Catherine Hill Bay Jetty

Engineering Assessment of Various Retention Options

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Version B

Contents

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1. Executive Summary

BG&E has completed an engineering assessment of various retention options for Catherine Hill Bay Jetty following the 2023 structural investigations and subsequent 2025 community consultation. The assessment considers three retention options, a revised estimate for full demolition, and further investigation of the internal condition of the steel Circular Hollow Section (CHS) piles. The purpose of this work is to identify whether elements of the existing jetty can be safely retained for visual and heritage interpretation purposes while managing structural, durability, maritime safety, environmental and cost risks.

The existing CHS piles are generally intact and remain structurally active elements, with no reported through-wall perforation, buckling, or global instability at the tested locations. However, the broader steelwork is affected by significant deterioration, particularly at connections, bracing members, bearing plates, splice locations and other details exposed to the tidal, splash and atmospheric marine zones. The retained structure would therefore require substantial repair, strengthening and durability intervention to achieve a 25-year residual life.

Option 1, comprising retention of the CHS piles only without bracing, is not considered structurally feasible. Removal of the lateral and longitudinal restraint systems changes the piles to an unbraced cantilever system. The capacity of the reinforced concrete portion of the pile is predicted to fail first, just below the tow of the steel, CHS portion of the pile. The section capacity of the CHS portion of the pile similarly fails due to the larger bending moment in the CHS pile but would occur after the concrete portion has failed. Cutting the piles down sufficiently to reduce the cantilever demand would place the pile tops approximately at low tide level, which would reduce visibility, undermine the purpose of retention, and introduce maritime safety concerns.

Option 2, comprising retention of the CHS piles, primary steel and bracing with removal of the precast concrete deck, secondary members and tertiary members, is the only assessed retention option found to be structurally viable in principle. This option requires retention, repair, and strengthening of the lateral bracing system, CHS vertical bracing, CHS tie beams, primary beams and key connections. Additional bracing is required between Bents 4 and 5 and between Bents 11 and 12 to restore restraint lost through removal of the deck and plan bracing. The assessment indicates that, with these works completed, the CHS piles, vertical bracing and raked piles have adequate capacity under the adopted ultimate limit state load cases.

Option 3, comprising retention of Bents 1, 2 and 3 and associated bracing only, is not considered structurally feasible in the configuration assessed. Removal of the longitudinal restraint provided by the raked piles between Bents 4 and 5 results in the reinforced concrete portion of the pile exceeding its bending capacity under longitudinal loading. Retention of Bents 1 to 3 may only be viable if Bents 4 and 5 and the associated raked piles are also retained, subject to further structural analysis and likely additional plan bracing.

For Option 2, the proposed durability strategy comprises galvanic anode cathodic protection for the submerged zone, petroleum tape and HDPE jacket wrapping for the tidal zone, and a high-build epoxy/polyurethane coating system for the atmospheric zone and retained horizontal steelwork. This approach is intended to achieve a 25-year residual life, provided routine inspection and maintenance are implemented. Key maintenance requirements include a full potential survey within one year of installation, periodic five-year diver or ROV inspections, two-year visual checks of tidal wraps, local coating touch-ups every five to seven years, and management of marine growth on anodes.

The Class 4 quantity surveying estimate for Option 2 indicates an initial capital cost of approximately $12.98 million, ongoing maintenance over 25 years of approximately $10.16 million, and end-of-life demolition of approximately $15.23 million. The resulting estimated lifecycle cost is approximately $38.36 million. By comparison, the revised Class 4 estimate for full demolition to the seabed is approximately $11.09 million, excluding future escalation.

Several material risks remain for any retention option. These include reliance on assumed seabed fixity and geotechnical capacity, exposure to tides and swell during construction, potential coating failures from flash rust and overcoating delays, environmental risks associated with corrosion inhibitor displacement and coating application, salt accumulation on horizontal steel members, and the need to maintain maritime exclusion through marker buoys and vessel prohibition signage. Further geotechnical investigation is recommended if any retention option proceeds to detailed design.

Internal CHS pile investigations confirmed the presence of an oil-like corrosion inhibitor in the piles at varying levels. Laboratory analysis identified the sample as a bitumen oil containing heavier hydrocarbons and some volatile organic compounds. Because the original Magnacote product is no longer available and safety information is unlikely to be obtained, any displacement, disposal or treatment of this material would need to be carefully managed. BG&E recommends filling the CHS piles, if retained and maintained, with concrete or grout to passivate the internal steel and reduce future internal corrosion risk, noting this would require management of approximately 130 cubic metres of fill and displacement of the existing inhibitor. If the piles are not retained, then the oil may be displaced with water prior to dismantling.

In summary, Option 1 and Option 3 are not recommended in their assessed forms because they do not provide adequate structural capacity. Option 2 is the only retention option assessed as structurally viable, but it carries substantial capital, maintenance, end-of-life demolition, constructability and environmental obligations. If retention is the preferred project outcome, BG&E recommends progressing Option 2 only after confirming seabed conditions, developing the repair and bracing design, resolving environmental controls for internal pile contents, and confirming that the lifecycle cost and ongoing maintenance commitments are acceptable to the asset owner.

2. Introduction

Following on from extensive investigations, structural engineering assessment of Catherine Hill Bay jetty in 2023, and community consultation in 2025, BG&E has conducted additional assessment of various retention options.

These options include:

- Option 1 – Retention of the Steel Circular Hollow Section Piles Only
- Option 2 – Retention of Piles, Primary Steel, Bracing, and Removal of Precast Concrete Deck
- Option 3 – Retention of Three Bents, Bent 1, 2, & 3, and Associated Steel Bracing

Furthermore, BG&E conducted the following works:

- Revision of cost estimates to fully demolish the jetty.
- Investigation of the internal condition of the CHS piles and determine if they are filled with corrosion inhibitor or not.

3. Background Information

The following summarises findings to date based on an extensive study in 2023 and a follow-up visual assessment in 2025.

- The CHS piles are generally intact but show localised coating failure, blistering and minor measurable wall loss, with no evidence of global instability or through-thickness perforation at the tested locations.
- The CHS piles carry large bearing plates and platforms at their heads, which exhibit significant corrosion and lamination at the welds and plate surfaces.
- Below water, piles are heavily fouled with marine growth; once cleaned they show discrete blistering and breakdown of the original coating rather than uniform bare steel exposure.
- Local pitting and lamination are present, particularly around attachments and within the tidal/splash zone, but there is no reported pile buckling, ovalisation, or gross deformation indicative of imminent failure.
- Underwater ultrasonic thickness measurements on three representative piles indicate wall thicknesses mostly around 19–19.7 mm for piles at Bents 1 and 5, with a thinner original section (approximately 14–14.5 mm) at Bent 11, suggesting different original wall sizes during construction rather than severe corrosion loss at those locations.
- The 2023 report notes that, because original wall thicknesses are not documented, precise percentage section loss cannot be quantified; however, measured values are consistent around the circumference at each elevation, implying localised rather than widespread deep wall loss.
- The piles remain structurally active elements; there is no recorded through-wall perforation or local buckling, and corrosion is described as local blistering and pitting rather than full-section compromise in the tested piles.
- From Bent 17 back to Bent 1, the steelwork is generally structurally present but affected by widespread corrosion at connections, bracing, and exposed details, with main girders (primary steel elements) typically in better condition than secondary elements.
- Primary beams are generally in good condition along their spans, but corrosion and section loss are evident at beam-to-column base plate, and splice connections, and many bolted connections are no longer fit for purpose.
- Secondary beams are similarly sound along their length but show local coating breakdown and corrosion at joints within the deck and at connections; ternary beams have notable bottom-flange lamination and heavily corroded connections.
- Flat/angle diagonal bracing between bents is often completely corroded, failed or missing, so the bracing system from Bent 17 to Bent 1 is substantially compromised.
- Circular Hollow Section (CHS) bracing shows coating loss, blistering and corrosion at connections, with lamination at the lower ends and section loss in stiffener plates; CHS brace splices are also corroded around the circular plates and nuts.

4. Option 1 - Retention of the Steel Circular Hollow Section Piles Only

This section summarises the structural engineering, construction engineering, and durability works required to achieve the proposed Catherine Hill Bay Jetty - Exploration of Retention Options, Option 1 - Retention of Vertical CHS Elements without Bracing.

The first option explored to address potential retention options involves the full demolition of the existing “landside” portion of the jetty to the movement joint at Bent 17, where vertical universal columns change to circular hollow section piles. Then, further demolition of the remaining jetty from Bent 17 to Bent 1 retaining only the vertical circular hollow sections without any bracing as illustrated in Figure 1 and Figure 2 below.

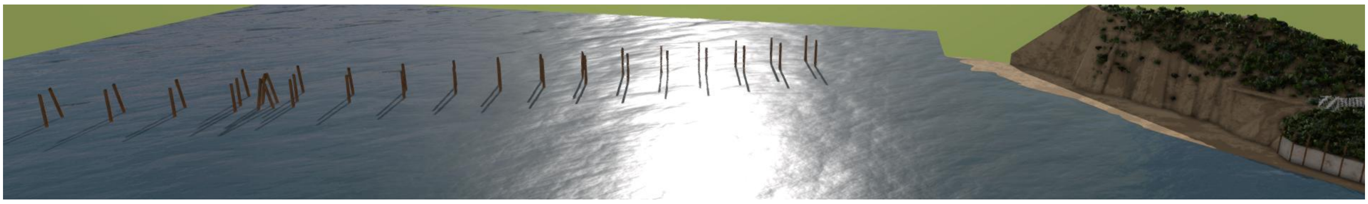


Figure 1 Visualisation of Retained CHS Piles North Looking South

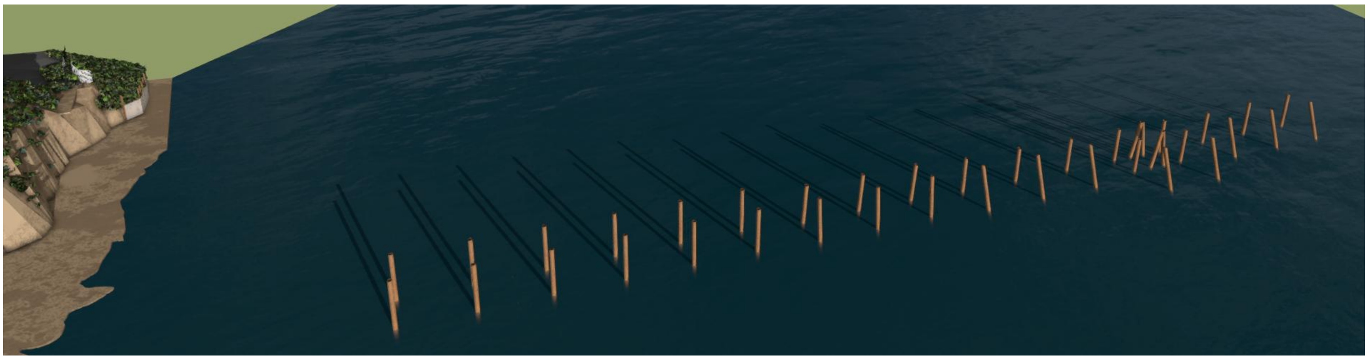


Figure 2 Visualisation of Retained CHS Piles South Looking North

Photo 1 Example Photo of Jetty

Photo 2 Example Photo of CHS Corrosion at Primary Beam Interface

Photo 3 Example of Underwater CHS Condition

Photo 4 Example of Underwater CHS Condition

4.1 General Arrangement

The following Figure 3 is provided to visualise the steel pile, reinforced concrete section, and embedment.

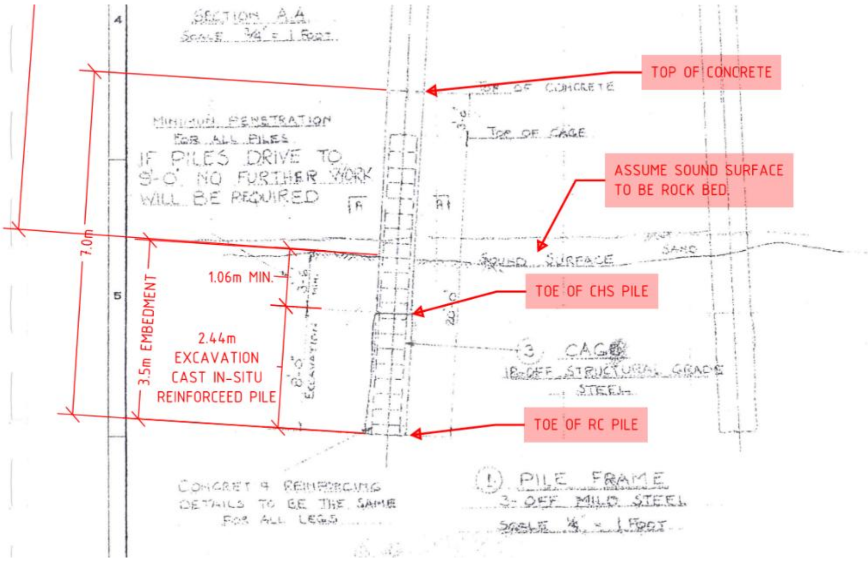


Figure 3 Visualisation of Pile Structural System

4.2 Structural Engineering Review

The commentary provided in 2023 is still relevant to this structural review and the capacity of elements has likely decreased over the last few years. To summarise the structural engineering review in 2023:

Key information for the steel circular hollow sections at Bents 1–17 is:

- For the 760 CHS vertical piles (Bents 1–17), the 2023 report finds they remain structurally adequate but with notable durability concerns.
- The piles are 760 mm CHS socketed into rock and infilled with reinforced concrete to about 7 metres above the toe of the CHS. Refer to Figure 4 below. The measured wall thicknesses are around 18.9–19.9 mm at Bents 1 and 5 and around 14.1–14.5 mm at Bent 11.
- External faces generally show scattered local corrosion blistering (about 100 mm by 100 mm patches) mostly in fair condition, with more severe corrosion concentrated at welded CHS brace/tie connections.
- CHS piles were historically managed with internal corrosion inhibitor, but the report notes this regime has lapsed and the actual internal condition has been verified in Section 8 of this report.
- The internal voids are expected to be at elevated corrosion risk due to loss of inhibitor, with internal steel and possibly the concrete infill–steel interface potentially experiencing accelerated section loss that is not yet quantified.

4.2.1 Basis of Structural Assessment

The main criterion for the structural assessment is that the CHS piles are to remain intact to the seabed under foreseeable ultimate limit state loads. Since the structure will not be serviceable, no serviceability assessments have been made, i.e. deflections and movements.

4.2.2 Jetty Lateral and Longitudinal Bracing System – Current

4.2.2.1 Lateral bracing system (across the jetty, perpendicular to the length of the jetty)

The current jetty typical lateral bracing system at each bent consists of k-bracing between the CHS piles. The lateral loading in this direction is shared between the two CHS piles via the K-bracing.

4.2.2.2 Longitudinal bracing system (along the jetty, parallel to the length of the jetty)

The current jetty longitudinal bracing system along the length of the jetty relies primarily on the strutting provided by the primary beams to transfer the load to the raked CHS piles located between bent 4 and 5.

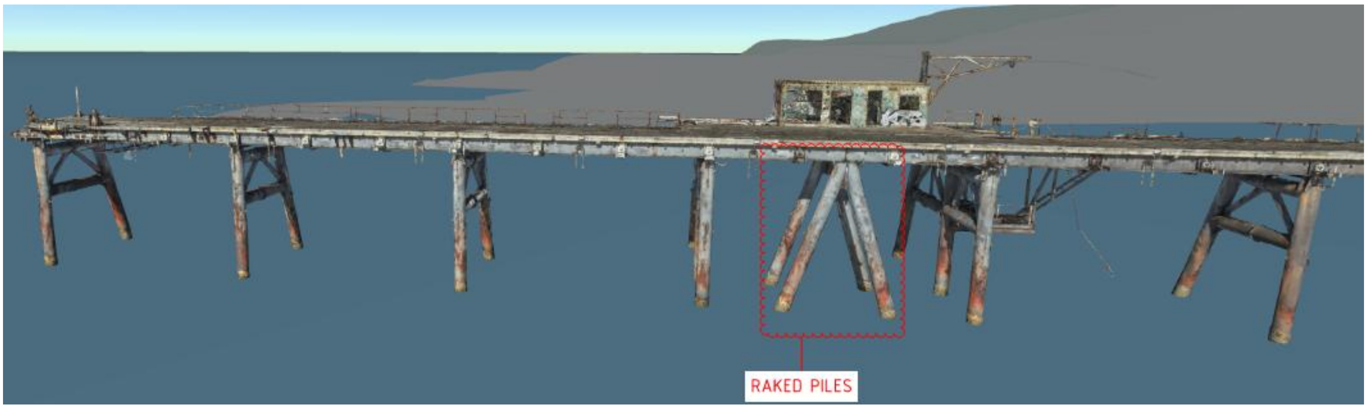


Figure 4 Raked Piles

4.2.2.3 Base Restraint

It is assumed that the piles will have some level of fixity at the base given that they are a composite socketed pile into the seabed. Considering that these piles were most likely required to cantilever from the seabed temporarily during construction, it further reinforces the assumption that some level of fixity exists at the seabed. Fixity at the base of piles provides rotational restraint, preventing free rotation and enabling the piles to act as cantilevers that resist lateral loads like waves, wind, and vessel impacts more effectively.

4.2.3 CHS Pile Lateral and Longitudinal Structural System – Option 1

The bracing system in the lateral (K-bracing) and longitudinal (primary beam) direction will be removed as part of this proposed option (Option 1). With the removal of the bracing system in both directions, this leaves the CHS piles unbraced. The CHS piles will be relying on the fixity at the seabed level, changing the structural system for the CHS piles from a braced structural system to a sway structural system.

4.2.3.1 Base Restraint

Without a geotechnical investigation into the seabed material, it is difficult to determine the level of the fixity and geotechnical capacity of the seabed. A geotechnical capacity assessment has not been completed. Assumptions have been made regarding the seabed material for the structural assessment. These assumptions are detailed below in Section 4.2.4.

4.2.4 Assumptions

The following assumptions have been made for the structural assessment:

- It is assumed that the lateral spring stiffness is constant along the 3.5m embedment depth of the pile into the seabed.
- It is assumed that the lateral spring stiffness of the seabed is similar for all CHS piles along the jetty.
- The seabed level has been assumed to be at a maximum distance of 14.4m below the tip of the CHS piles.
- A low tide level has been assumed to be at 6m above the seabed level.
- A high tide level has been assumed to be at 8.49m above the seabed level.
- A storm surge level has been assumed to be at 10.49m, which is 2m above the high tide level.
- A significant wave height of 4.03m has been adopted based on available historical data from NSW Nearshore Wave Tool.
- A wave period of 13.8 seconds has been adopted based on available historical data from NSW Nearshore Wave Tool.
- The CHS piles have been assumed to be installed in accordance with the original design drawings produced by Thiess.
- Mild steel grade of 250MPa has been assumed for all the CHS piles.
- Concrete grade of 25MPa has been assumed for all concrete along the length of the pile.
- Reinforcement grade of 500MPa has been assumed for all reinforcement.
- Due to the illegibility of the reinforced concrete detail at the bottom of the pile, the reinforcement detail has been assumed to be as shown below in Figure 55. A cover of 40mm in line with the current Australian Standard AS3600 for concrete structures for this exposure environment has been assumed.

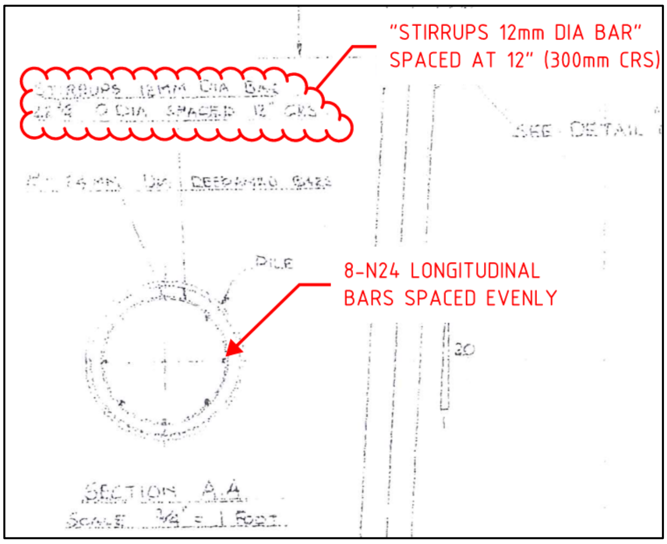


Figure 5 Lower Pile Reinforcement Detail

4.2.5 Design Loading and Combinations

The following design loading has been based on Australian Standard AS 4997 – Guidelines for the design of maritime structures and AS5100.2 – Bridge design Part 2: Design loads.

4.2.5.1 Wind Actions (W_u)

The following parameters has been adopted in determining the wind loading:

- Design life = 25 years
- Importance level = 1
- Ultimate limit state regional wind speed = 41m/s
- Wind drag coefficient for tubular piles, C_{wD} = 1.2
- Basic wind pressure = 1.14kPa

4.2.5.2 Current Actions ($F_{Current}$)

The following parameters has been adopted in determining the current loading:

- Current velocity, v = 1.64m/s. This has been determined based on a journal published by the University of Oslo which correlates the wind speed to current velocity. The journal has concluded that the current velocity equates to 3.1% to 3.4% of the wind speed, for this structural assessment, 4% has been adopted.
- Stream flow drag coefficient, CSD = 1.8 assuming heavy marine growth.
- Density of sea water, ρ = 1026 kg/m3 for sea water.

4.2.5.3 Debris Actions ($F_{Current,Debris}$)

- The following parameters has been adopted in determining the debris loading:
- Debris mat depth = 3m
- Debris mat length = 13.8m (equating to one half spacing of piles either side of a single pile in the direction along the jetty)
- Stream flow drag coefficient, CSD = 2.0
- Minimum uniform line load = 10kN/m along the length of the pile.

4.2.5.4 Wave Actions (F_{wave})

The following parameters has been adopted in determining the wave loading:

- Water depth, d = 15m
- Function category = 1
- Design life = 25 years
- Annual probability of exceedance of design wave events = 1/50
- Wave period = 13.8s
- Structure drag coefficient = 1.8
- Structure inertia coefficient = 2.0
- Significant wave height = 4.03m

4.2.5.5 Load Combinations

The following ultimate limit state load combination has been determined to be critical:

$1.2G + W_u + 1.5 (F_{Current} + F_{Current,Debris} + F_{Wave})$

4.2.6 Outcomes and Recommendations

The structural assessment has concluded that this option is not structurally feasible given that the lateral restraint system has been significantly altered. The capacity of the reinforced concrete portion of the pile is predicted to fail first, just below the tow of the steel, CHS portion of the pile. The section capacity of the CHS portion of the pile similarly fails due to the larger bending moment in the CHS pile but would occur after the concrete portion has failed. Figure 6 below is a snippet from the structural analysis software SpaceGass showing the bending moment diagram under an ultimate limit state load combination along the length of the CHS pile. This failure is also illustrated in Appendix A.1.

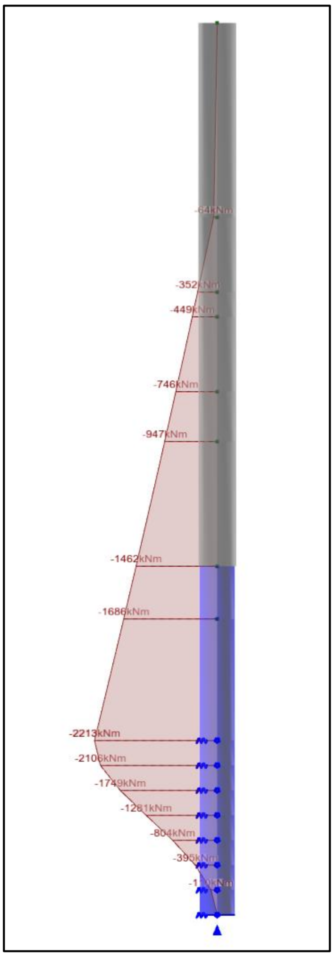


Figure 6 CHS Pile ULS Bending Moment Diagram

The following ultimate limit state bending moments and capacities apply to the scenario.

- Reinforced concrete portion of the pile:
  - Capacity: 410 kNm.
  - Maximum design bending moment: 1,856 kNm.
  - As the bending moment is greater than the capacity it fails.
- CHS portion of the pile:
  - Capacity: 1,704 kNm.
  - Maximum design bending moment: 2,213 kNm.
  - As the bending moment is greater than the capacity it fails.

An option to cut down the length of the CHS further has been explored, however, a significant length will need to be removed to reduce the cantilever to an acceptable length, which will significantly reduce visibility of the CHS piles. The CHS piles will need to be cut down to a level such that the top of the CHS pile is at a maximum of 6m above the seabed level. The top of the pile level will approximately be located at the low tide RL. This is considered unsafe for maritime activities and defeats the purpose of the retained jetty elements.

It is recommended to retain and rectify the existing lateral bracing system as a minimum to ensure that the CHS piles remain structurally adequate to resist the lateral loads. The existing lateral bracing system consists of the bracing between the CHS piles at each bent and the primary beams linking all the CHS piles at the bents together.

BG&E recommends that a geotechnical investigation to be conducted for the seabed to confirm the assumptions made in the structural assessment if this option was to be taken to a detailed design.

4.3 Construction Engineering Review and Sequencing

Based on the outcomes of the structural engineering review the retention of the CHS piles as cantilever members without any bracing is not considered viable. As such, Option 2, which explores the retention and introduction of additional structural steel bracing will include commentary on construction engineering and demolition sequencing. Refer to Section 5.

4.4 Durability

Based on the outcomes of the structural engineering review the retention of the CHS piles as cantilever members without any bracing is not considered viable. As such, Option 2, which explores the retention and introduction of additional structural steel bracing will include commentary on durability requirements to achieve a 25-year life. Refer to Section 5.

4.5 Quantity Surveying

Based on the outcomes of the structural engineering review the retention of the CHS piles as cantilever members without any bracing is not considered viable. As such, Option 2, which explores the retention and introduction of additional structural steel bracing will include a Class 3 quantity surveying assessment to estimate costs for both achieving the outcome and the end-of-life demolition of the structure. Refer to Section 5.

5. Option 2 – Retention of Piles, Primary Steel, and Bracing

This section summarises structural engineering, construction engineering, durability works, and estimated costs required to achieve the proposed Catherine Hill Bay Jetty - Exploration of Retention Options, Option 2 – Retention of Piles, Primary Steel, and Bracing.

The second option explored involves the full demolition of the existing “landside” portion of the jetty to the movement joint at Bent 17, where vertical universal columns change to circular hollow section piles. Then, further removal of the precast concrete deck, secondary and tertiary steel members from Bent 17 to Bent 1 as illustrated in Figure 7 and Figure 8 below.

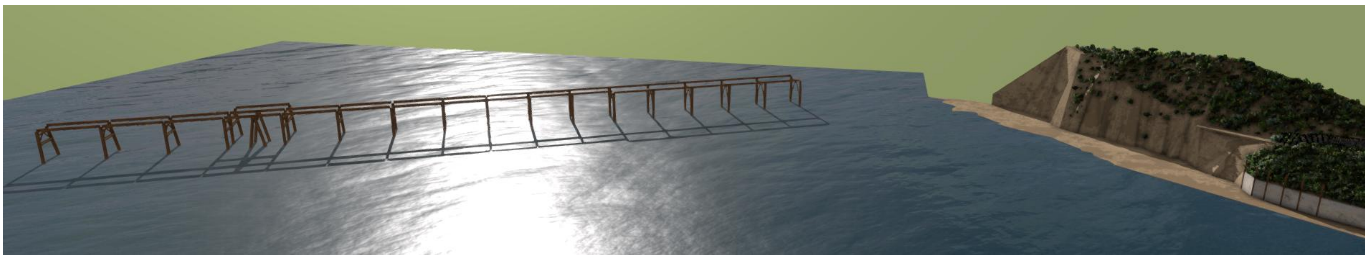


Figure 7 Visualisation of Retained Steel Jetty North Looking South

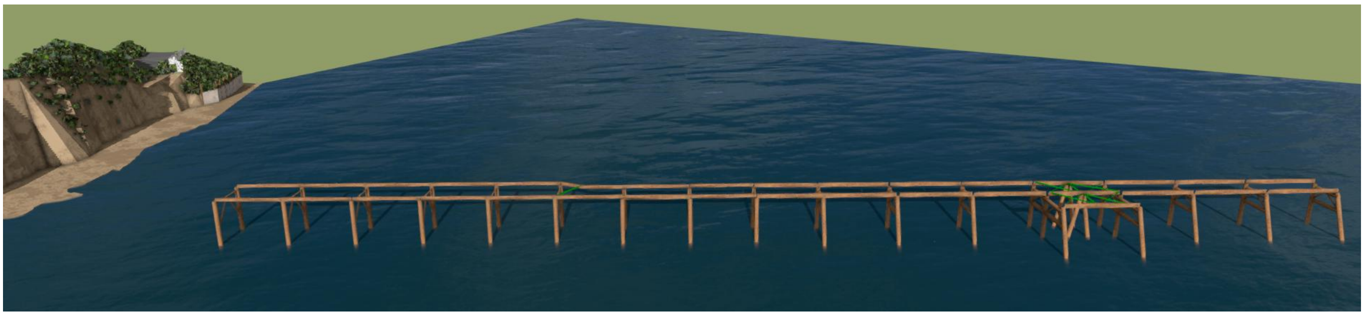


Figure 8 Visualisation of Retained Steel Jetty South Looking North

Photo 5 Example Photo of Bent Bracing

Photo 6 Example Photo of Steel Attachments

Photo 7 Example Photo of Movement Joint

Photo 8 Example Photo of Deteriorated Connection

5.1 Structural Engineering Review

The commentary provided in the 2023 report is still relevant to this structural review and the capacity of elements has likely decreased over the last few years. To summarise the structural engineering review in 2023:

Key findings for the steel elements comprising Bents 1–17 are:

- The CHS main support columns were assessed as having sufficient capacity to carry the self-weight of the steel and concrete elements comprising the jetty.
- Primary beams at the connection to the bents require repair where bottom flanges have experienced significant section loss; the 2023 report specifies replacement/rectification of the bottom flange at supports and addition of welded strengthening plates and end/side plates at each primary-beam-to-bent connection.
- CHS vertical bracing and CHS tie beams to the CHS columns are locally badly corroded at welded plate connections; these require cleaning, thickness verification and either strengthening or replacement where section loss is severe.
- The plan bracing within the deck between Bents 1–17 is assumed to be removed in addition to the precast planks which would otherwise provide lateral restraint to the primary beam top flange via the stud connection. With these two structural items removed (plan bracing and precast planks), the primary beam will be required to span about its weak axis between the CHS piles at the bents. The structural assessment of this is provided below.
- Smaller deteriorated steel items in this reach (knee braces, access walkways, suspended gantries, miscellaneous platforms) are recommended to be removed as they do not contribute to primary structural capacity but present safety risks.

5.1.1 Basis of Structural Assessment

The main criterion for the structural assessment is that the CHS piles are to remain intact to the seabed under foreseeable ultimate limit state loads. Since the structure will not be serviceable, no serviceability assessments have been made, i.e., deflections and movements.

The structural assessment for this option is a follow on from the structural assessment completed for Option 1 with additional consideration the retention of the lateral bracing system (CHS vertical bracing, CHS tie beams and primary beams).

5.1.2 Jetty Lateral and Longitudinal Bracing System – Option 2

The outcome of the structural assessment for Option 1 concluded that with the removal of the existing lateral and longitudinal bracing system resulted in structural deficiencies in resisting foreseeable lateral loads. Therefore, it was proposed in Option 1 for the current lateral structural system in both directions to be retained and maintained.

5.1.3 Assumptions

Assumptions made for the assessment of Option 1 is still applicable for the structural assessment of Option 2.

5.1.4 Design Loading and Combinations

The design loading and combinations detailed in the Option 1 are also adopted here with the addition of wind actions on the exposed primary beams and vertical bracing at the bent between the CHS piles.

5.1.4.1 Wind Actions (W_u)

The wind drag coefficient for the existing primary beams in the direction of the weak axis (perpendicular to the web) has been determined to be, C_{fig} = 2.05.

The wind drag coefficient for the existing vertical bracing members has been determined to be, C_{fig} = 1.2.

5.1.5 Outcomes and Recommendations

The structural assessment indicates that the existing CHS pile bracing system is adequate to resist the expected loads, provided the primary beams and CHS vertical bracing are repaired and strengthened to restore their original capacity. This includes the connections of the primary beam to the CHS piles at the bents and between bent 4 and 5.

5.1.5.1 Pile

The maximum ultimate limit state design bending moment induced in the pile is 1,337kNm with loading applied across the bent, as seen in the Figure 9 below. It is assumed conservatively that the connection of the pile at the seabed level is a pinned connection. Realistically, since the pile is embedded into the seabed, there will be a level of fixity, however, without geotechnical information on the seabed, a pinned connection can only be assumed at this stage of the assessment. The ultimate limit state bending moment capacity of the pile is 1,704kNm which is greater than the design bending moment, therefore the pile is structurally adequate.

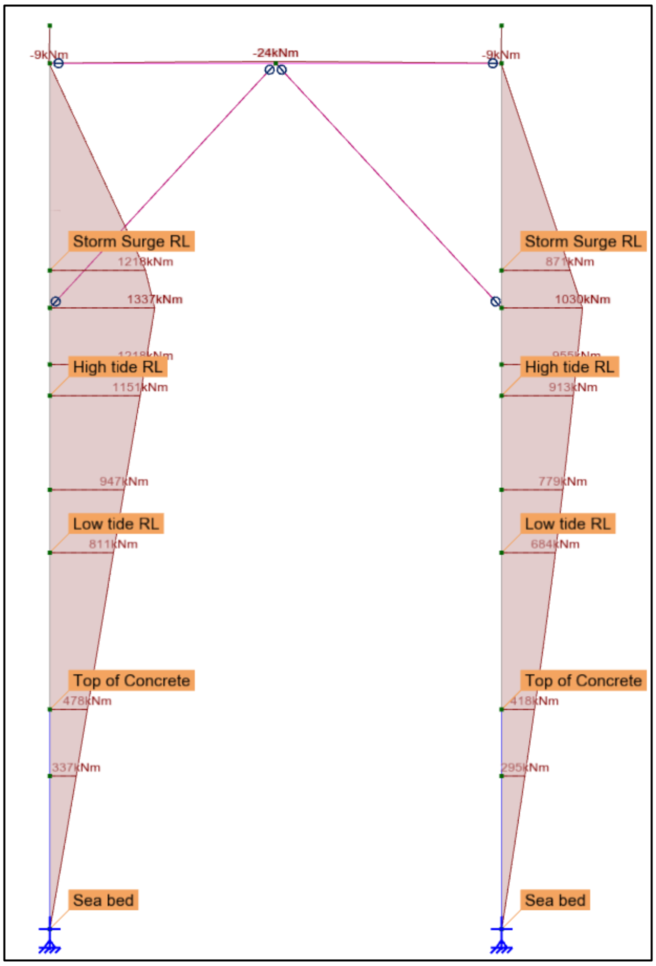


Figure 9 Pile Bending Moment Across Bent

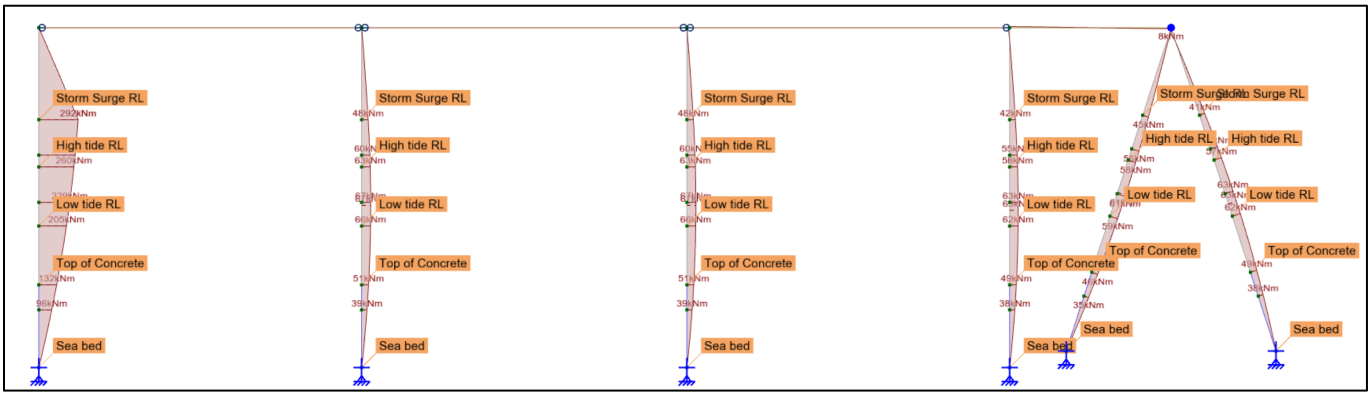


Figure 10 Pile Bending Moment Along Jetty

5.1.5.2 Vertical bracing

The maximum axial compression in the vertical bracing is 389kN as shown in the axial force diagram in Figure 11 below. The axial compression capacity of the vertical bracing is 1,614kN at 7,200mm effective length, hence the vertical bracing has sufficient structural capacity.

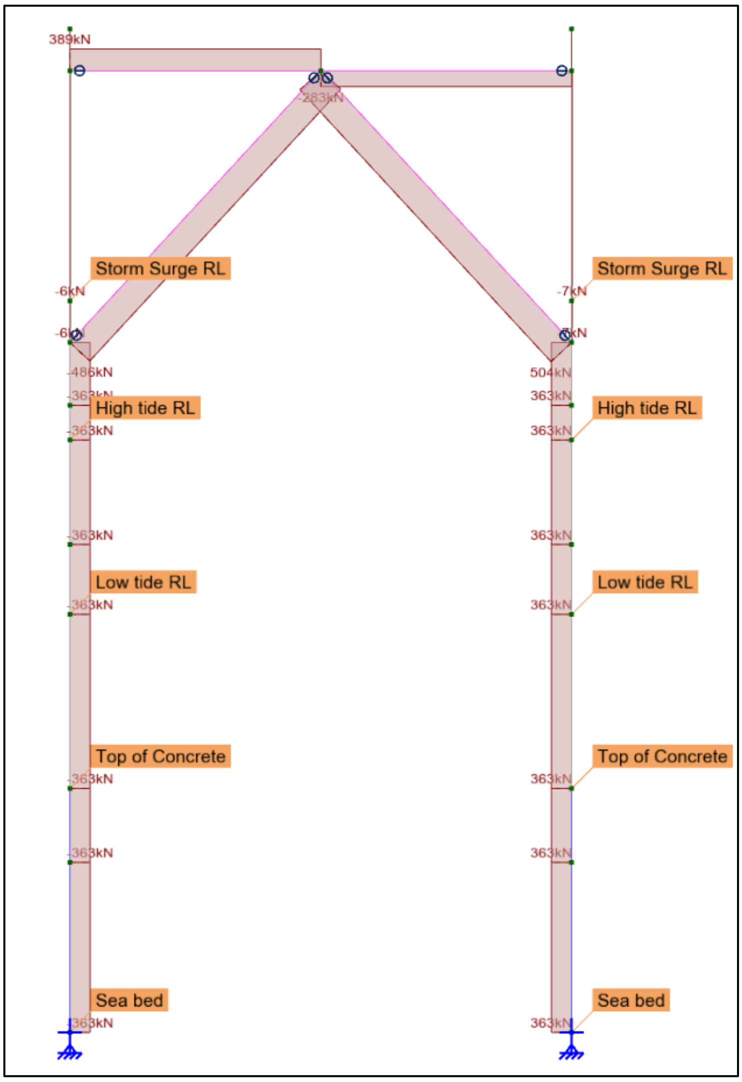


Figure 11 Pile Axial Force Diagram Across Bent

5.1.5.3 Raked pile

The maximum axial compression in the raked pile is 366kN, with an axial compression capacity of 2,682kN at 15,000mm effective length. Therefore, the raked pile is structurally adequate.

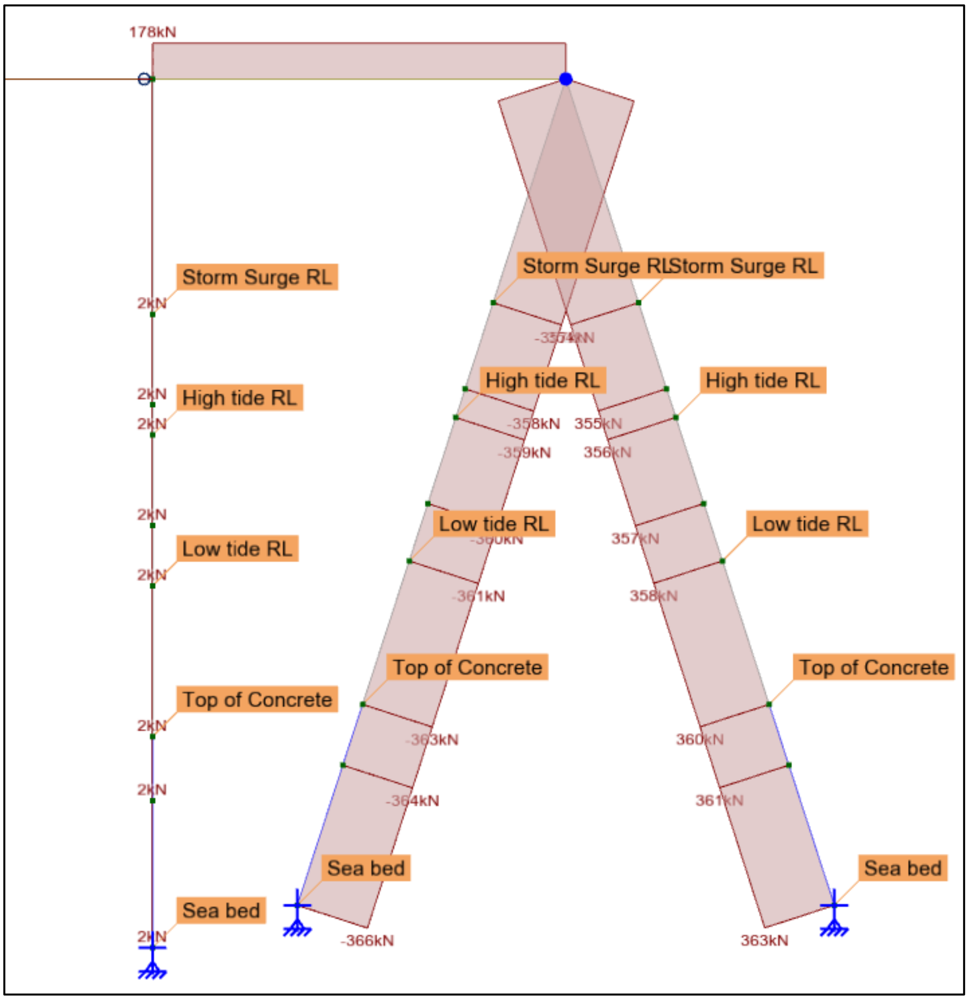


Figure 12 Raked Pile Axial Force Diagram

5.1.5.4 Additional bracing required

With the removal of the plan bracing and concrete planks, the restraint previously provided to the raked CHS piles and platform piles between bents 4 and 5 will need to be replaced to adequately restrain the piles in the transverse direction of the jetty. The recommended plan bracing framing arrangement is shown below in Figure 13 below.

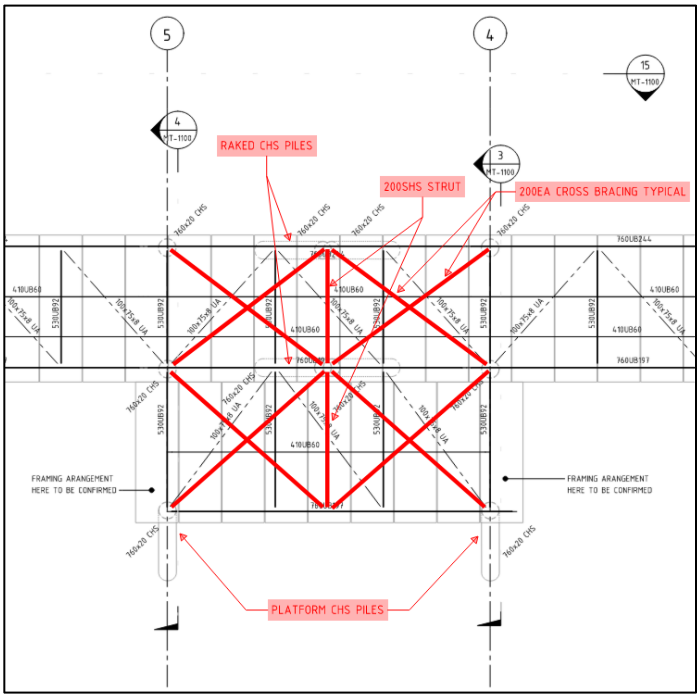


Figure 13 Additional Bracing Members between Bent 4 and 5

In addition to the bracing between bent 4 and 5, an additional strut will also need to be provided between bent 11 and 12 where there is a change in width to the jetty. This is shown below in Figure 1414.

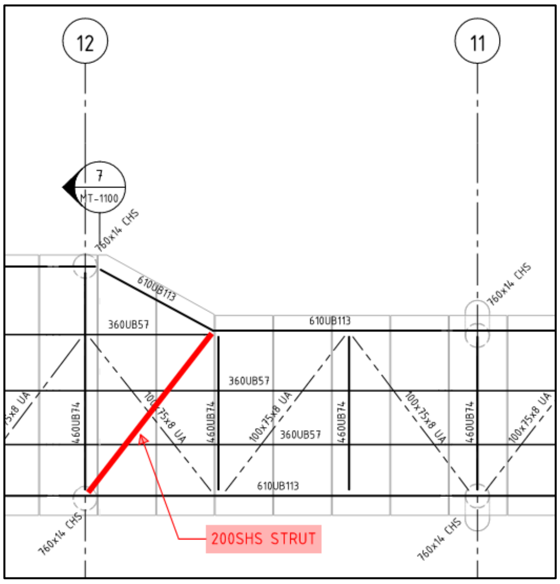


Figure 14 Additional Bracing between Bent 11 and 12

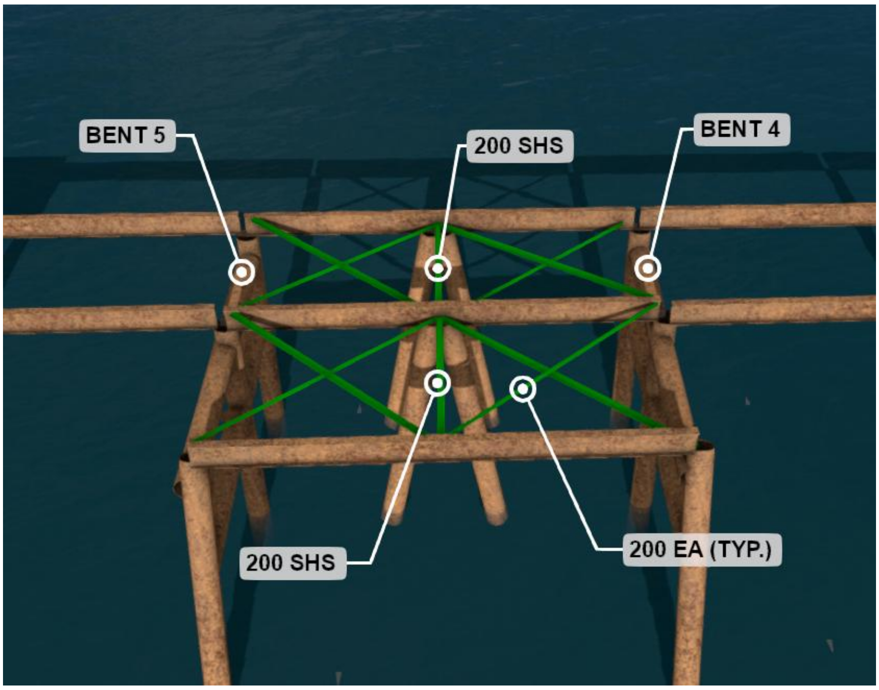


Figure 15 Visualisation of Additional Bracing at Bent 4 to 5 Jetty South Looking North

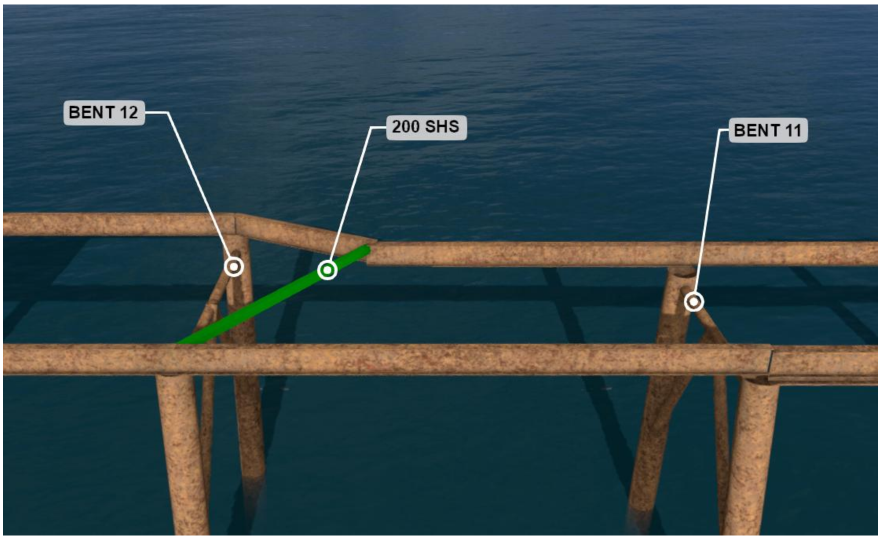


Figure 16 Visualisation of Additional Bracing at Bent 4 to 5 Jetty South Looking

The concept sketches for Option 2 can be found attached in Appendix A.2.

In addition to the above, recommendations around geotechnical investigations of the seabed as mentioned for Option 1 still applies.

5.2 Demolition to Achieve Option

5.2.1 Proposed Sequence of Works

The proposed sequence of works for Option 2 is as follows:

5.2.1.1 Full demolition of the jetty between bent 17 and bent 27

The proposed method of demolition is detailed in Technical Note CETN01 "Technical Note – Catherine Hill Bay Wharf Investigations – Demolition Option 1" dated 9th of June 2023. This deconstruction method is summarised as:

- Setup of a large, long-reach mobile crane at the jetty abutment to remove precast concrete planks.
- Setup of a 12T spider crane, elevated work platform, and excavator on the shore to reach precast concrete planks that the long-reach mobile crane cannot.
- Steel elements could then be cut from ternary, to secondary, to primary, then bracing between universal columns, then universal columns, then removal of concrete encasement below the sand line via excavator.
- Approximately, 56 precast concrete planks, 16 primary beams, 150 secondary and ternary members, and 32 bent columns and bracing groups would need to be removed.

5.2.1.2 Partial demolition of the jetty between bent 17 and 1

The proposed method of demolition is detailed in Technical Note CETN02 "Technical Note – Catherine Hill Bay Wharf Investigations – Demolition Option 2 – Full Demolition" dated 9th of June 2023 using a combination of different barges. The following elements between bent 17 and bent 1 are proposed to be removed and in the following sequence:

- The primary barge that would be required, would be a jack up barge that has an approximate total payload capacity of around 200T and suitable to work in waters up to 15m deep. The jack up barge would be used to support a crawler crane and elevated work platforms to undertake the demolition. The crawler crane would then be used to remove the wharf in small sections and transfer the removed materials to secondary barge(s). The secondary barge(s) would be a shallow draft landing barge with an approximate pay load capacity of 75-100T that can transfer the removed material to the beach and then using excavators the materials can be moved along the beach to within reach of the mobile crane
- Removal of existing ancillary items such as handrails, catwalks, platform 2 shed, hoarding and any other elements fixed to the main structural elements.
- Precast concrete deck, followed by the tertiary beams, secondary beams, and plan bracing. It is recommended that these elements be demolished sequentially between adjacent bents to avoid eliminating the diaphragm action provided by the precast concrete deck along the full length of the jetty.

5.2.1.3 Repairs/strengthening and new works

Following the demolition works, undertake repairs/strengthening on vertical CHS bracing / CHS tie beams and primary beams. This includes splices and connections between each of the elements and connections to the CHS piles. Once the repairs are completed, new bracing members as detailed can be installed.

Once all repairs and new works are completed, relevant works associated with durability to achieve a 25-year residual life can commence.

5.3 Durability

The following section describes recommendations for achieving a 25-year residual life for the circular hollow sections retained. This section is separated into three distinct environments:

- Submerged: Below the low tide mark.
- Tidal Zone: Between the low tide mark and the high tide mark.
- Atmospheric: Above the high tide mark.

5.3.1 Submerged

5.3.1.1 Existing Information

Based on the dives conducted in 2023 there appeared to be cathodic protection installed at the base of the CHS piles assessed. No comment was provided on their type or size, and no information can be found in historic records. It should be assumed that these are no longer fit for purpose and will need to be replaced. Divers also quoted that a plastic wrap had been installed and appeared to continue below the seabed and approx. 1 metre above the seabed. The base of the piles also appeared to sit within a concrete annulus, like that of the universal columns on the shore.

5.3.1.2 Proposed Durability System

The condition of the vertical CHS piles appears to be good with no significant loss of section. The presence of an existing cathodic protection system would have contributed significantly to this. It is recommended that the combined use of galvanic anodes plus coatings/wraps provide the best long-term results, i.e., to achieve a 25-year life.

Anodes would generally be aluminium or zinc/aluminium/indium alloys. Preferably with a mounting bracket below the low tide level i.e., always submerged. Direct connection of anodes to the steel pile is possible but not recommended as anodes cannot be replaced easily nor performance monitored, so using a fixing bracket is preferred. A cathode connection to the steel will need to be provided. All connections, etc need to be installed prior to painting/wrapping.

We would envisage that if a coating and wrap system was installed without any cathodic protection it is likely a residual life of only 10-15 years could be achieved.

BG&E have completed a concept design for cathodic protection of the steel CHS piles. This concept design has been completed as per the following technical guidelines:

- AS 2832.3-2005 Cathodic Protection of Metals
- DNV-RP-B401 Cathodic Protection Design
- Peabody’s Control of Pipeline Corrosion Third Edition

This concept applies to the steel CHS piles below the tidal zone.
The following assumptions have been made for this concept.

- 25-year service life.
- Pile diameter is 760mm.
- Depth below the low tide mark varies from 1,000mm to 9,500mm.
- 2,000mm below the mudline (seabed)
- The anode chosen for this system is Aluminium-Zinc-Indium, the industry standard for marine applications.
- This anode has a consumption rate of 4.4 kg(A*year) i.e., how much anode material is used up per year.
- Sea water temperature of 20 deg.c.
- Contingency factor of 1.2.
- Coating breakdown factor of 0.20 initially, followed by 0.40 as the mean, and 0.70 as the final.
- Consumption factor of 0.8.

The following table represents the distribution of anodes based on a current distribution balance (kg of Al-Zn-In) per pile.

Table 1 Cathodic Protection Calculations

Bent #	Diameter (m)	Depth Below Low Tide Mark (m)	Mudline below Seabed (m)	Surface Area External (m)	No. of Piles per Bent	Surface Area for No. of Piles	Current Distribution Balance per Pile in	CDB per Bent (kg)
Bent 1	0.76	1	2	7.16	2	14.33	30	61
Bent 2	0.76	1.5	2	8.36	2	16.71	36	71
Bent 3	0.76	2	2	9.55	2	19.10	41	81
Bent 4	0.76	2.5	2	10.74	3	32.23	46	137
Bent 5	0.76	3	2	11.94	4	47.75	51	203
Bent 6	0.76	3.5	2	13.13	3	39.40	56	168
Bent 7	0.76	4	2	14.33	2	28.65	61	122
Bent 8	0.76	4.5	2	15.52	2	31.04	66	132
Bent 9	0.76	5	2	16.71	2	33.43	71	142
Bent 10	0.76	5.5	2	17.91	2	35.81	76	152
Bent 11	0.76	6	2	19.10	2	38.20	81	163
Bent 12	0.76	6.5	2	20.29	2	40.59	86	173
Bent 13	0.76	7	2	21.49	2	42.98	91	183
Bent 14	0.76	7.5	2	22.68	2	45.36	97	193
Bent 15	0.76	8	2	23.88	2	47.75	102	203
Bent 16	0.76	8.5	2	25.07	2	50.14	107	213
Bent 17	0.76	9	2	26.26	2	52.53	112	224
Bent 18	0.76	9.5	2	27.46	2	54.92	117	234

This temperate environment and design life requirements result in a total weight of 2,856 kg of anode spread across each pile. As the piles vary in depth to the seabed the amount of anode material can be distributed. However, there is a risk that installed anodes can support electrically connected adjacent piles i.e., smaller anodes might be consumed faster, and thus it might be better suited to stick to one size anode across each pile. The weight of anode material per pile would then be 71kg.

Each pile will require an anode stand-off of 300mm to avoid “dumping” current into the steel directly behind the anode and to distance from marine growth which can “choke” the anodes output.

5.3.2 Tidal Zone

In the tidal zone we propose wrapping the steel CHS piles in a petroleum tape and HDPE jacket (or Denso system). This would include the following components:

- Cleaning of the piles surface to ST2/Sa2 via potable water jetting.
- Primer – A water-displacing petroleum-based paste.
- Tape – A heavy-duty petroleum-impregnated synthetic fabric wrap.
- Outer Jacket – A rugged, UV stabilised high-density polyurethane shell secured with 316 stainless steel bolts.

This system was chosen as it is constructable by divers in a wet environment.

5.3.3 Atmospheric

In the atmospheric zone we propose a high-build epoxy and polyurethane topcoat. This would include the following components:

- Cleaning of the piles surface to ST2/Sa2 via potable water jetting.
- Primer – Zinc-rich epoxy.
- Intermediate – High-build epoxy.
- Topcoat – Aliphatic polyurethane to provide UV resistance and maintain any aesthetic qualities.

This also applies to the horizontal steel members and bracing retained. Ensure all repair works for connections are completed prior to application.

5.4 Ongoing Maintenance Expectations

A galvanic anode cathodic protection system requires minimal but regular maintenance. This includes:

- Initial Survey – Requires a full potential survey within one year of installation to confirm that the jetty has "polarised" and reached the steady-state protection potential.
- Periodic Surveys – Recommend 5-year intervals for routine ROV or diver-assisted inspections.
- Visual Inspection of Anodes – During surveys, divers must estimate the percentage of anode consumption (e.g., how much lost as a percent). If consumption is significantly higher than 4% per year, the system is over-working and may need an early retrofit.

While the cathodic protection system takes minimal ongoing maintenance the wraps and atmospheric coatings require a more stringent maintenance schedule.

- Wrap Inspection – A visual check of the HDPE jackets every 2 years for impact damage or loose bolts.
- Atmospheric Coating Touch-ups – In the marine atmospheric zone, salt will eventually cause small blisters. A wash and treat routine every 5–7 years to treat localised rust before it spreads under the coating is recommended.
- Marine Growth Management – While a calcareous layer is beneficial, heavy hard growth (barnacles) on the anodes can eventually limit their current output. Any anodes found covered in thick deposits during surveys should be water-blasted clean.

5.5 Potential Risks

BG&E recognise the following limitations, risks, and potential issues for the retention of the jetty and works required to improve the durability to achieve a 25-year residual life.

5.5.1 Tides

The tides present a significant risk to the demolition, rectification, and durability works required to achieve this option. We would highlight the following:

- The application of the proposed protection system in the tidal zone is restricted to narrow low-tide windows for surface preparation and coating application on the upper portion of the piles.
- There is a high risk of a coating or wrap being applied and then immediately submerged by a rising tide before it has reached initial set. Careful planning and precise execution are required.
- Support barges must account for the Highest Astronomical Tide (HAT) over the proposed construction program to ensure they don't collide with the piles during installation. It is noted that historic data shows March, June, and July to be peak storm months in Central NSW.

5.5.2 Swell

Wave action and undercurrents pose a risk to cathodic protection installation and construction windows. We would highlight the following.

- During installation, swells can cause anodes to swing or vibrate, making it difficult to achieve continuous fillet welding required for the stand-off brackets.
- High swell conditions often lead to stand down days for divers, which can delay the installation of submerged anodes or the inspection of the seabed. This would also impact the ability for operations on jack-up barges and the use of lower payload dry-dock barges proposed to be used in the demolition sequence.
- Debris pushed by swells can damage freshly applied coatings or jackets before they have fully hardened.
- The is noted protection from Southerly swells by rock outcrops.

5.5.3 Flash Rust

In this type of environment salt-spray can be deposited on the bare steel directly after cleaning and begin to oxidise quickly. If the window between cleaning to Sa2 and the application of the zich-rich primer is too long flash rush will form and be trapped under the paint leading to blistering of the coating. This could lead to substantial rework costs.

5.5.4 Overcoating Timeframes

Coatings systems like those proposed require specific times to bond effectively to their previous layer. If layers are left too long salt spray can be deposited. It will be vital to ensure fresh water is available to clean between each layer before application.

5.5.5 Environmental Contamination

There is environmental contamination risk associated with the following items:

- Filling of the CHS piles with concrete and grout and depositing “oil” into the sea.
- Over spraying and spillage of coating systems.

5.5.6 Salt Pools on Horizontal Steel Members

Due to the geometry of I beams and the retained horizontal sections of this jetty they act as salt traps i.e., salt water or salt spray gets deposited on steel elements and gravity does not allow it to drain away. This deposit will eat into the coating. This risk can be reduced by:

- Installing drainage holes in horizontal members.
- Ensuring a requirement for hand painting edges, corners, and welds to ensure the dry film thickness is achieved.

5.5.7 Crevice Corrosion

This is considered a minor risk on the retained elements as a majority of the steelwork will be removed. Crevice corrosion is a significant issue in tight spaces, or close joints where cleaning and maintenance cannot be easily completed.

5.6 End-of-Life Demolition

At the end of the 25-year proposed service life. The following works would need to take place to remove the jetty.

- As per Section 4 of this report a jack-up barge, dry-dock barge, crane, and various EWP’s would be required to dismantle the bracing, secondary steel, and primary beams.
- Regarding the CHS piles we have assumed they will be concrete or grout filled making them act as a composite column and will be significantly heavier and harder to cut.
- The preferred method of demolition would be by diamond wire saw. An underwater frame could be setup at the seabed, and a diamond-tipped wire is hydraulically pulled through the cross section. This a considered a cold-cut and can be operated by a diver or ROV once installed. This is a much safe alternative to thermal methods like underwater oxyarc.
- The pile must remain rigged to a crane before the final cut is made with a light 10%-15% tension maintained to avoid snapping or jumping. The cut section might also need temporary framing to support it against currents and wave action to ensure diver safety.
- Any grout or steel shaving might need to be environmentally managed as per the POEO Act and other marine regulations.
- Regarding cutting the pile full length versus sectioning, there is currently not enough information to make that conclusion. Based on our experience we would recommend the piles be cut into two sections. The following comments apply:
    - For an assumed 20m pile with 760mm outer diameter, 17mm thick annulus, and grout filled, it will weigh roughly 1.3 tonnes per metre or 26 tonnes in total.

- While a 26-tonne pile is manageable with a large barge and crane there needs to be consideration on dynamic load factors like wave action, currents, and suction from the seabed. If the pile weight exceeds 70% of the cranes rated capacity at the required radius the pile will need to be sectioned.
- If the pile is cut in one piece, instead of two, the crane will need to have sufficient lifting height to allow for barge deck clearances to land the cut pile.
- Stability in wind will be a significant factor for lifting and manoeuvring the cut pile section.
- Transport and disposal of the piles will also be an issue and checks on truck, and other transport systems will need to be made.
- Internal shear is also potentially an issue i.e., will the internal grout bond to the steel shell, or slide out. This can be mitigated by installing shear pins through the cross section.
- Consideration needs to be given to raked piles as half of the piles retained are on an angle on the Southern side.

5.7 Class 4 Quantity Surveying Estimate

The following figures are extracted from the Quantity Surveyor's memorandum dated 14th of April 2026.

- Total estimated cost to achieve Option 2: $12.98m.
- Total estimated cost for ongoing maintenance over 25 years: $10.16m.
- Total estimated cost to demolish at the end-of-life: $15.23m.

5.7.1 Commentary on Cost Estimate

The following comments are made on the quantity surveying estimate.

- The total estimated cost for the lifecycle of this project is approximately $38.36 million.
- An escalation of 4% over 25 years has been allowed for in the ongoing maintenance phase and the end-of-life demolition phase and is a considerable cost factor in the estimate.
- The requirement of specialised marine plant is a significant cost.

5.8 Maritime Safety

To ensure safety for maritime activities 4 or 6 special (yellow) marker buoys with flashing lights would be required around the retained jetty structure and compliance stickers saying all vessel prohibited.

6. Option 3 - Retention of Three Bents, Bent 1, 2, & 3, and Associated Steel Bracing

This section summarises the structural engineering, construction engineering, and durability works required to achieve the proposed Catherine Hill Bay Jetty - Exploration of Retention Options, Option 3 - Retention of Three Bents, Bent 1, 2, and 3, and associated Steel Bracing.

The third option explored to address potential retention options involves the full demolition of the existing “landside” portion of the jetty to the movement joint at Bent 17, where vertical universal columns change to circular hollow section piles. Then, further demolition of the remaining jetty from Bent 17 to Bent 4 retaining only the vertical circular hollow sections with bracing as illustrated in Figure 17 and Figure 18 below.

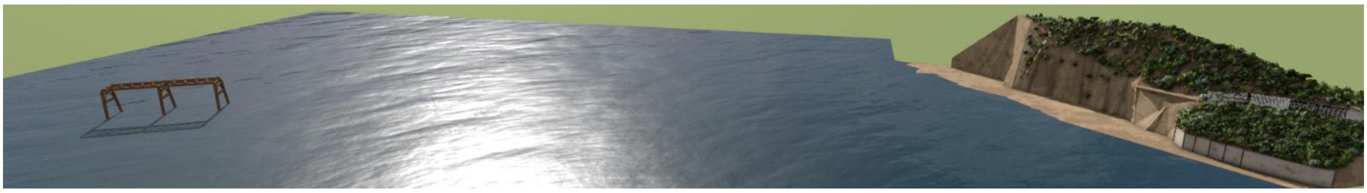


Figure 17 Visualisation of Retained Bent 1, 2, and 3 North Looking South

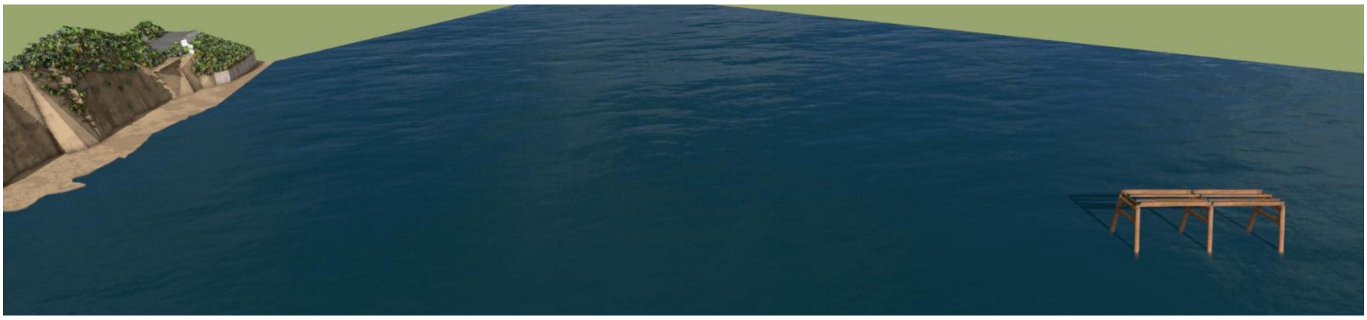


Figure 18 Visualisation of Retained Bent 1, 2, and 3 South Looking North

6.1 Structural Engineering Review

The commentary provided in the 2023 report is still relevant to this structural review and the capacity of elements has likely decreased over the last few years. To summarise the structural engineering review in 2023:

Key findings for the steel elements comprising Bents 1–3 are:

- The CHS main support columns were assessed as having sufficient capacity to carry the self-weight of the steel and concrete elements comprising the jetty.
- Primary beams at the connection to the bents require repair where bottom flanges have experienced significant section loss; the 2023 report specifies replacement/rectification of the bottom flange at supports and addition of welded strengthening plates and end/side plates at each primary-beam-to-bent connection.
- CHS vertical bracing and CHS tie beams to the CHS columns are locally badly corroded at welded plate connections; these require cleaning, thickness verification and either strengthening or replacement where section loss is severe.
- The plan bracing within the deck between Bents 1–3 is assumed to be removed in addition to the precast planks which would otherwise provide lateral restraint to the primary beam top flange via the stud connection. With these two structural items removed (plan bracing and precast planks), the primary beam will be required to span about its weak axis between the CHS piles at the bents. The structural assessment of this is provided below.

Smaller deteriorated steel items in this reach (knee braces, access walkways, suspended gantries, miscellaneous platforms) are recommended to be removed as they do not contribute to primary structural capacity but present safety risks.

6.1.1 Basis of Structural Assessment

The main criterion for the structural assessment is that the CHS piles are to remain intact to the seabed under foreseeable ultimate limit state loads. Since the structure will not be serviceable, no serviceability assessments have been made, i.e., deflections and movements.

6.1.2 Jetty Lateral and Longitudinal Bracing System – Current

The existing lateral and longitudinal bracing system are discussed in Section 4.

6.1.3 Jetty Lateral and Longitudinal Bracing System – Option 3

The bracing system in the lateral (k-bracing) direction is proposed to be retained. In the longitudinal direction however, the main longitudinal bracing system, namely the raked piles between bent 4 and 5 will be removed as part of this proposed option (Option 3). With the removal of the bracing system in the longitudinal direction, this leaves the CHS piles unbraced. The CHS piles will be relying on the fixity at the seabed level, changing the structural system for the CHS piles in the longitudinal direction from a braced structural system to an unbraced cantilevered structural system to resist the lateral loads.

6.1.3.1 Base Restraint

Without a geotechnical investigation into the seabed material, it is difficult to determine the level of the fixity and geotechnical capacity of the seabed. A geotechnical capacity assessment has not been completed. Assumptions have been made regarding the seabed material for the structural assessment. These assumptions are detailed in Section 4.

6.1.4 Assumptions

Assumptions made for the assessment of Option 1 are still applicable for the structural assessment of Option 3.

6.1.5 Design Loading and Combinations

The design loading and combinations detailed in both Option 1 and Option 2 are still applicable for the structural assessment of Option 3.

6.1.5.1 CHS Pile

The maximum ultimate limit state design bending moment induced in the pile is 1,337kNm with loading applied across the bent, as seen in the 20 below. It is assumed conservatively that the connection of the pile at the seabed level is a pinned connection. Realistically, since the pile is embedded into the seabed, there will be a level of fixity, however, without geotechnical information on the seabed, a pinned connection can only be assumed at this stage of the assessment. The ultimate limit state bending moment capacity of the pile is 1,704kNm which is greater than the design bending moment, therefore the pile is structurally adequate.

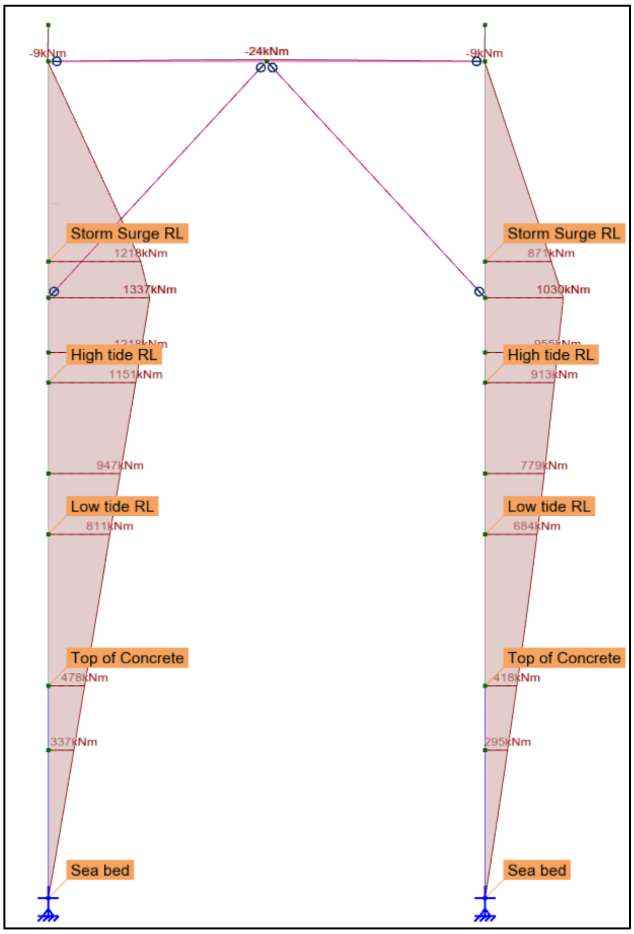


Figure 19 Pile Bending Moment across Bent

The assumption of a pinned connection at the connection of the pile to the seabed cannot be assumed for the lateral loading along the bent as the main longitudinal bracing system (raked piles between bent 4 and 5) is proposed to be removed for Option 3. Therefore, lateral support springs has been adopted along the pile starting at the seabed level to the bottom of the pile. Figure 2021 below shows the maximum ultimate limit state bending moment in the cantilevered pile:

- Maximum ultimate limit state bending moment in the CHS portion of the pile = 970kNm
- Ultimate limit state bending moment capacity of CHS pile = 1,704kNm
- Maximum ultimate limit state bending moment in the reinforced concrete portion of the pile = 604kNm
- Ultimate limit state bending moment capacity of the reinforced concrete portion of the pile = 410kNm

It is noted that the bending moment in the reinforced concrete portion of the pile exceeds the bending moment capacity, therefore the reinforced concrete portion of the pile is structurally inadequate for Option 3.

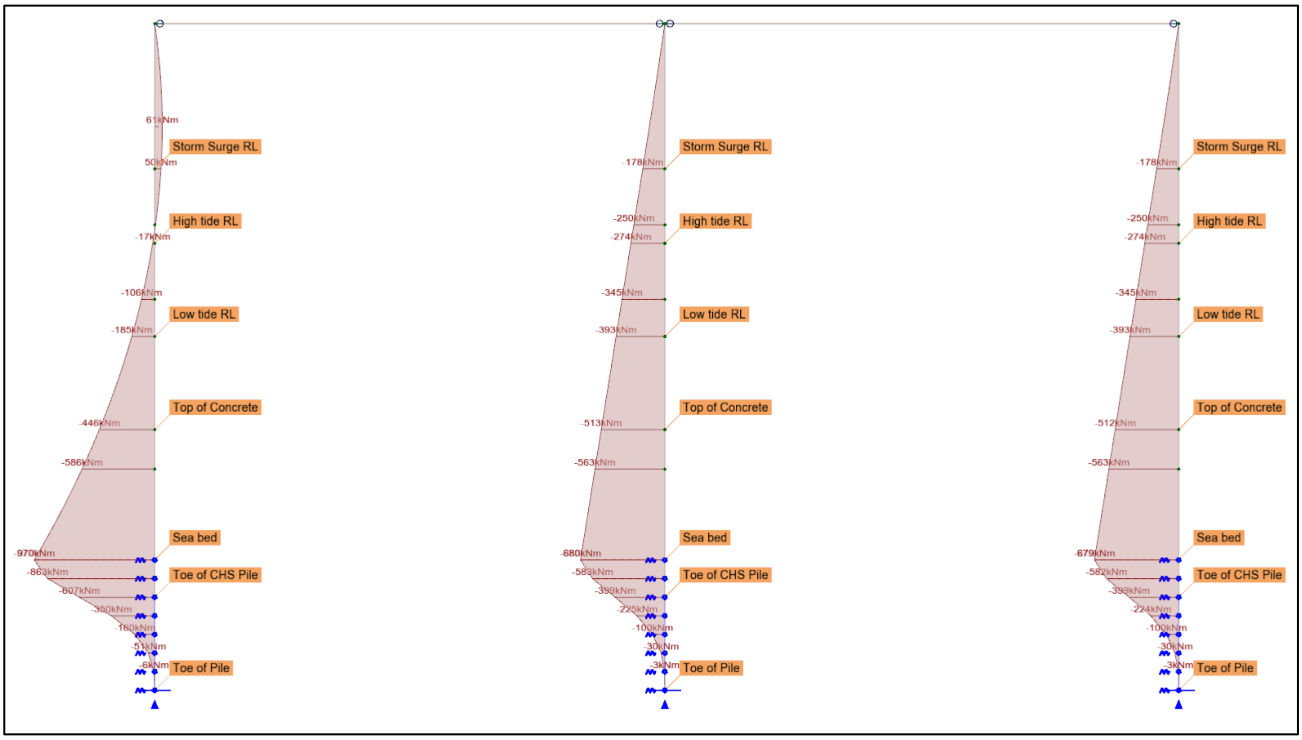


Figure 20 Pile Bending Moment along Bent

6.1.6 Outcomes and Recommendations

The structural assessment has concluded that this option is not structurally feasible given that the longitudinal restraint system (raked piles between bent 4 and 5) is proposed to be removed. For the longitudinal load case, the bending moment near the base of the pile exceeds the bending moment capacity of the pile. This failure is also illustrated in Appendix A.1.

It may be possible to retain bents 1 to 3 if bents 4 and 5 and the raked piles between are also retained. Some additional plan bracing may be required between the raked piles bents 4 and 5. BG&E recommends further structural analysis if this option is to be considered. If the option is to be further developed to detailed design stage, additional geotechnical investigation of the seabed is recommended to confirm the assumptions made in the structural assessment.

An option to cut down the length of the CHS further has been explored, however, a significant length will need to be removed to reduce the cantilever to an acceptable length, which will significantly reduce visibility of the CHS piles. The CHS piles will need to be cut down to a level such that the top of the CHS pile is at a maximum of 6m above the seabed level. The top of the pile level will approximately be located at the low tide RL. This may be considered unsafe for maritime activities and defeats the purpose of retaining the jetty elements for visual purposes.

6.2 Construction Engineering Review and Sequencing

Based on the outcomes of the structural engineering review the retention of the 6 CHS piles and associated bracing is not considered viable. As such, Option 2, which explores the retention and introduction of additional structural steel bracing will include commentary on construction engineering and demolition sequencing. Refer to Section 5.

6.3 Durability

Based on the outcomes of the structural engineering review the retention of the 6 CHS piles and associated bracing is not considered viable. As such, Option 2, which explores the retention and introduction of additional structural steel bracing will include commentary on durability. Refer to Section 5.

6.4 Quantity Surveying

Based on the outcomes of the structural engineering review the retention of the 6 CHS piles and associated bracing is not considered viable. As such, Option 2, which explores the retention and introduction of additional structural steel bracing will include commentary on quantity surveying. Refer to Section 5.

7. Revised Estimate for Full Demolition

The following figures are extracted from the Quantity Surveyor's memorandum dated 14th of April 2026.

- The total estimated cost to demolish the jetty to the seabed is $11,09m.
- BG&E would recommend adjusting this figure with a 4%-6% escalation each year to plan costs in the future.

8. Investigation of the Internal of CHS Piles

Previous documented evidence highlighted that the CHS pile internals were filled with corrosion inhibitors “oil” and the product “Magnacote” was noted in a quantity surveyors report. Approximately 310L of “oil” was added to the jetty each maintenance cycle. Please refer to the snapshot below.

Trade : 3	***Corrosion Inhibitor for Piles***				
	The inhibitor is called "Magnacote"				
1	Supply of "Magnacote" corrosion inhibitor for application to Steel PIles allowed as 344 litres per year (NB: Quantity required as per CSG Engineers Pty Ltd report dated July 2009/rate based on Ryan from Bulbecks advice of $8,000 for a 200 litre barrel)	3,440.00	Litres	40.00	137,600.00
		Corrosion Inhibitor for Piles	**Total :**		**137,600.00**

Figure 21 Extract from Historical Cost Estimate on Jetty Maintenance

To confirm the presence of corrosion inhibitor inside the CHS piles we attended site and inserted CCTV cameras through downpipes into the top of the CHS piles. Please refer to the below image as evidence of this activity. It was found that the piles did have an “oil” substance inside them at various heights, some were full and some were half-full. The substance was sampled and delivered to a laboratory for analysis.

Photo 9 Example of Oil Filling Pipes at Top of CHS

Photo 10 Substance Extracted from CHS Piles

Based on the laboratory analysis, which can be found in Appendix A.3, the following comments are made:

- The GCMS analysis indicates that the sample is a bitumen oil with heavier hydrocarbons.

- The sample contains some VOCs (volatile organic compounds). These include pentanal, hexanal, and 2,4-nonadienal (aldehydes), which may be harmful at certain exposure levels. It also contains di-n-octyl disulfide, a Sulphur compound with inhalation risks like H_2S.

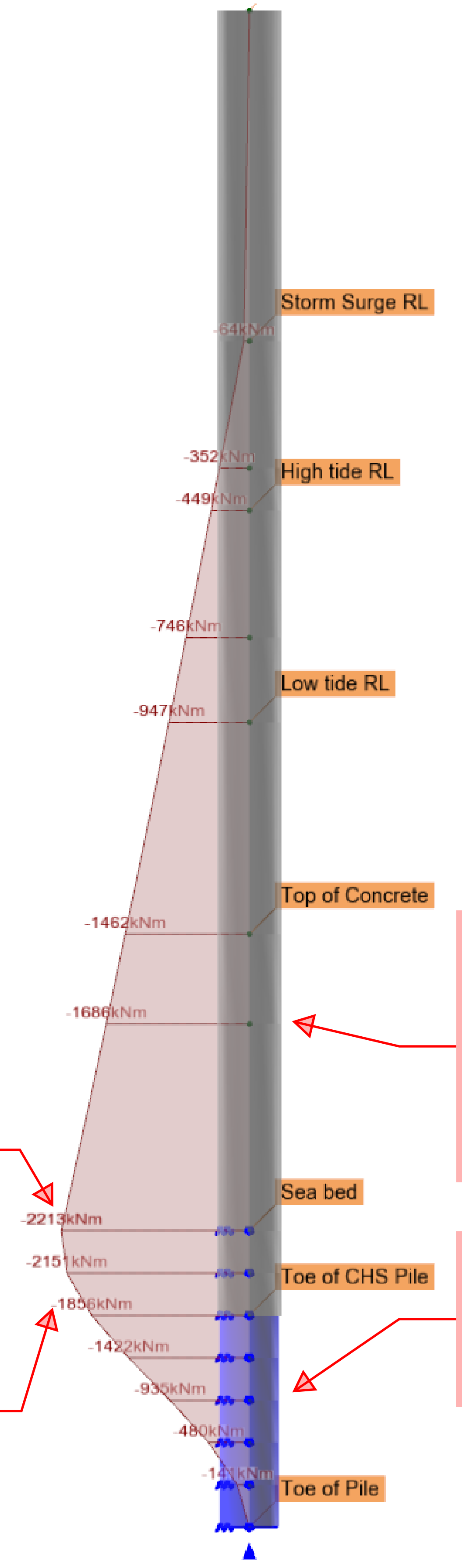
Unfortunately, “Magnacote” previously supplied by Bulbecks Marine, is now not an available product and Bulbecks Marine no longer operational. As such a comparison between the sampled substance and then historical documented substance could not be completed, and obtaining Safety Data Sheets to ensure correct handling and disposal is unlikely.

It is unlikely that the cathodic protection system proposed will extend to the internal surface of the CHS piles. The previous cathodic protection system was coupled with internal corrosion inhibitor. As such we would recommend filling the CHS piles with concrete or grout as the high alkalinity of the concrete passivates the steel, and the physical barrier prevents oxygen and chloride ingress. This is estimated at approximately 130 cubic metres of concrete/grout fill, assuming 2 metres below the seabed. This would displace the corrosion inhibitor inside the CHS piles and be considered an environmental risk. This would need to be closely managed by any contractor.

Appendices

Appendix A - Reference Documentation

A.1 Option 1 & Option 3 Failure Location Sketch



SECTION CAPACITY OF CHS PORTION OF THE PILE = 1704kNm, WHICH IS BELOW THE MAXIMUM BENDING MOMENT IN THE CHS PILE. HOWEVER, THE FAILURE WILL OCCUR AT THE REINFORCED CONCRETE PORTION OF THE PILE FIRST BEFORE THE CAPACITY OF THE CHS PORTION OF THE PILE IS EXCEEDED.

MAXIMUM BENDING MOMENT IN CHS SECTION OF THE PILE OF 2213kNm

SECTION CAPACITY OF THE REINFORCED CONCRETE PORTION OF THE PILE = 410kNm. THEREFORE, PILE FAILURE IS PREDICTED TO OCCUR JUST BELOW THE TOE OF THE CHS PORTION OF THE PILE.

MAXIMUM BENDING MOMENT IN REINFORCED CONCRETE PORTION OF THE PILE OF JUST UNDER 1856kNm.

OPTION 1 FAILURE POINT

REV	DATE	DESCRIPTION	RVD

REVISIONS

NOTES

CLIENT

PROJECT

Brisbane *Office*
Level 5, 180 Ann St
Brisbane QLD 4000
P / +61 7 3167 3300
E / info@bgeeng.com
bgeeng.com

BG&E

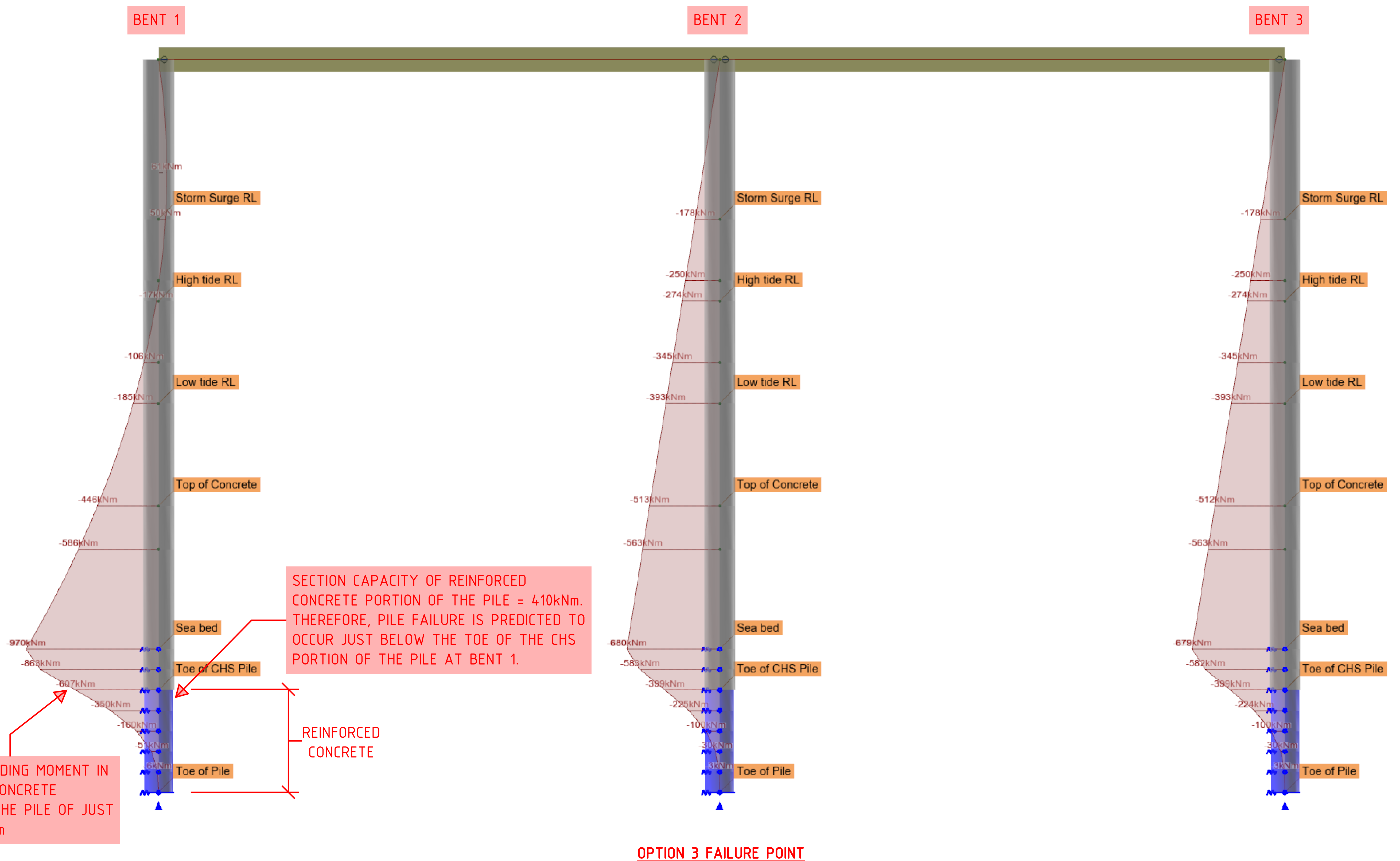
STATUS
CONCEPT ONLY - NOT FOR CONSTRUCTION

DRAWN	DESIGNED	CHECKED	APPROVED

DATUM	GRID	SCALE	SIZE
			A3

TITLE

PROJECT No.	SKETCH No.	REV.



OPTION 3 FAILURE POINT

REV	DATE	DESCRIPTION	RVD

REVISIONS

NOTES

CLIENT

PROJECT

Brisbane Office
Level 5, 180 Ann St
Brisbane QLD 4000
P / +61 7 3167 3300
E / info@bgeeng.com
bgeeng.com

STATUS
CONCEPT ONLY - NOT FOR CONSTRUCTION

DRAWN	DESIGNED	CHECKED	APPROVED

DATUM	GRID	SCALE	SIZE
			A3

TITLE

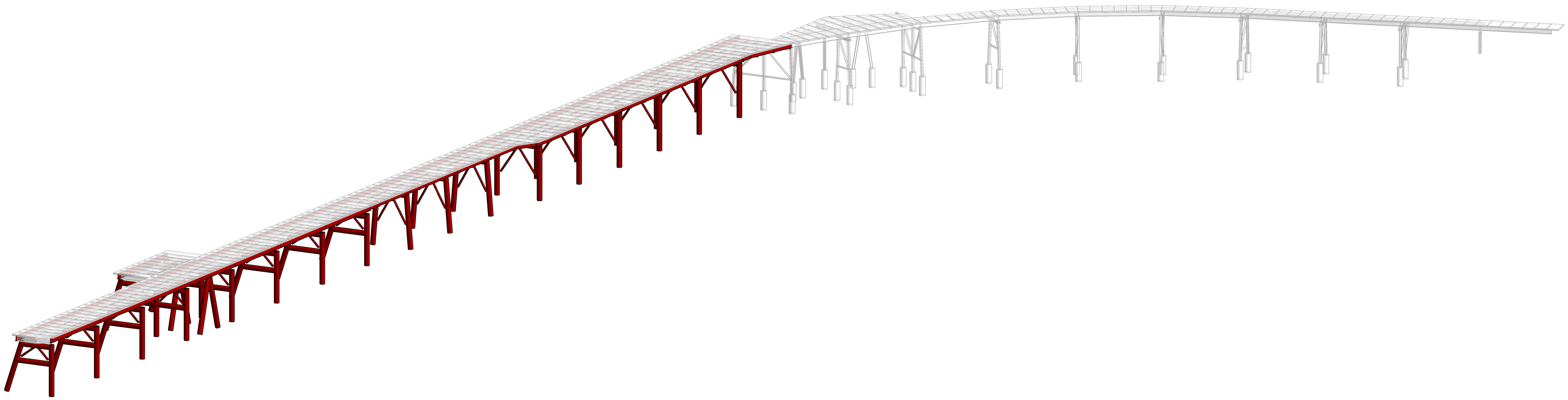
PROJECT No.	SKETCH No.	REV.

A.2 Option 2 – Drawings

CATHERINE HILL BAY JETTY

CATHERINE HILL BAY, NSW

DRAWING INDEX	
DRG No.	TITLE
MT-0001	TITLE AND DRAWING INDEX
MT-1000	OVERALL PLAN AND ELEVATION
MT-1010	GENERAL ARRANGEMENT PLAN - SHEET 1
MT-1011	GENERAL ARRANGEMENT PLAN - SHEET 2
MT-1100	GENERAL SECTIONS - SHEET 1



DRAWING TO BE PRINTED IN COLOUR

REV	DATE	DESCRIPTION	RVD
C	XX.02.26	ISSUED FOR INFORMATION - OPTION 2	HR
B	19.09.23	ISSUED FOR INFORMATION	MC
A	31.07.23	ISSUED FOR INFORMATION	MC

REVISIONS

CLIENT

Newcastle *Office* —
Suite 2, Level 3, 426 King St,
Newcastle NSW 2302
P /+61 2 4902 3000
E / info@bgeeng.com
bgeeng.com —

PROJECT
CATHERINE HILL BAY WHARF
CATHERINE HILL BAY, NSW

STATUS
ISSUED FOR INFORMATION
NOT TO BE USED FOR CONSTRUCTION

DRAWN	DESIGNED	CHECKED	APPROVED
MC	WT	NS	N/A

DATUM	GRID	SCALE
		NTS AT A0 SIZE

TITLE
TITLE AND DRAWING INDEX

PROJECT No.	DRAWING No.	REV.
N23028	MT-0001	C

18/02/2026 2:00:34 PM Catherine Hill Bay Jetty

OVERALL PLAN

SCALE 1 : 250

GENERAL ARRANGEMENT ELEVATION

ELEVATION A MT-1000

SCALE 1 : 250

DRAWING TO BE PRINTED IN COLOUR

REV	DATE	DESCRIPTION	RVD
C	XX.02.26	ISSUED FOR INFORMATION - OPTION 2	HR
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A	31.07.23	ISSUED FOR INFORMATION	MC

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PROJECT
CATHERINE HILL BAY WHARF
CATHERINE HILL BAY, NSW

STATUS
ISSUED FOR INFORMATION
NOT TO BE USED FOR CONSTRUCTION

DRAWN	DESIGNED	CHECKED	APPROVED
MC	WT	NS	N/A

DATUM | GRID | SCALE AT A0 SIZE

TITLE
OVERALL PLAN AND ELEVATION

PROJECT No. N23028 | DRAWING No. MT-1000 | REV. C

18/02/2026 2:00:35 PM Catherine Hill Bay Jetty

GENERAL ARRANGEMENT PLAN - PART A
SCALE 1 : 100

GENERAL ARRANGEMENT PLAN - PART B
SCALE 1 : 100

DISCLAIMER:
STEEL FRAMING INFORMATION TAKEN FROM EXISTING DRAWINGS AND LIMITED SITE INVESTIGATIONS.
ALL MEMBERS SIZE AND DETAILS OF CONNECTIONS TBC

CONCRETE DECK:
CONCRETE DECK PLANKS ARE TYPICALLY 180mm THICK AND 1820mm WIDE.

REV	DATE	DESCRIPTION	RVD
C	XX.02.26	ISSUED FOR INFORMATION - OPTION 2	HR
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A	31.07.23	ISSUED FOR INFORMATION	MC

REVISIONS

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PROJECT
CATHERINE HILL BAY WHARF
CATHERINE HILL BAY, NSW

STATUS
ISSUED FOR INFORMATION
NOT TO BE USED FOR CONSTRUCTION

DRAWN	DESIGNED	CHECKED	APPROVED
MC	WT	NS	N/A

DATUM	GRID	SCALE
		AT A0 SIZE

TITLE
GENERAL ARRANGEMENT PLAN - SHEET 1

PROJECT No.	DRAWING No.	REV.
N23028	MT-1010	C

18/02/2026 2:00:35 PM Catherine Hill Bay Jetty

DISCLAIMER:
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ALL MEMBERS SIZE AND DETAILS OF CONNECTIONS TBC

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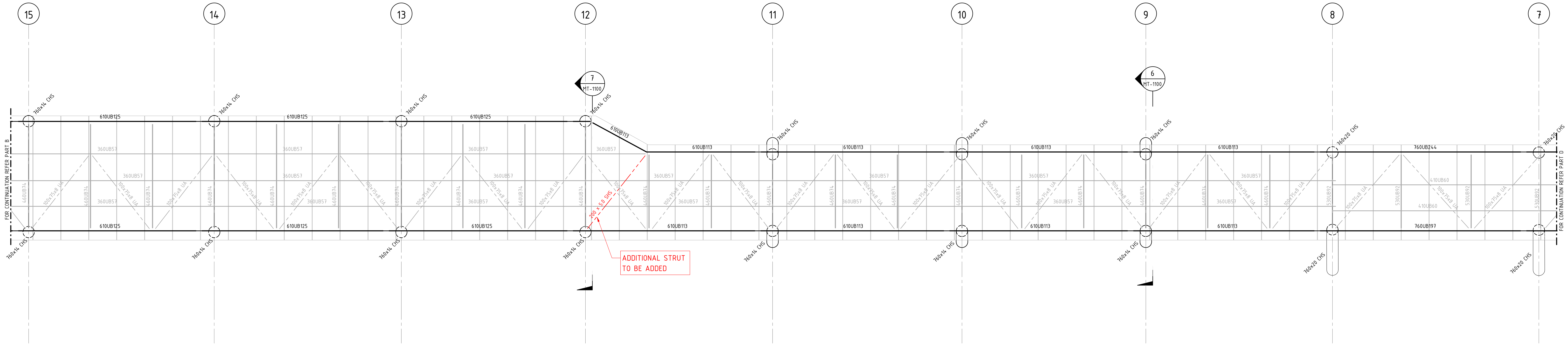
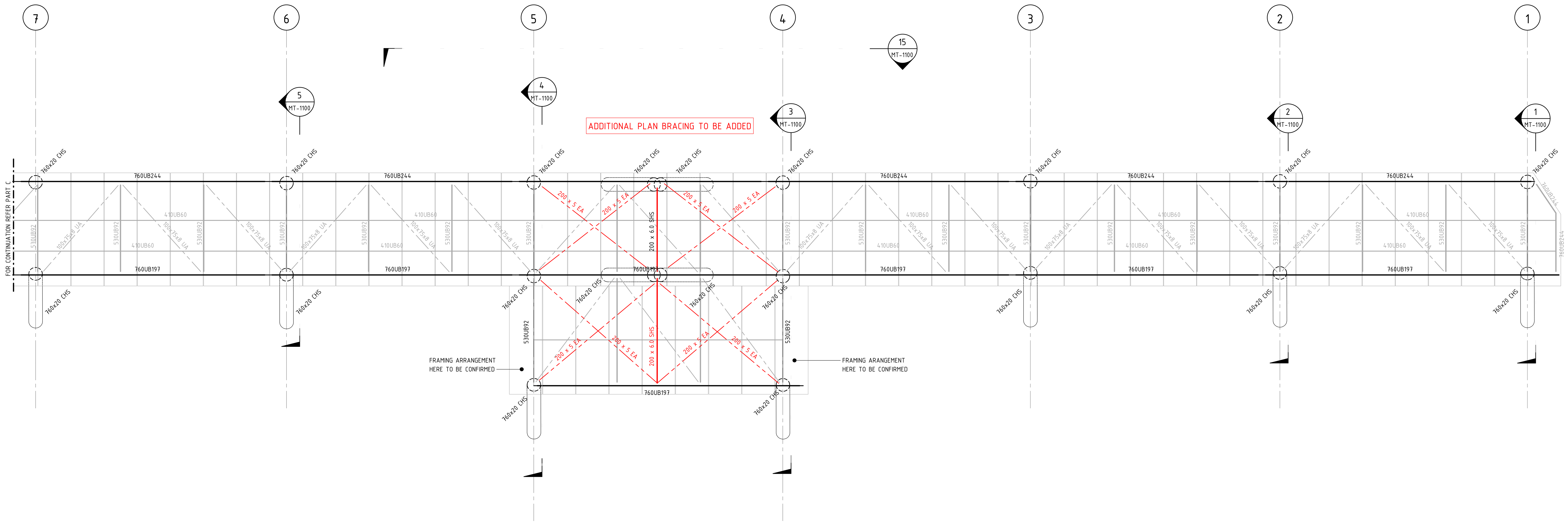
CONCRETE DECK PLANKS ARE TYPICALLY 180mm THICK AND 1820mm WIDE.



GENERAL ARRANGEMENT PLAN - PART C
SCALE 1 : 100



GENERAL ARRANGEMENT PLAN - PART D
SCALE 1 : 100

DRAWING TO BE PRINTED IN COLOUR

REV	DATE	DESCRIPTION	RVD
C	XX.02.26	ISSUED FOR INFORMATION - OPTION 2	HR
B	19.09.23	ISSUED FOR INFORMATION	MC
A	31.07.23	ISSUED FOR INFORMATION	MC

REVISIONS

CLIENT

Newcastle *Office* —
Suite 2, Level 3, 426 King St,
Newcastle NSW 2302
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bgeeng.com —

PROJECT	CATHERINE HILL BAY WHARF CATHERINE HILL BAY, NSW		
STATUS	ISSUED FOR INFORMATION NOT TO BE USED FOR CONSTRUCTION		
DRAWN MC	DESIGNED WT	CHECKED NS	APPROVED N/A
DATUM	GRID	SCALE AT A0 SIZE	
TITLE	GENERAL ARRANGEMENT PLAN - SHEET 2		
PROJECT No. N23028	DRAWING No. MT-1011	REV. C	

18/02/2026 2:00:36 PM Catherine Hill Bay Jetty

DISCLAIMER:
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ALL MEMBERS SIZE AND DETAILS OF CONNECTIONS TBC

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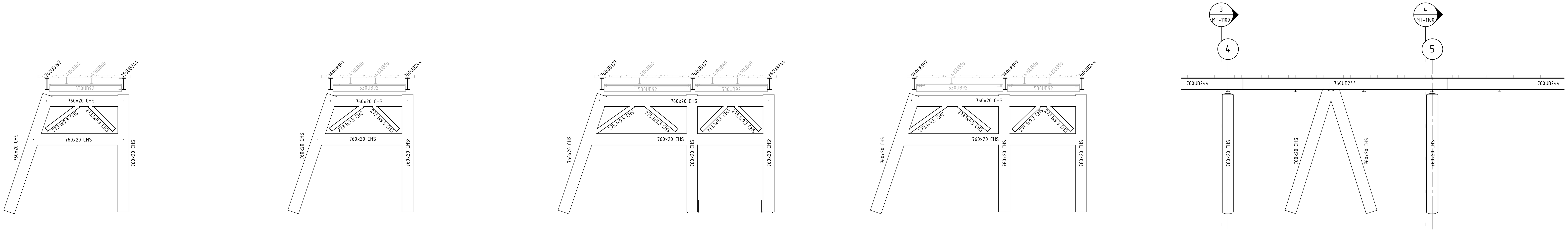


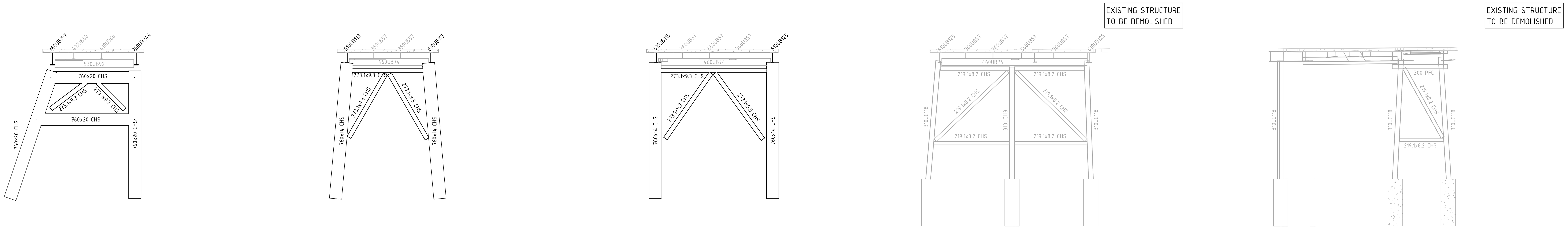
BENT 1
SECTION 1
SCALE 1 : 100 MT-1000

BENT 4
SECTION 2
SCALE 1 : 100 MT-1000

BENT 4
SECTION 3
SCALE 1 : 100 MT-1000

BENT 5
SECTION 4
SCALE 1 : 100 MT-1000

BETWEEN BENT 4-5
SECTION 15
SCALE 1 : 100 MT-1000



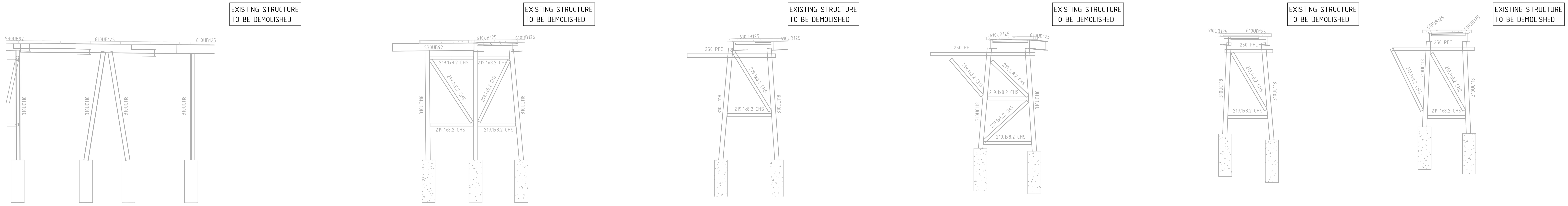
BENT 6-8
SECTION 5
SCALE 1 : 100 MT-1000

BENT 9-11
SECTION 6
SCALE 1 : 100 MT-1000

BENT 12-17
SECTION 7
SCALE 1 : 100 MT-1000

BENT 18
SECTION 8
SCALE 1 : 100 MT-1000

BENT 19
SECTION 9
SCALE 1 : 100 MT-1000



BETWEEN bent 19-20
SECTION 16
SCALE 1 : 100 MT-1000

BENT 20
SECTION 10
SCALE 1 : 100 MT-1000

BENT 21
SECTION 11
SCALE 1 : 100 MT-1000

BENT 22-23
SECTION 12
SCALE 1 : 100 MT-1000

BENT 24-25
SECTION 13
SCALE 1 : 100 MT-1000

BENT 26
SECTION 14
SCALE 1 : 100 MT-1000

DRAWING TO BE PRINTED IN COLOUR

REV	DATE	DESCRIPTION	RVD
C	XX.02.26	ISSUED FOR INFORMATION - OPTION 2	HR
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REVISIONS

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bgeeng.com —

BG&E

PROJECT
CATHERINE HILL BAY WHARF
CATHERINE HILL BAY, NSW

STATUS
ISSUED FOR INFORMATION
NOT TO BE USED FOR CONSTRUCTION

DRAWN	DESIGNED	CHECKED	APPROVED
MC	WT	NS	N/A

DATUM | GRID | SCALE | A1 A0 SIZE

TITLE
GENERAL SECTIONS - SHEET 1

PROJECT No.	DRAWING No.	REV.
N23028	MT-1100	C

A.3 Internal CHS Pile Investigation - Lab Report

INTERTEK GEOTECH
ANALYTICAL REPORT

CLIENT	BG&E Pty Limited Attn. Zachariah Arneil 10 Hastings Street Rocky Point, NSW 2259 Australia

JOB INFORMATION

JOB REFERENCE	:	2026-MDTN-000277 Rev.0
NO. OF SAMPLE	:	1
CLIENT ORDER NO.	:	NTL 2025-026
PROJECT	:	N25139/VA*01
DATE SAMPLED	:	13/03/2026
DATE RECEIVED	:	26/03/2026
DATE REPORTED	:	14/04/2026

REPORT NOTES

Sample received in good condition at a temperature of 24.3°C.

TESTED BY

Intertek Geotech
544 Bickley Road, Maddington
Western Australia, 6106
Tel: (08) 9263 0263
Email: ocm.perth@intertek.com

Noel Mellican
Laboratory Manager

Report Reviewed

List of revisions

REV.	DATE	REVISION DETAILS	PREPARED BY	APPROVED BY
0	14/04/2026	Document prepared	Eder Casanova	Noel Mellican

This document replaces and supersedes all previous versions.

Sample Information

SAMPLE DESCRIPTION	SAMPLE DATE	MATRIX	PRESERVATION	INTERTEK REFERENCE
Unknown Oil	13/03/2026 @09:00	Liquid	N/A	2026-MDTN-000277-001

Additional Sample Information

N/A

Methodology

N/A

Comments

The GCMS analysis indicates that the sample is bitumen oil with heavier hydrocarbons.

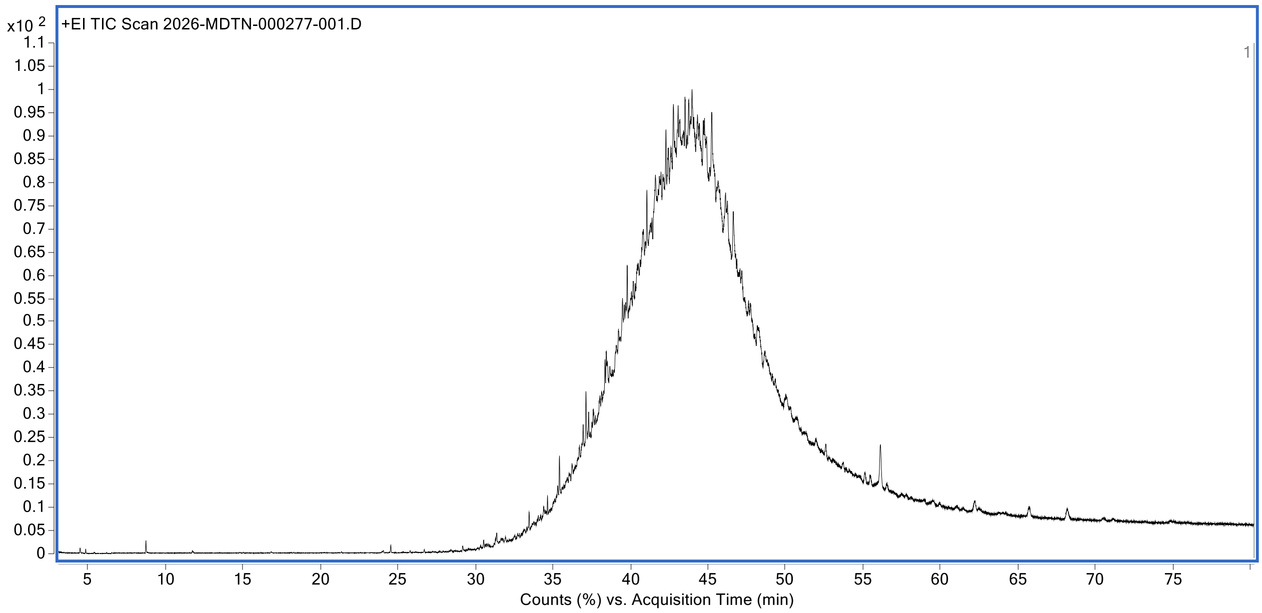
The sample contains some VOCs, including pentanal, hexanal, and 2,4-nonadienal (aldehydes), which may be harmful at certain exposure levels. It also contains di-n-octyl disulfide, a Sulphur compound with inhalation risks similar to H_2S.

Recommend sourcing the relevant bitumen oil Safety Data Sheet (SDS) and ensuring that handling and disposal are carried out in accordance with the recommended guidelines.

REULTS

Unknown Oil			**Intertek Reference: 2026-MDTN-000277-001**	
TEST	**METHOD**	**UNIT**	**LOR**	**RESULT**
Water content	Karl Fischer	% wt	0.01	0.31

GCMS Analysis



COMPOUNDS	**RETENTION TIME**	**MATCH QUALITY**	**UNIT**	**RESULT**
Pentanal	8.74	93	wt%	0.245
Hexanal	11.74	71	wt%	0.053
Di-n-octyl disulfide	24.07	94	wt%	0.053
2,4-nonadienal	24.54	88	wt%	0.164
Tridecane, C13	26.70	71	wt%	0.053
Tetradecane, C14	29.18	70	wt%	0.120
1H-Indene, octahydro-2,2,4,4,7,7-hexamethyl-, trans-	30.54	71	wt%	0.167
1-(2-Pyridyl)-2-pentene-1,4-dione-4-dimethylhydrazone	32.55	83	wt%	0.159
Cyclooctasiloxane, hexadecamethyl-	34.65	81	wt%	0.250
Pentadecane, 2,6,10,14-tetramethyl-	35.43	72	wt%	0.813
Hexadecane, 2,6,10,14-tetramethyl-	37.13	74	wt%	1.271
Nonadecane, C19	38.45	70	wt%	1.863
Eicosane, C20	39.80	71	wt%	0.858
Heneicosane, C21	41.07	74	wt%	1.403
Docosane, C22	42.30	70	wt%	1.298
Total (Identified Compounds)	**-**	**-**	**wt%**	**8.770**
Other Organic Compounds (Bitumen Oil)	**-**	**-**	**wt%**	**91.23**
Total	**3.27-80.39**		**wt%**	**100.000**

LEGEND

LOR = Limit of Reporting

NA = Not Analysed

N/A = Not Applicable

I/S = Insufficient Sample

Quality Control and Measurement Uncertainty reports are available upon request.

- End of Report -

At BG&E, we are united by a common purpose – we believe that truly great engineering takes curiosity, bravery and trust, and is the key to creating extraordinary built environments.

Our teams in Australia, New Zealand, South East Asia, the United Kingdom and the Middle East, design and deliver engineering solutions for clients in the Property, Transport, Ports and Marine, Water, Defence, Renewables and Resources sectors.

We collaborate with leading contractors, developers, architects, planners, financiers and government agencies, to create projects for today and future generations.

ABN 67 150 804 603

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