Berth No. 2 New Combi Wall and Shiploader Foundation

Concept Geotechnical Design Report

Neptune Bulk Terminals (Canada) Ltd.

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1 Introduction

1.1 Project Description

Neptune Bulk Terminals (Canada) Ltd. (NBTL) is planning to construct a new 7,000 Tonnes Per Hour (TPH) travelling shiploader within Berth No. 2 in their terminal in North Vancouver, British Columbia (BC) to replace the existing dual quadrant beam shiploaders. As part of the Definition phase of the project (Critical Decision Gate No. 2 [CD2]), Advisian recently completed the engineering related to the preliminary design of the foundation for the new travelling shiploader.

The site is at the location of the existing Berth No. 2 at Neptune terminals (the Site). The berth currently consists of a land back sheet pile wall (bulkhead wall), constructed around 1967. Analysis has shown that the existing bulkhead wall faces a number of geotechnical issues, the critical ones being:

- Full dynamic finite difference analyses (Advisian 2021b) indicated that there is a high potential for the Neptune Berth No. 2 bulkhead wall system to collapse or have large lateral displacements during a 1 in 2,475 seismic event. A collapse would involve some several meters of movement in the seaward direction and threatens to impact structures constructed on the seaward side of it; and
- There are elevated groundwater pressures on the land side of the bulkhead wall due to an ineffective wall drainage system. The drainage issues that exist at Berth No. 2 cause elevated stresses in the existing retaining wall system. Consequently, the wall has low factors of safety, especially at low tides and cannot tolerate additional loading such as heavy construction equipment operating near the wall at low tides.

For the above reasons and because the bulkhead wall is nearing the end of its useful service life, NBTL and the Port of Vancouver (owner of the bulkhead wall) decided to evaluate the prospect of replacing the existing bulkhead wall with a more robust retaining system that meets performance criteria in accordance with current practice and the National Building Code of Canada (NBCC 2015).

As a result, NBTL retained Worley Canada Services Ltd., operating as Advisian, to carry out a conceptual design of a Combi Wall that will retain the berth and support shiploader's rail beams.

The Site layout including the marine structures and proposed combi wall is shown in Drawing 317071-00041-01-MA-DSK-0001, which is included in Appendix 1.

This report presents a summary of geotechnical recommendations derived to support the conceptual design of the combi wall scope of work of this Technical Note is delineated in the next section.

1.2 Scope of Work

The geotechnical scope of work completed to support the conceptual design of the Combi wall and summarised in this technical note comprises the following:

- Summary of Soil stratigraphy;
- Geotechnical design parameters for improved and unimproved soils;

- Feasible ground improvement options;
- Ground improvement design;
- Assessment of static and seismic lateral earth pressures;
- Axial pile capacity with the consideration of ground improvement and pile group effects;
- Lateral behaviour of combi wall in seismic conditions;
- Pile drivability assessment; and
- Construction recommendations.

1.3 Previous Work and Desktop Review

Sources of background data as part of a desktop study are listed in Table 1-1.

Table 1-1 Desktop Study Resources

Document	Author	Date Published	Description
Swan Wooster D.L. 272 - Development Drawings (1966) (File No. U-1537-001)	Swan Wooster Engineering	1957	Provides site contour map at the original Neptune site development stage
Geotechnical Design Report, Cargill Overpass Extension Design- Allison Project-Neptune Bulk Terminals, report No. 1780811-040-R-Rev0	Golder Associates Ltd.	2018	Provides site specific seismic study at the Neptune North Vancouver terminals location
Golder Dynamic Testing and Analyses of Production Piles (19122046-001-R-Rev0; 19122046-002-R-Rev0; 19122046-003-R-Rev0)	Golder Associates Ltd.	2019 to 2020	Provides Pile Driving Analyzer (PDA) reports for production piles at Neptune Berth 1 Location
NTB Allison Project, Seismic Design Criteria, Document No NBT002A6-15-1100-CRI-0002	CWA Engineers (CWA)	2018	Provides seismic design criteria for the Allison Project at Neptune's Bulk Terminals
Berth No. 2 Shiploader Study, Geotechnical Investigation Report, Document No 317071-00041-00-SS-REP-0001 (Advisian 2021a)	Advisian	June 11, 2021	Provides 2020 site investigation information and results relevant to the Neptune Berth No. 2 location
Advisian Berth No. 2 Shiploader Study, Preliminary Geotechnical Design Report Document No 317071-00041-00-SS-TEN-0002 (Advisian 2021b)	Advisian	June 14, 2021	Provides geotechnical recommendations for Berth No. 2 Shiploader foundation design

1.4 Description of Existing Bulkhead Wall

The berth currently consists of a bulkhead wall, constructed around 1967, which has a top elevation of approximately +6 m above Chart Datum (mCD) and toe elevations of –10 mCD, except in the eastern corner, where it is locally extending to –15 mCD. The bulkhead wall is supported by two rows of anchors that tie back to a concrete dead-man anchor wall that is buried approximately 23.6 m north of the sheet pile wall. In the corner, where the sheets are lower, there is only one row of anchors. The lower part of the bulkhead wall's sheets is stabilised by a 2H:1V (Horizontal: Vertical) slope that extends down to the berth pocket, which is at approximately at –16 mCD. The slope was originally dredged lower than its current level, then covered with a layer of quarry tailings. It intersects the sheet piles at approximate elevation -2 mCD.

The bulkhead wall is shown schematically in the geotechnical cross-section (Drawing 317071-00041-00-SS-DSK-0005_R0) included in Appendix 2.

1.5 Description of Proposed Combi Wall

The proposed combi wall layout and cross-section is shown in the drawings included in Appendix A.

The combi wall consists of two rows of piles connected with cope beams. The north row consists of pipe piles that act as king posts, as well as sheet piles. The south row comprises pipe piles at a larger spacing than the north row of piles. The north piles are supported by one row of anchors that tie back to a concrete dead man anchor wall located 33 m north. There are two zones of ground improvement, one offshore on the passive side of the wall and one onshore, on the active side of the wall.

2 Subsurface Conditions Summary

Detailed description of subsurface conditions was provided in the Advisian geotechnical investigation report, Document No 317071-00041-00-SS-REP-0001 (Advisian 2021a) and in the Advisian preliminary geotechnical design report for the Berth No. 2 Shiploader Study, Document No 317071-00041-00-SS-TEN-0002 (Advisian 2021b).

The subsurface soil conditions encountered along the Shiploader site typically comprise the following:

- Fill (sand) placed during original reclamation of the Site onshore only, extending from underneath the existing pavement down to nominally –1 mCD (locally deeper behind the bulkhead wall), overlying;
- Granular marine sediments (sand/silty sand), extending to nominally –34 mCD, overlying;
- Fine grained marine sediments (clayey silt/silt/sandy silt/silty clay), extending to nominally –71 mCD, overlying;
- Glacial outwash sediments (gravel/sand/silty sand), extending to nominally –78 mCD, overlying; and
- Glacial till-like deposits (gravel/sand/silt) present for the remaining depth of investigation to –90 mCD and deeper.

Note that the above strata elevations are nominal. The strata depths vary both parallel and perpendicular to the berth and can be visualised on the geological sections included with the Advisian Geotechnical Investigation Report and also presented in Appendix 2 for ease of reference.

Being in a tidal environment, groundwater pressures on the marine side follow tidal elevations. On the land side of the existing bulkhead wall, the groundwater was originally intended to drain through drain holes in the bulkhead wall; however, these became ineffective many years ago, and the current Site investigation showed that groundwater elevation varies from approximately +3.5 mCD to +4.5 mCD. Groundwater elevations in the glacial till layer appear to be elevated to the extent that there is artesian pressure. The artesian pressures were the likely cause of heaving sands experienced during recent borehole drilling in the glacial outwash layer and probably arise because the overlying fine ground marine sediments have lower permeability and confine the aquifer, which probably has some connectivity to the hills to the North of the Site.

3 Seismic Considerations

For an understanding of the seismicity, liquefaction risk and other seismic considerations relevant to the Site, reference should be made to Advisian (2021b), specifically the seismic assessment presented in Section 2.

Key conclusions are the following:

- Due to the presence of potential liquefiable soil, the Site is classified as Site Class F according to NBCC 2015;
- For design, it is recommended to consider as horizontal acceleration design response spectrum at ground surface for 1:2,475 year events (probability of exceedance of 2% in 50 years) as the upper bound envelope of crustal earthquake ground motions derived by Site response analysis. Spectral accelerations are shown in Table 3-1 for ease of reference; and
- There is extensive liquefaction predicted within the Granular Fill layer and the naturally deposited sands underlying the Site (Granular Marine deposits) following the 1:2,475 years return period earthquake. Based on dynamic numerical modelling analysis performed by Naesgaard-Amini-Geotechnical Ltd. (NAGL), subcontracted to Advisian, during the shiploader CD2 design phase (Advisian 2021b), the liquefaction may extend to maximum elevation of approximately -30 mCD on the marine (offshore) side and approximately -22 mCD on the terrestrial (onshore) side. Below -30m CD there is still excess pore-pressure generation and strength softening ranging from 60% to 30%, extending to an elevation of -40 mCD.

Table 3-1 Site Response Spectral Horizontal Accelerations for the Site based on Site Response Analysis (Advisian 2021b)

S_a (Period [s])	Sa(0.2)	Sa(0.5)	Sa(1)	Sa(2)	Sa(5)	Sa(10)	PGA
Horizontal Acceleration Value (g)	0.38	0.68	0.60	0.60	0.08	0.04	0.29

4 Geotechnical Recommendations

This section provides geotechnical recommendations in support of the conceptual design of the combi wall.

4.1 Feasibility of Ground Improvement Options

As discussed in Section 3, extensive liquefaction is predicted in the existing granular fill and the native granular marine deposits to maximum elevation of approximately -30 mCD on the marine (offshore) side and approximately -22 mCD on the terrestrial (onshore) side.

Considering the aforementioned extent of liquefaction, expected large liquefaction induced loads on the proposed combi wall, and Advisian's experience with similar retaining structures in liquefiable soils, ground improvement is recommended to be implemented in front of and behind the combi wall.

The extent of ground improvement was assessed at this concept design stage but will be reviewed and optimized further, if possible, as part of the next phase of this project (preliminary design).

To evaluate the effectiveness and constructability of different ground improvement techniques, Advisian had meetings with local ground improvement contractors and received their feedback. Vibro replacement stone columns, deep soil mixing, vibro densification and timber piles were assessed as potential ground improvement options.

4.1.1 Stone Columns

The advantages and limitations of vibro replacement stone columns are summarized below:

- Stone columns have been applied by local ground improvement contractors in similar environments, in close proximity to existing infrastructure and have been successful in meeting performance criteria. The bottom-feed method is generally preferred, where the stone is inserted at the bottom of the soil profile and vibrated, before the stone column construction continues upwards (bottom-up method). In that way the soil between the stone columns gets densified (if generally sandy/gravelly with limited amount of fines). The wet top-feed method is an alternative for the onshore ground improvement but requires a bore to be formed that is then to be filled with stones, so it is less effective in improving the soil between columns, may induce more vibrations to existing infrastructure and creates a soil spoil that needs to be stockpiled or disposed. In summary, the bottom-feed stone column construction method is preferred:
- The method is relatively cost effective when compared with deep soil mixing or jet grouting;
- The improvement can be verified with Cone Penetration Tests (CPTs) post construction;
- A field trial prior to the production stone columns is recommended;
- Vibrations are expected, so it will be necessary to perform vibration and horizontal displacement/settlement monitoring and minimize the risk of impacting the existing bulkhead wall or the piles of the combi wall that have been installed prior to ground improvement; and

- The vibro replacement stone column method can offer savings if the same rig/crane and barge is utilized for stone columns and the combi wall piles.

4.1.2 Deep Soil Mixing

Deep soil mixing is a more expensive technique when compared to stone columns. In this method, it is important to encage the soil between cement columns and it is generally difficult to form good rigid geometry columns offshore. For this reason, it has uncertainty of success offshore and is more applicable to onshore applications. Spoil disposal and water management is required with this method. Finally, there is more uncertainty on estimating the deep soil mixing cost.

4.1.3 Vibro-Densification

Vibro-densification is an alternative to stone columns. With this method the native sandy soil is vibro-densified without inserting stone. The disadvantage it has when compared to other methods, including stone columns, is that it imposes higher vibrations. In addition, due to the expected soil conditions on Site, (silty sands at locations) it is advisable to introduce stones in the improvement solution as silty sands or finer soils will not densify.

4.1.4 Timber Piles

Timber piles have traditionally been a ground improvement method adopted in BC as they are cost-effective and can reduce vibrations. Nevertheless, timber piles have a length limit (likely less than 15 m), so not appropriate for the application at this Site. Also, higher risk of refusal is involved with this method, so pre-drilling may be necessary.

4.1.5 Conclusion

Based on the assessment summarized above, and based on feedback from local ground improvement contractors, it is concluded that vibro replacement stone columns are the most feasible ground improvement method for offshore and onshore areas of this project.

4.2 Ground Improvement Design

As indicated above, Advisian proposes a ground densification system utilizing vibro-replacement stone columns with the bottom-feed method. With this method, stone is added through the middle of the probe and compacted progressively to form stone columns. In this process, the adjacent granular soils (sands and gravels with limited amount of fines) are also densified.

Different stone column configurations were examined, and the following arrangement was suggested:

- Area replacement ratio (column area to total area) of about 10 %;
- Stone column diameter of approximately 1.0 m or approved equivalent so that replacement ratio of 10% is achieved;
- Stone column spacing of 3.0 m or approved equivalent sot that replacement ratio of 10% is achieved;

- Triangular stone column grid.

The improved soil properties were estimated using Priebe's method (1995) that involves determination of basic improvement factors and revising them by considering column compressibility, etc. A detailed description of this method can be found in Priebe (1995).

The following improved soil friction angles were derived and are considered for design:

- Improved friction angle of 38 degrees for granular marine deposits; and;
- Improved friction angle of 35 degrees for fine-grained silty deposits.

The friction angles above are based on a friction angle of 45 degrees for the stones and stone column area ratio of about 10%. The Ground Improvement Contractor must provide proper specifications for the stones assuring a rough angular coarse gradation to achieve the minimum target friction angle.

The vertical and lateral extent of stone column was studied based on considering the following criteria:

- Limiting the potential depth of liquefaction
- Estimating the ground improvement lateral extension from passive and active wedge zones in front of and behind the combi wall

We propose the following configuration which satisfies the criteria indicated above:

- Horizontal extent of stone column zone 30 m in front of the wall (offshore side), and extent of approximately 30 m behind the wall (onshore side).
- Vertical extent to Elevation. -35 mCD for the offshore zone and -25 mCD for the onshore zone. It should be noted that the vertical extents determined consider also the lateral variation of the thickness of the marine granular deposits (refer to geological cross-sections in Appendix 2).

The ground improvement zones are shown in drawings included in Appendix 1 (Drawings 317071-00041-01-MA-DSK-0004 and -0005).

The lateral and vertical extent of ground improvement will be reviewed and optimized further, if possible, during the next phase (preliminary design).

4.3 Soil Engineering Parameters

Soil engineering parameters (for improved and unimproved soils) are shown in Table 4-1 and Table 4-2 for the offshore and onshore sides, respectively. The soil engineering parameters of existing soils are in accordance with Advisian (2021b). The improved soil parameters are in accordance with Section 4.2. In addition to the existing sand fill material, a new well compacted structural fill is considered as part of the combi wall design.

Table 4-1 Interpreted Engineering Soil Parameters for Onshore Area (Combi Wall Active Zone)

Depth Below Ground (m)	Elevation (mCD)	Layer Description	Bulk Unit Weight (kN/m^3)	Submerged Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle ϕ' (°)	Average Undrained Shear Strength S_u (kPa)
0 to 6.5	+7.5 to +1	New Granular Fill (Sand)	20.0	10.2	0	35	
6.5 to 32.5	+1.0 to -25.0	Improved Granular Marine (Sand/gravelly sand)	20.0	10.2	0	38	
Refer to Appendix 2		Existing Granular Fill (Sand)	20.0	10.2	0	32	
Refer to Appendix 2		Granular Marine (Sand/gravelly sand)	19.0	9.2	0	32	
Refer to Appendix 2		Fine Grained Marine (Marine silt/clay)	18.5	8.7	2	30	110
Refer to Appendix 2		Outwash (Sandy silt/silty sand)	18.5	8.7	0	34	
Refer to Appendix 2		Till Like	19.0	9.2	0	36	

Table 4-2 Interpreted Engineering Soil Parameters for Offshore Area (Combi Wall Passive Zone)

Depth Below Ground (m)	Elevation (mCD)	Layer Description	Bulk Unit Weight (kN/m^3)	Submerged Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle ϕ' (°)	Average Undrained Shear Strength S_u (kPa)
0 to 19	-16.0 to -35.0	Improved Granular Marine (Sand/gravelly sand)	20.0	10.2	0	38	
14 to 19	-30.0 to -35.0	Improved Fine Grained Marine (Marine silt/clay)	20.0	10.2	0	35	
Refer to Appendix 2		Granular Marine (Sand/gravelly sand)	19	9.2	0	32	
Refer to Appendix 2		Fine Grained Marine (Marine silt/clay)	18.5	8.7	2	30	110
Refer to Appendix 2		Outwash (Sandy silt/silty sand)	18.5	8.7	0	34	
Refer to Appendix 2		Till Like	19	9.2	0	36	

Post earthquake-post liquefaction soil engineering properties are shown in Table 4-3 and Table 4-4. In deriving these properties at this conceptual stage, it was considered that some softening/strength reduction happens in the soils that do not liquefy, due to pore water pressure dissipation. In liquefiable soils, residual strength was considered in accordance with Advisian (2021b).

Table 4-3 Interpreted Engineering Soil Parameters Post Earthquake-Post Liquefaction for Onshore Area (Combi Wall Active Zone)

Depth Below Ground (m)	Elevation (mCD)	Layer Description	Bulk Unit Weight (kN/m^3)	Submerged Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle ϕ' (°)	Average Undrained Shear Strength S_u (kPa)
0 to 6.5	+7.5 to +1	New Granular Fill (Sand)	20.0	10.2	0	30	
6.5 to 32.5	+1.0 to -25.0	Improved Granular Marine (Sand/gravelly sand)	20.0	10.2	0	32	
Refer to Appendix 2		Existing Granular Fill (Sand)	20.0	10.2	0	9[1]	
Refer to Appendix 2		Granular Marine (Sand/gravelly sand)	19.0	9.2	0	9[1]	
Refer to Appendix 2		Fine Grained Marine (Marine silt/clay)	18.5	8.7	0	27	90
Refer to Appendix 2		Outwash (Sandy silt/silty sand)	18.5	8.7	0	34	
Refer to Appendix 2		Till Like	19.0	9.2	0	36	

Notes: [1] In accordance with Fayyazi et al. (2019). Alternatively, a residual strength of 0.1 of the vertical effective stress may be considered, in accordance with Advisian (2021b).

Table 4-4 Interpreted Engineering Soil Parameters Post Earthquake-Post Liquefaction for Offshore Area (Combi Wall Passive Zone)

Depth Below Ground (m)	Elevation (mCD)	Layer Description	Bulk Unit Weight (kN/m³)	Submerged Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle ϕ' (°)	Average Undrained Shear Strength S_u (kPa)
0 to 19	-16.0 to -35.0	Improved Granular Marine (Sand/gravelly sand)	20.0	10.2	0	32	
14 to 19	-30.0 to -35.0	Improved Fine Grained Marine (Marine silt/clay)	20.0	10.2	0	29	
Refer to Appendix 2		Granular Marine (Sand/gravelly sand)	19	9.2	0	9[1]	
Refer to Appendix 2		Fine Grained Marine (Marine silt/clay)	18.5	8.7	0	27	90
Refer to Appendix 2		Outwash (Sandy silt/silty sand)	18.5	8.7	0	34	
Refer to Appendix 2		Till Like	19	9.2	0	36	

Notes: [1] In accordance with Fayyazi et al. (2019). Alternatively, a residual strength of 0.12 of the vertical effective stress may be considered, in accordance with Advisian (2021b).

4.4 Static and Seismic Lateral Earth Pressures

Earth pressures will need to be considered in the design of the combi wall. Computation of earth pressures is recommended to be in accordance with the CFEM (2006).

4.4.1 Static Lateral Earth Pressures

Lateral earth pressures on below grade walls and retaining wall structures may be estimated as a triangular or trapezoidal earth pressure distribution using the following formula:

$$\sigma_h = K(\gamma z - u + q) + u$$

Where:

σ_h = lateral earth pressure (kPa)
K = coefficient of lateral earth pressure (K_a, K_o or K_p)
γ = unit weight of soil
z = depth below grade (m)

u = pore water pressure (KPa)
q = surcharge loading (kPa) (a minimum nominal pressure of 10 kPa is recommended)

The choice of the earth pressure coefficient (K_x) depends on the loading condition:

- K_a should be used where the wall or structure is allowed to move or rotate away from the soil load. i.e., the soil pushes on the wall;
- K_o should be used where the wall or structure is rigid and cannot move or rotate; and
- K_p should be used where the wall or structure moves into the soil (i.e., the wall pushes into the soil).

For the combi wall, which is considered flexible (yielding) and is designed to allow rotation, active earth pressures on the onshore wall side are considered applicable for design. If the wall is allowed to displace approximately 1-2 % of the wall retained height, then passive earth pressures may develop on the offshore wall side.

The unfactored soil parameters provided in Table 4-5 may be used for the combi wall design.

Table 4-5 Lateral Earth Pressure Coefficients in Static Conditions

Parameter	New Granular Fill (Sand)	Improved Granular Marine (Sand/gravelly sand)	Improved Fine Grained Marine (Marine silt/clay)	Granular Marine (Sand/Gravelly Sand)
Friction Angle, ϕ (degrees)	35	38	35	32
Interface Friction Angle, δ (degrees) [1]	17	17 (active side), 0 (passive side)	0 (passive side)	17
Coefficient of Active Earth Pressure (K_a)	0.25	0.22	0.25	0.28

Parameter	New Granular Fill (Sand)	Improved Granular Marine (Sand/gravelly sand)	Improved Fine Grained Marine (Marine silt/clay)	Granular Marine (Sand/Gravelly Sand)
Coefficient of Passive Earth Pressure (K_p)	3.69	4.20	3.69	3.26

Notes: [1] The interface friction angles are considered 0 for the passive wall side.

4.4.2 Seismic Lateral Earth Pressures

The combi wall should be designed to resist the earth pressures induced under seismic loading conditions.

The Mononobe-Okabe (M-O) method, which is included in CFEM (2006), is recommended to be used for the analysis in seismic conditions, as the wall is considered flexible (yielding) and is allowed to rotate.

Based on the M-O method, the total active (F_{AE}) and passive (F_{PE}) forces under seismic loading can be calculated using the following equations:

$$F_{AE} = \tfrac{1}{2} K_{AE} \gamma H^2 (1-k_v)$$

$$F_{PE} = \tfrac{1}{2} K_{PE} \gamma H^2 (1-k_v)$$

Where:

F_{AE} = Active lateral force considering both static and seismic earth pressures, in accordance with M-O method

F_{PE} = Passive lateral force considering both static and seismic earth pressures, in accordance with M-O method

H = height of wall

k_h = horizontal acceleration coefficient

k_v = vertical acceleration coefficient

γ = total unit weight, refer to Section 4.3

K_{AE} = active earth pressure coefficient (combined static and seismic), in accordance with the M-O method, refer to CFEM (2006)

K_{PE} = passive earth pressure coefficient (combined static and seismic), in accordance with the M-O method, refer to CFEM (2006)

The Horizontal acceleration coefficient, k_h for flexible (yielding) walls may be taken as half of the PGA, which is 0.29 g for the 2,475 years return period earthquake event (refer to Section 3). In addition, it is recommended that the k_h be corrected considering buoyant unit weight conditions in accordance with Kramer (1996). In this case K_h becomes 0.30 for the 2,475 years return period earthquake event.

The vertical acceleration coefficient, k_v is recommended to be considered as two-thirds of k_h, in accordance with CFEM (2006). Hence, k_v may be considered as 0.20.

The K_{AE}, K_{PE} values to be used for design are presented in Table 4-6.

Table 4-6 Lateral Earth Pressure Coefficients in Seismic Conditions

Parameter	Granular Fill (Sand)	<u>Improved</u> Granular Marine (Sand/Gravelly Sand)	<u>Improved</u> Fine Grained Marine (Marine Silt/Clay)	Granular Marine (Sand/Gravelly Sand)
Friction Angle, degrees	35	38	35	32
Interface Friction Angle, degrees [1]	17	17 (active side), 0 (passive side)	0 (passive side)	17
Coefficient of Active Earth Pressure (K_{AE})	0.55	0.50	0.55	0.62
Coefficient of Passive Earth Pressure (K_{PE})	2.88	3.36	2.88	2.48

Notes: [1] The interface friction angles are considered 0 for passive wall side.

4.5 Axial Pile Capacity in Static Conditions

Axial pile capacity of steel open-ended pipe piles with 1,219 mm outer diameter and 32 mm wall thickness was assessed. The main assumptions considered are the following:

- CPT data from CPTs 20-5 and 20-2 were used for the estimation of axial capacity. The details of CPT soundings are included in Advisian (2021a) and Advisian (2021b);
- It is understood that the north king post piles (that support the north rail beam and act as part of the retaining all of the berth) are at spaced 2.68 m centre to centre. I.e., they are spaced at approximately 2.2 pile diameters. For this spacing, a reduction factor of 0.77 was considered for the axial pile capacity. This is based on available literature and Advisian's experience with similar applications. It is considered conservative and appropriate for conceptual design;
- The south piles (that support the south beam) are spaced at spacing grated than 3 times the pile diameter. For this reason, no group effects need to be considered for these piles;
- Based on the proposed ground improvement design (refer to Section 4.2), friction angles of 38 and 35 degrees are considered for Granular Marine Deposits and Cohesive Marine Deposits, respectively. Back calculating the equivalent CPT tip resistance, an average conservative improvement factor of 1.5 applied on unimproved cone tip resistance values is considered attainable along the improved soil depth.

- The ultimate (unfactored) axial resistance of a single pile in compression was calculated as the sum of the ultimate shaft friction and ultimate end bearing resistance computed using the Laboratoire Central des Ponts et Chaussées (LCPC) CPT method summarized in CFEM (2004). The details of the method and corresponding assumptions were elaborated in Advisian (2021b);
- For normal static loading, geotechnical resistance factor of 0.5 was considered, assuming pile capacities are verified by Pile Driving Analyzer (PDA) and testing completed on 10 % of the piles installed;
- The axial capacity in compression was assessed considering the degree of soil plugging and the inner shaft friction of axial-loaded open-ended piles. Two cases were considered; Case 1 (plugged) considered soil plugging inside the steel pipe and Case 2 (unplugged) considered mobilization of the inner shaft friction. For Case 1, end bearing was computed based on total base area. For Case 2, end bearing was computed as the sum of the end bearing resistance of the annulus area of the steel pipe and the internal shaft friction, which was estimated as 60% (0.6) of the external skin friction. Pile end bearing resistance was then determined as the lesser of the plugged and unplugged cases and it was added to the external shaft friction to derive the total pile resistance in compression; and
- The Ultimate (unfactored) tensile geotechnical resistance of a single pile may be assumed as derived from the axial skin friction resistance and the weight of the pile itself, with no tension contribution from the pile base/toe. The factored uplift capacity may then be calculated by multiplying the ultimate skin friction resistance by the ULS Geotechnical Resistance factor in Tension of 0.3.

Charts of compression and tensile pile resistance versus depth, based on the above procedures are included in Appendix 3, with and without group effects. Considering the current combi wall design, the chart with the group effects should be used for the axial capacity of the north piles and the chart without group effects should be used for the axial capacity of the south piles.

4.6 Lateral Behaviour of Combi Wall in Seismic Conditions

The lateral performance of the combi wall system during/after the 2,475 years return period earthquake is critical for the performance of the berth and the overall design of the combi wall. To evaluate this at conceptual design stage, analyses were performed using the ProSheet (Ancelor Mittal, 2017) software as well with preliminary full dynamic finite difference analyses.

4.6.1 ProSheet Analyses

Lateral behaviour of the combi wall in seismic conditions was assessed using program ProSheet (ArcelorMittal, 2017). The following inputs and assumptions were taken into account:

- Horizontal inertial load 260 kN and vertical load of 580 kN at the top of combi wall as considered, provided by the Advisian marine team;
- Static lateral earth pressures were considered in accordance with Section 4.6;
- Application of differential seismic load (F_{AE} - F_A) as reverse triangular distributed pressures along the combi wall (see Section 4.4 for details);
- A Sheet Pile section of AZ28-700 was considered; and
- Tie rod (anchor) was considered at the elevation of 2.5 mCD.

The results of this preliminary analysis are summarized below:

- Maximum bending moment of 2,270 kNm/m was obtained at the wall section at anchor level; and
- Anchor force of 1,570 kN/m was obtained which translates to approximate total load of 4,210 kN for anchor spacing of 2.68 m.

The analysis shows wall performance and results comparable with the results of structural models set up by the Advisian structural team. It should be noted that the ProSheet analysis does not properly model the wall stiffness (combined sheet pile and king pile stiffness); the results are used for validation of the concept and a high level check of the wall performance with appropriate interpretation of the results.

4.6.2 Preliminary Full Dynamic Finite Difference Analysis

Because of the limitation of ProSheet program for the modelling of the combi wall (combined sheet pile and king pile) stiffness for deformation assessment, and in order to enhance the understanding of the soil structure interaction during/post the 2,475 years return period earthquake and associated liquefaction, a set of full dynamic finite difference analysis was performed. The dynamic two-dimensional finite difference model used the FLAC 2D software and was developed by NAGL, subcontracted to Advisian.

It should be noted that at this project phase (conceptual design to support CD1), the FLAC model has not accounted fully for the correct geometry and wall structural properties. It is still being developed and its results at this stage are considered preliminary. Nevertheless, the results of the analyses performed thus far can be used to help understand soil-structure interaction and especially the extent of liquefaction expected after the combi wall is installed.

Preliminary results of the full dynamic finite difference analysis show that liquefaction is limited to generally unimproved zones behind and in front of the combi wall and negligible liquefaction is expected under the combi wall system. Post liquefaction settlements (possible less than 1m) and horizontal displacements (possibly less than 1.5 m) are expected but the soil-wall system does not fail and maintains equilibrium.

The full dynamic finite difference analysis need to be continued and finalized in the next project phase (preliminary design).

4.6.3 Note on the Pile Length

Considering both axial and lateral pile performance, based on analyses performed to date (during the conceptual design), it is recommended to consider the below pile lengths:

- Pile tip at Elevation of -80 mCD for the north piles (piles supporting the north rail beam); and
- Pile tip at Elevation of – 85 mCD for the south piles (piles supporting the south rail beam).

This recommendation is mainly driven by the lateral pile capacity, based on analyses competed at this design stage. This will be further analysed, reviewed and optimized, if possible, at the next design phase.

4.7 Pile Drivability Analyses

A preliminary pile drivability was evaluated based on the wave analysis method, using the commercially available software GRLWEAP (2010 Version) developed by GRL Engineering Inc.

Static input parameters were adopted from the static pile analyses including steel pipe pile cross section geometry, and ultimate unit resistances obtained from axial pile analyses. The dynamic input parameters, including Smith damping and quake on the shaft and toe, were determined based on soil types encountered. The dynamic input parameters are listed in Table 4-7.

Table 4-7 Dynamic Input Parameters-Damping and Quake for Open Ended Driven Piles

Dynamic Parameters	Soil Type	Smith Damping (s/m)	Quake (mm)
Toe	All soils	0.49	2.54
Shaft	Sand	0.65	2.54
	Silt	0.40	2.54
	Clay	0.16	2.54

At this stage, the following diesel hammer was selected for the drivability analysis:

- APE D 225-42 with a rated energy of 757 kJ.

A driving system, including the hammer cushion and helmet as recommended by the manufacturer, was also included in the drivability analysis. The cushion material consists of 43% aluminum and 57% Conbest. The dimensions of the cushion and the properties of the driving system are shown in Table 4-8.

Table 4-8 Properties of Pile Driving System and Cushion Dimensions

Hammer Type	Cushion Elastic Modulus (MPa)	Cushion Thickness (mm)	Cushion Area (cm^2)	Cushion Coefficient of Restitution	Helmet Weight (kN)
APE D 225-42	3426.7	76.2	3,690.4	0.8	26.7

During the pile driving, the soil immediately adjacent to the pile shaft will be remoulded. Therefore, residual shear strength should be used for the pile drivability analysis to represent the soil conditions at the End of Driving (EOD). For this concept study, residual shear strength of 75% of the original shear strength was considered, representing a sensitivity of 1.3. This is a more conservative assumption in terms of blow count evaluation and ease of driving as a typical sensitivity value of 2 is common for EOD.

The drivability analysis results for 83 m pile penetration below the mudline are shown in Appendix 4. In the drivability model analyzed, the dense outwash sand and dense till are considered at a depth about 66 and 78 m below mudline, respectively. This means the analysis assumes 17 m of penetration into dense layers of outwash sand and till. The blow count results show that the blow count number is around 38 over 250 mm penetration at the top of outwash sand and approaches 48 blow count over 250 mm of penetration at the EOD where 5 m of penetration into the till layer was obtained.

The results also demonstrated that the maximum compressive stress of steel pipe reaches 230 MPa.

The aforementioned results are used as a base line at conceptual design stage. More drivability analysis is recommended to be undertaken at subsequent design phases, based on pile lengths considered and input from piling contractors on availability of available hammers.

4.8 Construction Recommendations

A suitable vibratory hammer can be employed to expedite the pile installation. The pilling contractor must perform a drivability assessment for their proposed piling equipment and submit the analysis results, as well as pile installation methods and equipment specifications to the Geotechnical Engineer of Record (EOR) for review, prior to mobilizing any construction equipment to Site. However, an impact hammer with a minimum energy rating of 700 kJ is recommended to be used to prove the ultimate pile resistance on restrike.

The Geotechnical EOR, or designated representative, must be present during pile installation and should be partly present during the ground improvement works (stone column construction), to verify that the piles are installed in accordance with the specifications and design requirements.

Potential impacts of piling and ground improvement works (stone column construction) on surrounding structures (including the existing bulk head wall) need to be assessed, taking account of each individual structure’s tolerance to vibrations and permanent ground deformations.

Overhead restrictions must be taken into account and safe operating distances need to be established, and Underground services to be retained. All underground services need to be identified and their positions positively confirmed by hydro vacuuming.

Condition of, and potential impacts of construction on the underground services need to be assessed, taking account each service line’s tolerance to surface loading, vibrations and permanent ground deformations.

Generally, vibration and horizontal/vertical displacement survey is recommended during the ground improvement works (stone column construction) and piling.

A field trial is recommended prior to the construction of the stone columns to verify the design assumptions and effectiveness of proposed construction methodology.

Due to high groundwater level in onshore area, dewatering by local sumps and/or well points will likely be required for temporary excavations below water level.

During pile/wall installation, depending upon the methods adopted and equipment capabilities, pre-drilling may be required. This need will be further evaluated in subsequent design phases and needs to be determined by the piling contractor. In case of predrilling, a reduced pile axial capacity needs to be considered. Advisian should be informed in that case to review the predrilling requirement and revise the estimated axial pile resistance.

Spoil and water management during ground improvement works is crucial. Contamination status of excavated spoil and extracted groundwater needs to be established, and a spoil and groundwater management plan developed. The bottom-feed stone column construction method is expected to generate negligible spoil, unless predrilling needs to be performed, in which case some spoil may be generated during drilling.

The stability of the sloped mudline where the shiploader foundation is situated might be impacted from piling operations or ground improvement works (stone column construction). The slope will likely settle and may require partial repair or reinforcement.

The stone column construction may cause, to some extent, horizontal displacement to the combi wall piles, if already installed. Preliminary feedback from ground improvement contractors has indicated that the stone columns have been installed successfully in similar applications, without causing unacceptable displacements or vibrations to neighbouring structures. Nevertheless, this is a risk to be further considered in subsequent design phases. A proper monitoring system and remedial measures need to be considered, in coordination with best practice.

Advisian

Neptune TERMINALS

5 Closure

We trust that this report satisfies your current requirements and provides suitable documentation for your records. If you have any questions or require further details, please contact the undersigned at any time.

Yours sincerely,

PROFESSIONAL
PROVINCE OF
Dr. K. KHAHESHI BANAB
KKB # 41303 Feb 3, 2022
BRITISH COLUMBIA
ENGINEER

Kasgin K. Banab, Ph.D., P.Eng.
Senior Geotechnical Engineer

Worley Canada Services Ltd.

Permit Number: 1000612

Senior Review by,

Panagiotis Berdousis, M.Sc., DIC, P.Eng.
Principal Geotechnical Engineer

6 References

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Appendix 1
Site Layout and Combi Wall General Conceptual Design Drawings

PLAN
1:750

PRELIMINARY
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m 0 10 20 30 40 m
1:750

DRAWING LIST

317071-00041-01-MA-DSK-0001	SITE LAYOUT
317071-00041-01-MA-DSK-0002	GENERAL ARRANGEMENT
317071-00041-01-MA-DSK-0003	DEMOLITION PLAN
317071-00041-01-MA-DSK-0004	GROUND IMPROVEMENT
317071-00041-01-MA-DSK-0005	TYPICAL SECTIONS - SHEET 1
317071-00041-01-MA-DSK-0006	TYPICAL SECTIONS - SHEET 2
317071-00041-01-MA-DSK-0007	TEMPORARY ACCESS LAYOUT

LEGEND

- PORTION OF TRACK TO BE REMOVE FROM SERVICE TO FACILITATE A TEMPORARY CONSTRUCTION ACCESS ROAD
- SECTION OF WALL TO BE REPLACED IN THE FUTURE DURING THE BERTH 3 REPLACEMENT PROJECT
- C32 AND DUMPER 2 PROJECT WORKS
- WALKWAY GRATING
- RAIL TRACK BOUNDARY

DRAWING NO.	DRAWING DESCRIPTION / TITLE
REFERENCE DRAWINGS / DESIGN STANDARDS	

PROJECT / W.O.: B2 MARINE WORKS
SYSTEM: POTASH
ASSET: 221
DISCIPLINE: STRUCTURAL

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TITLE:
BERTH 2 SHIPLOADER PROJECT
COMBI-WALL CONCEPT DESIGN
SITE LAYOUT

SCALE: SHOWN | SHEET 1 OF 1 | DRAWING NO. 01-MA-DSK-0001 | REV: A

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PLAN
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1:400

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TITLE:
BERTH 2 SHIPLOADER PROJECT
COMBI-WALL CONCEPT DESIGN
GENERAL ARRANGEMENT

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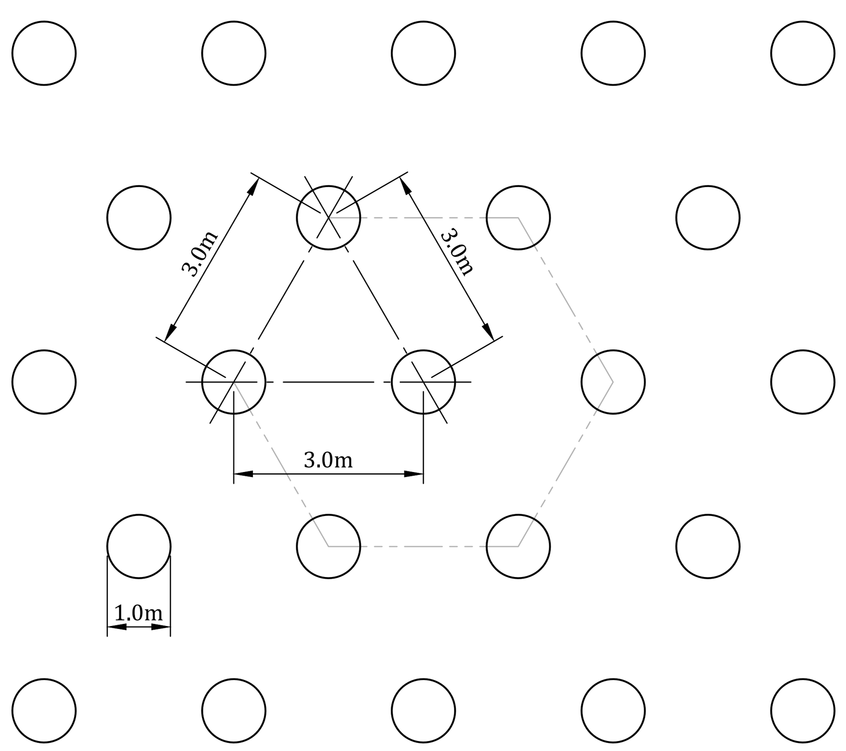

PLAN - VIBRO STONE COLUMNS
1:100

NOTES:
- STONE COLUMNS TO BE CONSTRUCTED WITH 'BOTTOM FEED' TECHNIQUE
- STONE COLUMN REPLACEMENT RATIO SHALL BE 10%, TO BE CONFIRMED BY A FIELD TRIAL
- STONE COLUMN NOMINAL SPACING = 3.0m ON AN EQUILATERAL TRIANGULAR GRID
- STONE COLUMN DIAMETER SHALL BE 1.0m
- STONE COLUMN GRID SPACING AND DIAMETER TO BE CONFIRMED BY A FIELD TRIAL

STONE COLUMNS

ZONE	AREA (m^2)	NO. OF COLUMNS
A	1420	156
B	6580	856
C	905	120
D	1410	176
E	5410	676
TOTAL	15725	1984

BERTH 2 MARINE INFILL

ZONE	AREA (m^2)
	1445

BERTH 2 RIPRAP TEMPORARY REMOVAL

ZONE	AREA (m^2)	VOLUME (m^3)
	7000	5040

BERTH 2 RIPRAP REINSTATE

ZONE	AREA (m^2)	VOLUME (m^3)
	5380	3880

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TITLE:
BERTH 2 SHIPLOADER PROJECT
COMBI-WALL CONCEPT DESIGN
GROUND IMPROVEMENT

SCALE: SHOWN | SHEET 1 OF 1 | DRAWING NO. 01-MA-DSK-0004 | REV: A

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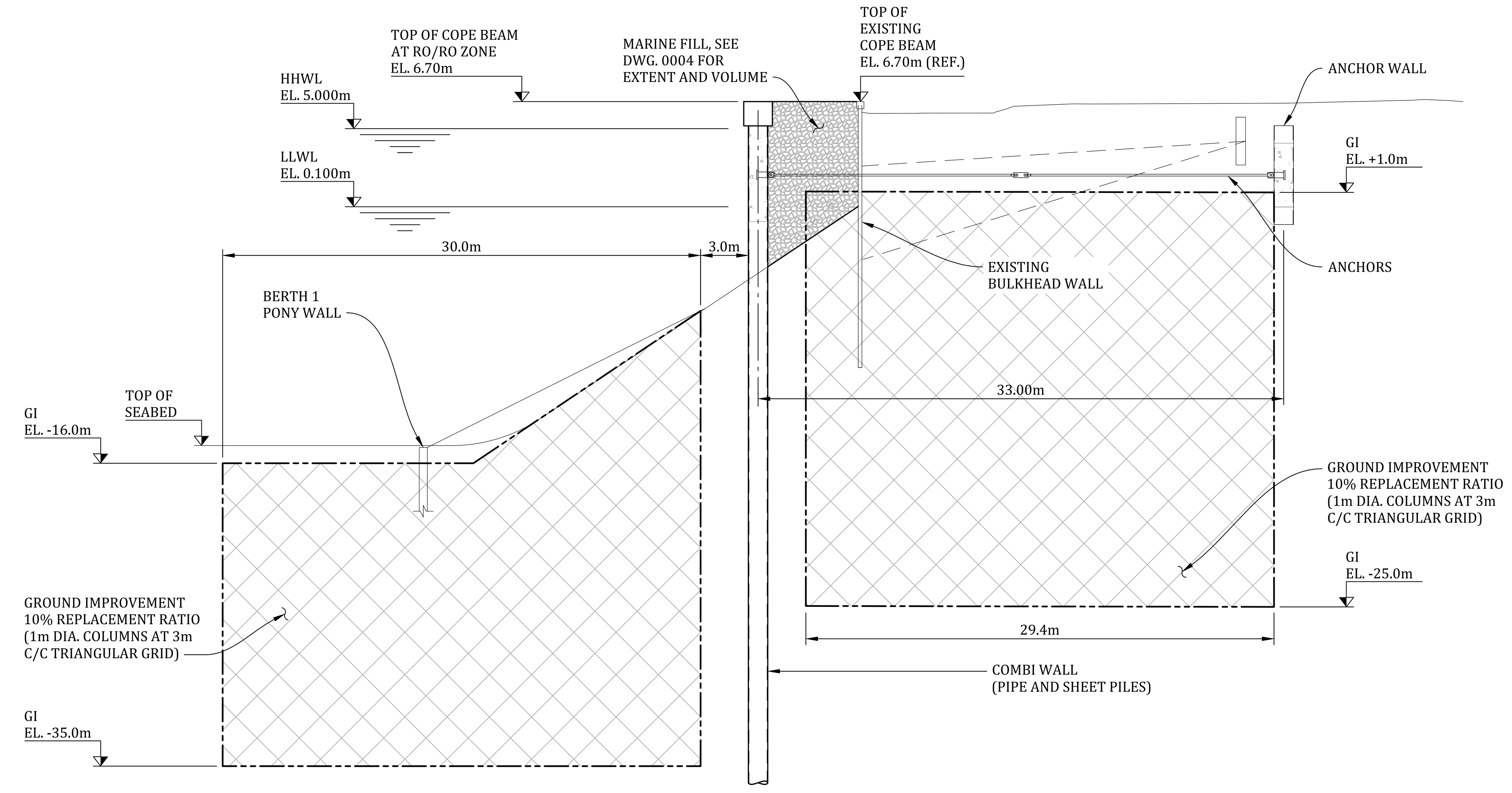

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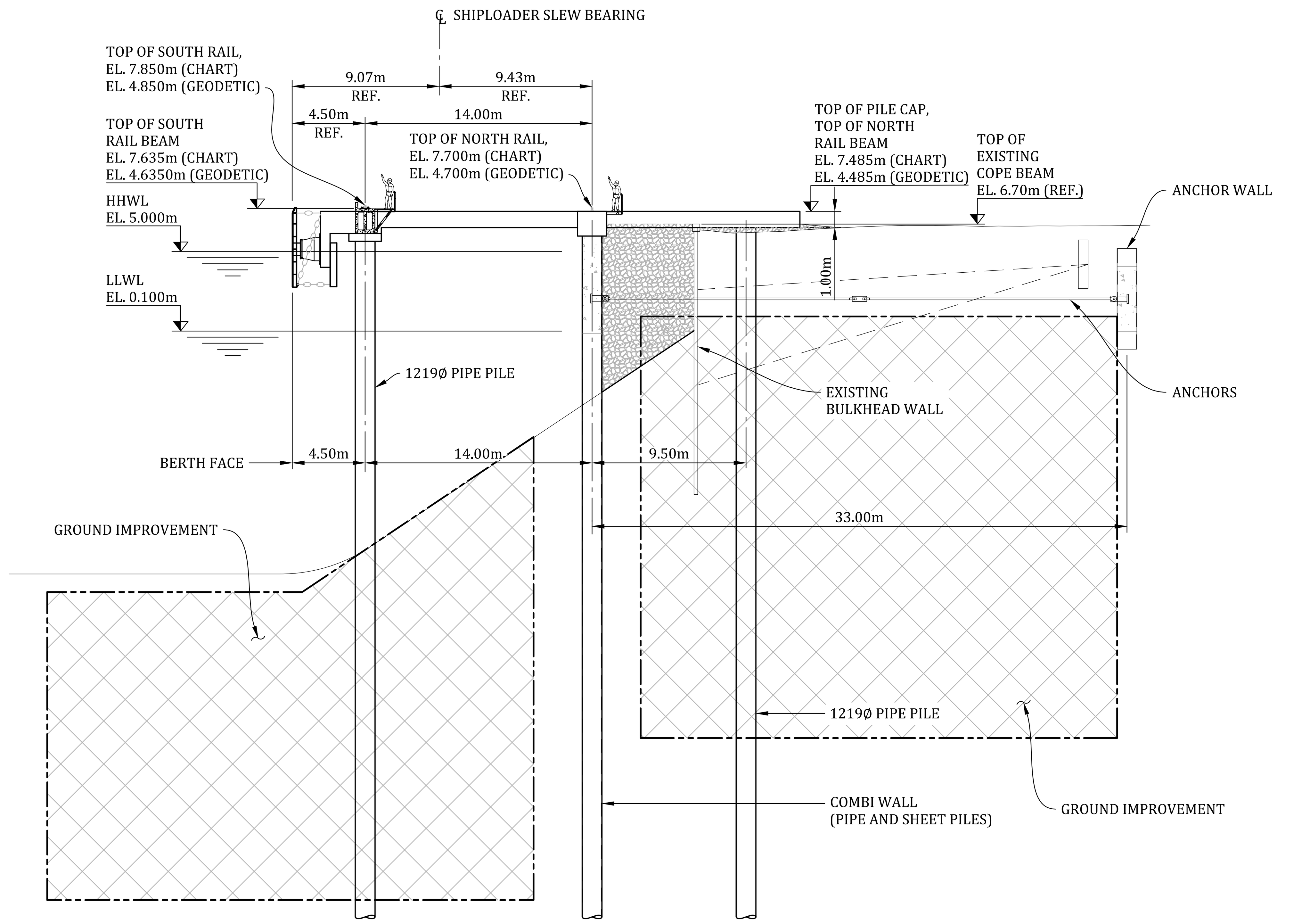

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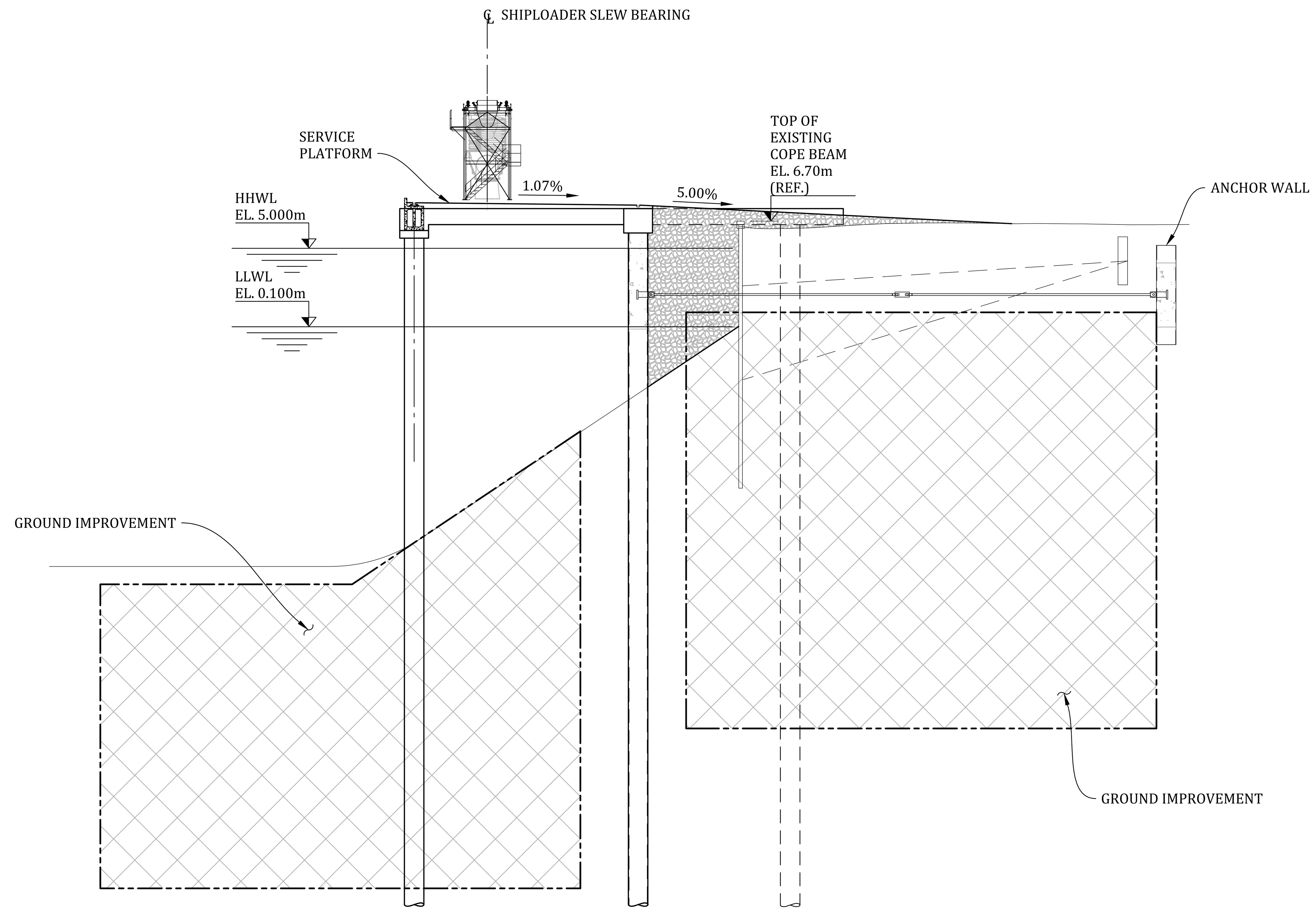

SECTION C
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NOTES:

1. ELEVATIONS ARE IN METRES AND ARE IN TIDE AND CHART DATUM, UNLESS NOTED OTHERWISE.

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COMBI-WALL CONCEPT DESIGN
TYPICAL SECTIONS - SHEET 1

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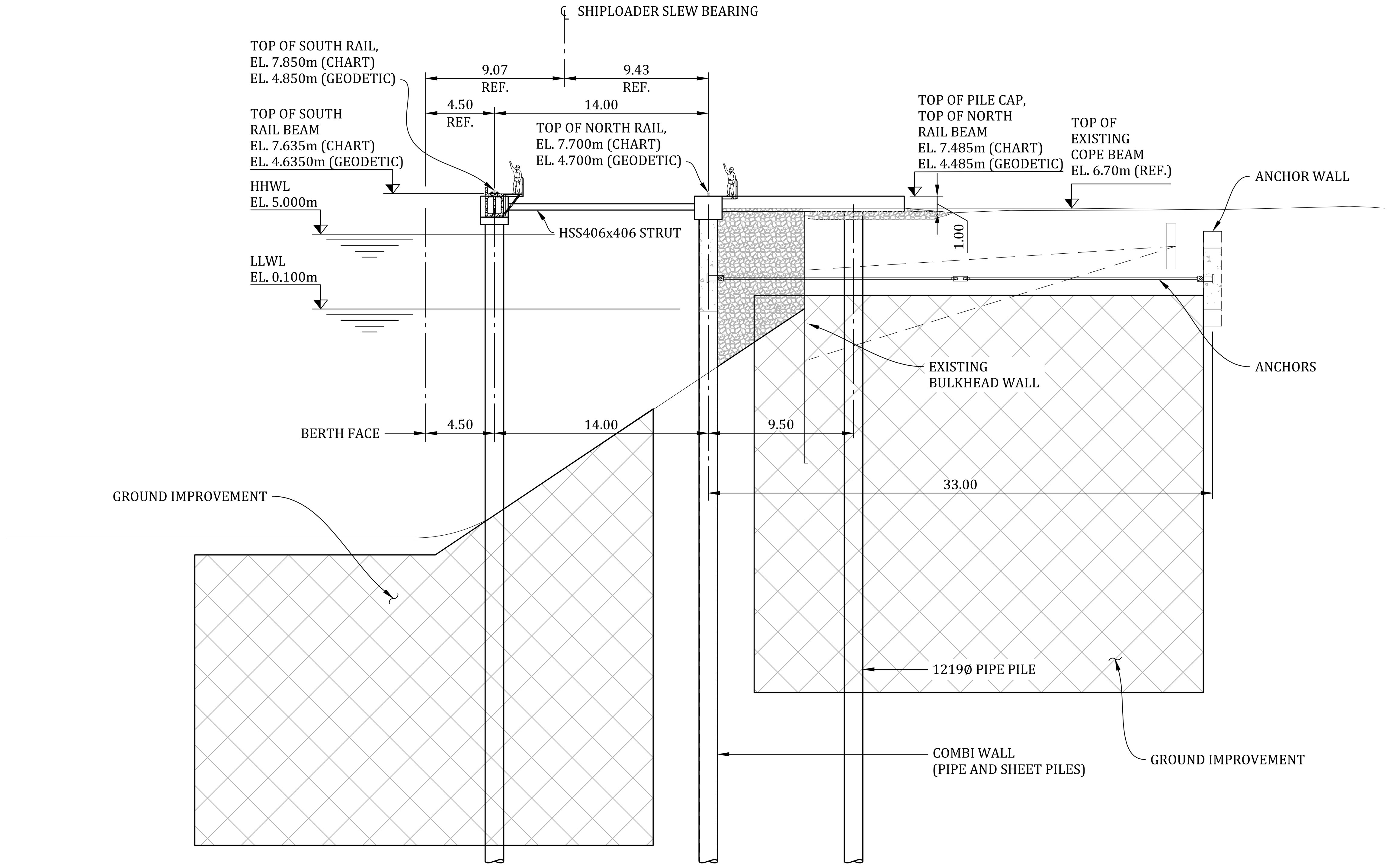

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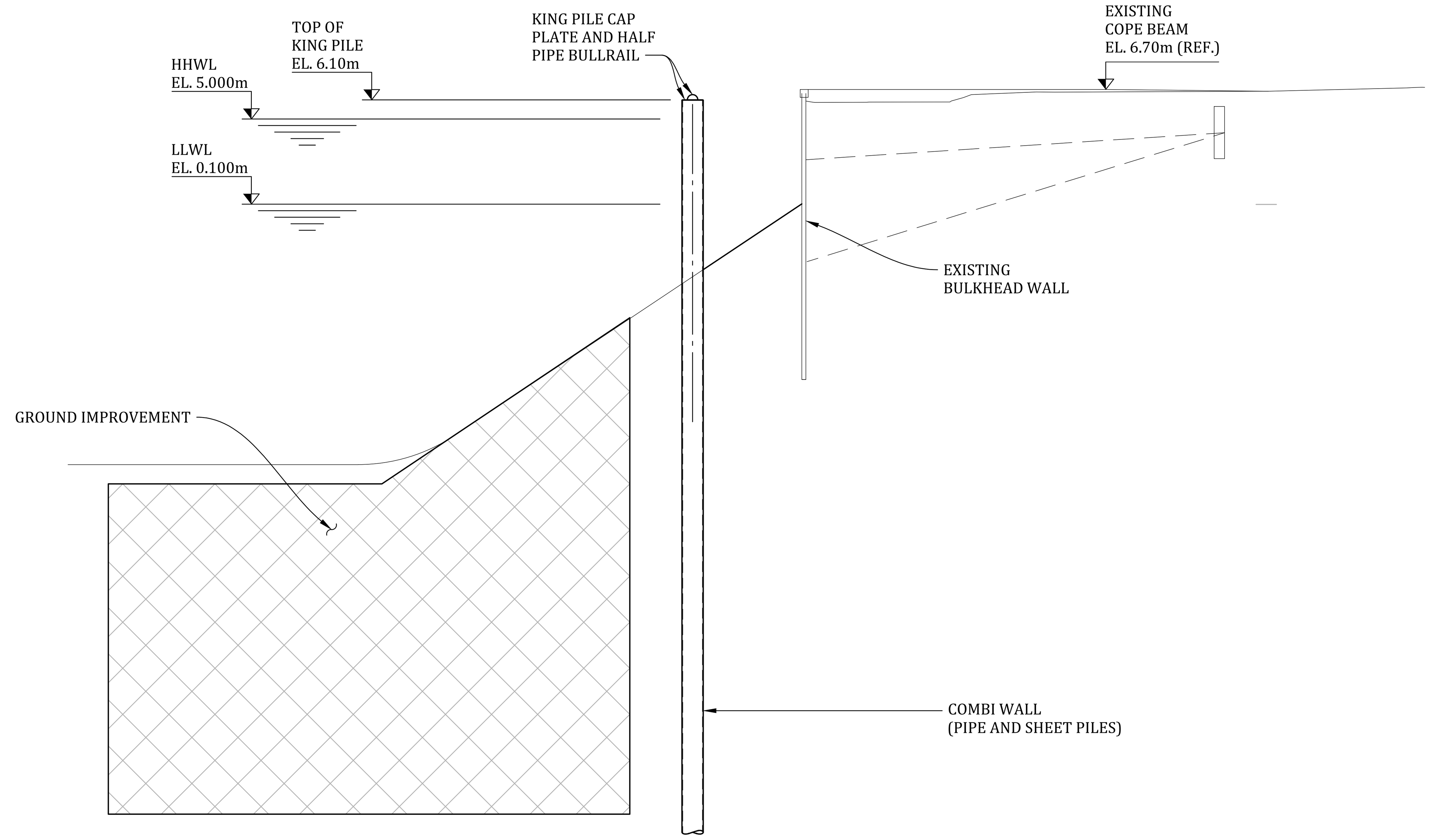

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SYSTEM: POTASH
ASSET: 221
DISCIPLINE: STRUCTURAL

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TYPICAL SECTIONS - SHEET 2

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Appendix 2
Geological Sections

2020 BOREHOLE LOCATIONS

BH#	NBTC NORTHING (m)	NBTC EASTING (m)	UTM NORTHING (m)	UTM EASTING (m)	SEABED OR GROUND ELEV. CD (m)	TOP ELEV. OF FINE GRAINED SEDIMENTS CD (m)	TOP ELEV. OF TILL-LIKE SEDIMENT CD (m)
BERTH 1 - INCLINOMETER BOREHOLE LOCATIONS							
B1-BH20-01	5461325.128	495938.851	5461324.988	495938.525	4.9	-18.1	-49.37
B1-BH20-02	5461306.143	495979.687	5461305.995	495979.377	5.3	-25.2	N/A
BERTH 2 - GEOTECHNICAL INVESTIGATION LOCATIONS							
SCPT20-BH20-01*	5461222.378	496169.120	5461222.559	496169.355	6.4	-26.0	-75.9
B2-BH20-01	5461223.893	496169.753	5461224.074	496169.987	6.5	-26.0	-75.9
SCPT20-BH20-02**	5461182.509	496249.634	5461182.706	496249.837	6.4	-33.1	-75.9
B2-BH20-02	5461184.232	496250.582	5461184.428	496250.784	6.5	-33.1	-75.9
SCPT20-BH20-03*	5461147.455	496324.345	5461147.666	496324.518	6.4	-16.2	-85.6
B2-BH20-03	5461149.128	496325.177	5461149.338	496325.350	6.4	-16.2	-85.6
SCPT20-BH20-04	5461188.795	496151.258	5461188.990	496151.500	-16.3	-33.1	-75.9
B2-BH20-04	5461186.895	496152.289	5461187.090	496152.530	-16.3	-28.5	-78.5
SCPT20-BH20-05	5461151.941	496234.571	5461152.150	496234.780	-15.8	-32.9	-82.5
B2-BH20-05	5461150.570	496234.471	5461150.780	496234.680	-15.8	-32.9	-82.5
SCPT20-BH20-06	5461107.053	496341.264	5461107.280	496341.430	-13.4	-35.5	-91.8
B2-BH20-06	5461107.843	496339.163	5461108.070	496339.330	-13.4	-35.5	-91.8
WWTP - CPT INVESTIGATION LOCATIONS							
WWTP-SCPT20-01***	5461315.203	496297.914	5461315.347	496298.097	6.4	-32.6	N/A
WWTP-SCPT20-02	5461344.576	496324.117	5461344.709	496324.290	6.5	-31.5	N/A

* CONTAINS SI CASING
** CONTAINS TWO VIBRATING WIRE PIEZOMETERS AND SI CASING
***CONTAINS ONE VIBRATING WIRE PIEZOMETER

LEGEND:

SCPT20-xx	2020 BERTH 2 SEISMIC CONE PENETRATION TEST
B2-BH20-xx	2020 BERTH 2 BOREHOLE LOCATION
B1-BH20-xx	2020 BERTH 1 BOREHOLE LOCATION
SCPT20-xx	2020 WWTP CPT/BOREHOLE LOCATION
DH-xx/CH-xx	HISTORICAL BOREHOLE LOCATION

PLAN
1:500

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NOTES:

1. NBTC GROUND LEVEL SURVEY INFORMATION

   THIS PLAN SHOWS GROUND MEASURED DISTANCES. TO CONVERT GROUND LEVEL NBTC COORDINATES TO UTM NAD83 ZONE10 NORTH COORDINATES FIRST ADD 2172.427 TO NORTHING AND 197.590 TO EASTING THEN SCALE ABOUT 0,0 BY A COMBINED SCALE FACTOR OF 0.99960240.

2. EXISTING BOREHOLE LOCATIONS WERE SCANNED FROM HISTORICAL DRAWINGS (SWAN WOOSTER ENGINEERING AND LEVELTON ENGINEERING SOLUTIONS) DATED 1957-1969 AND ARE THEREFORE SHOWN FOR REPRESENTATIVE PURPOSES ONLY.

3. SEABED CONTOURS DERIVED FROM 2020 BATHYMETRIC SURVEY CONDUCTED BY FRONTIER GEOSCIENCES INC. PROVIDED JANUARY 21, 2021.

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Neptune TERMINALS

TITLE:
BERTH 2 SHIPLOADER OPTIONS STUDY
7000 TPH TRAVELLING SHIPLOADER CONCEPT
BERTH 2 GEOTECHNICAL INVESTIGATION

SCALE:	SHEET	OF	DRAWING NO.	REV:
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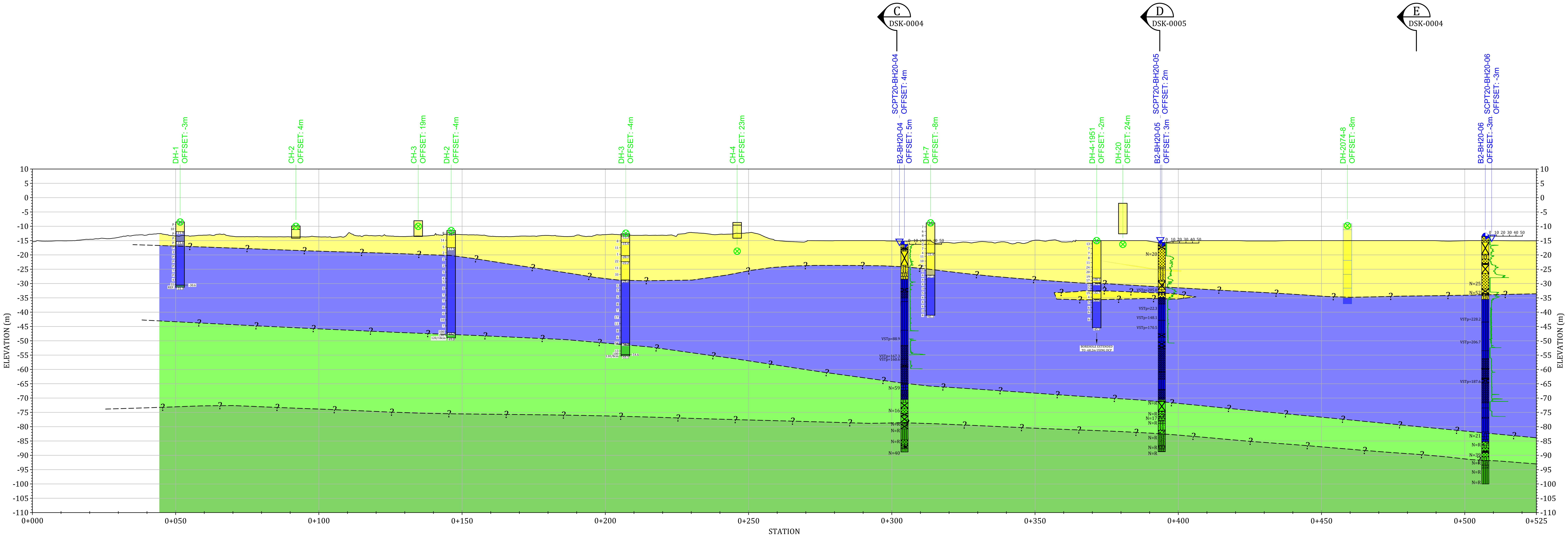
U:\YVR\317071\00041_NEP_B2SHIPLFDN\11_DRAWINGS\15_I_AND_E\04_GEOTECH\01_SKETCHES\317071-00041-00-SS-DSK-0001-0005.DWG DATE: 2021-02-25



PROFILE - BOREHOLE SECTION A

LEGEND:

SCPT20-xx	2020 BERTH 2 SEISMIC CONE PENETRATION TEST
B2-BH20-xx	2020 BERTH 2 BOREHOLE LOCATION
B1-BH20-xx	2020 BERTH 1 BOREHOLE LOCATION
SCPT20-xx	2020 WWTP CPT/BOREHOLE LOCATION
DH-xx/CH-xx	HISTORICAL BOREHOLE LOCATION

LEGEND - STRATIGRAPHY:

- INTERPOLATED STRATIGRAPHIC BOUNDARY
- HISTORICAL GROUND SURFACE

LEGEND - GEOLOGICAL UNITS:

- FILL
- GRANULAR MARINE
- FINE GRAINED MARINE
- OUTWASH
- TILL LIKE

LEGEND - BOREHOLE STRATIGRAPHY:

- ASPHALT
- HIGH PLASTICITY CLAY
- LOW PLASTICITY CLAY
- LOW PLASTICITY SILTY CLAY
- LOW PLASTICITY SANDY CLAY
- CORE LOSS
- FILL
- POORLY-GRADED GRAVEL
- WELL-GRADED GRAVEL/SANDY GRAVEL
- SILT
- SANDY SILT
- CLAYEY SAND
- SILTY SAND
- POORLY-GRADED SAND
- POORLY-GRADED GRAVELLY SAND
- N=XX STANDARD PENETRATION TEST
- VSTp=XX PEAK SHEAR STRENGTH FROM IN-SITU eVST

PRELIMINARY
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NOTES:

1. FOR GENERAL NOTES REFER TO DWG. 317071-00041-00-SS-DSK-0001.
2. FOR HORIZONTAL DATUM, REFER TO NOTE 1 ON DWG. 317071-00041-00-SS-DSK-0001.
3. ELEVATIONS ARE SHOWN IN METRES TO CHART DATUM.
4. DRAWINGS TO BE READ IN CONJUNCTION WITH THE GEOTECHNICAL SITE INVESTIGATION REPORT, REFER TO ADVISIAN REPORT 317071-00041-00-SS-REP-0001.
5. HISTORICAL BOREHOLE LOCATIONS WERE SCANNED FROM HISTORICAL DRAWINGS (SWAN WOOSTER ENGINEERING AND LEVELTON ENGINEERING SOLUTIONS) DATED 1957 TO 1969. GEOLOGICAL UNIT INTERPRETATION AT THESE BOREHOLES IS PROVIDED FOR REFERENCE ONLY.
6. INTERPRETATION BETWEEN BOREHOLES ARE REPRESENTATIVE. EXACT BOUNDARIES MAY DIFFER FROM THOSE SHOWN ON THE SECTIONS.
7. CPT PROFILES ARE SHOWN IN MPa.
8. SEABED CONTOURS DERIVED FROM 2020 BATHYMETRIC SURVEY CONDUCTED BY FRONTIER GEOSCIENCES INC. PROVIDED JANUARY 21, 2021.

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ENGINEERING SEAL:

VENDOR: Advisian Worley Group 317071-00041

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BERTH 2 GEOTECHNICAL INVESTIGATION

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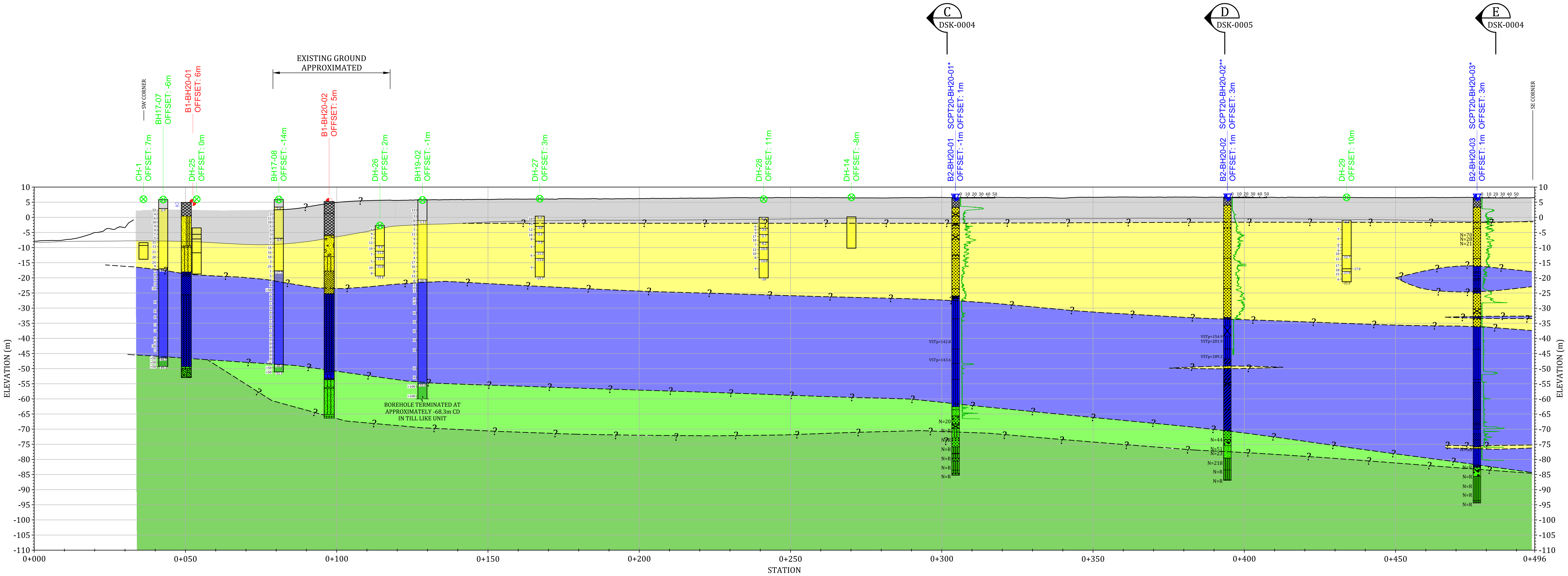

PROFILE - BOREHOLE SECTION B

LEGEND:

- SCPT20-xx — 2020 BERTH 2 SEISMIC CONE PENETRATION TEST
- B2-BH20-xx — 2020 BERTH 2 BOREHOLE LOCATION
- B1-BH20-xx — 2020 BERTH 1 BOREHOLE LOCATION
- SCPT20-xx — 2020 WWTP CPT/BOREHOLE LOCATION
- DH-xx/CH-xx — HISTORICAL BOREHOLE LOCATION

LEGEND - STRATIGRAPHY:

- INTERPOLATED STRATIGRAPHIC BOUNDARY
- HISTORICAL GROUND SURFACE

LEGEND - GEOLOGICAL UNITS:

- FILL
- GRANULAR MARINE
- FINE GRAINED MARINE
- OUTWASH
- TILL LIKE

LEGEND - BOREHOLE STRATIGRAPHY:

- ASPHALT
- HIGH PLASTICITY CLAY
- LOW PLASTICITY CLAY
- LOW PLASTICITY SILTY CLAY
- LOW PLASTICITY SANDY CLAY
- CORE LOSS
- FILL
- POORLY-GRADED GRAVEL
- WELL-GRADED GRAVEL/SANDY GRAVEL
- SILT
- SANDY SILT
- CLAYEY SAND
- SILTY SAND
- POORLY-GRADED SAND
- POORLY-GRADED GRAVELLY SAND
- N=XX STANDARD PENETRATION TEST
- VSTp=XX PEAK SHEAR STRENGTH FROM IN-SITU eVST

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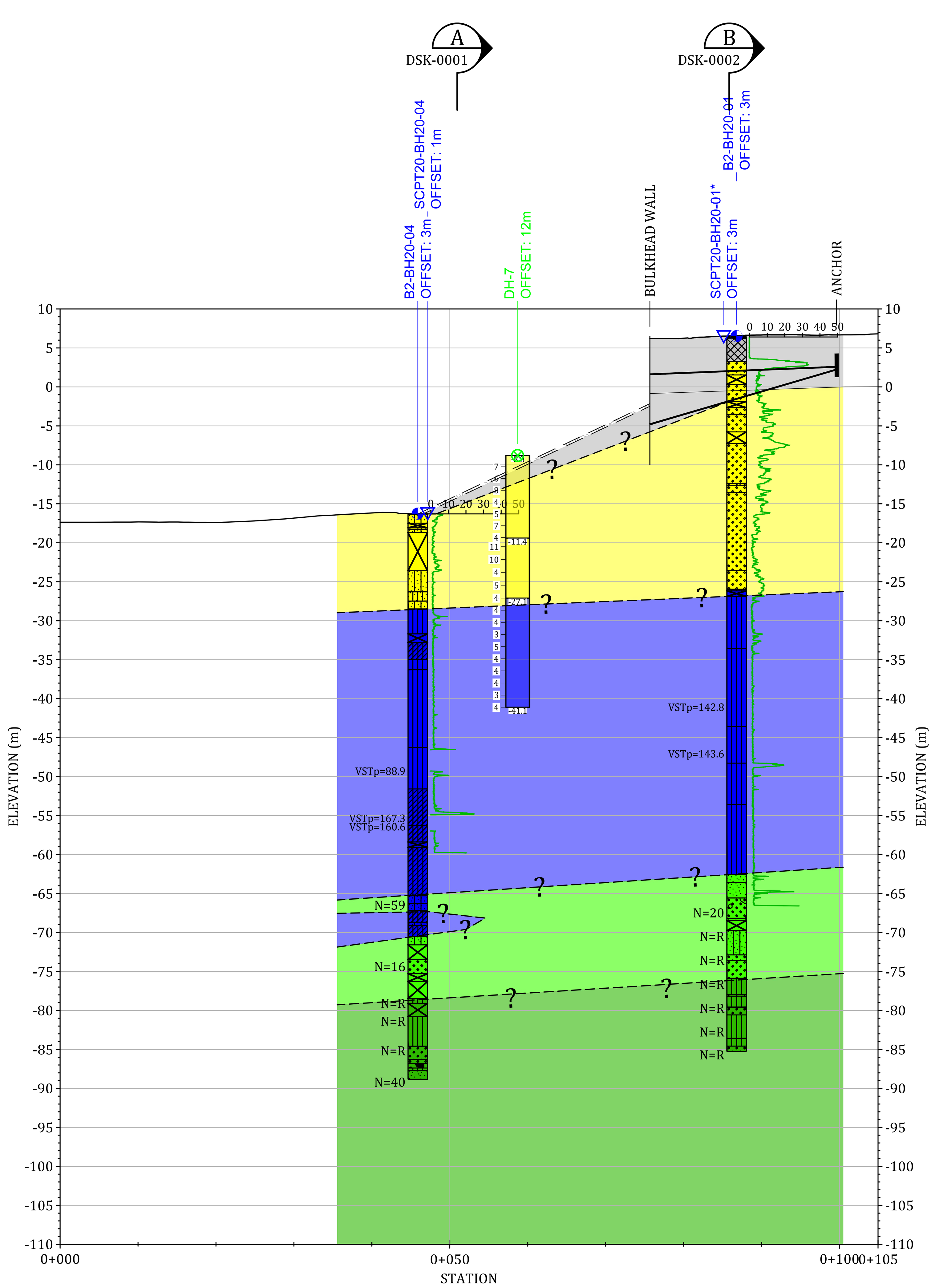
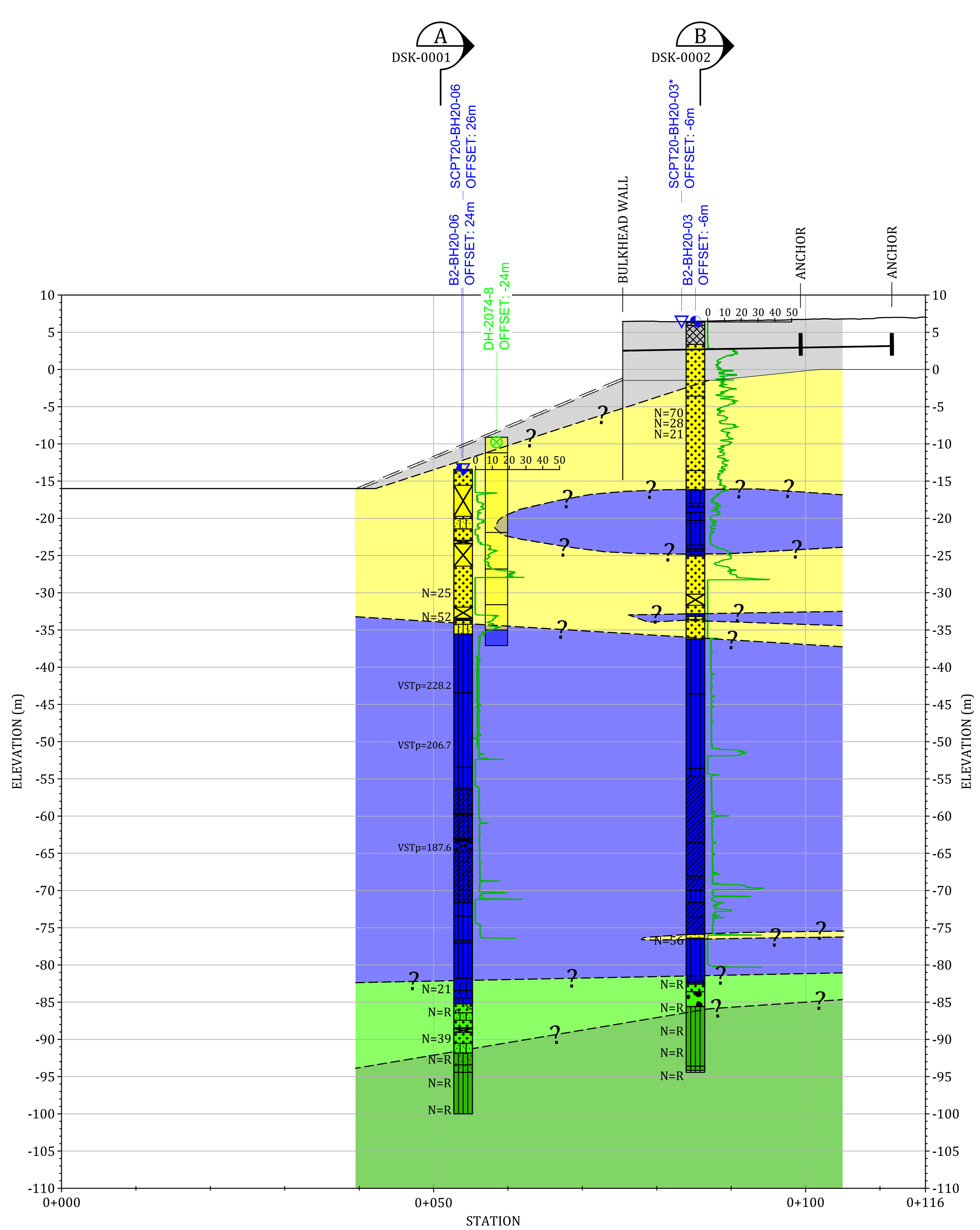


PROFILE - BOREHOLE SECTION C



PROFILE - BOREHOLE SECTION E

LEGEND:

- SCPT20-xx — 2020 BERTH 2 SEISMIC CONE PENETRATION TEST
- B2-BH20-xx — 2020 BERTH 2 BOREHOLE LOCATION
- B1-BH20-xx — 2020 BERTH 1 BOREHOLE LOCATION
- SCPT20-xx — 2020 WWTP CPT/BOREHOLE LOCATION
- DH-xx/CH-xx — HISTORICAL BOREHOLE LOCATION

LEGEND - STRATIGRAPHY:

- INTERPOLATED STRATIGRAPHIC BOUNDARY
- HISTORICAL GROUND SURFACE

LEGEND - GEOLOGICAL UNITS:

- FILL
- GRANULAR MARINE
- FINE GRAINED MARINE
- OUTWASH
- TILL LIKE

LEGEND - BOREHOLE STRATIGRAPHY:

- ASPHALT
- HIGH PLASTICITY CLAY
- LOW PLASTICITY CLAY
- LOW PLASTICITY SILTY CLAY
- LOW PLASTICITY SANDY CLAY
- CORE LOSS
- FILL
- POORLY-GRADED GRAVEL
- WELL-GRADED GRAVEL/SANDY GRAVEL
- SILT
- SANDY SILT
- CLAYEY SAND
- SILTY SAND
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- N=XX STANDARD PENETRATION TEST
- VSTp=XX PEAK SHEAR STRENGTH FROM IN-SITU eVST

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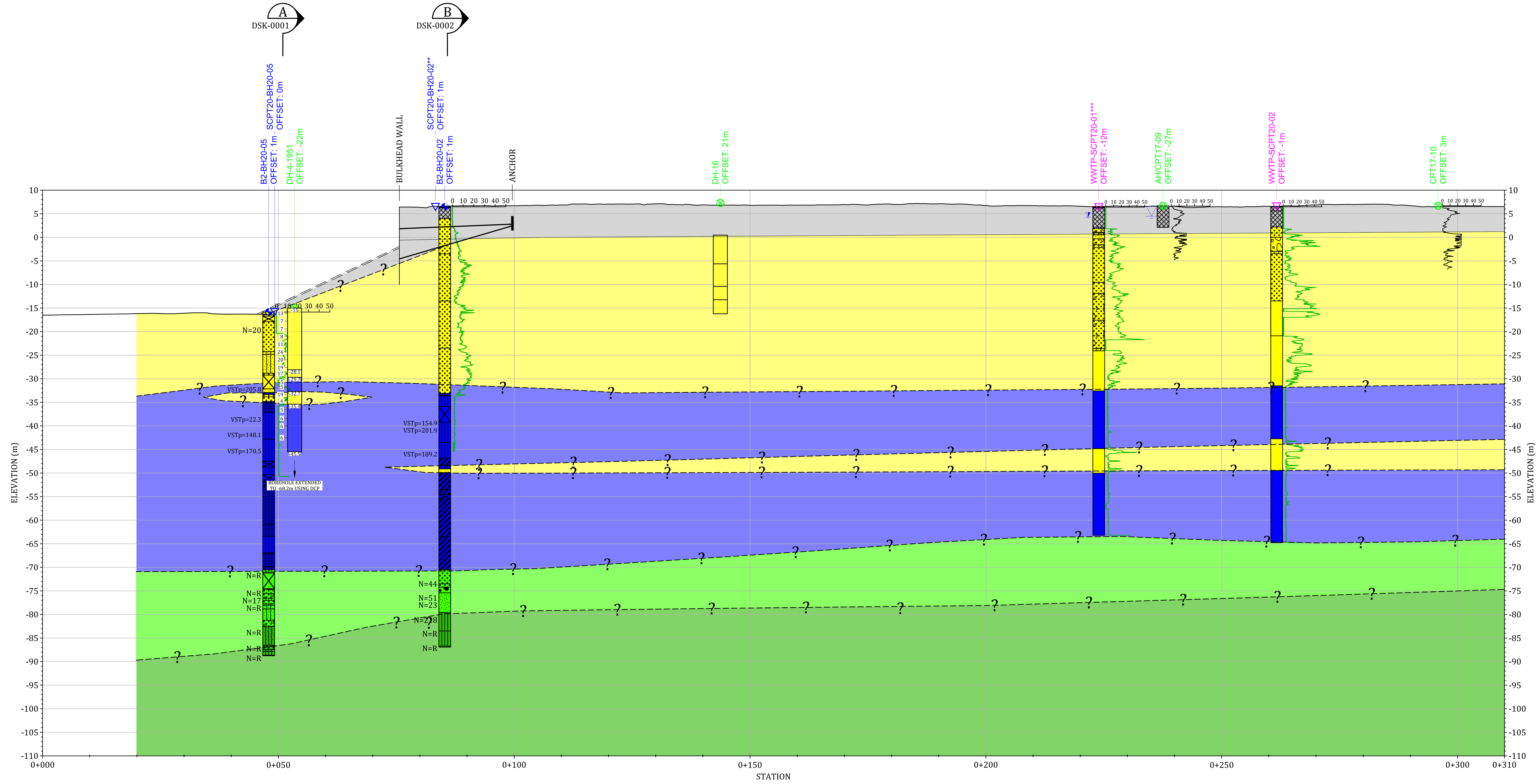

PROFILE - BOREHOLE SECTION D

LEGEND:

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- FINE GRAINED MARINE
- OUTWASH
- TILL LIKE

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- HIGH PLASTICITY CLAY
- LOW PLASTICITY CLAY
- LOW PLASTICITY SILTY CLAY
- LOW PLASTICITY SANDY CLAY
- CORE LOSS
- FILL
- POORLY-GRADED GRAVEL
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9. FILL BOUNDARY TAKEN FROM HISTORICAL GROUND SURFACE FROM HISTORICAL DRAWINGS (SWAN WOOSTER ENGINEERING AND LEVELTON ENGINEERING SOLUTIONS) DATED 1957 TO 1969.
10. SCPT20-01*** AND SCPT20-02 DRILLED TO DEPTHS OF 30.5mbgs AND 27.4mbgs RESPECTIVELY. CPT DATA USED FOR INTERPRETATION TO DEPTH OF TESTING.

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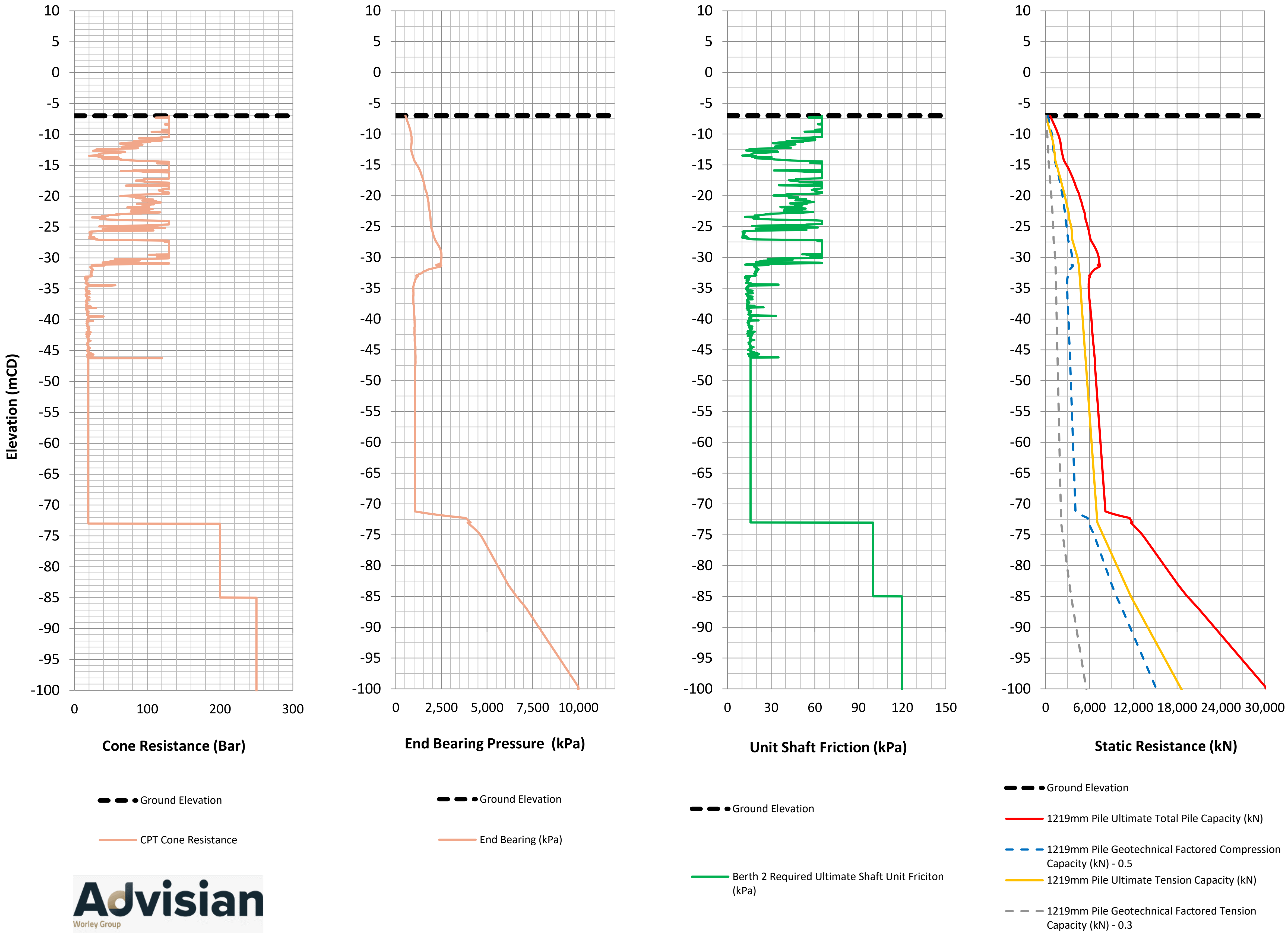
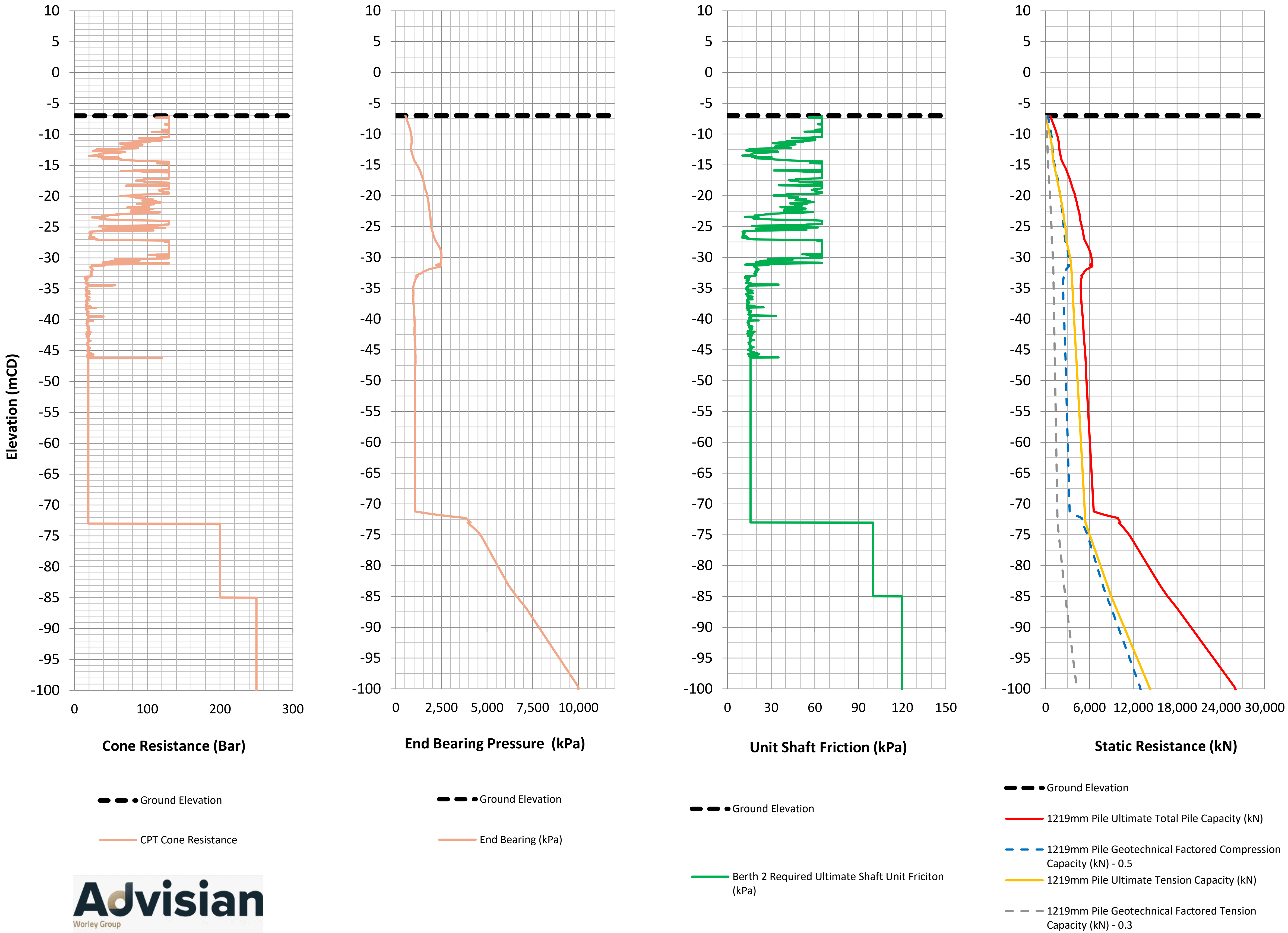
Appendix 3
Axial Pile Capacity Graphs

Ultimate and Factored Capacity as a Function of Elevation (Based on BH20-05/20-02) -Only Ground Improvement included



Advisian
Worley Group

Ultimate and Factored Capacity as a Function of Elevation (Based on BH20-05/20-02) -Ground Improvement and Pile Group Effects included



Appendix 4
Pile Drivability Analysis Graphs

Worley Parsons Services Ltd
APE D 225 BH 20-06

2021 Dec 08
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.750 / 0.750