APPENDIX 41

CONCEPT DESIGN REVIEW

Highly Confidential

MBIE
Northland Shipyard & Dry Dock

Concept Design Review

05-08-2026

NZ0074192.0767

Document distribution

MBIE

Northland Shipyard & Dry Dock – Concept Design Review

05-08-2026

Project Number: NZ0074192.0767

Prepared for

MBIE

Prepared by

WSP New Zealand Limited
12 Moorhouse Avenue, Christchurch, 8011
T +64 3 363 5507

Quality control	Name	Date	
Prepared by:	Zubair Nizamani, Jan Stanway	26/06/2026	
Reviewed by:	Campbell Keepa, Paul Westaway	26/06/2026	
Approved by:	Paul Westaway	26/06/2026	

Revisions

Rev	Date	Details
A	05/08/2026	Minor updates following review of 31/07/2026 Final DRAFT Beca report

Table of contents

Executive summary

MBIE is preparing a FTAA application for the Northland Shipyard and Dry Dock facility development, which comprises reclamation works, a dry dock facility, marine infrastructure, and associated buildings. This report presents a review of the Beca concept design, including an independent conceptual geotechnical assessment of the proposed combi-wall structure and high-level consideration of the revetment/bund and the open piled suspended deck structures.

The independent geotechnical assessment undertaken by WSP is based on historical and recent geotechnical investigations (1983, 2000, and 2019). These investigations include boreholes, laboratory testing, and interpreted ground models. The subsurface profile comprises reclamation fills overlying marine deposits, including Upper Loose Sand, Upper Sandy Silt-Clay, Lower Dense Sand, and deeper fine-grained soils. Bedrock is not encountered within the investigated depth.

The key geotechnical considerations influencing the engineering design are:

- **Liquefaction susceptibility** of Upper Loose Sand under seismic loading, with potential for significant ground deformation if untreated.
- **Compressibility of Upper Sandy Silt-Clay**, which governs long-term consolidation settlement and structural loading.
- **Variability and uncertainty in ground conditions**, particularly in deeper units and lateral extent of critical layers.

The Beca conceptual design adopts a revetment around two sides of the reclamation and a combi-wall retaining system, to the north face of the facility which is to be founded in Lower Dense Sand, supported by tie rods and a dead man wall, with staged reclamation and ground improvement using Vibroflot compaction.

Beca's concept design includes open ended 1,626 mm diameter tubular steel piles with an estimated toe depth of -30 m below Chart Datum ("CD") for the combi-wall and 914 mm diameter tubular steel piles for the jetty and suspended deck structure adjacent to the shiplift, with an estimated toe level around -45 mCD. Based on existing SPT data in the vicinity of the piling, WSP expects these piles can be installed using vibratory hammer driving techniques until a depth of around - 20 mCD to - 25 mCD and below this level, we expect impact hammer driving will be required.

WSP's independent geotechnical analysis confirms that the Beca conceptual design is an appropriate and viable engineering solution.

1. Introduction

The Ministry of Business, Innovation and Employment (MBIE) is preparing a Fast-track Approvals Act 2024 application for the Northland Shipyard and Dry Dock Facility, which is to include a shipyard, dry dock, marine facilities, and supporting buildings located west of Northport as shown in Figure 1. The facility will include either a floating dry dock or an alternative shiplift technology.

This report presents an independent review of the Beca concept design for the civils/marine infrastructure needed for the facilitation and operation of the proposed facility as well as consideration of the construction methodology and indicative construction works programme to support assessment of effects of the construction and use of this facility by others.

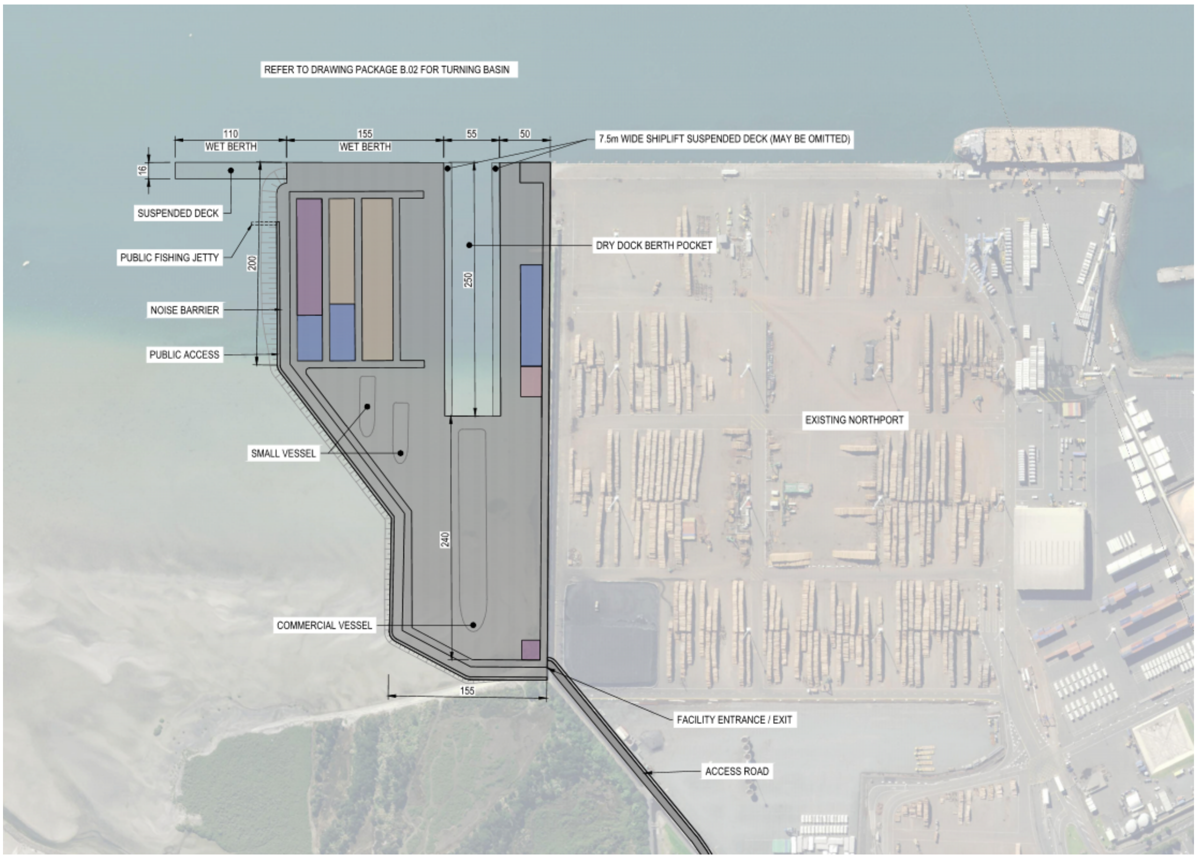

Figure 1: General arrangements for proposed development of Northland Shipyard and Dry Dock (figure source, Beca resource consent drawings)

1.1 Relevant reports & documentation

This report has been prepared based on the following reports and documentation:

- Beca, DRAFT Civil Engineering Resource Consent Drawings. Drawing numbers: 3237966-SK-0000 to 3237966-SK-4002, dated 08 May 2026.
- Beca, DRAFT Construction Staging Drawings. Drawing numbers: 3237966-SK-8001 to 8005, dated 27 May 2026.
- Beca, Draft Concept Design and Construction Report, dated 31 July 2026.
- Initia, Northport Western Reclamation Option – Concept Design Stage, Preliminary Geotechnical Report, Northport Limited, dated November 2019
- Beca, Addendum 1 Geotechnical Investigation Factual Report Marsden Point Port Development, Northland Port Corporation, dated 2000
- Beca, Geotechnical Investigation Factual Report Marsden Point Port Development, Northland Port Corporation, dated 1999
- Beca Carter Hollings & Ferner Ltd – Geotechnical Investigation for Proposed Marsden Point Port Development, dated August 1992

2. User Requirements

MBIE have provided the following key User Requirements (as taken from Beca report):

- Dredged pocket for a floating dry dock or alternative lifting device (shiplift). Dry dock/shiplift berth pocket design depth -16 mCD.
- A lifting device suitable to lift a vessel up to 230m in length with a displacement capacity 28,000 tonnes.
- A minimum distance of 50 m between Northport's existing berths and the entrance to the dry dock for safe navigation.
- A 265m long wet berth is required, to enable maintenance activities and vessel preparation prior to lifting. The wet berth is to accommodate vessels with maximum length overall (LOA) of 230m.
- Cranes/plant requirements alongside floating dry dock (one rail mounted electrical, or hybrid powered crane on each wing wall, capable of lifting 20 tonnes from the centreline on the floating dry dock).
- A system designed to transfer a vessel up to 200m in length to piled hardstand allowing multiple vessels to be worked on simultaneously.
- A suitable system to handle and store small vessels (e.g. white boats) up to 60m in length on the hardstand.
- Suspended deck adjacent to maintenance wet berth is to have capacity to support mobile harbour crane movements and lifting operations, as well as berthing and mooring forces.
- Maximum crest level of the finished hardstand level (approximately +5.7 m CD) to align with the adjacent Northport hardstand area.
- Heavy-duty pavement to enable mobile boat hoist to move white boats around hardstand working areas.
- Vessel handling systems (mules) to guide vessels into the dry dock pocket.
- A high-capacity winch at the centreline of the dock for hauling vessels into precise docking positions.
- To cater for high windage associated of large vessels in the dry dock the following bollards are to be provided for the dry dock:

  a) Standard bollards at 12 m spacing

  b) Storm bollards assumed to be at 45 m centres
- Fendering and bollards for the maintenance wet berth are to be designed for a range of vessels.
- An end wall to cater for mobile plant access to the dock floor.
- Quick connect couplings on the pocket wall attached with seismic bolts.

- Services to all berths including shore power, water (fresh and sea) and air supply (note this user requirement is outside of the scope of this report).
- Containment and treatment facilities for oily water, oil waste and wash water (note this user requirement is outside of the scope of this report).
- Salt-water firefighting pumps and potable water ring main (note this user requirement is outside of the scope of this report).
- Full utilities connections power, water, telecoms, etc. (note this user requirement is outside of the scope of this report).
- Buildings and other backland structures:
  a) Workshops and Warehousing / Covered storage areas
  b) Administration facilities
  c) Visiting ship Project office(s)
  d) Canteen(s)
  e) Ablutions and sickbay(s)
- Emergency services access.

MBIE have confirmed the following Design Criteria:

- The facility is to be designed to be IL2
- All marine/civil structures are to achieve 50-year design and 100-year durability life

3. Proposed Concept Design Solution

3.1 Proposed concept design

The concept design solution proposed by Beca includes enclosure of the reclamation area with a rock revetment to the south and west, and a combi-pile wall with anchors in the reclamation to the north and around the floating dry dock/shiplift berth pocket. It also includes piled structures with concrete deck for the jetty and suspended deck on each side to accommodate an alternative shiplift technology, refer to Figure 2 for locations of the different structures.

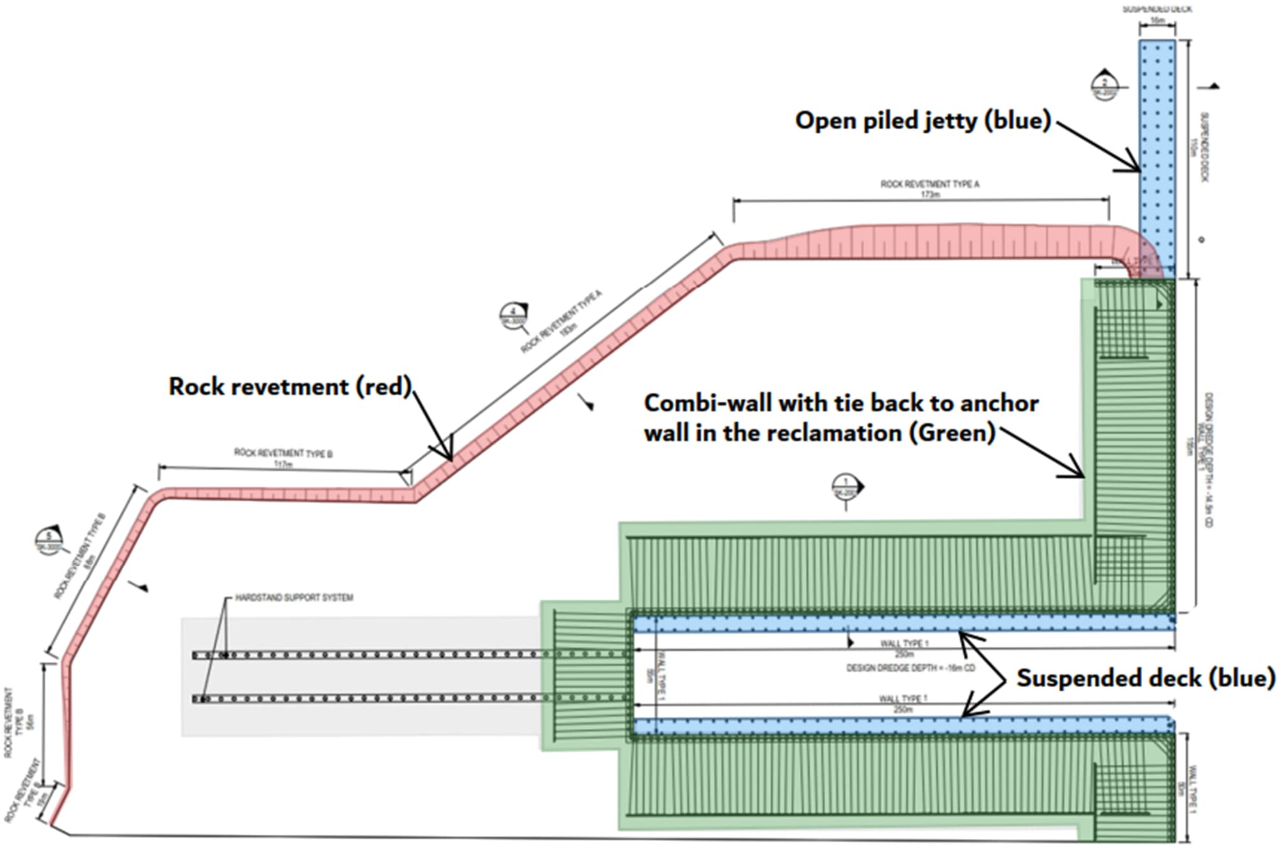

Figure 2: Proposed structures for the Northland Shipyard & Dry Dock facility

3.2 Proposed concept design for enclosure of the reclamation

The concept design solution proposed by Beca includes enclosure of the reclamation area with a rock revetment to the south and west and a combi-pile wall with anchors in the reclamation to the north and around the floating dry dock/shiplift berth pocket, refer to Figure 2 for locations of the rock revetment and combi-wall. A typical cross section of the revetment and combi-wall is shown in Figure 3.

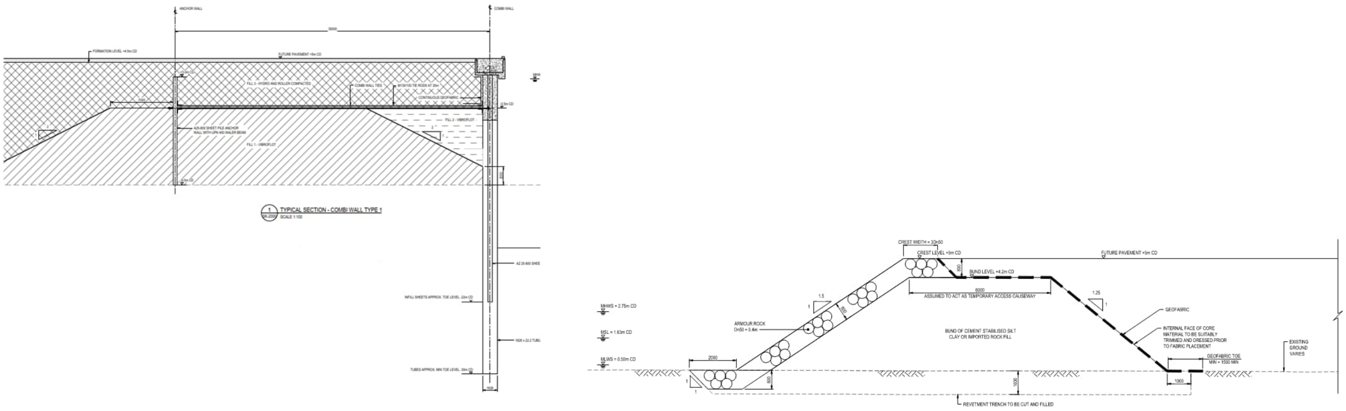

Figure 3: Typical combi-wall and revetment cross sections

3.3 Proposed concept design for jetty and suspended deck adjacent to shiplift

The concept design solution proposed by Beca includes an open piled structure with a concrete deck for the jetty located to the northwest of the dry dock facility and for the suspended deck on the eastern and western edges of the shiplift berth pocket, refer to Figure 2 for locations of the jetty and suspended deck. A typical cross section of the jetty is shown in Figure 4, note: the suspended deck associated with the shiplift comprises two piles per bent.

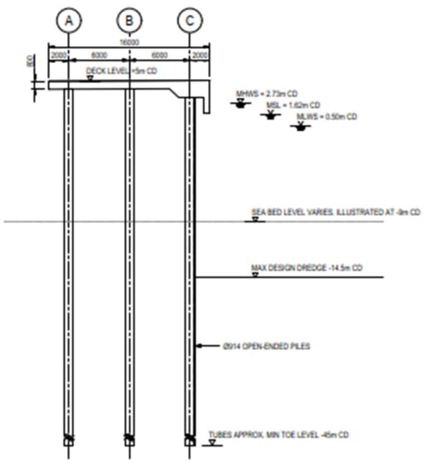

Figure 4: Typical jetty/suspended deck cross section

4. Consideration of Proposed Beca Conceptual Design

4.1 Design option considerations

Regarding the proposed reclamation:

- It is an efficient method of reclamation construction.
- The soils are contained on site along with any contaminants.
- The size and shape of the reclamation is based on the operational needs at the site. The size of the reclamation provides flexibility for detailed design.
- The effectiveness of vibro-compaction is greatest in clean sands. Silts and clays interbedded within the dredged soils may make vibro-compaction ineffective. As part of detailed design, we recommend investigations of the dredge area to confirm the suitability of dredged materials for vibo-compaction.
- Changes to the dredge and reclamation volumes through detailed design will impact the quantity of materials that need to be either disposed of off-site (if there is an excess of dredge material) or imported in the case that there is insufficient dredged material.
- Settlement may be greater and / or slower than anticipated. This may impact the construction programme, building foundations and pavements. Settlement risks will be managed as required.

Regarding the proposed revetment/bund structure to contain the reclamation:

- The proposed revetment/bund structure is a good solution as the geometry of the bund/revetment can be adjusted to respond to minor changes in User Requirements without necessitating a complete change in construction form.
- The proposed revetment/bund will achieve long term durability and resilience, and repairs can be made quickly when damaged in storms or earthquakes.
- It has a simple construction procedure.
- Cement mixing is not effective with sandy dredged soils. If the dredged material is not appropriate for cement mixing, material may need to be imported and used to construct the founding for the bund.
- The performance of the cementitious bund is sensitive to the local variability in the dredged material. As part of detailed design, we recommend investigations of the dredge area to confirm the suitability for use in the cementitious bund. If the dredged material is not appropriate for the cementitious bund, material may need to be imported.

- The seismic performance of the bund will be investigated during detailed design. Recommend at concept design that the proposed buildings near to the cementitious bund are piled.
- The depth of undercut required below the bund for stability is uncertain and could affect dredge and reclamation volumes.

Regarding the proposed open piled jetty and piled suspended deck:

- An open piled suspended deck structure is a good solution for the jetty and deck adjacent to the shiplift as this structural form can support large axial load demands.
- The geometry can be adjusted to respond to minor changes in User Requirements without necessitating a complete change in construction form.
- Durability requirements can be readily addressed using proven technologies.
- It is a simple construction procedure and easy verification of geotechnical capacity.
- Additional geotechnical investigations are required prior to detailed design to better estimate founding depth for the piles to achieve the required axial capacity.
- Hard founding (rock) is not expected to be found at the site. Piles can be readily lengthened to achieve required load capacity.
- Can be detailed to be resilient in earthquakes

Regarding the proposed combi-wall to contain the northern extent of the reclamation and surrounding the dry dock berth pocket:

- The combi-wall is a good solution as it enables vessels to berth alongside the wall whilst containing the dredged reclamation material behind the wall.
- The geometry can be adjusted to respond to minor changes in User Requirements without necessitating a complete change in construction form.
- Durability requirements can be readily addressed using proven technologies
- Has a simple construction procedure

Based on our review, the concepts for reclamation and marine structures are an appropriate and viable engineering solution.

4.2 Design risks, consequences and potential mitigation measures

Table 1 lists potential design risks and mitigation measures relevant to the overall project. WSP recommends that these are considered.

Table 1: Summary of design risk, consequences and mitigation measures

Risk Description	Risk consequence	Risk management options
Size or configuration of combi-wall	Need to increase pile sizes or tie-rod lengths once further investigations, and preliminary design is carried out	Additional capacity for the piles can be obtained by any or a combination of the following: A) Inc. grade of steel of pile B) Inc. wall thickness of pile C) Dec. spacing of piles (i.e. more tubular piles) and use smaller sheetpiles D) Move dead man back and make larger with larger tie rods E) Concrete fill piles
Deflection of Combi-wall is larger than anticipated	Displacement of Combi-walls impacts operations	Investigate and assess during detailed design, detail structures to accommodate movement, monitor movements during construction.
Effectiveness of ground improvement	Material not ideal for Vibroflot compaction due to presence of Clay/Silt material Mudcrete not feasible for bund construction	Consider alternative ground improvement methods, verify performance through field testing and monitoring
Ability to place and compact fill above anchors and below water	Heavy compaction applied to compact lower lifts of 'Fill 3' damaging anchors	Provide adequate cover to anchors, controlled fill construction, monitoring
Larger or slower rate of settlement than expected	Settlement is larger than expected or takes longer causing delays to construction or increased maintenance post construction	Investigate and assess in the next phase, include element to reduce settlement time, monitor during construction and adjust as required.
Corrosion	Corrosion rate higher than expected	Adopt ICCP system, as well as appropriate steel grades and corrosion protection systems for steel above MLW. Consider conservative corrosion allowances for steel above MLW even with corrosion protection.

Risk Description	Risk consequence	Risk management options
Climate change effects	Higher sea and groundwater levels, increased frequency of storms, increased microbial corrosion	Include sea level rise and more severe storm/loading scenarios in design, allow corrosion protection and provide monitoring/adaptive management provisions for long-term performance
Increased width of dry dock due to supplier requirements	Maximum extent of reclamation in FTAA application is exceeded on the basis that all increases will need to extend westwards to maintain the minimum distance of 50m between Northport's existing berths and the entrance to the dry dock for safe navigation.	Allow for the maximum reclamation extent on the western edge, to accommodate all hardstand and building areas west of the dry dock and allow margin for increased width of dry dock

5. Existing Geotechnical Information

Geotechnical site investigations and laboratory tests were carried out during 1983, 2000, and 2019 around the proposed development (refer to the relevant reports listed in Section 1.1). The geotechnical investigations and laboratory testing are summarised below, with the site investigations shown in Figure 5 and borehole information in Table 2.

- 20 boreholes with standard penetration test (SPT) data
    a. 6 boreholes from 2019
    b. 6 boreholes from 2000
    c. 8 boreholes from 1983 (only selected boreholes)
- 39 particle size distribution (PSD) in 1983 from boreholes (01 – 06, 08, and 11)
- 4 PSD in 2000 from MPT 29 borehole
- 15 wet PSD, 9 only for fines content and 1 for hydrometer in 2019 from boreholes (01 – 06)
- 07 Atterberg's in 2019 from boreholes (01 – 05)
- 07 Atterberg's in 1983 from boreholes (01, 03 – 06, and 08)
- 04 Oedometer in 1983 from boreholes (01, 03, 05, and 06)
- 02 consolidated undrained triaxial test in 1983 from boreholes (05 and 08)

Cone Penetration Testing (CPT) was attempted at the site; however, testing encountered difficulties and did not produce reliable results. We understand that further geotechnical investigation is planned as part of the detailed design process.

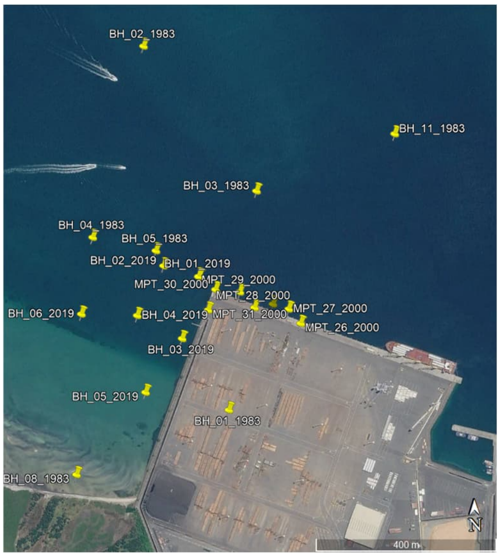

Figure 5: Site investigation plan at the proposed Northland Shipyard & Dry Dock facility location

Table 2: List of boreholes close to the proposed Northland Shipyard & Dry Dock location

Borehole ID	Easting (m)	Northing (m)	Elevation (Chart Datum, mCD)	Depth (below ground level, m)
BH_01_2019	274561.764	816089.4	-12.8	19.15
BH_02_2019	274484.264	816116.01	-11.7	19.95
BH_03_2019	274523.183	815954.67	-6.7	17.7
BH_04_2019	274424.215	816006.9	-6.2	18.45
BH_05_2019	274440.279	815837.14	0.3	22.95
BH_06_2019	274300.096	816011.66	-4	10.95
MPT_26_2000	274787	815985	-7.6	22.41
MPT_27_2000	274761	816017	-7.8	24.45
MPT_28_2000	274685	816021	-7.1	25.42
MPT_29_2000	274653	816056	-7	25.93
MPT_30_2000	274599	816062	-7.3	27.45
MPT_31_2000	274582	816017	-7.8	25.92
BH_01_1983	274624.5	815795.9	0.46	17.46
BH_02_1983	274446.7	816604.5	-7.9	8
BH_03_1983	274693.6	816279.33	-8.1	11
BH_04_1983	274327.61	816180.99	-4.9	14.5
BH_05_1983	274467.77	816149.57	-5.4	30
BH_06_1983	274726.01	816028.46	-6.1	23.5
BH_08_1983	274284.6	815658	2.15	17
BH_11_1983	275003.61	816402.36	-8.13	12.47

6. Geotechnical Conditions

6.1 Geological Sections

Geological sections were developed by WSP from the available site investigation data across the project area. Two sections were selected for independent geotechnical analysis: an east-west transverse section across the proposed Northland Shipyard and Dry Dock facility and a north-south section. These sections are generic representations of the ground conditions only. A more detailed and comprehensive ground model will be required for detailed design.

The engineering ground model has been interpreted in the context of previous and recent work by Beca (1983, 2000 and 2026) and Initia (2019). While it reflects all available information, the sections remain an interpretation rather than a definitive representation of subsurface conditions.

6.2 Ground Water

Due to proximity of the Northland Shipyard and Dry Dock infrastructure to the sea, a groundwater table consistent with the tidal level has been assumed. The mean sea level of approximately 1.63 mCD has been adopted to represent the groundwater conditions for the calculations. The groundwater levels are outlined in Table 3. Chart datum of the tidal levels is provided in the table are from LINZ (https://www.linz.govt.nz/guidance/marine-information/tide-prediction-guidance/standard-port-tidal-levels) at Marsden Point, last updated 3 June 2025. Potential effects of global warming and climate change along with tidal lag have not been considered in this assessment.

Table 3: Long-term groundwater levels

Tidal level	on June 2025 (mCD)
Mean sea level (MSL)	1.63
Highest astronomical tide (HAT)	3.04
Mean high water spring (MHWS)	2.74
Lowest astronomical tide (LAT)	0.17
Mean low water spring (MLWS)	0.51

6.3 Geological Units

The geological materials have been categorised into several project engineering units for this analysis, depending on their likely geotechnical properties.

Reclamation Fill

The Reclamation Fill is planned to be sourced from dredging in the turning basin, drydock berth pocket and imported fill, if required. This material consists of loose sand with some shells, which ranges from loose to dense. Three types of Reclamation Fill are proposed, as shown in Figure 6 and described below:

- Fill 1 and Fill 2 is expected to comprise dredged shelly sand placed behind the main sheet pile wall and densified using Vibroflot compaction. If the dredged sand contains a high fines content, suitable imported material may be required because Vibroflot compaction may not achieve the required densification.
- Fill 3 consists of dredged shelly sand placed as hydraulic fill and compacted with rollers above water level. This fill forms the upper reclamation layer above the tie rod level and is placed up to the finished platform level.

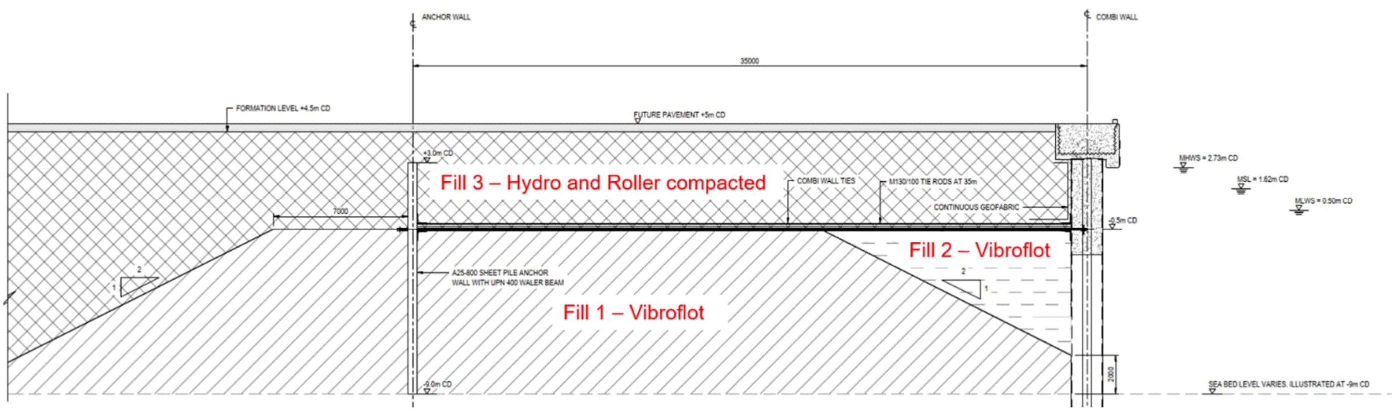

Figure 6: Typical Reclamation Fill cross-section

Marine Deposits

Marine deposits in the vicinity of the proposal comprise the seabed deposits (including Upper Loose Sand), and Upper Sandy Silt-Clay and Lower Dense Sand and Lower Silt-Clay.

Upper Loose Sand is fine to medium uniform sand with some shells, ranging from loose to very dense, and typically medium dense with SPT ‘N’ values ranging from 0 – 35 with an average of 11. This unit generally occurs immediately below the seabed, varies in thickness from 0 – 12 m and is expected to be dredged from the turning basin and dry dock areas and turned into the Reclamation Fill.

Upper Sandy Silt-Clay underlies the Upper Loose Sand and is comprised of a fine-grained layer with silty clay, sandy clay, sandy silt, and silt-clay and typically, stiff to very stiff. The SPT N values ranges from 0 – 35 with an average of 8. This unit generally occurs immediately below the Upper Loose Sand and varies in thickness from 1 – 7 m.

Lower Dense Sand is generally medium dense to very dense, with SPT N values ranging from 13 to 50+, and an average of about 40+. This unit is typically 1 m to more than 20 m thick and appears to be more prominent toward the north-west of the Northland shipyard and dry dock facility area. However, further geotechnical investigations should confirm the full extent of this layer.

Lower Silt-Clay is a fine-grained unit comprising silty clay, sandy clay, and sandy silt. Limited site investigation data are available for this layer, so its extent is currently not well defined. It was encountered in only two boreholes, BH_05_2019 and MPT_27_2000, where SPT N values greater than 35 indicate a stiff to very stiff layer. Beneath the Northport area, this unit is interpreted to be typically very stiff to hard.

Bedrock

The bedrock at Marsden Point is typically deep. It is inferred to consist of old greywacke. Elevated greywacke and argillite of the Waipapa Group outcropping to the north and south of the Whangarei harbour. Within the site boundary, bedrock was not encountered during borehole investigations drilled up to -50 mCD.

6.4 Material testing

This section presents a summary of the results for the in-situ and laboratory tests undertaken from the various investigations discussed in Section 5. The laboratory tests and results are presented in previous reports (refer to Section 1.1).

Because WSP did not receive the original digital data, it has necessarily relied on available data digitised from figures in PDF reports for various laboratory testing.

Particle size distribution

Summarised results for particle size distribution (PSD) samples are shown in Table 4 below, classified by engineering unit.

Table 4: PSD test results summary

Engineering Unit	Number of samples per investigation		Typical particle distribution range				
			Gravel (%)	Sand (%)	Silt and Clay (%)	Silt (%)	Clay (%)
Upper Loose Sand	2019	14	0-2 (0)	53-96 (83)	2-47 (16)	-	-
	2000	4	0-1 (1)	48-88 (74)	12-51 (25)	-	-
	1983	17	0-24 (2)	66-97 (87)	2-32 (10)	5-20 (14)	5-12 (10)
Upper Sandy Silt-Clay	2019	1	0	52	48	31	13
	2000	-	-	-	-	-	-
	1983	11	0-8 (2)	23-60 (41)	40-75 (56)	22-47 (33)	18-34 (23)
	2019	10	0-5 (1)	79-96 (89)	4-19 (10)	-	-

Engineering Unit	Number of samples per investigation		Typical particle distribution range					
			Gravel (%)	Sand (%)	Silt and Clay (%)	Silt (%)	Clay (%)	
Lower Dense Sand	2000	-	-	-	-	-		
	1983	10	0-32 (8)	60-95 (79)	4-40 (13)	3-30 (15)	2-13 (7)	
Lower Silt-Clay	2019	-	-	-	-	-	-	
	2000	-	-	-	-	-	-	
	1983	1	0	10	90	60	30	

Atterberg Tests

Atterberg classification test results from 2019 and 1983 (only selected boreholes) investigations are summarised in Figure 7. A total of 14 samples were tested, with 2 sample returning non-plastic (NP) results for Upper Loose Sand. The results show that the Upper Sandy Silt-Clay are generally of high plasticity. Upper Loose Sand exhibits some variability in plasticity behaviour reflecting nature of marine deposits; however, overall Upper Sandy Silt-Clay and Lower Silt-Clay remain high plastic.

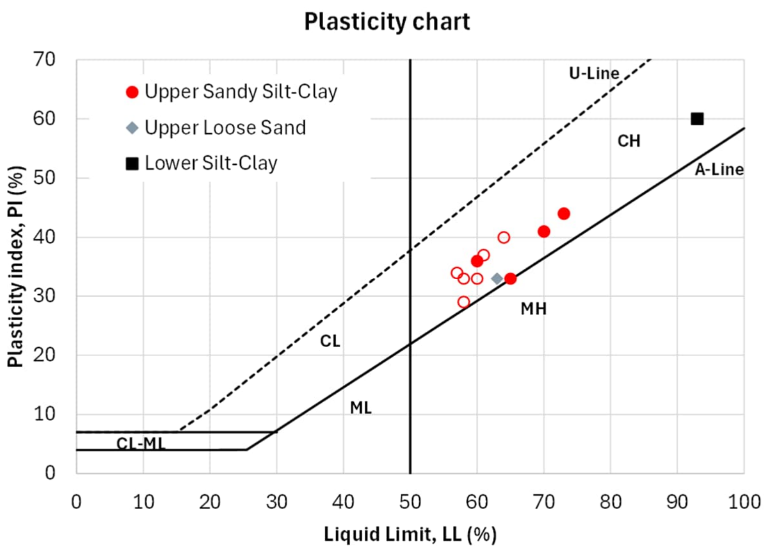

Figure 7: Summary of Atterberg test results. Solid markers represent data from the 1983 investigation, and open markers represent data from the 2019 investigation.

Consolidated-Undrained (CU) triaxial test

Results from 2 No. consolidated-undrained (CU) triaxial tests to inform effective stress parameters are provided in Table 5 below.

Table 5: CU triaxial test results summary

Borehole	Test depth (m CD)	Engineering Unit	Drained Cohesion, c' (kPa)	Friction angle, ϕ' (°)
BH_01_1983	-11.04	Upper Sandy Silt-Clay	30	33
BH_05_1983	-19.4		60	32

Oedometer Tests

Four No. oedometer tests were undertaken during 1983 site investigation (only selected boreholes). Three of the four tests were from Upper Sandy Silty-Clay and 1 was for the Upper Loose Sand to inform consolidation behaviour and settlement parameters. Summary results are shown in Table 6.

Table 6: Oedometer consolidation test results summary

Borehole	Test depth (m CD)	Engineering	pre-consolidation pressure, P'c (kPa)	Cc	Cs	e_0
BH_01_1983	-11.04	Upper Sandy-Silt Clay	500-600	0.59	0.02	1.2
BH_05_1983	-17		800-1000	0.48	0.05	1.2
BH_06_1983	-17.3		300-400	0.41	0.05	1.26
BH_03_1983	-4.1	Upper Loose Sand	-	0.20	0.007	0.98

Standard Penetration Test (SPT)

SPT N-field blow counts were generally recorded at 1.5 m intervals in the boreholes. These values were corrected assuming a hammer energy efficiency of 72 %, as the drilling contractor's hammer energy efficiency was not reported in the bore logs. A summary of the SPT N60 blows encountered during site investigations for each engineering unit is presented in Figure 8.

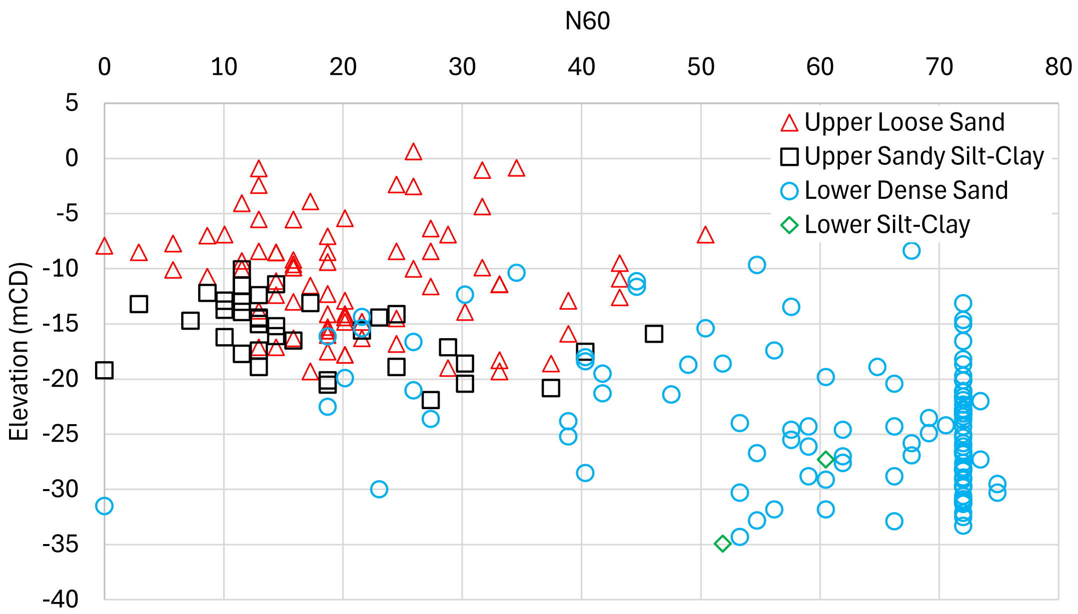

Figure 8: SPT N60 results from selected boreholes grouped by unit

6.5 Geotechnical Design Parameters

The geotechnical design parameters in Table 7 have been interpreted and developed based on in-situ testing data, laboratory testing data (refer to the relevant reports listed in Section 1.1), empirical correlations, and engineering judgment. Where available, empirically derived parameters have been validated against direct measurements and laboratory results. Interpreted settlement parameters for Upper Sandy Silt-Clay have also been provided separately in Table 8.

Further details of the selection and derivation of the design parameters are discussed below. Unit descriptions for adopted geotechnical design parameters are provided for each layer in the previous subsections.

Bulk Unit weight, γ – The unit weights are based on laboratory tests where available and typical published ranges.

Plasticity index, PI – Plasticity index (PI) is based on Atterberg tests and general soil behaviour where tests have not been performed.

SPT N_{60} – Refer to previous section.

Friction angle, ϕ – The friction angles are based on laboratory CU triaxial tests and SPT based correlation published by Wolf (1989). These values were compared against typical published ranges from Look (2007).

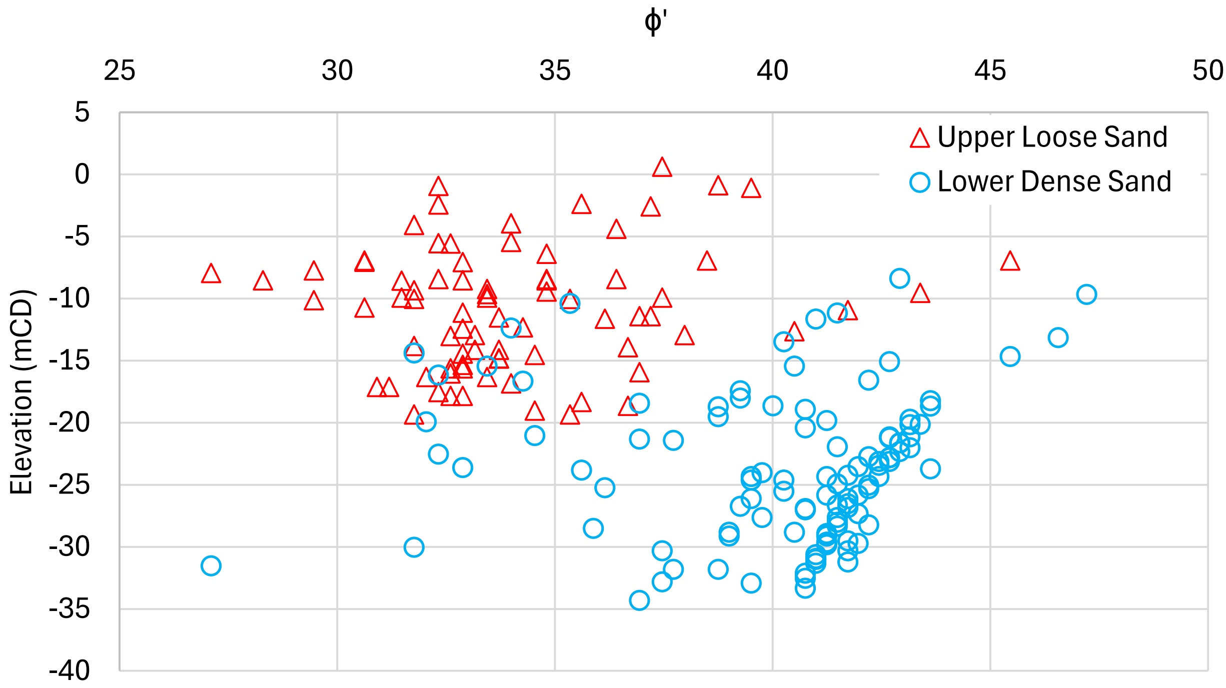

Figure 9: Friction angle derived from SPT N value using Wolf (1989) correlation

Undrained shear strength, Su – Undrained shear strength values have been derived from SPT interpretations and engineering judgment. These values were compared against typical published ranges from Look (2007). No CPT or shear vane testing carried out for the Upper Sandy Silt-Clay during the investigations, so Su of 40 kPa and 60 kPa have been adopted for Upper Sandy Silt-Clay and Lower Silt-Clay, respectively. A conservative value has been selected because Upper Sandy Silt-Clay is expected to be critical for settlement and lateral deformation, which could increase structural demands on the Combi-wall and the Deadman.

Shear wave velocity, Vs – No direct shear wave velocity measurements were carried out at the site; therefore, shear wave velocities were estimated from SPT using the correlations proposed by Kwak et al (2015). The average shear wave velocities are tabulated for the different units in Table 7.

At-rest earth pressure coefficient, K_0 – K_0 was derived using Jaky's equation $1 - \sin(\phi)$ (1944) considering normally consolidated soils, except for the Upper Sandy Silt-Clay which have been assessed as moderately to slightly overconsolidated with a corresponding K_0 of 1.0.

Permeability, k – Permeability is based on laboratory testing where available (e.g. Upper Sandy Silt-Clay) and typical published ranges.

Settlement Parameters – Interpreted settlement parameters for the compressible Upper Sandy Silt-Clay layer are shown in Table 8. The parameters are generally based on laboratory testing discussed in previous section, with numbers compared to typical published ranges.

Table 7: Design geotechnical parameters

Engineering Unit	γ (kN/m³)		SPT N60	PI	c/Su (kPa)	ϕ (°)	Vs (m/s)	K_0	E_{50}^{REF} (kPa)	E_{OED}^{REF} (kPa)	E_{UR}^{REF} (kPa)	OCR	k (m/day)
	γ_{unsat}	γ_{sat}											
Fill 1	16	18	-	NP	1	35	-	0.43	15000	15000	45000	1	0.864
Fill 2	16	18	-	NP	1	35	-	0.43	12000	12000	36000	1	0.864
Fill 3	16	18	-	NP	1	38	-	0.38	20000	20000	60000	1	0.864
Upper Loose Sand	15	17	5-20	NP	1	32	190	0.47	12000	12000	36000	1	0.864
Upper Sandy Silt-Clay	14.5	16.5	5-20	40	Su = 40	-	215	1	5000	3000	14550	1	2.419E-4
Lower Dense Sand	16.5	18.5	30-50+	NP	1	35	330	0.43	30000	30000	90000	1	0.8640
Lower Silt-Clay	15	17	-	40	60	-	-	1	7000	6000	21000	1	2.419E-4
Armour Rock	21	23	-	NP	10	42	-	-	E’=460E3	-	-	1	864
Cementitious Bund	21	23	-	N. A	Su = 500	-	-	-	E’= 50E3	-	-	1	2.419E-4

Table 8: Index and Settlement parameters for Upper Sandy Silt-Clay

Parameter	Recommended range
Natural water content	37 – 66 (46)
Liquid limit (%)	52 – 73 (62)
In-situ void ratio	1.2
Over-consolidation ratio, OCR	Short term > 4 (Overconsolidated Clay) Long term 1 (normal to slightly overconsolidated clay)
Coefficient of consolidation, Cv (m^2/yr)	20 – 60
Coefficient of volume compressibility, mv (m^2/MN)	0.2 – 1.5
Compression index, C_c	0.4 – 0.59
Coefficient of secondary compression, C_s	0.02 – 0.05

The parameter values are considered appropriate at this stage, based on the information available. Further investigations and laboratory testing will be undertaken in subsequent stages of the project to inform engineering soil properties for detailed design.

7. Geohazards

The following geohazards have been identified at the site and are relevant to the concept design. The consequences of these geohazards are discussed in this section.

7.1 Liquefaction and Cyclic Softening

Liquefaction and cyclic softening of the younger materials at the site is expected under seismic shaking, resulting in significant consequences for soil strength, stiffness, and lateral/vertical soil displacements. Independent liquefaction and cyclic softening assessments have been undertaken by WSP based on site-specific data and laboratory tests outlined in the previous reports, summarised in Section 6.

Liquefaction Assessment Methodology

The liquefaction susceptibility assessment was based on the Plasticity index from Atterberg limit tests, where PI ≤ 12 is susceptible to liquefaction.

The liquefaction triggering assessment is based on the SPT-based method following Boulanger & Idriss (2014; 2008) in CLiq version 3.5.4.6. The SPT N blow counts and fines contents were determined from the available particle size distributions and borehole log descriptions.

Cyclic Softening Assessment Methodology

Upper Sandy Silt-Clay layer can undergo cyclic softening, as they were not found to be susceptible to liquefaction due to high PI and fines contents. CPT and Cyclic triaxial testing may be required prior to preliminary design to assess the cyclic softening behaviour of this layer.

7.2 Assessment Results

An example assessment using the methods outlined in Section 7.1 shown in Figure 10 for BH_04_2019 SPT blow counts. Upper Loose Sand layer is predicted to be liquefiable and is generally consistent across different previous reports.

The extent of potential liquefaction and its consequences will require detailed cyclic testing on undisturbed samples and investigation at the detailed design stage.

For this assessment, a mean magnitude of 6.0 and a peak ground acceleration of 0.12 g for 1/500 annual exceedance probability have been adopted based on disaggregation results from the 2022 National Seismic Hazard Model (NSHM22) for Whangārei, the closest location available on the NSHM web tool.

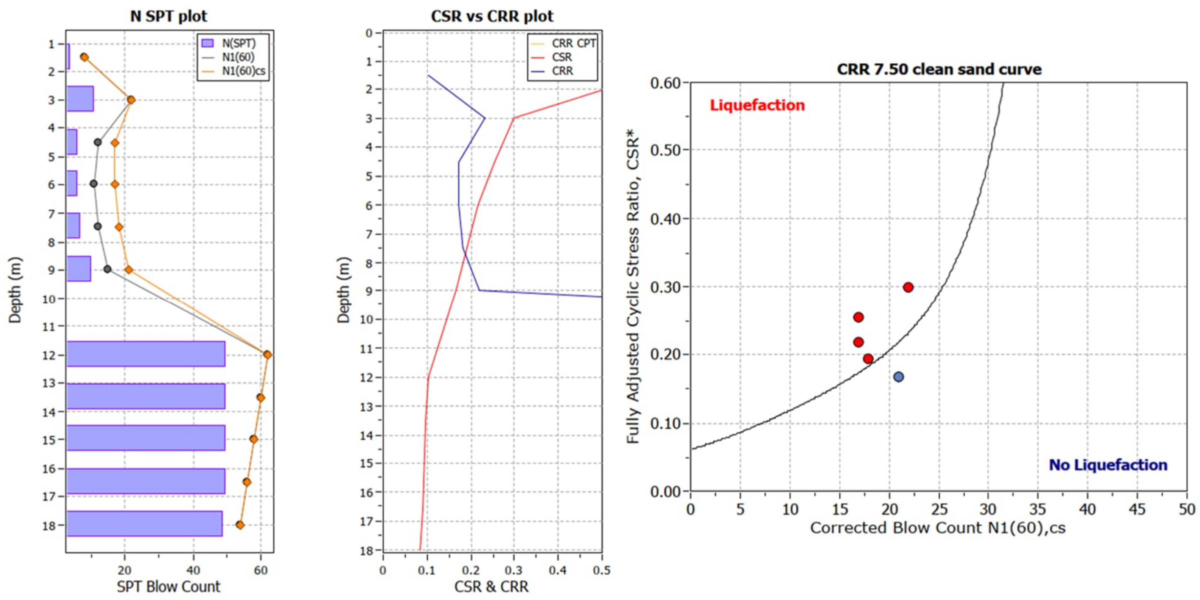

Figure 10: Summarised example of liquefaction in Upper Loose Sand layer for borehole BH_04_2019

Table 9: Liquefaction and cyclic softening hazard assessment summary

Unit	Susceptible to liquefaction/cyclic softening	Liquefaction/cyclic softening	Comment
Upper loose sands	Susceptible to liquefaction	Liquefaction triggers in approx. 1/250 y earthquake.	Recommend CPT testing to assess extent and treatment required
Upper Sandy Silt-Clay	Not susceptible to liquefaction Potentially Susceptible to cyclic softening	Cyclic softening potential undetermined due to lack of information.	Recommend cyclic triaxial testing and CPT to assess behaviour in earthquakes.
Lower dense sands	Susceptible	Not expected to trigger in a 500 y return period earthquake.	Unit is too dense to liquefy.
Lower Silt-Clay	Not Susceptible	N/A	Cyclic softening is not expected as this layer very stiff and deep

8. Assumed Construction Sequence

Beca's conceptual design report states that the overall construction period for the Northland Shipyard and Dry Dock facility would be approximately 3 years from mobilisation to completion of the civil infrastructure work. The sequence set out below is based on Beca's construction methodology described in their report (refer documents in Section 1.1) and Construction Staging drawings (refer documents in Section 1.1).

- Initial containment and reclamation to create construction compound area in southern corner of site.
- Extend the rock revetment and bund construction along the western extent of the reclamation.
- Construct the Combi-wall along the northern face, including around the dry dock berth.
- Place Reclamation Fill 1 from the turning basin within the reclamation area and densify it using vibroflot compaction.
- Construct the Deadman from +3.0 mCD to -9.0 mCD.
- Install 35 m long tie rods between the Combi-wall and the Deadman (no dredging prior to this stage).
- Place Fill 2 and dredge the dry dock to -16 mCD or to -14.5 m along the wet berth and densify it using vibroflot compaction
- In the final stage, place Fill 3 to +4.5 mCD and use hydro and roller compaction
- Construct the capping beam over the Combi-wall and foundation layer to achieve the final level of +5.0 mCD.
- Install piles to support the runway beams behind the dry dock
- Construction concrete runway support beams in hardstand behind the dry dock
- Install piles for jetty and suspended deck adjacent to shiplift
- Remove material from inside piles
- Concrete fill piles and construct suspended concrete deck
- Install wharf furnishings and services
- Construct utilities and pavements in backlands
- *The construction sequence/staging for the buildings is noted by Beca to either overlap with the construction sequence above or occur after completion of the infrastructure. The construction of the buildings and required foundations for the buildings is outside the scope of this report.*

The above construction methodology is considered reasonable and viable. We understand that the Beca methodology anticipates that the above construction methodology will be undertaken from water-based plant, e.g. jack-up barges where work is unable to be undertaken from land. For the purposes of the FTAA application, we recommend the application includes an allowance for temporary piled bridge staging to enable an alternative construction option for the contractor.

8.1 Pile installation

The estimated pile depths shown in the Beca concept design drawings are listed in Table 10. The estimated pile toe depths provided are around the pile toe depths that we would expect for the expected demands and based on available geotechnical information.

Table 10: Pile size and estimated toe depth from Beca conceptual design

Structure	Pile Size (Beca conceptual design)	Estimated pile toe depth (Beca conceptual design)
Combi-wall	1624 x 22 mm open ended tubular piles	-30 m CD
Jetty piles	914 mm open ended tubular piles	-45 m CD
Piles supported suspended deck adjacent to shiplift	914 mm open ended tubular piles	-4 5m CD

Based on the SPT data, refer to Section 6.4, and the estimated toe depths provided in the Beca conceptual design report, WSP expect the 1600 mm and the 900 mm piles may need to be driven with an impact hammer below -20 m to -25 m CD. On this basis, whilst Vibro hammer is expected to be used, Impact hammer driving is expected to be required at the end of driving for most combi-piles and to drive the 914 mm piles from -20 m/-25m CD to -45 m CD for the jetty and suspended deck adjacent to the shiplift. A detailed drivability assessment using wave analysis should be carried out in future stages.

9. Independent Conceptual Geotechnical Analysis for Combi-wall

9.1 Combi-wall structural properties

The Combi-wall supporting the reclamation fill will comprise 1,626 mm diameter tubular steel piles with a wall thickness of 22.2 mm, combined with AZ 25-800 sheet piles with a steel grade of 355 MPa. The tubular piles will be open-ended and driven from the top. Table 11 represents the structural properties used in the WSP independent analysis.

The primary piles will be installed at 3.2 m centre-to-centre spacing. The system will be tied back with 35 m long tie rods connected to Deadman wall formed from AZ 25-800 sheet piles with a steel grade of 355 MPa. No corrosion allowance, reduction factor, or fatigue effects have been included in the structural properties; these factors would reduce the capacity of the structural elements. In addition, no allowance for dredge or scour below the design dredge depth is considered.

The typical geometry of the Combi-wall section along with Reclamation Fill information is provided in Figure 7. The top of the Lower Dense Sand layer has been identified as a suitable founding stratum for the driven Combi-wall at approximately -30 mCD, where the soils comprise very dense sands. Given the variability in ground conditions, provision should be made to extend pile lengths during construction if required.

Table 11: Summary structural properties

Component	Material type	w (kN/m/m)	EA (kN/m)	EI (kN m^2/m)	Mp (kNm/m)	$L_{spacing}$ (m)	| Fmax, tens| (kN)
Combi-wall	Elastoplastic	3.22	9.0E6	2.35E6	4880		
Deadman	Elastoplastic	1.5	3.423E6	124.8E3	888.0		
M130/100 Tie Rod	Elastoplastic		1.600E6			3.2	4791

9.2 Numerical Modelling

Finite element model was developed in using Plaxis 2D (v.2024.1) and utilized to calculate vertical and lateral displacements of ground, structural demands on Combi-wall and Deadman, and the settlement of the ground during different phases of the construction. A sea level of 1.63 m mCD (mean sea level) is assumed for the base analysis. An unfactored surcharge of 25 kPa is applied in the analysis based on the Beca concept design report (Beca, 2026).

A general non-linear elasto-plastic constitutive model with stress-dependent stiffness have been (hardening soil (HS) used to model the soil layers in the Plaxis numerical analyses except of Cementitious Bund and Armour rock which have been modelled using Mohr Coulmb. The combi-wall and deadman have been modelled using the 'Plate element' structural model and the tie-rods have been modelled as node-to-node anchor elements with two-node elastic spring elements with a constant spring stiffness.

Construction of the proposed Northland Shipyard and Dry Dock reclamation enclosure components are expected to be completed over approximately 1.5 years. The construction sequence adopted in the numerical analysis is described in Section 7.

The engineering ground model has been developed from the available site investigation data and interpreted in the context of previous work by Beca (1983, 2000, and 2026) and Initia (2019). Although it reflects all available information, it remains an interpretation rather than a definitive representation of subsurface conditions. The final phase of the east-west transverse section model is shown in Figure 11 and north-south long section in Figure 12 below.

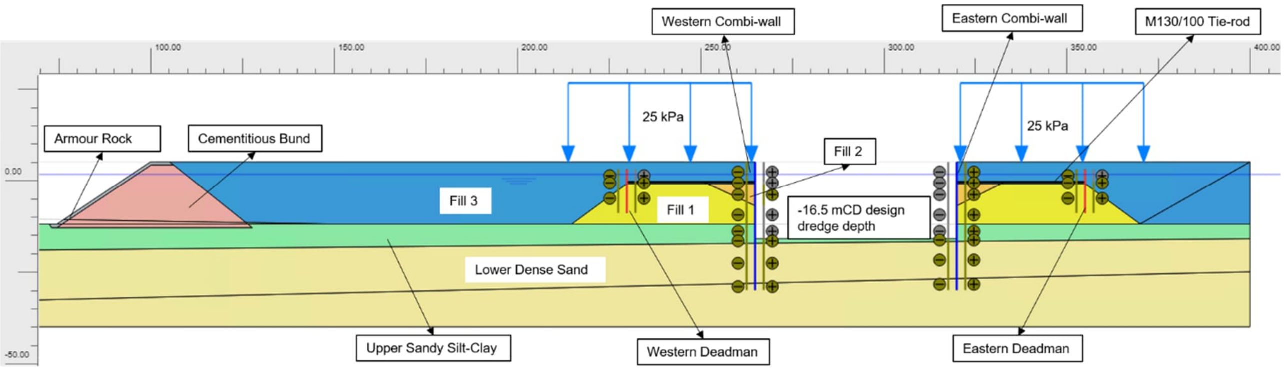

Figure 11: Example of east-west transverse section Plaxis model set up

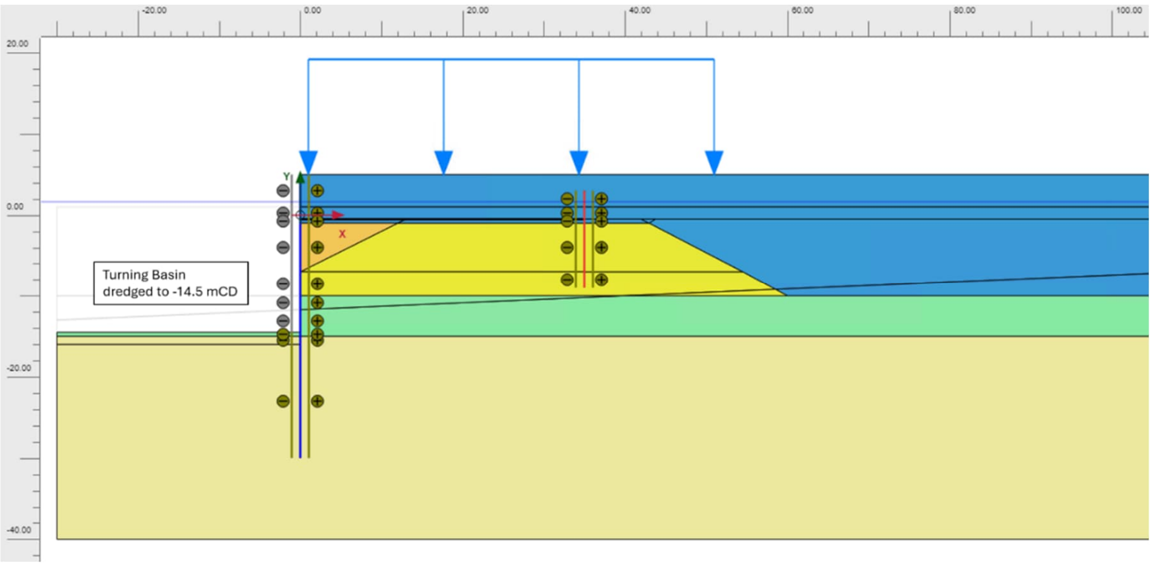

Figure 12: Example of north-south long section Plaxis model set up

The results of the PLAXIS static analysis for east-west transverse section are provided in terms of vertical and horizontal displacement of the ground (Figure 13 and Figure 14) as well as structural response such as lateral displacements (Ux), bending moment (M), shear force (Q) along the Combi-wall and Deadman (Figure 15 and Figure 16). The displacement contours from the static analysis for the north-south long section are presented in Figure 17 and Figure 18. Structural response, including lateral displacement (Ux), bending moment (M), and shear force (Q) in the Combi-wall and Deadman, is shown in Figure 19 and Figure 20.

Calculated total vertical displacements from the construction of the reclamation fill and drydock are anticipated up to 400-700 mm behind the Combi-wall and Deadman. Much of the settlement will occur during the construction of Combi-wall and reclamation. The calculated total horizontal displacements at the top of the wall are expected to 200-400 mm.

The factored structural demands on the Combi-wall and Deadman are within capacity when corrosion is not considered. However, allowing for corrosion and other factors could significantly reduce that capacity. We note that Beca’s conceptual design report states that a cathodic protection system and corrosion protection system where CP is not effective is expected to be included in the final design.

The rate of consolidation of the firm Upper Sandy Silt-Clay layer and magnitude of long-term settlement is uncertain and may affect demands on the walls structural elements and maintenance requirements for pavements and buildings constructed on the reclamation. Further investigation and laboratory testing should be carried out to assess settlement and develop settlement management strategies in detailed design.

No seismic analysis has been undertaken by WSP as part of this independent analysis. However, with ground improvement and compaction to prevent liquefaction of the loose sands and the reclamation fill, we expect the wall can be detailed to meet ULS requirements in a 1/500 AEP earthquake.

Based on the checks we have made, the proposed combi pile walls and reclamation construction are an appropriate and viable engineering solution.

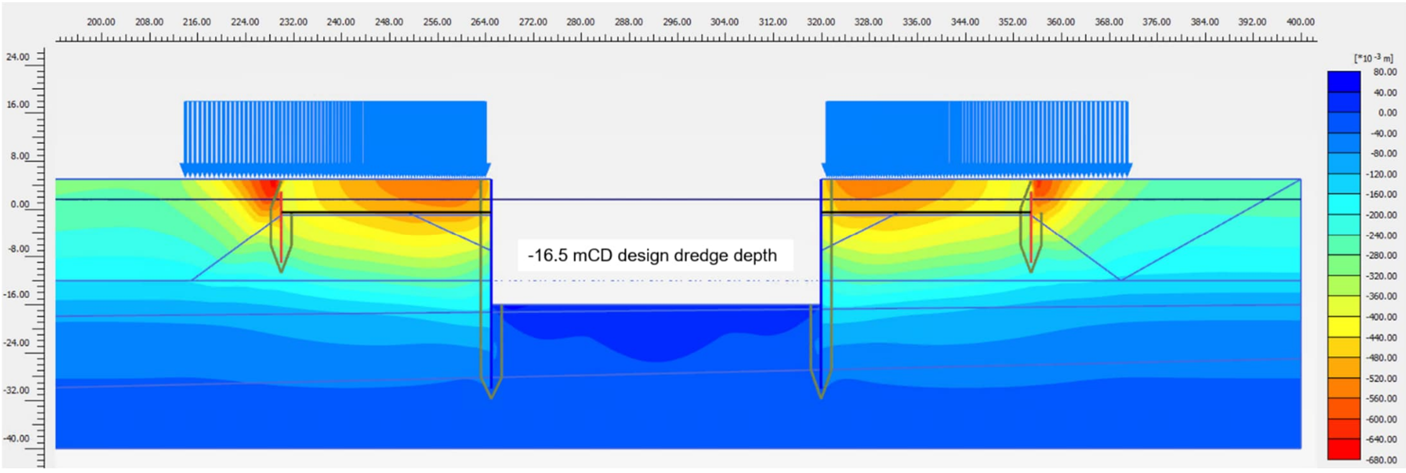

Figure 13: Example of east-west transverse section vertical displacements

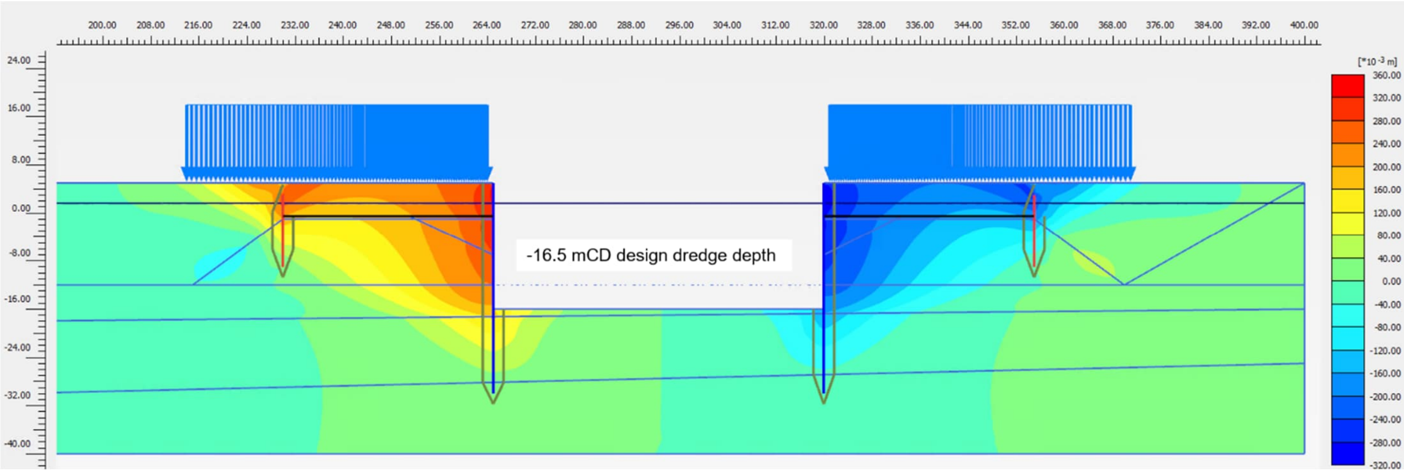

Figure 14: Example of east-west transverse section total horizontal displacements

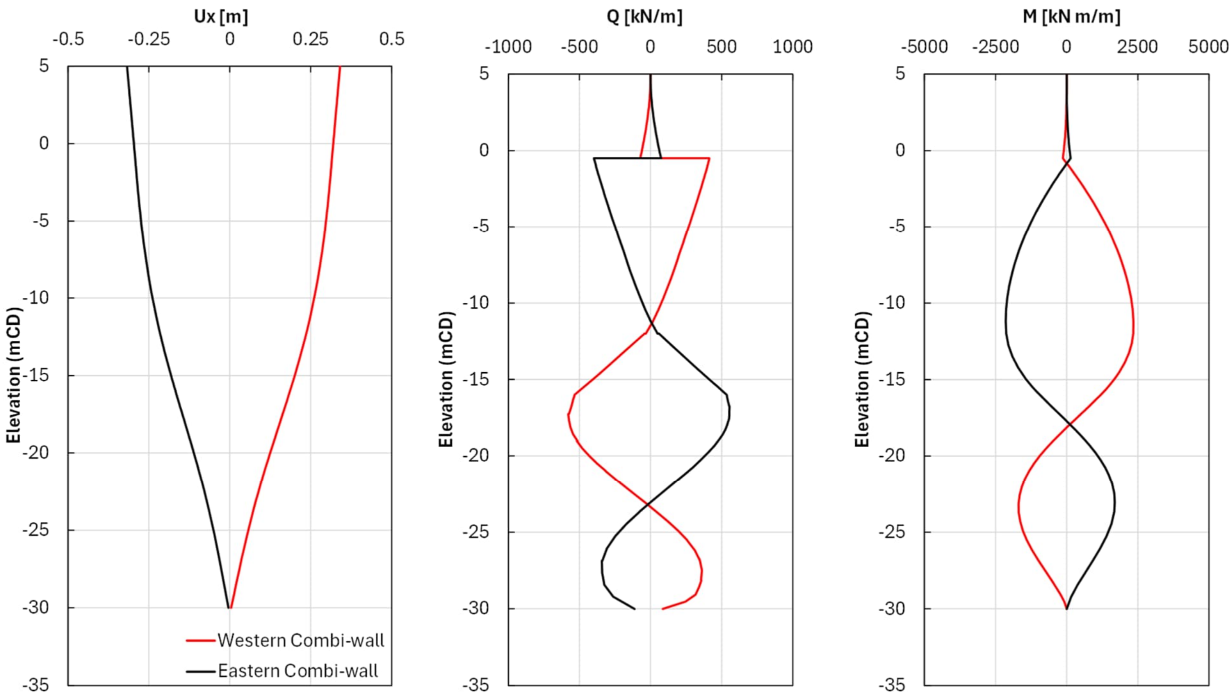

Figure 15: Lateral displacements (Ux), shear forces (Q), and bending moment (M) of Eastern and Western Combi-wall for east-west transverse section

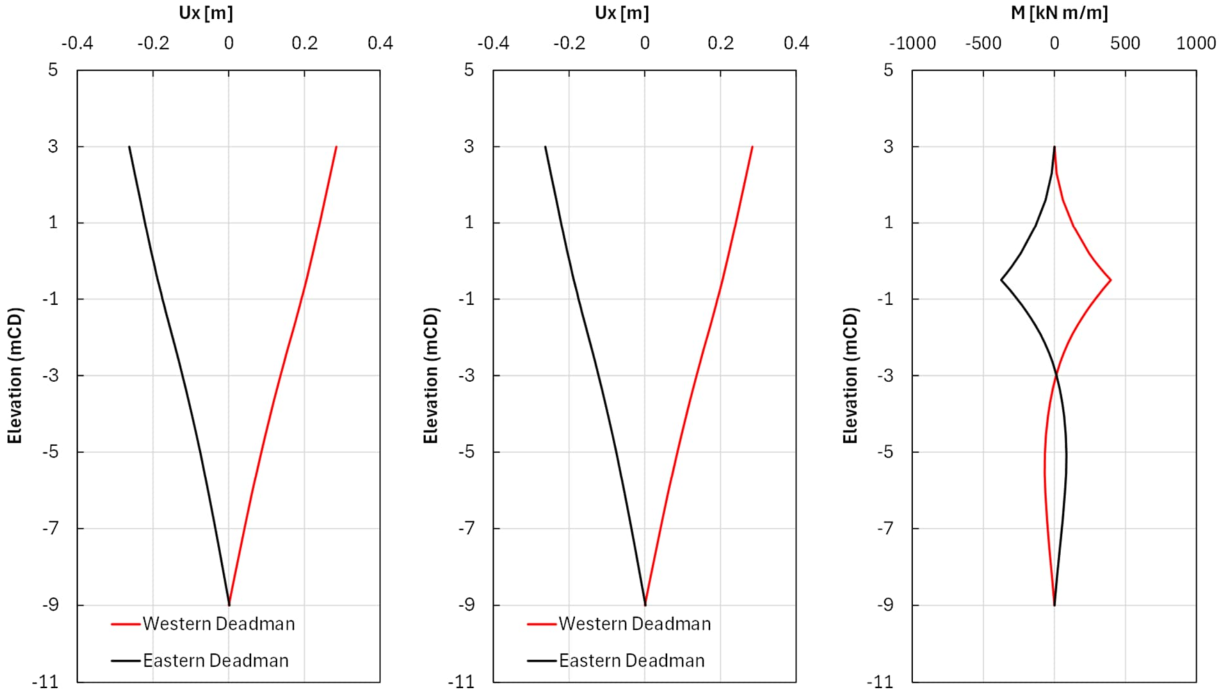

Figure 16: Lateral displacements (Ux), shear forces (Q), and bending moment (M) of Eastern and Western Deadman for east-west transverse section

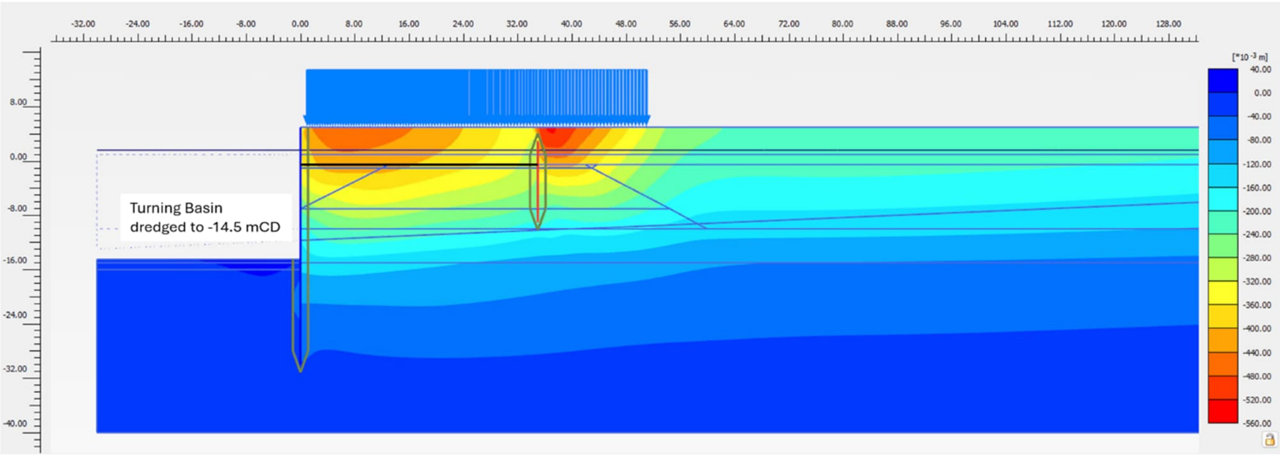

Figure 17: Example of north-south long section vertical displacements

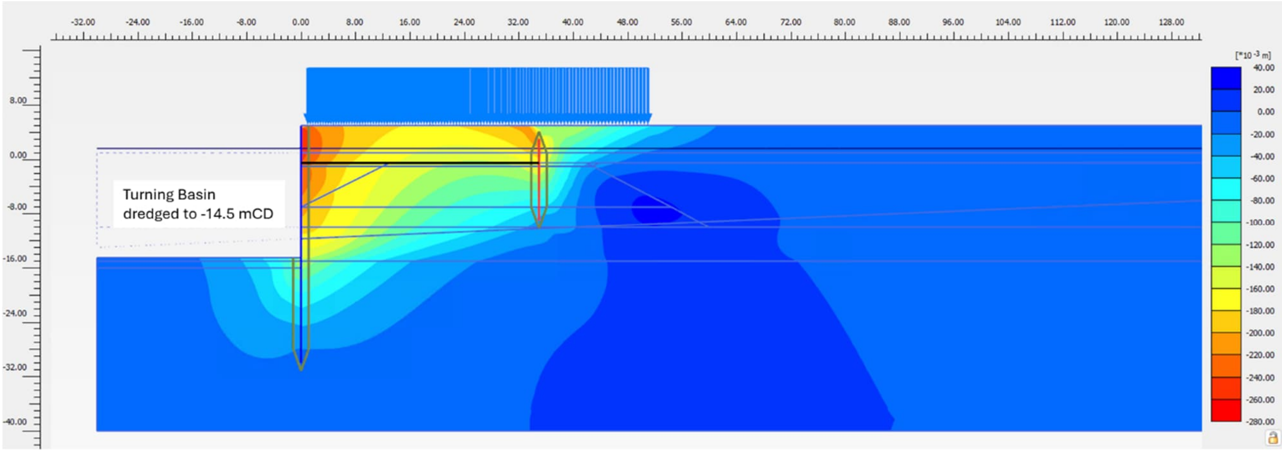

Figure 18: Example of north-south long section total horizontal displacements

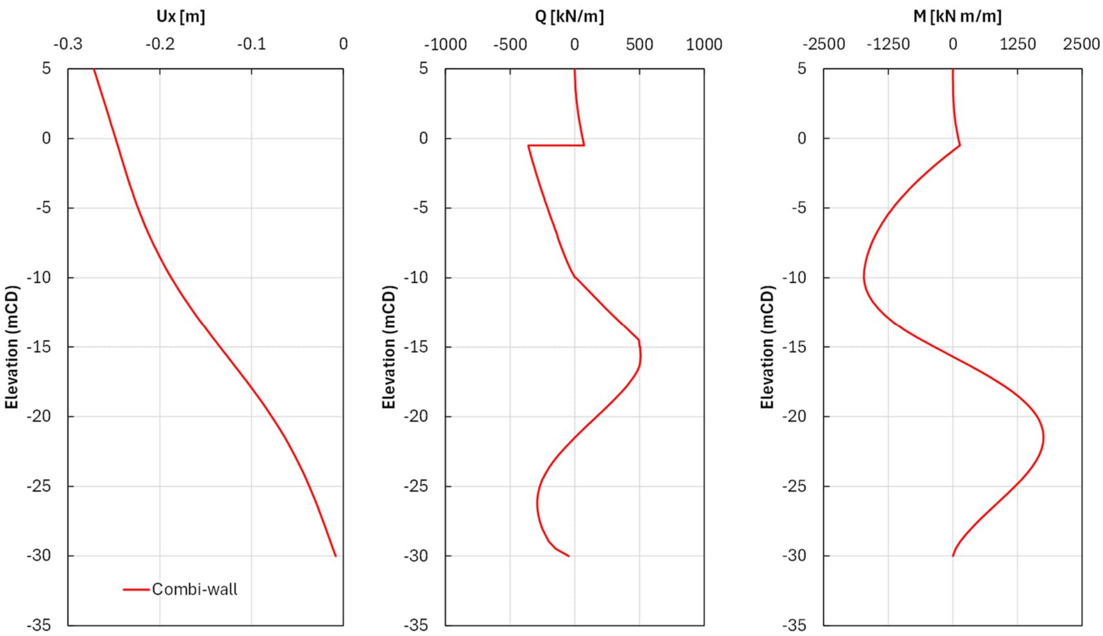

Figure 19: Lateral displacements (Ux), shear forces (Q), and bending moment (M) of Combi-wall for north-south long section

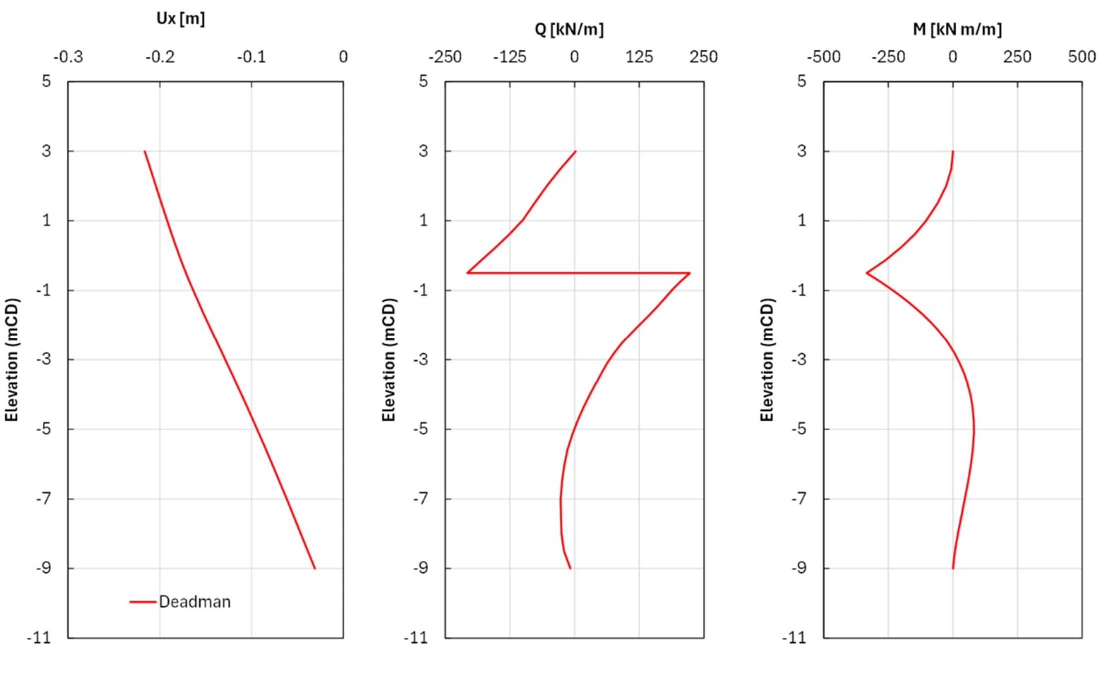

Figure 20: Lateral displacements (Ux), shear forces (Q), and bending moment (M) of Deadman for north-south long section

10. Conclusion

WSP has undertaken independent conceptual geotechnical assessment of the proposed Beca conceptual design of the Combi-wall retaining wall, and a high-level consideration of the revetment to enclose the western edge of the reclamation and the open piled jetty to the west and the piled suspended deck adjacent to the shiplift. WSP's independent analysis for the combi-wall and high-level assessment of the revetment, jetty and suspended deck indicates that the proposed Beca Northland Shipyard and Dry Dock civils/marine infrastructure design is an appropriate and viable engineering solution based on the information available.

Settlement from consolidation of the Upper Sandy Silt-Clay under the weight of the reclamation may affect construction and maintenance requirements. This will need to be investigated further and managed in future stages. In addition, the design relies on ground improvement to mitigate liquefaction. The effectiveness of different ground improvement techniques will be a key consideration in detailed design.

WSP's independent static numerical modelling suggests that the proposed Beca combi-wall system, which utilises 1,624 mm diameter tubular steel piles, can meet performance requirements under conceptual conditions; however, settlement, lateral displacement, and long-term durability (including corrosion) are key sensitivities. Due to the presence of a dense sand layer around -20 mCD to -25m CD, whilst Vibro hammer can be used to install the piles above the dense sand layer, impact hammering will be needed where piles need to be driven well into the dense sand layer.

Seismic loading and liquefaction-induced effects have not been assessed. These will increase ground deformation and structural demands that may have significant operational impacts. As such these aspects will need to be considered during future design stages.