DESIGN – TECHNICAL NOTE

JBA Project Code	2014s1358
Contract	Isle of Man Sea Defence Options
Client	Department of Infrastructure, Isle of Man Government
Day, Date and Time	26/08/2014
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Subject	Castletown Open Coast – Option COC3 – Rock Revetment

Project Title: Isle of Man Sea Defence Options			Sheet No: 1
Subject: Castletown Open Coast – Option COC3 – Rock Revetment			Calc No:
Job No: 2014s1358			Version:1.0
Developed By: Alec Dane	Date: 26/08/2014	Revised By: Alec Dane	Date: 04/12/2014
Checked By: Graham Kenn	Date: 03/12/2014	Approved By: Graham Kenn	Date: 04/12/2014

1 Aim

JBA have been commissioned by the Department of Infrastructure (DoI) to develop a number of technically viable solutions to address the still water level flooding in harbour environments and wave overtopping in open coast environments, at seven coastal sites across the Isle of Man.

This technical note covers the design assumptions, decision making process and methodology for the concept design of Option COC3, an option to place a rock revetment in front of the existing sea defences in Castletown, to reduce wave overtopping to the hinterland.

The scope of works does not include a formal options appraisal process. However a high level Multi Criteria Analysis will be undertaken with input from key stakeholders to help determine which option best satisfies the project criteria. The option proposed has been developed based on technical feasibility, engineering judgement, environmental impact, cost and consideration of the long term vision and key criteria determined by the project stakeholders.

2 Assumptions

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

2.1 Datum

All elevation and depth measurements presented in the conceptual design of defence options will be presented in Douglas02 datum.

2.2 Baseline conditions

The open coast defences at Castletown are frequently overtopped by waves during a storm event. In order to design an option that efficiently reduces the risk of wave overtopping damage to the hinterland, it is important to look at the baseline conditions.

Wave overtopping occurs where the waves run up the face of the coastal defence. Where this run up exceeds the defence crest level, water passes over the structure and inundates the land behind. This design option will therefore seek to reduce the volume of flood water travelling over the existing defences during a storm event.

2.2.1 Existing defence geometry

The existing defences are composed of a concrete sea wall fronted by a concrete stepped revetment. A small shingle beach fronts the stepped revetment, but offers little added protection against wave overtopping.

Attribute	Dimension
Sea wall crest level	6.86mD02
Sea wall toe level	2.86mD02
Sea wall height	4.0m

Table 2-1: Existing defence geometry summary

2.2.2 Current wave overtopping risk

Based on baseline modelling of the existing defences, Castletown promenade is currently offered a 1 in 100-year level of protection against wave overtopping. However, by including an allowance for climate change up to the year 2115, that standard of protection reduces to less than a 1 in 20-year. This highlights the requirements for defence improvements, to provide protection to Castletown promenade and the adjacent property.

2.2.3 Current still water level flood risk

Based on the predicted extreme water levels from the Environment Agency Coastal flood boundary conditions for UK mainland and islands project[1], a maximum SWL of 4.74mD02 for the 1 in 200-year event including an allowance for climate change is predicted. Based on this prediction, it is considered that there will be no risk of flooding to the town caused solely by static water / tide levels over the open coast defences, as the primary defence has a crest elevation of 5.60mD02.

2.3 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 (including climate change)

2.4 Climate change

By selecting a design life of 100 years, it is important to factor in the predicted effects of climate change. The latest UK Climate Projections (UKCP09) have been used to determine climate change allowance for:

- Still water levels;
- Wind driven waves; and
- Swell waves.

Within UKCP09 estimates for sea level rise are provided under low, medium and high emissions scenarios. Within the three scenarios the estimate is further refined by 5th, 50th and 95th percentile confidence ratings. 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 it is proposed that the medium emissions scenario is considered and that the 95th percentile confidence rating is used. This gives a projected sea level rise of 650mm by the year 2115 for Castletown.

UKCP09 acknowledges the difficulty in predicting changes in wind speeds over the next 100 years and concludes that there will be a negligible increase in wind speed. Therefore, the wind driven wave component of the numerical modelling has no direct increase in wave intensity due to climate change. However, as a result of the increased still water levels from relative sea level rise, there will be an indirect increase in wind driven wave height. As a result of the larger depth of water at the coastal defence toe, larger waves will be able to travel inshore before breaking, creating a higher intensity wave climate in the year 2115.

For changes in swell waves, UKCP09 gives a prediction of the change in annual maximum wave height for the year 2115 of up to **1.0m** for the UK. It should be noted that wave height increases could be limited by the water depth at the study location and therefore the full 1.0m increase is not applicable for all scenarios.

[1] Coastal flood boundary conditions for UK mainland and islands, Project: SC060064/TR2: Design sea-levels. Environment Agency, Feb 2011.

The 1.0m allowance has therefore been applied to offshore swell wave conditions, which was subjected to wave transformation modelling to determine the change in wave height at each individual site.

2.5 Hydrodynamic data

The hydrodynamic data, used to design the open coast defences to a 1 in 200-year standard of protection in 2115, has been sourced from three primary sources:

1. **Extreme sea levels** - The Environment Agency Coastal flood boundary conditions for UK mainland and islands project[2], which developed a consistent set of design sea levels for Scotland, England and Wales.
2. **Extreme winds** – Calculated using established methods in BS6399
3. **Extreme swell waves** - The extreme wave conditions were adopted based on the Environment Agency's *Coastal flood boundary conditions for UK mainland and islands* project[3] which developed a consistent set of design swell wave conditions around Scotland, England and Wales.

These three sources of data were combined using joint probability analysis to create the hydrodynamic input conditions for the design of these defences for any given return period.

2.6 Performance standards

For coastal defences, the performance standards can typically be split into two areas, the still water level performance and wave overtopping performance.

2.6.1 Performance standard 1 – still water level flood risk

As discussed above, the current defences are offering in excess of a 1 in 200-year level of protection in 2115 against still water level flooding. Hence this design option will not seek to raise the impermeable defence level to address still water level flood risk

2.6.2 Performance standard 2 – wave overtopping risk

Two thresholds have been used to limit the volume of overtopping that is deemed acceptable for the concept design options:

1. The first lower threshold was established for a common coastal storm event, considered to have a 1 in 1-year return period, based on a joint probability assessment.
2. The second higher threshold will be established for the design storm event, considered to have a 1 in 200-year return period, based on a joint probability assessment. During this event it is considered that general public use of the pavement and road immediately behind the structure will be discouraged and only trained personnel will be operating within the vicinity of the structure.

Table 2-2 below summarises the guidance for vehicles and pedestrians provided within the European Wave Overtopping Manual (EurOtop).

Table 2-2: Limits for overtopping for vehicles (source: EurOtop[4])

Hazard type and reason	Mean discharge	Max volume
	q (l/s/m)	V_{max} (l/m)
Driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed.	10 - 50[5]	100 – 1,000
Driving at moderate or high speed,	0.01 – 0.05[6]	5 – 50 at high level or

[2] Coastal flood boundary conditions for UK mainland and islands, Project: SC060064/TR2: Design sea-levels. Environment Agency, Feb 2011.

[3] Coastal flood boundary conditions for UK mainland and islands, Project: SC060064/TR3: Design swell-waves. Environment Agency, Feb 2011.

[4] Pullen, T., Allsop, W., Bruce, T., Kortenhaus, A., Schuttrumpf, H & van der Meer, J (2007) 'Wave overtopping of sea defences and related structure: Assessment manual'. Accessed from www.overtopping-manual.com

[5] Note: These limits relate to overtopping defined at highways.

[6] Note: These limits relate to overtopping defined at the defence, assumes the highway is immediately behind

impulsive overtopping giving falling or high velocity jets.		velocity
Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway	1-10	500 at low level
Aware pedestrian, clear view of the sea, not easily upset of frightened, able to tolerate getting wet, wider walkway	0.1	20-50 at high level or velocity

The following twofold tolerable discharge thresholds have been proposed for all concept options on open coast environments:

- 1 in 1-year event – <0.1l/s/m
- 1 in 200-year event – <10l/s/m.

These tolerable discharges are such that all structures will be considered safe for pedestrian access during the more regular storm event, while vehicular and emergency staff will be safe to inspect defences during the less frequent, higher magnitude storm.

By adopting a twofold approach to acceptable overtopping levels, the new defence options considered for the sites have a dual purpose of preventing the frequent overtopping caused by common storms while providing structural and overtopping protection during rare events. By incorporating dual overtopping targets the crest height of all structures can be minimised, reducing both construction cost and visual impact.

2.7 Ground conditions

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

2.8 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 will be reviewed by a structural engineer to confirm that the design principles adopted are acceptable.

2.9 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. However, the location of more critical services has been identified by DoI. These critical services were considered in the development of the concept design options. If the project progresses to outline and detailed design it will be essential that a full service plan is developed.

2.10 Environmental impact

This commission does not include any formal Environmental Impact Assessment or Landscape Visual Impact Assessment. If the project progresses to outline and detailed design, a more in depth study of the environmental impacts will be required.

2.11 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.

2.12 Contaminated land

No information regarding the location of areas of contaminated land has been provided as part of this commission. Therefore all design options have been developed with the assumption that none of the areas are subject to contaminated land constraints. An invasive contaminated land survey should be undertaken at all locations prior to detailed design to enable detailed assessment of suitable construction techniques and options for removal or re-use of excavated material.

To progress concept design options as part of this study the following have been assumed:

- No investigation of contamination issues at individual development sites; and
- Development flood defence options may require some contaminated land treatment depending on the result of the investigations.

2.13 Tie in details

Tie-in details between old and new defences have been considered at a conceptual level. The key consideration has been to develop an option that does not create an area of outflanking or weak point, where overtopped water can bypass the defences and flood the hinterland. Careful consideration of the connection between the existing and new defences will be required during the detailed design phase.

3 Standards, guidance & reference documents

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

- BS 6180 1999: Barriers in and about buildings, code of practice
- BS EN 206-1:2000 Concrete – Part 1: Specification, performance, production and conformity
- BS EN 12620:2002 Aggregates for concrete
- BS EN 6349-1-1:2013, Maritime works, General, Code of practise for planning and design.
- CIRIA (2010), The use of concrete in maritime engineering – a guide to good practise
- Cobb, F (2009), Structural Engineers Pocket Book (2nd Edition)
- DEFRA (2009) UK Climate Projections 09
- Environment Agency. (2010). Fluvial Design Guide
- HR Wallingford (2007), EurOtop, Wave Overtopping of Sea Defences and Related Structures: Assessment Manual
- US Army Corp of Engineers (2002), Coastal Engineering Manual

4 Design development

The following provides a brief summary of how the key design elements were selected.

4.1 General form of defence

This design option acts to dissipate the wave energy arriving at the coastline by placing a permeable structure in front of the existing sea wall. This has been achieved through the design of a rock armour revetment.

4.1.1 Defence crest level, crest width and slope

A defence crest level of 5.90mD02 has been proposed for the rock armour revetment. This has been defined by an iterative process using the EurOtop overtopping tool and engineering judgement. The initial design of this revetment considered a full suite of revetment crest elevation, widths and slopes which were all tested against a range of wave height and water level combinations that comprise a 1 in 200-year event including an allowance for climate change and a 1 in 1-year event again including an allowance for climate change. The primary aim of this modelling was to determine the worst case combination for anticipated overtopping volumes. This initial modelling provided a guide for the general defence geometry appropriate for this location. In this case, the greatest reduction in overtopping rates were seen for a crest elevation 1m below the existing sea wall height with the front slope, sloping a 1:2.

The Design Input Statement set out limits for overtopping and are again presented here, <0.1 l/m/s for a 1 in 1 year event and <10 l/m/s for a 1in 200 year event. Using the iterative overtopping process, defence crest widths were varied at 100mm increments to identify the required width of crest to achieve these tolerable overtopping limits. This iterative process showed that a crest width of 3m was sufficient to reduce the overtopping to those set out above.

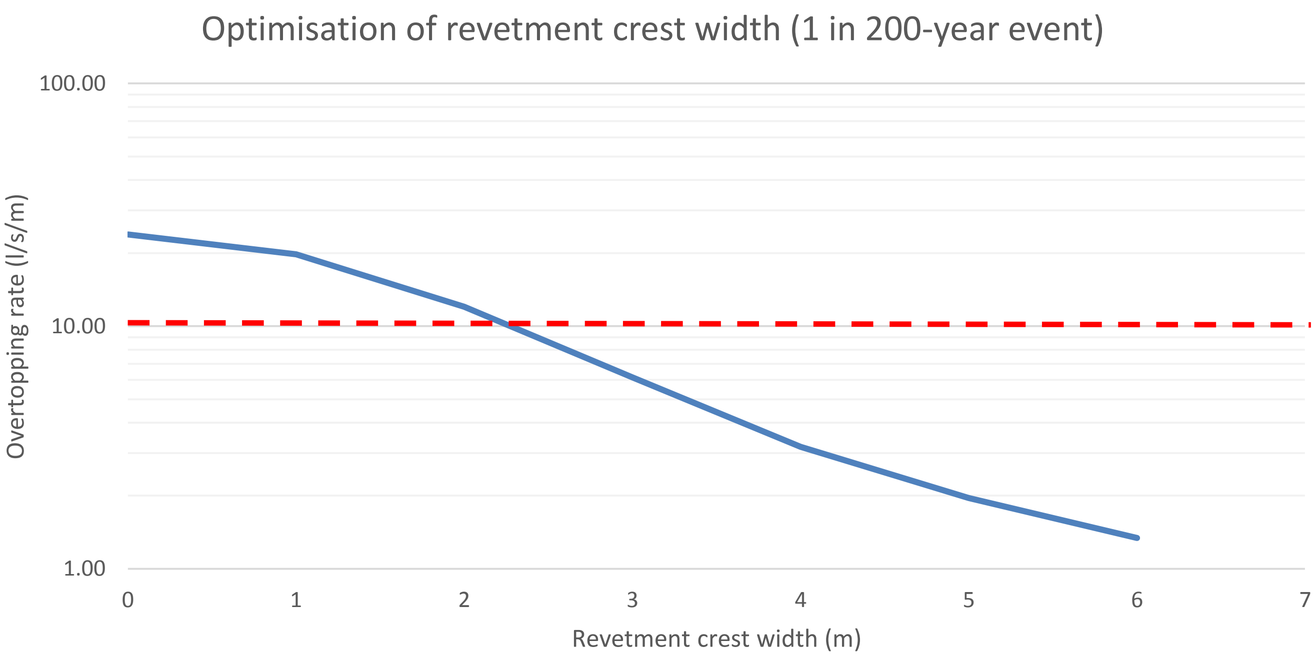

Figure 4-1: Optimisation of crest with for COC3

	Threshold Value	Value
Crest Elevation (mD02)	-	5.90
Crest Width (m)		3.00
Overtopping Rate 1:1 (l/m/s)	0.1	0.07
Overtopping Rate 1:200 (l/m/s)	10	6.15

Table 4.1: Defence configuration and overtopping rates for COC3

It should be noted that the EurOtop guidance suggests that the model is only suitable for the development of concept design options. Physical modelling is recommended for detailed design stages, if control of overtopping volumes forms one of the key design criteria.

4.2 Rock armour sizing

The following summarises the primary rock armour design process (further details can be found within the calculation sheet). The rock armour has been sized using the ultimate limit state, 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 transformed to the structure toe = 2.23m
- Period (T_m): 1 in 200 year plus climate change wave period = 5.44s
- Still Water Level (SWL): 1 in 200 year water level including climate change = 5.43mD02
- 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 armour stone

placed directly on a geotextile, therefore P=0.1 has been selected as an appropriate permeability factor.

- Slope Angle (α): 1 in 2 (27°) has been selected to represent a shallow 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.

Using the above input parameters within the Van der Meer calculation provides a median required rock mass (M_{50}) = **2.47t**. Therefore adopting a conservative approach by 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 armour stone. Figure 3 shows that less than 5% 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**.

However, the wave periods, extracted from the wave model, seem small given the orientation of the Castletown frontage. Given the criticality of these design parameters on the structural stability of the defence, a sensitivity analysis has been undertaken on the stability of this rock grading for a range of wave periods in the order of 5-7seconds. Figure 4-2 shows that for the wave with the highest energy, with a period of 7 seconds, the rock is still stable within the grading, with less than 15% being smaller than the required armour mass.

However, due to the importance of the hydrodynamic conditions on the defence design, it is recommended that the source of these small conditions is explored in more detail during detailed design. The source of this error could be due to the low resolution bathymetric data which is critical in calculating the wave height and period at a given location. During detailed design, it is recommended that further bathymetric data is collected, and the wave model is refined in order to better quantify the hydrodynamic conditions.

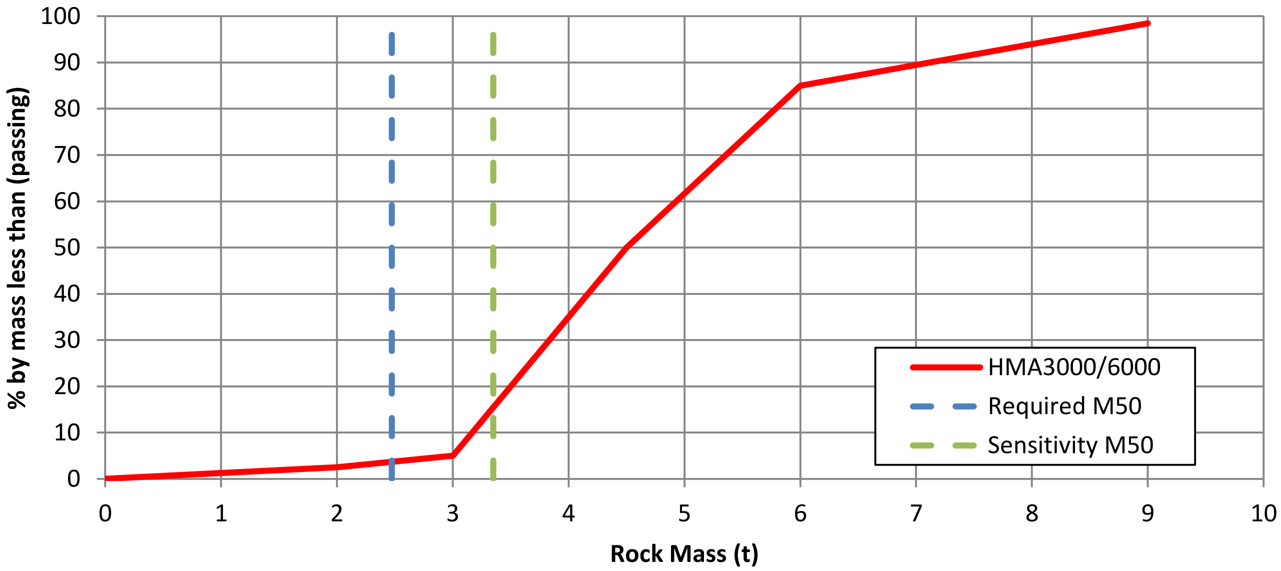

Figure 4-2: Grading curve for HMA 3000/6000 and the required and sensitvity M50

4.3 Structure toe and foundation level

Vertical sea walls show significant scour at the toe of the structures. This is caused by reflection at the vertical wall transmitting wave energy towards the toe. In addition, evidence suggests that for steep impermeable structures, the amount of expected scour is equivalent to that experienced at a vertical sea wall. For these types of structures, it is essential to found the structure toe at a level equivalent to the maximum expected scour depth from the design storm event. However, permeable rock revetments (as designed for Castletown) generally show little susceptibility to local scour and may even cause accretion (McConnell, 1998).

In addition, a rock toe has the benefit of flexibility, in being able to accommodate changes in beach level. When designing the rock toe and its foundation level, it is essential to consider the two principal failure mechanisms (Environment Agency, 2013):

- Displacement of rock,
- Loss of bed material through the rock matrix.

4.3.1 Mitigation measures to prevent displacement of rock

Revetment toes can be classified by their relative water depths, defined as the ratio of the water depth on the toe to the water depth of the whole structure (h_t/h).

During the design storm event, the structure can be described as having a high toe (for $h_t/h<0.5$). This equates to the toe of the revetment being directly exposed to wave induced loadings during the majority of the stages of the tide. Consequently, high toed structures have to have sufficiently large armour mass in order to maintain stability during the design storm event and prevent the expulsion of armour units. This can be calculated using the same formula as in Section 4.2, showing that the toe should be formed of 3-6 tonne rock.

The stability of the leading edge of the toe armour is likely to be reduced due to reduced friction between interlocking layers (Environment Agency, 2013). These rocks are more vulnerable to movement due to the absence of support from the seaward side.

In order to mitigate this potential failure mechanism, the design specifies using the largest rocks as the keystone (a minimum of 4.5 tonnes) and that the keystone should be layed a minimum of 1m below the beach surface. This small trench will ensure that the larger keystone is supported against the beach profile.

It should be noted that if excavations reach bedrock, it is recommended that the structure is founded on this substrate.

4.3.2 Mitgation measures to prevent the loss of bed material through voids in rock

Bed material washing out through the interstices of a rock matrix can lead to bed lowering causing lowering of the rock structure. The lowering continues until it becomes embedded similar to the process of liquefaction (Environment Agency, 2013).

To prevent the washout of fill material and fines which provide a defence function and are critical to the structural integrity of the revetment it is proposed that a geotextile layer is placed over the substrate. There has been no allowance for the design of an additional rock filter layer underneath the main rock armour in an attempt to limit the total amount of material. The geotextile will be directly overlain by the proposed rock armour. GEOfabrics HPS14 is recommended based on a maximum weight of 6t dropped from 1m (GEOfabrics Coastal and River Defence System Design Guidance). This has a level of conservatism built in, with a factor of safety of 2 built in.

4.4 Packing density

The packing density of the armour layer has a direct impact on the performance of the structure as well as the total volume occupied. The following assumptions have been made in calculating the packing density and total required volumes for the defence configuration 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 ~385m
- 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 an armour stone that is 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 requirements. It should be noted that the total required masses have been calculated based on a constant foundation depth of -0.25mD02 which is known to vary considerably. Consequently, the actual required mass may be significantly higher or lower than the values calculated here.

Required Median Mass ($M_{n50,a,req}$)	2.47t
Designed Median Mass ($M_{n50,a,des}$)	3-6t (4.5t median)
Required Median Diameter($D_{n50,a,req}$)	0.97m
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
Cross Sectional Area of Structure (A_a)	34m^2
Total Required Rock Mass ($W_{b,a}$)	23,000t

Table 4-2: Rock sizes and estimated requirements

It should be noted that these represent estimated material requirements and should not be used for ordering materials.

4.5 Tie in details

It is proposed that the structure will terminate at the start of the private defences to the west of the section, and at the point at which the sea front becomes less built up in the east of the section. The exact location of this tie in cannot be confirmed at this design stage. During detailed design, it is recommended that multiple sections are analysed to identify the exact location of the defence tie in.

4.6 Access for the public

By providing a physical barrier between the promenade and the beach, it is necessary to provide designated access routes for the public to access either side. This access could be achieved through placing stainless steel access steps at strategic locations. The best method for achieving this access should be developed during a latter design stage. This may be of particular importance to a slipway in the middle of the defence, where a revetment berm may be necessary to maintain slipway access.

4.7 Public safety

Public safety has formed a key consideration during the concept design development phase. The main risks associated with this option are the issues surrounding the future public usage of the structure. 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 a means of warning. In addition, it is recommended that stainless steel access steps are provided over the defence, to allow members of the public to access either side of the structure.

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.

5.2 Beach morphological evolution

This study has not included any assessment of the likely evolution of the cobble/shingle beach. Changes in beach morphology over the design life of the structure may cause increased settlement and steepening of the revetment slope. A rock revetment is flexible and adaptable by nature, so can cope with some fluctuations in the foundation layer.

While the design has ensured the rock revetment is founded below the maximum scour depth expected from the design storm event, no consideration for the long term evolution of the beach has been undertaken at this design stage. If the beach is showing a long term trend of erosion and lowering, it is possible that the defence may be undermined in the future. During detailed design, sediment transport modelling should be undertaken, to ensure the structure is founded at a sufficient depth with the inclusion of a consideration of the anticipated beach changes through time.

Due to the transient nature of beach levels, there is no certainty that the quoted levels will still be relevant during latter design stages. This should be checked at each stage of design. Some of this risk has been mitigated by ensuring all structures are founded at their appropriate minimum dimensions below the beach profile rather than founded at a fixed level.

5.3 Integrity of the sea wall

While the seawall face seems to be in reasonable condition, the placement of rock armour onto the existing structure represents a technical risk. Care should be taken to not drive plant on the landward side of the wall, minimise any required excavations and take care when rock is placed in direct contact with the existing defences. All attempts to prevent loading to the sea wall should be made. The geotextile should be lapped into the existing structure to reduce the risk of wash-out of fines.

The 100 year design life of the coastal defence is dependent on the structural integrity of the existing defences, as this new structure forms part of a composite coastal defence. This design assumes that the existing sea wall will not be allowed to deteriorate further as this may undermine the newly proposed superstructure. It is recommended that a full asset inspection be undertaken prior to detailed design, to quantify the residual life of the structure and allow for the development of more tailored remediation measures.

5.4 Low hydrodynamic conditions

The hydrodynamic conditions (wave height and period) calculated for Castletown appear to be relatively small. The hydrodynamic conditions are absolutely critical to the design of a coastal defence. Due to the importance of these conditions on the defence design, it is recommended that the source of these small conditions is explored in more detail during detailed design. The source of this error could be due to the low resolution bathymetric data which is critical in calculating the wave height and period at a given location. During detailed design, it is recommended that further bathymetric data is collected, and the wave model is refined in order to better quantify the hydrodynamic conditions.

5.5 Tie-ins with existing defence

The tie-ins have been considered at a conceptual level but will require careful consideration during detailed design.

5.6 Services

Limited 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.

5.7 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.

5.8 Stakeholder requirements

A Multi Criteria Analysis was completed as part of this study to try and determine the key considerations of the project stakeholders. It is anticipated that during the course of a formal options appraisal project stage that more in depth stakeholder consultation will be completed. The results of which may lead to changes in the concept designs that have already been developed.

5.9 Environmental impacts

No formal Environmental Impact Assessment was completed during this project stage. It is anticipated that during the course of an options appraisal stage that an in depth assessment of the environmental impacts associated with all proposed options would be considered. This process may result in changes being made to the proposed designs.