# APPENDIX 15

Maritime Design Report

Barangaroo Delivery Authority

# Barangaroo Headland Park Maritime Works, Concept Design Report

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# Barangaroo Delivery Authority

# Barangaroo Headland Park Maritime Works, Concept Design Report

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# 1 INTRODUCTION

## 1.1 EXECUTIVE OVERVIEW

As noted in the Design Brief "The Barangaroo public domain is a key component of the Barangaroo project that will help reinforce its status as a world-class waterfront renewal precinct. As part of the site's transformation an unprecedented opportunity exists for the design and creation of a significant harbour side headland park and vibrant public space.

The Headland Park will involve the creation of a naturalistic parkland, which will complement the existing harbour headlands, whilst at the same time establishing a counterpoint to the urban public domain envisaged to the south of the site. The entire public domain will help position Barangaroo as a high quality mixed use precinct with uninterrupted public access to the harbour."

This report covers the maritime concept design of the Headland Park shoreline and the adjacent harbour floor.

The key aspect of the design is the use of large sandstone blocks extracted from the site to act as a retaining and sea wall. Below this a rock ballast amour protects an embankment and new harbour floor.

The existing reinforced concrete caissons which retain the fill behind the quay line will cut back and will be used to retain the rock amour and the existing material.

The material behind the caisson is likely to be very free draining and accordingly wet excavation methods may have to be employed.

The concept design is based on broad assumptions on the nature of the existing material on the site. Geotechnical investigation currently underway and resultant geotechnical recommendations may impact on the design and as such due allowance should made for this.

# 2 SITE INFORMATION

## 2.1 THE EXISTING SITE

Barangaroo is located on the western side of Sydney's CBD, between Millers Point and Darling Harbour. Until the early $19^{th}$ century the site was a prominent natural headland. During the latter parts of the $19^{th}$ century and early $20^{th}$ century a number of timber wharves were constructed on the shoreline. During the 1970 reinforced concrete caisson walls were installed to provide a quayline for container ships and other large cargo ships. The hardstand behind the caissons was created by the placement of fill behind the caissons

The Headland Park site is bound by Hickson Road, Merriman Street, Dalgetty Road, Clyne Reserve and Moore's Wharf to the East, former Sydney Ports Corporation (SPC ) Wharf No 3 to the North, SPC Wharves No 3, 5 and 6 to the West and SPC Wharf No 6 to the South.

The existing site is essentially flat at levels varying between RL 2.5 AHD and RL 3.0 AHD

The north and west boundaries of the site are formed from concrete caissons placed in the 1970's and a rock ballast wall in the north west corner.

The fill material behind the walls is a variety of builders waste material and crushed sandstone from the Eastern Suburbs Railway tunnel excavation overlying alluvium and sandstone bedrock.

## 2.2 EXISTING CAISSON WALL INFORMATION

The information on the existing caissons is taken from the SPC drawings noted at the back of this document. The caissons are reinforced concrete 15.5 metres long (along the quay line) by 10 meters deep and 14.6 metres high. Each caisson has six compartments separated by 200mm thick reinforced concrete connector walls. The outside walls of the caisson are 300mm thick and the base is 660 mm thick reinforced concrete. Concrete strength nominated on the design drawings is 24MPa. The compartments are filled with sand. The caissons rest on a 1.5metre thick layer of rock ranging in size from 225mm to 300mm in size, which acts as a footing to the caissons and also allows the free passage of water under the caisson so that local landside water pressure can be dissipated.

The concrete seawall was constructed using the following methodology:

- A 15 metre wide trench was dredged to the underlying rock with sloping submerged batters of about 2H:1V. The existing drawings show the trench extending through the centres of many of the pre-existing timber wharves, which would suggest the wharves would have been demolished prior to dredging.
- Gravel fill was placed on the floor of the trench to form a levelling pad.
- Precast hollow units (caissons) were floated into position to form the seawall and then filled with sand.
- Backfill placed behind with wall, the varied depths up to 20 metres.

- Allowances were made for stormwater lines to extent through the caisssons.
- Development behind this wall compromised of a row of piles for adjacent structures and crane beams.
- Large reinforced concrete capping beams (approximately 1.5m deep x 5m wide) were placed on top of the caissons to accommodate fenders, bollards and any local cranes. The underside of the capping beam is at ISLW RL 1.52 or RL 0.595 AHD. Mean High Water Level (MHWL) is at ISWL RL 1.44 or RL 0.515 AHD therefore at very high tides the top of the caisson will be slightly submerged.

An inspection of the caissons by CTI Consultants Pty Ltd Report No C10656 Dated 12th October 2008 indicated that the caissons were in good condition, however chloride concentrations at the reinforcement depth were approximately three times higher than recommended and accordingly impressed current cathodic protection (ICCP) was recommended to be installed to achieve a design life greater than an additional 20 years.

## 2.3 EXISTING CURRENT GEOTECHNICLAL INFORMATION

With reference to ERM Pty Ltd report (reference 004432RP3 Rev Final) dated September 2006 which contained Jeffery and Katauskas Pty Ltd bore logs, the report identified the following geological units:

| Unit Number and Description | Description | Depth Below Ground Level (m) | Average thickness (m) |
|---|---|---|---|
| Unit 1: Fill | Comprised of a mixture of clay, sand and gravel in varied proportions contaminated with a variety of materials such as brick, concrete, rubble, wood, glass and slag materials. The fill is probably sourced from adjacent building sites, dredged material from the harbour and waste materials. The fill may contain timber members, large boulders and possibly buried structures, such as timbers wharves and tank structures. | 0 to 21.0 metres | 8.4 |
| Unit 2: Natural Marine Sediment | Comprised of alluvial/marine sediments comprised predominantly of sandy clays | 0.34 to 17.4 | 3.24 |
| Unit 3: Hawkesbury Sandstone | Extremely weak grading to high strength at depth | 0.5 to 31.04 | Unknown |

Based upon Hyder's previous knowledge of this site, we are also aware of the additional geotechnical considerations:

- Potential buried cliff lines across the site
- The influence of the Luna Park Fault on the caisson founding material

## 2.4 TIDAL LEVELS / SEA LEVEL RISE

### 2.4.1 Present Day Tidal Planes:

| | |
|---|---|
| Highest Astronomical Tide ("HAT") | +1.175AHD |
| Mean Sea Level ("MSL") | -0.035AHD |
| Lowest Astronomical Tide ("LAT") (ISLW) | -0.925AHD |

## 2.5 Sea level rises

The following  design sea level allowances have been made for this project

2050 + 0.4m

2100 + 0.9m

## 2.6 EXISTING SITE CONTAMINATION

Contamination has been identified on this site. The issue of ground water contamination and whether a cut off structure would be required was raised by Hyder. This was reviewed by  JBS Environmental who advised that " On the basis of the current groundwater conditions at the Headland Park Site and Northern Cove , there is not considered to be a need for groundwater containment as part of the proposed seawall at headland Park and North Cove". Accordingly cut off walls such as contiguous pile walls or diaphragm walls have not been employed for ground water cut off purposes.

## 2.7 EXISTING STRUCTURES ADJACENT TO MARINE WORKS

Moore Wharf and Building Structure: At the northern end of the site, the existing Moore's Wharf building at Miller's Point is to be retained. This sandstone structure constructed in the 1830's and it is our understanding that the building has been relocated to its current position. The building is currently in use. At this point of time, we do not have any drawings of this building however as the building is of masonry construction, it can be expected to be sensitive to movement. Accordingly any ground work near this building should be carried out is such a manner which will cause little or no movement to the footings of the building.

## 2.8 OTHER SITE SPECIFIC ASSUMPTIONS

- Existing caisson wall: It has been assumed the existing caisson wall has been constructed in accordance with the existing SPC drawings in Appendix B with no modifications and further the caissons exhibit on signs of major distress.
- Northern sandstone wall: It has been assumed this wall is structurally sound, however this is under investigation by Douglas Partners
- Moore's Wharf building: In the absence of any existing drawings and reports noting the structural condition of this building, it is assumed this structure is in a sound condition.

# 3 AUSTRALIAN STANDARDS AND SPECIFICATIONS

## 3.1 DESIGN CODES

The following standards are proposed for the design relating to maritime engineering of Headland Park:

- AS 1170.0 -2002 Structural Design Actions – General Principles
- AS 1170.1 -2002 Structural Design Actions –Permanent, Imposed and Other Actions
- AS 1170.2 -2002 Structural Design Actions – Wind Loads
- AS 1170.4 -2007 Loading Code- Earthquake Loads
- AS 2159 - 2009: Piling – Design and Installation
- AS 3600 - 2009 Concrete Structures
- AS 3798 - 2007 Guidelines on Earthworks for Commercial and Residential developments
- AS 3962 - 2001: Guidelines for Design of Marinas
- AS 4100 - 1998 Steel Structures
- AS 4678 - 2002 Earth Retaining Structures
- AS 4997 - 2005: Guidelines for Design of Maritime Structures
- AS 5100-2007 Bridge Design ( reference document)
- AS/NZS 1664 - 1997: Aluminium Structures
- AS/NZS 2312 - 2002: Guide to the protection of structural steelwork against atmospheric corrosion by use of protective coating
- AS/NZS 4586 - 2004: Slip Resistance Classification of New Pedestrian Surface Materials
- BS 6349-1:2000 British Standard Code of Practice for Maritime Structures
- NSW Maritime “Engineering Standards and Guidelines for Maritime Structures”
- PIANC – Guidelines for Design of Fenders Systems – 2002
- PIANC – Guidelines for the design of armoured slopes under open piled quay walls (Report Working Group 22) - 1997
- Public Works Department New South Wales “Boat Launching Ramps – Guidelines”
- USACE – Coastal Engineering Manual (“CEM”)
- “The Rock Manual”, CIRIA 2nd edition
- Environmentally Friendly Seawalls – A Guide to Improving the Environmental Value of Seawalls and Seawall-lined Foreshores in Estuaries (DECC, 2009)

# 4 DESIGN LOADS

## 4.1 IMPOSED DESIGN LOADS

The following loadings have been considered in the concept design of the maritime structures:

| Area or Facility | Superimposed Dead Load | Imposed Live Load |
|---|---|---|
| Rock Shore line | 40kPa ( Stacked Material ) not coincidental with live load | 20 kPa allowance for construction and emergency access vehicles |
| Paths adjacent to shoreline | 40kPa ( Stacked Material ) not coincidental with live load | 20 kPa allowance for construction and emergency access vehicles |
| Pontoon Structures | NA | 3 kPa |

## 4.2 LATERAL EARTH PRESSURE LOADINGS

Information on the geotechnical properties across the site is under development. Nominal properties used for the preliminary design are Ka= 0.3, Ko=0.5, Density of fill = 19kN/m3

## 4.3 HYDROSTATIC LOADINGS

A tidal lag of 1m has been allowed for in the preliminary design. This is based on the assumption that the fill material will be free draining. This assumption is to be confirmed by the geotechnical consultant.

## 4.4 ENVIRONMENTAL DESIGN PARAMETERS

Ref: Blumberg G. Et al, 2003, "Wave Climate Compliance at New Mooring Facility, Walsh Bay, Sydney Harbour", Coasts & Ports Australasian Conference 2003.

Sea State (local wind) Waves:

- 50yr ARI Hs=0.6m, Ts=2s
- Hmax = 1.86xHs (Ref: Coastal Engineering Manual)

Vessel Wake (adjusted design conditions from analysis of vessel wash monitoring records):

- Hs=0.36m Ts=2.5s
- Hmax = 0.51m (Blumberg)

Tidal current speed of 1.0 m/s

## 4.5 SERVICE AND ULTIMATE LOAD COMBINATIONS

Service and ultimate load combinations are in accordance with the recommendations of AS1170 and AS4997. The emphasis is on appropriate load combinations

# 5 STRUCTURAL DESIGN CRITERIA

## 5.1 STRENGTH

The structural strength of the maritime structures and its components shall be adequate to resist the load combinations in accordance with AS1170.0 and the relevant current code limit state provisions stated above.

## 5.2 STABILITY

Stability of the maritime structures and in particular the stability of the individual primary lateral load resisting structural elements when subjected to lateral earth pressure loading, using the code load combinations, shall be maintained in accordance with AS1170.0 Dead and Live Loads and Load Combinations and the relevant current code limit state provisions stated above.

## 5.3 DURABILITY

The project and site specific requirements to ensure the structural performance of the marine works are set out below.

### 5.3.1 DESIGN LIFE

The new maritime structures should have a design life of 100 years.

### 5.3.2 EXPOSURE CLASSIFICATION

Surface exposure classifications in accordance with AS4997 Table 6.3.are

- permanently submerged zone  Classification B2
- 1m below tidal to splash zone Classification C2
- spray zone Classification C1

### 5.3.3 PRESENCE OF AGGRESSIVE SOILS

With reference to ERM Pty Ltd report (reference 004432RP3 Rev Final) dated September 2006, the presence of sulphates soils provides an aggressive environment for concrete. This will influence the material design for elements located in this environment.

# 6 MATERIALS

## 6.1 CONCRETE

Proposed new concrete structure should have a design life of 100yrs. The concrete mix design should be based on a performance specification, with the concrete supplier to provide chloride diffusion modelling to provide a high level of assurance that the design life will be achieved. An indicative concrete would have a minimum compressive strength of 50mPa , Binder in excess of 500kg/m3 with high proportions (17% to 25%each ) of fly ash and blast furnace slag.

Limiting Crack Widths

Crack widths in marine structures shall be limited by means of limiting the allowable stress in reinforcement at serviceability state in accordance with AS 3600-2009 & AS 4997-2005 – Guideline for Design of Maritime Structure Section 6.3.7.2 for each of the exposure classification nominated therein

REINFORCEMENT

All reinforcing steel to be in accordance with AS 4671-2001 with material properties: Elastic Modulus, Es = 200 mPa    Yield Stress, fy = 500 mPa

Limiting Stresses

*(i) Ultimate Limit State Design*

Maximum stresses are as defined in AS 3600-2009, AS 4671-2001 and AS 4997-2005.

*(ii) Serviceability Limit State*

Stresses in reinforcement under unfactored service loads shall be limited as necessary to satisfy serviceability (crack width) requirements.

## 6.2 STRUCTURAL STEEL

It is not proposed to use structural steel on the maritime works, however if minor components are made from steel, it should be hot dip galvanised and painted with two coats of epoxy paint. Steelwork shall be designed to the requirements of AS 4997-2005. Regular maintenance will be required to ensure that these components achieve a 100 year design life.

## 6.3 STAINLESS STEEL

Stainless steel should be Marine grade 316 for components not located in the tidal or splash zones and super duplex stainless steel where components are located in tidal or splash zones.

## 6.4 ROCK AMOUR

The bedding layer rock and graded filter shall be basaltic igneous rock or equivalent rock.
Volcanic breccias or other low quality igneous rock shall not be used.
Bedding layer rock and graded filter shall have the following characteristics:
a) Minimum particle density as per AS 1141.6 – particle dry density 2.6 tonne/$m_3$
b) Maximum water absorption as per AS1141.6 – 1.5%;
c) Maximum loss – sodium sulphate as per AS 1141.24 – 6% over 5 cycles;
d) Maximum Los Angeles Abrasion as per AS1141.23 – 25%
e) Minimum wet strength as per AS 1141.22 – 150kN;
f) Maximum wet/dry strength variation as per AS 1141.22 – 25%;
g) Minimum wet unconfined compressive strength as per AS 4133.4.2 – 50MPa and
minimum wet over dry test result ratio of 25%; and
h) Minimum friction angle () of $38_o$.
For sandstone assume rock over sizing by 20% by weight to allow for weathering over 100 year design life and adopt a reduced rock density of 2400kg/m3

# 7 CONCEPT DESIGN

## 7.1 SHORELINE DESIGN

The concept design for the Headland Park is to create a headland similar to other major headlands in Sydney Harbour such as Lady Macquarie's Hair or Ball's Head. At the shoreline these parks are typified by horizontal rock platforms and pools stepping down into the harbour with underwater slopes and cliffs dropping to the harbour floor. The concept design replicates this form, with rock quarried on the site used to form the platforms. The existing quay line formed by concrete caissons is to be cut down and the fill material behind excavated to form the new harbour floor. The harbour floor and submerged embankment will be protected by rock armour.

The Guidelines "Environmentally Friendly Seawalls – A Guide to Improving the Environmental Value of Seawalls and Seawall-lined Foreshores in Estuaries (DECC, 2009)" have been considered in the concept design. The nature of the design is that it addresses many recommendations therein including,

- Low slopes
- Use of natural materials- local sandstone
- Form that assists habit diversity and complexity

The design is shown on Hyder Marine Works Concept Sketches AA003264 SK00, SK02, SK03, and SK04 in Appendix A. Sketches AA003264 SK06 and SK07 show sections with the steep embankment behind. Sketches SK01 and SK05 show the design for a shallower harbour option as noted in the Value Engineering Issues section of this report

### 7.1.1 HEADLAND PARK AND NORTH COVE

There are two primary sections of the shoreline: The visible section of the shoreline above the low tide mark and the section of shoreline below the low tide mark and not visible.

**Above Low Water:**

For visible section of the shoreline it is important for PWP and JPW to be able to achieve a naturalistic structure. A moderate slope (approximately 1 in 3 to 1 in 4 ) with large flat rocks forming steps and rock pools is considered by the design team to be an appropriate solution. The large sandstone rocks will be sourced from site as part of the quarrying activities.

Under the current marine concept design the rocks that form the edge are bedded on a gravel bed of approximately 1m, and then further bedded by forcing down with an excavator bucket. This can be done in the wet. Alternatively the rocks may be placed on a concrete bedding slab. This will require dewatering to achieve placement.

The bedding on a gravel layer will reduce the cost; however there will be a change in functionality of the pools as they will drain as the tide drops and fill back in as it rises. Further the rocks within the rock pool will have to be large approximately 500 mm so that the embankment is protected from wave action.

**Below Low Water:**

**Existing Caissons**

On the western and northern sides of the site, the proposed shoreline is close or above the existing caissons. These caissons retain the fill behind them, under the existing hardstand area and cannot be removed without the site slipping into Darling Harbour. It is proposed to retain as much of the existing caisson structures as possible. The proposed methodology is to remove sand fill within the front row of caisson compartments down to 0.8 m below the design finished levels, place 0.8 m of concrete over existing sand fill, pumped into place by tremie methods. This will lock the existing caisson structure together at that level. The caisson wall will then be demolished down to the design level. The back row of caisson compartments will be excavated to 1m below the design level and filled with of 1 m of rock armour over a geotextile.

The concrete in the outer cell is desirable as it provides

- Good load sharing to the total caisson,
- Provides a good point of stiffness for the cutting of the concrete caisson walls
- It provides a good surface for clean up from the concrete cutting

Further the top of the caissons are not structural so that local deterioration of the concrete and reinforcement will not be an issue

**Embankment and New Harbour Floor**

A number of structure types were examined for the section below low water including diaphragm walls, contiguous pile walls, secant pile walls, CFA pile with jet grout between, gravity walls such as counterforts, caissons, jet grout, however an embankment with rock ballast (amour, rip rap) facing solution was chosen as it was the most cost effective and could be placed in the wet if required. Refer Photo 1 and Concept Design Sections on SK02 andSK03 for details

**<u>Concept Design Rock Sizing Summary</u>**

**Bottom Protection** (assumed high quality sandstone and not to be placed at toe of revetment/seawall or above -4m AHD)**:**

- Design Armour Size: $M_{50}$=84kg & $D_{50}$=325mm
- Specified Armour Size: $M_{50}$=84kg & $D_{50}$=355mm (visual/sieve grading Ref: PIANC WG22 Table 5.1)
- Grading (rip rap grading 0.5 to 2 x $W_{50}$, Ref: PIANC WG22):

$M_{min}$= 40kg $M_{50}$= 85kg $M_{max}$= 170kg

$D_{min}$= 285mm $D_{50}$=355mm $D_{max}$= 465mm

- Min armour thickness = 1.5 to 1.8x$D_{50}$ = 600mm (no underlayer, rock placed directly on geofabric)

**Slope Protection and Wave Exposure** (assumed basalt and slope no steeper than 1 in 2)**:**

- Design Armour Size: 1.5x$D_{50}$ bottom protection min = $M_{50}$=235kg & $D_{50}$=450mm (Ref: PIANC WG22) therefore wave governs $M_{50}$=268kg & $D_{50}$=469mm
- Specified Armour Size: $M_{50}$=268kg & $D_{50}$=540mm (visual/sieve grading Ref: PIANC WG22 Table 5.1)
- Grading (rip rap grading 0.5 to 2 x $W_{50}$, Ref: PIANC WG22):

$M_{min}$= 135kg $M_{50}$= 270kg $M_{max}$= 540kg

$D_{min}$= 570mm $D_{50}$=540mm $D_{max}$= 685mm

Min armour thickness = 1.5 to 1.8xD50 = 1000mm (no underlayer, rock placed directly on geofabric )

For sandstone assume rock oversizing by 20% by weight will allow for weathering over 100 year design life and adopt a reduced rock density of 2400kg/$m^3$.

The rock sizes provided are the equivalent cube sizes determined through calculation.

The thickness of rock armour has been increased to 900mm in the base areas and 1400 thick on the slope to allow for placement under water. This will be revisited at detail design with the view of reducing it, however at this stage it provides a good contingency

Photo 1: Rock Amour: Large Rock Amour Rocks over geotextile protection layer over geotextile on embankment.

### 7.1.2 MOORE'S WHARF WALL

At Moore's Wharf the current design indicates a vertical wall to approximately RL -2 i.e. 1 m below low tide. In order to achieve this a diaphragm walls or contiguous pile wall would have to be installed. This type of construction is very expensive. It is therefore proposed to install a smaller gravity type wall to RL 0 and then place an embankment with rock ballast facing. Please refer Concept Design Sketch AA003264-SK03 Section 6

### 7.1.3 SOUTH SIDE OF NORTH COVE

The current design for the south side of North Cove is for a vertical similar to Moore's Wharf. As with Moore's Wharf it is proposed to install a smaller gravity type wall to RL 0 and then place an embankment with rock ballast facing. Please refer Concept Design Sketch AA003264-SK03 Section 4

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## 7.2 NORTH COVE

The depth and treatment of North Cove is driven by the appropriate vessel expected in North Cove.

**DESIGN VESSELS**

Design vessel which will access near shore area and new bay.

The Australian Code AS3962.1 2001 Guidelines for design of marinas Section 3.2 provides guidelines water depths as follows.

Design Depth = Vessel Draught + half wave height + allowance for siltation + clearance for hard surface

Vessel draughts :

| Length | Power | Yacht |
|---|---|---|
| 15m | 1.2m | 2.5m |
| 25m | 1.8m | 3.0m |
| 35m | 2.1m | 3.8m |

Half wave height= 0.5m, allowance for siltation=0.2m, clearance for hard surface=0.5m

Total Depth Required:

| Length | Power | Yacht |
|---|---|---|
| 15m | 2.4m | 3.7m |
| 25m | 3.0m | 4.2m |
| 35m | 3.3m | 5.0m |

The design depth chosen by the team is for the base of the cove at RL-3.0. This provide access for small craft such as water taxis at all times and access to larger craft ie up to 25 m for tides above mean Sea Water Level.

The base of North Cove will have a rock ballast layer approximately 900mm thick over a geofabric to provide scour protection to the floor of North Cove.

## 7.3 BEACH

The marine concept design for the beach is shown on Concept Design Sketch AA003264-SK04 Section 8 and comprises 600mm sand on 150mm gravel over geofabric. This is a very free draining formation. The fill material under this is builders rubble, crushed sandstone and a variety if fill which is likely to be very permeable. As the tide drops below the stone wall across the bay at RL 0.0 the seawater will continue to drain out through the substrata , and as the tide comes back in the water will travel back through the substrata and percolate up through the sand. If there are areas of greater permeability than others there could be springs of water entering the beach area. This is not a desirable effect. A simple solution to this would be to stagger the wall so that water could freely flow in and out.

Alternatively the beach could be placed on a water-resistant concrete slab which would be 300 to 400mmthick , fully reinforced with water stops . This would usually require the area to be dewatered. The concrete may be placed underwater, however the thickness would have to be increase to about 600mm to allow for dispersion effects of placement under water. In lieu of this a membrane may be used , however the chances of such a membrane being punctured are very high and it is not recommended.

The upper beach edge interface with the general area will require some sort of small retaining wall to stop wave wash out erosion, wind drift of sand up the beach on to the general area and contamination of sand by general fill. An embankment will often not work in this area. The sandstone from the existing wall could be used to form this wall. Such walls are generally 600mm to 900mm high. Please see attached photos from local sites Photos 4 and 5.

Typical beach slopes are 1 in 20 for fine grain sands i.e. 0.3mm diameter to 1 in 10 for coarse grain sands .Please refer to Photos 2 and 3.

Notwithstanding the above there are issues regarding water quality, safety and risk management as raised by Hyder which will have to be addressed in the future design and strategy.

Photo 2: Beach with  1 in 10 natural slope

Photo 3: Beach with 1 in 20 slope

Photo 4: Wall at back of beach to contain sand

Photo 5: Erosion on beach with no back wall

## 7.4 PONTOONS

It is proposed to use a proprietary brand pontoon as used in Darling Harbour and Port Botany Boat Ramp. The prime function of the pontoon is to provide drop off access for water taxis. It is not a mooring point for pleasure craft.

Please refer to Photos 6 and 7 for typical pontoons.

Photo 6: Pontoon for small craft such as water taxi.

Photo 7: Pontoon for larger vessels such as small ferries

## 7.5 ISSUES FOR FURTHER CONSIDERATION

- The stability of the rock amour embankment at the north western end of the site is under review by Douglas Partners.
- Stability of the caisson with a steep embankment behind is under review by Douglas Partners
- Cathodic protection has been nominated in the CTI report of 13th October 2008. At this stage an allowance should be made for this. The cathodic protection itself is not expensive however it will have to be installed by divers which can be expensive. There is an opportunity to make a saving on this and to this end it may be worthwhile getting a Durability consultant on board and divers to carry out a more extensive investigation, taking samples for chloride diffusion testing.

# 8 CONSTRUCTION CONSIDERATIONS

## 8.1 VARIED FILL MATERIAL

The fill behind the caissons is extremely varied and consists of a range of materials from crushed sandstone, bricks and rubble from CBD building sites, and the remains of previous wharves and plant over alluvial clays and rock. It is likely to be extremely permeable. Driving piles in particular sheet piles may not be possible due to the number of obstructions imbedded in the fill. Further the existing caissons rest on a rock bedding layer which will allow free passage of water into excavations. Excavation in the dry is likely to involve substantial dewatering and bunding operations.

## 8.2 VARIED DEPTH OF UNDERLYING SANDSTONE

The depth to bedrock varies across the site; this is unlikely to impact on the maritime works unless the rock is in the design zone. If this is the case a local rearrangement of the design would be the most cost effective method of dealing with this

## 8.3 STABILITY OF EXISING CAISSON WALL

During construction of the Headland Park, the earth behind the caissons is to be excavated and in general this action will reduce the load on the caisson and therefore will increase the caisson stability. This excavation does leave the caissons more vulnerable to wave actions and ship collision loads; however it expected that ships will be precluded from the area during construction and as wave heights within the cove are low at around 1m, these sea face loads are considered less critical than the earth pressures from the landward face.

The caisson is a gravity retaining structure and as such its stability is attained by the mass of the concrete walls and the sand fill within the caisson cells. A preliminary review of the stability of the existing caissons was calculated for various active earth pressure coefficients for its current height with a construction surcharge load of 20kPa applied. Checks for overturning, sliding and bearing failure were calculated and it was found that the caissons had a Factor Of Safety (FOS) of 1.5 for an earth pressure coefficient of up to Ka = 0.33. This was based on a founding allowable bearing capacity of 400kPa and a subgrade friction coefficient of 0.6. These factors along with the true active earth pressures would need to be provided by a geotechnical engineer to confirm the caisson stability, however it is expected that the caissons would be stable during construction and in the final condition. Further to this, the geotechnical consultant will be carrying out stability analysis and slip circle analysis of the wall and adjacent embankments

The most likely mode of failure for the caissons would be a gradual sliding of the caissons towards the sea. This failure mechanism has the tendency of reducing the earth pressure loads until the structure become stable again. This failure would likely be un- noticed in the final condition as the caissons are not visible

## 8.4 DRY/WET CONSTRUCTION AND DE-WATERING

Marine work of this nature is carried out in dry conditions where ever possible. Dry excavation is quicker, allows for the employment of a variety of plant, and provides good material handling outcomes. Finished levels and surface finishes are better as contractors and operators are able to physically see their work area. Large excavators are usually used to carry out the excavation as they hold large volumes, are accurate, can be guided by GPS and laser systems and can be used for a variety of tasks from excavation to trimming and rock placement. The back of the bucket can be used to forcefully bed armour down to achieve a relatively flat finished surface refer photo 11. Where the excavation extends below the water table, it is common to dewater using a sump and pumps.

For this site the fill behind the caisson wall is a mixture of crushed sandstone and builder's rubble and waste. This is likely to be a very permeable material. Further the existing caisson wall is founded on a 1.5m layer of large rock 200mm to 300 mm diameter which is very free draining. It is very unlikely that it will be possible to carry out dry excavation in large areas without large scale dewatering.

The alternatives to full dry excavation are local dry excavation whereby a local area is bunded off and local dewatering is carried out. This type of construction methodology may be employed by contactors for the shoreline edge structure where work will be visible and appearance of finished work is important. Please refer to Photos 9 and 10 for typical bunding arrangement.

The other alternative to dry excavation is full wet exaction and structure placement. Please refer to Photo 8 for typical wet excavation. This work usually involves the use of large long reach excavators. The excavation progresses back with excavation carried out flowed by placement of geofabric and rock armour. The geofabric often has to be placed by divers. A reasonable allowance has to be made for placement in the wet as the work area cannot be seen.

It is preferred to carry out wet excavation in protected waters; it is therefore proposed the caisson walls should be demolished as the last part of the works. Please refer to construction sequence below.


Photo 8 75 Tonne Excavator carrying out wet excavation

Photo 9 Typical bunded area for local dewatering

Photo 10 Bunded area after breakthrough

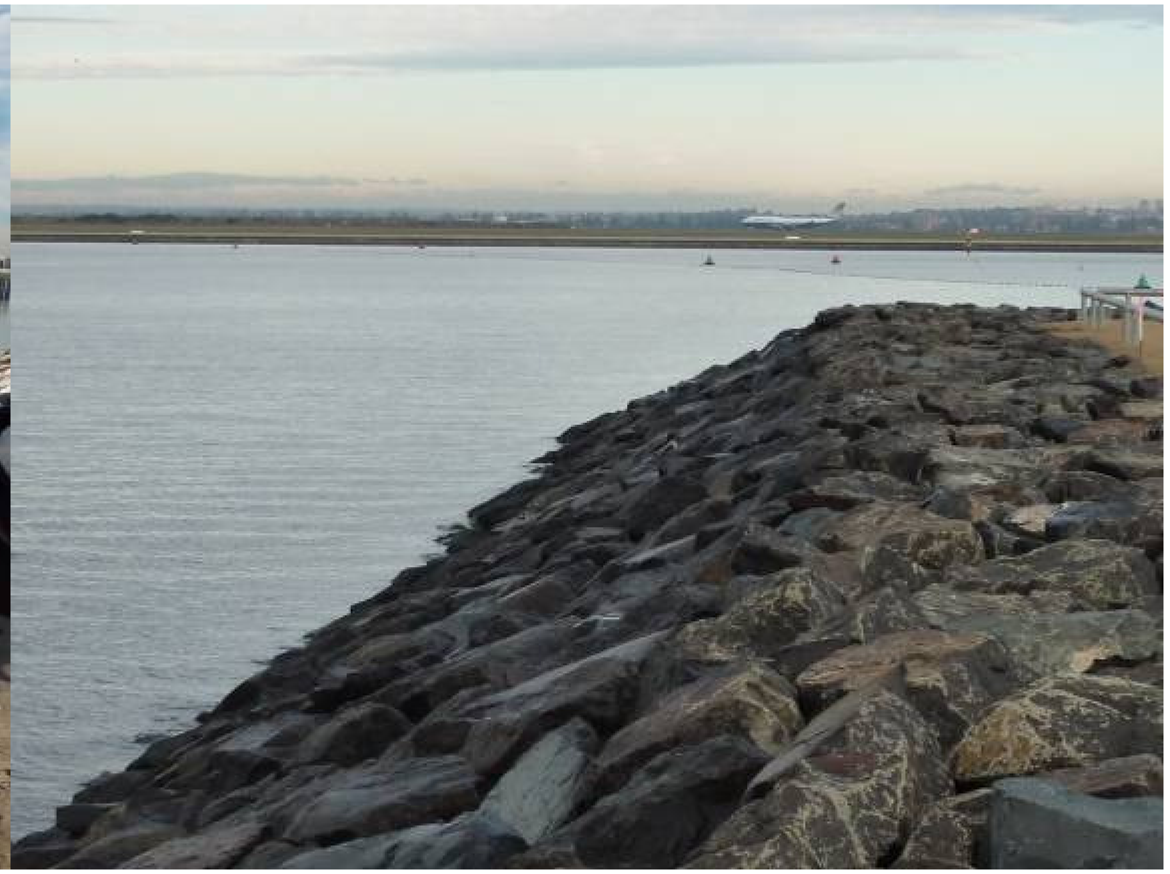

Photo 11: Relatively flat finished rock face achieved by the back of bucket of a large excavator.

# 8.5 EXISTING NORTHER SEA WALL

The existing sandstone sea wall is located at the northern end of Barangaroo and forms the majority of the western face of wharf three. It extends for approximately 120m and is founded on a ballast bank overlying rock. To the south of the wall is wharf 4 which consists of precast concrete caissons of approximately 15m in height. To the north is a small region of gravity 'L' shaped precast concrete retaining walls of a similar height to the sandstone retaining wall, which is founded on the same ballast embankment and is shown on drawing A1819 from 1979. The areas to the north and south are used as mooring areas for ships, however the area adjacent to the sandstone wall is not used as a mooring area.

There are no existing drawings for the construction of the sandstone retaining wall, however drawing A1-800 from 1977 shows the makeup of the wall and a scaled height of around 4m, with the toe of the wall at RL 0.000m. Drawing B6576 from 1968 shows the original extent of the wall and it is inferred from these drawings that the extent of the wall was reduced in the 1970's to its current size. Drawing B7390 from 1972 shows the wall founding on compacted ballast bank 13m high at a slope of 1:1.5. The stability of this bank has not been determined, however SKM have made a cursory comment regarding this, which will be outlined below .

"The exposed upper portion of the wall appears to be vertical and in fair condition. There appears to be some erosion above the high tide mark, which is likely due to salt re-crystalisation within the sandstone matrix from the wetting and drying cycle. The Seawall Assessment report produced by SKM (August 2005) report notes the following regarding the condition of the wall;

The sandstone wall on the western face is in fair condition. There are some rocks on the seabed in two areas along the length of the wall. The toe of the wall at the seabed appears to be stable."

The proposed detail for the area that encompasses the existing sandstone wall in the new Headland Park concept would involve the complete dismantling of the sandstone wall, and it is thought that the this sandstone could be salvaged for use around the foreshore. The proposed detail relies heavily on the stability of the ballast embankment that the wall is founded upon and it is essential to have a geotechnical assessment of the this for safe implementation of the concept.

# 8.6 WATER BLASTING OF STONE TO ACHIEVE NATURAL EFFECT

A trial of water blasting of sandstone shoreline blocks to achieve a natural effect was carried out on 2nd September 2010 at the Government Stone Masons yard at 92 Burrows Road, Alexandria. The effect achieved was similar to that of natural stone. Currently it is proposed to carry out the water blasting of the sandstone once the rock has been placed in its final position.

Please see Photo 12 below which shows the effect of the water blasting. It should be noted that the runoff from the water blasting should be natural local material and as such there should be no chemical impact from the water blasting activities. With regards to the silt in the runoff from the water blasting, although the exact details are not yet developed, the silt will be controlled during construction by a number of methods. For those areas of work carried out before the caissons are removed, the caissons will control any silt. For those areas of work which are carried out outside the caissons or after the caissons are removed, a turbidity curtain or similar will be used to control the silt.and turbitity. Please refer to 'Soil and Water Report to support the Environmental Assessment for Headland Park Barangaroo, Barangaroo Delivery Authority, September 2010" by WSP Environment and Energy section 3.5.2 Potential Impacts, Park Foreshore.

Photo 12 Water Blasted Sandstone

# 9 CONSTRUCTION SEQUENCE

A possible construction methodology is as follows

- Remove existing capping beam. It should be noted that the underside of the existing capping beam is at RL 0.595 AHD which is just above Mean High Water Level at RL 0.515 AHD so that at extreme tidal events the top of the existing caissons will be submerged. An allowance should be made for a temporary small rock embankment on top of the caissons during construction.
- Excavate sand from outer cells of caisson
- Dewater cell locally one at a time
- Place concrete in outer cell, alternatively the concrete may be placed by trammie without dewatering
- Leaving a berm of about 6 metres adjacent to the caissons start excavation behind the caissons of the North Cove and areas behind the caissons. This excavation may be carried out simultaneously on a number of fronts. By leaving the caissons in place the work area will be in protected waters which will make excavation and placement easier
- Once the areas behind the caissons have been completed the berm and the caisson can be cut back and rock armour placed by excavator, with the excavators retreating along the berm caisson peninsular . This work may also be done from each end.

# 10 VALUE ENGINEERING ISSUES

The following issues were raised during the value engineering review on 25th June 2010

- The raising of the floor of North Cove and the cove adjacent to Moore's Wharf will provide significant saving and are shown on SK01 SK05
- The contiguous pile wall adjacent to Moore's Wharf and to the south side of North Cove has been replaced with a gravity precast concrete wall , refer sections 4 and 6 of SK03 and SK05
- The PWP option of park at Moore's Wharf and leaving the sandstone wall in place will offer significant savings, please note that the gravity wall noted above can be used for park at Moore's Wharf for the short distance from the back if the caisson,. The caisson should be cut off at a cell wall to form a retaining wall for the last 10m to the outer face

# 11 FURTHER INVESTIGATIVE WORKS

**Other information required to complete the works:**

Final 3D ground survey, It is our understanding that this is underway

Hydrographical survey of adjacent harbour floor, It is our understanding that this will take place shortly.

Update geotechnical information and recommendations from Douglas Partners Pty Ltd including

Slip circle analysis /geotechnical global stability checks of the modified quay structures, proposed new foreshore structural configuration and proposed combination of modified quay structure and new foreshore edge structure.

- Recommendation on design parameters for backfill behind existing quay structures and new foreshore edge structures
- foundation recommendations for new edge structures and existing quay structures.
- factual report on stratum beneath the existing quay structures and proposed new foreshore edge structures.
- factual report on fill material under the hardstand area and behind existing quay walls.
- factual report on stratum below Moores Wharf.
- report on ground water and contaminates (for corrosion issues ) within the fill.

Condition survey of all existing quay structures which should include as a minimum:

- a report detailing existing defects type, location and recommended remedial repairs
- depth of chloride ingress and prediction of remaining serviceable life
- depth of carbonation and prediction of remaining serviceable life.
- inspection for sulphate attack on the concrete structures

This is best done by a specialist consultant with divers, Hyder diagnostic team carries out this work on a regular basis for projects of this nature

Durability Report: this is best done by a specialist consultant especially considering the 100 yr design life

# 12 EXCLUSIONS

The following work is not cover as part of this commission

- Design of structures above RL 2.0m
- Durability report on the existing quay structure and proposed foreshore edge structures
- Temporary works designs to enable the construction of the new foreshore structures and modification of the existing quay structures.
- Carrying out a condition survey of the existing quay structures.
- Design of cathodic protection system of the existing and new structures.

# 13 REFERENCE DRAWINGS AND REPORTS

## 13.1 ARCHITECTURAL DRAWINGS FROM JPW

| Drawing No. | Drawing title | File Name | Date | Revision | Author |
|---|---|---|---|---|---|
| - | - | 100518_4_Site_Plan_Morphology | | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Central Parklands Section 8/9 | 100518_CP_ Sc 8-9 | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Central Parklands Section 10 | 100518_CP_Sc 10 | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Central Parklands Promenade Section 11 | 100518_CP_Sc 11-12_Promenade | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Central Parklands Canal/Hickson Section 13 | 100518_CP_Sc 13_Canal | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Headland Park Section 1 | 100518_HP_Sc 1 | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Headland Park Section 2 | 100518_HP_Sc 2 | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Headland Park Section 3 | 100518_HP_Sc 3 | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Headland Park Section 4 | 100518_HP_Sc 4 | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Headland Park Section 5 | 100518_HP_Sc 5 | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Headland Park North Cove Section 6 | 100518_HP_Sc 6 | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Headland Park North Cove Section 7 | 100518_HP_Sc 7 | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | - | 100518_Plan_1_Diagramatic_landscape | - | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Diagramatic Plan | 100518_Plan_1_Diagramatic_Portrait | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | - | 100518_Plan_3_Descriptive Site Plan_Landscape | - | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Descriptive Site Plan | 100518_Plan_3_Descriptive Site Plan_Portrait | 13-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | - | 100518_Plan_4_Morphology_Landscape | - | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | - | 100518_Plan_5_Descriptive Site Plan w Trees_Landscape | - | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Descriptive Site Plan with Trees | 100518_Plan_5_Descriptive Site Plan w Trees_Portrait | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Potential Program Locations | 100518_Plan_6_Potential Program Locations | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Hickson Boulevard Option 1 and 2 | 100518_Plan_7_Hickson Options | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Canal Option 1 and 2 | 100518_Plan_8_Canal Options | 18-May-10 | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | Hickson Boulevard-Tram Left | 100518-C-SC-HICKSON-BLVD. | - | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | - | Barangaroo-board presentation#2reduced-100517 | - | - | Johnson Pilton Walker\|PWP Landscape Architecture |
| - | - | TITLE | - | - | Johnson Pilton Walker\|PWP Landscape Architecture |

## 13.2 BARANGAROO DELIVERY AUTHORITY DRAWING AND REPORT INFORMATION

The following information supplied by the BDA was referred to in this concept design.

### 13.2.1 REPORTS

The Maritime Services Board of NSW- 7,8 and 9 Darling Harbour Quay Face Wall – General Arrangement at Caissons 8,9,16 &17

CTI Consultants Pty Ltd – Concrete Condition Assessment Barangaroo Sea Wall – Job No: 2200 – Report No: C10656 dated 13th October 2008

Sinclair Knight Mertz Report "East Darling Harbour – Urban Design Competition Stage 1: Advice on seawall aspects of the Finalists" dated September 2005

Sinclair Knight Mertz Report "East Darling Harbour – Millers Point: Seawall Assessment" dated August Sear 2005

Sinclair Knight Mertz memo dated 18th August 2006 titled "East Darling Harbour Feasibility of Sea Wall/Edge Conditions" – Reference IN08953.200

Scanned Seawall feedback email dated 13/09/2006 prepared by CMH

Scanned Seawall Feasibility Options dated 28/08/2006 prepared by CMH

Rapid Metal Developments - The MSB of NSW Darling Harbour Berths 6 & 7 drawing N89/42-4a

### 13.2.2 DRAWINGS

A1-2-A
A1-3-B
A1-4-A
A1-6-B
A1-7-C
A1-9-B
A1-11-A
A1-12-A
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A1-819-B

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## Appendix A

# Hyder Concept Design Sketches

Hyder Marine Works Concept Sketches AA003264 SK00, SK01, SK02, SK03, SK04, SK05, SK07


| Issue | Description | Date |
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| A | CONCEPT ONLY | - |

Client

BARANGAROO DELIVERY AUTHORITY

Status

**PRELIMINARY**
NOT USED FOR CONSTRUCTION

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| Original Size | A1 | Designed TR |
| Height Datum | AHD | Checked |
| Grid | MGA | Approved PM |

Filename: SK00-AA003264-NSD-00.dwg

Project

BARANGAROO
HEADLAND PARK
AND PUBLIC DOMAIN
MARINE WORKS

Title

GENERAL ARRANGEMENT
EXISTING CAISSON BLOCK
LOCATION PLAN

Hyder


HYDER CONSULTING PTY LTD
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| Drawing No. | Project No. | Issue |
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Client

BARANGAROO DELIVERY AUTHORITY

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Project

BARANGAROO HEADLAND PARK AND PUBLIC DOMAIN MARINE WORKS

Title

GENERAL ARRANGEMENT EXISTING CAISSON BLOCK LOCATION PLAN RAISED HARBOUR FLOOR OPTION


Hyder

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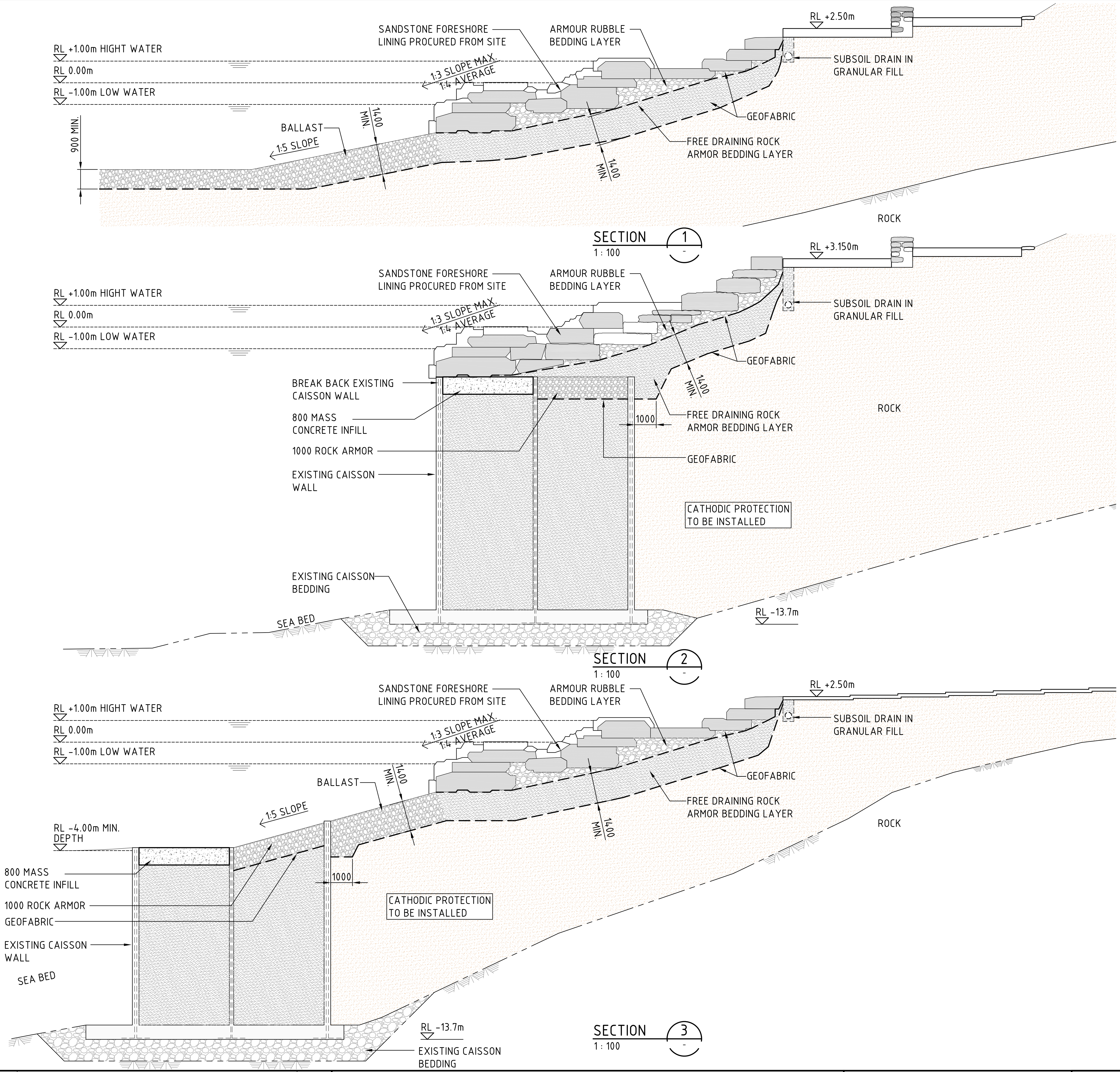


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Project: BARANGAROO HEADLAND PARK AND PUBLIC DOMAIN MARINE WORKS

Title: GENERAL ARRANGEMENT TYPICAL SECTIONS

SHEET 1


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| Drawing No. | Project No. | Issue |
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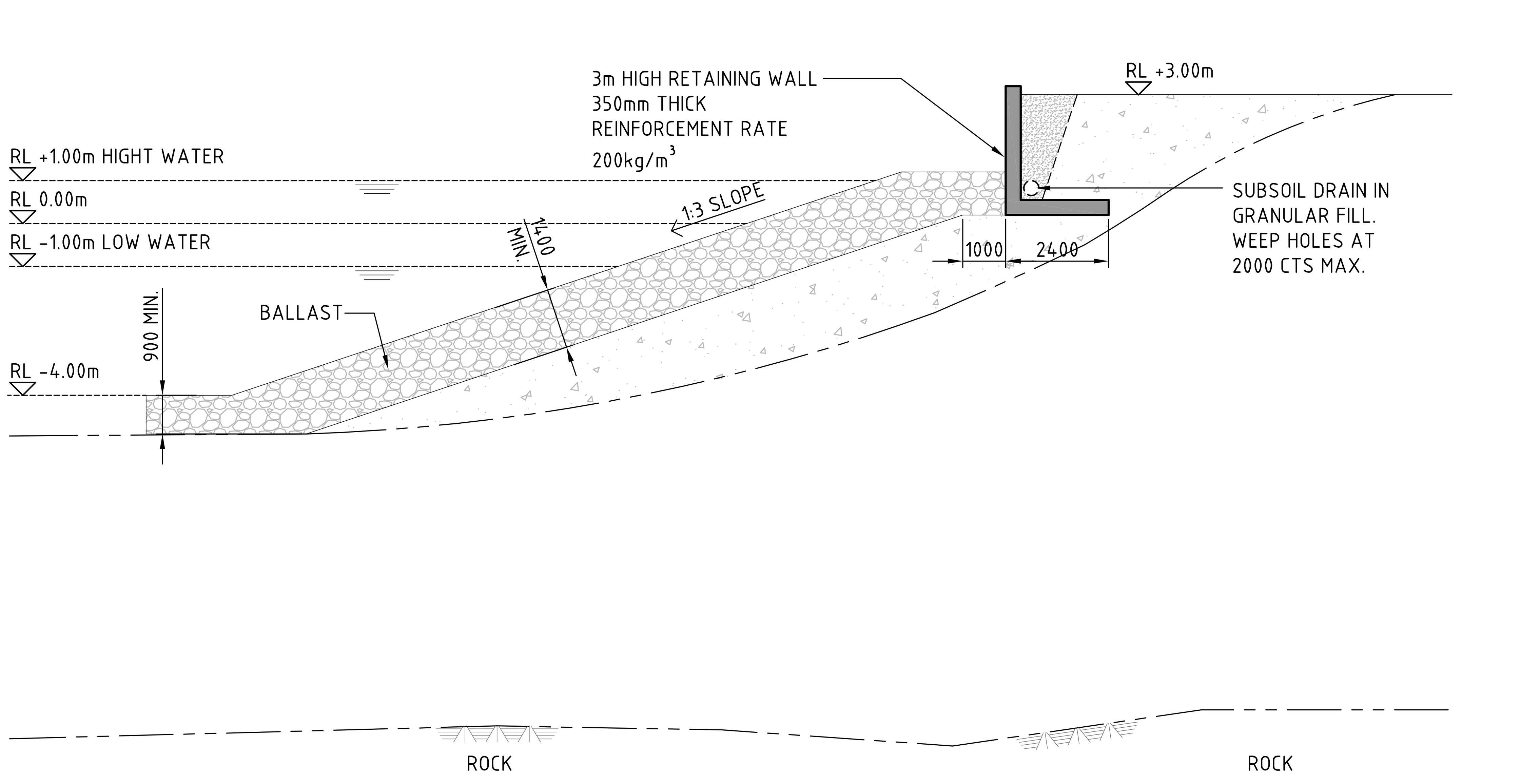


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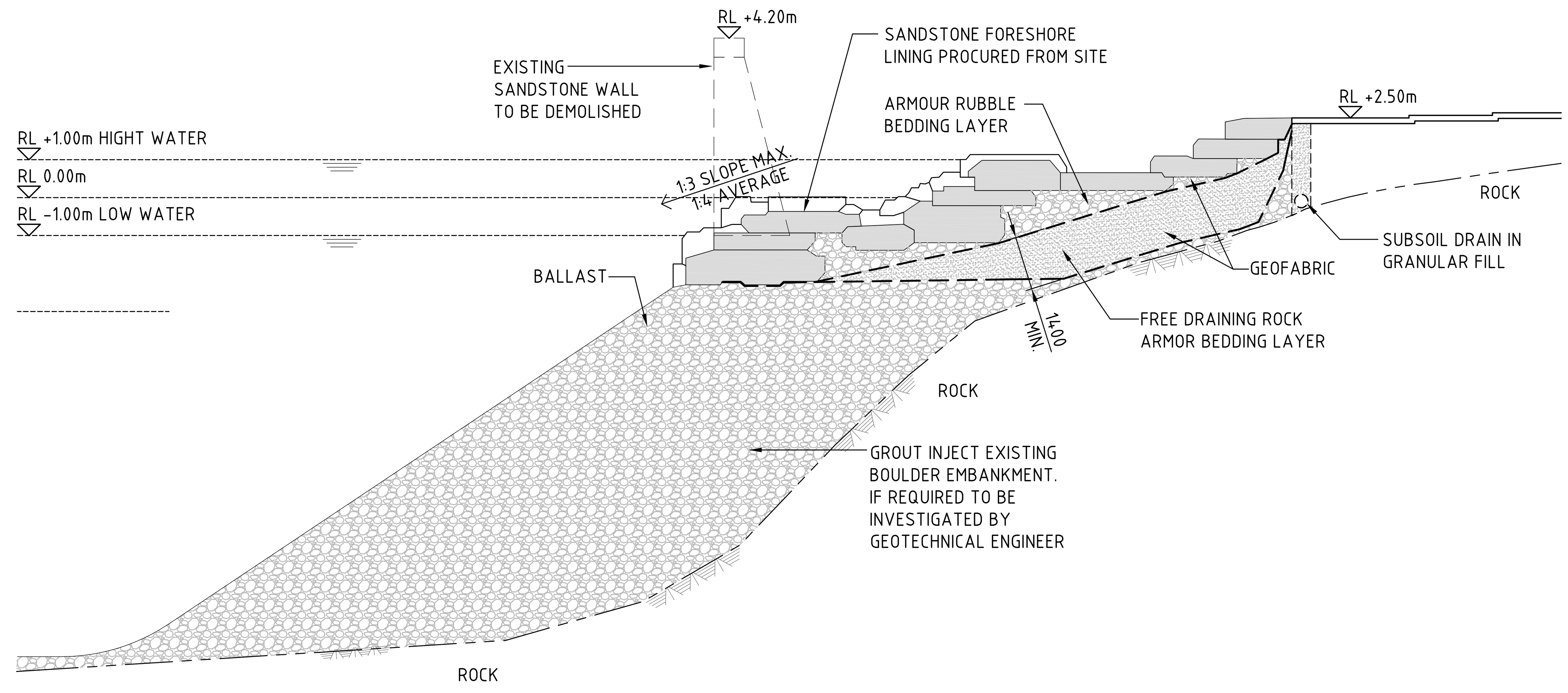


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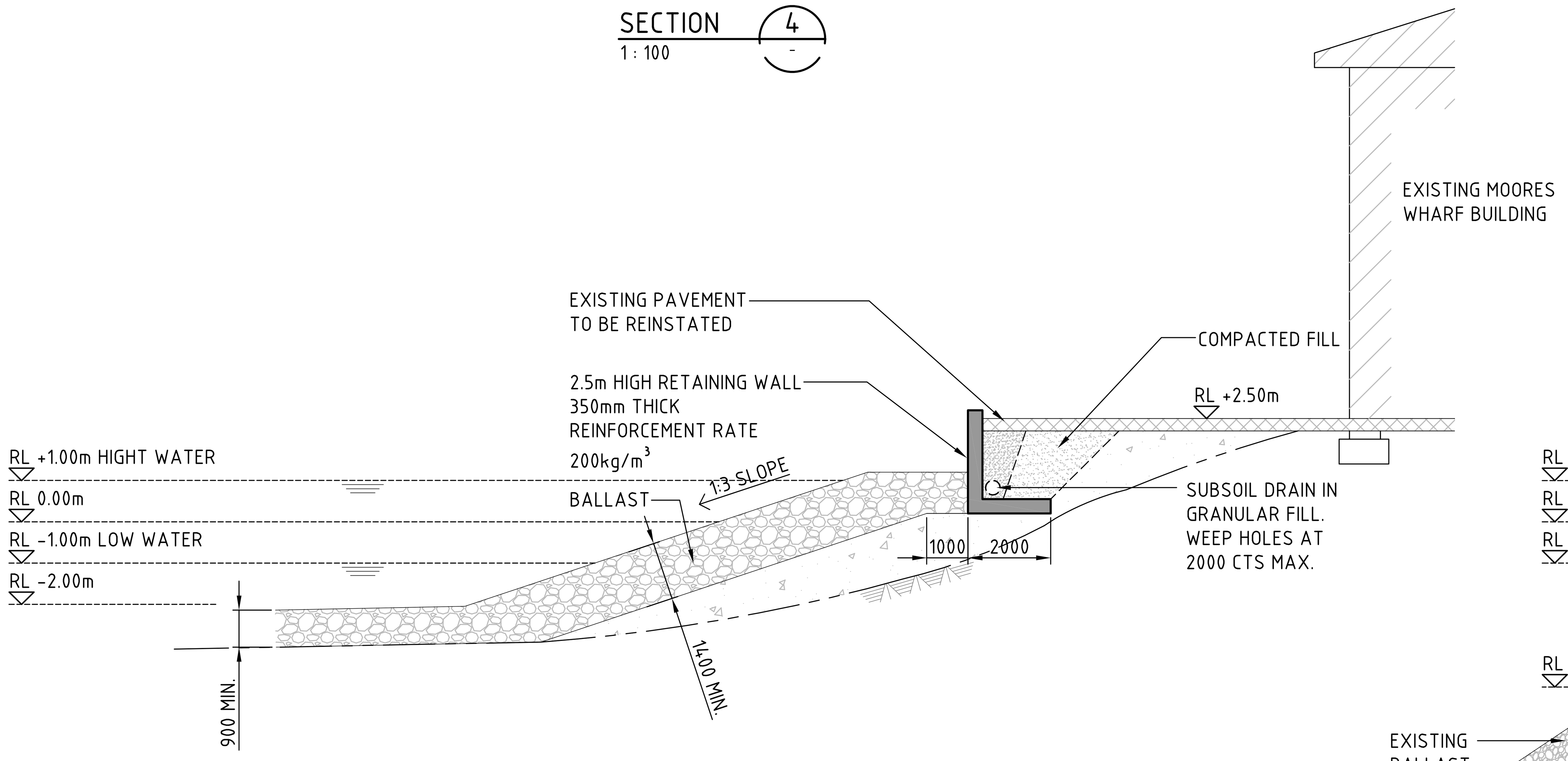


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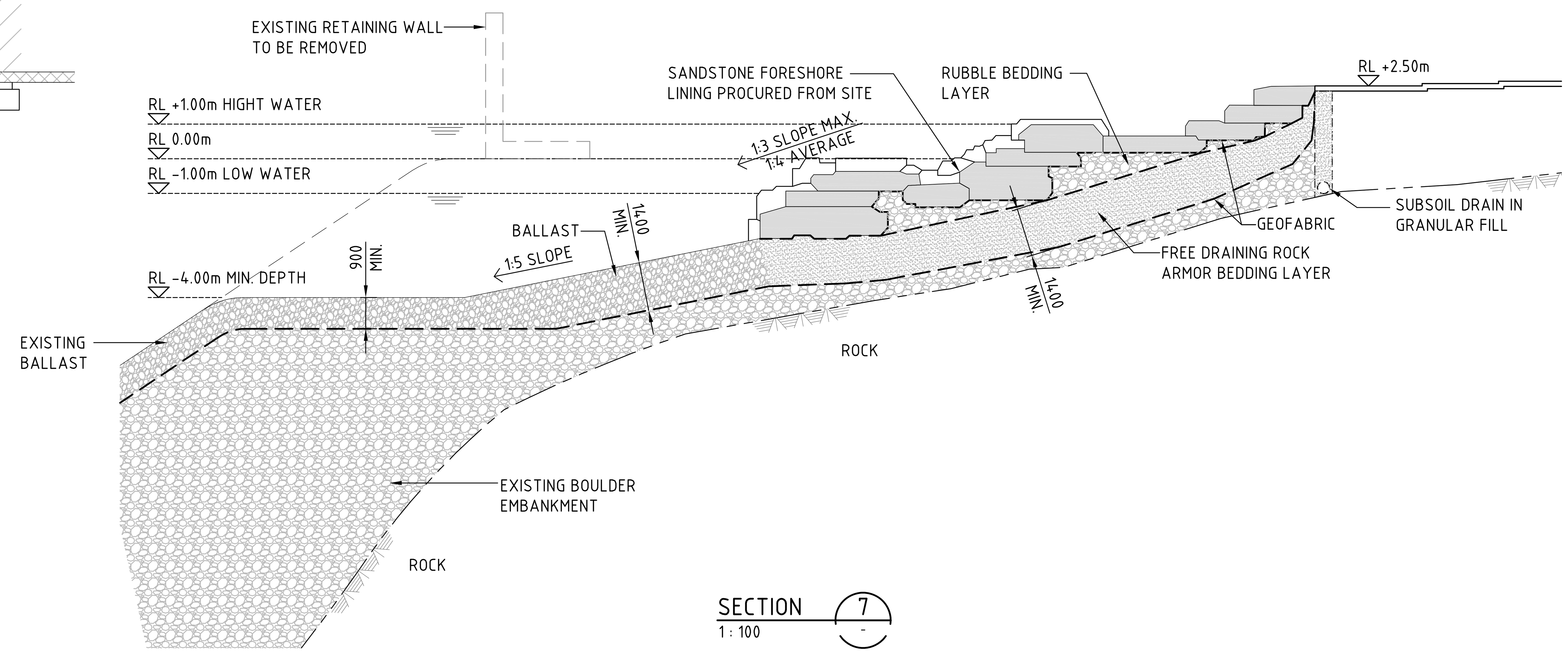


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Client
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Status
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| Grid | MGA | Approved PM | |

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Project
BARANGAROO
HEADLAND PARK
AND PUBLIC DOMAIN
MARINE WORKS

Title
GENERAL ARRANGEMENT
TYPICAL SECTIONS

SHEET 2

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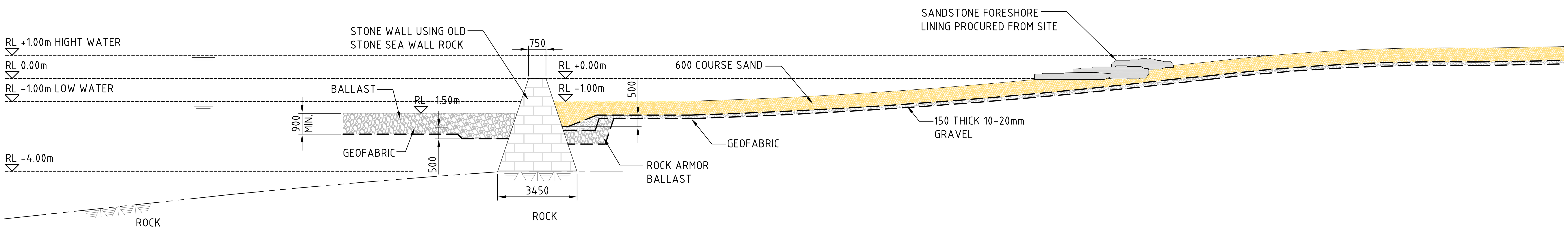


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Status: **PRELIMINARY**
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Project: BARANGAROO HEADLAND PARK AND PUBLIC DOMAIN MARINE WORKS

Title: GENERAL ARRANGEMENT TYPICAL SECTIONS

SHEET 3


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| Drawing No. | Project No. | Issue |
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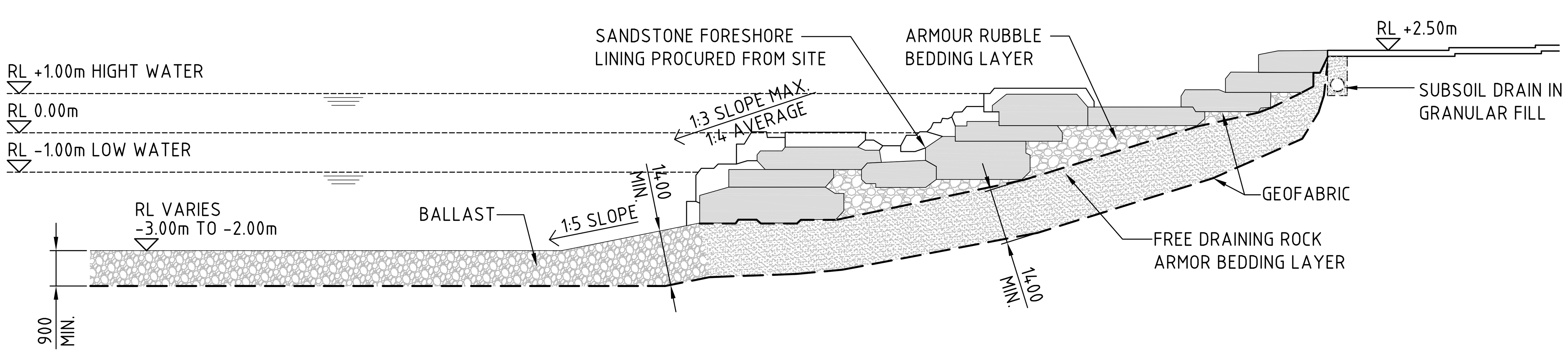


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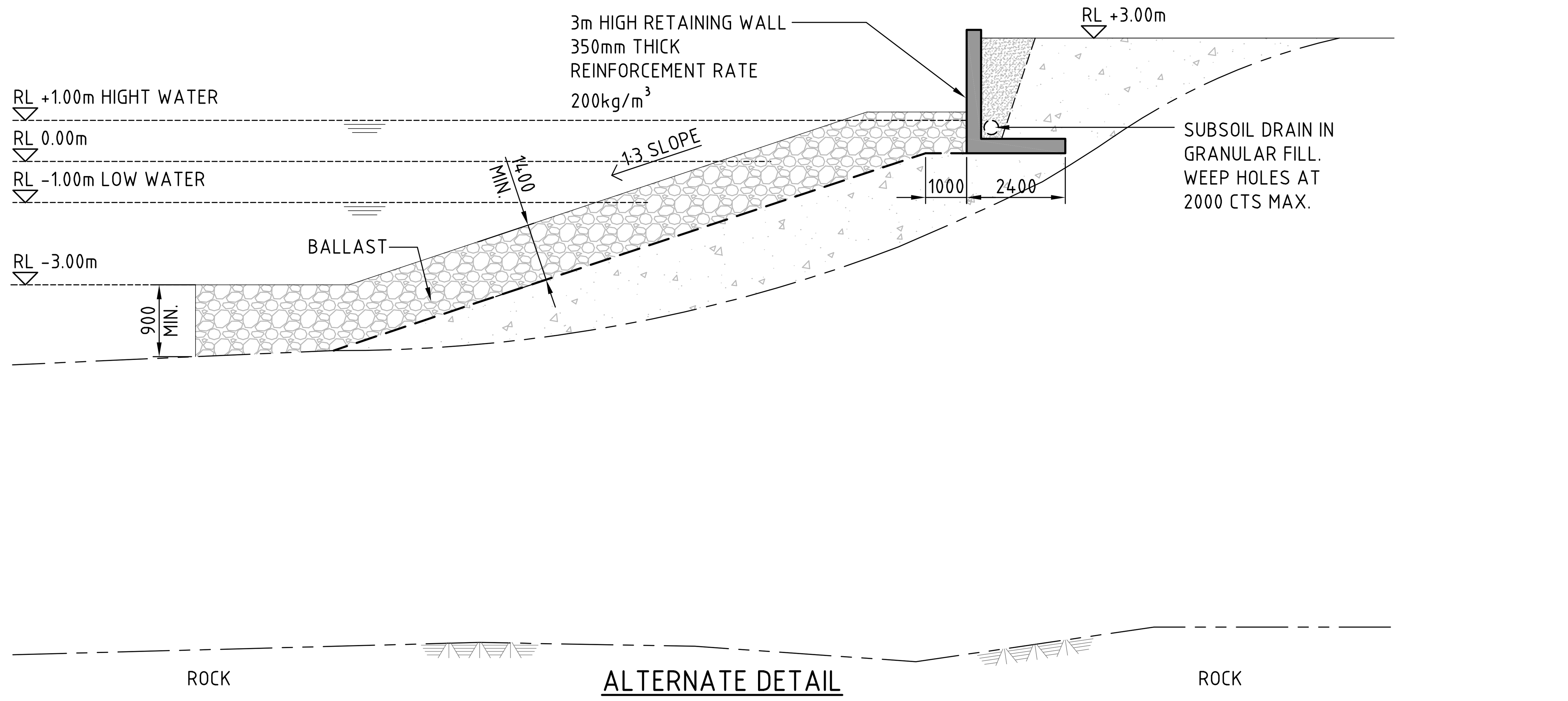


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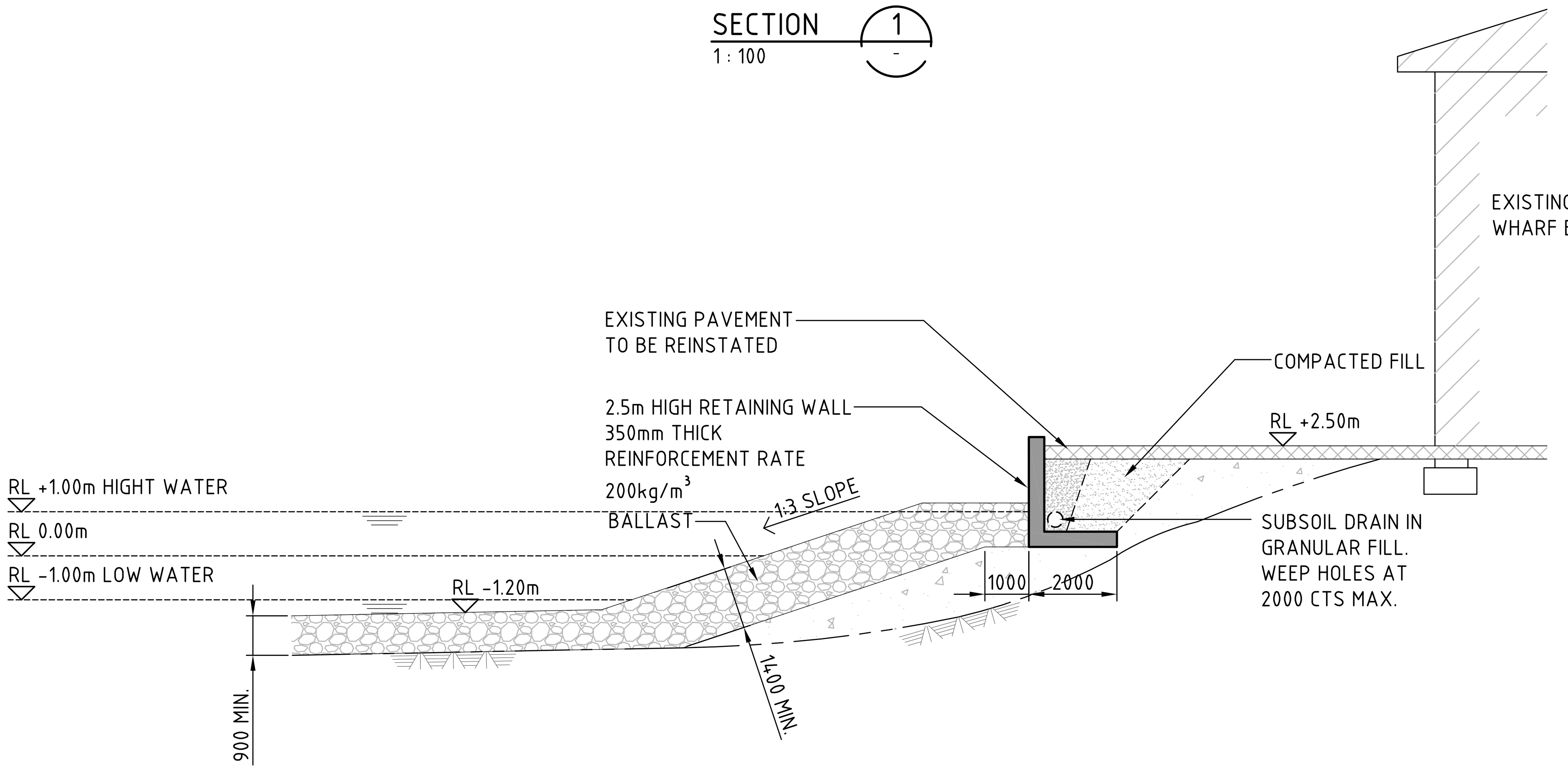


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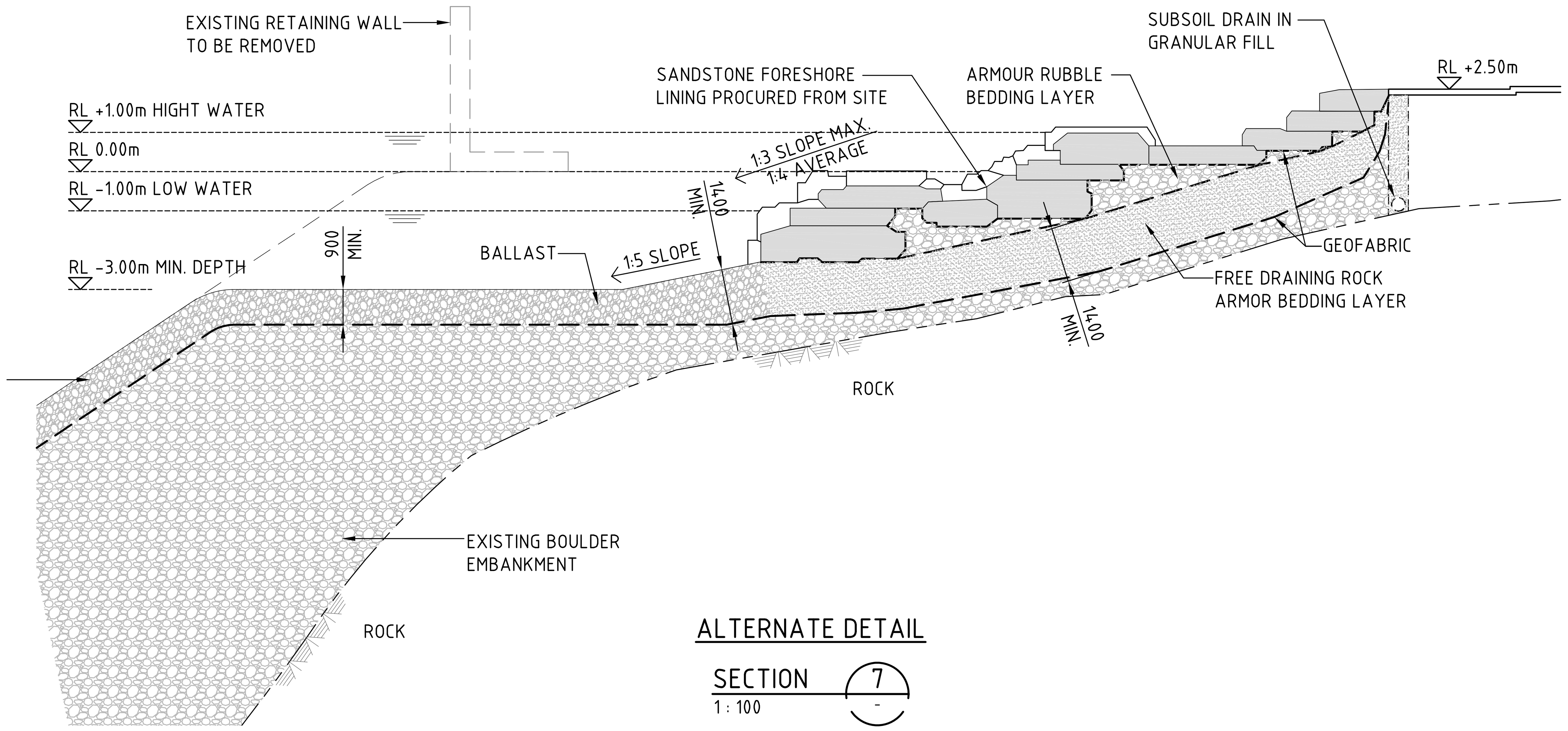


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Client: BARANGAROO DELIVERY AUTHORITY

Status: **PRELIMINARY**
NOT USED FOR CONSTRUCTION

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| Original Size | A1 | Designed TR |
| Height Datum | AHD | Checked |
| Grid | MGA | Approved PM |

Filename: SK05-AA003264-NSD-00.dwg

Project: BARANGAROO HEADLAND PARK AND PUBLIC DOMAIN MARINE WORKS

Title: GENERAL ARRANGEMENT TYPICAL SECTIONS

SHEET 4

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| Drawing No. | Project No. | Issue |
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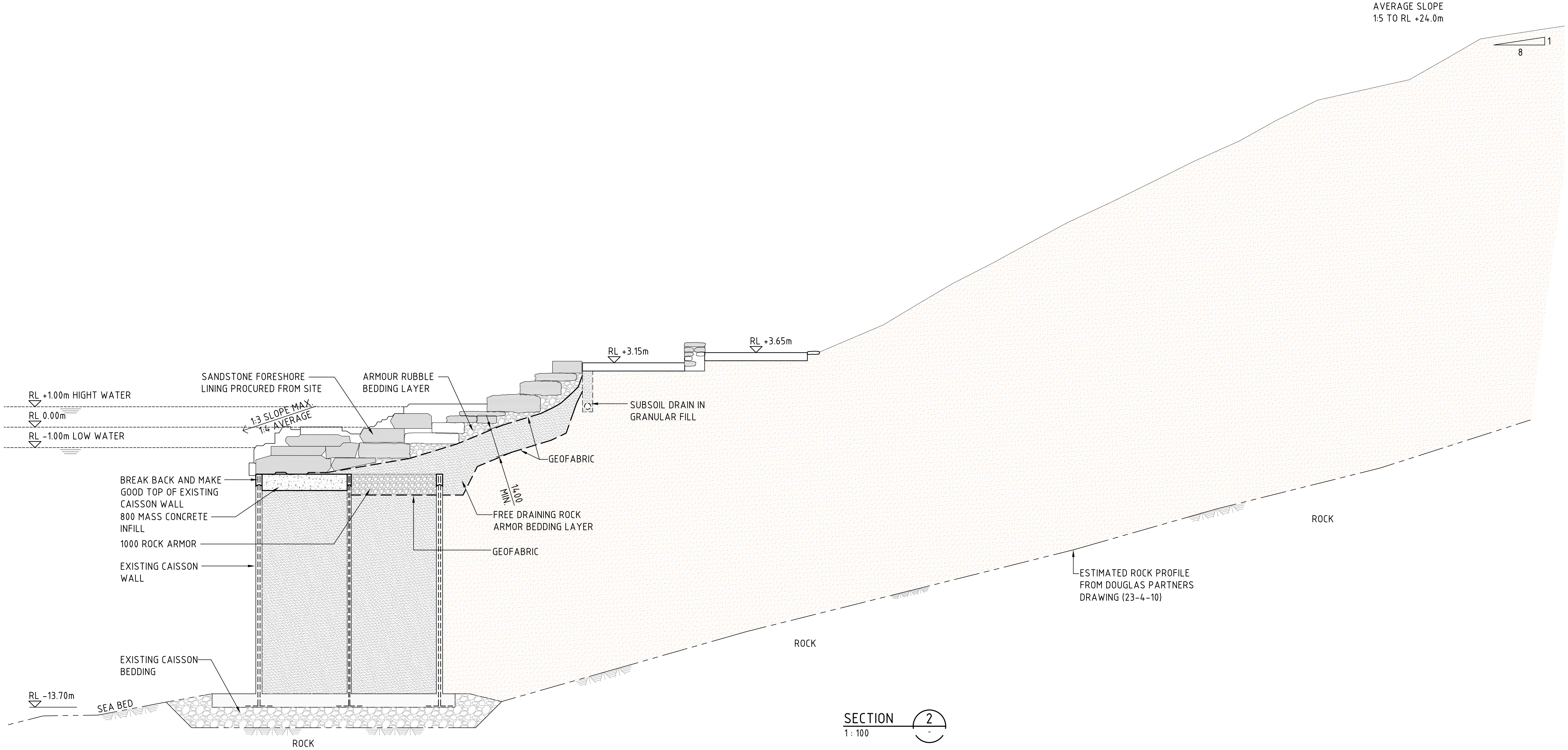


| Issue | Description | Date |
|---|---|---|
| A | CONCEPT ONLY | - |

Client

BARANGAROO DELIVERY AUTHORITY

| | | | |
|---|---|---|---|
| Status | **PRELIMINARY** NOT USED FOR CONSTRUCTION | | |
| Scales | AS SHOWN | Current Issue Signatures | |
| | | Drawn APT | |
| Original Size | A1 | Designed TR | |
| Height Datum | AHD | Checked | |
| Grid | MGA | Approved PM | |
| Filename: | SK06-AA003264-NSD-00.dwg | | |

Project

BARANGAROO HEADLAND PARK AND PUBLIC DOMAIN MARINE WORKS

Title

GENERAL ARRANGEMENT TYPICAL SECTIONS

SHEET 5

Hyder

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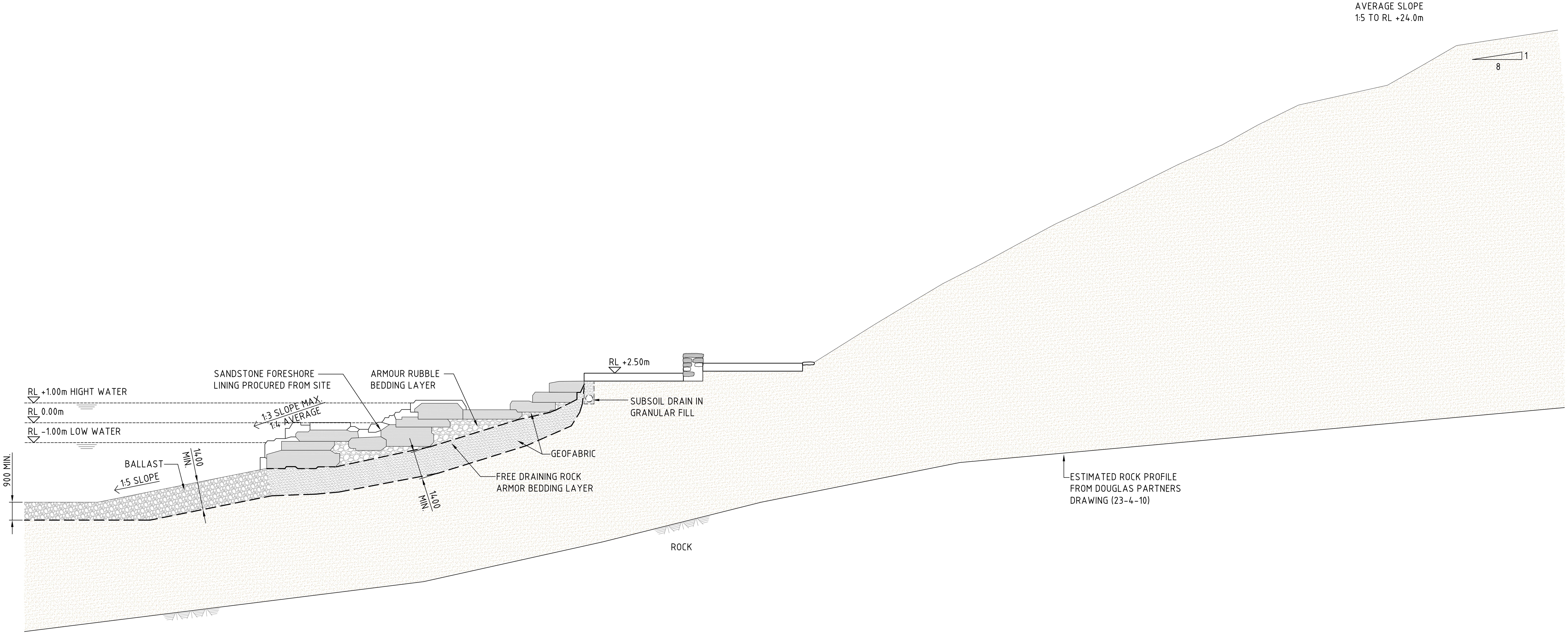
| Drawing No. | Project No. | Issue |
|---|---|---|
| SK06 | AA003264 | A |

100mm on Original

Date Plotted: 25 Jun 2010 - 05:07PM File Name: F:\AA003264\E-CAD\K-Sketches\SK06-AA003264-NSD-00.dwg V1




SECTION 1
1 : 100

| Issue | Description | Date |
|---|---|---|
| A | CONCEPT ONLY | - |

Client: BARANGAROO DELIVERY AUTHORITY

Status: **PRELIMINARY** NOT USED FOR CONSTRUCTION

| | | | |
|---|---|---|---|
| Scales | AS SHOWN | Current Issue Signatures | |
| | | Drawn APT | |
| Original Size | A1 | Designed TR | |
| Height Datum | AHD | Checked | |
| Grid | MGA | Approved PM | |
| Filename: | SK07-AA003264-NSD-00.dwg | | |

Project: BARANGAROO HEADLAND PARK AND PUBLIC DOMAIN MARINE WORKS

Title: GENERAL ARRANGEMENT TYPICAL SECTIONS

SHEET 6

Hyder

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| Drawing No. | Project No. | Issue |
|---|---|---|
| SK07 | AA003264 | A |

100mm on Original

Date Plotted: 25 Jun 2010 - 05:08PM File Name: F:\AA003264\E-CAD\K-Sketches\SK07-AA003264-NSD-00.dwg V1