Sunshine Coast Regional Council

Noosa Waters Revetment Walls

Condition Assessment & Concept Report

May 2013

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Appendices

1. Introduction

GHD Pty Ltd (GHD) has been commissioned by Sunshine Coast Regional Council (SCRC) to undertake a condition inspection of Noosa Waters revetment walls, with specific attention paid to three problem areas (Figure 1-1) at;

- 35/37 Masthead Quay (Site 1),
- 38/40 Saltwater Ave (Site 2), and
- At the park between Mermaid Quay and Seahorse Place (Site 3)

The report then outlines the site data gathered from a desktop review of available information. Site data includes;

- Water Levels,
- Winds,
- Waves,
- Currents,
- Geotechnical,
- Wall Design,
- Scour Protection, and
- As-Constructed / Hydrographic Survey.

The Condition Assessment and Concept Design Report then provides concept remediation options.

1.1 Location

Noosa Waters is located at Noosaville approximately 5 km west of Noosa Heads. The Noosa Waters waterway is connected to the Noosa River with boat access through a lock system shown in Figure 1-1. The lock and weir maintain a water level of approximately Mean High Water Springs (MHWS) and the weir is overtopped in tides larger than this. To maintain water quality water is also pumped in from the Noosa River to maintain mixing. Site conditions are discussed further in Section 3.

Figure 1-1 Noosa Waters Waterway (Google 2012)

2. Condition Inspection

On the 12th November 2012 a meeting was held on site between GHD (Adam Brook) and SCRC (Rod Williams) to inspect the site and discuss the project. Following the meeting GHD Senior Coastal Engineer Adam Brook undertook a detailed inspection of the condition of the revetment wall and toe protection at the three key sites and a number of other sites adjacent to parks or vacant blocks.

2.1 35/37 Masthead Quay (Site 1)

On the day of the inspection neither resident was home at the 35/37 properties so an inspection was undertaken of the adjacent properties from the land and 35/37 were inspected from the water (so no photos could be taken).

The revetment wall around the eastern end of Masthead Quay has a minimum of 1 m of rock protection and on the northern side (odd number properties 21 – 35) has up to 3 m of rock protection. The toe of the wall in a number of locations is up to 400 mm below the original design levels. Minor potholing has occurred in a few locations at either drainage holes or joins indicating some loss of material due to drainage Figure 2-1. At the toe varying amounts of scour are present but in all cases it was less than 50 mm so no exposure of the cutoff wall was present. The crest of the revetment wall mostly seemed to be close to that of the design level but there was some tilting forward of the wall at 35/37 Figure 2-2. The inspection identified the presence of geotextile underlying the rock protection and the rock protection was greater than 75 mm in diameter as specified in the original design. The batter material beyond the rock protection was mostly sand with some minor fines (Figure 2-3).

Figure 2-1 Potholing at drainage hole.

Figure 2-2 Revetment wall tilting forward at 37 Masthead Quay (SCRC 2012).

Figure 2-3 Sand with minor fines & Rock protection greater than 75mm .

2.2 38/40 Saltwater Av (Site 2)

Currently 38 Saltwater Av is a vacant block so provided good access for the inspection of the revetment wall. The inspection identified that very little of the rock protection remained in front of the wall in the middle of the block with only a few scattered rocks remaining (Figure 2-4). Digging down into the sand at the toe of the wall no rock could be found to have settled through the sand and there was no presence of geotextile. Going further across the bank into deeper water none of the original rock protection could be found either. The batter material was mostly sand with some minor fines. The properties at 36 and 40 Saltwater Av both have added additional rock protection placed at the toe of their walls (Figure 2-5 & Figure 2-6).

The entire wall in front of 38 has settled around 100 mm and the drainage holes are now just below the water surface. The toe level in front of the wall is approximately 300 mm below the design level but 150 mm or more of this could be accounted for due to the loss of the two layers of rock protection. Without rock protection at the toe of the wall, the toe of the wall is highly susceptible to scour and a couple of large scour holes have formed under the toe of the wall. With the drainage holes now below the water surface and no toe protection to protect against scour, the wall has begun to tilt forward slightly and potholes are forming at joins and drainage holes where material could be lost.

Figure 2-4 No rock scour protection at toe of wall 38 Saltwater Av

Figure 2-5 Additional Rock Scour Protection 40 Saltwater Av.

Figure 2-6 Additional Rock Scour Protection 36 Saltwater Av.

Figure 2-7 Wall sunken at 38 Saltwater Av, drainage holes are below water surface.

Figure 2-8 Sunken wall at 40 Saltwater Ave (Hall 2012).

2.3 Park between Mermaid Quay and Seahorse Place (Site 3)

The parks between Mermaid Quay and Seahorse Place were inspected. The toe of the wall is up to 300 mm below the original design levels in a number of locations . For the most part rock scour protection exists in front of the wall, however on one section the rock scour protection seems to have moved slightly down the slope leaving some of the toe exposed, and in this location no geotextile could be identified (Figure 2-9). There appeared to be minimal amounts of scour at the toe and the crest of the revetment wall mostly seemed to be close to that of the design level. Minor potholing has occurred in a few locations at either drainage holes or joins indicating some loss of material due to drainage (Figure 2-10). The rock protection was greater than 75 mm in diameter as specified in the original design. The batter material beyond the rock protection was mostly sand with slightly more fines than at the other 2 sites some (Figure 2-11).

Figure 2-9 Section of scour protection moved slightly down toe.

Figure 2-10 Potholing at drainage hole.

Figure 2-11 Sand with fines & Rock protection greater than 75mm .

2.4 Other Sites

2.4.1 Topsail Place

Topsail Place was inspected at a couple of vacant blocks that existed. As with the three main sites, some sections of the toe are up to 300 mm below the original design levels. The rock scour protection was in place with a 1m strip from the toe of the wall (Figure 2-12). Only very minor potholing was occurring at the joints and it was not determined whether geotextile existed.

Figure 2-12 Topsail Place

2.4.2 Seahorse Place

Seahorse Place was inspected at a vacant block at 13 Seahorse Place. In some locations the toe is approximately 100 mm below the original design level. Some of the scour protection had been displaced in front of the wall (Figure 2-13). Only very minor potholing was occurring at the joints and it was not determined whether geotextile existed.

Figure 2-13 Seahorse Place.

2.4.3 The Promontory

The Promontory was inspected at a couple of vacant blocks that existed at the end of the street. As with the three main sites, the wall in a number of locations is up to 300 mm below the original design levels. Some of the scour protection had been displaced in front of the wall. Also some potholing was occurring at the joints. It was not determined whether geotextile existed or how bad scour was at the toe of the wall.

2.4.4 Shorehaven Drive

Shorehaven Drive was inspected at a couple of vacant blocks that existed (147 & 163). There was no measureable difference in the toe level from the original design levels. The rock scour protection was in place varying from a 1 m to 3 m strip from the toe of the wall, and it was not determined whether geotextile existed. This section of wall seemed to generally be in good condition and does not require any immediate maintenance work.

2.4.5 Waterside Court

The wall at Waterside Court was inspected from a park. At this location a flexible concrete mat boatramp had been constructed extending into the water. There was 3+m of rock scour protection at the toe of the ramp and no measureable settlement of the wall (Figure 2-14). There appeared to be no geotextile extending out from under the boat ramp and it was not determined whether there was geotextile under the scour protection. This section of wall generally is in good condition and does not require any maintenance work.

Figure 2-14 Waterside Court flexible concrete mat boat ramp

3. Site Conditions

To enable design of the concept remediation options, site data and environmental design criteria are required, a summary of which is provided in the following sections.

3.1 Water Levels

The water level with the Noosa Waters waterway is controlled by a lock and weir to maintain a minimum level of 0.4 m (AHD). Higher tides and storm surge events have the ability to breach the weir and raise the water level within the system.

3.1.1 Tidal Planes

The astronomical tide levels for the study area will be between the tidal levels at Munna Point and Tewantin which have been determined using published tide tables which provide derived tidal planes and predictions of high and low tide water levels. The 2012 Tide Tables published by Maritime Safety Queensland (MSQ, 2012) provide data for standard tidal planes. The levels at Munna Point and Tewantin for water levels Highest Astronomical Tide (HAT) and Mean High Water Springs (MHWS) are presented in Table 1 in AHD datum.

Table 1 Tidal Planes for Munna Point and Tewantin

Tidal Plane	Munna Point	Tewantin
	m (AHD)	m (AHD)
Highest Astronomical Tide, HAT	0.68	0.55
Development Still Water Level	**0.40**	**0.40**
Mean High Water Springs, MHWS	0.36	0.27

3.1.2 Storm Surge

The Ocean Hazards Assessment Stage 2 Report (OHAR2) presents the work of a number of peak bodies in relation to the risk of inundation due to storm surge. The recommendations of the Ocean Hazards Stage 2 and 3 Reports (OHAR2 and OHAR3) provide storm surge levels (not including the potential for sea level rise) for a range of extreme weather events with varying annual recurrence intervals (ARI's). The water level frequency plot for Noosa Heads is shown in Figure 3-1. Given the locked nature of the Noosa Waters waterway and the distance up the Noosa River the effects of wave setup have not been included. The resulting storm surge levels for Noosa Heads are presented in Table 2.

Table 2 Storm Surge Levels (excluding Sea Level Rise) – Noosa Heads

	Storm Surge Water Level (excluding Sea level Rise)		
	ARI 100 years	ARI 500 years	ARI 1000 years
	1.20 m AHD	1.30 m AHD	1.40 m AHD
HAT = 1.16 m AHD	0.04 m above HAT	0.14 m above HAT	0.24 m above HAT

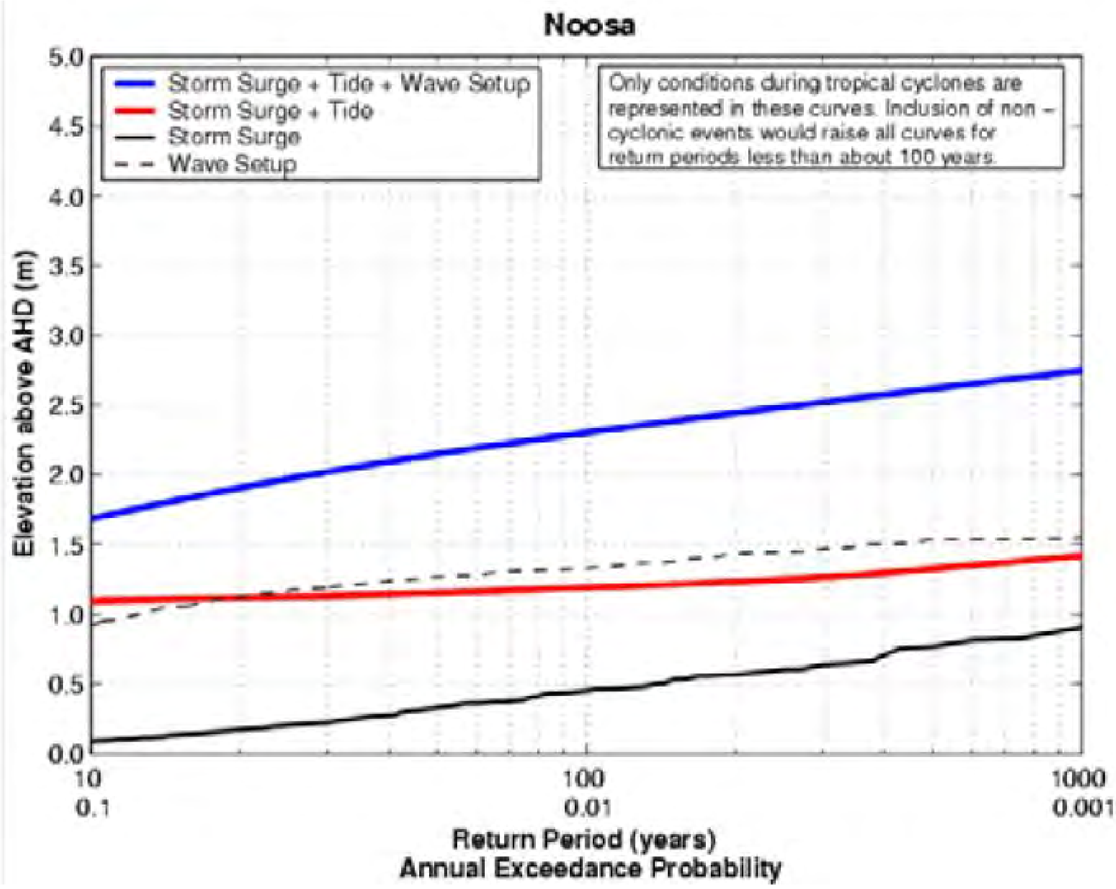


Figure 3-1 Noosa Storm Surge (Hardy et al 2004)

3.1.3 Sea Level Rise

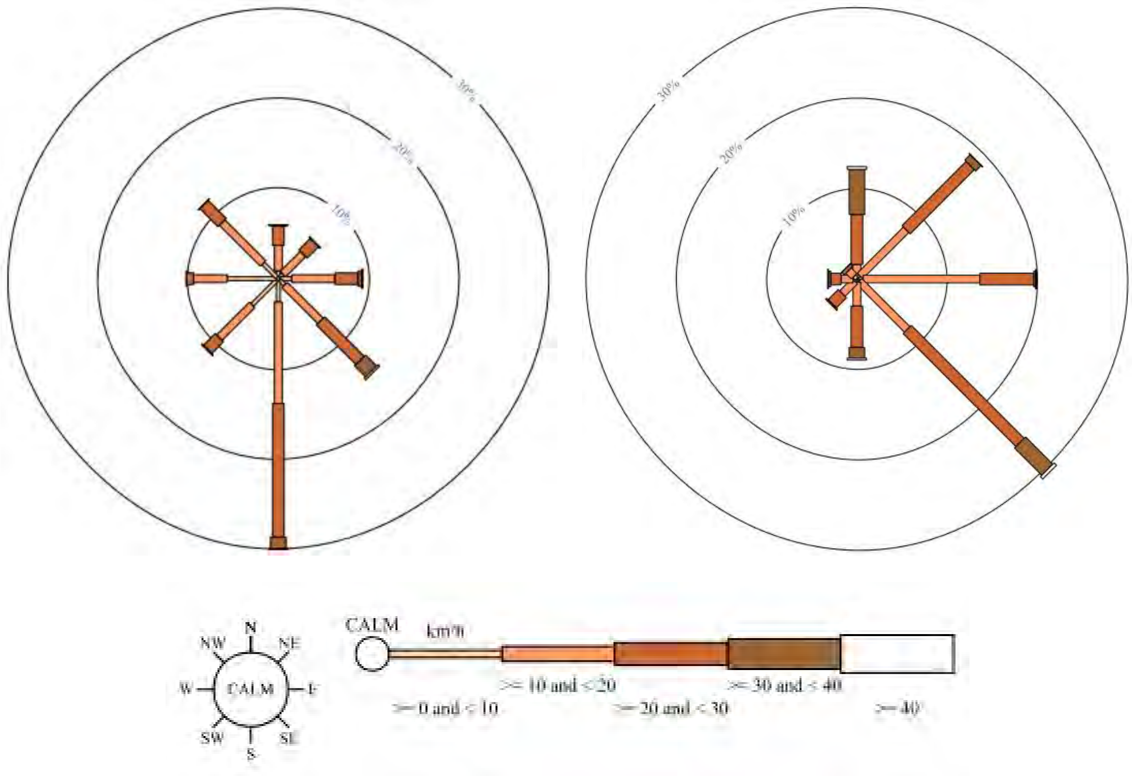
Sea Level Rise needs to be accounted for in the design life of the seawall and the *Queensland Coastal Plan* (DERM 2012) provides the projected sea-level rise for the year of the end of asset life (Table 3)

Table 3 Projected sea-level rise for the year of the end of asset life (DERM 2012)

Year of end of planning period	Projected sea-level rise
Year 2050	0.3 metres
Year 2060	0.4 metres
Year 2070	0.5 metres

3.1 Winds

The wind climate for the study area is based on wind records from the Bureau of Meteorology for the Sunshine Coast Aero approximately 30 km south of the study site. Figure 3-2 shows the annual 9 am and 3 pm wind climate in the form of wind roses for 06 Jul 1994 to 30 Sep 2010. As can be seen from the wind roses there is a dominance of winds from the south easterly sector.



9 am 5845 Total Observations Calm 1%	3 pm 5853 Total Observations Calm 1%

Figure 3-2 Wind Rose – Sunshine Coast Aero (BoM)

3.2 Waves

3.2.1 Wind Waves

With a maximum fetch within the waterway of approximately 350 m extreme event waves will be fetch limited. Using equations 3-39 and 3-40 from the Shore Protection Manual (CERC 1984) to calculate depth and fetch limited waves, wave heights were calculated for the site (Table 4). Under yearly type storm events where wind speeds of 40 km/hr are experienced wind waves could get up to Hs 0.15 m and typical wind generated waves within the waterway are generally less than Hs 0.1 m. It should be noted that the three key sites do not have long fetches to the southeast so are unlikely to regularly experience any waves over Hs 0.1m

Table 4 Significant Wave Heights

Average Return Period (Years)	Significant Wave Height Hs (m)
5 yr	0.2
10 yr	0.3
50 yr	0.4
100 yr	0.5

3.2.1 Boat Wake Waves

Although a speed limit of 6 knots no wash is imposed for the Noosa Waters waterway boat wake waves still have the potential to impact the revetment walls. On the 12th November 2012 while undertaking the inspection two separate vessels both passed travelling at speeds capable of producing a breaking wake, which produced a wave in excess of 0.5 m. Given the short fetches and hence small wind generated waves it is most likely that the most destructive waves will be produced by vessels not strictly adhering to the speed limit of 6 knots no wash as imposed.

3.3 Currents

Given the closed non-tidal waterway, currents experienced within the system are expected to be minimal. Although water velocities as low as 0.3 m/sec are capable of mobilising and eroding medium grain sands, scour protection has been provided around the stormwater outlets and at the edge of the waterway where the water is shallowest. Measurement of current velocities undertaken by Hall (2012) recorded a maximum current velocity of 0.12 m/s when a vessel travelling in excess of 6 knots passed the Acoustic Doppler Velocimenter (ADV) measuring device.

3.4 Geotechnical

The site predominately consists of fine to medium grained silty sand with densities ranging from loose to medium dense, overlying a marine clay layer. Typically this clay level was encountered at depths greater than -2.0 m. However the distribution of marine clay on the Sunshine Coast is known to be somewhat irregular and marine clays were encountered at higher levels in some areas. This is accounted for in the site classification for the development with the majority of the Noosa Waters development classified as Moderate to Highly reactive which can experience a high level of ground movement. From observations undertaken of the site during the site inspection, the development generally does not seem to have experienced any high levels of ground movement except at the revetment, as all roads appeared to be in good condition and building and fences did not appear to suffered significant differential movement.

3.5 Wall Design

The revetment wall is a high strength mass concrete structure with a concrete cut off wall and drainage holes. The wall is founded on imported compacted sand and scour protection is provided at the toe of the wall (see Figure 3-3). The wall was structurally designed for a surcharge of 2kPa and a bearing capacity of maximum 100kPa assuming no scour of the toe.

Originally when reviewing the design GHD (1991) advised that a design of surcharge 2kPa (adjacent wall) is less than GHD would normally recommend. Given this low surcharge design this may explain the failure of the wall at Leeside Drive due to machinery working above the wall. This may also leave the vacant blocks at risk if large rideon mowers are being used adjacent the walls.

GHD (1991) also advised that underlying marine clay will result in some residual consolidation and may cause differential settlement of adjoining wall sections. It is unsure if any further design measures were undertaken to address these comments.

Given that in some sections of the revetment wall the toe is below design level or no scour protection is visible, the toe of the wall is now susceptible to scour of the toe. Since the design was based on assuming no scour of the toe without remediation measures the long term stability of the wall cannot be guaranteed.

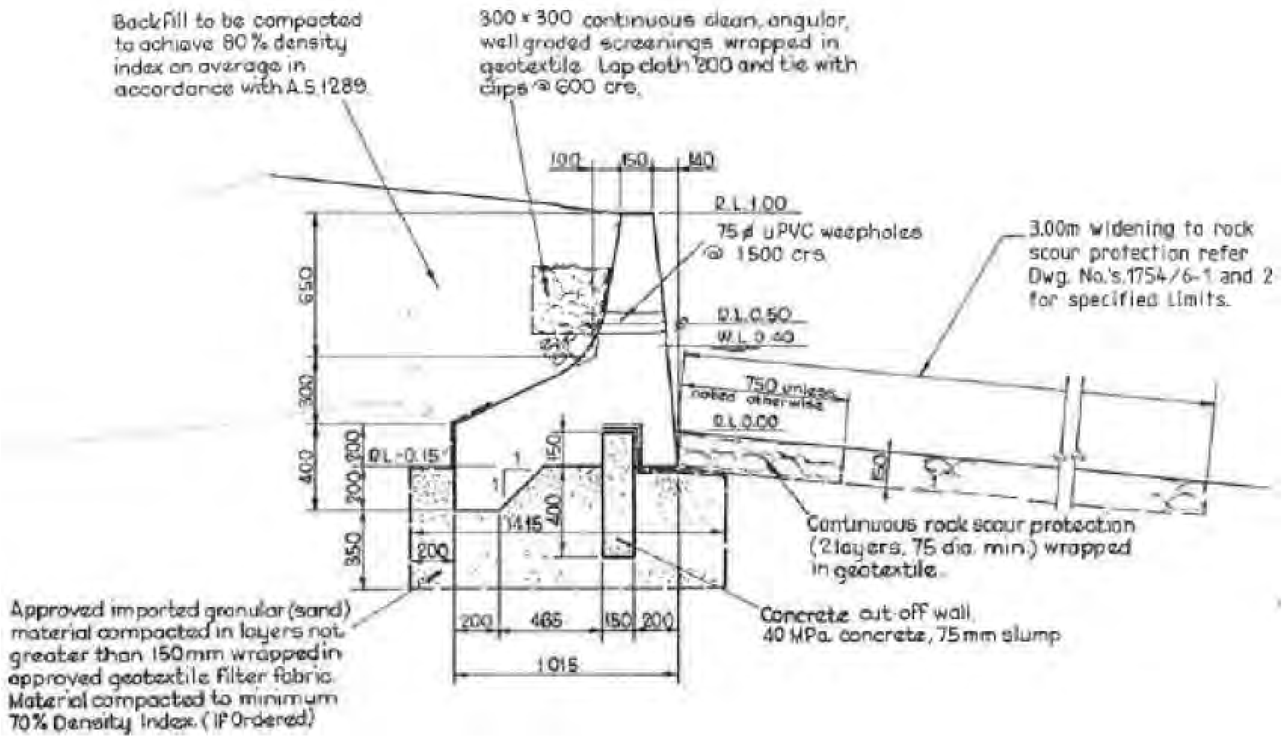


Figure 3-3 Typical revetment wall design (Cardno & Davies 1990)

3.6 Scour Protection

Calculation checks were undertaken on the scour protection for wave height and current velocity using Van der Meer (1987) armour stability against waves and Van Rijn (1984) bed load transport for stability against currents.

The current scour protection of two layers of minimum diameter 75 mm is suitable for wind induced waves of 0.5 m or less and any currents that would be experienced within the waterway. Waves of greater than 0.5 m do have potential to move armour units so boat wash needs to be kept to a minimum. Once exposed the sand under layer would be highly susceptible to wave induced scour and scour from currents greater than 1 m/s which could be produced by boat wash should vessels be travelling in close proximity to the revetment wall.

3.7 As Constructed / Hydrographic Survey

A comparison of the JFP Consulting Surveyors (1993) As-Constructed survey and the Port of Brisbane (2012) Hydrographic Survey showed that where available the as-constructed bed levels in the centre of the waterway fairly closely match the recent hydrographic survey. The as-constructed survey showed that in many areas the bed was excavated to a much greater depth than the design profile. The maximum departure from the design profile occurs in the waterway between Shorehaven Dr and Topsail Pl where the depth of the waterway is 8.8m below the design profile.

The Noosa Waters Residents Association (NWRA) undertook some independent measurements to produce two transects at 7 Masthead Quay and 37 Masthead Quay. However, at these transects no as-constructed surveys were available. Nevertheless, it is most likely that the bed was constructed at close to -3.7 m AHD and -4.2 m AHD as the Port of Brisbane (2012) Hydrographic Survey does not indicate any significant settlement occurring in the profiles.

Where the as constructed levels are available at the edge of the waterway they fairly closely match the recent hydrographic survey for the level of detail provided.

4. Remediation Concepts

On the 12th December 2012 a meeting was held between SCRC, NWRA, Soil Surveys and GHD to determine remediation concepts. From the meeting two remediation options where proposed. Option 1 for where the bed level is below the toe of the wall (4.1) and Option 2 for where the bed level is below the existing design level but above the toe of the wall (4.2).

The NWRA has conducted measurements throughout the estate at most properties classifying the wall into four categories. Measurements were taken from the top of the wall to the bed at the toe of the wall. Where the measurement from the top of the wall was greater than 1150mm the wall was classified either brown or red, indicating the toe was greater than 150mm below design level and below the toe of the wall was exposed. Option 1 has been proposed for sections of the wall classed brown or red.

Measurements of 1000 to 1150mm where the bed is below the design level but the toe of the wall is still covered were classified blue. Option 2 has been proposed for sections of the wall classed blue. Any measurements where the bed level was less than 1000mm and hence within 50mm of design was classified green.

NWRA & SCRC has proposed for Soil Surveys to conduct further geotechnical investigations of the site and subject to the findings of these investigations the remediation options suggested be implemented throughout the Noosa Waters development. Soil Surveys is also developing some site specific options for the rectification of 35/37 Masthead Quay, 38/40 Saltwater Av and another site at The Anchorage

From the inspection conducted by GHD it is evident that a number of different issues are occurring within the Noosa Water Estate. Any site where scour protection is 150 mm or more below the design level leaves the toe of the wall susceptible to scour and potential undermining. At a couple of sites there was movement of the scour protection rock or it was not present. Calculations have shown that the minimum diameter 75 mm should be suitable for wind waves and currents. However should a vessel pass travelling in excess of the 6 knots no wash that is imposed for the Noosa Waters waterway there could be potential for waves greater than 0.5 m to move the scour rock protection. The scour protection is critical to prevent undermining of the wall and hence scour protection rock of 75 to 150mm has been proposed where required.

Minor potholing is occurring at a number of sites adjacent to the drainage holes and revetment wall joints. This should be monitored for any sites where this potholing has reached a level where the ground surface is noticeably lower as it is likely sediment is being lost. It is not considered critical at this stage but should this potholing increase it is recommended at these drainage holes/joints a geotextile should be installed to prevent any further loss of material and the potholes topped up with a free draining sand/gravel.

At 38 Saltwater Av the entire wall has settled approximately 100 mm such that the drainage holes are just below the water's surface and the wall is beginning to tilt forward slightly. The drainage holes are important to the stability of the structure as they allow water to drain from behind the wall. For this section of wall it would be recommended to install new drainage holes at the joints in the wall. Soil Surveys is currently undertaking further investigation as to the stability of this section of wall and will propose a site specific remediation option.

4.1 Option 1 – Bed level below toe of wall (Red/Brown)

The recommended remediation option is to

(1) Install a geotextile on the existing bed near the revetment wall;

(2) .Place a layer of gravel (nominal 20mm to a level of RL 0.05m adjacent the wall; and

(3) Install scour protection in front of the wall to just below the water surface (see Appendix A).

4.2 Option 2 – Bed level below above toe of wall but below design level (Blue)

The recommended remediation option for this site is to install additional rock scour protection to a level above the original design level just below the water surface (see Appendix A).

4.3 Remediation (Post Soil Surveys geotechnical investigation)

Subsequent to the GHD Condition Assessment and Concept Report prepared December 2012 (contained herein) Soil Surveys (2013) undertook a geotechnical investigation of the Noosa Waters estate. The GHD proposed concepts (Appendix A) have been reviewed against Soil Survey investigation and remediation options are generally in accordance with those recommended by Soil Surveys. Both Soil Surveys and GHD recommend revetment rock at the toe of revetment walls where the bed level is below that of the original design. The revetment rock will provide an overburden pressure and increase stability.

Soil Surveys remediation recommendations are illustrated in Figure 3 which *indicates a diagrammatic recommended Typical Section of rock scour protection. The use of 20 mm aggregate, without geofabric underlay, is suggested to fill voids and re-establish ground support at the toe of the revetment wall where slumping has exposed or undermined the revetment wall base. Where the lake has not slumped below RL0.0m immediately adjacent to the wall a -150+75mm rock protection layer, with geofabric undelay, is recommended*"(Soil Surveys 2013)

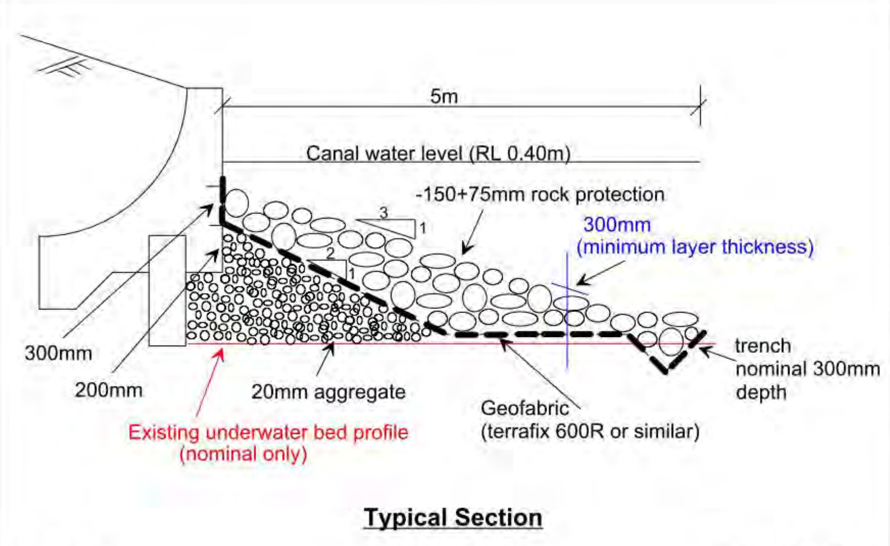


Figure 3

(Soil Surveys 2013)

Points of difference between the GHD (2012) remediation and Soil Surveys (2013) include;

4.3.1 Option 1 - Bed Level below top of wall (Soil Surveys - Bed below RL0.0m)

The geotextile layer is placed in a different location in relation to the 20mm gravel: Soil Surveys above 20 mm gravel, GHD below 20 mm gravel with additional gravel placed prior to geofabric to fill voids. The GHD & Soil Surveys placement of the geofabric provides a similar final outcome, the GHD option would have the benefit of potentially using less gravel as the geofabric stops the gravel sinking into the top layer and the Soil Surveys placement potentially offers some minor savings in construction time by being able to place the 20 mm gravel all in one go.

Either placement of the geotextile would be suitable to GHD for the remediation, with the decision probably best left to the contractor undertaking the works for their preferred construction methodology.

4.3.2 Option 2 - Bed level below above toe of wall but below design level (Soil Surveys - Bed above RL0.0m)

In cases were the bed has slumped less Soil Surveys have not provided a new concept sketch and have recommended the -150+75 mm layer as with the other case without the 20 mm gravel layer. GHD have provided a 3m width of rock as opposed to 5m by Soil Surveys.

In relation to the 3m width, GHD recommend that this only be used on areas where the bed is above the toe of the wall and for any borderline areas it is transitioned out to 5m as recommended by Soil Surveys. This assessment should also be made taking into account the adjacent surrounding areas. If adjacent areas require Option 1 a 5m width should be used, if adjacent areas are close to the original design profile rock protection this can be relaxed to 3m.

It also should be noted that reducing water levels within the estate would reduce the overall stability of the wall so when conducting rectification works the water level should not be reduced to aid construction.

5. Conclusion

The Noosa Waters revetment wall was designed to have no scour at the toe. Where the toe is below the original design level and exposed to scour the wall is no longer in accordance with the original design and is susceptible to an increased risk of failure in the future. It is recommended that, at any sites where the scour protection is more than 150 mm below the design level or is not present and the toe of the revetment wall is exposed, additional scour protection be installed to prevent potential undermining of the toe.

Although the scour protection rock has been sized sufficiently to protect from movement due to wave and current action, on the day of the inspection two vessels were observed to be travelling at speeds in excess of the 6 knots no wash imposed. Given the access restrictions through the lock at the entrance to Noosa Waters, it is most likely that vessels within the waterway belong to residents and it is recommended that residents be educated that travelling within the waterway at speeds producing a wake is detrimental to the revetment walls. Alternatively all the scour protection will need to be replaced with larger sized rock

The revetment wall design is only for a fairly minimal surcharge (2kPa) so are susceptible to potential failure should heavy loads be exerted near the crest of the wall (15kPa for rest of allotment). Residents should also be advised of this and recommended that vehicles and machinery should be kept off the backfill slope. This would include ride on mowers working closely to the wall on vacant blocks.

5.1 Post Geotechnical Investigation concluding remarks

Subsequent to the Soil Surveys (2013) investigation the GHD (2012) concepts were reviewed against the Soil Surveys investigation and remediation recommendations. Both Soil Surveys and GHD recommend revetment rock at the toe of revetment walls where the level is below that of the original design. The revetment rock will provide an overburden pressure and increase stability. Adding rock above the original design profile should increase the global stability from the original design when present in a profile above that of the original design. Installation of the rock some 200 mm above the original design level allows for settlement overtime while still providing an overburden pressure. Monitoring is however recommended to ensure the rock protection is at a level above that of the original designto ensure long-term stability.

6. References

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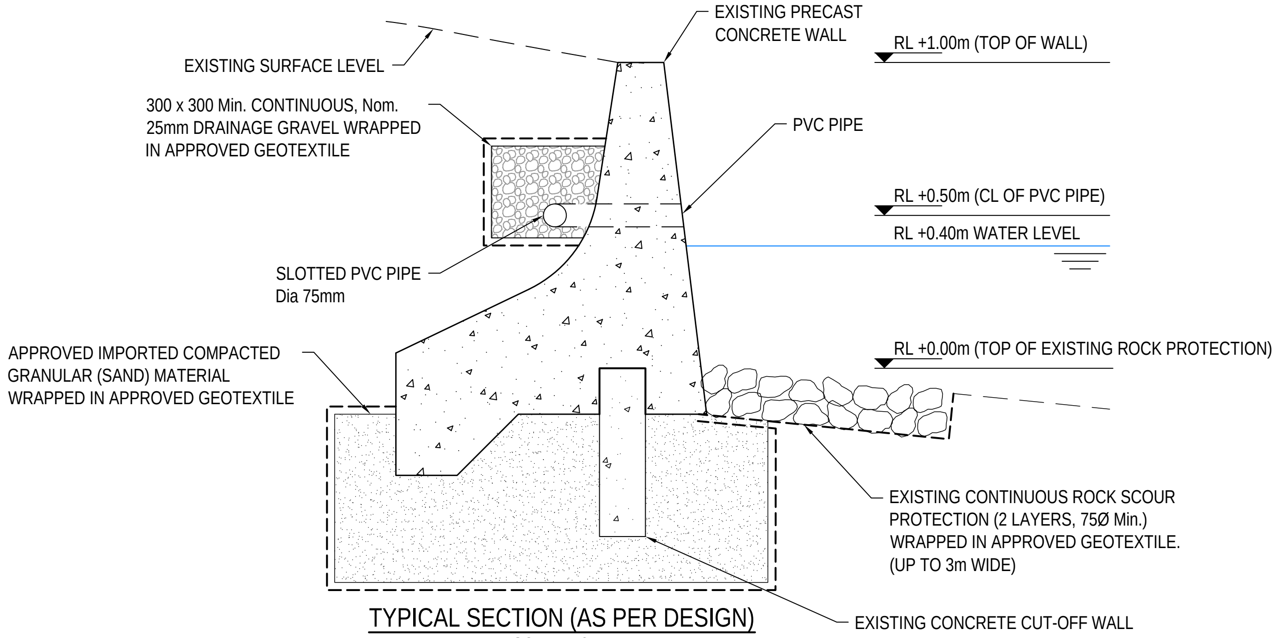
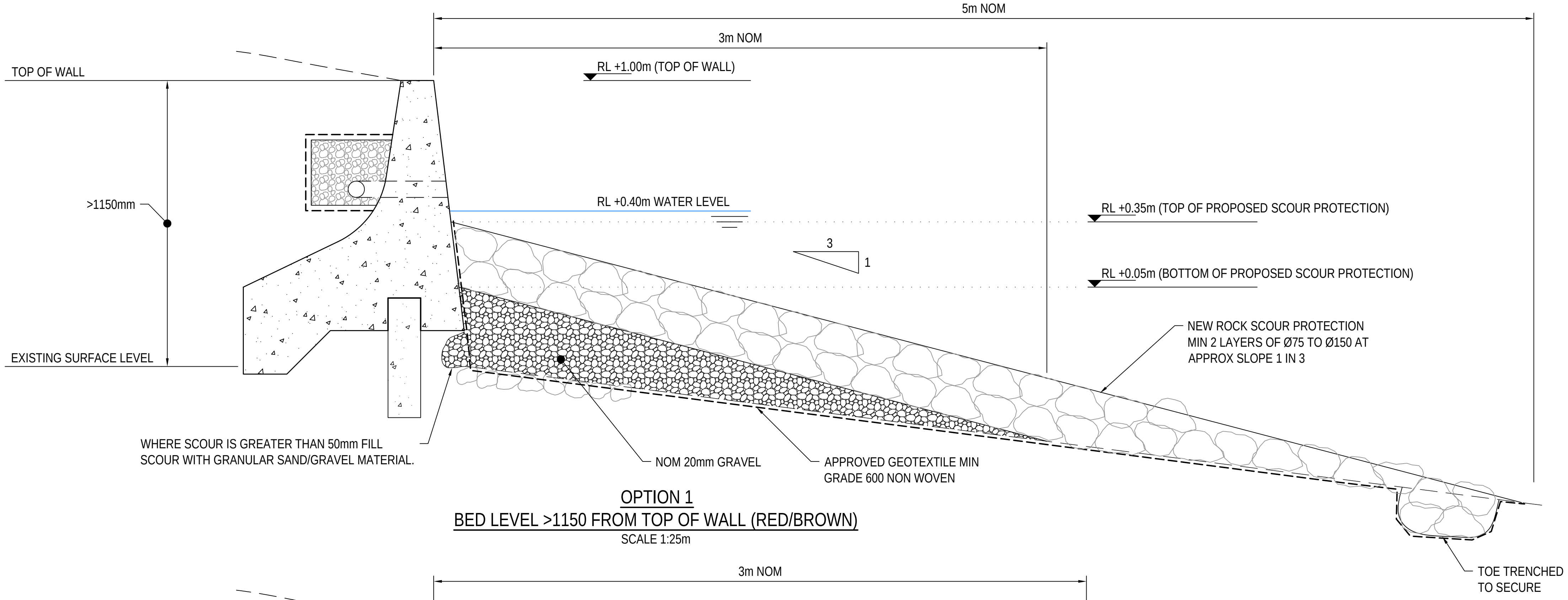
Appendix A - (Remediation Sketches)

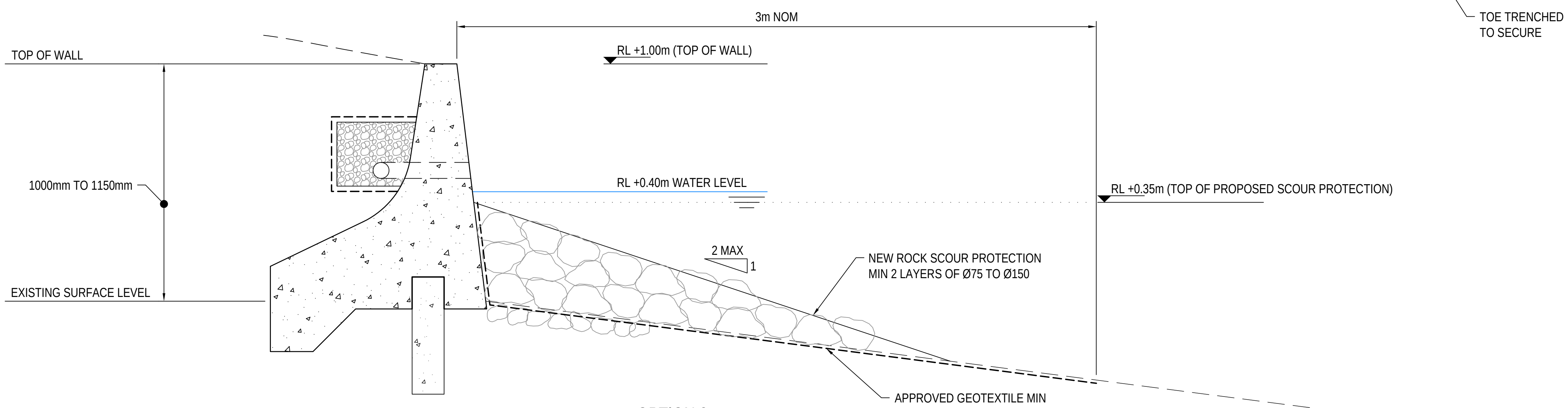
Appendices



TYPICAL SECTION (AS PER DESIGN)
SCALE 1:25m



OPTION 1
BED LEVEL >1150 FROM TOP OF WALL (RED/BROWN)
SCALE 1:25m



OPTION 2
BED LEVEL >1000 TO <1150 FROM TOP OF WALL (BLUE)
SCALE 1:25m

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE (U.N.O).
2. ALL LEVELS ARE SHOWN TO AUSTRALIAN HEIGHT DATUM (AHD).
3. WATER LEVEL IS 0.40m AHD APPROX.
4. DESIGNS ARE CONCEPTS ONLY AND SUBJECT TO FURTHER GEOTECHNICAL INVESTIGATION.
5. GEOTEXTILE SHALL BE ADEQUATELY WEIGHTED TO ENSURE IT CAN BE EFFECTIVELY PLACED UNDER WATER.
6. GEOTEXTILE SHALL BE LAID ON THE FORMATION WITHOUT WRINKLES, GAPS, FOLDS, SLACK, STRESSING OR DEFORMATION.

PRELIMINARY

rev	description	app'd	date
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SUNSHINE COAST REGIONAL COUNCIL
NOOSA WATERS REVETMENT WALLS

REMEDIATION CONCEPTS
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1	A.Brook	P. O'Keeffe	P. O'Keeffe*	P. O'Keeffe	P. O'Keeffe*	17/12/2012
2	A.Brook	P. O'Keeffe	P. O'Keeffe*	P. O'Keeffe	P. O'Keeffe*	19/12/2012
2	A.Brook	P. O'Keeffe		P. O'Keeffe		30/04/2013

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