DESIGN – TECHNICAL NOTE

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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 PSM2, an option to place a rock revetment and a raised rear sea wall in front of the existing sea defences in Port St Mary, to reduce both the still water level and wave overtopping flood risk.

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 Port St Mary are frequently overtopped by waves during a storm event and have also been shown to be at risk of still water level flooding during a high return period event. In order to design an option that efficiently reduces the wave overtopping and still water level flood risk 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. Still water level flooding occurs where the extreme water level exceeds the impermeable defence crest level and water spills over the defences, flooding 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 from both still water level and wave overtopping flood mechanisms.

2.2.1 Existing defence geometry

The existing defences are composed of a concrete armour unit revetment with a small raised concrete kerb behind the revetment crest. The low lying topography behind these defences mean that water travels over the defences and becomes trapped in a basin behind.

Attribute	Dimension
Concrete cap crest level	4.6mD02
Revetment crest level	4.2mD02
Cap height	0.2m
Revetment toe level	2.0mD02
Revetment slope	1:2.5

Table 2-1: Existing defence geometry summary

2.2.2 Current wave overtopping risk

Based on baseline modelling of the existing defences, Port St Mary is currently offered a 1 in 20-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 5-year. This highlights the requirements for defence improvements, to provide protection to Port St Mary 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 4.59mD02 for the 1 in 200-year event including an allowance for climate change is predicted. Based on this prediction, it is considered that there is a risk of flooding to the hinterland caused solely by static water / tide levels over the defences, as the impermeable defence has a crest elevation of 4.50mD02. The defence solution at this location, must therefore consider both the still water and wave overtopping flood risk.

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 Port St Mary.

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

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

the water depth at the study location and therefore the full 1.0m increase is not applicable for all scenarios. 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 less than a 1 in 200-year level of protection in 2115 against still water level flooding. Hence this design option will seek to raise the impermeable defence level to address still water level flood risk. This will be achieved, through ensuring that the impermeable defence crest is situated at the 1 in 200-year extreme water level plus a 150mm freeboard allowance.

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

[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.

Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets.	0.01 – 0.05[6]	5 – 50 at high level or 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 Integrity of the front sea wall

For the purposes of progressing this concept design, it is assumed that the wall will be repaired and strengthened to prevent further damage. This assumption states that the wall will be sheet piled to prevent further foundation undermining, as discussed on site with DoI. This has formed part of a conservative assumption as the repaired sea wall is likely to have a crest elevation greater than the failed sea wall, providing a level of conservatism in the overtopping rates calculated over the proposed defences.

No attempt to re-design the failed wall section has been made during this piece of work as it has currently been put forward for planning considerations. Assistance for the re-design of the failed wall can be provided if necessary.

2.9 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.

2.10 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 and have been highlighted on concept drawings and in hazard inventories. If the project progresses to outline and detailed

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

design it will be essential that a full service plan is developed.

2.11 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.

2.12 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.

2.13 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.

2.14 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. A small raised concrete cap has been placed behind the rock revetment, to raise the impermeable level of the defence to reduce the risk of still water level flooding.

4.1.1 Revetment crest level, crest width and slope

A defence crest level of 4.25mD02 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 comprises 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 at 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 5m was sufficient to reduce the overtopping to those set out above.

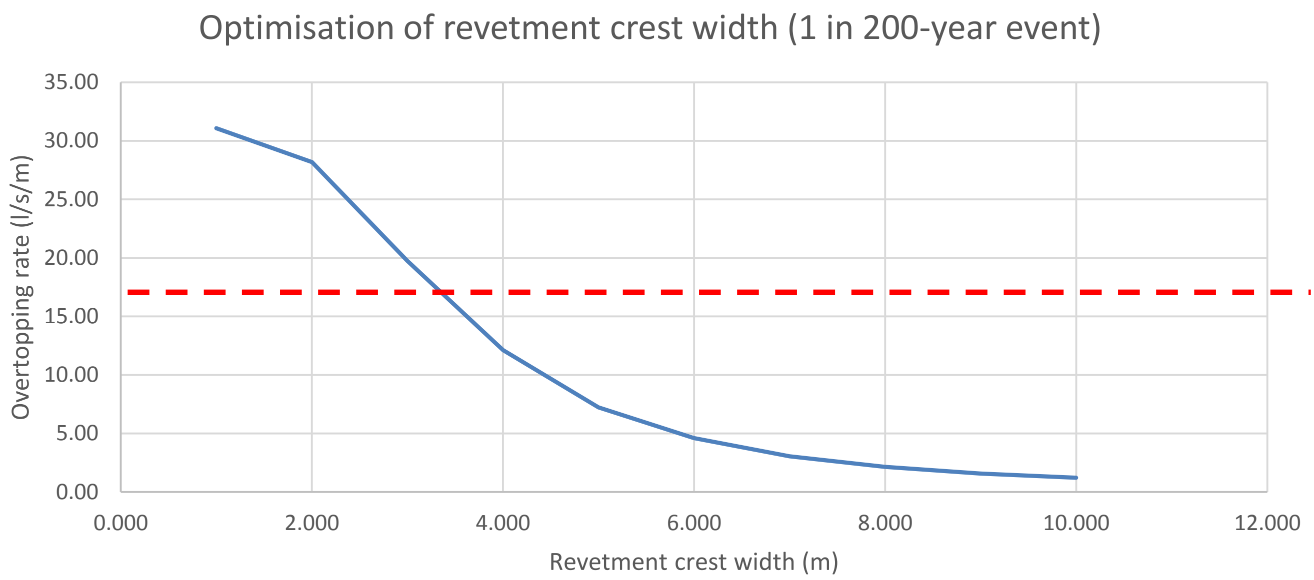

Figure 4-1: Optimisation of crest with for PSM2

	Threshold Value	Value
Crest Elevation (mD02)	-	4.25
Crest Width (m)		5.00
Overtopping Rate 1:1 (l/m/s)	0.1	2.43
Overtopping Rate 1:200 (l/m/s)	10	7.23

Table 4.1: Defence configuration and overtopping rates for PSM2

It should be noted, that this defence configuration does not conform to the tolerable threshold for the more regular 1 in 1-year storm (<0.1l/s/m). In order to meet this threshold, the crest width would have to be in the order of 18m which would be a considerable capital scheme. An overtopping rate of 2l/s/m is suggested as being acceptable for a well shod, trained member of staff to access the land behind[7]. Given Port St Mary's relatively industrial outlook, the DoI could consider this overtopping threshold acceptable for the more regular storm event. However, the lack of conformity to the design standard should be noted and considered when evaluating other options in the appraisal. It is important that the DoI understand the implications of accepting this higher overtopping rate and the need to plan for area cordoning during the storm event.

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.

4.1.2 Concrete cap crest elevation

Due to the need to provide a still water and wave overtopping performance standard, a rock revetment option in isolation will be insufficient to provide the adequate level of protection, due to the permeable nature of the defence.

The raised concrete cap has a crest level of 5.11mD02, forming a 500mm raise on the existing wall elevation. This includes a large freeboard allowance on the 4.59mD02 extreme still water level, which is for two reasons:

- Primarily, a raise of less than 500mm would be difficult to design to ensure structural stability while providing the minimum cover to reinforcement necessary for an exposed coastal environment such as this one.
- Raising the impermeable defence crest level would contribute to a reduction in the wave overtopping calculated over the Ramsey defences.

4.1.3 Wall height

The 500mm raise gives a wall height above ground level equal to a total wall height of 700mm above ground level, which is not in compliance with BS 6180 for the minimum height of concrete barriers and handrails for horizontal guarding (1100mm). To ensure compliance with this standard, the design specifies the use of a 400mm stainless steel hand rail, raising the guard rail to the necessary 1100mm. This has the added benefit of aiding in the dissuasion of the public from climbing on the rock armour.

4.1.4 Tie in to existing wall

The raised concrete cap will tie in to the existing wall through use of two 12mm diameter 500mm long stainless steel dowel bar located through the centre of the cap at 500mm centres. This has been estimated using engineering judgement, but should be considered in more depth should this option be taken forward to detailed design. The dowel bars should be sufficiently protected from the main reinforcement to avoid the interaction between the mild steel and stainless steel.

4.1.5 Wall thickness and reinforcement cover

The wall thickness has been defined, allowing for 200mm wide reinforcement cage with a minimum 100mm concrete cover. This allows for a wall thickness of minimum 400mm. A large minimum cover to concrete has been applied due to the exposed nature of the environment.

4.1.6 Structure reinforcement

The proposed new concrete cap will have a nominal 200mm wide steel reinforcement cage, this should be considered in more detail during the detailed design phase. The structural design of the proposed raised walls are beyond the scope of this study.

4.1.7 Concrete mix design

The concrete mix design should consider a number of factors, firstly issues associated with the heat of

[7] 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

hydration and thermal cracking as detailed above should be investigated. Secondly, the type of exposure that the concrete is subjected to and it's resistance to the ingress of chlorides which will cause corrosion to any reinforced elements must be assessed. The properties of the concrete for the raised harbour walls are suggested below based on guidance from EN 206-1:2000:

- **Density**: A typical concrete density of 2.4t/m^3
- **Grade**: C40/C50
- **Exposure class**: XS3 for concrete in a tidal, splash and spray zone
- **Aggregate diameter**: 20-40mm selected in accordance with EN 12620:2002
- **Workability**: Slump class S2 (50-90mm)

However, this specification will be subject to modification during refinement in detailed design.

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 = 1.96m
- Period (T_m): 1 in 200 year plus climate change wave period = 4.83s
- Still Water Level (SWL): 1 in 200 year water level including climate change = 4.74mD02
- 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}) = **1.60t**. 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**.

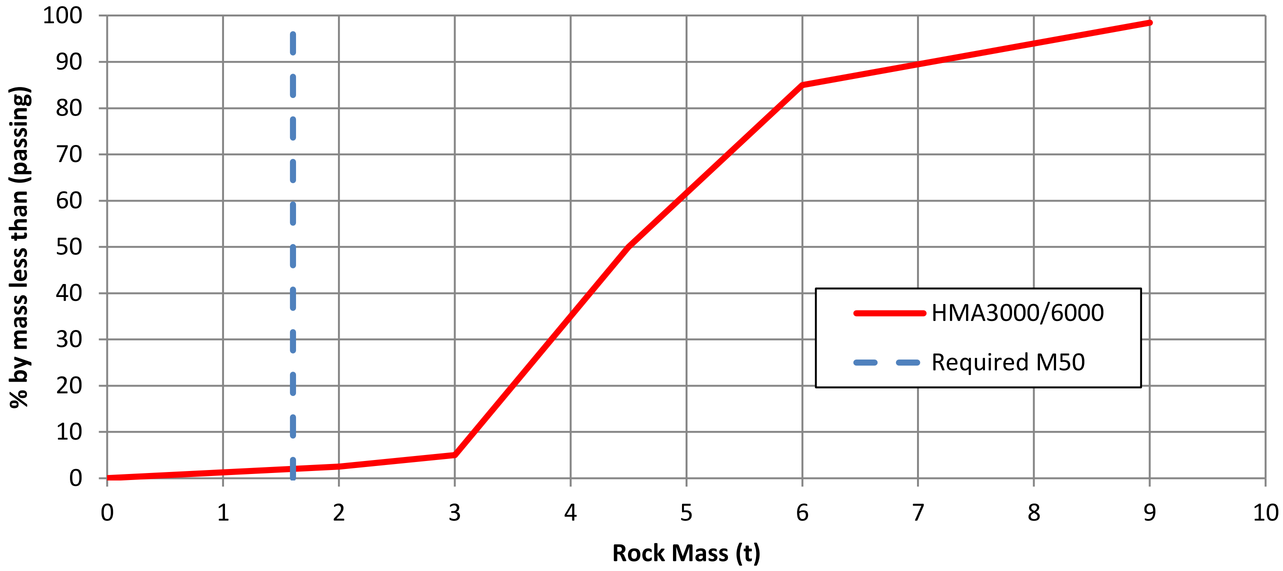

Figure 4-1: Grading curve for HMA 3000/6000 and the required M50

4.3 Structure toe and foundation level

The rock revetment has been designed to be placed directly on to the existing rock platform to reduce the risk of scour at the structure toe.

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).

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 ~250m
- 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 -0.34mD02 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 ($M_{n50,a,req}$)	1.60t
Designed Median Mass ($M_{n50,a,des}$)	3-6t (4.5t median)
Required Median Diameter($D_{n50,a,req}$)	0.84m
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)	24m^2
Total Required Rock Mass ($W_{b,a}$)	10,600t

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 form a lap with the harbour arms to either side of the site. This lap should be of sufficient length to prevent outflanking of the defences.

The exact location of these tie ins 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 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 interaction with the rock armour structure. Part of this risk has been mitigated by the inclusion of the concrete cap and handrail, which provides a physical barrier between the pavement and the revetment. 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 beach to access either side of the structure.

In addition, by offering a lower standard of protection or allowing a higher tolerable threshold, the risk of public interaction with water overtopping the defence is higher than if it conformed with the intended design standards. The DoI should implement a storm action plan to control these risks, to prevent pedestrians encountering the overtopped water.

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.

5.3 Integrity of the existing coastal defence

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 concrete revetment will not be allowed to deteriorate further as this may undermine the newly proposed superstructure. Significant asset maintenance is likely to be required in the next 20 years to ensure the condition does not become critical.

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 Tie-ins with existing defence

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

5.5 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.

5.6 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.

5.7 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.

5.8 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.