Kicking Horse River Bridges Replacement Project
Geotechnical Design Report
100% Detailed Design

PRESENTED TO

Urban Systems Ltd.
British Columbia Ministry of Transportation and Infrastructure

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ACRONYMS & ABBREVIATIONS

Acronyms/Abbreviations	Definition
AASHTO	American Association of State Highway and Transportation Officials
API	American Petroleum Institute
BC MoTI, MoTI	British Columbia Ministry of Transportation and Infrastructure
C_E	Energy Correction Factor
CFEM	Canadian Foundation Engineering Manual
CHBDC	Canadian Highway Bridge Design Code
CRR	Cyclic Resistance Ratio
CSR	Cyclic Stress Ratio
DoU	Degree of Understanding
Es	Soil Modulus of Elasticity
ER_m	Measured Energy Ratio for SPT
ETR	Energy Transfer Ratio
GRF	Geotechnical Resistance Factor
GSC	Geological Survey of Canada
iBPT	Instrumented Becker Penetration Test
k_h	Horizontal seismic coefficient
KHR	Kicking Horse River
LPT	Large Penetration Test
MSE	Mechanically Stabilized Earth
N	Field SPT Values
N_{60}	SPT Values for 60% Energy Ratio
NBCC	National Building Code of Canada
PGA	Peak Ground Acceleration
SPT	Standard Penetration Test
Tetra Tech	Tetra Tech Canada Inc.
USL	Urban System Ltd.
V_s	Shear Wave Velocity
ϕ'	Effective Peak Friction Angle

LIMITATIONS OF REPORT

This report and its contents are intended for the sole use of Urban System Ltd. (USL) and the British Columbia Ministry of Transportation and Infrastructure (BC MoTI) and their agents. Tetra Tech Canada Inc. (Tetra Tech) does not accept any responsibility for the accuracy of any of the data, the analysis, or the recommendations contained or referenced in the report when the report is used or relied upon by any Party other than USL and BC MoTI, or for any Project other than the proposed development at the subject site. Any such unauthorized use of this report is at the sole risk of the user. Use of this document is subject to the Limitations on the Use of this Document attached in Appendix A or Contractual Terms and Conditions executed by both parties.

1.0 INTRODUCTION

Tetra Tech Canada Inc. (Tetra Tech) presents Urban Systems Ltd. (USL) and the British Columbia Ministry of Transport and Infrastructure (BC MoTI) with the following Geotechnical Design Report for the 100% Detailed Design submission for the Highway 95 Kicking Horse River (KHR) Bridges Replacement Project. The project includes the replacement of the existing KHR Bridges 1 and 2 (including their approaches) and upgrading 800 m of the highway.

The existing Hwy. 95 consists of two bridges on the same alignment crossing Kicking Horse River and providing access to Gould's Island. The existing Bridge 1 crosses the main channel and Bridge 2 crosses the smaller (back) channel on the south side. The proposed improvement relocates the Hwy. 95 crossing to the east with the new Kicking Horse River Bridge No. 1 (KHR1); and provides separate access to Gould's Island with the new Kicking Horse River Bridge No. 2 (KHR2). The general arrangement is shown on Figure 1.

The contents of this report are limited to the geotechnical assessment that is carried out to define the geotechnical requirements and recommendations for the foundations of the proposed structures. The assessment and recommendations related to pavement design are not included in this report and are provided in a separate document.

For completeness, this report should be read in conjunction with our factual geotechnical data presented in "*Kicking Horse River Bridges Replacement Project Geotechnical Factual Data Report – Phases 1 and 2*" (Factual Data Report), dated May 2022, and Limitations on the Use of this Document included in Appendix A.

1.1 Design Requirements and Documents

The geotechnical design was carried out in accordance with the criteria outlined in BC MoTI Technical Circular T-04/17 and the Works/Services Schedule for Contract 872CS1667.

The following design codes and documents have been used to develop the design basis:

- BC MoTI Bridge Standards and Procedures Manual, Volume 1, Supplement to CAN/CSA S6-19.
- Canadian Highway Bridge Design Code (CHBDC), CAN/CSA S6-19.
- AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
- Canadian Foundation Engineering Manual (CFEM), 4th Edition, 2006.
- 100% Detail Design Drawings, R2-1131-000, April 6, 2023.
- Kicking Horse River Bridge 1 – 50% Detailed Design Criteria.
- Kicking Horse River Bridge 2 – 50% Detailed Design Criteria.
- Kicking Horse Bridge 1 Drawings, 90% Review Submission, November 22, 2022.
- Kicking Horse Bridge 2 Drawings, 90% Review Submission, November 22, 2022.

1.2 Design Criteria

The following criteria for specific project components have been used for developing the design.

1.2.1 Bridge Importance Category

Important categories have been defined as “Major Route” for KHR1 and “Other” for KHR2.

1.2.2 Seismic Performance for Walls and Embankments

The seismic performance criteria outlined in the BC MoTI Supplement to CHBDC S6-19, 6.14.2.1 and 6.14.2.3, are used. The following seismic performance criteria are used for the geotechnical systems within the bridge approach embankment interface zone:

- Major-route geotechnical systems shall have 100% of the travelled lanes available for use following ground motions with a return period of at least 475 years. Any repair work shall not cause service disruption.
- Other geotechnical systems shall have 50% of the travelled lanes, but not less than one, available for use following ground motions with a return period of at least 475 years. If damaged, normal service shall be restorable within one month.
- Major-route and other geotechnical systems shall meet the no-collapse requirement following ground motions with a return period of at least 2475 years and shall be possible to evacuate the bridge safely.

For geotechnical systems outside the bridge approach embankment interface zone and assigned a Seismic Performance Category 2, the following seismic performance criteria are used:

- Major-route geotechnical systems shall have at least 50% of the travelled lanes, but not less than one, available for use following ground motions with a return period of at least 475 years. If damaged, normal service shall be restorable within one month.
- Other geotechnical systems shall have at least 50% of the travelled lanes, but not less than one, restorable for use within one month following ground motions with a return period of at least 475 years.

The pseudo-static slope stability analyses use a horizontal seismic coefficient equal to 0.5 times the peak ground acceleration (PGA), as per CHBDC S6-19 (Clause 6.14.9.1).

The National Building Code of Canada (NBCC 2020) seismic hazard parameters have been used to obtain peak ground acceleration (PGA) and other hazard values.

1.2.3 Service Life Requirements

A design service life of 75 years was used for time-dependent geotechnical analyses.

1.2.4 Consequence Classification

The “Typical” consequence classification was used in the geotechnical design, as instructed by BC MoTI. As per Table 6.1 of CAN/CSA S6-19, the consequence factor for Typical Consequence is 1.0.

1.2.5 Mechanically Stabilized Earth Walls

The geotechnical resistance factors and factors outlined in CAN/CSA S6-19 and BC MoTI Supplement to CHBDC S6-19 will be applied to the Mechanically Stabilized Earth (MSE) wall design.

1.2.6 Degree of Understanding

Based on the geotechnical testing performed at the project site, a Typical Degree of Understanding (DoU) is used for the geotechnical design.

1.2.7 Traffic Loading

A traffic surcharge of 16 kPa is used for global stability evaluation.

1.2.8 200-Year Design Flood

Where applicable, slope stability analyses will consider the 200-year return period flood elevation of 788.9 m based on the information provided on the current design drawings by others. The flood condition will be checked against the required factor of safety presented in Table 6.2b of the BC MoTI Supplement to CHBDC CAN/CSA S6-19 for Temporary conditions. Due to high permeability of the MSE fill, drawdown of water levels from any flood elevation surrounding the retaining wall will be considered a drained condition.

1.2.9 Scour

Based on the input provided by hydrotechnical and structural engineers, the design of the pile foundation of the pier has considered a scour elevation of 782.5 m for axial requirements and 785.4 m for lateral load analysis; and no scour has been considered for the abutments. Details related to the design scour parameters will be presented by the USL project team in a separate document.

2.0 GEOLOGICAL AND GEOTECHNICAL CONDITIONS

2.1 Surficial Geology

The project site is situated at the confluence of the Kicking Horse River (KHR) and the Columbia River. Based on the information provided in Geological Survey of Canada Open File 7631 and Open File 8236 (Surficial Data Model V2.0), the surficial geology in the valley (including the town of Golden) comprises a thick and continuous glacial sediment (till) blanket (Tb). Below the glacial sediments are Cambrian to Ordovician age sedimentary rocks of the McKay Group (CmOM) consisting of argillite, shale, and limestone (BC Geological Survey Open File 2017-8).

2.2 Available Geotechnical Information

Previous geotechnical information available for the site were provided by BC MoTI and are presented in the following document:

- Highway 95 KHR Bridges Final Preliminary Geotechnical Report, Golder Associates Ltd. (Golder), October 1, 2020.

The preliminary site exploration program conducted in July 2020 (Golder, 2020) consisted of four drilling and sampling boreholes. The four boreholes were performed along the preliminary bridge alignments in the areas near the proposed foundations.

Tetra Tech reviewed the information presented in the Golder reports and determined a subsurface exploration program was necessary to obtain additional data for characterizing the subsurface conditions underlying the proposed bridge crossings, the new highway alignment, and connections with the local traffic network. The geotechnical data obtained during two phases of the 2021 program are presented in our Factual Data Report (Tetra Tech, 2022). The subsurface exploration program included one Instrumented Becker Penetration Test (iBPT), seven (7) sonic boreholes, and one additional hydrovac excavation location. The 2020 and 2021 borehole locations are shown on Figure 1.

2.3 Subsurface Ground Conditions

The subsurface soil characterization presented herein focuses on the bridge crossings and highway realignment. The general site plan showing the locations of the iBPT and borehole locations relevant to the proposed bridge structures and the north and south approaches are presented on Figure 2, which also provides the interpreted soil profile along the L100 alignment. The soil conditions along the L100 alignment (KHR1) are reasonably uniform and are considered applicable to the L400 alignment (KHR2). Five main soil units were identified to represent the soil stratigraphy at the project site.

2.3.1 Soil Units

Fill (Soil Unit I): A layer of fill was encountered in some boreholes to depths ranging from about 2 m to 5 m. This layer consists of mixtures of sand, gravel, and silt. Inclusions of bitumen were observed in both BH21-05 and BH21-08 to depths of about 0.5 m and 0.8 m, respectively.

Sand and Gravel with Cobbles (Soil Unit II): This unit was encountered underlying topsoil and/or the fill layer extending to depths of about 1 m to 15 m. The unit is mainly comprised of sand and gravel and contains some cobbles and fines. Cobbles up to 250 mm in diameter were observed in the testholes along with fine to coarse, subangular to rounded gravel, and fine to coarse sand. The size of the cobbles encountered during drilling is limited by the diameter of the drill string, such that particles larger than the core bit may be present. To install piles into this soil unit, the potential presence of large particles should be accounted for. The unit is generally dense to very dense, grey-brown in colour, and dry to wet.

Gravel (Soil Units IIIa and IIIb): This unit was encountered underlying Unit II and extends to depths of about 37 m to 42 m. The unit is generally sandy and dense to very dense to depths of about 22 m (Unit IIIa), then becomes less sandy and very dense (Unit IIIb). The gravel is fine to coarse, subangular to rounded and the sand is fine to coarse. Cobbles were recovered in the unit up to 100 mm in diameter. Similar to Unit II, the presence of larger particles is possible within this unit, since the size of the recovered samples is limited by the diameter of the drill string. The unit is damp to wet and grey-brown in colour.

Silty Sand (Soil Unit IV): This unit was encountered in BH20-02, BH21-01 and BH21-03 underlying Unit IV and extending to depths of about 40 m to 52 m. The unit consists of silty sand with trace to no gravel. Occasional silt laminations up to 30 mm in thickness were observed throughout the samples from this unit. The sand was fine, yellow-brown in colour, wet, and compact.

Silt and Sand (Soil Unit V): This unit was encountered at elevations between about El. 737 m and El. 743 m, below Unit IV. The layer extends to the maximum depth of exploration (i.e., 64 m in BH21-03). The unit is comprised of sand and silt to sandy silt. Interbedded silt layers were encountered in the lower part of the unit. The sand is fine to medium, and the unit is moist to wet and grey-brown in colour. The unit varies in consistency from compact to very dense, becoming very stiff to hard where the fines content increases. At the location of BH21-03, the transition from the above unit (Unit IV) is evident from the change in color from yellowish brown to grey at 45.5 m depth, along with an increase in silt content.

2.4 Groundwater Conditions

The groundwater level is expected to vary with the water level in the river as a result of seasonal changes. The data obtained from the 2020 and 2021 subsurface explorations indicate that the groundwater level was located at depths of about 3 m to 7.5 m below existing ground (depending on the location) at the time of the field work. In addition to the field observations, the historical data available on the BC Groundwater Wells and Aquifers database were reviewed. The wells that were in the vicinity of the project site recorded static water level at depths varying between 3 m and 4.6 m below ground surface.

As shown on the current design drawings, the "average August water level" in the river is estimated to be located at about El. 786.0 m.

For the geotechnical engineering analyses performed for the bridge and its approach embankments, the groundwater level is taken to be a reasonable average for the site, at an elevation of El. 788 m (i.e., about one meter below the Q200 high-water level).

2.5 Depth of Frost Penetration

The depth of frost penetration for the granular soils at the project site was estimated using the formulae presented in the 4th. edition of the Canadian Foundation Engineering Manual (2006). The parameters used in the analysis are listed as follows:

- Mean freezing index = 876.2 degree-days (obtained from Climate Atlas of Canada)
- Mean annual air temperature of 4.2 °C (Climate Atlas of Canada)
- Freezing duration of 191.5 days (Climate Atlas of Canada)
- Ground surface interface factor n = 0.95
- A dry unit density of 1800 kg/m^3 and a moisture content of 10%
- Frozen thermal conductivity of frozen soil = 2.3 W/(m-K) representative of the above soil properties

Based on the above parameters, a depth of frost penetration of 2.5 m was determined. With the groundwater level located at depths of more than 3 m and the fines content of the soil less than 20%, the frost susceptibility of the near-surface soil at the project site is determined to be low to medium. Considering the natural granular soils over the depth of frost penetration may have a sufficiently fine-grained nature that ice lenses could result, the natural granular soils over the frost penetration depth (i.e., 2.5 m) should be removed and replaced with non-frost-susceptible materials.

It is noted that depths of frost penetration are strongly dependent on soil type, ground cover and degrees of saturation of the near-surface soils. Where the near-surface, compacted fills are allowed to become saturated, the depth of frost penetration will increase. Therefore, it is recommended that measures be undertaken to minimize water ingress into the fills around the support structures (e.g., foundation footings).

3.0 DATA INTERPRETATION

The analysis and design for the project is carried out using soil parameters interpreted from the results of the in-situ and laboratory tests performed on soil samples recovered from the boreholes completed during the site exploration. Details of the in-situ and laboratory testing are presented in the referenced Factual Data Report, and the subsequent sections describe the methods used for interpreting the data.

3.1 Standard Penetration Test

The Standard Penetration Test (SPT) is a common sampling method that also provides an indication of soil consistency/density. The penetration values obtained in the field were used to correlate engineering parameters and to assess the resistance against liquefaction. SPTs were performed in all sonic boreholes at selected depth intervals. Testing was performed using a 63.5 kg (140 lb) SPT automatic trip hammer, except for BH21-03 and the uppermost 11 m of BH21-09 during which a 136 kg (300 lb) Large Penetration Test (LPT) hammer weight was used in the SPT automatic trip hammer. The data interpretation was carried out primarily using the SPT data and SPT-based correlations, supplemented by the penetration data obtained from the LPT.

For the work completed previously by Golder, the information provided to us indicated the Energy Transfer Ratio (ETR) for their SPT hammer varies between 92.6% and 98.4%. For the design, an average energy ratio of 95.8% was considered for the Golder testholes to correct the penetration values to standard values (N_{60}) corresponding to an energy ratio of 60%.

For the hammer used in the 2021 exploration, measurements of the ETR were taken by the drilling contractor. The energy ratios were measured to be about 75% of the theoretical maximum possible for the ASTM standard test using the 63.5 kg hammer weight. Details of the energy calibration are presented in the Factual Data Report. These values were used to correct the field-measured SPT N-values to standard values (N_{60}) corresponding to an energy ratio of 60% (Seed et al., 1984). The energy correction factor (C_E) is computed as follows:

$C_E = ER_m / 60$, where ER_m is the measured energy ratio of the SPT performed at the site.

Other standard correction factors that are required to convert the N-values to N_{60} were also used. The rod length correction varies with the test depth; and the length of the extension above ground was taken as 1.5 m. The SPT N_{60} values are presented on Figure 3.

3.2 Instrumented Becker Penetration Test

An Instrumented Becker Penetration Test (iBPT) was performed to collect downhole energy measurements and iBPT blow counts, which were correlated to equivalent SPT N_{60} values. Details of the iBPT testing, including the profiles, are presented in the Factual Data Report. The near-continuous iBPT data supplements the borehole data when defining the soil stratigraphy.

The correlated N_{60} values are presented on Figure 3 along with the SPT N_{60} values.

4.0 GEOTECHNICAL PARAMETERS

4.1 Soil Parameters

Geotechnical design parameters, including drained (effective) peak friction angle (ϕ') and soil modulus of elasticity (E_s) were defined using the results of penetration tests. The geotechnical parameters defined to carry out the engineering analyses for the bridge foundations and approaches are presented in Table 4-1.

The depth variations of the soil layers present at each foundation element are taken into consideration in the engineering analyses.

Table 4-1: Soil Parameters

Soil Unit	Soil Type	N_{60}	Unit Weight (kN/m³)	ϕ' (°)	E_s (MN/m²)
I	Fill	> 50	19	38 – 40	50 – 100
II	Sand and Gravel with Cobbles	30 – 60	19	38 – 40	30 – 60
III	Gravel	> 40	19	38 – 40	40 – 130
IV	Silty Sand	> 50	19	40	60 – 120
V	Sand and Silt	> 50	19	38	50 – 100

4.1.1 Total Unit Weight

Due to the limitations of sampling the gravelly soils at the project site, typical values were used for the identified soil units. The potential variation in total unit weight is not expected to have significant impact on the geotechnical evaluation.

4.1.2 Fines Content and Particle Size Distribution

The fines contents obtained from laboratory tests on recovered samples are presented on Figure 4 for the granular materials. The completed particle size distribution test results are presented in the Factual Data Report. The fines content is an important parameter in determining the liquefaction resistance of granular soils.

The fines contents from laboratory tests, where available, have been used in the soil characterization.

4.1.3 Friction Angle

The peak effective friction angle (ϕ') of the granular soils were determined using the empirical SPT-based correlations proposed by Schmertmann (1975), Peck, Hanson, and Thornburn (1974) and Hatanaka & Uchida (1996). The average strength values obtained from the three approaches are presented on Figure 5. The interpreted soil strength suggests the soils extending to the depth of exploration are generally dense to very dense with friction angles of 40° or higher. For practical reason, the friction angle used in the design is capped at 40°.

4.1.4 Soil Modulus of Elasticity

The N_{60} measurements provide an indication of the soil density and were also used to estimate the soil modulus of elasticity. Ranges of modulus of elasticity of the granular soils were estimated using the correlation proposed by Kulhawy and Mayne (1990).

4.1.5 Sulphate Content and Soil Resistivity

The soluble sulphate ion content was measured in five (5) soil samples that were recovered at depths between 0.5 m and 3.4 m across the site, based on the input received from structural and pipe designers in terms of types of tests and sample locations. The results of the chemical testing performed on all soil samples indicate low sulphate contents of 0.01% to 0.04%. Based on the results of sulphate testing, the class of exposure was determined to be negligible (CAN/CSA-A23.1, Table 3).

The results of all electrochemical tests are presented in Appendix B and are summarized in Table 4-2.

Table 4-2: Sulphate Ion Content and Soil Resistivity

Test	Results
Soluble Sulphate (CSA A23.2-2B and A23.2-3B)	0.01% - 0.04%
Water Soluble Chloride in Concrete (ASTM C1218)	0.000% - 0.009%
Organic Content in Soil (AASHTO T267)	0.5% - 3.8%
pH of Soil for Use in Corrosion Testing (AASHTO T289)	8.47 – 9.49
Resistivity using Two-Electrode Soil Box Method (AASHTO T288)	3,533 Ohm.cm – 15,333 Ohm.cm
Water-Soluble Chloride Ion Content (AASHTO T291)	11 ppm – 36 ppm
Water-Soluble Sulfate Ion Content (AASHTO T290)	6 ppm – 74 ppm

5.0 SEISMIC DESIGN CONSIDERATIONS

5.1 Seismic Hazard

The seismic hazard for the project site was obtained from Earthquakes Canada website considering the 6th generation seismic hazard model developed for NBCC 2020 (Natural Resources Canada), and is presented in Appendix C. The PGA for each of the three design seismic events are shown in Table 5-1 below for a Class C site (360 m/s < V_s < 760 m/s).

Table 5-1: Peak Ground Acceleration

Return Period (years)	PGA (g)
475	0.0489
975	0.0824
2475	0.149

The SPT data obtained from the site exploration indicate the average penetration resistance in the top 30 m is greater than 50, which designates the site as Class C (very dense soil). As per CHBDC CAN/CSA S6-19, the site coefficient is 1.00 and the above PGA values can be used to carry out the geotechnical analyses.

The response spectra corresponding to a Class C site for the three design earthquakes are presented on Figure 6.

The 6th generation seismic hazard model deaggregation data are not yet available. The 2015 seismic hazard deaggregation for the site was obtained from the Geological Survey of Canada (GSC). The seismologist from Canadian Hazards Information Service indicated the 2020 deaggregation data are expected to be similar to the 2015 deaggregation. Based on the 2015 deaggregation data corresponding to the PGA, an earthquake magnitude of 6.14 was considered. The 2015 deaggregation data provided by GSC are included in Appendix C.

5.2 Liquefaction Assessment

To evaluate the liquefaction triggering potential, the cyclic stress ratios (CSR) were determined in accordance with the procedure outlined by Boulanger and Idriss (2014) using the PGA values. The calculated CSR profile of the 2475-year earthquake is shown on Figure 7. A maximum CSR of 0.14 was determined for the 2475-year event.

The cyclic resistance ratios (CRR) were estimated using the SPT-based approach outlined in the Commentary (C4.14.8.1) to CAN/CSA S6-19, which is based on the liquefaction triggering analysis proposed by Boulanger and Idriss (2014). The high SPT penetration values (Figure 3) indicate the granular soils at the project site have CRR values higher than 0.5; therefore, these soils are determined to be not susceptible to liquefaction under the design earthquake loading.

6.0 PILE FOUNDATIONS

This section provides the geotechnical input that is required to develop the pile-supported foundations for KHR1 and KHR2.

The pile size and configuration were defined based on the results of pile group analyses and the structural loads provided to date by COWI for KHR1. Several iterations were required to converge on a pile layout. Detailed structural analyses and definition of foundation requirements for KHR1 have been completed by COWI using the geotechnical input presented herein.

For KHR2, pile group analyses were conducted by Tetra Tech using the foundation loads and pile layout provided by COWI to evaluate the pile group response and to determine the required pile embedment lengths. The pile responses obtained from the pile group analyses conducted by Tetra Tech were provided to COWI for structural design the structural requirements.

This report presents the axial capacity and lateral load-deflection characteristics for the use of pile foundations of KHR1, and the results of pile group analyses for the KHR2 foundation. Driven steel piles with a diameter of 762 mm and a wall thickness of 15.9 mm are used based on discussions with structural engineers (COWI) and BC MoTI team.

6.1 Axial Capacity

The static axial capacity (i.e., side resistance) within soil units was calculated using the approach outlined in CAN/CSA S6-19, and for comparison, the API RP2A (2003) method was also used. The results obtained from CAN/CSA S6-19 should be used for the design. The pile axial capacity values provided herein are applicable to both KHR1 and KHR2, considering the soil conditions are similar for design purposes and the same pile size is used for both bridges.

The variations with depth of the ultimate axial capacity in compression and tension for the 762 mm diameter piles are presented on Figures 8 and 9, respectively. The capacity curves presented herein consider only driven piles with limited soil disturbance. Other installation methods will be subject to review and approval. Vibratory and/or drilling methods may be considered for advancing the piles at shallow depths, but the final pile section should be driven to demonstrate the pile embedment is adequate for the design load, by achieving a pile set and/or performing high-strain dynamic tests (e.g., PDA).

At the pier location of KHR1, scour is considered for the pile foundation. Based on the information provided to Tetra Tech, scour elevation of El. 782.5 m should be used to evaluate the axial requirements at the pier location. The capacity curves in compression and tension accounting for a scour event that extends to 4.1 m below the top of the piles are presented on Figure 10.

Based on a typical degree of understanding of the geotechnical conditions, the factored axial pile capacity under static conditions should be evaluated using geotechnical resistance factors of 0.40 for compression and 0.30 for tension. For seismic conditions, the geotechnical resistance factors can be increased to 0.6 for compression and 0.5 for tension. A consequence factor of 1.0 (i.e., Typical) should be considered.

6.2 Lateral Load-Deflection Characteristics

For soil-structure interaction analyses, the response of the pipe piles to lateral loading was defined by means of non-linear p-y curves, determined in accordance with the recommendations provided by API RP2A (2003). It is understood that the p-y curves will be used to evaluate the performance of the pile-supported structure. Considering that soil degradation under the design earthquake is minimal, if any, the p-y curves that were developed using the representative parameters (Table 4-1) are applicable to both static and seismic conditions. The p-y curves for the 762-mm pile are presented in Appendix D.

The p-y curves are ultimate values since the geotechnical resistance factor has not been included. As per CAN/CSA S6-19, the geotechnical resistance factors below should be considered and be applied as p-multipliers. Reduction factors should not be applied to displacement-based analyses.

- Geotechnical resistance factor for ultimate limit state evaluation:
  - 0.50 (Typical degree of understanding) for static.
  - 0.70 for seismic.
- Consequence factor of 1.0 for both static and seismic conditions.

The pile group reduction factors have been determined using the pile layout provided in the design drawings and are attached in Appendix D.

6.3 KHR2 Pile Group Response

The pile group analyses conducted for KHR2 by Tetra Tech are discussed in this section. The pile size (i.e., 762 mm) and layout provided by COWI were used to evaluate the pile group response and required pile length. The pile layouts for the two abutments are shown in the COWI design drawings (Dwg. 6294-14 and -15). The foundation loads considered in the pile group analyses were provided by COWI and are presented in Appendix E.

The pile group analyses were performed using the commercially available program, GROUP, Version 2022.12.4. The soil response to pile loading is modeled using non-linear curves: p-y for lateral loading, and t-z and Q-z for axial loading. The solution is iterated to accommodate the nonlinear response of each of the piles in the group.

For the analyses performed, the pile head condition was modeled as a fixed connection. The results of the pile group analyses are presented in Appendix E for the eight (8) load cases. The maximum bending moment is generally considered critical to the structures. It is understood that the results were used by COWI to evaluate the structural requirements of the foundation.

The required pile embedment length was determined based on the axial demand obtained from the pile group analyses. With a maximum static axial reaction of 852 kN and a geotechnical resistance factor of 0.4, the required pile length for a 762-mm pipe pile to provide a factored geotechnical capacity of 852 kN was determined to be about 13 m.

6.4 Potential Impacts from Pile Driving

Vibrations from pile driving have a potential to cause settlement of granular soils. In-situ granular soils could settle about 100 mm within a radial distance of 1 m from driven piles. The settlements are expected to reduce rapidly with distance from the pile. By about 10 times diameter from the pile (i.e., approximately 8 m for a 762 mm diameter pile), the settlements are expected to be less than about 15 mm.

Pile driving can result in vibrations which could impact nearby structures and utilities depending on the distance from the pile being driven and ground conditions. The potential for damage is increased if obstructions (e.g., cobbles and boulders) are encountered, due to additional driving energy required.

A Peak Particle Velocity (PPV) of 25 mm/s is commonly used as a threshold for damage to existing structures and utilities. This threshold value of 25 mm/s is proposed but should be reviewed and accepted by the owners of the specific structures or utilities at the site.

Assuming a diesel hammer that is capable of driving the 762 mm piles to depths of about 20 m, the PPV induced from pile driving is estimated to be less than 25 mm/s for a horizontal distance of about 12 m. The ground vibrations caused by pile driving were estimated using the procedure outlined in Caltrans (2013). A summary of the estimated PPV values at various distances from the pile for an APE D80-42 hammer is presented in Table 6-1. If a different hammer is utilized, these calculated PPV values should be revised.

Table 6-1: Estimated Peak Particle Velocity from Pile Driving

Distance from pile (m)	Estimated PPV (mm/s)
5	60
7.5	39
10	28
12	23
15	18
20	13

As indicated by Table 6-1, pile driving will have a risk of impacting structures and utilities located up to about 12 m from pile driving activities based on the 25 mm/s threshold. During construction, instrumentation should be installed for all existing structures and utilities that are located within 25 m of pile driving to monitor the ground deformation, structural deformation, and vibrations.

The following measures can be considered to limit the construction impact associated with vibration from pile installation:

- Exposing and monitoring the underground utilities that may be affected. Input from utility owners is required with regards to the criteria and monitoring details.
- Pre-drilling to depths where vibration form pile driving can be limited to an acceptable level.
- Use of smaller equipment for the initial penetration.

Pre- and post-construction condition surveys are also recommended.

6.5 Additional Comments

The pile located in the northeast corner of the KHR1 north abutment will be placed near the existing timber piles. It is anticipated that the portion that is potentially in conflict with the existing timber piles will be installed by drilling. Then the pile will be driven to the design elevation and to achieve the required axial capacity (i.e., achieving the pile set). For evaluating the lateral performance of the northeast pile, the same p-y curves are considered appropriate as the soils are dense to very dense at the project site, such that the installation and presence of the timber piles are not expected to have significantly changed the stiffness of the soils.

The analyses conducted by COWI indicate very small pile movements of less than 5 mm is expected. Therefore, the interaction between the pile foundation and nearby structures (particularly the existing pile cap) is expected to be negligible.

No reduction factor has been considered for installation (e.g., predrilling) based on the consideration that care will be taken to prevent the adjacent soil outside the pile (or casing) to become disturbed, and adversely affect the structural integrity of the pile foundation.

7.0 EMBANKMENTS

The typical sections provided in the design drawings indicate the approach embankments to the bridge structures require raising the original ground by up to about 3.5 m. Mineral fill will be used to construct the approaches. The north approach of KHR1 will be retained by MSE walls. The design of the approach embankments should consider a total unit weight of 21 kN/m^3 and a soil strength of 36 degrees. For Bridge End Fill, the material should meet the requirements outlined in the BC MoTI 2020 Standard Specifications for Highway Construction, Section 202.02.04.

7.1 Global Stability

Stability analyses were completed using the limit equilibrium software Slope/W (GeoStudio 2021). The failure surfaces analyzed in Slope/W were created through the circular slip surface search method. The "Entry and Exit" method was used for determining the location of critical slip surfaces. The evaluation and design of the embankments under static and seismic conditions was performed considering:

- Factor of safety greater than or equal to 1.54 (Typical Consequence and DoU) under static conditions with a traffic surcharge of 16 kPa.
- Factor of safety greater than or equal to 1.3 under pseudo-static conditions considering a horizontal seismic coefficient (k_h) equal to one-half of the PGA, as per CSA-S6-19 C6.14.4.2.

The wall section with the maximum height of 3.75 m was used to evaluate the stability requirements. The results of the analyses for both static and seismic conditions are shown on Figure 11. The analyses indicate the typical minimum reinforcement length of the greater of 2.4 m or 70% of the wall height provides satisfactory stability.

For general road embankments, the side slopes to be considered should be 2H:1V or flatter.

7.2 Foundation Bearing Pressures

7.2.1 Ultimate Limit States (ULS)

The ultimate bearing resistance for shallow foundations was estimated using the general procedure recommended in CAN/CSA S6-19. For a minimum embedment of 0.6 m, the ultimate bearing resistance is estimated to be about 1,000 kPa. Using a geotechnical resistance factor of 0.5 combined with the typical consequence factor, the factored geotechnical resistance at ULS is 500 kPa.

7.2.2 Serviceability Limit States (SLS)

The SLS pressure corresponding to a settlement of about 25 mm was estimated to be about 300 kPa, based on drained moduli of 50 MPa for the fill (Soil Unit I) and 70 MPa for the underlying sand and gravel (Soil Units II and III).

7.3 Foundation Settlement

The new approaches to the bridge structures require raising the road grade by up to about 3.5 m using mineral fill. Given the soils at the project site are granular, the settlement of the approach fill is considered to be "immediate" and will occur during (or shortly after) the construction of the approach fill and abutment structures.

Settlement evaluations were performed using the commercially available program Rocscience Settle3. The soil compressibility was estimated based on the empirical correlations and the results from SPT. Lower and upper bound parameters were used to estimate the settlements.

The maximum settlements are anticipated to be less than 50 mm at the south approach, where the fill has the maximum height of about 3.5 m. The estimated settlements will occur during (or shortly after) construction of the approach, and the post-construction settlement is expected to be minimal. Settlement monitoring is recommended during and shortly after construction to confirm both the rate and magnitude of estimated settlements.

7.4 Retaining Wall

A Mechanically Stabilized Earth (MSE) retaining wall is required at the north approach of KHR1. The maximum wall height, including a nominal embedment of 600 mm, is about 3.75 m. Based on the discussion with the BC MoTI representatives on June 26, 2023, regarding the use of extensible reinforcement as per BC MoTI Supplement Clause 6.19.2.1.a – Mechanically Stabilized Earth (MSE) Walls at Bridge Abutments and the Embankment Bridge Interface Zone, the wall can be constructed using concrete (modular) blocks with geogrid (extensible) reinforcement.

The design of the modular wall was evaluated in accordance with the method provided in CAN/CSA S6-19. As per BC Supplement to S6-19 (Table 6.14.4.2-1), the proposed retaining wall was analyzed using forced-based method since the wall is less than 6 m high, assigned a Seismic Performance Category 2 and a "Major-route" structure.

The earthquake-induced loads were approximated using the Mononobe-Okabe equation considering a seismic horizontal acceleration coefficient equal to the site PGA, as the seismically induced lateral deformation is expected to be less than 25 mm.

As discussed in Section 7.3, the total vertical settlement is estimated to be less than 50 mm. The differential settlement is expected to be less than 25 mm.

The global, external and internal stability conditions were evaluated based on the following:

- Traffic load of 16 kPa was used for the static loading conditions.
- The pseudo-static horizontal earthquake load for the various earthquake scenarios was calculated following the recommendations provided in CAN/CSA S6-19.
- No water pressure is considered at the back of the wall. Drainage at the back of the wall is required to avoid any potential water pressure.
- Vertical facing (i.e., no batter) was considered.
- Reinforced fill that has a minimum friction angle of 34° and a total unit weight of 21 kN/m^3.
- The embedment depth was defined as per the recommendations provided in CAN/CSA S6-19 with a minimum value of 600 mm.
- A vertical spacing of 0.75 m was used for each reinforcement layer. This is equal to the height of a typical, individual concrete block. Geogrid reinforcement is required at the base of the wall and between the concrete blocks.
- Horizontal PGA at the base of the wall is 0.149g (Class C) for the 2475-year design earthquake.

In order to satisfy the stability requirements (both external and internal), the following should be met:

- The geogrid reinforcement should have a minimum reinforcing length of 70% of the wall height or 2.4 m, whichever is greater.
- The geogrid reinforcement should have a minimum ultimate tensile strength of 109 kN/m and a minimum tensile strength of 54 kN/m at 5% strain. The geogrid is to be installed with the strong axis perpendicular to the wall alignment.
- Wall drainage should include 150 mm diameter perforated PVC pipe surrounded by 150 mm thick layer of 19 mm clear gravel or drain rock wrapped with geotextile.

7.5 Multi-Use Pathway Underpass

This section provides the geotechnical input related to the design of the multi-use pathway (MUP) underpass structure.

Active, at-rest and passive earth pressures acting on the tunnel walls have been calculated following the recommendations given in the CFEM (2006) considering that the granular fill material to be used at the side of the tunnel is medium dense to dense sand (engineered fill). The static earth pressure coefficients for active (Ka), at-rest (Ko) and passive (Kp) conditions are estimated to be about 0.27, 0.43 and 3.69, respectively.

For seismic conditions, the magnitude and distribution of the earthquake-induced loads is approximated using the Mononobe-Okabe equation (1926) but considering a rigid wall condition and uses a seismic horizontal acceleration coefficient equal to the site-adjusted PGA. The coefficient of active earth pressure under seismic conditions was estimated to be 0.30 for the 475-yr event, 0.32 for the 975-yr event and 0.36 for the 2,475-yr event. The coefficient of passive seismic earth pressure was calculated as 3.59, 3.53 and 3.39 for the 475-yr, 975-yr and 2,475-yr events, respectively.

8.0 RECOMMENDATIONS FOR WATERMAIN INSTALLATION

A new 250 mm watermain is proposed, which will be structurally supported by the KHR1 and will connect to the existing watermain at about Sta. 103+46 on the north side and at the south approach of KHR2. Based on the results of the field and laboratory investigation, it is considered feasible to install the on-land sections of the proposed pipe using traditional open-trench construction method.

8.1 Watermain Installation

The proposed watermain installation will require excavation into the fill layer and possibly the sand and gravel layer below. All excavation work related to trenching should be completed in accordance with the WorkSafe BC Regulations (Part 20 Excavations) and Master Municipal Construction Document (MMCD) Platinum Edition Volume II (2019) Section 31 23 01. According to these regulations, excavations deeper than 1.2 m that are to be occupied by workers at any time during construction will require earth support in a form of sidewall shoring, bracing or grading.

The contractor should define the sideslope of the trenches to ensure stability. The sideslope should in no case be steeper than 1.5H:1V.

If the construction is performed during spring/summer months (i.e., March to September), the groundwater levels are expected not to differ much from the levels observed during the fieldwork.

8.2 Pipe Bedding and Trench Backfill

Pipes should be placed directly on imported, compacted or re-compacted granular fill to provide support to the pipe. The pipe should not be bedded directly on soft fine-grained soils, if encountered, due to the compressible nature of these materials. Native soils should be covered with compacted, free-draining granular pipe bedding material (Type 1 as specified in MMCD Section 31 05 17, 2.7) prior to pipe placement. The thickness of the pipe bedding layer should be a minimum of 100 mm. If the excavation intercepts pockets of unsuitable materials, over-excavation will be required to remove this material.

Bedding and backfilling material in contact with the pipe should consist primarily of imported crushed or graded angular gravel with no coarse particles greater than 25 mm. Backfill on the sides and above the top of the pipe should be compacted to 95% Modified Proctor Maximum Dry Density (MPMDD), in maximum 100 mm thick lifts. This is critical to avoid displacement of subsurface fill due to differing degrees of compaction, which can compromise stability of other existing utilities in the area. Vibratory disturbance during compaction of trench bedding and backfill materials could affect the stability of trench side walls. Therefore, a contingency should be considered for provision of temporary shoring for maintenance of trench side walls during backfill compaction. We recommend that only small hand-compactor type equipment such as walk-behind plate tampers should be allowed in the trench in order to reduce vibratory energy and potential disturbance of side wall and nearby utilities.

The existing granular fill originally excavated from the trench may be re-used as backfill, subject to review by Tetra Tech and approval.

9.0 SITE PREPARATION

Site preparation for the proposed bridge approach and abutment will require removal of topsoil. Significant excavation into the existing ground is not expected, but a nominal stripping of 0.3 m is likely required to remove the topsoil containing organics. The topsoil is not considered suitable for re-use as engineered fill due to its organic content. The materials below the topsoil may be re-used but may require some processing. Once removed, the materials should be stockpiled separately from the construction debris, organics and other unsuitable materials and should be approved by the geotechnical engineer for re-use as engineering fill.

No surface surcharge or temporary loading should be placed within a distance equal to twice the depth of any temporary excavation unless the excavation support system has been designed to accommodate such surface loading.

No vertical cuts higher than 1.2 m are considered. Any unsupported excavation above water level should not be steeper than 1.5H:1V. Any excavation requiring support will be a temporary works requirement for which the contractor is responsible.

10.0 CLOSURE

We trust this document meets your present requirements. If you have any questions or comments, please contact the undersigned.

Respectfully Submitted,
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CR/LC/AA/cy

PERMIT TO PRACTICE
TETRA TECH CANADA INC.

PERMIT NUMBER: 1001972

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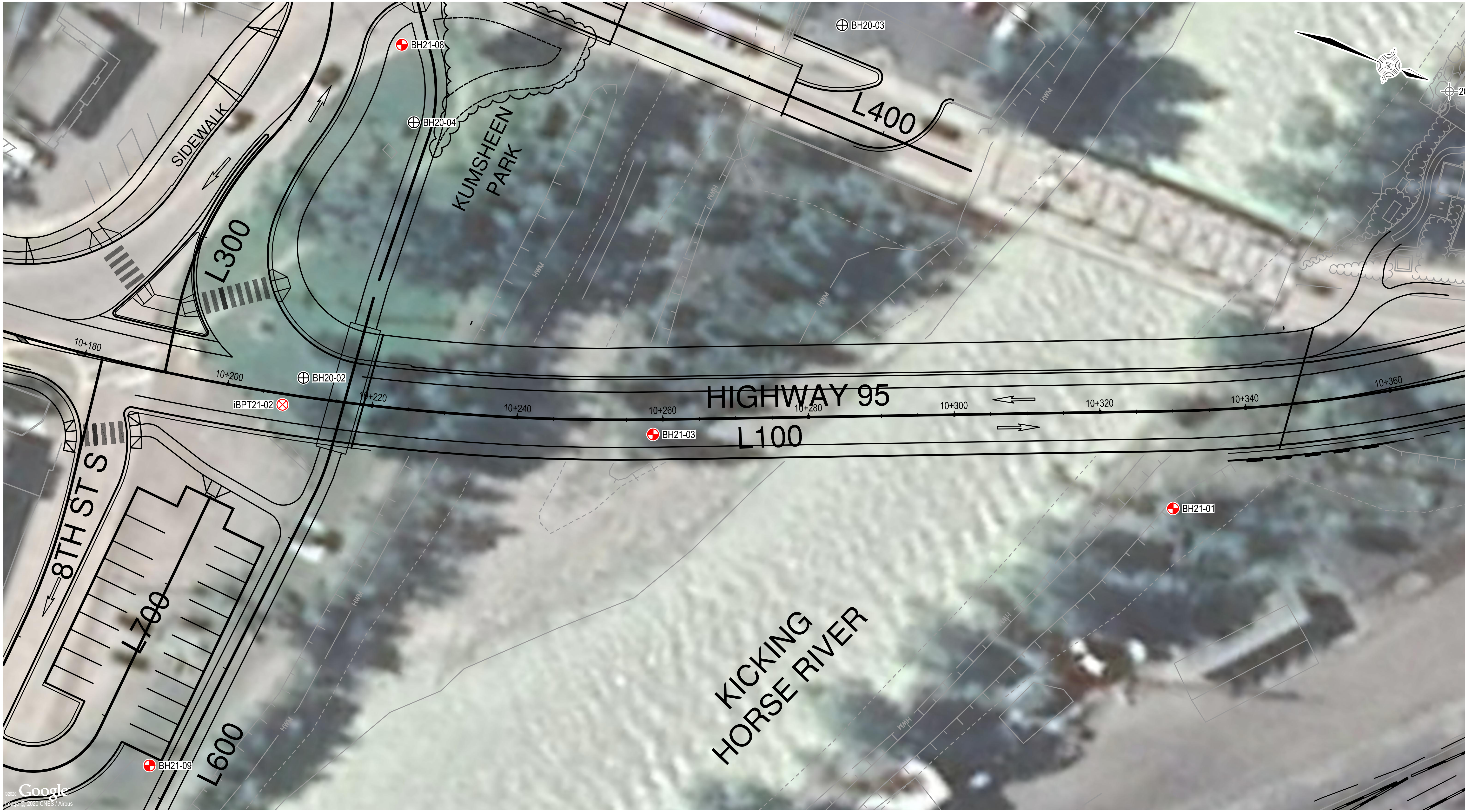
FIGURES



LEGEND
- Tetra Tech sonic testhole
- Tetra Tech iBPT
- Golder testhole locations

NOTES
1. Imagery from Google Earth Pro. Testhole locations on this image are approximate. See factual report for coordinates.
2. Layout from "Highway 95 - Kicking Horse River. Bridge 1 and 2. Replacement and Approaches" 100% functional design drawings by Urban Systems (June 2022).

ISSUED FOR USE

0 25m

Scale: 1:500 @ 11"x17"

CLIENT

British Columbia Ministry of Transportation and Infrastructure

TETRA TECH

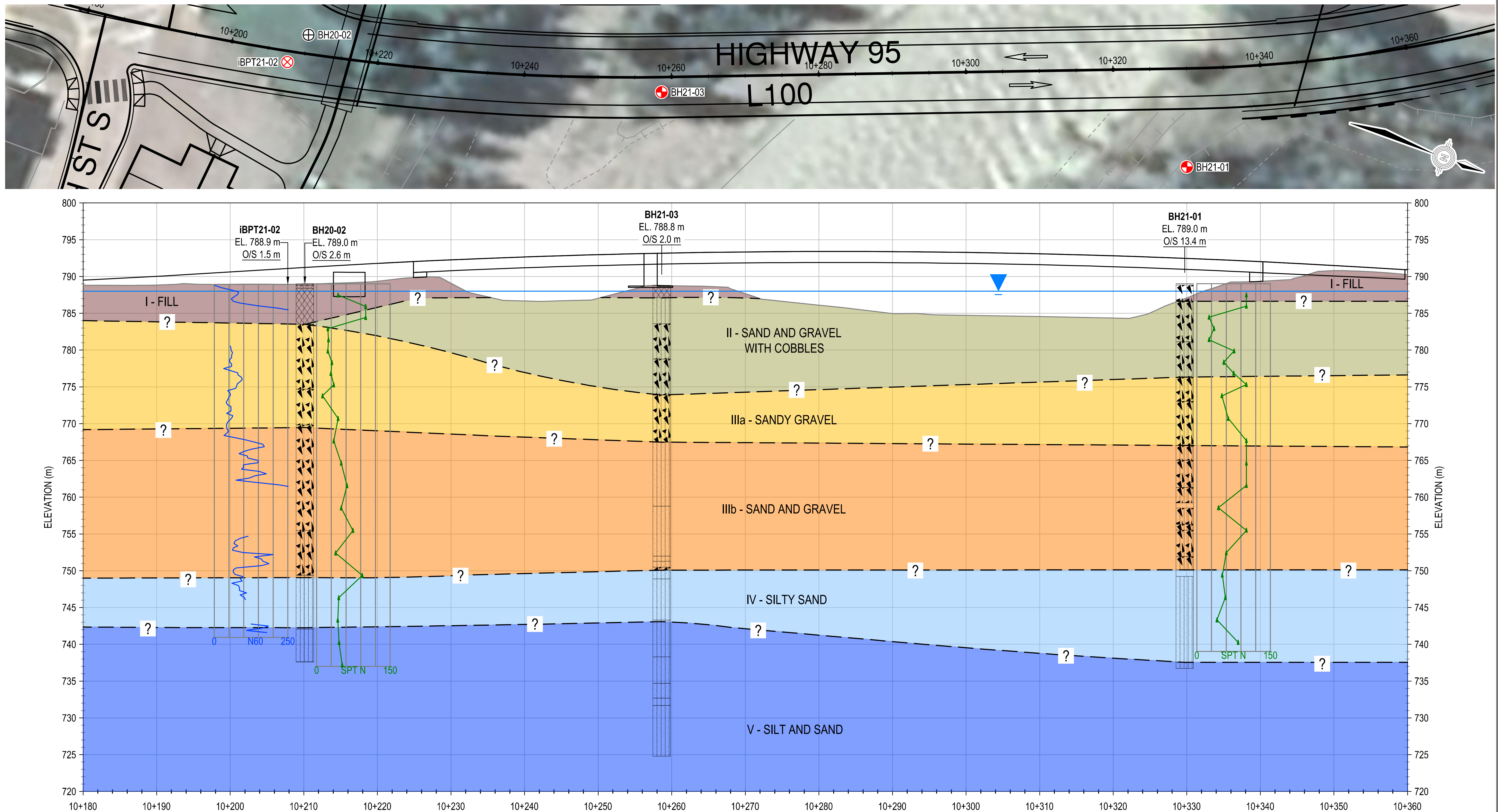
KICKING HORSE RIVER BRIDGES REPLACEMENT

PLAN VIEW

PROJECT NO.	DWN	CKD	REV
ENG.VGEO03793-01	RH	CR	0
OFFICE	DATE		
VANC	July 2023		

Figure 1

Q:\Vancouver\Drafting\Engineering\VGEO\ENG.VGEO03793-01\ENG.VGEO03793-01.115 Section R0f.dwg [FIGURE 1] September 26, 2022 - 10:45:23 am (BY: HALL, ROBERT J)



LEGEND

- Tetra Tech sonic testhole
- Tetra Tech iBPT
- Golder testhole locations
- Water level considered for geotechnical design

STICKLOG LEGEND

- Fill
- Sand and gravel
- Silt, sand, and gravel
- Silt and sand
- Silt

STICKLOG LEGEND

- I - Fill
- II - Sand and gravel with cobbles
- IIIa - Sandy gravel
- IIIb - Sand and gravel
- IV - Silty sand
- V - Silt and sand

NOTES

1. Imagery from Google Earth Pro. Testhole locations on this image are approximate. See factual report for coordinates.
2. Layout from "Highway 95 - Kicking Horse River. Bridge 1 and 2. Replacement and Approaches" 100% functional design drawings by Urban Systems (June 2022).

ISSUED FOR USE

0 25m

Scale: 1:500 @ 11"x17"

CLIENT

British Columbia Ministry of Transportation and Infrastructure

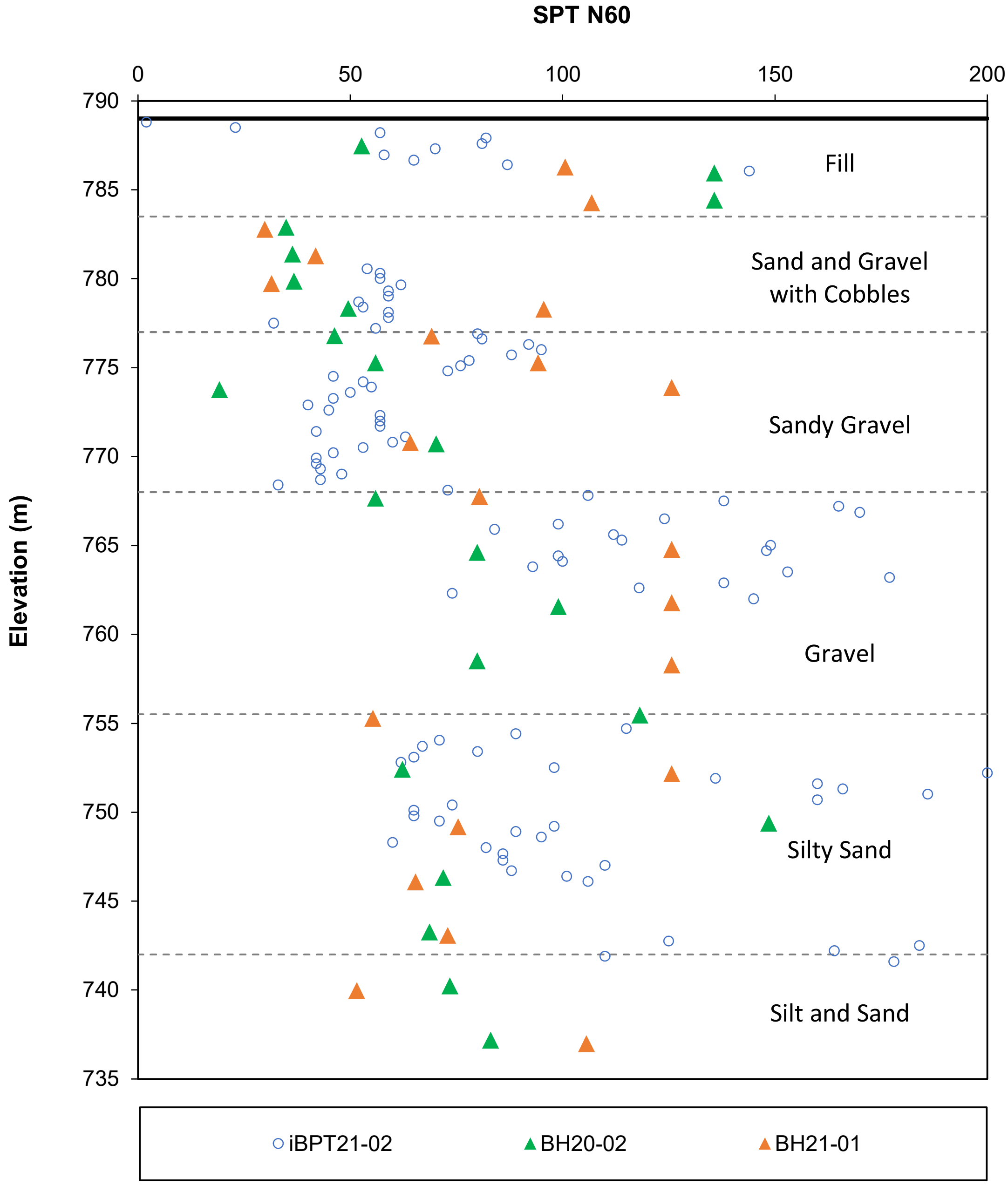
TETRA TECH

KICKING HORSE RIVER BRIDGES REPLACEMENT

L100 - HIGHWAY 95
PLAN AND PROFILE

PROJECT NO.	DWN	CKD	REV
ENG.VGEO03793-01	RH	CR	0
OFFICE	DATE		
VANC	July 2023		

Figure 2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100
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780
775
770
765
760
755
750
745
740
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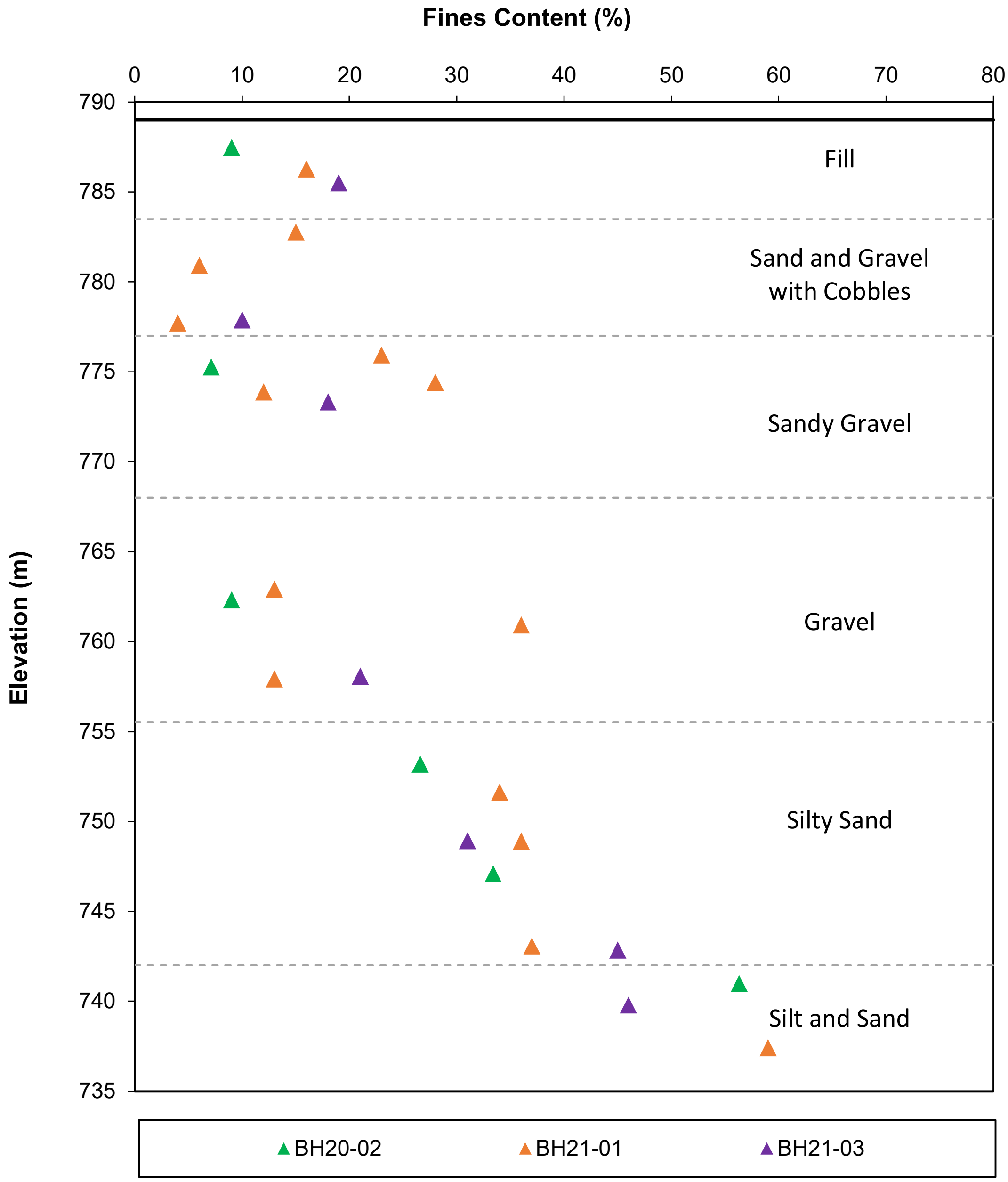
Sand and Gravel with Cobbles
Sandy Gravel
Gravel
Silty Sand
Silt and Sand
iBPT21-02
BH20-02
BH21-01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Elevation (m)
Fill
Sand and Gravel
with Cobbles
Sandy Gravel
Gravel
Silty Sand
Silt and Sand
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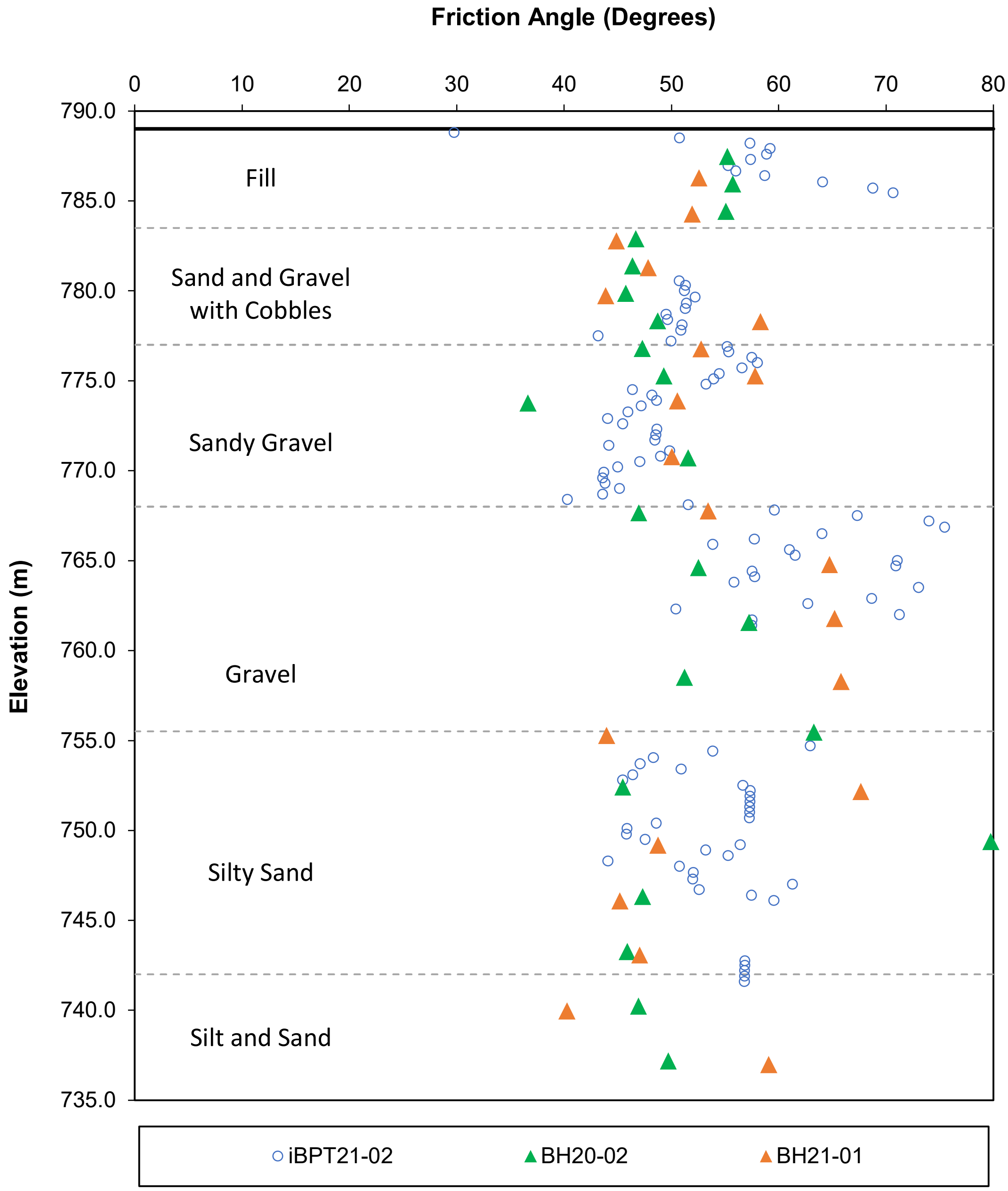
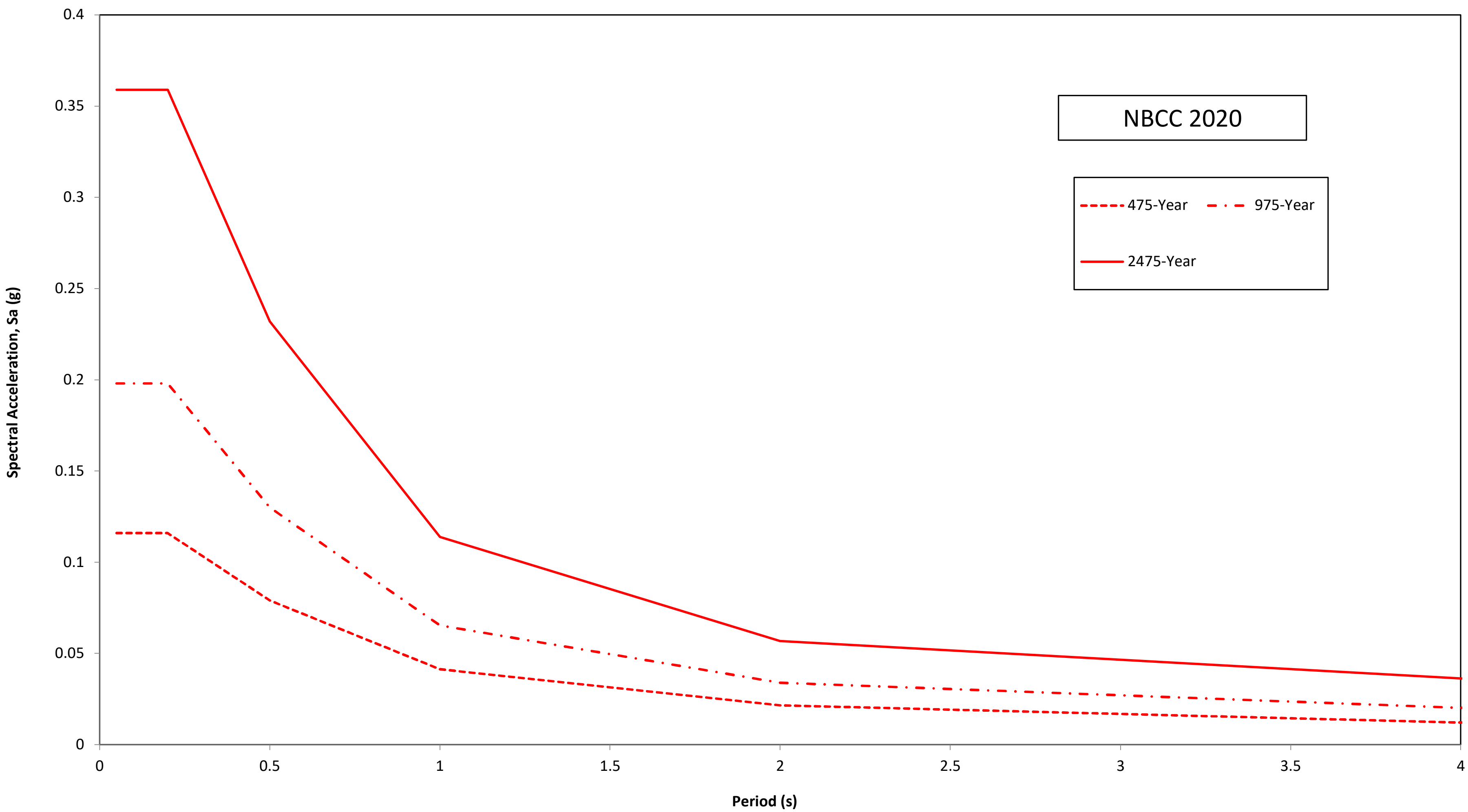
BH21-01
BH21-03



INTERPRETED FRICTION ANGLE

Figure 5



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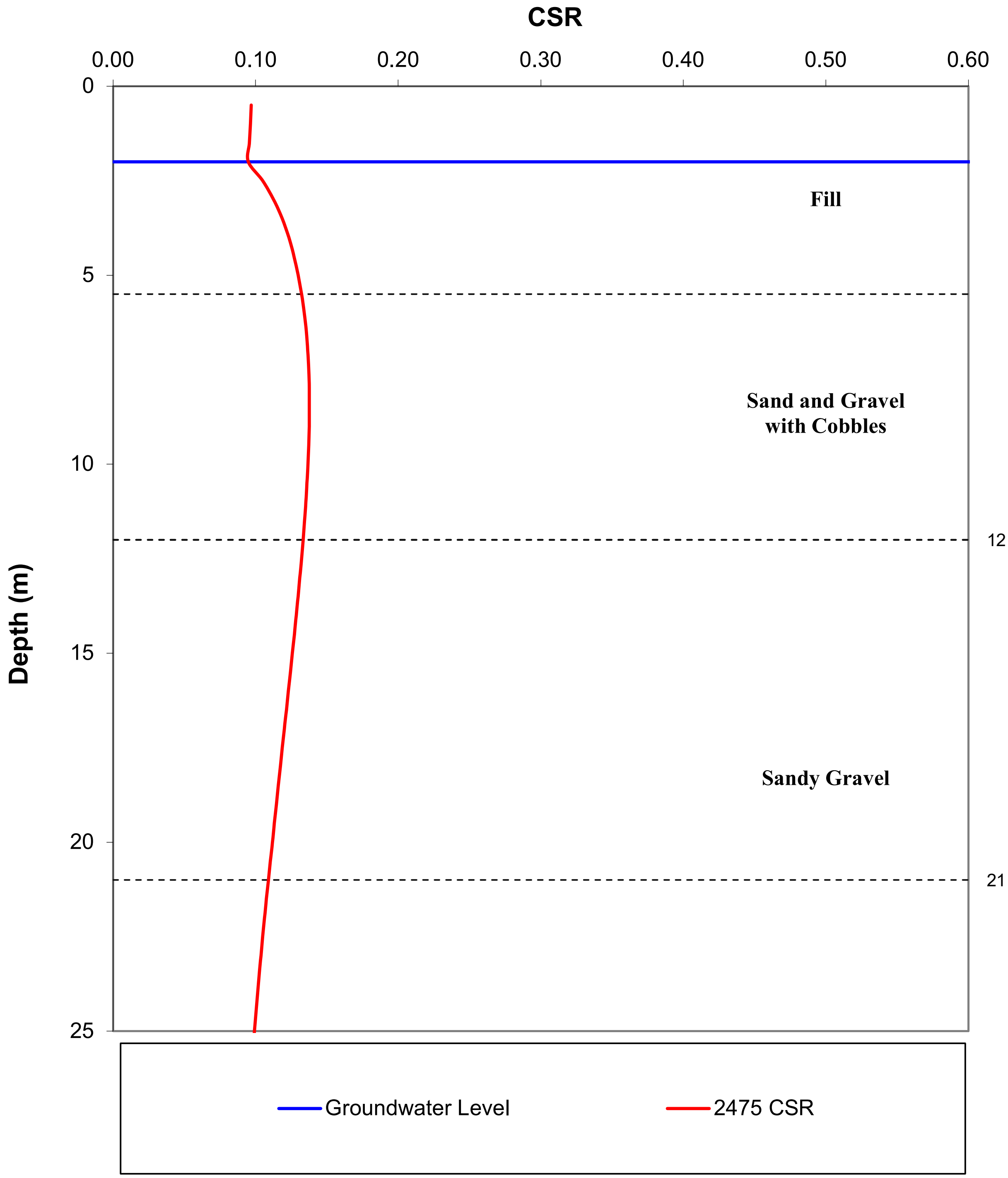
DESIGN SPECTRA
Site Class C

Figure 6



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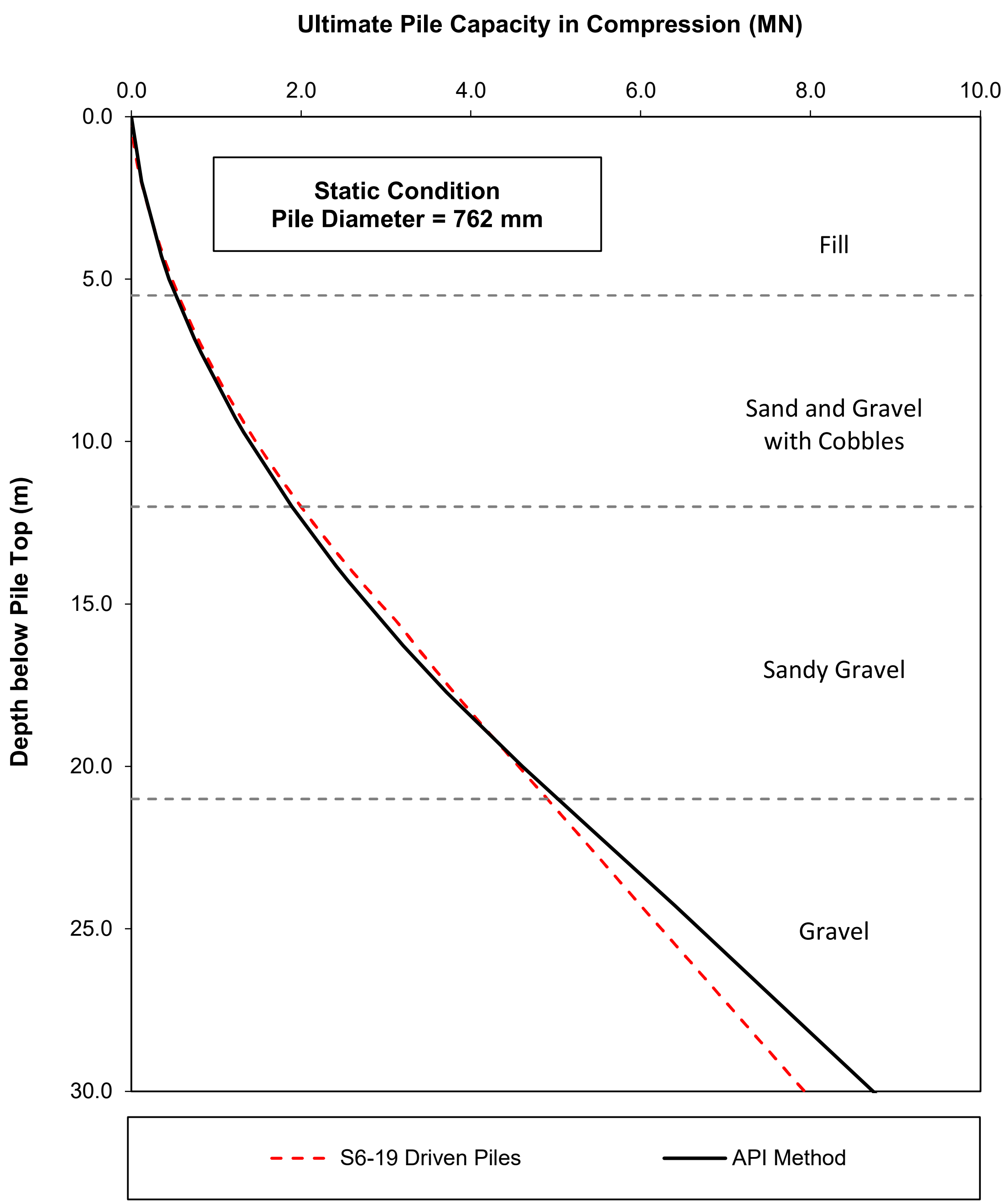
CYCLIC STRESS RATIO
2475-Year Earthquake

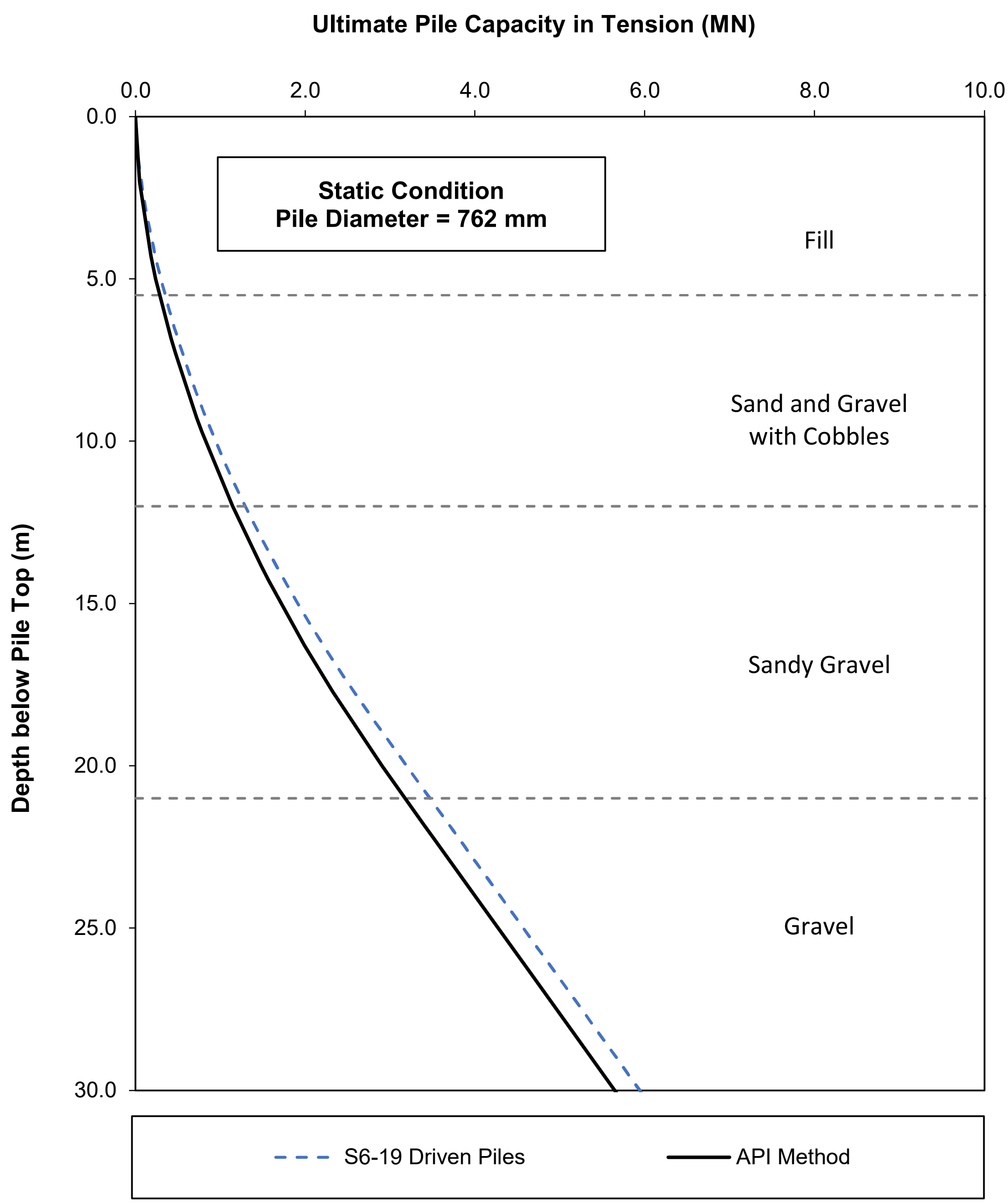
Figure 7



CAN/CSA S6-19 Driven Piles: Nq = 15, β = 0.60

ULTIMATE PILE AXIAL CAPACITY IN COMPRESSION
Static Condition
Diameter = 762 mm

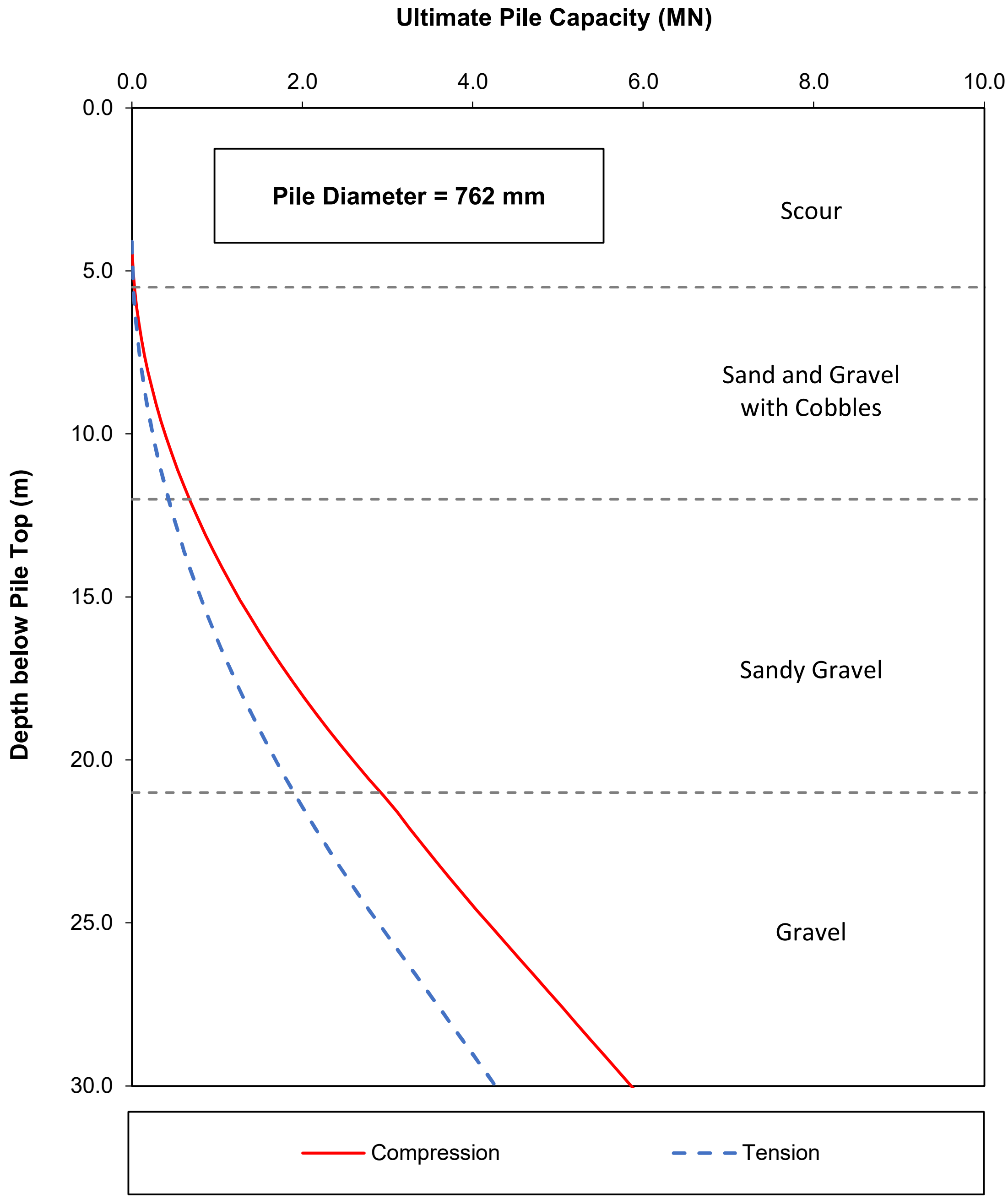
Figure 8



CAN/CSA S6-19 Driven Piles: Nq = 15, β = 0.60

ULTIMATE PILE AXIAL CAPACITY IN TENSION
Static Condition
Diameter = 762 mm

Figure 9



CAN/CSA S6-19 Driven Piles: Nq = 15, β = 0.60

Top of Pile at El. 786.6 m
Scoured Elevation at 782.5 m

ULTIMATE PILE AXIAL CAPACITY
Scoured Elevation at 782.5 m
Diameter = 762 mm

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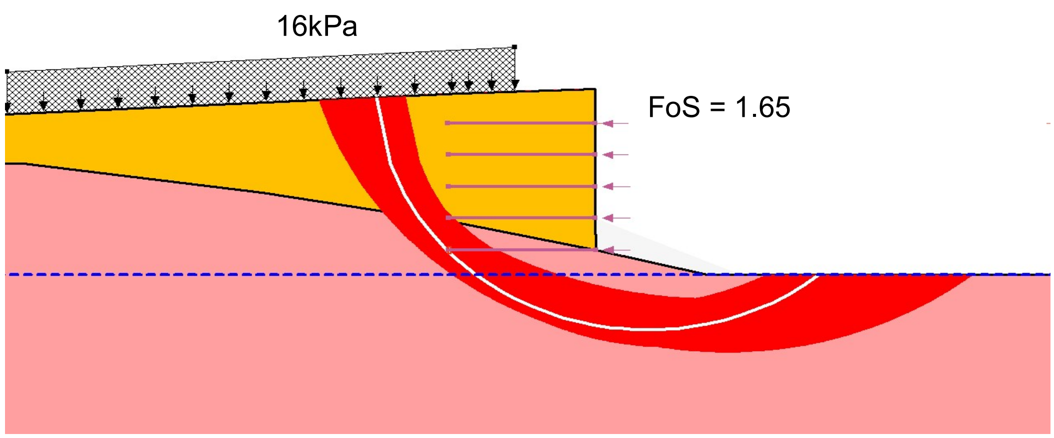
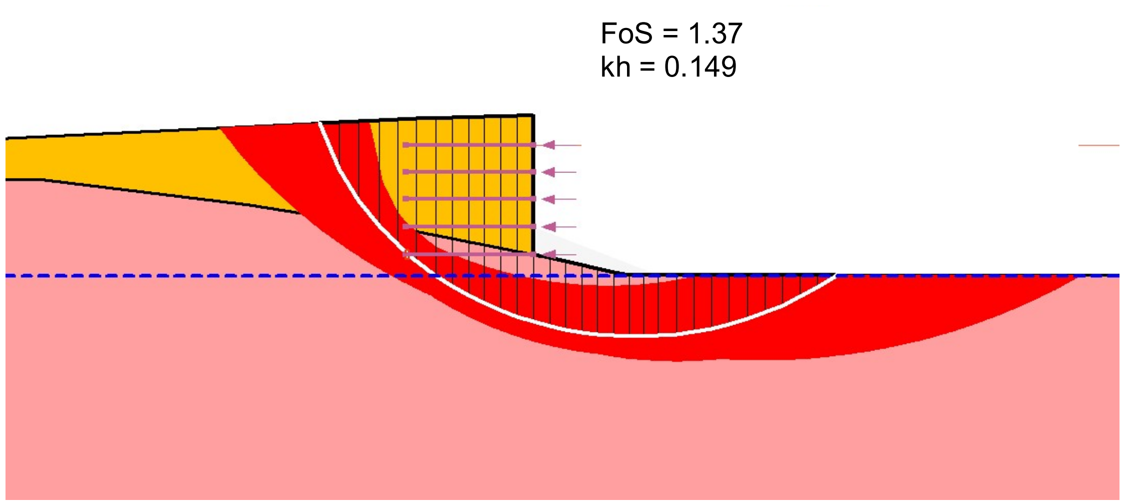
Figure 10

Static Conditions



Seismic Conditions



GLOBAL STABILITY EVALUATION
North Abutment

Figure 11

APPENDIX A

LIMITATIONS ON THE USE OF THIS DOCUMENT

LIMITATIONS ON USE OF THIS DOCUMENT

GEOTECHNICAL

1.1 USE OF DOCUMENT AND OWNERSHIP

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Both electronic file and/or hard copy versions of TETRA TECH's Instruments of Professional Service shall not, under any circumstances, be altered by any party except TETRA TECH. TETRA TECH's Instruments of Professional Service will be used only and exactly as submitted by TETRA TECH.

Electronic files submitted by TETRA TECH have been prepared and submitted using specific software and hardware systems. TETRA TECH makes no representation about the compatibility of these files with the Client's current or future software and hardware systems.

1.3 STANDARD OF CARE

Services performed by TETRA TECH for the Professional Document have been conducted in accordance with the Contract, in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practicing under similar conditions in the jurisdiction in which the services are provided. Professional judgment has been applied in developing the conclusions and/or recommendations provided in this Professional Document. No warranty or guarantee, express or implied, is made concerning the test results, comments, recommendations, or any other portion of the Professional Document.

If any error or omission is detected by the Client or an Authorized Party, the error or omission must be immediately brought to the attention of TETRA TECH.

1.4 DISCLOSURE OF INFORMATION BY CLIENT

The Client acknowledges that it has fully cooperated with TETRA TECH with respect to the provision of all available information on the past, present, and proposed conditions on the site, including historical information respecting the use of the site. The Client further acknowledges that in order for TETRA TECH to properly provide the services contracted for in the Contract, TETRA TECH has relied upon the Client with respect to both the full disclosure and accuracy of any such information.

1.5 INFORMATION PROVIDED TO TETRA TECH BY OTHERS

During the performance of the work and the preparation of this Professional Document, TETRA TECH may have relied on information provided by third parties other than the Client.

While TETRA TECH endeavours to verify the accuracy of such information, TETRA TECH accepts no responsibility for the accuracy or the reliability of such information even where inaccurate or unreliable information impacts any recommendations, design or other deliverables and causes the Client or an Authorized Party loss or damage.

1.6 GENERAL LIMITATIONS OF DOCUMENT

This Professional Document is based solely on the conditions presented and the data available to TETRA TECH at the time the data were collected in the field or gathered from available databases.

The Client, and any Authorized Party, acknowledges that the Professional Document is based on limited data and that the conclusions, opinions, and recommendations contained in the Professional Document are the result of the application of professional judgment to such limited data.

The Professional Document is not applicable to any other sites, nor should it be relied upon for types of development other than those to which it refers. Any variation from the site conditions present, or variation in assumed conditions which might form the basis of design or recommendations as outlined in this document, at or on the development proposed as of the date of the Professional Document requires a supplementary exploration, investigation, and assessment.

TETRA TECH is neither qualified to, nor is it making, any recommendations with respect to the purchase, sale, investment or development of the property, the decisions on which are the sole responsibility of the Client.

1.7 ENVIRONMENTAL AND REGULATORY ISSUES

Unless stipulated in the report, TETRA TECH has not been retained to explore, address or consider and has not explored, addressed or considered any environmental or regulatory issues associated with development on the subject site.

1.8 NATURE AND EXACTNESS OF SOIL AND ROCK DESCRIPTIONS

Classification and identification of soils and rocks are based upon commonly accepted systems, methods and standards employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Where deviations from the system or method prevail, they are specifically mentioned.

Classification and identification of geological units are judgmental in nature as to both type and condition. TETRA TECH does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

Where subsurface conditions encountered during development are different from those described in this report, qualified geotechnical personnel should revisit the site and review recommendations in light of the actual conditions encountered.

1.9 LOGS OF TESTHOLES

The testhole logs are a compilation of conditions and classification of soils and rocks as obtained from field observations and laboratory testing of selected samples. Soil and rock zones have been interpreted. Change from one geological zone to the other, indicated on the logs as a distinct line, can be, in fact, transitional. The extent of transition is interpretive. Any circumstance which requires precise definition of soil or rock zone transition elevations may require further investigation and review.

1.10 STRATIGRAPHIC AND GEOLOGICAL INFORMATION

The stratigraphic and geological information indicated on drawings contained in this report are inferred from logs of test holes and/or soil/rock exposures. Stratigraphy is known only at the locations of the test hole or exposure. Actual geology and stratigraphy between test holes and/or exposures may vary from that shown on these drawings. Natural variations in geological conditions are inherent and are a function of the historical environment. TETRA TECH does not represent the conditions illustrated as exact but recognizes that variations will exist. Where knowledge of more precise locations of geological units is necessary, additional exploration and review may be necessary.

1.11 PROTECTION OF EXPOSED GROUND

Excavation and construction operations expose geological materials to climatic elements (freeze/thaw, wet/dry) and/or mechanical disturbance which can cause severe deterioration. Unless otherwise specifically indicated in this report, the walls and floors of excavations must be protected from the elements, particularly moisture, desiccation, frost action and construction traffic.

1.12 SUPPORT OF ADJACENT GROUND AND STRUCTURES

Unless otherwise specifically advised, support of ground and structures adjacent to the anticipated construction and preservation of adjacent ground and structures from the adverse impact of construction activity is required.

1.13 INFLUENCE OF CONSTRUCTION ACTIVITY

Construction activity can impact structural performance of adjacent buildings and other installations. The influence of all anticipated construction activities should be considered by the contractor, owner, architect and prime engineer in consultation with a geotechnical engineer when the final design and construction techniques, and construction sequence are known.

1.14 OBSERVATIONS DURING CONSTRUCTION

Because of the nature of geological deposits, the judgmental nature of geotechnical engineering, and the potential of adverse circumstances arising from construction activity, observations during site preparation, excavation and construction should be carried out by a geotechnical engineer. These observations may then serve as the basis for confirmation and/or alteration of geotechnical recommendations or design guidelines presented herein.

1.15 DRAINAGE SYSTEMS

Unless otherwise specified, it is a condition of this report that effective temporary and permanent drainage systems are required and that they must be considered in relation to project purpose and function. Where temporary or permanent drainage systems are installed within or around a structure, these systems must protect the structure from loss of ground due to mechanisms such as internal erosion and must be designed so as to assure continued satisfactory performance of the drains. Specific design details regarding the geotechnical aspects of such systems (e.g. bedding material, surrounding soil, soil cover, geotextile type) should be reviewed by the geotechnical engineer to confirm the performance of the system is consistent with the conditions used in the geotechnical design.

1.16 DESIGN PARAMETERS

Bearing capacities for Limit States or Allowable Stress Design, strength/stiffness properties and similar geotechnical design parameters quoted in this report relate to a specific soil or rock type and condition. Construction activity and environmental circumstances can materially change the condition of soil or rock. The elevation at which a soil or rock type occurs is variable. It is a requirement of this report that structural elements be founded in and/or upon geological materials of the type and in the condition used in this report. Sufficient observations should be made by qualified geotechnical personnel during construction to assure that the soil and/or rock conditions considered in this report in fact exist at the site.

1.17 SAMPLES

TETRA TECH will retain all soil and rock samples for 30 days after this report is issued. Further storage or transfer of samples can be made at the Client’s expense upon written request, otherwise samples will be discarded.

1.18 APPLICABLE CODES, STANDARDS, GUIDELINES & BEST PRACTICE

This document has been prepared based on the applicable codes, standards, guidelines or best practice as identified in the report. Some mandated codes, standards and guidelines (such as ASTM, AASHTO Bridge Design/Construction Codes, Canadian Highway Bridge Design Code, National/Provincial Building Codes) are routinely updated and corrections made. TETRA TECH cannot predict nor be held liable for any such future changes, amendments, errors or omissions in these documents that may have a bearing on the assessment, design or analyses included in this report.

APPENDIX B

SULPHATE ION CONTENT AND SOIL RESISTIVITY TEST RESULTS

SOLUBLE SULPHATE ION CONTENT OF SOIL

(CSA Designation A23.2-2B & A23.2-3B)

Project:	KHR Bridges Replacement Project	Date Tested:	September 28, 2022
Project No.:	704-ENG.VGEO03793-01.203	Tested By:	EM
Client:	BC MOTI	Sample Source:	BH21-01
Location:	Golden, BC - East of 10th Ave N, north of river	Laboratory:	Calgary

Sample Number	SPT01	G02				
Borehole Number	BH21-01	BH21-01				
Depth (m)	2.5-2.7	3.2-3.4				
Sulphate Content %	0.04	0.01				
Degree of Exposure (Class)	Negligible (Neg)	Negligible (Neg)				

Class of exposure	Degree of exposure	Water-soluble sulphate (SO_4)† in soil sample, %	Sulphate (SO_4) in groundwater samples, mg/L‡	Water soluble sulphate (SO_4) in recycled aggregate sample, %	Cementing materials to be used§
S-1	Very severe	> 2.0	> 10 000	> 2.0	HS or HSb
S-2	Severe	0.20–2.0	1500–10 000	0.60–2.0	HS or HSb
S-3	Moderate	0.10–0.20	150–1500	0.20–0.60	MS, MSb, LH, HS, or HSb

**For sea water exposure, see Clause 4.1.1.5.*

†In accordance with CSA A23.2-3B.

‡In accordance with CSA A23.2-2B.

§Cementing material combinations with equivalent performance may be used (see Clauses 4.2.1.2, 4.2.1.3, and 4.2.1.4). Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates. Refer to Clause 4.1.1.6.3.

Limitations:

i) The degree of exposure class included herein are valid only if drainage and weeping systems meet the requirements of the site conditions.

ii) The degree exposure class should be re-verified if backfill soils for foundation walls originate from an unknown source.

Remarks:

Reviewed By: ______________ P.Geol.

SOLUBLE SULPHATE ION CONTENT OF SOIL

(CSA Designation A23.2-2B & A23.2-3B)

Project:	KHR Bridges Replacement Project	Date Tested:	September 28, 2022
Project No.:	704-ENG.VGEO03793-01.203	Tested By:	EM
Client:	BC MOTI	Sample Source:	BH21-03
Location:	Golden, BC - Gould's Island	Laboratory:	Calgary

Sample Number	G01	G02	G03			
Borehole Number	BH21-03	BH21-03	BH21-03			
Depth (m)	0.5-1.2	1.2-1.7	1.7-2.5			
Sulphate Content %	0.02	0.02	0.04			
Degree of Exposure (Class)	Negligible (Neg)	Negligible (Neg)	Negligible (Neg)			

Class of exposure	Degree of exposure	Water-soluble sulphate (SO_4)† in soil sample, %	Sulphate (SO_4) in groundwater samples, mg/L‡	Water soluble sulphate (SO_4) in recycled aggregate sample, %	Cementing materials to be used§
S-1	Very severe	> 2.0	> 10 000	> 2.0	HS or HSb
S-2	Severe	0.20–2.0	1500–10 000	0.60–2.0	HS or HSb
S-3	Moderate	0.10–0.20	150–1500	0.20–0.60	MS, MSb, LH, HS, or HSb

**For sea water exposure, see Clause 4.1.1.5.*

†In accordance with CSA A23.2-3B.

‡In accordance with CSA A23.2-2B.

§Cementing material combinations with equivalent performance may be used (see Clauses 4.2.1.2, 4.2.1.3, and 4.2.1.4). Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates. Refer to Clause 4.1.1.6.3.

Limitations:

i) The degree of exposure class included herein are valid only if drainage and weeping systems meet the requirements of the site conditions.

ii) The degree exposure class should be re-verified if backfill soils for foundation walls originate from an unknown source.

Remarks:

Reviewed By: ______________ P.Geol.

TETRA TECH

WATER-SOLUBLE CHLORIDE IN MORTAR AND CONCRETE

ASTM C1218

Project:	KHR Bridges Replacement Project	Date Tested:	Sept. 29, 2022 Tested By: EM
Project No.:	704-ENG.VGEO03793-01.203	Dates Sampled:	See remarks
Client:	BC MOTI	Laboratory:	110, 140 Quarry Park Blvd. S.E. Calgary, AB T2C 3G3

Sample No.	SPT01				Sample No.	G02			
Borehole Number	BH21-01				Borehole Number	BH21-01			
Depth (m)	2.5-2.7				Depth (m)	3.2-3.4			
Sample Location	Golden, BC - East of 10th Ave N, north of river				Sample Location	Golden, BC - East of 10th Ave N, north of river			
Chloride Content (ppm by Mass of Sample)	4				Chloride Content (ppm by Mass of Sample)	7			
Chloride Content (% by Mass of Sample)	0.000				Chloride Content (% by Mass of Sample)	0.001			

Sample No.	G01				Sample No.	G02			
Borehole Number	BH21-03				Borehole Number	BH21-03			
Depth (m)	0.5-1.2				Depth (m)	1.2-1.7			
Sample Location	Golden, BC - Gould's Island				Sample Location	Golden, BC - Gould's Island			
Chloride Content (ppm by Mass of Sample)	93				Chloride Content (ppm by Mass of Sample)	4			
Chloride Content (% by Mass of Sample)	0.009				Chloride Content (% by Mass of Sample)	0.000			

Sample No.	G03				Sample No.				
Borehole Number	BH21-03				Borehole Number				
Depth (m)	1.7-2.5				Depth (m)				
Sample Location	Golden, BC - Gould's Island				Sample Location				
Chloride Content (ppm by Mass of Sample)	15				Chloride Content (ppm by Mass of Sample)				
Chloride Content (% by Mass of Sample)	0.002				Chloride Content (% by Mass of Sample)				

Remarks: BH21-01 - September 15-21, 2021
BH21-03 - March 9-12, 2021

Reviewed By: ______________________ P.Geol.

TETRA TECH

Organic Content in Soils by Loss on Ignition

AASHTO T267

Project No:	704-ENG.VGEO03793-01.203	**Sample No.:**	G1b
Project:	KHR Bridges Replacement Project	**Date Sampled:**	14-Sep-21
Client:	BC MOTI	**Sampled By:**	-
		Date Tested:	30-Sep-22
Attention:	**Ph:**	**Tested By:**	EM
Email:		**Office:**	Suite 110, 140 Quarry Park Blvd SE, Calgary, AB T2C 3G3

Description: SILT and GRAVEL, sandy, trace clay, trace organics

Source: BH21-06

Sample Location: Golden, BC - 10th Ave N, north of river

Depth: 0.3-1.0m

A	Mass of dish + sample, before Ignition	(g)	161.575	164.851
B	Mass of dish + sample, after ignition	(g)	160.909	164.224
C	Mass of dish	(g)	54.929	56.841

*Furnace Temperature: **455 °C**

$$\textbf{ORGANIC MATTER} = \frac{A - B}{A - C} \times 100$$

0.6%	**0.6%**
0.6%	

Remarks:

Reviewed By: ____________ P.Geol.

TETRA TECH

Organic Content in Soils by Loss on Ignition

AASHTO T267

Project No:	704-ENG.VGEO03793-01.203	**Sample No.:**	SPT01
Project:	KHR Bridges Replacement Project	**Date Sampled:**	14-Sep-21
Client:	BC MOTI	**Sampled By:**	-
		Date Tested:	3-Oct-22
Attention:	**Ph:**	**Tested By:**	EM
Email:		**Office:**	Suite 110, 140 Quarry Park Blvd SE, Calgary, AB T2C 3G3

Description: SILT and GRAVEL, sandy, trace clay, trace organics

Source: BH21-06

Sample Location: Golden, BC - 10th Ave N, north of river

Depth: 1.5-2.0m

A	Mass of dish + sample, before Ignition	(g)	131.999	134.792
B	Mass of dish + sample, after ignition	(g)	131.537	134.263
C	Mass of dish	(g)	48.201	50.643

*Furnace Temperature: **455 °C**

$$\textbf{ORGANIC MATTER} = \frac{A - B}{A - C} \times 100$$

0.6%	**0.6%**
0.6%	

Remarks:

Reviewed By: ____________ P.Geol.

TETRA TECH

Organic Content in Soils by Loss on Ignition

AASHTO T267

Project No:	704-ENG.VGEO03793-01.203	**Sample No.:**	SPT02
Project:	KHR Bridges Replacement Project	**Date Sampled:**	14-Sep-21
Client:	BC MOTI	**Sampled By:**	-
		Date Tested:	3-Oct-22
Attention:	**Ph:**	**Tested By:**	EM
Email:		**Office:**	Suite 110, 140 Quarry Park Blvd SE, Calgary, AB T2C 3G3

Description: SILT and GRAVEL, sandy, trace clay, trace organics

Source: BH21-06

Sample Location: Golden, BC - 10th Ave N, north of river

Depth: 3.0-3.5m

A	Mass of dish + sample, before Ignition	(g)	141.376	143.998
B	Mass of dish + sample, after ignition	(g)	140.908	143.541
C	Mass of dish	(g)	55.634	56.673

*Furnace Temperature: **455 °C**

ORGANIC MATTER = $\frac{A - B}{A - C} \times 100$

0.5%	**0.5%**
0.5%	

Remarks:

Reviewed By: ______________ P.Geol.

TETRA TECH

Organic Content in Soils by Loss on Ignition

AASHTO T267

Project No:	704-ENG.VGEO03793-01.203	**Sample No.:**	G01
Project:	KHR Bridges Replacement Project	**Date Sampled:**	March 12-13, 2021
Client:	BC MOTI	**Sampled By:**	-
		Date Tested:	September 30, 2022
Attention:	**Ph:**	**Tested By:**	EM
Email:		**Office:**	Suite 110, 140 Quarry Park Blvd SE, Calgary, AB T2C 3G3

Description:	CLAY, silty (topsoil), sandy, gravelly, some organics
Source:	BH21-09
Sample Location:	Golden, BC - Kumsheen Park, east of parking lot
Depth:	0.5-1.1m

A	Mass of dish + sample, before Ignition	(g)	172.950	173.891
B	Mass of dish + sample, after ignition	(g)	168.725	169.748
C	Mass of dish	(g)	64.194	64.062

*Furnace Temperature: **455 °C**

$$\textbf{ORGANIC MATTER} = \frac{A - B}{A - C} \times 100$$

3.9%	**3.8%**
3.8%	

Remarks:

Reviewed By: ____________ P.Geol.

Organic Content in Soils by Loss on Ignition

AASHTO T267

Project No:	704-ENG.VGEO03793-01.203	**Sample No.:**	G02
Project:	KHR Bridges Replacement Project	**Date Sampled:**	March 12-13, 2021
Client:	BC MOTI	**Sampled By:**	-
		Date Tested:	September 30, 2022
Attention:	**Ph:**	**Tested By:**	EM
Email:		**Office:**	Suite 110, 140 Quarry Park Blvd SE, Calgary, AB T2C 3G3

Description: SAND and GRAVEL, some silt, trace clay, trace organics

Source: BH21-09

Sample Location: Golden, BC - Kumsheen Park, east of parking lot

Depth: 1.2-1.9m

A	Mass of dish + sample, before Ignition	(g)	182.758	179.519
B	Mass of dish + sample, after ignition	(g)	180.372	177.330
C	Mass of dish	(g)	62.678	59.442

*Furnace Temperature: **455 °C**

$$\textbf{ORGANIC MATTER} = \frac{A - B}{A - C} \times 100$$

2.0%	**1.8%**
1.9%	

Remarks:

Reviewed By: ______________ P.Geol.

Organic Content in Soils by Loss on Ignition

AASHTO T267

Project No: 704-ENG.VGEO03793-01.203
Project: KHR Bridges Replacement Project
Client: BC MOTI
Attention:
Ph:
Email:

Sample No.: G03
Date Sampled: March 12-13, 2021
Sampled By: -
Date Tested: September 30, 2022
Tested By: EM
Office: Suite 110, 140 Quarry Park Blvd SE, Calgary, AB T2C 3G3

Description: SAND & GRAVEL, some silt, some clay, trace organics
Source: BH21-09
Sample Location: Golden, BC - Kumsheen Park, east of parking lot
Depth: 3.5-4.2m

A	Mass of dish + sample, before Ignition	(g)	165.349	177.159
B	Mass of dish + sample, after ignition	(g)	164.691	176.378
C	Mass of dish	(g)	50.980	62.070

*Furnace Temperature: **455 °C**

ORGANIC MATTER = $\frac{A - B}{A - C} \times 100$

0.6%	**0.7%**
0.6%	

Remarks:

Reviewed By: ______ P.Geol.

TETRA TECH

Standard Method of Test for Determining pH of Soil for Use in Corrosion Testing

AASHTO T289

Project No.:	704-ENG.VGEO03793-01.203	Date Tested:	September 28, 2022 Tested By: EM
Project:	KHR Bridges Replacement Project	Date Sampled:	See remarks
Client:	BC MOTI	Laboratory:	Suite 110, 140 Quarry Park Blvd. S.E., Calgary, AB T2G 3G3
Attention:			

Sample No.	**G1b**	Sample No.	**SPT01**
Source	**BH21-06**	Source	**BH21-06**
Sample Location	Golden, BC - 10th Ave N, north of river	Sample Location	Golden, BC - 10th Ave N, north of river
Depth: (m)	0.3-1.0	Depth: (m)	1.5-2.0
Measured pH	9.29	Measured pH	9.11

Sample No.	**SPT02**	Sample No.	**G01**
Source	**BH21-06**	Source	**BH21-09**
Sample Location	Golden, BC - 10th Ave N, north of river	Sample Location	Golden, BC - Kumsheen Park, east of parking lot
Depth: (m)	3.0-3.5	Depth: (m)	0.5-1.1
Measured pH	9.29	Measured pH	8.47

Sample No.	**G02**	Sample No.	**G03**
Source	**BH21-09**	Source	**BH21-09**
Sample Location	Golden, BC - Kumsheen Park, east of parking lot	Sample Location	Golden, BC - Kumsheen Park, east of parking lot
Depth: (m)	1.2-1.9	Depth: (m)	3.5-4.2
Measured pH	8.79	Measured pH	9.49

Sample No.		Sample No.	
Source		Source	
Sample Location		Sample Location	
Depth: (m)		Depth: (m)	
Measured pH		Measured pH	

Remarks: BH21-06 - September 14, 2021
BH21-09 - March 12-13, 2021

Reviewed By: ______________ P. Geol.

Measurement of Soil or Aggregate Resistivity using Two-Electrode Soil Box Method

AASHTO T 288

Project No:	704-ENG.VGEO03793-01.203	**Sample No.:**	G1b
Project:	KHR Bridges Replacement Project	**Date Sampled:**	September 14, 2021
Client:	BC MOTI	**Sampled By:**	-
		Date Tested:	September 29, 2022
Attention:	**Ph:**	**Tested By:**	EM
Email:		**Office:**	Suite 110, 140 Quarry Park Blvd. SE, Calgary, AB T2C 3G3

Description:	SILT and GRAVEL, sandy, trace clay, trace organics	Temperature of sample;	
Source:	BH21-06	As collected :	°C
Sample Location:	Golden, BC - 10th Ave N, north of river	As Tested :	21 °C
Depth:	0.1-1.0m		
Model of meter:	Miller 400A		

Resistivity of Aggregate (saturated with distilled water)

Resistivity = R x S / L

Box Used: **TB 3**

Sample Temperature: **21.0 °C**

Where :				
	L = Length of sample box (cm)	L =	10.8	cm
	h = height of sample in box	h =	2.4	cm
	S = Cross-sectional area of sample (cm^2)	S =	7.20	cm^2
	R = Resistance (Ohms)	R =	8100	Ohms

Resistivity = 8100 x 7.20 ÷ 10.8

Resistivity = $\boxed{\textbf{5,400}}$ Ohm•cm

Remarks:

Reviewed By: ______________ P.Geol.

TETRA TECH

Measurement of Soil or Aggregate Resistivity using Two-Electrode Soil Box Method

AASHTO T 288

Project No:	704-ENG.VGEO03793-01.203	**Sample No.:**	SPT01
Project:	KHR Bridges Replacement Project	**Date Sampled:**	September 14, 2021
Client:	BC MOTI	**Sampled By:**	-
		Date Tested:	September 29, 2022
Attention:	**Ph:**	**Tested By:**	EM
Email:		**Office:**	Suite 110, 140 Quarry Park Blvd. SE, Calgary, AB T2C 3G3

Description:	SILT and GRAVEL, sandy, trace clay, trace organics	Temperature of sample;	
Source:	BH21-06	As collected :	°C
Sample Location:	Golden, BC - 10th Ave N, north of river	As Tested :	21 °C
Depth:	1.5-2.0m		
Model of meter:	Miller 400A		

Resistivity of Aggregate (saturated with distilled water)

Resistivity = R x S / L

Box Used: **TB 3**

Sample Temperature: **21.0 °C**

Where :				
	L = Length of sample box (cm)	L =	10.8	cm
	h = height of sample in box	h =	2.4	cm
	S = Cross-sectional area of sample (cm^2)	S =	7.20	cm^2
	R = Resistance (Ohms)	R =	9300	Ohms

Resistivity = 9300 x 7.20 ÷ 10.8

Resistivity = $\boxed{\textbf{6,200}}$ Ohm•cm

Remarks:

Reviewed By: ______________________ P.Geol.

TETRA TECH

Measurement of Soil or Aggregate Resistivity using Two-Electrode Soil Box Method

AASHTO T 288

Project No:	704-ENG.VGEO03793-01.203	**Sample No.:**	SPT02
Project:	KHR Bridges Replacement Project	**Date Sampled:**	September 14, 2021
Client:	BC MOTI	**Sampled By:**	-
		Date Tested:	September 29, 2022
Attention:	**Ph:**	**Tested By:**	EM
Email:		**Office:**	Suite 110, 140 Quarry Park Blvd. SE, Calgary, AB T2C 3G3

Description:	SILT and GRAVEL, sandy, trace clay, trace organics	Temperature of sample;	
Source:	BH21-06	As collected :	°C
Sample Location:	Golden, BC - 10th Ave N, north of river	As Tested :	21 °C
Depth:	3.0-3.5m		
Model of meter:	Miller 400A		

Resistivity of Aggregate (saturated with distilled water)

Resistivity = R x S / L

Box Used: **TB 3**
Sample Temperature: **21.0 °C**

Where :

L = Length of sample box (cm)	L =	10.8	cm
h = height of sample in box	h =	2.4	cm
S = Cross-sectional area of sample (cm^2)	S =	7.20	cm^2
R = Resistance (Ohms)	R =	18000	Ohms

Resistivity = 18000 x 7.20 ÷ 10.8

Resistivity = **12,000** Ohm•cm

Remarks:

Reviewed By: ______________ P.Geol.

TETRA TECH

Measurement of Soil or Aggregate Resistivity using Two-Electrode Soil Box Method

AASHTO T 288

Project No:	704-ENG.VGEO03793-01.203	**Sample No.:**	G01
Project:	KHR Bridges Replacement Project	**Date Sampled:**	March 12-13, 2021
Client:	BC MOTI	**Sampled By:**	-
		Date Tested:	September 29, 2022
Attention:	**Ph:**	**Tested By:**	EM
Email:		**Office:**	Suite 110, 140 Quarry Park Blvd. SE, Calgary, AB T2C 3G3

Description:	CLAY, silty (topsoil), sandy, gravelly, some organics	Temperature of sample;	
Source:	BH21-09	As collected :	°C
Sample Location:	Golden, BC - Kumsheen Park, east of parking lot	As Tested :	21 °C
Depth:	0.5-1.1m		
Model of meter:	Miller 400A		

Resistivity of Aggregate (saturated with distilled water)

Resistivity = R x S / L

Box Used: **TB 3**

Sample Temperature: **21.0 °C**

Where :

L = Length of sample box (cm)	L =	10.8	cm
h = height of sample in box	h =	2.4	cm
S = Cross-sectional area of sample (cm^2)	S =	7.20	cm^2
R = Resistance (Ohms)	R =	5300	Ohms

Resistivity = 5300 x 7.20 ÷ 10.8

Resistivity = **3,533** Ohm•cm

Remarks:

Reviewed By: P.Geol.

TETRA TECH

Measurement of Soil or Aggregate Resistivity using Two-Electrode Soil Box Method

AASHTO T 288

Project No:	704-ENG.VGEO03793-01.203	**Sample No.:**	G02
Project:	KHR Bridges Replacement Project	**Date Sampled:**	March 12-13, 2021
Client:	BC MOTI	**Sampled By:**	-
		Date Tested:	September 29, 2022
Attention:	**Ph:**	**Tested By:**	EM
Email:		**Office:**	Suite 110, 140 Quarry Park Blvd. SE, Calgary, AB T2C 3G3

Description:	SAND and GRAVEL, some silt, trace clay, trace organics	Temperature of sample;	
Source:	BH21-09	As collected :	°C
Sample Location:	Golden, BC - Kumsheen Park, east of parking lot	As Tested :	21 °C
Depth:	1.2-1.9m		
Model of meter:	Miller 400A		

Resistivity of Aggregate (saturated with distilled water)

Resistivity = R x S / L

Box Used: **TB 3**

Sample Temperature: **21.0 °C**

Where :

L = Length of sample box (cm)	L =	10.8	cm
h = height of sample in box	h =	2.4	cm
S = Cross-sectional area of sample (cm^2)	S =	7.20	cm^2
R = Resistance (Ohms)	R =	12000	Ohms

Resistivity = 12000 x 7.20 ÷ 10.8

Resistivity = **8,000** Ohm•cm

Remarks:

Reviewed By: P.Geol.

TETRA TECH

Measurement of Soil or Aggregate Resistivity using Two-Electrode Soil Box Method

AASHTO T 288

Project No: 704-ENG.VGEO03793-01.203
Project: KHR Bridges Replacement Project
Client: BC MOTI

Attention: **Ph:**
Email:

Sample No.: G03
Date Sampled: March 12-13, 2021
Sampled By: -
Date Tested: September 29, 2022
Tested By: EM
Office: Suite 110, 140 Quarry Park Blvd. SE, Calgary, AB T2C 3G3

Description: SAND & GRAVEL, some silt, some clay, trace organics
Source: BH21-09
Sample Location: Golden, BC - Kumsheen Park, east of parking lot
Depth: 3.5-4.2m
Model of meter: Miller 400A

Temperature of sample;
As collected : °C
As Tested : 21 °C

Resistivity of Aggregate (saturated with distilled water)

Resistivity = R x S / L

Box Used: **TB 3**
Sample Temperature: **21.0 °C**

Where :

L = Length of sample box (cm)	L =	10.8	cm
h = height of sample in box	h =	2.4	cm
S = Cross-sectional area of sample (cm^2)	S =	7.20	cm^2
R = Resistance (Ohms)	R =	23000	Ohms

Resistivity = 23000 x 7.20 ÷ 10.8

Resistivity = **15,333** Ohm•cm

Remarks:

Reviewed By: P.Geol.

TETRA TECH

Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil

AASHTO T291

Project No.:	704-ENG.VGEO03793-01.203	Date Tested:	September 29, 2022 Tested By: EM
Project:	KHR Bridges Replacement Project	Date Sampled:	See Remarks
Client:	BC MOTI	Laboratory:	Suite 110, 140 Quarry Park Blvd. S.E., Calgary, AB T2G 3G3

Sample No.	**G1b**	Sample No.	**SPT01**
Source	**BH21-06**	Source	**BH21-06**
Sample Location	Golden, BC - 10th Ave N, north of river	Sample Location	Golden, BC - 10th Ave N, north of river
Depth (m):	0.1-1.0	Depth (m):	1.5-2.0
Chloride Content (ppm by Mass of Sample)	36	Chloride Content (ppm by Mass of Sample)	27

Sample No.	**SPT02**	Sample No.	**G01**
Source	**BH21-06**	Source	**BH21-09**
Sample Location	Golden, BC - 10th Ave N, north of river	Sample Location	Golden, BC - Kumsheen Park, east of parking lot
Depth (m):	3.0-3.5	Depth (m):	0.5-1.1
Chloride Content (ppm by Mass of Sample)	17	Chloride Content (ppm by Mass of Sample)	15

Sample No.	**G02**	Sample No.	**G03**
Source	**BH21-09**	Source	**BH21-09**
Sample Location	Golden, BC - Kumsheen Park, east of parking lot	Sample Location	Golden, BC - Kumsheen Park, east of parking lot
Depth (m):	1.2-1.9	Depth (m):	3.5-4.2
Chloride Content (ppm by Mass of Sample)	15	Chloride Content (ppm by Mass of Sample)	11

Remarks: BH21-06 - September 14, 2021

BH21-09 - March 12-13, 2021

Reviewed By: ____________________ P.Geol.

TETRA TECH

Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil

AASHTO T290

Project No.:	704-ENG.VGEO03793-01.203	Date Tested:	September 28-29, 2022 Tested By: EM
Project:	KHR Bridges Replacement Project	Date Sampled:	See remarks
Client:	BC MOTI	Laboratory:	Suite 110, 140 Quarry Park Blvd. S.E., Calgary, AB T2G 3G3

Sample No.	**G1b**	Sample No.	**SPT01**
Source	**BH21-06**	Source	**BH21-06**
Sample Location	Golden, BC - 10th Ave N, north of river	Sample Location	Golden, BC - 10th Ave N, north of river
Depth (m):	0.1-1.0	Depth (m):	1.5-2.0
Sulphate Content (ppm by Mass of Sample)	10	Sulphate Content (ppm by Mass of Sample)	14

Sample No.	**SPT02**	Sample No.	**G01**
Source	**BH21-06**	Source	**BH21-09**
Sample Location	Golden, BC - 10th Ave N, north of rive	Sample Location	Golden, BC - Kumsheen Park, east of parking lot
Depth (m):	3.0-3.5	Depth (m):	0.5-1.1
Sulphate Content (ppm by Mass of Sample)	6	Sulphate Content (ppm by Mass of Sample)	74

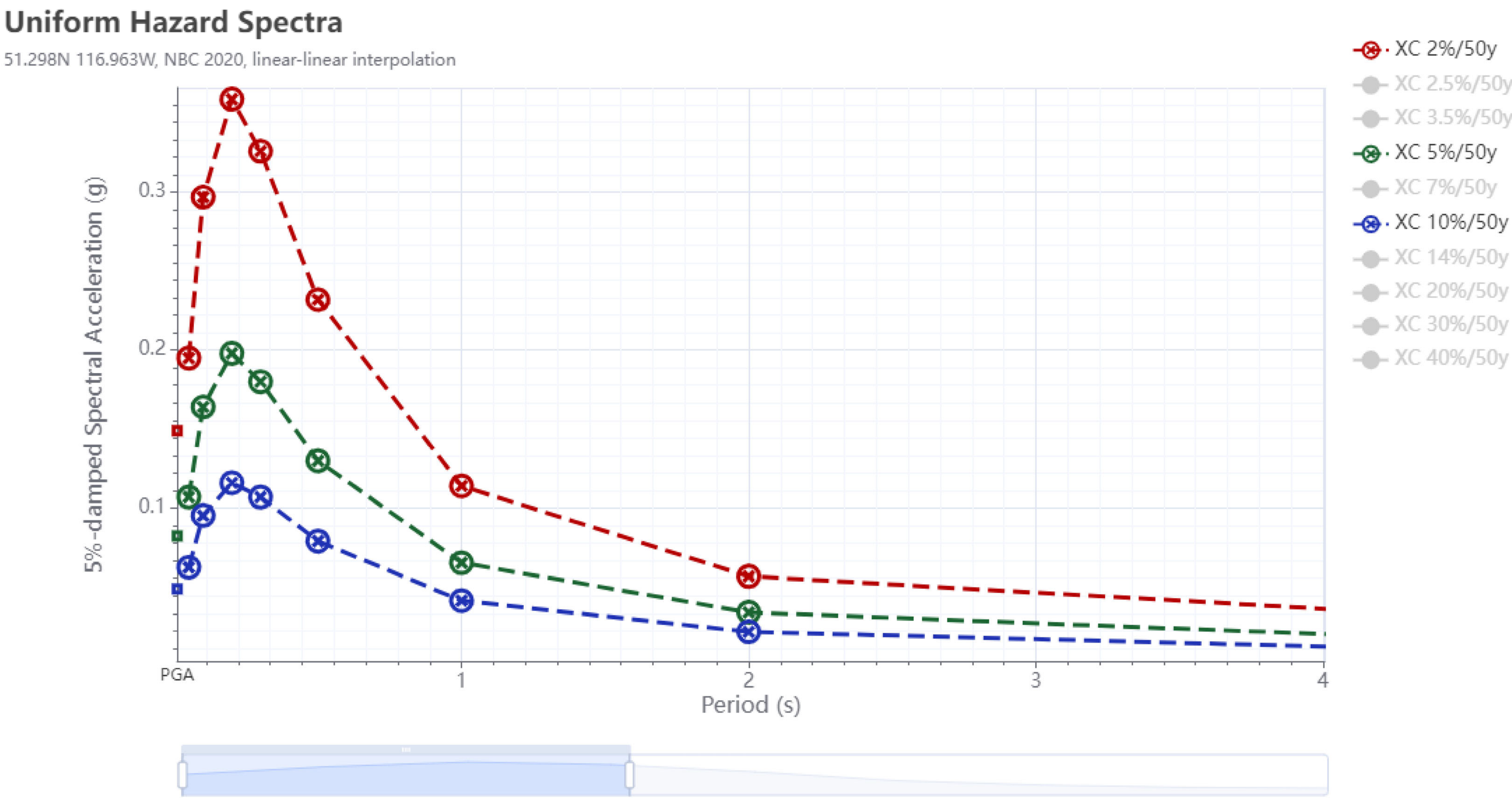
Sample No.	**G02**	Sample No.	**G03**
Sample Location	**BH21-09**	Source	**BH21-09**
Source	Golden, BC - Kumsheen Park, east of parking lot	Sample Location	Golden, BC - Kumsheen Park, east of parking lot
Sample Location	1.2-1.9	Depth (m):	3.5-4.2
Sulphate Content (ppm by Mass of Sample)	35	Sulphate Content (ppm by Mass of Sample)	21

Remarks: BH21-06 - September 14, 2021
BH21-09 - March 12-13, 2021

Reviewed By: ______________________ P.Geol.

APPENDIX C

SEISMIC HAZARD



Uniform Hazard Spectra
51.298N 116.963W, NBC 2020, linear-linear interpolation
XC 2%/50y
XC 2.5%/50y
XC 3.5%/50y
XC 5%/50y
XC 7%/50y
XC 10%/50y
XC 14%/50y
XC 20%/50y
XC 30%/50y
XC 40%/50y
5%-damped Spectral Acceleration (g)
0.3
0.2
0.1
PGA
1
2
3
4
Period (s)

Seismic Hazard Deaggregation

calculated by the Canadian Hazards Information Service

INFORMATION: *EarthquakesCanada.nrcan.gc.ca*

Eastern Canada (613) 995-5548 Western Canada (250) 363-6500

Requested by: Lothar Chan, Tetra Tech 2022/09/07

For site Golden, BC at 51.298 N 116.963 W

For ground motion parameter peak ground acceleration (PGA)

at a probability of 0.000404 per annum, seismic hazard = 0.119 g

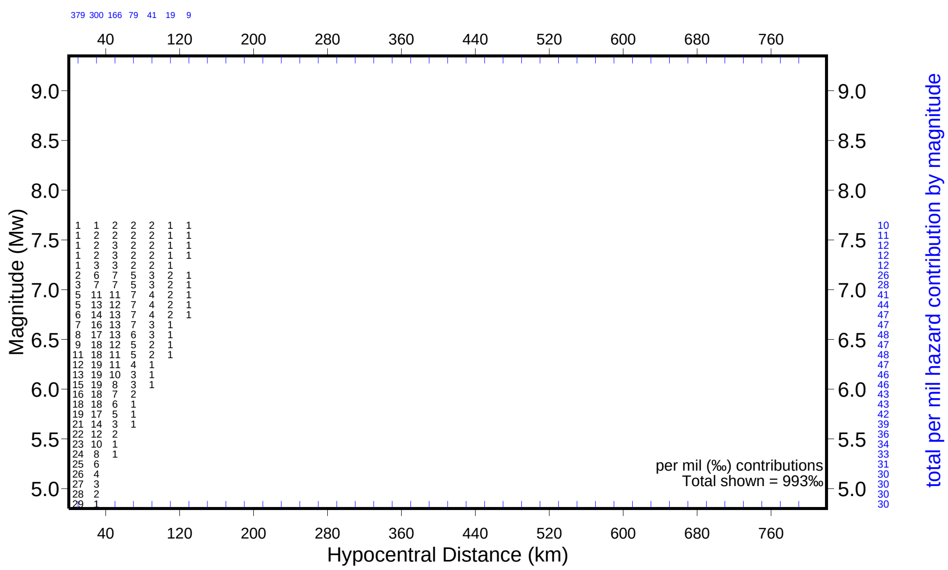
Soil Class C, 2015 Geological Survey of Canada 5th Generation model as prepared for NBCC2015

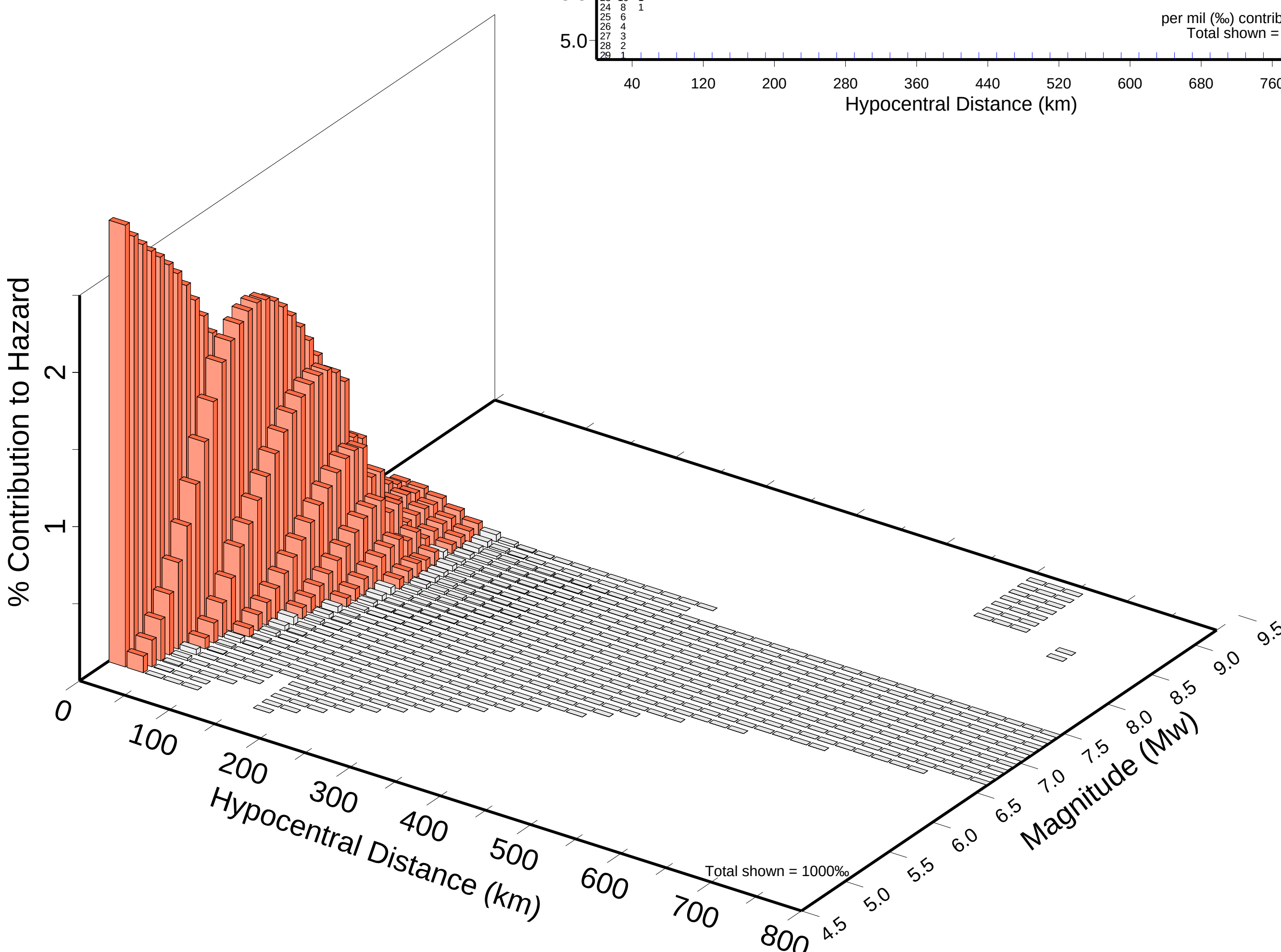
Mean magnitude (Mw) 6.14 Mean distance 35 km

Mode magnitude (Mw) 4.850 Mode distance 10 km

Deaggregation of mean hazard

Model: SWCan_2015clC.model





Aussi disponible en francais

Natural Resources Canada Ressources naturelles Canada

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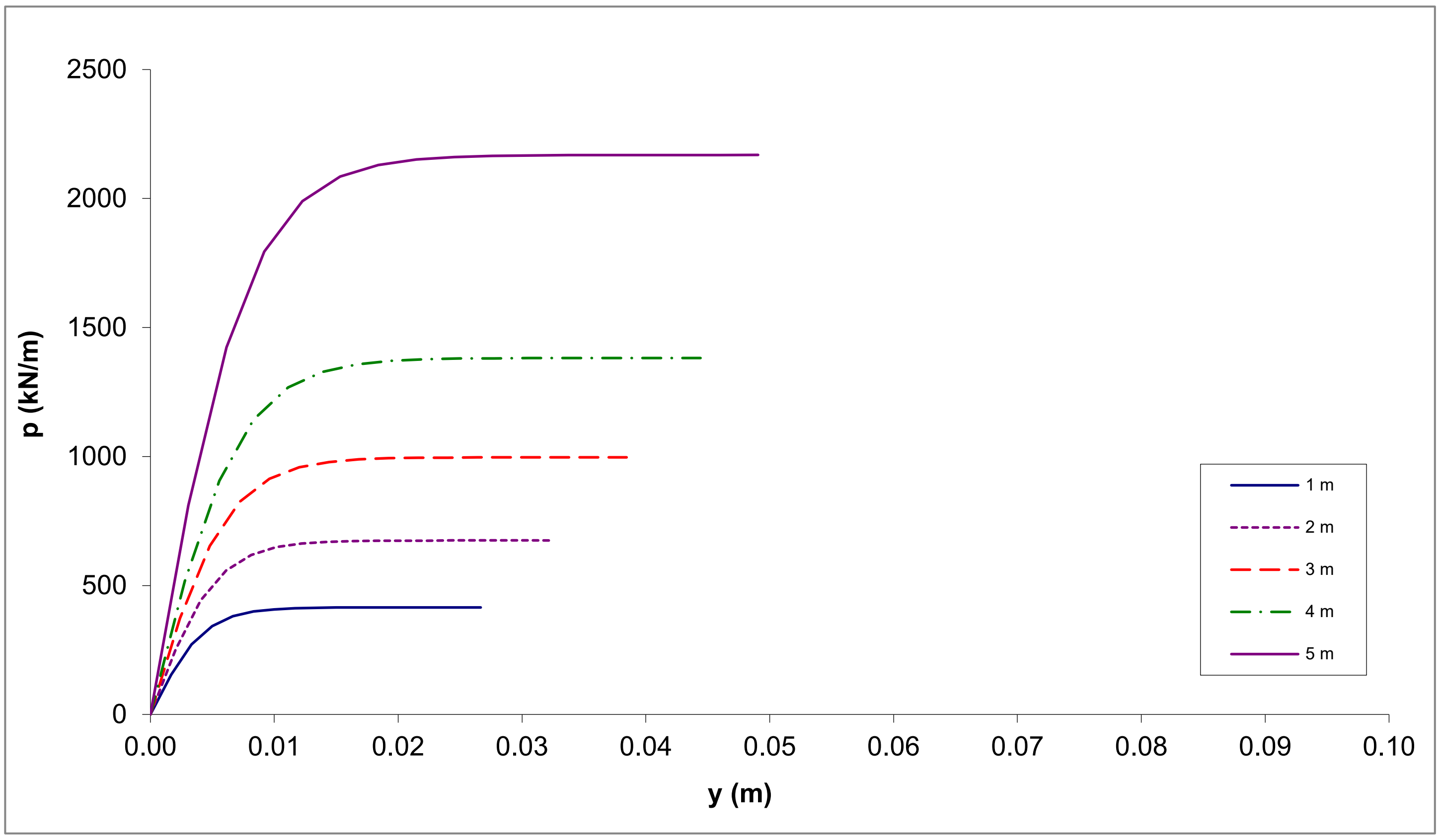
APPENDIX D

P-Y CURVES AND PILE GROUP REDUCTION FACTORS

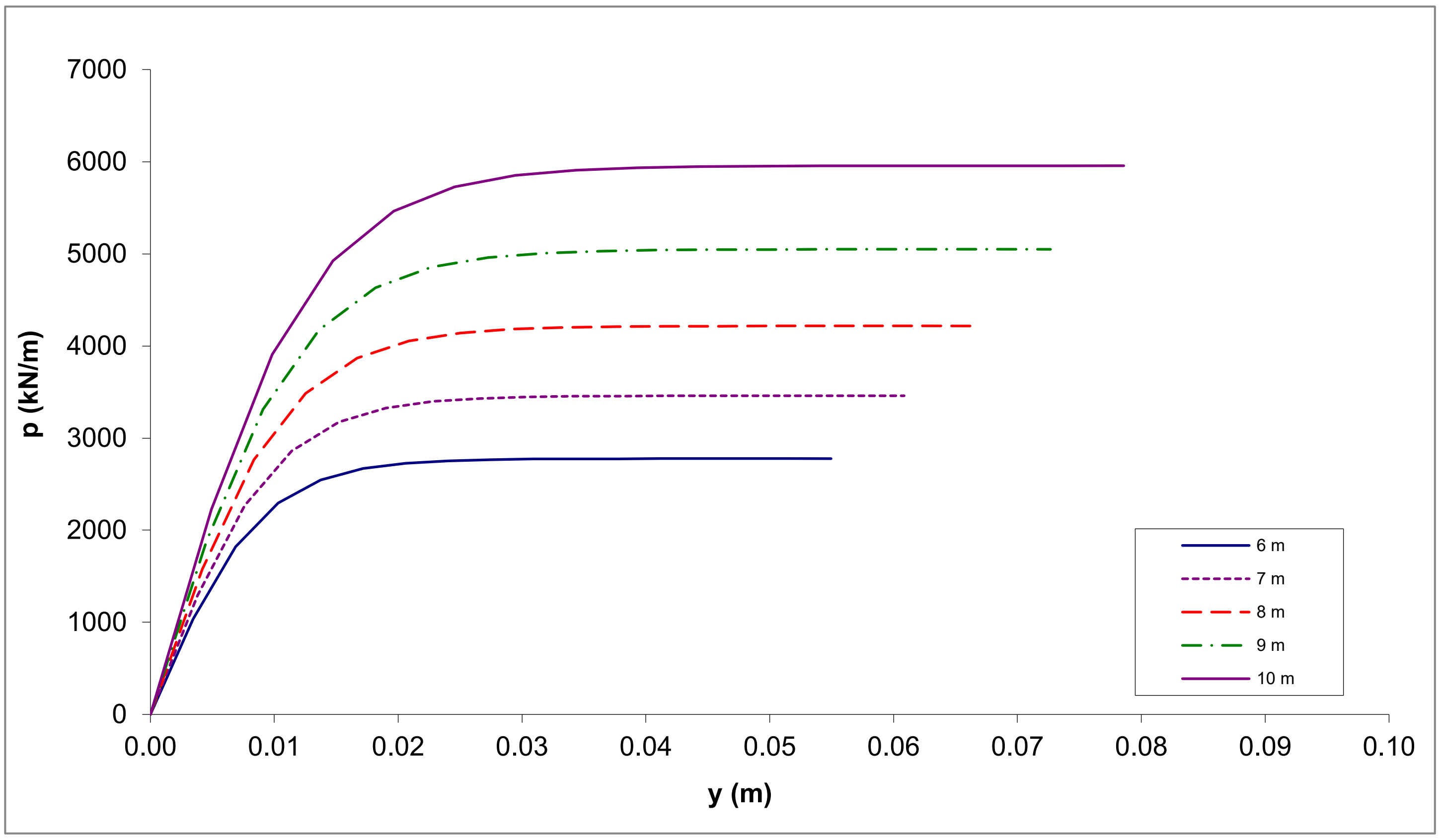


1 m		2 m		3 m		4 m		5 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00167	155.02	0.00203	252.03	0.00240	372.47	0.00278	516.34	0.00307	810.58
0.00333	272.05	0.00406	442.30	0.00481	653.66	0.00555	906.14	0.00613	1422.52
0.00500	343.02	0.00610	557.67	0.00721	824.16	0.00833	1142.50	0.00920	1793.57
0.00667	380.48	0.00813	618.57	0.00961	914.17	0.01110	1267.27	0.01227	1989.45
0.00833	398.81	0.01016	648.38	0.01201	958.23	0.01388	1328.35	0.01533	2085.34
0.01000	407.46	0.01219	662.44	0.01442	979.00	0.01665	1357.14	0.01840	2130.54
0.01167	411.46	0.01423	668.94	0.01682	988.62	0.01943	1370.48	0.02146	2151.47
0.01333	413.30	0.01626	671.93	0.01922	993.03	0.02221	1376.60	0.02453	2161.08
0.01500	414.14	0.01829	673.30	0.02162	995.05	0.02498	1379.40	0.02760	2165.47
0.01667	414.52	0.02032	673.92	0.02403	995.97	0.02776	1380.68	0.03066	2167.48
0.01833	414.70	0.02235	674.21	0.02643	996.40	0.03053	1381.26	0.03373	2168.40
0.02000	414.78	0.02439	674.34	0.02883	996.59	0.03331	1381.53	0.03680	2168.81
0.02167	414.82	0.02642	674.40	0.03124	996.67	0.03608	1381.65	0.03986	2169.00
0.02333	414.83	0.02845	674.42	0.03364	996.71	0.03886	1381.70	0.04293	2169.09
0.02500	414.84	0.03048	674.44	0.03604	996.73	0.04163	1381.73	0.04599	2169.13
0.02667	414.84	0.03252	674.44	0.03844	996.74	0.04441	1381.74	0.04906	2169.15

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.

Note - The depths are provided as depths below top of pile.

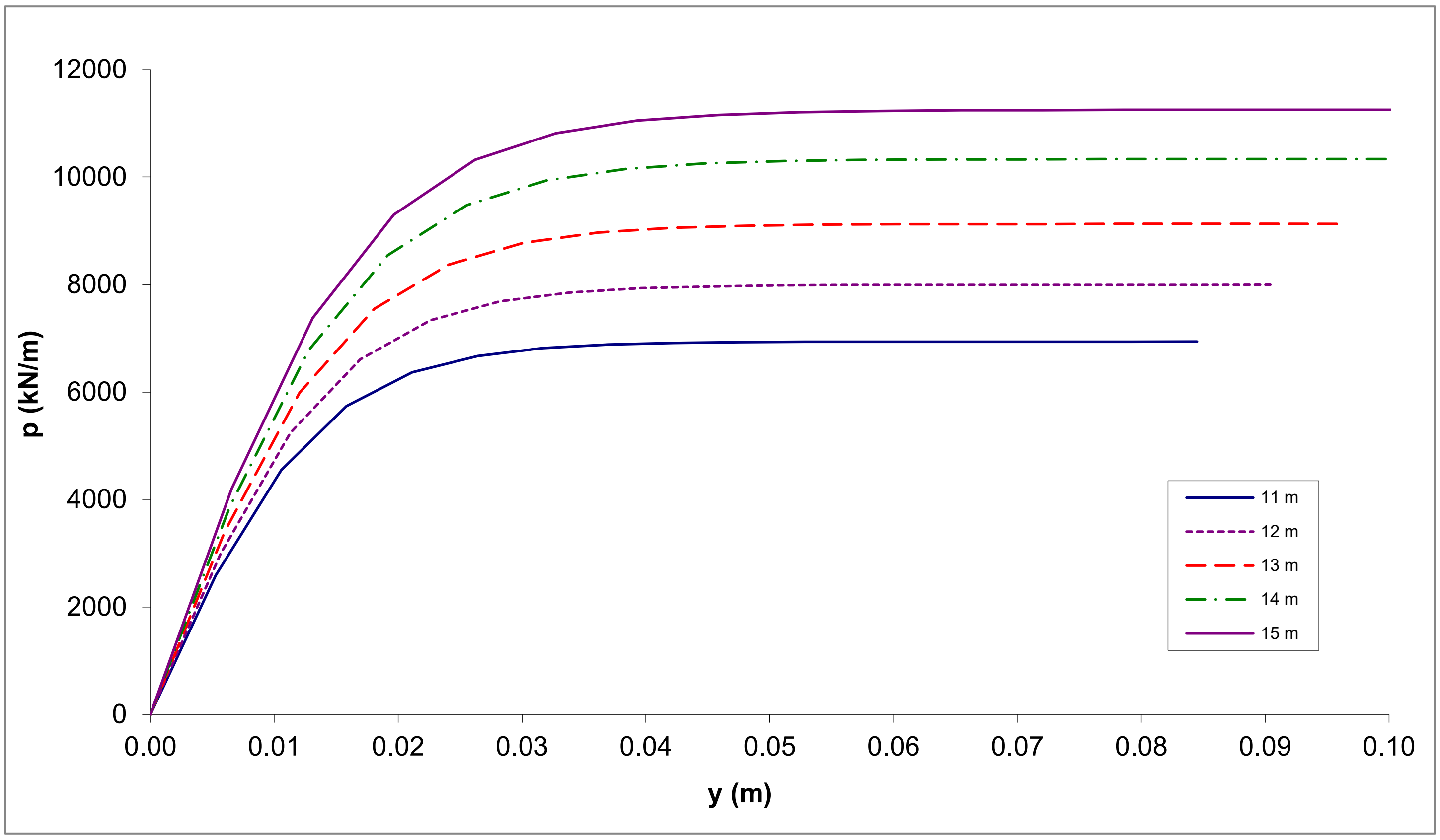
ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative Case (no Scour)



6 m		7 m		8 m		9 m		10 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00343	1037.63	0.00380	1292.66	0.00417	1575.69	0.00454	1886.71	0.00491	2225.72
0.00687	1820.97	0.00761	2268.55	0.00835	2765.24	0.00908	3311.06	0.00982	3906.01
0.01030	2295.95	0.01141	2860.27	0.01252	3486.52	0.01363	4174.72	0.01473	4924.85
0.01374	2546.70	0.01521	3172.64	0.01669	3867.29	0.01817	4630.64	0.01965	5462.70
0.01717	2669.44	0.01902	3325.55	0.02086	4053.68	0.02271	4853.82	0.02456	5725.98
0.02061	2727.30	0.02282	3397.64	0.02503	4141.55	0.02725	4959.03	0.02947	5850.09
0.02404	2754.09	0.02662	3431.01	0.02921	4182.23	0.03179	5007.75	0.03438	5907.56
0.02748	2766.39	0.03043	3446.34	0.03338	4200.91	0.03633	5030.11	0.03929	5933.95
0.03091	2772.02	0.03423	3453.35	0.03755	4209.46	0.04088	5040.35	0.04420	5946.02
0.03434	2774.59	0.03803	3456.55	0.04172	4213.36	0.04542	5045.02	0.04911	5951.53
0.03778	2775.76	0.04184	3458.01	0.04590	4215.14	0.04996	5047.15	0.05402	5954.04
0.04121	2776.30	0.04564	3458.68	0.05007	4215.95	0.05450	5048.12	0.05894	5955.19
0.04465	2776.54	0.04944	3458.98	0.05424	4216.32	0.05904	5048.56	0.06385	5955.71
0.04808	2776.65	0.05324	3459.12	0.05841	4216.49	0.06358	5048.77	0.06876	5955.95
0.05152	2776.70	0.05705	3459.18	0.06258	4216.57	0.06813	5048.86	0.07367	5956.06
0.05495	2776.72	0.06085	3459.21	0.06676	4216.60	0.07267	5048.90	0.07858	5956.11

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.
Note - The depths are provided as depths below top of pile.

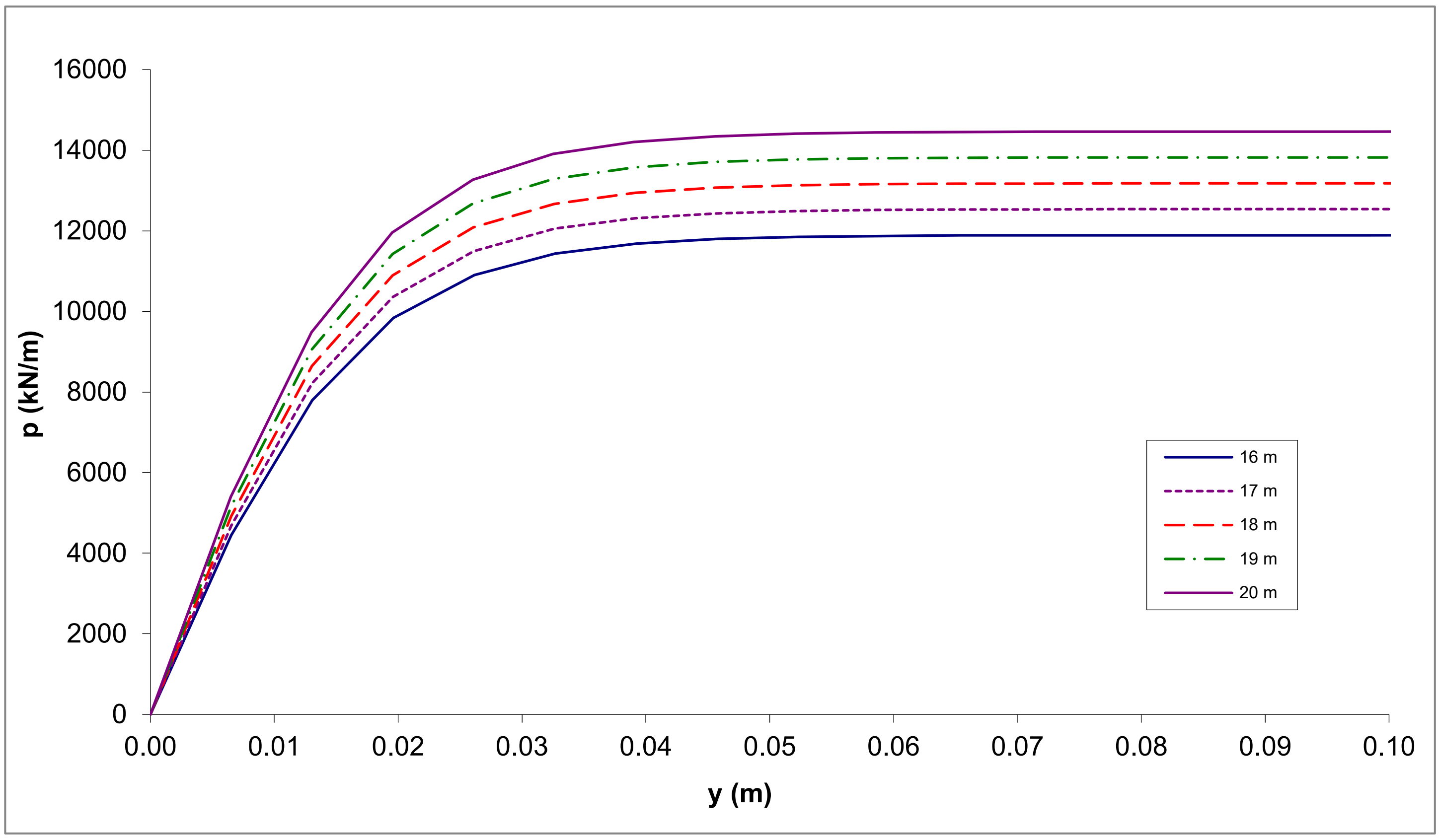
ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative Case (no Scour)



11 m		12 m		13 m		14 m		15 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00528	2592.73	0.00565	2987.72	0.00602	3410.71	0.00639	3861.69	0.00655	4203.73
0.01056	4550.08	0.01130	5243.27	0.01204	5985.59	0.01278	6777.03	0.01310	7377.30
0.01584	5736.91	0.01695	6610.92	0.01806	7546.86	0.01917	8544.75	0.01964	9301.58
0.02112	6363.45	0.02260	7332.91	0.02408	8371.07	0.02556	9477.93	0.02619	10317.42
0.02641	6670.15	0.02825	7686.33	0.03010	8774.52	0.03195	9934.73	0.03274	10814.68
0.03169	6814.72	0.03391	7852.93	0.03613	8964.71	0.03835	10150.07	0.03929	11049.09
0.03697	6881.67	0.03956	7930.07	0.04215	9052.78	0.04474	10249.78	0.04583	11157.63
0.04225	6912.41	0.04521	7965.50	0.04817	9093.21	0.05113	10295.56	0.05238	11207.47
0.04753	6926.47	0.05086	7981.70	0.05419	9111.71	0.05752	10316.50	0.05893	11230.27
0.05281	6932.89	0.05651	7989.10	0.06021	9120.16	0.06391	10326.06	0.06548	11240.68
0.05809	6935.82	0.06216	7992.47	0.06623	9124.01	0.07030	10330.43	0.07203	11245.43
0.06337	6937.15	0.06781	7994.01	0.07225	9125.77	0.07669	10332.42	0.07857	11247.59
0.06865	6937.76	0.07346	7994.71	0.07827	9126.57	0.08308	10333.32	0.08512	11248.58
0.07393	6938.04	0.07911	7995.03	0.08429	9126.93	0.08947	10333.74	0.09167	11249.03
0.07922	6938.17	0.08476	7995.18	0.09031	9127.10	0.09586	10333.93	0.09822	11249.23
0.08450	6938.22	0.09041	7995.25	0.09633	9127.18	0.10225	10334.01	0.10476	11249.33

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.
Note - The depths are provided as depths below top of pile.

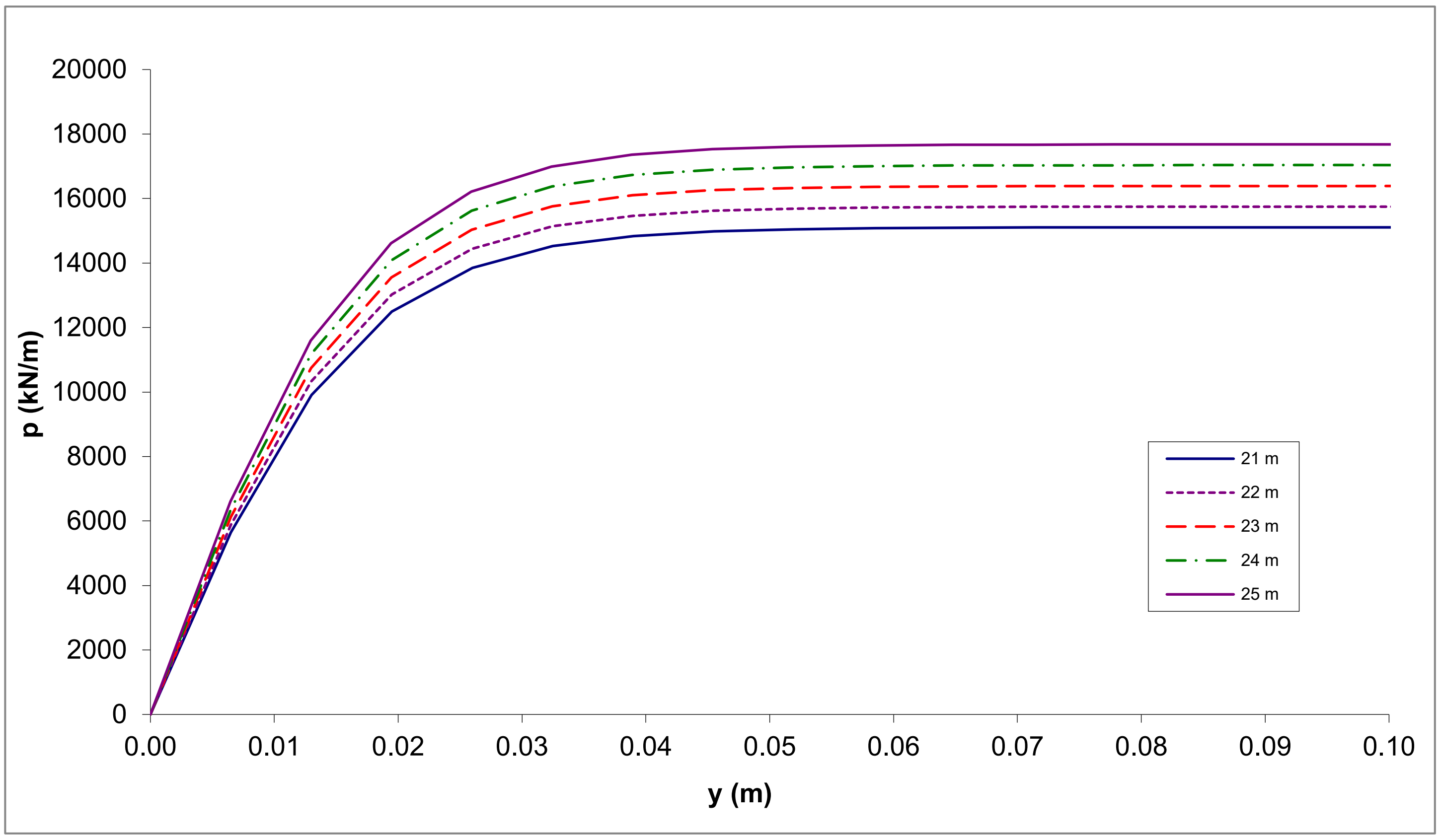
ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative Case (no Scour)



16 m		17 m		18 m		19 m		20 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00654	4443.95	0.00653	4684.16	0.00652	4924.37	0.00651	5164.58	0.00651	5404.80
0.01308	7798.86	0.01306	8220.42	0.01304	8641.98	0.01302	9063.53	0.01301	9485.09
0.01961	9833.10	0.01958	10364.62	0.01956	10896.14	0.01954	11427.65	0.01952	11959.17
0.02615	10906.99	0.02611	11496.55	0.02608	12086.12	0.02605	12675.69	0.02602	13265.25
0.03269	11432.66	0.03264	12050.64	0.03260	12668.63	0.03256	13286.61	0.03253	13904.59
0.03922	11680.47	0.03917	12311.85	0.03912	12943.22	0.03907	13574.60	0.03903	14205.98
0.04576	11795.21	0.04570	12432.79	0.04564	13070.37	0.04558	13707.95	0.04554	14345.53
0.05230	11847.90	0.05222	12488.32	0.05216	13128.75	0.05210	13769.18	0.05204	14409.60
0.05884	11872.00	0.05875	12513.73	0.05868	13155.46	0.05861	13797.18	0.05855	14438.91
0.06537	11883.00	0.06528	12525.32	0.06520	13167.65	0.06512	13809.97	0.06505	14452.30
0.07191	11888.02	0.07181	12530.62	0.07172	13173.21	0.07163	13815.81	0.07156	14458.40
0.07845	11890.31	0.07834	12533.03	0.07824	13175.75	0.07815	13818.47	0.07806	14461.19
0.08499	11891.35	0.08486	12534.13	0.08476	13176.91	0.08466	13819.68	0.08457	14462.46
0.09152	11891.83	0.09139	12534.63	0.09128	13177.43	0.09117	13820.24	0.09107	14463.04
0.09806	11892.05	0.09792	12534.86	0.09780	13177.67	0.09768	13820.49	0.09758	14463.30
0.10460	11892.15	0.10445	12534.97	0.10431	13177.78	0.10419	13820.60	0.10408	14463.42

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.
Note - The depths are provided as depths below top of pile.

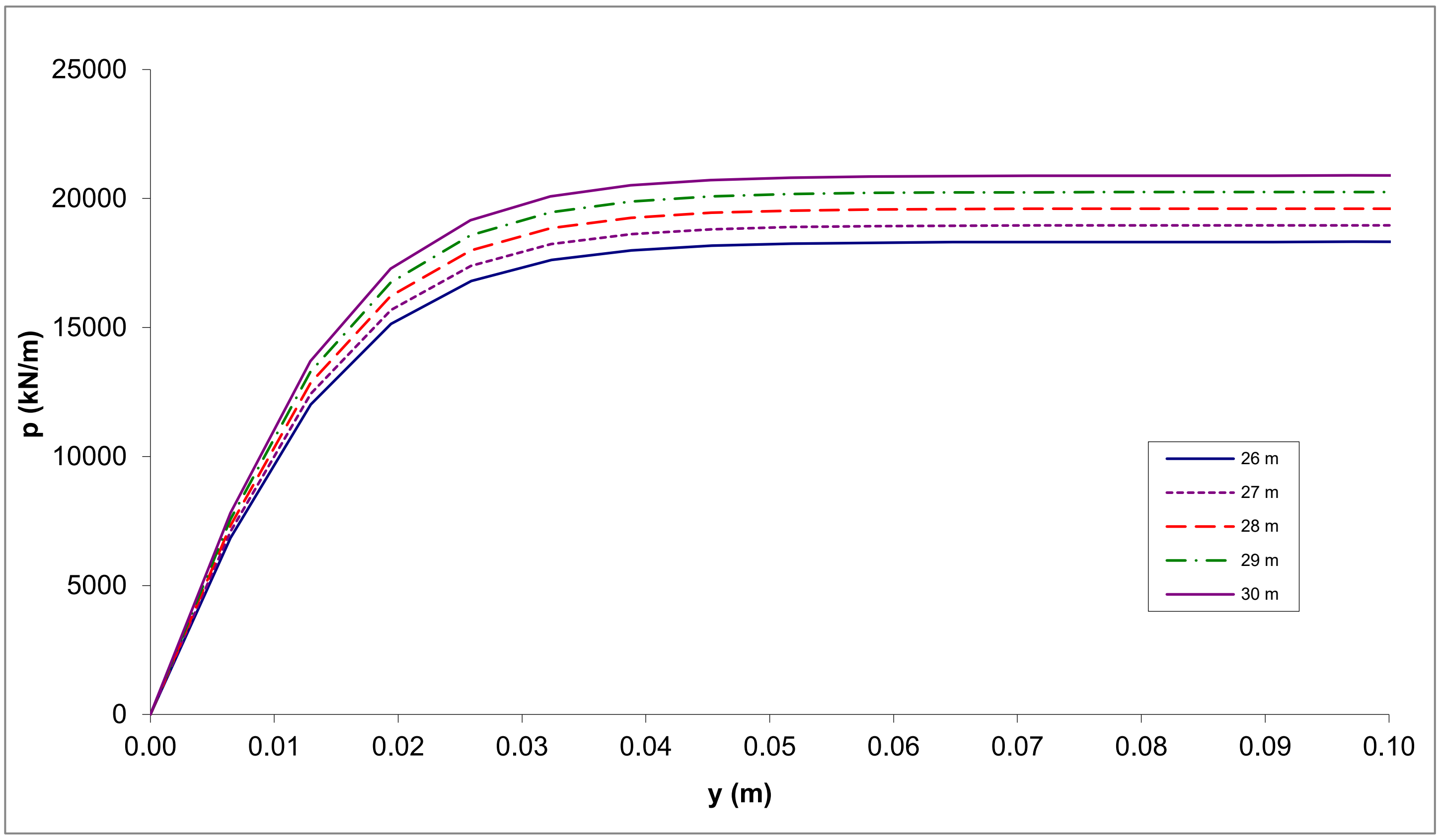
ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative Case (no Scour)



21 m		22 m		23 m		24 m		25 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00650	5645.01	0.00649	5885.22	0.00649	6125.44	0.00648	6365.65	0.00648	6605.86
0.01300	9906.65	0.01299	10328.21	0.01298	10749.77	0.01297	11171.33	0.01296	11592.89
0.01950	12490.69	0.01948	13022.21	0.01946	13553.73	0.01945	14085.25	0.01944	14616.77
0.02600	13854.82	0.02597	14444.39	0.02595	15033.95	0.02593	15623.52	0.02591	16213.09
0.03249	14522.57	0.03247	15140.55	0.03244	15758.53	0.03242	16376.52	0.03239	16994.50
0.03899	14837.35	0.03896	15468.73	0.03893	16100.11	0.03890	16731.48	0.03887	17362.86
0.04549	14983.10	0.04545	15620.68	0.04542	16258.26	0.04538	16895.84	0.04535	17533.42
0.05199	15050.03	0.05195	15690.46	0.05190	16330.88	0.05186	16971.31	0.05183	17611.74
0.05849	15080.64	0.05844	15722.37	0.05839	16364.10	0.05835	17005.83	0.05831	17647.56
0.06499	15094.62	0.06493	15736.95	0.06488	16379.27	0.06483	17021.59	0.06478	17663.92
0.07149	15101.00	0.07142	15743.60	0.07137	16386.19	0.07131	17028.79	0.07126	17671.38
0.07799	15103.91	0.07792	15746.63	0.07785	16389.35	0.07780	17032.07	0.07774	17674.79
0.08449	15105.23	0.08441	15748.01	0.08434	16390.79	0.08428	17033.56	0.08422	17676.34
0.09099	15105.84	0.09090	15748.64	0.09083	16391.44	0.09076	17034.24	0.09070	17677.05
0.09748	15106.11	0.09740	15748.93	0.09732	16391.74	0.09724	17034.55	0.09718	17677.37
0.10398	15106.24	0.10389	15749.06	0.10381	16391.88	0.10373	17034.70	0.10366	17677.52

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.
Note - The depths are provided as depths below top of pile.

ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative Case (no Scour)

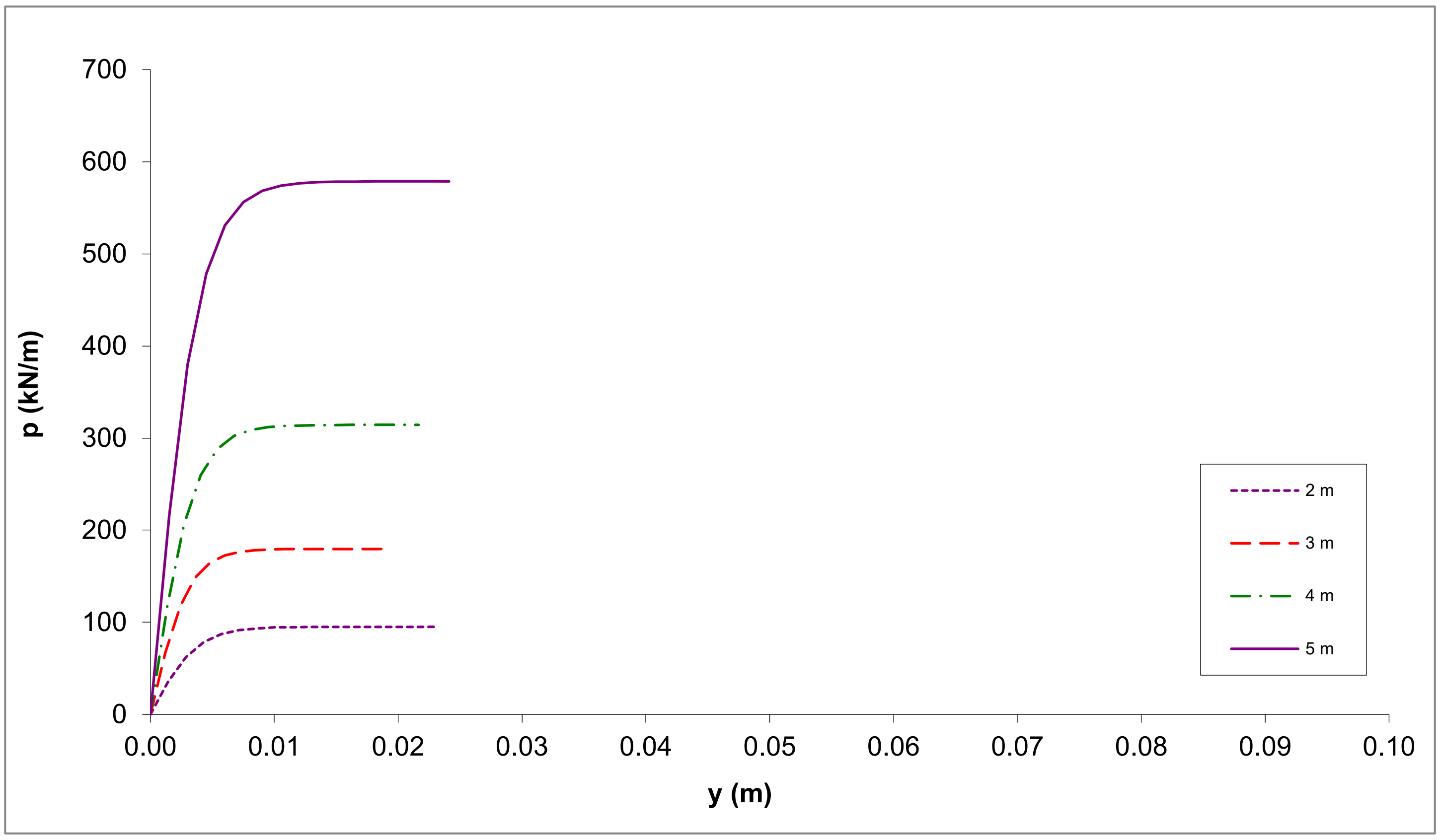


26 m		27 m		28 m		29 m		30 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00647	6846.08	0.00647	7086.29	0.00647	7326.50	0.00646	7566.72	0.00646	7806.93
0.01295	12014.45	0.01294	12436.01	0.01293	12857.57	0.01293	13279.13	0.01292	13700.69
0.01942	15148.29	0.01941	15679.81	0.01940	16211.32	0.01939	16742.84	0.01938	17274.36
0.02590	16802.65	0.02588	17392.22	0.02587	17981.79	0.02585	18571.35	0.02584	19160.92
0.03237	17612.48	0.03235	18230.46	0.03233	18848.44	0.03232	19466.42	0.03230	20084.41
0.03885	17994.24	0.03882	18625.61	0.03880	19256.99	0.03878	19888.37	0.03876	20519.74
0.04532	18171.00	0.04529	18808.58	0.04527	19446.16	0.04524	20083.74	0.04522	20721.31
0.05179	18252.16	0.05176	18892.59	0.05173	19533.02	0.05171	20173.45	0.05168	20813.87
0.05827	18289.29	0.05823	18931.02	0.05820	19572.75	0.05817	20214.48	0.05814	20856.21
0.06474	18306.24	0.06470	18948.57	0.06467	19590.89	0.06463	20233.22	0.06460	20875.54
0.07122	18313.98	0.07117	18956.57	0.07113	19599.17	0.07110	20241.76	0.07106	20884.36
0.07769	18317.51	0.07764	18960.22	0.07760	19602.94	0.07756	20245.66	0.07752	20888.38
0.08417	18319.11	0.08411	18961.89	0.08407	19604.67	0.08402	20247.44	0.08398	20890.22
0.09064	18319.85	0.09058	18962.65	0.09053	19605.45	0.09049	20248.25	0.09044	20891.05
0.09711	18320.18	0.09705	18963.00	0.09700	19605.81	0.09695	20248.62	0.09690	20891.44
0.10359	18320.33	0.10353	18963.15	0.10347	19605.97	0.10341	20248.79	0.10336	20891.61

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.

Note - The depths are provided as depths below top of pile.

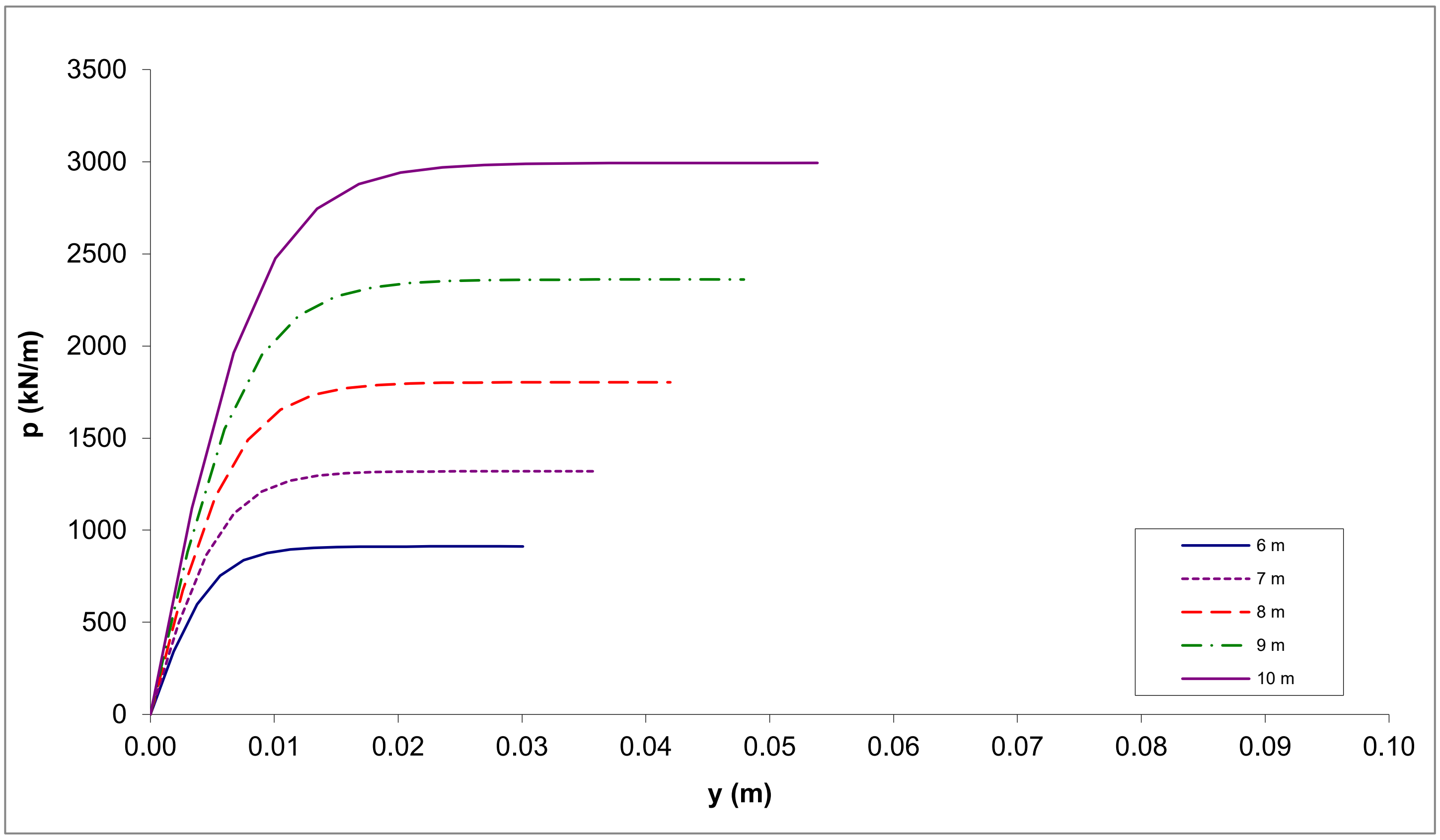
ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative Case (no Scour)



1 m		2 m		3 m		4 m		5 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
N/A	N/A	0	0.00	0	0.00	0	0.00	0	0.00
N/A	N/A	0.00143	35.56	0.00120	67.13	0.00135	117.46	0.00151	216.23
N/A	N/A	0.00287	62.40	0.00241	117.82	0.00271	206.13	0.00301	379.47
N/A	N/A	0.00430	78.68	0.00361	148.55	0.00406	259.90	0.00452	478.45
N/A	N/A	0.00573	87.27	0.00481	164.77	0.00541	288.28	0.00603	530.70
N/A	N/A	0.00717	91.48	0.00602	172.71	0.00677	302.17	0.00753	556.28
N/A	N/A	0.00860	93.46	0.00722	176.45	0.00812	308.72	0.00904	568.34
N/A	N/A	0.01004	94.38	0.00842	178.19	0.00947	311.76	0.01055	573.92
N/A	N/A	0.01147	94.80	0.00962	178.98	0.01082	313.15	0.01205	576.48
N/A	N/A	0.01290	94.99	0.01083	179.35	0.01218	313.79	0.01356	577.66
N/A	N/A	0.01434	95.08	0.01203	179.51	0.01353	314.08	0.01507	578.19
N/A	N/A	0.01577	95.12	0.01323	179.59	0.01488	314.21	0.01657	578.44
N/A	N/A	0.01720	95.14	0.01444	179.62	0.01624	314.27	0.01808	578.55
N/A	N/A	0.01864	95.15	0.01564	179.64	0.01759	314.30	0.01959	578.60
N/A	N/A	0.02007	95.15	0.01684	179.65	0.01894	314.31	0.02109	578.62
N/A	N/A	0.02150	95.15	0.01804	179.65	0.02030	314.32	0.02260	578.63
N/A	N/A	0.02294	95.16	0.01925	179.65	0.02165	314.32	0.02411	578.64

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.
Note - The depths are provided as depths below top of pile.

ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative 50% Scour Case

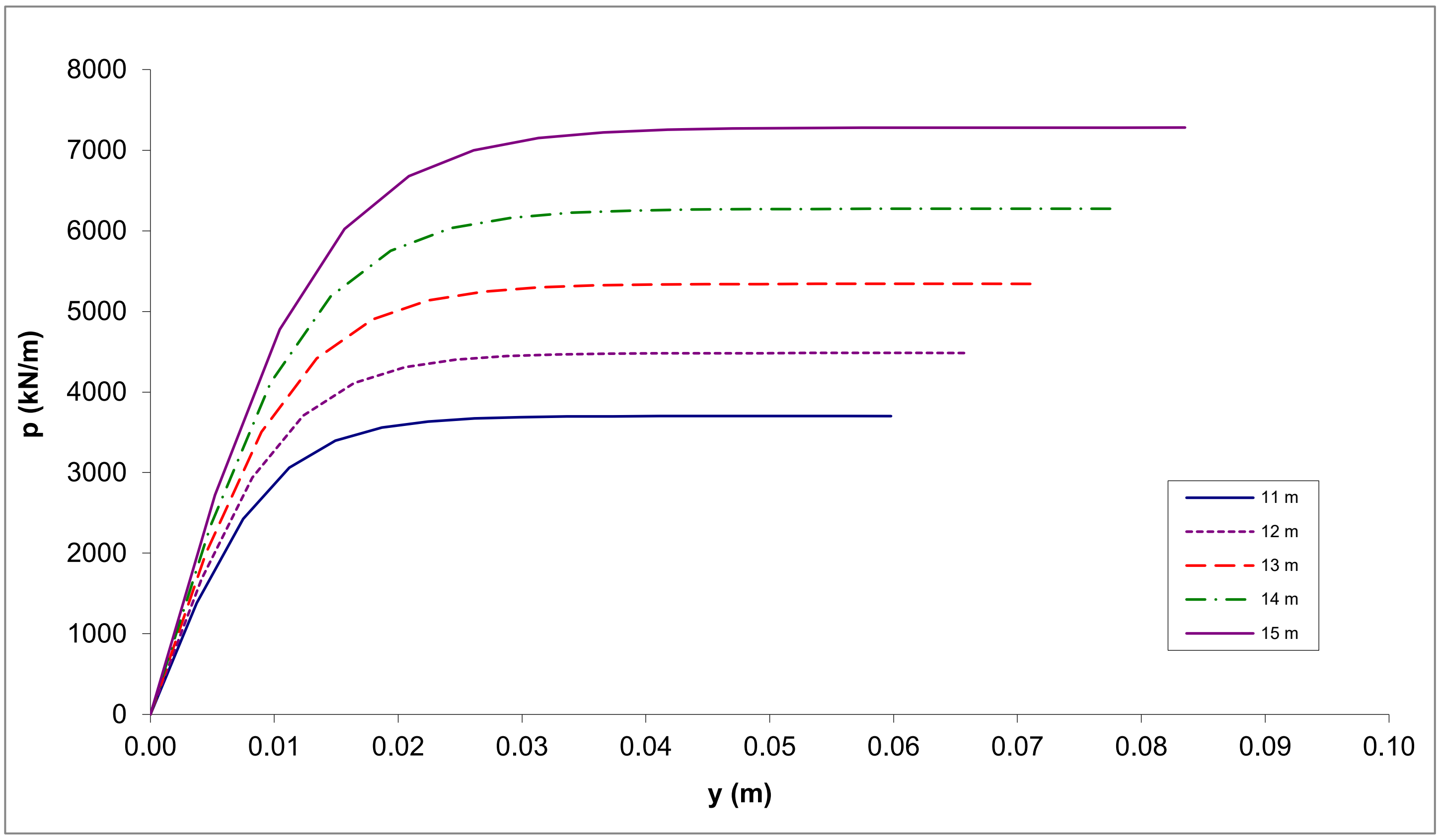


6 m		7 m		8 m		9 m		10 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00188	340.71	0.00225	493.19	0.00262	673.66	0.00300	882.12	0.00337	1118.57
0.00376	597.93	0.00450	865.52	0.00525	1182.23	0.00599	1548.06	0.00673	1963.02
0.00564	753.89	0.00676	1091.28	0.00787	1490.60	0.00898	1951.85	0.01010	2475.05
0.00752	836.23	0.00901	1210.46	0.01049	1653.39	0.01198	2165.02	0.01346	2745.35
0.00940	876.53	0.01126	1268.79	0.01312	1733.07	0.01497	2269.36	0.01683	2877.67
0.01128	895.53	0.01351	1296.30	0.01574	1770.64	0.01797	2318.55	0.02020	2940.04
0.01316	904.33	0.01576	1309.03	0.01836	1788.03	0.02096	2341.33	0.02356	2968.92
0.01504	908.37	0.01801	1314.88	0.02099	1796.02	0.02396	2351.79	0.02693	2982.19
0.01692	910.21	0.02026	1317.55	0.02361	1799.67	0.02695	2356.57	0.03029	2988.25
0.01880	911.06	0.02252	1318.77	0.02623	1801.34	0.02995	2358.76	0.03366	2991.02
0.02068	911.44	0.02477	1319.33	0.02886	1802.10	0.03294	2359.75	0.03702	2992.28
0.02255	911.62	0.02702	1319.58	0.03148	1802.45	0.03594	2360.21	0.04039	2992.86
0.02443	911.70	0.02927	1319.70	0.03410	1802.61	0.03893	2360.41	0.04376	2993.12
0.02631	911.73	0.03152	1319.75	0.03673	1802.68	0.04192	2360.51	0.04712	2993.24
0.02819	911.75	0.03377	1319.78	0.03935	1802.71	0.04492	2360.55	0.05049	2993.30
0.03007	911.76	0.03603	1319.79	0.04197	1802.73	0.04791	2360.57	0.05385	2993.32

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.

Note - The depths are provided as depths below top of pile.

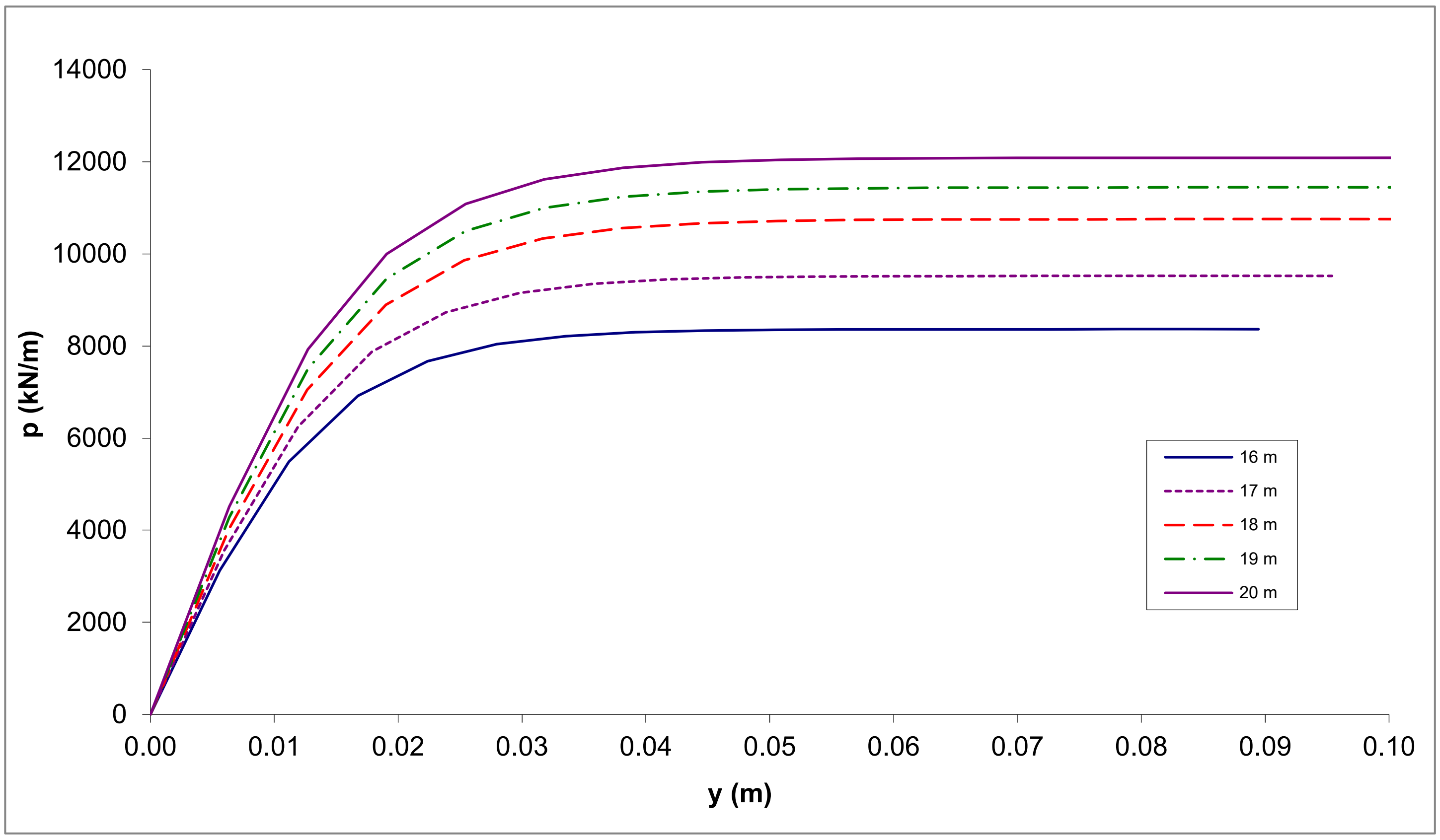
ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative 50% Scour Case



11 m		12 m		13 m		14 m		15 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00374	1383.01	0.00411	1675.45	0.00448	1995.87	0.00485	2344.29	0.00522	2720.71
0.00747	2427.10	0.00822	2940.31	0.00896	3502.64	0.00970	4114.09	0.01044	4774.67
0.01121	3060.18	0.01232	3707.25	0.01344	4416.26	0.01455	5187.21	0.01566	6020.09
0.01495	3394.39	0.01643	4112.13	0.01792	4898.57	0.01940	5753.71	0.02088	6677.56
0.01868	3557.99	0.02054	4310.32	0.02239	5134.66	0.02425	6031.02	0.02610	6999.39
0.02242	3635.11	0.02465	4403.74	0.02687	5245.96	0.02910	6161.74	0.03132	7151.10
0.02616	3670.82	0.02876	4447.00	0.03135	5297.49	0.03395	6222.27	0.03654	7221.35
0.02990	3687.21	0.03286	4466.87	0.03583	5321.15	0.03880	6250.07	0.04176	7253.61
0.03363	3694.71	0.03697	4475.95	0.04031	5331.98	0.04365	6262.78	0.04698	7268.36
0.03737	3698.14	0.04108	4480.10	0.04479	5336.92	0.04850	6268.58	0.05220	7275.10
0.04111	3699.70	0.04519	4482.00	0.04927	5339.17	0.05335	6271.23	0.05743	7278.17
0.04484	3700.41	0.04929	4482.86	0.05375	5340.20	0.05820	6272.44	0.06265	7279.58
0.04858	3700.74	0.05340	4483.25	0.05822	5340.67	0.06305	6272.99	0.06787	7280.21
0.05232	3700.88	0.05751	4483.43	0.06270	5340.88	0.06789	6273.24	0.07309	7280.51
0.05605	3700.95	0.06162	4483.51	0.06718	5340.98	0.07274	6273.36	0.07831	7280.64
0.05979	3700.98	0.06573	4483.55	0.07166	5341.03	0.07759	6273.41	0.08353	7280.70

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.
Note - The depths are provided as depths below top of pile.

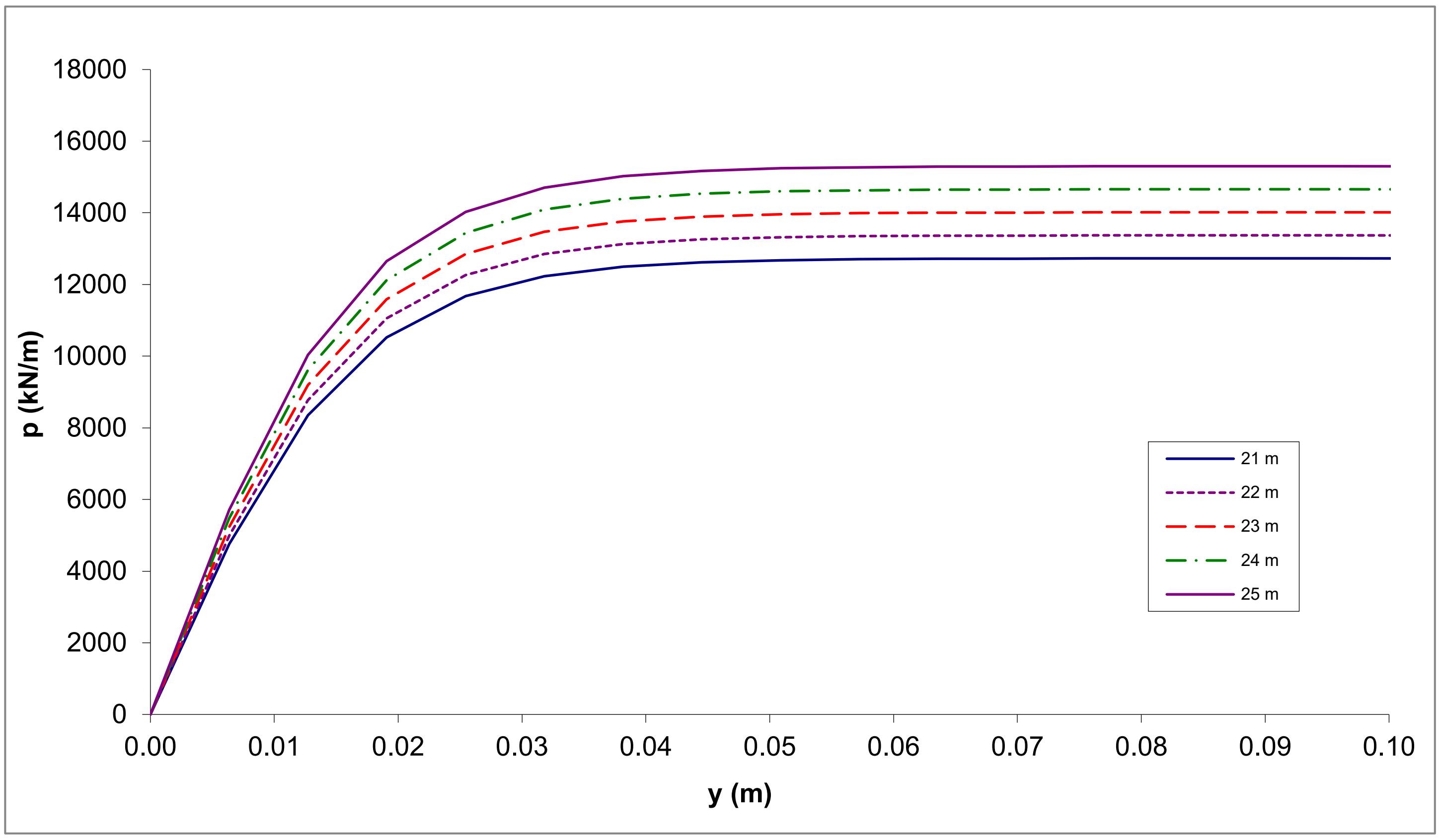
ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative 50% Scour Case



16 m		17 m		18 m		19 m		20 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00559	3125.11	0.00596	3557.51	0.00633	4017.89	0.00636	4275.80	0.00636	4516.01
0.01118	5484.38	0.01192	6243.21	0.01267	7051.16	0.01272	7503.76	0.01272	7925.32
0.01677	6914.92	0.01789	7871.68	0.01900	8890.37	0.01908	9461.04	0.01908	9992.55
0.02237	7670.10	0.02385	8731.35	0.02533	9861.31	0.02544	10494.29	0.02544	11083.86
0.02796	8039.77	0.02981	9152.17	0.03166	10336.58	0.03180	11000.08	0.03180	11618.06
0.03355	8214.04	0.03577	9350.55	0.03800	10560.63	0.03816	11238.51	0.03816	11869.88
0.03914	8294.73	0.04173	9442.40	0.04433	10664.37	0.04452	11348.91	0.04452	11986.48
0.04473	8331.78	0.04770	9484.58	0.05066	10712.01	0.05089	11399.60	0.05089	12040.02
0.05032	8348.73	0.05366	9503.87	0.05700	10733.80	0.05725	11422.79	0.05725	12064.51
0.05591	8356.47	0.05962	9512.68	0.06333	10743.75	0.06361	11433.37	0.06361	12075.70
0.06150	8360.00	0.06558	9516.70	0.06966	10748.29	0.06997	11438.20	0.06997	12080.80
0.06710	8361.61	0.07154	9518.53	0.07599	10750.36	0.07633	11440.41	0.07633	12083.13
0.07269	8362.34	0.07751	9519.37	0.08233	10751.30	0.08269	11441.41	0.08269	12084.19
0.07828	8362.68	0.08347	9519.75	0.08866	10751.73	0.08905	11441.87	0.08905	12084.67
0.08387	8362.83	0.08943	9519.92	0.09499	10751.93	0.09541	11442.08	0.09541	12084.89
0.08946	8362.90	0.09539	9520.00	0.10132	10752.02	0.10177	11442.17	0.10177	12084.99

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.
Note - The depths are provided as depths below top of pile.

ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative 50% Scour Case

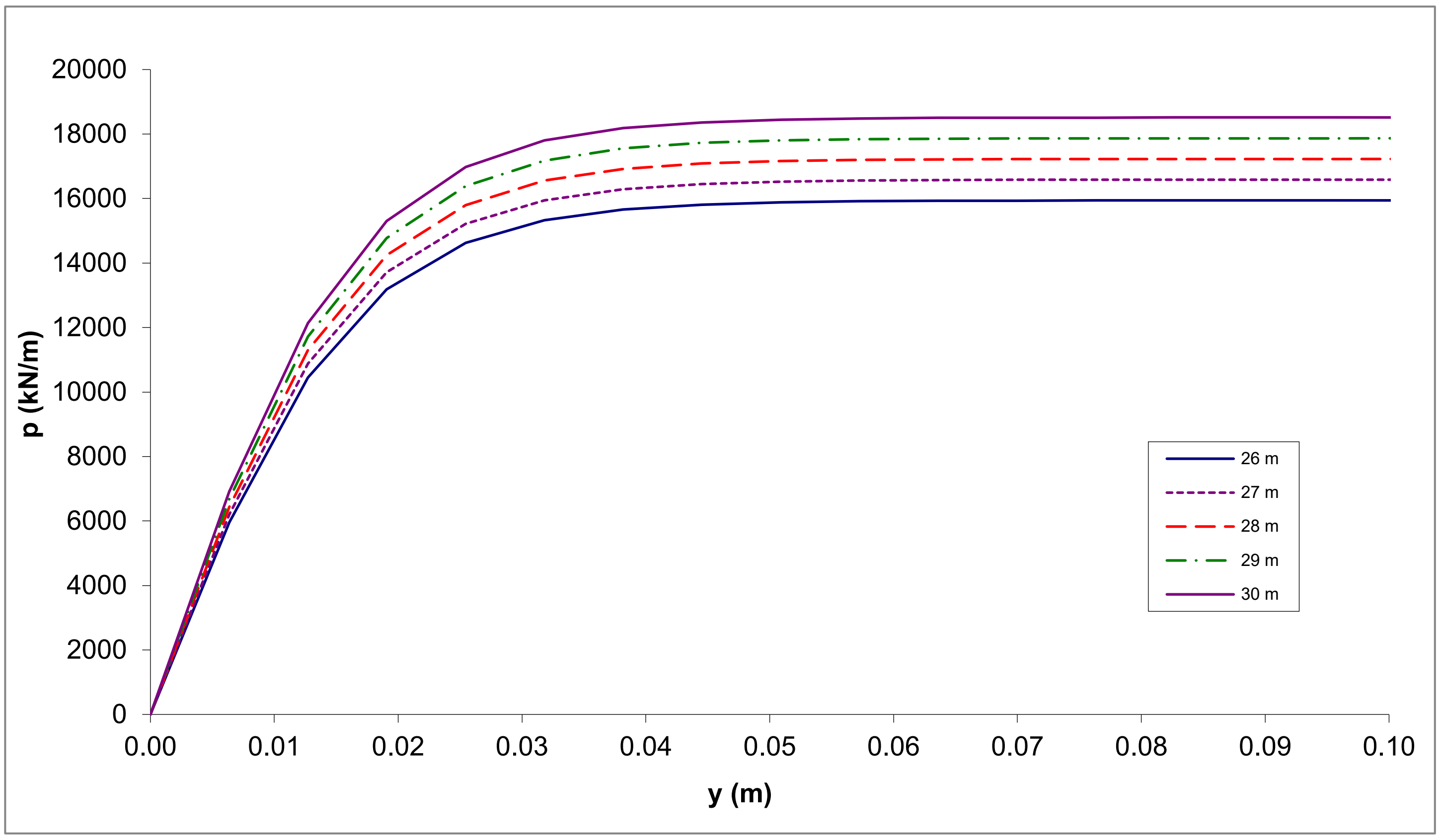


21 m		22 m		23 m		24 m		25 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00636	4756.22	0.00636	4996.44	0.00636	5236.65	0.00636	5476.86	0.00636	5717.08
0.01272	8346.88	0.01272	8768.44	0.01272	9190.00	0.01272	9611.56	0.01272	10033.12
0.01908	10524.07	0.01908	11055.59	0.01908	11587.11	0.01908	12118.63	0.01908	12650.15
0.02544	11673.42	0.02544	12262.99	0.02544	12852.56	0.02544	13442.12	0.02544	14031.69
0.03180	12236.04	0.03180	12854.02	0.03180	13472.00	0.03180	14089.98	0.03180	14707.97
0.03816	12501.26	0.03816	13132.64	0.03816	13764.01	0.03816	14395.39	0.03816	15026.77
0.04452	12624.06	0.04452	13261.64	0.04452	13899.22	0.04452	14536.80	0.04452	15174.38
0.05089	12680.45	0.05089	13320.88	0.05089	13961.31	0.05089	14601.73	0.05089	15242.16
0.05725	12706.24	0.05725	13347.97	0.05725	13989.70	0.05725	14631.43	0.05725	15273.16
0.06361	12718.02	0.06361	13360.35	0.06361	14002.67	0.06361	14644.99	0.06361	15287.32
0.06997	12723.40	0.06997	13365.99	0.06997	14008.59	0.06997	14651.18	0.06997	15293.78
0.07633	12725.85	0.07633	13368.57	0.07633	14011.28	0.07633	14654.00	0.07633	15296.72
0.08269	12726.96	0.08269	13369.74	0.08269	14012.52	0.08269	14655.29	0.08269	15298.07
0.08905	12727.47	0.08905	13370.27	0.08905	14013.08	0.08905	14655.88	0.08905	15298.68
0.09541	12727.71	0.09541	13370.52	0.09541	14013.33	0.09541	14656.15	0.09541	15298.96
0.10177	12727.81	0.10177	13370.63	0.10177	14013.45	0.10177	14656.27	0.10177	15299.09

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.

Note - The depths are provided as depths below top of pile.

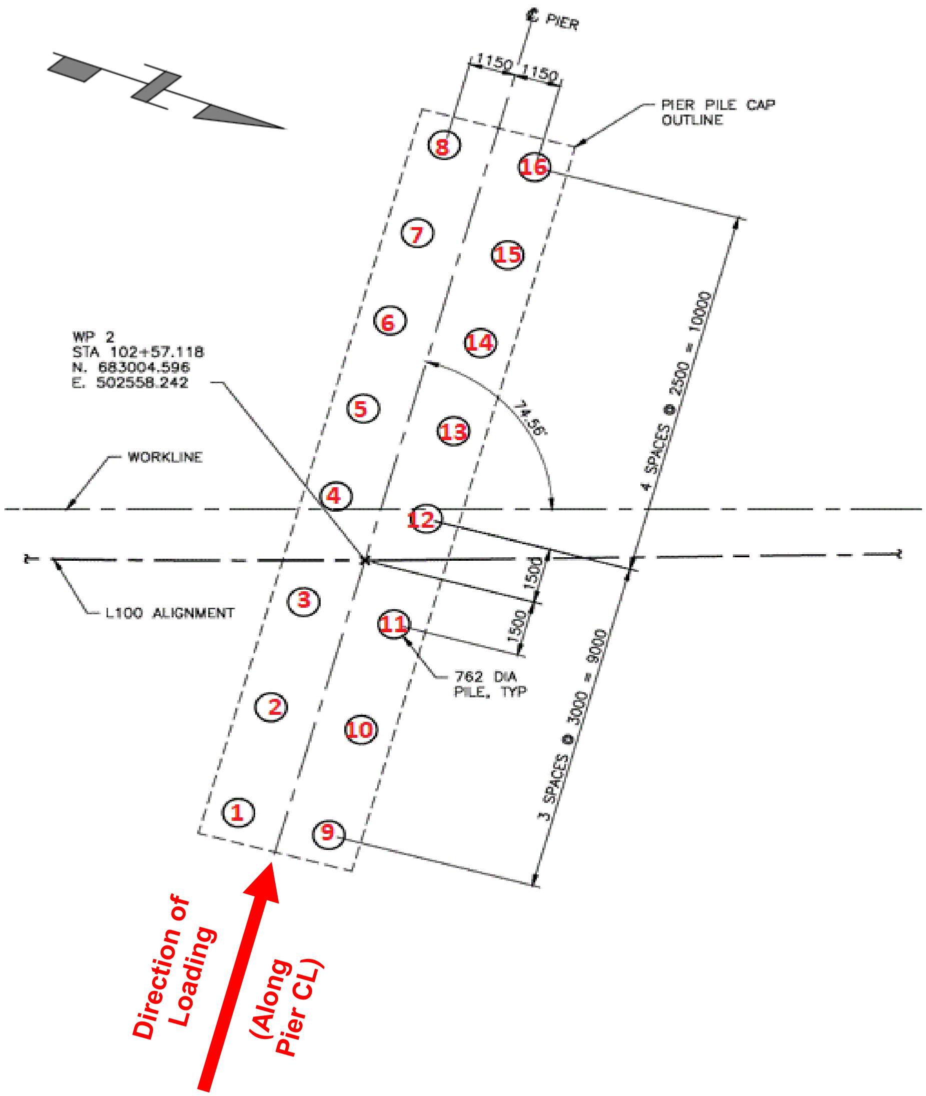
ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative 50% Scour Case



26 m		27 m		28 m		29 m		30 m	
y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)	y (m)	p (kN/m)
0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
0.00636	5957.29	0.00636	6197.50	0.00636	6437.72	0.00636	6677.93	0.00636	6918.14
0.01272	10454.68	0.01272	10876.24	0.01272	11297.80	0.01272	11719.36	0.01272	12140.92
0.01908	13181.67	0.01908	13713.19	0.01908	14244.70	0.01908	14776.22	0.01908	15307.74
0.02544	14621.26	0.02544	15210.82	0.02544	15800.39	0.02544	16389.96	0.02544	16979.52
0.03180	15325.95	0.03180	15943.93	0.03180	16561.91	0.03180	17179.89	0.03180	17797.87
0.03816	15658.14	0.03816	16289.52	0.03816	16920.90	0.03816	17552.27	0.03816	18183.65
0.04452	15811.96	0.04452	16449.54	0.04452	17087.12	0.04452	17724.69	0.04452	18362.27
0.05089	15882.59	0.05089	16523.01	0.05089	17163.44	0.05089	17803.87	0.05089	18444.29
0.05725	15914.89	0.05725	16556.62	0.05725	17198.35	0.05725	17840.08	0.05725	18481.81
0.06361	15929.64	0.06361	16571.97	0.06361	17214.29	0.06361	17856.62	0.06361	18498.94
0.06997	15936.37	0.06997	16578.97	0.06997	17221.57	0.06997	17864.16	0.06997	18506.76
0.07633	15939.44	0.07633	16582.16	0.07633	17224.88	0.07633	17867.60	0.07633	18510.32
0.08269	15940.84	0.08269	16583.62	0.08269	17226.39	0.08269	17869.17	0.08269	18511.95
0.08905	15941.48	0.08905	16584.28	0.08905	17227.08	0.08905	17869.89	0.08905	18512.69
0.09541	15941.77	0.09541	16584.59	0.09541	17227.40	0.09541	17870.21	0.09541	18513.03
0.10177	15941.90	0.10177	16584.72	0.10177	17227.54	0.10177	17870.36	0.10177	18513.18

Note - The combined effect of pile group factor and geotechnical resistance factors should be applied as p-multipliers on the ultimate p-y curves.
Note - The depths are provided as depths below top of pile.

ULTIMATE p-y CURVES
Pile Diameter = 762 mm
Representative 50% Scour Case

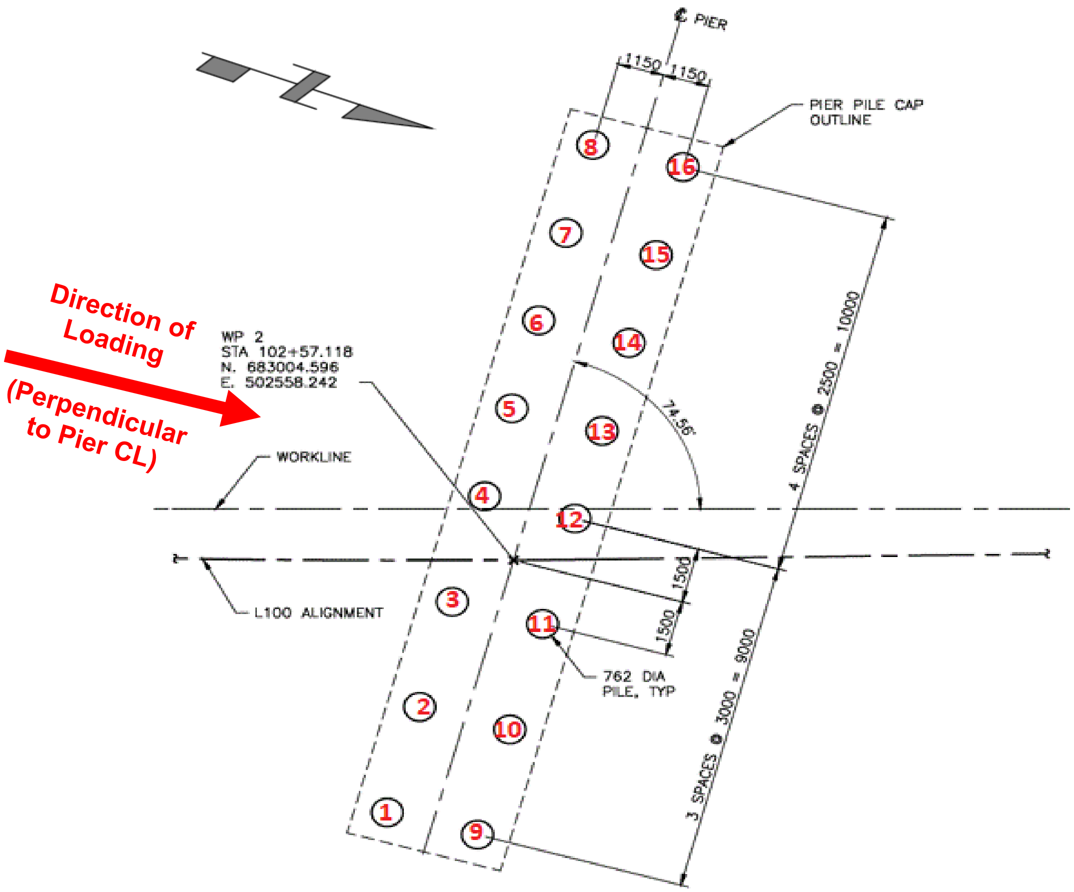


Pile No.	Reduction Factor
1	0.70
2	0.70
3	0.70
4	0.63
5	0.60
6	0.60
7	0.62
8	0.89
9	0.70
10	0.70
11	0.70
12	0.63
13	0.60
14	0.60
15	0.62
16	0.89

Note: reduction factor to be applied as p multiplier

TETRA TECH

PILE GROUP REDUCTION FACTORS
KHR1 Pier (762 mm Diameter)
Loading in Y Direction

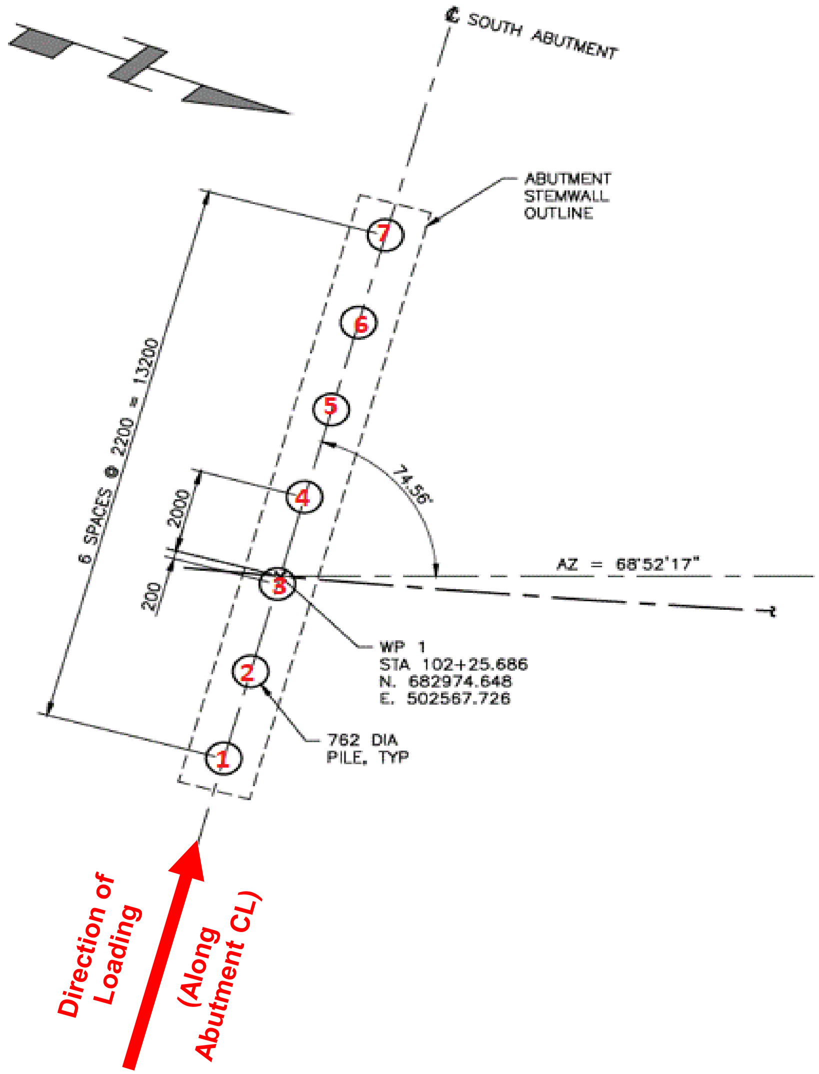


Pile No.	Reduction Factor
1	0.70
2	0.67
3	0.67
4	0.63
5	0.58
6	0.58
7	0.58
8	0.65
9	0.93
10	0.93
11	0.93
12	0.89
13	0.86
14	0.86
15	0.86
16	0.89

Note: reduction factor to be applied as p multiplier

PILE GROUP REDUCTION FACTORS
KHR1 Pier (762 mm Diameter)
Loading in Z Direction

TETRA TECH

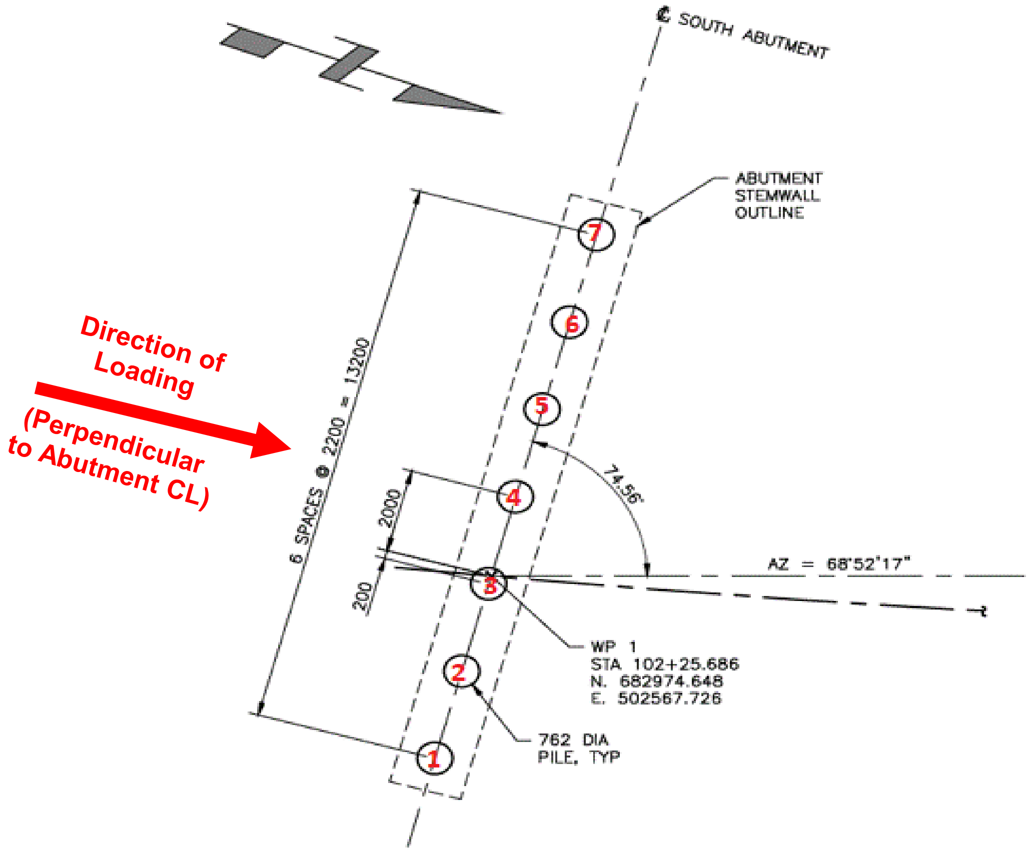


Pile No.	Reduction Factor
1	0.67
2	0.62
3	0.62
4	0.62
5	0.62
6	0.66
7	0.92

Note: reduction factor to be applied as p multiplier

PILE GROUP REDUCTION FACTORS
KHR1 South Abutment (762mm Diameter)
Loading in Y Direction

TETRA TECH

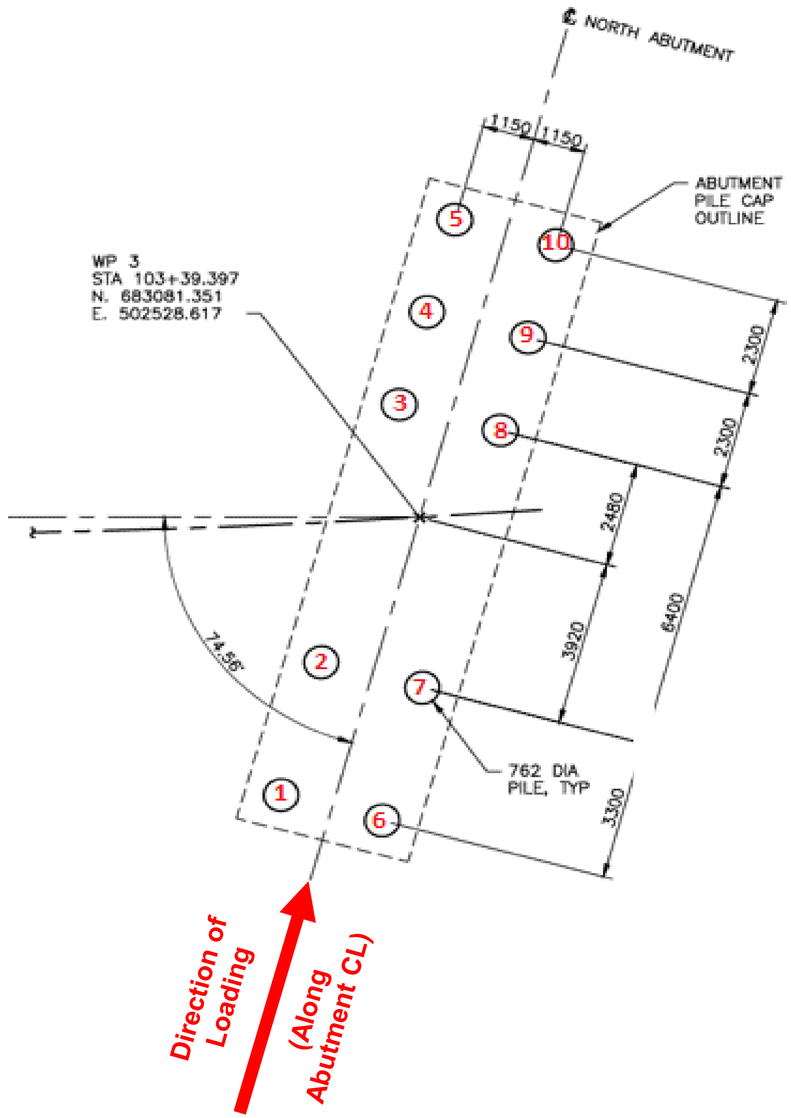


Pile No.	Reduction Factor
1	0.92
2	0.84
3	0.84
4	0.84
5	0.84
6	0.84
7	0.92

Note: reduction factor to be applied as p multiplier

PILE GROUP REDUCTION FACTORS
KHR1 South Abutment (762mm Diameter)
Loading in Z Direction

TETRA TECH



Pile No.	Reduction Factor
1	0.73
2	0.93
3	0.59
4	0.58
5	0.87
6	0.73
7	0.93
8	0.59
9	0.58
10	0.87

Note: reduction factor to be applied as p multiplier

PILE GROUP REDUCTION FACTORS
KHR1 North Abutment (762mm Diameter)
Loading in Y Direction

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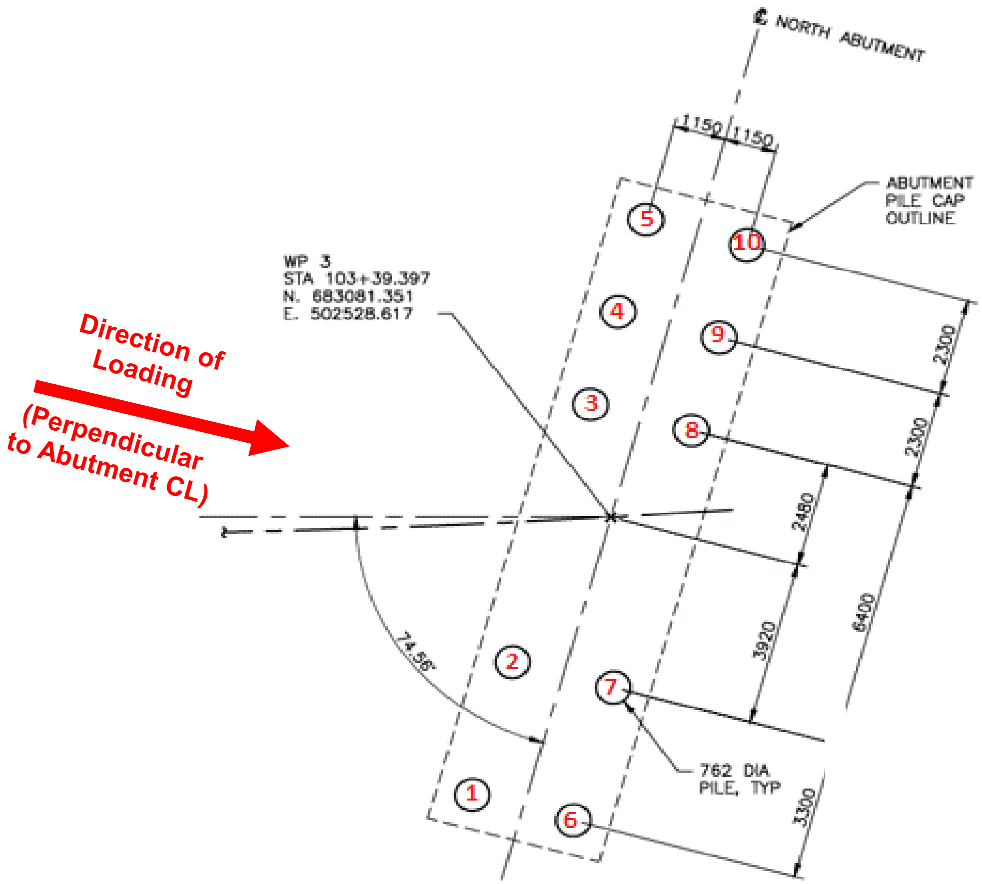


Pile No.	Reduction Factor
1	0.71
2	0.71
3	0.63
4	0.54
5	0.63
6	0.93
7	0.93
8	0.87
9	0.81
10	0.87

Note: reduction factor to be applied as p multiplier

PILE GROUP REDUCTION FACTORS
KHR1 North Abutment (762mm Diameter)
Loading in Z Direction

TETRA TECH

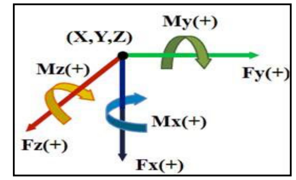
APPENDIX E

KHR2 PILE GROUP ANALYSES

KHR 2 - Foundation Loads: N./S. Abutments

Axis	F X	F Z	M Y	F Y	M Z
	Axial	Long. Shear	Long Moment	Trans Shear	Trans Moment
Load Case	kN	kN	kNm	kN	kNm
1 - ULS1	3409	995	1974	0	0
2 - ULS2	3356	1116	2316	0	0
3 - ULS4	2508	1294	2441	0	0
4 - ULS5a (+/+)	2665	0	0	682	1171
5 - ULS5a (+/-)	2665	0	0	682	-1171
6 - ULS5b	2665	845	1339	0	0
7 - ULS6 (+/+)	2508	441	611	113	298
8 - ULS6 (+/-)	2508	441	611	113	-298

Axis Orientation

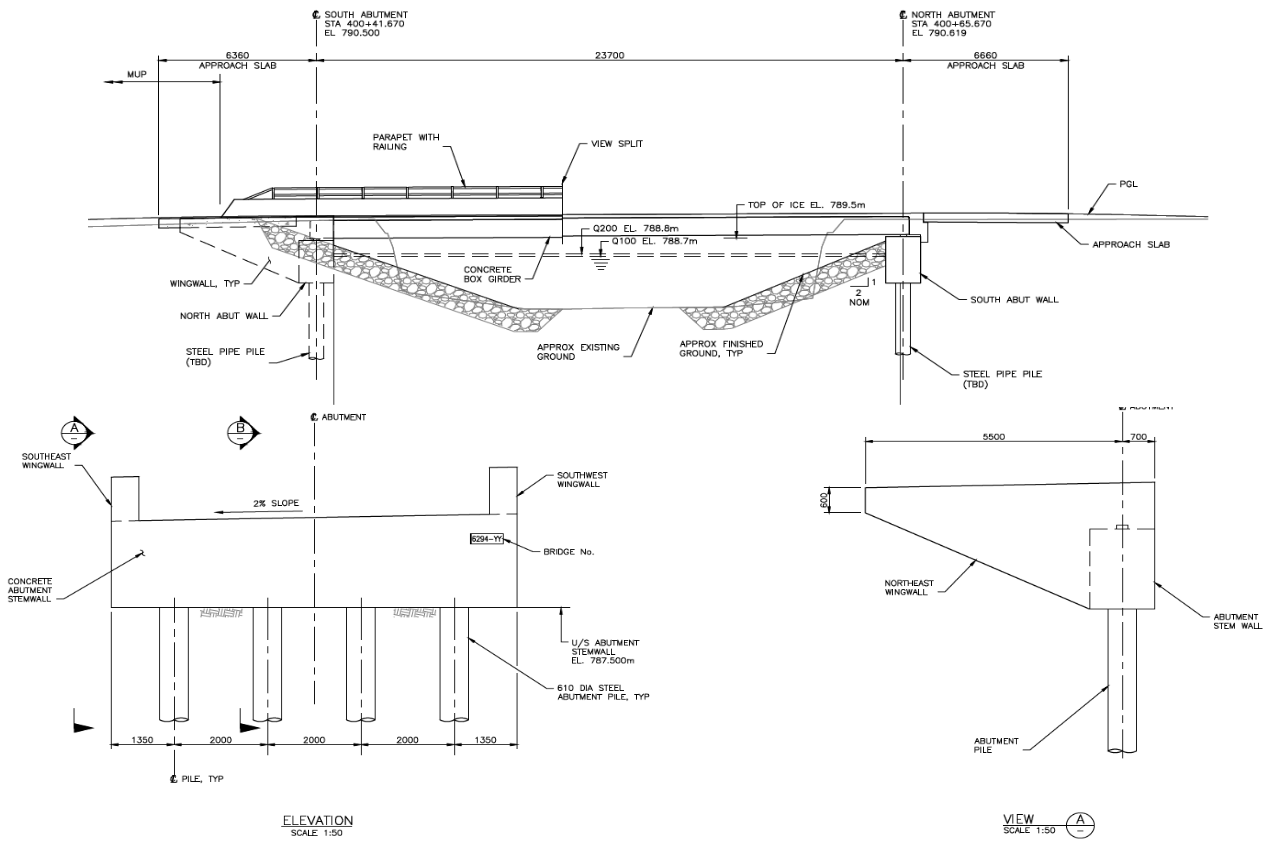


KHR 2 Modeled Elevations and Element thicknesses

Ground Surface El. (m)	Ground Surface El. (m)	Top of Pile Cap El. (m)	Top of Pile El. (m)	Top of Pile Cap Embedment - below Ground surface (m)	Pile Cap Thickness (m)
KHR 2 – Abutments	790.5	789.5	787.5	1.0	2
KHR 1 - North Abutment	788.4	788.5	787.1	-0.1	1.5
KHR 1 - South Abutment	789.7	789.6	788.2	0.1	1.5

Notes:
- Updated foundation loads for KHR2 provided by COWI on July 6, 2023.
- Preliminary pile section properties provided by COWI on May 9, 2022 with further clarification provided by COWI on May 11, 2023.
- Modeled elevations (ground surface, pile cap, and top of pile elevations) and element thicknesses based on COWI 50% Detailed Design drawings dated June 17, 2022 and updated values provided by COWI on November 8, 2022

KHR2 NORTH/SOUTH ABUTMENT



Note: - Foundation configurations and pile spacing provided in COWI 50% Detailed Design Drawings dated June 17, 2022.

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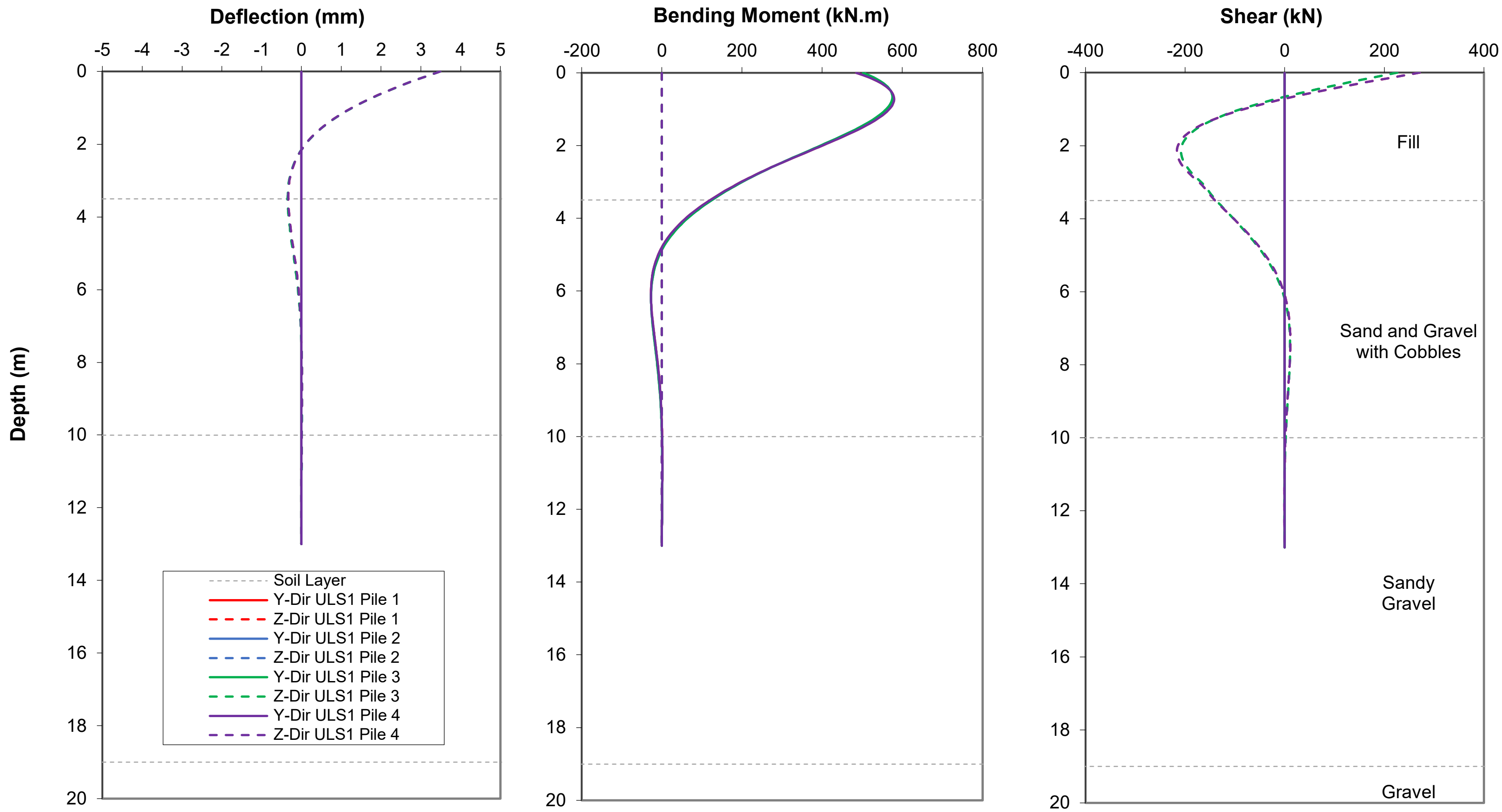
GROUP PILE ANALYSIS
KHR 2 Abutments Sections

Model Input Parameters				F X		F Z	M Y	F Y	M Z	M X	Resultant	
Pile Structure Location, Diameter, Wall Thickness, Pile Length, Pile Layout, Pile Head Connections	Pile Notes	Load Case	Load Case Name	Maximum Axial Reaction - Compression (kN)	Minimum Axial Reaction - Compression (kN)	Maximum Longitudinal Shear (kN)	Maximum Longitudinal Moment (kN.m)	Maximum Transverse Shear (kN)	Maximum Transverse Moment (kN.m)	Maximum Axial Moment (kN.m)	Maximum Lateral Deflection (mm)	Additional Comments
KHR2 Abutment, 762 mm, 15.9 mm, 13 m, 1 x 4 Fixed Connections	**Driven, Steel, No concrete infill**	1	ULS1	852	852	271	579	< 1	< 1	< 1	< 5	-
		2	ULS2	839	839	305	676	< 1	< 1	< 1	< 5	-
		3	ULS4	627	627	352	733	< 1	< 1	< 1	< 5	-
		4	ULS5a (+/+)	729	603	< 1	< 1	208	214	< 1	< 5	-
		5	ULS5a (+/-)	908	425	< 1	< 1	215	136	< 1	< 5	-
		6	ULS5b	666	666	229	412	< 1	< 1	< 1	< 5	-
		7	ULS6 (+/+)	651	603	119	194	29	34	< 1	< 5	-
		8	ULS6 (+/-)	679	575	119	194	30	14	< 1	< 5	-

Notes:
- All shear and moment values are absolute values.

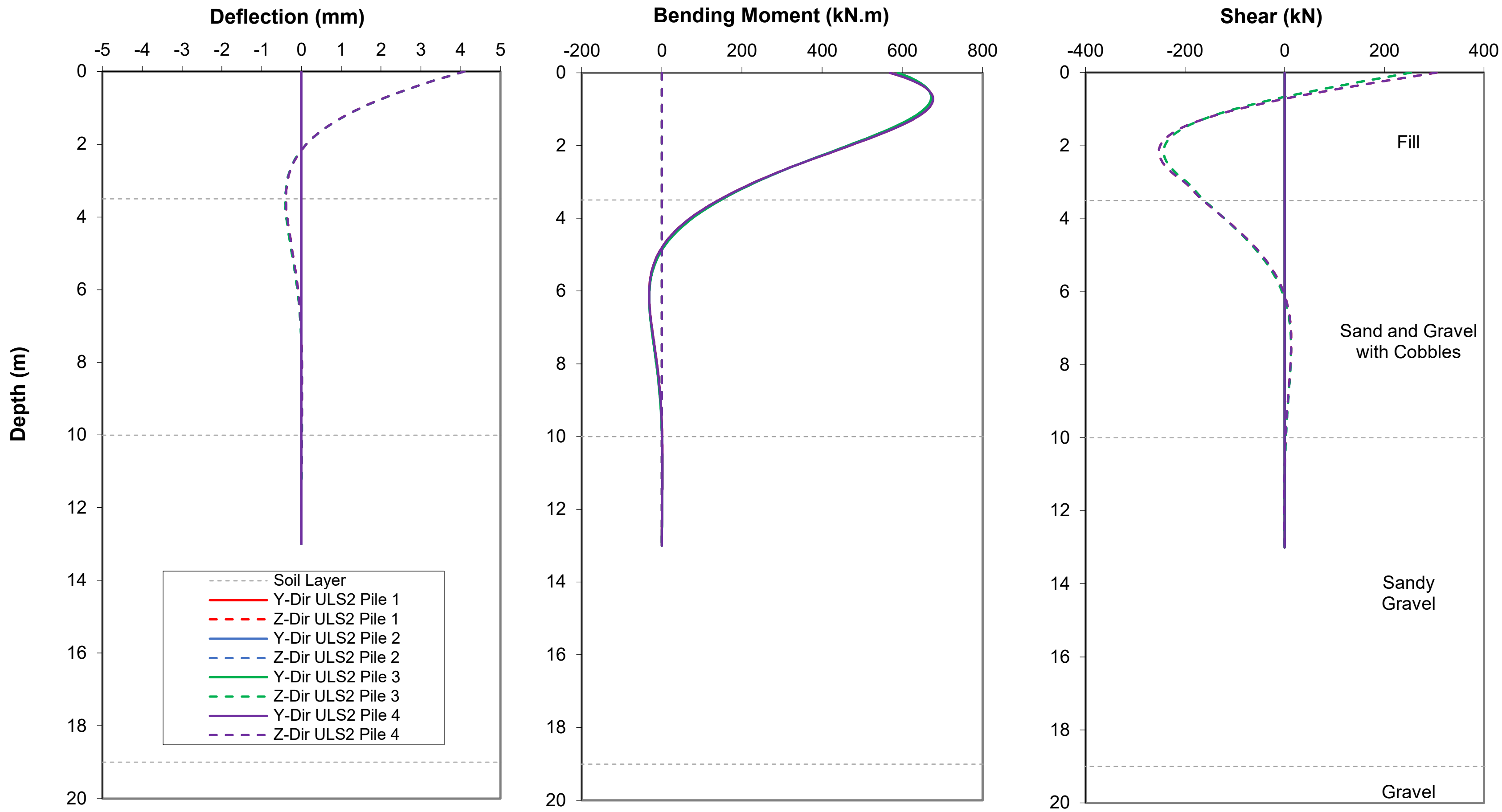


Notes:
- No Geotechnical Resistance Factor (GRF) considered in displayed plots.
- Considered wall thickness = 15.9 mm.
- Pile embedment = 13 m (considers GRF).
- No concrete infill considered.
- Top of piles considered to be fixed connections.

Maximum Deflection (mm)	3.5
Maximum Bending Moment (kN.m)	578.7
Maximum Shear (kN)	271.3

GROUP PILE ANALYSIS
KHR 2 Abutments 762 mm Steel Pipe Pile
Load Case 1: ULS1

TETRA TECH

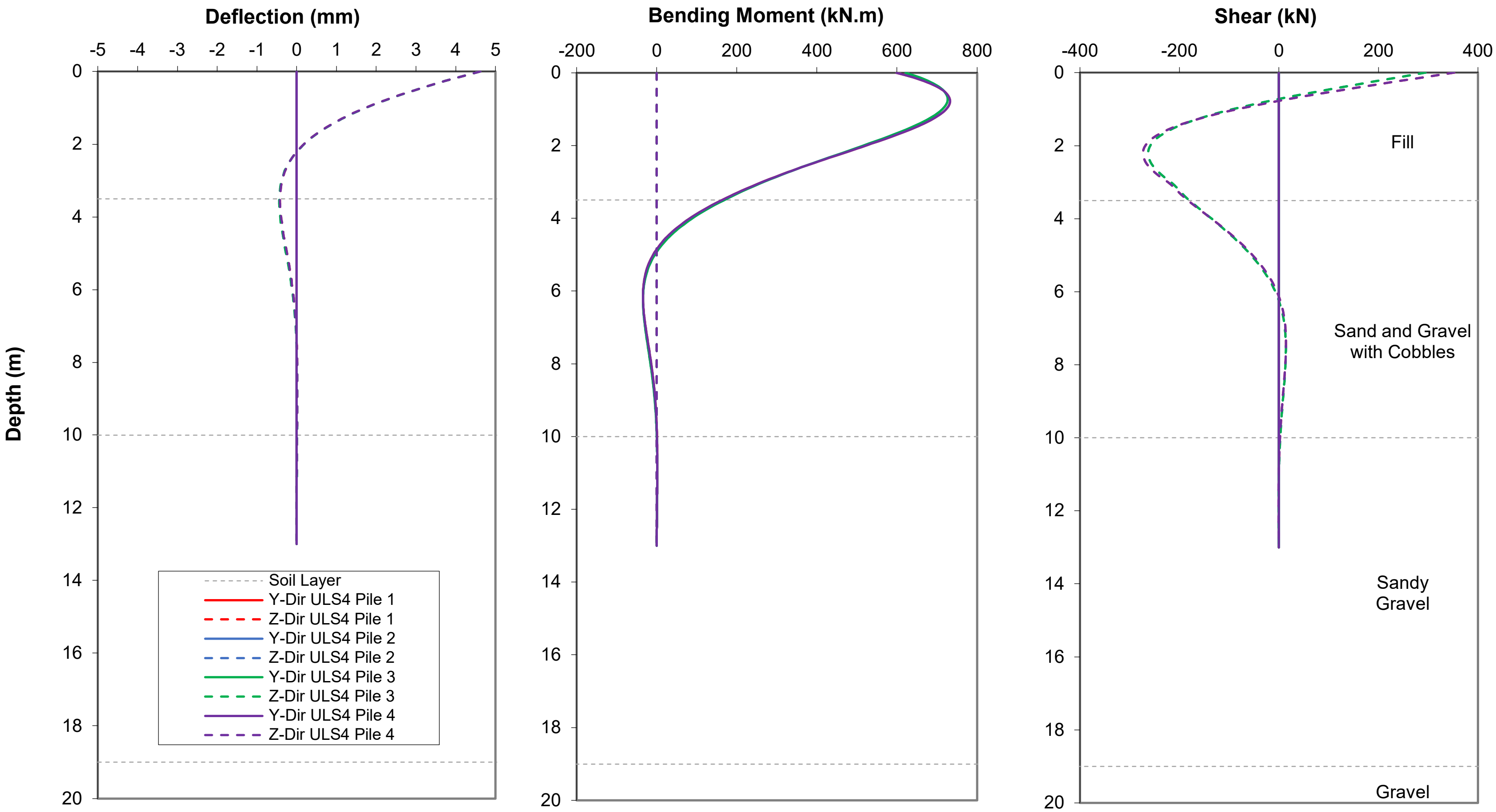


Notes: - No Geotechnical Resistance Factor (GRF) considered in displayed plots.
- Considered wall thickness = 15.9 mm.
- Pile embedment = 13 m (considers GRF).
- No concrete infill considered.
- Top of piles considered to be fixed connections.

Maximum Deflection (mm)	4.1
Maximum Bending Moment (kN.m)	676.0
Maximum Shear (kN)	304.7

GROUP PILE ANALYSIS
KHR 2 Abutments 762 mm Steel Pipe Pile
Load Case 2: ULS2

TETRA TECH

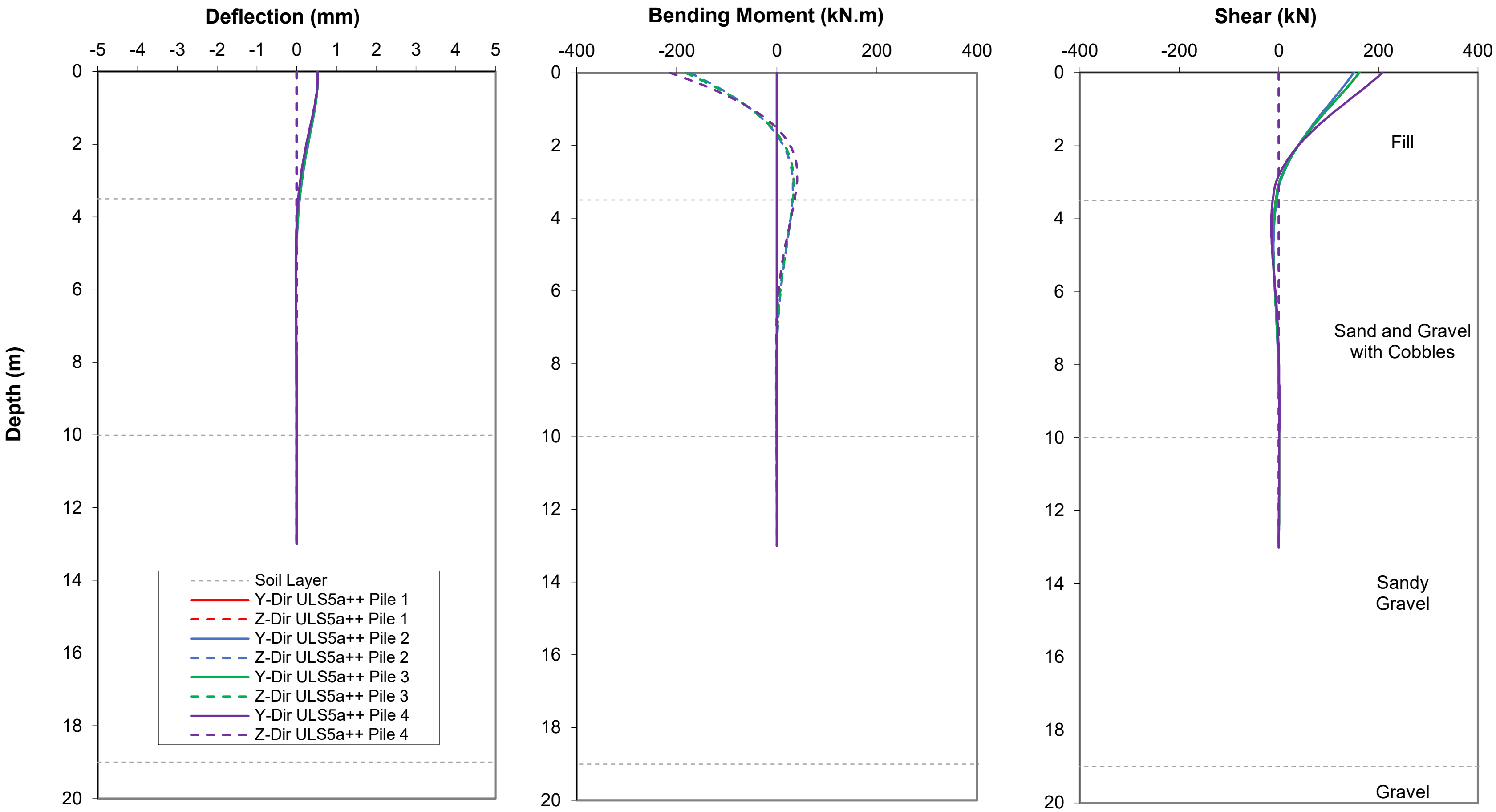


Notes: - No Geotechnical Resistance Factor (GRF) considered in displayed plots.
- Considered wall thickness = 15.9 mm.
- Pile embedment = 13 m (considers GRF).
- No concrete infill considered.
- Top of piles considered to be fixed connections.

Maximum Deflection (mm)	4.6
Maximum Bending Moment (kN.m)	732.5
Maximum Shear (kN)	352.2

GROUP PILE ANALYSIS
KHR 2 Abutments 762 mm Steel Pipe Pile
Load Case 3: ULS4

TETRA TECH

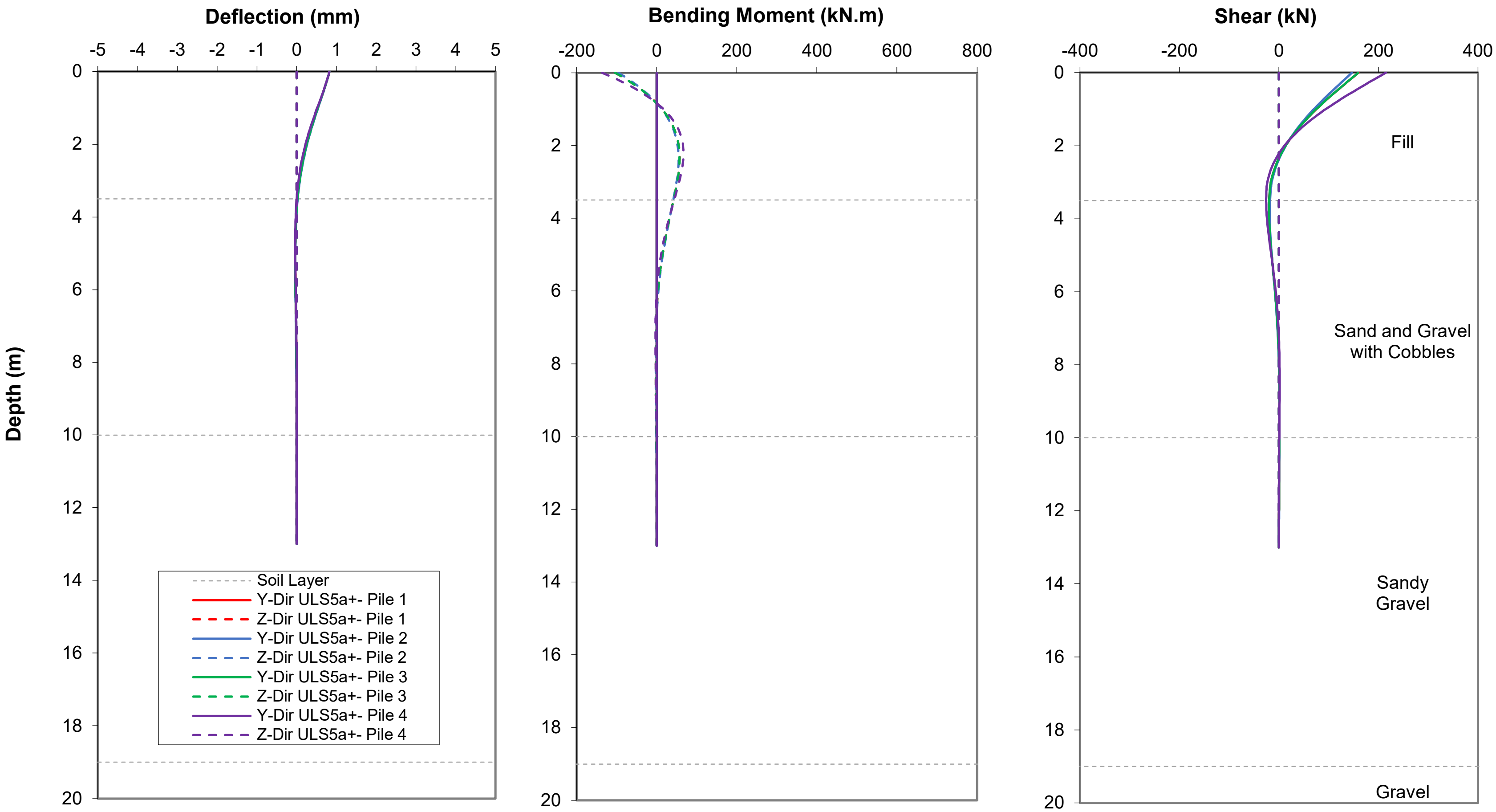


Notes: - No Geotechnical Resistance Factor (GRF) considered in displayed plots.
- Considered wall thickness = 15.9 mm.
- Pile embedment = 13 m (considers GRF).
- No concrete infill considered.
- Top of piles considered to be fixed connections.

Maximum Deflection (mm)	0.5
Maximum Bending Moment (kN.m)	213.6
Maximum Shear (kN)	208.3

GROUP PILE ANALYSIS
KHR 2 Abutments 762 mm Steel Pipe Pile
Load Case 4: ULS5a (+/+)

TETRA TECH

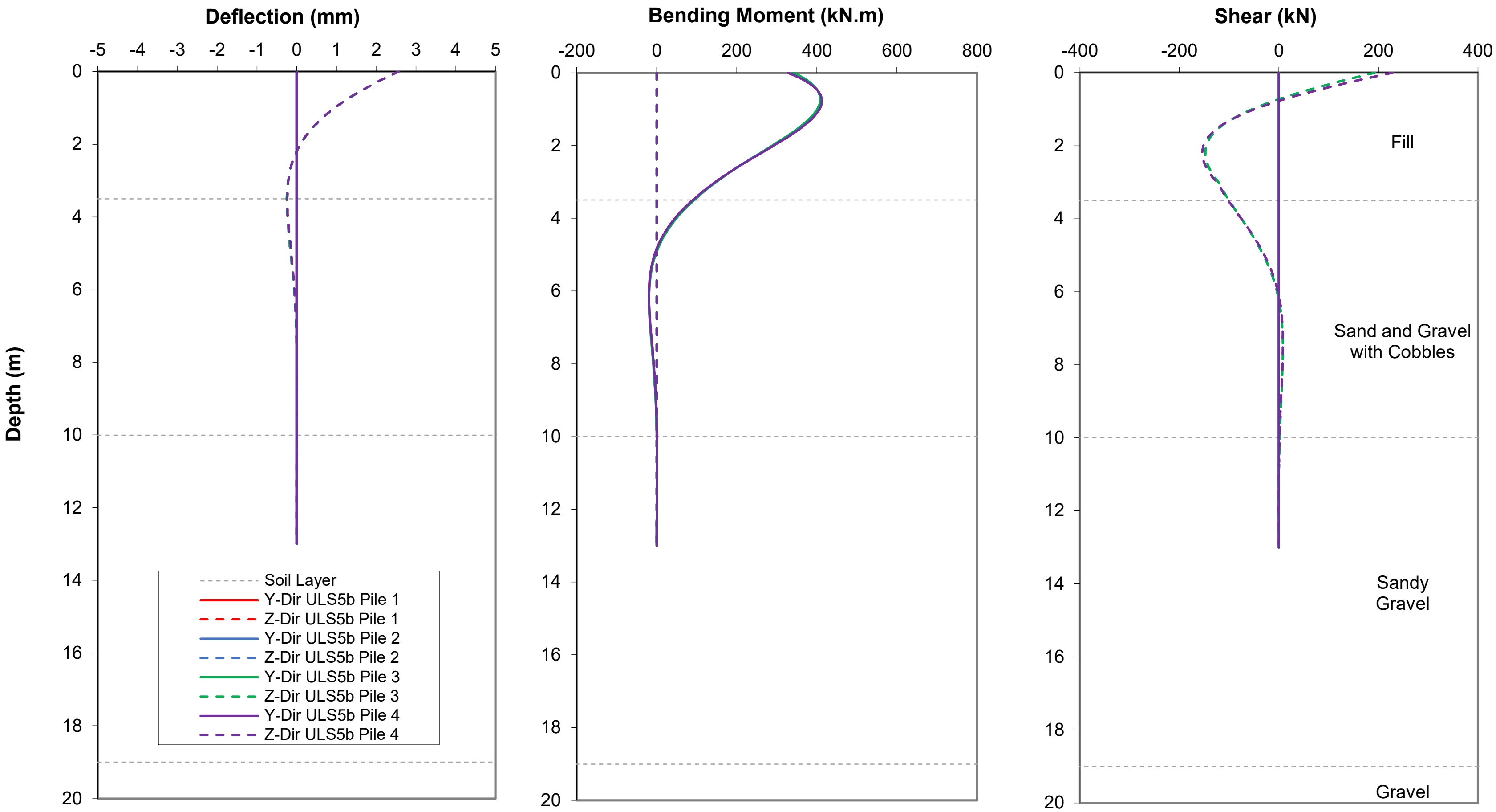


Notes: - No Geotechnical Resistance Factor (GRF) considered in displayed plots.
- Considered wall thickness = 15.9 mm.
- Pile embedment = 13 m (considers GRF).
- No concrete infill considered.
- Top of piles considered to be fixed connections.

Maximum Deflection (mm)	0.8
Maximum Bending Moment (kN.m)	136.3
Maximum Shear (kN)	215.2

GROUP PILE ANALYSIS
KHR 2 Abutments 762 mm Steel Pipe Pile
Load Case 5: ULS5a (+/-)

TETRA TECH

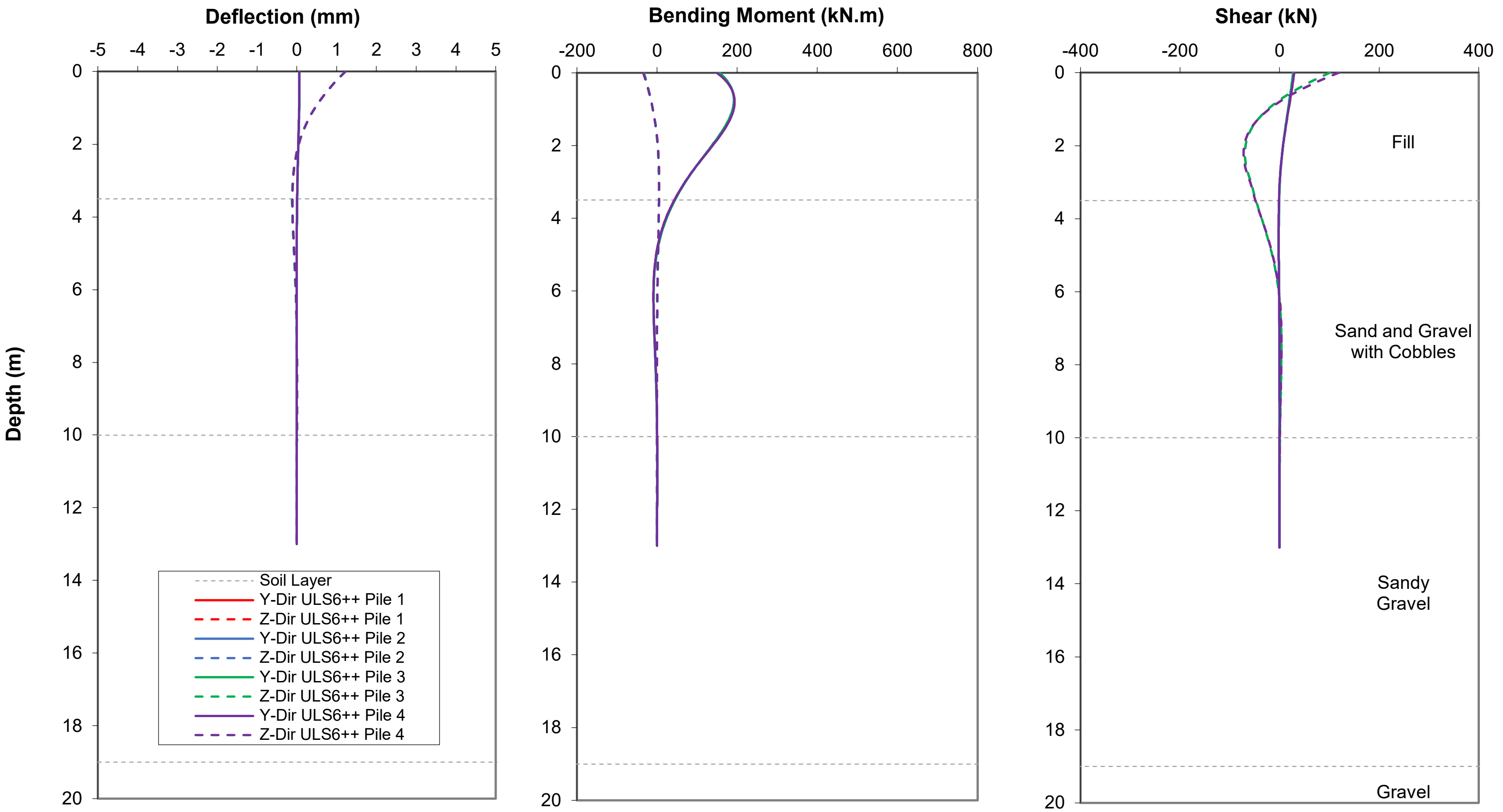


Notes: - No Geotechnical Resistance Factor (GRF) considered in displayed plots.
- Considered wall thickness = 15.9 mm.
- Pile embedment = 13 m (considers GRF).
- No concrete infill considered.
- Top of piles considered to be fixed connections.

Maximum Deflection (mm)	2.6
Maximum Bending Moment (kN.m)	412.0
Maximum Shear (kN)	229.1

GROUP PILE ANALYSIS
KHR 2 Abutments 762 mm Steel Pipe Pile
Load Case 6: ULS5b

TETRA TECH

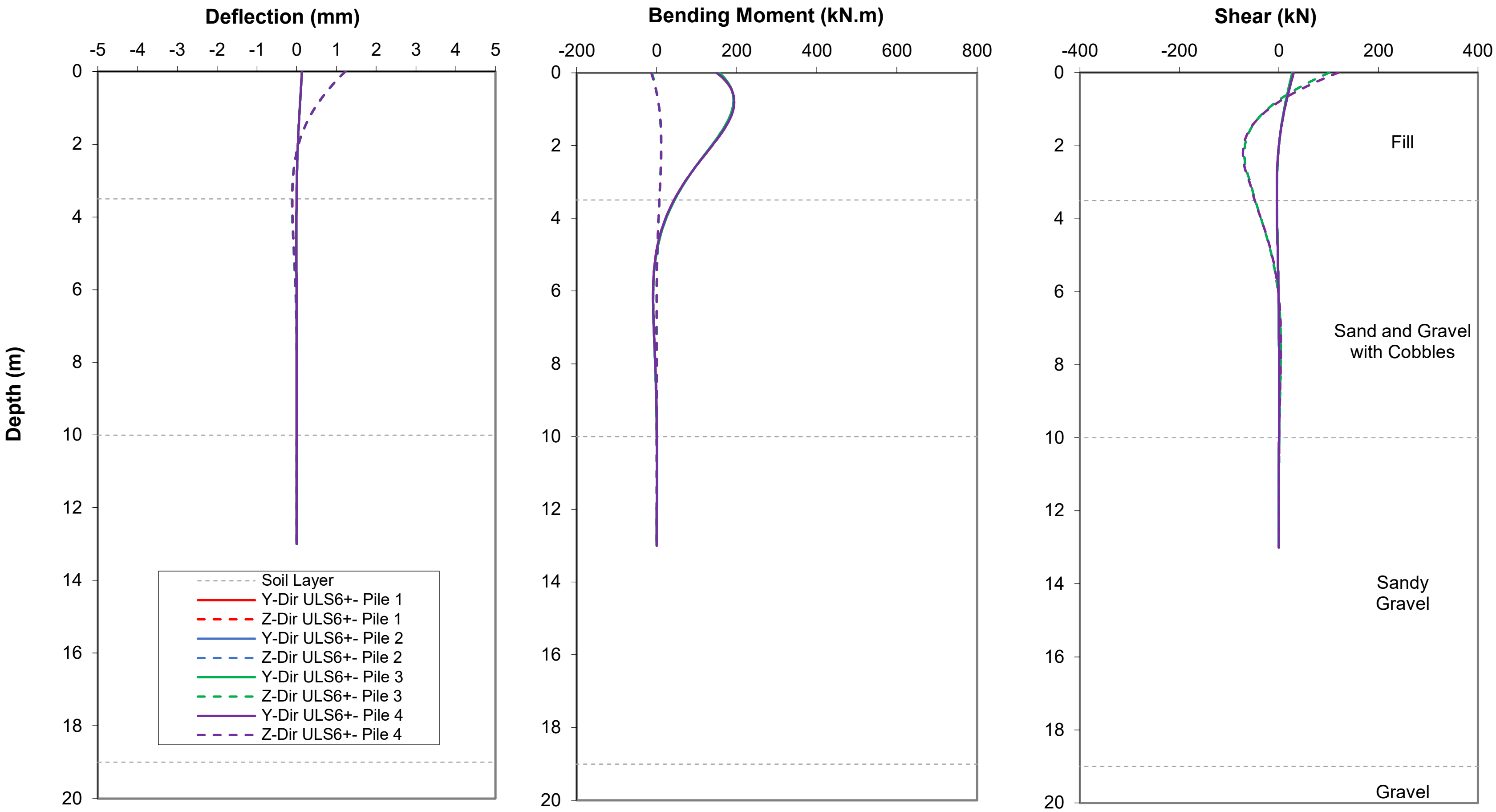


Notes: - No Geotechnical Resistance Factor (GRF) considered in displayed plots.
- Considered wall thickness = 15.9 mm.
- Pile embedment = 13 m (considers GRF).
- No concrete infill considered.
- Top of piles considered to be fixed connections.

Maximum Deflection (mm)	1.2
Maximum Bending Moment (kN.m)	193.5
Maximum Shear (kN)	119.2

GROUP PILE ANALYSIS
KHR 2 Abutments 762 mm Steel Pipe Pile
Load Case 7: ULS6 (+/+)

TETRA TECH



Notes: - No Geotechnical Resistance Factor (GRF) considered in displayed plots.
- Considered wall thickness = 15.9 mm.
- Pile embedment = 13 m (considers GRF).
- No concrete infill considered.
- Top of piles considered to be fixed connections.

Maximum Deflection (mm)	1.2
Maximum Bending Moment (kN.m)	193.6
Maximum Shear (kN)	119.3

GROUP PILE ANALYSIS
KHR 2 Abutments 762 mm Steel Pipe Pile
Load Case 8: ULS6 (+/-)

TETRA TECH