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DA CP16-001

Table R1 DA CP16-001 Timeline

DATE	EVENT
11 February 2016	DA submitted to NSW Coastal Panel
15 February 2016	DA received by the NSW Coastal Panel (DA16-001)
10 March 2016	Request for additional information
22 March 2016	Response to request for additional information
15 April 2016	Site visit with NSW Coastal Panel members: Mr Angus Gordon – Chair Professor Andrew Short - nominee of Local Government NSW Ms Jane Lofthouse - nominee of Local Government NSW Emeritus Professor Bruce Thom - nominee of Local Government NSW Dr Carolyn Davies - Office of Environment and Heritage and Deputy Chair Mr Stephen Wills - Department of Primary Industries (Lands) Ms Jane Gibbs - Coastal Panel Secretariat Support / Senior Manager Environmental Program Services Phil Watson - Coastal Panel Secretariat Support / Principal Coastal Specialist Dr Marc Daley - Coastal Panel Secretariat Support / Senior Coast & Estuaries Officer
4 April to 3 May 2016	Public exhibition period (Integrated Development)
19 April 2016	NSW Coastal Panel request for additional information
26 April 2016	NSW Office of Water (DPI) General Terms of Approval granted.
1 July 2016	Additional information submitted electronically to NSW Coastal Panel
12 August 2016	SEPP 26 Concurrence, additional aboriginal cultural heritage and ecological addendum report provided to the NSW Coastal Panel
24 August 2016	Seawall 'DWG" file provided to NSW Coastal Panel
22 September 2016	Umwelt Flood Study 2003 provided to NSW Coastal Panel
22 September 2016	Receive notification of NSW Coastal Panel determination meeting set down for 12 October 2016.
10 October 2016	Royal HaskoningDHV and City Plan Services assessment reports uploaded to the NSW Coastal Panel website.
10 October 2016	Letter to NSW Coastal Panel - Request determination be deferred.
12 October 2016	Letter to NSW Coastal Panel - Request determination be deferred and provide planning justification for proposed revetment – Landscape Plan and Ecological Assessment attached.

DATE	EVENT
20 October 2016	Royal HaskoningDHV consultant conducts site visit to verify desktop assessment.
21 October 2016	NSW Coastal Panel advice that determination of the DA is set down for 2 November 2016 and that any additional information must be provided by close of business on 28 October 2016.
28 October 2016	Submit Supplementary SoEE to NSW Coastal Panel.
2 November 2016	NSW Coastal Panel defer determination of the DA.
1 December 2016	NSW Coastal Panel's planning consultant, City Plan Services and engineering consultant, Royal Haskoning DHV provide amended assessment reports.
5 December 2016	Submit additional information in response to the panel consultant's assessment reports and recommendations. **Appendix R**.
6 December 2016	NSW Coastal Panel determine DA CP16-001.

Table P2 summarises matters raised and responses to the panel's requests for further information.

Please note that in the latter stages of the assessment process, the Panel's independent engineering consultant, Mr Paul Blumberg RHDHV, "closed out" most of the revetment design issues following the submission of explanatory information from the project engineers.

Table R2 Requests for Further Information – DA CP 16-001

NSW Coastal Panel Request	Response
March 2016	
How the relevant design wave and water level conditions advised have been determined for application at the proposed site;	The methodology applied to the design of the revetment by Coastal Engineering Solutions (now Water Technologies) has been to adopt the Design Event having the 100 year ARI storm tide and associated wave characteristics defined by the comprehensive modelling undertaken for the Coffs Harbour Coastal Processes and Hazards Definition Study (BMT WBM, 2011).
What design approach (or stability assessment) has been applied in order to propose the configuration of structure (slope, toe level, crest level, rock sizes, etc) to withstand the design coastal processes and hydraulic loadings envisaged;	The design techniques attributed to van der Meer in conjunction with the approach of placing armour to accommodate future climate influences have been applied by Coastal Engineering Solutions for the engineering design of the rock-armoured revetment. The extent of damage that is deemed to be acceptable under the 100 year ARI design criteria was selected as 5%.
Whether the scour levels at the toe of the proposed structure indicated are sufficient in this location. With a revetment structure in place to prevent channel or bank migration under flood outflows, there might be the potential for high velocities and associated sand losses from the near vicinity of the structure exceeding those considered relevant for normal beach scour levels on the open coast;	A row of Type B Armour (minimum 3 tonne rocks) is to be placed along the toe of the revetment to form a buttress for the armoured slope above. In the unexpected event of scour being greater than expected at a particular location on the revetment, it will be these large rocks that will first be "undermined". These large rocks then drop into the lower scoured bed, thereby armouring the edge of the scour hole that is immediately in front of the revetment. Whilst this can lead to some settlement or shifting of rocks within the matrix of the primary armour above, the structure will remain intact since the buttress rock is still able to serve its purpose.
What consideration (if any) has been given to the likelihood of wave overtopping of the structure (given the crest level advised is particularly low in parts). If there is overtopping of the structure, how will this be managed?	The design techniques outlined in the "*Wave Overtopping of Sea Defences and Related Structures: Assessment Manual*" (EurOtop, 2007) have been applied by Coastal

NSW Coastal Panel Request	Response
March 2016	
The crest levels advised might be sufficient to limit overtopping to acceptable levels but, there is no evidence this has been considered from the furnished information; and	Engineering Solutions for conditions associated with the 100 year ARI Design Event.
What maintenance regime (if any) is contemplated for such a structure (given what's proposed is a flexible rubble-mound structure) in order that it remain fit for purpose to meet desired level of protection following damage into the future. Further, how will it be adapted or maintained to accommodate projected sea level rise. Whilst sea level rise is acknowledged in the report there is no further information on how the structure has been designed to accommodate (or adapt to) such projections into the future.	The rock armour has been sized to accommodate the expected effects of future sea level rise, as well as changes to the regional wave climate. The rock armouring is more robust than it needs to be for present-day climate conditions, and will prove to be adequate during a more severe 100 year ARI storm tide and wave event occurring at some point in the future - when the predicted climate change effects have manifested themselves. The revetment design is such as to limit damage levels to 5%. In actual fact this may not necessarily require any significant repair works. The term "damage" nominated in such a way in coastal engineering designs accounts for the percentage of individual rocks which move from their initially placed position – which can be to a more stable position within the rock armour matrix. Often during severe storm events, the rock armour slope consolidates – resulting in a tightening of interlocking between individual rocks. So future 5% "damage" can also represent an improvement in structural stability at some locations within the revetment.

NSW Coastal Panel Request	Response
April 2016	
Information concerning the impact of the wall on the geometry of the Creek. The impact assessment provided does not address any potential offsite impacts the wall itself may have. These impacts may arise from the readjustment of the creek and estuary entrance to the construction of the wall. *The Panel notes the ecological significance of the land surrounding the proposed seawall.*	Additional modelling was carried out by Umwelt (Australia) Pty Ltd to determine flow velocity within the creek environment and the potential for offsite impacts. Umwelt prepared a 2D finite element mesh roughness value model to determine flow velocity under existing landform versus with the proposed revetment. The results are that there is essentially no change in flow velocity (**Appendix E**).

NSW Coastal Panel Request	Response
April 2016	
Given the footprint of the proposed seawall, any mitigation measures proposed to minimise impacts on existing vegetation both on site and offsite, and proposed mitigation measures for any Aboriginal cultural heritage items identified on the site.	The footprint of the proposed seawall has not altered. Vegetation within and surrounding the site has been comprehensively assessed by NatureCall and Ecosure. In the most recent assessment, Ecosure found that: ***Littoral Rainforest*** *The 7 part test found that it is not expected that the removal of the trees will have a negative impact on the surrounding EEC in the locality given the small number of trees being removed. A Species Impact Statement is not recommended.* ***Coastal Saltmarsh*** *Approximately 0.002 ha (20 m²) Coastal Saltmarsh will be removed by the proposal. It is considered that the Coastal Saltmarsh EECs within the Arrawarra estuary are unlikely to be significantly affected by the proposed action as the two small isolated patches do not form part of the community. A Species Impact Statement is not recommended.* The primary beneficial mitigation measure for impacts to existing vegetation will be compensatory planting of native species within the E2 zone. ***Aboriginal Cultural Heritage*** Everick Heritage Consultants are of the opinion that *given the extent of existing disturbance within the development footprint, the proposed rock revetment is unlikely to result in further harm to Aboriginal Heritage. No Aboriginal Objects were identified within the area of Proposed Works. One known Aboriginal site (Arrawarra Headland Site #22-1-0392) was recorded to extend into the Project Area however is well within the proposed environmental buffer zone and has been previously disturbed.* Mitigation measures are as listed on page 50 of the Aboriginal Cultural Heritage Assessment.

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Royal HaskoningDHV reviewed DA CP16-001 on behalf of the panel. Table P3 summarises RHDHV's comments and the applicant's responses.

Table R3 DA CP 16-001 RHDHV Assessment

Royal HaskoningDHV, Engineering Assessment 21 September 2016	**Applicant's Response 28 October 2016**	**Royal HaskoningDHV Memo 30 November 2016**	**Applicant's Response 5 December 2016**
1. The base survey by Newnham Karl Weir has not been provided, except that reproduced in the Coastal Engineering Solutions (CES) drawings. The contours are difficult to track. The survey date is uncertain, possibly April 2015 ***Outcome:*** *A base survey showing spot levels and contours is not provided.*	A supplementary site plan prepared by NKWP surveyors showing the revetment, site boundary, spot levels, site structures, existing embankment was provided to the pane in PDF format enabling separate data layers to be 'switched' on or off for ease of reference.	*"Closed out" ** ** taken to mean that the matter has been addressed to the satisfaction of RHDHV.*	No further action required.
2. Is there any history of flooding and wave impacts at the caravan park. Umwelt 2003 flood study may have information? It would be helpful for the Proponent if flooding impacts on the site have not been unduly problematic in the past. ***Outcome:***	Umwelt 2003 Flood Study emailed to the panel on 22/9/16.	*Closed out*	No further action required.

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Umwelt (2003) may provide useful information on flood history which does not appear to be included in the current application			
3. *The gabion wall runs along more than 50% of the shoreline to be protected. When was this installed, and how has it performed?* *Photos included in the application show damaged gabion cages. It would be very helpful to understand existing toe levels for the gabion wall. Our preference would be for the existing gabion wall including its underside/ toe level to be shown on the CES Annotated cross Section drawings.* ***Outcome:*** *Insufficient information is provided on the gabion wall.*	The existing gabion was constructed in 1990 pursuant to DA Consent 224/90 The gabion wall is founded on an unreinforced cast-insitu concrete apron, the top of which is at the sand level of the estuary bed. Since the gabion wall is to be demolished we see no point in having a structural audit of its characteristics, nor any merit in having those aspects included on the drawings for the new structure. Extract from *Geomorphic Impact Assessment for Proposed Seawall, Arrawarra Caravan Park*, Martens, 2007. *The current revetment structure is made up of rock gabion baskets*	*Closed out*	No further action required.

Royal HaskoningDHV, Engineering Assessment 21 September 2016	Applicant's Response 28 October 2016	Royal HaskoningDHV Memo 30 November 2016	Applicant's Response 5 December 2016
	that extend from chainage 175m to 380m. This wall has begun to deteriorate and in some places has completely corroded freeing rip rap. Some warping and settlement is also occurring as a result of wave attack sediment redistribution at the wall toe and tidal influences. *Minor bed sediment erosion (toe scour) was observed along parts of the gabion wall particularly along the eastern boundary where Arrawarra Creek flows entirely along the wall. Small bed scour pools were observed at each end of the existing wall, notably on the Arrawarra reach. Scour behind the existing gabion wall was observed between chainages*		

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	175 to 210. Erosion is localised and no impacts were observed at distance from the existing sea wall. However, erosion indicates that the present wall will need to be replaced sometime in the near future. The damaged gabion wall will be removed as part of the proposed works for the revetment. RHDHV correctly note that: *Along approximately one half of the length of this boundary is a gabion wall in a variable state of disrepair. Parts of this wall would currently pose or likely pose in the near future a hazard to persons who access the bed areas of the creek over the gabion wall. [The bed areas of the creek are*		

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	considered to be included in the definition of "beach" in the Coastal Protection Act 1979]. Removal of this gabion wall would remove a structure which is potentially hazardous, thereby providing a safety benefit. It is agreed that removal of the existing gabion will provide a safety benefit as a result of the proposal.		
4. *The crest levels for the proposed seawall appear quite low compared to the storm tide levels identified in Ref 8. The 50 and 100 year ARI storm tide levels in the estuary entrance are reported at 2.91 and 3.04 m AHD respectively, compared to proposed seawall crest levels of 2.0 to 3.0 m AHD.* ***Outcome:*** *If waves penetrate to the structure in design storms the*	Significant overtopping is expected and is stated clearly in the Revetment Design Report. Refer to subsequent comments addressing review comment #11.	*Further investigation on overtopping impacts and remedial strategy required.*	Response – Paul O'Brien, Water Technology: It is pertinent to appreciate that whilst this issue appears to have evolved to be one on which the decision regarding the technical veracity of the design turns, it is in reality easily resolved. A 0.3 metre increase in depth across the beach berm equates to an approximately 8% increase in design wave height, which equates to an approximately 11% increase in the dimension of the average

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structure will be significantly overtopped.			rock within the currently specified range of 350kg to 3.0 tonnes. This minor change will be readily achieved by simply amending the average rock size in the Technical Specification that will be issued to prospective contractors during the construction phase. If the Coastal Panel was to direct that a condition of consent was to adopt the RHDHV suggested saddle level of RL-0.1m AHD, then the rock specification will be changed to reflect the implication to rock armour size.
5. *No information provided on the availability of suitable rock. Rock dry density of 2.65 T/m3 noted in Ref 8, 4.2.1, but no consideration of rock quantities and potential sources.* ***Outcome:***	Suitable blue rock is of an Argillite type material is available from Woolgoolga Quarry. One months' notice is required for supply and delivery of the material.	*Closed out.*	No further action required.

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Insufficient information is provided on the availability of suitable rock.			
6. *Is the timber walkway bridge to be retained? If so, is it to be rebuilt? It is assumed that the proposed seawall could not be constructed without at least the part removal of the walkway bridge.* ***Outcome:*** *Insufficient information is provided on the retention or otherwise of the timber walkway bridge and how this is to be accommodated in the design.*	The timber walkway bridge is licensed (RI 553550) to Arrawarra Beach Pty Ltd who operate the Arrawarra Beach Holiday Park. A copy of the licence is provided at **Appendix K.** The expiry date of the licence is June 2030. The licence enables the structure to be repaired, rebuilt or partially rebuilt.	Closed out.	No further action required
7. *The Statement of Environmental Effects (SEE) Ref 1, 3.1.2, last para, states that the proposed seawall design takes account of natural processes impacting the site now and severe storms that may eventuate following climate change and sea level rise (SLR) predictions. However, Ref*	No longer relevant.		No further action required

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8, 3.5, 3rd bullet point, states that the rock size has been selected to cater for the present day climate, with the design tailored so that upgrading work can be readily undertaken should future climate change and SLR require modifications. It would appear that the statement in the SEE is not correct, or the proposal has been modified subsequent to the SEE.			
8. *SEE Ref 1, 3.14, p 42, Environmental Impacts, para 1, states" the proposed revetment wall will be constructed entirely within the subject site and as such, will not adversely affect neighbouring properties". In relation to direct property impacts at construction this may be correct, however the potential for wave reflections and "locking up" of foreshore sand to affect neighbouring*	Hydro dynamic modelling (Umwelt, October 2016) indicates that flow velocity with and without the revetment are essentially the same. (**Appendix E**) Issues relating to impacts on overall creek morphology were considered by Martens in their assessment of a previous seawall proposal. The previous design comprised rock armouring to the Yarrawarra Creek boundary and repairs to the existing	*Clarification required.*	

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properties would not appear to be considered. ***Outcome:*** *Insufficient information is provided on the quantum of referred erosion impacts to neighbouring properties and how these are to be managed.* *Comment: Ref 1, 3.3, discusses funding arrangements for ongoing maintenance of the seawall.* *Information is lacking on how these arrangements would be extended to manage potential offsite impacts to neighbouring properties.*	gabion sea wall at the Arrawarra Creek boundary. A copy of the Martens Geomorphic Impact Assessment at **Appendix M**.		
9. *SEE Ref 1, 3.16, para 1, refers to Council's "stormwater outlet". It is not clear where this is. It does not appear to be referred to elsewhere in the background briefing information.* ***Outcome:***	The stormwater outlet is located within Lot 101 DP 1122639 adjacent to the north-western corner of the site as shown in the image below (green circle): Continual erosion to the north-west area of the embankment is likely to have been compounded by the construction, presumably by	*Closed Out.*	No further action required

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Information is lacking on the details of Council's "stormwater outlet".	Council, of a stormwater pipe concentrating the stormwater runoff from Arrawarra Village homes and roadways into that part of the creek, the results have been undermining and collapse of the concrete public access stairs and the greatly intensified embankment erosion in this vicinity.  The stormwater outlet:		

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10. The MHWM boundary is not clearly defined on the CES design layout for the seawall. This should clearly mark start and end point of each line segment that defines this boundary, with numbering of the line segments cross-referenced to a table on the CES drawings. This table should indicate each line segment, line bearing and distance as shown on DP 1209371 registered 19/6/15 ***Outcome:*** *The definition of the MHWM property boundary is not adequate on the CES drawings.*	Whilst the line of MHWM is included in the design drawings, the necessary scale of the drawing makes it difficult to identify that particular line without masking other more important information. To address this issue, the sixteen drawings numbered 15-849nsw-03 to 18 all clearly show the position of the MHWM boundary, its location in relation to the proposed works and its location to existing land contours (**Appendix C**). Upon review, we cannot see how this could be made any clearer.	Closed out.	No further action required

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11. *Ref 8, 2.4, para 5, gives overtopping rates for varied damage behaviour behind a seawall, but no overtopping rates are reported for the developed site.* ***Outcome:*** *Design overtopping rates so overtopping impacts cannot be suitably assessed or a management strategy developed as there is insufficient information.*	A critical consideration of the design was the determination of overtopping rates along sections of the proposed wall. For the section of the revetment having the crest level of RL+3.0m AHD, the overtopping rate is dependent upon the incident wave period, but is in excess of 0.4m^3/sec/m. This indicates scour of the material behind the structure will occur. Since this will occur in the 20-metre (E2) buffer to infrastructure, this is deemed by the design process as an acceptable outcome (since the damage can be repaired) provided the crest armour does not suffer excessive damage or fail. The placement of rocks greater than 3 tonnes as buttress rocks to the rear of the crest is intended to achieve this performance outcome. For the section of revetment with a crest level at RL+2.0m AHD, the structure will be	*Design wave overtopping in excess of 400L/s per m predicted which is a very high value. Further investigation on overtopping impacts and remediation required.*	Response – Paul O'Brien, Water Technology: The concerns of RHDHV regarding the very high rates of overtopping are shared by the revetment designers. Indeed, as noted in the Design Report and subsequent responses to Requests for Information, it has been a significant focus of design efforts. The outcome of those design efforts is that the special armouring is required on crest of the structure and across the width of the E2 Zone. Such design techniques are widely used by coastal engineers to ensure the structural integrity of the revetment and land levels behind it during the 100 year ARI storm event. Any grassing or vegetation within the E2 Zone will be damaged, but not the revetment structure itself nor any essential infrastructure behind it. We note that the concerns of RHDV appear to relate to the

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	submerged by approximately 1 metre during the 100 year ARI storm tide / wave event, as well as by the 100 year ARI flood. The design of the armoured slope is such that it will remain structurally intact. It is intended to cause waves approaching and passing over the structure to break within the 20-metre wide buffer of the E2 zone. This is why 2 layers of 0.1 tonne rock armour is extended as a buried scour apron across the entire E2 zone on this type of wall. The extensive filling to otherwise raise the wall to mitigate this wave overtopping was deemed by the design process as an unacceptable modification to existing creek banks and the E2 zone.		"safety implications for severe wave overtopping to carry across the 20m E2 buffer". RHDHV have not opined that the structural design of the crest is in some way inadequate, but instead that there could be a safety issue associated with people being in the overtopping area during a 100 year ARI storm. Whilst that is certainly a valid safety issue (as it is for the entire shoreline during such events), it does not affect the structural characteristics of the revetment.
16. *Ref 8, 3.4.2, has selected a 130m wide scoured entrance channel with bed level +0.2 m AHD for assessment of design wave penetration to the site. We are concerned that +0.2 m AHD based on a single*	We do not believe that the methodology suggested (of using recorded water levels at other estuaries) to infer the scoured width and level of the entrance berm of Arrawarra Creek as a consequence of the 100 year ARI event is any	*More detailed assessment required to demonstrate selection of +0.2m AHD saddle level. In absence of further assessment and based on RHDHV review of comparable creek systems in NSW, it is*	Refer to response above.

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survey in April 2003 and understood to represent "average natural conditions for the purposes of the (1 in 100 year) flood study" (Umwelt, 2016) may not capture a suitable case for design wave penetration. To come to this view we have examined available water level data in coastal creek systems of comparable catchment size to Arrawarra Creek. *Unfortunately water level data is not collected at Arrawarra Creek and examining the behaviour of similar systems is a reasonable approach.* *The Estuaries Inventory of NSW (PWD, 1992) lists 91 NSW estuaries and characterises these according to parameters including catchment area. Arrawarra Creek is reported to have a catchment area of 20 km² which places it number 86*	more robust or accurate that that utilised for the design of the revetment. The entrance arrangement at Arrawarra Beach will be determined by the combined action of storm tide and extreme waves during such an oceanic storm event – not by any scour of flows exiting the estuary during a 100 year ARI Flood event. Additional modelling was carried out by Umwelt (Australia) Pty Ltd to determine flow velocity within the creek environment and the potential for offsite impacts. Umwelt prepared a 2D finite element mesh roughness value model to determine flow velocity under existing landform versus with the proposed revetment. The results are that there is essentially no change in flow velocity (**Appendix E**). RHDHV comment states an entrance berm level	*suggested that -0.1m AHD saddle level be adopted.*	

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in order of reducing catchment size. Other systems of comparable size and which we understand are not trained include Saltwater Creek (30 km²), Werri Lagoon (24 km²), Back Lagoon, Merimbula (23 km2), Belongil Creek (18 km²), Lake Cakora (11 km2) and Lake Arragan (10 km2). Of these six estuaries, MHL records water level data at Saltwater Creek, Werri Lagoon and Back Lagoon. The full water level records for these three lagoons are shown in Appendix A. Of particular interest, and the selected record for this is also shown, is the minimum water level at which tidal penetration occurs as this would represent the maximum bed level across the entrance saddle for the case of ocean water penetration, ie the saddle level could have been no higher, but was probably	comparable to Werri Lagoon entrance should be used. We disagree, and note that (unlike the estuary entrance of Arrawarra Creek) the entrance to Werri Lagoon is constrained by a rocky headland and has only around 75m maximum width. Irrespective of the concerns expressed above as to applying the suggested methodology for determining entrance conditions during a 100 year ARI storm tide/wave event, the narrower and constrained entrance at Werri Lagoon results in deeper channel flows during floods than at Arrawarra Beach. So in our view it is not appropriate to simply adopt the inferred entrance berm levels at Werri Lagoon as being those scoured by a 100 year ARI storm tide & wave event at Arrawarra Beach.		

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lower. Also included is a record of the water levels through the recent June 2016 storm event. *We can see that for these three systems of comparable size to Arrawarra Creek, this limiting saddle level was approximately -0.4 m AHD for Werri Lagoon, -0.1 m AHD for Back Lagoon and 0.45 m AHD for Saltwater Lagoon.* *Having regard therefore to the information before us, RHDHV would be suggesting that a saddle level of -0.4 m AHD should therefore represent* ***a maximum entrance saddle level*** *for the calculation of design storm wave penetration. This is 0.6 m* *below the +0.2 m AHD saddle level adopted by the proponent* *which could be expected to lead to significantly higher wave*			

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penetration. ***Outcome:*** *Insufficient justification is given regarding the selection of +0.2 m AHD as an appropriate entrance saddle level for design wave penetration. With water level data not collected for Arrawarra Creek, inspection of water level records for comparable NSW estuary systems indicates that a saddle level at least 0.6 m lower should reasonably apply.*			
19. *Ref 8 uses van der Meer (1988) to calculate armour rock sizes for a 100 year ARI storm event, finding that a 1 tonne (T) primary armour with minimum rock density 2.65 T/m3 placed to a slope of 1:1.5 meets the requirements of van de Meer for 5% damage. RHDHV is comfortable with the assumptions and general calculation approach, however we*	Refer to sample calculations indicating two layers of 1 tonne primary armour (**Appendix C**). Calculations refer to equation/page numbers in CIRIA (2007).	*RHDHV accepts vd Meer (1988) as reported in CIRIA (2007).* *Prudent to construct non-government owned seawall today to withstand 0.8m SLR to 2100. Adopted saddle level of +0.2m AHD should be reviewed.* *Ensure seawall setback to MHWM boundary.*	No further action required. Noted.

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find that the rock masses reported in Table 7 range between approximately one half to one third that specified using van de Meer (1988). We have applied Table VI-5-23 in Coastal Engineering Manual to complete our calculation checks, making reasonable assumptions of porosity (P=0.45), relative eroded area or damage (S=2.5) and applying the wave height modification for depth-limited waves (H=H2% /1.4). We have also applied Hudson (SPM, 1984) to cross-check our assessment. *Provisional calculations by RHDHV indicate that providing for primary armour comprising at least 2 layers of 2T median rock (all other parameters unchanged) should provide for suitable seawall*			The seawall is set back landward of the MHWM boundary.

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slope protection against a 100 year ARI storm event occurring today. ***Outcome:*** *While RHDHV is comfortable with van der Meer (1988) to calculate armour rock sizes, we are concerned that the specified 1.0 T primary armour is one half to one third the mass required by that assessment method, although this makes no allowance for the additional rock size required to accommodate larger wave penetration due to increased saddle water depths. Rock armour calculations to demonstrate compliance with van der Meer (1988) are not provided.*			
20. Ref 8, 3.56.2, last para, states that for 2100 with 0.8 m SLR, the breaking significant wave height at the revetment would be 20- 25% higher than the breaking wave heights in Table 6. Ref 8, 4.2.2, 2nd	Refer to sample calculations indicating three layers of 1 tonne primary armour (**Appendix C**). Calculations refer to equation/page numbers in CIRIA (2007).	*As stated above.*	No further action required.

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para, states that when 0.8 m depth added to the design storm tide and wave heights are increased accordingly, then one additional layer of primary armour rocks would accommodate the additional wave loads. *Provisional calculations by RHDHV indicate that providing for an additional 2 layers of 2T median rock (all other parameters unchanged) should provide for suitable seawall slope protection to the end of a structure life at 2100.* *The above assessment is based on the wave climate assessment assuming the +0.2 m AHD entrance saddle level. The additional water depth across the entrance saddle (Item 16) would lead to larger wave penetration and further increase the rock size in the revetment.*			

Royal HaskoningDHV, Engineering Assessment 21 September 2016	**Applicant's Response 28 October 2016**	**Royal HaskoningDHV Memo 30 November 2016**	**Applicant's Response 5 December 2016**
Outcome: *RHDHV notes that the Coastal Panel is receptive to an adaptive approach to increase the capacity of the seawall over its life (Ref 4, last bullet point). However, RHDHV is concerned that the adaptive modification proposed in Ref 8 may not suffice to provide suitable protection to the end of structure life. Suitably detailed calculations are not provided which demonstrate the acceptability of the proposed adaptive approach.* *Increasing the size of the additional (adaptive) rock layers, considered to be necessary by RHDHV, would require the seawall to be initially located further landward from the existing MHWM property boundary than is currently proposed.*			
21. *Ref 8, Figure 9 seems to show the NE end of the seawall terminating some 80 m short of the end*	The drawings show the accurate set-out of the proposed works and are drawn to scale. The figure in	Closed out.	

Royal HaskoningDHV, Engineering Assessment 21 September 2016	Applicant's Response 28 October 2016	Royal HaskoningDHV Memo 30 November 2016	Applicant's Response 5 December 2016
termination point shown in the design drawings. ***Outcome:*** *Certificate required*	the report shows indicative extents only. It is noted that the drawings are for DA purposes. Technical drawings would be provided with the CC documentation.		
22. *The seawall proposal assumes design scour levels at the toe of the wall ranging between -1.0 and -0.5 m AHD, based on this meeting a level equal to one median rock diameter below the local channel thalweg. The actual location of the thalweg was not reported.* *RHDHV is concerned that this approach may miss deeper localised scour that could occur during high creek flows, particularly during periods of low tail water levels. We have recently modelled, using MIKE21, creek outflows at another North Coast creek system where a design scour level of -1.5*	Refer comment #3 above (regarding the gabion wall).	Closed out.	No further action required.

Royal HaskoningDHV, Engineering Assessment 21 September 2016	Applicant's Response 28 October 2016	Royal HaskoningDHV Memo 30 November 2016	Applicant's Response 5 December 2016
m AHD was identified along a straight section of the protected creek bank, and locally deepened to -2m AHD and deeper at downstream spur walls. *Seven Google earth photos at Arrawarra Creek between 2004 and 2013 show the channel hard up against the gabion wall in the vicinity of SOP 22 and 23 for 4 of the 7 photo dates. It would be of interest to know what the minimum channel bed level may have been against the gabion wall during this period.* *Furthermore, if there is evidence of the gabion wall having settled in this area, it would be important to know the design toe level for the gabion wall and the existing settled toe level. Note that it is possible for a gabion wall to bridge a localised scour hole*			

Royal HaskoningDHV, Engineering Assessment 21 September 2016	Applicant's Response 28 October 2016	Royal HaskoningDHV Memo 30 November 2016	Applicant's Response 5 December 2016
although this should be evident by loss of backfill. **_Outcome:_** *To adequately define potential scour:* *(i) Information is required on the minimum channel bed level against the gabion wall in the vicinity of SOP 18 through SOP 29, including a review of the gabion toe level (design and settled)* *(ii) Consideration should be given to extending the existing flooding assessment to model channel velocities and scour along the toe of the rock revetment.*			
23. *It appears from the information provided that a geotechnical assessment has not been undertaken to investigate the ground conditions in the vicinity of the proposed seawall. Valuable information could be obtained from a geotechnical site investigation to characterise the*	The approach taken for the development of a design up to DA stage has been to develop what is considered to be a conservative arrangement at the toe. The review comments seem to take no account of the role that 3 tonne minimum-sized buttress rocks at the base of	Closed out.	No further action required.

Royal HaskoningDHV, Engineering Assessment 21 September 2016	Applicant's Response 28 October 2016	Royal HaskoningDHV Memo 30 November 2016	Applicant's Response 5 December 2016
subsurface conditions. The work could be expected to include a site walk over by an experienced geotechnical engineer, an assessment of the regional geotechnical context from geological mapping, and intrusive investigations. Techniques might include test pits along the footprint of the proposed seawall including along the toe of the gabion wall, boreholes, CPTs, and DCPs. A suitably designed geotechnical investigation will provide important baseline information to assist develop an appropriate toe detail for the seawall and provide design certainty. ***Outcome:*** *No geotechnical investigation would appear to be included in the background briefing information. To properly inform the seawall toe design it would be highly desirable to conduct*	the wall plays in self-armouring the foundation. A Geotechnical Assessment is provided at **Appendix O.**		

Royal HaskoningDHV, Engineering Assessment 21 September 2016	Applicant's Response 28 October 2016	Royal HaskoningDHV Memo 30 November 2016	Applicant's Response 5 December 2016
a suitably scoped geotechnical investigation.			
24. Item 11 refers. Ref 8, 4.4.1, overtopping assessment. Ref 8 refers to overtopping calculation that show discharges will scour any unprotected erodible materials in the area immediately behind the revetment, but no overtopping rates are reported. ***Outcome:*** *Overtopping quantities are not provided to enable a suitably thorough assessment of overtopping impacts and management.*	Refer response to review comment #11.	*Refer Item #4 and #11.*	
25. Seawall crest levels of 3.0 and 2.0 m AHD seem low for a site that can experience storm tide levels of 2.9 to 3.0 m AHD in a design storm plus breaking wave heights of 2 m or more. If wave heights can increase by around 20 - 25% in a 100 year storm at 2100, this could have a	Refer response to review comment #11. The adaptive response to extend the service life of the structure to the year 2100 is to simply add another layer of buried rocks within the 20 m wide E2 zone.	*Strategy required to manage wave overtopping impacts*	Response – Paul O'Brien, Water Technology As noted above, the concerns of RHDHV regarding the very high rates of overtopping are shared by the revetment designers. It has been a significant focus of design efforts. The outcome of those design efforts is that the special

Royal HaskoningDHV, Engineering Assessment 21 September 2016	Applicant's Response 28 October 2016	Royal HaskoningDHV Memo 30 November 2016	Applicant's Response 5 December 2016
significant impact on wave overtopping. ***Outcome:*** *Detailed information is lacking on the adaptive response to deal with what appear to be low seawall crest levels, particularly given the predicted 20-25% increase in design wave heights by year 2100.*			armouring is required on crest of the structure and across the width of the E2 Zone. We again note that the concerns of RHDHV appear to relate to the "safety implications for severe wave overtopping to carry across the 20m E2 buffer" rather than the structural design of the crest is in some way inadequate.
26. Design drawing 15-849NSW-02 B lists design parameters. Wave height H is given as 2.0 m with T = 7-12 s. Notwithstanding the issue of inadequate saddle depth and consequent larger wave penetration, it would be more accurate for H = 1.9 to 2.3 m to be added to the drawing to better reflect Table 6 in Ref 8. ***Outcome:*** Design drawing 15-849NSW- 02 B does not fully describe the design wave height	This issue is not considered critical in the determination of the DA. If necessary, any required adjustments to seawall design will be made to the final construction drawing set.	Closed out.	No further action required.

Royal HaskoningDHV, Engineering Assessment 21 September 2016	Applicant's Response 28 October 2016	Royal HaskoningDHV Memo 30 November 2016	Applicant's Response 5 December 2016
Conclusion by RHDHV: *Various matters have now been satisfactorily addressed to assist the Coastal Panel in its consideration of the application. RHDHV is satisfied with vd Meer (1988) as described in CIRIA (2007) for sizing the rock armour, however we remain concerned that a design entrance saddle level of +0.2 m AHD is too elevated.* *Investigation is required to demonstrate the acceptability of this level. It is noted that a lower saddle level would permit larger wave penetration and require larger armour size.* *Design wave overtopping reported by WT as exceeding 400 L/s per m is of concern. The consequences and management of design wave overtopping, particularly at the exposed north-eastern portion of the seawall, needs to be addressed.*			

Royal HaskoningDHV
Enhancing Society Together

Record Note / Memo

Haskoning Australia PTY Ltd.
Maritime & Aviation

To:	Mark Daley, Senior Coast and Estuaries Officer, OEH
From:	Gary Blumberg
Date:	30 November 2016
Copy:	Phil Watson, Principal Coastal Specialist OEH
Our reference:	PA1431_N003.F02
Classification:	Open

Subject: **Arrawarra Creek Revetment Development Application No CP16-001**
Assessment for NSW Coastal Panel
Considerations of Coastal Engineering Implications of Supplementary Information furnished by Applicant's Representative on 28 October 2016

Marc

Further to our recent discussions including those with Phil Watson, set out below please find RHDHV's addendum memo (PA 1431_N003) confirming my professional opinion on the coastal engineering matters relating to the above development application. This memo follows and responds to supplementary information provided to address our previous two memos N001.F02 and N002.F02.

The first memo itemised 26 comments which are responded to by Water Technology (Australia) Pty Ltd (WT) in the Supplementary Statement of Environmental Effects (Keiley Hunter Town Planning, 2016). We have reviewed these comments, and either closed them out where we have accepted the additional information, or have suggested that further information is required for us to make a suitable assessment.

A record of the updated comment log is provided in **Appendix A**. A summary overview of my current opinion is provided in the table below. Of the 26 items, 9 remain to be addressed. Of these 9, 3 deal with the entrance saddle level which we still consider to be too high, and 3 with wave overtopping. It is noted that lowering the entrance saddle level feeds back to higher design wave penetration, and potentially larger armour size and increased wave overtopping.

Item	Coastal Engineering Matter	Comment/ Opinion
1	Survey	Closed out
2	History of flooding	Closed out.
3	Information on gabion wall	Closed out
4	Wave overtopping	Further investigation on overtopping impacts and remedial strategy required.
5	Availability of suitable rock	Closed out.
6	Timber walkway bridge	Closed out.
7	Revetment design re climate change	Clarification required.

Item	Coastal Engineering Matter	Comment/ Opinion
9	Details of stormwater outlet	Closed out.
10	Description of MHWM boundary on seawall DA drawings	Closed out. Source of MHWM boundary to be noted on DA drawings
11	Wave overtopping	Design wave overtopping in excess of 400L/s per m predicted which is a very high value. Further investigation on overtopping impacts and remediation required. Item 4 refers.
12	Application of AS 4997	Comment only – no action.
13	Application of AS 4997	Comment only – no action.
14	2 scenario approach to selection of coastal design parameters	Comment only – no action.
15	Fit for purpose	Comment only – no action.
16	Selection of 0.2m AHD saddle level	More detailed assessment required to demonstrate selection of +0.2m AHD saddle level. In absence of further assessment and based on RHDHV review of comparable creek systems in NSW, it is suggested that -0.1m AHD saddle level be adopted.
17	Depth limited wave climate	Comment only – no action.
18	Approach to wave height selection	Comment only – reference to Item 16.
19 and 20	vd Meer (1988) rock armour calculation	RHDHV accepts vd Meer (1988) as reported in CIRIA (2007) and applied in Appendix S. Prudent to construct non-government owned seawall today to withstand 0.8m SLR to 2100. Adopted saddle level of +0.2m AHD should be reviewed. Ensure seawall setback to MHWM boundary.
21	Seawall end position	Closed out.
22	Channel velocities and toe scour level and	Closed out.
23	Geotechnical information	Closed out.
24	Wave overtopping	Item 4 and 11 refer.
25	Low crest level and wave overtopping, including effects of SLR	Strategy required to manage wave overtopping impacts
26	Wave height description in drawings	Closed out.

Particular mention should be made regarding the prediction by WT made at Item 11 that for the section of wall having a crest level of +3.0m AHD, the overtopping rate is dependent on the incident wave period but is in excess of 0.4m^3/s/m. We can only assume that this is a mean overtopping rate as is the normal practice of presenting this parameter and assessing its impact. Our concern here relates to our benchmarking of this overtopping rate. A particular example which we need to pay attention to is the video record of overtopping at Fairy Bower in Manly on 5 June 2016.

https://royalhaskoningdhv.box.com/s/vg276x84l75j2xbo2y2q06vsmpx8wz37

During this severe storm RHDHV understands that the wave overtopping rate in this event estimated by UNSW Water Research Laboratory was less than 10% of the minimum overtopping rate predicted by WT for the proposed Arrawarra seawall (Ian Coghlan pers comm). We have reviewed this estimate ourselves based on our observation of the video and agree with this value. By any reasonable review of this footage and also by comparative assessment with criteria for wave overtopping presented in Coastal Engineering Manual (USACE, 2002+) (refer **Appendix B**), it is clear that a high level of caution needs to be brought to his hazard and how it is proposed to be managed. At this point there would appear to be insufficient information before the Panel for it to be satisfied that wave overtopping of the seawall section with 3.0 m AHD crest level is adequately appraised or managed.

The findings of the site inspection made by Gary Blumberg on 20 October 2016 were reported in RHDHV memo N002.F02. Comment was made in this memo on the WBM Coastal Hazard Study which identifies a receding coastline. The proposed seawall is today located behind the beach berm and entrance saddle, but these features would progressively become less relevant over time as the coastline recedes and the seawall would become progressively more exposed to wave action. As is evident from the WBM figure attached to RHDHV memo N002.F02, approximately 90 m of the seawall is currently exposed to “almost certain” Immediate wave erosion hazard as defined by the Zone of Slope Adjustment. A length of the seawall longer than 90 m is also potentially impacted, but at a reduced level of likelihood than “almost certain”. This description of Immediate wave erosion hazard calculated by WBM using conventional wave erosion hazard assessment methods reinforces the level of scrutiny that should correctly be brought to developing an appropriately conservative design for the seawall.

Conclusion

RHDHV has reviewed the supplementary information provided to support the design of the proposed seawall at 46 Arrawarra Beach Road, Arrawarra. Our brief was to review the overall suitability of the seawall and determine whether it meets contemporary engineering design standards considered appropriate for this location. The purpose of our assessment is to assist the NSW Coastal Panel in making the DA determination for this structure.

Various matters have now been satisfactorily addressed to assist the Coastal Panel in its consideration of the application.

RHDHV is satisfied with vd Meer (1988) as described in CIRIA (2007) for sizing the rock armour, however we remain concerned that a design entrance saddle level of +0.2 m AHD is too elevated. Investigation is required to demonstrate the acceptability of this level. It is noted that a lower saddle level would permit larger wave penetration and require larger armour size.

Design wave overtopping reported by WT as exceeding 400 L/s per m is of concern. The consequences and management of design wave overtopping, particularly at the exposed north-eastern portion of the seawall, needs to be addressed.

Please do not hesitate to contact me should you require any further information or clarification.

Gary Blumberg
RHDHV, Manager Coastal, Maritime and Waterways Australia

References

CIRIA (2007)
The Rock Manual. The Use of Rock in Hydraulic Engineering (2nd ed)
C683. Published by CIRIA, London. ISBN 978-0-86017-683-1, June 2007

US Army Corps of Engineers (2002+)
Coastal Engineering Manual
EM 1110-2-1100

Keiley Hunter Town Planning (2016)
Statement of Environmental Effects, Coastal Protection Works
46 Arrawarra Beach Road, Arrawarra
Prelodgement Supplementary Report, October 2016

Van der Meer JW (1988)
Rock Slopes and Gravel Beaches under Wave Attack
Published as PhD thesis, also Delft Hydraulics Communication No 396

APPENDIX A

RESPONSES TO COMMENTS BY WT ON RHDHV ADVICE SET OUT IN THE SUPPLEMENTARY STATEMENT OF ENVIRONMENTAL EFFECTS

Keiley Hunter

STATEMENT OF ENVIRONMENTAL EFECTS
SUPPLEMENTARY – OCTOBER 2016

4.4 RoyalHaskoningDHV Assessment

RoyalHaskoningDHV (RHDHV) prepared a desktop assessment of the engineering design of the proposed revetment. The RHDHV assessment contains some anomalies that could have been addressed by, firstly a site visit; and secondly, liaison with the project consultants for access to background assessments that informed the design of the revetment.

The RHDHV assessment was completed on 21 September 2016, however, the assessment was not made available to the proponents until 10 October 2016. RHDHV's engineer conducted a site visit on 20 October 2016 and will be providing a supplementary report to the Panel, including any additional information which the site visit may have provided (Dr Marc Daley, OEH, email 28/10/16). Obviously the findings of the supplementary RHDHV report were not available for consideration in this supplementary SoEE.

On 12 October 2016, a letter of request was emailed to the panel requesting that determination of the DA be deferred until the applicants had a reasonable opportunity to respond to the matters raised by the independent consultants.

The panel secretariat advised by email received at 3.40 pm on 21 October 2016 that the matter was now set down for determination by the panel on 2 November 2016 and that any additional information must be received by close of business on 28 October 2016. This left five business days to provide additional information on technical matters relating to the design of the revetment and flood modelling.

The following table of responses to the matters raised by RHDHV has been prepared in the short time available to the applicants and their engineering consultants, Water Technologies, formerly Coastal Design Solutions and Umwelt (Australia) Pty Limited.

Set out below please find RHDHV response to responding information prepared by Water Technologies included in SoEE Supplementary report dated October 2016.

Table 4.3 RoyalHaskoningDHV comments and responses

Comments from Gary Blumberg, Royal HaskoningDHV, desktop independent engineering assessment:	**RESPONSE**
1. *The base survey by Newnham Karl Weir has not been provided, except that reproduced in the Coastal Engineering Solutions (CES) drawings. The contours are difficult to track. The survey date is uncertain, possibly April 2015* ***Outcome:*** *A base survey showing spot levels and contours is not provided.*	Refer to **Appendix V.** A supplementary site plan has been prepared by NKWP surveyors showing the revetment, site boundary, spot levels, site structures, existing embankment. The plan will be separately provided to the panel in a PDF format that enables the separate layers to be 'switched' on or off for ease of reference.

Response accepted by RHDHV. Comment closed out.

2. Is there any history of flooding and wave impacts at the caravan park. Umwelt 2003 flood study may have information? It would be helpful for the Proponent if flooding impacts on the site have not been unduly problematic in the past. ***Outcome:*** *Umwelt (2003) may provide useful information on flood history which does not appear to be included in the current application* — Umwelt 2003 Flood Study emailed to the panel on 22/9/16. Flood Study and emails attached at **Appendix N**.	Umwelt 2003 Flood Study shows maximum observed flood levels 2.6m AHD at east boundary, and 2.7m AHD at north boundary. Debris lines from east coast low 5/6/16 interpreted at 2.6m AHD in Photo 11. Information sufficient for RHDHV. Comment closed out.

3. *The gabion wall runs along more than 50% of the shoreline to be protected. When was this installed, and how has it performed?* *Photos included in the application show damaged gabion cages. It would be very helpful to understand existing toe levels for the gabion wall. Our preference would be for the existing gabion wall including its underside/ toe level to be shown on the CES Annotated cross Section drawings.* **Outcome:** *Insufficient information is provided on the gabion wall.*	Comment from Coastal Engineering Solutions (CES): The gabion wall is founded on an unreinforced cast-insitu concrete apron, the top of which is at the sand level of the estuary bed. Since the gabion wall is to be demolished we see no point in having a structural audit of its characteristics, nor any merit in having those aspects included on the drawings for the new structure. Extract from *Geomorphic Impact Assessment for Proposed Seawall, Arrawarra Caravan Park, Martens, 2007*. *The current revetment structure is made up of rock gabion baskets that extend from chainage 175m to 380m. This wall has begun to deteriorate and in some places has completely corroded freeing rip rap. Some warping and settlement is also occurring as a result of wave attack sediment redistribution at the wall toe and tidal influences.* *Minor bed sediment erosion (toe scour) was observed along parts of the gabion wall particularly along the eastern boundary where Arrawarra Creek flows entirely along the wall. Small bed scour pools were observed at each end of the existing wall, notably on the Arrawarra reach. Scour behind the existing gabion wall was observed between chainages 175 to 210. Erosion is localised and no impacts were observed at distance from the existing sea wall. However, erosion indicates that the present wall will need to be replaced sometime in the near future.* The damaged gabion wall will be removed as part of the proposed works for the revetment. RHDHV correctly note that:	RHDHV did not request a "structural audit". RHDHV observed gabion toe levels at its site inspection on 20/10/16. We also note that gabion crest levels are included in the survey attached to de Groot and Benson geotechnical investigation Job 0074 Rev B 2016 (App T). This information has been used to review the proposed seawall toe levels. RHDHV accepts that the present gabion wall will need to be replaced sometime in the near future. RHDHV has agreed that the removal of the existing gabion wall will provide a safety benefit. Comment closed out.

	Along approximately one half of the length of this boundary is a gabion wall in a variable state of disrepair. Parts of this wall would currently pose or likely pose in the near future a hazard to persons who access the bed areas of the creek over the gabion wall. [The bed areas of the creek are considered to be included in the definition of "beach" in the Coastal Protection Act 1979]. Removal of this gabion wall would remove a structure which is potentially hazardous, thereby providing a safety benefit. It is agreed that removal of the existing gabion will provide a safety benefit as a result of the proposal.	
4. The crest levels for the proposed seawall appear quite low compared to the storm tide levels identified in Ref 8. The 50 and 100 year ARI storm tide levels in the estuary entrance are reported at 2.91 and 3.04 m AHD respectively, compared to proposed seawall crest levels of 2.0 to 3.0 m AHD. ***Outcome:*** *If waves penetrate to the structure in design storms the structure will be significantly overtopped.*	Comment from CES: Significant overtopping is indeed expected and is stated clearly in the Revetment Design Report. Refer to subsequent comments addressing review comment #11.	At Item 11, design wave overtopping in excess of 400L/s per m is predicted for the section of revetment having a crest level of 3.0m AHD. This is a very significant overtopping rate. RHDHV is concerned for the sustainability of the landscaped zone immediately landward of the crest of the revetment in this zone. . The capacity and potential safety implications for severe wave overtopping to carry across the 20m E2 buffer (ie, extend 12m behind the seawall crest) and impact on the new subdivided foreshore properties should be investigated and a remedial strategy proposed. To gain an appreciation of the scale of the overtopping being discussed here, the reader's attention is drawn to the video of wave overtopping at Fairy Bower recorded by WRL on 5/6/16 and presented by B Modra to the NSW Coastal Conference in Coffs Harbour in November 2016. RHDHV understands that the estimated wave overtopping rate in the event was less 10% of the minimum overtopping rate predicted by WT for the proposed Arrawarra seawall (3.0m AHD crest portion). RHDHV has a copy of this video record which can be provided if required.

5. *No information provided on the availability of suitable rock. Rock dry density of 2.65 T/m3 noted in Ref 8, 4.2.1, but no consideration of rock quantities and potential sources.* ***Outcome:*** *Insufficient information is provided on the availability of suitable rock.* Suitable blue rock is of an Argillite type material is available from Woolgoolga Quarry. One months' notice is required for supply and delivery of the material.	Comment closed out.
6. *Is the timber walkway bridge to be retained? If so, is it to be rebuilt? It is assumed that the proposed seawall could not be constructed without at least the part removal of the walkway bridge.* ***Outcome:*** *Insufficient information is provided on the retention or otherwise of the timber walkway bridge and how this is to be accommodated in the design.* The timber walkway bridge is licensed (RI 553550) to Arrawarra Beach Pty Ltd who operate the Arrawarra Caravan Park. A copy of the licence is provided at **Appendix O.** The expiry date of the licence is June 2030. The licence enables the structure to be repaired, rebuilt or partially rebuilt. The timber bridge is not part of the works for this DA or for the subdivision DA. However, as shown on the Landscape Plan at **Appendix P**, a pathway 2 m wide will be constructed to Council's public pathway specifications to meet with the bridge. The proposed access pathway will provide permanent legal public access to the footbridge.	RHDHV accepts the explanation, but notes that there would be a need to remove at least part of the existing walkway bridge to build the seawall under its landside approach. Comment closed out.

7. *The Statement of Environmental Effects (SEE) Ref 1, 3.1.2, last para, states that the proposed seawall design takes account of natural processes impacting the site now and severe storms that may eventuate following climate change and sea level rise (SLR) predictions. However, Ref 8, 3.5, 3rd bullet point, states that the rock size has been selected to cater for the present day climate, with the design tailored so that upgrading work can be readily undertaken should future climate change and SLR require modifications. It would appear that the statement in the SEE is not correct, or the proposal has been modified subsequent to the SEE.* This section of the SoEE has been modified to reflect the design methodology in the Revetment Design Report.	Reference to 3.1.2 in the original comment should read 3.12. Please advise where SoEE has been modified to reflect the design methodology in the Revetment Design Report.
8. *SEE Ref 1, 3.14, p 42, Environmental Impacts, para 1, states" the proposed revetment wall will be constructed entirely within the subject site and as such, will not adversely affect neighbouring properties". In relation to direct property impacts at construction this may be correct, however the potential for wave reflections and "locking up" of foreshore sand to affect neighbouring properties would not appear to be considered.* ***Outcome:*** *Insufficient information is provided on the quantum of referred erosion impacts to neighbouring properties and how these are to be managed.* *Comment: Ref 1, 3.3, discusses funding arrangements for ongoing maintenance of the seawall.* *Information is lacking on how these arrangements would be extended to* Hydro dynamic modelling (Umwelt, October 2016) indicates that flow velocity with and without the revetment are essentially the same. (**Appendix U**) Issues relating to impacts on overall creek morphology were considered by Martens in their assessment of a previous seawall proposal. The previous design comprised rock armouring to the Yarrawarra Creek boundary and repairs to the existing gabion sea wall at the Arrawarra Creek boundary. A copy of the Martens Geomorphic Impact Assessment and deGroot and Benson seawall design are provided at **Appendix Q**.	RHDHV is prepared to close this out on the basis that a condition of consent similar to that noted at p13 of SoEE Supplementary Report could be made by CHCC.

9. *SEE Ref 1, 3.16, para 1, refers to Council's "stormwater outlet". It is not clear where this is. It does not appear to be referred to elsewhere in the background briefing information.* ***Outcome:*** *Information is lacking on the details of Council's "stormwater outlet".*	The stormwater outlet is located within Lot 101 DP 1122639 adjacent to the north-western corner of the site as shown in the image below (green circle): Continual erosion to the north west area of the embankment is likely to have been compounded by the construction, presumably by Council, of a stormwater pipe concentrating the stormwater runoff from Arrawarra Village homes and roadways into that part of the creek, the results have been undermining and collapse of the concrete public access stairs and the greatly intensified embankment erosion in this vicinity. See images below.  The stormwater outlet:	Comment closed out.
		

10. The MHWM boundary is not clearly defined on the CES design layout for the seawall. This should clearly mark start and end point of each line segment that defines this boundary, with numbering of the line segments cross-referenced to a table on the CES drawings. This table should indicate each line segment, line bearing and distance as shown on DP 1209371 registered 19/6/15 ***Outcome:*** *The definition of the MHWM property boundary is not adequate on the CES drawings.*	Comment from CES: We disagree with this *Outcome*. Perhaps this comment is made in relation to drawing number15-849nsw-01 which has the purpose of presenting information for the layout of the works by surveyors. Whilst the line of MHWM is included on that drawing, the necessary scale of the drawing makes it difficult to identify that particular line without masking other more important information. To address this issue, the sixteen drawings numbered 15-849nsw-03 to 18 all clearly show the position of the MHWM boundary, its location in relation to the proposed works and its location to existing land contours (**Appendix C**). Upon review, we cannot see how this could be made any clearer. The information in the requested "table" referred to in the comments is included on the survey drawings by Newnham Karl Weir (**Appendix K**).

RHDHV is comfortable with the Crown Lands survey and set out, and can only assume that the MHWM is correctly positioned on the CES drawings.

Please add a note to the CES drawings set 15-849nsw giving the source for the MHWM.

With this note added, RHDHV happy to close out this comment.

11. Ref 8, 2.4, para 5, gives overtopping rates for varied damage behaviour behind a seawall, but no overtopping rates are reported for the developed site. ***Outcome:*** *Design overtopping rates so overtopping impacts cannot be suitably assessed or a management strategy developed as there is insufficient information.*	Comment from CES: A critical consideration of the design was the determination of overtopping rates along sections of the proposed wall. For the section of the revetment having the crest level of RL+3.0m AHD, the overtopping rate is dependent upon the incident wave period, but is in excess of 0.4m³/sec/m. This indicates scour of the material behind the structure will occur. Since this will occur in the 20-metre (E2) buffer to infrastructure, this is deemed by the design process as an acceptable outcome (since the damage can

The words "not provided" were omitted in the original RHDHV Outcome comment, between the words "Design overtopping rates" and "so overtopping impacts…."

As noted at Item 4, design wave overtopping in excess of 400L/s per m is predicted for the section of revetment having a crest level of 3.0m AHD. This is a very significant overtopping rate. RHDHV is concerned for the sustainability of the landscaped zone immediately landward of the crest of the revetment in this zone. The capacity and potential safety

	be repaired) provided the crest armour does not suffer excessive damage or fail. The placement of rocks greater than 3 tonnes as buttress rocks to the rear of the crest is intended to achieve this performance outcome. For the section of revetment with a crest level at RL+2.0m AHD, the structure will be submerged by approximately 1 metre during the 100 year ARI storm tide / wave event, as well as by the 100 year ARI flood. The design of the armoured slope is such that it will remain structurally intact. It is intended to cause waves approaching and passing over the structure to break within the 20-metre wide buffer of the E2 zone. This is why 2 layers of 0.1 tonne rock armour is extended as a buried scour apron across the entire E2 zone on this type of wall. The extensive filling to otherwise raise the wall to mitigate this wave overtopping was deemed by the design process as an unacceptable modification to existing creek banks and the E2 zone.	implications for severe wave overtopping to carry across the 20m E2 buffer (ie, extend 12m behind the seawall crest) and impact on the new subdivided foreshore properties should be investigated and a remedial strategy proposed. To gain an appreciation of the scale of the overtopping being discussed here, the reader's attention is drawn to the video of wave overtopping at Fairy Bower recorded by WRL on 5/6/16 and presented by B Modra to the NSW Coastal Conference in Coffs Harbour in November 2016. RHDHV understands that the estimated wave overtopping rate in the event was less 10% of the minimum overtopping rate predicted by WT for the proposed Arrawarra seawall (3.0m AHD crest portion). RHDHV has a copy of this video record which can be provided if required.
12. Not used – draft comment in relation to AS 4997 Guidelines for Design of Maritime Structures is not valid since rock armoured walls are expressly excluded from the Standard (refer Clause 1.1 – Scope) ***Outcome:*** Not used	Comment from CES: We are aware of, (and acknowledge the comments regarding) applicability of AS 4997 to rock-armoured structures. The reference was simply made to more fully inform the discussion concerning selecting an appropriate design life for the revetment.	Comment closed out.
13. Not used – draft comment in relation to AS 4997 Guidelines for Design of Maritime Structures is not valid since rock armoured walls are expressly excluded from the Standard (refer Clause 1.1 – Scope) ***Outcome:*** Not used	As above.	Comment closed out.

14. Seawalls in NSW are often designed to be stable over their working lives against 50 to 100 year ARI storm events. RHDHV considers the 2 scenario approach by CES to consider combinations of 100 year and 50 year ARI waves and storm tides (and vice-versa), with mean wave periods between 7 and 14 s, as reasonable. ***Outcome:*** Comment Noted. No response required.	Comment closed out.
15. In Ref 2, 2c, the Coastal Panel has sought confirmation from the Proponent that the design of the seawall is "fit for purpose". Please be aware that this requirement may not be covered by the professional indemnity insurance policies of the Proponent's professional engineer advisors. In our experience offering such a warranty goes beyond the industry standard and is not insurable. The standard practice is that the guarantee in respect of engineering services should be to comply with a professional standard of care. The common law obligation for the performance of services is that of reasonable skill, care, diligence and that sound professional principles are applied. ***Outcome:*** Comment Noted. Refer to **Section 2.5** Fit for Purpose.	Comment closed out.

16. *Ref 8, 3.4.2, has selected a 130m wide scoured entrance channel with bed level +0.2 m AHD for assessment of design wave penetration to the site. We are concerned that +0.2 m AHD based on a single survey in April 2003 and understood to represent "average natural conditions for the purposes of the (1 in 100 year) flood study" (Umwelt, 2016) may not capture a suitable case for design wave penetration. To come to this view we have examined available water level data in coastal creek systems of comparable catchment size to Arrawarra Creek.* Comment from CES: We do not believe that the methodology suggested (of using recorded water levels at other estuaries) to infer the scoured width and level of the entrance berm of Arrawarra Creek as a consequence of the 100 year ARI event is any more robust or accurate that that utilised for the design of the revetment. The entrance arrangement at Arrawarra Beach will be determined by the combined action of storm tide and extreme waves during such an oceanic storm event – not by any scour of flows exiting the estuary during a 100 year ARI Flood event. Additional modelling was carried out by Umwelt (Australia) Pty Ltd to determine flow velocity within the creek environment and the potential for offsite	Given the uncertainty associated with the matter of the saddle level, RHDHV remains unconvinced that +0.2m AHD is a suitably conservative saddle level for assessment of design storm wave penetration to the site. We accept that Werri Lagoon may have morphological differences to Arrawarra Creek that limits its consideration here, and on that basis is happy to discard the site for the comparative assessment. No comment however is provided by WT in regard to the other creeks. Without more detailed assessment by WT it is suggested that -0.1m AHD (Back Lagoon) is a more appropriate (precautionary) parameter to adopt, 300 mm lower than the saddle level used to date for the Arrawarra seawall design.

Unfortunately water level data is not collected at Arrawarra Creek and examining the behaviour of similar systems is a reasonable approach. *The Estuaries Inventory of NSW (PWD, 1992) lists 91 NSW estuaries and characterises these according to parameters including catchment area. Arrawarra Creek is reported to have a catchment area of 20 km^2 which places it number 86 in order of reducing catchment size. Other systems of comparable size and which we understand are not trained include Saltwater Creek* *(30 km^2), Werri Lagoon (24 km^2), Back Lagoon, Merimbula (23 km2), Belongil Creek (18 km^2), Lake Cakora (11 km2) and Lake Arragan (10 km2). Of these six estuaries, MHL records water level data at Saltwater Creek, Werri Lagoon and Back Lagoon. The full water level records for these three lagoons are shown in Appendix A. Of particular interest, and the selected record for this is also shown, is the minimum water level at which tidal penetration occurs as this would represent the maximum bed level across the entrance saddle for the case of ocean water penetration, ie the saddle level could have been no higher, but was probably lower. Also included is a record of the water levels through the recent June 2016 storm event.* *We can see that for these three systems of comparable size to Arrawarra Creek, this limiting saddle level was approximately -0.4 m AHD for Werri Lagoon, -0.1 m AHD for Back Lagoon and 0.45 m AHD for Saltwater Lagoon.* *Having regard therefore to the information before us, RHDHV would be suggesting that a saddle level of -0.4 m AHD should therefore represent* ***a maximum entrance saddle level*** *for*	impacts. Umwelt prepared a 2D finite element mesh roughness value model to determine flow velocity under existing landform versus with the proposed revetment. The results are that there is essentially no change in flow velocity (**Appendix U**). RHDHV comment states an entrance berm level comparable to Werri Lagoon entrance should be used. We disagree, and note that (unlike the estuary entrance of Arrawarra Creek) the entrance to Werri Lagoon is constrained by a rocky headland and has only around 75m maximum width. Irrespective of the concerns expressed above as to applying the suggested methodology for determining entrance conditions during a 100 year ARI storm tide/wave event, the narrower and constrained entrance at Werri Lagoon results in deeper channel flows during floods than at Arrawarra Beach. So in our view it is not appropriate to simply adopt the inferred entrance berm levels at Werri Lagoon as being those scoured by a 100 year ARI storm tide & wave event at Arrawarra Beach.	

the calculation of design storm wave penetration. This is 0.6 m below the +0.2 m AHD saddle level adopted by the proponent which could be expected to lead to significantly higher wave penetration. **Outcome:** *Insufficient justification is given regarding the selection of +0.2 m AHD as an appropriate entrance saddle level for design wave penetration. With water level data not collected for Arrawarra Creek, inspection of water level records for comparable NSW estuary systems indicates that a saddle level at least 0.6 m lower should reasonably apply.*	
17. Ref 8, 3.4.2, states that "wave climate at the revetment during severe storms is depth-limited". This would seem to be reasonable. **Outcome:** Comment	Noted No response required.
18. Ref 8, 3.4.2, lists in Table 6 design significant breaking wave heights at the estuary entrance for 1 and 6 hour duration 100 year ARI storms, for mean wave periods between 7 and 14 s, and for 100 and 50 year ARI storm tide levels (32 wave heights listed). The design wave heights range from 1.9 to 2.4 m, with average 2.2 m. While this approach would seem to be reasonable, the wave heights are likely to be low given the reduction in the entrance saddle bed level (see Item 16). **Outcome:** Comment	Comment from CES: This review comment is based on that made in Item 16. We disagree with the underlying premise expressed by that earlier comment; and hence maintain that the wave characteristics listed in Table 6 of the Revetment Design Report are appropriate.

Comment closed out.	
Our comment above at Item 16 refers.	

19. *Ref 8 uses van der Meer (1988) to calculate armour rock sizes for a 100 year ARI storm event, finding that a 1 tonne (T) primary armour with minimum rock density 2.65 T/m3 placed to a slope of 1:1.5 meets the requirements of van de Meer for 5% damage. RHDHV is comfortable with the assumptions and general calculation approach, however we find that the rock masses reported in Table 7 range between approximately one half to one third that specified using van de Meer (1988). We have applied Table VI-5-23 in Coastal Engineering Manual to complete our calculation checks, making reasonable assumptions of porosity (P=0.45), relative eroded area or damage (S=2.5) and applying the wave height modification for depth-limited waves (H=H2% /1.4). We have also applied Hudson (SPM, 1984) to cross-check our assessment.* *Provisional calculations by RHDHV indicate that providing for primary armour comprising at least 2 layers of 2T median rock (all other parameters unchanged) should provide for suitable seawall slope protection against a 100 year ARI storm event occurring today.* ***Outcome:*** *While RHDHV is comfortable with van der Meer (1988) to calculate armour rock sizes, we are concerned that the specified 1.0 T primary armour is one half to one third the mass required by that assessment method, although this makes no allowance for the additional rock size required to accommodate larger wave penetration due to increased saddle water depths. Rock armour calculations to demonstrate compliance with van der Meer (1988) are not provided.*	Comment from CES: Refer to sample calculations indicating two layers of 1 tonne primary armour (**Appendix S**). Calculations refer to equation/page numbers in CIRIA (2007).

RHDHV made its assessment of vd Meer using CEM (2002+). It appears that simplifications are introduced into the CEM assessment methodology for shallow water waves which are more thoroughly treated in CIRIA (2007). We have checked the calculation undertaken by WT using CIRIA (2007) and obtain similar results. Accordingly, RHDHV is prepared to accept the rock mass values presented in Table 7 of WT Rock Armoured Revetment report (30/6/16).

In regard to the extra layer of 1T rock to account for climate change effects to 2100 (calc at p2 of App S), RHDHV accepts this approach, however for the proposed seawall it is recommended that the 3 layers of armour be placed as part of the initial construction of the seawall. This would seem to be an appropriate measure for a seawall structure which is not government-owned and which is in line with the “prudent” approach described in Section 3.5 para 4 of WT s report (30/6/16).

Placing an extra layer of armour rock may require that the seawall is setback further from the MHWM boundary. It is noted that the current design shows the 3T toe rock set back 1.5 m from the seaward property boundary.

To close out on this comment, there is the matter of saddle level which for us requires further examination and confirmation. Based on the information provided by WT, RHDHV is not satisfied that a saddle level of 0.2 m AHD is suitably low to assess design storm wave penetration to the seawall site (Item 16 refers).

20. *Ref 8, 3.56.2, last para, states that for 2100 with 0.8 m SLR, the breaking significant wave height at the revetment would be 20- 25% higher than the breaking wave heights in Table 6. Ref 8, 4.2.2, 2nd para, states that when 0.8 m depth added to the design storm tide and wave heights are increased accordingly, then one additional layer of primary armour rocks would accommodate the additional wave loads.* *Provisional calculations by RHDHV indicate that providing for an additional 2 layers of 2T median rock (all other parameters unchanged) should provide for suitable seawall slope protection to the end of a structure life at 2100.* *The above assessment is based on the wave climate assessment assuming the +0.2 m AHD entrance saddle level. The additional water depth across the entrance saddle (Item 16) would lead to larger wave penetration and further increase the rock size in the revetment.* ***Outcome:*** *RHDHV notes that the Coastal Panel is receptive to an adaptive approach to increase the capacity of the seawall over its life (Ref 4, last bullet point). However, RHDHV is concerned that the adaptive modification proposed in Ref 8 may not suffice to provide suitable protection to the end of structure life. Suitably detailed calculations are not provided which demonstrate the acceptability of the proposed adaptive approach.* *Increasing the size of the additional (adaptive) rock layers, considered to be necessary by RHDHV, would require the seawall to be initially located further landward from the existing MHWM property boundary than is currently proposed.*	Comment from CES: Refer to sample calculations indicating three layers of 1 tonne primary armour (**Appendix S**). Calculations refer to equation/page numbers in CIRIA (2007).

RHDHV accepts the CIRIA (2007) calculation presented at page 2 of Appendix S.

This comment would be closed out when Item 19 is closed out.

21. *Ref 8, Figure 9 seems to show the NE end of the seawall terminating some 80 m short of the end termination point shown in the design drawings.* **Outcome:** *Certificate required*	The drawings show the accurate set-out of the proposed works and are drawn to scale. The figure in the report shows indicative extents only. It is noted that the drawings are for DA purposes. Technical drawings would be provided with the CC documentation.

RHDHV accepts the explanation. Comment may be closed out.

22. *The seawall proposal assumes design scour levels at the toe of the wall ranging between -1.0 and -0.5 m AHD, based on this meeting a level equal to one median rock diameter below the local channel thalweg. The actual location of the thalweg was not reported.* *RHDHV is concerned that this approach may miss deeper localised scour that could occur during high creek flows, particularly during periods of low tail water levels. We have recently modelled, using MIKE21, creek outflows at another North Coast creek system where a design scour level of -1.5 m AHD was identified along a straight section of the protected creek bank, and locally deepened to -2m AHD and deeper at downstream spur walls.* *Seven Google earth photos at Arrawarra Creek between 2004 and 2013 show the channel hard up against the gabion wall in the vicinity of SOP 22 and 23 for 4 of the 7 photo dates. It would be of interest to know what the minimum channel bed level may have been against the gabion wall during this period.* *Furthermore, if there is evidence of the gabion wall having settled in this area, it would be important to know the design toe level for the gabion wall and the existing settled toe level. Note that it is possible for a gabion wall to bridge a localised scour hole although this should be evident by loss of backfill.* ***Outcome:*** *To adequately define potential scour:* *(i) Information is required on the minimum channel bed level against the gabion wall in the vicinity of SOP 18 through SOP 29, including a review of the gabion toe level (design and settled)* *(ii) Consideration should be given to extending the existing flooding*	CES response to the issue is raised as review comment #3 above (regarding the gabion wall) and also relates to this comment #22. Additional modelling was carried out by Umwelt (Australia) Pty Ltd to determine flow velocity within the creek environment and the potential for offsite impacts. Umwelt prepared a 2D finite element mesh roughness value model to determine flow velocity under existing landform versus with the proposed revetment. The results are that there is essentially no change in flow velocity (**Appendix U**).	RHDHV notes the results of the additional modelling carried out by Umwelt. Having regard to the further information provided here and at Item 23, this comment may be closed out.

assessment to model channel velocities and scour along the toe of the rock revetment.		
23. *It appears from the information provided that a geotechnical assessment has not been undertaken to investigate the ground conditions in the vicinity of the proposed seawall. Valuable information could be obtained from a geotechnical site investigation to characterise the subsurface conditions. The work could be expected to include a site walk over by an experienced geotechnical engineer, an assessment of the regional geotechnical context from geological mapping, and intrusive investigations. Techniques might include test pits along the footprint of the proposed seawall including along the toe of the gabion wall, boreholes, CPTs, and DCPs. A suitably designed geotechnical investigation will provide important baseline information to assist develop an appropriate toe detail for the seawall and provide design certainty.* ***Outcome:*** *No geotechnical investigation would appear to be included in the background briefing information. To properly inform the seawall toe design it would be highly desirable to conduct a suitably scoped geotechnical investigation.*	Comment from CES: The approach taken for the development of a design up to DA stage has been to develop what is considered to be a conservative arrangement at the toe. The review comments seem to take no account of the role that 3 tonne minimum-sized buttress rocks at the base of the wall plays in self-armouring the foundation. A Geotechnical Assessment is provided at **Appendix T.**	RHDHV has reviewed the geotechnical investigation prepared by de Groot & Benson (2016 update), in particular the borehole and DCP logs. Having regard to this information, the incidence of stiff clay at around -0.5m AHD over the most exposed portion of the seawall, and the position and performance of the existing gabion wall structure, RHDHV is comfortable with the adopted toe design. This comment may be closed out.

24. *Item 11 refers. Ref 8, 4.4.1, overtopping assessment. Ref 8 refers to overtopping calculation that show discharges will scour any unprotected erodible materials in the area immediately behind the revetment, but no overtopping rates are reported.* ***Outcome:*** *Overtopping quantities are not provided to enable a suitably thorough assessment of overtopping impacts and management.*	Refer response to review comment #11.	Refer response to RHDHV review comment at Item 11.
25. *Seawall crest levels of 3.0 and 2.0 m AHD seem low for a site that can experience storm tide levels of 2.9 to 3.0 m AHD in a design storm plus breaking wave heights of 2 m or more. If wave heights can increase by around 20 - 25% in a 100 year storm at 2100, this could have a significant impact on wave overtopping.* ***Outcome:*** *Detailed information is lacking on the adaptive response to deal with what appear to be low seawall crest levels, particularly given the predicted 20-25% increase in design wave heights by year 2100.*	Refer response to review comment #11. The adaptive response to extend the service life of the structure to the year 2100 is to simply add another layer of buried rocks within the 20 m wide E2 zone.	To clarify the response made by WT, the additional layer of rocks would extend approximately 12 m behind the crest of the seawall. This approach is acceptable to RHDHV for the low crest level (2.0m AHD) section of seawall to be situated in Arrawarra Creek which is relatively protected from ocean wave penetration. The strategy for addressing the impacts from the predicted 400 L/s per m wave overtopping potentially affecting the remainder of the seawall (crest level 3.0 m AHD) needs to be addressed (Item 11 refers).
26. Design drawing 15-849NSW-02 B lists design parameters. Wave height H is given as 2.0 m with T = 7-12 s. Notwithstanding the issue of inadequate saddle depth and consequent larger wave penetration, it would be more accurate for H = 1.9 to 2.3 m to be added to the drawing to better reflect Table 6 in Ref 8. ***Outcome:*** Design drawing 15-849NSW- 02 B does not fully describe the design wave height	This issue is not considered critical in the determination of the DA. If necessary, any required adjustments to seawall design will be made to the final construction drawing set.	RHDHV accepts the response here. This comment may be closed out.

RHDHV incorrectly states that: *An existing footpath runs along parts of the eastern boundary leading to the footbridge. This footpath would need to be demolished to construct the rock revetment. The proposal does not appear to provide for the reconstruction of this footpath, although it is unlikely that this footpath, being located within the caravan park, would have been accessible to the general public in any case.*	
 **Nearmap May 2016** As shown in the aerial image, there is not an existing footpath along the eastern boundary leading to the wooden footbridge. The footbridge can only be accessed via the caravan park.	Correction noted. No further response required for this matter.
RHDHV states that: *There is a public footpath that leads to Arrawarra Creek located along the western boundary of the site off Arrawarra Beach Road. From this path, the public must cross the creek to get to the beach. This access should not be affected by the proposed revetment.*	

The photos below show the only public footpath to the beach, located adjacent to the western boundary of the subject land.  Right of Footway (ROF) 1 wide created by DP 789002 located along the western boundary of Lot 12 DP 835612. This ROF may be extinguished, with the consent of Council, and replaced with superior right of public footway via the proposed community title accessway and footpath to meet with the footbridge, provided safe and legible access to the beach. Under the terms of the footbridge licence, the structure may be repaired or rebuilt. Obviously, modifications to the footbridge will be required to accommodate the revetment.	Noted.

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Existing public ROF, narrow, does not meet Council specifications for public footpaths and is poorly maintained.

Under the existing public footpath arrangements, the public must traverse the creek to access the beach.
As stated previously, and clearly shown on the Landscape Plan (Appendix P), a public footpath will be created within the community land providing legal, safe public access to the foreshore and the footbridge via a 2 m wide concrete footpath within a 3 m wide easement.
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<td>Noted.</td>
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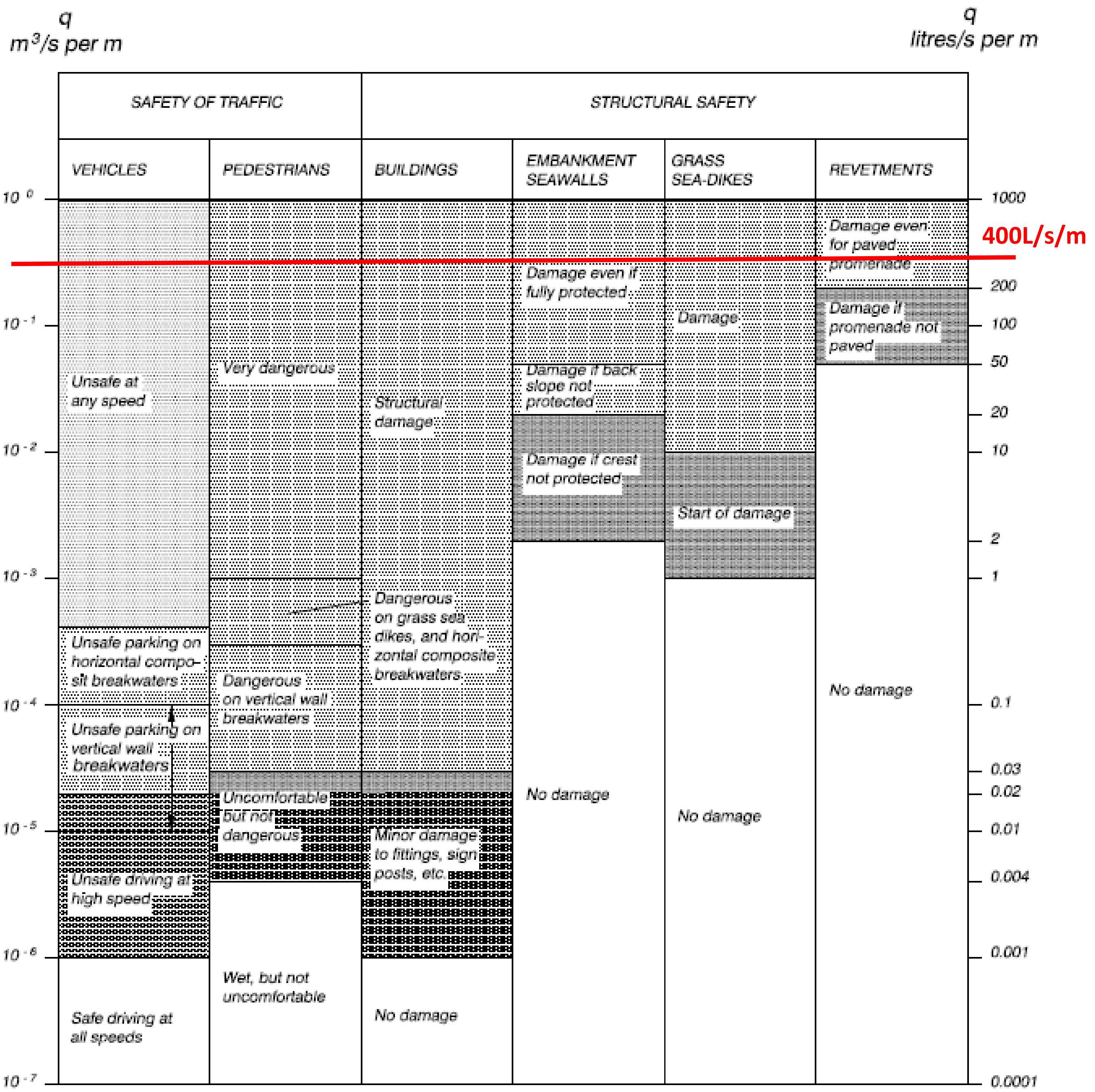
APPENDIX B

CRITICAL VALUES OF WAVE OVERTOPPING DISCHARGES

Table VI-5-6
Critical Values of Average Overtopping Discharges



Source: CEM (2002+)