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PORT OF HASTINGS DEVELOPMENT PROJECT

DESIGN AND ENGINEERING

Interim Structural Design Options Assessment Report

Document Ref: AGH-CEP0-EG-REP-0015

In May 2016 the Special Minister of State asked Infrastructure Victoria to provide advice on the future capacity of Victoria's commercial ports. Specifically, the Minister has asked for advice on when the need for a second container port is likely to arise and which variables may alter this timeline. The Minister has also asked for advice on where a second container port would ideally be located and under what conditions, including the suitability of, and barriers to investing in, sites at the Port of Hastings and the Bay West location.

In undertaking this task, Infrastructure Victoria reviewed work that was completed as part of the Port of Hastings development project before it was cancelled in 2014. This document forms part of the initial work undertaken for the proposed port development at Hastings. Infrastructure Victoria considers that much of the previous Hastings work, although preliminary in nature, is relevant and suitable for informing a strategic assessment. Therefore, Infrastructure Victoria has requested that preliminary and draft reports previously commissioned for the development project be reissued to form part of the evidence base on which Infrastructure Victoria will use in providing the Minister with advice.

The opinions, conclusions and any recommendations in this document are based on conditions encountered and information reviewed at the date of preparation of the document and for the purposes of the Port of Hastings Development Project.

Infrastructure Victoria and its consultants have used the information contained in these reports as an input but have not wholly relied on all the information presented in these reports.

DRAFT

Port of Hastings Development Project – Design and Engineering

Interim Structural Design Options Assessment Report

Client: Infrastructure Victoria

ABN: 44 128 890 975

Prepared by the AECOM + GHD Joint Venture

AECOM Australia Pty Ltd
Level 9, 8 Exhibition Street, Melbourne VIC 3000, Australia
T +61 3 9653 1234 F +61 3 9654 7117 www.aecom.com
+

GHD Pty Ltd
Level 8, 180 Lonsdale Street, Melbourne VIC 3000, Australia
T +61 3 8687 8000 F +61 3 8687 8111 www.ghd.com

JV ABN: 57 194 323 595

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The opinions, conclusions and any recommendations in this Report are based on assumptions made by the AECOM + GHD Joint Venture described in this Report. The AECOM + GHD Joint Venture disclaims liability arising from any of the assumptions being incorrect.

DRAFT

Quality Information

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Prepared by	A. McArthur, S. Muniswamy & B. Gray
Reviewed by	Venket Naidu, Richard Clarke & Richard Hill

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1.0 Introduction

1.1 Background Information

The Victorian Government has identified the Port of Hastings as the suitable site for additional capacity for international container trade as the Port of Melbourne reaches capacity. The Victorian Government established the Port of Hastings Development Authority on 1 January 2012 as the first key step in the development of Hastings as a future container port.

The Authority's objectives for the Project are to:

- Make a positive contribution to Victorian and national economic growth and productivity over the long term;
- Create a commercially viable container port which services the long term needs of Victoria;
- Deliver a world-class competitive port which is attractive to shipping lines and supply chain owners and operators;
- Achieve a minimum port capacity of nine million TEU by 2060 which is integrated into the Victorian freight network;
- Construct, operate and maintain a safe and sustainable port;
- Optimise benefits and manage impacts of the project on the community, existing customers and the environment; and
- Maximise whole-of-life value for money for government.

1.2 Purpose of this Report

This report presents the preliminary assessment of options for the relevant structure types and wharf concepts for three shortlisted berth alignments. The various structure types have been prepared based on engineering judgement and preliminary structural and geotechnical analysis and the findings have been included in this report for comparison based on input from other disciplines. At this stage the preferred structure types have not been shortlisted and it is intended that this will be done following review and workshops with PoHDA and other work package teams.

This report is intended to form the basis for a qualitative evaluation of each of the wharf types taking into consideration operational requirements, likely capital and maintenance cost, serviceability, settlement, constructability, future proofing, geotechnical and environmental issues and dredging and reclamation requirements.

A workshop with the Hydrodynamic/DMM and Environment & Social workstreams will be held to compare and assess each of the options against the established criteria prior to preparation of the final Summary Report for the Structural Design Options.

1.3 Scope and Limitations

The scope of work for the Structural Design Options Assessment report as outlined in the submission for work package WP-013 is as follows:

- Derive Quay Wall Concept Design Loads
  - Preliminary berthing analysis;
  - Preliminary mooring analysis;
  - Wharf crane and live load analysis;
- Preliminary Review of Wharf Types

- Review options for quay wall and edge structures and identify suitable or appropriate solutions;
- Preliminary concept design of wharf and edge structures sufficient to compare relative merits and impact on construction and staging;

- Prepare a summary report on broad structural forms that is to form the basis for a qualitative evaluation of the wharf options.

1.4 Assumptions and Logic

The preliminary assessment of wharf concept designs is based on the following assumptions and logic:

- Quay structural options are based on the alignments outlined in Section 2.0.
- Functional and engineering criteria are taken from Rev B of the Basis of Design unless specified otherwise.
- Ground conditions are based on the information in the Geotechnical Interpretive Report.
- Dredging and reclamation strategy is based on the working group discussions for which reports are still being prepared by the other workstreams.

2.0 Berth Alignment Options

Three berth alignments have been considered for the development of the quay structural design options. These are:

- Along the Shore Alignment, - refer to figure AGH-CEP0-DE-FIG-0097 in Appendix A;
- Basin Alignment – refer to figure AGH-CEP0-DE-FIG-0098 in Appendix A;
- Straight Alignment – refer to figure AGH-CEP0-DE-FIG-0096 in Appendix A;

All alignments share a common starting point which is 250m north of the centre point of the Long Island Point (LIP) jetty as outlined in memo AGH-CEP0-EG-MEM-0015 - Vessel Clearances and Safety Zones at LIP.

The Along the Shore and Basin alignments are the same for the first 2,200m in that they are both aligned with the existing Long Island Point Jetty and BlueScope RORO wharf berth line. At about this point the berth line changes to form the following two different alignments that define two of the three options that are currently being considered:

- Along the Shore Option – berth line follows the general shape of the shoreline and bed contours;
- Basin Option – a basin is excavated behind the shore line with berths potentially located on both sides;

For the Along the Shore Option the berth alignment changes by approximately 44 degrees to the east and generally follows the -5m CD contour. The length of this face could be potentially up to 3,000m which would bring the total berth length to 5,200m. For the Basin Option the proposed berth alignment changes by approximately 46 degrees to the west to maintain a 100m clearance between the back of a 600m deep terminal and the centreline of the existing road around the north of the BlueScope facility. In this option the length of the basin proposed is 2,500m with an additional 800m of berths on the northern side of the basin. The width of the basin is currently nominated as 450m. The total berth length proposed in this alignment is 5,500m.

The third option that has been considered is based on a Straight Alignment starting from the same southern point as the other options. This alignment is 5,200m long and generally follows the -10m CD contour.

The berth positions and lengths adopted are indicative and are subject to modification pending other studies. The selection of a suitable location of the berth line will be influenced by ongoing related studies, these include:

- Hydrodynamic studies,
- Dredging and reclamation studies,
- Environmental studies,
- Vessel simulation studies,
- Terminal systems analysis including the number of operators,
- Ongoing development of the engineering options and
- Operational safety considerations.

3.0 Principal Functional Requirements

3.1 Berth Levels

A berth cope level of +6.0mCD and depth alongside of -17mCD has been adopted in the preliminary analysis, in line with the Basis of Design.

3.2 Design Vessel

In line with the Basis of Design the design vessels are outlined in Table 3-1.

The minimum design vessel is a small feeder ship. At present there are no feeder services out of Victoria but there is a chance that part of the Tasmania service will convert to LoLo. Several countries are also actively encouraging coastal shipping services as a way of minimising the environmental impact of transport and it is likely that Australia will follow this trend.

The minimum vessel in Version 4 of the PoH Draft Demand dated 5 December 2014 has been revised to a 5,000-6,000 TEU ship which has typical dimensions of 280m LOA, 40m beam and a maximum design draught (T_{max}) of 14.0m. The change in the minimum design vessels primarily affects the fender design while the larger vessel generally governs for other design aspects. This minimum vessel service the Pacific Island trades and corresponds to the smallest vessels that currently visit Swanson Dock.

The Maximum Design Draught shown is the structural design draught of the ship. In practice these ships never sail with draughts in excess of 15.5m.

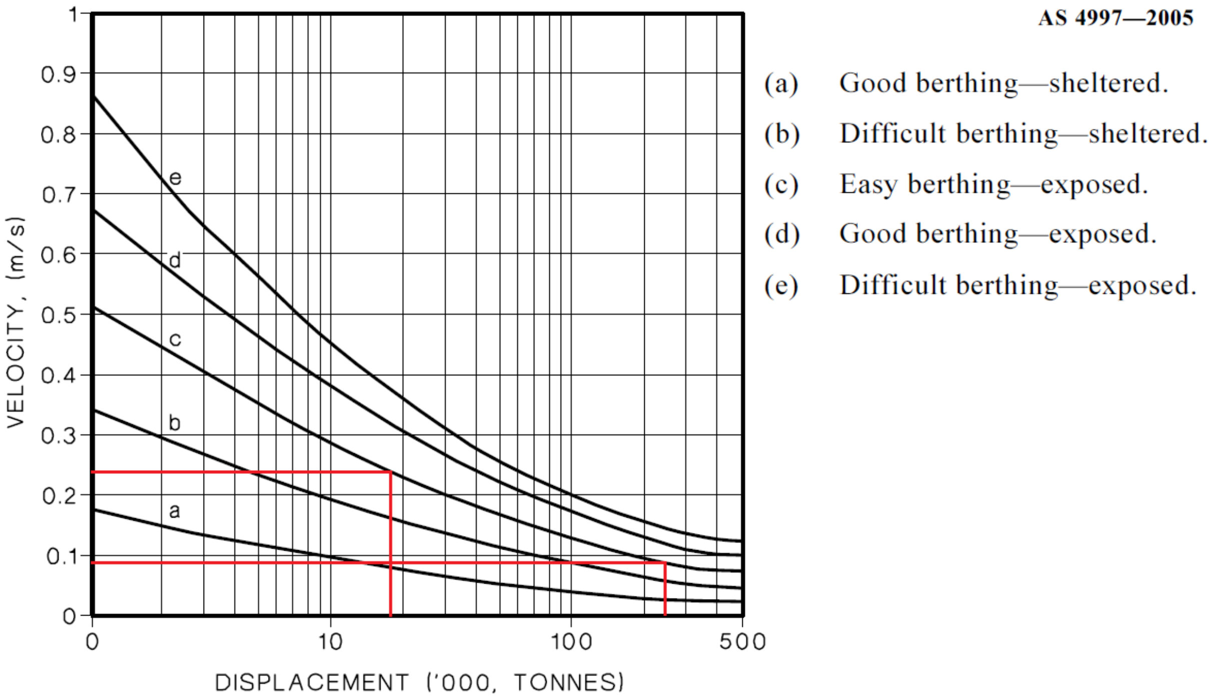
Table 3-1. Design Vessels

Characteristic	Maximum Design Vessel	Minimum Design Vessel
Dead Weight Tonnage	195,000	12,600
Full Load Displacement (tonnes)	249,000	17,200
Length Overall (m)	400	144
Length Between Perpendiculars (m)	376	135
Maximum Beam (m)	59	22.6
Depth (m)	30.3	10.8
Maximum Design Draught (T_{max}) (m)	16	8

3.3 Berthing and Mooring of Vessels

3.3.1 Preliminary Berthing Assessment

The design berthing speeds have been derived taking into account the recommendations from AS4997 including Brolsma et al (1977) as outlined in Figure 3-1.



Source: Brolsma et al, (1977) *Design berthing velocity (mean value) as a function of navigation conditions and size of vessel*)

Figure 3-1 - Design Berthing Speed – Brolsma 1977

Preliminary berthing analysis of the design vessels has been based on the parameters in Table 3-2.

Table 3-2. Design Vessel Berthing Parameters

Characteristic	Maximum Design Vessel	Minimum Design Vessel
Berthing conditions – Brolsma (refer AS4997 Appendix B)	Easy berthing, Partial exposure – Condition C	Easy berthing, Partial exposure – Condition C
Berthing Velocity – Brolsma (m/s)	0.10	0.25
Berthing Angle (deg.)	10	10
Abnormal Berthing Energy Ratio	1.75	1.75
Maximum Hull Pressure (kPa) – (PIANC Guidelines for the Design of Fenders) to be confirmed with ship manufacturers	200	400

3.3.2 Fender Arrangement Requirements

Preliminary berthing analysis suggests that the normal and abnormal berthing energies and fender reactions for design will be approximately as outlined in Table 3-3 below. The fender reaction force based on the design berthing energy is in the order of 220 tonnes based on the preliminary fender arrangement that has been considered.

Table 3-3 - Preliminary Fender Arrangement Requirements

Characteristic	Maximum Design Vessel	Minimum Design Vessel
Normal Energy (t-m)	130	55
Abnormal Energy (t-m)	228	95
Fender	SCN 1600 (E1.2)	
Fender Spacing (m)	Up to 14	
Rated Energy (t-m)	165	
Rated Reaction (t)	199	
Rated Energy less tolerance (t-m)	148	
Rated Reaction plus tolerance (t)	220	

3.3.3 Mooring Conditions during Severe Wind Conditions and Crane Operations

The PoMC have recently indicated that container terminals may be required to cater for vessels remaining alongside during 30 second wind gusts of up to 60 knots, a wind speed that was exceeded only on 12 occasions over a period of almost 19 years based on BOM wind data at Fawkner Beacon. This would equate to a mean hourly wind speed of approximately 45knots (23.5m/s). These guidelines are likely to be followed by the Port of Hastings in the future.

Modern container cranes are designed to operate in steady wind speeds of 35 knots (18m/s) which is likely to include gusts up to 45 knots (23.5m/s). To allow some latitude in an increasing wind, the analysis will be based on a 48 knot gust.

3.3.4 Mooring Analysis Criteria

Mooring loads are to be assessed in accordance with AS 4997 – 2005 for the full vessel spectrum.

Bollard groups for international container vessels are to be designed to independently service at least 4 bow and 4 stern lines from adjacent ships where applicable considering all possible berthing configurations. Nominal mooring line working capacity of 75tonnes / line is assumed based on the use of 80mm diameter Euroflex polypropylene/polyester mooring lines and up to 55% utilisation of minimum breaking loads.

Mooring loads on bollards are to be limited to between +75° and -15° to the horizontal plane and in any direction from the forward arc of the wharf. The mooring assessment has considered extreme winds only, and not in-combination with effects of passing vessels.

3.3.4.1 Extreme Mooring – No Cargo Transfer Operations

The following extreme mooring conditions have been considered:

- 60 knots wind speed (30 second gust) from any direction;
- 25 year design wave (Hs = 1.5m T = 4sec);
- Maximum current (2 knots) at MHWS (+2.84mCD) or MLWS (+0.61mCD) concurrent with maximum wind and wave condition;
- 50% maximum current (1 knot) at HAT (+3.31mCD) or LAT (0.0mCD) concurrent with maximum wind and wave condition (this scenario takes account of what will happen under surge conditions); and
- With and without sea level rise and surge allowances (combined sea level rise and 20 year return period storm surge of 1.4m in 2070).

3.3.4.2 Operational Mooring – Cargo Transfer Operations

The following limits of cargo transfer operation mooring conditions have been considered:

- 48 knots wind speed (30 second gust) from any direction;
- 25 year design wave (Hs = 1.5m T = 4.0sec);

- Maximum current (2 knots) at MHWS (+2.84mCD) or MLWS (+0.61mCD) concurrent with maximum wind and wave condition;
- 50% maximum current (1knot) at HAT (+3.31mCD) or LAT (0.0mCD) concurrent with maximum wind and wave condition; and
- With and without sea level rise and surge allowances (combined sea level rise and 20 year return period storm surge of 1.4m in 2070).

3.3.5 Preliminary Mooring Assessment

Two mooring scenarios were considered in the preliminary mooring assessment. The first scenario was for mooring during cargo transfer operations with 30 sec wind speeds of 48 knots and the second was for the limiting mooring scenario with 30 second wind speed of 60 knots. Mooring line configurations assumed were as illustrated below in Figure 3-2 and Figure 3-3.

Initially bollards with a capacity of 150 tonnes were modelled with a spacing of 15m for both scenarios with additional storm bollards utilised in the extreme case.

Twin mooring lines were utilised on each bollard in each scenario with a total combined working capacity of 150 tonnes per bollard.

3.3.5.1 Limiting Cargo Handling Operations – Severe Wind 48 Knots

Four pairs of mooring lines were used forward and four aft with two pairs of mooring spring lines in each direction. Winches were assumed for all lines with limited pretension except for spring lines which were not pre-tensioned.

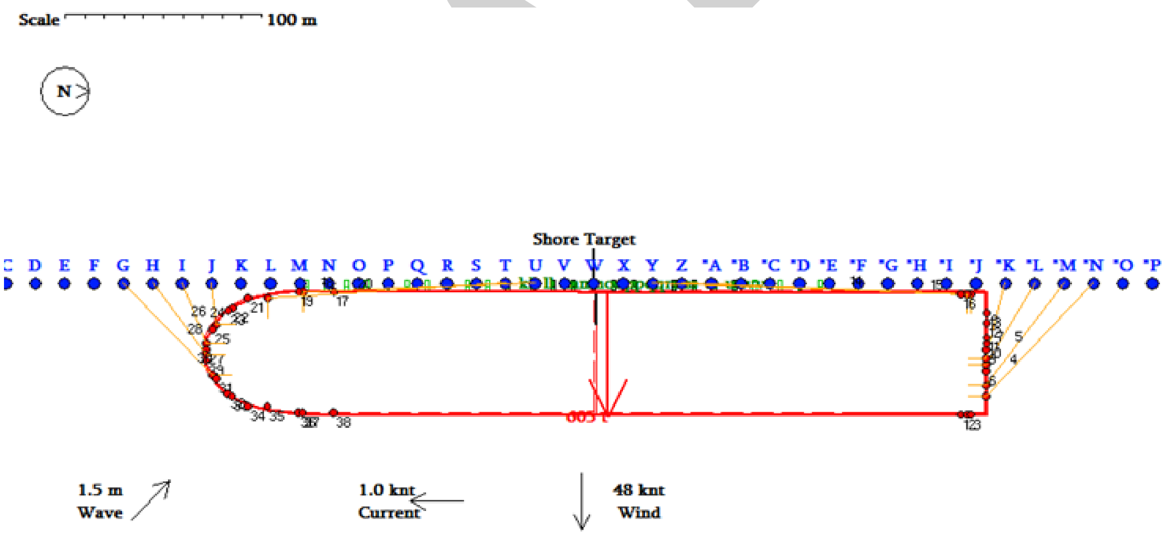


Figure 3-2 Design Mooring Scenario with Cargo Transfer Operations - 48 Knots

3.3.5.2 Limiting Mooring Scenario – Extreme Wind 60 Knots

Four pairs of mooring lines were used forward and four aft with two pairs of spring lines were utilised in each direction. These were supplemented with two additional pairs of mooring lines forward and aft to storm bollards (labelled *Q, *R, *S and *T in Figure 3-3) positioned at the landside of the berth (43m landward of the cope line). The ability to provide these storm moorings without compromising the operation of the terminal will have to be reviewed at the design stage. Winches were assumed for all lines with pretension on all mooring lines except spring lines.

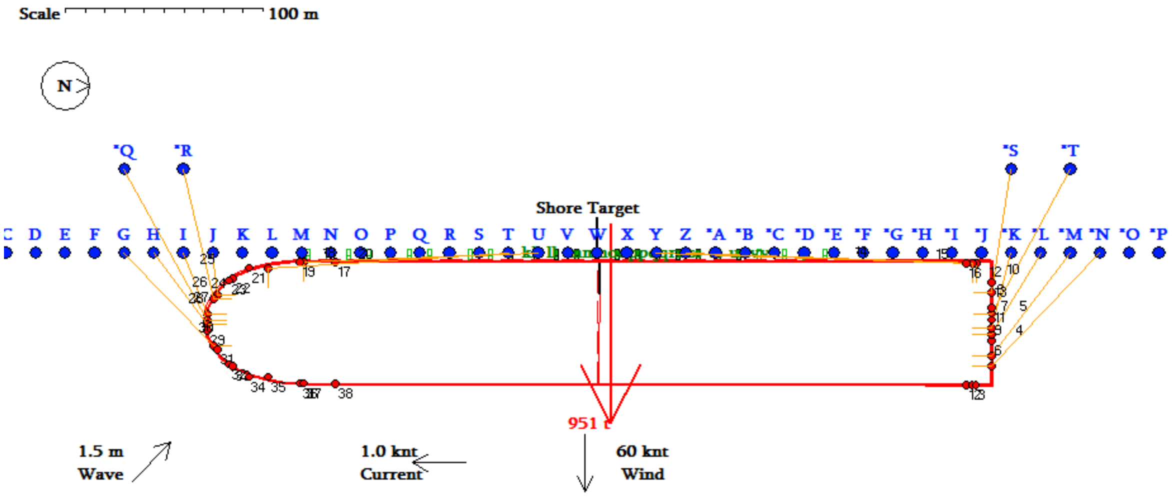


Figure 3-3 Design Mooring Scenario in Severe Wind Conditions - No Cargo Transfer Operations – 60 Knots

Mooring line loads and bollard loads were all within the respective working capacities. Vessel movements off the berth were up to 0.8m and along berth up to approximately 2.9m, however actual movements will depend on line types, numbers, age and any pretension.

3.3.6 Bollard Sizes, Spacing and Capacity

The results of preliminary mooring analysis suggest that an alternative arrangement with smaller and more closely spaced bollards may be preferred, nominally 150 tonne bollards at 15m centres. This was adopted for the purpose of the structural assessment.

3.4 Preliminary Operational Loads and Limitations

3.4.1 Quay Crane Loads

Operational quay crane wheel loads have been assumed to be 136 tonnes at a spacing of 1.1m with 10 wheels per quay crane leg based on experience and assuming a 35m quay crane gauge to cater for 59m beam container vessels.

3.4.2 Other Operational Loads

The following uniformly distributed loads have been assumed:

- In front of the front crane rail - 10kPa (Class 10 emergency and service vehicles as per Table 5.1, AS 4997).
- Between the two crane rails – 50kPa (Class 50 primary port, international gateway container terminal as per Table 5.1, AS 4997).
- Behind the rear crane rail for a distance of 95m – 30kPA for ASC operations.
- Beyond 95m from the rear crane rail – 60kPA for ASC stacks.

4.0 Berth Structure Options

4.1 General

A long list of quay structure options was identified and presented to the Authority on 25 November 2014 and these options represent a relatively large range of solutions covering three alternative generic land backed quay structural forms as outlined below.

In line with the Basis of Design the nominal depth alongside for stability assessment was adopted as -17mCD and the cope line is +6.0mCD, subject to further assessment on vessel spectrum and berth elevation. This assessment adopts a quay crane rail gauge of 35m for vessels with a beam of 59m with 23 containers across.

In addition, taking into account preliminary findings of the terminal systems assessment, this assessment also adopts a seaside crane rail setback distance from cope line of 5m and a fender line offset seaward of the cope line of 2m.

The figures in this section show the existing surface along the stage 1 development. Appendix B contains additional plans and cross sections with the existing surface for the second facet of the along the shore alignment and the basin option.

4.2 Open Piled Quay Structure with Cut-off Sheet pile Wall

The open piled quay structure or suspended deck typically comprises a reinforced concrete deck supported by a combination of vertical and raking piles overlying a sloping rock revetment. The option considered uses steel tubular piles with a retaining wall on the landside of the quay deck, as dictated by the depth alongside, revetment slope and quay structure width. An alternative system with prestressed reinforced concrete piles was not considered due to the inherent risks with pile damage during installation in a marine environment.

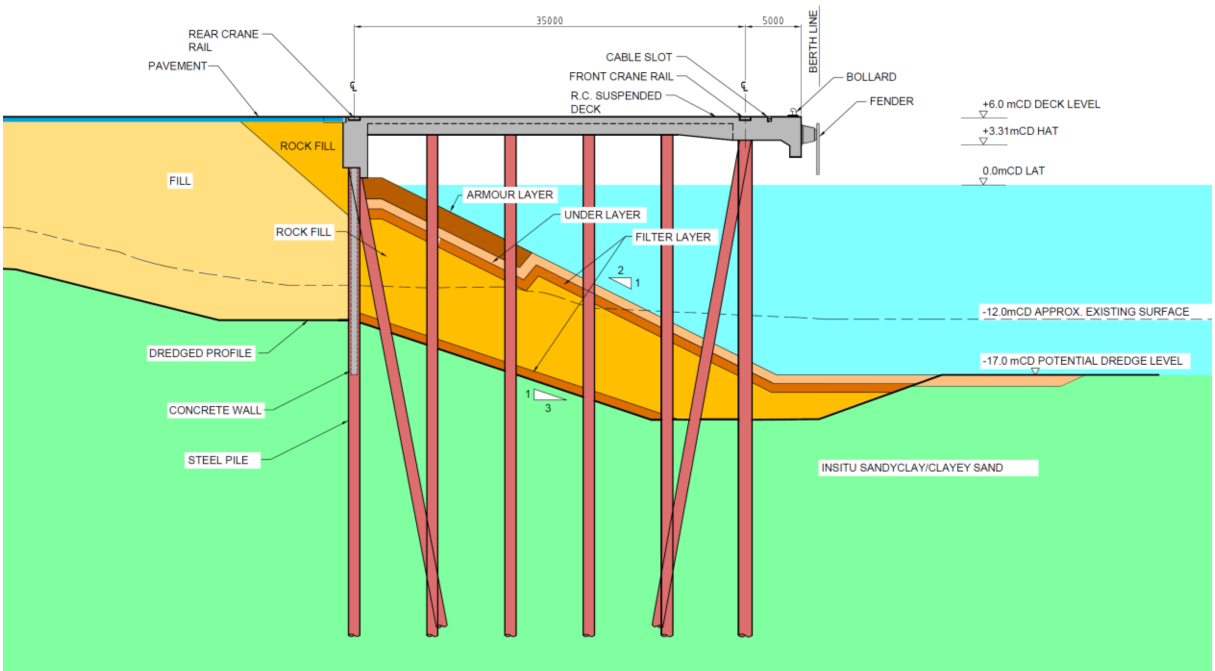


Figure 4-1 - Open Piled Structure Overlying Sloping Revetment

The quay deck can be in the form of a reinforced concrete flat slab, one way beam and slab or two way beam and slab system. The superstructure option used in this evaluation is a two way beam and slab system which provides dedicated longitudinal beams beneath quay crane rails and intermediate longitudinal and transverse beams at pile grid lines. Steel tubular piles are driven to a prescribed set to achieve the required capacity and the exposed top section of piles may be infilled with reinforced concrete to carry pile loads, assuming the top

steel sections are allowed to corrode in the long term. The rear retaining wall may be formed with steel sheet piles or reinforced concrete retaining wall with tie back anchors where necessary or propped against the rear edge of the quay structure.

If steel sheet piles are used to form the retaining wall, they will have to be designed for the required 100 year life. This may require a combination of concrete casing and cathodic protection. A revetment slope of 1 vertical in 2 horizontal is proposed for geotechnical slope stability. The revetment is formed with rock armour to protect the slope from waves and propeller scour.

4.3 Bulkhead Wall

The bulkhead wall comprises a vertical land backed retaining wall tied back to a dead man anchor system. The main retaining wall can be steel or reinforced concrete or a combination of the two. A capping beam spreads loads along the wall and overhangs to provide the required spacing between the crane rail and the cope line. The crane rail is directly over the wall which takes the vertical loads. The tie back system typically has tie rods at regular intervals connected to a similar but shallower deadman wall or longitudinal beam with raking piles. Lateral loads from retained reclamation fill and water level differentials across the structure are carried by the tie back system and the embedded portion of the toe of the main wall. The wall also takes the fender loads, the forces from the mooring bollards and the lateral loads on the crane rail. The landside quay crane rail is supported on a longitudinal crane beam on either vertical and/or raking steel tubular piles. The landside crane rail beam may also be part of the tie back system.

Two bulkhead wall options were considered as described below.

The first option uses large diameter steel tubular piles interlocked by clutches to form the main wall. Steel tie rods are anchored to a steel tubular pile dead man wall. The tie bars are articulated with hinges at each end to take out the bending stresses resulting from settlement of the fill behind the wall. Staging of the backfill behind the bulkhead wall is critical to the strength and stability of the structure and an option is to construct the wall within a bund and then remove the fill on the seaward side of the wall.

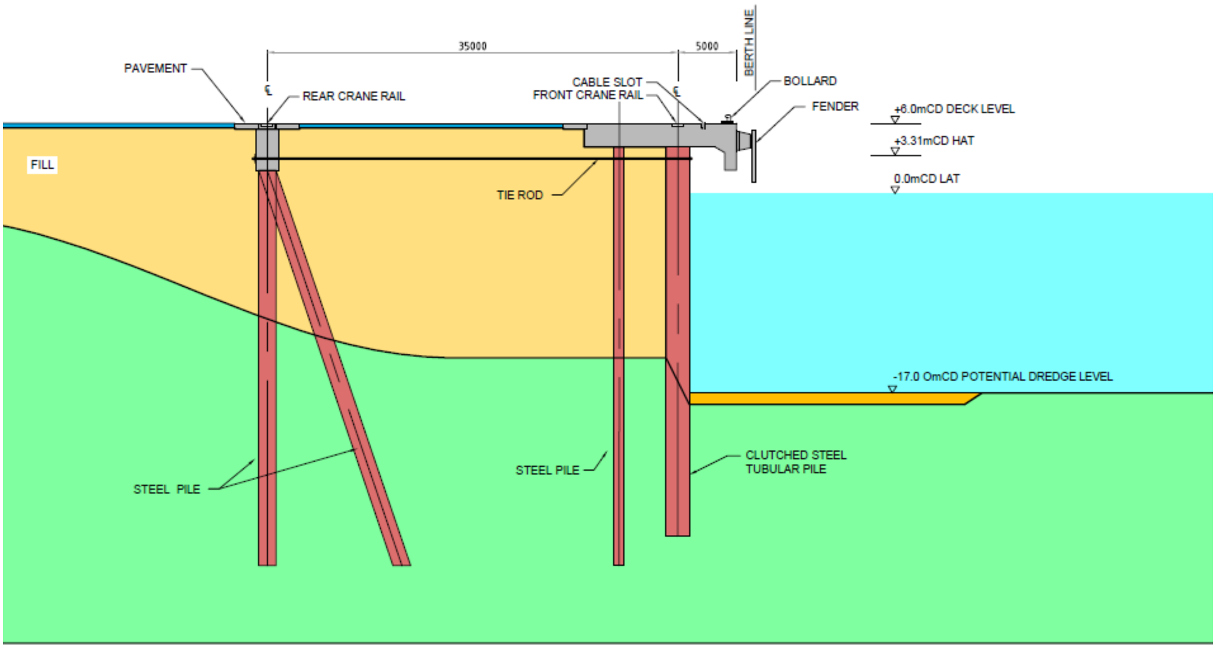


Figure 4-2 – Bulkhead Wall using Steel Pipe Pile Wall

The second option is to form the face with a reinforced concrete diaphragm wall with steel tie bars back to an anchor point. The anchor point can consist of a concrete beam supported on steel piles or a second diaphragm

wall. In other respects the details are similar to the steel wall.

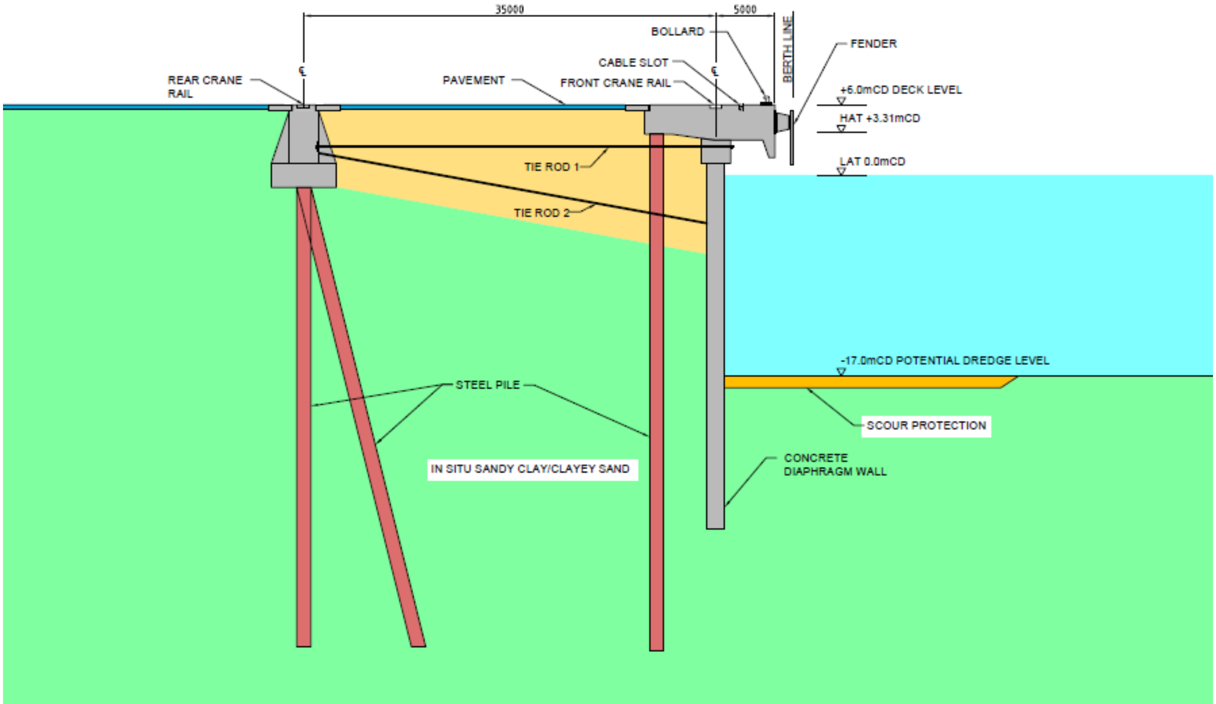


Figure 4-3 **– Concrete Bulkhead WallGravity Retaining Structure**

Gravity retaining structures principally rely on their self-weight for stability and may be formed as reinforced concrete caissons, counterfort walls, mass concrete blockwork walls or steel sheet pile cofferdams. The options considered in the assessment assume full height gravity structures although there are hybrid versions which combine gravity structures with narrower open piled quay deck arrangements. Four full height gravity retaining structure forms have been considered as described below.

The first option considered uses a multi cell reinforced concrete caisson as shown in Figure 4-4. The caisson base slab is founded on a rock fill mound at approximately 2m below the level of the berth pocket and extends close to the cope level. Above this is a reinforced concrete coping to support the fenders and bollards. The rockfill mound is founded at a level where ground conditions, enhanced where needed by ground treatment, are adequate to support the distributed loads from the base of the caisson. These are typically highest at the seaward edge. Each cell of the caisson is filled with granular non compressible material which, together with the weight of the caisson and overlying pavement provides stability against overturning and sliding. The horizontal forces are transferred to the ground by friction on the underlying rockfill mound. The land behind the caisson is backfilled with granular material to minimise residual settlement and lateral forces on the caisson. The caisson structure serves to provide a stable edge structure to the reclamation whilst supporting direct quay loads, including the seaside crane rail and lateral loads from the quay crane, mooring and berthing. The landside quay crane rail is supported on an independent longitudinal crane beam on steel tubular piles.

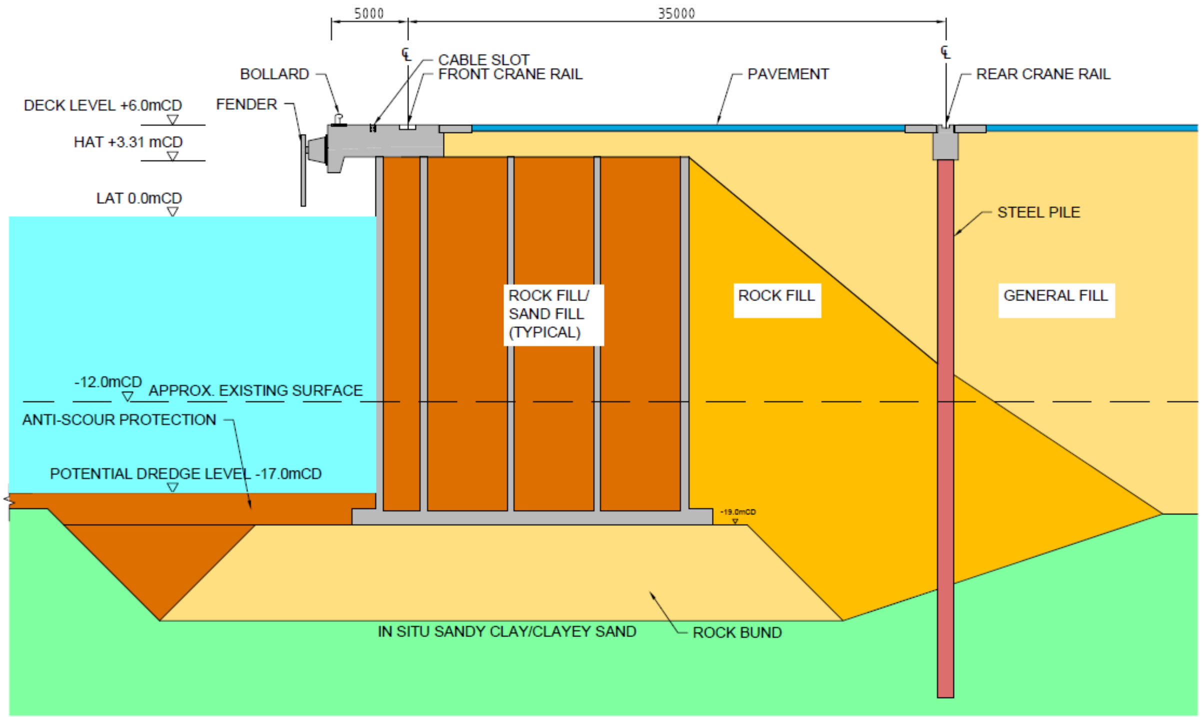


Figure 4-4 – Reinforced Concrete Caisson and Independent Piled Landside Crane Beam

The second gravity retaining structure is a reinforced concrete counterfort wall as shown in Figure 4-5. This is formed by a reinforced concrete vertical retaining wall and base slab connected with reinforced concrete buttress walls. Similar to the caisson, the counterfort wall structure is backfilled with granular material to minimise settlement and lateral loads and relies on the self-weight together with overlying back fill to resist overturning and sliding forces. The counterfort wall provides a stable edge structure to the reclamation whilst supporting direct quay loads, including the seaside crane rail and lateral loads from the quay crane, mooring and berthing. The landside quay crane rail is supported on an independent longitudinal crane beam on steel tubular piles.

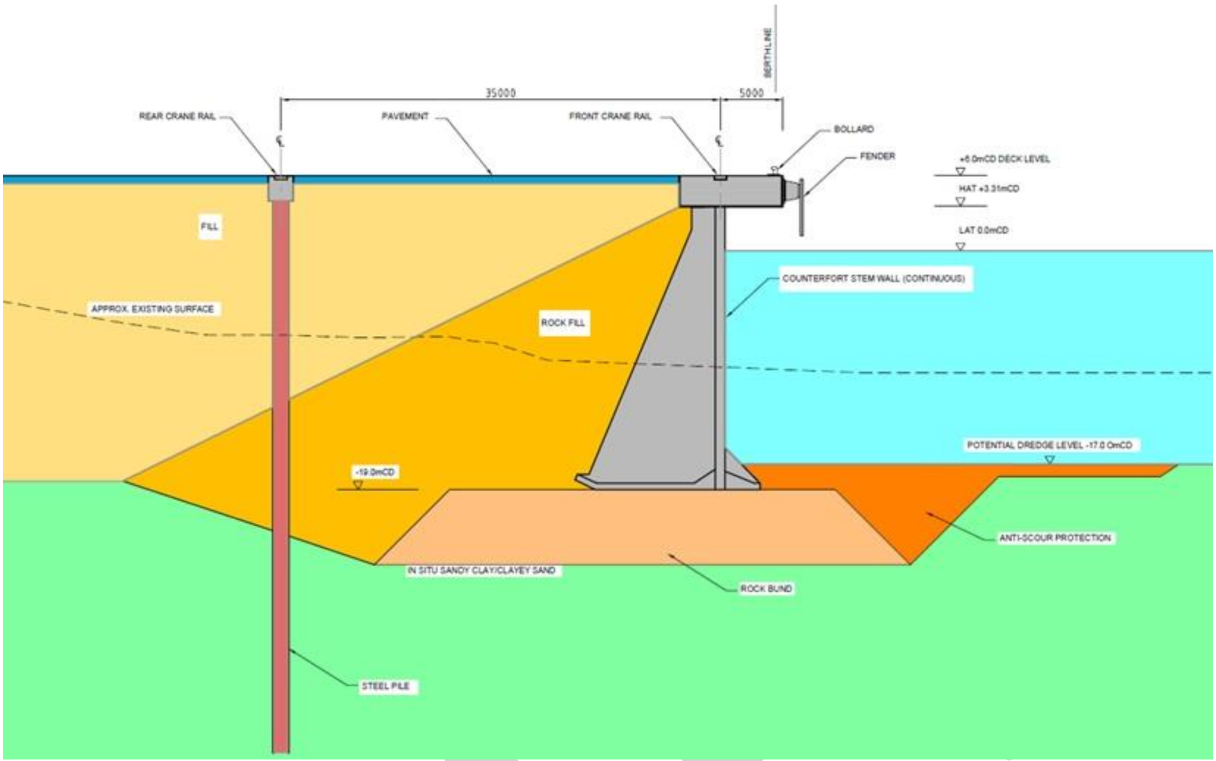


Figure 4-5 – Reinforced Concrete Counterfort Retaining Wall and Independent Piled Landside Crane Beam

The third gravity retaining wall structure is the mass concrete blockwork wall as shown in Figure 4-6. These walls consist of multiple layers of mass concrete blocks supported on a rockfill mound. Similar to the caisson and counterfort wall, the landside of the blockwork wall is backfilled with granular material to minimise settlement and lateral loads and relies on the self-weight together with overlying back fill to resist overturning. Sliding forces are resisted by friction at each horizontal joint s and at the base by friction of the base blocks on the underlying rockfill mound. The blockwork wall provides a stable edge structure to the reclamation whilst supporting direct quay loads, including the seaside crane rail and lateral loads from the quay crane, mooring and berthing. The landside quay crane rail is supported on an independent longitudinal crane beam, usually on steel tubular piles.

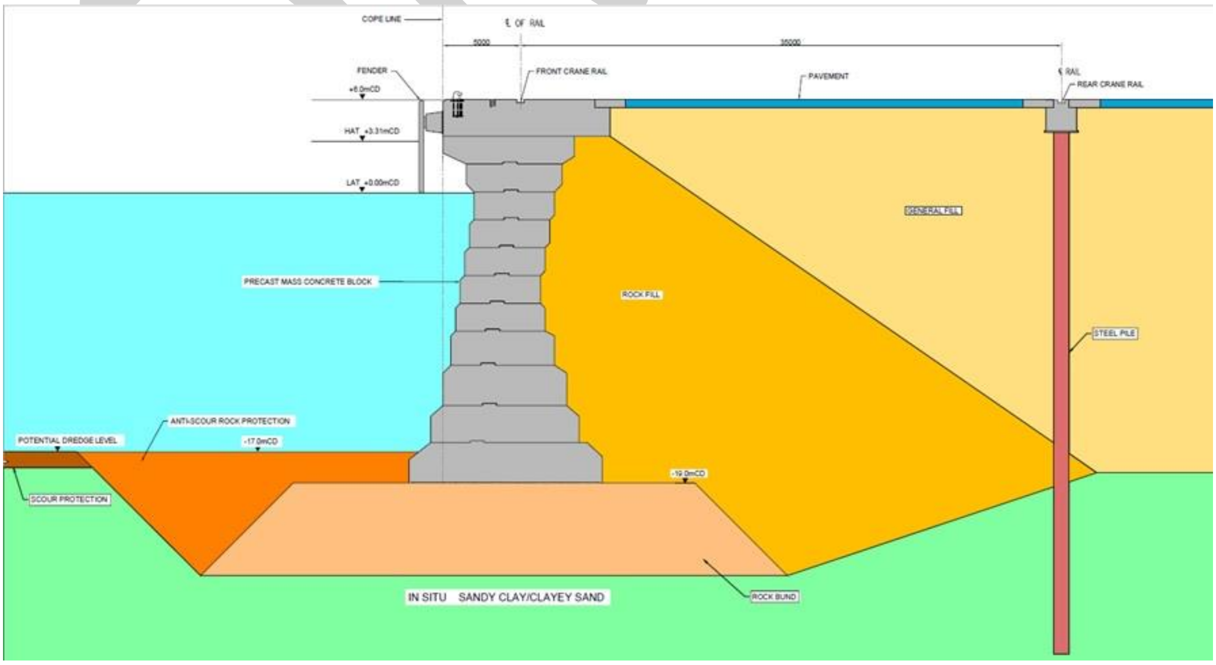


Figure 4-6 – Mass Concrete Blockwork Wall and Independent Piled Landside Crane Beam

The fourth gravity retaining wall structure is the steel sheet pile cofferdam wall as shown in Figure 4-7 and Figure 4-8. These are either circular or rectangular cofferdams infilled with granular non compressible material to minimise settlement and lateral loads. Sheet piles are driven into competent seabed material to support the toe of the cofferdam wall. Circular cofferdams are constructed in plan as a series of independent circular cells interlinked by arc walls with flat sheets. The main cells and arc walls rely on hoop stresses to retain the internal backfill.

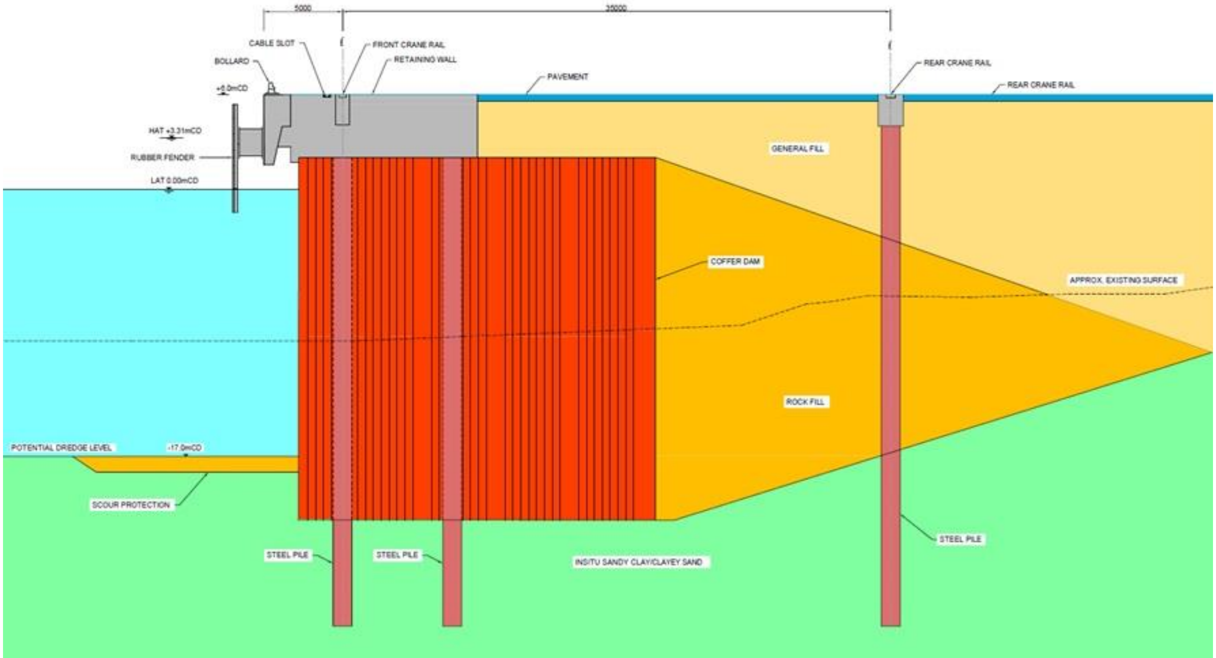


Figure 4-7 – Steel Sheet Piled Circular Cofferdam and Piled Seaside and Landside Crane Beams

However sheet piles forming rectangular cofferdams would be profiled and require tie rods and horizontal whaler beams potentially at multiple levels between opposing walls to retain the internal back fill material. Where rectangular cofferdams are constructed, the return walls may not be required and the cofferdam may be one continuous structure with tie rods between the seaward and landside walls orientated in one direction only. Similar to the caisson and counterfort wall, the landside of the cofferdam wall is backfilled with granular material to minimise settlement and lateral loads and relies on the self-weight of cofferdam infill together with overlying back fill to resist overturning. Sliding forces are resisted at the base principally by the shear strength of material into which the sheet piles are founded. The cofferdam retaining wall provides a stable edge structure to the reclamation whilst supporting direct quay loads, including lateral loads from the quay crane, mooring and berthing. The seaside crane rail and landside quay crane rail are supported on independent longitudinal crane beams normally founded on steel tubular piles. Potential construction issues which may arise with sheet pile cofferdam wall are the practicality of installing long sections of sheet pile to close tolerances required to form the cofferdam and the potential risks with declutching and unzipping of cofferdams during construction or due to in service impact.

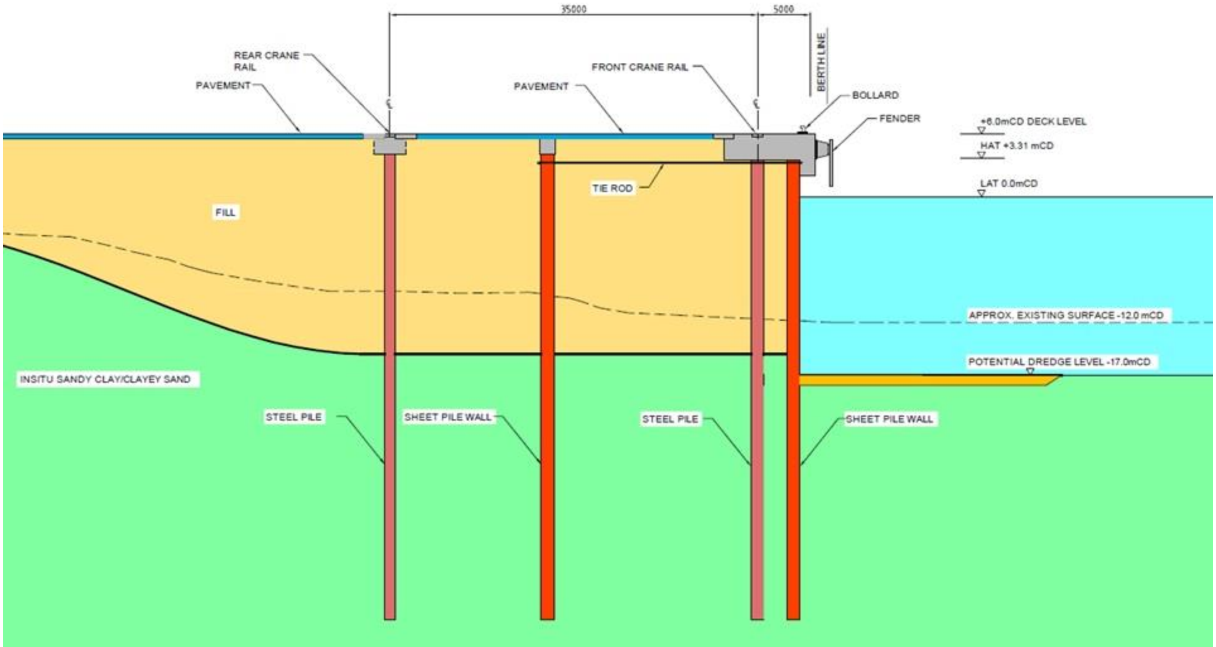


Figure 4-8 – Steel Sheet Piled Rectangular or Linear Cofferdam with Piled Seaside and Landside Crane Beams

5.0 Preliminary Comparison of Quay Structure Options

The following sections outline the comparative advantages and disadvantages for each of the quay structure options.

5.1 Open Piled Quay Deck over a Sloping Revetment

An open piled structure may be suited where ground conditions are poor or where water depths and berth structure heights are significant. Where the quay deck is built over a full height sloping revetment, the berth structure may not carry significant lateral load from the edge structure. Alternatively the quay structure width may be reduced if supplemented with a cut-off retaining wall, which is typically located on the landside edge and which may be independent or be propped to the berth structure. Where designed as independent structures the berth structure would need to be designed for vertical quay loads together with lateral loads from mooring and berthing of vessels.

Advantages

- Provides and maintains a fixed separation between the front and rear crane rail within a rigid structure.
- Eliminates residual settlement and residual differential settlement along and between the crane rails.
- Sloping seawall section tailored to underlying ground conditions without need for soil improvement.
- Minimises the height of structural cut-off wall needed to retain the reclamation.
- Low wave reflectivity from revetment slope relative to vertical quay wall.
- Provides environment for new marine habitat within the rocks and around the piles.
- Minimises impact on the currents and flow of the channel.
- Minimal need for removal of soft underlying material within the footprint of the structure.
- Normal piling equipment which is readily available.

Disadvantages

- Relatively large surface areas of the steel pile exposed to marine environment and maintenance burden requires pile encasement above water and cathodic protection system below water or extensive reinforced concrete pile infills.
- Difficulty in maintaining seawall under quay deck requires target revetment design to be very low maintenance.
- Piling works are required to be undertaken over water.
- Piling installation impacts i.e. noise.
- Cannot be built in advance of land reclamation or serve as a temporary works bund to land reclamation activities.

5.2 Bulkhead Wall

5.2.1 Steel Pile Bulkhead Wall

Large diameter steel tubular piles interlocked by clutches to form the main wall with fully structural reinforced concrete infill in the upper tidal section for durability reasons and large diameter steel tie rod anchored to a steel tubular pile dead man wall. Landside quay crane rail is supported on an independent longitudinal crane beam on either vertical and/or raking steel tubular piles.

Advantages

- Wall can be designed and built in advance of berth dredging, except for pre-removal of soft sediment in the vicinity of the wall.

- Relatively fast to construct after pile materials arrive on site.
- Moderate size construction and piling equipment can be used compared to gravity structures.

Disadvantages

- If wall is installed before the fill is placed, extensive over water work is needed.
- Tie rods and steel sheet piles are difficult to maintain particularly above the tidal level.
- Differential settlement can potentially be an issue between tie rods and anchor wall
- Piling installation impacts i.e. noise.
- For reinforced concrete options, high quality controls are required on concrete production and placement.

5.2.2 Diaphragm wall

The diaphragm wall bulkhead option utilities a reinforced concrete diaphragm wall to form the main wall and large diameter steel tie rods anchored to either a reinforced concrete beam supported by raking steel pipe piles below or a second diaphragm wall.

Advantages

- Wall is constructed in existing ground or through a bund allowing all ground treatment to be done before the wall is built, minimising risk to the wall.
- Number of joints in the wall is less which improves the water tightness.
- There is no over water work.
- It can be used for retaining very deep excavations and it can be designed for very high structural loads.
- Less noise and vibration during installation compared to driving of piles and sheet piles.
- Diaphragm wall controls the movement of ground during construction.

Disadvantages

- Differential settlement can potentially be an issue between tie rods and anchor wall with crane rail.
- Requires specialist construction equipment.

5.3 Gravity Structures

Gravity structures are generally the more robust and durable form of construction, and also resist abnormal lateral loads like vessel berthing and mooring loads. Gravity structures are more suitable where the founding soil underlying the seabed is of good quality. If the founding soil layer is not suitable the caisson would need to be founded on a rock fill mound prepared after removing soft soil layers. If necessary the founding layer for the rockfill mound may require ground treatment to support the distributed loads from the base of the caissons. Rubble or free draining granular fill would be required immediately behind the caisson wall so that the effects of tidal lag and earth pressure are reduced.

5.3.1 Caissons

Advantages

- Following construction, failures tend to be local such as differential settlement. This possibility can be reduced by preloading the structure before casting the cope beam.
- The gravity structure and rear crane rail are independent structures, however differential settlement of backfill has to be accommodated.

Disadvantages

- Additional dredging and imported rockfill is required below the gravity structure with underwater operations that also requiring use of divers.

- Aggregates and rock material have to be imported.
- When the subsoil is loose and there is excess pore water pressure in the subsoil, this can cause significant deformation in the foundation soil, potentially leading to a large seaward movement and settlement.
- Suitable ground improvement is required to take the bearing pressure from the caisson foundation.
- Weights of gravity units are very high and requires specialised floating equipment for transportation of the precast units and installation.
- Fabrication of caissons needs a dry dock or a submersible dock.
- A high standard of concrete production and placement is required.

5.3.2 Concrete Counterfort Retaining Wall

Disadvantages

- Additional dredging and imported rockfill is required below the structure with under water operations that also require the use of divers.
- Aggregates and rock material would have to be imported.
- Fabrication of retaining wall needs facilities for a casting yard and specialist transportation to the site.
- Weights of precast retaining wall units are very high and may have handling difficulties and require very large floating equipment

5.3.3 Concrete Block wall

Advantages

- Construction process will be faster for precast concrete blocks.
- Heavy Precast concrete blocks provides a robust maintenance free structure.
- Block wall is suitable where the seabed is of good quality, mostly suitable for rocky sea bed or very competent founding layer.

Disadvantages

- Considerable underwater works by divers.
- Additional dredging and imported rockfill is required below the structure with under water operations that also requiring use of divers.
- Suitable ground improvement maybe required to take the bearing pressure from the Block wall base unit and or the rock rubble mound.
- Weights of heavy precast blocks are high. Handling difficulties and require large floating equipment.
- Fabrication of the block wall requires facilities for casting yard.

5.3.4 Steel Sheet Piled Circular Cofferdam

Advantages

- Medium sized construction equipment can be used to drive the sheet piles.

Disadvantages

- Significant piled circular temporary structures required to drive sheet pile cofferdam.
- Piling installation impacts i.e. noise.
- Fill for the cells should be of granular material having a high bulk density to aid stability and a high angle of internal friction to provide internal shear strength and sliding resistance at the base. Aggregates and rock material for the infill have to be imported.

- Large surface area of sheet pile is exposed to severe marine environment and would require concrete skirting above the water level with cathodic protection to the entire structure to achieve the specified design life.
- Pile clutches are prone to damage in operation
- Cells do not tolerate differential settlement

5.3.5 Rectangular or Linear Steel Sheet Piled Cofferdam

Advantages

- Moderate construction equipment can be used to drive the sheet pile.
- Wall can be built in advance of berth dredging, except for pre-removal of soft sediment in vicinity of the wall.

Disadvantages

- Aggregates and rock material have to be imported.
- Piling installation impacts i.e. noise.
- Large surface area of steel pile exposed to marine environment and maintenance burden requires cathodic protection system.
- Tie rods and steel sheet piles are difficult to maintain.
- Differential settlement can potentially be an issue between tie rods and anchor wall with crane rail.

6.0 Summary and Next Steps

This report is intended to present descriptions of various structural forms that were considered based on the known site conditions at Hastings at the time writing this report for discussion with the Authority and the other disciplines with a view to shortlisting preferred options for further analysis for costing purposes. Preliminary operating loads and structural assessment were undertaken for the primary structure types in order to assess the approximate key member sizes, founding depths, dredging and backfill requirements for comparison of options.

The primary structure types that were assessed include:

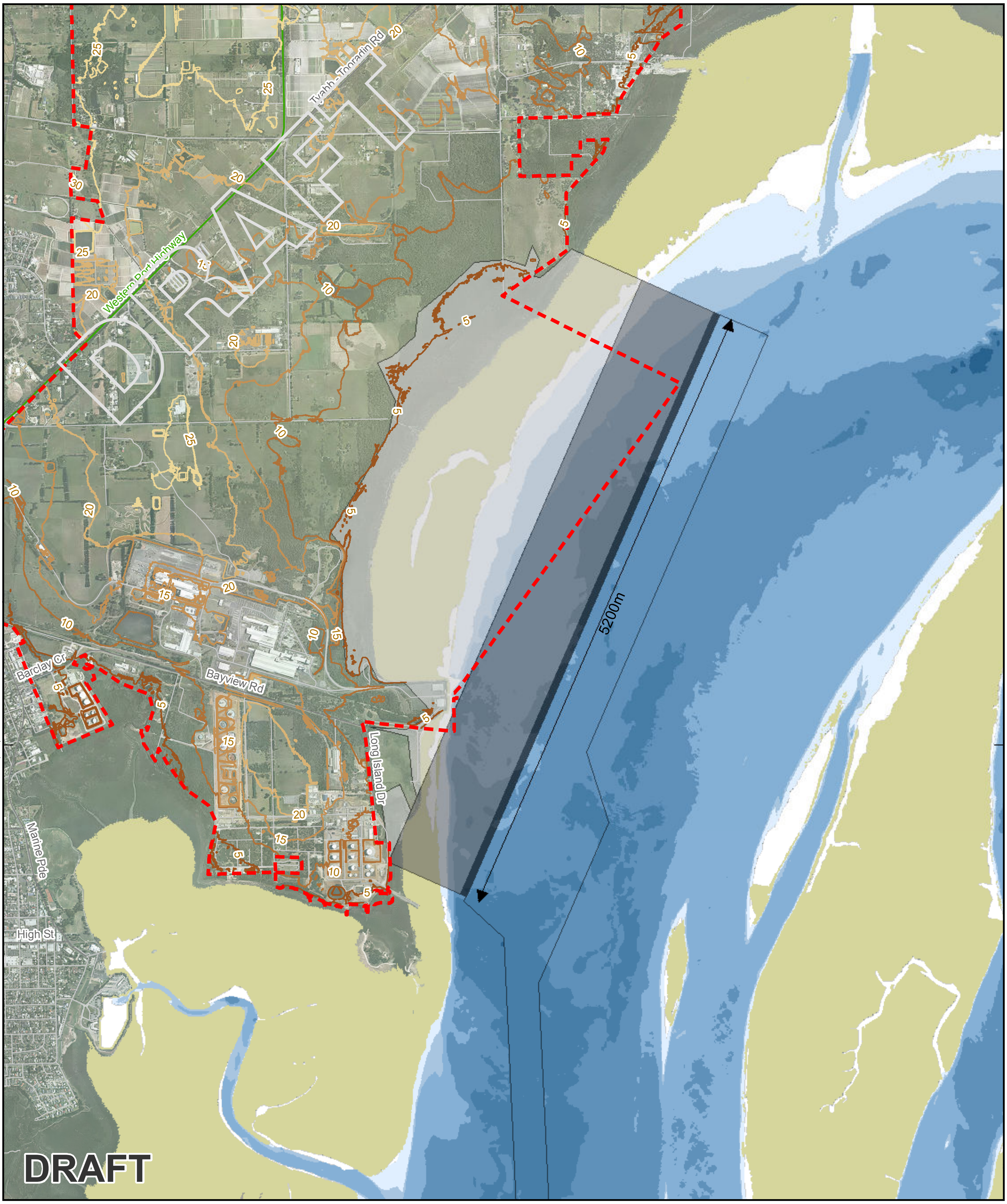
- Open piled quay deck over a sloping revetment;
- Bulkhead wall;
  - Steel pile bulkhead wall
  - Diaphragm wall
- Gravity Structures;
  - Caissons
  - Concrete counterfort retaining wall
  - Concrete block wall
  - Steel sheet piled circular cofferdam
  - Rectangular or linear steel sheet piled cofferdam

It is conceivable that considering the variability of the site geology and bathymetry for each layout and alignment and for the future stages of the layout, more than one structure type or a combination of structure types might be more appropriate. At this stage no attempt has been made to shortlist options to enable open discussion based on multi-criteria assessment with input from other relevant workstreams and reports which are in progress.

A preliminary comparison of the options has been undertaken to outline technical advantages and disadvantages of the options as a precursor for a more detailed evaluation involving the Hydrodynamic/DMM and Environment and Social workstreams. This will be qualitative evaluation of each of the wharf types taking into consideration criteria such as likely capital and maintenance cost, serviceability, residual and differential settlement, constructability, future proofing, geotechnical issues, environmental issues and dredging and reclamation considerations.

Following this evaluation a report on the concept designs and preliminary options review outlining the preferred engineering scheme derived from the list of options considered along with a broad indication of construction methodology and staging of works.

Appendix A Berth Alignments



Dimensions	Special Use Zone 1	**Contour Level (m, Chart Datum)**	10	-10 - -5
Berth Apron	Highway	30	5	-15 - -10
Channel	Arterial	25	>0	-20 - -15
Reclamation Behind Terminal	Local	20	-2 - 0	< 20
Terminal		15	-5 - -2	No bathymetry available

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Metres
Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 55

N

PORT OF HASTINGS DEVELOPMENT AUTHORITY

Port of Hastings Development Project

Full Port Development
Straight Berth Alignment

Doc No. AGH-CEP0-DE-FIG-0096
Revision B
Date 21 Jan 2015

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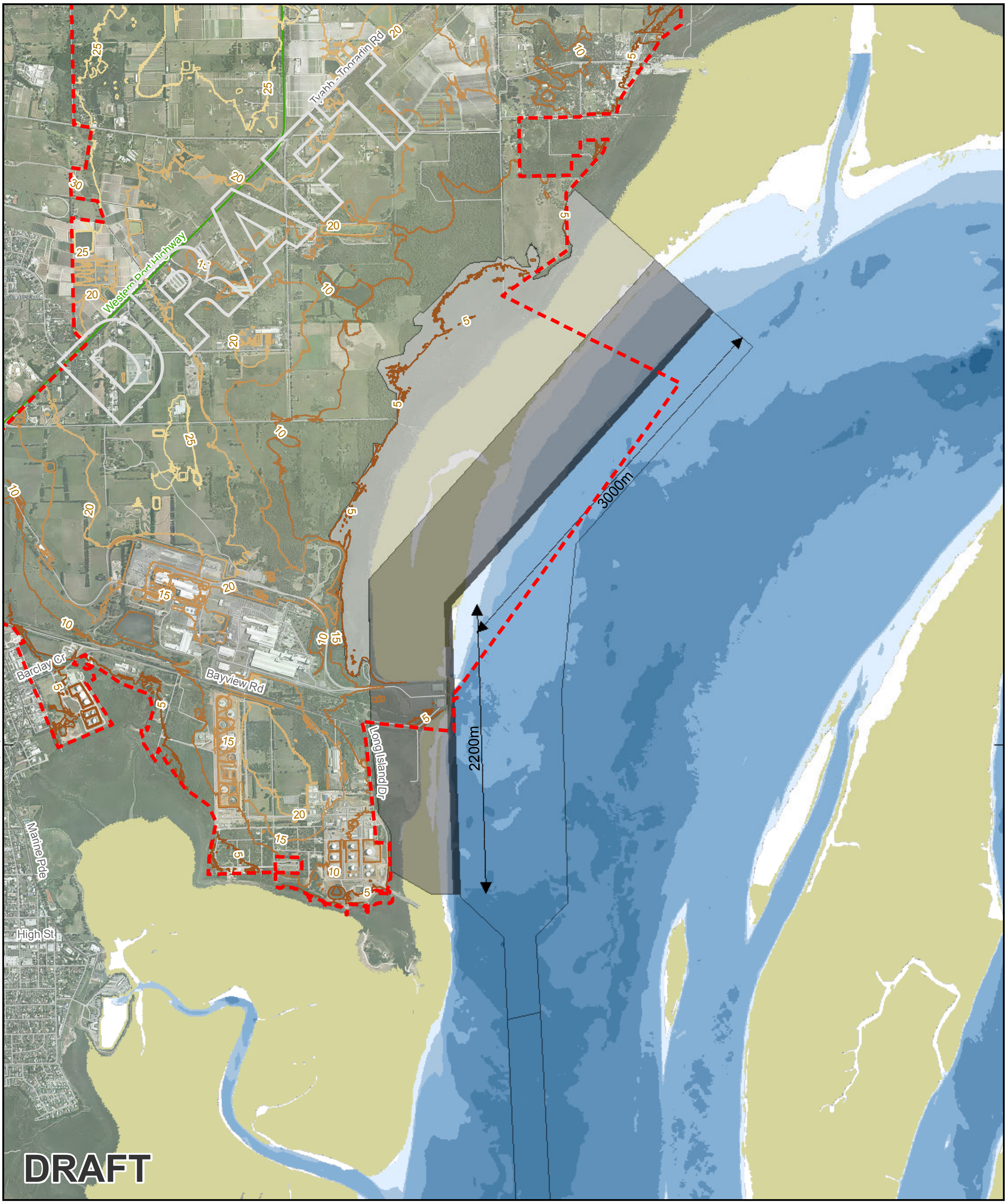
2/34 High Street Hastings VIC 3915 Australia T 1300 149 478 F 61 3 5979 5500 E enquiries@portofhastings.com W www.portofhastings.com

Data source: VicMap Background Layers(DEPI, 2014); Port Design (Aecom GHD Joint Venture, 2014) Created by: cillingworth



Berth Apron
Channel
Reclamation Behind Terminal
Terminal
Special Use Zone 1

Highway
Arterial
Local
Dimensions

Contour Level (m, Chart Datum)
30
25
20
15
10
5

>0
-2 - 0
-5 - -2
-10 - -5
-15 - -10
-20 - -15
< 20
No bathymetry available

Paper Size A3
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Metres
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N

PORT OF HASTINGS
DEVELOPMENT AUTHORITY

Port of Hastings Development Project

Full Port Development
Along the Shore Berth Alignment

Doc No. AGH-CEP0-DE-FIG-0097
Revision B
Date 21 Jan 2015

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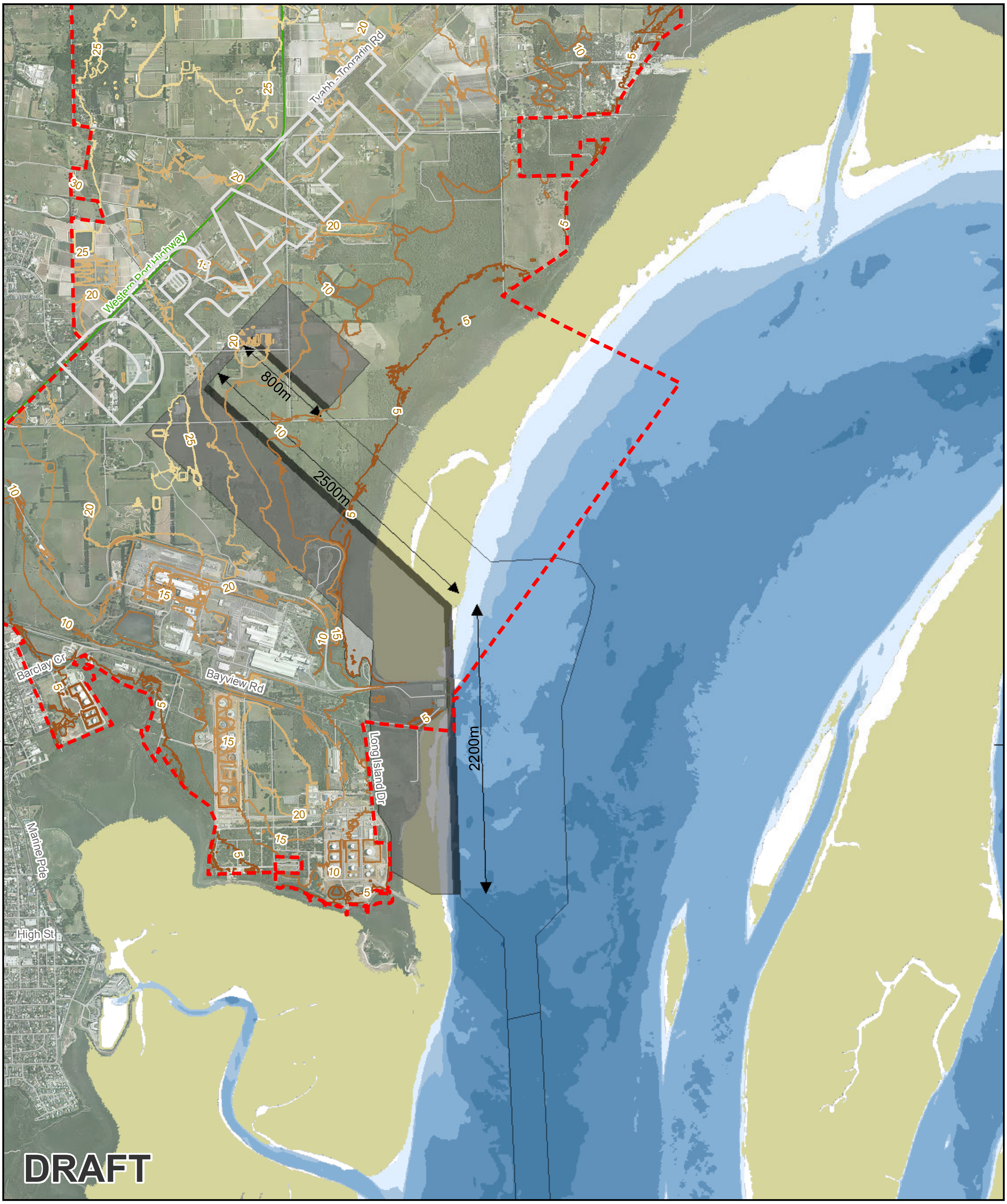
Data source: VicMap Background Layers(DEPI, 2014); Port Design (Aecom GHD Joint Venture, 2014) Created by: cillingworth



Berth Apron	Highway	**Contour Level (m, Chart Datum)**	10	-10 - -5
Channel	Arterial	30	5	-15 - -10
Reclamation Behind Terminal	Local	25	>0	-20 - -15
Terminal	Dimensions	20	-2 - 0	< 20
Special Use Zone 1		15	-5 - -2	No bathymetry available

Paper Size A3
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Metres
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N

PORT OF HASTINGS
DEVELOPMENT AUTHORITY

Port of Hastings Development Project

Full Port Development
Basin Berth Alignment

Doc No. AGH-CEP0-DE-FIG-0098
Revision B
Date 21 Jan 2015

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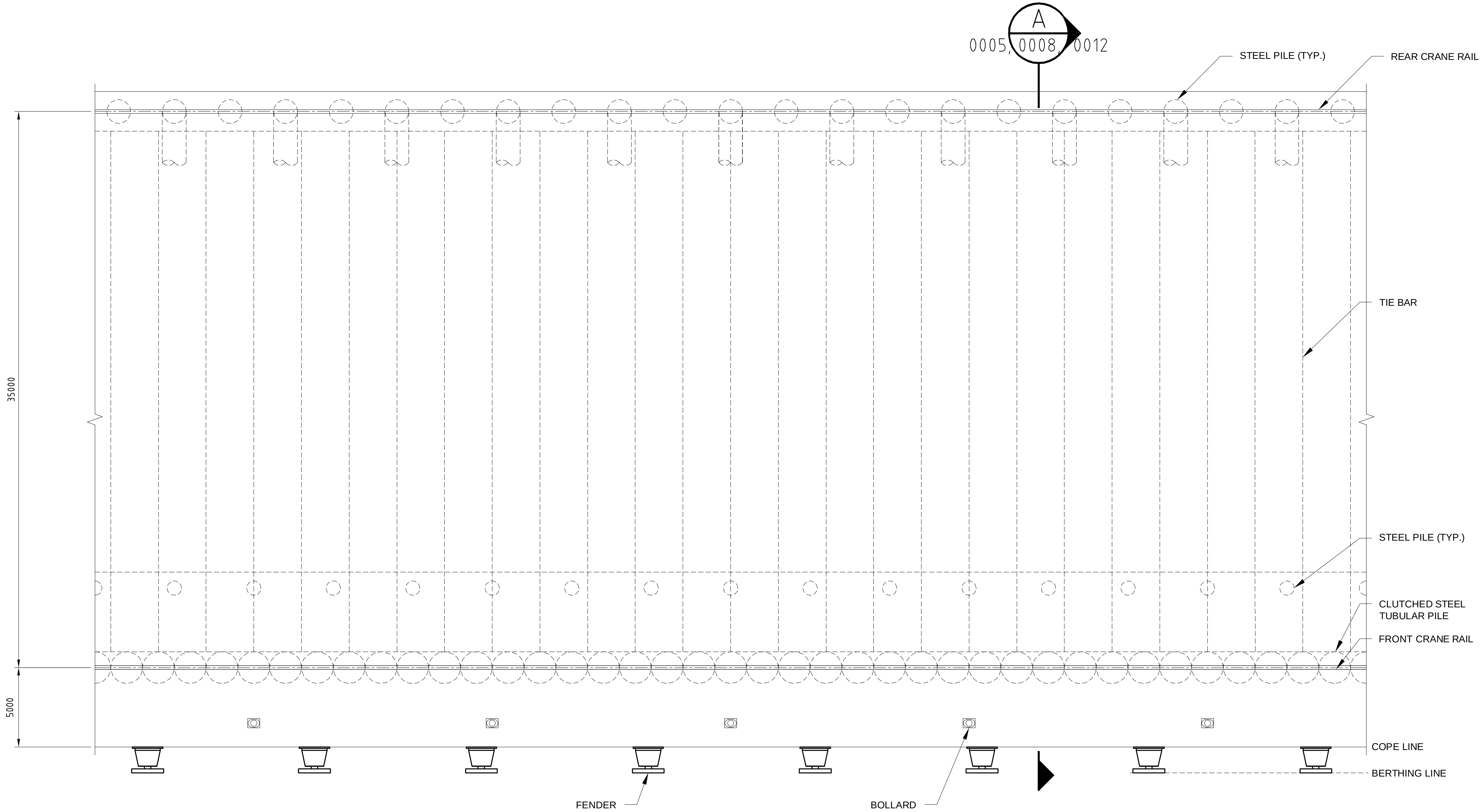
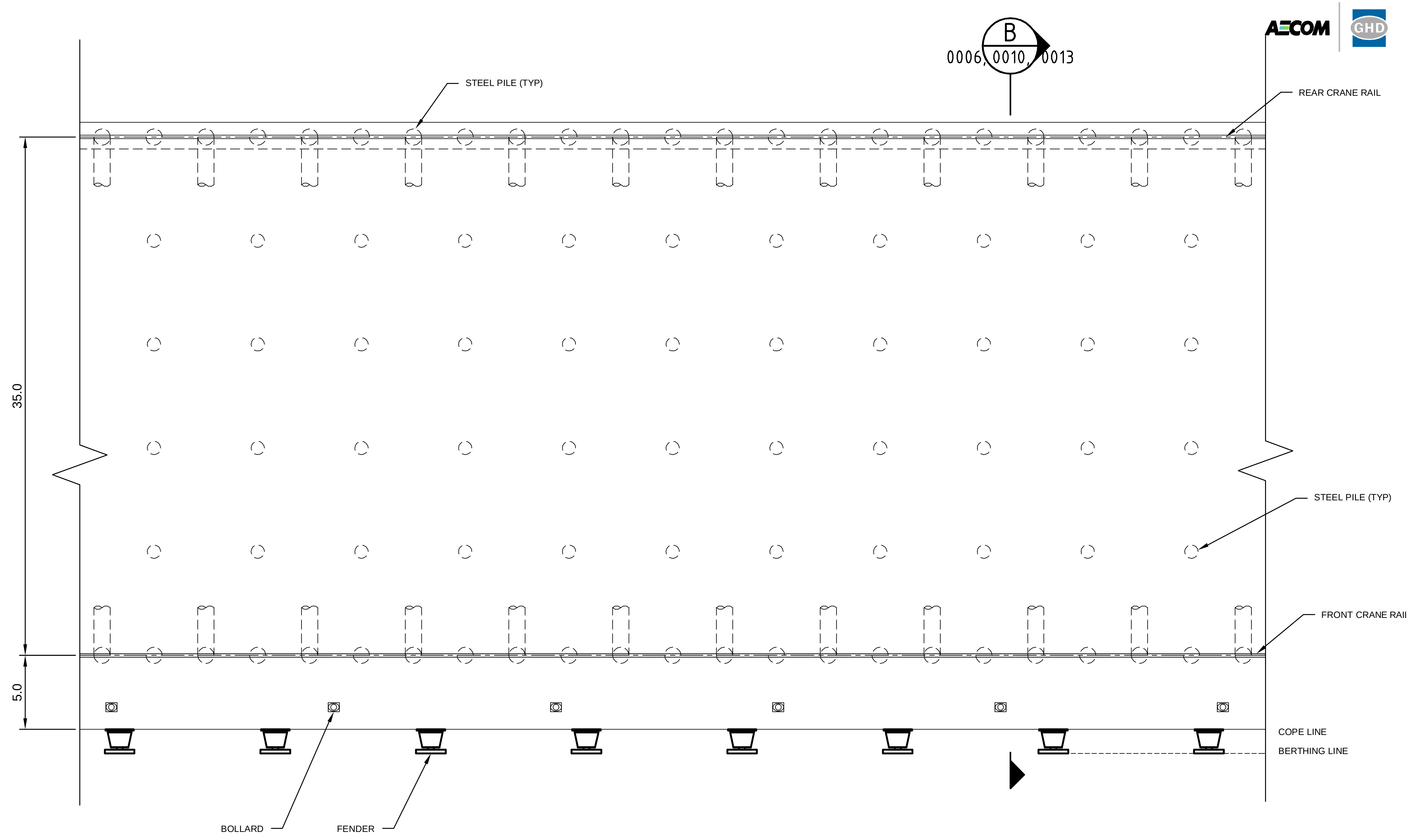
Appendix B Concept Drawings



STEEL PIPE PILE BULKHEAD WALL - PLAN

(STAGE 1A - ESSO TO BLUESCOPE)
(STAGE 2A - BASIN OPTION)
(STAGE 2E - COASTLINE OPTION)

Client: PORT OF HASTINGS DEVELOPMENT AUTHORITY
Project: PORT OF HASTINGS DEVELOPMENT PROJECT
Status: DRAFT CONCEPT
Sketch: AGH-CEP0-EG-SKE-0001



OPEN PILE QUAY DECK - PLAN

(STAGE 1B - ESSO TO BLUE SCOPE)
(STAGE 2C - BASIN OPTION)
(STAGE 2F - COASTLINE OPTION)

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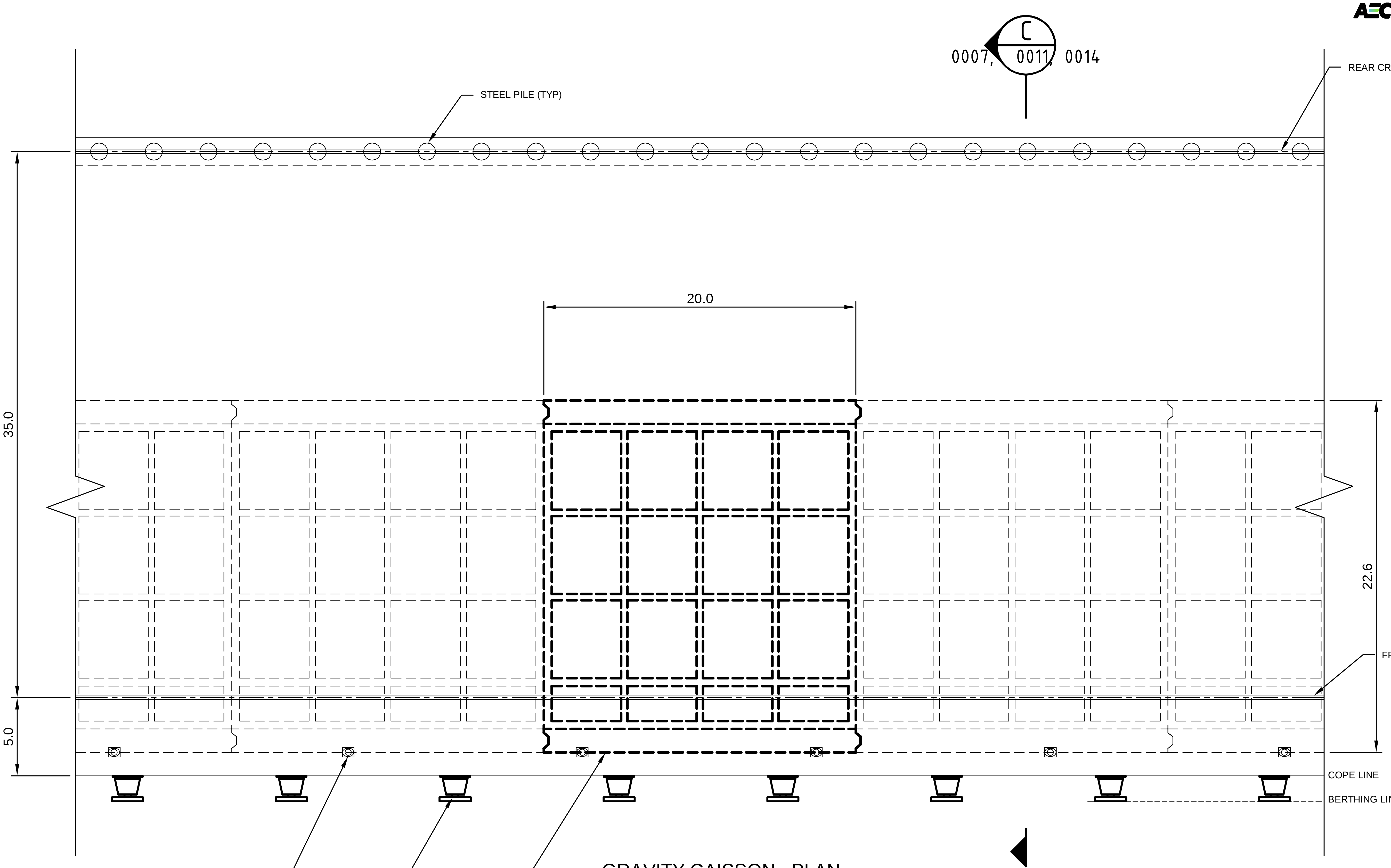
Project: PORT OF HASTINGS DEVELOPMENT PROJECT
Status: DRAFT CONCEPT
Sketch: AGH-CEP0-EG-SKE-0002



GRAVITY CAISSON - PLAN

(STAGE 1C - ESSO TO BLUE SCOPE)
(STAGE 2D - BASIN OPTION)
(STAGE 2G - COASTLINE OPTION)

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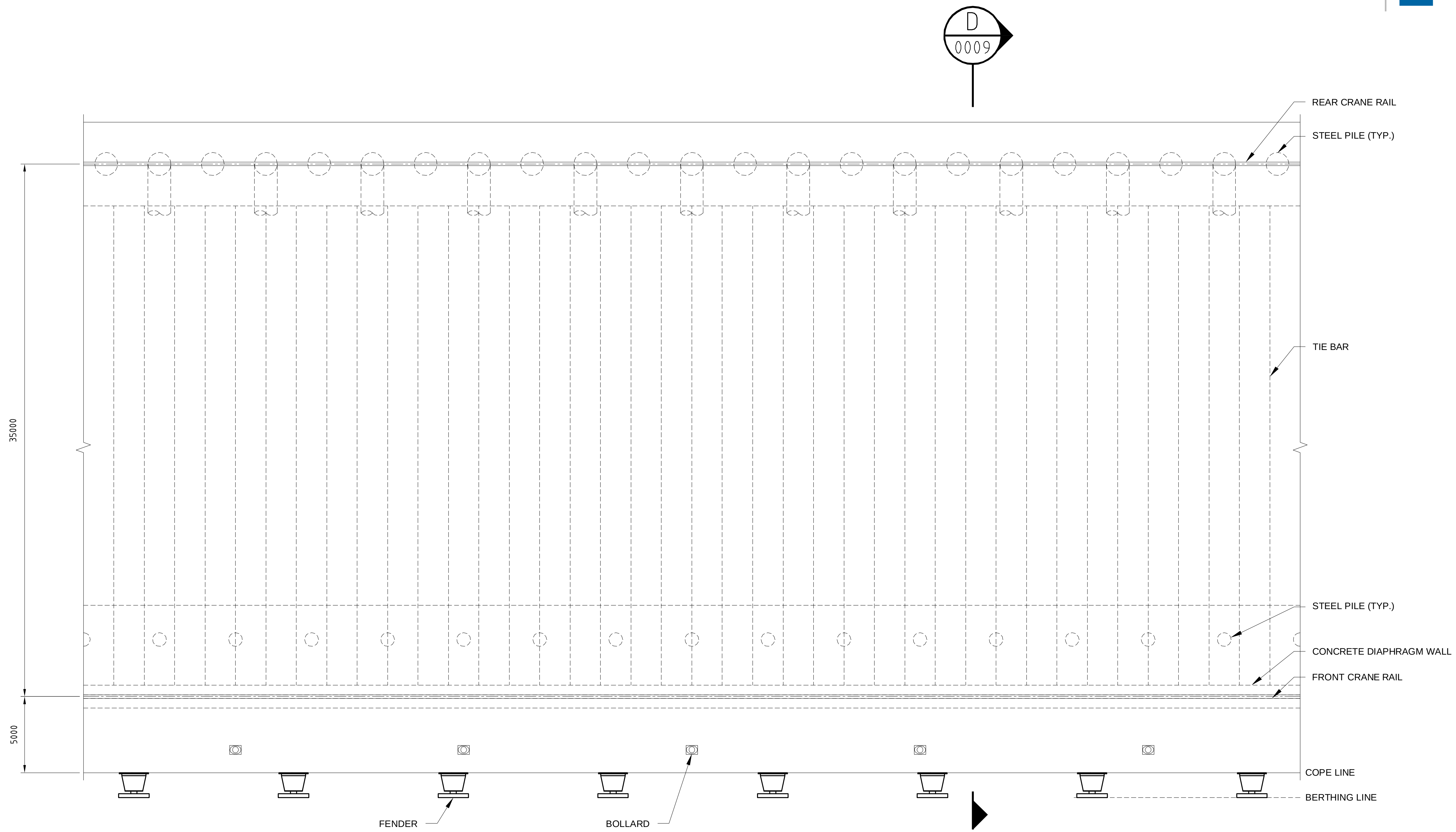
Project: PORT OF HASTINGS DEVELOPMENT PROJECT
Status: DRAFT CONCEPT
Sketch: AGH-CEP0-EG-SKE-0003



DIAPHRAGM WALL - PLAN

(STAGE 2B - BASIN OPTION)

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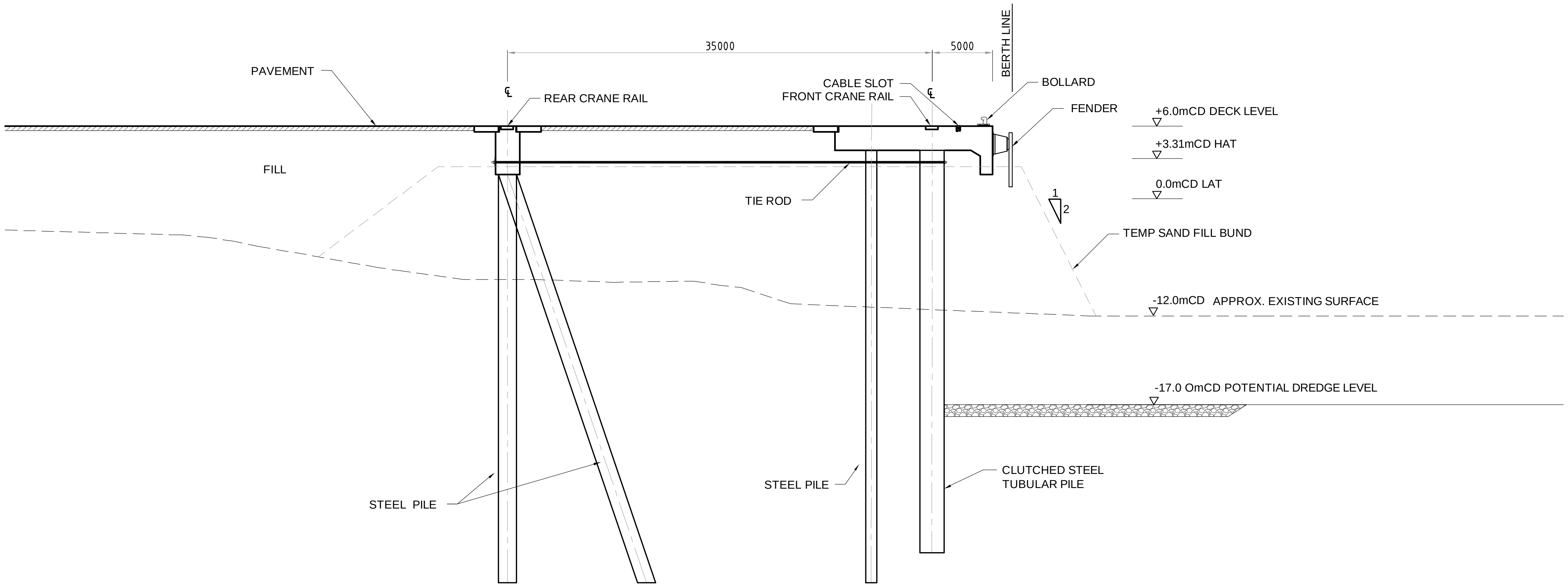
Project: PORT OF HASTINGS DEVELOPMENT PROJECT
Status: DRAFT CONCEPT
Sketch: AGH-CEP0-EG-SKE-0004



STEEL PIPE PILE BULKHEAD WALL - SECTION

(STAGE 1A - ESSO TO BLUESCOPE)

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Project: PORT OF HASTINGS DEVELOPMENT PROJECT
Status: DRAFT CONCEPT
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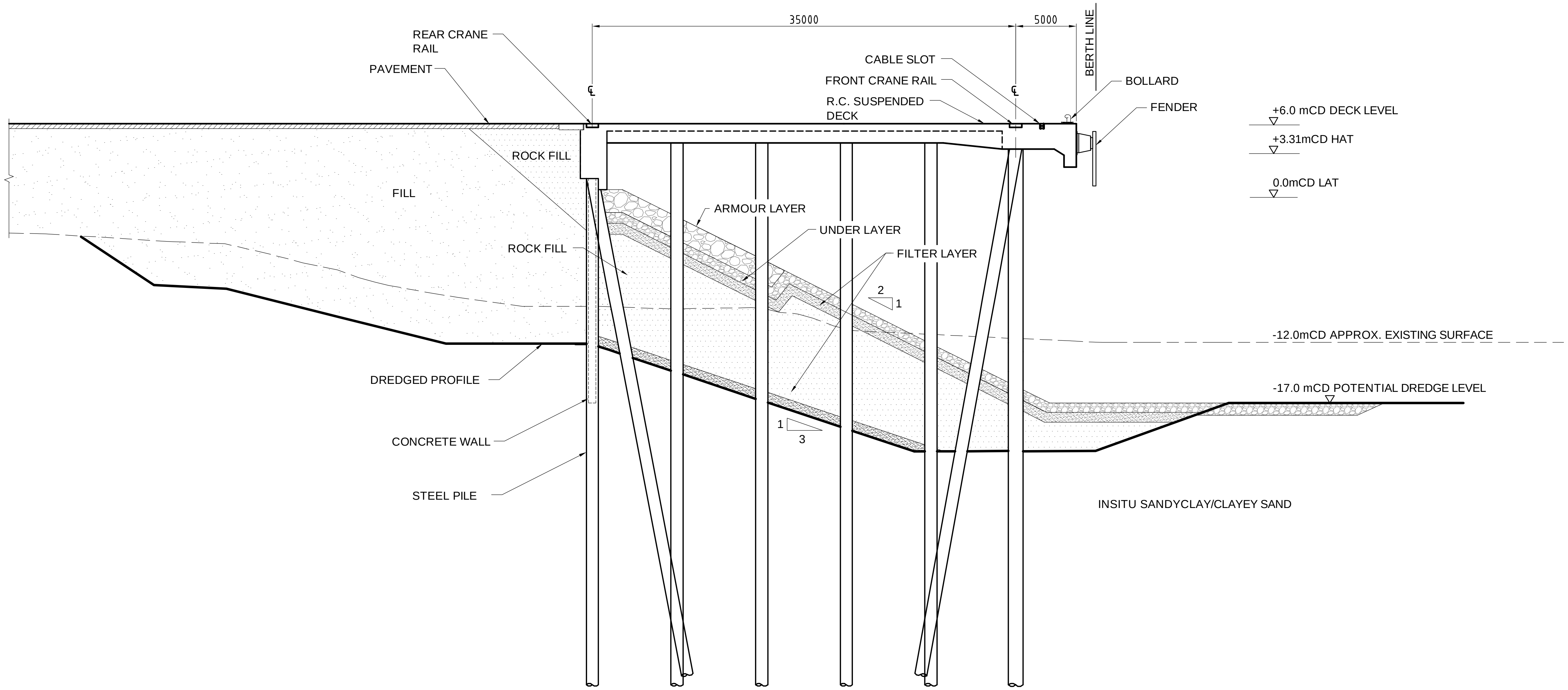

PILED QUAY DECK AND ROCK REVETMENT - SECTION

(STAGE 1B - ESSO TO BLUE SCOPE)

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Project: PORT OF HASTINGS DEVELOPMENT PROJECT
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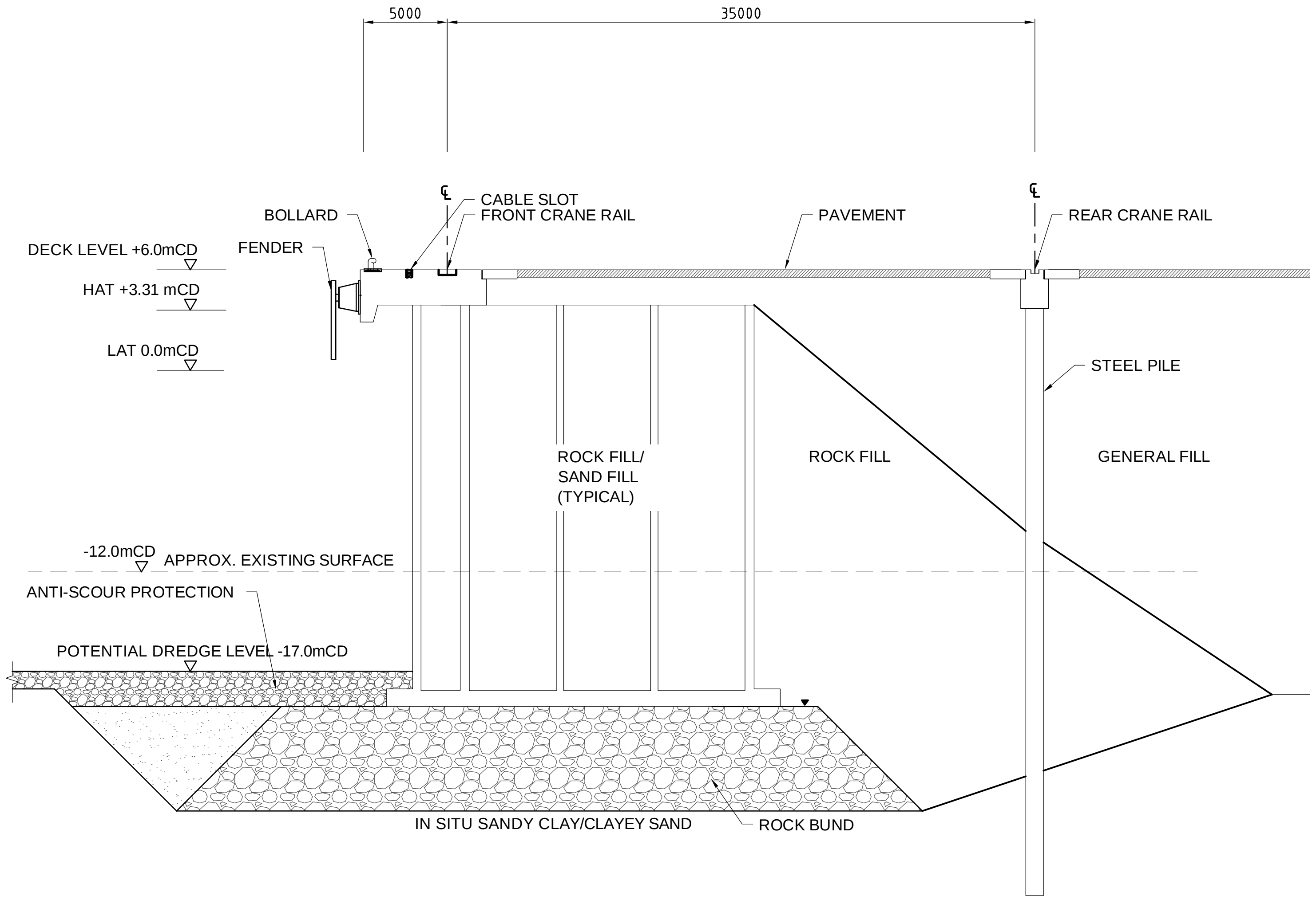

GRAVITY CAISSON - SECTION

(STAGE 1C - ESSO TO BLUE SCOPE)

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Project: PORT OF HASTINGS DEVELOPMENT PROJECT
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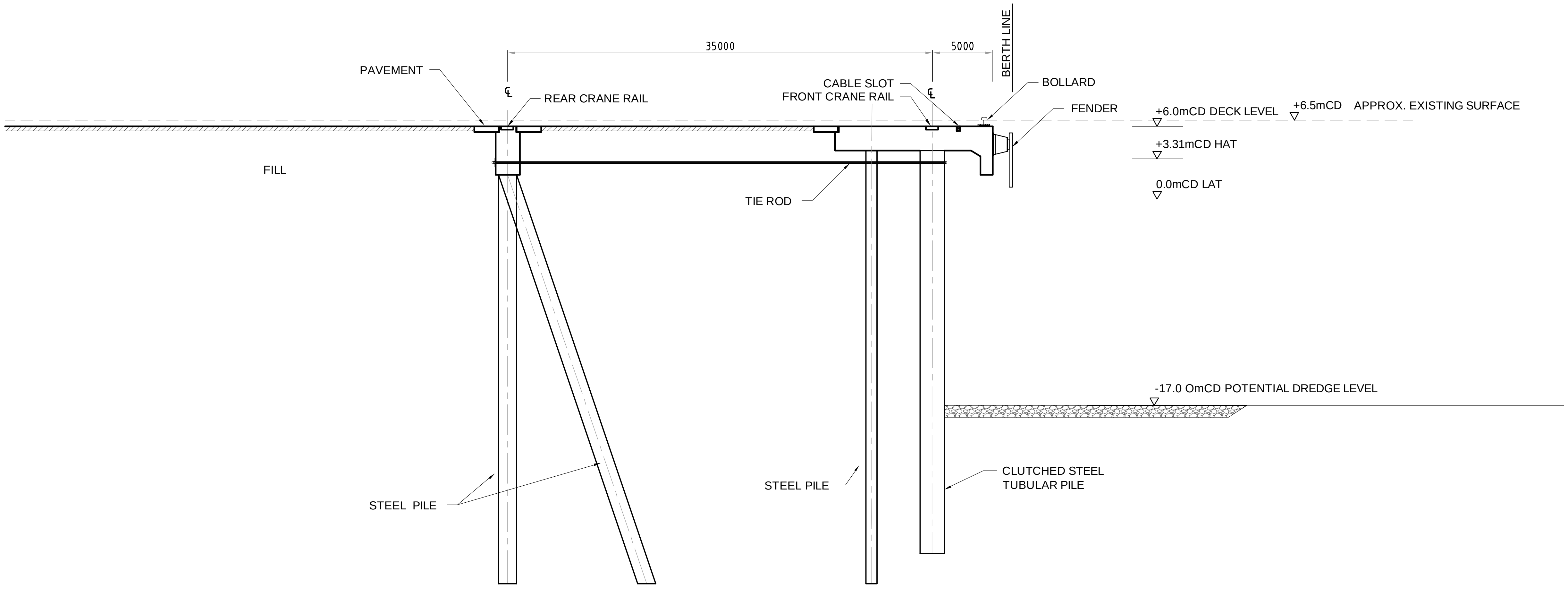
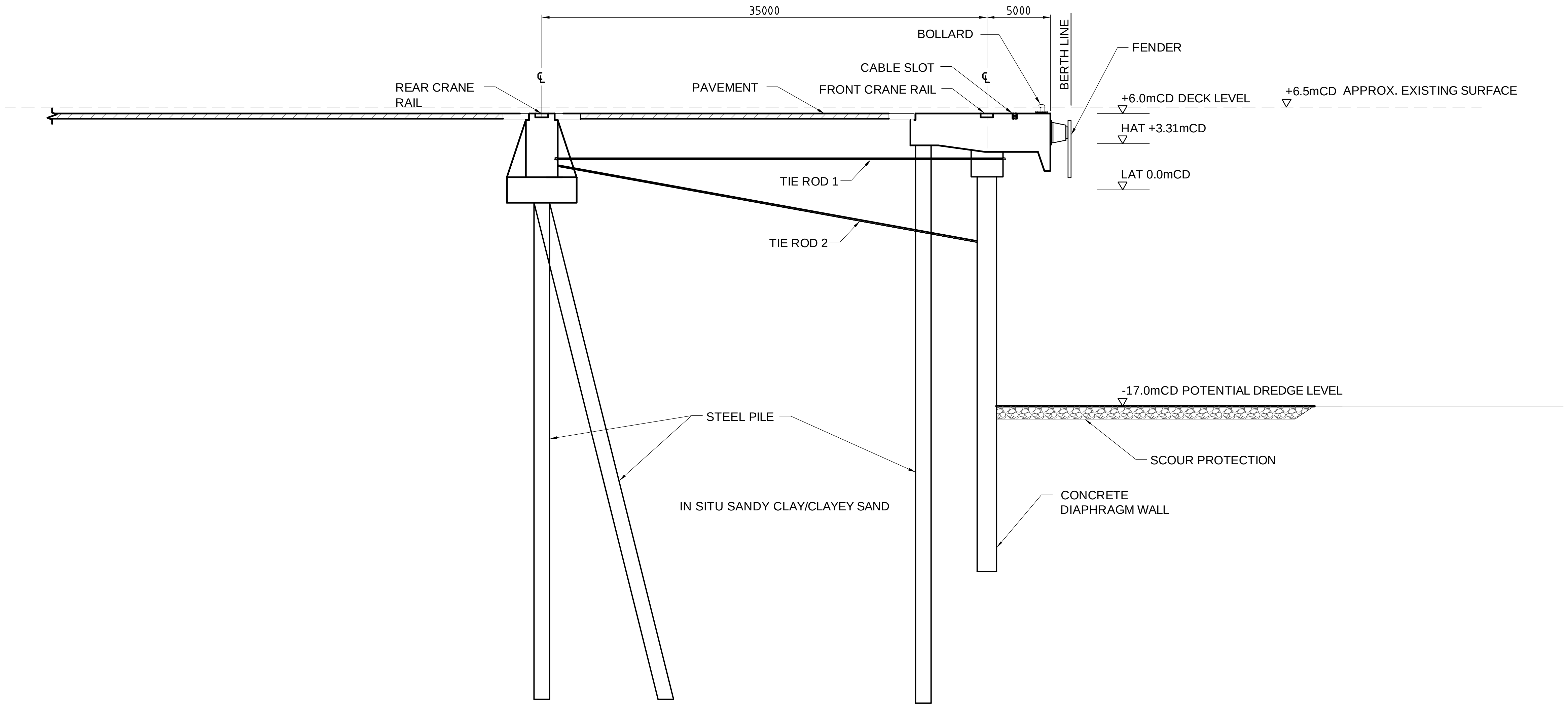


STEEL PIPE PILE BULK HEAD WALL - SECTION

(STAGE 2A - BASIN OPTION)



DIAPHRAGM WALL - SECTION

(STAGE 2B - BASIN OPTION)

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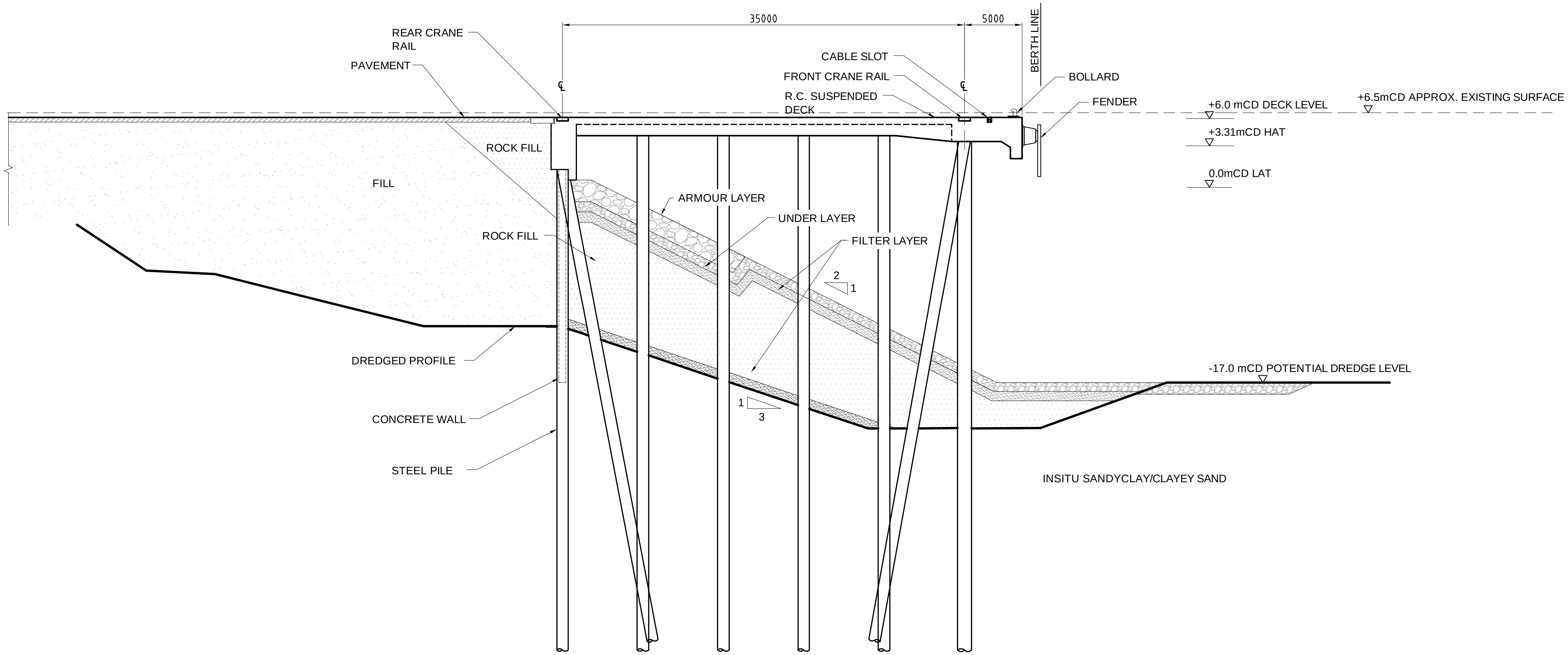

PILED QUAY DECK AND ROCK REVETMENT - SECTION

(STAGE 2C - BASIN OPTION)

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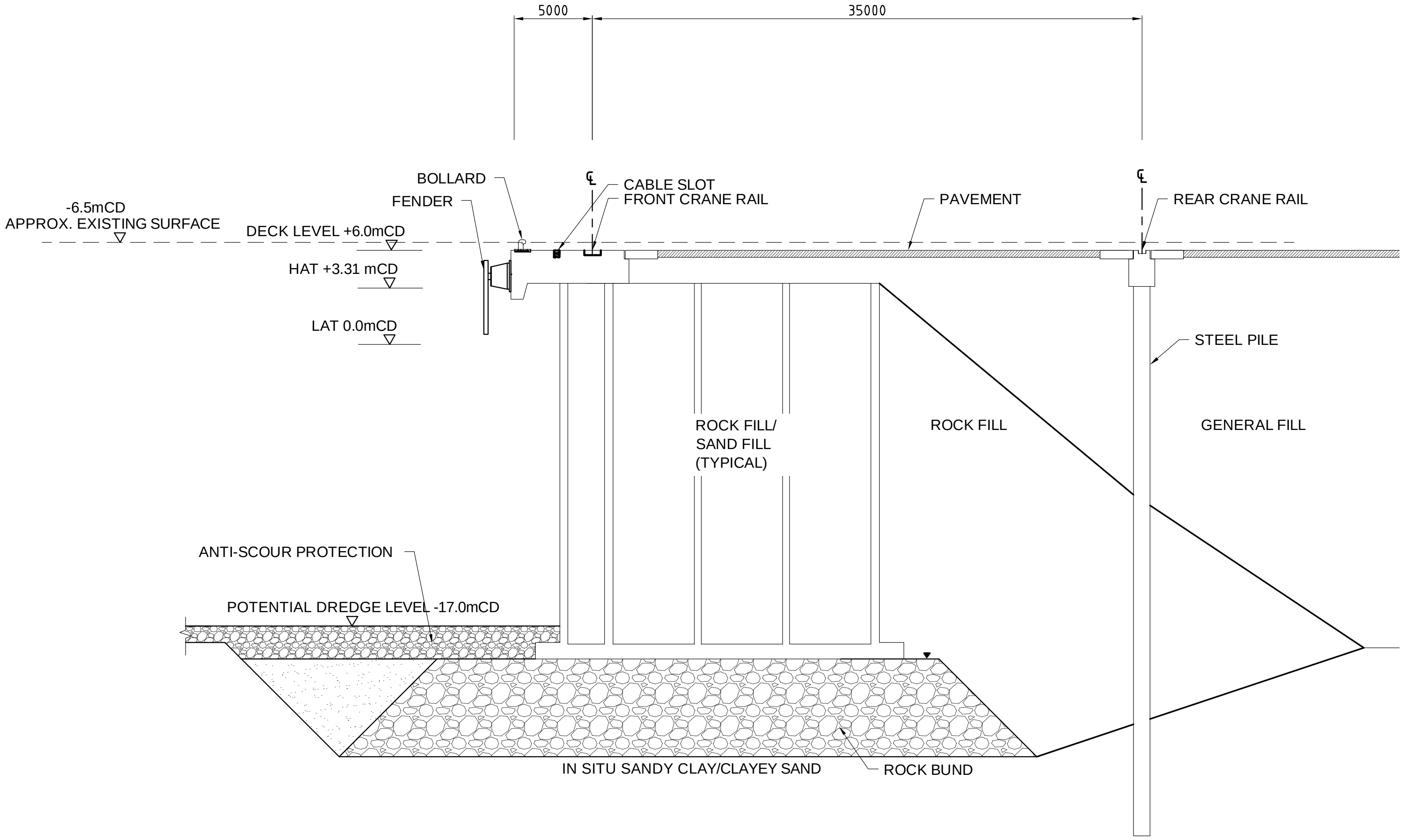

GRAVITY CAISSON - SECTION

(STAGE 2D - BASIN OPTION)

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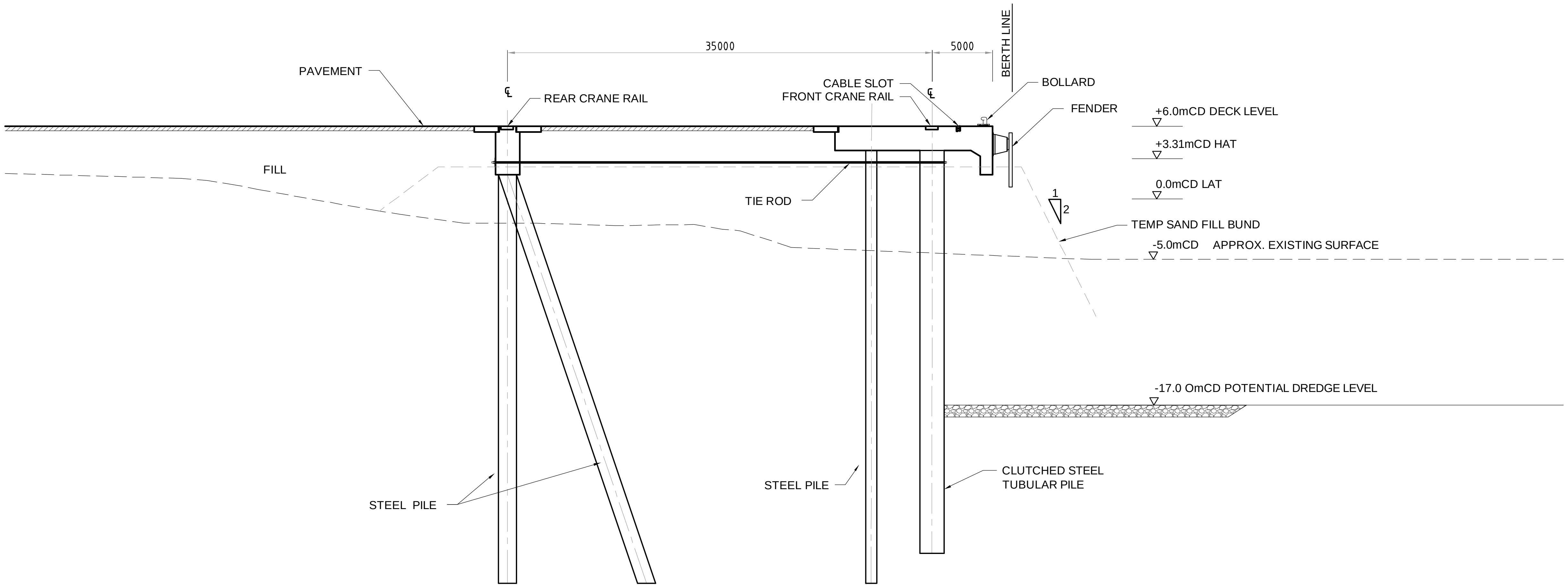

STEEL PIPE PILE BULKHEAD WALL - SECTION

(STAGE 2E - COASTLINE OPTION)

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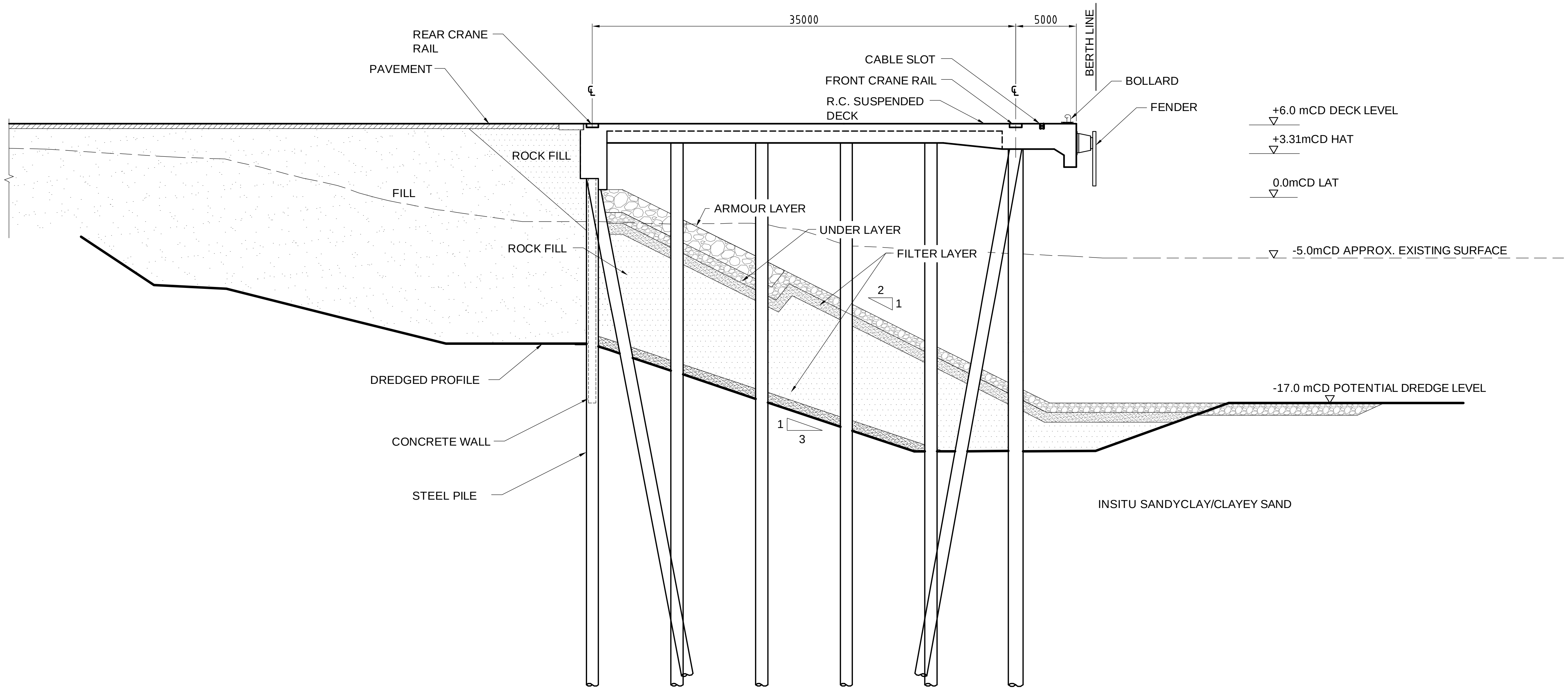

PILED QUAY DECK AND ROCK REVETMENT - SECTION

(STAGE 2F - COASTLINE OPTION)

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Project: PORT OF HASTINGS DEVELOPMENT PROJECT
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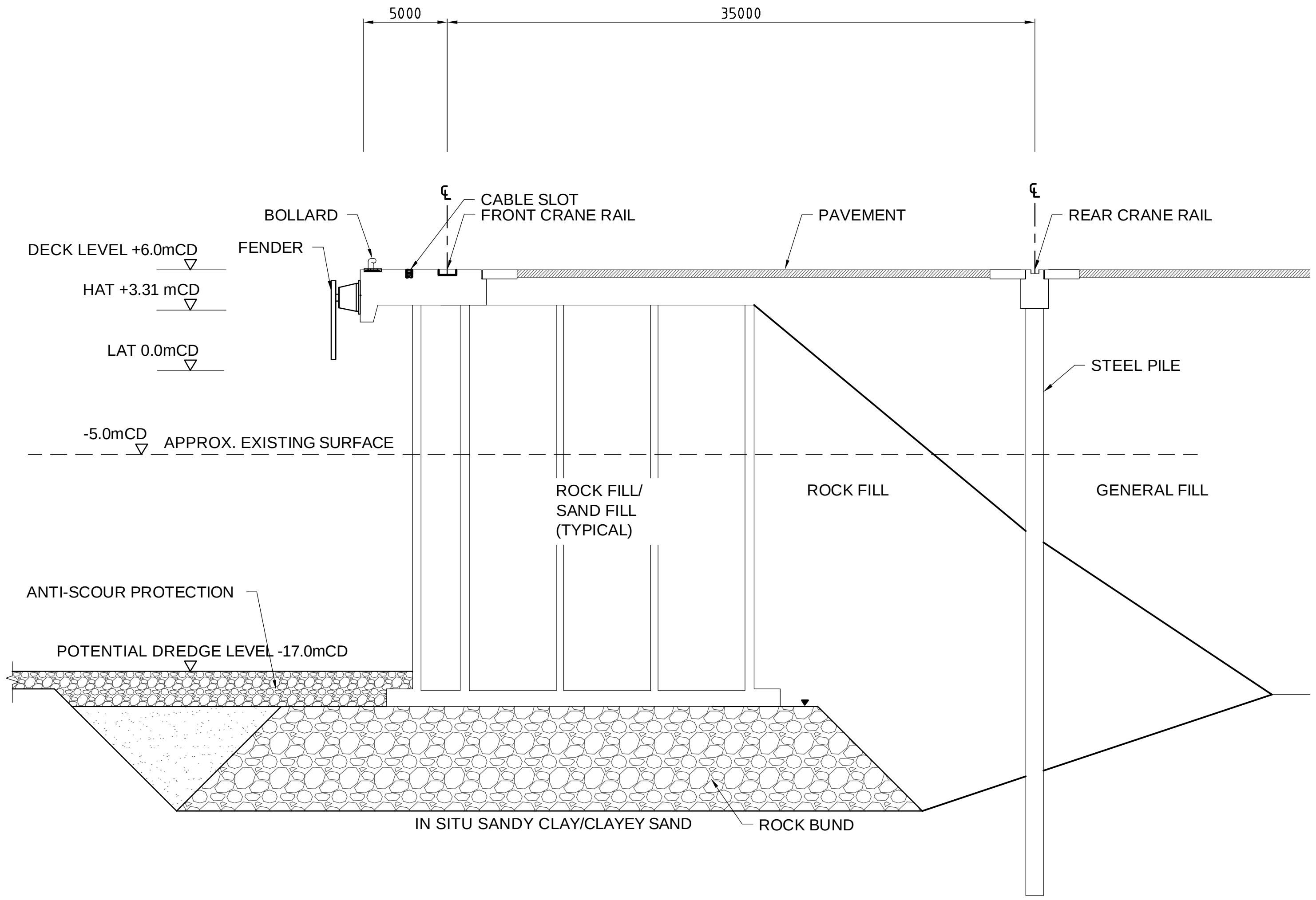

GRAVITY CAISSON - SECTION

(STAGE 2G - COASTLINE OPTION)

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Status: DRAFT CONCEPT
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