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
Author	Alec Dane
Subject	Douglas Open Coast – Option DOCA3 – Beach Recharge

Project Title: Isle of Man Sea Defence Options			Sheet No: 1
Subject: Douglas Open Coast – Option DOCA3 – Beach Recharge			Calc No:
Job No: 2014s1358			Version:1.0
Developed By: Alec Dane	Date: 26/08/2014	Revised By: Alec Dane	Date: 01/12/2014
Checked By: Graham Kenn	Date: 28/11/14	Approved By: Graham Kenn	Date: 02/12/2014

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 DOCA3, an option to undertake a beach recharge in Douglas, to reduce wave overtopping to the hinterland. Douglas has been split into two distinct areas due to the differing wave overtopping mechanisms experienced. This design option covers a beach recharge option in the main Douglas Bay, backed by Central and Queens Promenade.

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 coastal defences at Douglas 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.

As stated above, the options proposed to reduce wave overtopping in Douglas have been split into two distinct sections, due to the differing wave overtopping mechanisms experienced. In the south of Douglas Bay, defence elevations are low, but are fronted by a large sand beach in places. Here, the wave overtopping occurs due to the run-up of broken waves exceeding the defence crest elevation, inundating the hinterland. However, in the north of the bay, wave overtopping occurs due to impulsive conditions. Here, the large vertical walls allow for large waves to break on the structures, causing jets of up-rush water to pass over the structure. This design option seeks to reduce the volume of wave overtopping in the south of Douglas Bay.

2.2.1 Existing defence geometry

The existing defences are composed of a low-level concrete recurve sea wall fronted by a sandy beach. To the north and south of the site, beach elevations are low with anecdotal evidence suggesting the overtopping rates are higher in these areas.

Attribute	Dimension
Sea wall crest level	4.9mD02
Beach crest level	3.4mD02
Beach crest width	0m

Table 2-1: Existing defence geometry summary

2.2.2 Current wave overtopping risk

Based on baseline modelling of the existing defences, Douglas Section A 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 the road and property behind.

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 5.52mD02 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 risk of flooding caused solely by static water / tide levels over the open coast defences, as the impermeable defence has a crest elevation of 4.91mD02. The defence solution at this location must therefore consider both the still water level 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 Douglas.

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 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 q (l/s/m)	Max volume 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 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.

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

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

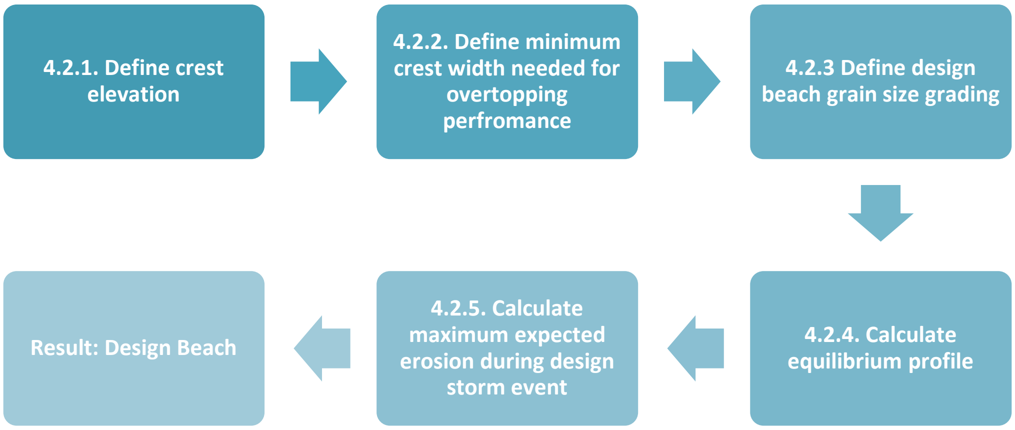
The following provides a brief summary of how the key design elements were selected.

4.1 General form of defence

This design option involves the placement of sediment on the existing beach in order to ‘top up’ the existing beach to create a higher and wider beach. This will dissipate wave energy over the beach as opposed to over the existing coastal defences.

4.2 Design beach

The design beach has been defined in the following process. More information on these stages are given in the following subsections.



4.2.1 Crest elevation

A range of defence crest levels were considered as part of the design of the recharged beach. The design beach has a selected crest level of 4mD02, selected to represent the most feasible design while providing an amenity value to the public.

4.2.2 Minimum beach crest width for overtopping performance

A minimum beach crest width of 10.00m has been proposed. This has been defined by an iterative process using the EurOtop overtopping tool and engineering judgement. The initial design of this beach considered a full suite of beach 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 of 4mD02.

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, beach 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 10m was sufficient to reduce the overtopping to those set out above.

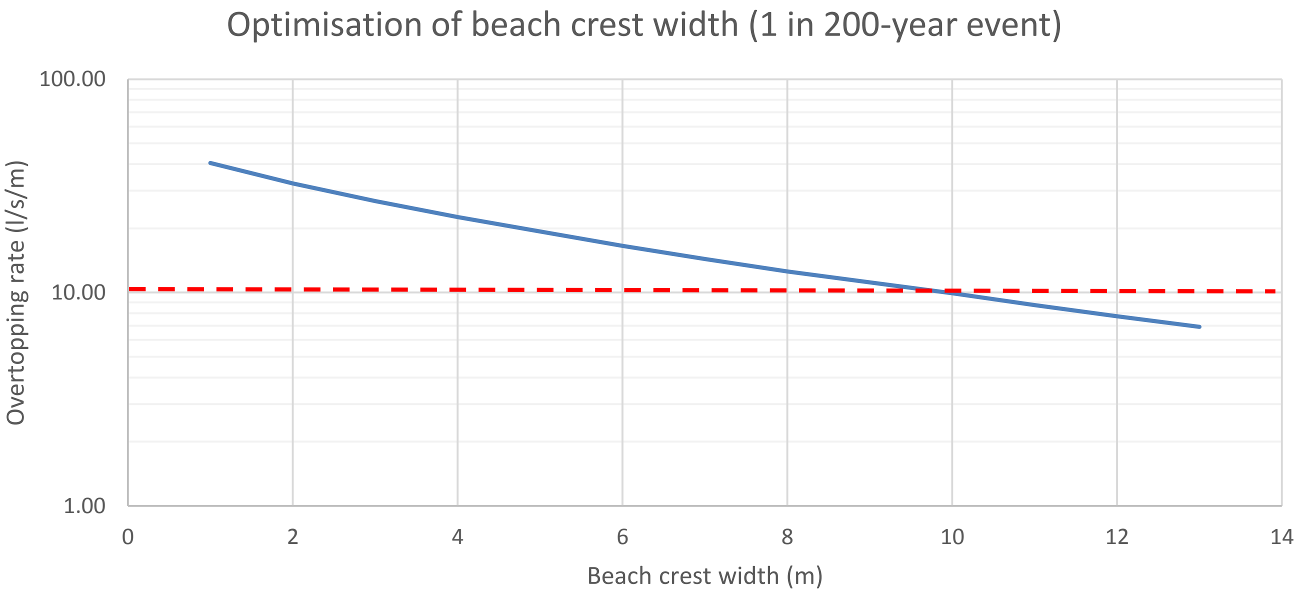


Figure 4-1: Optimisation of beach crest

	Threshold Value	Value
Crest Elevation (mD02)	-	4.00
Crest Width (m)		10.00
Overtopping Rate 1:1 (l/m/s)	0.1	2
Overtopping Rate 1:200 (l/m/s)	10	8

Table 4.1: Defence configuration and overtopping rates for DOCA3

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 beach crest width would have to be in the order of 25m which would be a considerable capital investment. 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]. 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.

4.2.3 Definition of recharged grain size

A fundamental consideration in the design of a beach recharge scheme is the grain size distribution of the recharged material. This has significant implications on both the stability of the beach, the long term performance and the equilibrium profile. Coarse sediment sizes can expect to have a longer design life, being more difficult to entrain and showing reduced losses when compared to finer sand sized particles. However, coarse gravel provides little amenity value. On most beaches, annual sorting of sediment by waves and tides produces a dynamically stable grain size distribution which is expected to show stability during the normal hydrodynamic conditions. The native grain size therefore provides a good representation of the required grain size for stability of the recharged beach.

[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

It is expected that the recharged material should have a grain size distribution of at least that found native on site. Full specification of this grading has not been undertaken during conceptual design as it was considered to be outside the scope of works. For the purposes of estimating costs of material, it should be assumed that the grain size distribution should be similar or courser than the one found on site, reflective of a medium sand.

4.2.4 Equilibrium slope

The equilibrium slope is directly proportional to the grain size with finer sediment lying at a flatter slope than coarse material. This is a direct consequence of the reduced permeability of fine sediment beach. In coarse gravel beaches, wave energy can be dissipated through the permeable beach, with backwash percolating through the beach surface and slowly returning to sea. However, in sandy beaches, backwash runs down the main beach face, carrying entrained sediment resulting in a longer and flatter profile. Typical stable slopes are provided in the table below. An estimated median grain size of 0.5mm suggests the equilibrium profile will be in the order of 1:20, used for the design of this beach.

Table 3: Typical beach gradients for a range of median grain sizes

Median sediment size	Mean beach gradient	
	From	To
0.2	1:50	1:100
0.3	1:25	1:50
0.5	1:20	1:40

4.2.5 Maximum expected erosion during design storm event

During storm conditions the impact of extreme waves and water levels will induce erosion losses that will reduce the beach and result in larger than expected overtopping rates. These erosion losses must be added to the minimum beach width calculated above to ensure overtopping does not exceed the desired thresholds. The erosion losses have been assessed conceptually. This conceptual design has allowed for 50% erosion during the design storm event, equal to a crest recession of 5m. Coupled with the required 10m beach crest width, this creates a design beach crest width of 15m.

Table 4: Design beach crest

Proposed beach elevation (mD02)	Required beach crest width (m)	Potential erosion during storm (m)	Required pre-storm beach crest width (m)
4.0	10	5	15

4.2.6 Design beach

The design process above results in the design beach shown in drawings OP4-1001 and OP4-1002 which is necessary to achieve the design standards outlined in the Design Input Statement.

4.3 Capital replenishment

The design beach would require an initial capital replenishment to build the beach up to the desired levels. This would likely be achieved through use of a trailing suction hopper dredger to ‘rainbow’ sand onto the beach.

During construction of the beach, losses are to be expected. Losses in volume can occur from two main sources:

- Losses due to compaction
- Loss of fines

During placement of beach (particularly through rainbowing), the density of the material can often be far lower than would occur on the native beach – a process known as bulking. On a well-established beach, fines migrate to the core and fill interstices in the structure. Over the course of three or four tides, large losses of volume can occur as the density increases under wave action. In addition, recharged material (especially dredged aggregate) contains a significant portion of fines. These fines are often lost from the beach as they are too small for the hydrodynamic environment.

These losses need accounting for, in order to achieve the desired beach profile. Typical losses are expected to be in the order of 20%. This should be included when estimating the costs associated with the scheme.

4.4 Maintenance replenishment

In addition to the capital losses, annual losses are to be expected. Typical losses on recharged beaches have been shown in the order of 5% annually which are a combination of both cross shore and longshore losses. Due to the largely swash aligned nature of Douglas Bay, it is expected that longshore losses would be less than cross-shore losses. The beach would likely form into a crenulated bay which mimics the natural headland-bay arrangement shown in Douglas. Consequently cross shore losses are expected to dominate. Without maintenance recharge, the total beach volume would fall far below, the required volume to offer the standard of protection set out in the design input statement.

Instead, it is recommended that the beach recharge scheme would incorporate a staged maintenance replenishment programme. This programme would consist of 5 yearly cycles of a deposition of further recharged sand. This is to maintain the total beach volume at the critical volume required to meet the design standards presented above. The requirement for staged maintenance reflects the less durable nature of a beach when compared to more permanent defences such as breakwaters and revetments. This comes at a considerable extra cost when compared with other options presented in this study.

4.5 Beach control structures

The nature of the existing small beach suggests that the area doesn't naturally support the build-up of beach grade material. In sites such as this, the long-term process inducing the loss is not mitigated by simply adding more beach. As explained above, losses are expected which will require maintenance to maintain the design standard. These losses can be mitigated on drift aligned frontages by the installation of beach control structures, such as groynes. The design of beach control structures has not been considered with this option. During detailed design, numerical modelling would provide a better indication for the requirements of long shore control structures.

4.6 Impermeable defence crest elevation

This design option seeks to make no increase in the impermeable defence level of the composite coastal protection. It is expected that an additional raise of 500mm of the existing sea wall will be necessary to offer the required standard of protection against still water level flooding.

4.7 Public safety

Public safety has formed a key consideration during the concept design development phase. The main public safety design issue relates to the construction of the beach. The area should be cordoned off during placement of beach material.

In addition, by offering a lower standard of protection or allowing a higher tolerable threshold during the regular storm event, the risk of public interaction with water overtopping the defence is higher than if it conformed with the intended design standards. If this option is progressed, 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 Prevention of cliffing

Following recharge, the beach profile will alter in response to wave action to reach its dynamic equilibrium profile. 'Cliffing' poses a significant problem on recharged beaches, where erosion of the beach at the high water mark creates a near vertical cliff in the beach profile. This poses a significant health and safety risk to beach users. The cause of cliffs is not well understood. However, it is thought that chemical processes within the recharged material, in combination with mechanical compaction cause a very impermeable beach surface that can be held against the natural angle of repose. In order to mitigate this risk, it is essential that the proportion of fines on the lower limb of the grading curve are limited, and that mechanical compaction is limited wherever possible.

5.3 Still water level flood risk

This design option does not address the still water level flood risk in Douglas. The use of a raised concrete cap should be considered in combination with the beach recharge option in order to offer the required design standard.

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 Promenade closure during a storm event

The higher tolerable thresholds offered by the beach mean that the promenade area behind the defence will be unsuitable for public pedestrian access during the regular storm event (1 in 1-year). Consequently, DoI should provide emergency on-the-ground manpower during this storm event, to cordon off and close parts of the frontage to reduce the risk of public interaction with wave overtopping. This should be factored when considering the suitability of each defence option.

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.