THURBER ENGINEERING LTD.

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Attention: Ken Savage, P.Eng.

DP WORLD FRASER SURREY INC.
CANOLA OIL TRANSLOAD FACILITY, FRASER SURREY TERMINAL
GEOTECHNICAL RECOMMENDATIONS REPORT

Dear Ken:

Thurber Engineering Ltd. (Thurber) is submitting this letter report that summarizes the findings of our desktop study and geotechnical site investigation, and provides geotechnical recommendations for the proposed Canola Oil Transload facility at the Fraser Surrey Terminal in Surrey, B.C.

This report had been prepared exclusively for the use of Sacré-Davey Engineering Inc. (SDE) and DP World Fraser Surrey Inc. (DP World). Any use that a third party makes of this report, or any reliance on decisions based on it are the responsibility of such third parties. Thurber accepts no responsibility for damages incurred by third parties as a result of decisions made or actions taken based on this report. It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

1. INTRODUCTION

The Fraser Surrey Terminal is a multi-purpose marine port that is located along the banks of the Fraser River at 11060 Elevator Road in Surrey, B.C. The terminal site is generally flat and is largely an open paved area with the exception of some light warehouse structures as shown on the Dwg. 34098-1 (attached).

DP World is contemplating the feasibility of developing a canola oil transshipment facility from rail to ship with intermediate storage at the Fraser Surrey Terminal. DP World retained SDE as the prime consultant and Thurber as the geotechnical consultant to provide engineering design services. We understand that detailed design and construction of the facility will be completed through a design-build project delivery method.

The proposed Canola Oil Transload facility, as shown on Dwg. 34098-2, will include:

- A tank farm area to be located in the existing container parking area, south of the rail lines. According to SDE Dwg. 7704-GA-001 Rev. 5 (see Appendix A), the tank farm will initially comprise three, 37 m wide circular tanks with a 15,000 metric ton holding capacity and a 2.4 m high containment wall. The drawing indicates a potential for a future expansion that will include two additional tanks of similar size and holding capacity, and an additional extension to the east of the tank farm area that will comprise three, 15 m

wide circular tanks with 2,000 metric ton holding capacity and a similar containment wall extension. The proposed layout of the tank farm area encroaches on an existing spur line to the south.
- A berth facility that will comprise a load arm deck and trestle, access catwalk, bollard and pile cap.
- Buried load and recycles lines that will connect the tank farm to the berth, and will traverse under the spur lines and across Yard 10.

Relevant SDE drawings are included in Appendix A of the report.

In support of the design for the proposed facility, Thurber completed a desktop review of existing geotechnical information and carried out a geotechnical investigation limited to the tank farm area and the alignment of the load/recycle lines. This report summarizes the desktop review of available onshore and offshore geotechnical information, the findings of the geotechnical investigation and provides geotechnical design input in general accordance with the requirements outlined in the request for quotation (RFQ) package.

Assessment of soil and groundwater contamination is not included in our scope of work.

2. EXISTING GEOTECHNICAL INFORMATION

Several geotechnical investigations were completed at the Fraser Surrey Terminal and at the adjacent sites in support of various projects. Copies of four reports were provided by SDE for our use. Below are brief descriptions of the previous investigations that include the parties tasked with completing the work and an outline of the relevant geotechnical investigation completed.

- WSP Canada Group Limited (WSP) was retained by Vancouver Fraser Port Authority (VFPA) to provide engineering services for the new roadway alignment at the Fraser Surrey Port Land Transportation Improvements project. The investigation comprised sixteen auger test holes that were completed south of the tank farm area and along south Timberland Road. The test holes were advanced to depths ranging between 6.1 m and 9.1 m. The findings of the investigation are summarized in WSP's "*Geotechnical Design Report (30% Design) – Fraser Surrey Ports Land Transportation Improvement – Greater Vancouver Gateway Program*" report (No. 20-0173), dated December 19, 2020.
- EXP Services Inc. (EXP) was retained by BC Pacific Enterprises Ltd. (BCP) to provide engineering services for the proposed Bulk Material Facilities at 10610 Timberland Road in Surrey, B.C. The investigation comprised ten auger test holes and nine cone penetration test (CPT) soundings that were completed along the perimeter of Yard 10, along south Timberland Road, and east of the Westran Intermodal building. The test holes were advanced to depths ranging between 6.1 m and 9.1 m, and the CPT soundings were advanced to 20 m depth. The findings of the investigation are summarized in EXP's "*Proposed Bulk Material Facilities – 10610 Timberland Road, Surrey, BC"* geotechnical assessment report (No. VAN-00236988-A0), dated February 14, 2017.
- MEG Consulting Ltd. (MEG), now Tetra Tech, was retained by Ausenco to carry out an onshore and offshore geotechnical investigation for a proposed outbound potash export

facility at the Fraser Surrey Docks. The investigation comprised four offshore CPT soundings to depths ranging between 25 m and 40 m, four onshore CPT soundings to depths ranging between 30 m and 35 m, two onshore seismic cone penetration test (SCPT) soundings to 40 m and 60 m depths, and two onshore mud rotary test holes to about 25 m depth. The onshore investigation CPT soundings were completed at the Berth 9 location which includes the location of the proposed Berth for the Canola Oil Transload facility. The onshore portion of the investigation was completed west of Yard 10. The findings of the investigation are summarized in MEG's "*FSD Potash Export Terminal – Geotechnical Engineering Assessment for Conceptual Design of Berth Structure and Storage Building*" report (No. 16-011-05), dated June 6, 2017.

- Tetra Tech Canada Inc. (Tetra Tech) was retained by Hatch Bantrel Joint Venture (HBJV) to carry out a geotechnical and hydrological investigation for the North Surrey Interceptor (NSI) sewer alignment and Berth 9 upgrade project at the Fraser Surrey Terminal. The investigation comprised three CPT soundings to depths ranging between 33 m and 63 m, twenty-five auger test holes to depths ranging between 6.1 m and 9.1 m, and dynamic cone penetration test (DCPT) profiling to 6.1 m depth at sixteen locations. Shallow standpipe piezometers were installed at 10 test hole locations. The investigation was completed along the proposed alignment of the NSI sewer which included the yard area west of Yard 10, along Robson Road, and at and around the tank farm area. The findings of the investigation are summarized in Tetra Tech's "*SC6212 – Geotechnical and Hydrological Survey*" Geotechnical Factual Report (No. 704-ENG.VGEO03979-01_01 REV. A), dated March 12, 2021.

3. GEOLOGIC CONDITIONS

The Fraser Surrey Terminal is located along the south bank of the Fraser River, at the onset of Annieville Channel. According to Surficial Geology Map 1484A (New Westminster), the surficial geology is expected to consist of a sequence of:

- SAb – Quaternary postglacial swamp deposits comprising lowland peat.
- Fb/Fc – Quaternary postglacial Fraser River sediments comprising overbank sandy to silt loam deposits up to 2 m thick, overlying deltaic and distributary channel fill deposits (Fd).
- Fa – Quaternary postglacial Fraser River Sediments comprising fine to medium sand with minor silt content.
- Fd - Deltaic and distributary channel fill deposits consisting mainly of interbedded fine to medium sand, silt and silty clay that measure between 10 m and 40 m in thickness and potentially including organic material.
- Ce – Pleistocene Capilano sediments comprising silt loam to clay loam, up to 60 m in thickness.
- PVf – Pre-Vashon glacial deposits (till, glaciofluvial, glaciomarine and glaciolacustrine).

4. GEOTECHNICAL INVESTIGATION

Given the available geotechnical information which included CPT soundings located within the proposed layout of the offshore structures, our geotechnical investigation was limited to the tank farm area and along the alignment of the load and recycle lines.

Prior to the investigation, Thurber submitted BC One Call tickets (#20221402580 and #20221402622), Vancouver Fraser Port Authority (VFPA) category A project permit application (#22-043), and a request to MetroVancouver to locate underground facilities (#220536). A private utility locator was retained by Thurber on April 1, 2022, to confirm the locations of the various underground and above grade utilities in the vicinity of the test hole and SCPT locations. The test hole and SCPT locations were also surveyed during utility locating.

The geotechnical investigation was completed on April 6 and 7, 2022, and included the completion of five auger test holes, two in the tank farm area and three along the alignment of the load and recycle lines in Yard 10, and three SCPT soundings, two in the tank farm area and one along the alignment of the load and recycle lines.

The auger test holes were advanced using an MPP Geotek 60, track mounted auger drill rig operated by Earth Drilling Co. Ltd. (Earth Drilling). The SCPT soundings were advanced using a standard 10 cm^2 electronic CPT cone, operated by Earth Drilling, that was mounted on the drill rig using a custom ramset. The SCPT is a direct push method that obtains information on tip resistance, sleeve friction, pore water pressure and shear wave velocity.

Table 1 summarizes the recently completed test holes and SCPT soundings. The locations of the test holes and SCPT soundings are shown in Figure 1. The test hole logs are attached in Appendix A and include the soil descriptions, backfill details, and laboratory soil test results. Appendix B includes the cone penetration test report that includes individual SCPT sounding logs, dissipation test results and in-situ shear wave measurement results.

Table 1 - Summary of Completed Test Holes and CPT Soundings

Location	**Test Hole / SCPT ID**	**Northing**	**Easting**	**El. (m)**	**Termination depth (m)**	**Comments**
Tank Farm Area	SCPT22-1	5448494.49	506993.97	4.1	30.0	-
	TH22-1				7.6	-
	SCPT22-2	5448431.33	506948.38	4.0	19.2	Refusal due to rod flex
	TH22-2				7.6	-
Load and Recycle Lines (Yard 10)	SCPT22-3	5448607.02	506710.96	4.2	27.9	Refusal due to rod flex
	TH22-3				10.7	-
	TH22-4	5448680.55	506616.57	3.7	7.6	-
	TH22-5	5448530.59	506828.82	3.7	7.6	-

The soil and groundwater conditions were logged in the field by a Thurber engineer. Disturbed soil samples were collected off the auger flights, typically at 1.5 m intervals, and transported to Thurber's Vancouver laboratory for routine moisture content determination and visual classification testing. Atterberg limits and washed sieves were completed on selected soil samples.

Shear wave velocities (V_s) were measured typically at 1 m intervals in all SCPT soundings. Pore-pressure dissipation (PPD) tests were also completed in the SCPT soundings at select locations to estimate the consolidation characteristics of cohesive soil units and assist in the evaluation of the stabilized groundwater levels.

The completed test holes and SCPT soundings were backfilled with quartz sand and bentonite chips as required in the VFPA project permit and were subsequently patched using cold mix asphalt. The drill cuttings from the test holes were collected in drums and disposed of off-site.

5. SUBSURFACE CONDITIONS

The results of the site investigation and laboratory testing are summarized on the attached test hole logs and CPT logs. The soil descriptions on the logs should be used in preference to the generalized soil profile descriptions given below.

Where the investigation did not include recovery of soil samples, the SCPT allows for the classification of the soil stratigraphy using various established methods that utilize the measured cone parameters (tip resistance, sleeve friction and pore pressure response) to identify the soil types and anticipated behaviour. The normalized Soil Behaviour Type (SBTn) method, which uses SCPT parameters that are normalized to the effective stress, is considered a more reliable method particularly for deeper SCPT soundings as it accounts for the overburden pressure effects (Robertson, 2010).

According to the SCPT soundings and test holes completed during the recent and previous geotechnical investigations, the subsurface stratigraphy generally comprises surface asphalt that is underlain by about 300 mm of granular fill (sand with varying amounts of gravel and silt) encountered in some of the test holes, over a sequence of:

- Sand fill (inferred).
- Peat (encountered only in test holes in the tank farm area and south end of Yard 10).
- Cohesive soils of varying composition.
- Sand with trace silt.
- Interbedded silty sand, sandy silt, silty clay and sand with trace fines.

The subsurface stratigraphy at depth, based on our desktop review of deeper cone penetration test (CPT) soundings, comprises silty clay deposits overlying dense to very dense till, glacio-fluvial and glaciomarine deposits.

Sand Fill

The sand fill was encountered below the pavement structure to 6.5 m depth (~ El. -2.5 m) in the tank farm area, and to 2.8 m and 4 m depth (El. +0.9 m to -0.2 m) at Yard 10. The layer includes variable silt and gravel content, and traces of organics. It is characterized as compact to loose based on the SCPT tip resistance.

Peat

The peat was encountered below the sand fill in TH22-1 and TH22-2 in the tank farm area, and in TH22-5 at the south end of Yard 10. The peat layer extended to 4.2 m depth (El. -0.5 m) in TH 22-5 at the south end of Yard 10 and is inferred to extend to about 9.5 m depth (El. -5.5 m) in the tank farm area based on the SCPT tip resistance and sleeve friction measurements. Previous test hole information also indicates that the thickness of the peat deposit generally increases further away from the shoreline, towards the South Fraser Perimeter Road (Highway 17).

Silt

Silt, varying from silt with a trace of clay to clayey silt to silt with a trace of sand to sandy, with trace organics and wood debris to sandy silt with trace clay and organics, was encountered below the sand fill in TH22-3 and TH22-4 and below the peat in TH22-1, TH22-2 and TH22-5. The silt is characterized as firm to stiff based on CPT tip resistance. The overconsolidation ratio (i.e., ratio of equivalent maximum past pressure to in situ pressure) of this unit was up to about 4 near the top reducing to 2 at depth.

In TH22-1 and TH22-2 in the tank farm area, the silt was homogenous with minor sand lenses and extended to about 16.5 m depth (El. -12.5 m) based on the SCPT soundings. At Yard 10, the silt is inferred to extend to 17 m depth (~ El. -13 m) based on SCPT22-3, TH22-3, TH22-4 and TH22-5, and includes interbedded silt and sand layers up to 2 m thick.

Sand with trace silt

Compact, fine to medium sand with trace silt and gravel and organics, and interbedded silty sand and silty clay lenses was encountered below the silt layer. Based on the CPT soundings completed by MEG, these Fraser River sediments extend to about El. -32 m.

Interbedded silty sand, sandy silt, silty clay, and sand

Based on test holes, CPTs and SCPTs completed by others, Fraser River sediments comprising interbedded silty sand, sandy silt, silty clay and sand are expected to be present below the sand. These deposits are characterized as loose to compact/very stiff to hard, based on CPT tip resistance and are expected to extend to nominally El. -50 m.

Silty clay

Below the sand and interbedded silty sand to silty clay and sand comprising the Fraser River Sediments, a marine silty clay deposit with silty sand laminations is inferred. This deposit is characterized as low to high sensitivity, low to medium plastic and is expected to be normally to lightly over-consolidated. Deep CPT soundings by others in the general vicinity indicate that the marine silty clay deposit extends to nominally El. -85 m and is underlain by dense to very dense, till, glacio-fluvial and glaciomarine deposits.

6. DESIGN CONSIDERATIONS

VFPA requires the design for all buildings and structures to be completed in accordance with the National Building Code of Canada (NBCC). With the recent release of the 2020 NBCC, the recommendations provided in this report address the new code requirements. The 2020 NBCC code requires the structures to be designed for a seismic event with 2% in 50 years probability (2475 Year Return Period).

Further, VFPA requires that offshore structures be designed in accordance with ASCE/COPRI 61-14 “Seismic Design of Piers and Wharves” standard. According to the standard, structures must be designed according to one of the following three performance levels under seismic loading:

- Life Safety Protection performance level where the structure is expected to continue to support gravity loads (i.e., collapse-prevention), the sustained damage does not prevent human egress, and there is no loss of containment of materials in a manner that would pose a public hazard.
- Controlled and Repairable Damage performance level where the structure responds primarily in a controlled and ductile manner, loss of serviceability is limited to several months to complete required repairs, and there is no loss of containment of materials in a manner that would pose a public hazard.
- Minimal Damage performance level where the structure exhibits near-elastic structural response with minor or no residual deformation, there is no loss of serviceability of the structure, and there is no loss of containment of materials in a manner that would pose a public hazard.

The selection of the appropriate performance level depends on the magnitude of the earthquake event and the design classification assigned for a given structure, which varies between high, moderate and low. A high design classification is assigned to structures that are deemed essential to the region’s economy or post-event recovery which require a level of seismic performance beyond life safety protection. A moderate design classification is assigned to structures of secondary importance to the regional economy and not essential to post-event recovery, but which require a level of seismic performance beyond life safety protection. Finally, a low design classification, is assigned to structures that do not fall under the “high” or “moderate” classification.

For all three design classifications, the performance level under a design earthquake event, which we infer to correspond to the 1:2,475 year earthquake event, is life safety protection. Where a high design classification is assigned, the performance levels are controlled and repairable damage under a contingency level earthquake corresponding to a 1:475 year earthquake event, and minimal damage under an operating level earthquake corresponding to a 1:72 year earthquake event. Where a moderate design classification is assigned, the performance level under a contingency level earthquake, which corresponds to a 1:225 year earthquake event, is controlled and repairable damage. A low design classification does not require any analysis for a contingency or operating level earthquakes.

6.1 Limit States Design and Geotechnical Resistance Factors

Limit States Design is the recommended design methodology in the National Building Code of Canada. According to the Canadian Foundation Engineering Manual (CFEM, 2006), limit states are defined as conditions under which a structure or its components no longer perform their intended function. The limit state design methodology includes addressing the ultimate limit state (ULS), which is concerned with collapse mechanisms of the structure (i.e., safety), and serviceability limit state (SLS), which represent conditions of mechanics that restrict or constrain the intended use or function of the structure under anticipated working loads.

In ULS design, the geotechnical resistance is multiplied by a geotechnical resistance factor that is less than unity, to account for variabilities in geotechnical parameters and analysis uncertainties when evaluating the geotechnical resistance, and the structural loads are multiplied by load factors that are generally greater than unity to account for uncertainties in loads and their probability of occurrence (CFEM, 2006). Table 2 summarizes the recommended geotechnical resistance factors in CFEM (2006) for shallow and deep foundations.

Table 2 - Geotechnical Resistance Values for Shallow and Deep Foundations based on CFEM (2006)

Application	Limit State	Test method/model	Resistance Factors
Shallow Foundations	Bearing	Analysis	0.5
	Base Sliding	Analysis	0.8 (frictional base foundation) 0.6 (cohesive base foundation)
	Passive Resistance	Analysis	0.5
Deep Foundations	Compression	Analysis	0.4
		Dynamic Testing	0.5
		Static Testing	0.6
	Tension	Analysis	0.3
		Static Uplift Testing	0.4
	Lateral	Analysis	0.5

As shown in Table 2, a higher geotechnical resistance factor can be used where field testing is completed to reduce the uncertainty that is inherent in static analysis. Depending on the type of field testing, the geotechnical resistance factors for deep foundations can be increased by up to 0.2 (i.e., 0.6 in lieu of 0.4 where full-scale static loading testing is completed).

For seismic (pre-liquefaction) loading conditions, NBCC 2015 indicates that the factored geotechnical resistance of various foundation systems can be increased to capture the overstrength of the soil. Part 4 of the NBCC, including the Notes to Part 4, and Commentary J of the Structural Commentaries to the NBCC document should be consulted for detailed discussion and guidance on the required structural analysis.

As CFEM (2006) does not provide geotechnical resistance factors explicitly for retaining systems, Table 3 outlines the geotechnical resistance factors recommended in the 2019 Canadian Highway Bridge Design Code (CHBDC 2019) for retaining walls, assuming a typical degree of understanding.

Table 3 - Geotechnical Resistance Values for Retaining Systems based on CHBDC (2019)

Application	Limit State	Test method/model	Resistance Factors[1]
Retaining Systems	Bearing	Analysis	0.6
	Overturning	Analysis	0.55
	Base Sliding	Analysis	0.9
	Settlement	Analysis	0.9
	Deflection/tilt	Analysis	0.9

Notes:

1) The geotechnical resistance factors are based on a typical degree of understanding.

7. SEISMIC ASSESSMENT

Fraser River sediments, and potentially the sand fill, are known to be susceptible to liquefaction under seismic loading. Soils that are prone to liquefaction are assigned a X_F seismic site designation that requires the completion of a site-specific response analysis (SSRA) to establish the peak ground acceleration (PGA) and cyclic stress ratio profile for liquefaction triggering assessment, and a design response spectrum. However, where the fundamental period of vibration of structures is equal to or less than 0.5 s, the PGA and spectral accelerations can be determined using the 2020 NBCC Seismic Hazard Tool by assuming that the soils are non-liquefiable.

Table 4 below provides the key values for the design response spectra determined using the 2020 NBCC Seismic Hazard Tool. An average shear wave velocity over the top 30 m of soil column ($V_{s,30}$) of 165 m/s was used, which was evaluated based on the in-situ shear wave velocity measurements.

Table 4 - Spectral Accelerations for Site Class X_F for 5% Damping and 2% / 50 Years Probability (2475 Year Return Period)

Sa (0.2)	Sa (0.5)	Sa (1.0)	Sa (2.0)	Sa (5.0)	Sa (10.0)	PGA (0.05)	PGV (0.05)
(g)							(m/s)
1.16	1.16	0.871	0.529	0.161	0.0593	0.506	0.777

As mentioned earlier, the spectral accelerations provided in Table 4 must only be used where the fundamental period of a structure does not exceed 0.5 s. Otherwise, the design response spectra developed using the SSRA must be used.

7.1 Site-Specific Response Analysis

A one-dimensional (1D) SSRA was carried out for the 1:2,475 year earthquake event using the software program DEEPSOIL published by the University of Illinois. A total of 30 motion records, 10 records for each of the three earthquake sources (crustal, intraslab and interface [i.e., Cascadia subduction event]) were used in the analysis. These ground motion records were spectrally matched to the George Massey Tunnel site, designated as the reference site for the Lower Mainland area, and were subsequently linearly scaled based on the seismic hazard for the site. The SSRA analysis was completed using the equivalent-linear model and the Darendeli (2001) reference curves for both the sand and clay layers.

The average response spectra for 5% structural damping for each of the three earthquake sources are shown for the 1:2,475 year earthquake event in Figure C-1 of Appendix C, along with our recommended design envelope. The design response spectra corresponding to $V_{s,30}$ of 165 m/s is also included for reference. As the SSRA produced markedly lower spectral accelerations than the code-based approach, the design envelope was capped at 80% of the code-based response spectra as recommended in NBCC.

7.2 Liquefaction Triggering Assessment

Liquefaction triggering assessments are commonly carried out using a stress-based approach, where the earthquake-induced cyclic shear stresses, defined as the Cyclic Stress Ratio (CSR), are compared against the cyclic resistance of the soil, defined as the Cyclic Resistance Ratio (CRR). In general, if the CSR is greater than the CRR, then the soil is considered to be susceptible to liquefaction.

CRR is typically evaluated using semi-empirical relationships that correlate the in-situ CRR of sand and the results of in-situ tests (Idriss and Boulanger, 2008). The earthquake-induced CSR is typically evaluated using the Seed-Idriss simplified procedure. However, this procedure is only recommended for depths not exceeding 20 m. For relatively thick deposits, it is recommended that the CSR be evaluated using a SSRA (Idriss and Boulanger, 2008). Considering the subsurface stratigraphy at the Site, the CSR profiles for liquefaction triggering assessment were evaluated based on the results of the SSRA.

The liquefaction assessment was carried out using the software program CLiq (v.2.2.1.4), published by Geologismiki. The assessment followed the methods described by Idriss and Boulanger (2008 and 2014) to evaluate the CRR profiles. The CSR profiles were evaluated using the maximum stress ratio profiles developed from the SSRAs that were multiplied by a 0.65 reference stress level factor (Idriss and Boulanger, 2008). The CRR and CSR profiles were evaluated using a moment magnitude of 7.1 for the crustal and intraslab earthquake sources and 9 for the interface earthquake source.

The assessment indicates that the surficial fills and Fraser River sediments, particularly the sand with trace silt layer, will experience liquefaction in 1:2,475 year earthquake event. The estimated depth of liquefaction in the Fraser River sediments is estimated to extend to El. -47 m or 34 m depth below mudline El. -13 m at the proposed location of the berth facility. Further, the assessment indicates the potential for limited zones of liquefaction (<150 mm in thickness) to occur below this depth.

Hazards involved with liquefiable soils include loss of side shear and end bearing resistance for pile foundations in the liquefiable zone, vertical deformation, and potential for lateral flow slides that will impact near and offshore structures. The impact of potential lateral flow slides on the tank farm will be limited considering relative distance to the shoreline, which is about 400 m.

8. GEOTECHNICAL AND SEISMIC CONSIDERATIONS

The subsurface stratigraphy at the tank farm and Yard 10 is anticipated to comprise a sequence of compact to loose sand fill, peat in the tank farm area that decreases in thickness towards Yard 10, nominally overconsolidated, firm to stiff silt of varying composition, Fraser River sediments (compact sand over loose to compact/very stiff to hard interbedded silty sand, sandy silt, clay and sand), normally to lightly over-consolidated marine silty clays over dense to very dense till, glacio-fluvial and glaciomarine deposits.

The subsurface stratigraphy at the proposed offshore structure locations is anticipated to comprise a sequence of Holocene Fraser deltaic deposits to about El. -50 m, over marine silty clays underlain by Pleistocene (glacial till) deposits.

The geologic conditions at the site are considered challenging with near-surface and deep liquefaction susceptible sand layers, and compressible peat, and shallow and deep silts layers.

Provided in the following sections are axial and lateral pile analyses for static and kinematic seismic loading conditions for the offshore structures, and foundation input for the tank farm, ancillary structures and secondary containment.

9. OFFSHORE STRUCTURES

The offshore structures, as shown on the attached structural drawings in Appendix A, comprise a loading arm deck and trestle, access catwalk, bollard and pile cap. The structures are shown to be supported on 60 m long, 1,219 mm diameter, open-ended steel pipe piles except for the trestle abutment piles that will be 40 m long. Based on the mudline elevation and pile cut-off elevations reported on SDE’s drawings in Appendix A, the pile embedment will be between about 40 m and 42 m.

As noted in Section 7.2, the estimated depth to liquefaction could extend to El. -47 m. Considering the sloping ground conditions at the proposed berth location, a flow slide failure is likely to occur which would result in very large displacements (> 2 m) within the liquefied zone and induce kinematic loading on the berthing dolphin piles. Kinematic loading typically results in the highest demands on nearshore piles.

Input for lateral and axial design, and installation of the offshore piles is provided below.

9.1 Lateral Pile Resistance

The performance of the proposed offshore piles under kinematic lateral loading was assessed using Group software by Ensoft Inc. The software analyses the behaviour of single piles and pile groups subjected to axial and lateral loads using two- or three- dimensional models. The soil resistance along the embedded length of a pile is modelled using a series of discrete, non-linear soil springs in the form of t-z and q-w curves for axial loading, t-r curves for torsional loading and p-y curves for lateral loading.

The piles, having a maximum vertical embedment of about 42 m, will be founded entirely in the Fraser River deposits. Tables 5 and 6 summarize the soil parameters and soil spring models used to develop the p-y curves for the loading arm deck pile group and trestle abutment piles, under kinematic loading in a 1:2,475 earthquake event.

Table 5 –Soil Parameters and Spring Models for Lateral Pile Analysis of Loading Arm Deck Piles

Design Loading Condition	**El. (m)**	**Soil Deposit**	**Soil Model**	**γ_{bulk}[1] (kN/m^3)**	**$\gamma_{submerged}$[2] (kN/m^3)**	**S_u[3] (kPa)**	**$S_{u,res}/\sigma'$[4]**
Kinematic Loading[5] Seismic (1:2,475 year)	-15 m to - 47 m	Fraser River Deposits (Liquefied)	Soft Clay (Matlock)	18	8	-	0.12
	> -47 m	Fraser River Deposits (Non-Liquefied)	Stiff Clay without Free Water			100	-

Notes:

1) Bulk unit weight.
2) Submerged unit weight.
3) Normalized residual shear strength of liquefied soils.
4) No external loading (i.e., ship impact) was assumed under seismic loading conditions.

Table 6 – Soil Parameters and Spring Models for Lateral Pile Analysis of Trestle Abutment Piles

Design Loading Condition	El. (m)	Soil Deposit	Soil Model	γ_{bulk}[1] (kN/m^3)	$\gamma_{submerged}$[2] (kN/m^3)	ϕ[3] (°)	S_u[4] (kPa)	$S_{u,res}/\sigma'$[5]
Kinematic Loading[5] Seismic (1:2,475 year)	0 m to -10 m	Fill/Sand/silt	API Sand	18	8	32	-	-
	-10 m to -47 m	Fraser River Deposits (Liquefied)	Soft Clay (Matlock)			-	-	0.12
	> -47 m	Fraser River Deposits (Non-Liquefied)	Stiff Clay without Free Water			-	100	-

Notes:

1) Bulk unit weight.
2) Submerged unit weight.
3) Internal angle of shearing resistance.
4) Normalized residual shear strength of liquefied soils.
5) No external loading (i.e., ship impact) was assumed under seismic loading conditions.

As recommended in the Ensoft (2019) Group software Technical Manual and CSA S6.1-19 “Commentary on CSA S6-19, Canadian Highway Bridge Design Code” document, the lateral response of the liquefied soils was modelled using the soft clay p-y curve model by Matlock (1970), with the undrained shear strength equal to the residual strength of the liquefied soils ($s_{u,res}/\sigma'$ of 0.12).

Additional assumptions and conditions adopted in our analysis included:

- The kinematic loading for the 1:2,475 year earthquake event was modelled by subjecting the piles to external displacements of 2 m within the liquefied zone.
- The pile head was modelled as a fixed connection.
- The pile was modelled as an elastic section.
- Pile group effects were accounted for in the analysis in the software.

Analysis outputs for the arm loading deck pile group and trestle abutment piles are summarized in Appendices F and E, respectively, and include general GROUP model view and profile plots of pile deflections, bending moments and shear for the individual pile.

The results of our analysis indicate that the proposed pile lengths are generally feasible with the exception of the trestle abutment piles where the embedment must be increased to about 45 m instead of the currently envisioned 40 m. Further, it should be noted that while it is anticipated that the foundations will experience substantial movement in the vertical and horizontal directions, the collapse-prevention design criterion is expected to be achieved provided that the piles are properly sized and designed.

9.2 Axial Pile Resistance

The axial pile resistance was assessed for static and non-liquefied seismic (1:2,475 year earthquake event) loading conditions based on high-strain dynamic test (HSDT) results from nearby projects that included testing of offshore piles of similar size in the Fraser River.

Appendix G includes Figures G-1 and G-2 which provide estimated unfactored geotechnical resistance profiles in axial compression loading for offshore piles installed at approximately mudline El. -15 m, which represents the piles supporting the loading arm deck piles, and at approximately mudline El. 0 m, which represents the piles supporting the trestle abutment. The figures also include profiles of the estimated unfactored, cumulative side shear resistance that can be used for the assessment of the uplift (tensile) resistance.

Further, the figures include unfactored side shear resistance and the total combined geotechnical resistance (i.e., side shear plus end bearing) profiles for two potential installation procedures. The first installation procedure assumes that the piles will be installed by means of vibratory driving only (i.e., no impact hammer will be used to advance the piles). The second installation procedure assumes that the pile sections penetrating the interbedded silt and sand deposit (i.e., below El. -32 m) will be installed by means of impact driving which would be expected to lessen friction fatigue effects and result in higher mobilized side shear resistances compared to vibratory driving.

Table 7 summarizes the estimated unfactored geotechnical resistances in both compression and tension and for both means of installation. These values must be confirmed during detailed design.

Table 7 – Unfactored Geotechnical Resistance in Compression and Tension for Loading Arm Deck and Trestle Abutment Piles

Pile Locations	Estimated Vertical Embedment (m)	Unfactored Geotechnical Resistance in Compression (kN)	Unfactored Geotechnical Resistance in Tension (kN)
Installation Procedure 1: Piles to be Installed by Means of Vibratory Driving Only			
Loading Arm Deck	~41 m	4,600	2,300
Trestle Abutment	~44 m	8,050	5,700
Installation Procedure 2: Piles to be Installed by Means of Impact Driving below El. -32m			
Loading Arm Deck	~41 m	7,400	5,100
Trestle Abutment	~44 m	9,950	7,600

To mitigate the risk of damage during driving, API Recommended Practice 2S-WSD (2007) recommends a minimum wall thickness, t, equal to the diameter divided by 100 plus 6.35 mm. For 1,219 mm pipe, this results in 19.1 mm wall pipe. Further, to reduce the potential for pile damage during driving, we recommend that the rated hammer energy during driving be limited to

440 kJ for 1219 mm x 19.1 mm piles. To facilitate pile installation, consideration could be given to the use of driving shoes to increase the wall thickness over a length of about 300 mm at the pile toe.

9.3 Analysis Summary

The analysis indicates that the proposed offshore foundations for the offshore structures are expected to be capable of serving their intended purposes under static and kinematic seismic loading conditions. The exception is the trestle abutment where the currently envisioned 40 m embedment length will likely need to be increased to 45 m. Further, the minimum required wall thickness of the piles should be assessed using the bending moment and shear profile plots in Appendices E and F.

10. TANK FARM

As mentioned in Section 1, the tank farm will include construction of three tanks and a secondary containment initially, with the potential for construction of additional tanks in the future. According to input from SDE, the structural demands of the 37 m diameter tanks include dead and total loads of 4,370 kN and 151,370 kN, respectively. This equates to a foundation pressure of about 140 kPa under serviceability loading condition.

The site preparation and/or ground improvement for the tank farm were initially envisaged to be completed for the whole area, including the area of the future tanks. However, it is understood that the work may be completed in two phases in an effort to reduce initial capital costs associated with the project.

Provided below is a discussion on the feasibility of different foundation support options for the tanks, and recommendations on the preferred foundation system.

10.1 Feasibility of Various Foundation Support Options

Geotechnically challenging conditions at the tank farm include:

- Near-surface (i.e., between 2 m and 6 m depth) and deep (i.e., between 17 m and 36 m depth) liquefiable soils. The near-surface liquefiable soils will affect the seismic performance of a grade supported foundation whereas the deep liquefiable soils will affect the seismic performance of deep foundations and the shallow foundation but to a lesser extent.
- Shallow silt deposits that, while somewhat over-consolidated, will be loaded beyond their pre-consolidation pressure by a grade supported foundation system and will experience relatively large settlements.
- Highly compressible peat that is prone to undergo significant secondary settlement that will affect the long-term performance of a raft slab foundation.
- Deep seated silts that will likely be prone to settlement. The settlement is expected to be greater where a deep foundation system is adopted.

Below is a discussion on four potential foundation support system options.

Raft Slab Foundation on Stone Columns with Preload Surcharging

For this option, the tanks would be supported on a raft slab foundation founded on stone columns. The columns would typically be 900 mm in diameter, extend to about 18 m depth and be installed at nominally 2.5 m spacing. The zone of ground improvement would likely have to extend about 10 m beyond edge of tanks and must include the secondary containment area. The stone columns would need to be installed ahead of preloading to enhance soil drainage.

Preloading by surcharge fill placement would be required to reduce primary and long-term secondary consolidation settlements under the tanks. A preload surcharge fill of at least 15 m with a preload duration of at least one year would likely be required. The preload surcharge would need to extend, full height, laterally at least 4.5 m beyond the edge of the tanks. Geogrid reinforced, lock-block walls would be required to retain the fill and limit the footprint of the preload area. As offsite settlements during preload surcharging are expected to be significant, it will be necessary to conduct a detailed pre-construction survey of all nearby facilities and utilities and plan on monitoring and repairing as required.

The estimated post-construction long-term settlement could be in the range of 300 mm and will be governed by how long the surcharge fill is left in place and the actual amount of surcharge fill removed.

Raft Slab Foundation on Preload Surcharged Ground with Wick Drains and Rapid Impact Compaction (RIC)

For this option, the tanks would be supported on a raft slab foundation. Preload surcharging would need to be conducted to reduce settlement to tolerable levels along with the application of rapid impact compaction (RIC) to densify and treat the near-surface liquefiable soils. To facilitate drainage during preload surcharging, wick drains would need to be installed to nominally 16 m depth and at nominally 2 m spacing. The zone of ground improvement (i.e., wick drains and RIC) would likely need extend about 10 m beyond edge of tanks.

Similar to the previous option, a preload surcharge fill of at least 15 m with a preload duration of at least one year would likely be required. The preload surcharge would need to extend, full height, laterally about 4.5 m beyond the edge of the tanks and would require geogrid reinforced, lock-block walls to limit the footprint of the preload area. Survey monitoring of all nearby facilities and utilities will also be required.

The estimated post-construction, long-term settlement would likely be in the range of 300 mm and will be governed by how long the surcharge fill is left in place and the actual amount of surcharge fill removed.

Deep Soil Mixing

For this option, the tank would also be on a raft slab foundation supported by continuous soil mix panels with up to 35% to 40% plan coverage. The panels would typically measure up to 1 m in width and would need to achieve minimum compressive strengths in the range of 5 MPa after 7 days. This method of ground treatment would typically need to extend to nominally 18 m depth and at least 5 m laterally beyond the edge of the tank foundations.

A combination of shallow and deep soil mixing may be advantageous to control secondary settlement in peat and primary consolidation settlement in the shallow silt.

The estimated long-term settlement would be expected to be in the range of 150 mm to 350 mm and will be governed by compression of the deep, interbedded silt and sand and underlying clay. Consideration could be given to preload surcharging to pre-consolidate the deep-seated silts in advance deep soil mixing and tank construction to reduce the magnitude of post-construction settlements.

Deep Foundations Support with Preload Surcharging and RIC

For this option, the tanks would be supported on deep foundations comprising close-ended, driven steel pipe piles. RIC would likely be required to treat the near-surface liquefiable soils to improve the lateral and vertical performance of the piles under seismic loading.

The estimated long-term total and differential settlements under the tanks could measure up to 400 mm and 150 mm, respectively, and will be governed by compression of the deep, interbedded silt and sand and underlying clay. If this magnitude of settlement is deemed excessive for the tanks’ performance, consideration should be given to preload surcharging to pre-consolidate the deep-seated silts in advance of tank construction.

Following discussions with SDE and DP World, we understand that the deep foundation option currently is the preferred option. Further, while initial discussions considered both driven and cast-in-place piles, it is our understanding that driven piles are the preferred deep foundation option.

SDE and DP World have also indicated that preloading is not a preferred option from a scheduling perspective. As such, the recommendations will address the anticipated settlements where no preloading using surcharge fill is completed ahead of pile installation.

10.2 Axial Pile Resistance for Tank Farm Area

The assessment of axial pile resistance for static and non-liquefied seismic (1:2,475 year earthquake event) loading conditions was based on HSDT results from nearby projects with similar ground conditions. Based on the anticipated subsurface conditions at the tank farm, we recommend that the piles be terminated at about 18 m to 20 m embedment (from ground El. +4 m) which, according to the CPT soundings, corresponds to a zone of compact to dense sand that underlies the silt deposit. Further, we recommend that the piles be driven closed-ended to engage higher end bearing (toe) resistance.

The use of an end plate (i.e., closed-end) limits the practical size of piles that can be installed due to the increased driving resistance introduced by the substantial soil displacement that occurs during installation in comparison with an open-end pipe pile. Our experience with pile driving indicates that closed-end piles measuring up to 762 mm in diameter have been successfully installed on other projects with similar ground conditions with only minor impact on construction productivity. As such, our recommendations will be provided for 610 mm and 762 mm, closed-end pipe piles.

Figures H-1 and H-2 (see Appendix H) provide estimated unfactored geotechnical resistance profiles in axial compression loading for closed-end pipe piles measuring 610 mm and 762 mm in diameter, respectively. The figures also include profiles of the unfactored, cumulative side shear resistance that can be used for the assessment of the uplift (i.e., tensile) resistance.

To mitigate the risk of damage during driving, the minimum wall thickness according to API Recommended Practice 2S-WSD (2007) is 12.7 mm for 610 mm piles and 15.9 mm for the 762 mm piles. Further, to reduce the potential for pile damage during driving, we recommend that the rated hammer energy during driving be limited to 146 kJ for 610 mm piles and 228 kJ for 762 mm piles.

As discussed in Section 10.1 above, we recommend that the deep foundation option include the completion of RIC to treat the near-surface liquefiable soils to improve the lateral and vertical performance of the piles under seismic loading.

Consideration should be given for the completion of a full-scale static loading test to optimize the foundation design by directly measuring the actual geotechnical resistance and by thereby take advantage of a higher geotechnical resistance factor.

10.3 Pile Group Settlement

Pile group settlement will be primarily governed by compression of the deep, interbedded silt and sand and underlying clay. The estimated long-term total and differential settlements under the tanks could be up to 400 mm and 150 mm, respectively, where the site is not preloaded using surcharge fill. Where preloading using surcharge fill is carried out, the estimated long-term total and differential settlements under the tanks could likely be reduced to less than 150 mm and 20 mm per 10 m lineal distance, respectively.

The height of the preload surcharge fill and preload duration will be governed by the required performance of the tanks and how the soil behaves during preloading. For preliminary purposes, it is envisaged that, if completed, the preload surcharge fill will measure about 11 m in height and laterally extend about 4.5 m beyond the edge of the tanks. Geogrid reinforced, lock-block walls would likely be required to limit the footprint of the preload area. Survey monitoring of all nearby facilities and utilities will also be required.

11. ANCILLARY STRUCTURES AND SECONDARY CONTAINMENT

Provided below is a discussion on ground settlement, foundation recommendations for ancillary structures and secondary containment wall, and input for site preparation, slab-on-grade, sub-drainage and lateral earth pressure. This section also includes a discussion on the impact of construction on adjacent third-party structures and provides preliminary input for the envisaged phased construction.

11.1 Ground Settlement

Long-term settlements due to compression of the peat adjacent to the pile supported structures should be anticipated. This settlement will be non-uniform and entirely differential to the pile-supported structures. Non-seismic ground settlement of up to 150 mm over a period of 20 years should be considered. Where RIC is completed, post-liquefaction settlement will be limited to the reconsolidation settlement of the Fraser River sands below 17 m depth (based on ground El. +4 m) and is expected to be limited to about 150 mm based on our analyses.

Flexible service connections should be provided at the edge of the area of pile support. Ideally, these connections should be located in readily accessible areas such as landscaping to allow for future maintenance. Further, the proposed non-woven geotextile blankets and PVC liner membrane should include sufficient slack to accommodate differential movements of up to 300 mm that is anticipated between the tanks and adjacent ground surface where preloading is not completed.

11.2 Secondary Containment Wall Structure

The secondary containment wall structure can be supported on deep foundations using our pile design recommendations for the tank structures. Alternatively, the secondary containment wall could be grade supported provided that RIC is carried out to densify the surficial fill and reduce the likelihood of liquefaction under seismic loading in the near-surface soils, and the wall base is supported on a minimum 1 m thick compacted granular fill pad.

Table 8 below provides the geotechnical input for the design of a grade supported secondary containment wall. The table includes unfactored and factored bearing resistance values using the geotechnical resistance values provided in Table 2 and 3.

Table 8 - Geotechnical Input for Secondary Containment Wall Structure

Design Condition[1]	Geotechnical Input for Design
Bearing	Unfactored Ultimate Resistance = 200 kPa Factored Ultimate Resistance = 100 kPa
Base Sliding	Sliding Coefficient = 0.7

Notes:
1) Settlement and deflection/tilt design conditions to be checked once the configuration has been finalized.

The zone of RIC must laterally extend a minimum of 2 m beyond the edge of the wall base. The actual distance will need to be determined during final design.

The granular fill must comprise 19 mm crushed gravel (less than 5% passing the 75 µm sieve), compacted to at least 100% Standard Proctor Maximum Dry Density (SPMDD). The compacted granular fill should extend laterally a minimum of 1 m beyond the edge of the base wall. The backfill zone should also be backfilled using compacted granular fill. The wall base should have a minimum embedment depth of 450 mm minimum below adjacent finished grade for frost protection.

The magnitude of the total and differential settlements under service limit states (SLS) conditions will primarily depend on the thickness of the peat along the wall alignment. Where peat is anticipated to be present along the wall alignment, anticipated total settlements under service limit states (SLS) conditions of up to 200 mm should be expected. Differential settlement is expected to be small (i.e., less than 20 mm per 10 lineal meters) unless the thickness of the peat layer along the alignment is substantially variable. Where peat is not anticipated to be present, total and differential settlements in the range of 25 mm and 15 mm, respectively, under service limit states (SLS) should be expected.

Based on the anticipated non-seismic settlement of up to 200 mm and post-liquefaction settlement of up to 150 mm, consideration should be given to increasing the height of the wall by an additional 300 mm (height of secondary containment wall is currently proposed at 2.7 m) to maintain the freeboard height of the wall.

11.3 Ancillary Structures

Ancillary structures can be supported on deep foundations, using our pile design recommendations for the tank structures. Alternatively, it is feasible to support lightly-loaded ancillary structures on shallow foundations, comprising strip and pad footings, provided they are founded on a minimum 1 m thick compacted granular fill pad. Table 9 below summarizes preliminary factored ultimate and serviceability bearing resistance values for strip and pad footings.

Table 9 – Foundation Design Bearing Resistances and Pressures

Foundation Soil	**Factored Ultimate Bearing Resistance[1] (kPa)**		**Serviceability Bearing Resistance (kPa)**	
Compact Granular Fill	**Strip Footings**	**Pad footings**	**Strip Footing**	**Pad footing**
	200	250	100	100

[1] Factored ultimate bearing resistance values include a geotechnical resistance factor $\Phi=0.5$

The bearing resistance was evaluated for strip and pad footings assuming minimum widths of 450 and 600 mm, respectively. Footings should have a minimum embedment depth of 450 mm minimum below adjacent finished grade for frost protection.

The granular fill, comprising 19 mm crushed gravel (less than 5% passing the 75 µm sieve), should be compacted to at least 100% SPMDD and should typically laterally extend a minimum of 1 m beyond the edge of the footings.

The magnitude of the total and differential settlements under service limit states (SLS) conditions will primarily depend on the thickness of the peat. Where peat is anticipated to be present under the footprint of a structure, anticipated total settlements under service limit states (SLS) conditions of up to 200 mm should be expected. However, differential settlement is expected to be small (i.e., generally in the range of 25 mm) for lightly-loaded structures unless the thickness of the peat layer under the footprint of the structure is substantially variable. Where peat is not anticipated to be present, total and differential settlements in the range of 25 mm and 15 mm, respectively, under service limit states (SLS) should be expected. These settlement estimates and bearing resistances must be confirmed during detailed design when the structural loads become available.

Completion of RIC to treat the near-surface liquefiable soils is recommended under structures supported on shallow foundations to reduce the likelihood of near-surface liquefaction under seismic loading. Consideration should also be given to structurally connecting the footings and floor slab together to achieve the collapse-prevention under seismic loading.

11.4 Site Preparation and Fill Placement Requirements

All construction work must be completed in safe manner and must conform to the all applicable regulations such as WorkSafeBC, laws, codes and any other relevant regulations in the Province of British Columbia and to any applicable company-specific regulations.

Site preparation should proceed with the removal of any existing structures and landscaping within the area of the proposed addition. Any underground services and utilities crossing this area should also be relocated or terminated appropriately. Excavated material from trenches must be removed and replaced with compacted granular fill.

Excavation should be carried out using excavators equipped with a smooth-edge trimming buckets. The base of all excavations should be free of loose, organic, or disturbed material. All water must be drained away to prevent ponding. Large-sized granular particles protruding above the bearing surface must be eliminated, either by removal or splitting, to avoid hard-points on the underside of the foundations.

Some of the foundation soils will typically be sensitive to changes in moisture content and disturbance by construction and repeated pedestrian traffic. Therefore, unless the footing concrete will be placed within 24 hours of exposing the bearing surface, a concrete blinding layer (or equivalent) should be placed on the bearing surface to reduce the likelihood of disturbance.

Grade restoration fill should typically comprise free draining (<5% passing the 75 µm sieve) granular material, and must be free of organics and other deleterious material. Suitable materials include MMCD minus 75 mm well graded pit run sand and gravel. Other granular material may also be acceptable but samples or representative gradation curves of the material should be submitted to a qualified geotechnical engineer for review and approval prior to use.

Grade restoration and granular fills should typically be placed in maximum 300 mm thick loose lifts and compacted to at least 100% Standard Proctor Maximum Dry Density (SPMDD).

Unless walls supporting soil are specifically designed to support compaction-induced lateral stresses, backfill placed within 1 m of a wall should be compacted using light weight equipment such as a plate tamper to avoid build-up of excessively high lateral soil pressure on the wall.

11.5 Slab-On-Grade

For slab-on-grade, we recommend excavating a minimum of 300 mm and then bringing the site up to design grade using granular fill, comprising 19 mm crushed gravel (less than 5% passing the 75 µm sieve), placed and compacted as outlined in Section 11.4. All loose material, organic, soft or wet soils, or other deleterious material at the base of the subgrade must be removed and replaced with compacted granular fill.

A vapour barrier comprising 6-mil (minimum) polyethylene sheeting should be placed on top of the gravel layer. Adjacent sheets of polyethylene should be overlapped by a minimum of 300 mm. The underslab gravel layer should be hydraulically connected to the perimeter sub-drainage system.

The exposed subgrade surface must be reviewed by a qualified geotechnical engineer to assess conformance with the soil conditions expected.

11.6 Sub-Drainage

A perimeter sub-drainage system should be installed for building structures. The sub-drain should comprise 100 mm to 150 mm diameter perforated PVC pipe (with perforations down) surrounded by a minimum of 150 mm of drain rock. The drain rock should be fully separated from the general backfill by a non-woven geotextile (such as Nilex 4553 or approved equivalent). The sub drainage system should be installed with an invert level at or nominally below the underside of the clear crush gravel layer. The perimeter sub-drains should be connected to a suitable point of gravity discharge or a pumped sump.

Within 2 m of a building, the yard grade should be sloped to provide surface drainage away from the structures.

11.7 Underground Services and Utilities

Underground services and utilities, including sub-drains that run parallel to the footings should not be located within a zone defined by a plane sloping down and away from the bottom perimeter edge of footing at 1 horizontal to 1 vertical (1H:1V). If services cannot be relocated, they must be fully encased in concrete or the affected footing must be lowered.

11.8 Lateral Earth Pressure

Foundation walls should be designed using the lateral earth pressure distribution shown on Figure I-1 in Appendix I. The pressure distribution assumes fully-drained conditions, a “non-yielding” wall (i.e., a wall that is unable to rotate at least 0.005H) under static loading condition and a “yielding” wall (i.e., a wall that is able to rotate at least 0.005H) under seismic loading condition, no surcharge loading and that the wall backfill is comprised of material that conforms to the requirements of Section 11.4 and that it is hydraulically connected to the sub-drainage system.

11.9 Third-Party Structures

Construction activities, particularly pile driving and RIC, are anticipated to affect adjacent third-party structures. Further, there is a likelihood that adjacent third-party structures may also sustain permanent settlement by the loading imposed from the tanks and preload surcharges, if completed. As such, it is imperative to complete survey monitoring of all nearby facilities before the onset of construction to establish a baseline, and during and following completion of construction to evaluate the effect of construction on the adjacent third-party structures. We also recommend the completion of vibration monitoring adjacent to these structures to evaluate the effect of the various construction activities.

11.10 Considerations for Phased Construction

As mentioned previously, the site preparation and/or ground improvement for the tank farm were initially envisaged to be completed for the whole area, including the area of the future tanks. However, it is understood that the work may be completed in two phases in an effort to reduce initial capital costs associated with the project. Future construction activities that are anticipated to affect the performance of the initial tanks are pile installation and RIC.

<u>Pile Installation</u>

One of the main references on vibration monitoring is British Standard (BS) 5228. The standard categorizes vibrations, which are measured in terms of peak particle velocity (PPV), as being continuous, transient or intermittent. The vibrations generated during pile driving using an impact hammer are classified as intermittent vibrations and are primarily a function of the delivered energy per blow, pile impedance and ground conditions.

For nearby structures, acceptable levels for intermittent or transient vibrations to limit cosmetic damage are provided in Table 10 below.

Table 10 – Transient Vibration Guide Values for Cosmetic Damage According to BS-5228

Type of Building	**Peak Particle Velocity in Frequency Range of Predominant Pulse**	
	4 Hz to 15 Hz	**15 Hz and above**
- Reinforced or framed structures - Industrial and heavy commercial buildings	50 mm/s at 4 Hz and above	
- Unreinforced or light framed Structures - Residential or light commercial buildings	15 mm/s at 4 Hz increasing to 20 mm/s at 15 Hz	20 mm/s at 15 Hz increasing to 50 mm/s at 40 Hz and above

Minor damage to structures is likely to occur where the vibration levels are greater than twice the limit for cosmetic damage. For instance, a reinforced or framed structure will sustain minor damage where the measured peak particle velocity exceeds 100 mm/s, regardless of the predominant frequency.

The concern with the above limits is that they do not take into consideration the potential for soil settlement, particularly in sandy soils, due to ground vibrations, which could result in permanent structural damage. To reduce the risk of potential settlement in sand due to ground vibration, the maximum peak particle velocity adjacent to the phase 1 tanks will likely need to be limited to 20 mm/s during construction of the future tanks. The selected peak particle velocity limit is based on the relation introduced by Massarch (2002) for a shear wave velocity of 200 m/s in the sand and a low risk tolerance for settlement.

For preliminary purposes, the vibration attenuation formula proposed by Fellenius (2021) was used to estimate the required setback from the tanks to maintain peak particle velocity levels below 20 mm/s. Assuming the use of a Pileco D62-22 diesel hammer operating at fuel setting 4, it is anticipated that a minimum setback of at least 10 m between the tanks and nearest piles would be required. It is strongly recommended, however, that vibration monitoring be completed at the onset of construction, or during a pile testing program, to measure the actual vibrations at various distances away from the piling activity.

Measures to reduce pile driving induced vibrations include pre-drilling of the upper 6 m at the pile locations, which corresponds to the compact to dense sand fill layer. Another measure includes operating the hammer at a lower fuel setting (i.e., reducing the hammer energy).

RIC

RIC is typically carried out using a 7 ton or 9 ton mass that is dropped from controlled heights ranging from 1 m to 1.5 m. Based on local vibration monitoring data completed during RIC installations, the measured peak particle velocity is not expected to exceed 20 mm/s for a setback of at least 10 m between the tanks and the source (i.e., the RIC equipment). Similarly, it is strongly recommended that vibration monitoring be completed at the onset of construction to measure the actual vibrations at various distances away from the construction activity.

12. CLOSURE

We trust this information meets your present needs. If you have any questions, please contact the undersigned at your convenience.

Yours truly,
Thurber Engineering Ltd.

David Tara, P.Eng.
Review Engineer

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Attachments

- Statement of Limitations and Conditions (1 page)
- Symbols and Terms for Soil Description and Test Hole Logs (2 page)
- Figure 1 – Site Plan (1 page)
- Figure 2 – Proposed Site Project Layout (1 page)
- Appendix A – SDE Drawings (6 pages)
- Appendix B – Test Hole Logs (6 pages)
- Appendix C – Cone Penetration Test Report by Earth Drilling (23 pages)
- Appendix D – Design Response Spectra (1 page)
- Appendix E – Lateral Analysis of Arm Loading Deck Group Pile under Seismic Loading Kinematic (22 pages)
- Appendix F – Lateral Analysis of Trestle Abutment Piles Pile under Seismic Kinematic Loading (5 pages)
- Appendix G – Unfactored Axial Geotechnical Resistance Plots for Offshore piles (2 pages)
- Appendix H – Unfactored Axial Geotechnical Resistance Plots for Tank Farm Piles (2 pages)
- Appendix I – Lateral Earth Pressure Distribution Diagram for Basement Walls (1 page)

STATEMENT OF LIMITATIONS AND CONDITIONS

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This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

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b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.

d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

7. INDEPENDENT JUDGEMENTS OF CLIENT

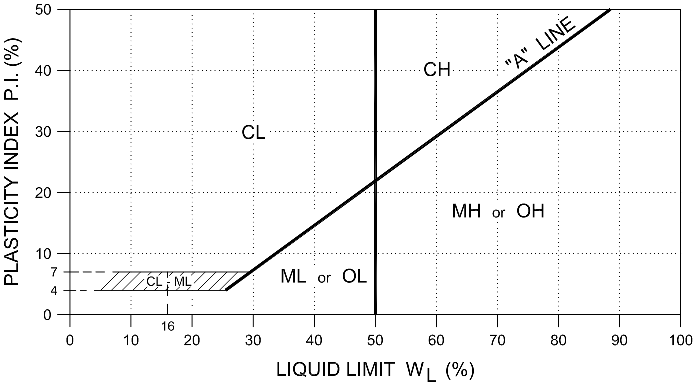
The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

THURBER

UNIFIED CLASSIFICATION SYSTEM FOR SOILS (ASTM D2487)

MAJOR DIVISION			SYMBOLS		TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
			GROUP	GRAPH		
COARSE-GRAINED SOILS (MORE THAN 50% BY WEIGHT RETAINED ON No. 200 SIEVE)	GRAVELS MORE THAN 50% COARSE FRACTION RETAINED ON No. 4 SIEVE	CLEAN GRAVELS (< 5% FINES)	**GW**	▨	WELL GRADED GRAVEL and WELL GRADED GRAVEL with SAND.	$C_U = \frac{D_{60}}{D_{10}} \geq 4$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$
			GP	▨	POORLY GRADED GRAVEL and POORLY GRADED GRAVEL with SAND.	NOT MEETING ABOVE REQUIREMENTS
		GRAVELS WITH FINES (> 12% FINES)	**GM**	▨	SILTY GRAVEL, GRAVEL - SAND - SILT MIXTURES.	FINES CLASSIFY AS ML or MH (3)
			GC	▨	CLAYEY GRAVEL, GRAVEL - SAND - CLAY MIXTURES.	FINES CLASSIFY AS CL or CH (3)
	SANDS MORE THAN 50% COARSE FRACTION PASSES No. 4 SIEVE	CLEAN SANDS (< 5% FINES)	**SW**	▨	WELL GRADED SAND and WELL GRADED SAND with GRAVEL	$C_U = \frac{D_{60}}{D_{10}} \geq 6$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$
			SP	▨	POORLY GRADED SAND and POORLY GRADED SAND with GRAVEL.	NOT MEETING ABOVE REQUIREMENTS
		SANDS WITH FINES (> 12% FINES)	**SM**	▨	SILTY SAND, SAND - SILT MIXTURES.	FINES CLASSIFY AS ML or MH (3)
			SC	▨	CLAYEY SAND, SAND - CLAY MIXTURES.	FINES CLASSIFY AS CL or CH (3)
FINE-GRAINED SOILS (MORE THAN 50% BY WEIGHT PASSES No. 200 SIEVE)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	**ML**	▨	INORGANIC SILTS, SILTS with SAND and SILTS with GRAVEL and SANDY or GRAVELLY SILTS.	P.I. < 4 or PLOTS BELOW THE "A" LINE
		$W_L > 50\%$	**MH**	▨	INORGANIC SILTS, SILTS with SAND & SILTS with GRAVEL & SANDY or GRAVELLY SILTS, FINE SANDY or SILTY SOILS.	P.I. PLOTS BELOW THE "A" LINE
	CLAYS ABOVE "A" LINE ON PLASTICITY CHART NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	**CL**	▨	INORGANIC CLAYS of LOW PLASTICITY, GRAVELLY, SANDY, or SILTY CLAYS, LEAN CLAYS.	P.I. > 7 and PLOTS ON OR ABOVE THE "A" LINE
		W_L near 50%	**CL-CH**	▨	BORDERLINE INORGANIC CLAYS and SILTY CLAYS with LIQUID LIMITS NEAR 50%.	(only used for visual identification)
		$W_L > 50\%$	**CH**	▨	INORGANIC CLAYS of HIGH PLASTICITY, FAT CLAYS.	P.I. PLOTS ON OR ABOVE THE "A" LINE
	ORGANIC SILTS and CLAYS	$W_L < 50\%$	**OL**	▨	ORGANIC SILTS and ORGANIC SILTY CLAYS of LOW PLASTICITY.	$\frac{W_L \text{ (oven dried)}}{W_L \text{ (not dried)}} < 0.75$
		$W_L > 50\%$	**OH**	▨	ORGANIC CLAYS OF HIGH PLASTICITY.	$\frac{W_L \text{ (oven dried)}}{W_L \text{ (not dried)}} < 0.75$
HIGHLY ORGANIC SOILS			**PT**	▨	PEAT and other HIGHLY ORGANIC SOILS.	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE.

PLASTICITY CHART FOR SOILS PASSING No. 40 SIEVE



NOTES:

1. ALL SIEVE SIZES ARE U.S. STANDARD, A.S.T.M. E11-04.
2. COARSE GRAINED SOILS WITH 5 TO 12% FINES REQUIRE DUAL SYMBOLS (GW-GM, GW-GC, GP-GM, GP-GC, SW-SM, SW-SC, SP-SM, SP-SC).
3. IF FINES CLASSIFY CL-ML USE DUAL SYMBOL (GC-GM or SC-SM).
4. WHERE TESTING IS NOT CARRIED OUT, THE IDENTIFICATIONS ARE DETERMINED BY VISUAL-MANUAL PROCEDURES DESCRIBED IN ASTM D2488-06.

Client: TEL Standard Detail
File No.: n/a
E-File: UCSFS-20140807.dwg

Revised: August 07, 2014

THURBER

SYMBOLS AND TERMS
USED ON TEST LOGS

1. PARTICLE SIZE CLASSIFICATION OF MINERAL SOILS

DESCRIPTION		APPARENT PARTICLE SIZE
BOULDERS		> 200 mm
COBBLES		75 mm to 200 mm
GRAVEL	coarse	19 mm to 75 mm
	fine	4.75 mm to 19 mm
SAND	coarse	2 mm to 4.75 mm
	medium	0.475 mm to 2 mm
	fine	0.075 mm to 0.475 mm
SILT		Non-plastic particles, not visible to the naked eye
CLAY		Plastic particles, not visible to the naked eye

NOTE: Metric Conversion is approximate only

2. TERMS DESCRIBING CONSISTENCY (Cohesive Soils Only)

DESCRIPTION	APPROXIMATE UNDRAINED SHEAR STRENGTH
Very Soft	Less than 10 kPa (250 psf)
Soft	10 to 25 kPa (250 - 500 psf)
Firm	25 to 50 kPa (500 - 1000 psf)
Stiff	50 to 100 kPa (1000 - 2000 psf)
Very Stiff	100 to 200 kPa (2000 - 4000 psf)
Hard	Greater than 200 kPa (4000 psf)

NOTE: Metric Conversion is approximate only

3. TERMS DESCRIBING DENSITY (Cohesionless Soils Only)

DESCRIPTION	STANDARD PENETRATION TEST Number of blows per foot (300 mm) *
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

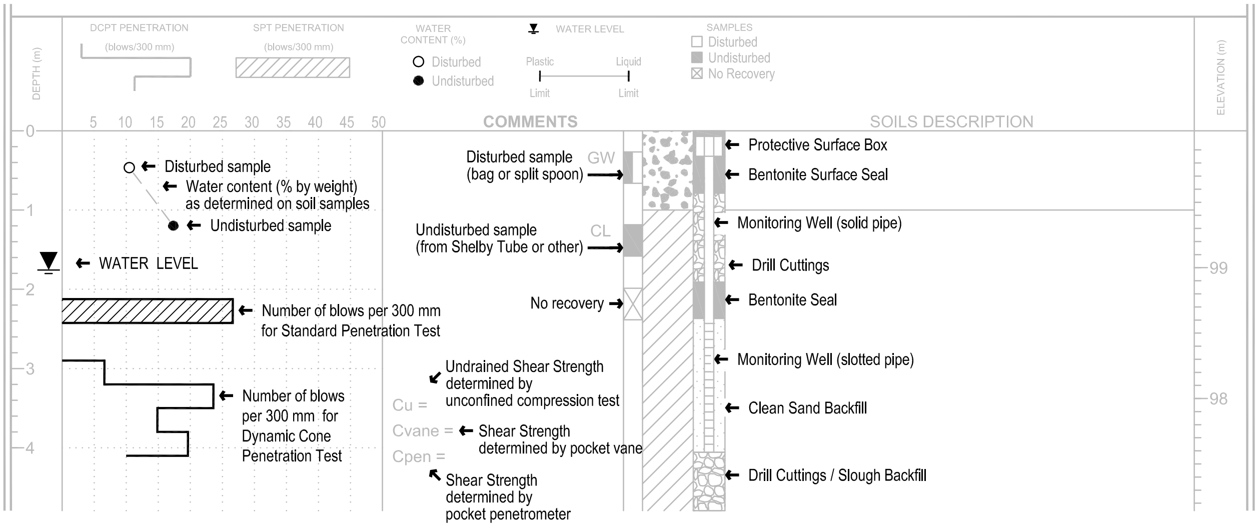
** Directly applicable to sands and, with interpretation, to gravels*

4. PROPORTION OF MINOR COMPONENTS BY WEIGHT

DESCRIPTION	PERCENT BY WEIGHT
and	35 to 50 %
y / ey	20 to 35 %
some	10 to 20 %
trace	less than 10 %

EXAMPLE: Silty SAND, trace of gravel = Sand with 20 to 35% silt and up to 10% gravel, by dry weight. (Percentages of secondary materials are estimates based on visual and tactile assessment of samples).

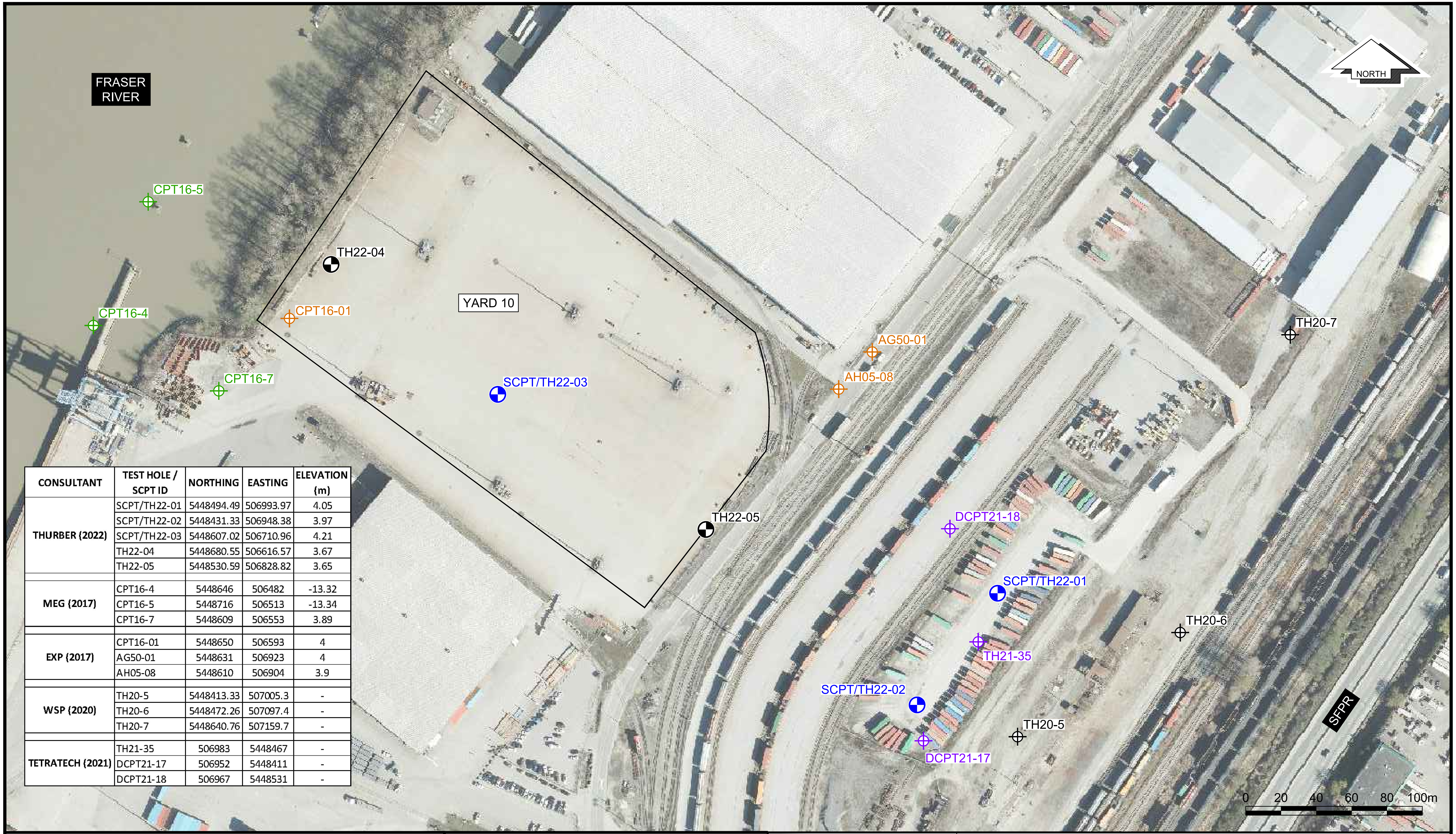
5. LEGEND FOR TEST HOLE LOGS *(Typical only showing commonly included elements)*



Client: TEL Standard Detail
File No.: n/a
E-File: UCSFS-20140807.dwg

Revised: August 07, 2014
UCSFS, page 2 of 2

Plotted: June 15, 2022



CONSULTANT	TEST HOLE / SCPT ID	NORTHING	EASTING	ELEVATION (m)
THURBER (2022)	SCPT/TH22-01	5448494.49	506993.97	4.05
	SCPT/TH22-02	5448431.33	506948.38	3.97
	SCPT/TH22-03	5448607.02	506710.96	4.21
	TH22-04	5448680.55	506616.57	3.67
	TH22-05	5448530.59	506828.82	3.65
MEG (2017)	CPT16-4	5448646	506482	-13.32
	CPT16-5	5448716	506513	-13.34
	CPT16-7	5448609	506553	3.89
EXP (2017)	CPT16-01	5448650	506593	4
	AG50-01	5448631	506923	4
	AH05-08	5448610	506904	3.9
WSP (2020)	TH20-5	5448413.33	507005.3	-
	TH20-6	5448472.26	507097.4	-
	TH20-7	5448640.76	507159.7	-
TETRATECH (2021)	TH21-35	506983	5448467	-
	DCPT21-17	506952	5448411	-
	DCPT21-18	506967	5448531	-

LEGEND:

◑ TEST HOLE (THURBER)
◑ SCPT / TEST HOLE (THURBER)
⊕ TEST HOLE / CPT (WSP)
⊕ TEST HOLE / CPT (MEG)
⊕ TEST HOLE / CPT (EXP)
⊕ TEST HOLE / CPT (TETRATECH)

NOTES:

1. AERIAL IMAGE TAKEN FROM THE CITY OF SURREY OPEN DATA CATALOGUE.
2. THURBER TEST HOLE LOCATIONS WERE SURVEYED ON APRIL 1, 2022 BY SPITFIRE DRONE SURVEY.
3. TEST HOLES BY WSP, MEG, EXP AND TETRATECH ARE APPROXIMATE.

THURBER ENGINEERING LTD.

SACRÉ - DAVEY ENGINEERING INC.

TEST HOLE LOCATION PLAN

DP WORLD FRASER SURREY
CANOLA OIL TRANSLOAD FACILITY
SURREY, BC

DESIGNED	DRAWN	APPROVED	DATE	SCALE	PROJECT No.	DWG. NO.	REV.
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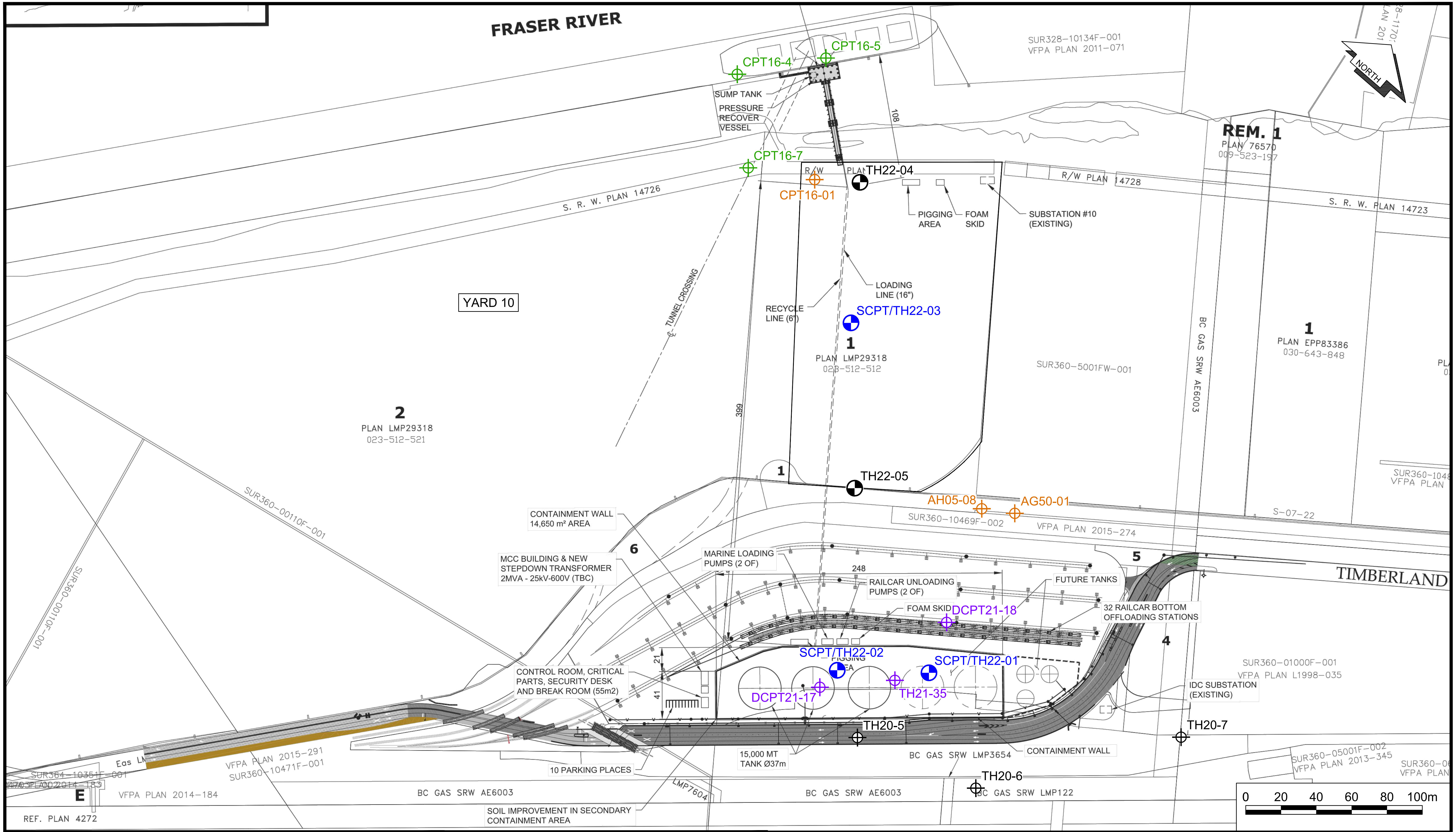
Plotted: June 30, 2022



LEGEND:

◐ TEST HOLE (THURBER)
◐ SCPT / TEST HOLE (THURBER)
⊕ TEST HOLE / CPT (WSP)
⊕ TEST HOLE / CPT (MEG)
⊕ TEST HOLE / CPT (EXP)
⊕ TEST HOLE / CPT (TETRATECH)

NOTES:

1. BASEPLAN TAKEN FROM "7704-GA-001-RF", DATED APRIL 26, 2022, BY SACRÉ - DAVEY ENGINEERING INC.
2. THURBER TEST HOLE LOCATIONS WERE SURVEYED ON APRIL 1, 2022 BY SPITFIRE DRONE SURVEY.
3. TEST HOLES BY WSP, MEG, EXP AND TETRATECH ARE APPROXIMATE.

THURBER ENGINEERING LTD.

SACRÉ - DAVEY ENGINEERING INC.

TEST HOLE LOCATION PLAN

DP WORLD FRASER SURREY
CANOLA OIL TRANSLOAD FACILITY
SURREY, BC

DESIGNED	DRAWN	APPROVED	DATE	SCALE	PROJECT No.	DWG. NO.	REV.
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THURBER

APPENDIX A

SDE Drawings (6 pages)

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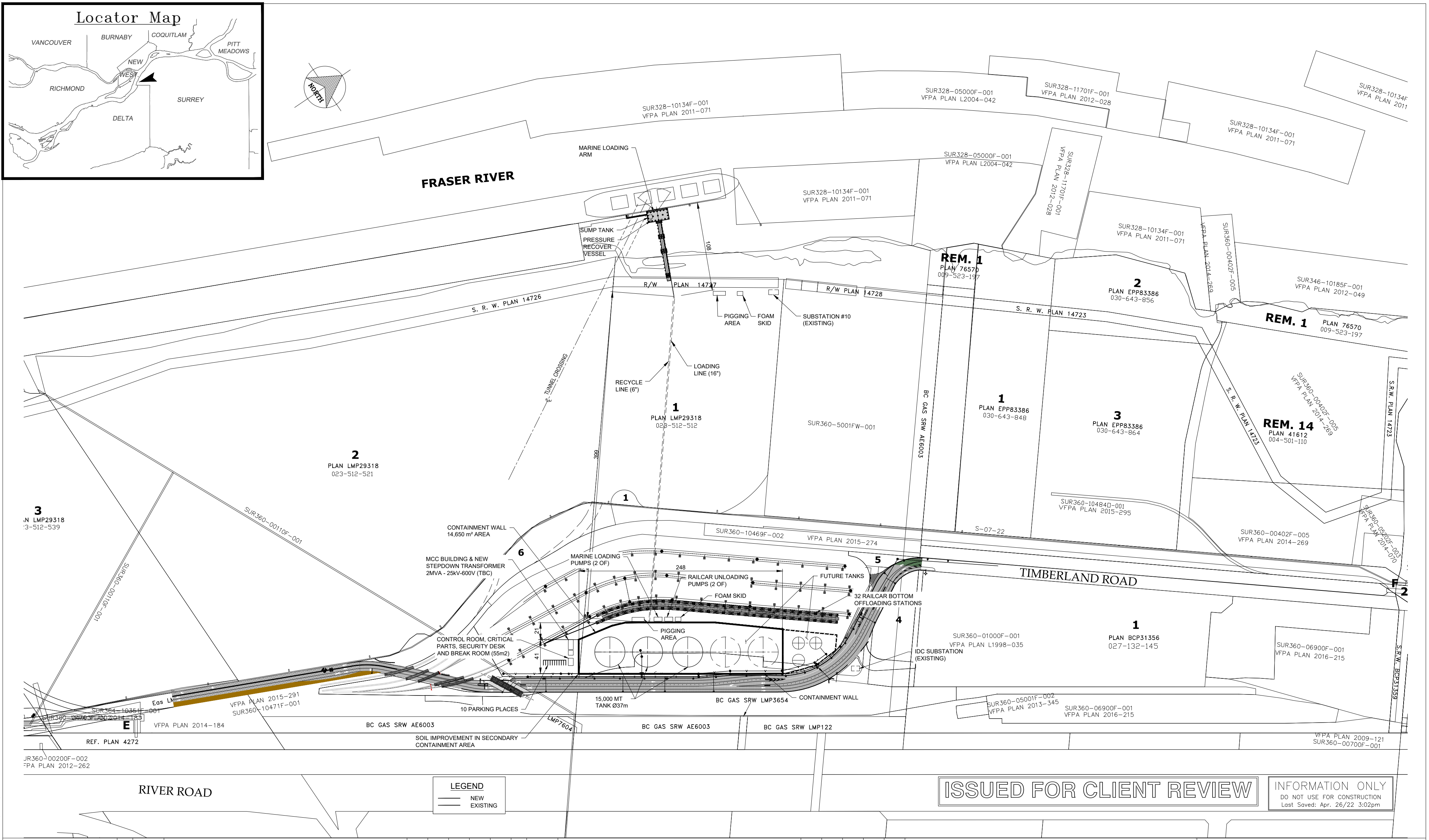

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INFORMATION ONLY
DO NOT USE FOR CONSTRUCTION
Last Saved: Apr. 26/22 3:02pm

PROFESSIONAL SEAL

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E	MAR17/22	ISSUED FOR CLIENT REVIEW	RC				DP
D	MAR14/22	ISSUED FOR CLIENT REVIEW	RC				DP
C	DEC03/21	ISSUED FOR CLIENT REVIEW	RC				DP
B	NOV26/21	ISSUED FOR CLIENT REVIEW	RC				DP
A	NOV12/21	ISSUED FOR CLIENT REVIEW	SN				DP

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FRASER SURREY

CONSULTANT
SACRÉ DAVEY

TITLE
CANOLA OIL TRANSLOAD FEASIBILITY STUDY
TRANSLOAD FACILITY – SITE PLAN
–

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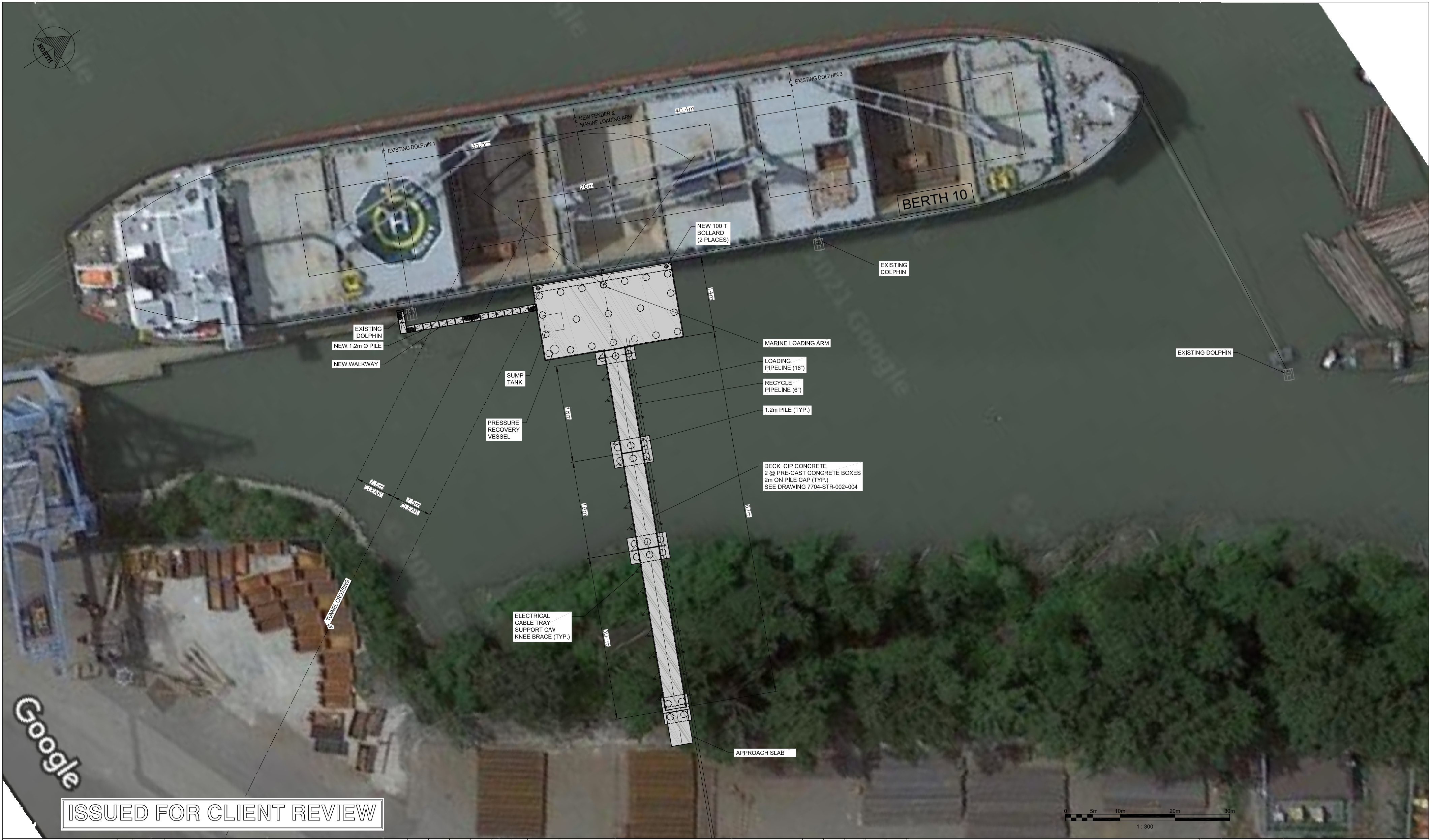

ISSUED FOR CLIENT REVIEW

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A	NOV12/21	ISSUED FOR CLIENT REVIEW	RC	PC	PC	–	DP

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TITLE

CANOLA OIL TRANSLOAD FEASIBILITY STUDY
LOADING DECK
GENERAL ARRANGEMENT

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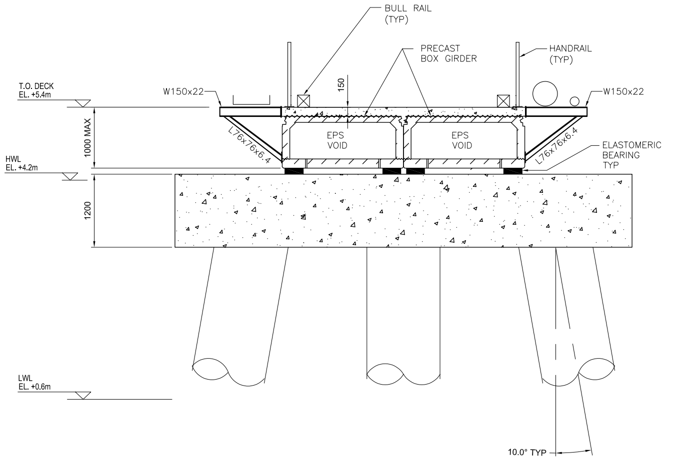

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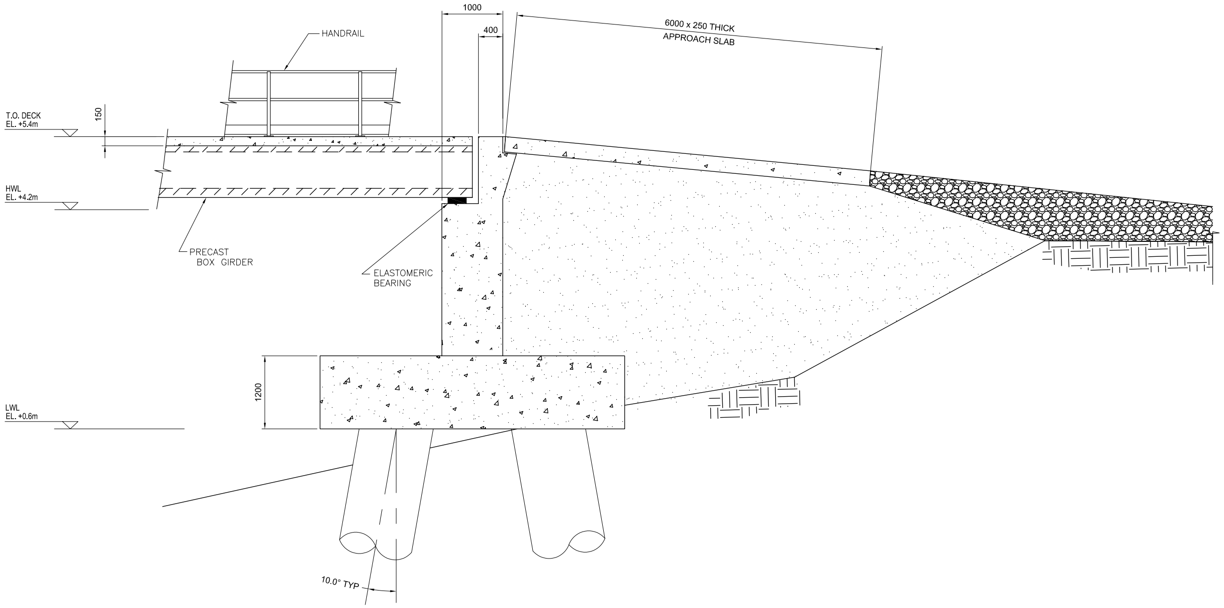
7704-STR-004



DETAIL A
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7704-STR-004

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PROFESSIONAL SEAL

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		ISSUE / REVISIONS					

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		ISSUE / REVISIONS					

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SACRÉ
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TITLE
CANOLA OIL TRANSLOAD FEASIBILITY STUDY
NEW BOLLARD PILE CAP

DRAWING SCALE	PROJECT NUMBER	DRAWING NUMBER	REV.
NOTED	7704	7704–STR–002	B

Filename: Z:\Jobs\A To H\DP World Canada\7704 Canola Oil Study\9 Drawings\5 CAD\4 Struct\7704–STR–002.dwg – Layout
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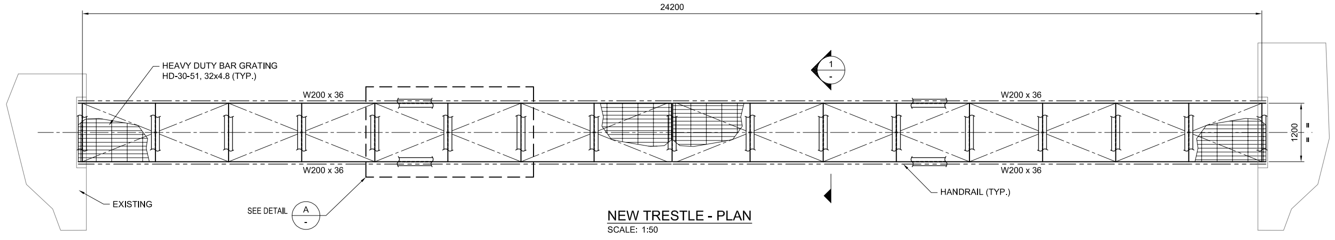

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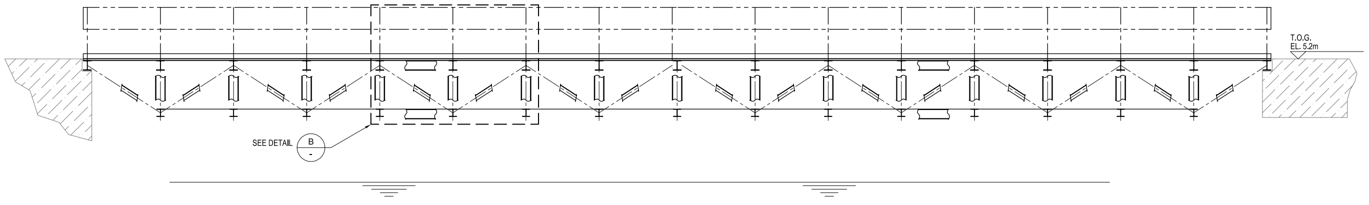

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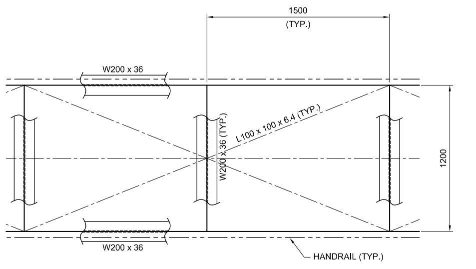

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-
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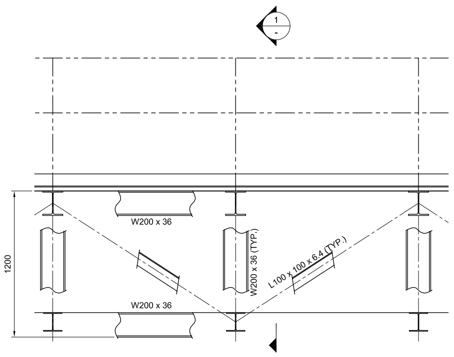

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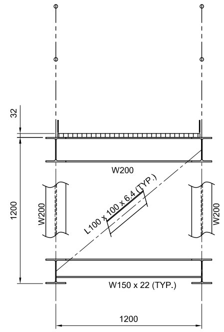

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SACRÉ DAVEY

TITLE
CANOLA OIL TRANSLOAD FEASIBILITY STUDY
NEW TRESTLE

DRAWING SCALE	PROJECT NUMBER	DRAWING NUMBER	REV.
NOTED	7704	7704–STR–003	B

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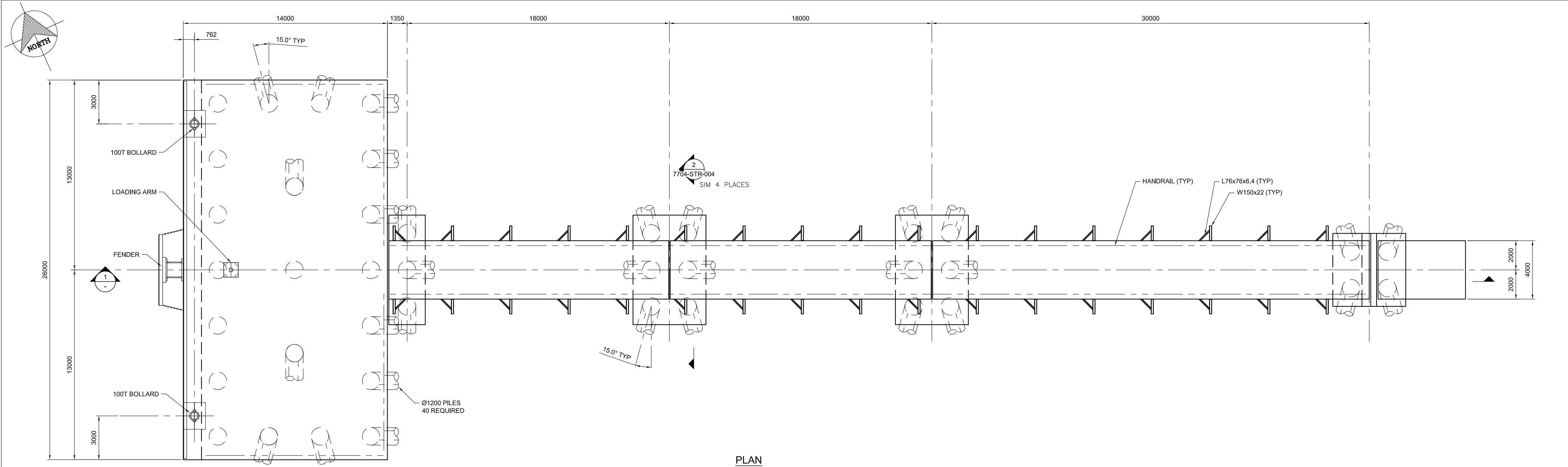

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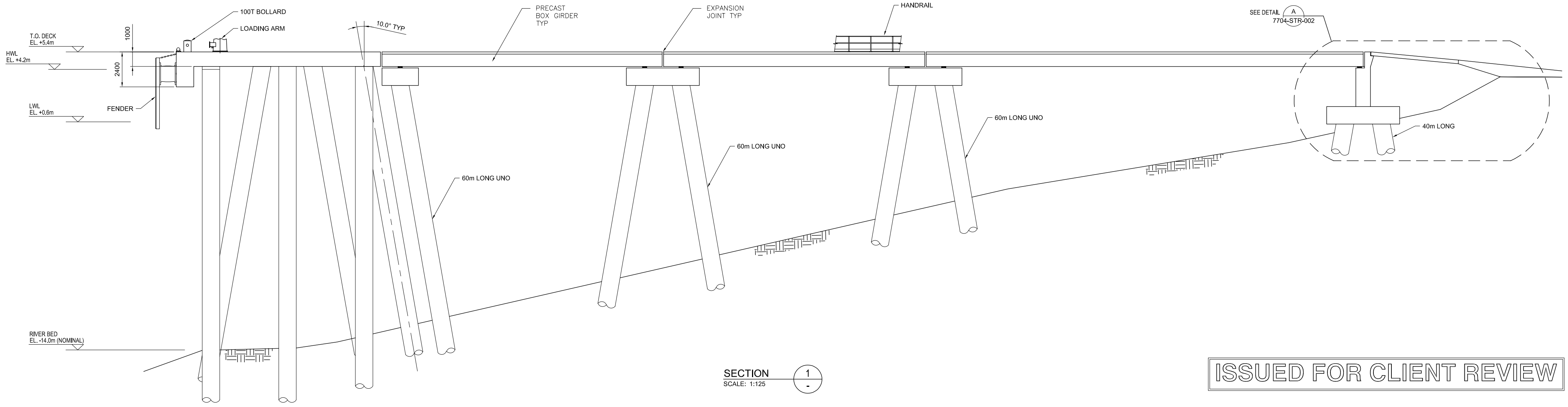

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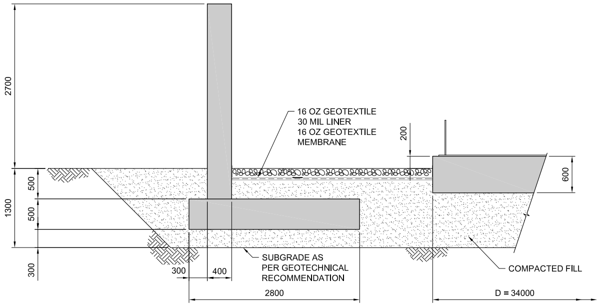
TITLE
CANOLA OIL TRANSLOAD FEASIBILITY STUDY
NEW LOADING ARM DECK AND TRESTLE

DRAWING SCALE	PROJECT NUMBER	DRAWING NUMBER	REV.
NOTED	7704	7704–STR–004	B



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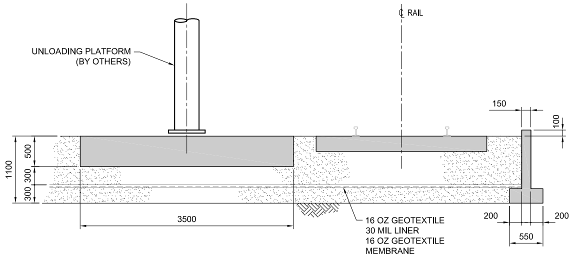

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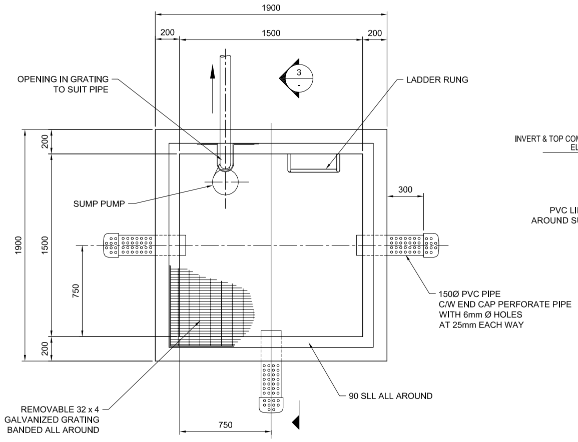
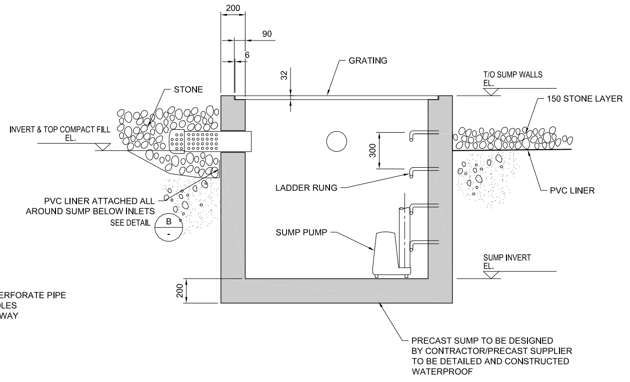


PLAN - SUMP

DETAIL A -
SCALE: 1:20



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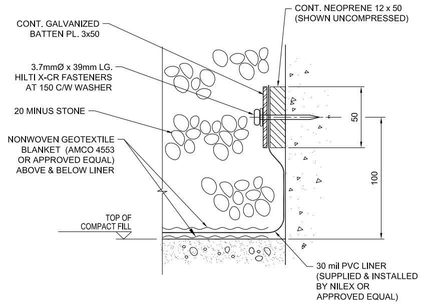

DETAIL B -
SCALE: 1:2

TYPICAL LINER AT CONTAINMENT WALL , FOOTINGS & CONCRETE SUMP

ISSUED FOR CLIENT REVIEW

PROFESSIONAL SEAL

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B	MAR25/22	ISSUED FOR CLIENT REVIEW	TH	PC	PC	–	DP
A	NOV12/21	ISSUED FOR CLIENT REVIEW	RC	PC	PC	–	DP

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SACRÉ DAVEY

TITLE
CANOLA OIL TRANSLOAD FEASIBILITY STUDY
STRUCTURAL DETAILS
SHEET 1

DRAWING SCALE	PROJECT NUMBER	DRAWING NUMBER	REV.
NOTED	7704	7704–STR–005	B

Filename: Z:\Jobs\A To H\DP World Canada\7704 Canola Oil Study\9 Drawings\5 CAD\4 Struct\7704–STR–005.dwg – Layout
Last Saved: Nov. 23/21 5:25pm Plotted: Mar. 25/22

THURBER

APPENDIX B

Test Hole Logs (6 pages)

Sheet 1 of 1

LOG OF TEST HOLE

TEST HOLE NO. TH22-01

LOCATION: See DWG. 34098-1
N 5448494.49, E 506993.97

TOP OF HOLE ELEV: 4.05 m

METHOD: Solid Stem Auger

DRILLING CO.: Earth Drilling Co. Ltd.

INSPECTOR: DA

THURBER

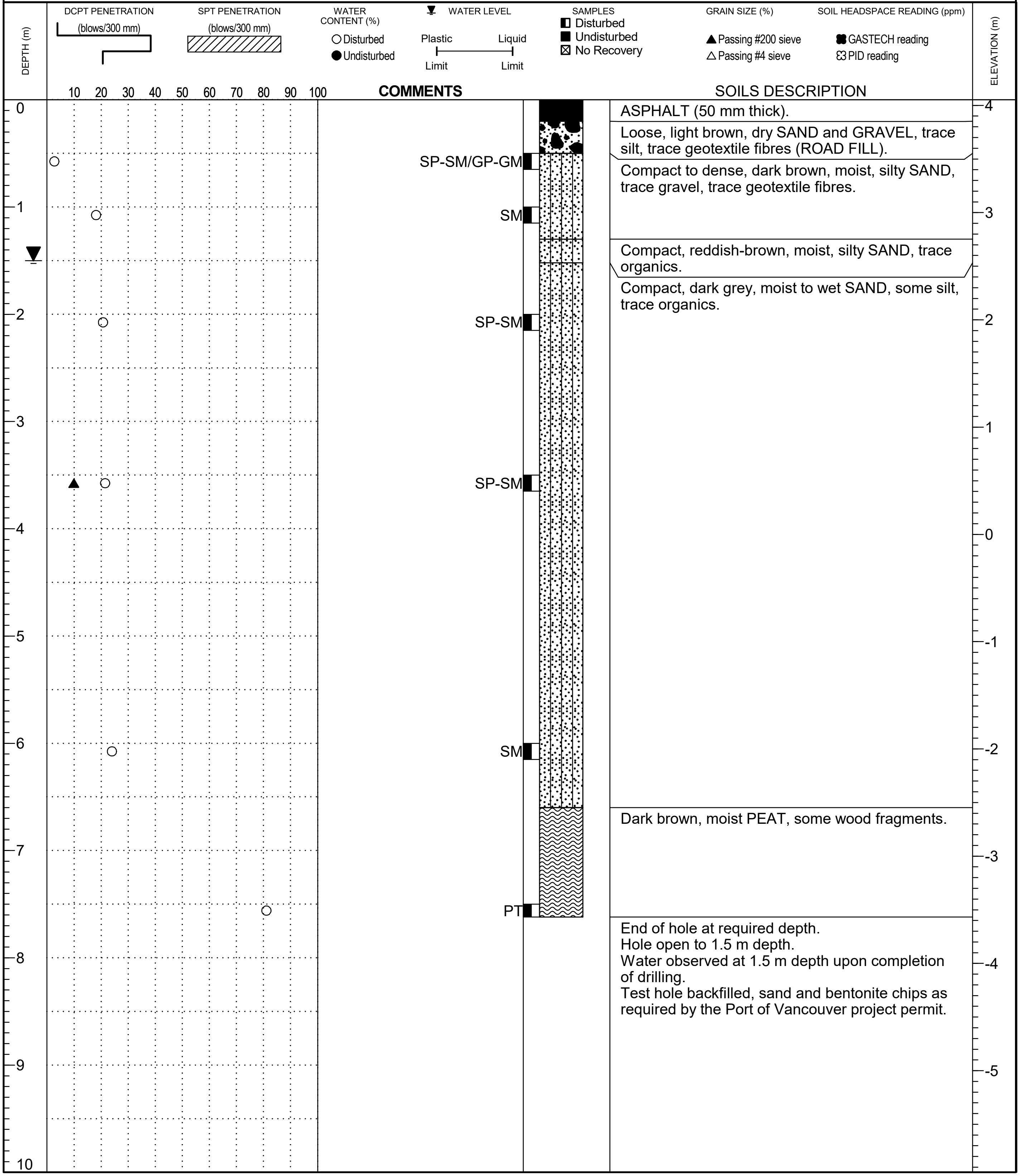
CLIENT: Sacré - Davey Engineering Inc.

PROJECT: DP World Fraser Surrey
Canola Oil Transload Facility

DATE: April 5, 2022

FILE NO.: 34098

REVIEWED BY: TFD



ASPHALT (50 mm thick).

Loose, light brown, dry SAND and GRAVEL, trace silt, trace geotextile fibres (ROAD FILL).

Compact to dense, dark brown, moist, silty SAND, trace gravel, trace geotextile fibres.

Compact, reddish-brown, moist, silty SAND, trace organics.

Compact, dark grey, moist to wet SAND, some silt, trace organics.

Dark brown, moist PEAT, some wood fragments.

End of hole at required depth.
Hole open to 1.5 m depth.
Water observed at 1.5 m depth upon completion of drilling.
Test hole backfilled, sand and bentonite chips as required by the Port of Vancouver project permit.

Sheet 1 of 1

LOG OF TEST HOLE

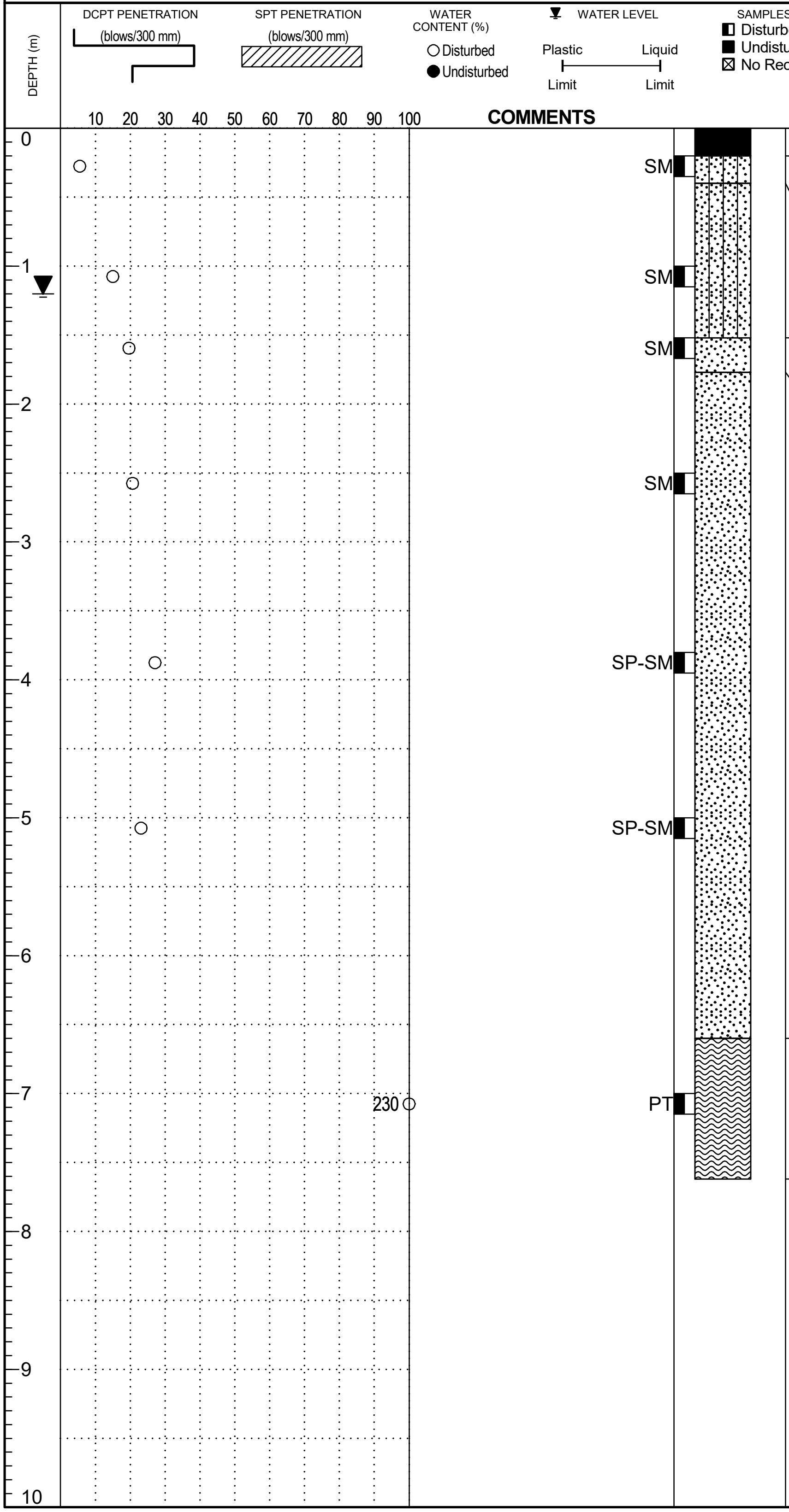
TEST HOLE NO.
TH22-02

LOCATION: See DWG. 34098-1
N 5448431.33, E 506948.38

TOP OF HOLE ELEV: 3.97 m
METHOD: Solid Stem Auger
DRILLING CO.: Earth Drilling Co. Ltd.
INSPECTOR: DA

CLIENT: Sacré - Davey Engineering Inc.
PROJECT: DP World Fraser Surrey
Canola Oil Transload Facility

DATE: April 5, 2022
FILE NO.: 34098
REVIEWED BY: TFD



GRAIN SIZE (%)
▲ Passing #200 sieve
△ Passing #4 sieve

SOIL HEADSPACE READING (ppm)
GASTECH reading
PID reading

ELEVATION (m): 3, 2, 1, 0, -1, -2, -3, -4, -5

SOILS DESCRIPTION

ASPHALT (50 mm thick).

Light brown, dry SAND, some silt to silty, some gravel, trace geotextile fragments.

Dark grey, moist SAND, some silt to silty, trace gravel.

- red-brown mottling, wet below 1.3 m depth

Dark grey, brown mottling, wet SAND, gravelly to some gravel.

Compact, dark grey, wet SAND, trace to some silt.

- loose below 5.5 m depth

Dark to light red-brown, moist PEAT, trace of wood debris.

End of hole at required depth.
Hole open to 1.4 m depth.
Water observed at 1.2 m depth upon completion of drilling.
Test hole backfilled, sand and bentonite chips as required by the Port of Vancouver project permit.

Sheet 1 of 2

LOG OF TEST HOLE

TEST HOLE NO.
TH22-03

LOCATION: See DWG. 34098-1
N 5448607.02, E 506710.96

TOP OF HOLE ELEV: 4.21 m

METHOD: Solid Stem Auger

DRILLING CO.: Earth Drilling Co. Ltd.

INSPECTOR: DA

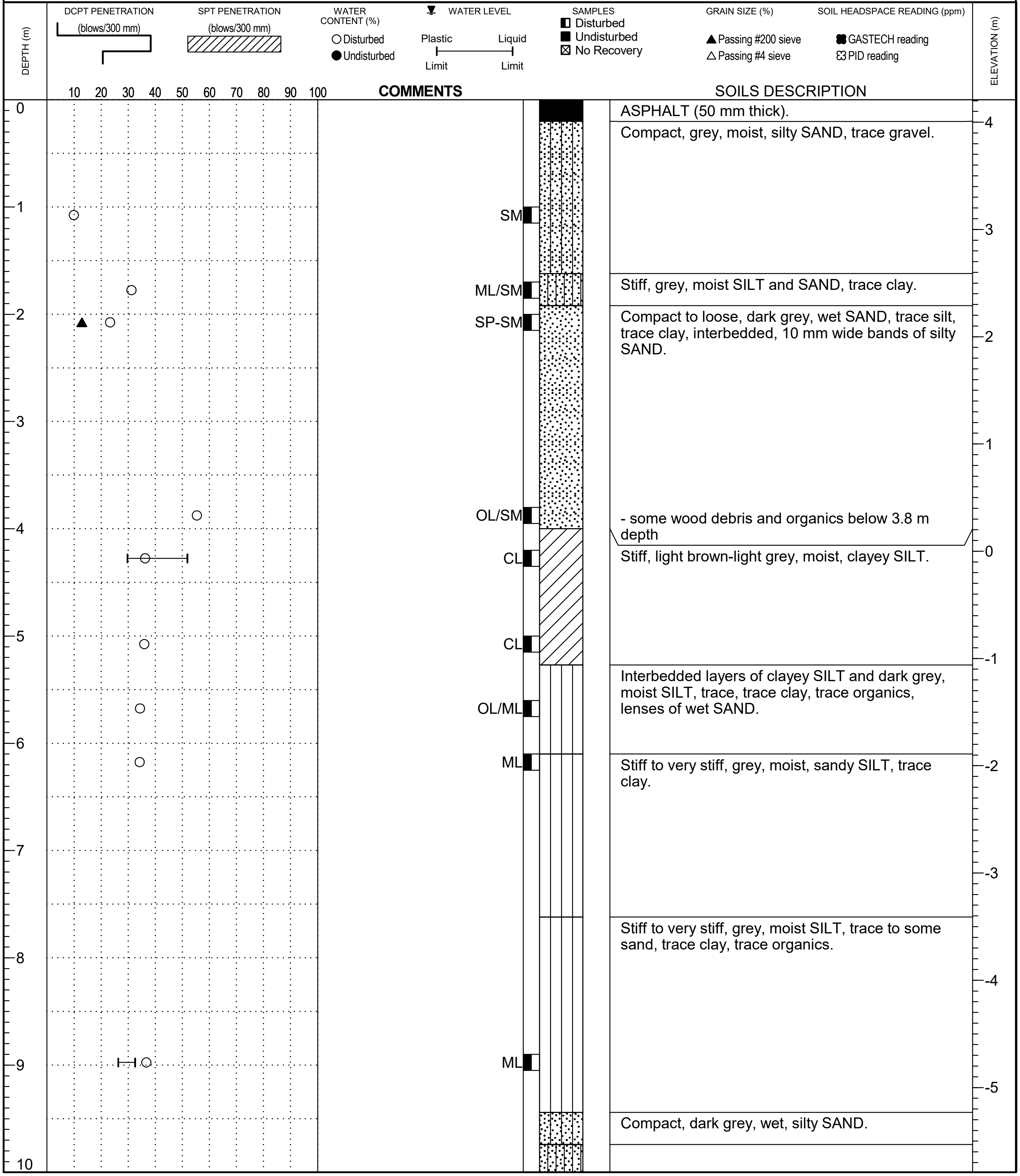
CLIENT: Sacré - Davey Engineering Inc.

PROJECT: DP World Fraser Surrey
Canola Oil Transload Facility

DATE: April 6, 2022

FILE NO.: 34098

REVIEWED BY: TFD



ASPHALT (50 mm thick).

Compact, grey, moist, silty SAND, trace gravel.

Stiff, grey, moist SILT and SAND, trace clay.

Compact to loose, dark grey, wet SAND, trace silt, trace clay, interbedded, 10 mm wide bands of silty SAND.

- some wood debris and organics below 3.8 m depth

Stiff, light brown-light grey, moist, clayey SILT.

Interbedded layers of clayey SILT and dark grey, moist SILT, trace, trace clay, trace organics, lenses of wet SAND.

Stiff to very stiff, grey, moist, sandy SILT, trace clay.

Stiff to very stiff, grey, moist SILT, trace to some sand, trace clay, trace organics.

Compact, dark grey, wet, silty SAND.

Sheet 2 of 2

LOG OF TEST HOLE

TEST HOLE NO. **TH22-03**

LOCATION: See DWG. 34098-1
N 5448607.02, E 506710.96

TOP OF HOLE ELEV: 4.21 m

METHOD: Solid Stem Auger

DRILLING CO.: Earth Drilling Co. Ltd.

INSPECTOR: DA

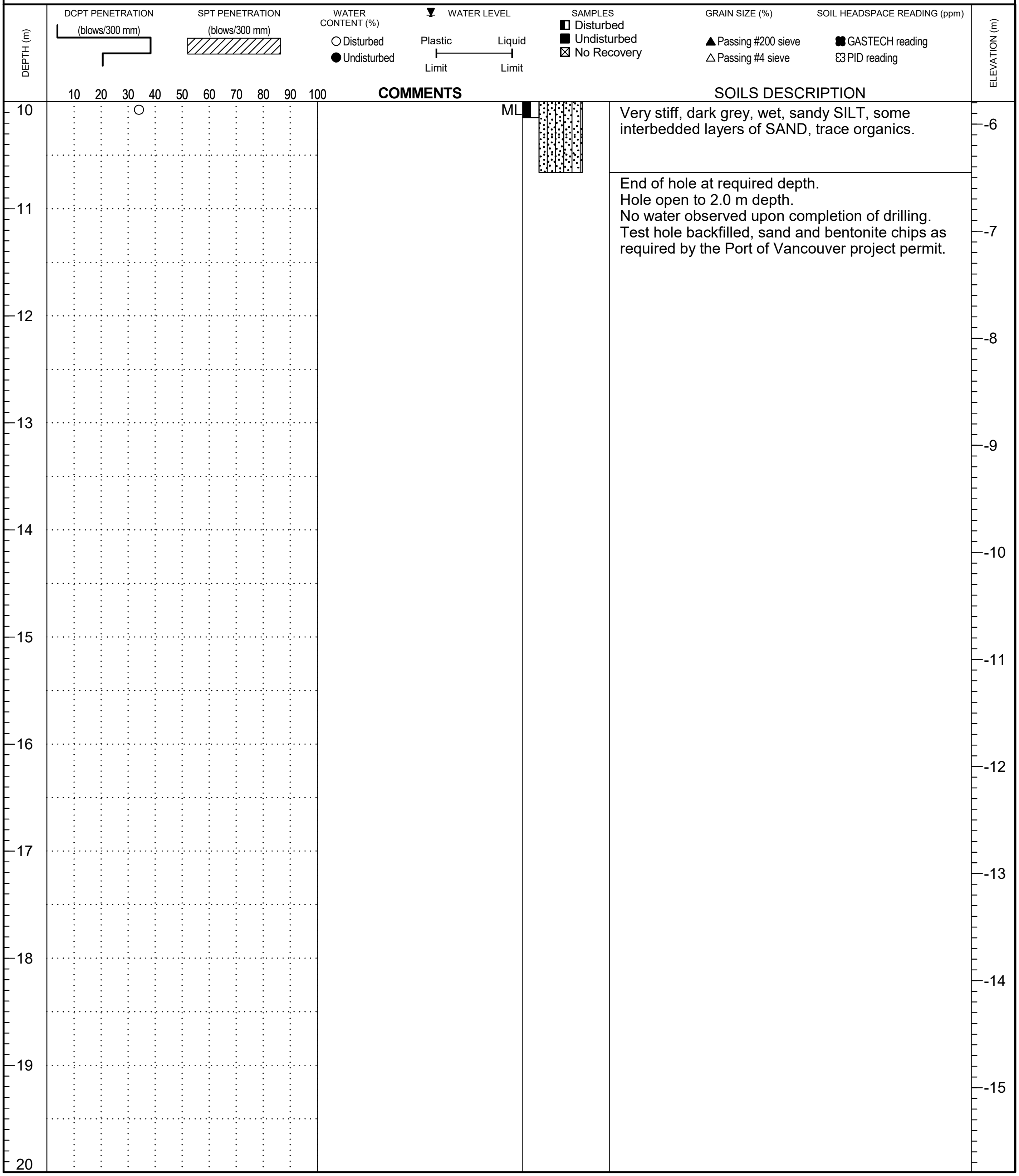
CLIENT: Sacré - Davey Engineering Inc.

PROJECT: DP World Fraser Surrey
Canola Oil Transload Facility

DATE: April 6, 2022

FILE NO.: 34098

REVIEWED BY: TFD



Very stiff, dark grey, wet, sandy SILT, some interbedded layers of SAND, trace organics.

End of hole at required depth.
Hole open to 2.0 m depth.
No water observed upon completion of drilling.
Test hole backfilled, sand and bentonite chips as required by the Port of Vancouver project permit.

Sheet 1 of 1

LOG OF TEST HOLE

TEST HOLE NO.
TH22-04

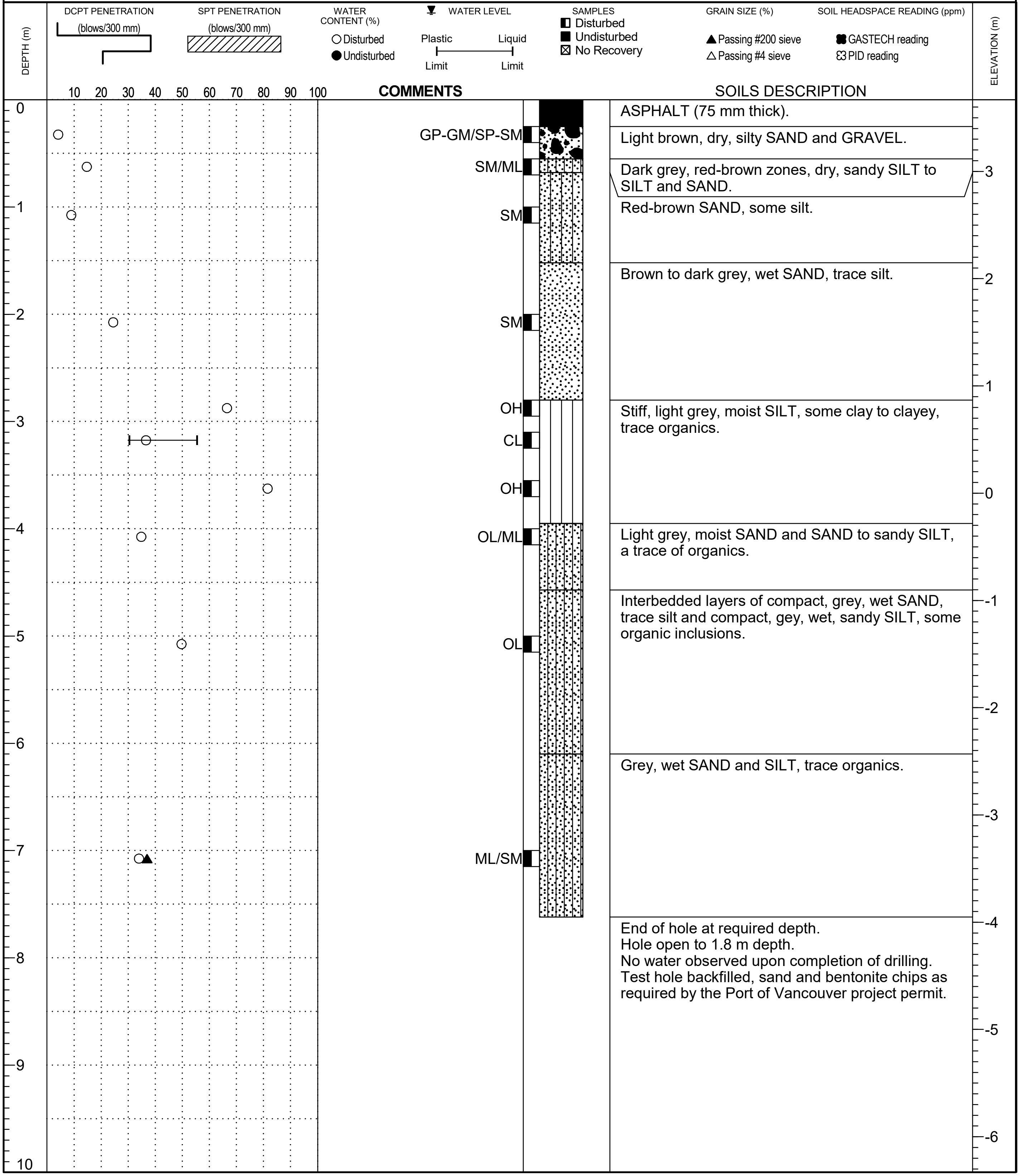
LOCATION: See DWG. 34098-1
N 5448680.55, E 506616.57

TOP OF HOLE ELEV: 3.67 m
METHOD: Solid Stem Auger
DRILLING CO.: Earth Drilling Co. Ltd.
INSPECTOR: DA

THURBER

CLIENT: Sacré - Davey Engineering Inc.
PROJECT: DP World Fraser Surrey
Canola Oil Transload Facility

DATE: April 6, 2022
FILE NO.: 34098
REVIEWED BY: TFD



ASPHALT (75 mm thick).

Light brown, dry, silty SAND and GRAVEL.

Dark grey, red-brown zones, dry, sandy SILT to SILT and SAND.

Red-brown SAND, some silt.

Brown to dark grey, wet SAND, trace silt.

Stiff, light grey, moist SILT, some clay to clayey, trace organics.

Light grey, moist SAND and SAND to sandy SILT, a trace of organics.

Interbedded layers of compact, grey, wet SAND, trace silt and compact, gey, wet, sandy SILT, some organic inclusions.

Grey, wet SAND and SILT, trace organics.

End of hole at required depth.
Hole open to 1.8 m depth.
No water observed upon completion of drilling.
Test hole backfilled, sand and bentonite chips as required by the Port of Vancouver project permit.

Sheet 1 of 1

LOG OF TEST HOLE

TEST HOLE NO.
TH22-05

LOCATION: See DWG. 34098-1
N 5448530.59, E 506828.82

TOP OF HOLE ELEV: 3.65 m

METHOD: Solid Stem Auger

DRILLING CO.: Earth Drilling Co. Ltd.

INSPECTOR: DA

THURBER

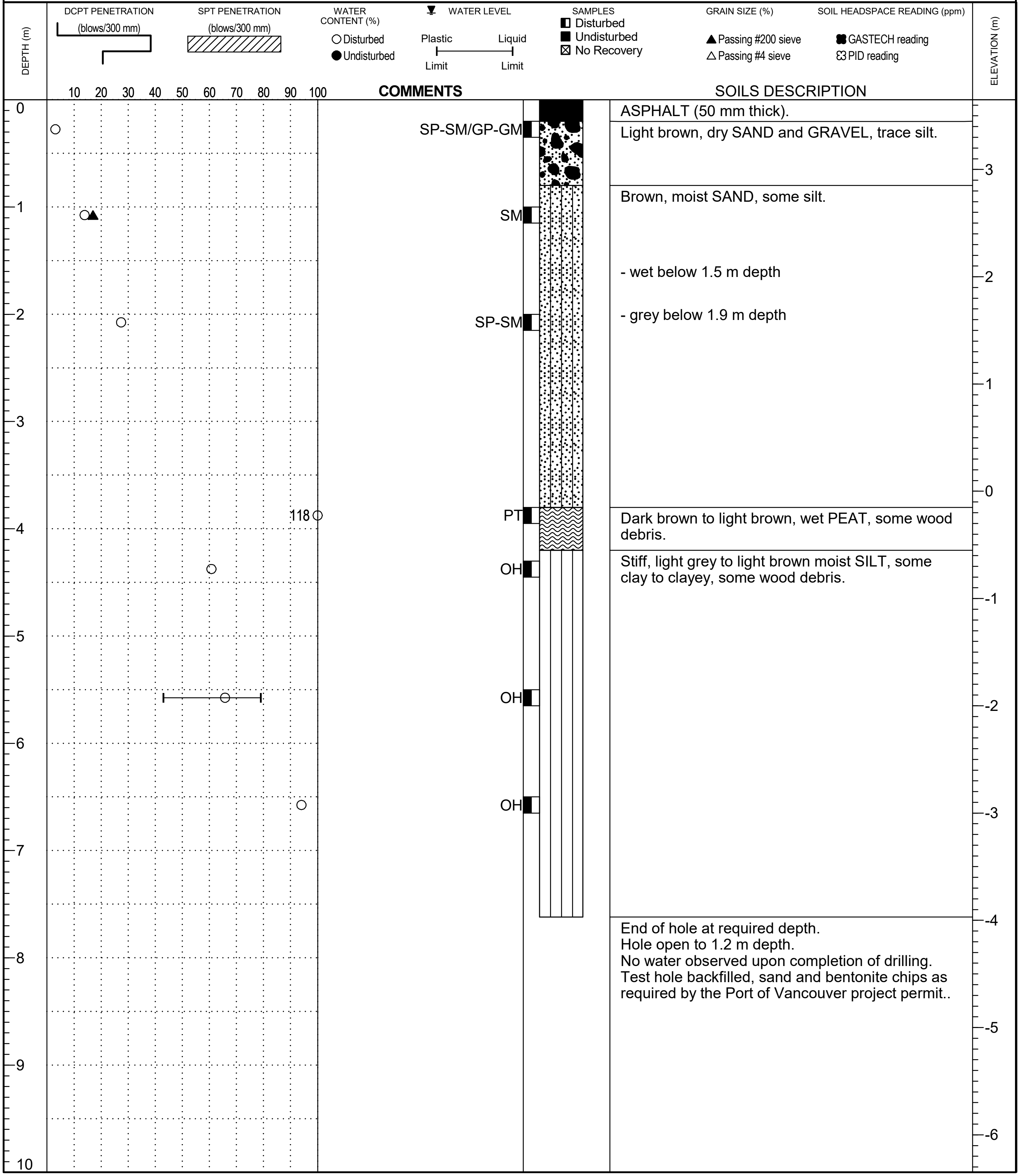
CLIENT: Sacré - Davey Engineering Inc.

PROJECT: DP World Fraser Surrey
Canola Oil Transload Facility

DATE: April 6, 2022

FILE NO.: 34098

REVIEWED BY: TFD



ASPHALT (50 mm thick).

Light brown, dry SAND and GRAVEL, trace silt.

Brown, moist SAND, some silt.

- wet below 1.5 m depth

- grey below 1.9 m depth

Dark brown to light brown, wet PEAT, some wood debris.

Stiff, light grey to light brown moist SILT, some clay to clayey, some wood debris.

End of hole at required depth.
Hole open to 1.2 m depth.
No water observed upon completion of drilling.
Test hole backfilled, sand and bentonite chips as required by the Port of Vancouver project permit..

THURBER

APPENDIX C

CPT Report (23 pages)

CONE PENETRATION TEST REPORT

Prepared for:

THURBER ENGINEERING LTD.

Site: DP World Fraser Surrey Docks – Canola Oil Transload Facility
Surrey, BC

Date Drilled: April 5th – 6th, 2022

Prepared by:

On Track Drilling
20626 Mufford Crescent
Langley, BC
V2Y 1N8

Phone: 604-523-1200
zach@ontrackdrilling.com
www.ontrackdrilling.com

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Cone Penetration Testing (CPT) Equipment & Calculated Geotechnical Parameters

On Track Drilling Inc. owns and operates a cone penetration test (CPT) system, supplied by Vertek – A Division of Applied Research and Associates. The Hogentogler electronic system is used with a 10 cm^2, 10 ton cone that records tip resistance, sleeve friction, pore pressure, inclination and temperature at desired intervals chosen by the operator. The cone penetrometers are designed with equal end area friction sleeves, a net end area ratio 0.8 and 60° apex angle on the tip. The cone consists of two strain gauge transducers, with the cone electronics packaged directly behind the transducers. The cone can be stopped at desired depths and dissipation tests can be completed to determine the groundwater pressures.

All testing is performed in accordance with the current ASTM D5778 standards.

The CPT calculations displayed on the plots are based on the measured tip resistance, sleeve friction and pore water pressure recorded at each specified data point. The recorded tip resistance (qc) is corrected for pore pressure effects (qt) and is used for all the calculations.

The following empirical correlations have been used to calculate the geotechnical parameters used in the CPT plots:

Corrected cone tip resistance:

$$q_t = q_c + (1\text{-}a) \bullet u2$$

where: q_c = the recorded tip resistance
a = net area ratio for cone (0.8)
u2 = the recorded dynamic pore pressure

Soil Behavior Type (Normalized): based on SBTn Robertson (1990) (Linear normalization)

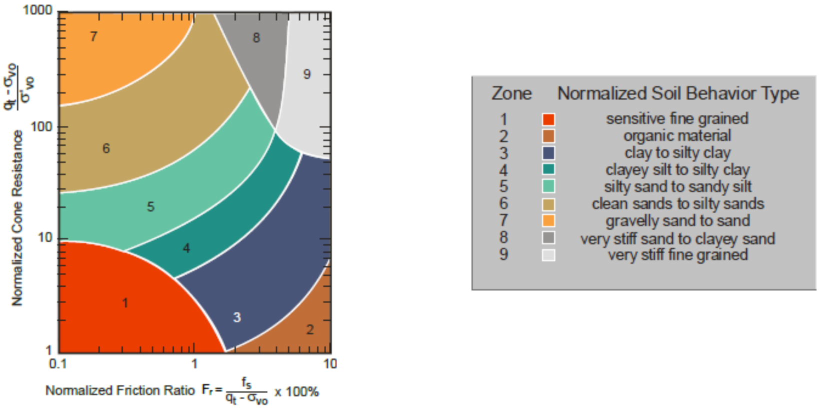

Figure 1: Normalized Soil Behavior Type (SBTn) Classification Chart

Undrained Shear Strength (Su):

$$Su = \frac{(q_t - \sigma_v)}{N_{kt}}$$

where: q_t = the corrected tip resistance
σ_v = the effective overburden stress
N_{kt} = cone constant (user selectable)

Standard Penetration Test Correlation $N_{1(60)}$:

$$(N_1)_{60} = C_n N_{60}$$

The SPT N_{60} value corrected for overburden pressure (C_n)

Equivalent SPT N_{60}, (blows/30cm) Lunne et al. (1997) :

$$\frac{\left(\frac{q_t}{p_a}\right)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$$

Over Consolidation Ratio (OCR):

$$OCR = k_{OCR}\ Q_{t1}$$

Only SBTn 1, 2, 3, 4, & 9 (see Lunne et al., 1997)

Shear Wave Velocity (Vs) Testing:

Shear wave velocity measurements can be recorded at desired intervals in conjunction with the cone penetrometer test. The shear waves are typically generated by using a heavy hammer to horizontally strike a beam that is held in place on the ground by a normal force, in this case the outriggers of the drill rig. Two accelerometers mounted directly to the source are used as the contact triggers to initiate the recording of the seismic wave traces. The seismic source is oriented parallel to the axis of the active geophone being used.

The geophones are located 0.2 meters behind the cone tip and the source offset to the cone is recorded for each test.

The velocities of each interval are calculated by choosing a first arrival feature of each recorded wave set and taking the difference in ray path, divided by the time difference between subsequent first arrival times.

All testing is performed in accordance with the current ASTM D7400 standards.

All calculations have been carried out automatically using the software program CPeT-IT v.3.0.3.2. supplied by Geologismiki. The parameters selected are based on current published CPT correlations and are subject to change to reflect the current state of practice. On Track Drilling does not warrant the correctness or the applicability of any of the calculations carried out by the software and does not assume liability for the use of the data in any design or review.

References:

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-12.

ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM International, West Conshohocken, PA. DOI: 10.1520/D7400_D7400M-19.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “Cone Penetration Testing in Geotechnical Practice”, Blackie Academic and Professional.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158. DOI: 10.1139/T90-014.

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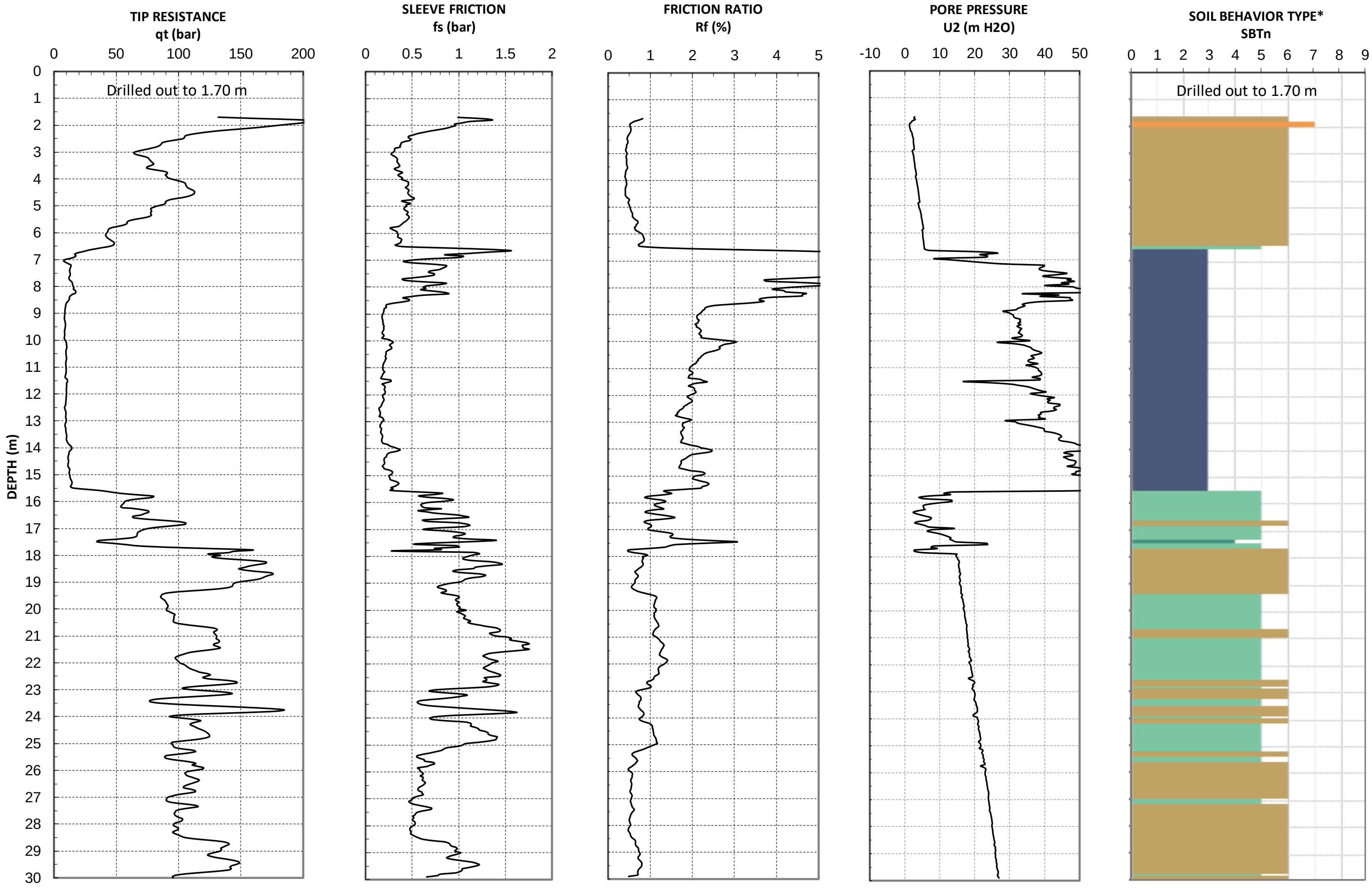
Sounding: SCPT22-01	Client: Thurber Engineering Ltd.
05-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC



* Based on Robertson et. al 1990

- 🟥 1. Sensitive Fine Grained
- 🟫 2. Organic Material
- 🟦 3. Clay to Silty Clay
- 🟦 4. Clayey Silt to Silty Clay
- 🟩 5. Silty Sand to Sandy Silt
- 🟨 6. Clean Sand to Silty Sand
- 🟧 7. Gravely Sand to Sand
- ⬛ 8. Very Stiff Sand to Clayey Sand
- ⬜ 9. Very Stiff Fine Grained

Depth Increment: 0.05 m
Geodetic Elevation: N/A
Maximum Depth: 30.00 m

Cone ID: DDG1521
Operator: ZH

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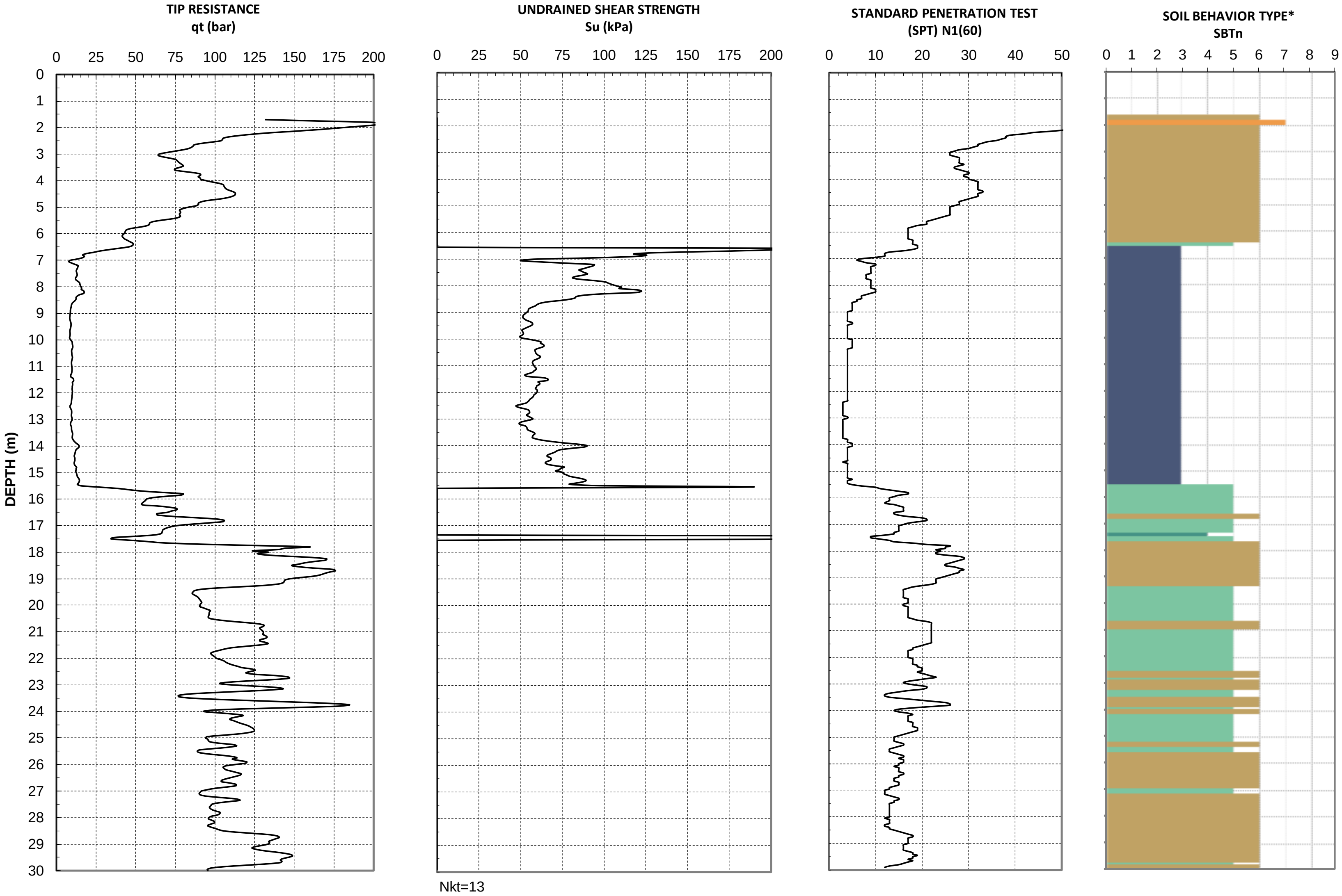
Sounding: SCPT22-01	Client: Thurber Engineering Ltd.
05-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC



* Based on Robertson et. al 1990

- 🟥 1. Sensitive Fine Grained
- 🟫 2. Organic Material
- 🟦 3. Clay to Silty Clay
- 🟩 4. Clayey Silt to Silty Clay
- 🟩 5. Silty Sand to Sandy Silt
- 🟨 6. Clean Sand to Silty Sand
- 🟧 7. Gravely Sand to Sand
- ⬛ 8. Very Stiff Sand to Clayey Sand
- ⬜ 9. Very Stiff Fine Grained

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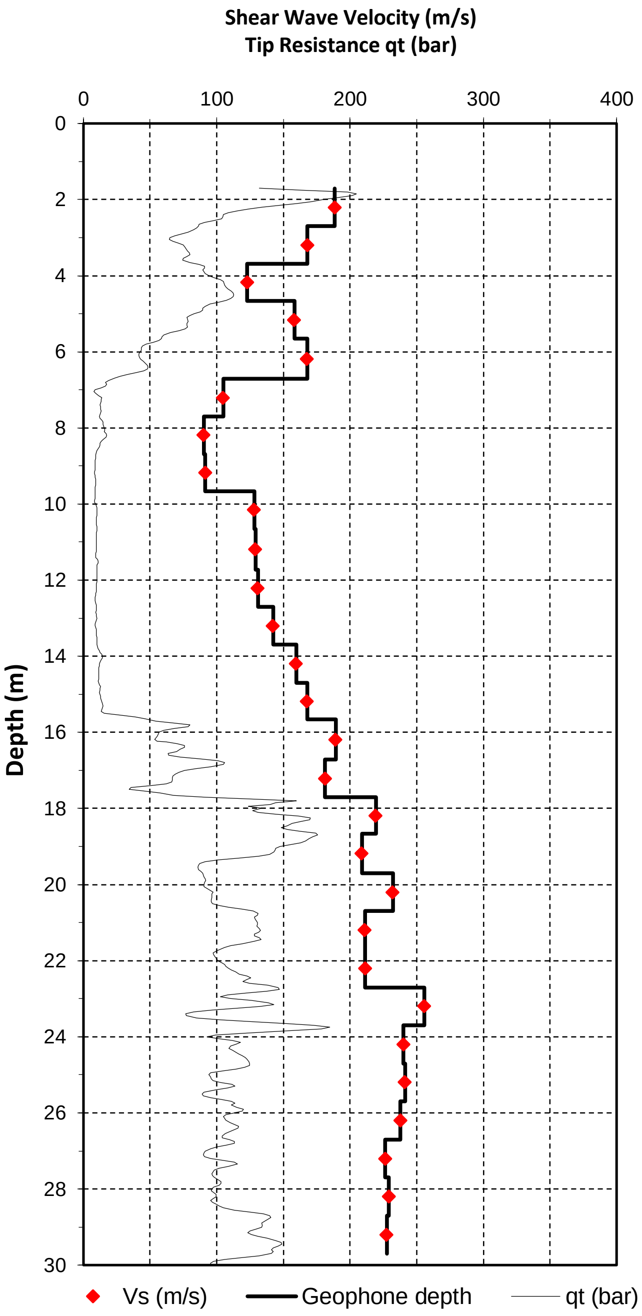
Sounding: SCPT22-01	Client: Thurber Engineering Ltd.
05-Apr-22	Site: DP World - Fraser Surrey Docks, Surrey, BC

Seismic Source: Beam
Source to cone (m): 1.2

Geodetic Elevation: N/A
Cone ID: DDG1521
Operator: ZH

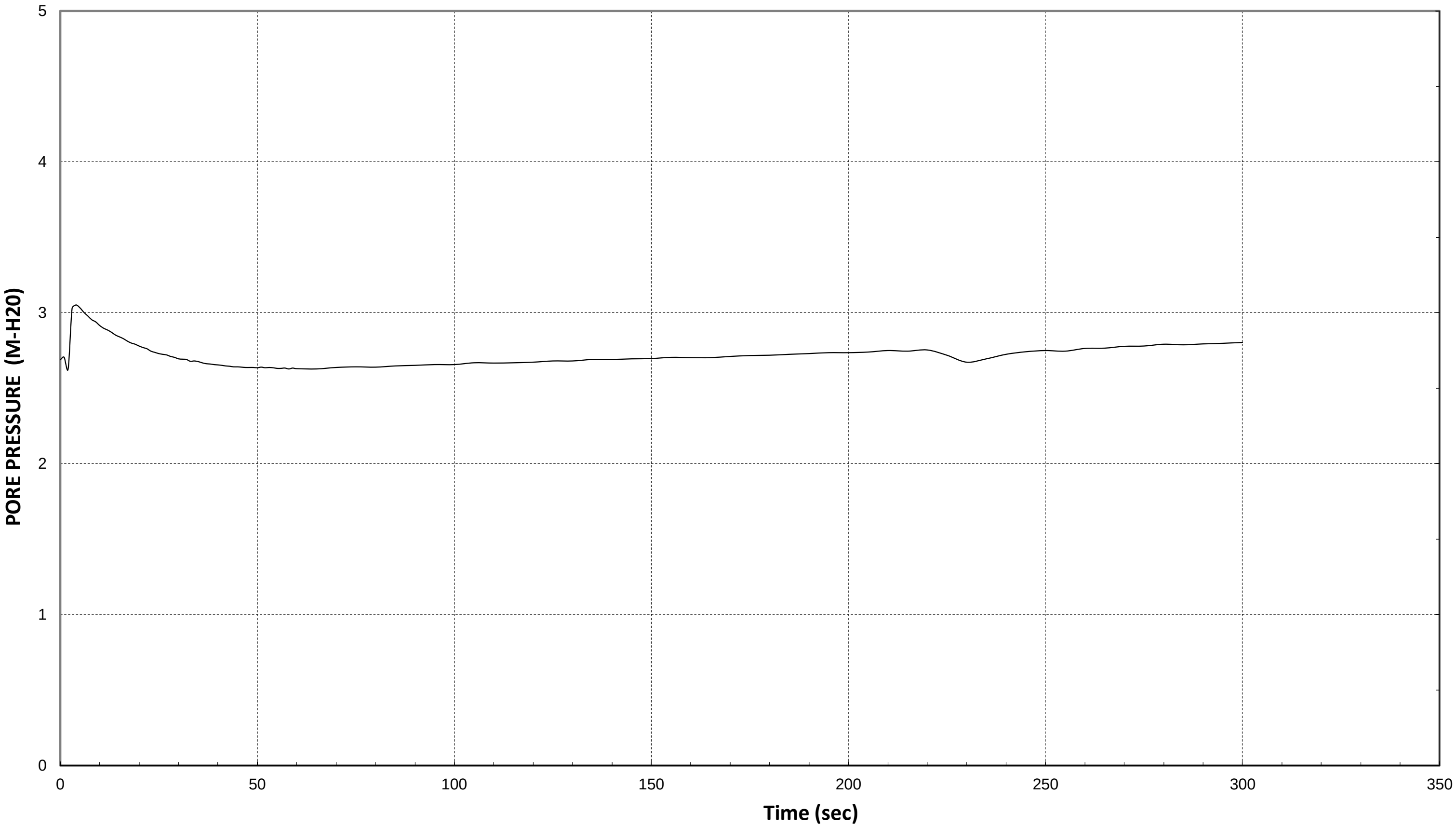
Shear Wave Velocity Data (Vs)

Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Time Difference (ms)	Shear Wave Velocity Vs (m/s)
1.91	1.71	2.09			
2.90	2.70	2.95	0.87	4.59	**189**
3.89	3.69	3.88	0.93	5.51	**168**
4.87	4.67	4.82	0.94	7.66	**123**
5.86	5.66	5.79	0.96	6.10	**158**
6.91	6.71	6.82	1.03	6.15	**168**
7.90	7.70	7.79	0.98	9.32	**105**
8.89	8.69	8.77	0.98	10.84	**90**
9.87	9.67	9.74	0.97	10.62	**91**
10.85	10.65	10.72	0.97	7.60	**128**
11.92	11.72	11.78	1.06	8.25	**129**
12.91	12.71	12.77	0.99	7.53	**131**
13.90	13.70	13.75	0.99	6.94	**142**
14.90	14.70	14.75	1.00	6.25	**160**
15.86	15.66	15.71	0.96	5.71	**168**
16.92	16.72	16.76	1.06	5.59	**189**
17.90	17.70	17.74	0.98	5.40	**181**
18.87	18.67	18.71	0.97	4.41	**219**
19.90	19.70	19.74	1.03	4.92	**209**
20.90	20.70	20.73	1.00	4.30	**232**
21.90	21.70	21.73	1.00	4.73	**211**
22.90	22.70	22.73	1.00	4.72	**211**
23.90	23.70	23.73	1.00	3.91	**256**
24.90	24.70	24.73	1.00	4.16	**240**
25.90	25.70	25.73	1.00	4.14	**241**
26.90	26.70	26.73	1.00	4.20	**238**
27.90	27.70	27.73	1.00	4.41	**227**
28.90	28.70	28.73	1.00	4.36	**229**
29.90	29.70	29.72	1.00	4.39	**228**


Shear Wave Velocity (m/s)
Tip Resistance qt (bar)
0
100
200
300
400
Depth (m)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Vs (m/s)
Geophone depth
qt (bar)

Sounding: SCPT22-01	Client: Thurber Engineering Ltd.
05-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC

PORE PRESSURE DISSIPATION TEST

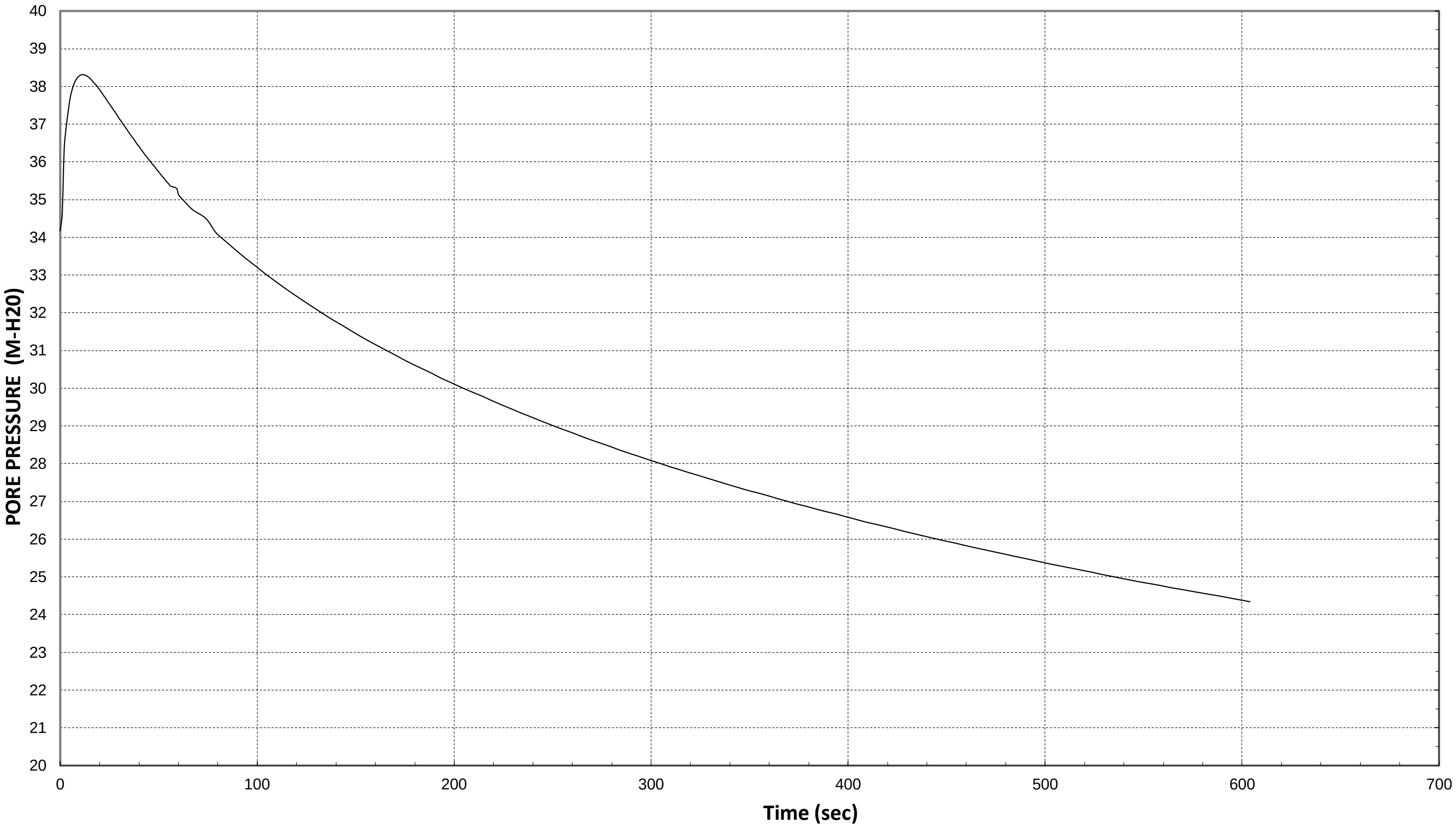


Test Depth:	5.00 m
Duration:	300 s
Cone Area:	10 cm^2
Cone ID:	DDG1521

u Min:	2.63 m
u Max:	3.05 m
u Final:	2.80 m

Sounding: SCPT22-01	Client: Thurber Engineering Ltd.
05-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC

PORE PRESSURE DISSIPATION TEST

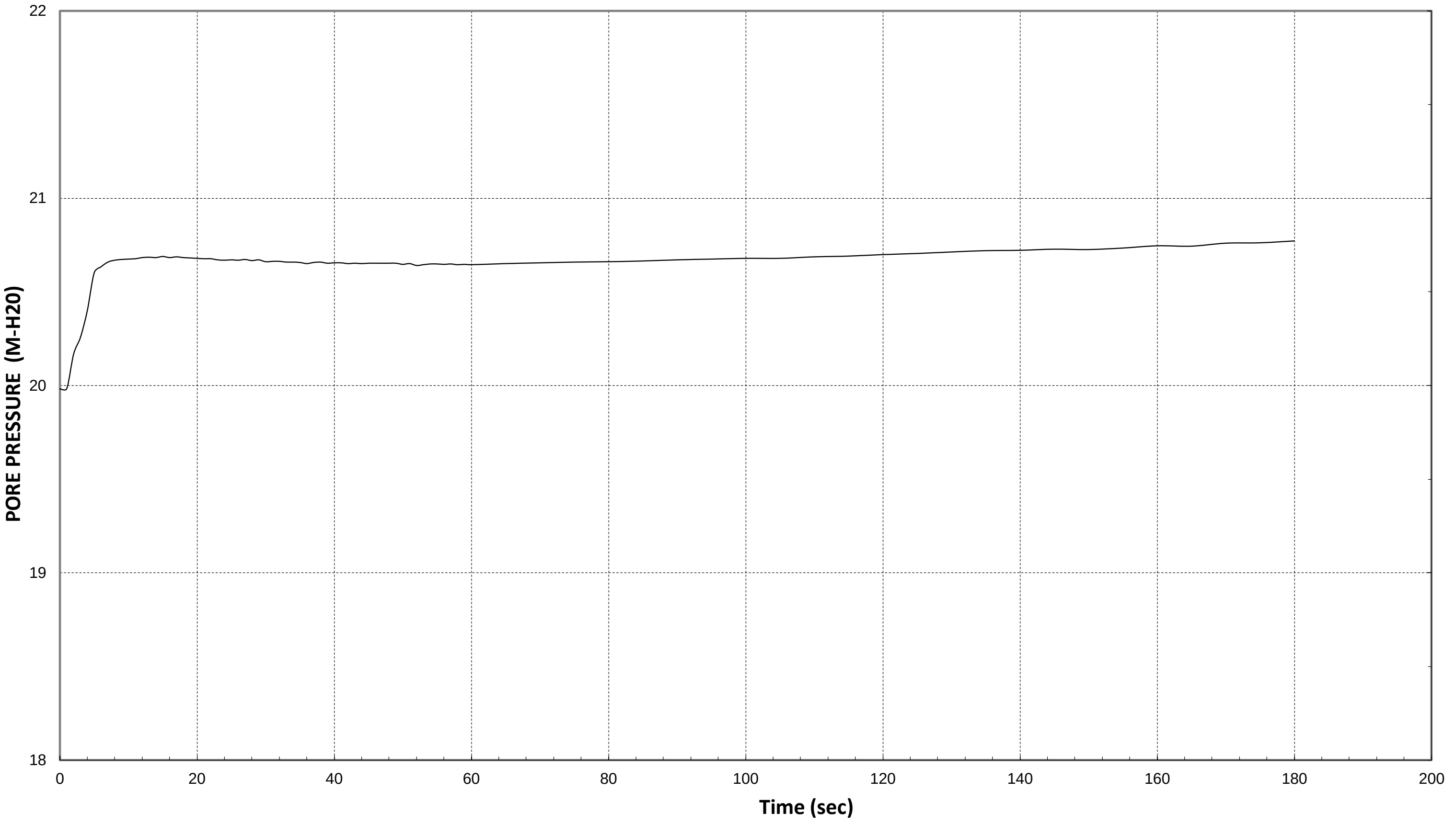


Test Depth:	10.00 m
Duration:	604 s
Cone Area:	10 cm^2
Cone ID:	DDG1521

u Min:	24.34 m
u Max:	38.31 m
u Final:	24.34 m

Sounding: SCPT22-01	Client: Thurber Engineering Ltd.
05-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC

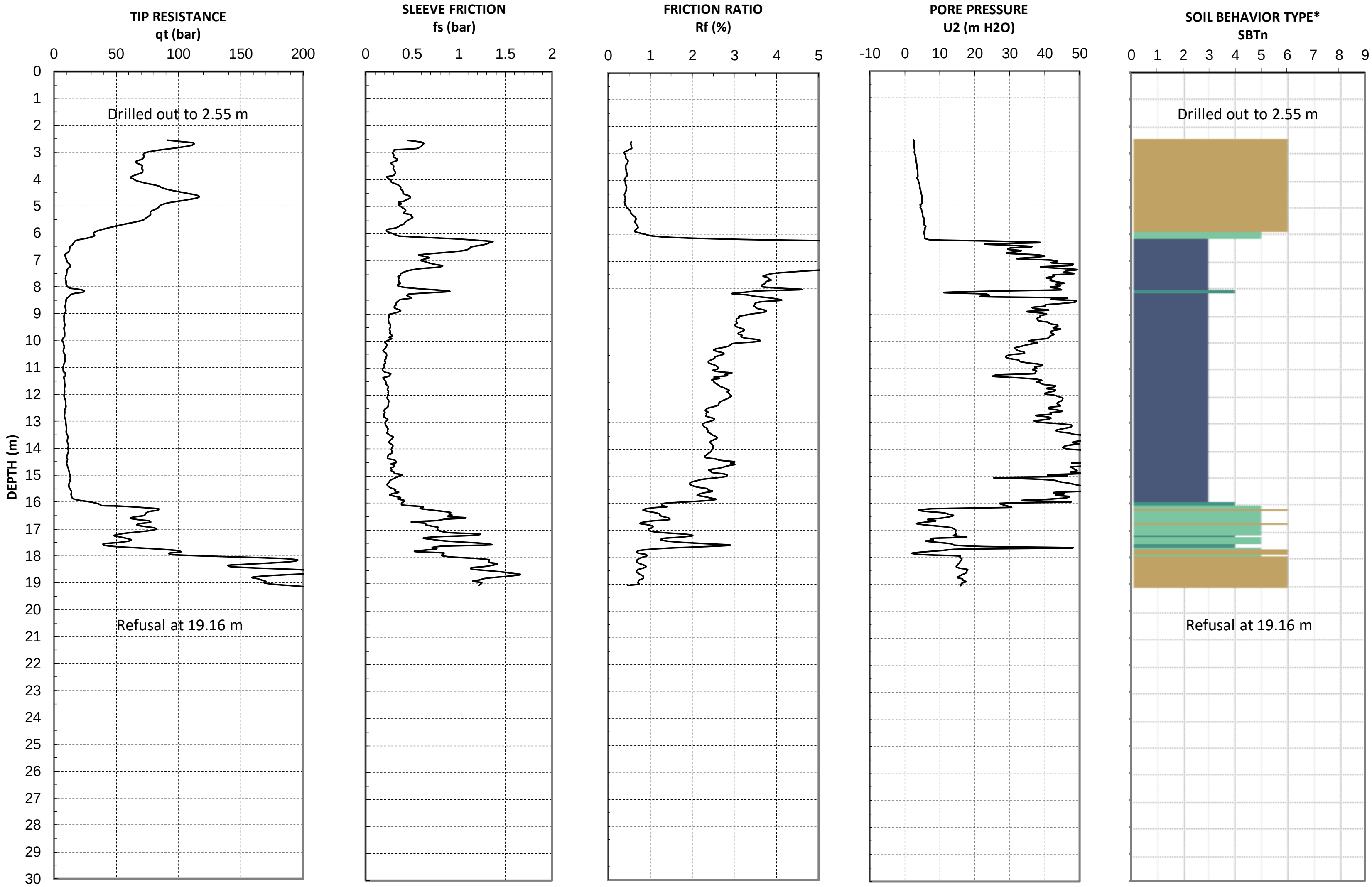
PORE PRESSURE DISSIPATION TEST



Test Depth:	25.10 m
Duration:	180 s
Cone Area:	10 cm^2
Cone ID:	DDG1521

u Min:	20.77 m
u Max:	38.31 m
u Final:	20.77 m

Sounding: SCPT22-02	Client: Thurber Engineering Ltd.
05-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC



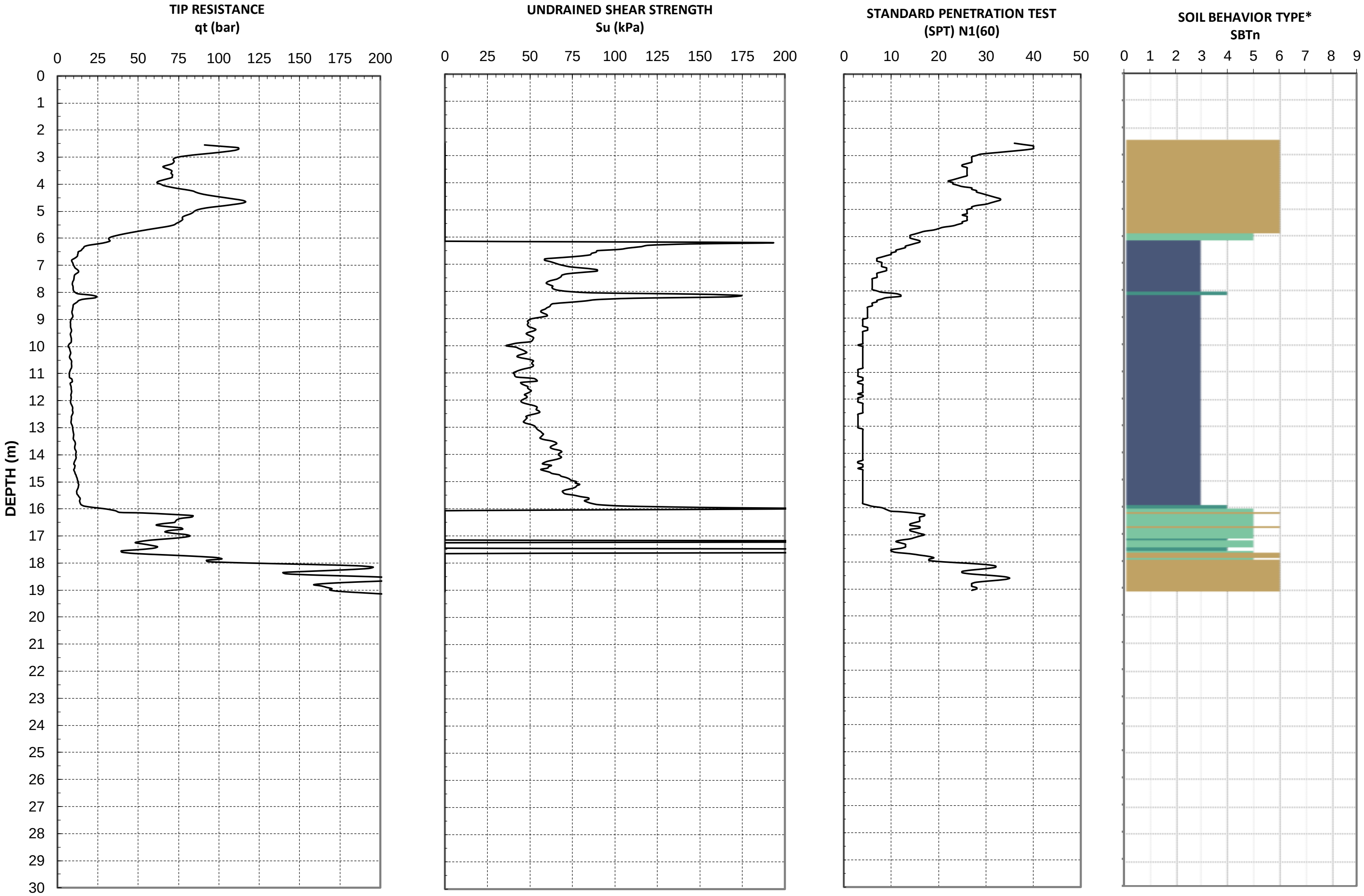
* Based on Robertson et. al 1990

- 🟥 1. Sensitive Fine Grained
- 🟫 2. Organic Material
- 🟦 3. Clay to Silty Clay
- 🟦 4. Clayey Silt to Silty Clay
- 🟩 5. Silty Sand to Sandy Silt
- 🟨 6. Clean Sand to Silty Sand
- 🟧 7. Gravely Sand to Sand
- ⬛ 8. Very Stiff Sand to Clayey Sand
- ⬜ 9. Very Stiff Fine Grained

Depth Increment: 0.05 m
Geodetic Elevation: N/A
Maximum Depth: 19.16 m

Cone ID: DDG1521
Operator: ZH

Sounding: SCPT22-02	Client: Thurber Engineering Ltd.
05-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC



Nkt=13

* Based on Robertson et. al 1990

🟥 1. Sensitive Fine Grained
🟫 2. Organic Material
🟦 3. Clay to Silty Clay
🟦 4. Clayey Silt to Silty Clay
🟩 5. Silty Sand to Sandy Silt
🟨 6. Clean Sand to Silty Sand
🟧 7. Gravely Sand to Sand
⬛ 8. Very Stiff Sand to Clayey Sand
⬜ 9. Very Stiff Fine Grained

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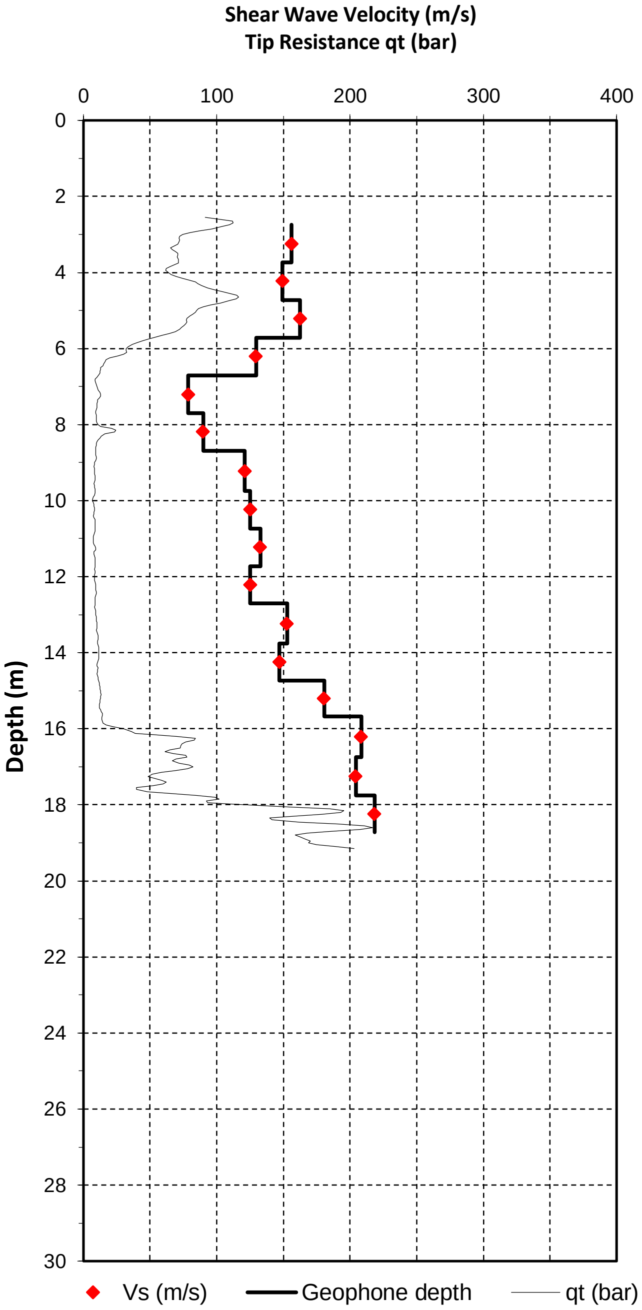
Sounding: SCPT22-02	Client: Thurber Engineering Ltd.
05-Apr-22	Site: DP World - Fraser Surrey Docks, Surrey, BC

Seismic Source: Beam
Source to cone (m): 1.2

Geodetic Elevation: N/A
Cone ID: DDG1521
Operator: ZH

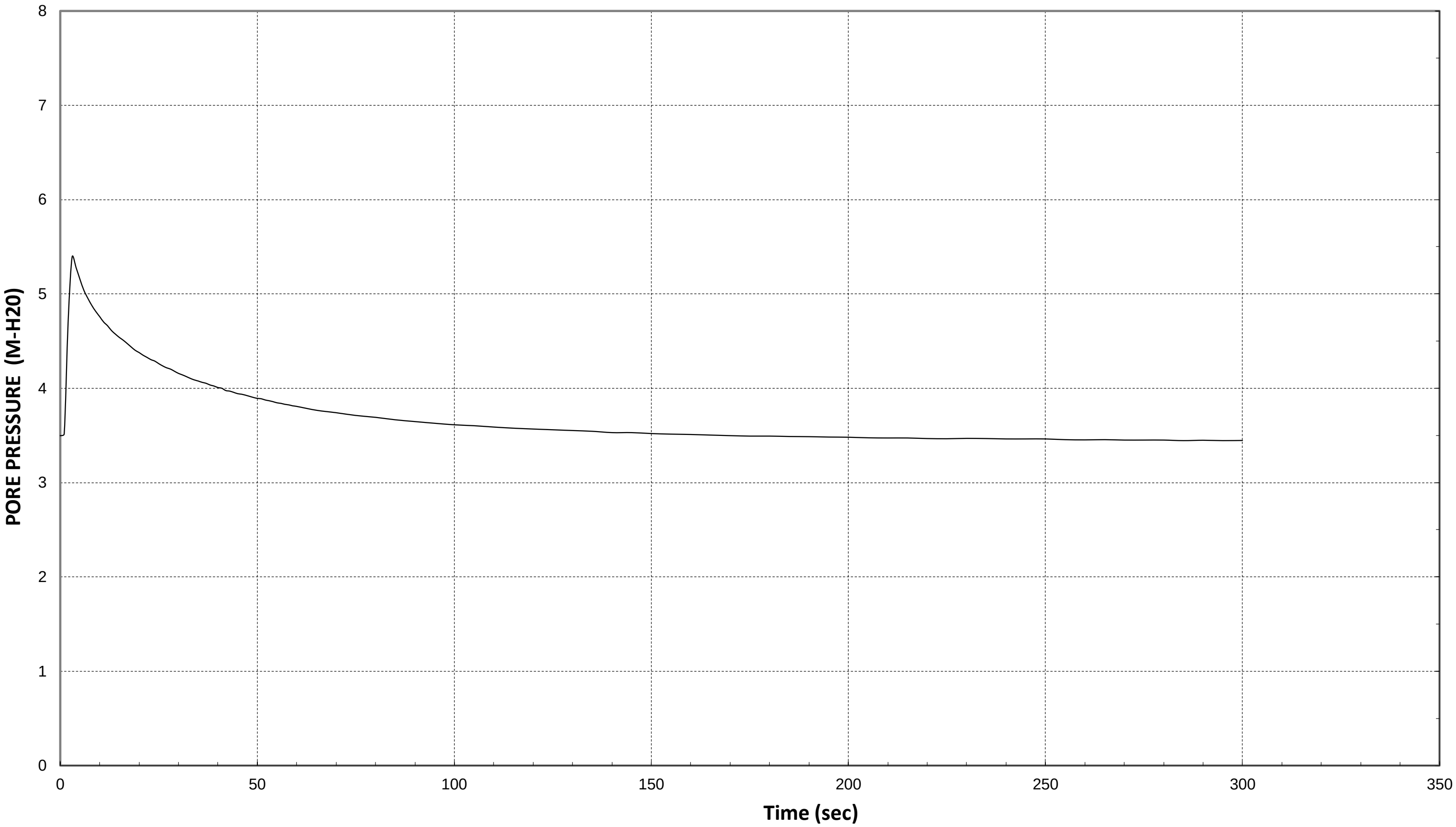
Shear Wave Velocity Data (Vs)

Depth (m)	**Geophone Depth (m)**	**Ray Path (m)**	**Ray Path Difference (m)**	**Time Difference (ms)**	**Shear Wave Velocity Vs (m/s)**
2.95	2.75	3.00			
3.94	3.74	3.93	0.93	5.94	**156**
4.92	4.72	4.87	0.94	6.31	**149**
5.91	5.71	5.83	0.96	5.94	**163**
6.91	6.71	6.82	0.98	7.58	**130**
7.90	7.70	7.79	0.98	12.41	**79**
8.89	8.69	8.77	0.98	10.90	**90**
9.95	9.75	9.82	1.05	8.69	**121**
10.93	10.73	10.80	0.97	7.78	**125**
11.92	11.72	11.78	0.98	7.42	**133**
12.91	12.71	12.77	0.99	7.87	**125**
13.95	13.75	13.80	1.04	6.77	**153**
14.94	14.74	14.79	0.99	6.71	**147**
15.88	15.68	15.73	0.94	5.19	**181**
16.95	16.75	16.79	1.07	5.12	**208**
17.95	17.75	17.79	1.00	4.88	**204**
18.92	18.72	18.76	0.97	4.43	**218**


Shear Wave Velocity (m/s)
Tip Resistance qt (bar)
0
100
200
300
400
Depth (m)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Vs (m/s)
Geophone depth
qt (bar)

Sounding: SCPT22-02	Client: Thurber Engineering Ltd.
05-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC

PORE PRESSURE DISSIPATION TEST

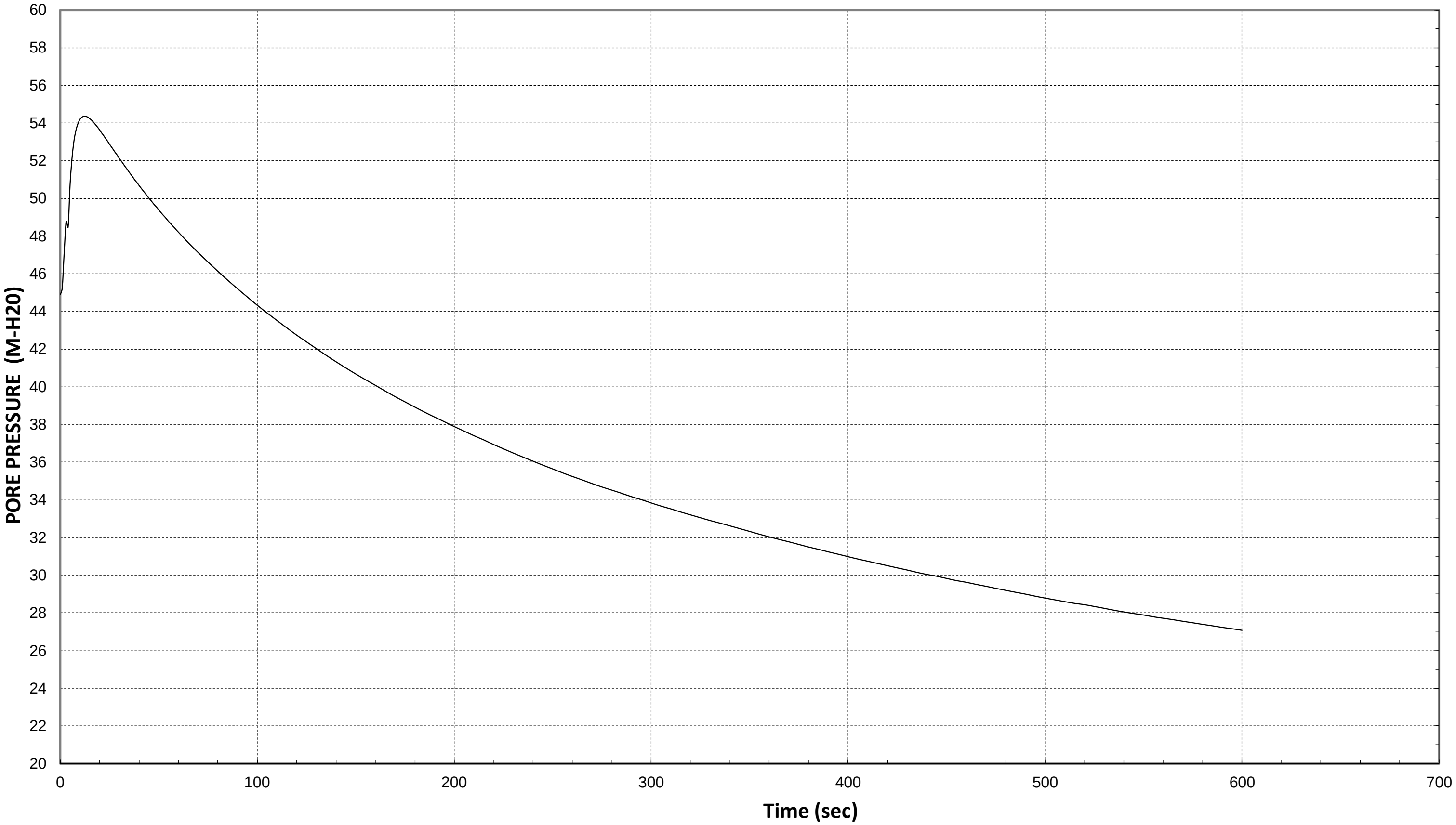


Test Depth:	5.00 m
Duration:	300 s
Cone Area:	10 cm^2
Cone ID:	DDG1521

u Min:	3.45 m
u Max:	5.39 m
u Final:	3.45 m

Sounding: SCPT22-02	Client: Thurber Engineering Ltd.
05-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC

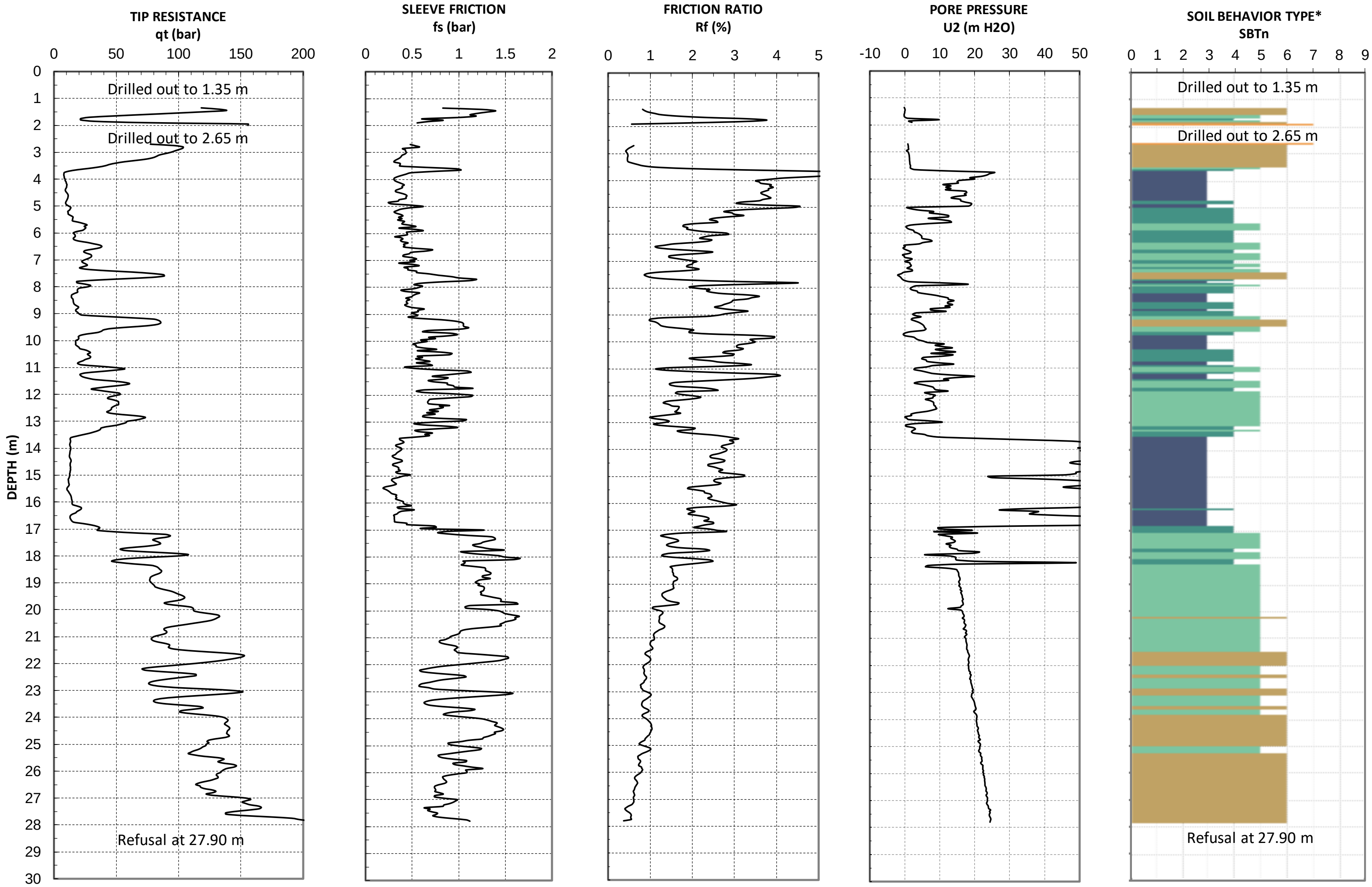
PORE PRESSURE DISSIPATION TEST



Test Depth:	15.00 m
Duration:	600 s
Cone Area:	10 cm^2
Cone ID:	DDG1521

u Min:	27.08 m
u Max:	54.36 m
u Final:	27.08 m

Sounding: SCPT22-03	Client: Thurber Engineering Ltd.
06-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC



* Based on Robertson et. al 1990

- 🟥 1. Sensitive Fine Grained
- 🟫 2. Organic Material
- 🟦 3. Clay to Silty Clay
- 🟩 4. Clayey Silt to Silty Clay
- 🟩 5. Silty Sand to Sandy Silt
- 🟨 6. Clean Sand to Silty Sand
- 🟧 7. Gravely Sand to Sand
- ⬛ 8. Very Stiff Sand to Clayey Sand
- ⬜ 9. Very Stiff Fine Grained

Depth Increment: 0.05 m
Geodetic Elevation: N/A
Maximum Depth: 27.93 m

Cone ID: DDG1521
Operator: ZH

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An earth DRILLING Company
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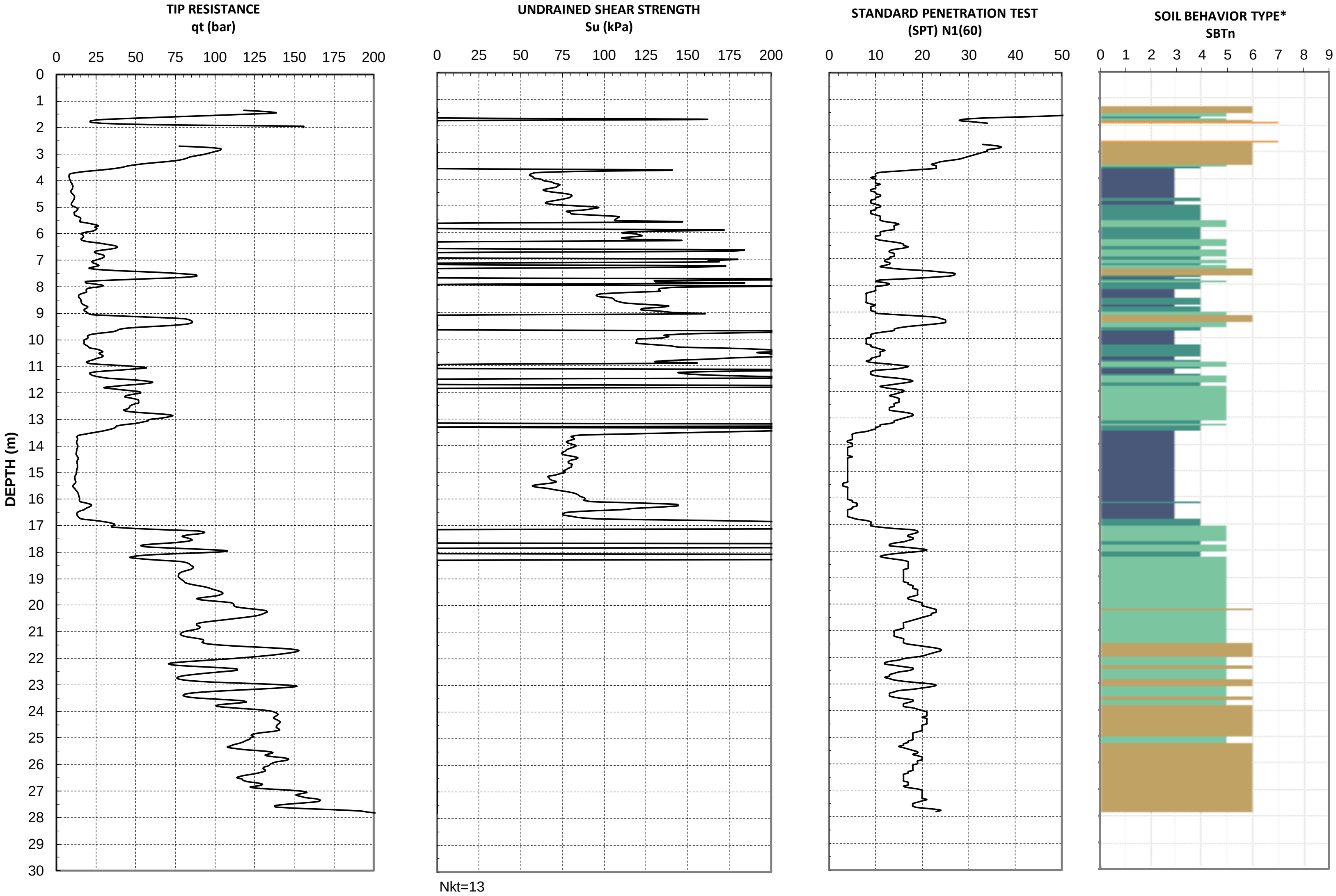
Sounding: SCPT22-03	Client: Thurber Engineering Ltd.
06-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC



* Based on Robertson et. al 1990

🟥 1. Sensitive Fine Grained	🟩 4. Clayey Silt to Silty Clay	🟧 7. Gravely Sand to Sand
🟫 2. Organic Material	🟩 5. Silty Sand to Sandy Silt	⬛ 8. Very Stiff Sand to Clayey Sand
🟦 3. Clay to Silty Clay	🟨 6. Clean Sand to Silty Sand	⬜ 9. Very Stiff Fine Grained

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Sounding: SCPT22-03

Client: Thurber Engineering Ltd.

06-Apr-22

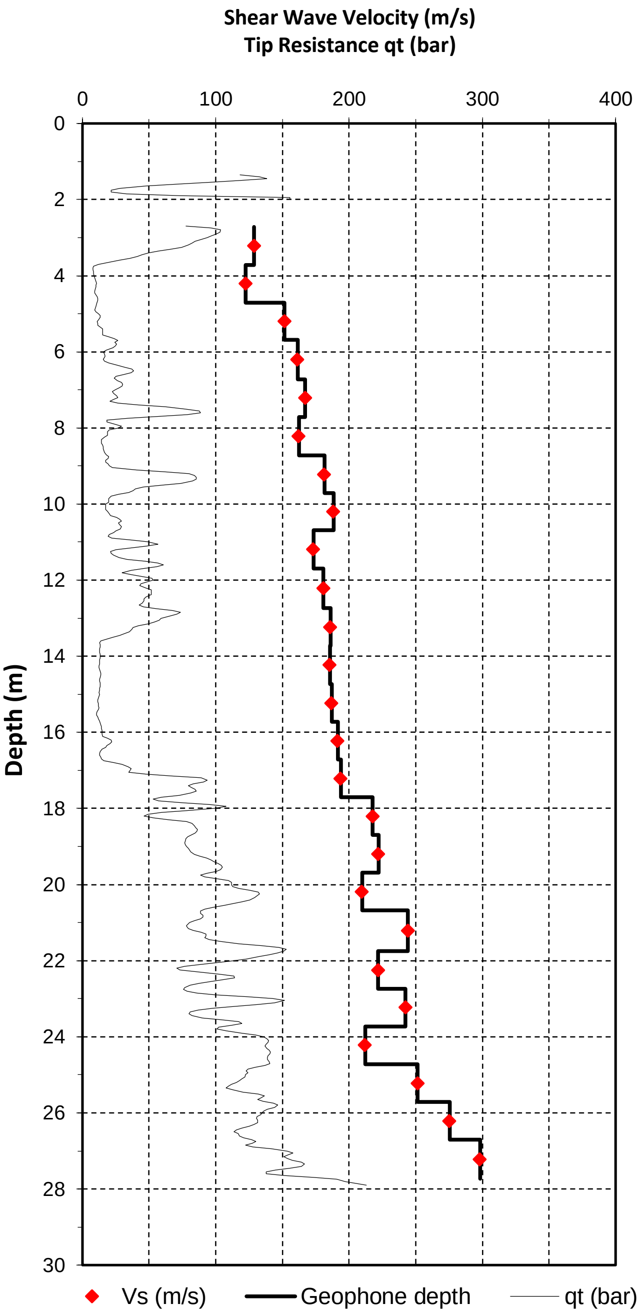
Site: DP World - Fraser Surrey Docks, Surrey, BC

Seismic Source: Beam
Source to cone (m): 1.2

Geodetic Elevation: N/A
Cone ID: DDG1521
Operator: ZH

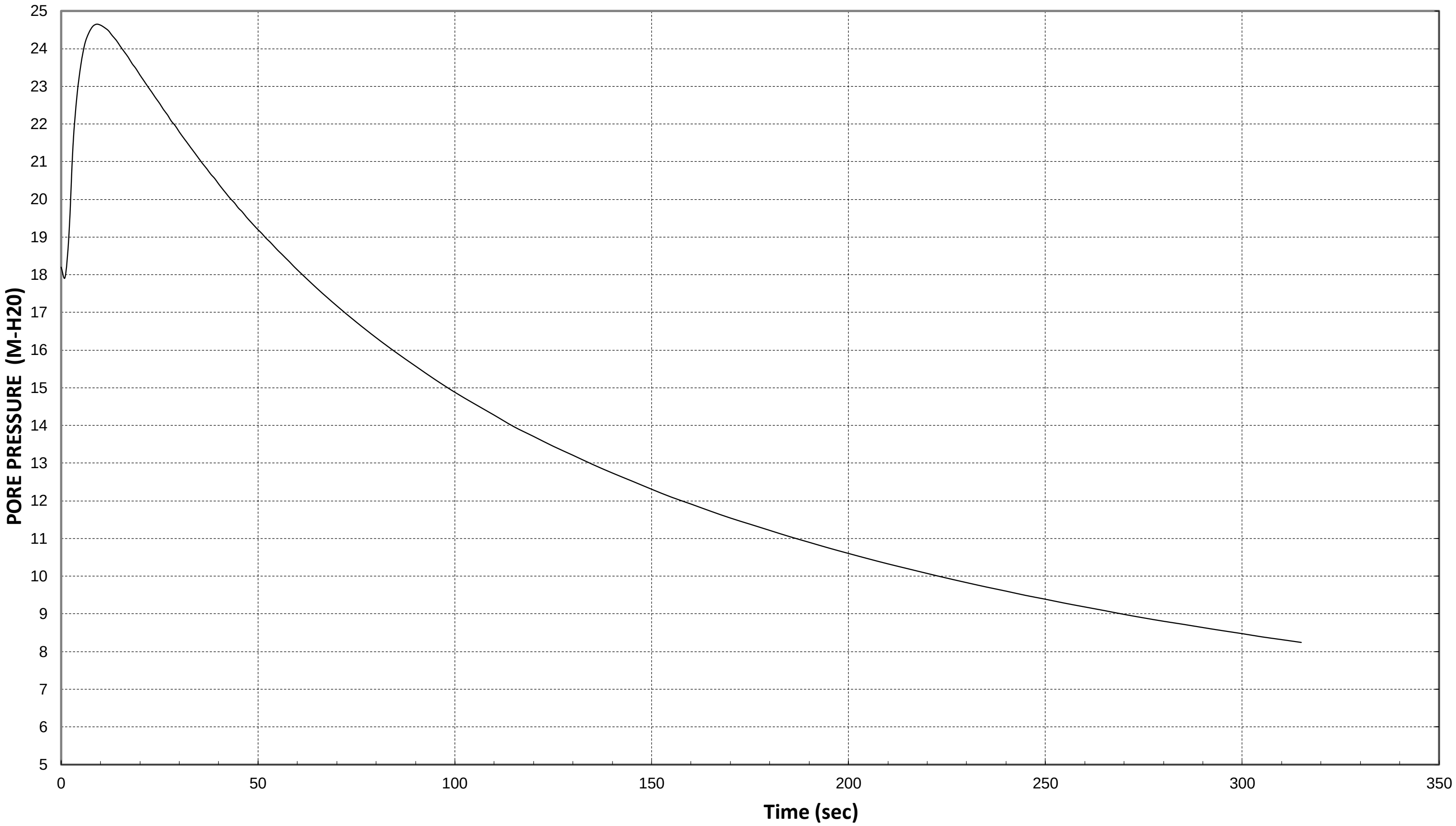
Shear Wave Velocity Data (Vs)

Depth (m)	**Geophone Depth (m)**	**Ray Path (m)**	**Ray Path Difference (m)**	**Time Difference (ms)**	**Shear Wave Velocity Vs (m/s)**
2.92	2.72	2.97			
3.92	3.72	3.91	0.94	7.27	**129**
4.91	4.71	4.86	0.95	7.78	**122**
5.88	5.68	5.81	0.94	6.23	**152**
6.92	6.72	6.83	1.02	6.33	**161**
7.92	7.72	7.81	0.99	5.90	**167**
8.93	8.73	8.81	1.00	6.16	**162**
9.92	9.72	9.79	0.98	5.41	**181**
10.89	10.69	10.76	0.96	5.12	**188**
11.89	11.69	11.75	0.99	5.74	**173**
12.93	12.73	12.79	1.04	5.73	**181**
13.93	13.73	13.78	1.00	5.35	**186**
14.93	14.73	14.78	1.00	5.37	**186**
15.93	15.73	15.78	1.00	5.33	**187**
16.92	16.72	16.76	0.99	5.16	**192**
17.91	17.71	17.75	0.99	5.10	**194**
18.90	18.70	18.74	0.99	4.54	**218**
19.89	19.69	19.73	0.99	4.45	**222**
20.88	20.68	20.71	0.99	4.71	**210**
21.95	21.75	21.78	1.07	4.38	**244**
22.94	22.74	22.77	0.99	4.46	**222**
23.93	23.73	23.76	0.99	4.08	**242**
24.92	24.72	24.75	0.99	4.66	**212**
25.92	25.72	25.75	1.00	3.97	**252**
26.91	26.71	26.74	0.99	3.59	**275**
27.93	27.73	27.76	1.02	3.41	**298**


Shear Wave Velocity (m/s)
Tip Resistance qt (bar)
0
100
200
300
400
Depth (m)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Vs (m/s)
Geophone depth
qt (bar)

Sounding: SCPT22-03	Client: Thurber Engineering Ltd.
06-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC

PORE PRESSURE DISSIPATION TEST

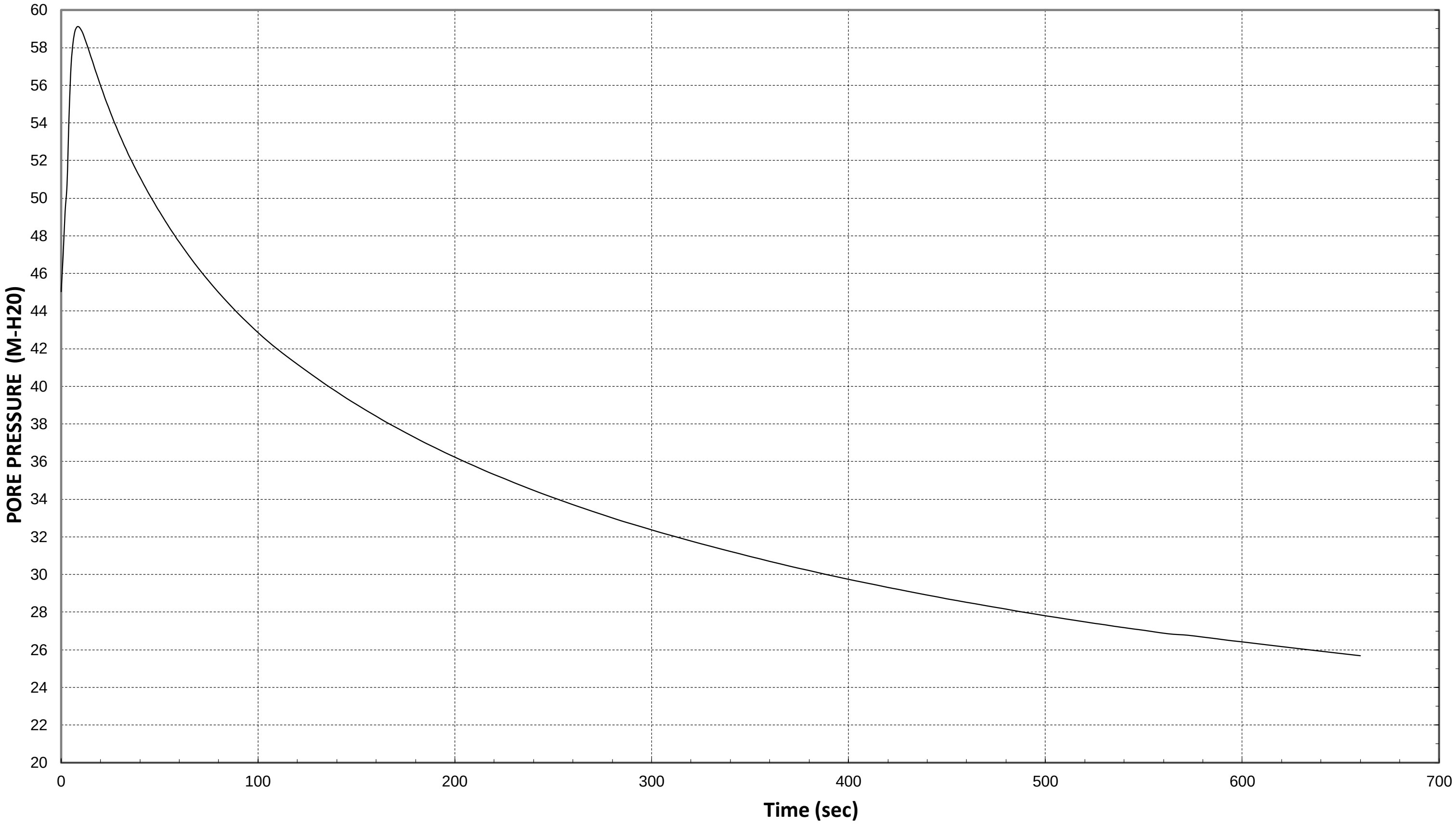


Test Depth:	5.00 m
Duration:	315 s
Cone Area:	10 cm^2
Cone ID:	DDG1521

u Min:	8.24 m
u Max:	24.65 m
u Final:	8.24 m

Sounding: SCPT22-03	Client: Thurber Engineering Ltd.
06-Apr-2022	Site: DP World - Fraser Surrey Docks, Surrey, BC

PORE PRESSURE DISSIPATION TEST



Test Depth:	15.00 m
Duration:	660 s
Cone Area:	10 cm^2
Cone ID:	DDG1521

u Min:	25.69 m
u Max:	59.10 m
u Final:	25.69 m

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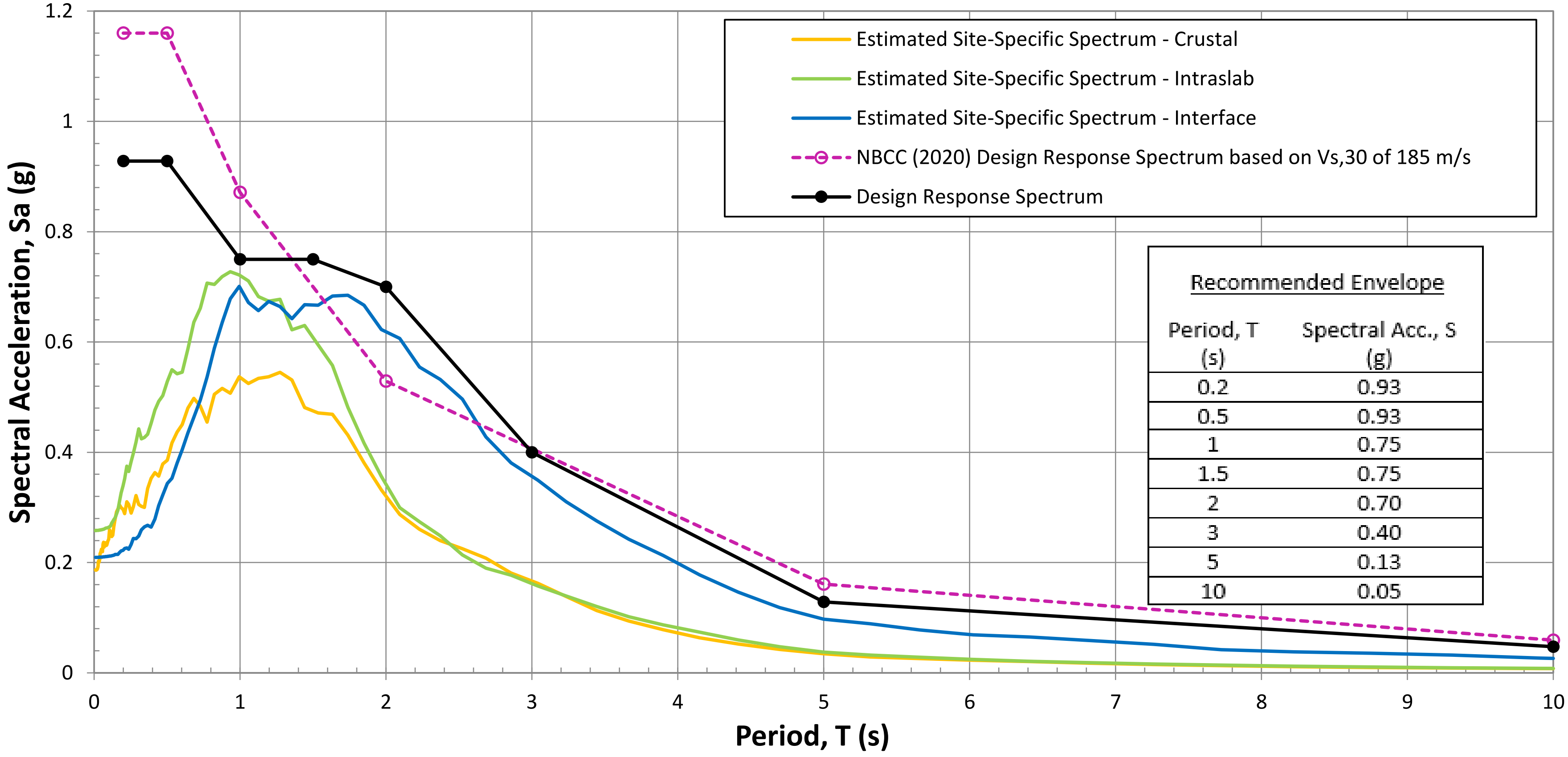
APPENDIX D

Design Response Spectra (1 page)

2% Probability of Exceedance in 50 years (1:2,475 Yr Earthquake Event)



Recommended Envelope

Period, T (s)	Spectral Acc., S (g)
0.2	0.93
0.5	0.93
1	0.75
1.5	0.75
2	0.70
3	0.40
5	0.13
10	0.05

Client: DP World Fraser Surrey
File No.: 34098

Figure D1 - Design Site-Specific Response Spectra at Ground Surface
[5% Damping and 2% / 50 Years Probability (2475 Year Return Period)]

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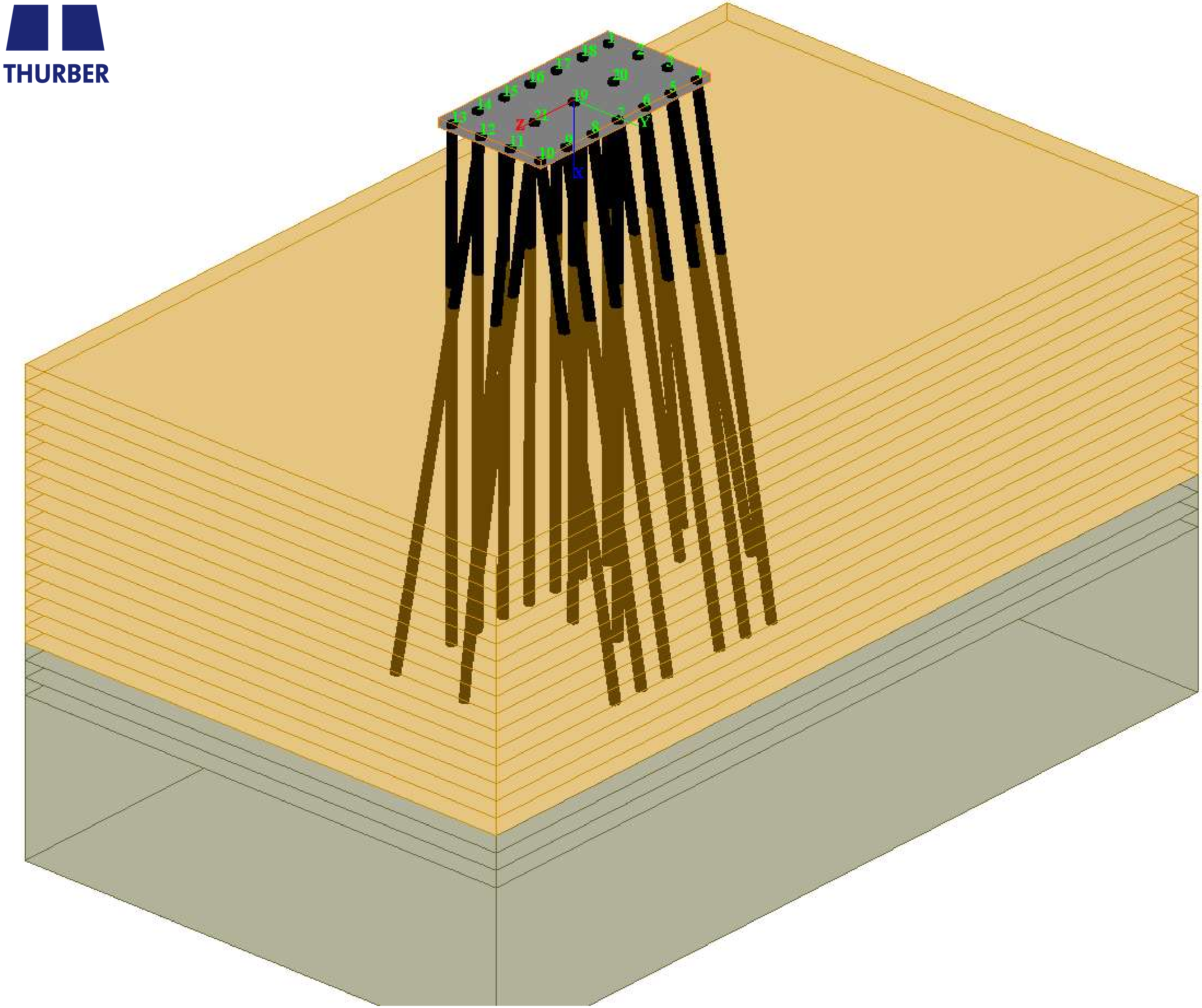
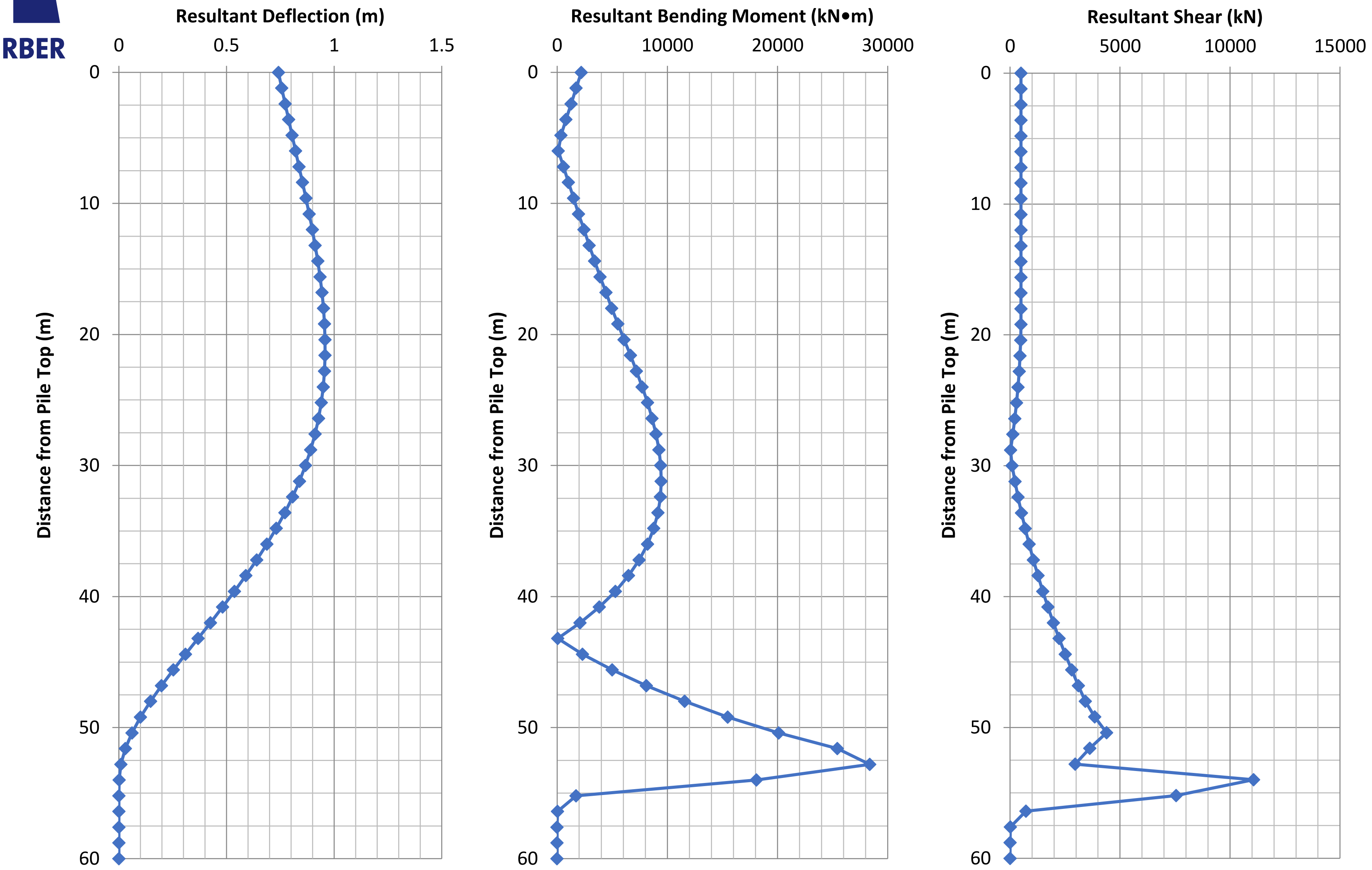
APPENDIX E

Lateral Analysis of Arm Loading Deck Group Pile under Seismic Loading Kinematic (22 pages)


1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
Z
Y
X



Client: DP World Fraser Surrey
File No.: 34098

Figure E1 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P1 - Loading Arm Deck

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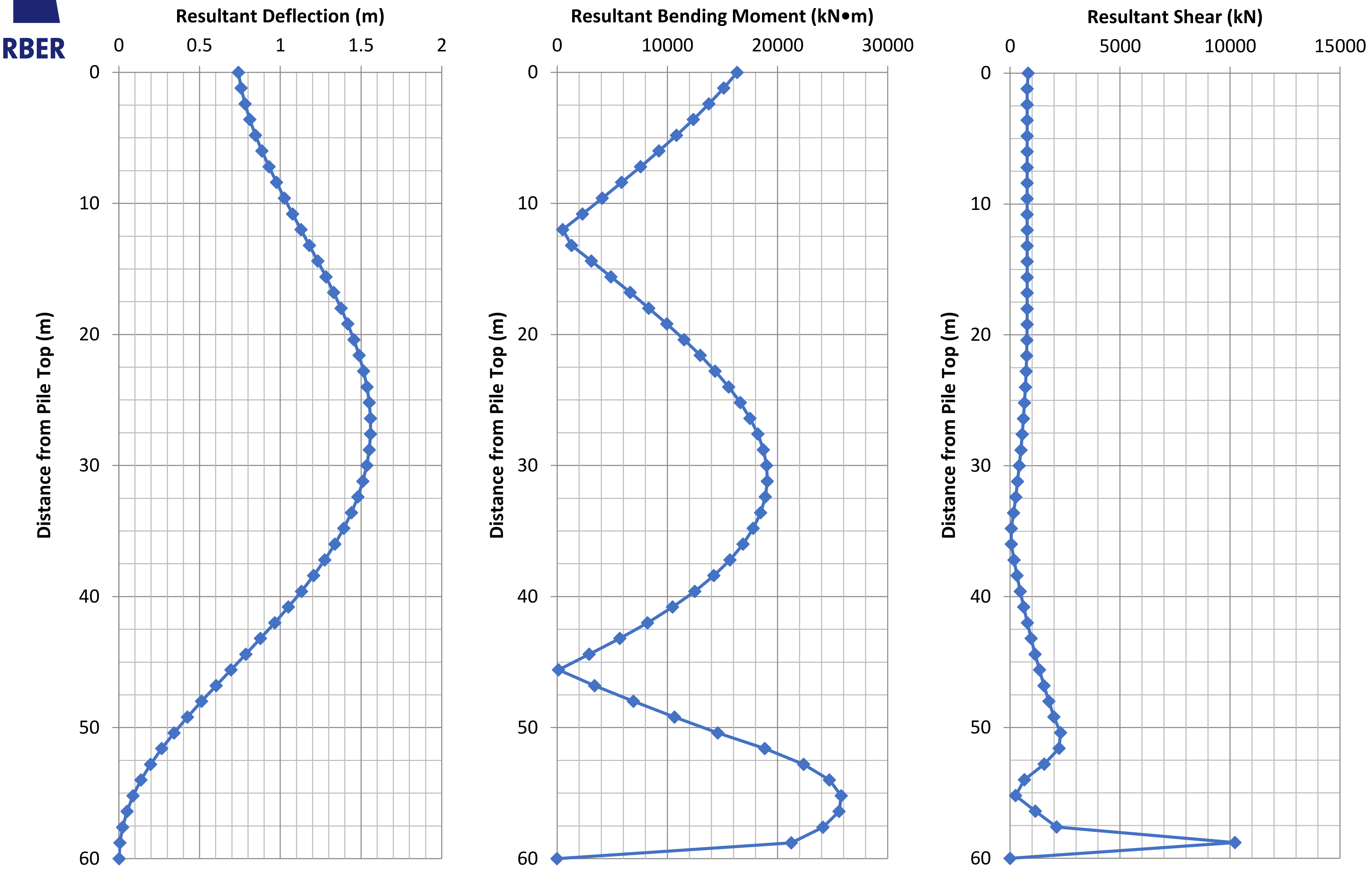


Figure E2 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P2 - Loading Arm Deck

Client: DP World Fraser Surrey
File No.: 34098

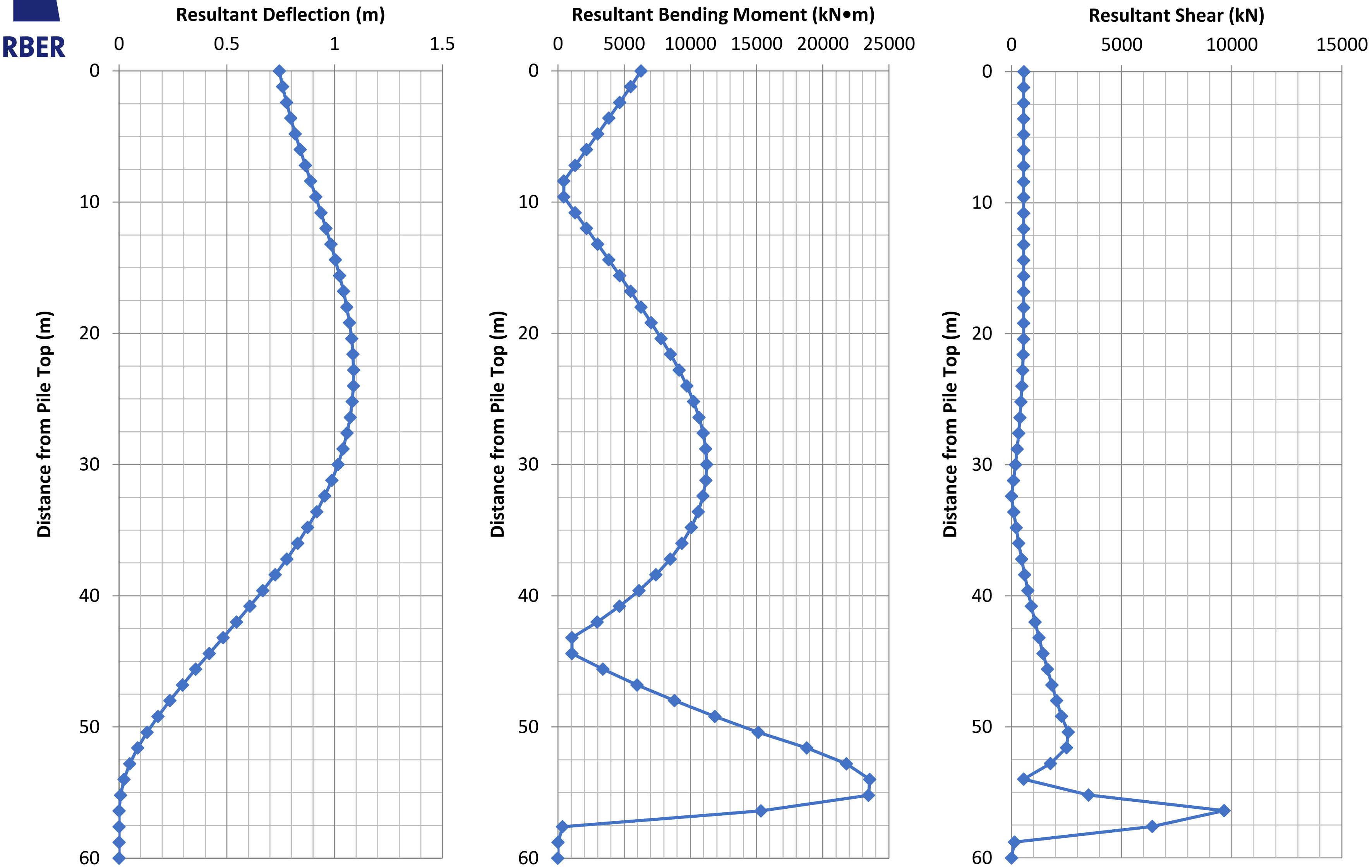
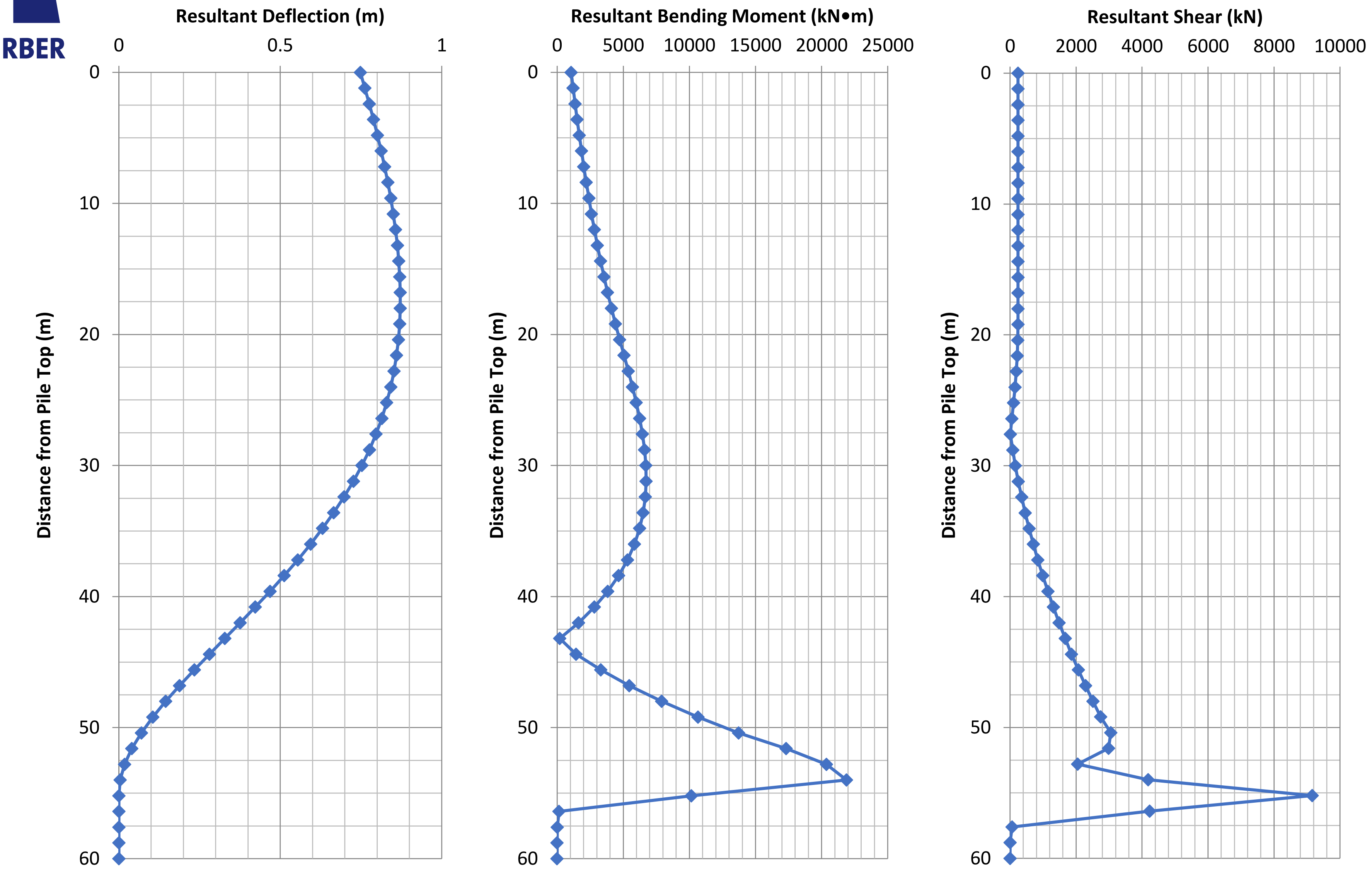


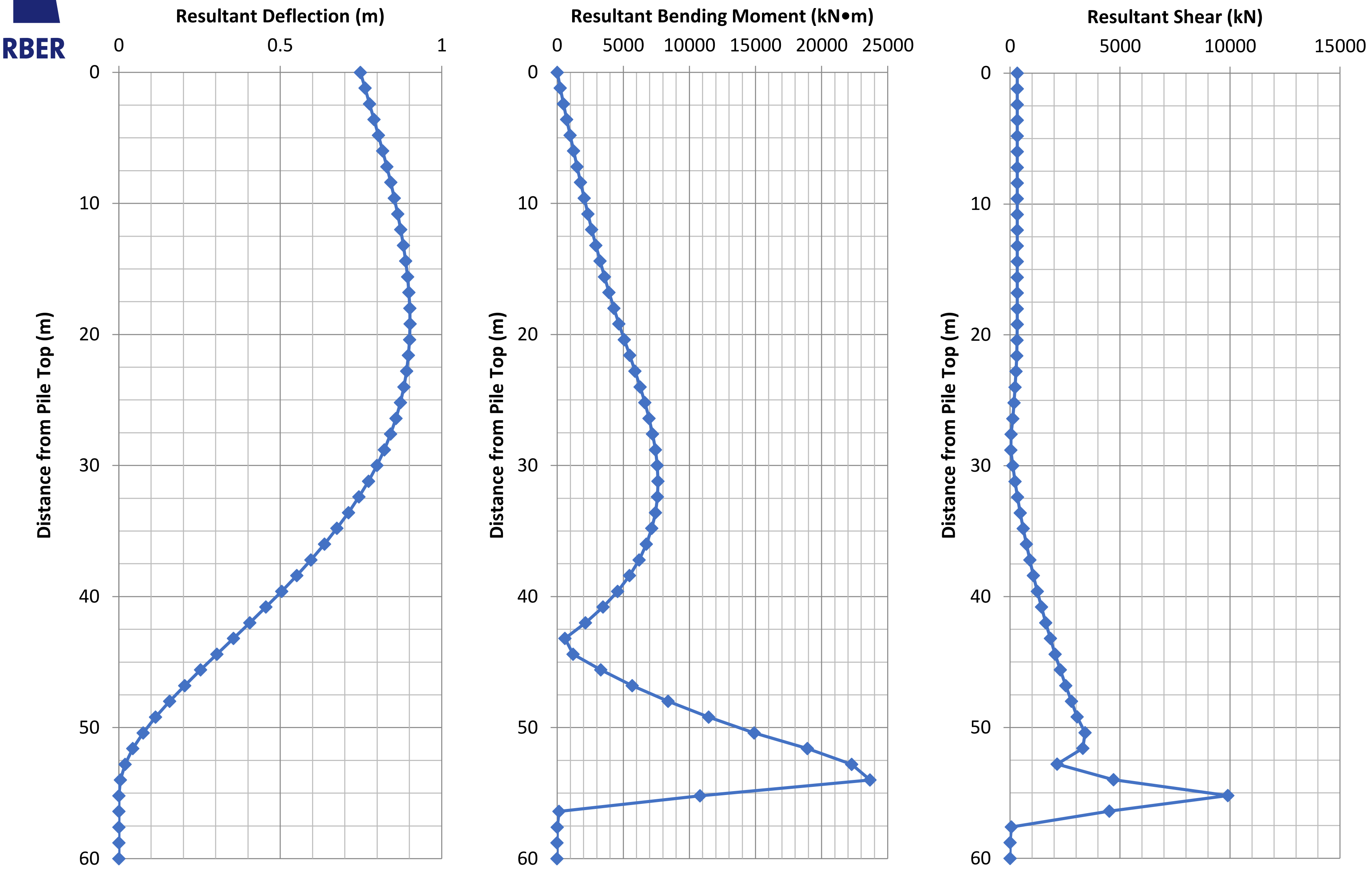
Figure E3 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P3 - Loading Arm Deck

Client: DP World Fraser Surrey
File No.: 34098



Client: DP World Fraser Surrey
File No.: 34098

Figure E4 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P4 - Loading Arm Deck



Client: DP World Fraser Surrey
File No.: 34098

Figure E5 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P5 - Loading Arm Deck

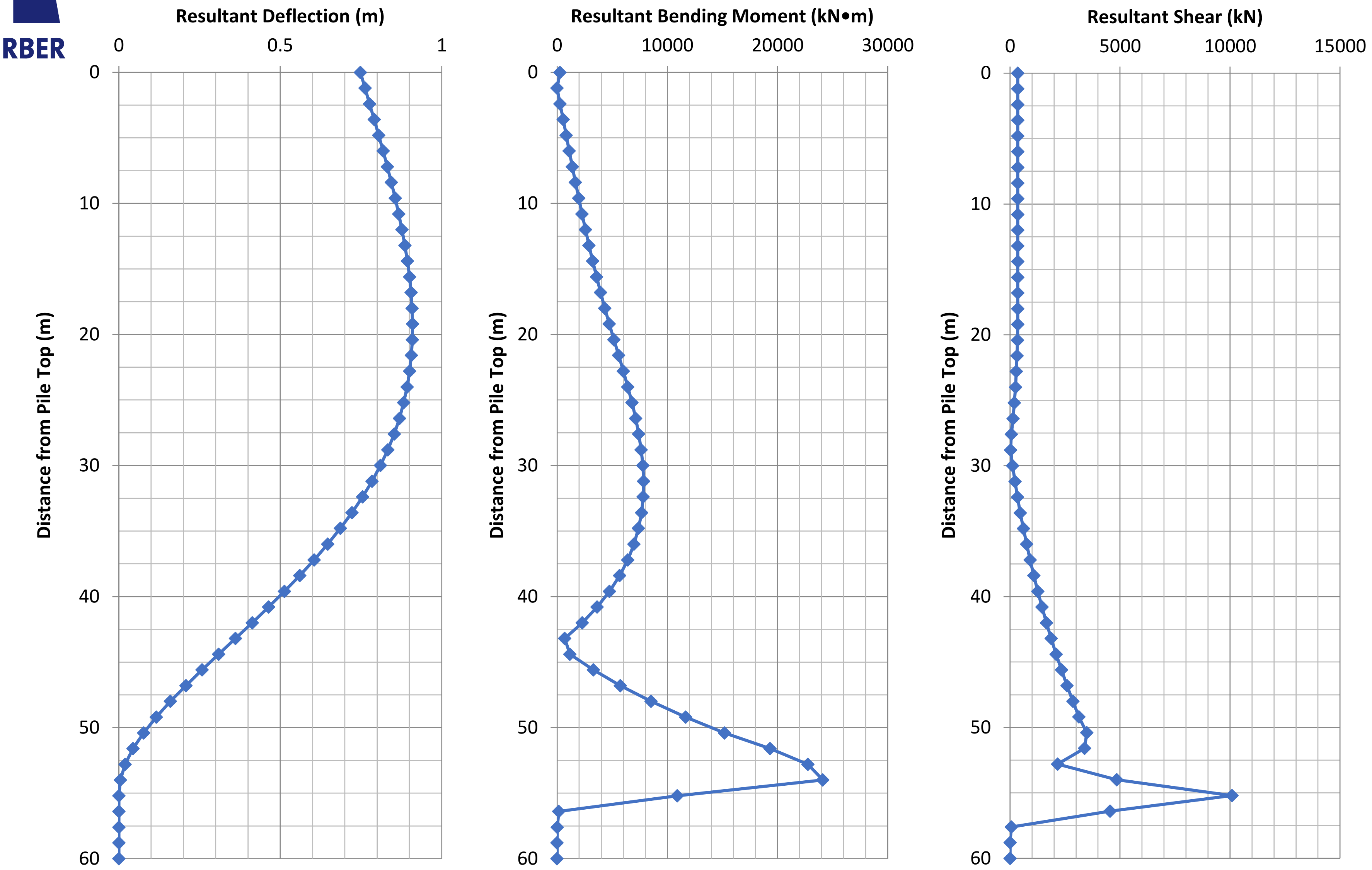
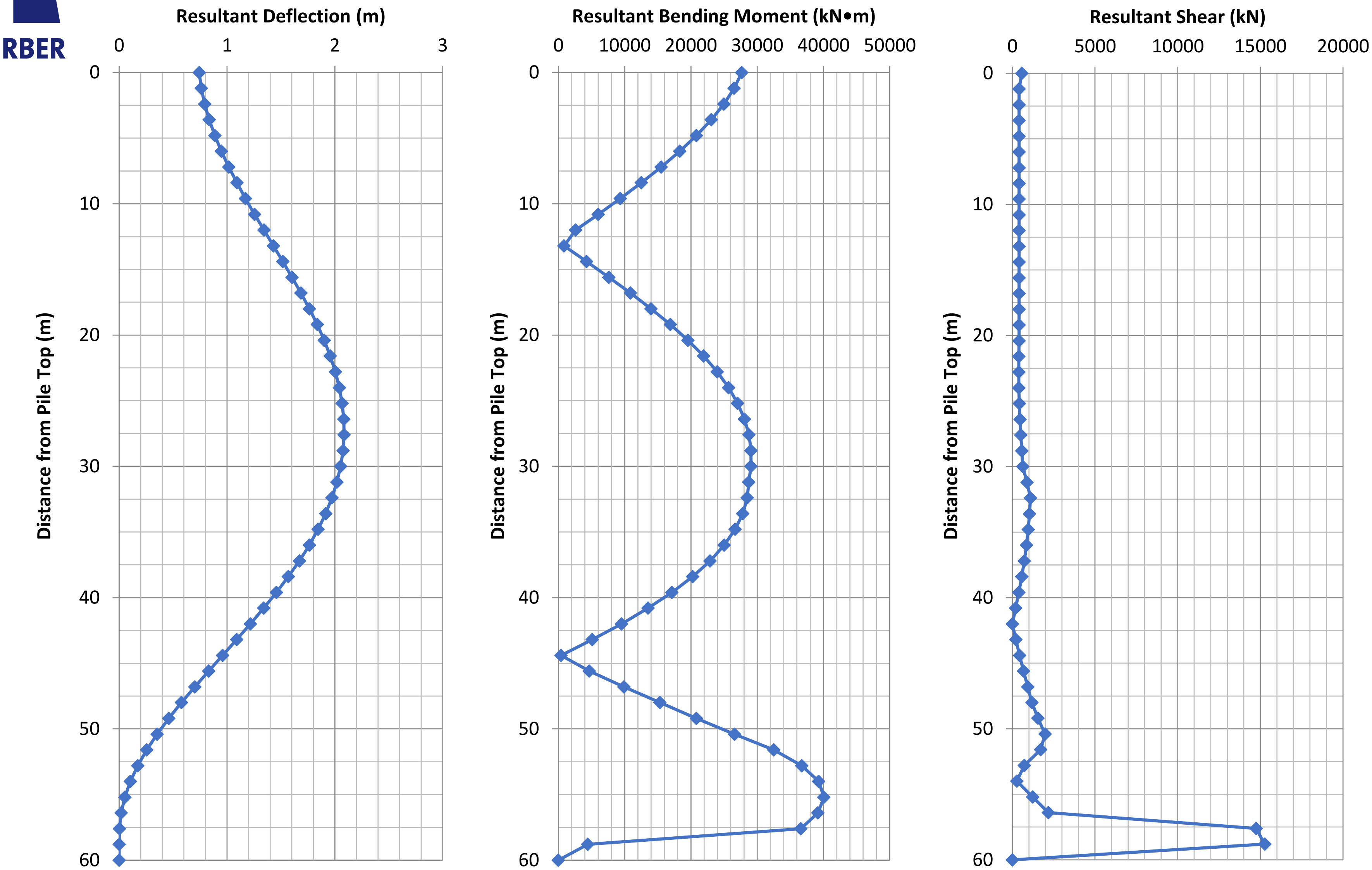


Figure E6 - Lateral Pile Analysis

Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)

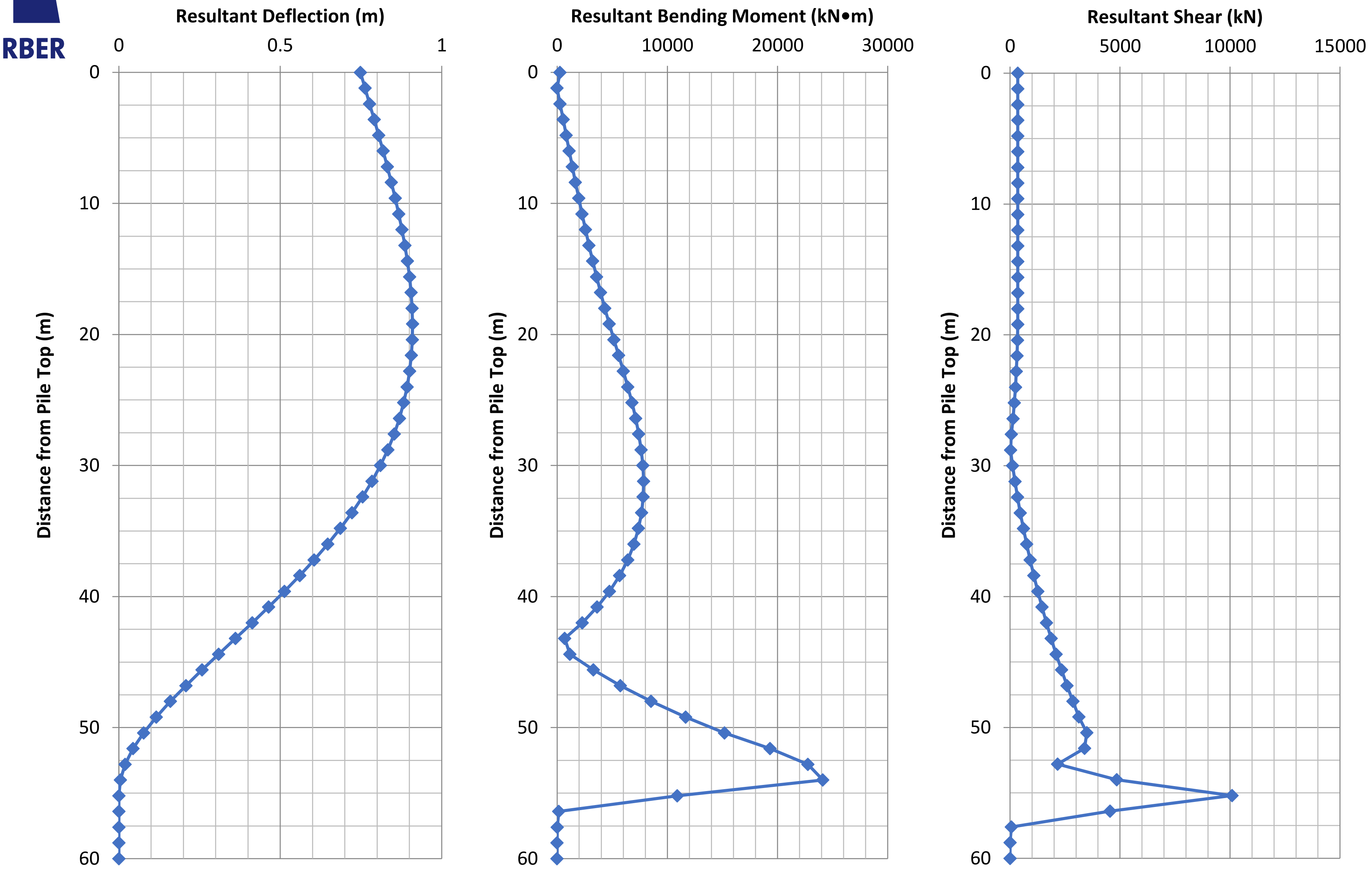
Pile P6 - Loading Arm Deck

Client: DP World Fraser Surrey
File No.: 34098



Client: DP World Fraser Surrey
File No.: 34098

Figure E7 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P7 - Loading Arm Deck



Client: DP World Fraser Surrey
File No.: 34098

Figure E8 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P8 - Loading Arm Deck

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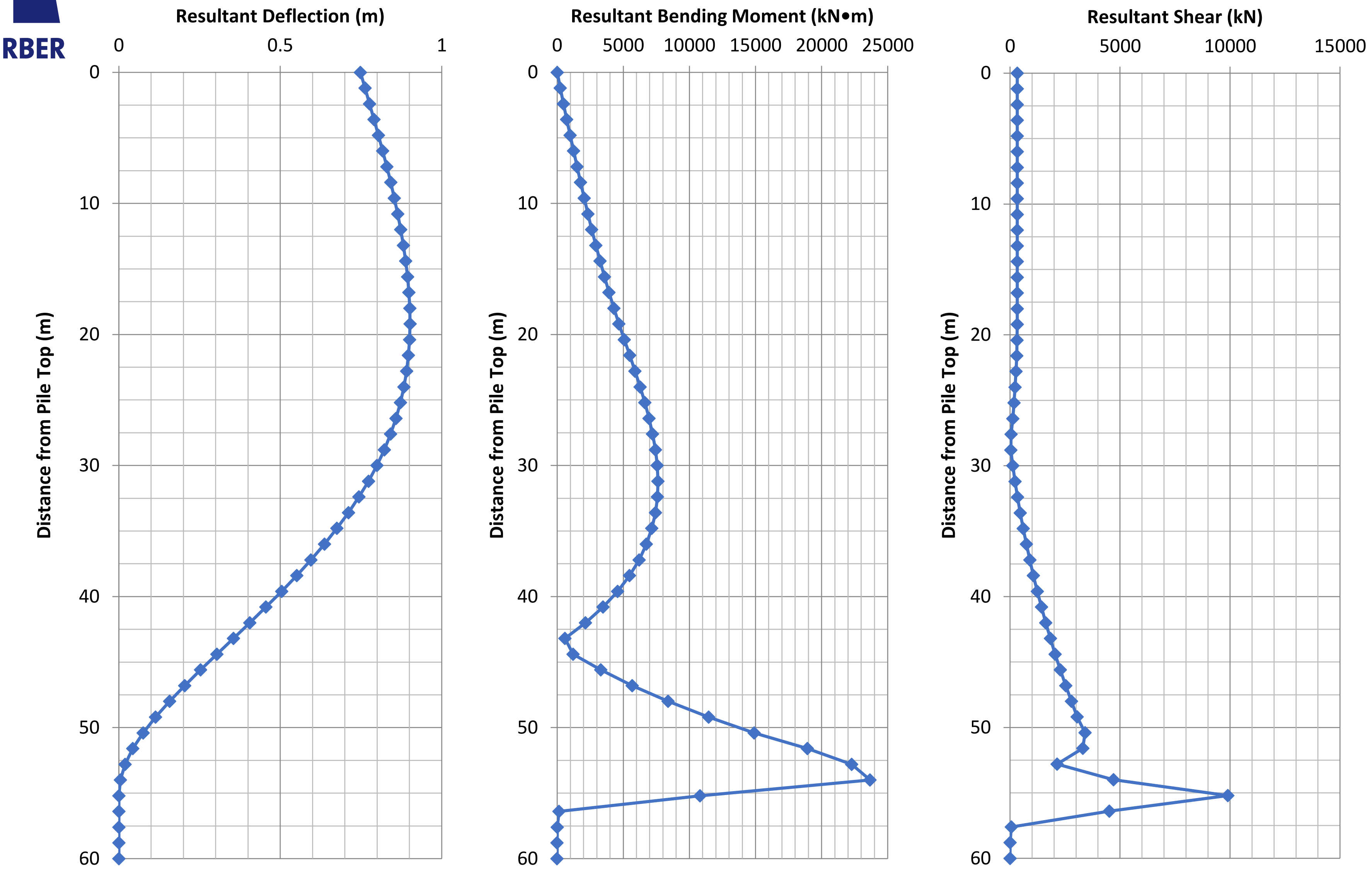


Client: DP World Fraser Surrey
File No.: 34098

Figure E9 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P9 - Loading Arm Deck

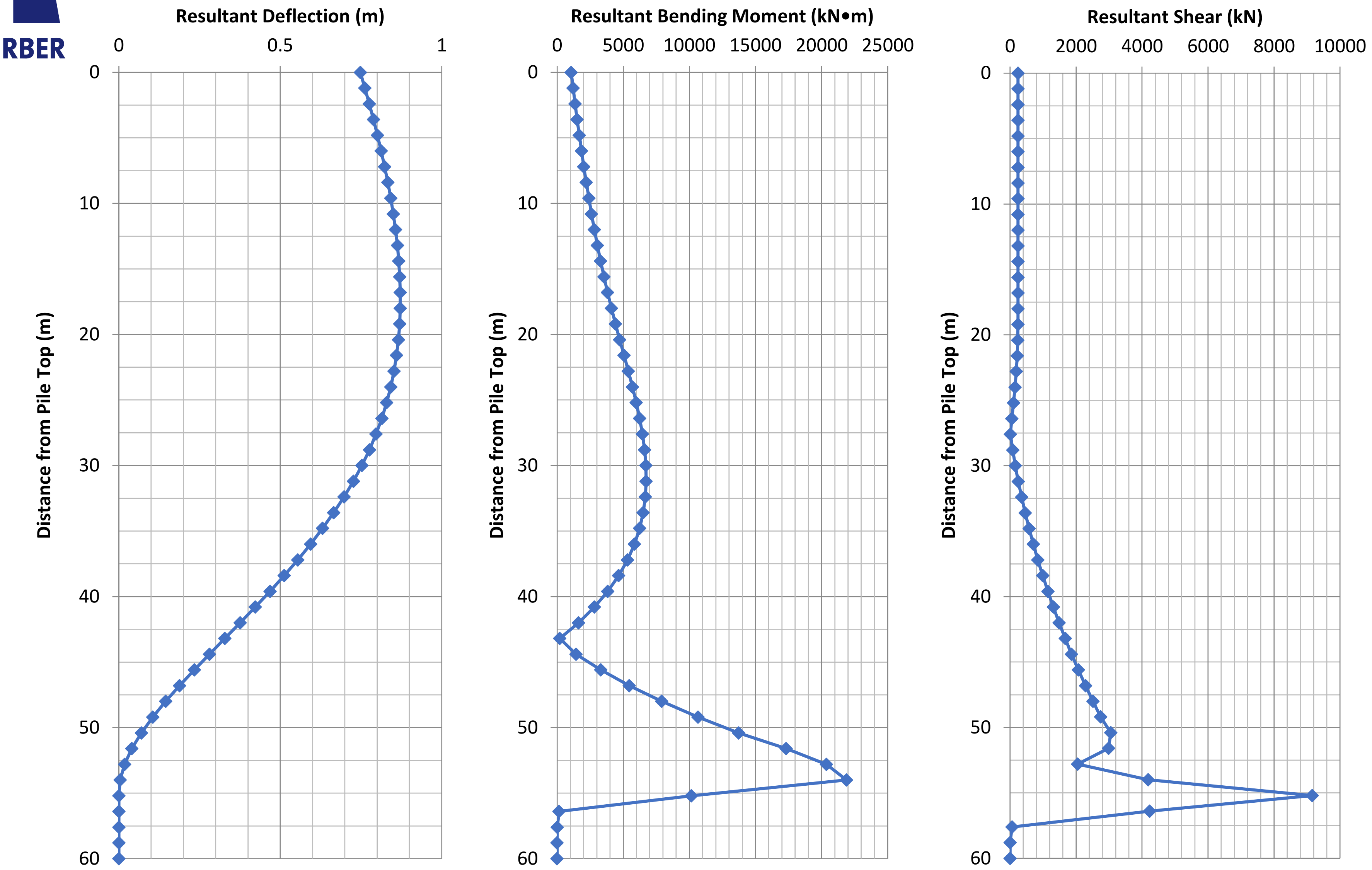


Figure E10 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P10 - Loading Arm Deck

Client: DP World Fraser Surrey
File No.: 34098

THURBER

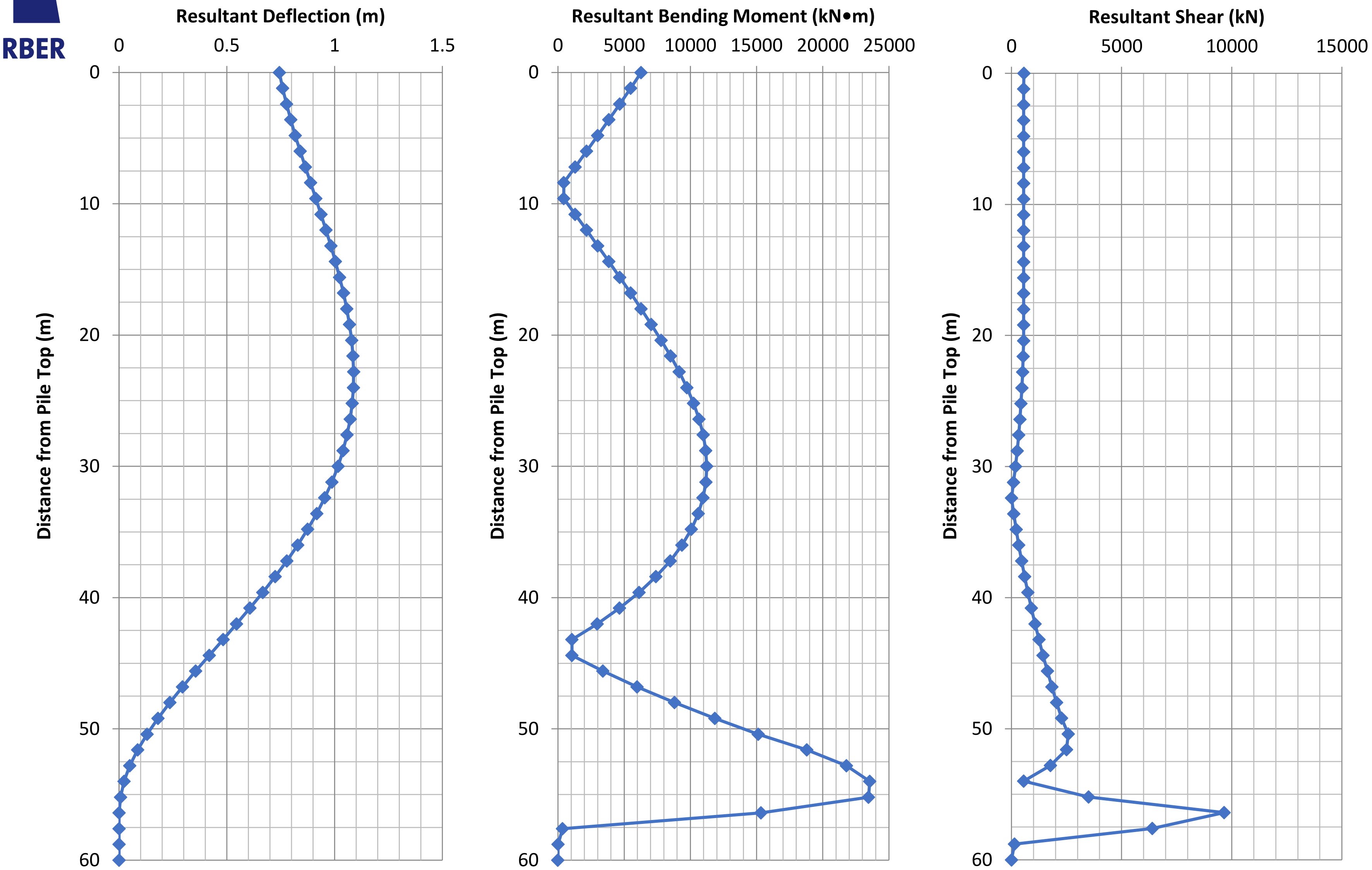
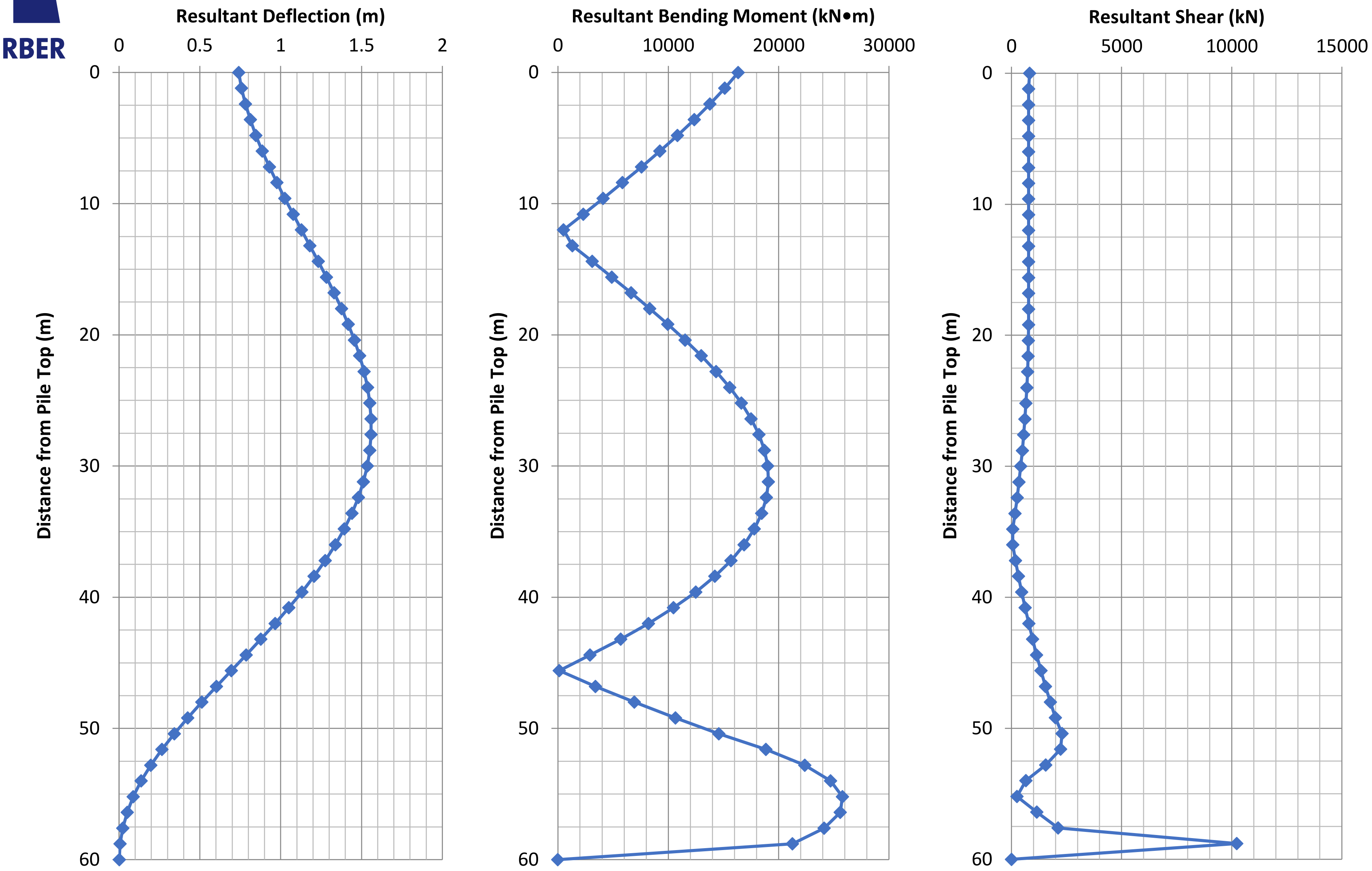


Figure E11 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P11 - Loading Arm Deck

Client: DP World Fraser Surrey
File No.: 34098



Client: DP World Fraser Surrey
File No.: 34098

Figure E12 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P12 - Loading Arm Deck

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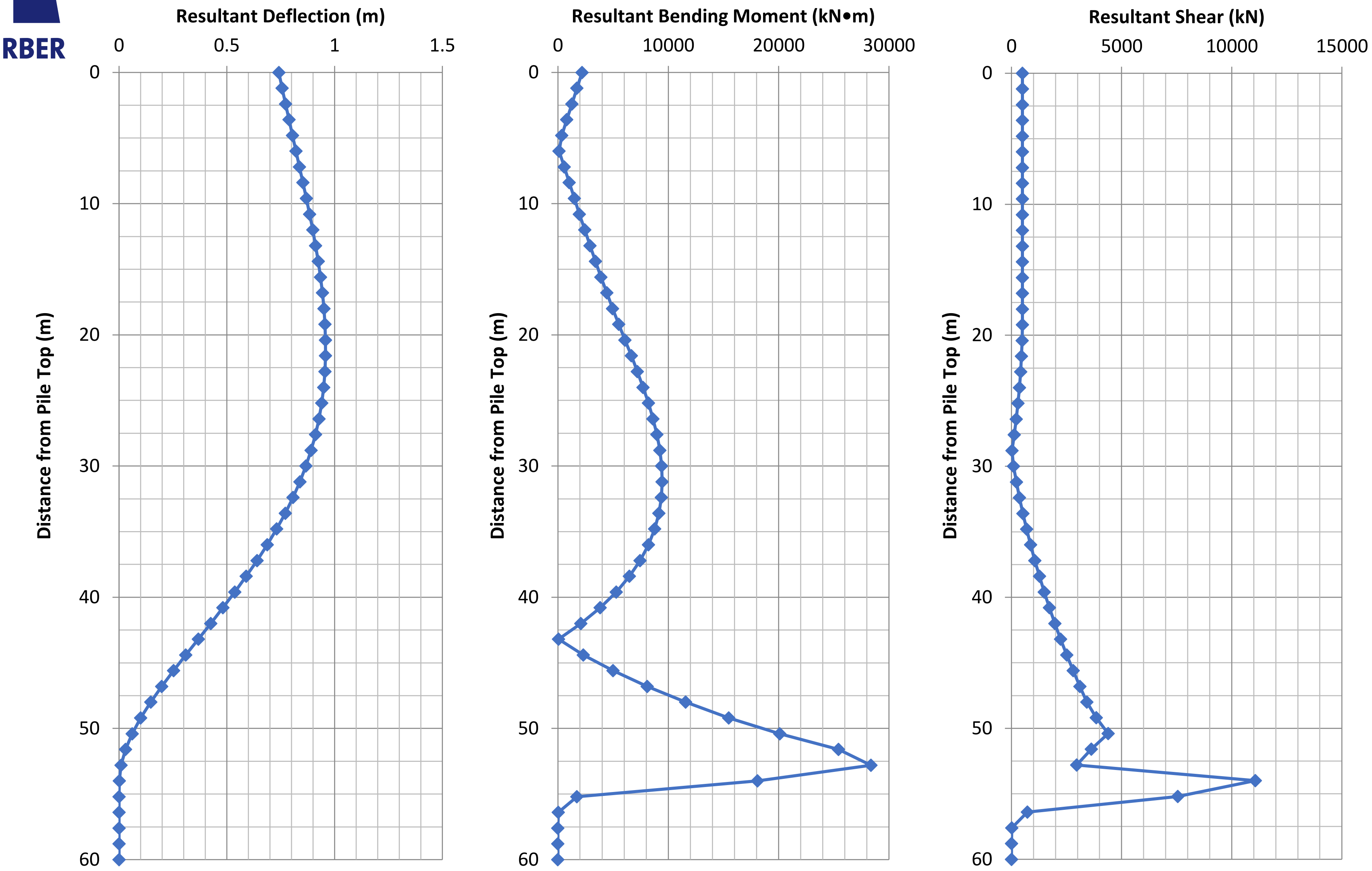

Client: DP World Fraser Surrey
File No.: 34098

Figure E13 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P13 - Loading Arm Deck

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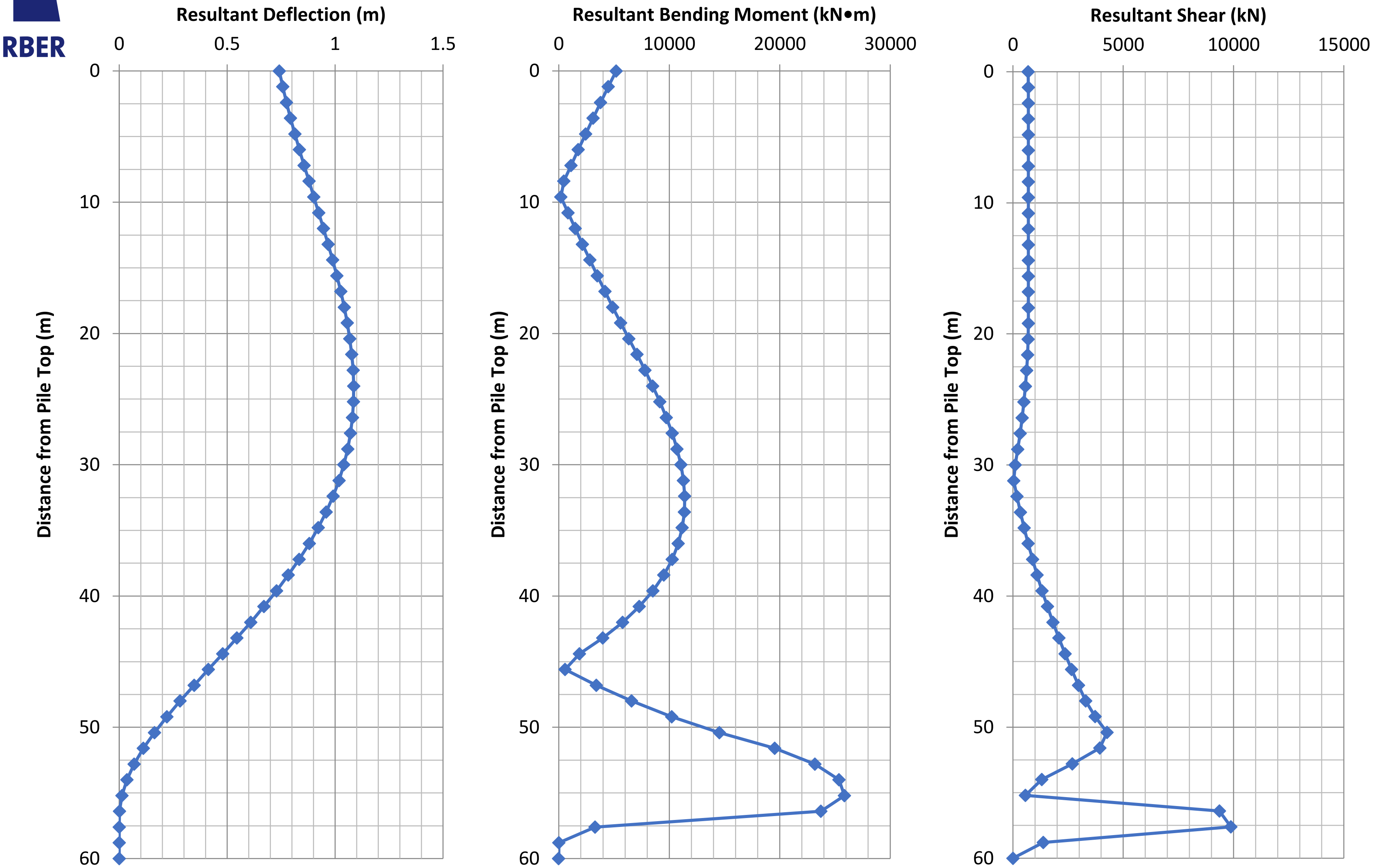


Client: DP World Fraser Surrey
File No.: 34098

Figure E14 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P14 - Loading Arm Deck

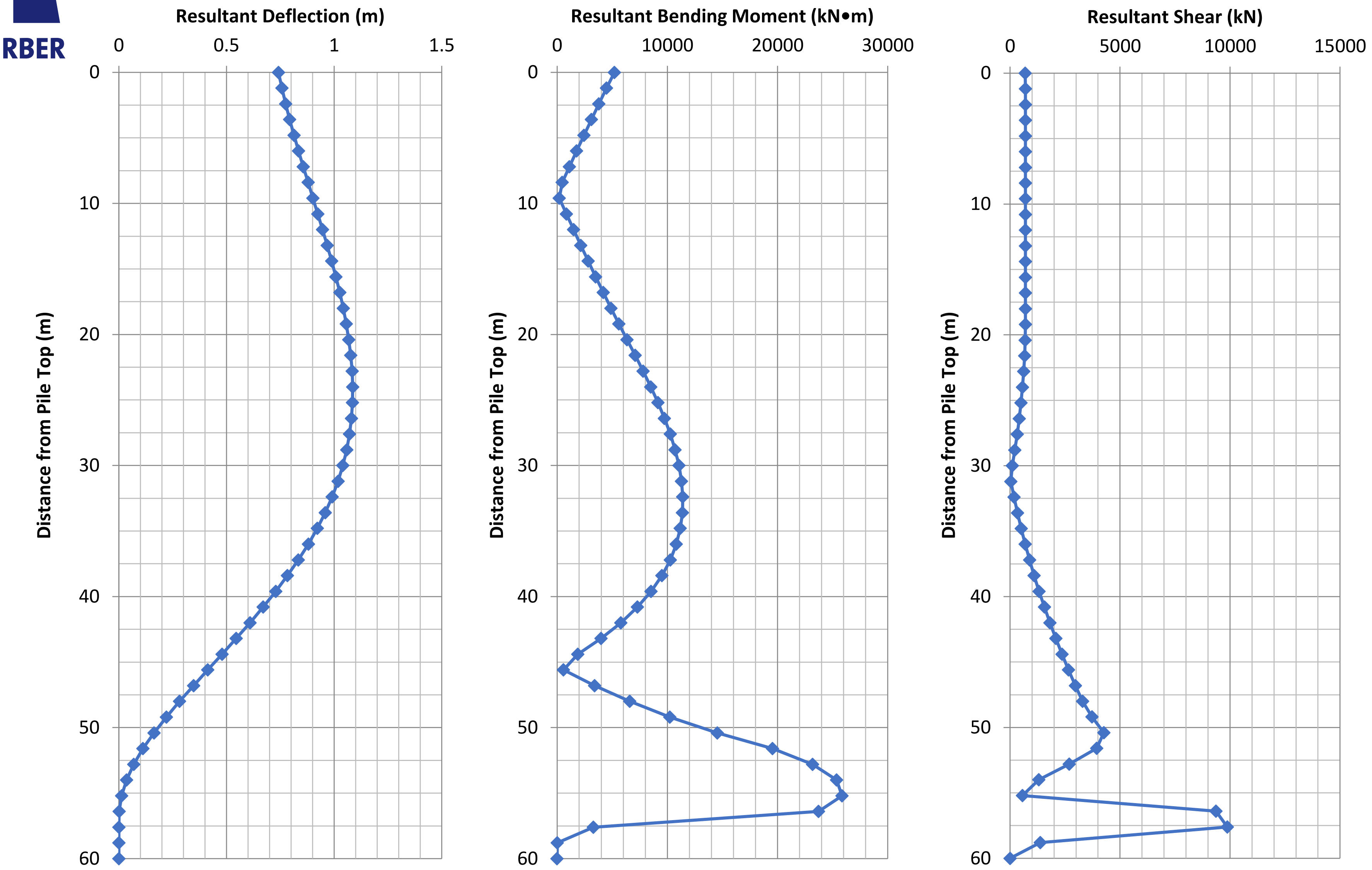


Figure E15 - Lateral Pile Analysis

Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)

Pile P15 - Loading Arm Deck

Client: DP World Fraser Surrey
File No.: 34098

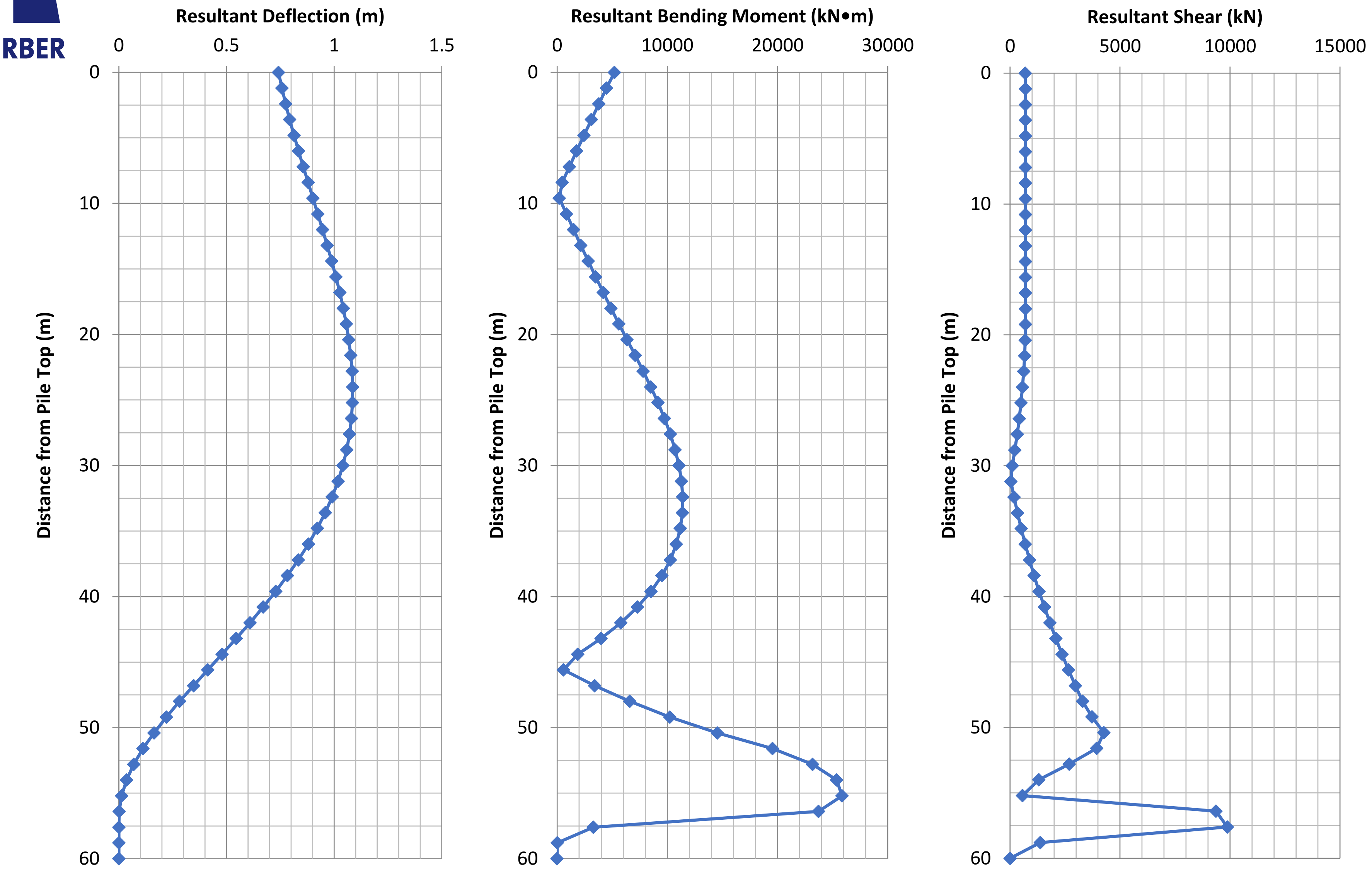


Figure E16 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P16 - Loading Arm Deck

Client: DP World Fraser Surrey
File No.: 34098

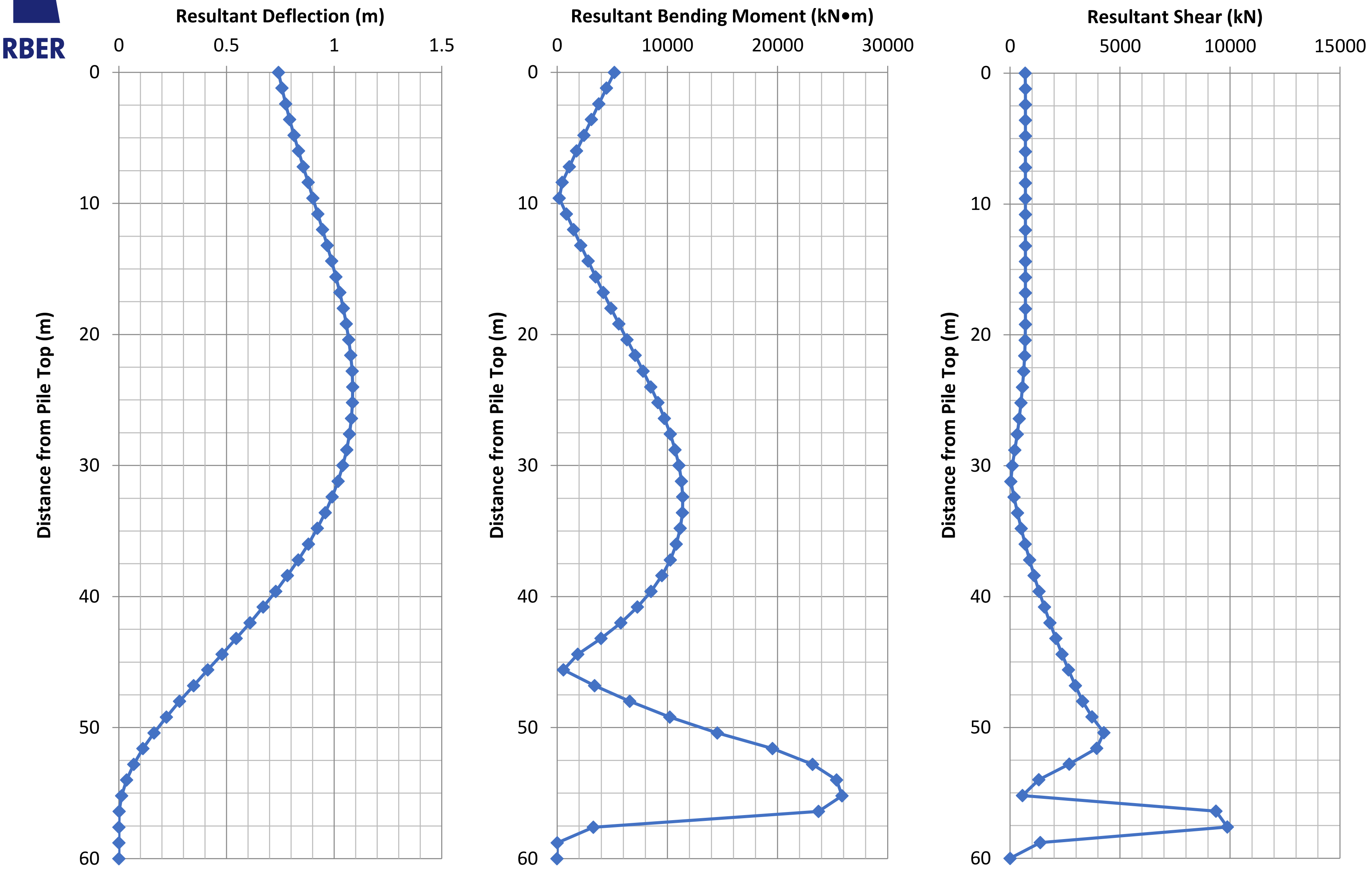
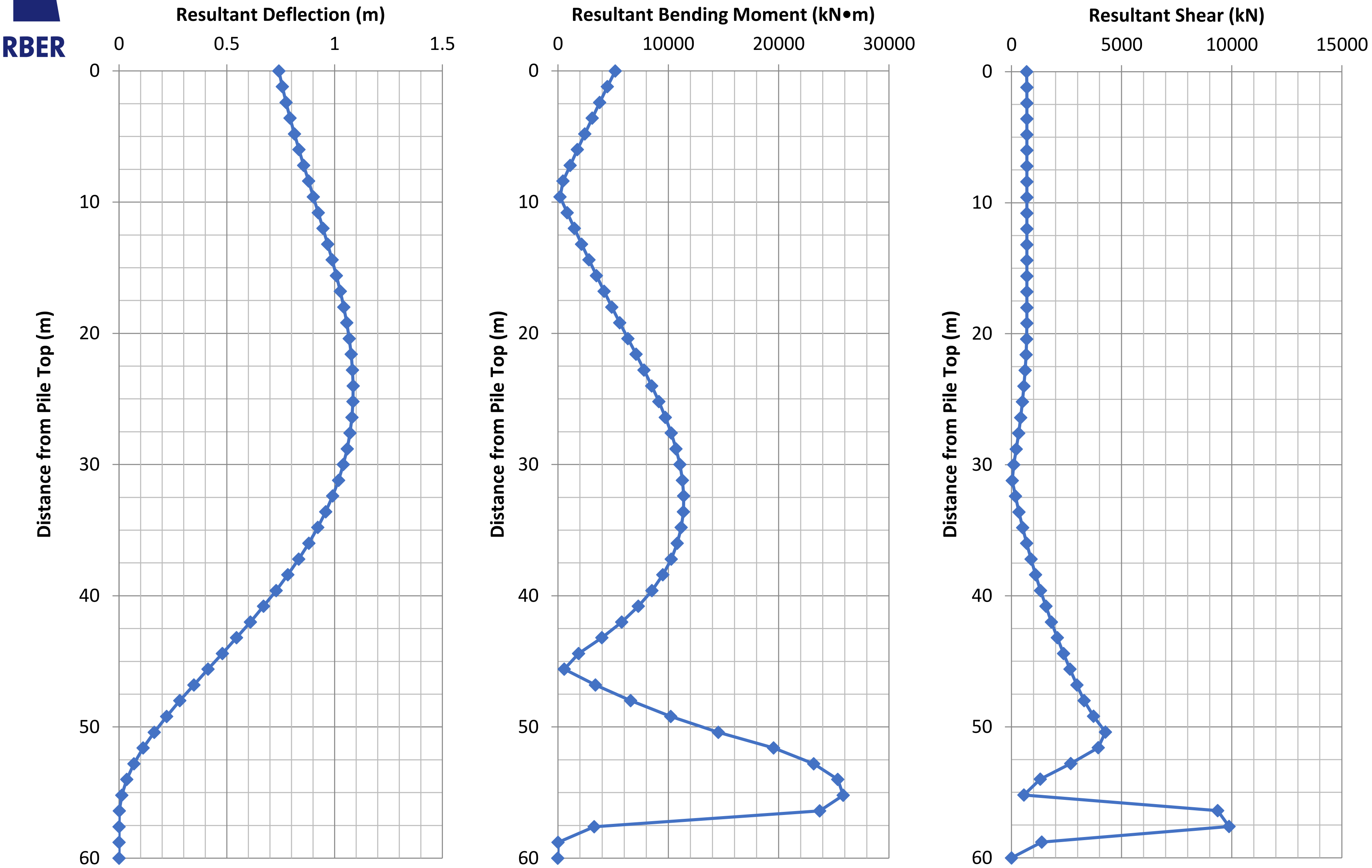


Figure E17 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P17 - Loading Arm Deck

Client: DP World Fraser Surrey
File No.: 34098



Client: DP World Fraser Surrey
File No.: 34098

Figure E18 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P18 - Loading Arm Deck

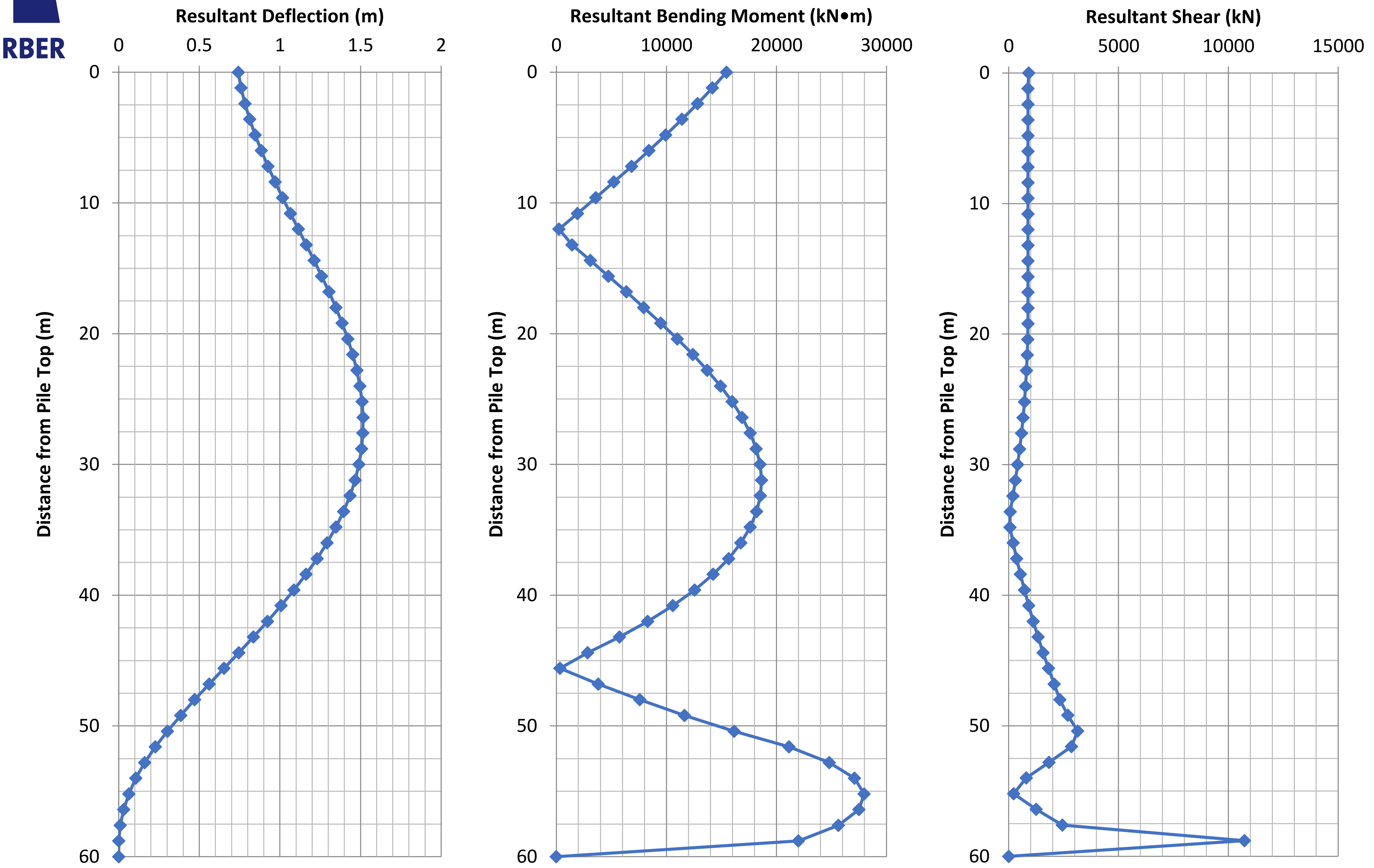


Figure E19 - Lateral Pile Analysis

Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)

Pile P19 - Loading Arm Deck

Client: DP World Fraser Surrey
File No.: 34098

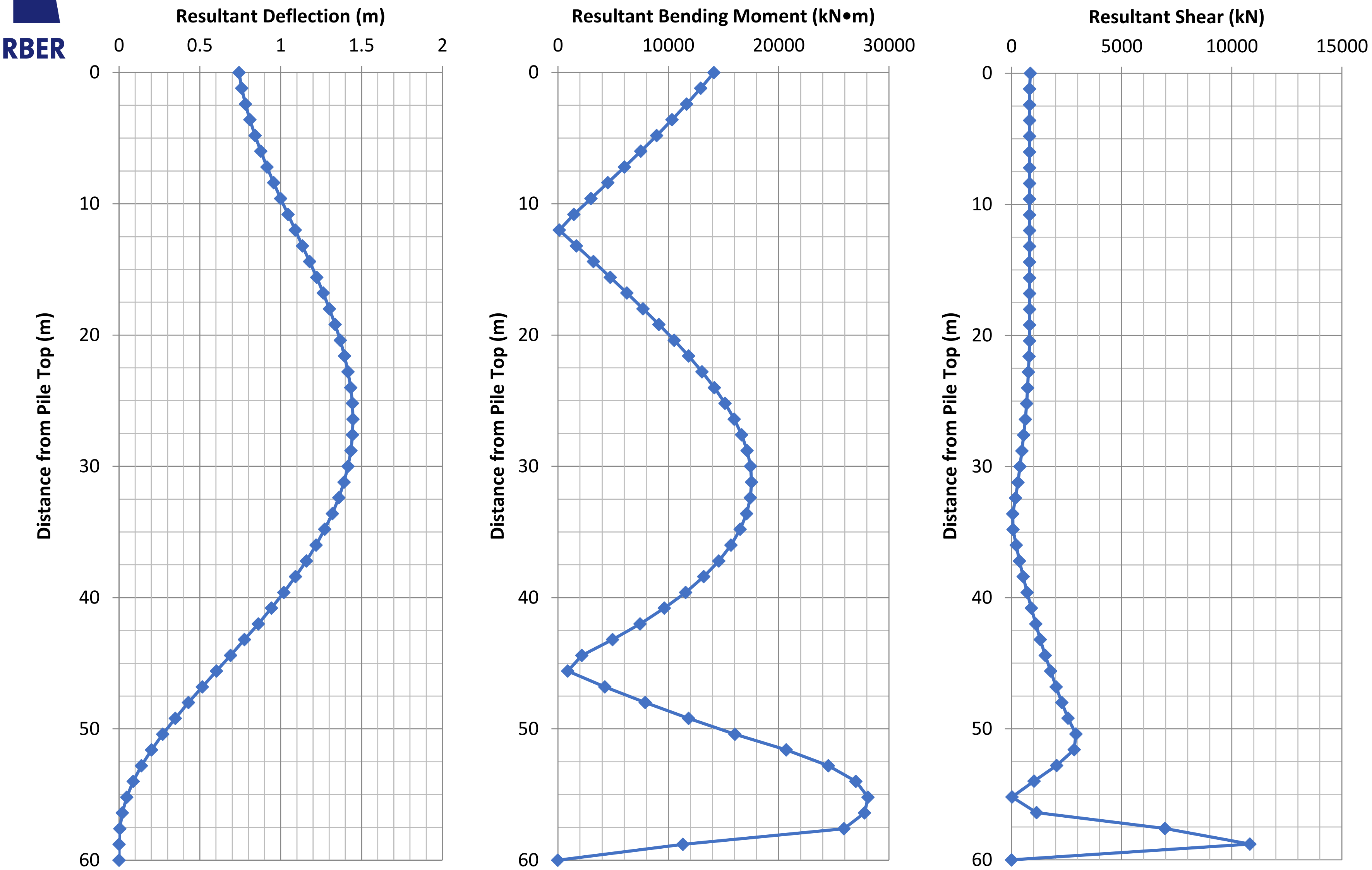


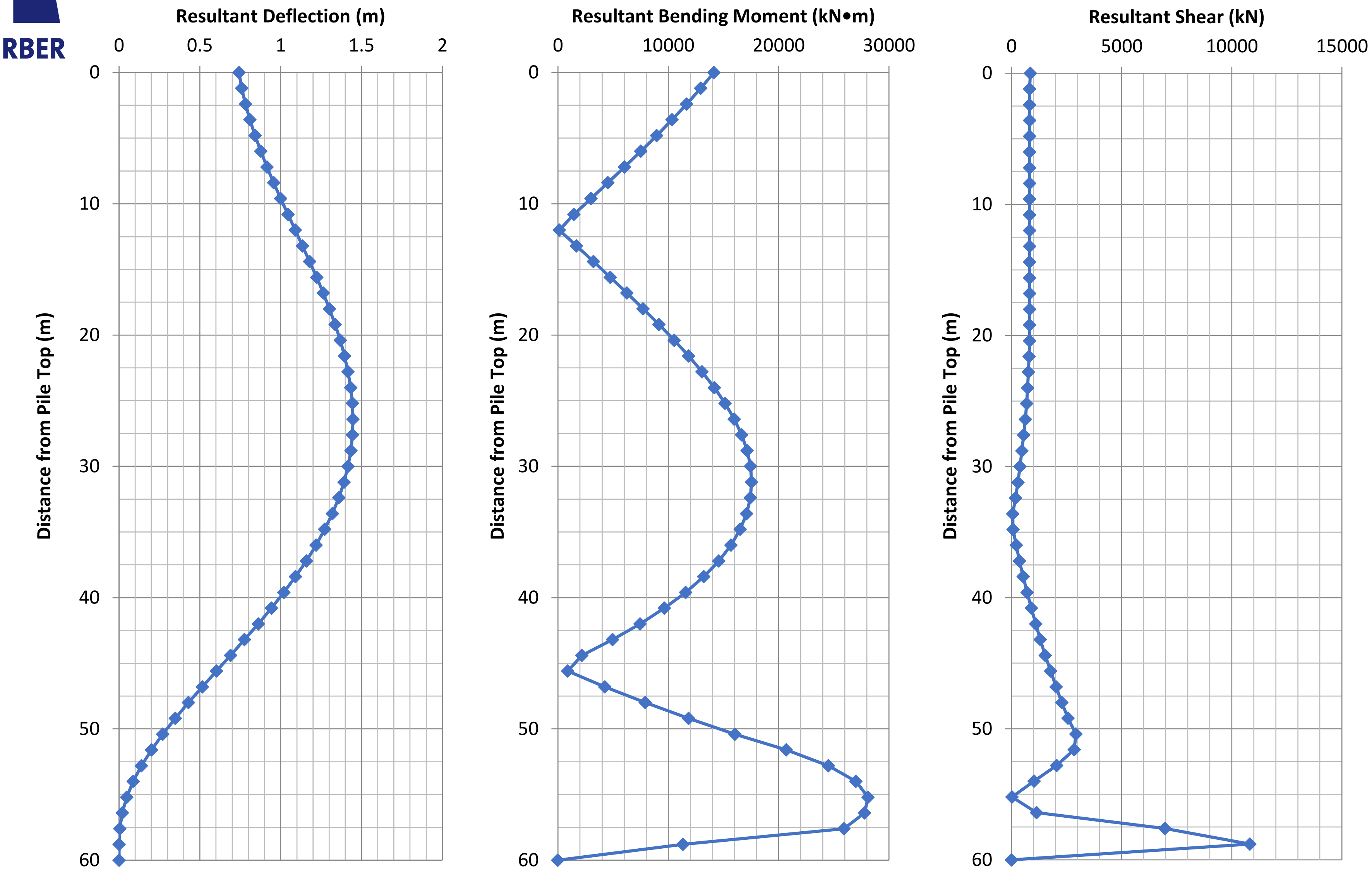
Figure E20 - Lateral Pile Analysis

Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)

Pile P20 - Loading Arm Deck

Client: DP World Fraser Surrey
File No.: 34098

THURBER

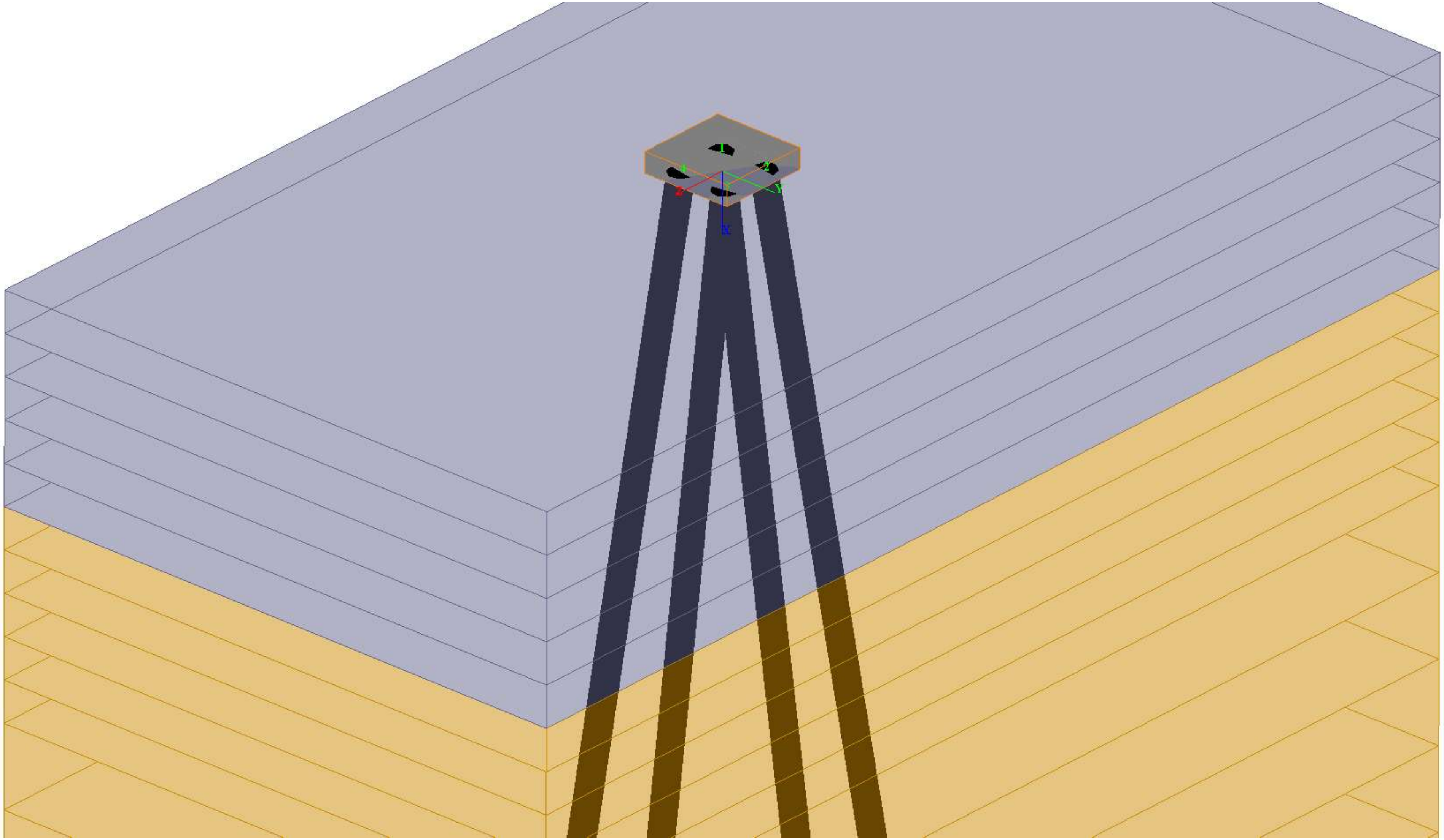


Client: DP World Fraser Surrey
File No.: 34098

Figure E21 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P21 - Loading Arm Deck

APPENDIX F

Lateral Analysis of Trestle Abutment Piles Pile under Seismic Kinematic Loading (5 pages)


1
4
2
Z
Y
X

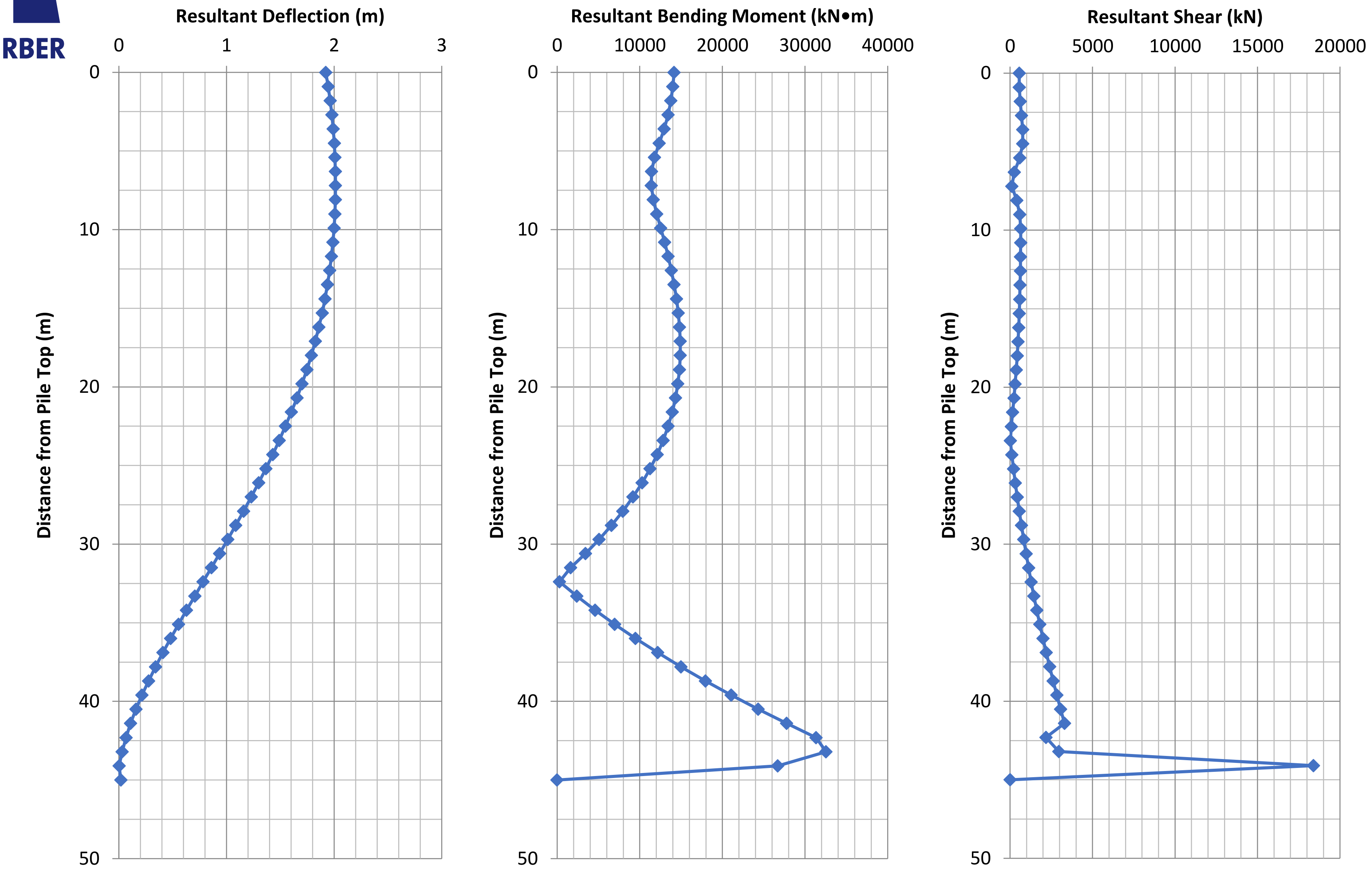


Figure F1 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P1 - Trestle Abutment

Client: DP World Fraser Surrey
File No.: 34098

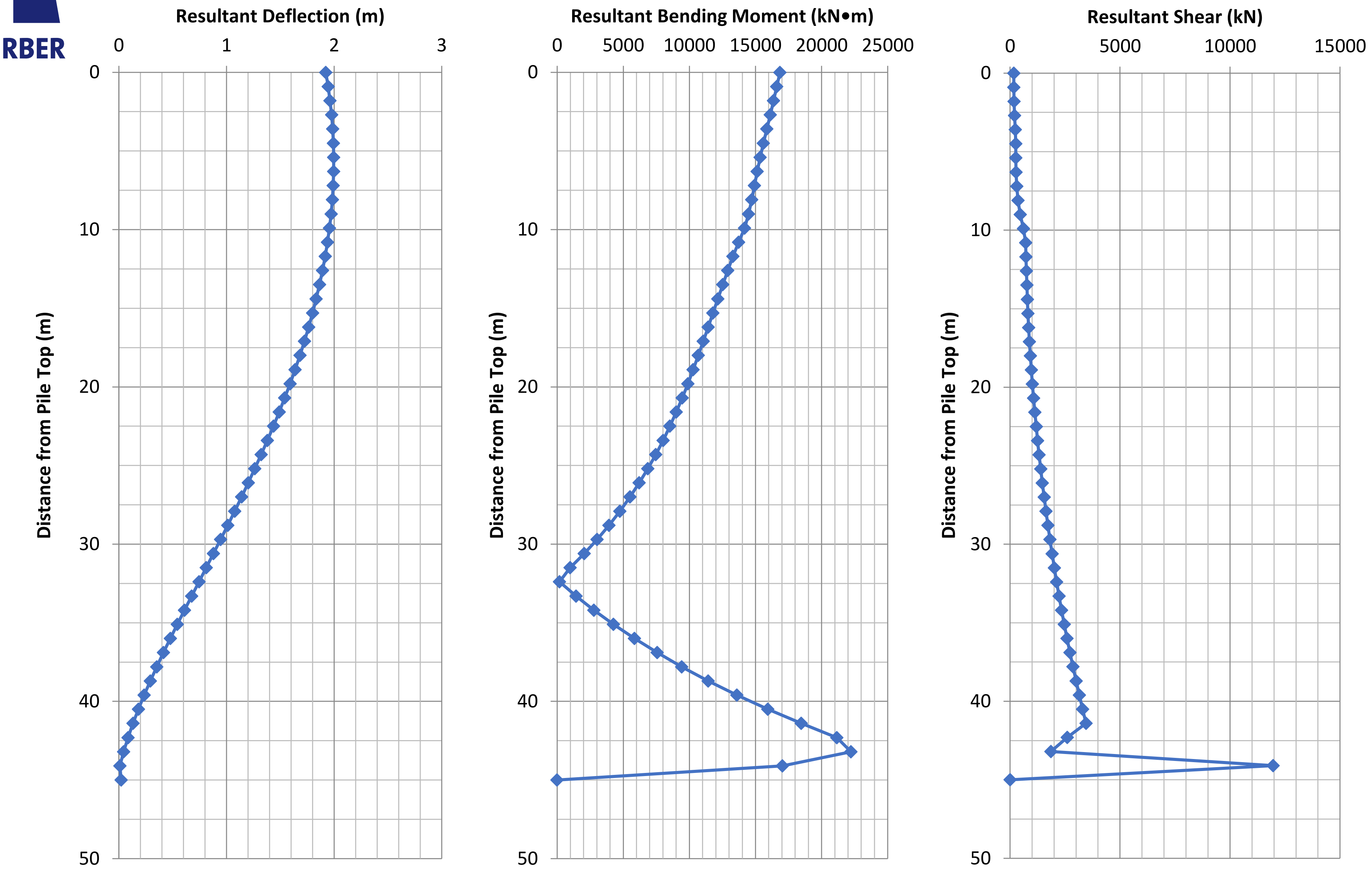


Figure F2 - Lateral Pile Analysis

Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)

Pile P2 - Trestle Abutment

Client: DP World Fraser Surrey
File No.: 34098

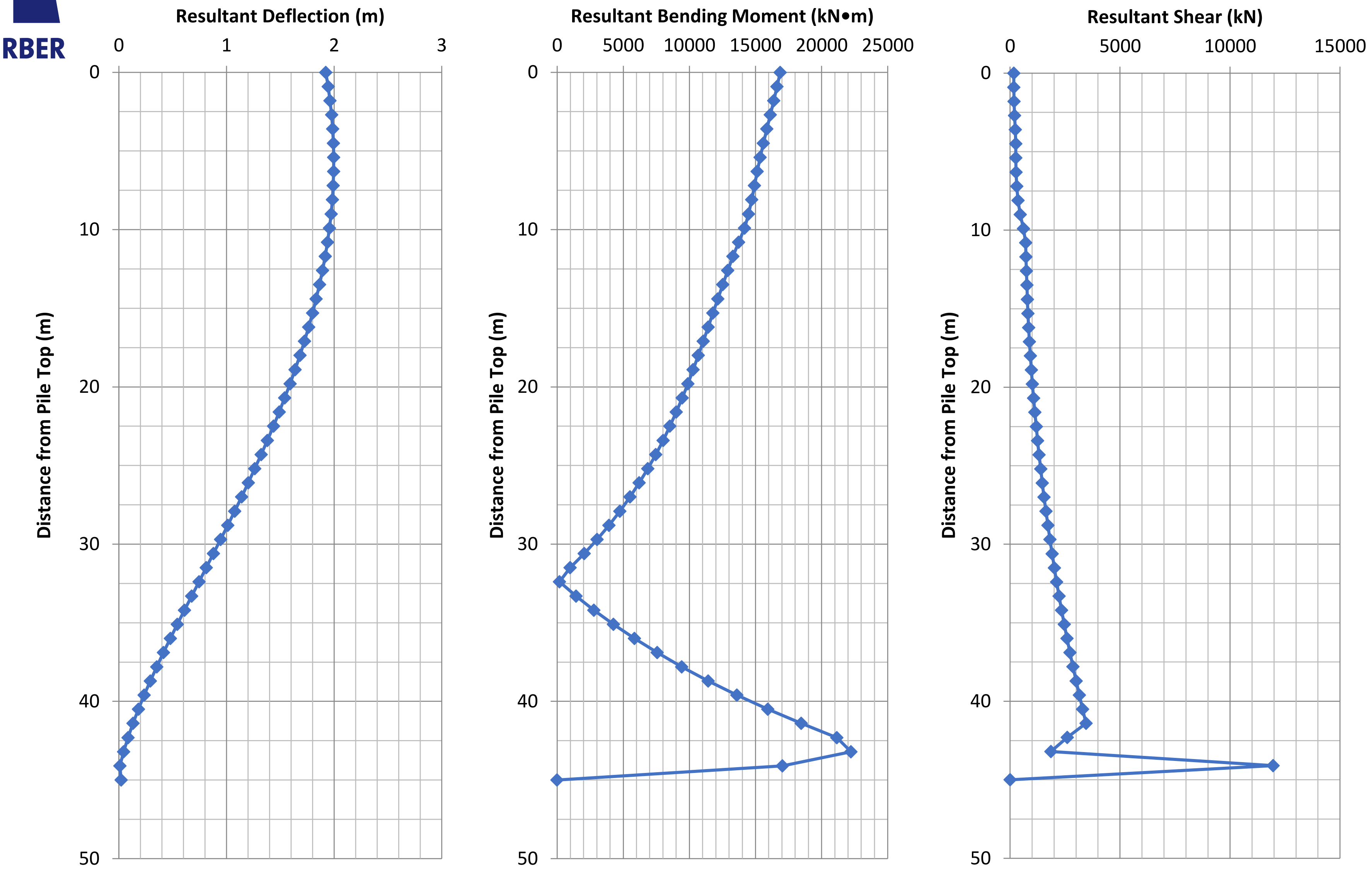


Figure F3 - Lateral Pile Analysis

Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)

Pile P3 - Trestle Abutment

Client: DP World Fraser Surrey
File No.: 34098

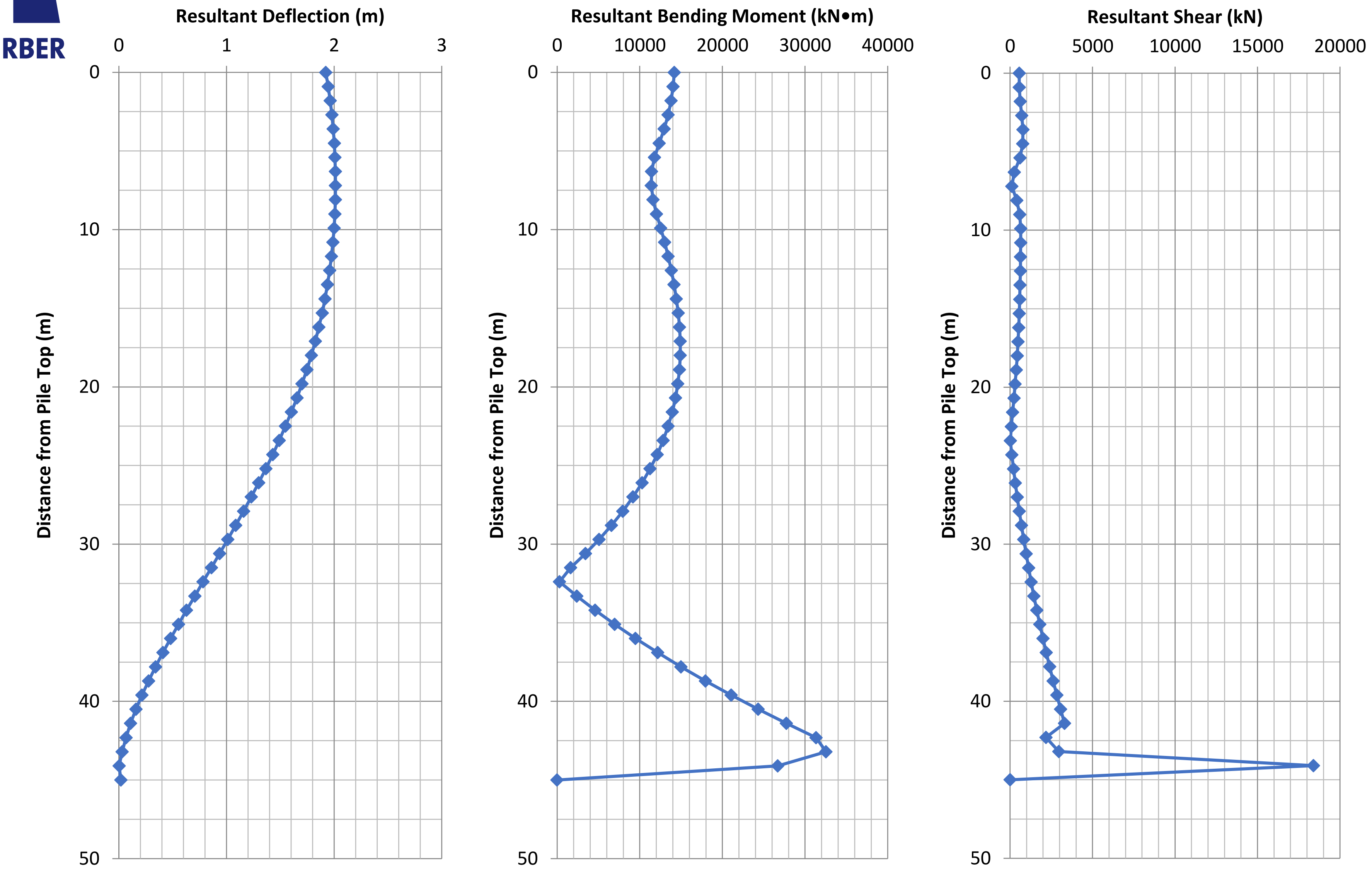


Figure F4 - Lateral Pile Analysis
Kinematic Seismic Loading Condition (1:2,475 Earthquake Event)
Pile P4 - Trestle Abutment

Client: DP World Fraser Surrey
File No.: 34098

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APPENDIX G

Unfactored Axial Geotechnical Resistance Plots for Offshore piles (2 pages)

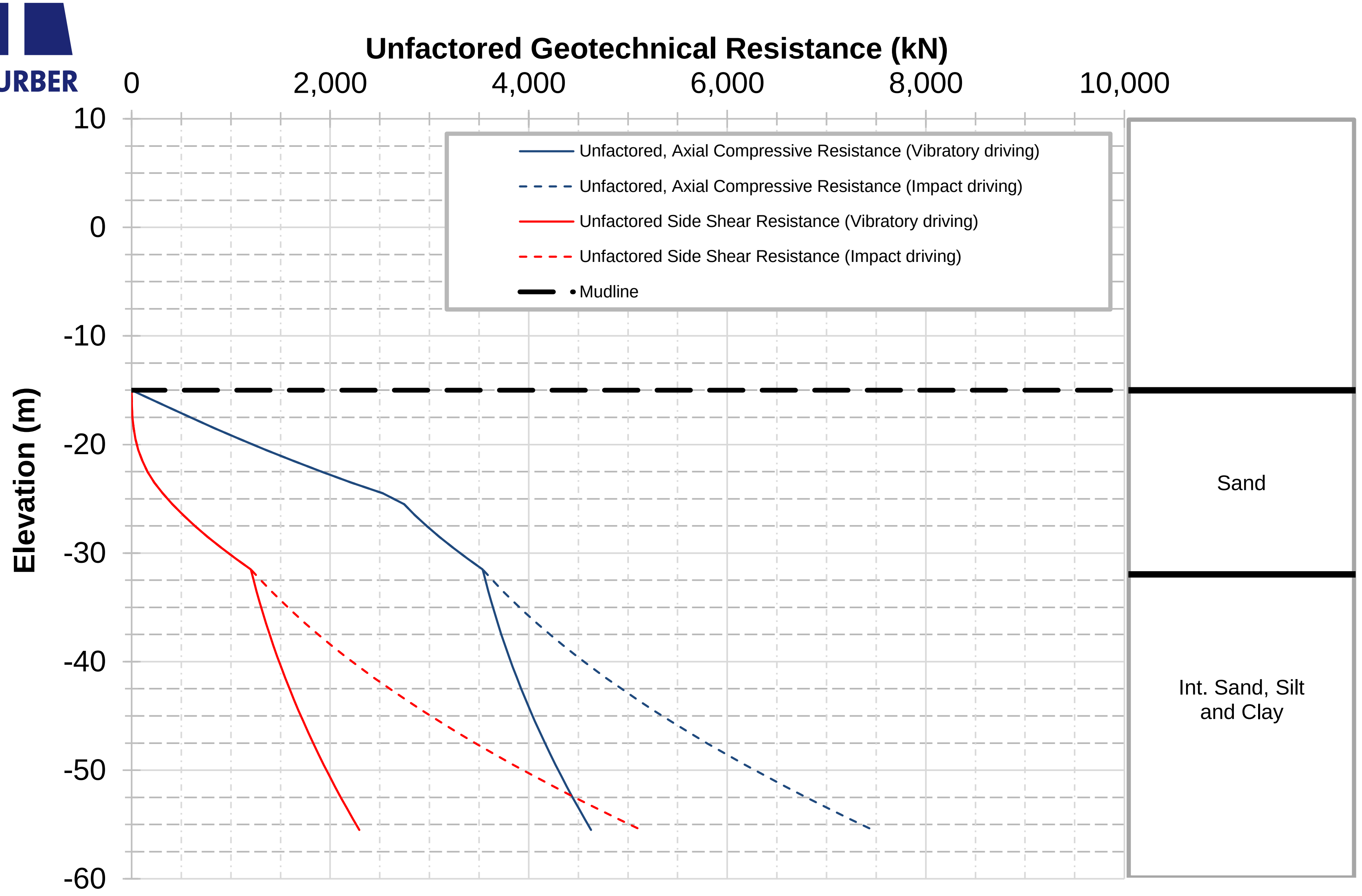

Figure G1 - Unfactored Axial Pile Resistance versus Depth
Static and Non-Liquefied Seismic Loading Conditions
1,219 mm Open-Ended Steel Pipe Pile
Mudline at El. -15 m

Client: DP World Fraser Surrey
File No.: 34098

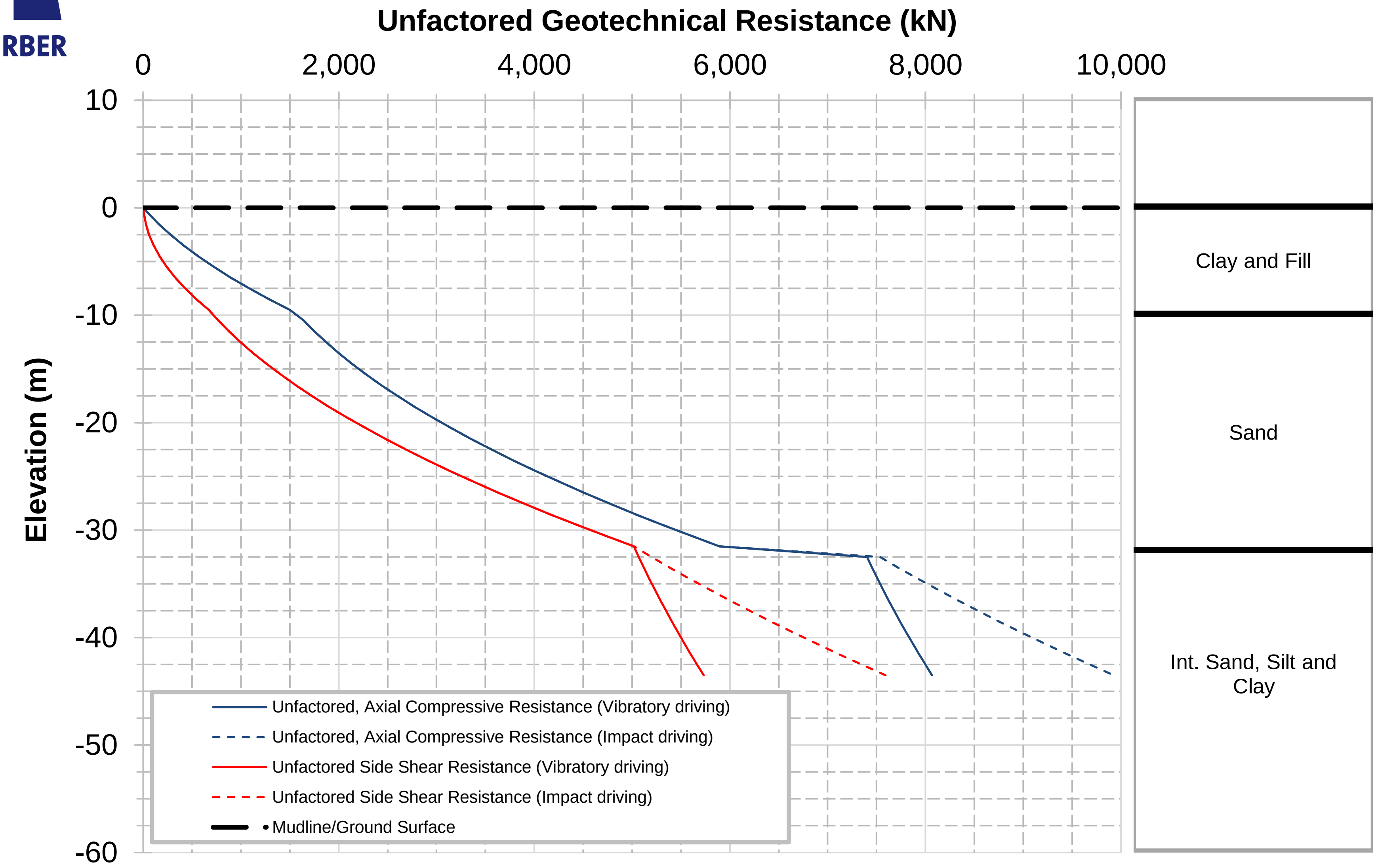

Figure G2 - Unfactored Axial Pile Resistance versus Depth
Static and Non-Liquefied Seismic Loading Conditions
1,219 mm Open-Ended Steel Pipe Pile
Mudline at El. 0 m

Client: DP World Fraser Surrey
File No.: 34098

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APPENDIX H

Unfactored Axial Geotechnical Resistance Plots for Tank Farm Piles (2 pages)

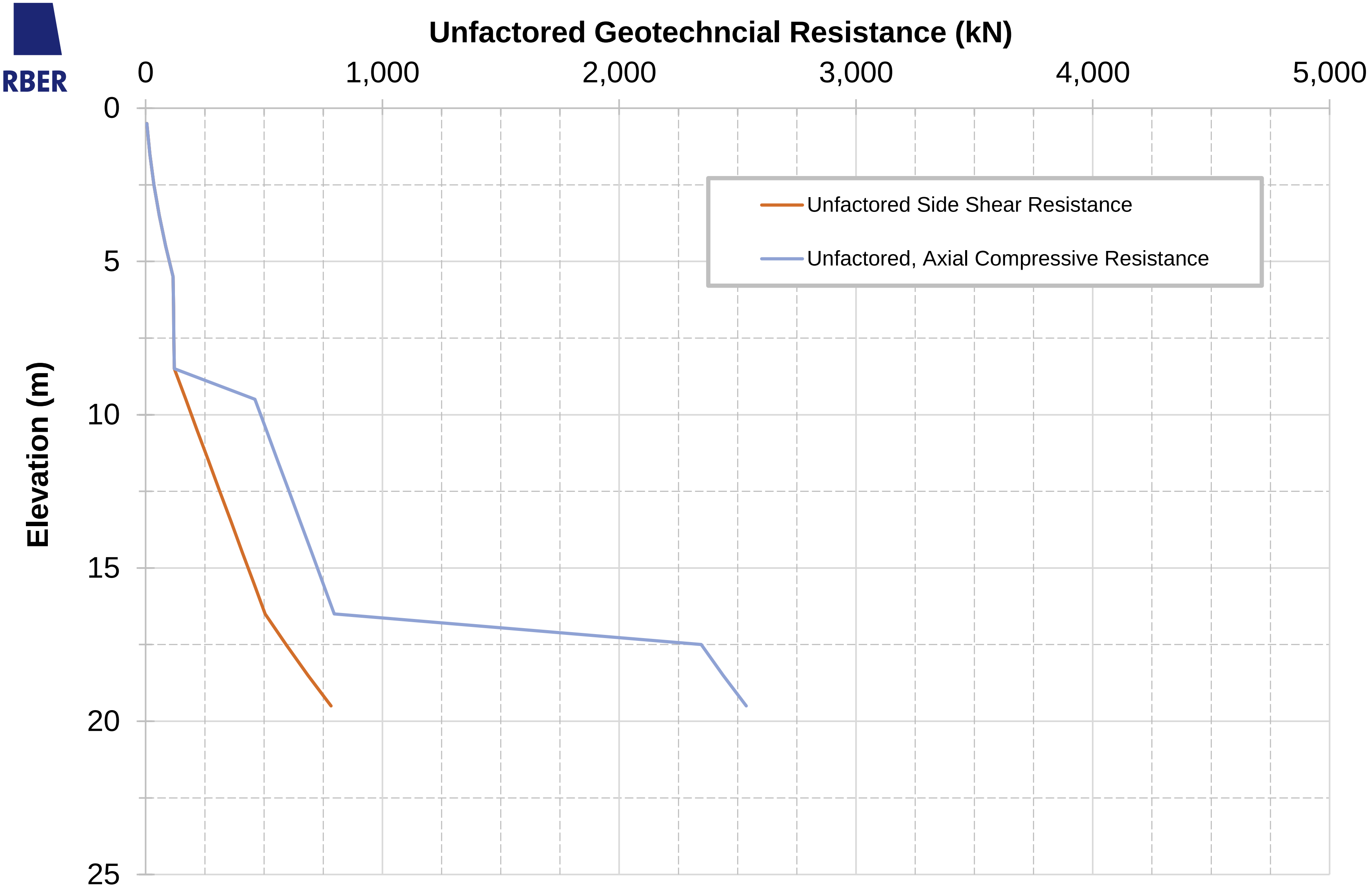

Figure H1 - Unfactored Axial Pile Resistance versus Depth
Static and Non-Liquefied Seismic Loading Conditions
610 mm Closed-End Pipe Piles

Client: DP World Fraser Surrey
File No.: 34098

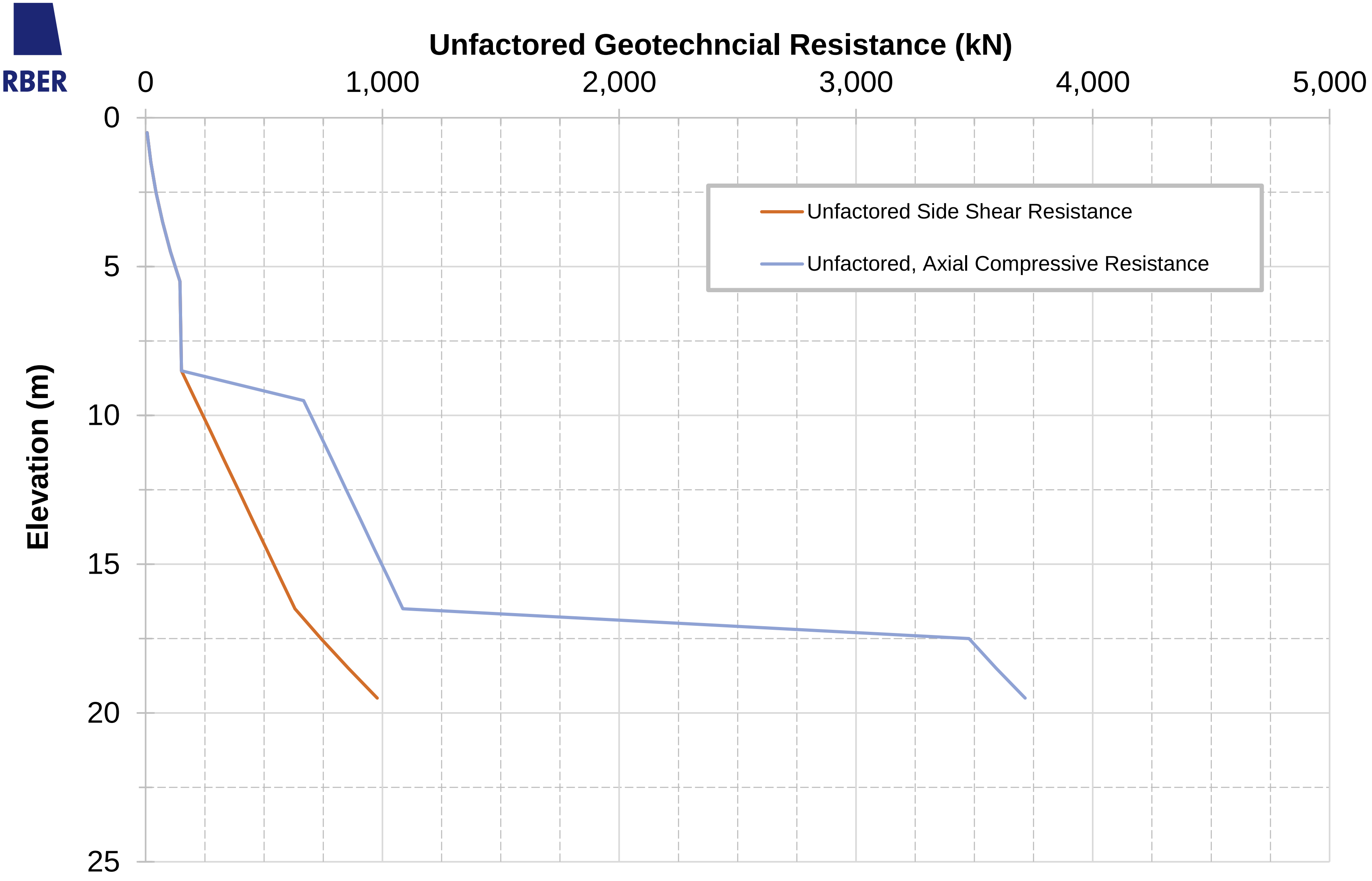

Figure H2 - Unfactored Axial Pile Resistance versus Depth
Static and Non-Liquefied Seismic Loading Conditions
762 mm Closed-End Pipe Piles

Client: DP World Fraser Surrey
File No.: 34098

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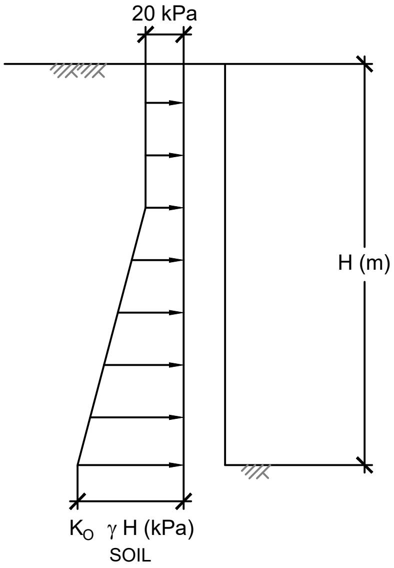
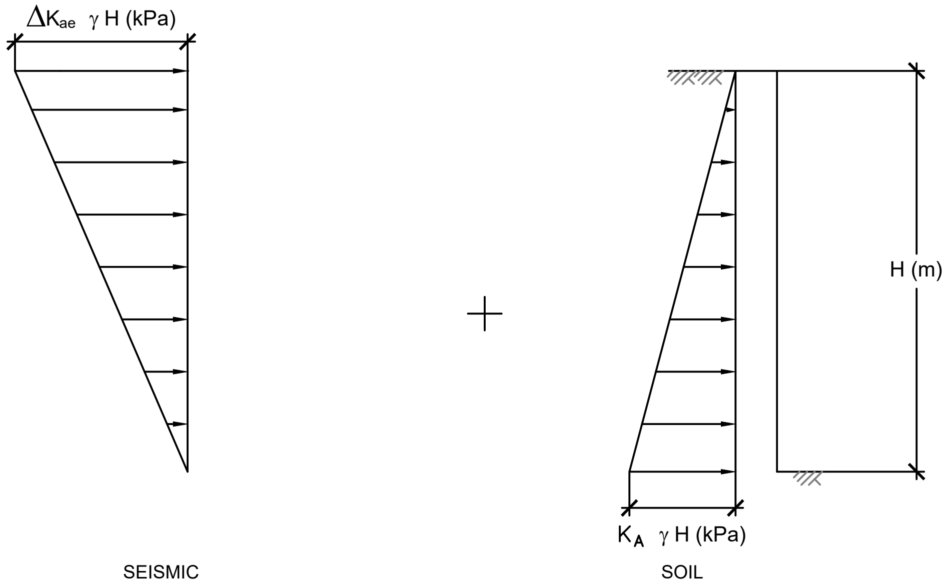
APPENDIX I

Lateral Earth Pressure Distribution Diagram for Basement Walls (1 page)

Plotted: June 30, 2022



STATIC LOADING CONDITION



SEISMIC LOADING CONDITION

LOADING CONDITIONS			
γ (kN/m^3)	K_O	K_A	ΔK_{ae} (2475 YR EQ)
20	0.43	0.27	0.46

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THURBER ENGINEERING LTD.

SACRÉ - DAVEY ENGINEERING INC.

LATERAL EARTH PRESSURE DIAGRAM

DP WORLD FRASER SURREY
CANOLA OIL TRANSLOAD FACILITY
SURREY, BC

DESIGNED	DRAWN	APPROVED	DATE	SCALE	PROJECT No.	FIGURE. NO.	REV.
TFD	MOM		JUNE 15, 2022	N.T.S.	34098	I-1	0