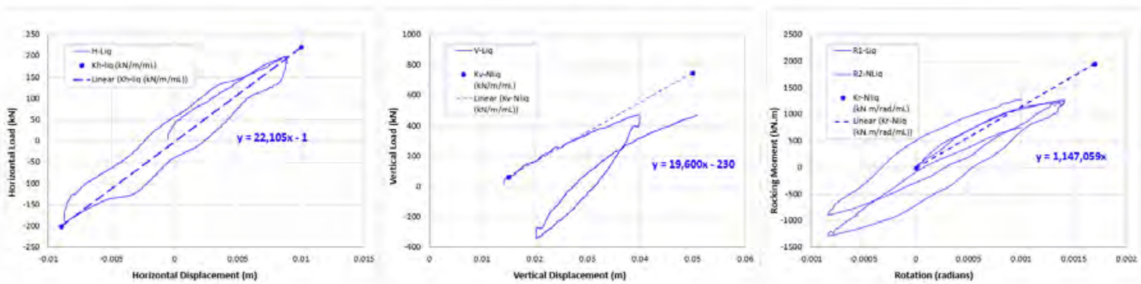


Location= Inbound
Soil Type= IB1
Densification= No
Footing Dimension (m)= 9 In the plane of analysis
Distance between loading columns (m)= 7

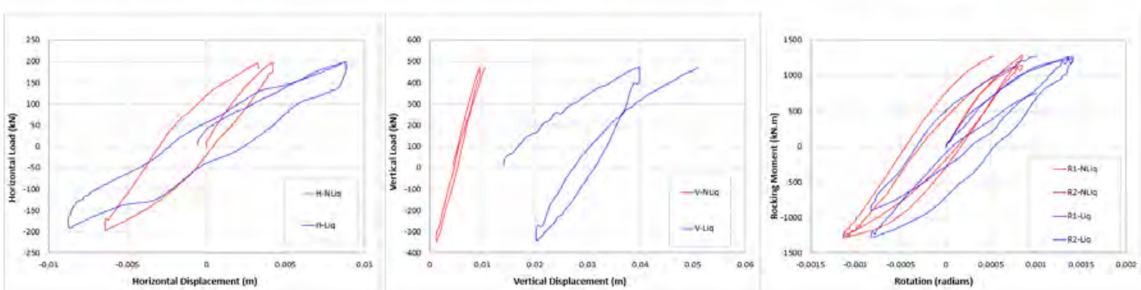
Non-Liq



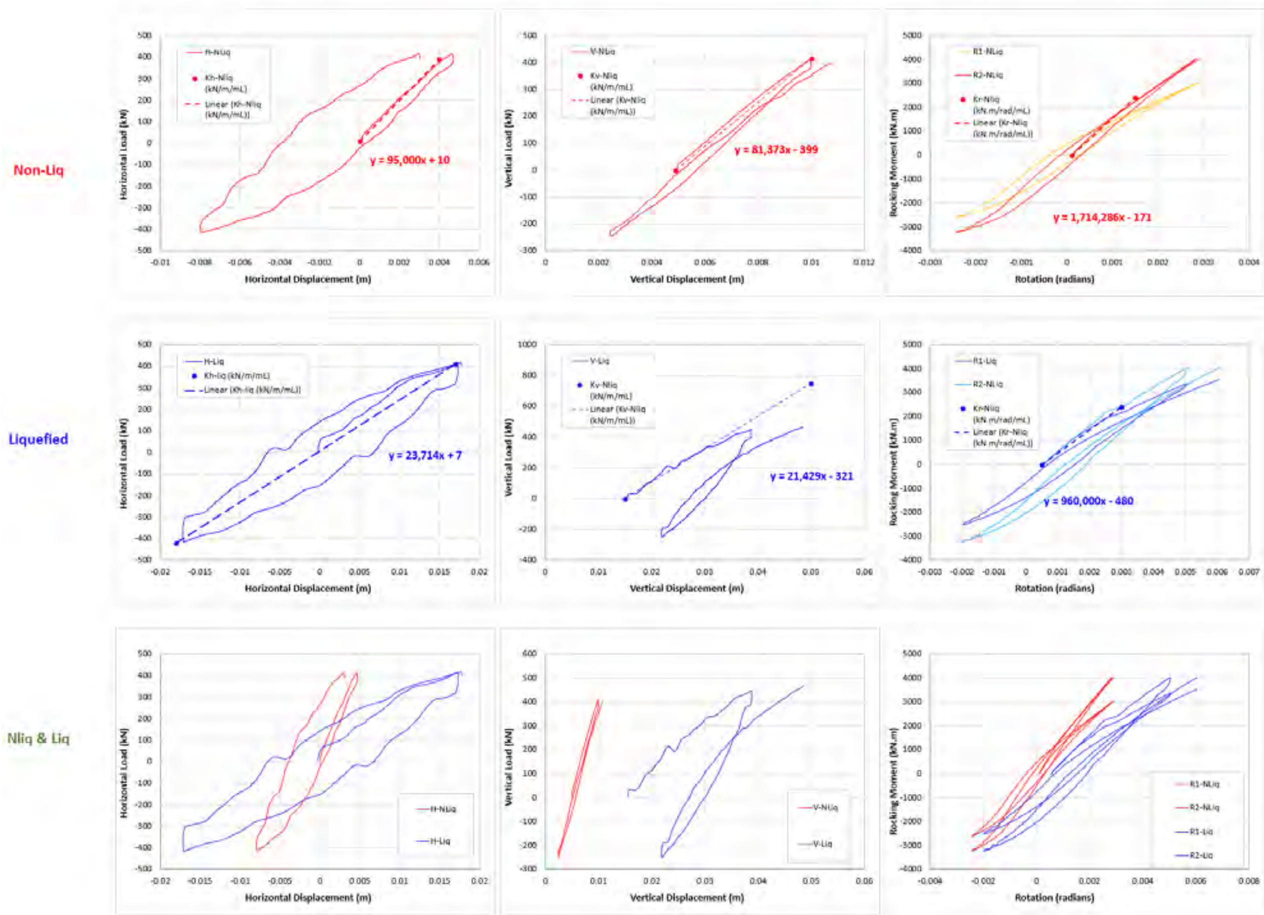

Liquefied




Nliq & Liq



Location= Inbound
Soil Type= IB1
Densification= No
Footing Dimension (m)= 41 In the plane of analysis
Distance between loading columns (m)= 3



# Appendix K

## Load-Displacement Curves In-Bound Footings

## For Non-Linear dynamic Structural Analysis

**Load-Displacement Curves**

1. This update provides the estimated load-displacement curves for liquefied and non-liquefied cases for 10 in-bound structure footings.
    a) See attached the load-displacement curves graphs.
    b) Digital data is provided in a zip file that includes the excel files listed here:

| Name | Size |
|---|---|
| P40-B2-Footing-Load-displ-Rev0-2021-03-08.xlsx | 7,677 KB |
| P40-B3-Footing-Load-displ-Rev0-2021-03-08.xlsx | 7,691 KB |
| P40-B4-Footing-Load-displ-Rev0-2021-03-08.xlsx | 7,347 KB |
| P42-Footing-Load-displ-Rev1-2021-04-14.xlsx | 7,525 KB |
| P45-B1-Footing-Load-displ-Rev0-2021-03-08.xlsx | 8,953 KB |
| P45-B2-Footing-Load-displ-Rev0-2021-03-08.xlsx | 7,688 KB |
| P45-B3-Footing-Load-displ-Rev0-2021-03-08.xlsx | 7,686 KB |
| P45-B4-Footing-Load-displ-Rev0-2021-03-08.xlsx | 7,772 KB |
| P47-Footing-Load-displ-Rev0-2021-03-08.xlsx | 12,070 KB |
| P50-B1-Footing-Load-displ-Rev0-2021-03-08.xlsx | 7,929 KB |
| P52-Footing-Load-displ-Rev0-2021-03-26.xlsx | 8,394 KB |

2. Figure K-1 shows the content of one of the excel files as an example.
3. Note the following items:
    a) Dead loads & footing self-weight were added to the numerical model and solved. Loads were then zeroed to represent the initial conditions before push-over analyses.
    b) The load displacement curves (attached) represent best estimate curves. It is prudent to check the response of the system within an upper and lower range of 0.5 to 2 times best estimate curves (ASCE-SEI-41-2017) due to uncertainties. The lower and upper range can be derived by using a multiplier of 0.5 and 2, respectively, on the vertical axis (load).
    c) The undulations on some of the load-displacement curves (Figure K-2) are due to the continued isolation of the ground after earthquake shaking (the FLAC model was analyzed in the dynamic mode). Simplified load-displacement curves may be fitted to the calculated curves as shown on Figure K-2, if needed.
    d) Vertical loading for liquefied case results in accumulation of vertical displacements with increasing number of cycle. This is due to the plastic flow of liquefied soils from underneath of the footing resulting in permanent vertical displacements in each loading cycle.
4. See Appendix J for general methodology, uncertainties and background information of estimation of soil springs and load-displacement curves.

# Figure K-1- An example of the content of a load-displacement Excel file

Structure: P40-B2
B= 7.9 m
L= 15.2 m
Date: 2021-03-08
Rev: 0

Notes:
1. Use the corrected force (P. CF) vs Δ or corrected moment (M.CF) vs ϴ for footing non-linear curves
2. The vertical load -displacement calculated in B and L directions are different. It is suggested to use the stiffer cuver for non-Liquefied case and to use the softer curve for liquefied case.
3. The trendlines on the graphs are for general review and are not for use as stiffness values.
4. The information in this spreadsheet to be used in conjuction with Doc# 20-8543-UPDATE-003-Rev0 dated 2021-03-08

Correction Factors (CF)

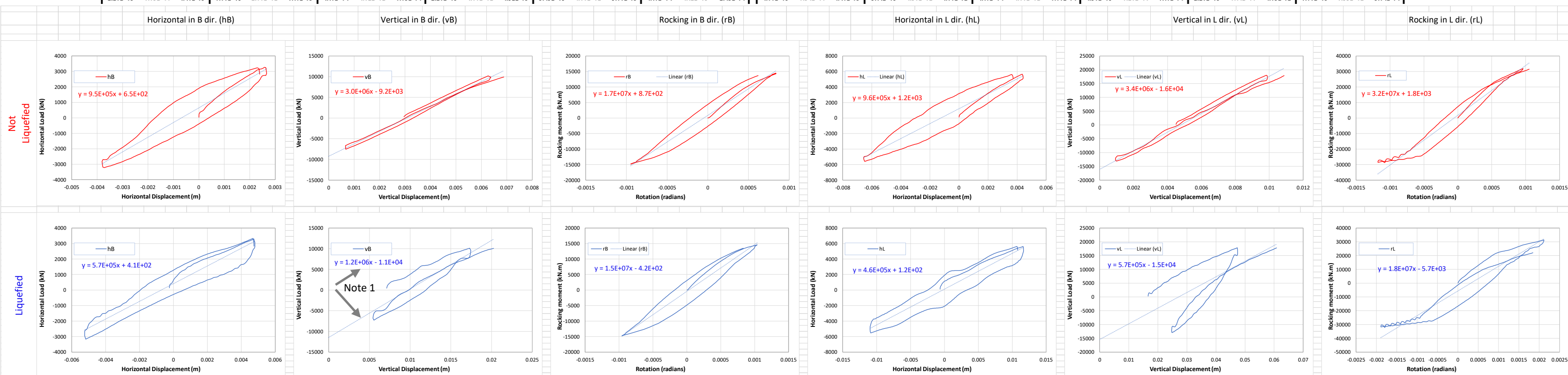
| vB | hB | rB | v | hL | rL |
|---|---|---|---|---|---|
| 36.6 | 28.1 | 24.6 | 28.5 | 21.5 | 10.4 |

**Not-Liquefied**

| hB P.CF. (kN) | hB P (kN) | hB Δ (m) | vB P.CF. (kN) | vB P (kN) | vB Δ (m) | rB M.CF. (kN.m) | rB M (kN.m) | rB ϴ (Rad.) | hL P.CF. (kN) | hL P (kN) | hL Δ (m) | vL P.CF. (kN) | vL P (kN) | vL Δ (m) | rL M.CF. (kN.m) | rL M (kN.m) | rL ϴ (Rad.) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 4.08E+01 | 1.45E+00 | -9.07E-07 | 3.49E+02 | 9.53E+00 | 2.96E-03 | 1.50E+01 | 6.11E-01 | 4.97E-07 | 1.24E+02 | 5.77E+00 | 1.90E-06 | 2.10E+02 | 7.36E+00 | 4.50E-03 | 5.10E+01 | 4.88E+00 | -3.09E-07 |
| 2.59E+02 | 9.21E+00 | 1.06E-05 | 8.43E+02 | 2.30E+01 | 3.05E-03 | 1.28E+02 | 5.19E+00 | 1.64E-05 | 2.87E+02 | 1.33E+01 | 1.47E-05 | 5.98E+02 | 2.10E+01 | 4.52E-03 | 3.08E+02 | 2.94E+01 | 5.05E-06 |
| 3.71E+02 | 1.32E+01 | 5.22E-05 | 1.20E+03 | 3.28E+01 | 3.16E-03 | 5.53E+02 | 2.25E+01 | 3.69E-05 | 4.04E+02 | 1.88E+01 | 4.46E-05 | 8.90E+02 | 3.12E+01 | 4.56E-03 | 8.52E+02 | 8.16E+01 | 1.39E-05 |
| 4.19E+02 | 1.49E+01 | 1.12E-04 | 1.51E+03 | 4.12E+01 | 3.27E-03 | 9.70E+02 | 3.95E+01 | 5.28E-05 | 4.87E+02 | 2.27E+01 | 8.69E-05 | 1.13E+03 | 3.96E+01 | 4.60E-03 | 1.25E+03 | 1.20E+02 | 2.25E-05 |
| 4.73E+02 | 1.68E+01 | 1.69E-04 | 1.83E+03 | 4.99E+01 | 3.38E-03 | 1.39E+03 | 5.68E+01 | 7.02E-05 | 5.54E+02 | 2.58E+01 | 1.35E-04 | 1.34E+03 | 4.71E+01 | 4.64E-03 | 1.66E+03 | 1.59E+02 | 3.20E-05 |
| 5.61E+02 | 2.00E+01 | 2.17E-04 | 2.15E+03 | 5.86E+01 | 3.49E-03 | 1.82E+03 | 7.39E+01 | 8.76E-05 | 6.19E+02 | 2.88E+01 | 1.84E-04 | 1.50E+03 | 5.27E+01 | 4.69E-03 | 2.02E+03 | 1.94E+02 | 4.13E-05 |
| 6.62E+02 | 2.35E+01 | 2.62E-04 | 2.47E+03 | 6.74E+01 | 3.60E-03 | 2.24E+03 | 9.11E+01 | 1.05E-04 | 6.90E+02 | 3.21E+01 | 2.30E-04 | 1.62E+03 | 5.68E+01 | 4.75E-03 | 2.39E+03 | 2.29E+02 | 5.00E-05 |
| 7.53E+02 | 2.68E+01 | 3.09E-04 | 2.79E+03 | 7.62E+01 | 3.71E-03 | 2.66E+03 | 1.08E+02 | 1.22E-04 | 7.67E+02 | 3.57E+01 | 2.75E-04 | 1.75E+03 | 6.13E+01 | 4.80E-03 | 2.79E+03 | 2.67E+02 | 5.85E-05 |
| 8.32E+02 | 2.96E+01 | 3.60E-04 | 3.11E+03 | 8.50E+01 | 3.83E-03 | 3.08E+03 | 1.25E+02 | 1.40E-04 | 8.45E+02 | 3.93E+01 | 3.19E-04 | 1.89E+03 | 6.62E+01 | 4.85E-03 | 3.20E+03 | 3.06E+02 | 6.68E-05 |
| 9.04E+02 | 3.22E+01 | 4.12E-04 | 3.43E+03 | 9.36E+01 | 3.94E-03 | 3.50E+03 | 1.43E+02 | 1.58E-04 | 9.20E+02 | 4.28E+01 | 3.64E-04 | 2.03E+03 | 7.12E+01 | 4.91E-03 | 3.61E+03 | 3.46E+02 | 7.52E-05 |
| 9.75E+02 | 3.47E+01 | 4.65E-04 | 3.76E+03 | 1.03E+02 | 4.05E-03 | 3.92E+03 | 1.60E+02 | 1.76E-04 | 9.92E+02 | 4.62E+01 | 4.10E-04 | 2.18E+03 | 7.64E+01 | 4.96E-03 | 4.02E+03 | 3.85E+02 | 8.35E-05 |
| 1.05E+03 | 3.74E+01 | 5.17E-04 | 4.11E+03 | 1.12E+02 | 4.15E-03 | 4.34E+03 | 1.77E+02 | 1.94E-04 | 1.06E+03 | 4.95E+01 | 4.57E-04 | 2.34E+03 | 8.20E+01 | 5.01E-03 | 4.43E+03 | 4.24E+02 | 9.17E-05 |
| 1.13E+03 | 4.02E+01 | 5.68E-04 | 4.45E+03 | 1.21E+02 | 4.26E-03 | 4.76E+03 | 1.94E+02 | 2.12E-04 | 1.13E+03 | 5.27E+01 | 5.04E-04 | 2.50E+03 | 8.76E+01 | 5.06E-03 | 4.84E+03 | 4.64E+02 | 1.00E-04 |
| 1.21E+03 | 4.29E+01 | 6.19E-04 | 4.78E+03 | 1.30E+02 | 4.37E-03 | 5.19E+03 | 2.11E+02 | 2.31E-04 | 1.20E+03 | 5.60E+01 | 5.51E-04 | 2.66E+03 | 9.33E+01 | 5.11E-03 | 5.25E+03 | 5.03E+02 | 1.08E-04 |
| 1.28E+03 | 4.55E+01 | 6.71E-04 | 5.10E+03 | 1.39E+02 | 4.48E-03 | 5.61E+03 | 2.28E+02 | 2.49E-04 | 1.27E+03 | 5.93E+01 | 5.98E-04 | 2.82E+03 | 9.90E+01 | 5.16E-03 | 5.66E+03 | 5.42E+02 | 1.17E-04 |
| 1.35E+03 | 4.80E+01 | 7.24E-04 | 5.43E+03 | 1.48E+02 | 4.59E-03 | 6.03E+03 | 2.45E+02 | 2.69E-04 | 1.35E+03 | 6.27E+01 | 6.44E-04 | 2.99E+03 | 1.05E+02 | 5.21E-03 | 6.06E+03 | 5.81E+02 | 1.25E-04 |
| 1.42E+03 | 5.05E+01 | 7.77E-04 | 5.76E+03 | 1.57E+02 | 4.71E-03 | 6.44E+03 | 2.62E+02 | 2.88E-04 | 1.42E+03 | 6.60E+01 | 6.91E-04 | 3.15E+03 | 1.10E+02 | 5.26E-03 | 6.47E+03 | 6.19E+02 | 1.33E-04 |
| 1.49E+03 | 5.30E+01 | 8.30E-04 | 6.09E+03 | 1.66E+02 | 4.82E-03 | 6.86E+03 | 2.79E+02 | 3.08E-04 | 1.49E+03 | 6.92E+01 | 7.38E-04 | 3.31E+03 | 1.16E+02 | 5.31E-03 | 6.87E+03 | 6.58E+02 | 1.42E-04 |
| 1.56E+03 | 5.55E+01 | 8.84E-04 | 6.41E+03 | 1.75E+02 | 4.93E-03 | 7.28E+03 | 2.96E+02 | 3.28E-04 | 1.56E+03 | 7.24E+01 | 7.85E-04 | 3.47E+03 | 1.22E+02 | 5.36E-03 | 7.27E+03 | 6.96E+02 | 1.50E-04 |
| 1.63E+03 | 5.80E+01 | 9.37E-04 | 6.74E+03 | 1.84E+02 | 5.04E-03 | 7.70E+03 | 3.13E+02 | 3.48E-04 | 1.62E+03 | 7.56E+01 | 8.33E-04 | 3.64E+03 | 1.28E+02 | 5.41E-03 | 7.67E+03 | 7.35E+02 | 1.59E-04 |
| 1.70E+03 | 6.04E+01 | 9.91E-04 | 7.05E+03 | 1.93E+02 | 5.15E-03 | 8.12E+03 | 3.30E+02 | 3.69E-04 | 1.69E+03 | 7.88E+01 | 8.81E-04 | 3.81E+03 | 1.34E+02 | 5.46E-03 | 8.07E+03 | 7.73E+02 | 1.67E-04 |
| 1.76E+03 | 6.28E+01 | 1.04E-03 | 7.38E+03 | 2.02E+02 | 5.26E-03 | 8.53E+03 | 3.47E+02 | 3.89E-04 | 1.76E+03 | 8.19E+01 | 9.29E-04 | 4.00E+03 | 1.40E+02 | 5.51E-03 | 8.48E+03 | 8.12E+02 | 1.76E-04 |
| 1.83E+03 | 6.52E+01 | 1.10E-03 | 7.70E+03 | 2.10E+02 | 5.37E-03 | 8.95E+03 | 3.64E+02 | 4.11E-04 | 1.83E+03 | 8.51E+01 | 9.77E-04 | 4.19E+03 | 1.47E+02 | 5.55E-03 | 8.89E+03 | 8.52E+02 | 1.84E-04 |
| 1.90E+03 | 6.75E+01 | 1.15E-03 | 8.02E+03 | 2.19E+02 | 5.48E-03 | 9.37E+03 | 3.81E+02 | 4.32E-04 | 1.89E+03 | 8.82E+01 | 1.03E-03 | 4.38E+03 | 1.54E+02 | 5.60E-03 | 9.31E+03 | 8.91E+02 | 1.93E-04 |
| 1.96E+03 | 6.97E+01 | 1.21E-03 | 8.34E+03 | 2.28E+02 | 5.59E-03 | 9.78E+03 | 3.98E+02 | 4.60E-04 | 1.96E+03 | 9.12E+01 | 1.07E-03 | 4.56E+03 | 1.60E+02 | 5.65E-03 | 9.72E+03 | 9.31E+02 | 2.01E-04 |
| 2.02E+03 | 7.18E+01 | 1.27E-03 | 8.65E+03 | 2.36E+02 | 5.71E-03 | 1.02E+04 | 4.15E+02 | 4.87E-04 | 2.02E+03 | 9.43E+01 | 1.12E-03 | 4.74E+03 | 1.66E+02 | 5.70E-03 | 1.01E+04 | 9.70E+02 | 2.10E-04 |
| 2.07E+03 | 7.38E+01 | 1.32E-03 | 8.96E+03 | 2.45E+02 | 5.82E-03 | 1.06E+04 | 4.32E+02 | 5.14E-04 | 2.09E+03 | 9.73E+01 | 1.17E-03 | 4.92E+03 | 1.72E+02 | 5.75E-03 | 1.05E+04 | 1.01E+03 | 2.19E-04 |
| 2.14E+03 | 7.60E+01 | 1.38E-03 | 9.27E+03 | 2.53E+02 | 5.93E-03 | 1.10E+04 | 4.49E+02 | 5.41E-04 | 2.15E+03 | 1.00E+02 | 1.22E-03 | 5.09E+03 | 1.78E+02 | 5.79E-03 | 1.09E+04 | 1.05E+03 | 2.27E-04 |
| 2.20E+03 | 7.82E+01 | 1.43E-03 | 9.58E+03 | 2.62E+02 | 6.04E-03 | 1.14E+04 | 4.65E+02 | 5.69E-04 | 2.22E+03 | 1.03E+02 | 1.27E-03 | 5.26E+03 | 1.84E+02 | 5.84E-03 | 1.13E+04 | 1.08E+03 | 2.36E-04 |
| 2.26E+03 | 8.05E+01 | 1.49E-03 | 9.89E+03 | 2.70E+02 | 6.16E-03 | 1.18E+04 | 4.82E+02 | 6.03E-04 | 2.29E+03 | 1.06E+02 | 1.32E-03 | 5.43E+03 | 1.90E+02 | 5.89E-03 | 1.17E+04 | 1.12E+03 | 2.45E-04 |

**Liquefied**

| hB P.CF. (kN) | hB P (kN) | hB Δ (m) | vB P.CF. (kN) | vB P (kN) | vB Δ (m) | rB M.CF. (kN.m) | rB M (kN.m) | rB ϴ (Rad.) | hL P.CF. (kN) | hL P (kN) | hL Δ (m) | vL P.CF. (kN) | vL P (kN) | vL Δ (m) | rL M.CF. (kN.m) | rL M (kN.m) | rL ϴ (Rad.) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.40E+02 | 4.99E+00 | -2.43E-04 | 4.32E+02 | 1.18E+01 | 7.12E-03 | 1.50E+01 | 6.11E-01 | 2.26E-06 | 1.09E+02 | 5.05E+00 | -6.43E-04 | 2.89E+02 | 1.01E+01 | 1.67E-02 | 4.98E+01 | 4.77E+00 | 1.68E-06 |
| 3.21E+02 | 1.14E+01 | -2.21E-04 | 1.01E+03 | 2.76E+01 | 7.20E-03 | 1.58E+02 | 6.44E+00 | 1.65E-05 | 2.77E+02 | 1.29E+01 | -6.33E-04 | 6.28E+02 | 2.20E+01 | 1.67E-02 | 4.24E+02 | 4.06E+01 | 7.98E-06 |
| 3.98E+02 | 1.42E+01 | -1.70E-04 | 1.35E+03 | 3.69E+01 | 7.31E-03 | 6.05E+02 | 2.46E+01 | 3.49E-05 | 3.98E+02 | 1.85E+01 | -6.05E-04 | 9.09E+02 | 3.19E+01 | 1.67E-02 | 8.89E+02 | 8.52E+01 | 1.64E-05 |
| 4.38E+02 | 1.56E+01 | -1.08E-04 | 1.60E+03 | 4.36E+01 | 7.43E-03 | 1.04E+03 | 4.25E+01 | 5.26E-05 | 4.80E+02 | 2.24E+01 | -5.63E-04 | 1.14E+03 | 4.00E+01 | 1.68E-02 | 1.29E+03 | 1.24E+02 | 2.48E-05 |
| 5.07E+02 | 1.81E+01 | -5.47E-05 | 1.82E+03 | 4.98E+01 | 7.55E-03 | 1.49E+03 | 6.05E+01 | 7.14E-05 | 5.45E+02 | 2.54E+01 | -5.14E-04 | 1.31E+03 | 4.58E+01 | 1.68E-02 | 1.68E+03 | 1.61E+02 | 3.40E-05 |
| 6.08E+02 | 2.16E+01 | -1.03E-05 | 1.98E+03 | 5.40E+01 | 7.68E-03 | 1.92E+03 | 7.84E+01 | 9.08E-05 | 6.11E+02 | 2.85E+01 | -4.67E-04 | 1.40E+03 | 4.92E+01 | 1.69E-02 | 2.00E+03 | 1.91E+02 | 4.34E-05 |
| 7.05E+02 | 2.51E+01 | 3.52E-05 | 2.08E+03 | 5.69E+01 | 7.82E-03 | 2.36E+03 | 9.61E+01 | 1.11E-04 | 6.84E+02 | 3.19E+01 | -4.21E-04 | 1.46E+03 | 5.11E+01 | 1.69E-02 | 2.31E+03 | 2.21E+02 | 5.27E-05 |
| 7.83E+02 | 2.79E+01 | 8.62E-05 | 2.18E+03 | 5.95E+01 | 7.95E-03 | 2.80E+03 | 1.14E+02 | 1.31E-04 | 7.60E+02 | 3.54E+01 | -3.77E-04 | 1.49E+03 | 5.22E+01 | 1.70E-02 | 2.63E+03 | 2.52E+02 | 6.21E-05 |
| 8.46E+02 | 3.01E+01 | 1.41E-04 | 2.27E+03 | 6.19E+01 | 8.09E-03 | 3.22E+03 | 1.31E+02 | 1.52E-04 | 8.32E+02 | 3.87E+01 | -3.31E-04 | 1.49E+03 | 5.21E+01 | 1.71E-02 | 2.92E+03 | 2.80E+02 | 7.19E-05 |
| 9.07E+02 | 3.23E+01 | 1.97E-04 | 2.31E+03 | 6.31E+01 | 8.23E-03 | 3.64E+03 | 1.48E+02 | 1.73E-04 | 8.96E+02 | 4.17E+01 | -2.82E-04 | 1.46E+03 | 5.10E+01 | 1.72E-02 | 3.19E+03 | 3.05E+02 | 8.21E-05 |
| 9.72E+02 | 3.46E+01 | 2.52E-04 | 2.36E+03 | 6.46E+01 | 8.37E-03 | 4.05E+03 | 1.65E+02 | 1.94E-04 | 9.50E+02 | 4.43E+01 | -2.30E-04 | 1.42E+03 | 4.96E+01 | 1.72E-02 | 3.44E+03 | 3.29E+02 | 9.26E-05 |
| 1.04E+03 | 3.70E+01 | 3.06E-04 | 2.44E+03 | 6.68E+01 | 8.51E-03 | 4.45E+03 | 1.81E+02 | 2.15E-04 | 9.99E+02 | 4.65E+01 | -1.76E-04 | 1.37E+03 | 4.80E+01 | 1.73E-02 | 3.68E+03 | 3.53E+02 | 1.03E-04 |
| 1.10E+03 | 3.93E+01 | 3.60E-04 | 2.50E+03 | 6.83E+01 | 8.65E-03 | 4.84E+03 | 1.97E+02 | 2.36E-04 | 1.04E+03 | 4.86E+01 | -1.20E-04 | 1.32E+03 | 4.62E+01 | 1.74E-02 | 3.92E+03 | 3.75E+02 | 1.14E-04 |
| 1.16E+03 | 4.13E+01 | 4.17E-04 | 2.56E+03 | 7.00E+01 | 8.79E-03 | 5.23E+03 | 2.13E+02 | 2.57E-04 | 1.09E+03 | 5.06E+01 | -6.37E-05 | 1.27E+03 | 4.46E+01 | 1.75E-02 | 4.15E+03 | 3.98E+02 | 1.25E-04 |
| 1.21E+03 | 4.30E+01 | 4.77E-04 | 2.65E+03 | 7.24E+01 | 8.93E-03 | 5.63E+03 | 2.29E+02 | 2.79E-04 | 1.13E+03 | 5.25E+01 | -6.49E-06 | 1.24E+03 | 4.33E+01 | 1.76E-02 | 4.39E+03 | 4.20E+02 | 1.36E-04 |
| 1.25E+03 | 4.44E+01 | 5.39E-04 | 2.75E+03 | 7.50E+01 | 9.06E-03 | 6.02E+03 | 2.45E+02 | 3.01E-04 | 1.16E+03 | 5.42E+01 | 5.31E-05 | 1.21E+03 | 4.25E+01 | 1.76E-02 | 4.63E+03 | 4.43E+02 | 1.47E-04 |
| 1.27E+03 | 4.53E+01 | 6.05E-04 | 2.84E+03 | 7.76E+01 | 9.20E-03 | 6.41E+03 | 2.61E+02 | 3.24E-04 | 1.19E+03 | 5.53E+01 | 1.17E-04 | 1.20E+03 | 4.22E+01 | 1.77E-02 | 4.88E+03 | 4.67E+02 | 1.58E-04 |
| 1.31E+03 | 4.65E+01 | 6.68E-04 | 2.95E+03 | 8.06E+01 | 9.33E-03 | 6.79E+03 | 2.77E+02 | 3.46E-04 | 1.20E+03 | 5.60E+01 | 1.83E-04 | 1.21E+03 | 4.23E+01 | 1.78E-02 | 5.13E+03 | 4.91E+02 | 1.70E-04 |
| 1.36E+03 | 4.82E+01 | 7.26E-04 | 3.09E+03 | 8.45E+01 | 9.46E-03 | 7.18E+03 | 2.92E+02 | 3.69E-04 | 1.22E+03 | 5.69E+01 | 2.49E-04 | 1.23E+03 | 4.31E+01 | 1.79E-02 | 5.39E+03 | 5.16E+02 | 1.81E-04 |
| 1.42E+03 | 5.05E+01 | 7.80E-04 | 3.26E+03 | 8.90E+01 | 9.59E-03 | 7.59E+03 | 3.09E+02 | 3.91E-04 | 1.25E+03 | 5.83E+01 | 3.10E-04 | 1.27E+03 | 4.44E+01 | 1.79E-02 | 5.66E+03 | 5.42E+02 | 1.92E-04 |
| 1.48E+03 | 5.25E+01 | 8.37E-04 | 3.42E+03 | 9.35E+01 | 9.72E-03 | 8.00E+03 | 3.26E+02 | 4.14E-04 | 1.29E+03 | 6.02E+01 | 3.67E-04 | 1.33E+03 | 4.67E+01 | 1.80E-02 | 5.95E+03 | 5.70E+02 | 2.03E-04 |
| 1.51E+03 | 5.37E+01 | 9.00E-04 | 3.58E+03 | 9.79E+01 | 9.85E-03 | 8.41E+03 | 3.42E+02 | 4.41E-04 | 1.34E+03 | 6.22E+01 | 4.24E-04 | 1.42E+03 | 4.98E+01 | 1.80E-02 | 6.25E+03 | 5.99E+02 | 2.14E-04 |
| 1.53E+03 | 5.45E+01 | 9.67E-04 | 3.75E+03 | 1.02E+02 | 9.97E-03 | 8.83E+03 | 3.59E+02 | 4.71E-04 | 1.38E+03 | 6.40E+01 | 4.82E-04 | 1.52E+03 | 5.34E+01 | 1.81E-02 | 6.56E+03 | 6.29E+02 | 2.26E-04 |
| 1.56E+03 | 5.55E+01 | 1.03E-03 | 3.92E+03 | 1.07E+02 | 1.01E-02 | 9.24E+03 | 3.76E+02 | 4.99E-04 | 1.41E+03 | 6.55E+01 | 5.43E-04 | 1.63E+03 | 5.72E+01 | 1.82E-02 | 6.88E+03 | 6.59E+02 | 2.37E-04 |
| 1.61E+03 | 5.72E+01 | 1.09E-03 | 4.11E+03 | 1.12E+02 | 1.02E-02 | 9.65E+03 | 3.93E+02 | 5.26E-04 | 1.43E+03 | 6.65E+01 | 6.07E-04 | 1.74E+03 | 6.11E+01 | 1.82E-02 | 7.19E+03 | 6.89E+02 | 2.48E-04 |
| 1.66E+03 | 5.90E+01 | 1.15E-03 | 4.29E+03 | 1.17E+02 | 1.04E-02 | 1.01E+04 | 4.10E+02 | 5.55E-04 | 1.45E+03 | 6.75E+01 | 6.72E-04 | 1.85E+03 | 6.49E+01 | 1.83E-02 | 7.51E+03 | 7.19E+02 | 2.59E-04 |
| 1.70E+03 | 6.05E+01 | 1.21E-03 | 4.47E+03 | 1.22E+02 | 1.05E-02 | 1.05E+04 | 4.27E+02 | 5.84E-04 | 1.47E+03 | 6.85E+01 | 7.36E-04 | 1.95E+03 | 6.84E+01 | 1.83E-02 | 7.82E+03 | 7.49E+02 | 2.70E-04 |
| 1.73E+03 | 6.15E+01 | 1.27E-03 | 4.64E+03 | 1.27E+02 | 1.06E-02 | 1.09E+04 | 4.44E+02 | 6.15E-04 | 1.50E+03 | 6.98E+01 | 7.98E-04 | 2.04E+03 | 7.16E+01 | 1.84E-02 | 8.12E+03 | 7.77E+02 | 2.81E-04 |
| 1.75E+03 | 6.24E+01 | 1.34E-03 | 4.82E+03 | 1.32E+02 | 1.07E-02 | 1.13E+04 | 4.61E+02 | 6.45E-04 | 1.53E+03 | 7.13E+01 | 8.59E-04 | 2.12E+03 | 7.45E+01 | 1.85E-02 | 8.41E+03 | 8.05E+02 | 2.92E-04 |
| 1.78E+03 | 6.34E+01 | 1.40E-03 | 5.01E+03 | 1.37E+02 | 1.09E-02 | 1.17E+04 | 4.78E+02 | 6.77E-04 | 1.56E+03 | 7.28E+01 | 9.19E-04 | 2.20E+03 | 7.71E+01 | 1.85E-02 | 8.70E+03 | 8.33E+02 | 3.04E-04 |




Note 1: Trendlines are for general checking purposes only.

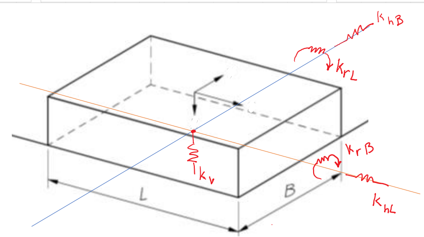


Figure K-2- An example of undulations in load-displacement curves

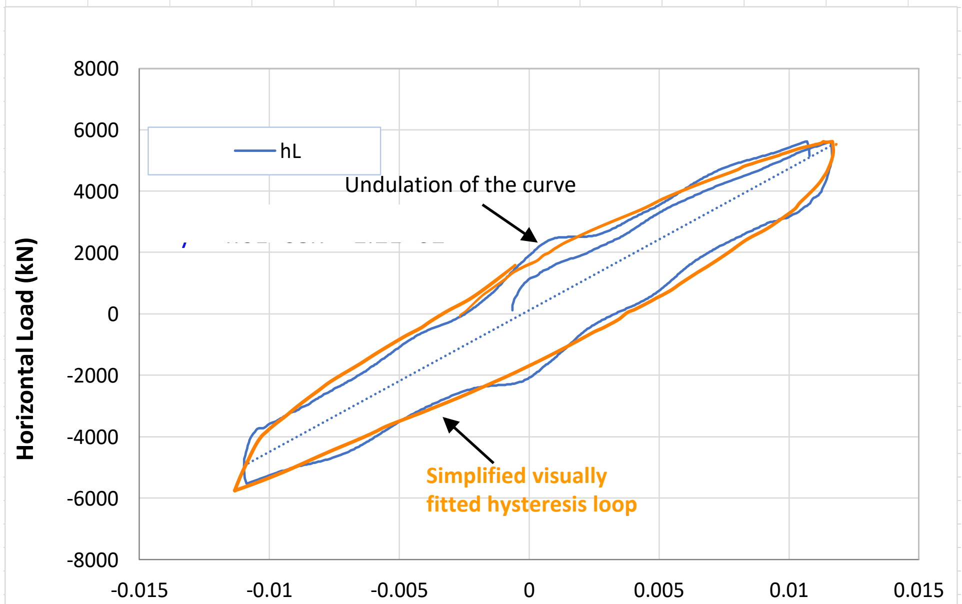
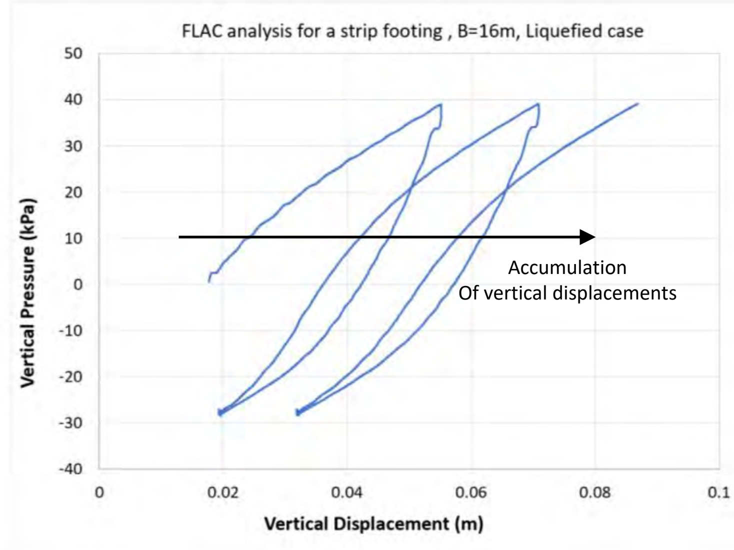
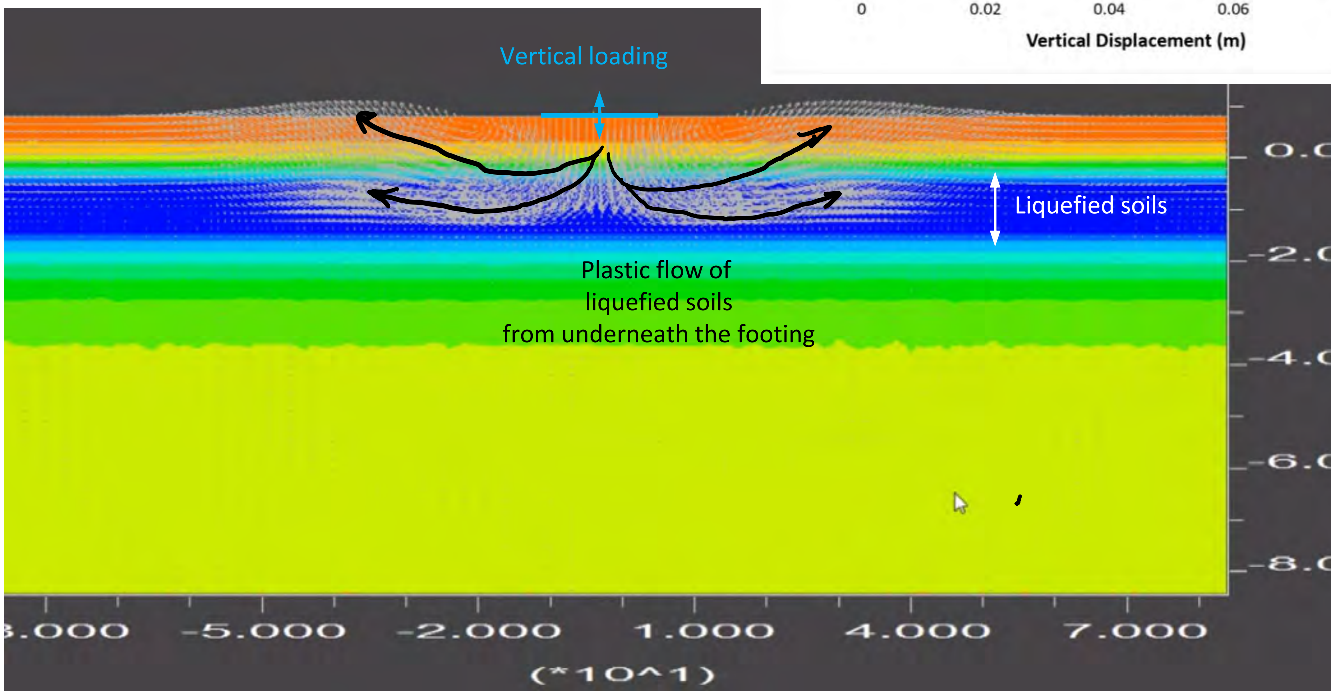


Figure K-3- An example of vertical cyclic response of a strip footing in liquefied conditions in the FLAC model

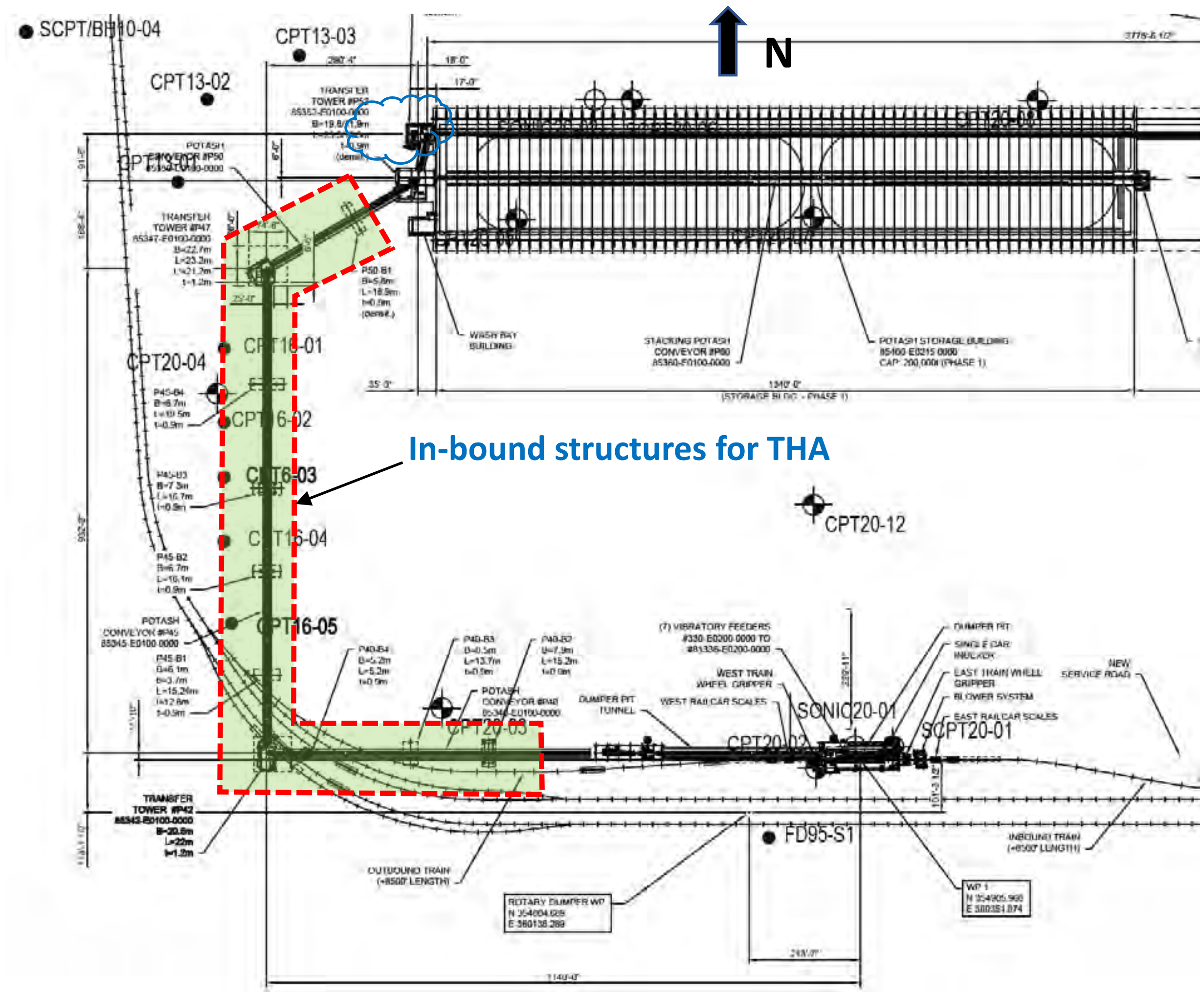
The liquefied soil flows away from underneath the footing and results in permanent displacement in each cycle which accumulates with increasing number of cycles.



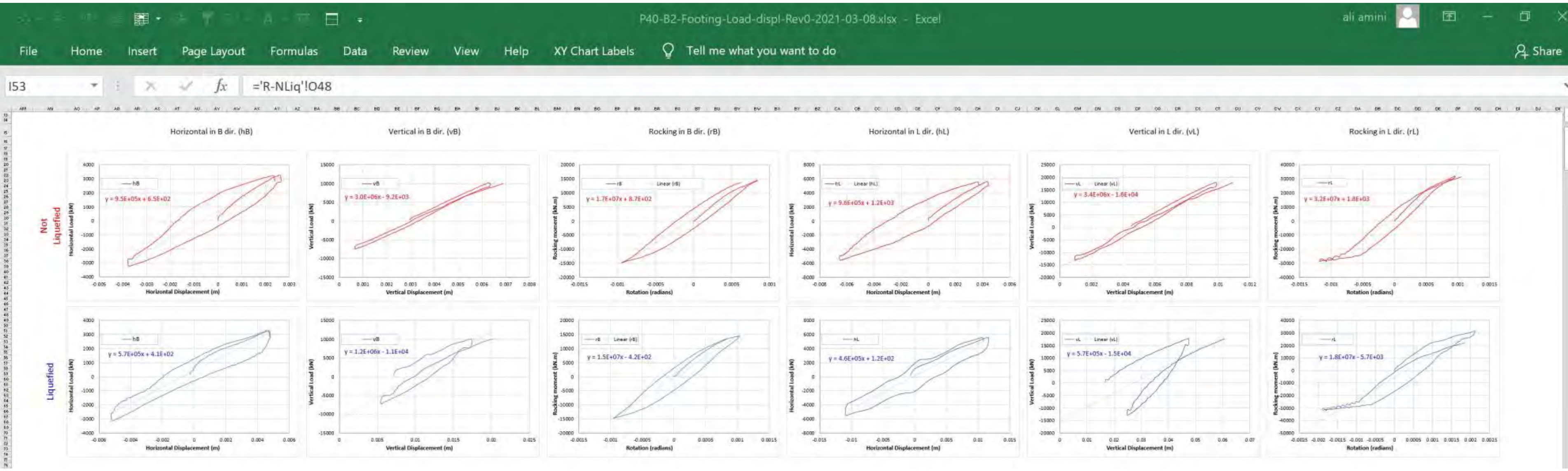



# Load-Displacement Curves for In-bound Footings

Site location plan of the In-bound structure



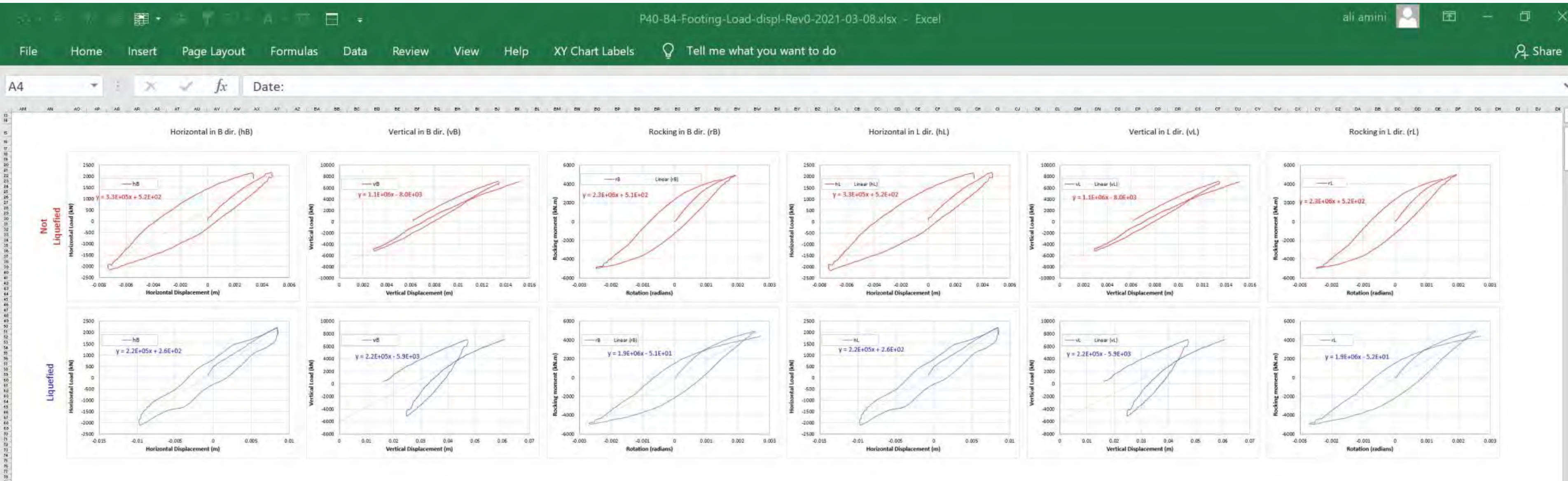
# P40-B2


P40-B2-Footing-Load-displ-Rev0-2021-03-08.xlsx - Excel
File
Home
Insert
Page Layout
Formulas
Data
Review
View
Help
XY Chart Labels
Tell me what you want to do
Share
I53
='R-NLiq'!O48
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied
y = 9.5E+05x + 6.5E+02
y = 3.0E+06x - 9.2E+03
y = 1.7E+07x + 8.7E+02
y = 9.6E+05x + 1.2E+03
y = 3.4E+06x - 1.6E+04
y = 3.2E+07x + 1.8E+03
y = 5.7E+05x + 4.1E+02
y = 1.2E+06x - 1.1E+04
y = 1.5E+07x - 4.2E+02
y = 4.6E+05x + 1.2E+02
y = 5.7E+05x - 1.5E+04
y = 1.8E+07x - 5.7E+03
Horizontal Displacement (m)
Vertical Displacement (m)
Rotation (radians)

# P40-B3


P40-B3-Footing-Load-displ-Rev0-2021-03-08.xlsx - Excel
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied
y = 9.8E+05x + 8.7E+02
y = 3.1E+06x - 9.6E+03
y = 2.0E+07x + 1.0E+03
y = 8.6E+05x + 1.1E+03
y = 2.4E+07x + 1.3E+03
y = 5.9E+05x + 4.3E+02
y = 1.2E+06x - 1.2E+04
y = 1.7E+07x - 4.8E+02
y = 4.1E+05x + 1.0E+02
y = 5.1E+05x - 1.4E+04
y = 1.3E+07x - 4.2E+03

# P40-B4


P40-B4-Footing-Load-displ-Rev0-2021-03-08.xlsx - Excel
File
Home
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Formulas
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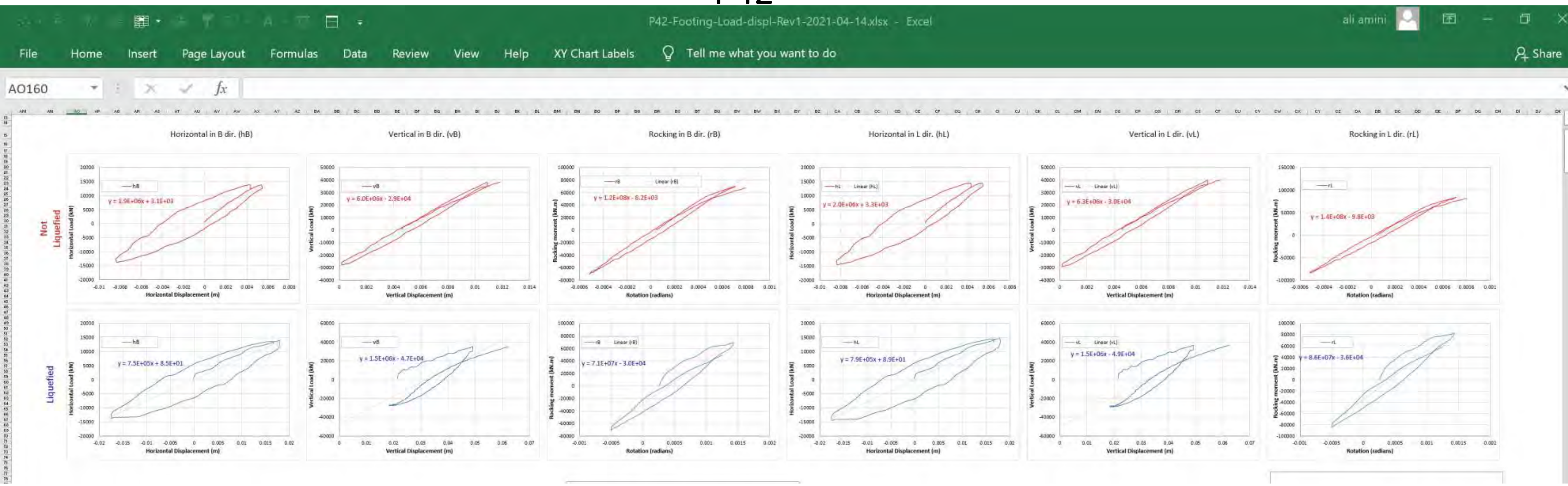
XY Chart Labels
Tell me what you want to do
Share
A4
Date:
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied

# P42


P42-Footing-Load-displ-Rev1-2021-04-14.xlsx - Excel
ali amini
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Page Layout
Formulas
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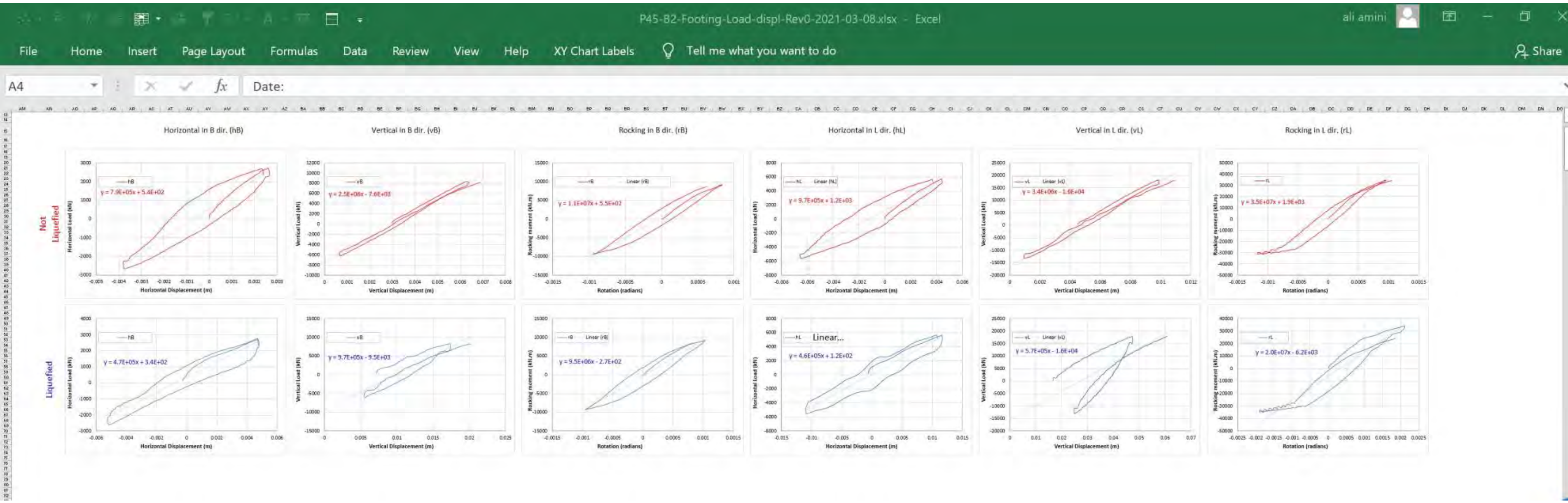
XY Chart Labels
Tell me what you want to do
Share
AO160
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied

# P45-B1


P45-B1-Footing-Load-displ-Rev0-2021-03-08.xlsx - Excel
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied

# P45-B2


P45-B2-Footing-Load-displ-Rev0-2021-03-08.xlsx - Excel
ali amini
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Page Layout
Formulas
Data
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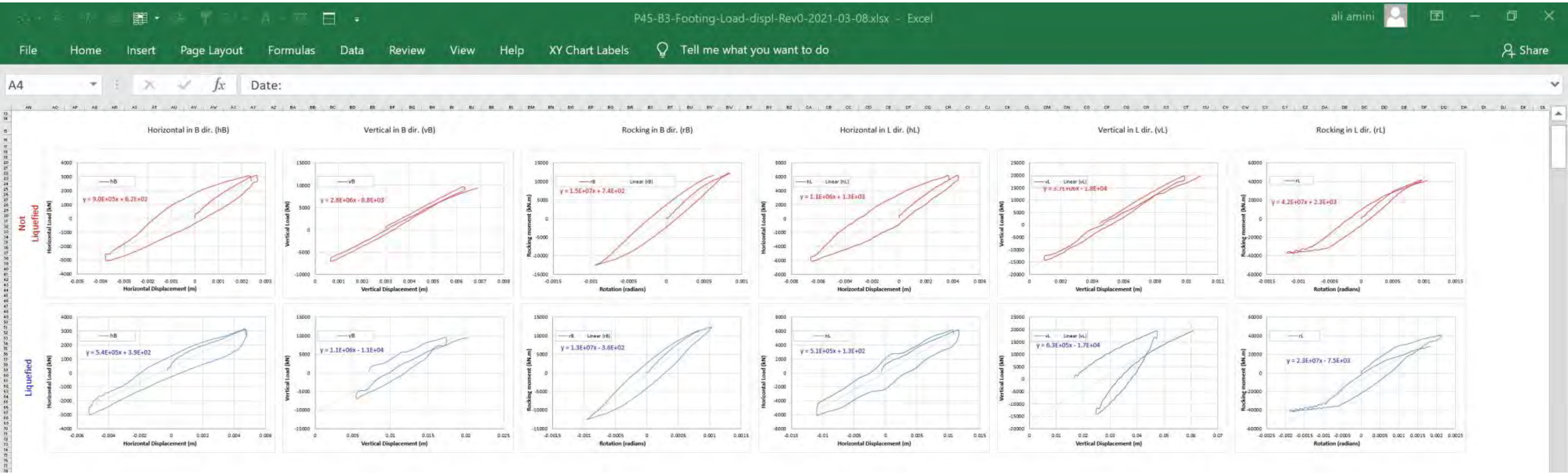
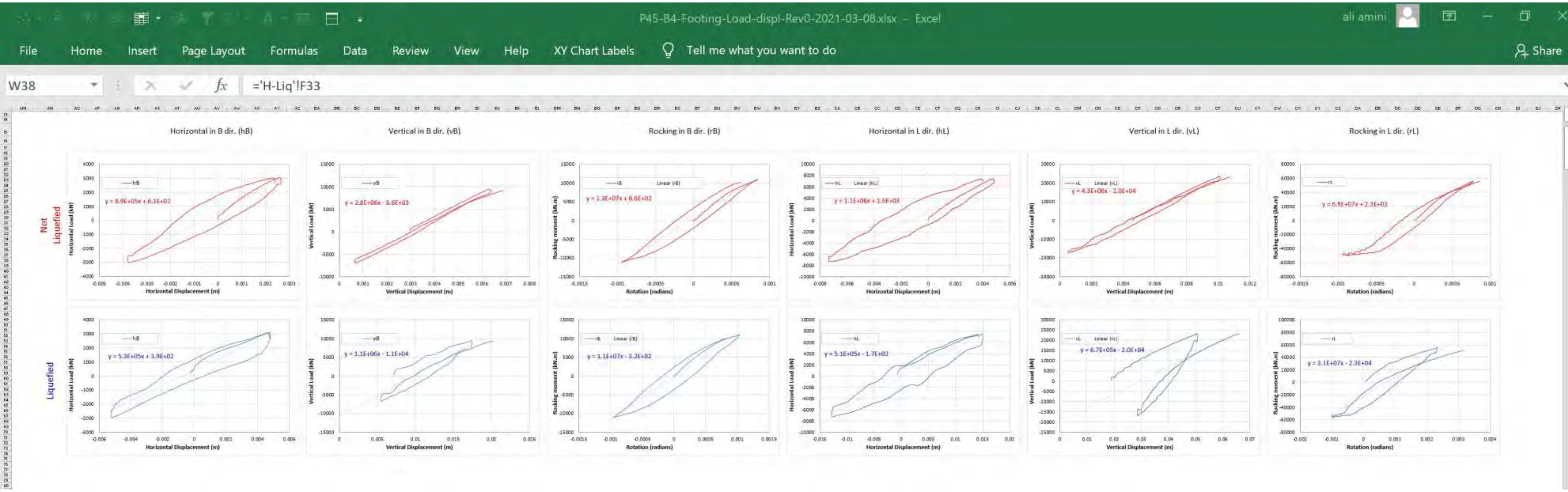
Help
XY Chart Labels
Tell me what you want to do
Share
A4
Date:
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied

# P45-B3


P45-B3-Footing-Load-displ-Rev0-2021-03-08.xlsx - Excel
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied

# P45-B4


P45-B4-Footing-Load-displ-Rev0-2021-03-08.xlsx - Excel
File
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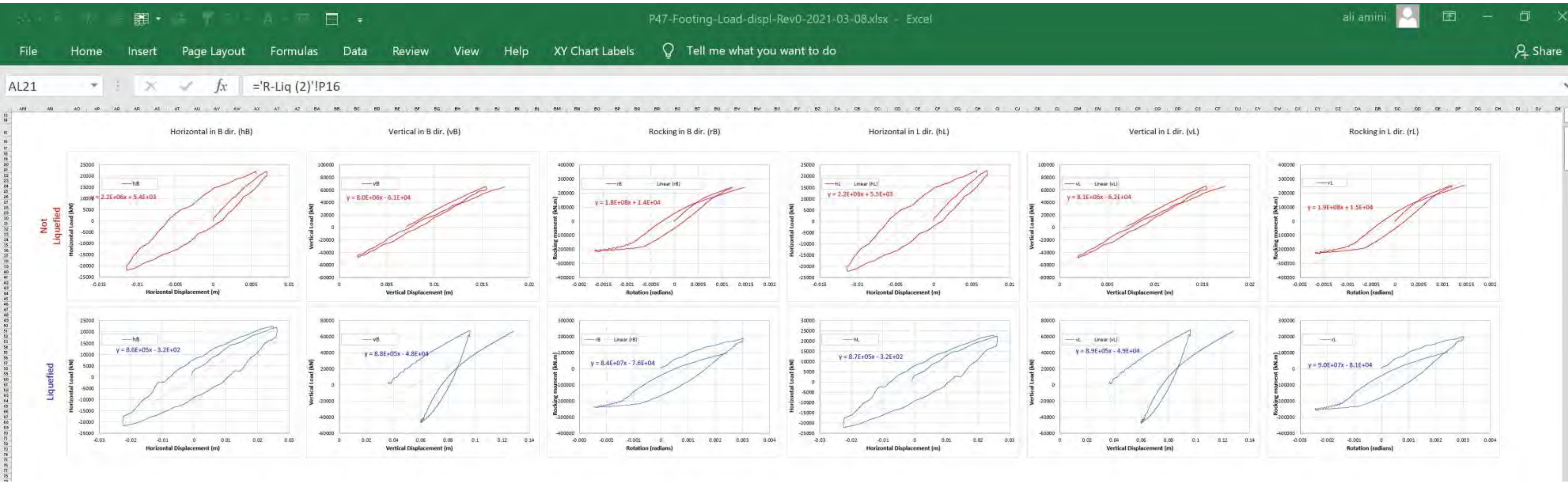
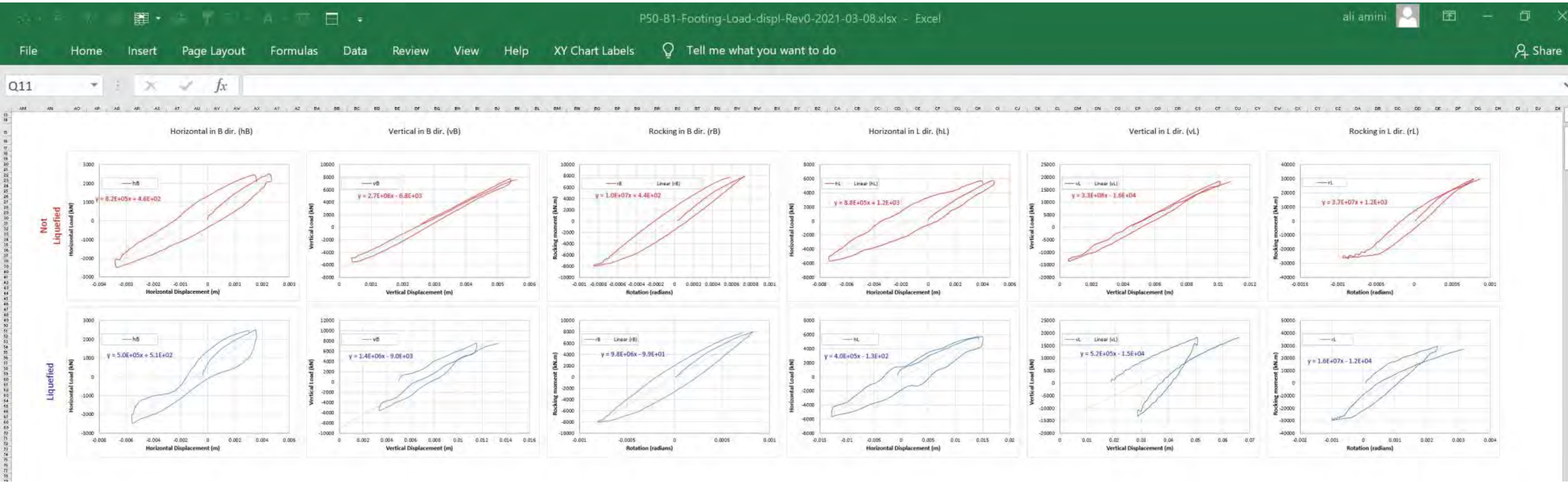
Page Layout
Formulas
Data
Review
View
Help
XY Chart Labels
Tell me what you want to do
Share
W38
='H-Liq'!F33
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied

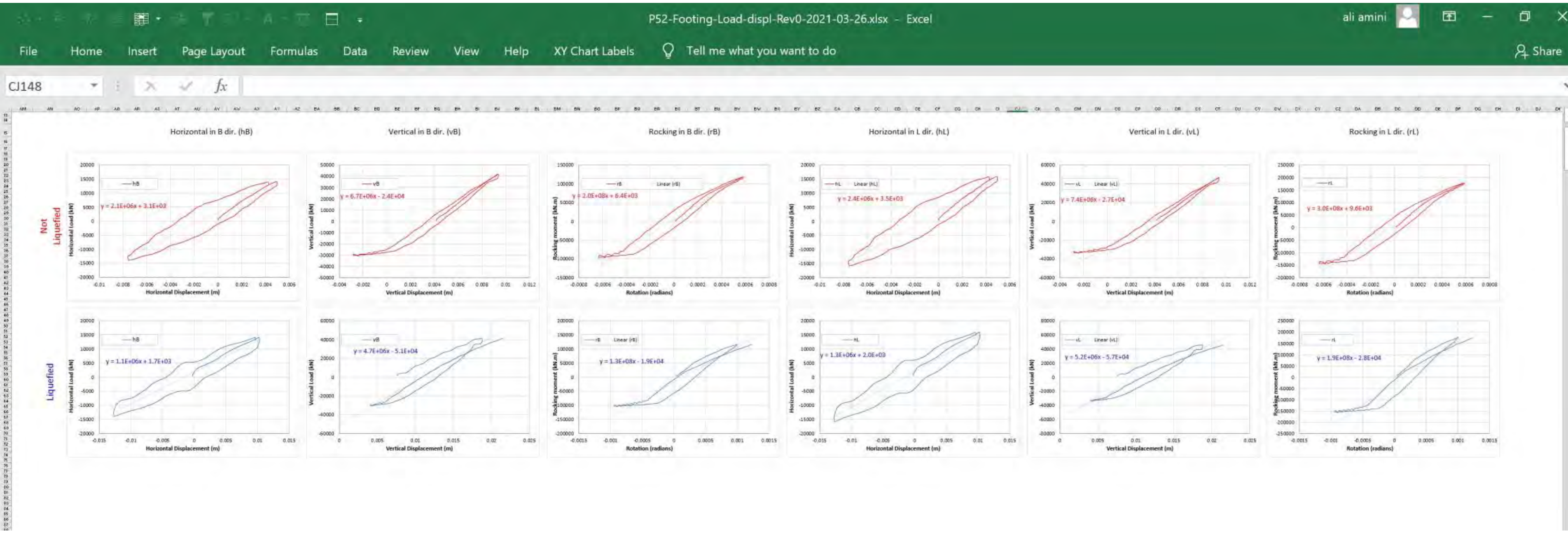
# P47


P47-Footing-Load-displ-Rev0-2021-03-08.xlsx - Excel
File
Home
Insert
Page Layout
Formulas
Data
Review
View
Help
XY Chart Labels
Tell me what you want to do
Share
AL21
='R-Liq (2)'!P16
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied
y = 2.2E+06x + 5.4E+03
y = 8.0E+06x - 6.1E+04
y = 1.8E+08x + 1.4E+04
y = 2.2E+06x + 5.5E+03
y = 8.1E+06x - 6.2E+04
y = 1.9E+08x + 1.5E+04
y = 8.6E+05x - 3.2E+02
y = 8.8E+05x - 4.8E+04
y = 8.4E+07x - 7.6E+04
y = 8.7E+05x - 3.2E+02
y = 8.9E+05x - 4.9E+04
y = 9.0E+07x - 8.1E+04

# P50


P50-B1-Footing-Load-displ-Rev0-2021-03-08.xlsx - Excel
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied

# P52


P52-Footing-Load-displ-Rev0-2021-03-26.xlsx - Excel
File
Home
Insert
Page Layout
Formulas
Data
Review
View
Help
XY Chart Labels
Tell me what you want to do
Share
CJ148
Horizontal in B dir. (hB)
Vertical in B dir. (vB)
Rocking in B dir. (rB)
Horizontal in L dir. (hL)
Vertical in L dir. (vL)
Rocking in L dir. (rL)
Not Liquefied
Liquefied

# Appendix L

## Geotechnical Input for Non-Linear Structural Analysis of In-Bound Structures

**Content**

1. The objective of this document is to provide free field horizontal (East-West and North-South) and vertical time histories of displacement and acceleration at the location of the proposed in-bound structure footings for 6 sets of selected A2475 design ground motions. These time histories will be used by HPC in the structural time history analyses.
2. The shaded area on Figure L-1 shows the locations of the inbound structures.
3. The provided displacement time histories are “total” displacements. The time histories at the base of the FLAC models are also provided so the relative displacements can be derived. As discussed, HPC will calculated the relative displacement time histories.
4. A methodology to derive relative displacement time histories (correction for the base movements) is suggested as follows.
    - Relative horizontal displacement time history can be obtained by subtracting the ground surface horizontal displacement time history from a best fit curve on the base horizontal displacement time history. This may be done for E-W and N-S directions separately. See Figure L-5 as an example.
    - Relative vertical displacement time history can be obtained using the same procedure as above but includes an extra step of adding the base corrected vertical displacement time history from one direction to the best fit on the vertical displacement time history from the orthogonal direction. The reason for this extra step is that there are two surface time history for vertical displacements for each location, i.e. one from E-W FLAC model and one from N-S FLAC model. Figure L-6 illustrates the suggested method.
5. Ground motions: Table L-1 presents the list of design ground motions selected (by HPC) for time history analysis. Design ground motions have been developed by Golder 2016 for Massey Tunnel Replacement Project (the Golder report is publicly available). The ground motions were scaled linearly (by Braun/NAGL) by a factor of 1.09 to account for a higher design PGA and spectral accelerations at the Westshore site relative to Massey Tunnel site.
6. Footing Springs: Refer to Braun/NAGL Update #1, 20-8543-2021-02-11-Rev2 for the methodology and results of footing springs. The provided springs are linear (interpreted from the push over analysis) as per request of CWA. These linear springs can be used for initial time history analysis. The non-linear load-displacement curves for each footing will be provided under a separate cover at a later date.

**Methodology for development of time histories**

1. Select FLAC analysis cases with free field conditions from the geotechnical assessment phase (see Geotechnical assessment report Doc # : 20-8543-2020-12-16 for more information).
    - WS-44-2475 For the east-west (E-W) direction see Figure L-2 (This includes the coal stockpile)
    - WS-N-S-15-2475 For the north-south (N-S) direction see Figure L-3 (This does not include the coal stockpile)
2. Apply the vertical component and the 1st horizontal component of each set of design ground motion to the base of the E-W FLAC model.
3. Apply the vertical component and the 2nd horizontal component of each set of design ground motion to the base of the N-S FLAC model.
4. Repeat steps 2 and 3 for the 6 sets of selected ground motions.
5. Run the analyses to the end of earthquake shaking. It was not necessary to run the analyses for the post-earthquake conditions as the structures are outside the unstable (flow slide failure) zones of the slopes.
6. Extract select time histories at the locations of the structures.
    a) Table L=2-1 & L-2-2 present the list of provided time histories; a total of 960 text files. Each text file includes displacements in meters or accelerations in m/s^2 versus time in seconds. The time interval is about 0.01 second. Table L-4 shows an example.
    960 time histories = 10 locations x 6 sets of ground motions x 2 directions x 8 histories per direction
    a) Table L-3 shows an example of the list of provided time histories for one structure and one set of ground motion
    b) The location of each structure in FLAC models was determined based on its distance from the west or south slope crest (Figure L-1, L-2 & L-3).
    c) Sign Convention of displacements:
        - Horizontal Displacement: (+) means displacement to East or to South
        - Vertical Displacement: (+) means upward displacement

**Uncertainties and limitations:**

1. Seismic analyses generally include considerable uncertainty and simplifications.
2. Settlements from static, post-earthquake reconsolidation, bearing capacity shear strain pattern , soil-structure interaction and potential loss of material from underneath the footings (ejecta) are not included in the displacement time histories.
3. Differential displacements between footings due to uncertainties (+/- 15% of the total displacements assumed in the design) are not included in the displacement time histories.
4. Time histories are from 2D analyses. It is assumed that the displacement time histories from the analyses in E-W and N-S occur simultaneously.
5. Ground improvement at P50-B1 is not included in the analyses (negligible effect).
6. The E-W and N-S base case analyses have been used for the entire extent of the in-bound structures. The effects of local variation of ground conditions are not included.
7. The effect of the coal stockpiles are not considered in the north-south direction. This is justifiable for most footings. Coal stockpiles will likely cause some more permanent displacement to some footings e.g. P40-B2 & B3.

Figure L-1: Site location plan of the in-bound structures for time history analysis

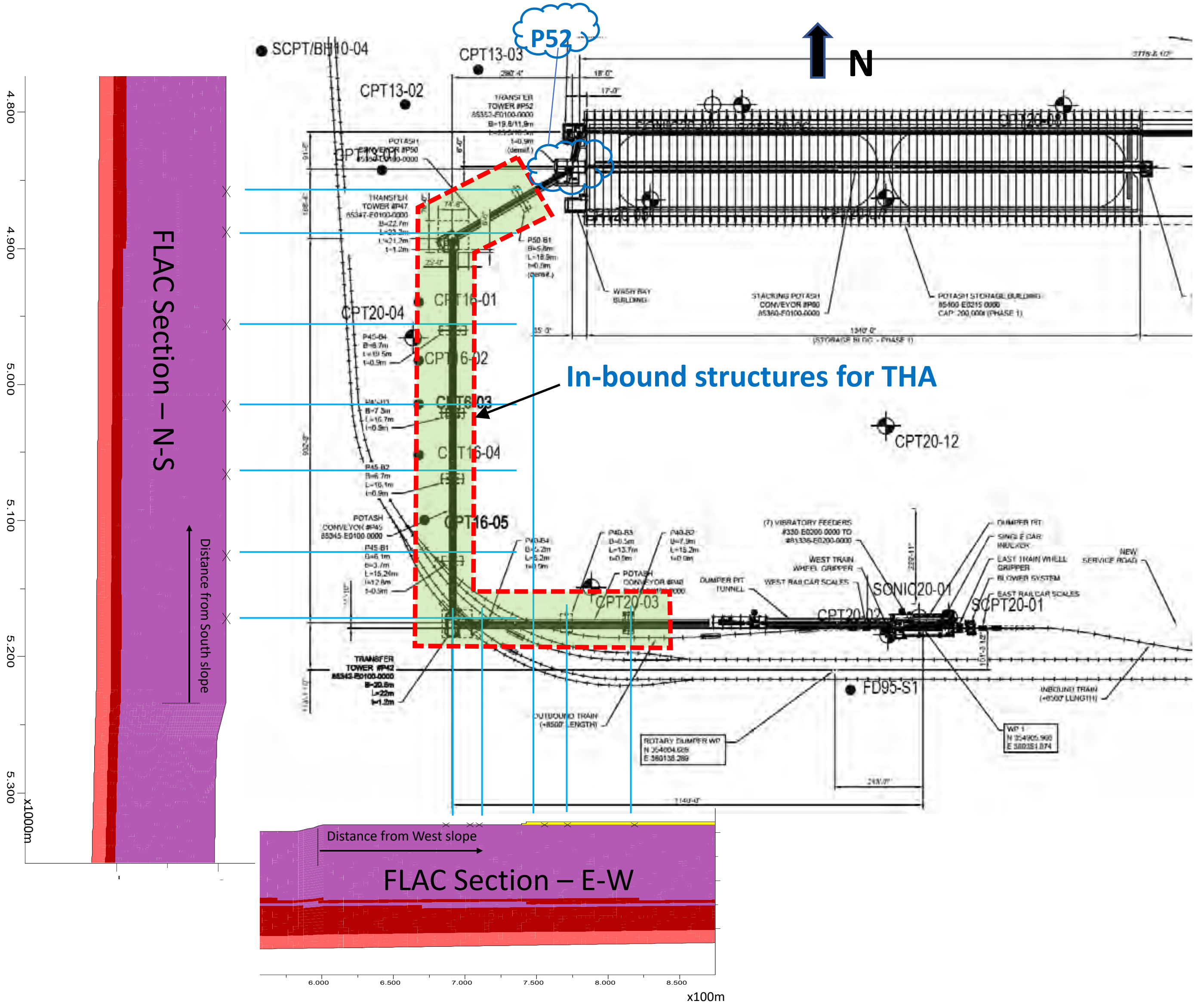
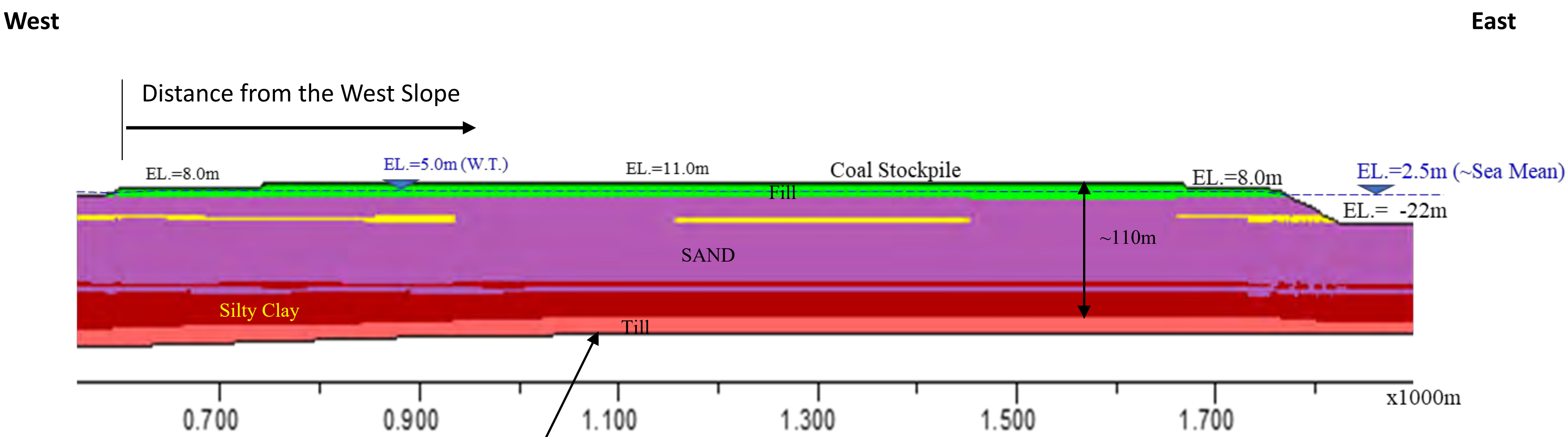


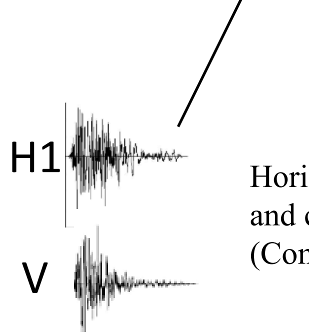
Table L-1: Ground motions selected by HPC for time history analysis

| Type | GM | Component | FLAC Run Time (s) | Duration (s) | Ground Motion Description | No. of Points | Time Interval (s) | FLAC Section |
|---|---|---|---|---|---|---|---|---|
| **Crustal** | CRU031 | Horizontal 1 | 100 | 98.0 | 2475-LANDERS1992-MVP000n | 19595 | 0.005 | E-W |
| | CRU032 | Vertical | 100 | 98.0 | 2475-LANDERS1992-MVP-up (vertical) | 19595 | 0.005 | Both |
| | CRU033 | Horizontal 2 | 100 | 98.0 | 2475-LANDERS1992-MVP090n | 19595 | 0.005 | N-S |
| **In-Slab** | INS021 | Horizontal 1 | 65 | 62.5 | 2475-ElSalvador2001-R113_RF-180n | 12500 | 0.005 | E-W |
| | INS022 | Vertical | 65 | 62.5 | 2475-ElSalvador2001-R113_RF-up (vertical) | 12500 | 0.005 | Both |
| | INS023 | Horizontal 2 | 65 | 62.5 | 2475-ElSalvador2001-R113_RF-90n | 12500 | 0.005 | N-S |
| | INS041 | Horizontal 1 | 92 | 91.5 | 2475-Nisqually2001-R75-125n | 18300 | 0.005 | E-W |
| | INS042 | Vertical | 92 | 91.5 | 2475-Nisqually2001-R75-Up (vertical) | 18300 | 0.005 | Both |
| | INS043 | Horizontal 2 | 92 | 91.5 | 2475-Nisqually2001-R75-215n | 18300 | 0.005 | N-S |
| **Interfcae** | INTF021 | Horizontal 1 | 210 | 207.5 | 2475-Tohoku_2011_M9.0_R209_YMT008_EWn | 20752 | 0.01 | E-W |
| | INTF022 | Vertical | 210 | 207.5 | 2475-Tohoku_2011_M9.0_R209_YMT008_up (vertical) | 20752 | 0.01 | Both |
| | INTF023 | Horizontal 2 | 210 | 207.5 | 2475-Tohoku_2011_M9.0_R209_YMT008_NSn | 20752 | 0.01 | N-S |
| | INTF031 | Horizontal 1 | 220 | 218.0 | 2475-Tohoku_2011_M9.0_R230-IWT022-EWn | 21800 | 0.01 | E-W |
| | INTF032 | Vertical | 220 | 218.0 | 2475-Tohoku_2011_M9.0_R230-IWT022-up (vertical) | 21800 | 0.01 | Both |
| | INTF033 | Horizontal 2 | 220 | 218.0 | 2475-Tohoku_2011_M9.0_R230-IWT022-NSn | 21800 | 0.01 | N-S |
| | INTF041 | Horizontal 1 | 130 | 128.5 | 2475-Tokachioki2003_R152-EWn | 12845 | 0.01 | E-W |
| | INTF042 | Vertical | 130 | 128.5 | 2475-Tokachioki2003_R152-up (vertical) | 12845 | 0.01 | Both |
| | INTF043 | Horizontal 2 | 130 | 128.5 | 2475-Tokachioki2003_R152-NSn | 12845 | 0.01 | N-S |

THA-Inboud.xlsx

Figure L-2 E-W FLAC Section
(Only a portion of the FLAC model is shown for clarity)

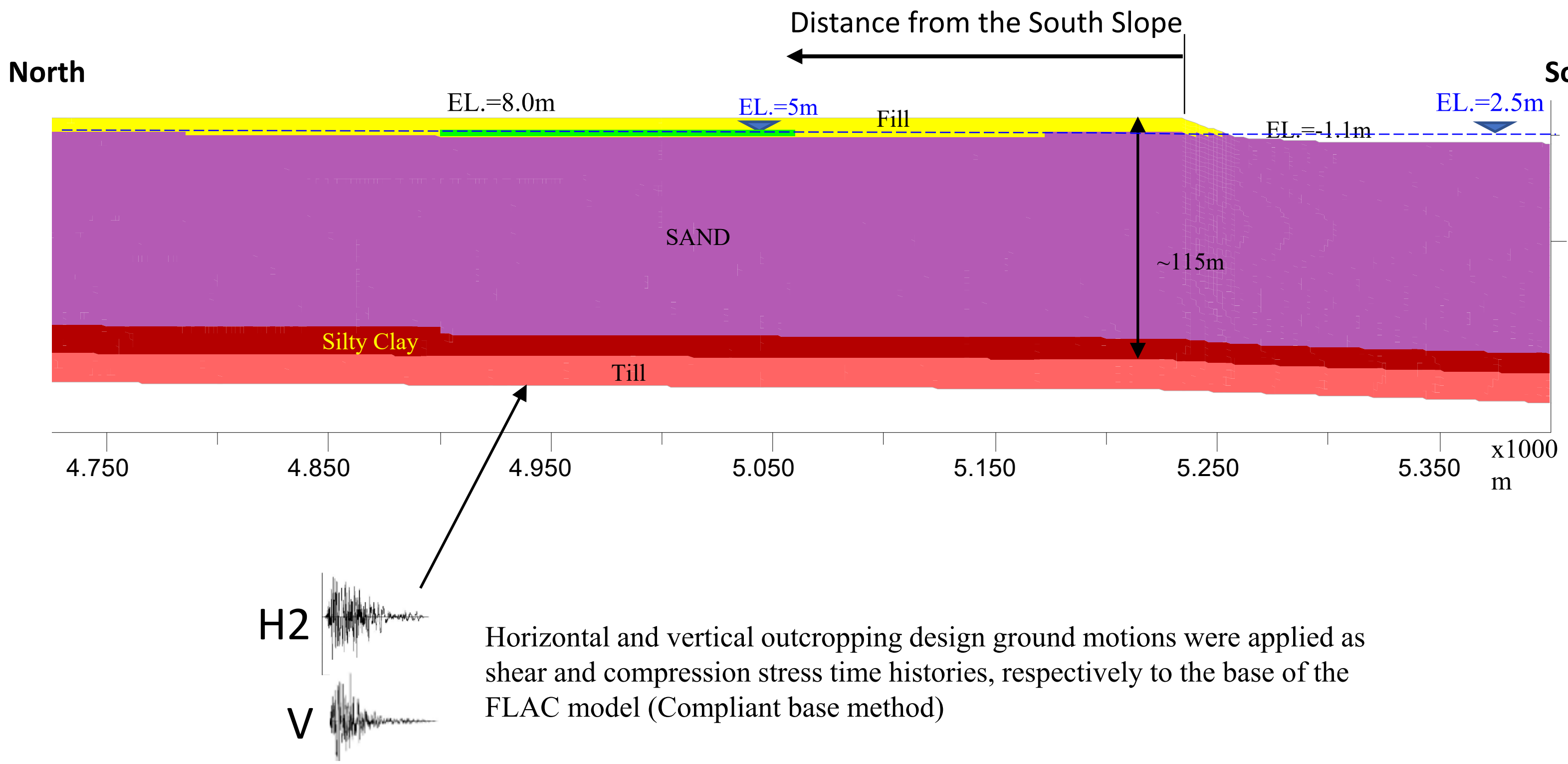



Horizontal and vertical outcropping design ground motions were applied as shear and compression stress time histories, respectively to the base of the FLAC model (Compliant base method)

**Figure L-3 N-S FLAC Section**

(Only a portion of the FLAC model is shown for clarity)




Horizontal and vertical outcropping design ground motions were applied as shear and compression stress time histories, respectively to the base of the FLAC model (Compliant base method)

Table L-2: List of output time histories (provided as .log and .jpg files)

| Ground Motion | | Time History Data | | | P40-B2 | P40-B3 | P40-B4 | P42 | P45-B1 | P45-B2 | P45-B3 | P45-B4 | P47 | P50-B1 | P52 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Time History Filenames (.log text files) | | | | | | | | | | |
| Distance from the West Slope (m) | | | | | 216 | 171 | 110 | 86 | 90 | 94 | 96 | 100 | 105 | 158 | 196 |
| Distance from the South Slope (m) | | | | | 62 | 62 | 62 | 62 | 107 | 168 | 217 | 278 | 346 | 375 | 397 |
| CRU03 | EW | CRU031 | Horiz. Disp. Top | xd_t | EW_CRU031_P40-B2_xd_t | EW_CRU031_P40-B3_xd_t | EW_CRU031_P40-B4_xd_t | EW_CRU031_P42_xd_t | EW_CRU031_P45-B1_xd_t | EW_CRU031_P45-B2_xd_t | EW_CRU031_P45-B3_xd_t | EW_CRU031_P45-B4_xd_t | EW_CRU031_P47_xd_t | EW_CRU031_P50-B1_xd_t | EW_CRU031_P52_xd_t |
| | | CRU031 | Horiz. Disp. Bott | xd_b | EW_CRU031_P40-B2_xd_b | EW_CRU031_P40-B3_xd_b | EW_CRU031_P40-B4_xd_b | EW_CRU031_P42_xd_b | EW_CRU031_P45-B1_xd_b | EW_CRU031_P45-B2_xd_b | EW_CRU031_P45-B3_xd_b | EW_CRU031_P45-B4_xd_b | EW_CRU031_P47_xd_b | EW_CRU031_P50-B1_xd_b | EW_CRU031_P52_xd_b |
| | | CRU031 | Vertical Disp. Top | yd_t | EW_CRU031_P40-B2_yd_t | EW_CRU031_P40-B3_yd_t | EW_CRU031_P40-B4_yd_t | EW_CRU031_P42_yd_t | EW_CRU031_P45-B1_yd_t | EW_CRU031_P45-B2_yd_t | EW_CRU031_P45-B3_yd_t | EW_CRU031_P45-B4_yd_t | EW_CRU031_P47_yd_t | EW_CRU031_P50-B1_yd_t | EW_CRU031_P52_yd_t |
| | | CRU031 | Vertical Disp. Bott | yd_b | EW_CRU031_P40-B2_yd_b | EW_CRU031_P40-B3_yd_b | EW_CRU031_P40-B4_yd_b | EW_CRU031_P42_yd_b | EW_CRU031_P45-B1_yd_b | EW_CRU031_P45-B2_yd_b | EW_CRU031_P45-B3_yd_b | EW_CRU031_P45-B4_yd_b | EW_CRU031_P47_yd_b | EW_CRU031_P50-B1_yd_b | EW_CRU031_P52_yd_b |
| | | CRU031 | Horiz. Acc. Top | xa_t | EW_CRU031_P40-B2_xa_t | EW_CRU031_P40-B3_xa_t | EW_CRU031_P40-B4_xa_t | EW_CRU031_P42_xa_t | EW_CRU031_P45-B1_xa_t | EW_CRU031_P45-B2_xa_t | EW_CRU031_P45-B3_xa_t | EW_CRU031_P45-B4_xa_t | EW_CRU031_P47_xa_t | EW_CRU031_P50-B1_xa_t | EW_CRU031_P52_xa_t |
| | | CRU031 | Horiz. Acc. Bott | xa_b | EW_CRU031_P40-B2_xa_b | EW_CRU031_P40-B3_xa_b | EW_CRU031_P40-B4_xa_b | EW_CRU031_P42_xa_b | EW_CRU031_P45-B1_xa_b | EW_CRU031_P45-B2_xa_b | EW_CRU031_P45-B3_xa_b | EW_CRU031_P45-B4_xa_b | EW_CRU031_P47_xa_b | EW_CRU031_P50-B1_xa_b | EW_CRU031_P52_xa_b |
| | | CRU031 | Vertical Acc. Top | ya_t | EW_CRU031_P40-B2_ya_t | EW_CRU031_P40-B3_ya_t | EW_CRU031_P40-B4_ya_t | EW_CRU031_P42_ya_t | EW_CRU031_P45-B1_ya_t | EW_CRU031_P45-B2_ya_t | EW_CRU031_P45-B3_ya_t | EW_CRU031_P45-B4_ya_t | EW_CRU031_P47_ya_t | EW_CRU031_P50-B1_ya_t | EW_CRU031_P52_ya_t |
| | | CRU031 | Vertical Acc. Bott | ya_b | EW_CRU031_P40-B2_ya_b | EW_CRU031_P40-B3_ya_b | EW_CRU031_P40-B4_ya_b | EW_CRU031_P42_ya_b | EW_CRU031_P45-B1_ya_b | EW_CRU031_P45-B2_ya_b | EW_CRU031_P45-B3_ya_b | EW_CRU031_P45-B4_ya_b | EW_CRU031_P47_ya_b | EW_CRU031_P50-B1_ya_b | EW_CRU031_P52_ya_b |
| | NS | CRU033 | Horiz. Disp. Top | xd_t | NS_CRU033_P40-B2_xd_t | NS_CRU033_P40-B3_xd_t | NS_CRU033_P40-B4_xd_t | NS_CRU033_P42_xd_t | NS_CRU033_P45-B1_xd_t | NS_CRU033_P45-B2_xd_t | NS_CRU033_P45-B3_xd_t | NS_CRU033_P45-B4_xd_t | NS_CRU033_P47_xd_t | NS_CRU033_P50-B1_xd_t | NS_CRU033_P52_xd_t |
| | | CRU033 | Horiz. Disp. Bott | xd_b | NS_CRU033_P40-B2_xd_b | NS_CRU033_P40-B3_xd_b | NS_CRU033_P40-B4_xd_b | NS_CRU033_P42_xd_b | NS_CRU033_P45-B1_xd_b | NS_CRU033_P45-B2_xd_b | NS_CRU033_P45-B3_xd_b | NS_CRU033_P45-B4_xd_b | NS_CRU033_P47_xd_b | NS_CRU033_P50-B1_xd_b | NS_CRU033_P52_xd_b |
| | | CRU033 | Vertical Disp. Top | yd_t | NS_CRU033_P40-B2_yd_t | NS_CRU033_P40-B3_yd_t | NS_CRU033_P40-B4_yd_t | NS_CRU033_P42_yd_t | NS_CRU033_P45-B1_yd_t | NS_CRU033_P45-B2_yd_t | NS_CRU033_P45-B3_yd_t | NS_CRU033_P45-B4_yd_t | NS_CRU033_P47_yd_t | NS_CRU033_P50-B1_yd_t | NS_CRU033_P52_yd_t |
| | | CRU033 | Vertical Disp. Bott | yd_b | NS_CRU033_P40-B2_yd_b | NS_CRU033_P40-B3_yd_b | NS_CRU033_P40-B4_yd_b | NS_CRU033_P42_yd_b | NS_CRU033_P45-B1_yd_b | NS_CRU033_P45-B2_yd_b | NS_CRU033_P45-B3_yd_b | NS_CRU033_P45-B4_yd_b | NS_CRU033_P47_yd_b | NS_CRU033_P50-B1_yd_b | NS_CRU033_P52_yd_b |
| | | CRU033 | Horiz. Acc. Top | xa_t | NS_CRU033_P40-B2_xa_t | NS_CRU033_P40-B3_xa_t | NS_CRU033_P40-B4_xa_t | NS_CRU033_P42_xa_t | NS_CRU033_P45-B1_xa_t | NS_CRU033_P45-B2_xa_t | NS_CRU033_P45-B3_xa_t | NS_CRU033_P45-B4_xa_t | NS_CRU033_P47_xa_t | NS_CRU033_P50-B1_xa_t | NS_CRU033_P52_xa_t |
| | | CRU033 | Horiz. Acc. Bott | xa_b | NS_CRU033_P40-B2_xa_b | NS_CRU033_P40-B3_xa_b | NS_CRU033_P40-B4_xa_b | NS_CRU033_P42_xa_b | NS_CRU033_P45-B1_xa_b | NS_CRU033_P45-B2_xa_b | NS_CRU033_P45-B3_xa_b | NS_CRU033_P45-B4_xa_b | NS_CRU033_P47_xa_b | NS_CRU033_P50-B1_xa_b | NS_CRU033_P52_xa_b |
| | | CRU033 | Vertical Acc. Top | ya_t | NS_CRU033_P40-B2_ya_t | NS_CRU033_P40-B3_ya_t | NS_CRU033_P40-B4_ya_t | NS_CRU033_P42_ya_t | NS_CRU033_P45-B1_ya_t | NS_CRU033_P45-B2_ya_t | NS_CRU033_P45-B3_ya_t | NS_CRU033_P45-B4_ya_t | NS_CRU033_P47_ya_t | NS_CRU033_P50-B1_ya_t | NS_CRU033_P52_ya_t |
| | | CRU033 | Vertical Acc. Bott | ya_b | NS_CRU033_P40-B2_ya_b | NS_CRU033_P40-B3_ya_b | NS_CRU033_P40-B4_ya_b | NS_CRU033_P42_ya_b | NS_CRU033_P45-B1_ya_b | NS_CRU033_P45-B2_ya_b | NS_CRU033_P45-B3_ya_b | NS_CRU033_P45-B4_ya_b | NS_CRU033_P47_ya_b | NS_CRU033_P50-B1_ya_b | NS_CRU033_P52_ya_b |
| INS02 | EW | INS021 | Horiz. Disp. Top | xd_t | EW_INS021_P40-B2_xd_t | EW_INS021_P40-B3_xd_t | EW_INS021_P40-B4_xd_t | EW_INS021_P42_xd_t | EW_INS021_P45-B1_xd_t | EW_INS021_P45-B2_xd_t | EW_INS021_P45-B3_xd_t | EW_INS021_P45-B4_xd_t | EW_INS021_P47_xd_t | EW_INS021_P50-B1_xd_t | EW_INS021_P52_xd_t |
| | | INS021 | Horiz. Disp. Bott | xd_b | EW_INS021_P40-B2_xd_b | EW_INS021_P40-B3_xd_b | EW_INS021_P40-B4_xd_b | EW_INS021_P42_xd_b | EW_INS021_P45-B1_xd_b | EW_INS021_P45-B2_xd_b | EW_INS021_P45-B3_xd_b | EW_INS021_P45-B4_xd_b | EW_INS021_P47_xd_b | EW_INS021_P50-B1_xd_b | EW_INS021_P52_xd_b |
| | | INS021 | Vertical Disp. Top | yd_t | EW_INS021_P40-B2_yd_t | EW_INS021_P40-B3_yd_t | EW_INS021_P40-B4_yd_t | EW_INS021_P42_yd_t | EW_INS021_P45-B1_yd_t | EW_INS021_P45-B2_yd_t | EW_INS021_P45-B3_yd_t | EW_INS021_P45-B4_yd_t | EW_INS021_P47_yd_t | EW_INS021_P50-B1_yd_t | EW_INS021_P52_yd_t |
| | | INS021 | Vertical Disp. Bott | yd_b | EW_INS021_P40-B2_yd_b | EW_INS021_P40-B3_yd_b | EW_INS021_P40-B4_yd_b | EW_INS021_P42_yd_b | EW_INS021_P45-B1_yd_b | EW_INS021_P45-B2_yd_b | EW_INS021_P45-B3_yd_b | EW_INS021_P45-B4_yd_b | EW_INS021_P47_yd_b | EW_INS021_P50-B1_yd_b | EW_INS021_P52_yd_b |
| | | INS021 | Horiz. Acc. Top | xa_t | EW_INS021_P40-B2_xa_t | EW_INS021_P40-B3_xa_t | EW_INS021_P40-B4_xa_t | EW_INS021_P42_xa_t | EW_INS021_P45-B1_xa_t | EW_INS021_P45-B2_xa_t | EW_INS021_P45-B3_xa_t | EW_INS021_P45-B4_xa_t | EW_INS021_P47_xa_t | EW_INS021_P50-B1_xa_t | EW_INS021_P52_xa_t |
| | | INS021 | Horiz. Acc. Bott | xa_b | EW_INS021_P40-B2_xa_b | EW_INS021_P40-B3_xa_b | EW_INS021_P40-B4_xa_b | EW_INS021_P42_xa_b | EW_INS021_P45-B1_xa_b | EW_INS021_P45-B2_xa_b | EW_INS021_P45-B3_xa_b | EW_INS021_P45-B4_xa_b | EW_INS021_P47_xa_b | EW_INS021_P50-B1_xa_b | EW_INS021_P52_xa_b |
| | | INS021 | Vertical Acc. Top | ya_t | EW_INS021_P40-B2_ya_t | EW_INS021_P40-B3_ya_t | EW_INS021_P40-B4_ya_t | EW_INS021_P42_ya_t | EW_INS021_P45-B1_ya_t | EW_INS021_P45-B2_ya_t | EW_INS021_P45-B3_ya_t | EW_INS021_P45-B4_ya_t | EW_INS021_P47_ya_t | EW_INS021_P50-B1_ya_t | EW_INS021_P52_ya_t |
| | | INS021 | Vertical Acc. Bott | ya_b | EW_INS021_P40-B2_ya_b | EW_INS021_P40-B3_ya_b | EW_INS021_P40-B4_ya_b | EW_INS021_P42_ya_b | EW_INS021_P45-B1_ya_b | EW_INS021_P45-B2_ya_b | EW_INS021_P45-B3_ya_b | EW_INS021_P45-B4_ya_b | EW_INS021_P47_ya_b | EW_INS021_P50-B1_ya_b | EW_INS021_P52_ya_b |
| | NS | INS023 | Horiz. Disp. Top | xd_t | NS_INS023_P40-B2_xd_t | NS_INS023_P40-B3_xd_t | NS_INS023_P40-B4_xd_t | NS_INS023_P42_xd_t | NS_INS023_P45-B1_xd_t | NS_INS023_P45-B2_xd_t | NS_INS023_P45-B3_xd_t | NS_INS023_P45-B4_xd_t | NS_INS023_P47_xd_t | NS_INS023_P50-B1_xd_t | NS_INS023_P52_xd_t |
| | | INS023 | Horiz. Disp. Bott | xd_b | NS_INS023_P40-B2_xd_b | NS_INS023_P40-B3_xd_b | NS_INS023_P40-B4_xd_b | NS_INS023_P42_xd_b | NS_INS023_P45-B1_xd_b | NS_INS023_P45-B2_xd_b | NS_INS023_P45-B3_xd_b | NS_INS023_P45-B4_xd_b | NS_INS023_P47_xd_b | NS_INS023_P50-B1_xd_b | NS_INS023_P52_xd_b |
| | | INS023 | Vertical Disp. Top | yd_t | NS_INS023_P40-B2_yd_t | NS_INS023_P40-B3_yd_t | NS_INS023_P40-B4_yd_t | NS_INS023_P42_yd_t | NS_INS023_P45-B1_yd_t | NS_INS023_P45-B2_yd_t | NS_INS023_P45-B3_yd_t | NS_INS023_P45-B4_yd_t | NS_INS023_P47_yd_t | NS_INS023_P50-B1_yd_t | NS_INS023_P52_yd_t |
| | | INS023 | Vertical Disp. Bott | yd_b | NS_INS023_P40-B2_yd_b | NS_INS023_P40-B3_yd_b | NS_INS023_P40-B4_yd_b | NS_INS023_P42_yd_b | NS_INS023_P45-B1_yd_b | NS_INS023_P45-B2_yd_b | NS_INS023_P45-B3_yd_b | NS_INS023_P45-B4_yd_b | NS_INS023_P47_yd_b | NS_INS023_P50-B1_yd_b | NS_INS023_P52_yd_b |
| | | INS023 | Horiz. Acc. Top | xa_t | NS_INS023_P40-B2_xa_t | NS_INS023_P40-B3_xa_t | NS_INS023_P40-B4_xa_t | NS_INS023_P42_xa_t | NS_INS023_P45-B1_xa_t | NS_INS023_P45-B2_xa_t | NS_INS023_P45-B3_xa_t | NS_INS023_P45-B4_xa_t | NS_INS023_P47_xa_t | NS_INS023_P50-B1_xa_t | NS_INS023_P52_xa_t |
| | | INS023 | Horiz. Acc. Bott | xa_b | NS_INS023_P40-B2_xa_b | NS_INS023_P40-B3_xa_b | NS_INS023_P40-B4_xa_b | NS_INS023_P42_xa_b | NS_INS023_P45-B1_xa_b | NS_INS023_P45-B2_xa_b | NS_INS023_P45-B3_xa_b | NS_INS023_P45-B4_xa_b | NS_INS023_P47_xa_b | NS_INS023_P50-B1_xa_b | NS_INS023_P52_xa_b |
| | | INS023 | Vertical Acc. Top | ya_t | NS_INS023_P40-B2_ya_t | NS_INS023_P40-B3_ya_t | NS_INS023_P40-B4_ya_t | NS_INS023_P42_ya_t | NS_INS023_P45-B1_ya_t | NS_INS023_P45-B2_ya_t | NS_INS023_P45-B3_ya_t | NS_INS023_P45-B4_ya_t | NS_INS023_P47_ya_t | NS_INS023_P50-B1_ya_t | NS_INS023_P52_ya_t |
| | | INS023 | Vertical Acc. Bott | ya_b | NS_INS023_P40-B2_ya_b | NS_INS023_P40-B3_ya_b | NS_INS023_P40-B4_ya_b | NS_INS023_P42_ya_b | NS_INS023_P45-B1_ya_b | NS_INS023_P45-B2_ya_b | NS_INS023_P45-B3_ya_b | NS_INS023_P45-B4_ya_b | NS_INS023_P47_ya_b | NS_INS023_P50-B1_ya_b | NS_INS023_P52_ya_b |
| INS04 | EW | INS041 | Horiz. Disp. Top | xd_t | EW_INS041_P40-B2_xd_t | EW_INS041_P40-B3_xd_t | EW_INS041_P40-B4_xd_t | EW_INS041_P42_xd_t | EW_INS041_P45-B1_xd_t | EW_INS041_P45-B2_xd_t | EW_INS041_P45-B3_xd_t | EW_INS041_P45-B4_xd_t | EW_INS041_P47_xd_t | EW_INS041_P50-B1_xd_t | EW_INS041_P52_xd_t |
| | | INS041 | Horiz. Disp. Bott | xd_b | EW_INS041_P40-B2_xd_b | EW_INS041_P40-B3_xd_b | EW_INS041_P40-B4_xd_b | EW_INS041_P42_xd_b | EW_INS041_P45-B1_xd_b | EW_INS041_P45-B2_xd_b | EW_INS041_P45-B3_xd_b | EW_INS041_P45-B4_xd_b | EW_INS041_P47_xd_b | EW_INS041_P50-B1_xd_b | EW_INS041_P52_xd_b |
| | | INS041 | Vertical Disp. Top | yd_t | EW_INS041_P40-B2_yd_t | EW_INS041_P40-B3_yd_t | EW_INS041_P40-B4_yd_t | EW_INS041_P42_yd_t | EW_INS041_P45-B1_yd_t | EW_INS041_P45-B2_yd_t | EW_INS041_P45-B3_yd_t | EW_INS041_P45-B4_yd_t | EW_INS041_P47_yd_t | EW_INS041_P50-B1_yd_t | EW_INS041_P52_yd_t |
| | | INS041 | Vertical Disp. Bott | yd_b | EW_INS041_P40-B2_yd_b | EW_INS041_P40-B3_yd_b | EW_INS041_P40-B4_yd_b | EW_INS041_P42_yd_b | EW_INS041_P45-B1_yd_b | EW_INS041_P45-B2_yd_b | EW_INS041_P45-B3_yd_b | EW_INS041_P45-B4_yd_b | EW_INS041_P47_yd_b | EW_INS041_P50-B1_yd_b | EW_INS041_P52_yd_b |
| | | INS041 | Horiz. Acc. Top | xa_t | EW_INS041_P40-B2_xa_t | EW_INS041_P40-B3_xa_t | EW_INS041_P40-B4_xa_t | EW_INS041_P42_xa_t | EW_INS041_P45-B1_xa_t | EW_INS041_P45-B2_xa_t | EW_INS041_P45-B3_xa_t | EW_INS041_P45-B4_xa_t | EW_INS041_P47_xa_t | EW_INS041_P50-B1_xa_t | EW_INS041_P52_xa_t |
| | | INS041 | Horiz. Acc. Bott | xa_b | EW_INS041_P40-B2_xa_b | EW_INS041_P40-B3_xa_b | EW_INS041_P40-B4_xa_b | EW_INS041_P42_xa_b | EW_INS041_P45-B1_xa_b | EW_INS041_P45-B2_xa_b | EW_INS041_P45-B3_xa_b | EW_INS041_P45-B4_xa_b | EW_INS041_P47_xa_b | EW_INS041_P50-B1_xa_b | EW_INS041_P52_xa_b |
| | | INS041 | Vertical Acc. Top | ya_t | EW_INS041_P40-B2_ya_t | EW_INS041_P40-B3_ya_t | EW_INS041_P40-B4_ya_t | EW_INS041_P42_ya_t | EW_INS041_P45-B1_ya_t | EW_INS041_P45-B2_ya_t | EW_INS041_P45-B3_ya_t | EW_INS041_P45-B4_ya_t | EW_INS041_P47_ya_t | EW_INS041_P50-B1_ya_t | EW_INS041_P52_ya_t |
| | | INS041 | Vertical Acc. Bott | ya_b | EW_INS041_P40-B2_ya_b | EW_INS041_P40-B3_ya_b | EW_INS041_P40-B4_ya_b | EW_INS041_P42_ya_b | EW_INS041_P45-B1_ya_b | EW_INS041_P45-B2_ya_b | EW_INS041_P45-B3_ya_b | EW_INS041_P45-B4_ya_b | EW_INS041_P47_ya_b | EW_INS041_P50-B1_ya_b | EW_INS041_P52_ya_b |
| | NS | INS043 | Horiz. Disp. Top | xd_t | NS_INS043_P40-B2_xd_t | NS_INS043_P40-B3_xd_t | NS_INS043_P40-B4_xd_t | NS_INS043_P42_xd_t | NS_INS043_P45-B1_xd_t | NS_INS043_P45-B2_xd_t | NS_INS043_P45-B3_xd_t | NS_INS043_P45-B4_xd_t | NS_INS043_P47_xd_t | NS_INS043_P50-B1_xd_t | NS_INS043_P52_xd_t |
| | | INS043 | Horiz. Disp. Bott | xd_b | NS_INS043_P40-B2_xd_b | NS_INS043_P40-B3_xd_b | NS_INS043_P40-B4_xd_b | NS_INS043_P42_xd_b | NS_INS043_P45-B1_xd_b | NS_INS043_P45-B2_xd_b | NS_INS043_P45-B3_xd_b | NS_INS043_P45-B4_xd_b | NS_INS043_P47_xd_b | NS_INS043_P50-B1_xd_b | NS_INS043_P52_xd_b |
| | | INS043 | Vertical Disp. Top | yd_t | NS_INS043_P40-B2_yd_t | NS_INS043_P40-B3_yd_t | NS_INS043_P40-B4_yd_t | NS_INS043_P42_yd_t | NS_INS043_P45-B1_yd_t | NS_INS043_P45-B2_yd_t | NS_INS043_P45-B3_yd_t | NS_INS043_P45-B4_yd_t | NS_INS043_P47_yd_t | NS_INS043_P50-B1_yd_t | NS_INS043_P52_yd_t |
| | | INS043 | Vertical Disp. Bott | yd_b | NS_INS043_P40-B2_yd_b | NS_INS043_P40-B3_yd_b | NS_INS043_P40-B4_yd_b | NS_INS043_P42_yd_b | NS_INS043_P45-B1_yd_b | NS_INS043_P45-B2_yd_b | NS_INS043_P45-B3_yd_b | NS_INS043_P45-B4_yd_b | NS_INS043_P47_yd_b | NS_INS043_P50-B1_yd_b | NS_INS043_P52_yd_b |
| | | INS043 | Horiz. Acc. Top | xa_t | NS_INS043_P40-B2_xa_t | NS_INS043_P40-B3_xa_t | NS_INS043_P40-B4_xa_t | NS_INS043_P42_xa_t | NS_INS043_P45-B1_xa_t | NS_INS043_P45-B2_xa_t | NS_INS043_P45-B3_xa_t | NS_INS043_P45-B4_xa_t | NS_INS043_P47_xa_t | NS_INS043_P50-B1_xa_t | NS_INS043_P52_xa_t |
| | | INS043 | Horiz. Acc. Bott | xa_b | NS_INS043_P40-B2_xa_b | NS_INS043_P40-B3_xa_b | NS_INS043_P40-B4_xa_b | NS_INS043_P42_xa_b | NS_INS043_P45-B1_xa_b | NS_INS043_P45-B2_xa_b | NS_INS043_P45-B3_xa_b | NS_INS043_P45-B4_xa_b | NS_INS043_P47_xa_b | NS_INS043_P50-B1_xa_b | NS_INS043_P52_xa_b |
| | | INS043 | Vertical Acc. Top | ya_t | NS_INS043_P40-B2_ya_t | NS_INS043_P40-B3_ya_t | NS_INS043_P40-B4_ya_t | NS_INS043_P42_ya_t | NS_INS043_P45-B1_ya_t | NS_INS043_P45-B2_ya_t | NS_INS043_P45-B3_ya_t | NS_INS043_P45-B4_ya_t | NS_INS043_P47_ya_t | NS_INS043_P50-B1_ya_t | NS_INS043_P52_ya_t |
| | | INS043 | Vertical Acc. Bott | ya_b | NS_INS043_P40-B2_ya_b | NS_INS043_P40-B3_ya_b | NS_INS043_P40-B4_ya_b | NS_INS043_P42_ya_b | NS_INS043_P45-B1_ya_b | NS_INS043_P45-B2_ya_b | NS_INS043_P45-B3_ya_b | NS_INS043_P45-B4_ya_b | NS_INS043_P47_ya_b | NS_INS043_P50-B1_ya_b | NS_INS043_P52_ya_b |

Table L-3: List of output time histories (provided as .log and .jpg files) (continues in the next table)

| | | | | | Time History Filenames (.log text files) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Ground Motion** | | | **Time History Data** | | **P40-B2** | **P40-B3** | **P40-B4** | **P42** | **P45-B1** | **P45-B2** | **P45-B3** | **P45-B4** | **P47** | **P50-B1** | **P52** |
| Distance from the West Slope (m) | | | | | 216 | 171 | 110 | 86 | 90 | 94 | 96 | 100 | 105 | 158 | 196 |
| Distance from the South Slope (m) | | | | | 62 | 62 | 62 | 62 | 107 | 168 | 217 | 278 | 346 | 375 | 397 |
| INTF02 | EW | INTF021 | Horiz. Disp. Top | xd_t | EW_INTF021_P40-B2_xd_t | EW_INTF021_P40-B3_xd_t | EW_INTF021_P40-B4_xd_t | EW_INTF021_P42_xd_t | EW_INTF021_P45-B1_xd_t | EW_INTF021_P45-B2_xd_t | EW_INTF021_P45-B3_xd_t | EW_INTF021_P45-B4_xd_t | EW_INTF021_P47_xd_t | EW_INTF021_P50-B1_xd_t | EW_INTF021_P52_xd_t |
| | | INTF021 | Horiz. Disp. Bott | xd_b | EW_INTF021_P40-B2_xd_b | EW_INTF021_P40-B3_xd_b | EW_INTF021_P40-B4_xd_b | EW_INTF021_P42_xd_b | EW_INTF021_P45-B1_xd_b | EW_INTF021_P45-B2_xd_b | EW_INTF021_P45-B3_xd_b | EW_INTF021_P45-B4_xd_b | EW_INTF021_P47_xd_b | EW_INTF021_P50-B1_xd_b | EW_INTF021_P52_xd_b |
| | | INTF021 | Vertical Disp. Top | yd_t | EW_INTF021_P40-B2_yd_t | EW_INTF021_P40-B3_yd_t | EW_INTF021_P40-B4_yd_t | EW_INTF021_P42_yd_t | EW_INTF021_P45-B1_yd_t | EW_INTF021_P45-B2_yd_t | EW_INTF021_P45-B3_yd_t | EW_INTF021_P45-B4_yd_t | EW_INTF021_P47_yd_t | EW_INTF021_P50-B1_yd_t | EW_INTF021_P52_yd_t |
| | | INTF021 | Vertical Disp. Bott | yd_b | EW_INTF021_P40-B2_yd_b | EW_INTF021_P40-B3_yd_b | EW_INTF021_P40-B4_yd_b | EW_INTF021_P42_yd_b | EW_INTF021_P45-B1_yd_b | EW_INTF021_P45-B2_yd_b | EW_INTF021_P45-B3_yd_b | EW_INTF021_P45-B4_yd_b | EW_INTF021_P47_yd_b | EW_INTF021_P50-B1_yd_b | EW_INTF021_P52_yd_b |
| | | INTF021 | Horiz. Acc. Top | xa_t | EW_INTF021_P40-B2_xa_t | EW_INTF021_P40-B3_xa_t | EW_INTF021_P40-B4_xa_t | EW_INTF021_P42_xa_t | EW_INTF021_P45-B1_xa_t | EW_INTF021_P45-B2_xa_t | EW_INTF021_P45-B3_xa_t | EW_INTF021_P45-B4_xa_t | EW_INTF021_P47_xa_t | EW_INTF021_P50-B1_xa_t | EW_INTF021_P52_xa_t |
| | | INTF021 | Horiz. Acc. Bott | xa_b | EW_INTF021_P40-B2_xa_b | EW_INTF021_P40-B3_xa_b | EW_INTF021_P40-B4_xa_b | EW_INTF021_P42_xa_b | EW_INTF021_P45-B1_xa_b | EW_INTF021_P45-B2_xa_b | EW_INTF021_P45-B3_xa_b | EW_INTF021_P45-B4_xa_b | EW_INTF021_P47_xa_b | EW_INTF021_P50-B1_xa_b | EW_INTF021_P52_xa_b |
| | | INTF021 | Vertical Acc. Top | ya_t | EW_INTF021_P40-B2_ya_t | EW_INTF021_P40-B3_ya_t | EW_INTF021_P40-B4_ya_t | EW_INTF021_P42_ya_t | EW_INTF021_P45-B1_ya_t | EW_INTF021_P45-B2_ya_t | EW_INTF021_P45-B3_ya_t | EW_INTF021_P45-B4_ya_t | EW_INTF021_P47_ya_t | EW_INTF021_P50-B1_ya_t | EW_INTF021_P52_ya_t |
| | | INTF021 | Vertical Acc. Bott | ya_b | EW_INTF021_P40-B2_ya_b | EW_INTF021_P40-B3_ya_b | EW_INTF021_P40-B4_ya_b | EW_INTF021_P42_ya_b | EW_INTF021_P45-B1_ya_b | EW_INTF021_P45-B2_ya_b | EW_INTF021_P45-B3_ya_b | EW_INTF021_P45-B4_ya_b | EW_INTF021_P47_ya_b | EW_INTF021_P50-B1_ya_b | EW_INTF021_P52_ya_b |
| | NS | INTF023 | Horiz. Disp. Top | xd_t | NS_INTF023_P40-B2_xd_t | NS_INTF023_P40-B3_xd_t | NS_INTF023_P40-B4_xd_t | NS_INTF023_P42_xd_t | NS_INTF023_P45-B1_xd_t | NS_INTF023_P45-B2_xd_t | NS_INTF023_P45-B3_xd_t | NS_INTF023_P45-B4_xd_t | NS_INTF023_P47_xd_t | NS_INTF023_P50-B1_xd_t | NS_INTF023_P52_xd_t |
| | | INTF023 | Horiz. Disp. Bott | xd_b | NS_INTF023_P40-B2_xd_b | NS_INTF023_P40-B3_xd_b | NS_INTF023_P40-B4_xd_b | NS_INTF023_P42_xd_b | NS_INTF023_P45-B1_xd_b | NS_INTF023_P45-B2_xd_b | NS_INTF023_P45-B3_xd_b | NS_INTF023_P45-B4_xd_b | NS_INTF023_P47_xd_b | NS_INTF023_P50-B1_xd_b | NS_INTF023_P52_xd_b |
| | | INTF023 | Vertical Disp. Top | yd_t | NS_INTF023_P40-B2_yd_t | NS_INTF023_P40-B3_yd_t | NS_INTF023_P40-B4_yd_t | NS_INTF023_P42_yd_t | NS_INTF023_P45-B1_yd_t | NS_INTF023_P45-B2_yd_t | NS_INTF023_P45-B3_yd_t | NS_INTF023_P45-B4_yd_t | NS_INTF023_P47_yd_t | NS_INTF023_P50-B1_yd_t | NS_INTF023_P52_yd_t |
| | | INTF023 | Vertical Disp. Bott | yd_b | NS_INTF023_P40-B2_yd_b | NS_INTF023_P40-B3_yd_b | NS_INTF023_P40-B4_yd_b | NS_INTF023_P42_yd_b | NS_INTF023_P45-B1_yd_b | NS_INTF023_P45-B2_yd_b | NS_INTF023_P45-B3_yd_b | NS_INTF023_P45-B4_yd_b | NS_INTF023_P47_yd_b | NS_INTF023_P50-B1_yd_b | NS_INTF023_P52_yd_b |
| | | INTF023 | Horiz. Acc. Top | xa_t | NS_INTF023_P40-B2_xa_t | NS_INTF023_P40-B3_xa_t | NS_INTF023_P40-B4_xa_t | NS_INTF023_P42_xa_t | NS_INTF023_P45-B1_xa_t | NS_INTF023_P45-B2_xa_t | NS_INTF023_P45-B3_xa_t | NS_INTF023_P45-B4_xa_t | NS_INTF023_P47_xa_t | NS_INTF023_P50-B1_xa_t | NS_INTF023_P52_xa_t |
| | | INTF023 | Horiz. Acc. Bott | xa_b | NS_INTF023_P40-B2_xa_b | NS_INTF023_P40-B3_xa_b | NS_INTF023_P40-B4_xa_b | NS_INTF023_P42_xa_b | NS_INTF023_P45-B1_xa_b | NS_INTF023_P45-B2_xa_b | NS_INTF023_P45-B3_xa_b | NS_INTF023_P45-B4_xa_b | NS_INTF023_P47_xa_b | NS_INTF023_P50-B1_xa_b | NS_INTF023_P52_xa_b |
| | | INTF023 | Vertical Acc. Top | ya_t | NS_INTF023_P40-B2_ya_t | NS_INTF023_P40-B3_ya_t | NS_INTF023_P40-B4_ya_t | NS_INTF023_P42_ya_t | NS_INTF023_P45-B1_ya_t | NS_INTF023_P45-B2_ya_t | NS_INTF023_P45-B3_ya_t | NS_INTF023_P45-B4_ya_t | NS_INTF023_P47_ya_t | NS_INTF023_P50-B1_ya_t | NS_INTF023_P52_ya_t |
| | | INTF023 | Vertical Acc. Bott | ya_b | NS_INTF023_P40-B2_ya_b | NS_INTF023_P40-B3_ya_b | NS_INTF023_P40-B4_ya_b | NS_INTF023_P42_ya_b | NS_INTF023_P45-B1_ya_b | NS_INTF023_P45-B2_ya_b | NS_INTF023_P45-B3_ya_b | NS_INTF023_P45-B4_ya_b | NS_INTF023_P47_ya_b | NS_INTF023_P50-B1_ya_b | NS_INTF023_P52_ya_b |
| INTF03 | EW | INTF031 | Horiz. Disp. Top | xd_t | EW_INTF031_P40-B2_xd_t | EW_INTF031_P40-B3_xd_t | EW_INTF031_P40-B4_xd_t | EW_INTF031_P42_xd_t | EW_INTF031_P45-B1_xd_t | EW_INTF031_P45-B2_xd_t | EW_INTF031_P45-B3_xd_t | EW_INTF031_P45-B4_xd_t | EW_INTF031_P47_xd_t | EW_INTF031_P50-B1_xd_t | EW_INTF031_P52_xd_t |
| | | INTF031 | Horiz. Disp. Bott | xd_b | EW_INTF031_P40-B2_xd_b | EW_INTF031_P40-B3_xd_b | EW_INTF031_P40-B4_xd_b | EW_INTF031_P42_xd_b | EW_INTF031_P45-B1_xd_b | EW_INTF031_P45-B2_xd_b | EW_INTF031_P45-B3_xd_b | EW_INTF031_P45-B4_xd_b | EW_INTF031_P47_xd_b | EW_INTF031_P50-B1_xd_b | EW_INTF031_P52_xd_b |
| | | INTF031 | Vertical Disp. Top | yd_t | EW_INTF031_P40-B2_yd_t | EW_INTF031_P40-B3_yd_t | EW_INTF031_P40-B4_yd_t | EW_INTF031_P42_yd_t | EW_INTF031_P45-B1_yd_t | EW_INTF031_P45-B2_yd_t | EW_INTF031_P45-B3_yd_t | EW_INTF031_P45-B4_yd_t | EW_INTF031_P47_yd_t | EW_INTF031_P50-B1_yd_t | EW_INTF031_P52_yd_t |
| | | INTF031 | Vertical Disp. Bott | yd_b | EW_INTF031_P40-B2_yd_b | EW_INTF031_P40-B3_yd_b | EW_INTF031_P40-B4_yd_b | EW_INTF031_P42_yd_b | EW_INTF031_P45-B1_yd_b | EW_INTF031_P45-B2_yd_b | EW_INTF031_P45-B3_yd_b | EW_INTF031_P45-B4_yd_b | EW_INTF031_P47_yd_b | EW_INTF031_P50-B1_yd_b | EW_INTF031_P52_yd_b |
| | | INTF031 | Horiz. Acc. Top | xa_t | EW_INTF031_P40-B2_xa_t | EW_INTF031_P40-B3_xa_t | EW_INTF031_P40-B4_xa_t | EW_INTF031_P42_xa_t | EW_INTF031_P45-B1_xa_t | EW_INTF031_P45-B2_xa_t | EW_INTF031_P45-B3_xa_t | EW_INTF031_P45-B4_xa_t | EW_INTF031_P47_xa_t | EW_INTF031_P50-B1_xa_t | EW_INTF031_P52_xa_t |
| | | INTF031 | Horiz. Acc. Bott | xa_b | EW_INTF031_P40-B2_xa_b | EW_INTF031_P40-B3_xa_b | EW_INTF031_P40-B4_xa_b | EW_INTF031_P42_xa_b | EW_INTF031_P45-B1_xa_b | EW_INTF031_P45-B2_xa_b | EW_INTF031_P45-B3_xa_b | EW_INTF031_P45-B4_xa_b | EW_INTF031_P47_xa_b | EW_INTF031_P50-B1_xa_b | EW_INTF031_P52_xa_b |
| | | INTF031 | Vertical Acc. Top | ya_t | EW_INTF031_P40-B2_ya_t | EW_INTF031_P40-B3_ya_t | EW_INTF031_P40-B4_ya_t | EW_INTF031_P42_ya_t | EW_INTF031_P45-B1_ya_t | EW_INTF031_P45-B2_ya_t | EW_INTF031_P45-B3_ya_t | EW_INTF031_P45-B4_ya_t | EW_INTF031_P47_ya_t | EW_INTF031_P50-B1_ya_t | EW_INTF031_P52_ya_t |
| | | INTF031 | Vertical Acc. Bott | ya_b | EW_INTF031_P40-B2_ya_b | EW_INTF031_P40-B3_ya_b | EW_INTF031_P40-B4_ya_b | EW_INTF031_P42_ya_b | EW_INTF031_P45-B1_ya_b | EW_INTF031_P45-B2_ya_b | EW_INTF031_P45-B3_ya_b | EW_INTF031_P45-B4_ya_b | EW_INTF031_P47_ya_b | EW_INTF031_P50-B1_ya_b | EW_INTF031_P52_ya_b |
| | NS | INTF033 | Horiz. Disp. Top | xd_t | NS_INTF033_P40-B2_xd_t | NS_INTF033_P40-B3_xd_t | NS_INTF033_P40-B4_xd_t | NS_INTF033_P42_xd_t | NS_INTF033_P45-B1_xd_t | NS_INTF033_P45-B2_xd_t | NS_INTF033_P45-B3_xd_t | NS_INTF033_P45-B4_xd_t | NS_INTF033_P47_xd_t | NS_INTF033_P50-B1_xd_t | NS_INTF033_P52_xd_t |
| | | INTF033 | Horiz. Disp. Bott | xd_b | NS_INTF033_P40-B2_xd_b | NS_INTF033_P40-B3_xd_b | NS_INTF033_P40-B4_xd_b | NS_INTF033_P42_xd_b | NS_INTF033_P45-B1_xd_b | NS_INTF033_P45-B2_xd_b | NS_INTF033_P45-B3_xd_b | NS_INTF033_P45-B4_xd_b | NS_INTF033_P47_xd_b | NS_INTF033_P50-B1_xd_b | NS_INTF033_P52_xd_b |
| | | INTF033 | Vertical Disp. Top | yd_t | NS_INTF033_P40-B2_yd_t | NS_INTF033_P40-B3_yd_t | NS_INTF033_P40-B4_yd_t | NS_INTF033_P42_yd_t | NS_INTF033_P45-B1_yd_t | NS_INTF033_P45-B2_yd_t | NS_INTF033_P45-B3_yd_t | NS_INTF033_P45-B4_yd_t | NS_INTF033_P47_yd_t | NS_INTF033_P50-B1_yd_t | NS_INTF033_P52_yd_t |
| | | INTF033 | Vertical Disp. Bott | yd_b | NS_INTF033_P40-B2_yd_b | NS_INTF033_P40-B3_yd_b | NS_INTF033_P40-B4_yd_b | NS_INTF033_P42_yd_b | NS_INTF033_P45-B1_yd_b | NS_INTF033_P45-B2_yd_b | NS_INTF033_P45-B3_yd_b | NS_INTF033_P45-B4_yd_b | NS_INTF033_P47_yd_b | NS_INTF033_P50-B1_yd_b | NS_INTF033_P52_yd_b |
| | | INTF033 | Horiz. Acc. Top | xa_t | NS_INTF033_P40-B2_xa_t | NS_INTF033_P40-B3_xa_t | NS_INTF033_P40-B4_xa_t | NS_INTF033_P42_xa_t | NS_INTF033_P45-B1_xa_t | NS_INTF033_P45-B2_xa_t | NS_INTF033_P45-B3_xa_t | NS_INTF033_P45-B4_xa_t | NS_INTF033_P47_xa_t | NS_INTF033_P50-B1_xa_t | NS_INTF033_P52_xa_t |
| | | INTF033 | Horiz. Acc. Bott | xa_b | NS_INTF033_P40-B2_xa_b | NS_INTF033_P40-B3_xa_b | NS_INTF033_P40-B4_xa_b | NS_INTF033_P42_xa_b | NS_INTF033_P45-B1_xa_b | NS_INTF033_P45-B2_xa_b | NS_INTF033_P45-B3_xa_b | NS_INTF033_P45-B4_xa_b | NS_INTF033_P47_xa_b | NS_INTF033_P50-B1_xa_b | NS_INTF033_P52_xa_b |
| | | INTF033 | Vertical Acc. Top | ya_t | NS_INTF033_P40-B2_ya_t | NS_INTF033_P40-B3_ya_t | NS_INTF033_P40-B4_ya_t | NS_INTF033_P42_ya_t | NS_INTF033_P45-B1_ya_t | NS_INTF033_P45-B2_ya_t | NS_INTF033_P45-B3_ya_t | NS_INTF033_P45-B4_ya_t | NS_INTF033_P47_ya_t | NS_INTF033_P50-B1_ya_t | NS_INTF033_P52_ya_t |
| | | INTF033 | Vertical Acc. Bott | ya_b | NS_INTF033_P40-B2_ya_b | NS_INTF033_P40-B3_ya_b | NS_INTF033_P40-B4_ya_b | NS_INTF033_P42_ya_b | NS_INTF033_P45-B1_ya_b | NS_INTF033_P45-B2_ya_b | NS_INTF033_P45-B3_ya_b | NS_INTF033_P45-B4_ya_b | NS_INTF033_P47_ya_b | NS_INTF033_P50-B1_ya_b | NS_INTF033_P52_ya_b |
| INTF04 | EW | INTF041 | Horiz. Disp. Top | xd_t | EW_INTF041_P40-B2_xd_t | EW_INTF041_P40-B3_xd_t | EW_INTF041_P40-B4_xd_t | EW_INTF041_P42_xd_t | EW_INTF041_P45-B1_xd_t | EW_INTF041_P45-B2_xd_t | EW_INTF041_P45-B3_xd_t | EW_INTF041_P45-B4_xd_t | EW_INTF041_P47_xd_t | EW_INTF041_P50-B1_xd_t | EW_INTF041_P52_xd_t |
| | | INTF041 | Horiz. Disp. Bott | xd_b | EW_INTF041_P40-B2_xd_b | EW_INTF041_P40-B3_xd_b | EW_INTF041_P40-B4_xd_b | EW_INTF041_P42_xd_b | EW_INTF041_P45-B1_xd_b | EW_INTF041_P45-B2_xd_b | EW_INTF041_P45-B3_xd_b | EW_INTF041_P45-B4_xd_b | EW_INTF041_P47_xd_b | EW_INTF041_P50-B1_xd_b | EW_INTF041_P52_xd_b |
| | | INTF041 | Vertical Disp. Top | yd_t | EW_INTF041_P40-B2_yd_t | EW_INTF041_P40-B3_yd_t | EW_INTF041_P40-B4_yd_t | EW_INTF041_P42_yd_t | EW_INTF041_P45-B1_yd_t | EW_INTF041_P45-B2_yd_t | EW_INTF041_P45-B3_yd_t | EW_INTF041_P45-B4_yd_t | EW_INTF041_P47_yd_t | EW_INTF041_P50-B1_yd_t | EW_INTF041_P52_yd_t |
| | | INTF041 | Vertical Disp. Bott | yd_b | EW_INTF041_P40-B2_yd_b | EW_INTF041_P40-B3_yd_b | EW_INTF041_P40-B4_yd_b | EW_INTF041_P42_yd_b | EW_INTF041_P45-B1_yd_b | EW_INTF041_P45-B2_yd_b | EW_INTF041_P45-B3_yd_b | EW_INTF041_P45-B4_yd_b | EW_INTF041_P47_yd_b | EW_INTF041_P50-B1_yd_b | EW_INTF041_P52_yd_b |
| | | INTF041 | Horiz. Acc. Top | xa_t | EW_INTF041_P40-B2_xa_t | EW_INTF041_P40-B3_xa_t | EW_INTF041_P40-B4_xa_t | EW_INTF041_P42_xa_t | EW_INTF041_P45-B1_xa_t | EW_INTF041_P45-B2_xa_t | EW_INTF041_P45-B3_xa_t | EW_INTF041_P45-B4_xa_t | EW_INTF041_P47_xa_t | EW_INTF041_P50-B1_xa_t | EW_INTF041_P52_xa_t |
| | | INTF041 | Horiz. Acc. Bott | xa_b | EW_INTF041_P40-B2_xa_b | EW_INTF041_P40-B3_xa_b | EW_INTF041_P40-B4_xa_b | EW_INTF041_P42_xa_b | EW_INTF041_P45-B1_xa_b | EW_INTF041_P45-B2_xa_b | EW_INTF041_P45-B3_xa_b | EW_INTF041_P45-B4_xa_b | EW_INTF041_P47_xa_b | EW_INTF041_P50-B1_xa_b | EW_INTF041_P52_xa_b |
| | | INTF041 | Vertical Acc. Top | ya_t | EW_INTF041_P40-B2_ya_t | EW_INTF041_P40-B3_ya_t | EW_INTF041_P40-B4_ya_t | EW_INTF041_P42_ya_t | EW_INTF041_P45-B1_ya_t | EW_INTF041_P45-B2_ya_t | EW_INTF041_P45-B3_ya_t | EW_INTF041_P45-B4_ya_t | EW_INTF041_P47_ya_t | EW_INTF041_P50-B1_ya_t | EW_INTF041_P52_ya_t |
| | | INTF041 | Vertical Acc. Bott | ya_b | EW_INTF041_P40-B2_ya_b | EW_INTF041_P40-B3_ya_b | EW_INTF041_P40-B4_ya_b | EW_INTF041_P42_ya_b | EW_INTF041_P45-B1_ya_b | EW_INTF041_P45-B2_ya_b | EW_INTF041_P45-B3_ya_b | EW_INTF041_P45-B4_ya_b | EW_INTF041_P47_ya_b | EW_INTF041_P50-B1_ya_b | EW_INTF041_P52_ya_b |
| | NS | INTF043 | Horiz. Disp. Top | xd_t | NS_INTF043_P40-B2_xd_t | NS_INTF043_P40-B3_xd_t | NS_INTF043_P40-B4_xd_t | NS_INTF043_P42_xd_t | NS_INTF043_P45-B1_xd_t | NS_INTF043_P45-B2_xd_t | NS_INTF043_P45-B3_xd_t | NS_INTF043_P45-B4_xd_t | NS_INTF043_P47_xd_t | NS_INTF043_P50-B1_xd_t | NS_INTF043_P52_xd_t |
| | | INTF043 | Horiz. Disp. Bott | xd_b | NS_INTF043_P40-B2_xd_b | NS_INTF043_P40-B3_xd_b | NS_INTF043_P40-B4_xd_b | NS_INTF043_P42_xd_b | NS_INTF043_P45-B1_xd_b | NS_INTF043_P45-B2_xd_b | NS_INTF043_P45-B3_xd_b | NS_INTF043_P45-B4_xd_b | NS_INTF043_P47_xd_b | NS_INTF043_P50-B1_xd_b | NS_INTF043_P52_xd_b |
| | | INTF043 | Vertical Disp. Top | yd_t | NS_INTF043_P40-B2_yd_t | NS_INTF043_P40-B3_yd_t | NS_INTF043_P40-B4_yd_t | NS_INTF043_P42_yd_t | NS_INTF043_P45-B1_yd_t | NS_INTF043_P45-B2_yd_t | NS_INTF043_P45-B3_yd_t | NS_INTF043_P45-B4_yd_t | NS_INTF043_P47_yd_t | NS_INTF043_P50-B1_yd_t | NS_INTF043_P52_yd_t |
| | | INTF043 | Vertical Disp. Bott | yd_b | NS_INTF043_P40-B2_yd_b | NS_INTF043_P40-B3_yd_b | NS_INTF043_P40-B4_yd_b | NS_INTF043_P42_yd_b | NS_INTF043_P45-B1_yd_b | NS_INTF043_P45-B2_yd_b | NS_INTF043_P45-B3_yd_b | NS_INTF043_P45-B4_yd_b | NS_INTF043_P47_yd_b | NS_INTF043_P50-B1_yd_b | NS_INTF043_P52_yd_b |
| | | INTF043 | Horiz. Acc. Top | xa_t | NS_INTF043_P40-B2_xa_t | NS_INTF043_P40-B3_xa_t | NS_INTF043_P40-B4_xa_t | NS_INTF043_P42_xa_t | NS_INTF043_P45-B1_xa_t | NS_INTF043_P45-B2_xa_t | NS_INTF043_P45-B3_xa_t | NS_INTF043_P45-B4_xa_t | NS_INTF043_P47_xa_t | NS_INTF043_P50-B1_xa_t | NS_INTF043_P52_xa_t |
| | | INTF043 | Horiz. Acc. Bott | xa_b | NS_INTF043_P40-B2_xa_b | NS_INTF043_P40-B3_xa_b | NS_INTF043_P40-B4_xa_b | NS_INTF043_P42_xa_b | NS_INTF043_P45-B1_xa_b | NS_INTF043_P45-B2_xa_b | NS_INTF043_P45-B3_xa_b | NS_INTF043_P45-B4_xa_b | NS_INTF043_P47_xa_b | NS_INTF043_P50-B1_xa_b | NS_INTF043_P52_xa_b |
| | | INTF043 | Vertical Acc. Top | ya_t | NS_INTF043_P40-B2_ya_t | NS_INTF043_P40-B3_ya_t | NS_INTF043_P40-B4_ya_t | NS_INTF043_P42_ya_t | NS_INTF043_P45-B1_ya_t | NS_INTF043_P45-B2_ya_t | NS_INTF043_P45-B3_ya_t | NS_INTF043_P45-B4_ya_t | NS_INTF043_P47_ya_t | NS_INTF043_P50-B1_ya_t | NS_INTF043_P52_ya_t |
| | | INTF043 | Vertical Acc. Bott | ya_b | NS_INTF043_P40-B2_ya_b | NS_INTF043_P40-B3_ya_b | NS_INTF043_P40-B4_ya_b | NS_INTF043_P42_ya_b | NS_INTF043_P45-B1_ya_b | NS_INTF043_P45-B2_ya_b | NS_INTF043_P45-B3_ya_b | NS_INTF043_P45-B4_ya_b | NS_INTF043_P47_ya_b | NS_INTF043_P50-B1_ya_b | NS_INTF043_P52_ya_b |

THA-Inboud.xlsx

An example of provided time histories

for
Structure P40-B2
&
Ground Motion CRU03-2475

Table L-4- List of time history files provided for P40-B2, Ground motion CRU03-2475

**Acronyms**

| | |
|---|---|
| Horiz. Disp. Top | xd_t |
| Horiz. Disp. Bott | xd_b |
| Vertical Disp. Top | yd_t |
| Vertical Disp. Bott | yd_b |
| Horiz. Acc. Top | xa_t |
| Horiz. Acc. Bott | xa_b |
| Vertical Acc. Top | ya_t |
| Vertical Acc. Bott | ya_b |

Note:
Time histories at the bottom are "within motion".

| | | |
|---|---|---|
| EW-CRU031_P40-B2_xa_b.jpg | JPG File | 39 KB |
| EW-CRU031_P40-B2_xa_t.jpg | JPG File | 40 KB |
| EW-CRU031_P40-B2_xd_b.jpg | JPG File | 38 KB |
| EW-CRU031_P40-B2_xd_t.jpg | JPG File | 35 KB |
| EW-CRU031_P40-B2_ya_b.jpg | JPG File | 37 KB |
| EW-CRU031_P40-B2_ya_t.jpg | JPG File | 37 KB |
| EW-CRU031_P40-B2_yd_b.jpg | JPG File | 36 KB |
| EW-CRU031_P40-B2_yd_t.jpg | JPG File | 36 KB |
| NS-CRU033_P40-B2_xa_b.jpg | JPG File | 39 KB |
| NS-CRU033_P40-B2_xa_t.jpg | JPG File | 40 KB |
| NS-CRU033_P40-B2_xd_b.jpg | JPG File | 37 KB |
| NS-CRU033_P40-B2_xd_t.jpg | JPG File | 35 KB |
| NS-CRU033_P40-B2_ya_b.jpg | JPG File | 37 KB |
| NS-CRU033_P40-B2_ya_t.jpg | JPG File | 37 KB |
| NS-CRU033_P40-B2_yd_b.jpg | JPG File | 38 KB |
| NS-CRU033_P40-B2_yd_t.jpg | JPG File | 35 KB |
| EW-CRU031_P40-B2_xa_b.log | Text Document | 539 KB |
| EW-CRU031_P40-B2_xa_t.log | Text Document | 539 KB |
| EW-CRU031_P40-B2_xd_b.log | Text Document | 539 KB |
| EW-CRU031_P40-B2_xd_t.log | Text Document | 539 KB |
| EW-CRU031_P40-B2_ya_b.log | Text Document | 539 KB |
| EW-CRU031_P40-B2_ya_t.log | Text Document | 539 KB |
| EW-CRU031_P40-B2_yd_b.log | Text Document | 539 KB |
| EW-CRU031_P40-B2_yd_t.log | Text Document | 539 KB |
| NS-CRU033_P40-B2_xa_b.log | Text Document | 533 KB |
| NS-CRU033_P40-B2_xa_t.log | Text Document | 533 KB |
| NS-CRU033_P40-B2_xd_b.log | Text Document | 533 KB |
| NS-CRU033_P40-B2_xd_t.log | Text Document | 533 KB |
| NS-CRU033_P40-B2_ya_b.log | Text Document | 533 KB |
| NS-CRU033_P40-B2_ya_t.log | Text Document | 533 KB |
| NS-CRU033_P40-B2_yd_b.log | Text Document | 533 KB |
| NS-CRU033_P40-B2_yd_t.log | Text Document | 533 KB |

Table L-5- An example of a time history text file for Bent P40-B2, ground motion CRU03-2475 in E-W direction
(Filename: EW-CRU031_P40-B2_xd_t.log)

**Time (s)** | **Horizontal Displacement (m)**

```
History *** vs   2
-----------------------------------
   Dynamic time                 X displacement( 214,  47)
     ----------------------      ----------------------
            4.6246635E-004            -6.4350098E-009
            1.0174260E-002            -6.8756044E-008
            1.9886053E-002            -1.3233339E-007
            2.9597847E-002            -1.8564737E-007
            3.9309640E-002            -2.4063297E-007
            4.9021434E-002            -2.8898248E-007
            5.8733227E-002            -3.4222581E-007
            6.8445021E-002            -3.9182198E-007
            7.8156815E-002            -4.4823379E-007
            8.7868609E-002            -5.0112804E-007
            9.7580403E-002            -5.6067146E-007
            1.0729220E-001            -6.1568471E-007
            1.1700399E-001            -6.7696061E-007
            1.2671578E-001            -7.3278340E-007
            1.3642757E-001            -7.9003584E-007
            1.4613936E-001            -8.2735669E-007
            1.5585115E-001            -8.3836351E-007
            1.6556294E-001            -8.2772202E-007
            1.7527472E-001            -8.2286174E-007
            1.8498651E-001            -8.3554744E-007
            1.9469830E-001            -8.5011812E-007
            2.0441009E-001            -8.4989645E-007
            2.1412188E-001            -8.3502656E-007
            2.2383367E-001            -8.0882361E-007
            2.3354546E-001            -7.6469138E-007
            2.4325726E-001            -7.1332101E-007
            2.5296905E-001            -6.4755735E-007
            2.6268084E-001            -5.7975203E-007
            2.7239263E-001            -4.9565976E-007
            2.8210443E-001            -4.0810430E-007
            2.9181622E-001            -2.9673474E-007
            3.0152801E-001            -1.8552322E-007
            3.1123980E-001            -6.4414110E-008
            3.2095160E-001             3.7027281E-008
            3.3066339E-001             1.4641617E-007
            3.4037518E-001             2.6050708E-007
            3.5008698E-001             4.1438479E-007
            3.5979877E-001             5.7686644E-007
            3.6951056E-001             7.6895774E-007
            3.7922235E-001             9.4540100E-007
            3.8893414E-001             1.1263035E-006
            3.9864592E-001             1.2634514E-006
            4.0835769E-001             1.3808131E-006
            4.1806945E-001             1.4795074E-006
```

Figure L-4- Location of Conveyor Bent P40-B2 and location of provided time histories

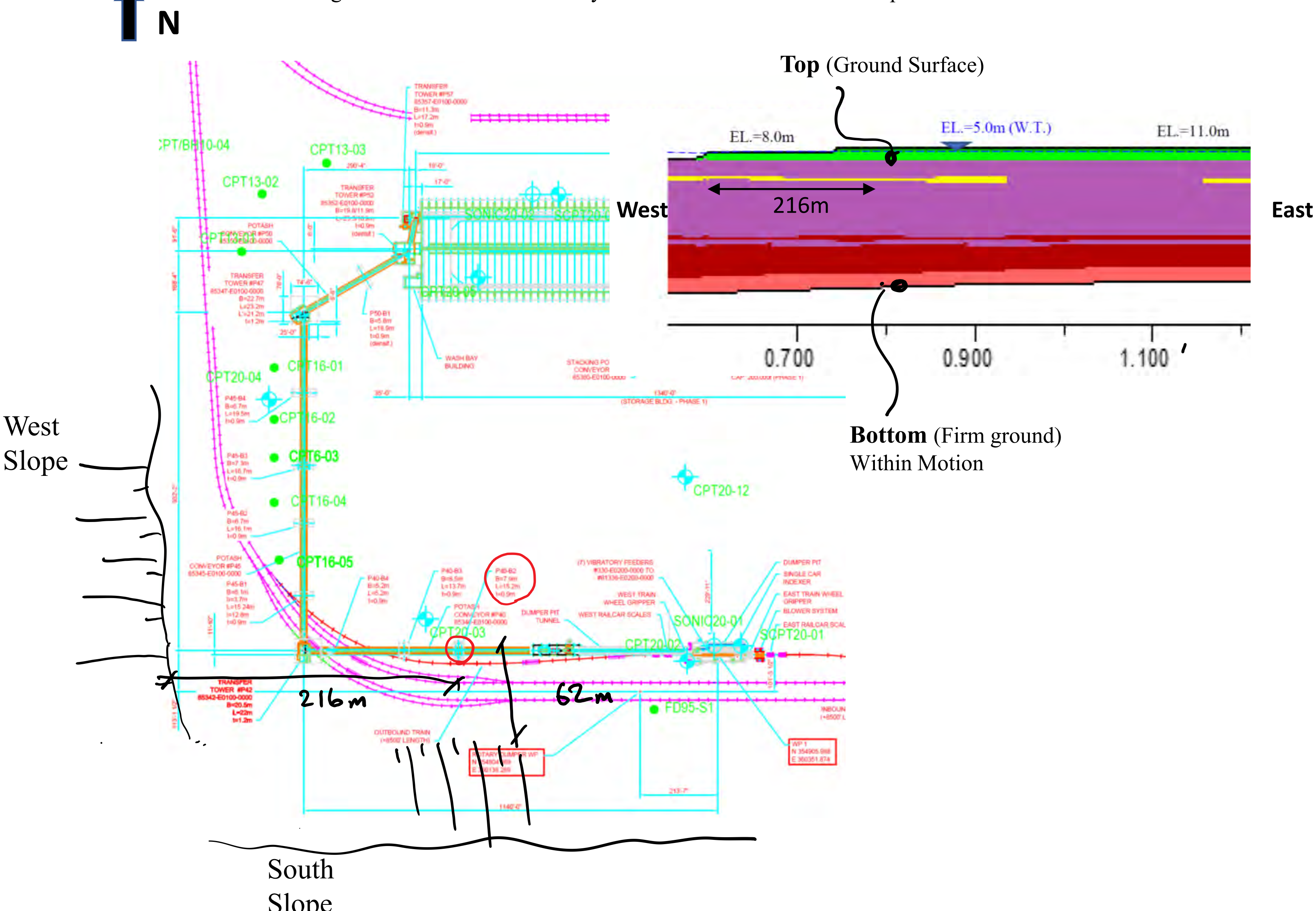
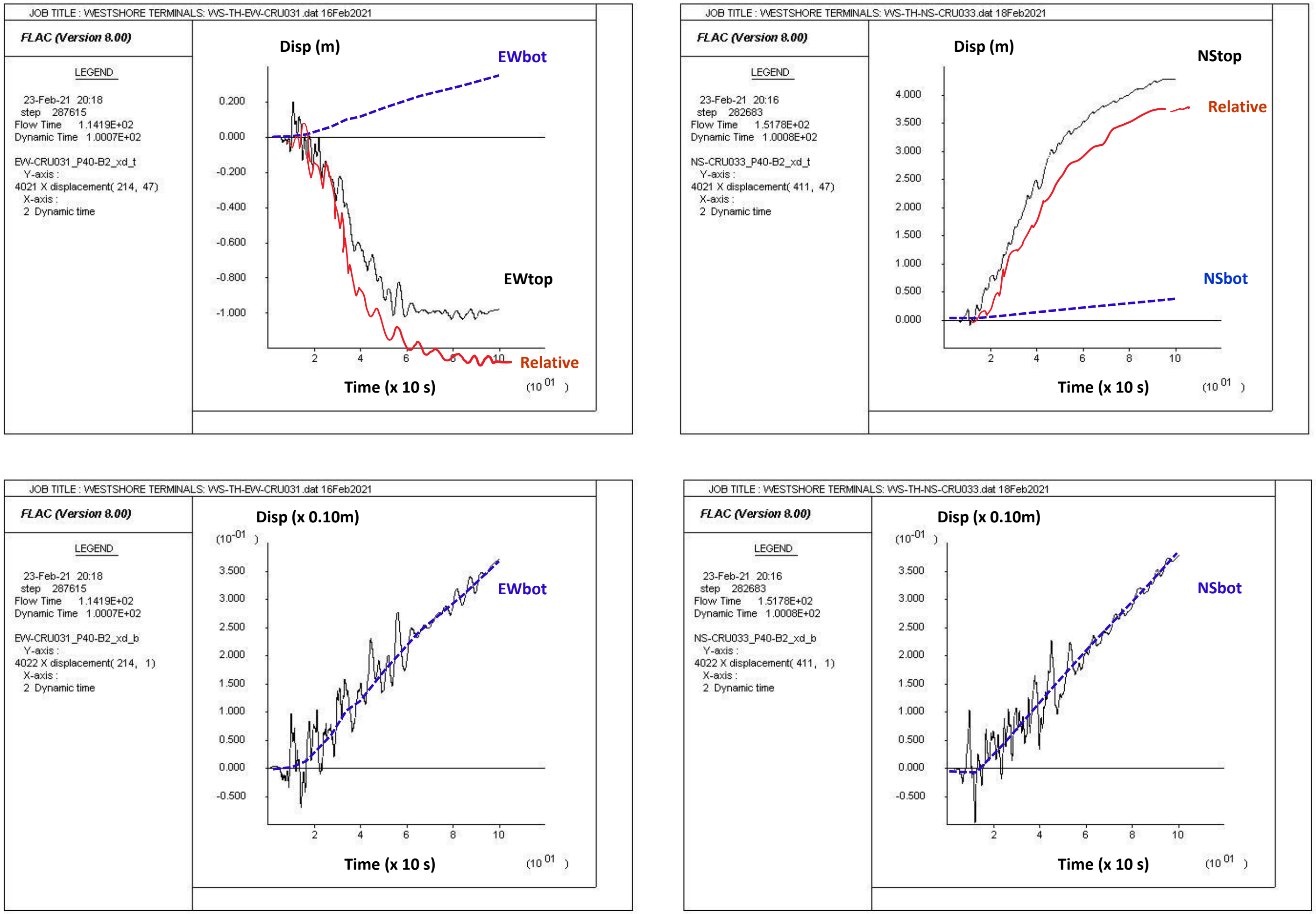


Figure L-5- Time histories of Horizontal Displacements

**E-W** **N-S**

**Top**
(Ground Surface)

**Bottom**
(Firm Ground)
(With-in Motion)




- EW-CRU031_P40-B2_xa_b.log
- EW-CRU031_P40-B2_xa_t.log
- EW-CRU031_P40-B2_xd_b.log
- EW-CRU031_P40-B2_xd_t.log
- EW-CRU031_P40-B2_ya_b.log
- EW-CRU031_P40-B2_ya_t.log
- EW-CRU031_P40-B2_yd_b.log
- EW-CRU031_P40-B2_yd_t.log
- NS-CRU033_P40-B2_xa_b.log
- NS-CRU033_P40-B2_xa_t.log
- NS-CRU033_P40-B2_xd_b.log
- NS-CRU033_P40-B2_xd_t.log
- NS-CRU033_P40-B2_ya_b.log
- NS-CRU033_P40-B2_ya_t.log
- NS-CRU033_P40-B2_yd_b.log
- NS-CRU033_P40-B2_yd_t.log

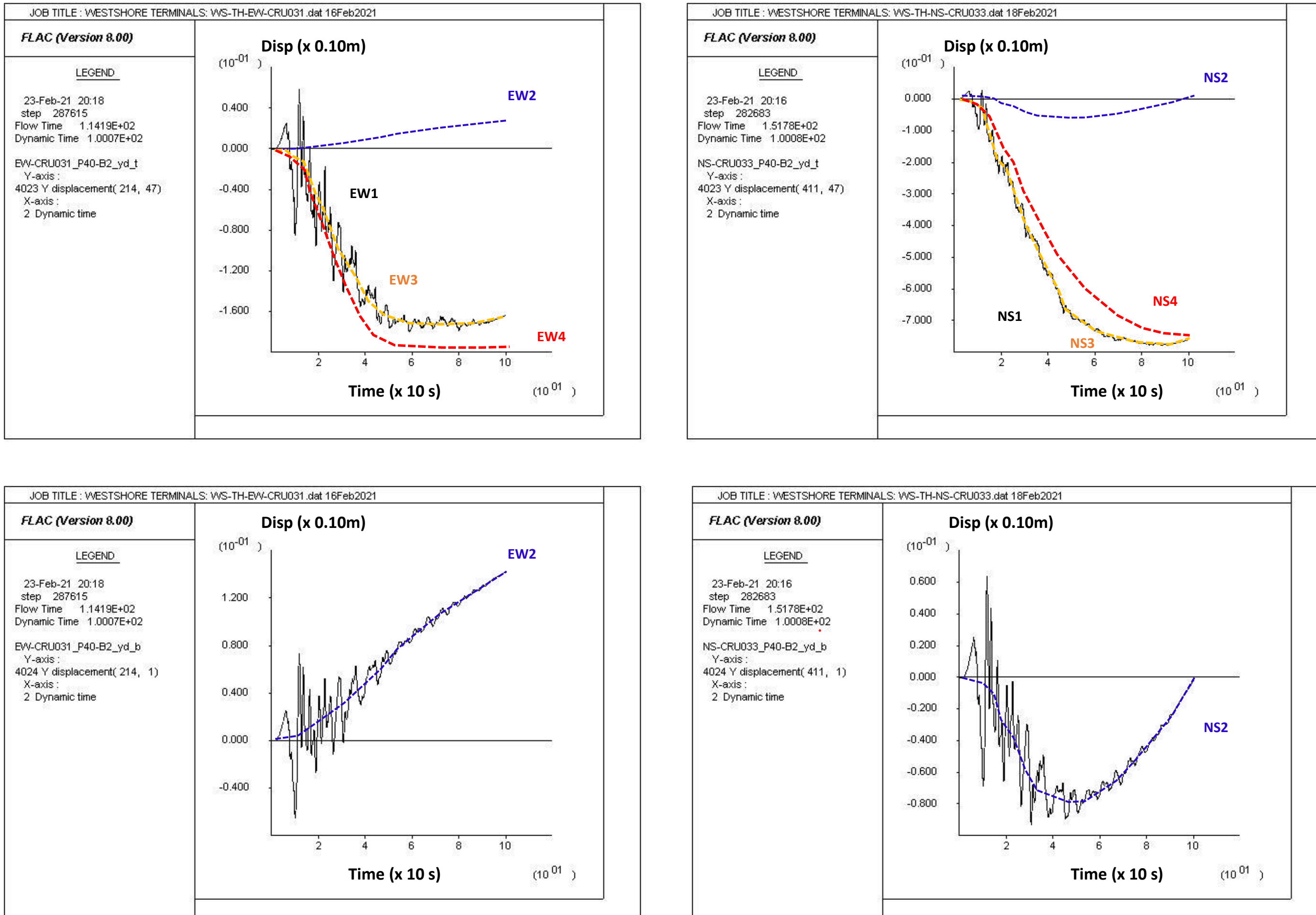
EWtop: Horiz. Displace. time history @ ground surface E-W (before correction)
EWbot: Horiz. Best fit on Displace. time history @ bottom
**Relative EW time history @ ground surface = EWtop – EWbot**
**Relative NS time history @ ground surface = NStop – NSbot**

Figure L-6- Time histories of Vertical Displacements

**E-W** **N-S**

**Top**
(Ground Surface)

**Bottom**
**(**Firm Ground)
(With-in Motion)




EW-CRU031_P40-B2_xa_b.log
EW-CRU031_P40-B2_xa_t.log
EW-CRU031_P40-B2_xd_b.log
EW-CRU031_P40-B2_xd_t.log
EW-CRU031_P40-B2_ya_b.log
EW-CRU031_P40-B2_ya_t.log
EW-CRU031_P40-B2_yd_b.log
EW-CRU031_P40-B2_yd_t.log
NS-CRU033_P40-B2_xa_b.log
NS-CRU033_P40-B2_xa_t.log
NS-CRU033_P40-B2_xd_b.log
NS-CRU033_P40-B2_xd_t.log
NS-CRU033_P40-B2_ya_b.log
NS-CRU033_P40-B2_ya_t.log
NS-CRU033_P40-B2_yd_b.log
NS-CRU033_P40-B2_yd_t.log

EW1: Ver. Disp. time history from the E-W FLAC @ G.S. before correction
EW2: Best fit curve to Ver. Disp. time history from the E-W FLAC @ Bott
EW3: Best fit curve to Ver. Disp. Time history @ G.S.
EW4: EW3 – EW2

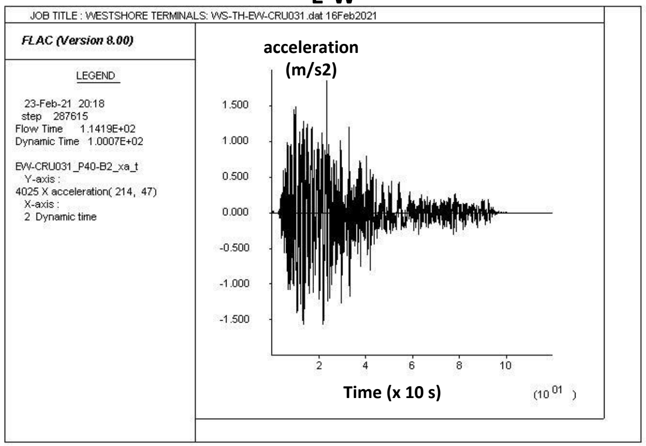
NS1: Ver. Disp. time history from the N-S FLAC @ G.S. before correction
NS2: Best fit curve to Ver. Disp. time history from the N-S FLAC @ Bott
NS3: Best fit curve to Ver. Disp. Time history @ G.S.
NS4: NS3 – NS2

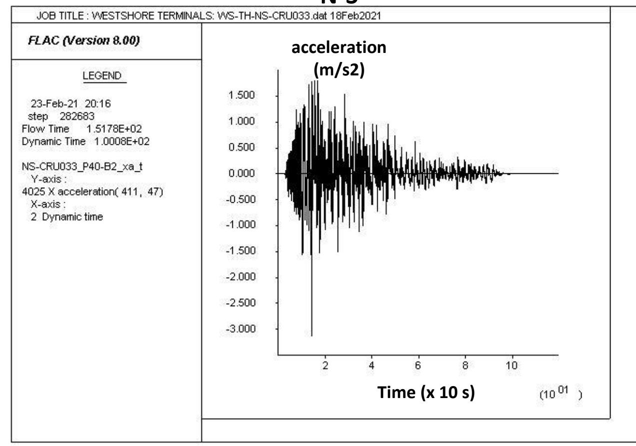
**Corrected vertical disp. Time history for analysis = NS1 - NS2 + EW4**

Figure L-7- Time histories of Horizontal Accelerations

**E-W** | **N-S**

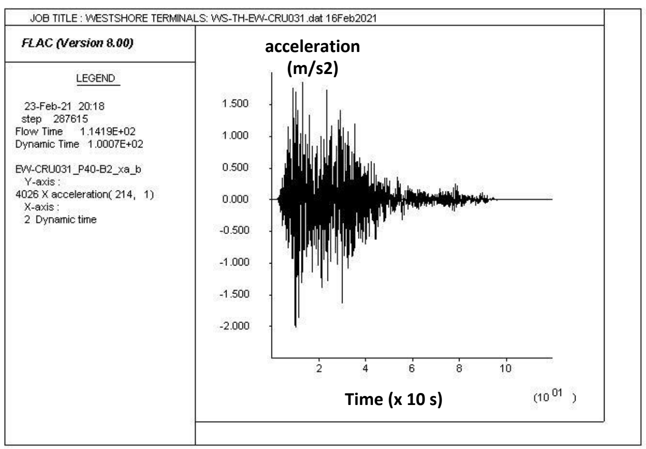
**Top**
(Ground Surface)

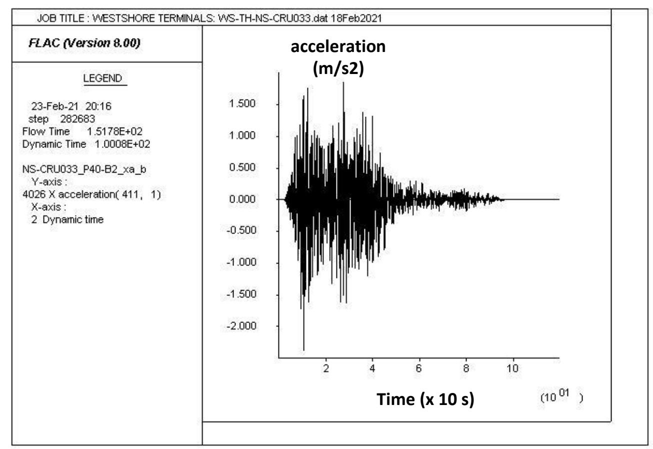


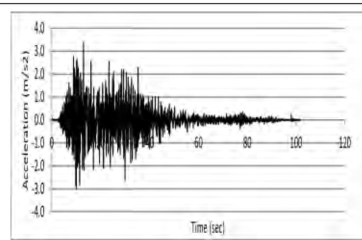

- EW-CRU031_P40-B2_xa_b.log
- EW-CRU031_P40-B2_xa_t.log
- EW-CRU031_P40-B2_xd_b.log
- EW-CRU031_P40-B2_xd_t.log
- EW-CRU031_P40-B2_ya_b.log
- EW-CRU031_P40-B2_ya_t.log
- EW-CRU031_P40-B2_yd_b.log
- EW-CRU031_P40-B2_yd_t.log
- NS-CRU033_P40-B2_xa_b.log
- NS-CRU033_P40-B2_xa_t.log
- NS-CRU033_P40-B2_xd_b.log
- NS-CRU033_P40-B2_xd_t.log
- NS-CRU033_P40-B2_ya_b.log
- NS-CRU033_P40-B2_ya_t.log
- NS-CRU033_P40-B2_yd_b.log
- NS-CRU033_P40-B2_yd_t.log

**Bottom**
**(Firm Ground)**
(With-in
Motion)




**(Outcrop Ground motion)**
**To be multiplied by 1.09**



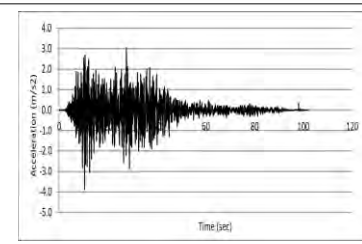
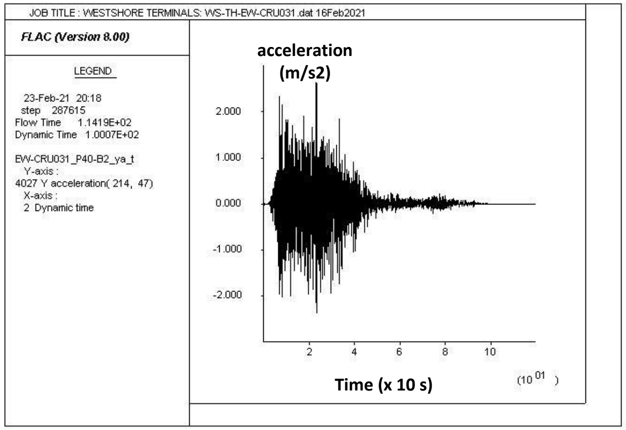
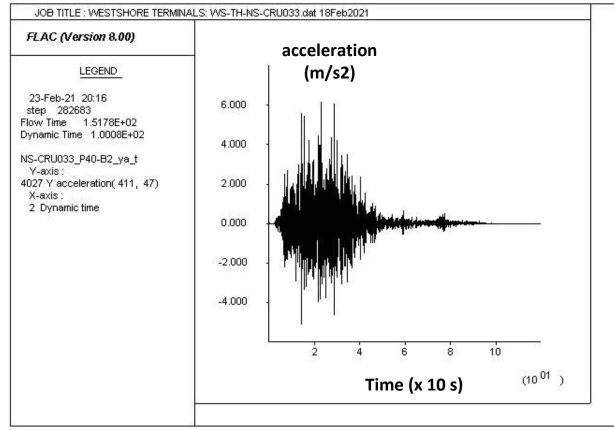



Figure L-8- Time histories of Vertical Accelerations

**E-W** **N-S**

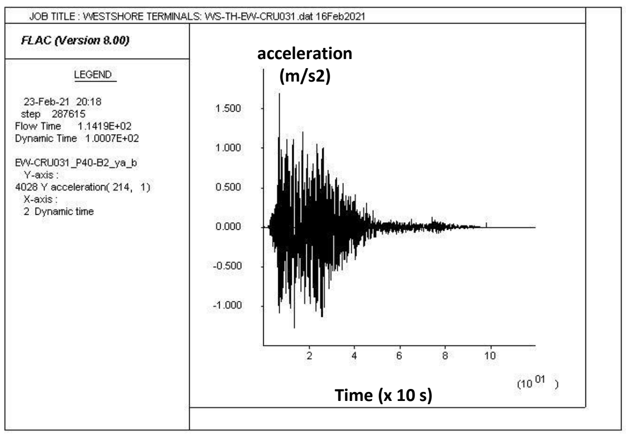
**Top**
(Ground Surface)

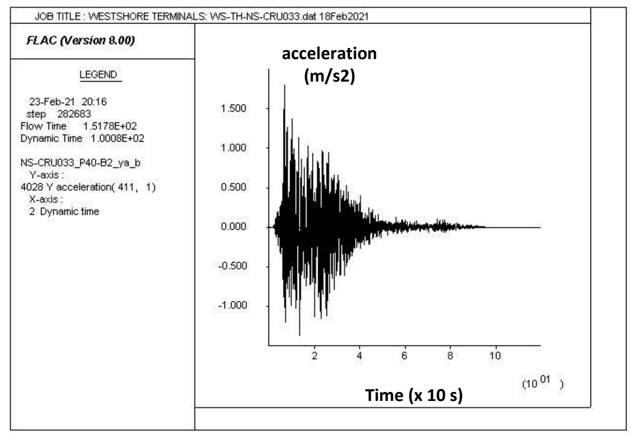



EW-CRU031_P40-B2_xa_b.log
EW-CRU031_P40-B2_xa_t.log
EW-CRU031_P40-B2_xd_b.log
EW-CRU031_P40-B2_xd_t.log
EW-CRU031_P40-B2_ya_b.log
EW-CRU031_P40-B2_ya_t.log
EW-CRU031_P40-B2_yd_b.log
EW-CRU031_P40-B2_yd_t.log
NS-CRU033_P40-B2_xa_b.log
NS-CRU033_P40-B2_xa_t.log
NS-CRU033_P40-B2_xd_b.log
NS-CRU033_P40-B2_xd_t.log
NS-CRU033_P40-B2_ya_b.log
NS-CRU033_P40-B2_ya_t.log
NS-CRU033_P40-B2_yd_b.log
NS-CRU033_P40-B2_yd_t.log

**Bottom**
**(**Firm Ground)
(With-in
Motion)




**(Outcrop Ground motion)**
**To be multiplied by 1.09**