DESIGN – TECHNICAL NOTE

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1 Aim

JBA have been commissioned by Aberdeenshire Council to develop a number of concept design options to prevent the propagation of waves up the channel of the River Carron in Stonehaven, Aberdeen. This technical note covers the design assumptions, decision making process and methodology for the concept design of a detached breakwater.

The scope of works does not include a formal options appraisal process. The option proposed has been developed based on technical feasibility, engineering judgement, cost and consideration of the long term vision and key criteria determined by the client.

2 Assumptions

The following assumptions have been used during the development of the concept design.

2.1 Existing training wall arrangement

The River Carron drains through the southern extent of Stonehaven beach. The river was stabilised with rock armour in the past to prevent the migration of the channel and reduce wave transmission into the river. However, for the existing channel layout, waves have been observed in the river during the period of high tide, propagating into the channel from the sea.This concept design seeks to reduce this wave propagation by constructing a nearshore detached breakwater in Stonehaven Bay.

The existing training walls direct the channel east, with the river mouth opening out towards the dominant wave direction. The walls have been constructed out of rock armour and have a crest level of 3mAOD. The existing defences have been provided in Figure 2.1.

Figure 2-1: Plan layout (above), Photo 1 looking southeast (top right) Photo 2 looking northwest (bottom right).

2.2 Design life and level of protection

The structure has been designed to achieve the following design standards:

- **Design life**: 100 years
- **Design storm event:** 1 in 200-year event

2.3 Climate change

Extreme wave conditions at the structure have been estimated using a numerical wave transformation model. The model has used the latest UK Climate Projections 09 (UKCP09) to incorporate an allowance for future climate change of the structures design life. Within UKCP09 estimates for sea level rise are provided under three emissions scenarios; low, medium and high. Within the three scenarios the estimate is further refined by percentile confidence ratings of 5%, 50% and 95%. In simple terms this should be interpreted as the relative likelihood of the projected change being at, or less than the given change. For this study, the medium emissions scenario for the 95th percentile was used for the expected sea level rise. This gives a projected sea level rise of **0.66m** for the year 2100.

The modelling has made no allowance for the increase in wave intensity due to climate change. However, as a result of sea level rise the greater water depths will allow larger waves to arrive at the coastline.

2.4 Design performance standards

The breakwater has been designed to meet two performance targets:

- An Ultimate Limit State (ULS) design standard for structural components of 200-year waves, water levels and climate change estimates occurring simultaneously,
- A Serviceability Limit State (SLS) design standard to achieve a reduction in the in-channel wave height below the River Carron FAS design wall height.

2.4.1 Ultimate limit state standard

The breakwater has been designed to achieve structural stability during an ULS scenario, defined as the 1 in 200-year wave height coinciding with a 1 in 200-year Still Water Level (SWL), including an allowance for climate change factored (refer to Section 2.3). The overall likelihood of an event of this magnitude occurring will have a probability greater than 200-years, incorporating a preliminary level of safety into the critical design elements. This has ensured any structures will withstand wave conditions with an extremely low recurrence interval.

2.4.2 Serviceability Limit State standard

A key performance driver for the breakwater is to reduce the wave propagation within the River Carron channel to an acceptable limit. Numerical modelling has been used to optimise the breakwater design to ensure the maximum reduction of wave height in the channel, whilst limiting the defence footprint (and therefore cost). For more information on this process and methodology, please refer to the River Carron Rock Armour Study, Draft Report.

Unlike typical breakwater designs, this structure has not been required to address typical wave overtopping performance standards during an extreme (e.g. 200-year) coastal event. Instead, it has been designed based on a 200-year joint probability fluvial-dominated event where 200-year fluvial flows coincide with 1-year coastal waves and water levels. During this event the structure must achieve a reduction in the in-channel wave height to an acceptable level below the River Carron FAS design wall height

2.5 Ground conditions

No geotechnical or ground condition information has been made available as part of this study. Therefore, all designs of the defence structure have been progressed assuming poor ground conditions e.g. low bearing capacity. This provides a conservative approach to the development of the concept design. The levels presented in the all drawings represent finished defence levels, so some consideration of the settlement during construction should be taken into account during detailed design.

2.6 Services information

No detailed services information was provided as part of this study and a services search is not included within the scope of works. If the project progresses to outline and detailed design it will be essential that a full service plan is developed.

2.7 Structural design

A full structural design has not been included within this study as the scope of works did not include geotechnical investigation or analysis. All designs have been reviewed by a chartered civil engineer to confirm that the design principles adopted are acceptable.

2.8 Environment and landscape

This commission does not include any formal Environmental Impact Assessment or Landscape Visual Impact Assessment. If the project progresses to a detailed design phase, a more in depth study of the environmental impacts is recommended. This is particularly important regarding the location of this proposed option, with the structure designed to be constructed on top of the existing rocky foreshore, containing a number of unique natural habitats for intertidal flora and fauna. The environmental implications of founding the structure on the rock reef should be investigated further should this option be progressed.

2.9 Reinstatement and finish details

The development of landscape and architectural enhancements are outside the current project scope of works. It is assumed that following construction the surrounding area will be re-instated to a condition similar to the present. However, during the detailed design stage further architectural and landscape enhancements could be considered.

3 Standards, guidance & reference documents

All design assumptions have been developed using the following reference material:

- JBA Consulting (2014), River Carron Rock Armour Assessment, Draft Report
- BS EN 6349-1-1:2013, Maritime works, General, Code of practise for planning and design
- CIRIA (2007), The Rock Manual: The Use of Rock In Hydraulic Engineering (second edition)
- CIRIA (2010), The Beach Management Manual (second edition)
- DEFRA (2009) UK Climate Projections 09
- HR Wallingford (2007), EurOtop, Wave Overtopping of Sea Defences and Related Structures: Assessment Manual
- McConnell, K (1998), Revetment Systems Against Wave Attack – A design manual
- BS EN 13383-1:2002 Armourstone – Part 1: Specification
- BS EN 13383-2:2002 Armourstone – Part 2: Test methods
- Environment Agency (2013), Toe Structures Management Manual. Document Reference SC070056/R. Bristol: Environment Agency
- GEOfabrics Coastal and River Defence System Design Guidance.

4 Design development

The following sub-sections provide a brief summary of how the key design elements were selected.

4.1 General form of defence

The 50m long rock armour breakwater has been designed to be constructed perpendicular to the dominant wave direction to prevent waves from entering the channel directly. The function of the structure will be to allow the waves to break on the structure itself, and therefore prevent unbroken wave trains reaching the river mouth.

The defence takes the form of a 3-6tonne armourstone breakwater formed in a double interlocking layer. The breakwater has a 1:2 sloped front face and a 1:1.5 sloped rear face with a 3.6m wide crest with an elevation of at least 4.7mAOD. On the rear side of the embankment, the slope angle has been increased to 1:1.5 to reduce total material quantities.

4.2 Defence planform and length

Both the defence planform and length have been optimised through an iterative process. In this way, the distance offshore, the angle and the length of the breakwater was varied to identify the layout that offered the best reduction in nearshore energy. For more details on this process, please refer to River Carron Rock Armour Study, Draft Report.

A suite of breakwater positions, lengths and orientations were tested using a numerical model to assess their effectiveness. Breakwaters positioned nearer to the coastline were found to result in the greatest reduction of in-channel wave height. However, the final chosen breakwater location was selected by considering other more practical aspects such as:

- Ease of construction
- Volume of required materials (and so cost)
- Health and safety
- Engineering judgement

The selected breakwater location considers these four practical issues to provide the maximum reduction in in-channel wave height whilst ensuring the design is feasible with both the site constraints and project budget in mind.

4.2.1 Tombolos and salient

In addition, the position of the detached breakwater has been considered to limit the impacts of the breakwater on the sediment transport processes. Detached breakwaters typically cause a reduction of nearshore wave energy in the lee of the structure which increases deposition of fine to medium sediment sizes. This transport of material can occur to such an extent that a salient (depositional feature) or tombolo (a connection between the beach and the breakwater) is formed. Design guidance (CIRIA, 2007) suggests that a tombolo will be formed where:

$$tombolo\ \frac{l}{s} > 2$$

$$salient\ \frac{l}{s} < 2$$

Where l is the length of the breakwater and s is the distance offshore. A tombolo will have a significant control on sediment transport rates, preventing material being transported downdrift. To limit the disturbance on the existing sediment regime, the detached breakwater has been designed to be a minimum of 50m from the beach toe. With the breakwater being 50m in length, $l/s = 1$ therefore reducing the potential for a tombolo to form.

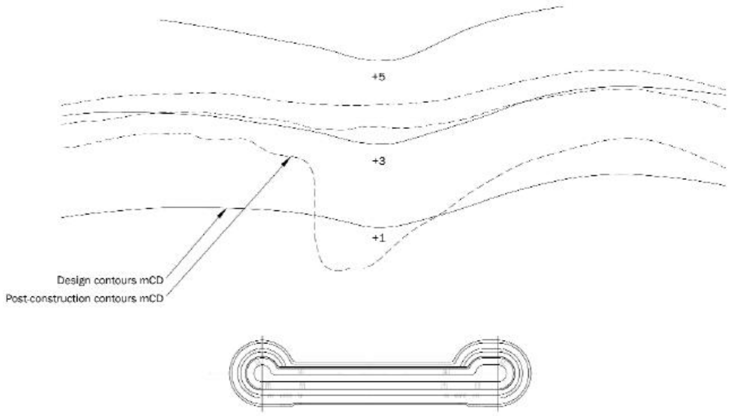

Figure 15.18 *Beach response after one year (solid line shows design profile contours, dashed line shows actual beach contours +1 year)*

Figure 4-1: Example salient formation in the lee of a detached breakwater at Newbiggin Bay (CIRIA, 2010)

A salient is likely to be formed in the lee of the structure due to the lower energy environment created. A typical salient formation is shown in Figure 4-1 at Newbiggin Bay, Northumberland. It is possible that this deposition may block the River Carron channel or cause a southerly migration of the river.

However, the likely behaviour of the beach is an inherently natural and stochastic process which is difficult to predict, being dependent on a large number of site specific variables. This simplified rule is suitable for the purposes of concept design but would require detailed physical and numerical modelling during detailed design, to better quantify the processes.

On the other hand, the development of a tombolo or salient may provide benefits to the coastline which should be investigated further should this option be taken forward.

4.3 Defence crest level

As the brewakwater is not required to limit overtopping during a coastal dominated 200-year event, the defence crest level has been designed to limit the possibility of an unbroken wave travelling over the breakwater and entering the channel. In this way, the new structure has been designed to disrupt and break a wave train travelling into the coast, but will allow some turbulent wave energy to enter the mouth. This allows the defence crest level to be optimised, to reduce the total height of the structure and so the cost. The crest has therefore been designed as follows:

$$Defence\ crest\ level = SWL + \frac{1}{2}Hs$$

Where *SWL* = Still Water Level and *Hs* = wave height calculated as the maximum total elevation from joint probability analysis of the design storm event. The joint probability analysis for Stonehaven is provided in Table 4-1.

Table 4-1: Joint probability analysis for defence crest level

Joint probability analysis of 1:200-year event			Design crest level (mAOD)
SWL (mAOD)	**Hs (m)**	**1/2 Hs**	**SWL +1/2H**
3.16	2.05	1.06	4.19
3.24	2.08	1.04	4.28
3.33	2.12	1.06	4.39
3.40	2.13	1.06	4.46
3.47	2.13	1.06	4.53
3.56	2.13	1.06	4.62
3.64	2.10	1.05	4.69
3.70	2.02	1.01	4.71
3.79	**1.88**	**0.94**	**4.73**
3.86	1.67	0.84	4.70
3.92	1.46	0.73	4.65

The defence crest level has therefore been set at 4.7mAOD reflecting the highest combination of wave amplitude and still water level from the joint probability analysis. While this crest height will allow some wave energy to be transmitted through and over the structure, it is considered to represent a conservative method to reduce overall costs of the new defence. The wave transmission through and over the structure should be further investigated during detailed design, possibly with the incorporation of a physical model to better quantify the reduction in nearshore wave energy.

4.4 Rock armour sizing

The rock armour has been sized using the ULS, or the upper limit for the structural stability of the proposed defence components. This limit state has been used to ensure the rock armour units will withstand 200-year wave conditions in combination with 200-year extreme sea-levels, including the effects of climate change to the year 2115. The overall likelihood of an event of this magnitude occurring will have a probability greater than 200-years, incorporating a preliminary level of safety into the critical design elements. This has ensured any structures will withstand wave conditions with an extremely low recurrence interval.

- Rock armour sizing calculation used: Van der Meer & Hudson
- Significant wave height (H_s): 1 in 200-year ultimate limit state wave height plus climate change = 2.32m
- Period (T_m): 1 in 200 year plus climate change wave period = 7.52s
- Still Water Level (SWL): 1in 200-year water level including climate change = 3.92mAOD
- Permeability Factor (P). HR Wallingford (1998) Revetment systems against wave attack - A design manual (page 89). The proposed structure will be formed with a minimum of 2 layers of armourstone placed directly on an impermeable core, therefore P = 0.1 has been selected as an appropriate permeability factor.
- Slope Angle (α): 1 in 2.5 (22°) has been selected to represent a stable slope while attempting to limit the total required volume of rock
- Damage Number (S_d). HR Wallingford Revetment Systems Against Wave Attack - A Design Manual (page 89) states: *'For most cases, design damage is set at S_d=2 as equivalent to the "no damage" limit'*.
- Storm duration was set at 4 hours to cover two hours before and after high tide (The sensitivity of this was varied and was considered to be the worst case while reflecting a typical tidal curve over the storm event).

Using the above input parameters within the Van der Meer calculation and assuming an igneous rock source provides a median required rock mass (M_{50}) = **2.85t**. Therefore adopting a conservative approach and selecting a standard rock grading above the median predicted rock size results in a standard rock grading of **3 - 6 tonnes** being selected for the armourstone. Figure 4.1 shows that less than 10% of the rock in this grading will be smaller than the required M_{50} and so represents a suitable material grade for the environment. The revetment, therefore has a designed median required rock mass ($M_{50,des}$) = **4.50t** and a designed median required rock diameter ($D_{n50,des}$) = **1.19m**.

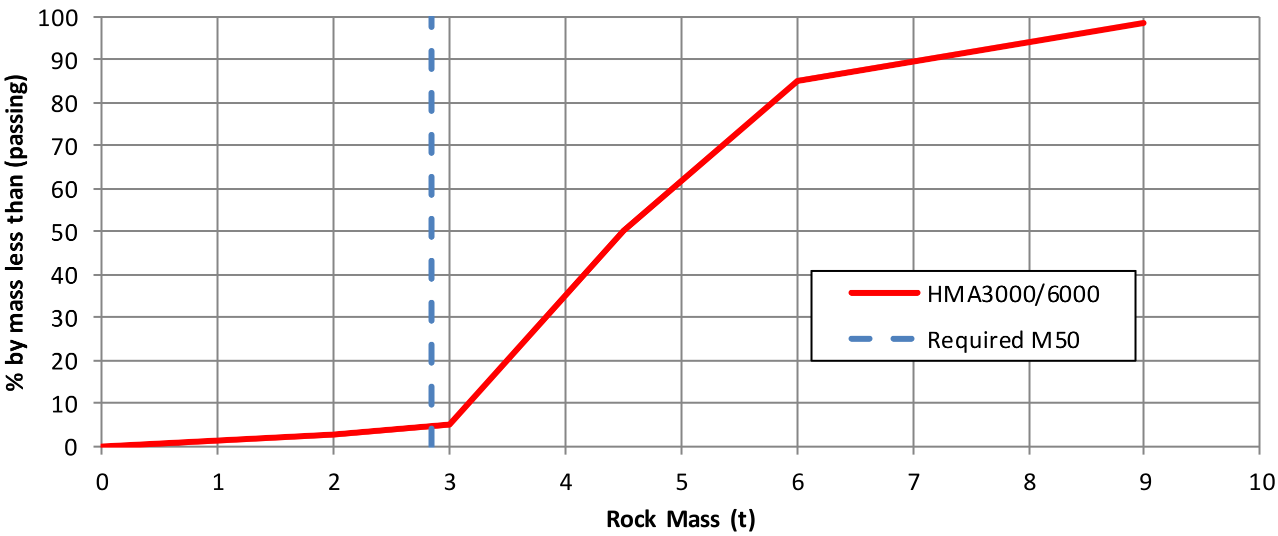

Figure 4-2: Grading curve for HMA 3000/6000 and the required M50

4.5 Defence crest width

The defence crest width is 3.6m calculated to allow three rocks to form the training wall crest.

4.6 Secondary armour units and breakwater core layers

In an attempt to reduce total material requirements, the design specifies the use of only one primary material type and size and that the breakwater should be placed directly on top of the rock foreshore. The breakwater core should be constructed of suitably graded quarry run.

4.7 Packing density

The packing density of the primary armour layer has a direct impact on the performance of the structure as well as the total volume occupied, so is essential to consider during concept design. The following assumptions have been made in calculating the packing density and total required volumes and weight of the rock outlined above:

- Packing density calculation used - CIRIA -The Rock Manual - The Use of Rock in Hydraulic Engineering (page 124).
- Chainage of total length to protect ~82m
- Volumetric layer porosity (n_v). CIRIA - The Rock Manual - The Use of Rock in Hydraulic Engineering (page 126) states that: *'for a double layer of irregular rock placed in standard packing, a value of 32% should be used for the volumetric layer porosity'.*
- Layer thickness coefficient (k_t). CIRIA - The Rock Manual - The Use of Rock in Hydraulic Engineering (page 126) states that: *'for a double layer of irregular rock placed in standard packing, a coefficient of 0.87 should be used for the layer thickness coefficient'.*
- The rock is assumed to have a porosity (p) of 0.1.
- The degree of saturation (S_r). CIRIA - The Rock Manual - The Use of Rock in Hydraulic Engineering (page 97) states that: *'for armourstone not in permanent contact with water, a saturation of 0.25 should be used'.*

Using the above input parameters, the following packing densities and total required volumes have been calculated. In addition, the previous calculations have been summarised, to provide a lookup table for the rock sizes. It should be noted that the total required masses have been calculated based on a constant foundation depth of +5.0mOD which is known to vary considerably. Consequently, the actual required mass may be significantly higher or lower than the values calculated here. A more in depth analysis would be possible if geotechnical information was collected.

Armour Layer		
Primary	Required Median Mass ($M_{n50,a,req}$)	2.85t
	Designed Median Mass ($M_{n50,a,des}$)	3-6t (4.5t median)
	Required Median Diameter($D_{n50,a,req}$)	1.02m
	Designed Median Diameter ($D_{n50,a,des}$)	1.19m
	Thickness of Layer ($t_{d,a}$)	2.06m
	Packing Density ($\rho_{b,a}$)	1.77t/m^3
	Total Required Rock Mass ($W_{b,a}$)	**9,000t**

Table 4-2: Armour requirements

4.8 Structure toe and foundation level

The breakwater has been designed with a small rock armour toe, two rocks wide by two rocks deep, positioned on the seaward side of the structure. This serves a dual purpose of dissipating wave energy and adding toe weighting to increase the stability of the structure.

A large rocky foreshore is located at between 0.5 and -0.5mAOD in Stonehaven Bay. The breakwater has been designed to be constructed on top of the hard rocky foreshore to prevent issues associated with scour in front of the breakwater toe. The structure therefore has a conservative estimated foundation depth of -0.5mAOD.

Breakwaters constructed on rock foreshores can include a toe detail that specifies that some of the rock should be broken out to bed the toe below the hard substrate. This detail has not been included in this design, due to the relatively small total height of the structure. During detailed design, slope stability analysis will provide a better indication of whether this toe detail will be required.

4.9 Breakwater roundhead

The breakwater roundhead shows reduced stability when compared to the main trunk section, due to the large velocities and wave forces generated as a wave breaks over the roundhead. To maintain stability of the armour layer under the design limit state there are two options available; ether by increasing the armourstone mass or by reducing the slope.

The required armour unit mass was investigated for a 1:2 slope using the following design principles (Note:

1:2.5 was not available, due to the model being reliant on the outputs of physical modelling):

- Roundhead armour sizing calculation used: Carver and Heimbaugh (1989) and Hudson (1959)
- Significant wave height (H_s): 1 in 200 year ULS wave height plus climate change transformed to the structure toe = 2.32m
- Period (T_m): 1 in 200 year plus climate change wave period = 7.53s
- Still Water Level (SWL): 1in 200 year water level including climate change = 3.92mAOD
- Storm duration was set at four hours to cover two hours before and after high tide.

Using the above input parameters within the Carver and Heimbaugh calculation and assuming an igneous rock source showed that the 3-6 tonne armourstone grading would be stable on the breakwater roundhead for the 1:2.5 slope.

In addition, for armour units reliant on their mass for stability (as in armourstone), the roundhead should have a minimum radius of two times the design wave height at the design water level, in order to ensure stability. For this option, the breakwater roundhead should have a minimum radius of 4.63m. In order to meet this requirement, the crest radius has been doubled to lengthen the radius at the extreme SWL to 5.5m. These design formula should be used during concept design only. Three dimensional physical modelling should be undertaken during detailed design to investigate the stability of the armour on the roundhead section.

4.10 Adaptability

A rock structure has the benefit of flexibility in both plan layout and section. The structure itself has some natural flexibility and can adjust to fluctuations in the foundation profile, and can accommodate some reshaping following wave activity. It can also be easily modified in the future dependent on the type of coastal morphological evolution that occurs.

4.11 Public safety

The main public safety design issue relates to the future public interaction with the proposed structure. While this structure is situated away from the main public beach, the breakwater is still accessible during low tide. The public should be discouraged from climbing on the rock structure as there is a risk of injury. It is suggested that signage is used as means of warning.

For further information on all the risks considered, mitigated or reduced please refer to the Designers Hazard Inventory.

5 Technical risks summary

The following are considered to represent the key risks highlighted during the development of this concept design.

5.1 Unknown ground conditions

Due to the unknown ground conditions it is possible that the current design will require modification in order to achieve structural and geotechnical stability. To progress this option further, geotechnical information must be provided to assess the stability of the structure.

5.2 Beach morphological evolution

This study has not included any assessment of the likely evolution of the shingle beach south and north of the River Carron.

By placing a detached breakwater in Stonehaven Bay, there are likely to be significant changes in the sediment transport processes. This has not been quantified at this design stage but would require detailed numerical and physical modelling to assess the impacts. The chances of developing a tombolo (causing the greatest disturbance to the sediment transport processes) have been reduced by ensuring the detached breakwater is situated a significant distance offshore. However, there can be no certainty on the likely processes following construction of the breakwater. Detailed physical and numerical modelling will be required should this option be taken forward to assess the likely impacts.

In addition, the client should provide guidance on whether or not the opportunity to develop a salient or tombolo represents an attractive prospect. The advantages and disadvantages of the deposition of beach material in the lee of the structure are provided in the table below:

Table 5-1: Advantages and disadvantages of depositional feature in the lee of the breakwater

Advantage	Disadvantage
Amenity value of increased beach	Starvation of sediment for downdrift beaches
Greater protection against wave overtopping	Pathway for public access to breakwater (associated H&S risks)
Potentially better disruption of wave trains entering the Carron channel	Blockage of the River Carron channel which may cause a migration southwards and increase fluvial flood risk

5.3 Downtime

The location of the detached breakwater provides a considerable construction risk associated with downtime due to high sea levels and stormy conditions. The need to work at low levels may increase the price of works due to the associated risks of project delays and budget overrun.

5.4 Services

No services information has been provided as part of this study. If the project progresses to outline and detailed design it will be essential that a full service plan is developed.

5.5 Construction accessibility

Prior to the development of outline designs it would be advisable to appoint a construction contractor to provide constructability advice. Although the site is considered reasonably accessible it would be beneficial to confirm the proposed methods of construction and temporary works required.

5.6 Stakeholder requirements

No consideration of other stakeholder requirements was made at this stage. However, this should be addressed during detailed design.

5.7 Environmental impacts

No formal Environmental Impact Assessment was completed during this project stage. The environmental impacts of constructing on top of the environmentally sensitive rocky foreshore should be investigated in further depth should this option be taken forward to detailed design. This option may prove to be unfeasible if the necessary licenses and consents cannot be obtained, due to environmental impacts associated with this scheme.