Portgordon Flood Study

Moray Council

Options Appraisal and Business Case Report

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Contents

Executive Summary

Stewart Street in Portgordon suffers from waves overtopping the existing sea defence structure. The resultant flooding to the houses is exacerbated by the water becoming trapped behind a small setback wall along the seaward edge of the pavement. This study identifies and appraises options aimed at reducing overtopping of the existing defences, and subsequently reducing the flood risk to the surrounding properties along the seafront. Following completion of this initial options appraisal an additional sensitivity study looks at solutions that could be developed for lower return period option, with the initial study focussing on 200-year return period solutions.

Based on a visual inspection undertaken by Jacobs in December 2016, the structural condition of the existing defences does not appear to have deteriorated significantly since a previous visual inspection undertaken by Jacobs in 2013[1]. Discrete locations along the setback wall require maintenance and the rock armour along the toe of the defences appears to have slumped to a lower angle in locations along the length of the defence and, in some places, is partially buried by sand, shingle and seaweed; this is likely to have a detrimental effect on the ability of the rock armour to dissipate wave energy.

In order to develop options~~,~~ aimed at reducing the overtopping and flooding to an acceptable level, wave modelling and overtopping analysis of the existing defence was first undertaken. The overtopping assessment was carried out by ABPmer, using the Neural Network Tool (NNT) developed under the European Overtopping Manual (EurOtop (2007))[2].

A range of return periods from 1 in 10-year to 1 in 1,000-year are considered in the hydraulic modelling for the baseline ‘failed existing defence’ and ‘maintained existing defence’ options. In line with Scottish Planning Policy[5], the enhanced defence options are considered for a 1 in 200-year return period event. In addition, the effects of climate change are considered over the design life of the structure. The wave overtopping performance of the structure is assessed for 2022 and at the end of its 100-year design life for the year 2122.

Five defence options have been identified with the aim of alleviating the overtopping issues at Stewart Street. These are:

- Option 1: Rock armour berm over upper slope
- Option 2: High rock armour berm over lower seawall
- Option 3: Rock armour berm extended seaward.
- Option 4: Stepped revetment
- Option 5: Wave return wall

The first three options comprise a rock armour berm built up over the existing defences to differing levels and differing seaward extents. Options 1 and 3 demolish the existing setback wall and construct a new, higher level wall in its place, whereas Option 2, which has a higher berm crest, maintains the existing setback wall. The remaining two options are concrete structures for comparison with the rock armour options. Option 4 is a composite structure comprising a rock armour revetment leading up to a concrete stepped revetment. Option 5 is an enlarged wave return wall, encompassing the existing wave return wall in-situ.

Each of the options modelled were found to result in a significant reduction in the overtopping rate in comparison to the existing defence. Of the options considered, Option 2 was found to give the greatest reduction in the rate of overtopping, with a peak rate of approximately 2 l/s/m during the 1 in 200-year event: over 100 times lower than the rate for the existing scenario. The peak rate is calculated as the mean rate over a 5-minute time period. This rate is below the acceptable overtopping limit for trained pedestrian staff, according to EurOtop guidance[2].

[1] Portgordon Sea Defences Options Study Report, March 2013, Rev 0, Jacobs UK Ltd.
[2] EurOtop 2007, Wave Overtopping of Sea Defences and Related Structures: Assessment Manual

The overtopping results for each option were applied to a hydraulic model to predict the maximum flood extent and depth during the 1 in 200-year event, while accounting for drainage via the existing and proposed baffles and the control point at the western end of the setback wall. A degree of flooding still occurs for each of the five options considered, although property damage is only expected for Options 1 and 3. Option 2 experiences greater flooding than Options 4 and 5, yet the extent is predicted to be restricted to Stewart Street and Lennox Place, without affecting the nearby properties. It is envisaged that further refinement of the drainage design may offer improved performance over the layout considered which may in turn reduce the flood extent and depth further.

Given the size and scale of the proposed options, it is expected that any of the options considered would involve major construction works with associated disruption to the local area. This is particularly apparent for Options 4 and 5 where large concrete structures would be constructed on site. These options are likely to result in a degree of disruption to the local road network, construction noise and air pollution and visual impact during the construction phase. Thereafter, the relatively large concrete structures would have a long-term visual impact on road users and residents, particularly for Option 4 which has the highest crest level.

The estimated capital cost developed for each of the five options and their associated benefit cost ratios are shown in Table ES-1.

Table ES-1: Cost estimates and benefit cost ratio (BCR) for each defence option

Option	Total Cost Estimate (£k)	BCR (60% Optimum Bias)	BCR (30% Optimum Bias)%	BCR (Without Optimum Bias)
Do Minimum	307	2.9	3.6	4.7
Option 1	11,477	0.87	1.07	1.39
Option 2	10,895	0.99	1.22	1.59
Option 3	16,585	0.62	0.76	0.99
Option 4	12,280	0.88	1.08	1.40
Option 5	16,698	0.64	0.79	1.03

It is evident from Table ES-1 that, of the options considered, Option 2 is the preferred option, achieving a benefit cost ratio (BCR) of 0.99 with a 60% Optimism Bias.

It is noted that Do Minimum does achieve a higher BCR of 2.9. However, this option maintains the existing defences as they are currently and will not alleviate the flooding risk that the properties are currently subjected to. To put this into perspective, by maintaining the existing defences, it is predicted that approximately £1.431m of damages will be avoided at Portgordon over 60 separate storm events with return periods of up to 1:200-years, over the 100-year design period under consideration. Notwithstanding this, it is anticipated that damages totalling £15.961m will still be incurred at the site over this same period.

By contrast, it is predicted that Option 2 would result in the avoidance of £17.271m of damages with only £122k of damage being incurred over the same 100-year period. This highlights the added benefit of Option 2 over the Do Minimum option and supports the recommendation of Option 2 as the preferred option.

1. Introduction

1.1 Project Background

The village of Portgordon, on the Moray Coast, is periodically subjected to extreme waves combined with high water levels, resulting in overtopping of the existing coastal defences. The overtopping causes temporary flooding of adjacent roads and properties, principally along Stewart Street.

In 2013, Jacobs undertook a study[1], which identified potential solutions to address the coastal flooding problem. A condition survey was carried out and options appraisal undertaken for upgrades to the 713m long sea defences at Portgordon to alleviate the problem of coastal flooding and reduce the structure's maintenance burden. A topographical survey of the sea defences was also procured as part of the project. There has been no significant change to the defence structure since the 2013 study, with the exception of an additional baffle structure being constructed within the setback wall.

Since the 2013 study[1], there have been a number of storm events at Portgordon causing overtopping of the existing wall. This leads to flooding of the properties, which is exacerbated by the overtopped water being trapped on the landward side of the existing setback wall due to being unable to drain back to the sea at a rate that would prevent the build-up of water. The main areas experiencing flooding are Stewart Street and its junction with Lennox Place, due to naturally occurring low points at these locations.

As a result, the Moray Council objectives are to:

- Deliver a coastal flood protection scheme in the most cost-effective manner;
- Identify a preferred option with the aim of reducing overtopping and flooding at Portgordon;
- Better understand the potential flood risk; and
- Develop a business case for the preferred option.

Jacobs has undertaken a cursory visual inspection of the existing coastal defences at Portgordon in order to inform an options appraisal study on behalf of the Moray Council. The purpose of this study is to identify a preferred option to mitigate flooding of the houses at Portgordon currently affected by wave overtopping. This project includes hydrodynamic modelling of potential options to assess the performance of each at reducing overtopping. Furthermore, improved seaward drainage on the performance of each option is accounted for in the hydraulic drainage modelling with the aim of reducing the extent of flood damage. An additional topographical survey of the area landward of the setback wall was procured as part of this stage of the project to inform the hydraulic drainage model. A business case is provided including cost estimates and the likely benefit cost ratio for each option, accounting for an optimum bias which is appropriate at this stage of the development. It should be noted that optimism bias is generally reduced as projects such as this are developed and more factual information becomes available upon which to base the design of the preferred scheme.

1.2 200-year Return Period for Initial Options Study

The probability of an event occurring is characterised by a return period. It is a statistical definition of the average time that separates two occurrences of an event of the same or greater magnitude that is typically based on historical data. In general terms, the more extreme an event is, the longer its return period will be.

BS6349-1:2000[3] Maritime Structures observes that structures can generally be designed to withstand a range of extreme conditions. However, in order to withstand the more extreme events, costs could often become prohibitively expensive, and so an appropriate trade-off between cost and functionality is often sought. As such, an appropriate degree of risk usually has to be accepted when determining the design conditions for use in a flooding assessment such as this. For the purposes of this study, a 200-year return period storm event has

[3] BS6349-1:2000 Maritime Structures - Part 1: Code of practice for general criteria

been selected. This results in a 0.5% probability of occurrence in any one year (inclusive of the year following an event of the same return period).

1.3 Project Location

Portgordon is located on the Moray Coast, approximately 2 miles west of Buckie and 3 miles east of the mouth of the River Spey. This report considers the seafront area extending from the west of the harbour to the western end of Stewart Street along which the existing defences run.

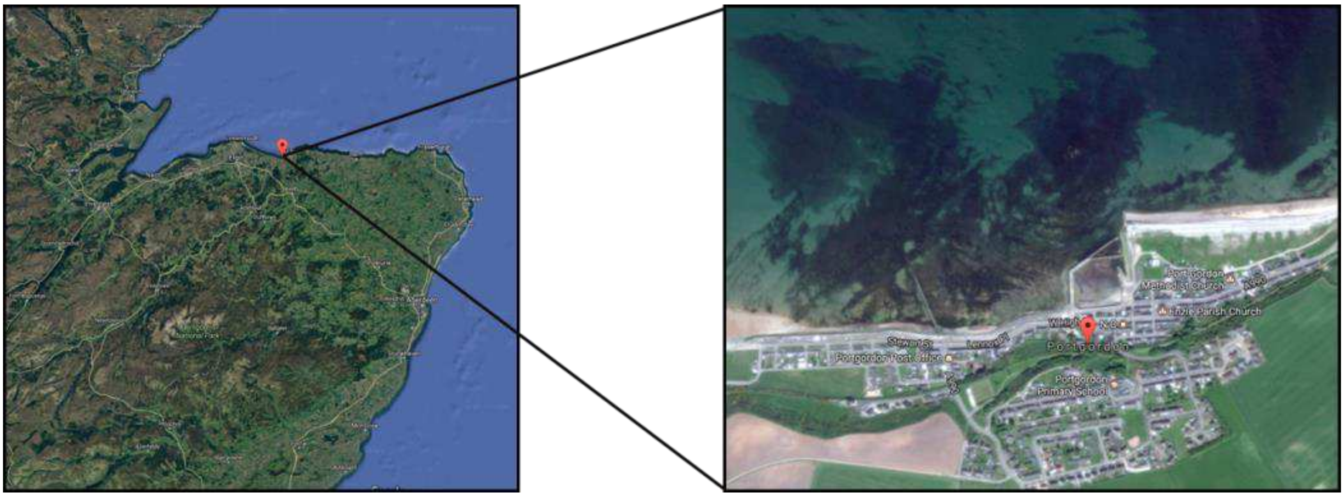

Figure 1-1: Images taken from Google Earth[4] showing an aerial view of Portgordon (right) and its location with respect to the North East of Scotland.

1.4 Report Objectives

This report includes a review of the condition of the existing coastal defences at Portgordon. An overtopping assessment is conducted for the existing and proposed sea defence options, before calculating the corresponding flood extent and depth expected. An economic assessment is provided for each of the proposed options including a business case for the preferred option.

[4] Main image map data © 2017 DigitalGlobe © Google. Imagery Date: 20/05/2014.

2. Review of Condition of Structure

A visual inspection of the sea defences at Portgordon was carried out by Jacobs on 8th March 2013. The coastal defences considered during this inspection extended approximately 713m west from Portgordon Harbour, along the A990 (Lennox Place) and Stewart Street. On 22nd December 2016, Jacobs undertook a cursory visual inspection of the same defences in order to assess the change in condition since they were last viewed in 2013. This section of the report compares the general condition of the structure observed in December 2016, with the more detailed assessment provided in the 2013 report[1].

The coastal defences comprise rock armour placed against the face of a low level concrete wave return wall. To the landward side of the concrete wall is a mortared stonework revetment, which rises to road level. At the top of the revetment slope there is a small vertical setback wall. Further details of the current arrangement are shown in Figure 2-1.

Figure 2-1: General view of coastal defences looking east towards the harbour (2016).

The current structure has been developed over a number of years. It is believed that the setback wall along the top of the current structure formed the upper section of a vertical seawall and that the revetment, wave return wall and rock armour were added in an attempt to reduce noise, vibration and wave overtopping of the original vertical wall.

2.1 Rock Armour

The rock armour is located seaward along the full length of the lower wave return wall. In 2013, it was observed that there was no obvious evidence of individual rocks having split, broken or becoming significantly rounded; comparing the condition of the rocks in 2016, it appeared as though the rocks are in similar condition.

Figure 2-2: General view of rock armour in reasonable condition (2016).

Although the individual rocks appear to be in reasonable condition, numerous sections of the rock slope look to have slumped to a shallower slope angle than previously observed. In addition, over large areas of the rock slope, sand, shingle, pebbles and seaweed has filled the gaps between the rocks; this was particularly evident at the east end of the defence, as illustrated in Figure 2-3 and Figure 2-4. The partial burial of sections of rock armour is likely to have a detrimental effect on the effectiveness of the rock armour in absorbing wave energy by reducing the permeability of the structure. The 2013 survey identified evidence of the process of burial with further deposition of material noted in December 2016.

Figure 2-3: View of rock armour at the east end of the defences, showing significant burial of the rock slope to the left of the wall and significant deposition of pebbles on the right of the wall (2016).

Figure 2-4: Rock armour has slumped and is partially buried at the western end of the coastal defences (2016).

2.2 Wave Return Wall

The wave return wall is located at the base of the revetment slope, and is generally observed to be in good condition. However, there were a number of discrete sections along the wall displaying signs of degradation. Primarily, this degradation takes the form of spalling of the concrete wall (Figure 2-5). Pitting of sections suggests that the damage may have been caused by abrasion, rather than erosion, as a result of pebbles being cast against the wall by waves during high energy wave conditions.

There is also a large volume of smaller diameter loose material on the landward side of the wave return wall. This material generally consists of shingle and pebbles, and is likely to have been repeatedly cast against the face of the wall, abrading its surface.

Rust staining was observed at points where the damage to the concrete surface of the wall was more significant (Figure 2-6). This damage appears to have been caused by impacts that have exposed reinforcing steel, which, in turn, has rusted. No evidence was observed that might suggest widespread corrosion of the steel reinforcement within the wall.

Figure 2-5: Spalling of concrete wall, exposing aggregate (2016).

Figure 2-6: Rust-staining to concrete wall (2016).

Figure 2-7: Damage to face of wave return wall (2016).

The damage to the wave return wall is similar to that identified in the 2013 report[1]; it does not appear to have deteriorated significantly since the 2013 survey:

- Widespread abrasion to the seaward and upper faces resulting in exposed aggregate and loss of aggregate;
- Concrete patch repairs to the seaward face;
- Rust staining; and
- Honeycombing on surface of concrete.

A step in the line of the wall was observed to the west of the junction of Station Road, Stewart Street and Lennox Place. The 2013 report[1] noted that this appeared to have been a design detail, rather than as a result of settlement.

2.3 Mortared Rubble Stone Revetment

Similar to the 2013 survey, the arrangement and condition of the revetment was found to vary over its length. A number of patch repairs have been carried out on an ad hoc basis, which has led to a non-uniform finish. The repairs vary from small patches to large areas of concrete, some of which extend over the full width of the revetment. It appears as though the repairs have been undertaken to address surface damage as opposed to damage of a more structural significant nature. No visual evidence was observed in this area that would suggest significant subsidence or movement in the revetment.

Figure 2-8: Concrete patch repairs on the revetment slope (2016).

Figure 2-9: Extensive concrete patch repairs. Drains are placed periodically along the length of the wall (2016).

It is assumed that the revetment was constructed using the same mortared stonework arrangement over the full length and that the larger concrete areas (Figure 2-9) represent more recent repairs.

It was also noted that a significant amount of beach material (seaweed, sand, shingle, pebbles) was observed on the revetment at the east end of the structure, adjacent to the harbour (Figure 2-10). This effect has worsened significantly since the 2013 inspection and will continue to have negative implications for the structural condition of the revetment slope. A number of the drainage holes enabling water to drain back through the wave return wall to the sea were partially blocked by this material.

Figure 2-10: Significant deposition of shingle, pebbles and seaweed on the revetment slope (2016).

2.4 Setback Wall

The setback wall along the crest of the revetment was found to be in reasonable condition, given its age and level of exposure. Numerous minor cracks and deterioration of the construction joints were observed over the full length of the wall. Although neither is thought to be of significant structural concern, both are likely to result in progressive damage if not repaired.

A number of significant cracks and areas of damage were observed (Figure 2-11 and Figure 2-12).

Figure 2-11: Damage to crest and cracking of the setback wall (2016).

Figure 2-12: Further damage and cracking to setback wall (2016).

Additionally, there are a number of features along the length of the wall that may have an impact of the way in which the coastal defences operate during overtopping. There are a number of flap valves in place that should allow water that has overtopped the wall to flow seaward in a flood event (Figure 2-13). However, some of the valves have been covered with concrete and the holes through the wall filled. Others are missing the flap valve leaving a hole through the setback wall.

Figure 2-13: Flap valve and gap in the wall blocked off by wooden planks (2016).

There are two baffle arrangements built into the setback wall; one of these was in place during the 2013 inspection, but the second is a new addition (Figure 2-14). These open baffle arrangements have angled extensions, allowing storm water to drain seaward, rather than collecting on the road behind the setback wall. The location of these two baffle walls coincides with the lowest ground level along the length of the setback wall, which will encourage water to drain naturally through these gaps in the wall.

Figure 2-14: View looking towards the Harbour end of the coastal defences, showing the older baffle arrangement in the foreground and the newer arrangement further along the setback wall (2016).

Three further gaps were observed in the setback and wall; these had wooden boards held in position in steel channel sections cast into or fixed against the setback wall (Figure 2-13).

The condition of the setback wall did not seem to differ significantly in December 2016 in comparison to the observations made during the 2013 inspection.

3. Engineering Options

In order to identify a preferred option to combat wave overtopping, it is important to understand the baseline level of protection offered by the existing defence. A hydrodynamic model predicts the performance of the existing defence for a range of return periods, the results of which are summarised in Section 4.2.1. The options modelled for the existing defence are as follows:

- Failed existing defence; and
- Maintained existing defence.

Three rock armour defence options were developed during an initial options study with the aim of reducing wave overtopping and subsequent flooding of the area behind the defences. These are as follows:

- Option 1: Rock armour berm over upper slope;
- Option 2: High rock armour berm over existing lower seawall; and
- Option 3: Rock armour berm extended seaward.

The initial three options utilise rock armour due to the energy dissipating qualities of rock armour structures that are well suited to address the overtopping issues currently experienced at Portgordon. Furthermore, the rock armour is in keeping with the existing sea defence and current rock armour may be re-used as part of the development. The distance between the seaward edge of the existing wave return wall and the setback wall varies from 11-14m, which allows a low approach angle to extract energy from incoming waves over a longer distance.

It was initially felt that a solid concrete wall would need to be of significant height to limit overtopping from spray thrown up upon wave impact with the wall. Furthermore, the cost and visual impact of such a structure were considered to be less attractive than rock armour options, which could potentially achieve a similar overtopping performance with a lower crest height. However, in order to provide a complete comparison of the available options, two concrete defence options have been identified for detailed comparison with the rock armour options as follows:

- Option 4: Stepped revetment; and
- Option 5: Wave return wall.

A preliminary investigation was undertaken by ABPmer to refine the geometry for each defence option, refer to Section 4.2.2 for further details. However, should an option be taken forward to the detailed design stage, the levels identified herein may be refined further to optimise the overtopping performance.

3.1 Design Tidal Event

This study considers defence options for a design flood level based on a 200-year return period (0.5% Annual Exceedance Probability (AEP)) tidal event, defined as suitable for residential, institutional, commercial and industrial development by the Scottish Planning Policy[5]. In addition, the performance of the baseline 'Failed Existing Defence' and 'Maintained Existing Defence' options have been considered for a range of events, from 10-year to 1,000-year return periods, as a sensitivity study. The results are detailed in Section 4.2.1.

It has been assumed that construction would be complete by 2022 and any new sea defence structure would have a design life of 100-years. Therefore, a 100-year allowance for climate change and sea level rise has been applied allowed for using UKCP09[6] medium emissions 95th percentile predictions.

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[5] Scottish Planning Policy, June 2014, The Scottish Government
[6] United Kingdom Climate Projections, 2009

3.2 Failed Existing Defence (Do Nothing)

This option represents a scenario in which all maintenance ceases and no repairs to the existing structure are undertaken. It is expected that the structure deteriorates further in this scenario as regular maintenance work is not carried out. Historically, repairs to the structure have typically been undertaken as a response to storm damage and general deterioration. Failure to maintain this approach is likely to lead to accelerated damage and deterioration of the revetment slope and wave return wall in particular.

The failed existing defence assumes that gaps and voids between the rock armour are filled with loose sand, shingle and pebbles reducing the effectiveness of the rock amour in absorbing wave energy. The lower concrete wave return wall will continue to deteriorate due to abrasion. This option assumes that there will be an increase in the level of damage to the mortared rubble stone revetment. The setback wall is assumed to be partially collapsed for the purposes of the overtopping assessment but conservatively assumed to be in place to restrict drainage towards the sea.

3.3 Maintained Existing Defence (Do Minimum)

This option represents the current scenario, including ongoing maintenance of the existing defences. There are no significant changes to the layout of the existing structure. Repairs are undertaken to stabilise the condition of the structure and to mitigate further deterioration; these are envisaged to include:

- Replacement of sections of the setback wall where there are significant cracks and where the concrete is chipped and broken. These sections are broken out and new sections of the wall cast.
- Less significant cracks in the setback wall are grouted in an effort to mitigate further deterioration of the wall.
- More substantial concrete patch repairs to areas of damage on mortared rubble stone revetment slope. This includes breaking out areas of damage to a reasonable depth and removing any loose material. Mass concrete is considered to be suitable for this work.

3.4 Option 1: Rock Armour Berm Over Upper Slope

Option 1 involves the construction of a new rock armour berm structure over the existing defence, as shown in Figure 3-1. This option may take in the region of 12 months to construct and includes the following:

- Demolish the existing setback wall adjacent to the pavement.
- Install new drainage pipes beneath the new setback wall location. The drainage pipes are required to drain surface water from the road back towards the sea and should be designed such that the pipes can be cleared of blockages and be readily maintained throughout their design life. The outlets should also be protected from damage from misplaced rock armour or otherwise.
- Construct a new reinforced concrete setback wall with a crest level of +5.48m ODN. Incorporate baffle structures, similar to that photographed in Figure 2-14, at approximately 50m centres to aid the flow of flood water from the road towards the sea. It is understood from The Moray Council that the three baffles installed in the current defence have offered mixed success in their effectiveness at providing a flow path back to the sea for water that has overtopped the setback wall. Although a limited volume of water can flow landward through the baffles due to wave action, it is considered that the benefit of draining overtopped flood water outweighs the drawback the baffles offer overall.
- Following construction of the new setback wall, reinstate the pavement along the length of the wall.
- Move the existing rock armour and reuse within the scheme. Remove any loose material between the existing rock armour units and use to replenish the beach.

- Excavate a trench seaward of the existing wave return wall in order to toe the new berm into the seabed. There is potential for rock excavation to be necessary to achieve the required depth.

- Leave the existing wave return wall in place beneath the new rock armour.

- Place granular fill seaward of the existing wave return wall and compact to form a core for the new rock armour.

- Build up rock armour to a berm elevation level of +5.44m ODN, extending out a distance of approximately 12.5 metres from the setback wall before sloping down to the excavated toe below seabed level. This comprises a layer of geotextile, two layers of smaller, secondary rock armour, and two layers of primary rock armour formed of larger rocks. The rocks should be appropriately sized so as to resist the force of the waves. Depending on the size of rock armour required and the range of sizes of rock armour found on site, some of the existing rock may be required to be crushed to make them suitable for use as secondary amour.

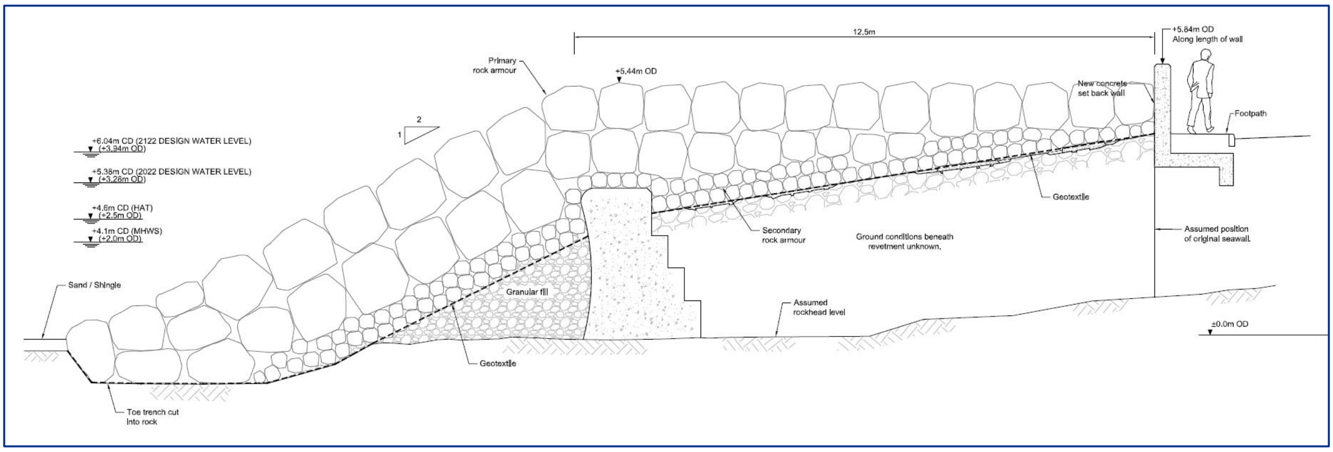

Figure 3-1: Drawing showing proposed cross section of Option 1.

3.5 Option 2: High Rock Armour Berm Over Existing Lower Seawall

Option 2 is a rock armour berm with a four-metre-wide crest, built over the existing wave return wall, which may also take in the region of 12 months to construct. The key aspects of the scheme are illustrated in Figure 3-2 and are as follows:

- As with Option 1, move the existing rock armour and reuse within the scheme. Remove any loose material between the existing rock armour units and use to replenish the beach.

- Excavate a trench seaward of the existing wave return wall in order to toe the new berm into the seabed. There is potential for rock excavation to be necessary to achieve the required depth.

- Leave the existing wave return wall in place beneath the new rock armour.

- Place granular fill seaward of the existing wave return wall and compact to form a core for the new rock armour.

- Build up rock armour to a crest elevation of +6.34m ODN, which is 500mm higher than Option 1. The crest is four metres wide, sloping down the existing revetment slope on the landward side and into the excavated toe on the seaward side of the wave return wall. As with Option 1, this comprises a layer of geotextile, a double layer of secondary rock armour and a double layer of primary rock armour. As discussed above, the rocks need to be appropriately sized for the wave climate therefore some of the existing rock may need to be resized to make it suitable for reuse.

- Unlike Option 1, the existing setback wall is not demolished. Patch repairs are undertaken where required along the length of the wall.

- Retain the existing road drainage system as the rock armour berm for Option 2 is expected to terminate seaward of the existing outfalls, facilitating unrestricted maintenance of the system.

- Install baffle structures at approximately 50m centres along the length of the existing setback wall in addition to those already installed to aid the flow of flood water from the road towards the sea.

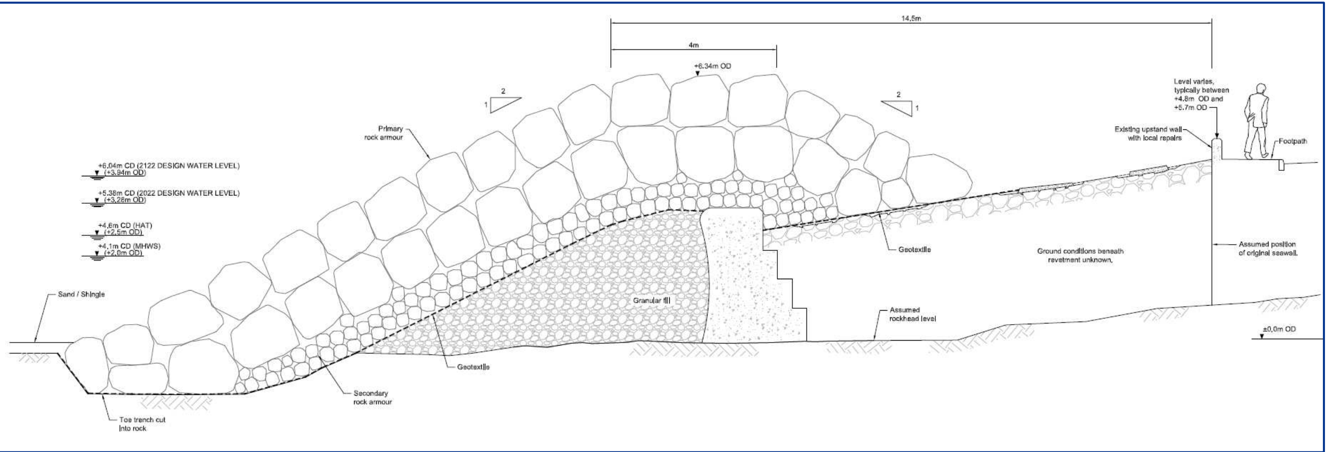

Figure 3-2: Drawing showing proposed cross section of Option 2.

3.6 Option 3: Rock Armour Berm Extended Seaward

Option 3 is similar to Option 1, although the berm is at a lower height and extends further seaward before sloping back down into an excavated toe (Figure 3-3). This option may take in the region of 12 to 18 months to construct. Option 3 consists of the following:

- Similar to Option 1, demolish the existing setback wall adjacent to the pavement

- Install new drainage pipes beneath the new setback wall location. As with Option 1, the new drainage system should be designed such that the pipes can be cleared of blockages and be readily maintained throughout their design life. The outlets should also be protected from damage from misplaced rock armour or otherwise.

- Construct a new reinforced concrete setback wall with a crest level of +5.34m ODN, which is marginally lower than the proposed wall for Option 1. Incorporate baffle structures at approximately 50m centres along the length of the setback wall to aid the flow of flood water from the road towards the sea.

- Following the construction of the new setback wall, reinstate the pavement along the length of the wall.

- Similar to the previous two options, move the existing rock armour and reuse within the scheme. Remove any loose material between the existing rock armour units and use to replenish the beach.

- Excavate a trench seaward of the existing wave return wall in order to toe the new berm into the seabed. There is potential for rock excavation to be necessary to achieve the required depth.

- A significantly larger core is required for Option 3, due to the berm being approximately 10 m to 12 m wider than Options 1 and 2. Leave the existing wave return wall in place to form part of this core.

- Place granular fill seaward of the existing wave return wall and compact to complete the core for the new rock armour.

- Build up rock armour to a berm elevation level of +4.94m ODN, which is the lowest of the three options. The crest extends 21.5m seaward of the setback wall before sloping down into the excavated toe below seabed level. The rock armour berm is made up of the same constituents as both Options 1 and 2.

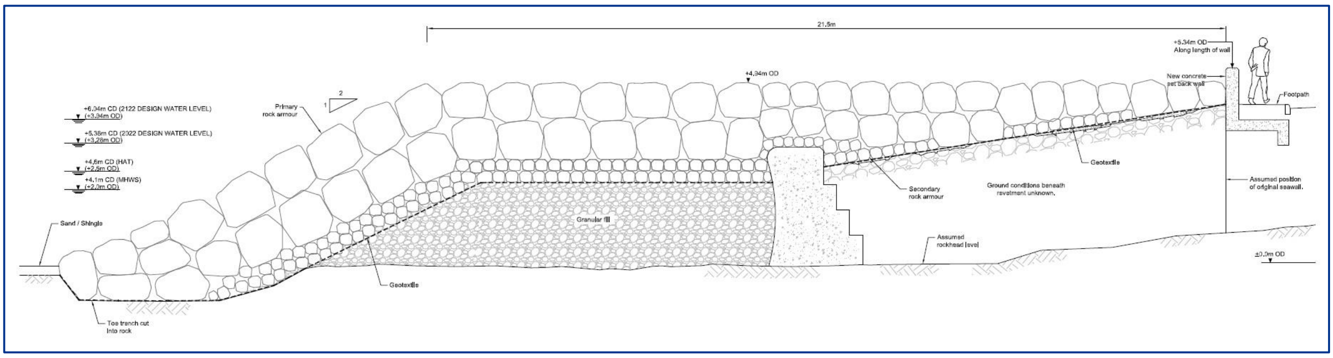

Figure 3-3: Drawing showing proposed cross section of Option 3.

3.7 Option 4: Stepped Revetment

Option 4 is a composite solution comprising a rock armour revetment seaward of the existing wave return wall and a concrete stepped revetment beginning at the existing wave return wall and rising landward. The top step of the revetment incorporates a bullnose wave return wall, as shown in Figure 3-4. This option may take in the region of 12 to 18 months to construct and includes the following:

- Move the existing rock armour and reuse within the scheme. Remove any loose material between the existing rock armour units and use to replenish the beach.
- Excavate landward of the existing wave return wall to a depth of approximately 1.5m. Excavate a trench seaward of the existing wave return wall in order to toe the new berm into the seabed. There is potential for rock excavation to be necessary to achieve the required depth.
- Build up rock armour from the excavated toe rising landward to form a revetment leading up to the top seaward edge of the existing wave return wall.
- Place 1.5m of granular fill behind the existing wave return wall and compact as required.
- Install a concrete stepped revetment structure, rising landward from the existing wave return wall, incorporating a bullnose wave return wall as part of the top step with a crest elevation of +6.84m ODN. The make-up of the revetment structure includes 450mm diameter drainage pipes at 5m centres, which returns overtopped water to the sea. The drainage pipes through the stepped revetment are designed with flap valves or similar to prevent flow of water through the defence in a landward direction during a high water event.
- Undertake patch repairs to the existing setback wall where required.
- Retain the existing road drainage system as the stepped revetment is expected to terminate seaward of the existing outfalls, facilitating unrestricted maintenance of the system.
- Install baffle wall structures at approximately 50m centres along the length of the existing setback wall in addition to those already installed to aid the flow of flood water from the road towards the sea.

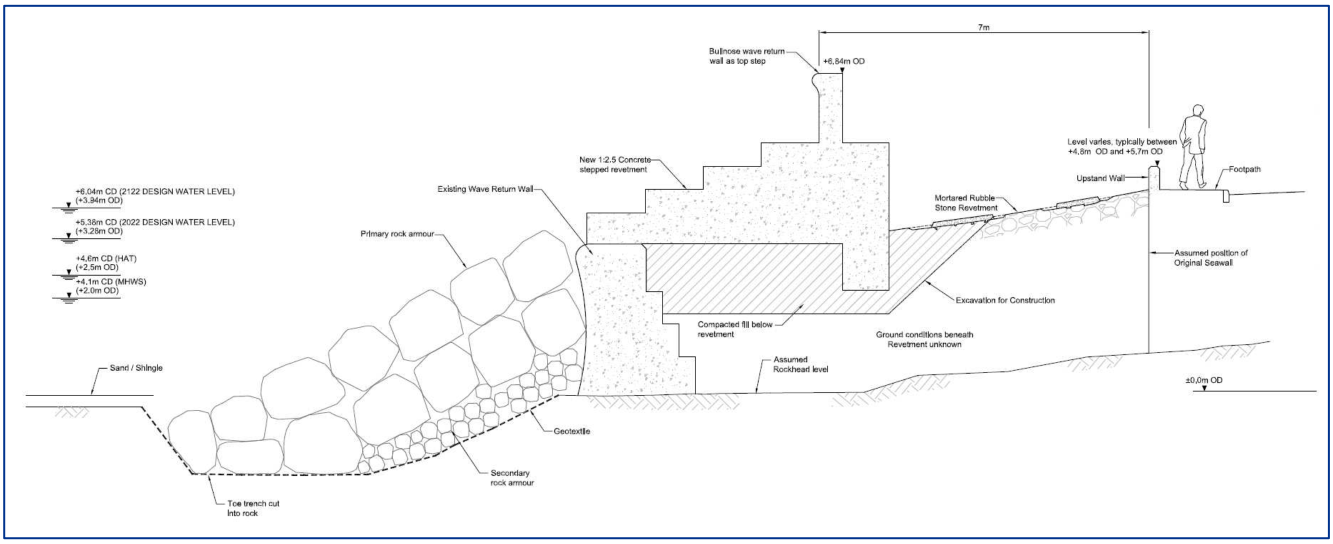

Figure 3-4: Drawing showing proposed cross section of Option 4.

3.8 Option 5: Wave Return Wall

Option 5 is an enlarged wave return wall, encompassing the existing wave return wall, as shown in Figure 3-5. Option 5 utilises an embedded toe and raised fill level behind the wall for stability purposes. This option may take in the region of 12 to 18 months to construct and includes the following:

- Remove the existing rock armour.
- Excavate the material behind the existing wave return wall, as indicated in Figure 3-5.
- Excavate a trench seaward of the existing wave return wall in order to toe the new wave return wall into the seabed. There is potential for rock excavation to be necessary to achieve the required depth.
- Construct a new wave return wall with a crest elevation of +6.34m ODN, encompassing the existing wave return wall and embedded into rock seaward of the existing wall, as indicated in Figure 3-5. The make-up of the wave return wall includes circa 450mm diameter drainage pipes at 5m centres, which return overtopped water to the sea. The drainage pipes through wave return wall would be designed with flap valves or similar to prevent flow of water through the defence in a landward direction during a high water event.
- Undertake patch repairs to the existing setback wall where required.
- Alter the existing road drainage system as appropriate to allow water to flow seaward through the new wave return wall.
- Install baffle wall structures at approximately 50m centres along the length of the existing setback wall in addition to those already installed to aid the flow of flood water from the road towards the sea.

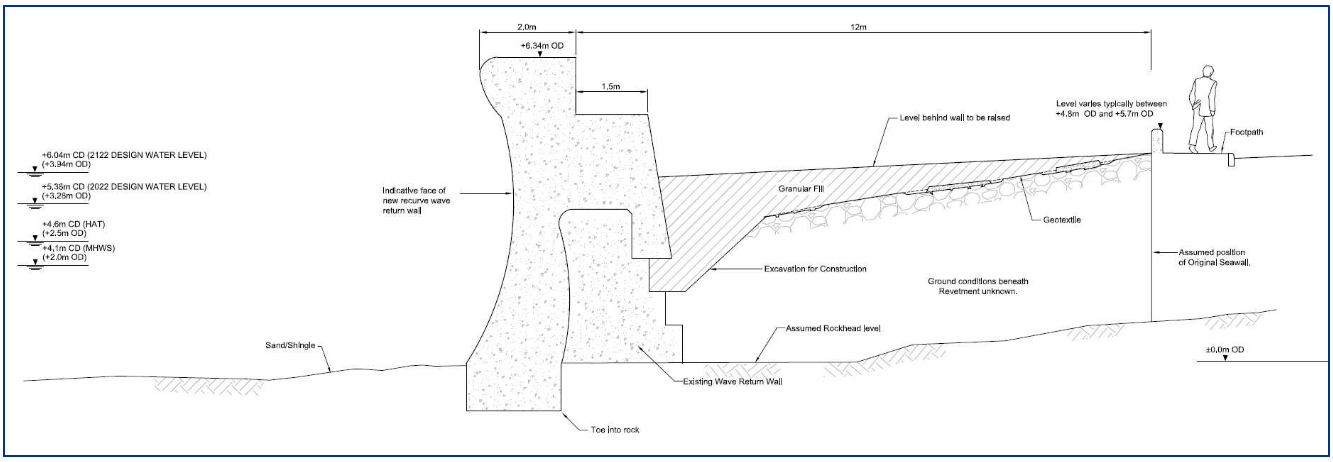

Figure 3-5: Drawing showing proposed cross section of Option 5.

3.9 Other Potential Options

A number of alternative options are considered below, though in less detail than Options 1 to 5; for various reasons discussed, they are not considered further in this study.

1. **Offshore reef:** This option comprises the construction of an offshore breakwater structure with the intention of dissipating the energy of the approaching waves and reducing overtopping. The structure is likely to consist of concrete or natural blocks sunk offshore to alter the wave direction and take the energy out of the waves. They are fairly durable and do not generally require much maintenance. The construction of this solution is likely to require a large volume of material. Furthermore, offshore construction can be complex and expensive. Offshore options are outwith the scope of the modelling study; therefore, it is not appropriate to put forward an option in this report without modelling it.

2. **Individual Property Protection:** It is understood from photographs that there are varying levels of individual property protection currently in place at Portgordon. Installing property protection at each property along the seafront, such as temporary flood gates on the doors, does not reduce overtopping, but it could reduce the flood risk for the properties on the seafront. However, it should be noted that flood gates to individual properties would not necessarily reduce the risk of damage occurring to windows, doors or walls due to impact from spay and debris. Furthermore, emergency access to properties in case of emergency would still be restricted during an overtopping event.

   This measure should be considered by the Moray Council, but is not considered further in this report due the fact that it does not alter the overtopping scenario and cannot be modelled. This option would need further discussion and consideration to determine its feasibility at the location.

3. **Set-back defences along western 300-400 m of the seafront:** At the western half of the site, there is a wide area of land between the existing defences and the properties in which some form of set-back defence structure could be built. The form of this structure is not considered fully within this report. Insertion of a new defence closer to the properties could affect the individual drainage systems, should the area flood in an extreme event. Therefore, if a defence were considered in this location, it would require careful monitoring during its operational phase. Furthermore, it would offer no protection to the properties located closer to the sea adjacent to the eastern half of the site.

4. **Pumping options**: There is also an opportunity to pump floodwater from behind the setback wall to alleviate flooding. The drainage system would need to be updated with the addition of a new combined kerb drainage system along the road connected to a new pumping station. Although this option warrants consideration, it is not preferable for the following reasons: it requires regular maintenance; the mechanical equipment is likely to be prone to maintenance; this option does not reduce the volume of overtopped water and therefore may not alleviate the short term flooding problem.

3.10 Miscellaneous Details

There are a number of details common to the five options considered herein, which require consideration. These include: existing manholes, road drainage through the proposed defence, and the technical detail at the eastern and western extents of the revetments.

There is currently a series of manholes in place on the existing revetment slope. Details of the manholes are unknown, but assuming that access would need to be preserved for maintenance purposes, they would remain uncovered by rock armour at their discrete locations.

There are currently a series of drainage pipes leading from roadside gullies under the setback wall, with t-head outfalls on the revetment slope. For the purposes of this study, it is assumed that these will remain in place for Options 2 and 4 and replaced with a higher capacity drainage system for Options 1 and 3. The road drainage system for Option 5 will be altered as appropriate to allow water to flow through the new wave return wall. The condition of the current system is unknown; pipes may be blocked and / or be damaged or corroded. In any case, the roadside gullies cannot be relied upon to assist with the drainage of overtopped water back towards the sea.

The structural detail at the western end of the defence requires further development. At the eastern end, the defence naturally meets the harbour wall. In contrast, the setback wall gradually slopes downward towards the western end and terminates abruptly so that the ground level is lower than the typical height of the setback wall. This has the potential to both increase and decrease the flood level locally. Firstly, there is a possibility that water may flow around the end of the defence during an event, inundating the area behind the setback wall. However, the low ground level in comparison to the setback wall acts as a control point so that overtopped water can drain seaward.

4. Summary of Hydrodynamic Modelling

Jacobs procured Associated British Ports Marine Environmental Research (hereafter referred to as “ABPmer”) to undertake wave modelling and overtopping analysis to support the development and appraisal of the options. This section summarises the *Portgordon Wave and Overtopping Modelling Report* (Appendix A) produced for the initial three defence options and the *Portgordon Sea Defence Options Overtopping Assessment* (Appendix B) produced for the additional two defence options by ABPmer.

4.1 Wave Modelling

In order to undertake an overtopping assessment of the defence at Portgordon, wave and still water levels at the toe of the structure are required. To determine the wave conditions at the toe of the defence, wave modelling was undertaken to transform offshore wave conditions inshore. Water levels taken from ABPmer (UKCS) tide and surge model, which is based on data from the period 1979 to 2015, have been adjusted to account for climate change using UKCP09 medium emissions 95th percentile predictions. To account for sea level rise between present day and 2022, an increase of 0.03m was applied; for 2022 to 2122, a further increase of 0.65m was applied.

Although the defence is of similar construction along the length, the height of the setback wall and wave return wall vary along the length. To determine which cross section of the structure to model, the defence length was split into three sections (Figure 4-1) and associated wave parameters calculated. The largest waves occur at the Defence 3 (“Def_3”) western section of the wall due to a combination of this defence section being lower than the other two sections and its more exposed orientation and location. The maximum water levels and associated wave heights at the toe of the wall are shown in Table 4-1.

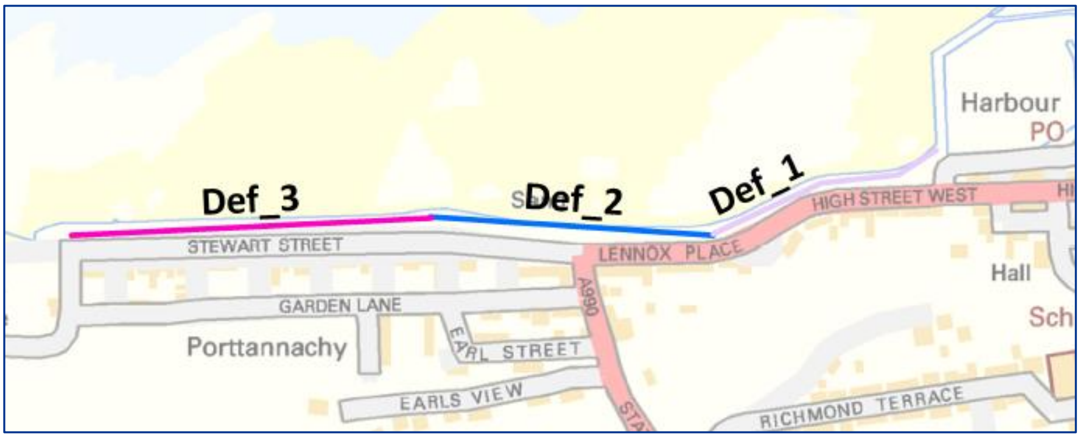

Figure 4-1: Indicative location of the three defence sections.

Table 4-1: Maximum water levels and wave heights from ABPmer wave model (based on 37 years of hindcast data).

Year	Maximum water level at the toe of the structure (m ODN)	Maximum predicted significant wave height at the toe of the structure (m ODN)
2022	+3.28	+2.99
2122	+3.94	+3.31

4.2 Overtopping Assessment

The wave model results are used as an input into the overtopping assessment. 37 years of data were extracted from ABPmer's SEASTATES hindcast[7] and wave parameters paired with the coincident water level. This allows a continuous time-series record of overtopping rate for both 2022 and 2122, which means that the overtopping assessment is based on a large set of historical combinations rather than extreme water levels that are defined offshore.

ABPmer calculated the overtopping using the Neural Network Tool (NNT) developed under EurOtop (2007)[2]. The NNT assesses the overtopping performance of sea defences and related structures. The event combinations that create the largest overtopping rates were used to produce the hydrographs for an event duration of 25 hours as per Environment Agency (2011)[8] methodology, which was originally developed with the support of the Scottish Environment Protection Agency (SEPA). The mean overtopping rate for each 5-minute increment during the 25-hour event is calculated. The results presented in the following sections represent the peak 5-minute mean overtopping rates from the model. During the 25-hour event period modelled, there are two tidal cycles, resulting in two peak water level events.

ABPmer validated the results of the overtopping assessment of the existing defence with photographs[9] from previous flood events and EurOtop (2007)[2] guidance. EurOtop (2007)[2] provide guidance on safe limits for overtopping rates which are detailed in full in Appendix A.

As the overtopping assessment was only undertaken for the Def_3 cross section for Options 1-3 , approximate reduction factors based on experience and two validation events were applied to the rates in order to calculate the volume of overtopping along the length of Defence 1 ("Def_1") and Defence 2 ("Def_2"). These factors reflect that the crest elevations of Def_1 are higher than Def_2, and those of Def_2 are higher than Def_3. For Options 4 and 5, the wave overtopping flow hydrographs and the associated tidal hydrographs were provided at each of the three defence sections. Scaling factors were not required to be applied to these.

Table 4-2: Approximate reduction factors at each section of sea defences.

Defence Section	Reduction Factor (%)
Def_1	35
Def_2	20
Def_3	0

Due to waves reflecting from the harbour arm, the area of the defence at the eastern end of the site may experience increased overtopping. Areas such as this may require further consideration if an option is taken forward to detailed design stage.

4.2.1 Existing Defence

ABPmer ran the overtopping assessment for the maintained and failed existing defence. The results below indicate that the overtopping rate at the setback wall for the existing defence is greater than 10 l/s/m which is the limit suggested by EurOtop (2007)[2] for well-prepared pedestrians and driving vehicles at low speeds. This level highlights the poor performance of the existing defences at Portgordon.

[7] www.seastates.net
[8] Environment Agency. 2011. Coastal flood boundary conditions for UK mainland and islands. Project: SC060064/TR4: Practical guidance design sea levels
[9] Photographs provided by the Moray Council from storm events in05/12 2013, 09/102014 and 13/01/2017

Table 4-3: Calculated "Failed Existing Defence" peak (5-minute mean) overtopping rates at the setback wall during a 25-hour event duration for Defence 3 per return period for base year 2022 and climate change scenario 2122.

Return Period (years)	Mean Overtopping Rate (l/s/m)	
	2022	2122
10	75.21	155.17
50	125.79	248.14
100	153.58	299.58
200	185.84	359.55
1,000	282.19	539.99

Table 4-4: Calculated "Maintained Existing Defence" peak (5-minute mean) overtopping rates at the setback wall during a 25-hour event duration for Defence 3 per return period for base year 2022 and climate change scenario 2122.

Return Period (years)	Mean Overtopping Rate (l/s/m)	
	2022	2122
10	58.65	133.74
50	101.54	213.52
100	125.62	256.38
200	153.94	305.46
1,000	240.51	448.61

4.2.2 Defence Options

ABPmer initially modelled the three rock armour defence options to determine the overtopping rates for both the 2022 and 2122 epochs. An initial sensitivity study investigated the effect of variation of the crest height and berm height for each option. The results from the initial study can be found in Appendix A. The parameters used in the overtopping assessment are outlined in Section 3.

ABPmer subsequently modelled the additional two concrete defence options for comparison with the rock armour options for the 2122 epoch only. An initial sensitivity study investigated the effect of variation of crest height and revetment composition for Option 4, and crest height for Option 5. The results from the initial study can be found in Appendix B.

The results from the overtopping assessment for the five options considered in this study for the 1 in 200-year event are shown in Table 4-5. The overtopping assessment results for the five options considered indicate that Option 2 reduces the rate of overtopping more than the other options considered herein.

It should be noted that due to restrictions in the EurOtop assessment method, the overtopping rates for Options 2, 4 and 5 are for the point at the crest of the berm or concrete structure. Therefore, a further calculation has been undertaken to determine the approximate overtopping at the location of the setback wall. Therefore, the setback wall is not considered in the assessment for Options 2, 4 and 5. However, overtopping rates shown are considered conservative and appropriate for this stage of the study.

Table 4-5: Calculated peak (5-minute mean) overtopping rates at the setback wall during a 1 in 200-year event of 25-hour duration for each defence option design for year 2122 including climate change.

Defence Option	Mean Overtopping Rate (l/s/m)
1	67.35
2	2.16
3	27.35
4	7.57
5	4.42

4.3 Overtopping Model Limitations

It should be noted that the model does not account for a number of processes contributing to both the rate of overtopping and the volume of water accumulated behind the setback wall during an overtopping event event. Therefore, the results tabulated above should be considered as guidance only.

The effects of wind cannot be modelled with EurOtop; it is likely that high onshore winds combined with high water level conditions would cause the rate of overtopping to increase.

The NNT used by ABPmer to calculate overtopping is not capable of recognising a permeable rock armour material. Instead, it utilises a roughness reduction factor on an impermeable surface. The roughness factors used are based on empirical data from experimental investigations which are considered to be appropriate for preliminary design estimates. However, physical modelling is recommended if the project proceeds to subsequent stages, in line with industry guidance[10].

Physical modelling is further recommended where wave overtopping is critical as overtopping is affected by several factors whose individual and combined influences are difficult to predict[2]. Theoretical or numerical approaches are appropriate to provide overtopping estimates at this stage; yet physical modelling should be considered in subsequent stages to validate and refine the preferred option.

[10] CIRIA C683 - The Rock Manual, 2nd Edition, CIRIA, 2007

5. Flood Modelling and Mapping

In order to inform an economic assessment for each option, hydraulic modelling is used to translate the overtopping assessment results into flood extents and flood depth grids. The resulting flood maps are used as a basis for calculating the extent of potential flood damages to adjacent properties.

5.1 Outline Methodology

A two-dimensional (2D) hydraulic model was built using Tuflow (version 2016-03-AE-iDP-w64). The model extent and key features used in the model schematization are shown in Figure 5-1. As shown, the sea defence is split into the same three sections used for the overtopping assessment (refer to Section 4.1).

The model represents the wave overtopping flows over the setback wall calculated by ABPmer (refer to Section 4), the resulting overland runoff, back towards the sea, by gravity and the impact of the baffles and the drainage system (i.e. pipes through and beneath the setback and wave return walls). The resulting model outputs are water depth, overland velocities and flows. Maximum flood water depth maps are produced from the results and the outputs are used to inform the economic assessment and business case. Refer to Appendix D for details of the model verification process.

5.1.1 Assumptions and Limitations

The accuracy and validity of the flood model results is dependent on the accuracy of the hydrodynamic model and topographic data included in the model. While appropriate available information has been used to construct the model, there are assumptions and limitations associated with this work. These are as follows:

- The LiDAR data used to inform the model with ground elevation information has a horizontal resolution of 1m. In the model, this was further resampled using a 2m square grid in Tuflow. This resolution is deemed appropriate for predicting the flooding mechanisms over the study area to a sufficient level of accuracy for this stage of the project;
- The floodplain downstream boundaries of the model assume free flow. They are located far enough from the area of interest to have a negligible effect;
- The baffles were modelled using a 1D weir approach. This is deemed to be an appropriate representation;
- The road gulley drainage and pipes through the existing setback wall have been conservatively assumed to have a negligible impact on flood depth and flood extent on the basis that the drainage baffles and the control point at the western extent of the setback wall will be the principal drainage route;
- The model has not been quantitatively calibrated. Model performance has been checked as well as the consistency of model results; and
- For Options 1 and 3, it is assumed that the rock armour around the baffles can be arranged such that it does not impede the return flow. A flow width reduction of 10% has been applied to the drainage model to capture any minimal restricting effect.

The overtopping performance of each defence option has been assessed with calculations undertaken by ABPmer (refer to Section 4). Reference should be made to the associated reports (Appendix A and Appendix B) for assumptions used therein. For the baseline scenarios and Options 1, 2 and 3 overtopping volumes have been assessed at Defence 3. Reduction factors have been applied to estimate the overtopping volumes at Defences 1 and 2. Separate overtopping volume calculations at each of the defences may be more representative. For Options 4 and 5, reduction factors are not required as overtopping volumes have been assessed separately at Defences 1-3.

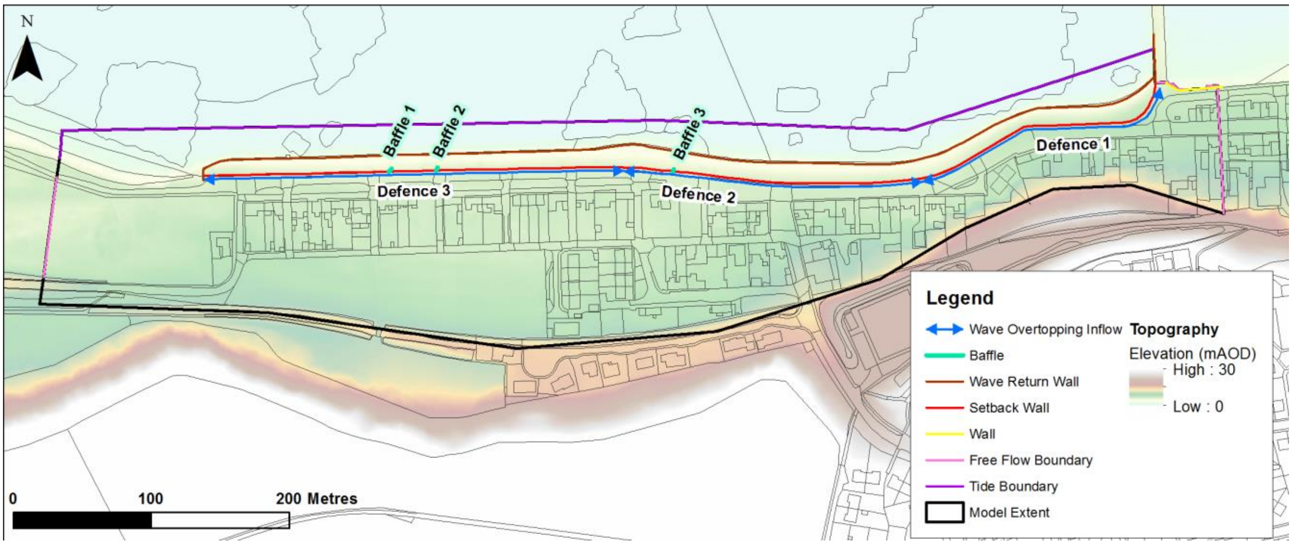

Figure 5-1: Drainage model schematisation for baseline scenario.

5.1.2 Input Data

The data used to build the hydraulic model is summarised in Table 5-1.

Table 5-1: Data used to build the hydraulic model.

Data	Description	Source
LiDAR	Filtered LiDAR (Light Detection And Ranging) data. Used to inform the hydraulic model with ground elevation information. Refer to Section 5.3	Open LiDAR from the Scottish Remote Sensing Portal https://remotesensingdata.gov.scot/
OS maps	MasterMap data 1 to 10,000 Scale Raster Refer to Section 5.3	Ordnance Survey
Sea defences survey	Survey of Portgordon seafront including topographic levels for wave return wall, revetment and setback wall, including the baffle structures and drainage system levels and locations Refer to Section 5.3	Jacobs
Wave overtopping hydrographs	Time series of wave overtopping flows on two tidal cycles Refer to Section 5.2	Associated British Ports Marine Environmental Research (ABPmer)
Tidal hydrographs	Time series of tidal levels corresponding to the overtopping hydrographs Refer to Section 5.2	Associated British Ports Marine Environmental Research (ABPmer)
Design of the options	Option 1: ND800401_002 Rev P02 Option 1.dwg Option 2: ND800401_003 Rev P02 Option 2.dwg Option 3: ND800401_004 Rev P02 Option 3.dwg Option 4: ND800401_005 Rev P01 Option 4.dwg Option 5: ND800401_006 Rev P01 Option 5.dwg Refer to Appendix C.	Jacobs

5.2 Wave Overtopping and Tide

ABPmer has undertaken wave modelling and overtopping analysis to support the development and appraisal of the options. Refer to Section 4 for further details.

5.2.1 Wave Overtopping

For the baseline scenarios and for Options 1, 2 and 3, ABPmer provided wave overtopping hydrographs (in litres/second/metre units) and the associated tidal hydrographs (head-time) along the length of Defence 3. Reduction factors were applied to the Defence 3 flow hydrographs to determine wave overtopping volumes along Defences 1 and 2 in line with Table 4-2.

For Options 4 and 5, the wave overtopping hydrographs and the associated tidal hydrographs were provided at each of the three defences. Therefore, scaling factors were not required.

5.2.2 Tide

The tidal hydrographs were provided by ABPmer for the 1 in 200-year event for all the modelled scenarios. Although the hydrographs include two tidal cycles, each model simulation was run for the first tidal cycle (15hrs) only. This provides the largest wave overtopping volumes and resulting flood depth predicted by the Tuflow model.

Table 5-2 gives the maximum tide level at each defence for all the modelled scenarios. The overtopping assessment was conducted for a collection of events based on hind cast data to determine the worst case overtopping rate for each structure (refer to Section 4.2). Since the severity of each event is a product of both tide level and wave height, no single event produced the highest overtopping rate for every option. The same tidal data was applied to the 2022 epoch for the baseline scenarios (refer to Section 5.3.1).

Table 5-2: Maximum sea levels at the defences for the 2122 epoch.

Scenario	Maximum Tide Level (m ODN) at 2122 Epoch		
	Defence 1	Defence 2	Defence 3
Baseline	+2.90	+2.90	+2.90
Do Nothing	+2.90	+2.90	+2.90
Option 1, 2 and 3	+2.90	+2.90	+2.90
Option 4	+3.56	+3.56	+3.56
Option 5	+3.29	+2.91	+2.91

5.3 Baseline Model Schematisation

5.3.1 Do Minimum

The Do Minimum scenario represents the current status of the site. This was based primarily on the topographic survey of the key features and LiDAR data.

Topography

The 2D domain covers an area of $0.11km^2$ (Figure 5-1). The topography is represented using a 2m resolution square grid in Tuflow. The levels for the grid cells are based on a Digital Terrain Model (DTM) derived from the LiDAR dataset.

No modifications were made to the model to apply a threshold level for buildings.

Sea Defences

The elements of the existing sea defences which have been modelled comprise:

- The wave return wall located at the base of the revetment slope;
- The revetment slope which rises up to the road level; and
- The vertical setback wall at the top of the revetment slope next to the road.

Levels of the sea defences, including the revetment slope, the wave return wall crest and the setback wall crest were extracted from the survey data.

Drainage system

Three baffles through the setback wall were modelled using a 1D (ESTRY) approach because the openings for the baffles were smaller than the 2D grid cell size. ESTRY weir units were used and their dimensions were extracted from the survey dataset. The most restrictive opening of the baffles has been considered for the weir width. A wall was modelled on the seaward side of the baffles which height was extracted from the survey dataset.

Other drainage systems in the study area, including road gulley drainage and drainage pipes through the existing setback wall, were assumed to have a negligible impact on flood depth and flood extent, and therefore were not included in the model.

Hydraulic Friction

Hydraulic roughness coefficients were applied over each grid cell of the 2D domain, as shown in Table 5-3, depending on land use taken from OS MasterMap data.

Buildings have been represented as high roughness polygons defined by the building footprint. The ground levels at property locations have not been modified (uplifted) to account for threshold levels. By applying a high roughness as opposed to raising the building footprint it allows for water to pond in the building footprint while still forming an obstruction to flow.

Table 5-3: Manning's 'n' coefficients.

Land use	Feature Code	Manning's n
Roads, tracks and paths, manmade structures	10185, 10172, 10123, 10119, 10054	0.025
Roadside (short grass)	10183	0.035
Buildings	10021	1.000
Land, trees, rough grassland	10111	0.100
Property gardens	10053	0.050
Land, slope, manmade, embankment	10096	0.050
Open land, general surface, revetment slope	10056	0.055
Tidal water	10203	0.060

Boundary Conditions

- Wave Overtopping Inflows

Overtopping hydrographs assessment conducted by ABPmer (refer to Section 4) provided wave i.e. flow time series due to defences wave overtopping (refer to Figure 5-2 and Figure 5-3). These were used as inflows in the model at the landward side of the setback wall.

In the model, the wave overtopping inflow is applied at each grid cell along the boundary line as a flow vs time boundary (2d_bc_ST). Factor were applied to take into account the length of the defences relative to the number of Tuflow cells that the boundary line is intersecting.

- Tide Levels

ABPmer provided tidal hydrographs i.e. sea level time series corresponding to the associated overtopping hydrographs (refer to Figure 5-2 and Figure 5-3). Those levels were used as boundary conditions along the seaward border of the 2D model.

The maximum tide level applied to the baseline model is +2.90 mODN for the year 2122 in the Do Minimum scenario. This level is much lower than all outlet invert levels (minimum of +3.439 mODN) of the drainage system discharging on the revetment slope. Therefore, this tidal hydrograph has also been used for the Do Minimum scenario for the year 2022.

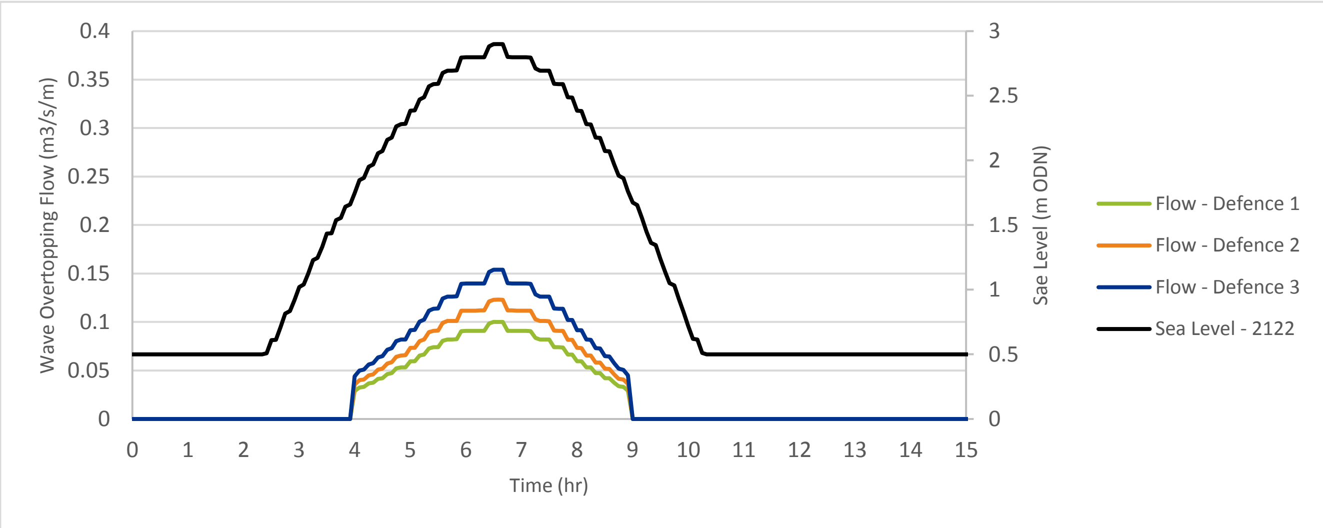

Figure 5-2: Wave overtopping hydrographs and tidal hydrograph – Do Minimum, 2022 epoch.

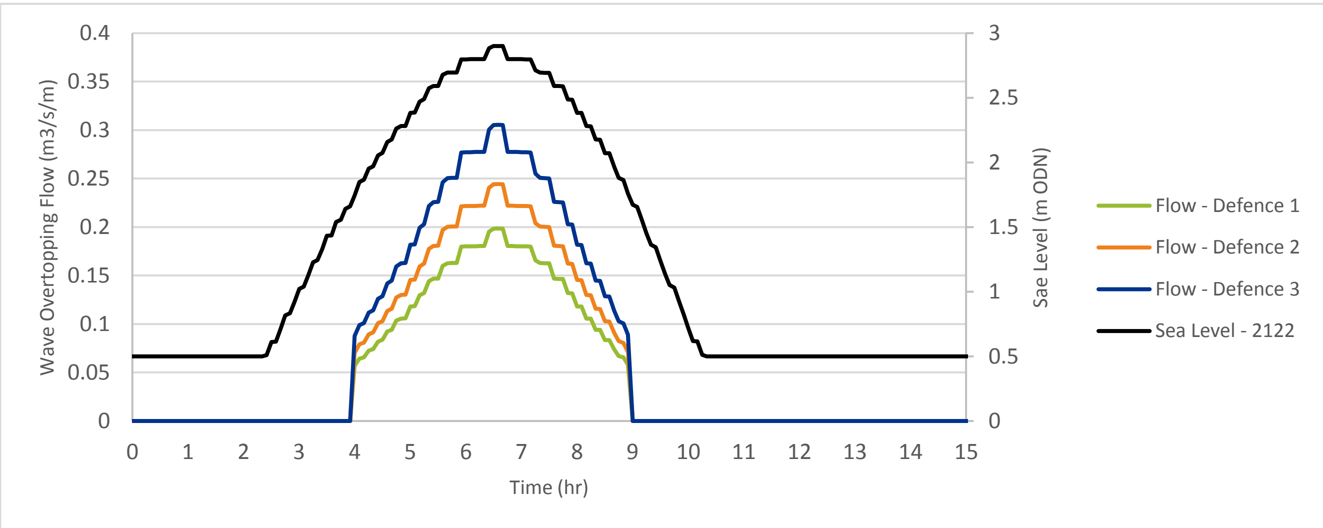

Figure 5-3: Wave overtopping hydrographs and tidal hydrograph – Do Minimum, 2122 epoch.

- Floodplain

A free flow condition has been applied at the east and west boundaries of the model domain located in the floodplain, which means that there is no downstream control and water is assumed to leave freely the model domain.

5.3.2 Do Nothing

The Do Nothing scenario assumes a deterioration of the sea defences allowing larger volumes of water due to wave overtopping of the defences. For both epochs, ABPmer provided the wave overtopping hydrographs for the Do Nothing scenario which were used as inflows in the model (refer to Figure 5-4 and Figure 5-5).

The sea defences were modelled as in the Do Minimum scenario i.e. no deterioration of the defences has been simulated, which is a conservative assumption i.e. setback wall will still slow down the return of water from road to revetment.

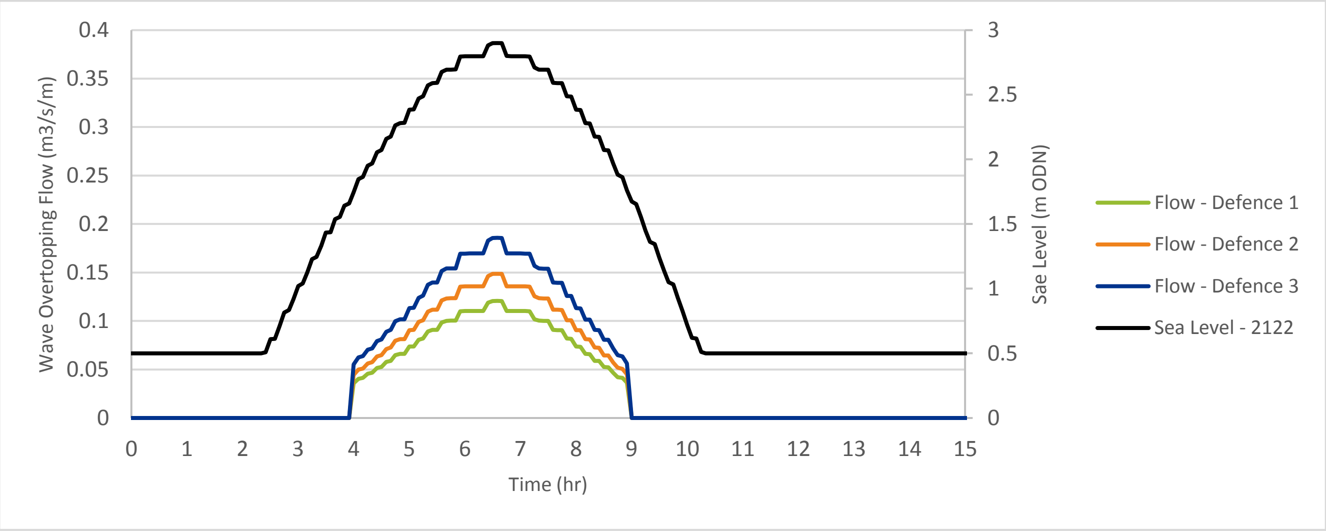

Figure 5-4: Wave overtopping hydrographs and tidal hydrograph – Do Nothing, 2022 epoch.

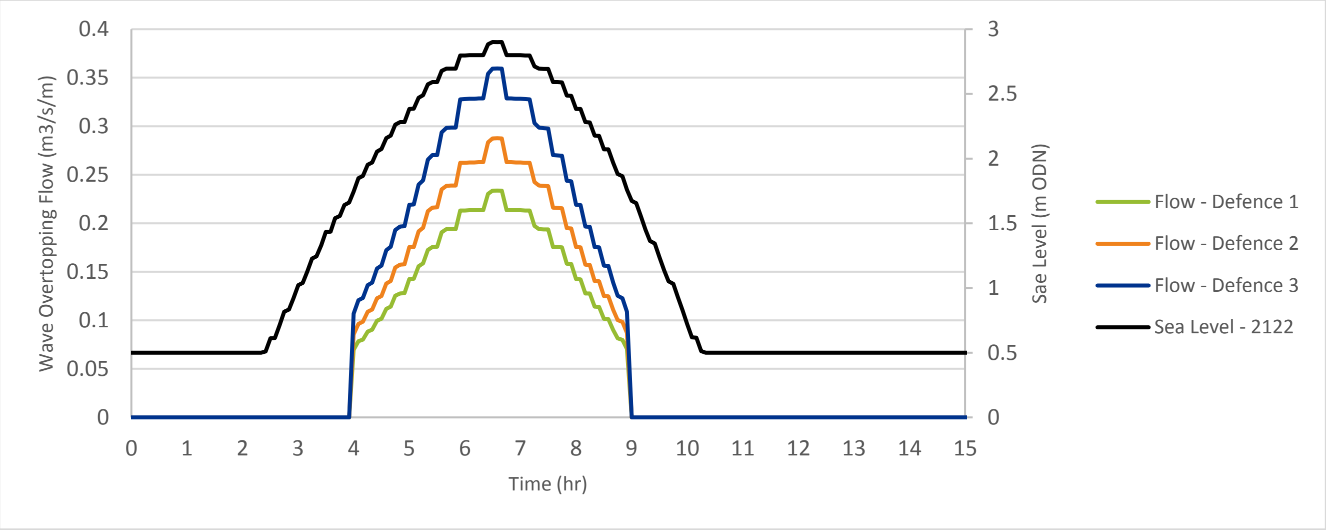

Figure 5-5: Wave overtopping hydrographs and tidal hydrograph – Do Nothing, 2122 epoch.

5.4 Defence Options Model Schematisation

Each of the five defence options, representing an improvement of the existing sea defences, were investigated for the 1 in 200-year event. The impact of the proposed defence options on flood depth was estimated at the end of their design life i.e. for the year 2122.

Each of the five options are modelled according to the design described in Section 3 and the drawings contained in Appendix C.

5.4.1 Option 1

Figure 5-6 shows the model schematisation for Option 1. Updates to the baseline model are as follows:

- Removal of the existing setback wall and replacement with a new setback wall at the same location with a crest level of +5.84m ODN;
- Proposed baffle structures have been modelled with an average 50m spacing along the length of the new setback wall. To optimise their drainage capacity, they have been placed at the low points along the road and aligned with the main overland flow paths;
- Ground levels below and behind the new rock armour is assumed to be the same as the existing revetment slope and the rock armour is assumed to be permeable;
- At the location of the rock armour i.e. the area between wave return wall and setback wall, the roughness has been increased (Manning's n of 0.105[11]) to represent the impact of the rocks and the flow obstruction that they induce;
- It is assumed that the rock armour around the baffles can be arranged such that it does not significantly affect the return flow. To capture any restricting effect, a total flow width reduction of 10% has been implemented at the seaside outlet of the baffles; and
- The existing drainage pipes through the wave return wall have been removed from the model as maintenance of these pipes will not be possible due to the rock armour.

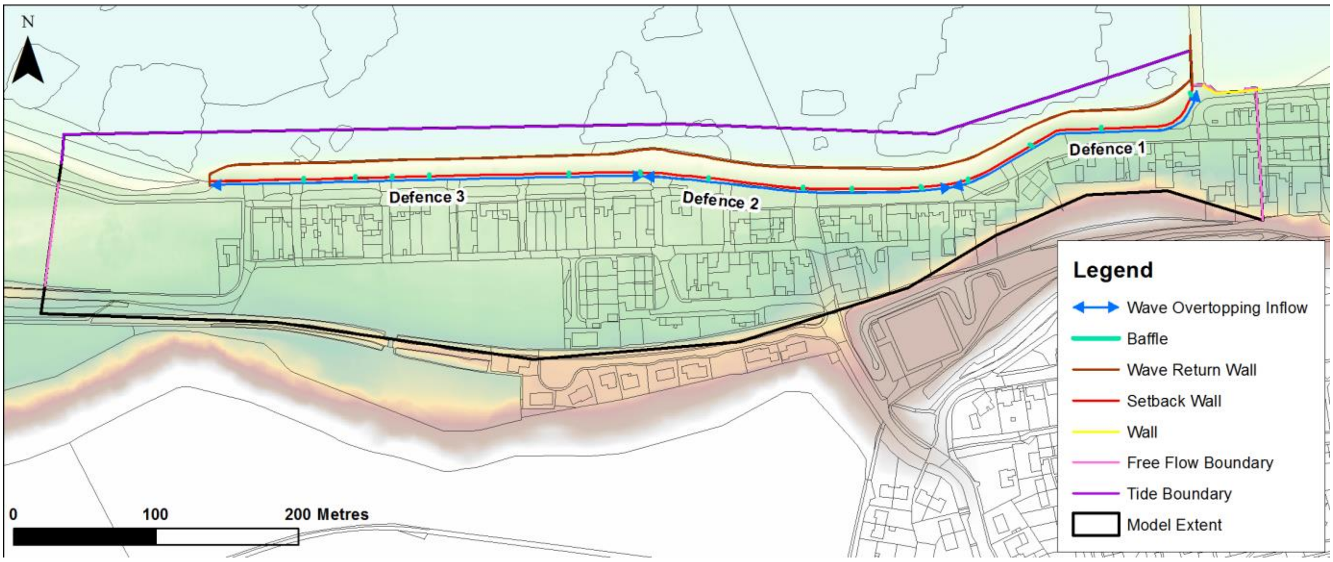

Figure 5-6: Options 1 and 3 – model schematisation.

For Option 1, ABPmer provided the wave overtopping flow hydrographs at the setback wall. They were used as inflows in the model (Figure 5-7) and placed at the landward side of the setback wall.

[11] Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains, U.S. Geological Survey, Water Supply Paper 2339, 1989

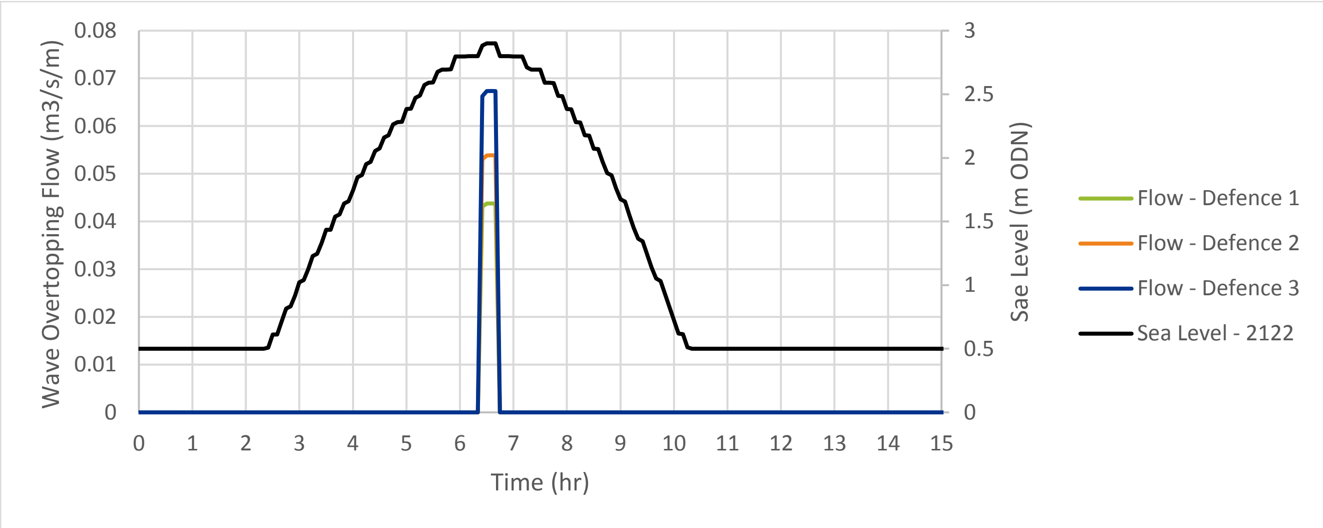

Figure 5-7: Wave overtopping hydrographs and tidal hydrograph for 2122 – Option 1.

5.4.2 Option 2

Figure 5-8 shows the model schematisation for Option 2. Updates to the baseline model are as follows:

- In contrast to Option 1, the existing setback wall remains in place;
- Baffle structures were modelled with an average 50m spacing along the length of the setback wall in addition to the existing ones. To optimise their drainage capacity, they have been placed at the low points along the road and aligned with the main overland flow paths;
- The rock armour is assumed to have no impact on the flow coming out of the baffles;
- Ground levels below/behind the new rock armour is assumed to be the same as the existing revetment slope and the rock armour is assumed to be permeable; and
- The drainage pipes through the wave return wall have been removed from the model as maintenance of these pipes will not be possible due to the rock armour.

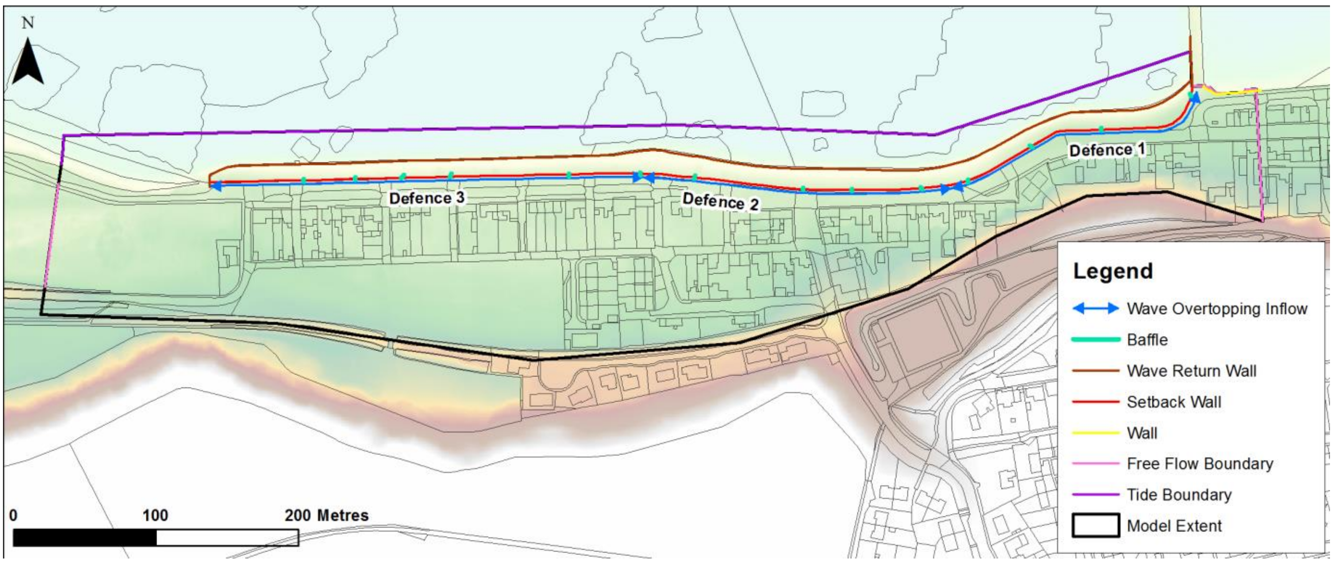

Figure 5-8: Option 2 – model schematisation.

For Option 2, ABPmer provided the wave overtopping hydrographs at the setback wall. They were used as inflows in the model (Figure 5-9) and placed at the landward side of the setback wall.

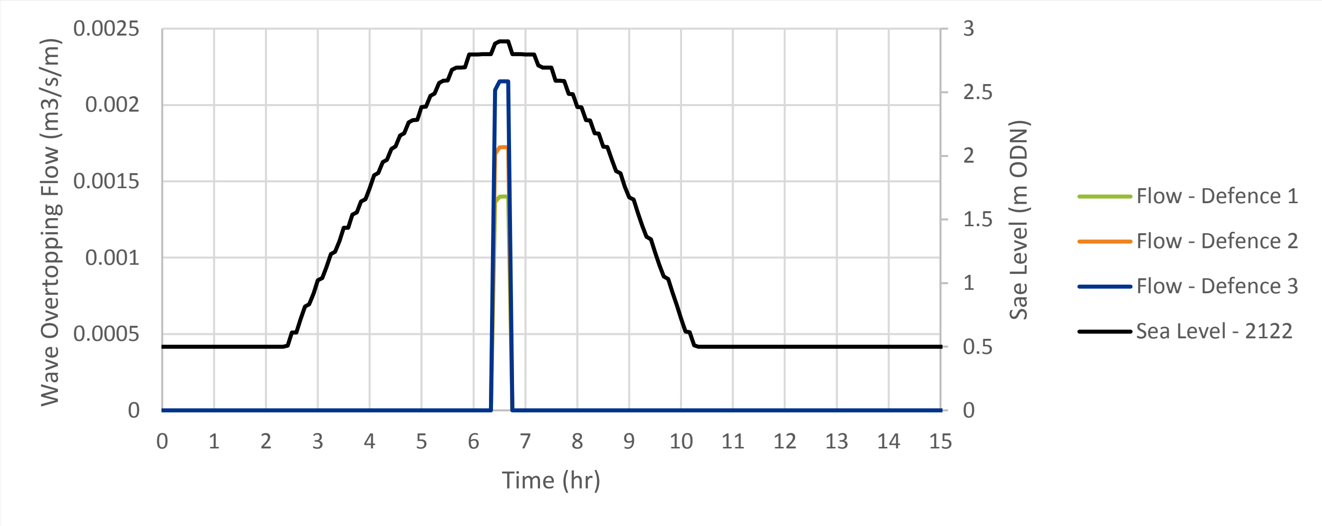

Figure 5-9: Wave overtopping hydrographs and tidal hydrograph for 2122 – Option 2.

5.4.3 Option 3

Figure 5-6 shows the model schematisation for Option 3 (in addition to Option 1). Updates to the baseline model are the same as in Option 1 with the difference that the crest level of the new setback wall is +5.34m ODN.

For Option 3, ABPmer provided the wave overtopping hydrographs at the setback wall. They were used as inflows in the model (Figure 5-10) and placed at the landward side of the setback wall.

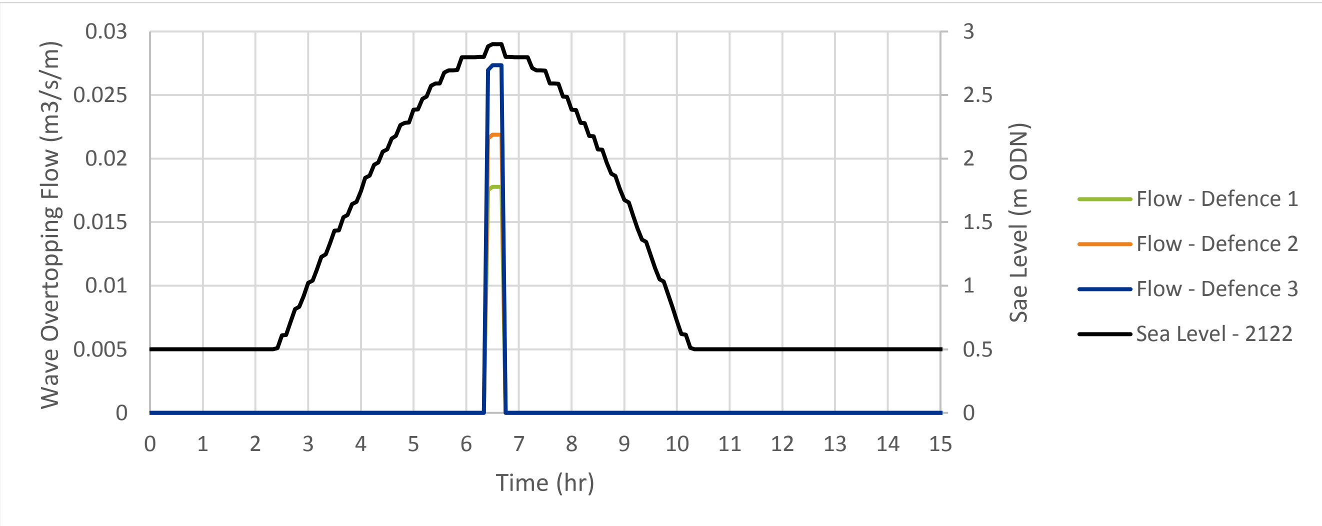

Figure 5-10: Wave overtopping hydrographs and tidal hydrograph for 2122 – Option 3.

5.4.4 Option 4

Figure 5-12 shows the model schematisation for Option 4. Updates to the baseline model are as follows:

- A new wave return wall is installed as the top step of the stepped revetment, with a crest level of +6.84m ODN, 5.5m seaward from the existing setback wall to the landward face of the stepped revetment;
- 300mm pipes with a 5m spacing are modelled through this new wave return wall. The pipes are 6.5m long based on the wall width at the appropriate level, and they have a slope of 3.1% in general. The

downstream invert levels for all the pipes are above +3.57m ODN which is the maximum tide level for this option provided by ABPmer;

- The existing setback wall is not modified; and
- Ground levels between the new wave return wall and the setback wall are assumed to be the same as the Do Minimum scenario.

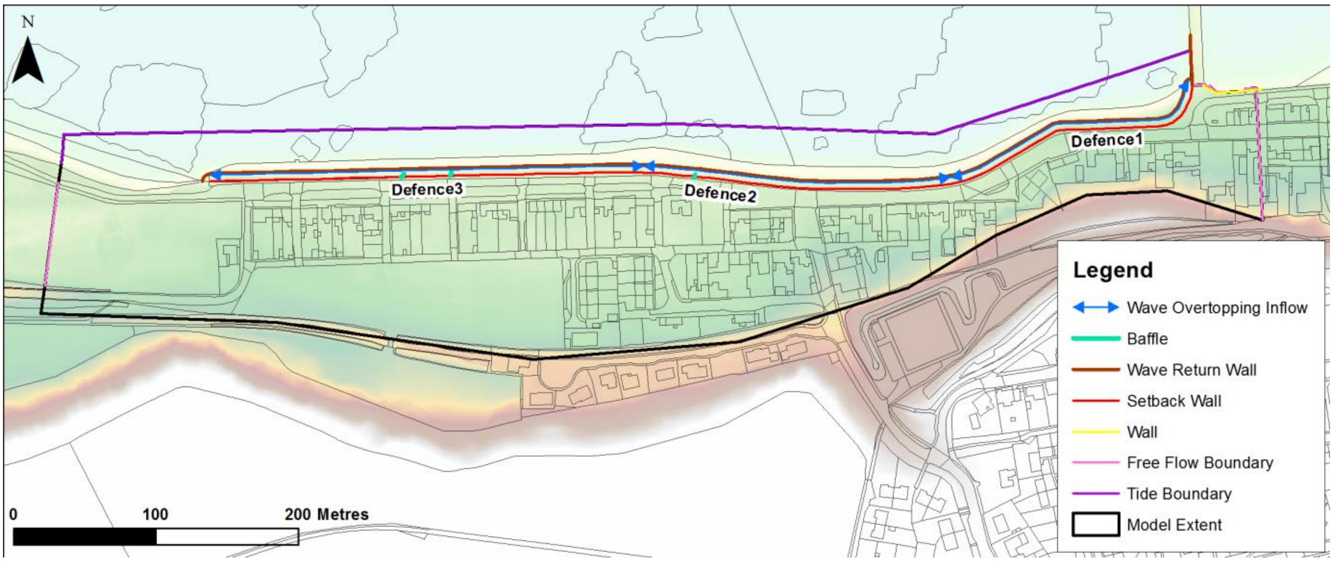

Figure 5-11: Option 4 – model schematisation.

For Option 4, ABPmer provided the wave overtopping hydrographs at the new wave return wall. They were uses as inflows in the model (Figure 5-12). The wave overtopping flows in this option were applied on the revetment next to the new wave return wall, which is different from the baseline and defence Options 1-3 in which the wave overtopping flows were applied next to the setback wall.

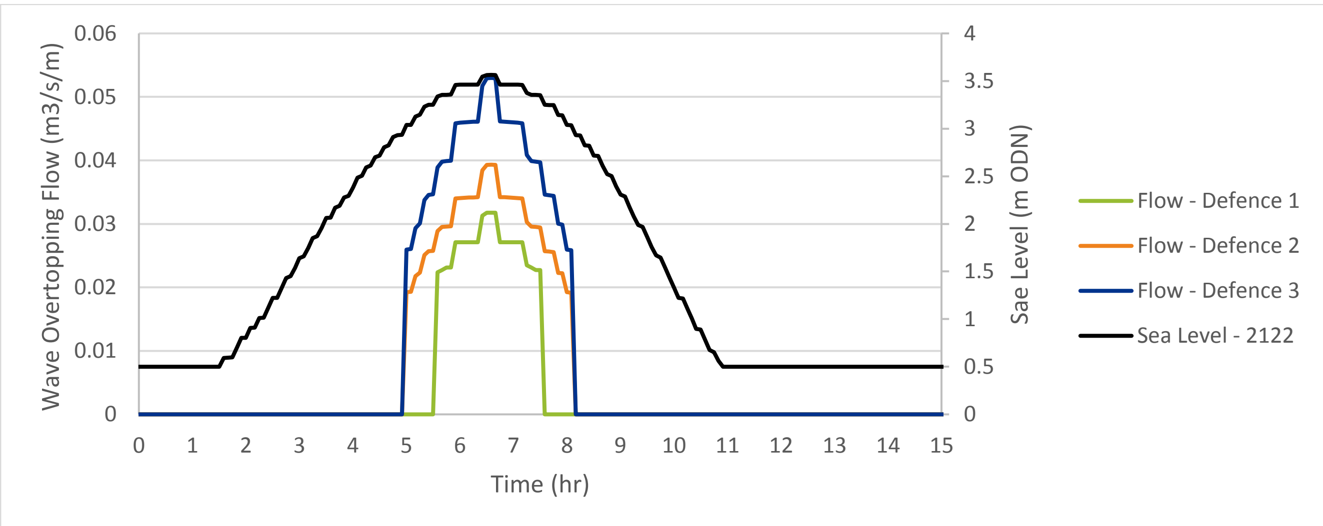

Figure 5-12: Wave overtopping hydrographs and tidal hydrograph for 2122 – Option 4.

5.4.5 Option 5

Figure 5-13 shows the model schematisation for Option 5. Updates to the baseline model are as follows:

- A new wave return wall with a crest of +6.34m ODN is installed approximately 10m seaward from the setback wall to the landward face of the wave return wall;

- 300mm drainage pipes with a 5m spacing are modelled through the new wave return wall. The pipes are 3.6m long based on the wall width at the appropriate level, and have a slope of 2.8% in general. The downstream invert levels for all the pipes are above +3.29m ODN which is the maximum tide level for this option provided by ABPmer;
- The existing setback wall and the existing drainage system beneath and through the setback wall are not modified; and
- The ground level between the setback wall and the new wave return wall falls through a drop 0.5m;
- In contrast to the baseline scenarios, different tide levels time series provided by ABPmer have been applied for this option. The same tide levels were applied for Defence 2 and 3 and a different tide was applied for Defence 1 (refer to Section 4).

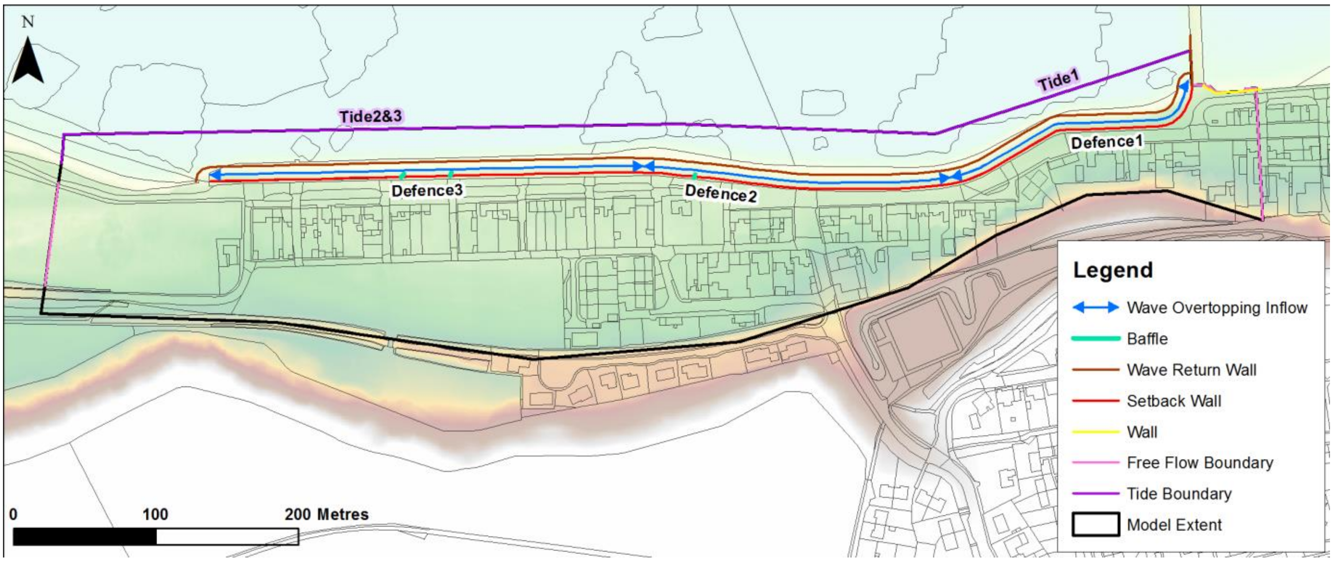

Figure 5-13: Option 5 – model schematisation.

For Option 5, ABPmer provided wave overtopping hydrographs at the wave return wall. They were used as inflows in the model. Similar to Option 4, the wave overtopping flows in this Option 5 were applied on the revetment next to the new wave return wall.

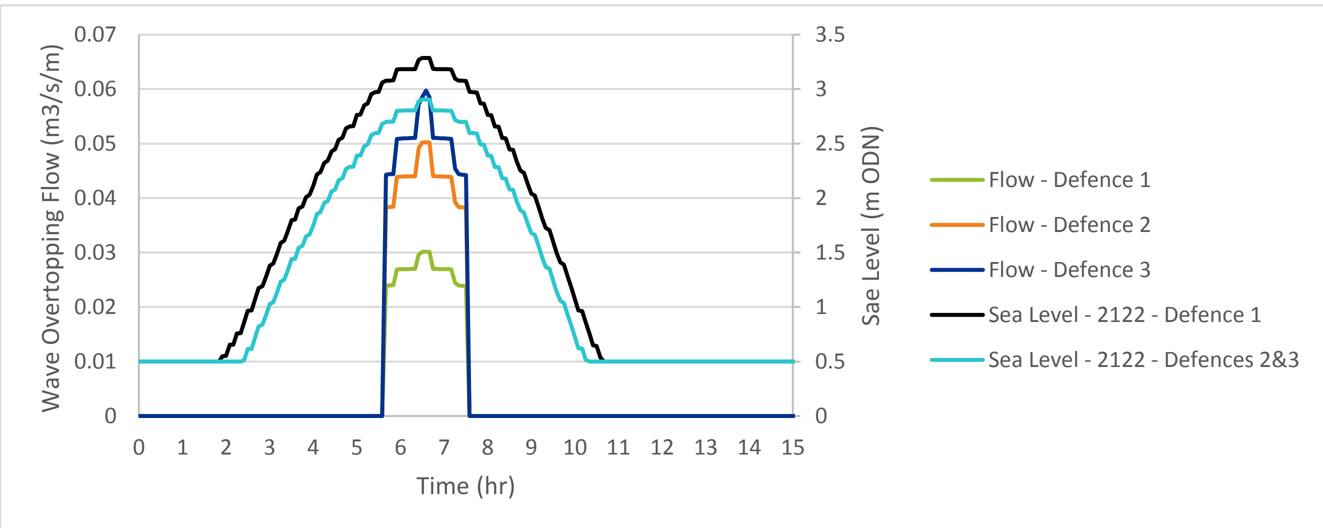

Figure 5-14: Wave overtopping hydrographs and tidal hydrographs for 2122 – Option 5.

5.5 Model Results

5.5.1 Do Minimum

The volume of water which overtops the Portgordon sea defences has the potential to flood the seafront between the setback wall and the high ground further south in the residential area. The gradient of the topography also encourages flood water to flow towards the west end of the study area. Figure 5-15 and Figure 5-16 show the maximum flood extent and maximum flood depth for the years 2022 and 2122 respectively for Do Minimum scenario.

In the year 2022, water depths at the properties are generally between 0.25m and 0.75m. On the road they are between 0.75m and 1.00m in the vicinity of the Defence 3 baffles.

In the year 2122, water depths are generally between 0.50m and 0.75m at the location of the properties and locally between 0.75m and 1.00m at a few locations. On the road, water depths are generally above 0.75m, and between 1.00m and 1.50m in the vicinity of the Defence 3 baffles.

Table 5-4 provides the peak flow values through the baffles across the setback wall as flood flows drain back to the sea. Model results also indicate that Defences 1, 2 and 3 are overtopped by the returning flow. Associated peak flow values are also reported in Table 5-4.

Table 5-4: Flows returning to the sea through the baffles and on top of the defences – Do Minimum scenario.

Epoch	Peak flows returning to the sea (m^3/s)					
	Baffle 1	Baffle 2	Baffle 3	Defence 1	Defence 2	Defence 3
2022	1.3	1.2	1.0	9.2	19.9	39.2
2122	1.6	1.5	1.3	17.5	45.2	77.0

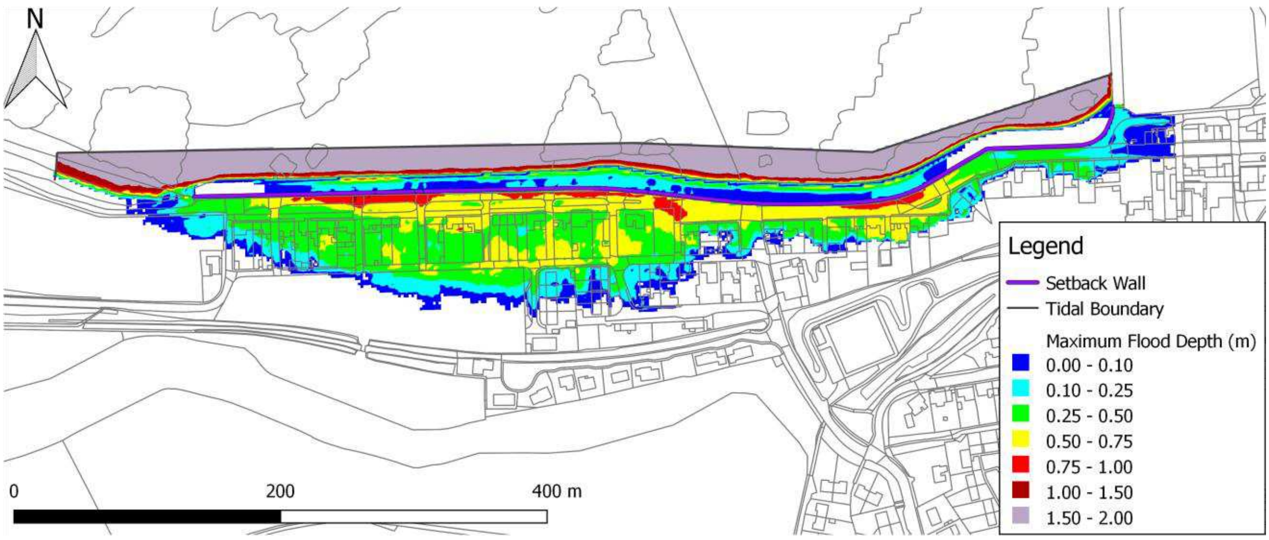

Figure 5-15: Maximum flood depths – Do Minimum scenario, 2022 epoch 1 in 200-year event.

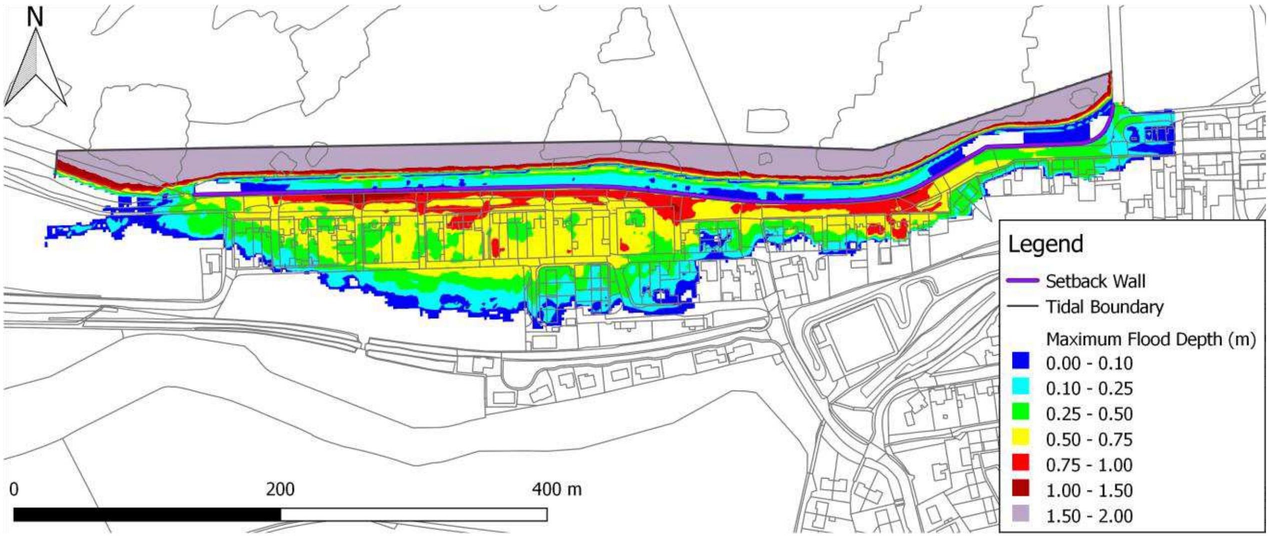

Figure 5-16: Maximum flood depths – Do Minimum scenario, 2122 epoch 1 in 200-year event.

5.5.2 Do Nothing

Failure of the sea defence causes a greater flood volume as a result of wave overtopping. Figure 5-17 and Figure 5-18 indicate the maximum flood extent and maximum flood depths for the 2022 and 2122 epoch respectively for the Do Nothing scenario.

For both epochs 2022 and 2122, in the Do Nothing scenario, the maximum flood extent and the maximum flood depths are slightly greater than in the Do Minimum scenario. However, the maximum water depths and number of properties affected are still in the same order as in the Do Minimum scenario.

Table 5-5 gives the flows which return to the sea through the baffles and over the top of the sea defence. Flows through the baffles are in the same order as in the Do Minimum scenario while Defences 1, 2 and 3 are overtopped by more significant return flows than in the Do Minimum scenario.

Table 5-5: Flows returning to the sea through the baffles and on top of the defences – Do Nothing scenario.

Epoch	Flows returning to the sea (m^3/s)					
	Baffle 1	Baffle 2	Baffle 3	Defence 1	Defence 2	Defence 3
2022	1.4	1.3	1.1	10.8	25.3	47.2
2122	1.6	1.6	1.3	20.9	54.2	90.6

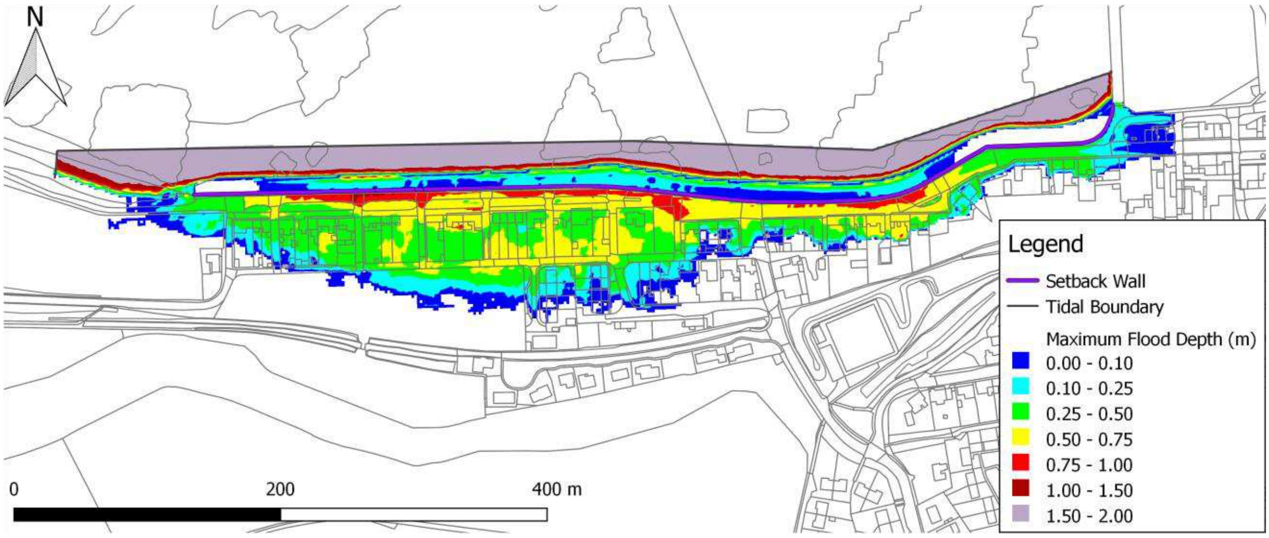

Figure 5-17: Maximum flood depths – Do Nothing scenario, 2022 epoch 1 in 200-year event.

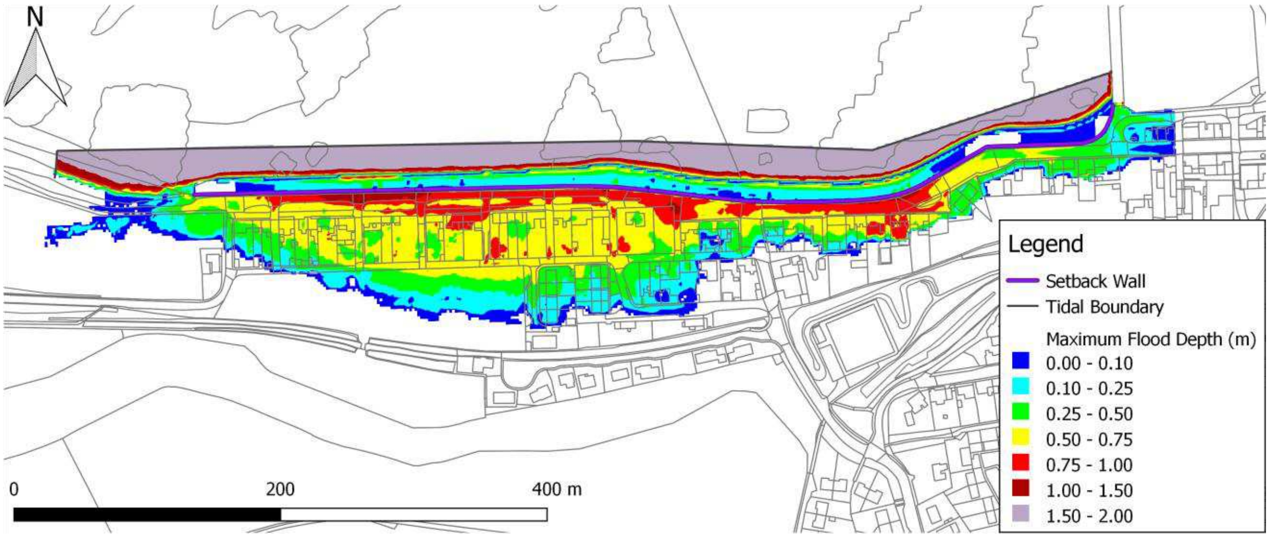

Figure 5-18: Maximum flood depths – Do Nothing scenario, 2122 epoch 1 in 200-year event.

5.5.3 Option 1

In Option 1, wave overtopping occurs for a period of around 30min at high tide while it occurs for 5hrs in the Do Minimum scenario. This reduces considerably the duration of submersion, from 6hrs in the Baseline scenario to approximately 1hr in the Option 1 scenario. However, since the replacement setback wall is higher than the existing setback wall, it is not overtopped by the returning flows. Therefore, the new setback wall holds back flood water on the landside, causing prolonged flooding. Drainage flows through the baffles vary from 0.2 m^3/s to 2.4 m^3/s.

Figure 5-19 presents the resulting maximum flood extent and maximum flood depths for the 2122 epoch. Figure 5-20 compares the maximum flood extent and indicates the properties affected between Option 1 and the Do Minimum scenario for the 2122 epoch.

The maximum flood extent, maximum flood depths and number of properties affected are similar to the Do Minimum even though there is significantly larger number of baffles modelled. In comparison with the Do Minimum scenario, flood depths increase by 0.00m to 0.10m at the properties between No. 9 and No. 23

Stewart Street and decrease by 0.00m to 0.10m at the other properties of this street. Flood depths decrease by 0.30m to 0.20m along West High Street.

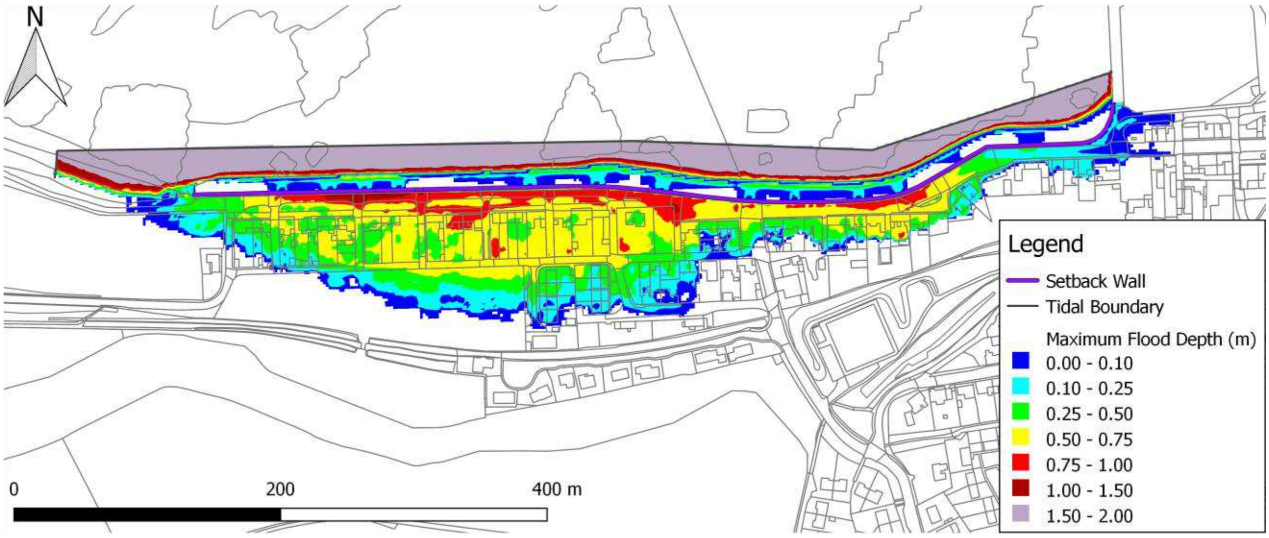

Figure 5-19: Maximum flood depths – Option 1, 2122 epoch 1 in 200-year event.

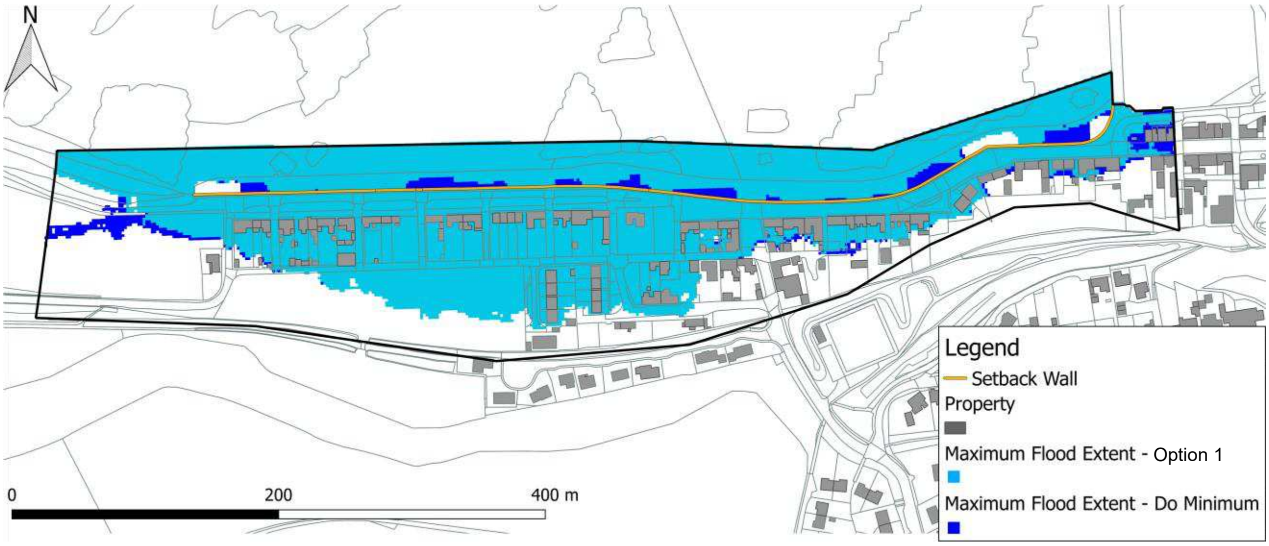

Figure 5-20: Comparison between Option 1 and Do Minimum maximum flood extents, 2122 epoch 1 in 200-year event (where dark blue is over and above light blue extent).

5.5.4 Option 2

In Option 2, wave overtopping occurs for approximately 30min. Wave overtopping volumes are smaller than in the Do Minimum scenario. The maximum flood extent is restricted to the road with no properties predicted to experience flooding. The setback wall is not overtopped by return flows and the baffles are able to drain the overtopping volumes back to the sea. Flows going through the baffles vary from 0.0 m^3/s to 0.2 m^3/s.

Figure 5-21 presents the resulting maximum flood extent and maximum flood depths for the 2122 epoch. Figure 5-22 compares the maximum flood extent and properties affected between Option 2 and the Do Minimum scenario for the 2122 epoch.

It can be seen that even though the number of baffles in Option 2 is much smaller than Option 1, yet the flooding is significantly less in Option 2, which is as a result of the reduced wave overtopping flows applied.

This means that the impact of the mitigation measures that reduce the wave overtopping is much more significant than the measures to improve the drainage of the overtopping volumes.

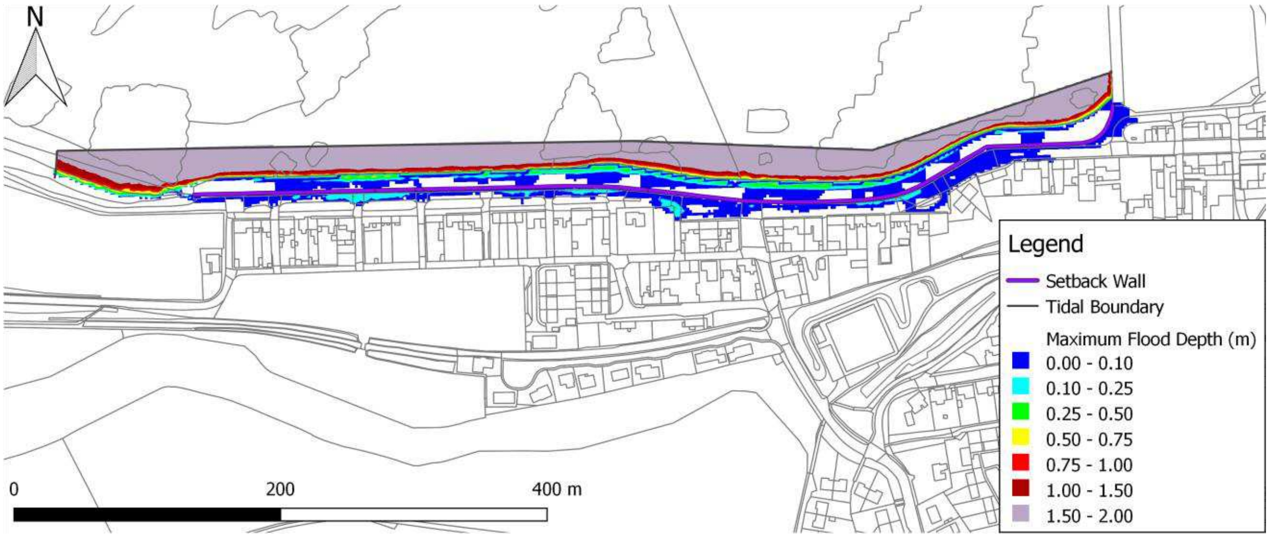

Figure 5-21: Maximum flood depths – Option 2, 2122 epoch 1 in 200-year event.

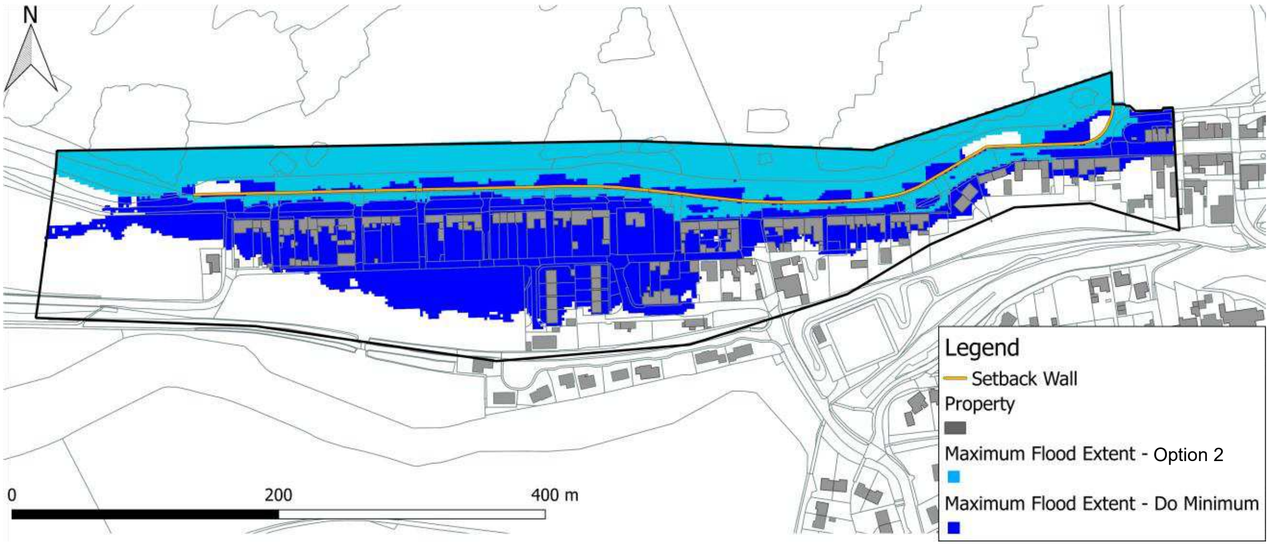

Figure 5-22: Comparison between Option 2 and Do Minimum maximum flood extents, 2122 epoch 1 in 200-year event (where dark blue is over and above light blue extent).

5.5.5 Option 3

In Option 3, wave overtopping occurs for approximately 30min, which is similar to Option 1 and 2. It reduces considerably the duration of submersion, from 6hrs in the Baseline scenario to 1hr approximately in the Option 3 scenario. At some locations along Defence 1, the new setback wall is lower than the existing setback wall and it is overtopped by returning flows with a peak value of 0.2 m^3/s. Defences 2 and 3 are not overtopped. Flows going through the baffles vary from 0.0 m^3/s to 1.5 m^3/s.

Figure 5-23 presents the resulting maximum flood extent and maximum flood depths for the 2122 epoch. Figure 5-24 compares the maximum flood extent and properties affected between Option 3 and the Do Minimum scenario for the 2122 epoch.

The maximum flood extent and maximum flood depths are reduced in comparison with the Do Minimum scenario. Flood depths at the properties are generally between 0.10m and 0.50m. They have been reduced by

0.30m to 0.40m in comparison with the Do Minimum scenario at the properties in Stewart Street and West High Street and by 0.40m to 0.50m at the properties in Lennox Place.

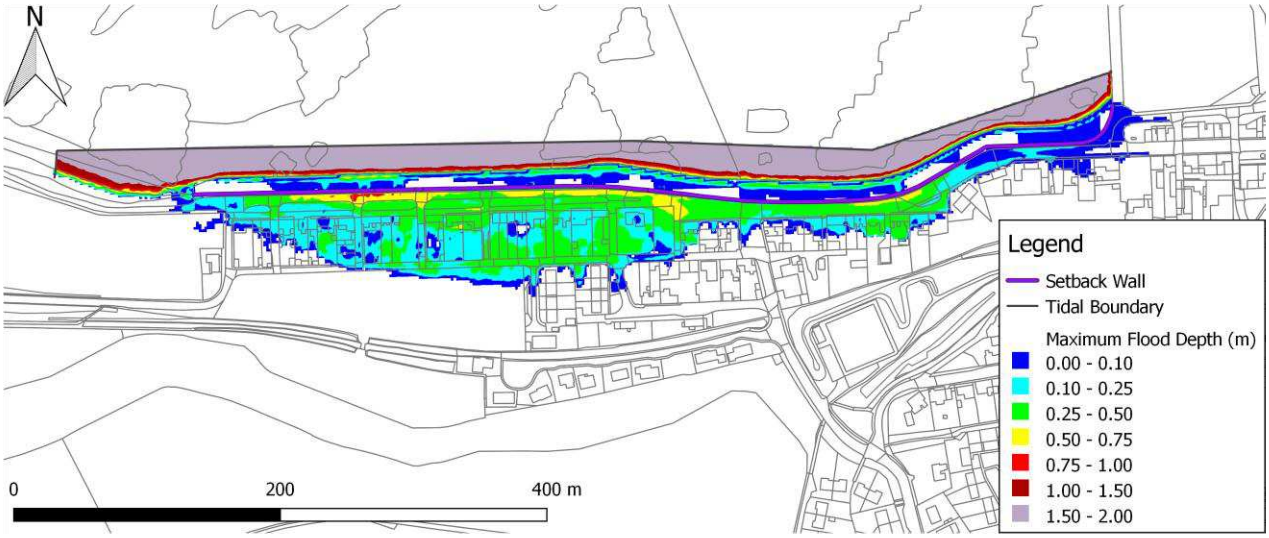

Figure 5-23: Maximum flood depths – Option 3, 2122 epoch 1 in 200-year event.

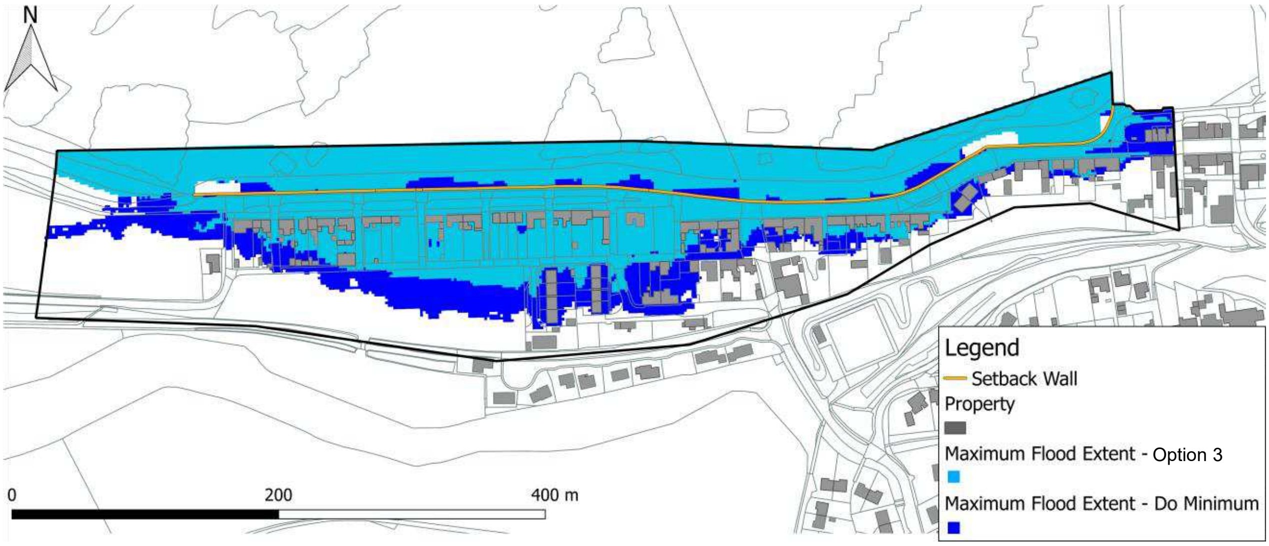

Figure 5-24: Comparison between Option 3 and Do Minimum maximum flood extents, 2122 epoch 1 in 200-year event (where dark blue is over and above light blue extent).

5.5.6 Option 4

Different arrangements of the drainage through the new wave return wall were tested for Option 4:

- 300mm diameter pipes with a 50m spacing;
- 300mm diameter pipes with a 5m spacing; and
- 450mm diameter pipes with a 5m spacing.

The last arrangement (450mm pipes with a 5m spacing) appears to provide sufficient capacity to drain overtopping volumes back to the sea whilst preventing build up of water between the new wave return wall and the setback wall. In the arrangement with 300mm diameter pipes mentioned above, flooding to properties occurs due to water flowing landwards through the baffles in Defence 3. In the 450mm diameter pipes arrangement, the flood extent is restricted to the vicinity of the baffles with no properties experiencing flooding.

Figure 5-25 provides the resulting maximum flood extent and maximum flood depths for the 2122 epoch. Figure 5-26 compares the maximum flood extent and properties affected between Option 4 and the Do Minimum scenario for the 2122 epoch.

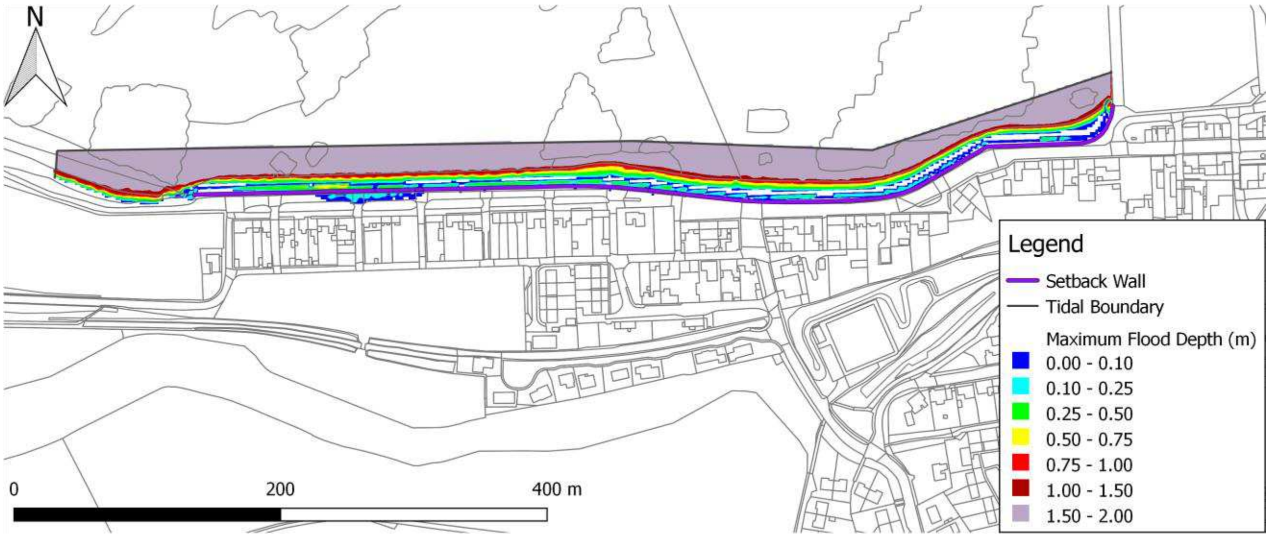

Figure 5-25: Maximum flood depths – Option 4, 2122 epoch 1 in 200-year event.

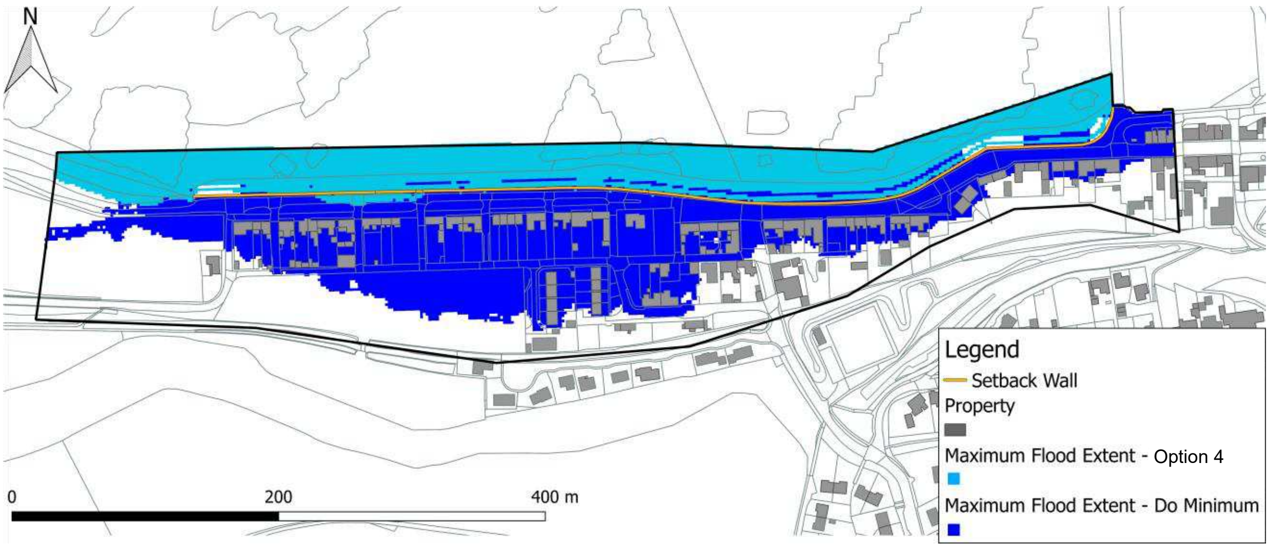

Figure 5-26: Comparison between Option 4 and Do Minimum maximum flood extents, 2122 epoch 1 in 200-year event (where dark blue is over and above light blue extent).

5.5.7 Option 5

As for Option 4, different through wall drainage options were modelled for Option 5, as follows:

- 300mm diameter pipes with a 50m spacing;
- 300mm diameter pipes with a 5m spacing; and
- 450mm diameter pipes with a 5m spacing.

The results were found to be similar to Option 4 with the last arrangement (450mm pipes with a 5m spacing) appearing to provide sufficient capacity to drain overtopping volumes back to the sea preventing a build-up of water between the wave return wall and the setback wall and consequently avoid flooding of properties.

Figure 5-27 presents the resulting maximum flood extent and maximum flood depths for the 2122 epoch. Figure 5-28 compares the maximum flood extent and properties affected between Option 5 and the Do Minimum scenario for the 2122 epoch.

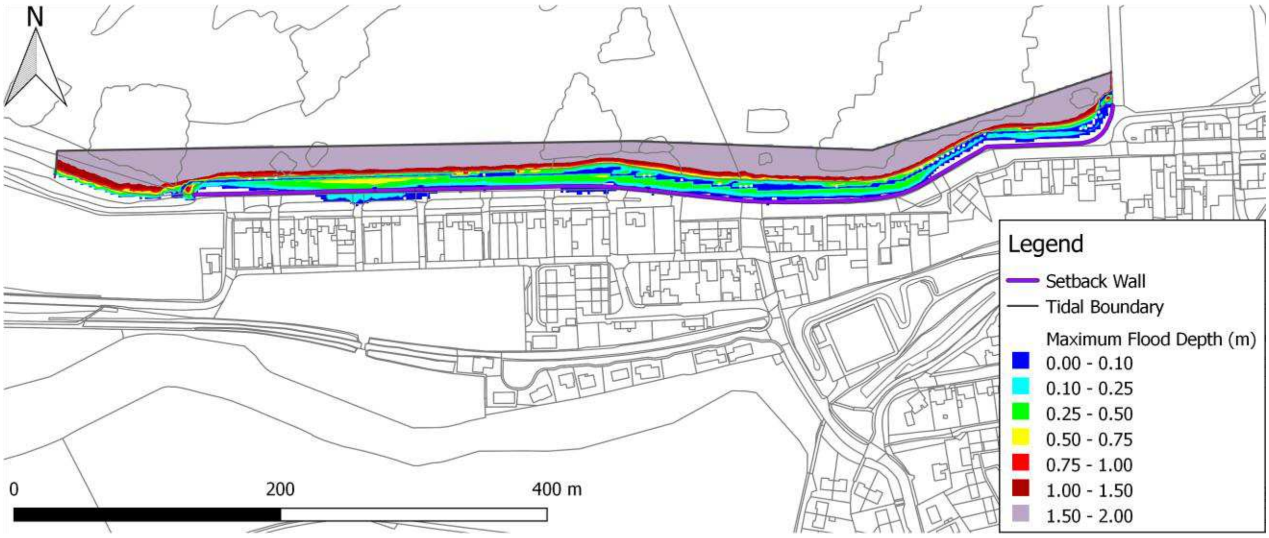

Figure 5-27: Maximum flood depths – Option 5, 2122 epoch 1 in 200-year event.

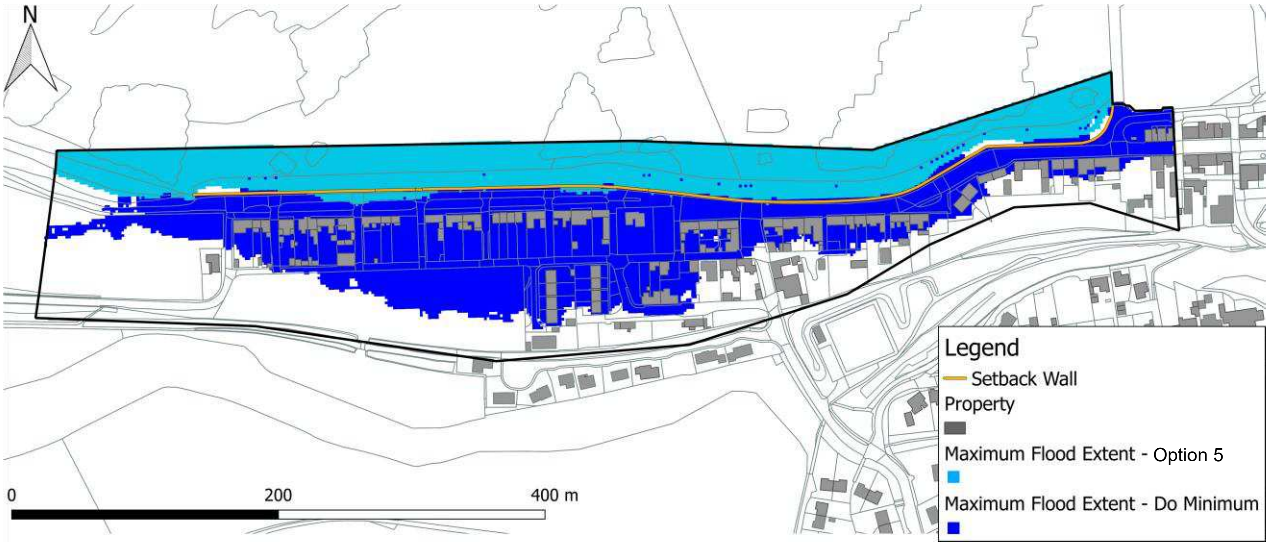

Figure 5-28: Comparison between Option 5 and Do Minimum maximum flood extents, 2122 epoch 1 in 200-year event (where dark blue is over and above light blue extent).

5.6 Discussion

Flooding due to wave overtopping the existing and proposed defences along Portgordon seafront has been assessed using hydraulic modelling. The model takes into account performance of the sea defences in combination with the performance of the drainage system.

In the baseline scenarios, due to the configuration of the topography, the area between the setback wall and the high ground in the south is found to flood under a 1 in 200-year event. The maximum water depth landward of the setback wall is estimated to be between 0.25m and 1.00m at the properties for the Do Minimum scenario for the 2122 epoch. Mechanisms for the return of overtopped water back towards the sea are limited to the existing baffles and by seaward overtopping of the setback wall.

Options 2, 4 and 5 appear to be effective in preventing flooding of the properties along Portgordon seafront. Options 1 and 3 considerably decrease the duration of submersion, albeit properties along the seafront would still be predicted to experience flooding. Flood depths are in the same order in Option 1 as in the Do Minimum scenario. In Option 3, flood extent is predicted to be reduced with flood water depths also dropping from around 0.30m to 0.50m.

A degree of wave overtopping is predicted to occur in Option 2 for a period of around 30 minutes coinciding with high tide. However, the additional baffles in the setback wall are able to drain the overtopped water sufficiently such that the maximum flood extent is restricted to the road with no properties experiencing flooding.

If the project progresses to the detailed design stage, it is recommended that sensitivity testing is conducted in the drainage arrangement. The configuration of the existing baffles is currently proposed to be repeated at 50m centres along the setback wall. However, an improved configuration may result in better drainage performance. It is further recommended that the detailed design takes into account the limitations of the modelling as described in Section 5.1.1.

6. Cost Estimates

Preliminary cost estimates have been developed for the five defence options identified in Section 3. These are based on approximate quantities informed by topographical survey information and engineering judgement. The rates used within the cost estimates have been derived from published cost data and supplier rates.

At this stage in the project, the cost estimates are based on concept designs, which will require review and refinement if taken forward. It should be noted that no preliminary design work has been undertaken at this stage. It is expected that the estimates within this report have an accuracy range of -30% to +60%. Costs include an inflationary rise between 2018 and the projected construction date of 2022 and are based on the following assumptions:

- All works are undertaken during normal working hours.
- No contaminated or hazardous materials are present on site.
- The contract will be competitively tendered.
- Suitable access is available for all plant and machinery.
- No allowances have been made for hand excavations.
- No live services are affected by the works.
- For Options 1-4, the cost estimates allow for rock armour, but concrete armour block units would be an alternative.
- The rock armour costs could vary significantly and are heavily dependent on from where the rock is sourced, and the size of rock available.
- A proportion of the existing rock armour can be reused for the proposed options.

When preparing the cost estimates, the following items have been specifically excluded from the figures:

- Value Added Tax (VAT)
- Design fees
- Ground investigation work
- Planning and environmental approvals
- Consents and licences
- Consultancy fees supporting the Moray Council during the tender and construction phases
- Additional surveys recommended for the next phase of the project
- Local authority fees
- Legal or funding costs

The preliminary cost estimate and accuracy range are presented in the Table 6-1.

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Table 6-1: Cost estimates for each defence option.

Option	Approximate Capital Cost		
	Cost Estimate -30%	Cost Estimate	Cost Estimate +60%
Option 1	£ 7,794,000	£ 11,133,000	£ 18,927,000
Option 2	£ 7,420,000	£ 10,600,000	£ 18,020,000
Option 3	£ 11,292,000	£ 16,131,000	£ 27,423,000
Option 4	£ 8,323,000	£ 11,889,000	£ 19,023,000
Option 5	£ 11,485,000	£ 16,407,000	£ 26,252,000

7. Planning and the Environment

7.1 Introduction

The purpose of this section is to provide high level commentary on the planning requirements and the key environmental designations of both the site and the immediate surrounding area. This section also identifies the potential scope of further studies and environmental consents that may be required at subsequent stages of the project development.

Given the high level approach to the planning and environment assessment, which is appropriate for this stage of the project, no specific study area is defined. A broader overview of the Portgordon area is adopted to identify the key planning and environmental designations and constraints.

7.2 Methods and Scope of Review

7.2.1 Methods

The desk-based review utilises publicly accessible information and is supplemented by studies undertaken within other sections of this report.

As noted in Section 7.1, using professional judgement of potential impacts considered at this stage, a broad area is assumed as the “Study Area” although where deemed necessary for some environmental topics (e.g. ecological Habitat Regulation Assessment) a specific area is reviewed (and defined) accordingly to identify designations.

7.2.2 Assumptions and Limitations

The section provides general advice in relation to the flood defence options set out in Section 3. Where differentiators in relation to potential environmental impacts or consents are identified between options, these are noted below in Section 7.5.

No engagement with any statutory or determining authorities has been undertaken to inform this study. This will be required at a later stage to seek confirmation of any assumptions.

A number of assumptions have been made that will require confirmation upon review of the chosen detailed design. Where applicable, these assumptions are identified throughout this section.

7.3 Planning Context

7.3.1 Policy Context

The Moray Council formally adopted The Moray Local Development Plan (MLDP) in July 2015[12], which details the planning policy requirements of the site, and replaces the previous Moray Structure Plan (2007)[13] and Moray Local Plan (2008)[14].

The strategy for the distribution of development across Moray is a continuation of that taken by the 2008 Local Plan[14] which identified Portgordon as a “third tier settlement”.

The objectives for Portgordon of relevance to the study area, outlined in the MLDP[12], include:

- To encourage new house building and take long term view.
- Control the direction of growth; avoid spread along the coast.

[12] The Moray Local Development Plan, The Moray Council July 2015
[13] Moray Development Plan, Moray Structure Plan, The Moray Council April 2007
[14] Moray Local Plan, 2008

- Provide support for proposals to re-use the harbour.

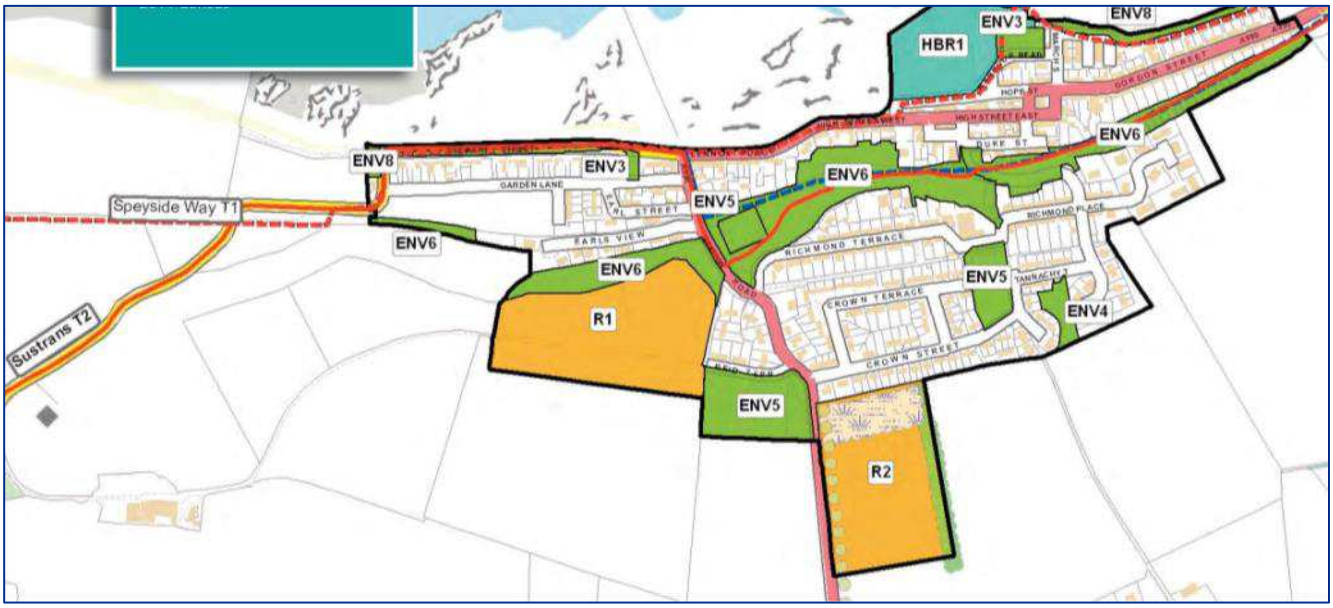

Figure 7-1: Portgordon settlement map[12].

As noted in Figure 7-1 from the MLDP[12], a number of key development opportunity allocations are identified in the area, primarily for Environment and Tourism. These identify sites of potential planned growth, per use time, and are noted as:

- **ENV3 – Amenity Greenspace**
  - Grassed areas at Stewart Street; east of harbour.
- **ENV4 – Playspace for Children and Teenagers**
  - Tannachy Terrace.
- **ENV5 – Sports Areas**
  - Bowling green; football pitch, school playing field.
- **ENV6 – Green Corridors / Natural / Semi Natural Greenspaces**
  - Old railway line; North of Reid Terrace.
- **ENV8 – Foreshore Areas**
  - Area at east and west end of village.
- **HBR1 – Foreshore Areas**
  - The harbour and its immediate hinterland will be retained for potential tourist use involving recreational sailing; pontoons; increased berthing and ancillary facilities. However, the prospects for future use are felt to be more related to sailing/tourist activities.
- **T1 – Speyside Way**
  - The route of the Speyside Way Long Distance Footpath and the Moray Coast Trail through Portgordon will be safeguarded and protected from development. While this route does pass along Stewart Street and Lennox Place, it does not utilise the sea wall area and should not be affected by the proposed works.
- **T2 – Sustrans**

- The route of the SUSTRANS long distance cycle route through Portgordon will be safeguarded and protected from development. As with T1 above, it is not anticipated that this will be affected by the proposed works.

Whilst it is considered that the majority of the proposed works will occur within the ENV3, ENV8, T1 and T2 allocations, consideration has also been given to any cumulative impacts on other allocations (refer to Section 7.3.4).

7.3.2 Extant Planning Applications and Permissions

A high level review of extant planning applications and permissions determines whether any planned developments may be impacted by the proposed works. Consideration of consented developments, or yet to be determined planning applications, in a three-year period (01 January 2015 and valid up to 31 December 2017) are assessed. Minor or procedural type applications were excluded from the assessment.

Table 7-1 outlines the extant planning applications located within the settlement of Portgordon.

Table 7-1: Extant planning applications in Portgordon.

Moray Council Reference	Overview	Approximate distance from closest point of proposed site
16/01446/APP	Erection of a 12m high steel lattice mast with three antenna two micro-wave dishes installation of equipment cabinet ancillary equipment and fencing	>150m
15/00840/APP	Erection of dwellinghouse	>140m
15/01626/PPP	Renew planning consent 12/01264/APP (original consent re 06/01874/OUT) to erect dwellinghouse	>55m
15/01144/APP	Erection of dwellinghouse	>120m
16/00610/APP	Revised house design including detached garage (previous planning consent 15/01144/APP)	>120m

Given the scope and nature of works, it is not anticipated that there will be any direct impacts on any of the extant planning applications or consents. However, further review will be required at the next stage to confirm.

7.3.3 Marine Planning

The Marine (Scotland) Act 2010[15] provisions that the jurisdiction of Marine Planning is defined as the territorial "sea" which includes any area submerged at Mean High Water Spring tide mark (MHWS). All of the proposed options extend below MHWS, with the following summary of the options:

- Option 1 extends approximately 5m seaward of the existing toe of the defence structures;
- Option 2 extends approximately 9m seaward of the existing toe of the defence structures;
- Option 3 extends approximately 13m seaward of the existing toe of the defence structures;
- Option 4 remains within the lateral extent of the existing defence structures; and
- The wave return wall in Option 5 extends by approximately 2m seaward of the existing wave return wall, yet the overall lateral extent of the defence reduces by 4m due to removal of the existing rock armour.

[15] Marine (Scotland) Act 2010

As the boundaries of the existing defences are being extended for Options 1-3, it is considered likely that a Marine Licence will be required. However, there are certain activities exempt from consent for works performed by, or on behalf of, local authorities to maintain coastal protection and flood defence. Further details on consents required are set out below in Section 7.4.3.

In line with Planning Circular[16], marine and terrestrial planning authorities should consult one another formally. Therefore, the proposals and the potential to impact the marine environment should be assessed in consultation with Marine Scotland during subsequent stages of the project.

7.3.4 Further Studies and Recommendations

It is recommended that:

- Early consultation with Marine Scotland takes place to confirm the requirement for a Marine Licence;
- A detailed review of any extant planning applications in the local area is undertaken, particularly around potential allocation sites (refer to Figure 5-1); and
- Upon understanding the full extent of the proposed works and any residual impacts, contact should be made with the Moray Council Planning Department to discuss any potential impacts, namely on the cycle route and footpath, and to seek confirmation that Permitted Development rights would apply to the proposed works (refer to Section 7.4).

7.3.5 Heritage Designations

Whilst the extent of any proposed works will be specified in the next phase of work and the option selected, it is assumed that the construction works will take place within the broader (Easting / Northing Coordinates) area of **NGR 38814 64215** to the West and **NGR 339508 864257** to the East of the seafront.

As can be seen in Figure 7-2, the following designations have been identified in the study area:

- Spey Bay Site of Special Scientific Interest (SSSI) (Mixed);
- Spey Bay Geological Conservation Review (GCR) site;
- One B and one C Listed Building; and
- Other heritage aspects recorded in the National Monument Records of Scotland (not shown on figure).

[16] Planning Circular. The relationship between the statutory land use planning system and marine planning and licensing. http://www.gov.scot/topics/ marine/seamanagement/national/circular

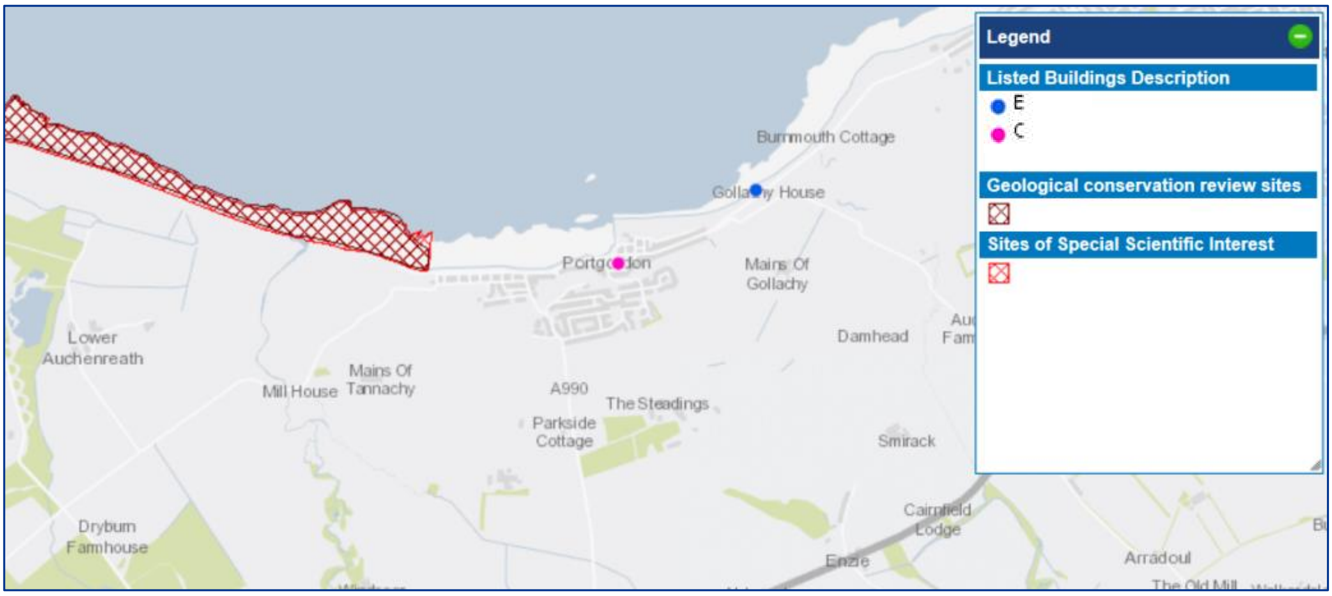

Figure 7-2: Identified fesignations in the local area[17].

Within the study area, the following statutory designated sites have been identified:

- Tynet Burn GCR site;
- Lower River Spey SSSI;
- River Spey SSSI;
- Moray and Nairn Coast Special Protection Area (SPA);
- Moray and Nairn Coast Ramsar;
- Lower River Spey GCR site;
- Lower River Spey – Spey Bay Special Area of Conservation (SAC); and
- River Spey SAC.

As can be seen in Figure 7-2, two heritage assets have been identified; outlined in Table 7-2 below.

Table 7-2: Listed buildings in Portgordon.

Feature	Category (HES Reference)	NGR	Description
Portgordon 2 East high Street	C (LB15522)	39650 64244	Early 19th century. Single storey, 4-bay cottage with single bay return elevation to Gordon Square (W). Rendered rubble with later long and short detailing. Entrance with panelled door flanked by windows and varied glazing; blocked doorway in outer bay at right; single window in W elevation (to Gordon Square).
Portgordon, Gollachy Ice House	B (LB15546)	40260 64565	Earlier 19th century. Rectangular rubble ice house with long elevations E and W, and off-centre entrance in E. Modern pinkish harl.

[17] "Scotland's Environment", accessed 22/05/17 from here http://map.environment.scotland.gov.uk/seweb/map.htm?menutype=1

Due to their location and the scope or works, the proposed works are not expected to directly impact any of the statutory designated sites or heritage assets. However, potential impacts on qualifying features would be assessed at the next stage when the extent and construction methods of the proposed works are known.

7.3.5.1 Further Studies Recommended

A high level heritage assessment should be undertaken to confirm that there will be no impacts on listed buildings and other key receptors should the project be taken forward to the next stage.

7.3.6 Noise, Vibration and Air Quality

It is considered unlikely that there will be operational noise or air quality impacts on sensitive receptors. Therefore, given the nature of the scheme, an assessment is not undertaken in this study. Upon understanding the full extent of the proposed works, there will be a requirement to undertake an assessment at the next stage to further understand potential impacts.

It is considered that, during the construction period of the proposed works, there would be the potential for temporary disturbance to local receptors from noise, vibration and dust resulting from construction activities. However, these issues can be mitigated through the development of an appropriate Construction Environmental Management Plan. It is not anticipated that any formal surveys will be required, although it is recommended that liaison with the Moray Council's Environmental Health service is undertaken to confirm this.

7.3.7 Marine Ecology and Nature Conservation

A desk based review of freely available ecological data carried out of the study area includes statutory designated sites and other protected and notable ecological features that may be potentially impacted by the proposed works.

The statutory designated sites identified in Figure 7-2 (Spey Bay SSSI and Spey Bay GCR) are located on the edge of the proposed site. Once the extents of the proposed works are confirmed, further review will be required at the next stage to confirm potential impacts on these and other designated sites.

It is unlikely, given the nature of the scheme, that there will be potential impacts from operational noise or air quality. However, a further review will be required to confirm this should the project be taken forward to the next stage.

7.3.7.1 Further Studies Recommended

As noted above, the assessment comprises a desk based review of publically available information and previous published reports. A terrestrial site walkover and full biological data centre search have not been undertaken to confirm the potential for protected species and habitats.

There is also no detailed plan regarding construction footprints and construction methods, which would be required to fully understand the predicted impacts of the proposed works on ecological receptors.

Table 7-3 lists the further studies recommended.

Table 7-3: Recommended ecological studies.

Recommendation	Justification
Full biological record centre search	To obtain detailed protected species, habitat and site (statutory and non-statutory) information to inform a judgement on potential impacts from the scheme and inform the decision to conduct more targeted ecological surveys.

Recommendation	Justification
Preliminary Ecological Appraisal of the site	Obtain up to date habitat information and on what is likely to be impacted by the proposed works. Undertaken through a walkover and desk based survey. The results of this appraisal will determine whether further targeted ecological surveys are required.
Consultation with Local Planning Authority (LPA) and Scottish Natural Heritage (SNH)	Obtain opinion regarding likely effects from the scheme and potential impacts to the designated sites.
A Habitat Regulations Appraisal (HRA) Stage 1 Screening Report	Due to the proximity of the Natura 2000 sites (SACs and SPA) it is highly likely that a Stage 1 Habitats Regulations Assessment Screening report would be required to determine if there is the potential to have an adverse effect on site integrity of the designated sites.

Should European Protected Species, i.e. Otters, be identified in the next stage, then appropriate mitigation licences would be required.

7.3.8 Water Quality and Drainage

A desk based review of watercourses within the study area uses Ordnance Survey mapping and the SEPA Water Environment Hub[18]. The following Water Framework Directive (WFD) water bodies fall within the study area:

- Burn of Tynet - a river in the Banff Coastal catchment which is approximately 11.0km in length;
- Lossiemouth to Portgordon - a coastal water body with an area of 79.0km^2;
- Portgordon to Findochty - a coastal water body with an area of 37.1km^2; and
- No undesignated minor watercourses were noted.

The status of these water bodies and quality is set out in Table 7-4 on a condition rating scale ranging from "High" to "Bad", with "Good" being the second highest status of water condition.

Table 7-4: WFD status of coastal and estuary water bodies potentially impacted.

Water Body (ID)	Component	2014	2021	2027	Long Term
Burn of Tynet (23047)	**Overall**	**Good**	**Good**	**Good**	**Good**
	Access for fish migration	High	High	High	High
	Water flows and levels	Good	Good	Good	Good
	Physical condition	Good	Good	Good	Good
	Freedom from invasive species	High	High	High	High
	Water quality	High	High	High	High
Lossiemouth to Portgordon (200147)	**Overall**	**Good**	**Good**	**Good**	**Good**
	Physical condition	High	High	High	High

[18] http://www.sepa.org.uk/data-visualisation/water-environment-hub/

Water Body (ID)	Component	2014	2021	2027	Long Term
	Freedom from invasive species	High	High	High	High
	Water quality	Good	Good	Good	Good
Portgordon to Findochty (200146)	**Overall**	Good	Good	Good	Good
	Physical condition	High	High	High	High
	Freedom from invasive species	High	High	High	High
	Water quality	Good	Good	Good	Good

7.3.8.1 Information Gaps

The following information is outstanding and would be required at the next phase of the project:

- Detailed drainage information (surface waters and foul waters) for the whole area potentially affected by the flood alleviation works. In particular, drainage system of the properties (and their curtilage) in the vicinity of the works would be required to ensure that any interactions with the works are considered within the designs and construction process;
- Information on potentially contaminated land sites in the immediate vicinity that may be disturbed by the works (including the contaminants that they may contain);
- Any water quality sampling/monitoring data from the SEPA monitored water bodies. Consultation with the SEPA will be required;
- Review of existing topographical information available to determine potential flow pathways for flood waters. If required, a further detailed topographical study to be undertaken; and
- Information on the Moray Council's existing surface water and foul water drainage networks that may be connected to the drainage within the working area. Consultation with the local authority will be required.

This information will indicate existing pollutant pathways to sensitive receptors.

7.3.8.2 Further Studies Recommended

Details of the location and extent of the works will inform the scope of future studies which will be required to ensure consents.

7.4 Consenting Requirements

7.4.1 Environmental Impact Assessment

The Town and Country Planning (Environmental Impact Assessment) (Scotland) Regulations 2017[19] ("the EIA Regulations") define an "EIA development" as either a Schedule 1, EIA is mandatory, or Schedule 2, development that is likely to have significant effects on the environment by virtue of factors such as its nature, size or location.

Based on the nature and scale of the potential works options, the project does not currently fall under Schedule 1 development requirements. Schedule 2 listed activities, which are considered relevant to the proposed works, are listed below:

Schedule 2, Class 10 "Infrastructure Projects" has two categories related to the type of development proposed:

- Class 10 (h) "*Inland-waterway construction, canalisation and flood-relief works – applicable if the area of the works exceeds 1 hectare*"

[19] Town and Country Planning, The Town and Country Planning (Environmental Impact Assessment) (Scotland) Regulations 2017, Scottish Statutory Instruments

- Class 10 (m) "*Coastal work to combat erosion and maritime works capable of altering the coast through the construction, for example, of dykes, moles, jetties and other sea defence works, excluding the maintenance and reconstruction of such works*"

At this stage in the project, without exact project extents, it is not possible to determine whether Spey Bay SSSI would be directly affected. If the proposed development is within an environmentally sensitive location (SSSIs are defined in Regulation 2(1) as "sensitive areas"), the development would be required to be screened for the need for EIA.

7.4.2 Planning Consents

At this stage in the project, it is assumed that the preferred options will likely require a planning consent to be granted. While this will be considered and confirmed at a future stage, it is worth noting at this point that there is the potential that the works could be undertaken without formal consent under The Town and Country Planning (General Permitted Development)(Scotland) Order 2011[20] ("PD rights").

PD rights[20] enable certain works to be exercised without formal planning approval, providing they fall within the criteria and thresholds set out in the order.

A number of rights are afforded to development by Local Authorities (i.e. the Moray Council) that have the potential to cover other aspects of any works, including:

> "**Class 30: The erection or construction and the maintenance, improvement or other alteration by a local authority of** –
>
> (a) *Any building, works or equipment not exceeding 4 metres in height or 200 cubic metres in capacity on land belonging to or maintained by them, being building works or equipment required for the purposes of any function exercised by them on that land otherwise than as statutory undertakers*;"

However, as noted above, as it is expected that the capacity will exceed this threshold, it would be considered that a formal planning application would be required.

7.4.3 Marine Licence

As noted in 7.3.3, all options involve the boundaries of the existing defences to be extended below MHWS. As a result, it is considered likely that a Marine Licence will be required. Under The Marine Licensing (Exempted Activities) (Scottish Inshore Region) Order 2011[21] there are a series of exemptions afforded to certain works, including:

> **"Maintenance of coast protection, drainage and flood defence works**
>
> *20.—(1) This article applies to an activity carried on by or on behalf of a local authority for the purpose of maintaining any—*
>
> *a) coast protection works;*
>
> *b) drainage works; or*
>
> *c) flood defence works.*
>
> *(2) This article is subject to the condition that the activity is carried on within the existing boundaries of the works being maintained.*
>
> *(3) This article does not apply in relation to any beach replenishment."*

[20] Town and Country Planning, The Town and Country Planning (General Permitted Development) (Scotland) Amendment Order 2011, Scottish Statutory Instruments
[21] Marine Licensing (Exempted Activities) (Scottish Inshore Region) Order 2011

As noted in 20(2), this exemption is on the condition that the activity is carried out within the existing boundary of the current footprint. Given that all options on the proposed works exceed the current footprint it would be considered that this exemption may not be applicable and a formal Marine Licence will be required from the licensing authority, Marine Scotland Licensing Operations Team (MS-LOT). It is recommended that early discussions are initiated with MS-LOT to confirm assumptions and agree the licencing requirements. This would include clarity regarding timescales, fees and consultation. However, as it is not considered that the proposed works fall within the prescribed activities as identified in Part 2 of the Marine Scotland Guidance (2015), it is not anticipated that this activity will require pre-application consultation undertakings. This would be confirmed in discussions with MS-LOT.

7.4.4 CAR Licence

It is anticipated that the works will not require to be licenced under the Water Environment (Controlled Activities) (Scotland) Regulations 2011[22] (as amended) given the location. However, once further detailed information is available, a scoping exercise will be required, which will include consultation with SEPA. Under the WFD, no deterioration of water quality is permitted and therefore any further studies will need to demonstrate that there will be no adverse impacts on the water bodies identified.

7.5 Summary

As noted in Section 7.1, the purpose of this section is to review potential planning or environmental constraints of the site and identify potential future survey work and consents that may be required to deliver the proposed scheme. This is based on the current information on the proposed works and may be subject to change.

Given the information available at this date, there are no clear environmental constraints that will prevent the proposed works being undertaken. However, acknowledging that the preferred flood mitigation option is to still be confirmed, a number of additional assessments will be required at future stages of the project, as noted throughout this section. As summarised in Table 7-5 below, a series of assumptions would require resolution prior to confirmation of consents requirements. This would need to be undertaken in due course when the designs have been confirmed.

[22] Environmental Protection, Water, The Water Environment (Controlled Activities) (Scotland) Regulations, 2011 (as amended), Scottish Statutory Instruments

Table 7-5: Summary of next stage requirements.

Topic	Work required in next stage
Terrestrial and Marine Planning	• A detailed review of any extant planning applications in the local area is undertaken, particularly around potential allocations sites (refer to Figure 5-1) • Upon understanding the full extent of the proposed works and any residual impacts, contact should be made with the Moray Council Planning Department to discuss any potential impacts, namely on the cycle route and footpath, and to seek confirmation that Permitted Development rights would not apply to the proposed works and as such a formal planning application would be required. • Consultation with MS-LOT to confirm the requirement for a Marine Licence.
Heritage, Noise, Vibration and Air	• A high level heritage assessment should be undertaken to confirm that there will be no impacts on listed and other key receptors. • A high level Noise, Vibration and Air assessment based on further detail of design, including discussions with the Moray Council's Environmental Health Officer (EHO).
Water Quality	• Detailed drainage information (surface waters and foul waters) for the whole area potentially affected by the flood alleviation works. • Information on potentially contaminated land sites in the immediate vicinity that may be disturbed by the works (including the contaminants that they may contain). • Review of any water quality sampling/monitoring data from the SEPA monitored water bodies. Consultation with the SEPA will be required. • Detailed topographical information to determine potential flow pathways for flood waters. • Information on the Moray Council's existing surface water and foul water drainage networks that may be connected to the drainage within the working area. Consultation with the Moray Council will be required. • Details of the location and extent of works will indicate the scope of future studies (Environmental Appraisal or similar).
Ecology (Marine and Terrestrial)	• Full biological record centre search • Preliminary Ecological Appraisal of the site • Consultation with the Moray Council and Scottish Natural Heritage (SNH) • A Habitat Regulations Appraisal (HRA) Stage 1 Screening Report

8. Options Appraisal Assessment

The hydrodynamic model gives a mean overtopping rate at the setback wall for the maintained existing defence of 305.46 l/s/m during a 1 in 200-year event in 2122. This translates to a widespread flooding of properties with a maximum flood depth of up to 1.50m. Five options aimed at reducing overtopping, and improving drainage where appropriate, are identified in Section 3. This section considers each of the five options in turn and appraises each option with respect to its potential performance in terms of reducing overtopping, its capital and maintenance cost, and its benefit cost ratio.

8.1 Failed Existing Defence (Do Nothing)

Over time, further damage to the revetment slope is inevitable and the condition of the setback wall is expected to deteriorate if no repairs are carried out. Increased deterioration of the structure may lead to progressive failure of the worst affected areas and an increase in the wave overtopping that currently occurs. This would increase the vulnerability of the houses behind the revetment and may compromise the structural integrity of the road. Sand and gravel would be expected to continue to collect between the rock armour, reducing its effectiveness. If the structure is not maintained regularly, it is assumed that the rate of damage will accelerate. This option is not recommended; it embodies high risk as it is likely that a large storm might cause significant damage. There is no financial outlay associated with this option.

8.2 Maintained Existing Defence (Do Minimum)

This option is intended to help prolong the life of the existing sea defence structure, mitigate further deterioration and increase the effectiveness of the existing rock armour, if maintained. However, these steps have a limited beneficial impact on the wave overtopping performance of the structure and, as with the failed existing structure, it would not protect the properties against a 1 in 200-year event. The modelling indicates that the flooding extent and depths would only be slightly reduced in comparison to the Failed Existing Defence option. A significant volume of water would still be trapped behind the setback wall during the 200-year event, causing flooding of the road and adjacent properties. There would be no capital expenditure associated with this option, but maintenance works would be required periodically.

8.3 Option 1: Rock Armour Berm Over Upper Slope

Option 1, as described fully in Section 3.4, comprises a rock armour berm that is constructed over the existing wave return wall and extends landward to a new setback wall. The new setback wall is built to a higher level that the existing setback wall and higher than the rock armour berm crest. This is beneficial in terms of reducing the overtopping rate, but has negative implications for the aesthetic appeal of the seafront.

During a 1 in 200-year event, this defence option is expected to reduce the overtopping rate at the setback wall to a mean of 67.35 l/s/m in 2122. This remains significantly higher than the recommended limit for safe pedestrian and vehicle usage according to EurOtop guidance[2] (Appendix A).

Whilst this option does not fully eliminate the high levels of overtopping expected, it does significantly reduce the modelled rate of overtopping in comparison to the existing conditions. However, a significant volume of water could still potentially accumulate behind the setback wall over the course of a 25-hour storm event period, albeit at a slower rate than with the existing defences in place.

It is proposed that additional drainage baffles are installed along the setback wall at approximately 50m centres. The baffles allow drainage of overtopped water back towards the sea which reduces the retained water level landward of the setback wall. In comparison to the Do Minimum option, the drainage modelling predicts that the flood extent and depths will be slightly reduced as a result. This is particularly apparent towards the east end of the seafront where some properties would be fully protected from flooding during a 1 in 200-year event under Option 1.

Although there may not be free flow of return water through the baffles in Option 1 due to presence of rock armour, the drainage modelling indicates that installing additional baffles still reduces the volume of water accumulating behind the setback wall. Refer to Section 5 for further details of the drainage modelling methodology followed and results produced.

The road gully drainage system is proposed to be replaced while renewing the setback wall, although it has been conservatively assumed that the road gully drainage system does not contribute to the drainage of overtopped water. Therefore, it is expected that the volume of water calculated to accumulate behind the wall is conservative at this stage and represents a worst case scenario. It is recognised that the rock armour arrangement around the road gulley drainage outfalls would require further development during detailed design to allow unrestricted access for maintenance.

The capital cost estimate for this option is in the region of £11.1 million, which is the second least expensive of the options considered in this study. The cost to build and maintain this option, which the initial modelling work predicts will not meet the EurOtop (2007)[2] 10 l/s/m limit suggested by for well-prepared pedestrians, could be prohibitive.

8.4 Option 2: High Rock Armour Berm Over Existing Lower Seawall

The rock armour berm is the main element of Option 2 as the existing setback wall is not replaced. The berm is the highest of the rock armour options, resulting in a greater visual impact as the line of sight from the properties to the sea is further blocked by the high berm level. However, it is lower than the crest level proposed for Option 4 and equal to that for Option 5.

During a 1 in 200-year event, this option is expected to reduce the overtopping rate at the setback wall to a mean of 2.16 l/s/m in 2122; this reduces the overtopping rate to within a safe, acceptable limit for trained pedestrian staff, according to EurOtop guidance[2] (Appendix A).

The modelling indicates that Option 2 reduces overtopping most of the options considered, and achieves less than 10 l/s/m together with Options 4 and 5. Furthermore, it is reasonably expected that events of a lesser magnitude than the 1 in 200-year climate change event conditions are likely to have reduced overtopping and corresponding flood risk.

It is proposed that additional drainage baffles are installed along the setback wall at approximately 50m centres. The baffles allow drainage of overtopped water back towards the sea which reduces the retained water level landward of the setback wall. Although Option 2 still results in a limited volume of overtopped water, the drainage modelling indicates that the additional baffles limit the extent of flooding to the road area only, with no flooding of properties. The flood extent is vastly improved in comparison to the other rock armour options, yet Options 4 and 5 reduce the flood extent further. Refer to Section 5 for further details of the drainage modelling methodology followed and results produced.

As with Option 1, the road gully drainage system is proposed to be replaced while renewing the setback wall, although it has been conservatively assumed that the road gully drainage system does not contribute to the drainage of overtopped water. Therefore, it is expected that the volume of water calculated to accumulate behind the wall is conservative at this stage and represents a worst case scenario.

The construction phase of this option is likely to cause the least disruption to the town of Portgordon, as the existing setback wall remains in place. Options 1 and 3 require the pavement to be excavated so that the foundations of the new wall could be laid, whereas for Option 2, patch repairs are undertaken on the wall and baffle details are installed at discrete locations only. In addition, this option has the advantage that the sloping revetment can still be partially accessed along the length of the wall, which would be beneficial for public amenity. Furthermore, the berm terminates downslope of the existing gulley drainage outfalls which allows unrestricted access for maintenance if required.

The capital cost estimate for this option is in the region of £10.6 million, which is the least expensive of those considered.

8.5 Option 3: Rock Armour Berm Extended Seaward

Option 3 has the lowest berm crest of all the options, but extends the farthest seaward. As with Option 1, the presence of rock armour on the existing revetment slope restricts pedestrian access and use of the slope. It requires the largest volume of rock armour of all the options, the transport of which will have greater negative impact. The construction period is likely to be the longest of the rock armour options, up to 18 months as opposed to 12 months for the others; this is due principally to the size of the berm compared to the other options.

During a 1 in 200-year event, Option 3 is expected to reduce the overtopping rate at the setback wall to a mean of 27.35 l/s/m. The option performs better than Option 1, but not as well as Options 2, 4, or 5 and it reduces the rate to a level acceptable to traffic, according to EurOtop guidance[2] (Appendix A).

It is proposed that additional drainage baffles are installed along the setback wall at approximately 50m centres. The baffles allow drainage of overtopped water back towards the sea which reduces the retained water level landward of the setback wall. The results of the drainage modelling indicate that there is residual flooding given the volume of water that still overtops the setback wall. Flooding extent and depths are reduced in comparison to Option 1, yet not nearly as reduced as Option 2. Multiple properties would still be expected to experience flooding during a 1 in 200-year event. Although there may not be free flow of return water through the baffles in Option 1 due to presence of rock armour, the drainage modelling indicates that installing additional baffles still reduces the volume of water accumulating behind the setback wall. Refer to Section 5 for further details of the drainage methodology followed and results produced.

As with Option 1, the road gully drainage system is proposed to be replaced while renewing the setback wall, although it has been conservatively assumed that the road gully drainage system does not contribute to the drainage of overtopped water. Therefore, it is expected that the volume of water calculated to accumulate behind the wall is conservative at this stage and represents a worst case scenario. It is recognised that the rock armour arrangement around the road gulley drainage outfalls would require further development during detailed design to allow unrestricted access for maintenance.

The capital cost estimate for this option is in the region of £16.1 million, which is the most expensive rock armour option and the second most expensive of all options considered.

8.6 Option 4: Stepped Revetment

Option 4 is a composite structure comprising a rock armour revetment seaward of the existing wave return wall and concrete stepped revetment rising landward from the wave return wall. The top step of the revetment incorporates a bullnose wave return wall with a crest elevation of +6.84m ODN, which is the highest of all the options, resulting in the greatest visual impact to local residents and road users.

During a 1 in 200-year event, Option 4 is expected to reduce the overtopping rate at the setback wall to a mean peak rate of 7.57 l/s/m at the setback wall. This is within the safe, acceptable limit for trained pedestrian staff, according to EurOtop guidance[2] (Appendix A).

It is proposed that additional drainage baffles are installed along the setback wall at approximately 50m centres. The baffles allow drainage of overtopped water back towards the sea which reduces the retained water level landward of the setback wall. The results of the drainage modelling indicate that Option 4 is more effective than the other options considered in terms of reducing flooding beyond the setback wall. Flooding is limited to a small section towards the western end of the road, with no flooding of nearby properties. It has been conservatively assumed that the road gully drainage system does not contribute to the drainage of overtopped water. Therefore, it is expected that the volume of water calculated to accumulate behind the wall is conservative at this stage and represents a worst case scenario.

A key disadvantage of the stepped revetment solution is the visual impact of such a large, imposing structure on the view from the road and nearby properties. The revetment has the highest crest level of options considered under this study which would likely restrict the sea view from the road and ground floor of nearby properties.

Furthermore, the concrete stepped revetment structure is less in keeping with the natural surroundings than the proposed rock armour solutions.

The construction phase for Options 4 and 5 are likely to cause the most disruption to the town of Portgordon of the options considered. Although it is anticipated that the majority of the existing rock armour may be reused for the development of Option 4, the concrete section of the revetment would have to be transported to site in pre-cast sections or by concrete trucks and cast on site. Alternatively, a batching plant could be established on-site due to the high volume of concrete required and the relatively remote site location. The logistics of transporting materials for the construction of such a large structure would be likely to cause disruption to the local road network and residents. However, Options 4 and 5 do not require the setback wall to be demolished and re-built, as is the case with Options 1 and 3, which would reduce disruption. Furthermore, the concrete stepped revetment terminates downslope of the existing gulley drainage outfalls which allows unrestricted maintenance if required.

Drainage is required through the concrete stepped revetment to allow overtopped water to drain seaward. Therefore, there will be an ongoing requirement to maintain the drainage system with associated financial commitment. Furthermore, as the drainage is sized approximately for a 1 in 200-year event, there is a risk that an event of greater magnitude than a 1 in 200-year event could result in overtopped water being trapped. The drainage pipes through the stepped revetment would require flap valves or similar to prevent flow of water through the defence in a landward direction during a high water event.

The capital cost estimate for this option is in the region of £11.9 million, which is the least expensive concrete option considered, yet more expensive than rock armour Options 1 and 2.

8.7 Option 5: Wave Return Wall

Option 5 involves removing the existing rock armour and enlarging the existing wave return wall to a crest elevation of +6.34m ODN, which is equal to that of the highest rock armour option yet lower than the stepped revetment in Option 4.

During a 1 in 200-year event, Option 5 is expected to reduce the overtopping rate at the setback wall to a mean of 4.42 l/s/m. This is within acceptable limit for trained pedestrian staff, according to EurOtop guidance[2] (Appendix A).

It is proposed that additional drainage baffles are installed along the setback wall at approximately 50m centres. The baffles allow drainage of overtopped water back towards the sea which reduces the retained water level landward of the setback wall. The results of the drainage modelling indicate that Option 5 is the second most effective of the options considered to reduce flooding beyond the setback wall. Similar to the best performing Option 4, flooding is limited to a small section towards the western end of the road, with no flooding of nearby properties. It has been conservatively assumed that the road gully drainage system does not contribute to the drainage of overtopped water. Therefore, it is expected that the volume of water calculated to accumulate behind the wall is conservative at this stage and represents a worst case scenario.

Similar to Option 4, an enlarged wave return wall is likely to have a significant visual impact on the view from the road and nearby properties. Although the crest level is lower than for Option 4, it is still likely to restrict the sea view from the road and ground floor of nearby properties. Furthermore, the wave return wall is less in keeping with the natural surroundings than the proposed rock armour solutions.

The construction phase for Options 4 and 5 are likely to cause the most disruption to the town of Portgordon. The existing rock armour would have to be removed entirely prior to construction of the wave return wall. Thereafter, the materials required for the construction of the wall could be transported to site by concrete mixer and cast on site. Alternatively, a batching plant could be established on-site due to the high volume of concrete required and the relatively remote site location. The logistics of transporting such a large volume of concrete or constituent materials has the potential to cause disruption to the local road network and residents. However, Options 4 and 5 do not require the setback wall to be demolished and re-built, as is the case with Options 1 and 3, which would reduce disruption.

Similar to Option 2, this option has the advantage that the sloping revetment can still be partially accessed along the length of the wall, which would be beneficial for public amenity. Unlike Options 1-4, Option 5 requires removal of the existing rock armour and does not re-use any within the scheme.

Drainage is required through the solid wave return wall to allow overtopped water to drain seaward. Therefore, there will be an ongoing requirement to maintain the drainage with associated financial commitment. Furthermore, as the drainage is sized approximately for a 1 in 200-year event, there is a risk that an event of greater magnitude than a 1 in 200-year event could result in overtopped water being trapped. The drainage pipes through the stepped revetment would require flap valves or similar to prevent flow of water through the defence in a landward direction during a high water event.

Based on the proposed cross section of Option 5 at this stage (refer to Appendix C), the existing drainage system is unlikely to be compatible due to the raised embankment level between the new wave return wall and the setback wall. An upgraded drainage system consisting of extended outfalls passing through the new wave return wall is likely to be required, the details of which would require development should Option 5 be taken forward.

It may be possible to review the area between the wave return wall and the setback wall in order to provide a catchment zone for overtopped water. Removal of the drainage baffles in the setback wall would retain water in the catchment zone and allow drainage towards the sea via the drainage incorporated in the wave return wall. However, further analysis would be required to establish the impact of an event of greater magnitude than the 1 in 200-year event and the risk of trapping overtopped water on the road. The distance between the wave return wall and the setback wall would also require to be assessed to determine if splash and spray is likely to still reach the road.

A large portion of the costs associated with Option 5 are attributed to the specialist formwork required to cast the curved return face. However, it may be possible to revisit the overtopping modelling for a straight wall of higher crest elevation which would be likely to result in a reduced construction cost. Alternatively, the curved wall section may be precast in sections and delivered to site for integration with the cast section.

The capital cost estimate for this option is in the region of £16.4 million, which is the most expensive of the options considered.

8.8 Summary

Based on performance in terms of reducing the rate of overtopping and the resulting flooding extent, whilst considering the estimated capital and maintenance costs, Option 2 is the preferred option of those considered herein. Although the drainage modelling indicates that Options 4 and 5 reduce the flood extent further than Option 2, flooding during the 1 in 200-year event is limited to the Stewart Street and Lennox Place with no damage to nearby properties. Furthermore, the added advantages in terms of reduced disruption during construction, reuse of the existing rock armour and compatibility with the local environment in comparison with the concrete defence options add weight to the choice of preferred option.

Based on the findings of the overtopping assessment, Option 2 has been shown to reduce the overtopping to the lowest of the options considered. Only three of the five options were found to satisfy the 10 l/s/m limit for pedestrian traffic in accordance with the EurOtop guidance[2]. Options 1-3 incorporate a fully engineered rock armour berm with a uniform slope angle and toe stabilisation trench, which may result in better wave energy absorption and subsequently lower overtopping rates than are predicted by the conservative modelling technique.

Recognising the magnitude of the current flooding issue at the Portgordon seafront, due to overtopping waves, it is expected that any of the options considered would incur relatively significant capital cost and involve major construction works. Initial enquiries indicate that rock of sufficient size and strength for Options 1-4 would be available in the north of Scotland. However, if this is not the case, it may be more cost effective to use concrete armour units rather than rock armour to construct enhanced sea defences at Portgordon. However, it should be noted that rock armour may look more natural at Portgordon and is in keeping with what is currently in place. The concrete required to construct Options 4 and 5 would likely be transported to site by articulated trucks, or a batching plant could be established on-site due to the high volume required and the remote site location. The logistics of transporting such a large volume of concrete or constituent materials would cause significant disruption to the local road network and residents. This is also applicable to Option 3 which requires a far greater volume of rock armour than Options 1 or 2, resulting in a greater level of disruption.

Excavation is required for each of the options to toe the proposed defences into the seabed for stability purposes. The geotechnical conditions are unknown at this stage and so geotechnical investigation would be required prior to construction. Geotechnical investigation may also be required to confirm the bearing capacity of the existing ground to support the large added mass associated with each of the options. If rock excavation is required at any point, this may have an impact on construction costs and duration.

From a visual impact perspective, all five options considered herein would have an impact on the view out to sea. Option 4 has the most significant impact with the highest crest elevation and being of concrete construction. It is likely that road users, pedestrians and seafront properties would no longer have an unrestricted view out to the sea. Option 5 would also have a significant visual impact, being of concrete construction, yet it has an equal crest elevation equal to Option 2.

Of the rock armour solutions, Option 3 which has a lower, longer berm has been included in this study for comparison with the higher crested Options 1 and 2. However, although it performs reasonably well from an overtopping perspective, the initial analysis suggests that it is less effective than the higher crested Option 2 which is likely to have a greater visual impact from the land.

Maintenance of Options 1-3 is likely to be similar. It is anticipated that minor maintenance works might be required every five years, with more major maintenance works required every 20 years. Maintenance is likely to involve the replacement of displaced rock armour units to improve the stability of the structure. As Option 3 involves a larger volume of rock armour, it is likely to incur higher associated maintenance costs. Maintenance of the concrete structures for Options 4 and 5 is expected to be minimal, consisting of periodic visual inspection throughout the design life and patch repairs if required. However, the drainage system associated with Options 4 and 5 to drain overtopping water towards the sea would require routine cleaning to ensure that it is free of debris.

The health and safety aspects of each option are likely to be similar. Any water that overtops the structure has the potential to create a health and safety risk to the public.

No one option is considered to be particularly onerous from a planning or environmental perspective.

The five options modelled were chosen to offer a range of defence solutions, while considering variations of defence crest elevation and setback wall elevation, as well as construction materials. Different combinations of the widths of berms and levels of both rock armour and setback wall are used in the options considered within the study and the variation in overtopping results shown. Although some preliminary refinement of the options has been undertaken in terms of the elevation and geometry of each defence, it is suggested that the options are refined further if the project is taken forward by undertaking further detailed numerical and physical modelling.

The options considered in this study are large structures that involve significant associated construction costs and disruption, due to the magnitude of the events they are designed to protect against. They have been developed to provide an enhanced degree of physical protection along the coastline at Portgordon whilst taking into consideration the visual impact from both sea and shore. Although higher, more imposing structures would be likely to reduce the overtopping rates further, the outcomes of the initial modelling work indicate that the preferred Option 2 reduces flooding to acceptable levels for the 1 in 200-year design event.

9. Economic Assessment

9.1 Introduction

This section presents an economic assessment of the reduction in flooding of Portgordon for each of the options considered. The methodology is based upon assessing the frequency and volume of seawater overtopping the existing and proposed coastal defences under a range of sea level and wave magnitudes a defined appraisal period of 100-years (refer to Section 3.1), and the monetary value of damages these events will cause to property over this period. The results of the benefit cost analysis (BCA) will be used to rank all options to assist in determining a preferred option.

The economic damages (costs) and benefits (avoided costs) of the following options are considered within this assessment:

- Do Nothing Scenario (all maintenance ceases and no repairs are undertaken)
- Do Minimum Scenario (ongoing maintenance)
- Option 1: Rock Armour Berm Over Upper Slope
- Option 2: High Rock Armour Berm Over Existing Lower Seawall
- Option 3: Rock Armour Berm Extended Seaward
- Option 4: Stepped Revetment
- Option 5: Wave Return Wall

Details of the above options are provided in Section 3.

The methodology used for the economic assessment follows UK Treasury Green Book: Appraisal and Evaluation in Central Government[23], which is the industry standard method for economically assessing flood risk projects. Guidance provided in the Multi-coloured Manual (MCM), and its supporting Handbook[24], is used to assess the relationship between the benefits and costs of the options, allowing for the calculation of benefit cost ratios (BCRs), incremental benefit cost ratio (IBCR) and the Net Present Value (NPV) for each defence option. This section sets out the types of flood damages, the method of assessment and how they are incorporated into the BCA. The results of the BCA will identify the preferred option from an economic perspective.

9.2 Appraisal Period

As discussed in Section 3.1, this report considers a 100-year allowance for climate change. The new defence options will have a design life of 100-years with active maintenance.

These factors indicate that a **100-year appraisal period** is appropriate for the cost/benefit analysis and economic assessment of the proposed options.

9.3 Flood Receptors and Damage Type

The economic damages associated with flooding can be split into four categories; direct damages, indirect damages, tangible and intangible damages[25].

[23] HM Treasury (2016). The Green Book: appraisal and evaluation in central government.
[24] Penning-Rowsell *et al*., 2013. Handbook for Economic Appraisal
[25] Penning-Rowsell *et al*., 2013. Handbook for Economic Appraisal, Table 3.1

Direct damages are those resulting from physical damage caused by flooding, such as damage to residential buildings, risk to life and physical road repair. For non-residential properties, financial losses refer only to direct damage to stock or property as it is assumed that the financial loss at one flooded business would be offset by a financial increase at another nearby unaffected business, as the requirement for this trade would not be impacted by the flooding. Indirect damages are those which occur as a result of the flooding such as the cost of temporary accommodation.

Intangible damages are calculated for residential properties following guidance from the Defra Supplementary Note on 'Appraisal of Human Related Intangible Impacts of Flooding' (R&D Technical Report FD2005/TR)[26]. These account for the human-related impacts of flooding such as increased stress, health effects and loss of memorabilia.

Environmental damage to habitats and ecosystems was not considered as no impact to habitats or ecosystems was identified as a result of flood damage in Portgordon. Emergency services will be required to attend in both an emergency works and clean-up capacity.

The damages included in this assessment are shown in Figure 9-1.

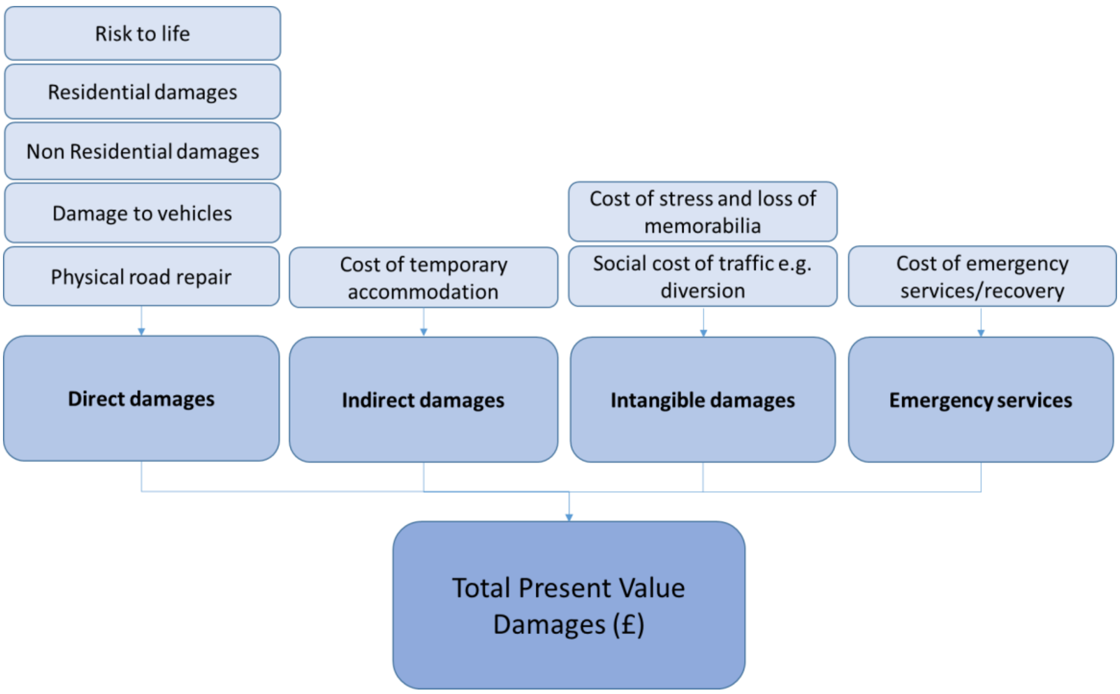

Figure 9-1: Damages considered in economic assessment.

9.4 Direct Damages

9.4.1 Residential Properties

Calculation of the direct damages to residential properties as a result of flooding considers the geographical location of the property, the depth of the flood water, the saltwater uplift factor and the market value of the property. These factors are discussed in detail in the following sub-sections.

9.4.1.1 Property Dataset and Threshold Levels

The Corporate Address Gazetteer (CAG) has been used as a basis for the Portgordon property dataset. This dataset provided the address information and geographical location of each property within Portgordon.

[26] Defra Supplementary Note on 'Appraisal of Human Related Intangible Impacts of Flooding' (R&D Technical Report FD2005/TR)

A review of the dataset was undertaken to check that the information contained therein is current. The property points are updated to reflect recent developments, and information such as property use (residential or non-residential) and property type (detached, terraced, etc.) was incorporated. This data was required for the calculation of flood damages, as discussed later in Section 9.9. The review was based upon Ordnance Survey mapping and street-level photography freely available online.

Property threshold levels are a key factor in assessing vulnerability to flooding. The threshold level for each property has been estimated by using the Digital Elevation Model (LiDAR) to provide local ground levels and estimating the relative level of the thresholds based on street-level photography. Approximately 40% of properties located within the study area were found to have a floor level raised above street level. Estimated property thresholds ranged from 50mm to 300mm where there was a significant step up into the property. This has been reflected in the receptor dataset.

9.4.1.2 Flood Depths

Flood depths were extracted from the flood depth grids generated for this study using a property footprint method (refer to Section 5.5). Values were extracted based on Ordnance Survey Mastermap building footprints using the QGIS-Zonal Statistics tool.

For the purposes of the economic assessment a review of mean and maximum flood depths was carried out. These results indicated that due to property thresholds, certain properties were predicted to flood based on the maximum depth in the property, but not the mean depth, because of a few millimetres depth difference between the two results. As such, determining property damages based on the maximum depth within each property was considered to be a conservative but appropriate approach.

Jacobs' in-house tool EcMap has been used to determine the flood damages at each property. The tool uses hydraulic model predictions from a 2D flood model (Tuflow[27]) which are intersected with the National Receptor Database (NRD) of properties. Each unique property point is assigned a flood depth for each modelled probability event. This is based on the parameters from the Multi-Coloured Manual (MCM) (2014) guidance (updated to a base date of 2022).

9.4.1.3 Saltwater Uplift Factor

Additional damages to properties may be experienced due to saltwater flooding compared to freshwater flooding. To account for this, an uplift factor of 1.22 has been applied within EcMap to the damages calculated for both residential and non-residential properties, as per MCM guidance[28].

9.4.1.4 Property Valuations (Capped Damages)

Residential property values for Portgordon have been derived from the Land Registry[29] website and are based on the average property values within the Moray Council administrative area. Residential property values use 2014 average sale prices which are uplifted to 2022 prices using the Consumer Price Index (CPI) within the EcMap model.

Where the direct damage of a residential property exceeds the property value, the direct damage will be capped at the above valuations. This is standard economic practice and effectively caps the economic damage for each property at its current market value.

9.4.2 Non-Residential Properties

The value of non-residential properties is calculated using the following equation[30]:

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27 Tuflow 2D hydraulic model - version 2016-03-AE-iDP-w64
28 MCM-Online (2013): The Manual
29 HM Land Registry (2017), https://www.gov.uk/government/organisations/land-registry
30 Penning-Rowsell *et al*., 2013. Handbook for Economic Appraisal

$$Market\ Value = Rateable\ Value * \left(\frac{100}{Equivalent\ Yield}\right)$$

Non-residential property rateable values[31] have been determined from the Scottish Assessors Association (SAA) website[32]. Only two non-residential properties are located within the maximum flood extent in Portgordon, two retail properties: a hairdressing salon and a pub. The total rateable value of each property was provided by the SAA, and the floor area for each property was used to calculate the rateable value for each property to be £39/m^2. The average rateable value based on both properties was taken forward, and was uplifted using the Consumer Price Index (CPI) to January 2018 values.

Equivalent yield has been derived from the Office for National Statistics (ONS), and taken as the average across the whole of England, excluding London. ONS does not provide equivalent yield values for Scotland. The equivalent yield for the non-residential properties is 6.5%.

9.4.3 Motor Vehicles

The MCM recommends that for an "overview" project appraisal, the ratio of average property to vehicle damage should be considered. It assumes that the total number of vehicles likely to be damaged will equate to 28% of the total number of residential and non-residential properties at risk from flooding. This is then multiplied by £3,100, the damage sustained by the vehicle[33].

9.4.4 Physical Road Damage

The latest guidance from the SPONS manual[34] and DMRB[35], combined with professional experience for similar scale road reinstatement, has been used to estimate the cost per km for the reinstatement of affected roads in Portgordon. For this assessment, A-roads have been valued at £1.25 million per km and minor roads (B-roads, side lanes etc.) are valued at £625,000 per km (2018 values). The assessment has been conducted at a high level and therefore excludes earthworks and structures such as footpaths, junctions, signs, road restraints etc.

9.4.5 Risk to Life

Risk to Life assesses the likely hood of injuries and fatalities during a flood event. These fatalities and injuries are assigned a monetary value as indicated in the Treasury Green Book[36].

Defra's guidance[37] on assessing and valuing risk to life was used to calculate intangible damages of flooding on an individual or society. This method is based on determining flood hazard, area vulnerability and people vulnerability resulting in an economic value of average annual individual or risk of fatality due to flooding. This Reference Valuation is applied within the cost-benefit analysis.

NOMIS data provided population statistics including the prevalence of an elderly population, the number of residents suffering from long term illness and those receiving personal independence payments (PIP) and disability living allowance (DLA) payments, which was used to determine the 'at-risk' population. The value of cost per fatality and cost per serious injury were sourced from the Department for Transport (DfT), in line with Defra's guidance. DfT values the reduction of risk of death at £1,618,000 and the risk of injury at £128,650 (2017 prices).

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[31] Rateable value is a value assigned to a commercial building based on its size location etc. The value is used to determine the rates payable by the owner of the building.

[32] SAA (2017), https://www.saa.gov.uk/

[33] Penning-Rowsell *et al*., 2013. Handbook for Economic Appraisal, Table 4.4

[34] Spain, B., Spon's Estimating Costs Guide to Minor Works, Alterations and Repairs to Fire, Flood, Gale and Theft Damage: Unit Rates and Project Costs. 2003. CRC Press.

[35] Highways Agency. 2002. Design Manual for Roads and Bridges. Volume 6. TD9/93

[36] HM Treasury (2016)

[37] Defra (2008). Assessing and valuing the risk to life from flooding for use in appraisal of risk management measures.

9.5 Indirect Damages

9.5.1 Temporary Accommodation (Evacuation Costs)

The indirect cost of flooding to residential properties is considered in this assessment. This includes costs of temporary accommodation and the costs of additional electricity consumption associated with drying and heating a property following each flood event.

9.6 Human-Related Impacts (Intangible Damages)

9.6.1 Intangible Damages

As per Defra guidance the intangible damages of flooding consist of increased stress, health effects and loss of personal belongings.

The intangible damages have been calculated in direct relation to the onset of flooding (taken as the event as which water levels first exceed property threshold levels) for every affected residential property in the study area. The reported value of avoiding intangible impacts of flooding is £213 per residential property per year (August 2004).

9.6.2 Social Cost of Traffic

The estimation of social costs associated with flood damage was also considered and is based on diversion-value methodology approved by the MCM which is based on Highways Agency and Department for Transport[38] data on the estimated values associated with traveller's time. The diversion value method assumes an eight-hour flood event with 150 cars per hour, travelling at a constant speed of 40kmph, will be diverted to neighbouring roads surrounding Portgordon. Calculations can be found in Appendix G.

9.7 Emergency Services

9.7.1 Cost of Emergency Services and Recovery

Emergency and recovery costs during flood events vary between local authorities but often consist of the repair and construction of infrastructure assets including the electrical supply.

The MCM guidance on the calculation of emergency service cost during a flood event was utilised as part of the BCA. A value of 10.7% of the total flood damages for a flood event[39] was incorporated into the model. A review of the study area, using street level photography suggested that a flood warning reduction factor also be applied to account for the properties who have precautionary measures in place such as sandbags. This applies a two percent reduction to all residential and non-residential properties.

9.8 Cost Estimates of Proposed Defence Options

Details of the estimated capital cost for each of the options considered are provided in Section 6. These values have been uplifted to 2022 prices. Maintenance costs have been attributed to the relevant subject year. These base values have been uplifted to the subject year values.

It is noted that there are significant differences in the total operational costs of the scenarios, ranging from £307,000 for the Do Minimum scenario and £16.7 million for Option 5. Option 2 has the lowest PV of the defence options considered herein.

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[38] Department for Transport (2012) UNIT 3.5.6: Values of time and vehicle operating costs. In Transport Analysis Guidance (TAG).
[39] MCM Handbook (2018) Table 6.23

Table 9-1: Calculation of total PV cost.

	Capital cost (£k)	Maintenance cost (£k)	Total PV cost (£k)
Do Nothing			
Do Minimum		307	307
Option 1	11,133	344	11,477
Option 2	10,600	295	10,895
Option 3	16,131	454	16,585
Option 4	11,889	391	12,280
Option 5	16,407	291	16,698

9.9 Flood Damages

9.9.1 Climate Change

The impacts of climate change over the 100-year appraisal period have been considered as part of the economic assessment by factoring in sea level rise predictions as per MCM guidance (refer to Section 3.1).

9.9.2 Residential Properties at Risk

The number of residential properties affected by floods significantly diminishes with the construction of flood defences options, as shown in Table 9-2. This is due to no flood damage occurring across Options 1-5 during 1 in 10 to 1 in 100-year flood events. Furthermore, damage to residential property is mitigated entirely with Options 2,3 and 5 across all flood events up to the 1 in 200-year event.

Table 9-2: Number of Properties at Risk for Varying Return Period Events.

	Properties at Risk						
	Do Nothing	**Do Minimum**	**Option 1**	**Option 2**	**Option 3**	**Option 4**	**Option 5**
1:10	51	49	-	-	-	-	-
1:50	56	53	-	-	-	-	-
1:100	59	57	-	-	-	-	-
1:200	60	59	55	-	34	-	-

9.10 Economic Damages of Flooding with Options

9.10.1 Breached Defences and Methodology

For the purposes of the economic assessment, it has been assumed that the each of the proposed defence options would be maintained to such a standard that failure of the defence would not occur within the 100-year appraisal period. Therefore, deterioration was not factored into the assessment.

The damages for each year were then summed by factoring the impact of climate change to provide the total PV damages over the 100-year appraisal period.

9.10.2 Damages and Avoided Damages

The benefits (damages avoided) for each option (including the Do Minimum scenario) have been calculated by using the Do Nothing option as the baseline against which the other options are compared. The results are presented in Table 9-3. Note that the do minimum scenario incurs £15,691,000 damages over the 100-year period, which is only slightly lower (8% less) than the baseline damage levels. All 'Do-something' options reduce damages incurred with the best performance achieved by Option 2 which is estimated to eliminate over 99% of damages compare to the baseline scenario.

Based on the modelling projections (up to a 1:200 flood), over the 100-year study period the Do Minimum scenario will reduce the number of flood years by 3 relative to the Do-Nothing option. This will result in a flood event occurring on average twice in every five years (40%). The likelihood of a flood event occurring for Options 2,4 and 5 have been eliminated.

Table 9-3: Options damages and avoided damages (benefits).

	PV (£)						
	Do Nothing	Do Minimum	Option 1	Option 2	Option 3	Option 4	Option 5
Damages	17,393	15,961	1,433	122	908	200	247
Avoided Damages	-	1,431	15,959	17,271	16,485	17,192	17,146
Flood Years	43	40	33	0	14	0	0

9.11 Benefit Cost Analysis

9.11.1 NPV, BCR & IBCR

The Benefit Cost Ratio (BCR) for an option is derived by dividing PV benefits by PV costs. A BCR of unity indicates a net neutral opportunity, equivalent to there being no financial incentive or disincentive to progress that option. A BCR greater than unity indicates a positive return on investment from a financial standpoint and should be further investigated.

The NPV is obtained by subtracting the PV costs from the PV benefits. A NPV of zero indicates a net neutral opportunity, that is there is no financial incentive or disincentive to progress that option. A NPV greater than zero indicates a positive return on investment from a financial standpoint and should be further investigated.

The IBCR is derived by $\frac{Expensive\ Option\ Benefits - Cheap\ Option\ Benefits}{Expensive\ Option\ Costs - Cheaper\ Option\ Costs}$ and is used to determine if the additional benefits attributed to an option merits the additional spend compared to cheaper option. The IBCR is used to rank mutually exclusive options in order of preference. A IBCR greater than unity indicates that the more expensive option is preferred over the cheaper scenario.

9.11.2 Optimism Bias 60%

As per EA guidance[40] a 60% optimism bias has been applied to the PV costs to allow for optimism and uncertainty at the project appraisal stage. As shown in Table 9-4, the only option of those considered herein to achieve a positive NPV and a BCR greater than unity when a 60% optimism bias is modelled is the Do Minimum scenario. Option 2 achieves a BCR of 0.99 which indicates that it is on the verge of resulting in an economically viable solution. If construction cost estimates could be reduced by 1.8%, Option 2 would produce a positive NPV. Other options would require construction costs to reduce by 12.9% (Option 4) through 38.9% (Option 3) to achieve a BCR of unity. The Do Minimum option appears to yield the most favourable BCR of the options considered at 60% optimism bias. However, it is important to recognise that the Do Minimum is maintaining the

[40] Environment Agency. Flood and Coastal Erosion Risk Management Appraisal Guidance.

current sea defence arrangement at Portgordon which is recognised as being incapable of sufficiently preventing wave overtopping with regular flooding and associated damages as a result. Furthermore, cognisance should be taken of the additional factors discussed in Section 8.8 which are non-economic advantages. It is envisaged that the optimum bias will reduce as the project and design develops which will ultimately yield a more accurate BCR.

Table 9-4: NPV, BCR & IBCR of all scenarios at 60% optimism bias.

	NPV (£)	BCR	IBCR
Do Minimum	939	2.9	-
Option 1	-2,403	0.87	0.8
Option 2	-161	0.99	0.9
Option 3	-10,051	0.62	0.6
Option 4	-2,455	0.88	0.8
Option 5	-9,570	0.64	0.6

9.11.3 Optimism Bias 30%

In order to investigate the effect of a reduced optimism bias on the NPV and BCR figures which is likely to occur as the project develops, a 30% optimum bias has been applied to the PV costs. When a 30% optimism bias is modelled, in addition to the Do Minimum scenario, Options 1, 2 and 4 achieve positive NPV and a BCR of above unity. Although the Do Minimum scenario still achieves the highest BCR, Option 2 has the highest NPV of the options considered. The IBCR for Option 2 is 1.2 which indicates that although this scenario is more expensive than the Do Minimum option, the additional benefits justify this extra cost. Therefore, Option 2 is the preferred option of those considered herein from an economic perspective at 30% optimism bias.

Table 9-5: NPV, BCR & IBCR of all scenarios at 30% optimism bias.

	NPV (£)	BCR	IBCR
Do Minimum	1,032	3.6	-
Option 1	1,040	1.07	1.0
Option 2	3,107	1.22	1.2
Option 3	-5,076	0.76	0.7
Option 4	1,229	1.08	1.0
Option 5	-4,561	0.79	0.7

9.11.4 NPV and BCR of all Scenarios without Optimism Bias

When no optimism bias is accounted for, all options, except for Option 3, achieve positive NPV and a BCR of greater than unity. The Do Minimum scenario still achieves the highest BCR, whereas Option 2 has the greatest NPV. The IBCR of Option 2 is 1.5 which indicates that although this scenario is more expensive than the Do Minimum option the additional benefits justify this extra spend. Therefore, Option 2 is the preferred option of those considered herein from an economic perspective when no optimism bias is modelled.

Table 9-6: NPV, BCR & IBCR of all scenarios without optimism bias.

	NPV (£)	BCR	IBCR
Do Minimum	1,124	4.7	-
Option 1	4,483	1.39	1.3
Option 2	6,376	1.59	1.5
Option 3	-100	0.99	0.9
Option 4	4,913	1.40	1.3
Option 5	448	1.03	1.0

9.12 Discussion

The limited total expenditure for the Do Minimum scenario results in the largest BCR value. However, the benefits derived are modest (£1.43 million) in comparison to the alternative options, as the extent of the benefits are limited to only 2 fewer residential buildings being impacted during any single flood event (see Table 9-2) compared to the baseline (Do Nothing). As such, the Do Minimum Option provides limited protection to properties and, as it is maintenance only, does not improve the present day risk of flood overtopping. It is due to the limited benefits from the Do Minimum scenario that the NPV remains fairly static when altering optimism bias.

The total capital expenditure estimate for Option 2 is the lowest of the options considered, except for the Do Minimum scenario, and, along with Options 4 and 5, offers the greatest benefits (£17.3 million) of the options considered. It is due to these benefits that Option 2 achieves the greatest NPV in the 30% and 0% optimism bias models. The 30% and 0% optimism bias models show Option 2 achieves the highest IBCR, and therefore is the preferred option under these conditions. In the 60% optimism bias model, Option 2 achieves a BCR of 0.99 and a negative NPV. However, a reduction in capital expenditure of 1.8% would result in a BCR of greater than unity and a positive NPV. The required 1.8% reduction in capital costs falls within the 60% optimism bias margin of error. It is due to the minimal variance required in either costs and or benefits to occur, combined with the benefits incurred, that Option 2 is considered the preferred option.

10. Public Consultation

Following the submission of Revision 1 of this report, a public consultation was carried out whereby the local population were presented with the initial five options to gain their feedback. The public consultation took place on the 21 February 2018.

Twenty-one residents signed in and six feedback forms were received on the day. Additional feedback was provided by email directly to the Moray Council following the event.

The residents living along the coastal front explained their concerns and issues experienced with the existing arrangement, these include:

- Lack of drainage through setback wall prolongs flood duration;
- Concerns that erosion is occurring at the western end of Stewart Street;
- Water lapping up towards properties at the western end of Stewart Street is relatively common; and
- Wooden planks within the existing setback wall were washed away during the most recent flood event, which could allow waves to wash up through the gaps in the setback wall.

The five proposed solutions were put forward to the residents, with explanation that Options 1 and 3 do not do not protect against flooding to the same extent as Options 2, 4 and 5 in their current arrangement. Option 3 was generally the preferred option from an aesthetic perspective by most residents. The scale of the solutions was a significant talking point, and a concern to many residents. Opinion generally varied, with some resident's primary concern being protecting their property from the risk of flooding, something that is becoming frequent for several households. However, some residents were resistant to any significant defence structure being put in place and that their unhindered sea view was paramount to them. Concerns for the solutions put forward included:

- Scale of proposed solutions may detract from the current sea view;
- The solutions may obstruct the view of children, wheelchair users and some ground level properties;
- Aesthetics may discourage tourists, an industry the town is trying to develop;
- Aesthetics may put off new buyers and effect current property values;
- Rock armour solutions would be burdened by issues consistent with the existing defence, such as seaweed build up leading to unpleasant smells;
- Concerns regarding noise associated with waves impacting the large wave return structure. This was a recognised problem with the existing wave return wall prior to the rock armour being placed in front; and
- Restricted access to the beach.

The residents offered suggestions of solutions that they would prefer to see implemented, which frequently focussed on offshore breakwater type structures. Whilst it could be expected that these type structures might have a lesser impact to local residents, they can have significant environmental impacts, such as altering the transport of sediment, potentially affecting coastlines elsewhere. Suitable offshore breakwater type structures are likely to be relatively expensive and, depending on form, could require specialist plant. Such structures are outwith the scope of this study, as discussed further in Section 3.9.

Residents also suggested they would like to see improved drainage to allow water to escape through the setback wall, such as the baffle structures currently used. This was something that residents felt should be carried out soon as the water retention is currently deemed as exacerbating flooding events. It is felt that these works would be modest in scale and could be readily implemented.

There were some comments that grants should be sought for protecting individual properties and sandbags should be delivered in the event of flood warnings.

Whilst the solutions put forward were generally deemed too intrusive there was appreciation from residents that the issue is being investigated by the Council. However, there were concerns that little in the way of physical improvements have been carried out thus far and the residents are keen to be kept informed of future developments.

11. Conclusions and Recommendations

The condition of the sea defences, to the west side of the harbour at Portgordon, is not considered to have deteriorated significantly since they were previously inspected by Jacobs in 2013[1]. However, the issue remains that they do not provide adequate protection against overtopping and subsequent flooding of the adjacent streets and properties during storm conditions.

The results from the wave modelling, overtopping analysis and drainage modelling of the existing defence structure indicate that widespread flooding of the area landward of the setback wall would occur during a 1 in 200-year event. Multiple properties would be affected by the overtopped water which reaches a maximum water depth of 1.5m. This confirms that a significant redesign of the existing defences is required to reduce the overtopping rates and to improve the drainage from the road back towards the sea to alleviate the flooding problem.

Three rock armour and two concrete design options have been identified and assessed for Portgordon with the with the aim of reducing the rate and overall retained volume of overtopped water during the 200-year design event with climate change, in line with industry practice. In addition, improved drainage by means of additional baffles installed along the setback wall is proposed with the aim of reducing the flood extent and depth due to overtopped water. All five options reduce the overtopping rate significantly in comparison to the Maintained Existing Defence and Failed Existing Defence scenarios.

The overtopping assessment conducted by ABPmer indicates that Option 2, “high rock armour berm over existing seawall”, has the most favourable performance out of the options considered in terms of reducing overtopping during the design event. Acknowledging the limitations of the modelling work undertaken to date, the study concludes that Option 2, together with Options 4 and 5, reduce the overtopping rate at the setback wall to an acceptable level for pedestrians located behind the setback wall based on EurOtop guidance[2]. Options 1 and 3 are predicted to reduce the rate of overtopping significantly from the existing scenario, but not to a level that is considered safe for pedestrians.

The overtopping results for each option were applied to a hydraulic model to predict the maximum flood extent and depth during the 200-year event, while accounting for drainage via the existing and proposed baffles and the control point at the western end of the setback wall. The results indicate that Option 4 is the most effective at reducing flooding, performing slightly better than Option 5. Both Options 4 and 5 are overtopped during the 200-year event but the extent is limited to a small section of the road towards the western end of the defence. In comparison, Option 2 is overtopped during the 200-year event with the flood extent affecting the majority of Stewart Street and Lennox Place, albeit no properties are affected.

An economic assessment was carried out (see Section 9) for all assessed options, and found that, for a 60% optimism bias, the Do Minimum yielded the best benefit cost ratio (BCR) and net present value (NPV) figures. However, this option does not apply any flood protection measures, and merely maintains the defences in their current form, which have been subjected to overtopping on occasion during recent years, leading to flooding of adjacent properties. The benefits gained for a Do Minimum Option over the Do Nothing baseline are limited to only two residential properties during a single flood event. As such, it is unlikely that this option can be considered preferable to the installation of a new defence system that will provide greater protection to the affected properties in Portgordon.

Consequently, Option 2 is considered to be the preferred option of those identified and assessed herein. With respect to capital cost, Option 2 is estimated to be the least expensive of the new defence options considered herein at an estimated cost of £10.6 million. In the 60% optimism bias model, Option 2 achieves a BCR of 0.99 and a negative NPV. However, a reduction in capital expenditure of 1.8% would result in a BCR of greater than unity and a positive NPV. The required 1.8% reduction in capital costs falls within the 60% optimism bias margin of error. The 30% and 0% optimism bias models show Option 2 achieves the highest IBCR. The avoided damages of Option 2 are of a similar magnitude to Options 4 and 5, and greater that Do Minimum, Option 1 and Option 3.

Option 2 is an imposing structure in its current arrangement and will have an impact on the landscape of Portgordon, which may prove unpopular to local residents. However, Options 1 and 3, which have a lesser visual impact, do not appear to perform nearly as well in terms of damage prevention.

Should Option 2 be progressed, it is recommended that the parameters should be refined by undertaking additional detailed numerical and/or physical modelling to refine the design. Furthermore, the optimisation of the design of the defence could potentially lead to a lower crest level, reducing the visual impact of the structure. However, this would require further investigation.

It is noted that the capital costs and visual impact of Option 2 are relatively significant. Should the preferred option be considered inappropriate and not developed further, it is suggested that consideration could be given to identifying additional options that protect the residents from a lesser return period than that adopted in this report (1:200 with 100-year climate change). The residents are currently subject to a risk of flooding during relatively low return period storm events and it may prove beneficial to explore lower levels of protection should the preferred option from this report not be taken forward. The figures in Table 9-2 indicate that there are 51 properties at risk during a 1:10-year event for the baseline Do Nothing Option. These alternate options would likely be of a smaller scale than those identified in this report, with reduced capital costs. However, their appropriateness for developing a business case around, and associated BCR values, would require further investigation.

JACOBS

Appendix A. ABPmer Portgordon Wave and Overtopping Modelling Final Report

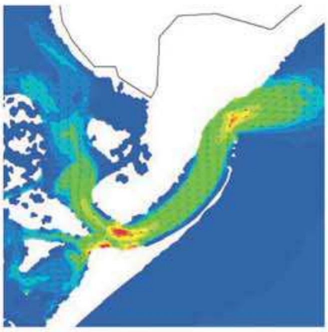
Jacobs

Portgordon Wave and Overtopping Modelling

Final report

June 2017




Innovative Thinking - Sustainable Solutions

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Portgordon Wave and Overtopping Modelling

Final report

June 2017

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Contents

Appendices

Tables

Figures

1 Introduction

1.1 Study

Jacobs is undertaking a study to review options for the sea defences at Portgordon (Figure 1), on behalf of Moray Council. The revetment at Portgordon often suffers from wave overtopping which results in the flooding of the road and properties behind the sea defence. To assist Jacobs, ABPmer has undertaken wave modelling and overtopping analysis to support the development and appraisal of the options. This report details the modelling that was undertaken for the assessment of the baseline condition, under 'Maintained existing defence' and 'Failed existing defence' scenarios and the overtopping appraisal of three different defence options.

Source: Map data ©2016 Google. Image © 2016 Terrametrics. Data © SIO, NOAA, U.S. Navy, NGA, GEBCO

Figure 1. Aerial imagery of Portgordon

1.2 Existing defences

Portgordon is defended from coastal flooding by linear defences, which extend around 700 m in length from the western harbour arm. These defences are further backed by the main coastal road though Portgordon. The existing defences differ only very slightly along the frontage, with small differences related to crest height and defence alignment. The study area has been characterised into three defence lengths (Figure 2 and Table 1). The model build, validation and initial overtopping analysis has been completed for all three defence lengths. Due to the similarity of defences along the study area, only the worst case section has been taken forward as the basis for the “Maintained existing defence” and “Failed existing defence” assessment.

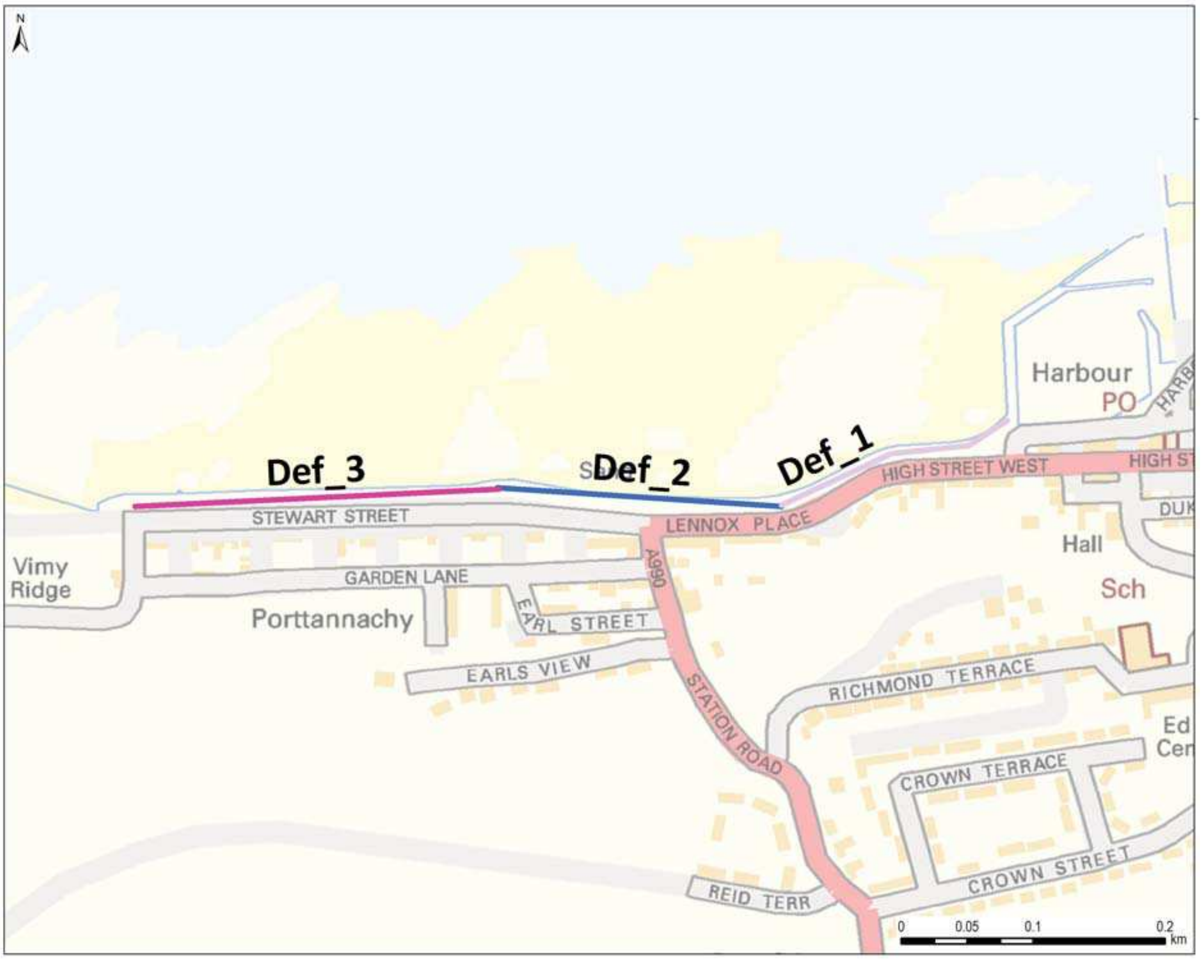


Figure 2. Indicative location of the three defence sections

Table 1. Defence lengths

Defence Section	Length (m)
Defence 1	194.02
Defence 2	215.58
Defence 3	303.61

2 Wave Modelling

2.1 Introduction

To calculate the overtopping along the Portgordon frontage, a numerical wave model was constructed to transform offshore wave conditions inshore, to the toe of the defences. This wave modelling was undertaken using the MIKE21 SW model developed by DHI (Danish Hydraulic Institute). In this study 37 years of hourly offshore wave heights, from ABPmer's SEASTATES hindcast (www.seastates.net) were transformed to the nearshore. The calibration report for ABPmer's SEASTATES metocean model can be found here (http://www.seastates.net/downloads/).

2.1.1 Model domain

To generate the nearshore wave time series a model domain, referred to as the local model, was developed. Figure 3 and Figure 4 show the local model domain including some of the local bathymetric features.

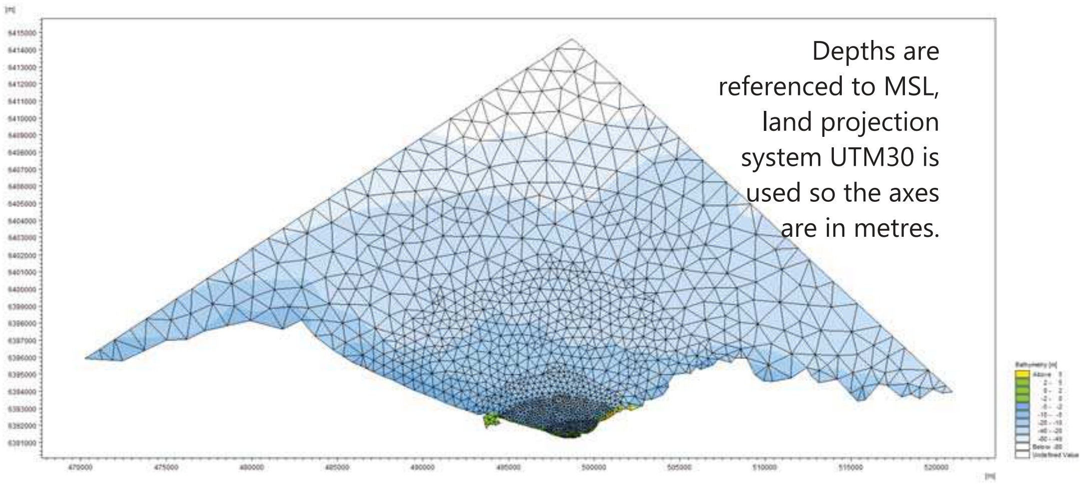


Figure 3. Local model domain

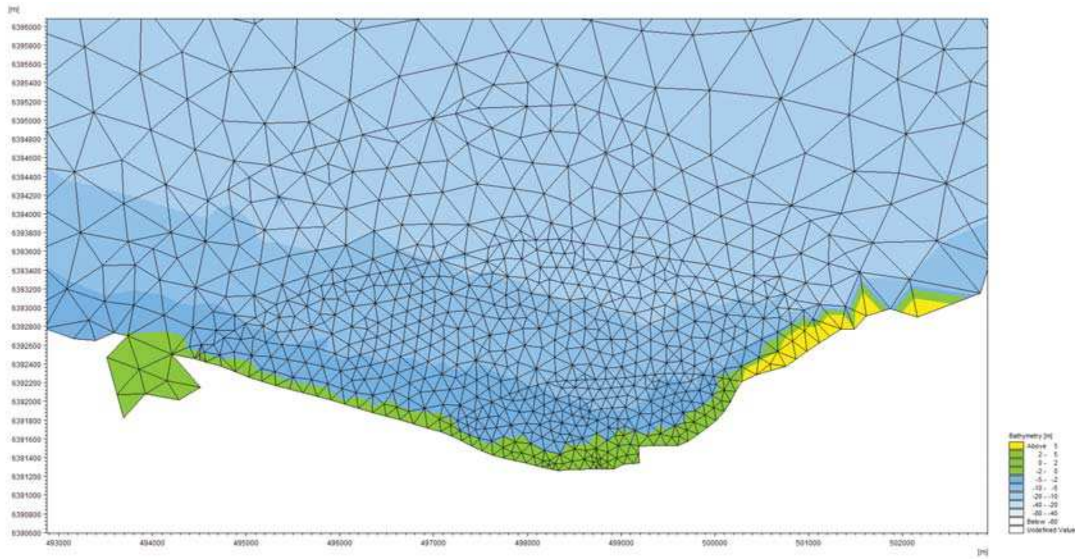


Figure 4. Close-up of Portgordon in the local model domain

The local model has boundaries approximately 17-24 km offshore of Portgordon with a resolution of around 500 m at the offshore boundary, increasing to 25 – 35 m at the Portgordon frontage. The resolution of the model is considered appropriate for use at initial option appraisal stage. For more detailed design studies, it may however be necessary to refine the model mesh further and undertake additional model calibration.

2.2 Source data

The data sources utilised in the modelling study are summarised below with further details provided in the subsequent sections:

- Nearshore bed/ground levels from Moray Council (Light Detection And Ranging (LiDAR) flights;
- Bathymetry survey data from United Kingdom Hydrographic Office (UKHO) surveys and charts;
- Offshore spectral wave conditions from ABPmer's SEASTATES hindcast;
- Water levels at Portgordon from ABPmer UKCS (UK Continental Shelf) surge and tide model (ABPmer, 2017);
- Spatial varying Climate Forecast System Reanalysis (CFSR) wind field data.

2.2.1 LiDAR

LiDAR (Light Detection And Ranging) flights were flown over Portgordon between 2007 and 2014. The 2 m resolution surveys were flown in 2007 and the 1 m resolution survey was flown between November 2012 and April 2014. These data cover the majority of the nearshore frontage of Portgordon. The LiDAR, provided by the Moray Council, is projected in OSGB36 and uses ordnance datum Newlyn (ODN) as the vertical reference datum with vertical values in metres. The vertical datum was converted to mean sea level (MSL) and the projection was changed to UTM30 for the wave model.

2.2.2 Bathymetric data

The approaches to Portgordon have been surveyed by the UKHO. The coverage of those surveys is shown in Figure 5, which also indicates the approximate years of survey. All of the UKHO data was converted to UTM30 projection, and converted from chart datum (CD) to MSL, using Vertical Offshore Referencing Frame (VORF).

GEBCO data is used in the outer areas of the model domain where the other data sets, described above, do not provide cover. The GEBCO dataset is provided in WGS84 projection with a vertical reference to CD (the latter is assumed as not details are provided for the data set). The data was re-projected in UTM30 and the vertical reference to CD was checked by assessing the tie in with adjacent data sets. Once checked the data was then converted to MSL using conversion values from the Admiralty Tide Tables.

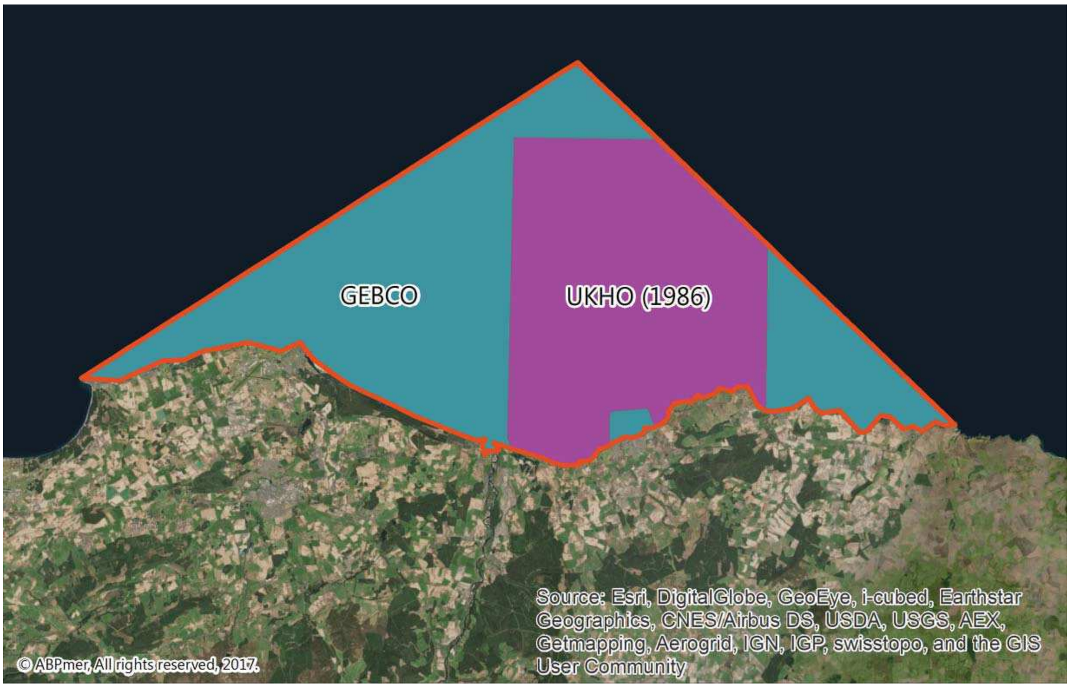


Figure 5. Extent of bathymetry data within model domain

2.2.3 Offshore boundary conditions: SEASTATES

The local model's offshore boundaries are identified in Figure 5. The offshore boundaries were driven by hourly wave spectral parameters (Hs, Tp, MDir and Direction Standard Deviation) spectra spanning the time period 1979 to 2015 inclusive. This spectral data was obtained from ABPmer's SEASTATES hindcast database and varies spatially along the model boundaries. SEASTATES is a fully spectral wave model which covers the North Atlantic and European shelf seas, and is driven by the spatially and temporally varying Climate Forecast System Reanalysis (CFSR) wind fields developed by National [American] Oceanic and Atmospheric Administration (NOAA). The SEASTATES hindcast has been previously validated against 28 buoys within UK and European waters (ABPmer, 2013); the closest validation point is located in the Moray Firth.

2.2.4 Water levels

The water level in the local model is spatially constant but varies temporally. In the absence of locally available data, total water levels (tide + surge) data was extracted from the ABPmer (UKCS) tide and surge model. This data covers the period 1979 to 2015 inclusive. The ABPmer UKCS tide and surge model is calibrated and validated successfully in UK and European waters (ABPmer, 2017). The resulting time series of water levels is the most complete record readily available to the study.

Adjustments for climate change

To support the wave and overtopping assessment, which examines overtopping for two epochs (2022 and 2122 with climate change) the local model was used to transform the waves to the nearshore for both epochs. For 2022, an increase in level from present day (2015) of 0.03 m was applied. For 2122 Epoch, the water level will be increased by a further 0.65 m to account for Sea Level Rise using UKCP09 medium emissions 95th percentile predictions from 2022.

2.2.5 CFSR wind field data

The local model is driven by wind fields sourced from the National Centers for Environmental Prediction (NCEP) Climate Forecast System Reanalysis (CFSR) (http://rda.ucar.edu/datasets/ds093.1/) and Climate Forecast System v2 (CFSv2) (http://rda.ucar.edu/datasets/ds094.1/) hindcast databases. The data archive is managed by the National Center for Atmospheric Research (NCAR).

Wind is referenced to 10 m above surface and is assumed to represent the hourly mean value. The data are available at hourly time steps.

The spatial resolution of the source data varies with parameter and year, with winds prior to 2010 being available at 0.3 degrees spatial resolution, and data post 2009 at 0.3 degrees. All of the data were linearly interpolated onto a standardised 0.2 degree grid prior to use.

Due to the size of the local model domain, the wind data used was temporally, but not spatially, varying. The time series was taken from a location 7.5 km from the Portgordon frontage.

2.3 Model parameters

The local model has a boundary approximately 17-25 km offshore off Portgordon and is set up as follows:

- Run in directionally decoupled parametric mode with quasi stationary time formulation;
- Logarithmic frequency discretisation with 25 frequency bins, minimum frequency of 0.055 Hz and a frequency factor of 1.1;
- Twenty two directional frequency bins with a minimum direction of 250 degrees and a maximum direction of 110 degrees, and no wind-sea and swell separation;
- Wind forcing from the CFSR time series;
- Depth-induced wave breaking gamma constant of 1.3; and
- Bottom friction Nikuradse roughness of 0.02 m.

The wave breaking and bottom friction values have been chosen to be more representative of nearshore wave conditions, as the default model values are optimised for offshore environments where are considered to lead to an underestimation of the wave conditions at nearshore sites. The use of the amended values should provide more conservative estimates of the nearshore wave conditions, however, without appropriate data to calibrate the model, the accuracy of the model cannot be confirmed.

2.4 Model results

The local model provides output of spatially varying significant wave height, mean period, peak period, mean wave direction and wave velocity components. These outputs provide the input to the overtopping calculations.

Example model outputs for significant storm events are shown in Figure 6 to Figure 8, for storm events from the NW, N and NE respectively. In each case, the results are shown for the conditions at high water.

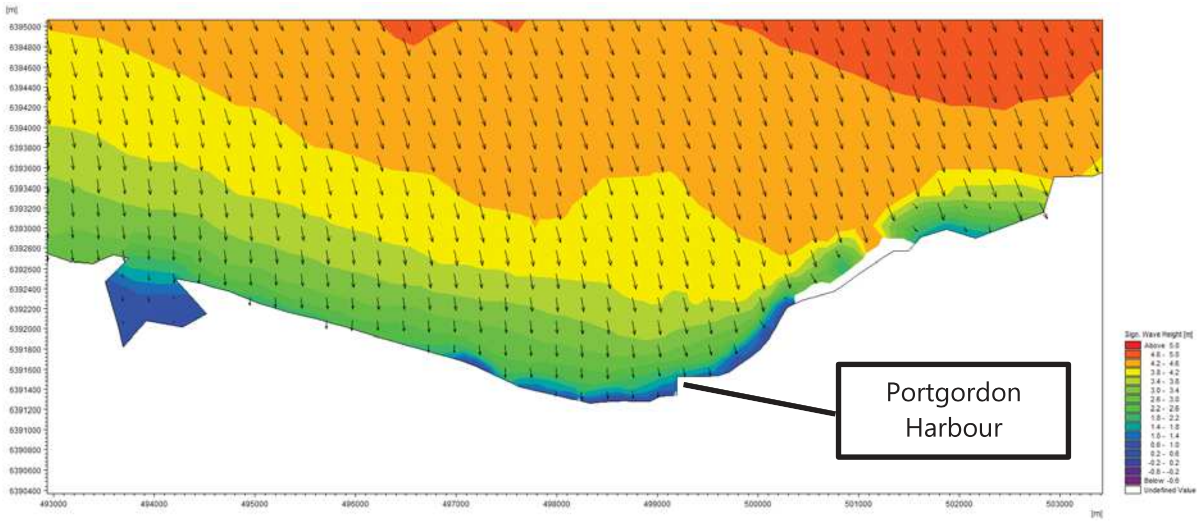


Figure 6. NW storm event (direction = 313 degrees)

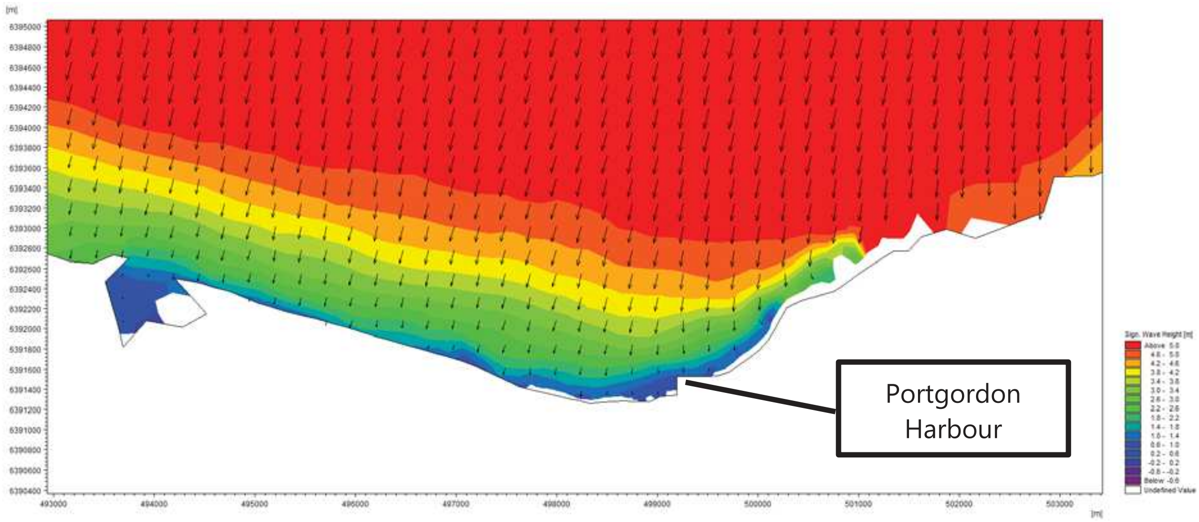


Figure 7. N storm event (direction = 3 degrees)

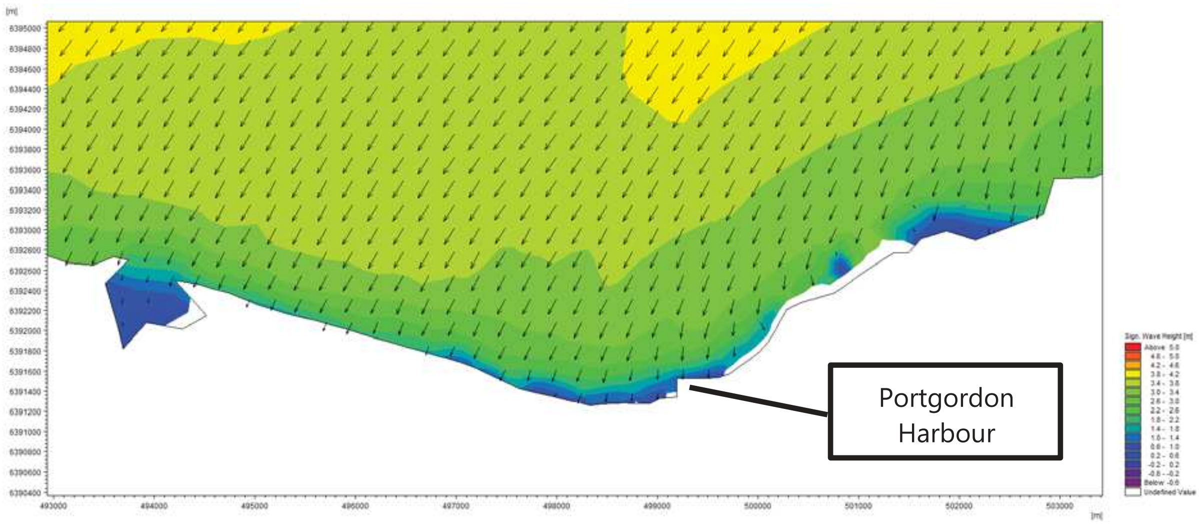
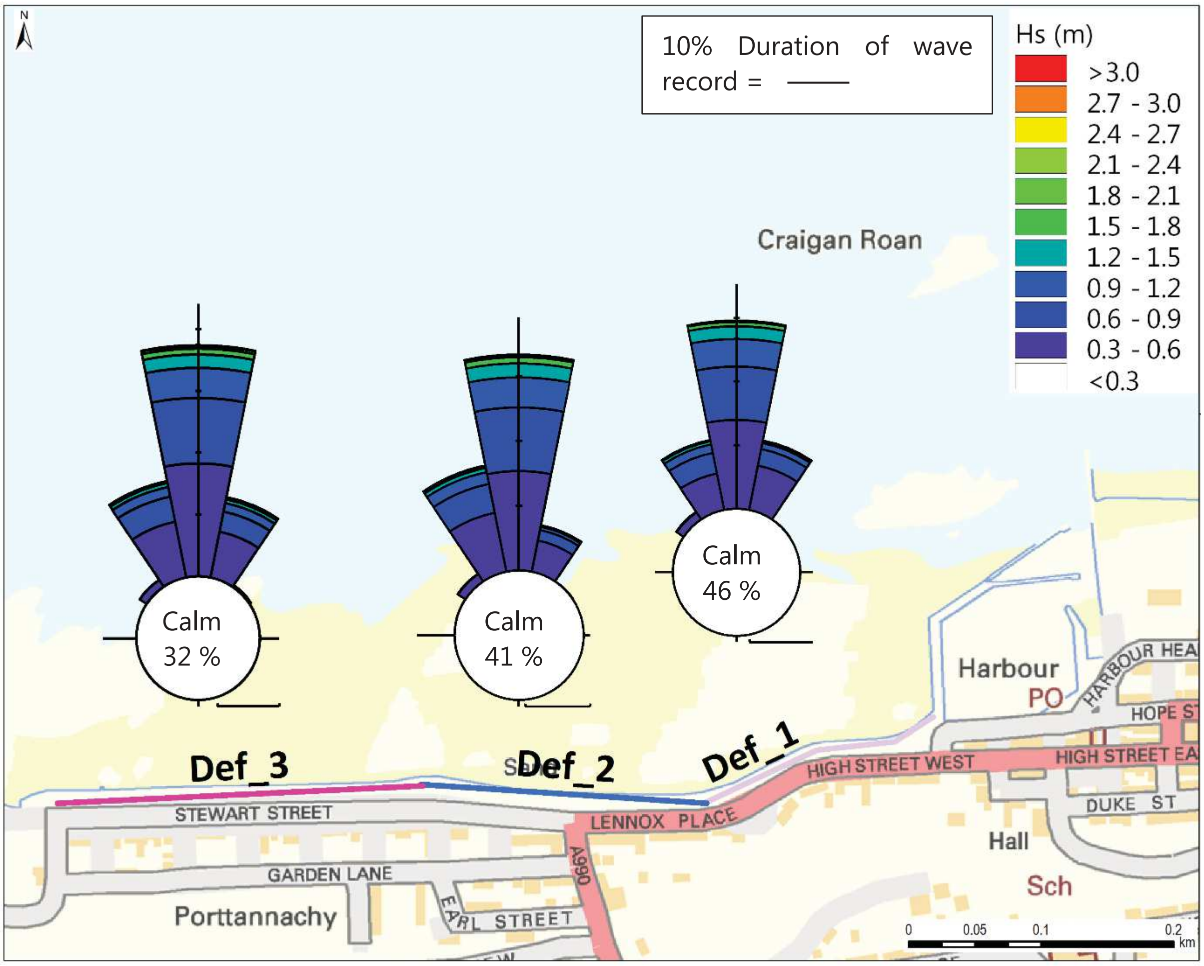


Figure 8. NE Storm event (direction = 43 degrees)

Time series of wave and water level conditions covering the full 37 years hindcast, have been extracted at three locations along the toe of the defence sections to be examined. The wave climates for the 2022 epoch simulations are shown as wave roses Figure 9. The largest predicted waves along the frontage were found to occur at the toe of Defence Section 3, with significant wave heights of 2.99 m and 3.31 m predicted for the 2022 and 2122 epochs respectively (see Table 2). The water levels at the defences predicted at the frontage are 3.28 mODN and 3.94 mODN for the 2022 and 2122 epochs respectively.



Contains Ordnance Survey data © Crown copyright and database right 2017

Figure 9. Wave rose plots showing wave climate at the toes of the defences (2022 epoch)

Table 2. Maximum predicted significant wave height (Hs) at the toe of the structures for epoch 2022 and 2122

Defence Section	2022	2122
1	2.79	3.10
2	2.87	3.21
3	2.99	3.31

3 Overtopping

3.1 Methodology

Overtopping has been calculated using the Neural Network Tool (NNT) developed under EurOtop (2007). 37 years of nearshore waves were extracted at the toe of the structure from the numerical modelling exercise described in Section 2. These waves were paired with the coincident water levels that were derived for the site (Section 2.2.4). These levels were also adjusted for sea level rise according to the epoch considered.

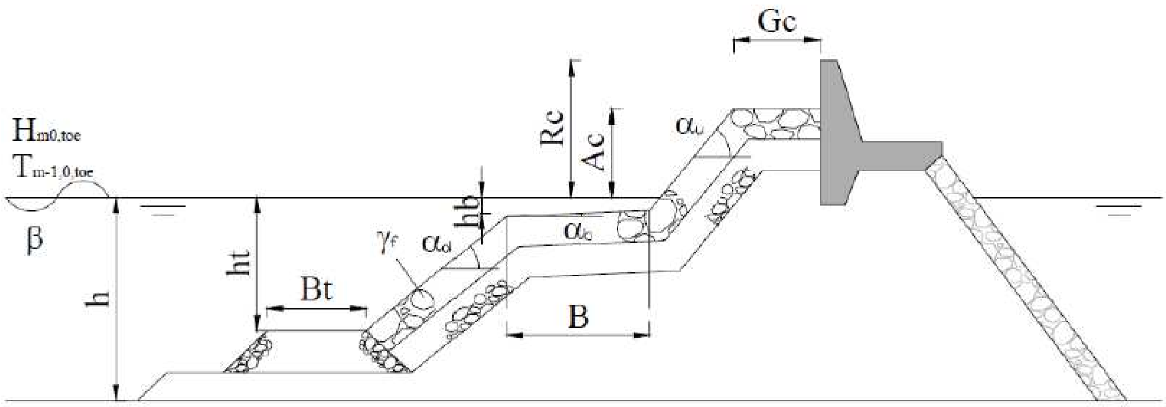
Coincident waves and water levels were therefore derived for the two epochs, 2022 and 2122. These long-term, 37 year, data sets of coincident wave and water levels were then run through the NNT to obtain a continuous time-series record of overtopping rate for two scenarios “Maintained existing defence” and “Failed existing defence”.

The overtopping rates acquired were then then used to extrapolate extreme overtopping discharges for a range of return periods. The significant advantage of the proposed method is that overtopping was calculated from over 300,000 historical combinations of waves and water levels, rather than relying on extreme water levels defined offshore (as found in the Environment Agency extreme coastal flood boundary data (2011)). The unique event combinations that resulted in the largest overtopping rates are then used to construct a design overtopping hydrographs as per Environment Agency (2011) methodology.

EurOtop (2007) describes the NNT as the most suitable tool for calculating overtopping in this instance and it was thus used in the study. The NNT has some limitations and requires engineering judgement when applying the tool and interpreting the overtopping results. Ideally, suitable data on past overtopping events should also be available to help validate the schematisation of the sections within the tool.

3.2 Schematisation of defences

Within the NNT defences are schematised using 15 geometric parameters which include; crest height (Rc), armour height (Ac), armour width (Gc), berm elevation (hb), berm width (B), upper slope (α_u), lower slope (α_d) and roughness (γ_f) (See Figure 10).



Source: Coeveld *et al*, 2005: CLASH Database

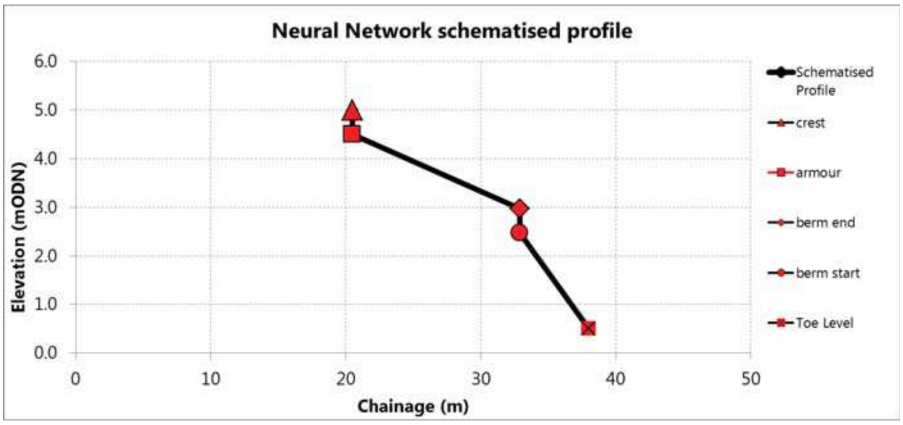
Figure 10. Schematisation descriptors for a defence profile using Neural Networks overtopping tool

For the present study, representative profiles (Figure 11) for each of the three identified defence sections were schematised (Figure 12) based on the topographic survey procured for Jacobs in 2013, by Property & Land Surveys (HLDS) LTD, and the LiDAR data collect between November 2012 and April 2014 by the Moray Council. Where differences occurred between the LiDAR and the 2013 topographic survey, the survey levels are taken as primary. The levels of the schematised defences are presented in Table 3.

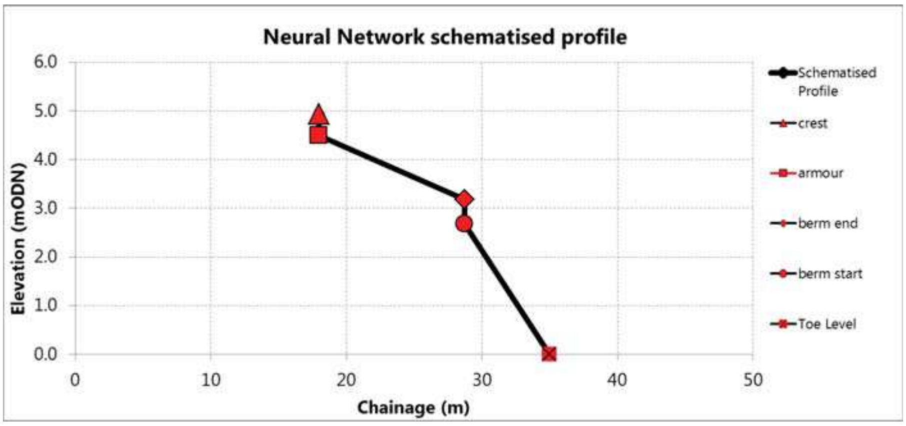
Figure 11. Defences of Portgordon

Table 3. Schematised defence levels

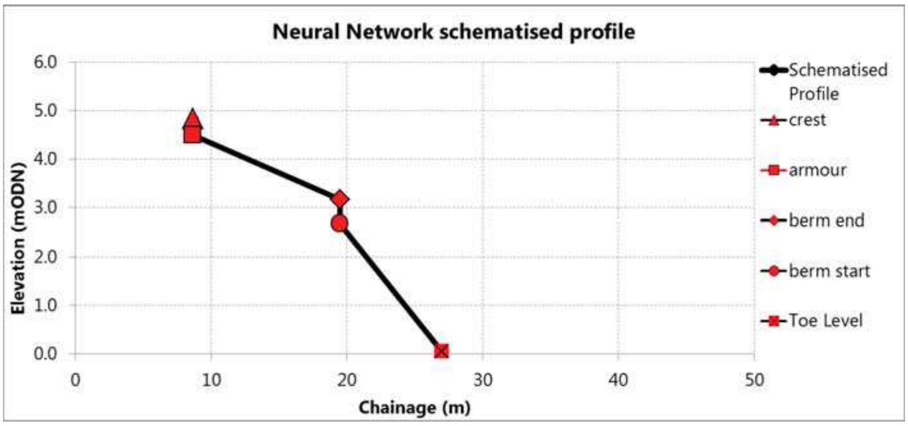
Defence	Defence Descriptor	Level (mODN)
Defence 1	Toe	0.50
	Berm start	2.47
	Berm finish	2.97
	Crest level	5.00
Defence 2	Toe	0.00
	Berm start	2.68
	Berm finish	3.18
	Crest level	4.94
Defence 3	Toe	0.05
	Berm start	2.67
	Berm finish	3.17
	Crest level	4.84



Defence Section 1



Defence Section 2



Defence Section 3

Figure 12. NNT schematised defences of Portgordon

3.3 NNT validation

3.3.1 Validation approach

To validate the schematisation of the defence sections within the NNT tool, it is important to have known or indicative overtopping rates from past events with which to compare the results of the overtopping assessment.

In reality, actual quantified overtopping rates are not usually available, however, past anecdotal and photographic records often exist when significant events have occurred, and this information can be used in conjunction with EurOtop (2007) guidance to estimate overtopping rates. EurOtop (2007) provides useful guidance that relates overtopping rate to hazardous situations and volumes. This guidance provides overtopping thresholds in relation to pedestrian (Table 4) and vehicles (Table 5) and to the damage of defence structures (Table 6).

Table 4. Limits for overtopping for pedestrians

Hazard Type and Reason	Mean Discharge Q (l/s/m)	Max. Volume Vmax (l/m)
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 sea, not easily upset or frightened, able to tolerate getting wet, wide walkway.	0.1	20 – 50 at high level or velocity

Source: EurOtop, 2007

Table 5. Limits for overtopping for vehicles (EurOtop, 2007)

Hazard Type and Reason	Mean Discharge Q (l/s/m)	Max. Volume Vmax (l/m)
Driving at low speed, overtopping by pulsating flows at low depths, no falling jets, vehicle not immersed.	10 – 50*	100 – 1,000
Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets.	0.01 – 0.05**	5 – 50 at high level or velocity
* Overtopping limit is related to the effective overtopping at the highway location. ** Overtopping limit is related to the effective overtopping at the defence location with the highway/motorway immediately behind the defence.		

Source: EurOtop, 2007

Table 6. Limits for overtopping for property and damage to the defences

Hazard Type and Reason	Mean Discharge Q (l/s/m)
Damage to building structural elements.	1*
Damage to equipment set back 5 – 10 m.	0.4**
No damage to embankment/seawall if crest and rear slope are well protected.	50 – 200
No damage to embankment/seawall crest and rear face of grass covered embankment of clay.	1 – 10
Damage to paved or armoured promenade behind a seawall.	200
Damage to grassed or lightly protected promenade.	50
* Overtopping limit is related to the effective overtopping at the location of the building. ** Overtopping limit relates to the overtopping rates at the defence location.	

Source: EurOtop, 2007

Using these thresholds it is expected that overtopping events of the order of 10's-100's of l/s/m, will have received some noticeable attention as damage to structures may have occurred at these rates and this is likely to have been reported.

Reports, including photographic and video evidence, on passed overtopping events has been provided by Moray Council for three separate events. These events occurred on the 05/12/2013, 09/10/2014 and the 13/01/2017. The 13/01/2017 was excluded from the assessment as it falls outside the available hindcast period. Photographs of the overtopping at the time of the 05/12/2013 and 09/10/2014 events are shown in Figure 13 and Figure 14 respectively.

The 2013 event is understood to have been more severe and from the overtopping reported / observed; the peak overtopping rates during the event are likely to be in the order of 10-200 l/s/m. It is understood that there was a restriction on vehicle access at the time but no damage to the sea wall was noted.

The event in 2014 is understood to have been less severe, and the available information suggests the peak overtopping rates during the event are likely to be in the order of <10 l/s/m.

To validate the model these two events have then been run through the NNT and the outputs compared against the estimated overtopping rates identified above. This validation, exercise was undertaken for three profiles, representative of the three defence sections identified in Figure 2. The results of this assessment are provided in Section 3.3.2.

Photographs courtesy of Moray Council

Figure 13. Photographic evidence of wave overtopping along the Portgordon frontage on the 05/12/2013

Photographs courtesy of Moray Council

Figure 14. Photographic evidence of wave overtopping along the Portgordon frontage on the 09/10/2014

3.3.2 Validation results

The results of the validation exercise are presented in Table 7, with mean overtopping rates being calculated during the peak of the two events for the three representative profiles identified.

Table 7. Modelled overtopping rates predicted during two validation events

Defence Section	Mean Overtopping Rate for 05/12/2013 Event	Mean Overtopping Rate for 09/10/2014 Event
Defence 1	36.73 l/s/m	0.46 l/s/m
Defence 2	44.48 l/s/m	0.57 l/s/m
Defence 3	55.27 l/s/m	0.72 l/s/m

The results shown for the two events are consistent with those estimated in Section 3.3.1 from the available reports and photographs at the time. For the 2013 event the rates are between 10-200 l/s/m and for the 2014 event the rates are less than 10 l/s/m. This provides confidence that the wave modelling and NNT are providing realistic overtopping rates for the present condition is therefore fit for purpose.

In addition, from Table 7, it can be seen that the predicted overtopping rates increase as you head west along the frontage, with the profile at Defence 3, resulting in the greatest overtopping. This difference is accredited to very small changes in the defence form/crest levels, the orientation of the shoreline and wave exposure. From Table 2 it is noted that the most extreme waves are also predicted to occur towards the western end of the frontage. The difference in predicted overtopping rate along the frontage also appears to agree with observations made from photographs during the 2013 event. From Figure 15, it appears that driving restriction may have been in place along the western end of the frontage, but not the eastern end, although, the evidence to confirm if this was the case is lacking.

Junction of Defence 2 and Defence 3 driving restriction applied

Defence 2 before driving restrictions

Defence 1 before driving restrictions

Photographs courtesy of Moray Council

Figure 15. Photographic evidence of overtopping along the frontage for the 05/12/2013

3.3.3 Conclusions from validation exercise

On the basis of this validation exercise, the wave modelling and NNT is considered fit for purpose, and the profile from Defence Section 3 has been selected for use in the “Maintained existing defence” and “Failed existing defence” assessment, as this defence section is considered most vulnerable to overtopping.

Since the remainder of the assessment examines overtopping for Defence Section 3, Table 8, has been developed to show the relative overtopping rates predicted at Defence Section 1 and 2, in relation to Defence Section 3, during the two validation events. These, reduction factors provide a rough indication as to how overtopping rates during other events, including more extreme future events, might vary along the frontage. The use of these reduction factors has limited accuracy. If the overtopping performance along other section needs to be examined in more detail it is recommended that formal overtopping calculation are undertaken along these sections,

Table 8. Overtopping rates for Defence Section 1 and 2 as a factor those for Defence Section 3 (based on results from validation events)

Defence Section	Reduction Factor (%)
Defence 1	35
Defence 2	20
Defence 3	0

It should be noted that immediately adjacent to the harbour arm, there is potential for increased overtopping as waves reflecting from the harbour arm may combine with incident waves to increase the level of overtopping in this area. This effect will only be of importance for a small section of frontage immediately adjacent to the harbour arm and may need further consideration during later design stages.

3.4 “Maintained existing defence” and “Failed existing defence” assessment

Having validated the NNT, the overtopping assessment has been undertaken for two epochs (2022 and 2122), for two baseline defence scenarios, these are “Maintained existing defence” and “Failed existing defence” based on Defence Section 3. These are described further below.

3.4.1 “Maintained existing defence” scenario

In the “Maintained existing defence” scenario, the defences have been schematised as per the present day situation, with the levels taken from a representative profile along Defence Section 3. Under this scenario, it is assumed that the defence would be maintained as they presently are.

The representative profile has been selected at a location were the parapet crest wall is close to its lowest level, having a level of 4.84 mODN (Table 9). From the base of the 0.50 m high raised parapet wall a cemented single slope then extends down to a lower concrete wall. In front of the lower concrete wall there is then a vertical drop down to the top of a rock-armoured revetment (Figure 11 and Figure 12).

Table 9. “Maintained existing defence” schematisation values

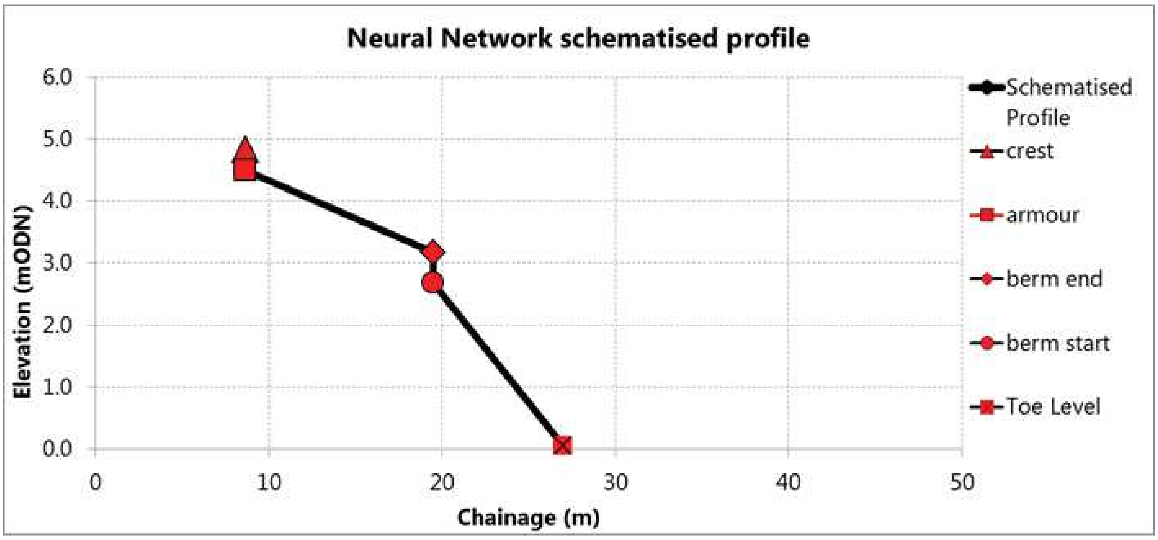
Defence Descriptor	Value
Toe (mODN)	0.05
Berm start (mODN)	2.67
Berm finish (mODN)	3.17
Armour crest level (mODN)	4.50
Crest level (mODN)	4.84
Toe to berm horizontal distance (m)	7.52
Berm to crest horizontal distance (m)	10.83

3.4.2 "Failed existing defence" scenario

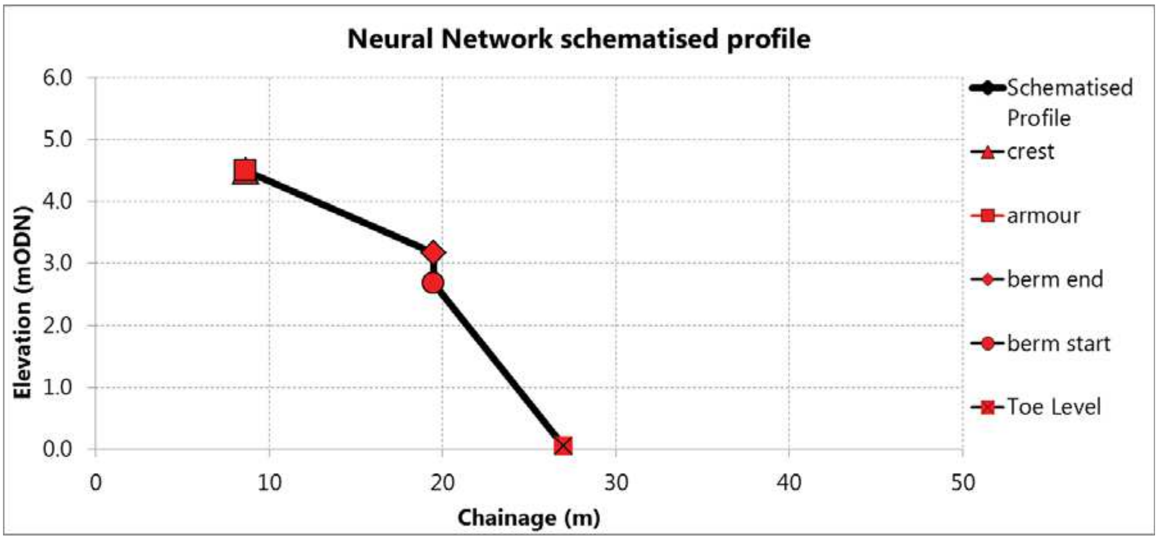
In the "Failed existing defence" scenario, the defences have been schematised according to the following assumptions:

- Assumes that no repair / refurbishment work is undertaken on the structure;
- The gaps and voids between the rock armour will be filled with sand, shingle, pebbles and seaweed, thus the rock armour would continue to lose effectiveness in absorbing energy due to greater volumes of material filling in the voids and gaps. The roughness which represents the rock armour (0.55) will be smoothed in NNT to represent "shingle" (0.8);
- The lower concrete wall will deteriorate due to abrasion but no changes are proposed;
- The cemented shingle slope would be likely to continue to experience abrasion, resulting in significant damage to the slope. However, it has been assumed that the roughness of the cemented slope would remain unaltered at (0.8);
- It is expected that the raised parapet at the landward edge of the defences will be damaged and its stability compromised, leading to partial collapse. Therefore, the raised parapet is partially removed in the "Failed existing defence" defence schematisation. The maximum crest level is therefore reduced from 4.84 mODN to 4.50 mODN.

Figure 16 shows the schematisation of the defence section (as per requirements of the NNT), for both the "Maintained existing defence" and "Failed existing defence" scenarios.



"Maintained existing defence"



"Failed existing defence"

Figure 16. "Maintained existing defence" and "Failed existing defence" defence profile and Neural Network schematised profile

3.5 Overtopping predictions

Having schematised and validated the NNT, overtopping rates were then derived using the NNT for Defence Section 3 for both the "Maintained existing defence" and "Failed existing defence" scenarios, for both the 2022 and 2122 epochs.

In order to do this the incident wave conditions at the toe of the structure were taken from the nearshore wave model results, with the data extracted at the most western extraction point (Section 2.4). The 37 year time series of coincident wave and water levels were then passed through the NNT to develop a 37 year time series of mean overtopping rates. This was repeated for each defence scenario and epoch. The results from these predictions were then subsequently analysed as part of the extremes overtopping assessment (Section 3.6) to derived overtopping rates for the required return periods.

3.6 Extremes overtopping analysis

To derive overtopping rates for different return periods, the 37 year record of overtopping for each scenario were run through a Generalised Pareto Distribution extremes package. The overtopping events from the hindcast time series above a threshold are selected and are plotted against return period. The Pareto distribution of the overtopping and return periods were then fitted using the software package in2extremes (see Gilleland & Katz, 2016) to the overtopping events. In this process the shape and scale parameters of the fitted data are determined. The Pareto fit to the data is visually assessed, and if necessary the threshold is reselected and the extrapolation refitted to the data to improve the fit quality. This is a subjective process guided by the behaviour of the scale and shape parameters at various thresholds, and by the experience of the practitioner. Further details on threshold selection can be found in Coles (2001). The final shape and scale parameters are used to extrapolate the theoretical fit to the data in order to determine extreme conditions for various return periods.

This process has been followed for the two scenarios and the resultant fits are provided in Appendix A and B for the "Maintained existing defence" and "Failed existing defence" scenarios respectively.

For the "Maintained existing defence" scenario, the results for the base year of 2022 and the climate change scenario of 2122 are presented in Table 10 and Table 11 respectively. The equivalent results for the "Failed existing defence" scenarios are presented in Table 12 and Table 13.

Neural Networks provides the mean wave overtopping rates. Table 10 to Table 13 provide the mean predicted overtopping rates that would occur at the peak of the event.

The resultant overtopping rates for each defence section can also be related to the guidance given in the EurOtop (2007) manual (see Section 3.3).

Overtopping rates over 10 l/s/m are highlighted in red to indicate the overtopping rates are above the EurOtop (2007) guidance limit for the 'Hazard type and reason' - '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'.

Table 10. Calculated "Maintained existing defence" mean overtopping rates for Defence 3 per return period for base year 2022

Return Period	Mean Overtopping Rate l/s/m
	Defence 3
10	58.65
50	101.54
100	125.62
200	153.94
1,000	240.51

Table 11. Calculated "Maintained existing defence" mean overtopping rates for Defence 3 per return period climate change scenario 2122

Return Period	Mean Overtopping Rate l/s/m
	Defence 3
10 cc	133.74
50 cc	213.52
100 cc	256.38
200 cc	305.46
1,000 cc	448.61

Table 12. Calculated "Failed existing defence" mean overtopping rates for Defence 3 per return period for base year 2022

Return Period	Mean Overtopping Rate l/s/m
	Defence 3
10	75.21
50	125.79
100	153.58
200	185.84
1,000	282.19

Table 13. Calculated "Failed existing defence" mean overtopping rates for Defence 3 per return period climate change scenario 2122

Return Period	Mean Overtopping Rate l/s/m
	Defence 3
10 cc	155.17
50 cc	248.14
100 cc	299.58
200 cc	359.55
1,000 cc	539.99

3.7 Extreme overtopping hydrographs

3.7.1 Design tide hydrographs

To estimate overtopping volumes during a storm event, idealised design tide hydrographs have been generated for each return period event, for each defence section under all scenarios. To generate these design tide hydrographs the Environment Agency preferred method (Environment Agency,

2011) was adopted. This method was developed with the support of the Scottish Environment Protection Agency

To achieve this, a design tide hydrograph was constructed using a distance weighted mean high water spring (MHWS) base astronomical tide extracted from Total Tide software for Buckie and Lossiemouth. This base astronomical tide was combined with a scaled Moray Firth design surge shape profile (Environment Agency, 2011) such that the water level at the peak of the overtopping event from the hindcast period is equivalent to the required water level for the overtopping event. The total event length considered is 25 hours covering two consecutive high tides.

3.7.2 Design overtopping hydrographs

To create design overtopping hydrographs the design tidal hydrograph (described in Section 3.7.1) was run through the NNT as described below. This provided an overtopping profile shape for each epoch and scenario.

In this assessment a uniform wave condition was used over the design tide hydrograph. This wave event equated to the worst overtopping condition obtained at the defence section in the hindcast 37 year overtopping record. This is a derivation of the method set out in Environment Agency (2011), but we believe is in line with the forthcoming Environment Agency 'State of the Nation' approach.

In this approach, the wave and water levels conditions associated with the largest overtopping event from the 37 hindcast period are identified for each epoch. The wave conditions for this event are then run through the NNT over the design tidal hydrograph (Section 3.7.1), providing a 25 hour overtopping hydrograph.

In this assessment, the overtopping hydrograph is then scaled to the calculated overtopping extreme return periods (see Section 3.6) providing a wave overtopping design hydrograph for each return period. This assessment was undertaken for each of the scenarios examined. The overtopping hydrographs for each return period were provided in digital format to be used in inundation model inputs.

The method therefore focuses upon defining the *result* of the extreme event (i.e. the actual overtopping of a defence) rather than defining the event itself. We believe that this approach more closely reflects latest industry advances in flood risk assessment.

3.8 Defence designs

To provide future protection of the Portgordon frontage three different design options were proposed. For each design option, various configurations were initially identified, with varying crest heights, berm heights and berm widths.

To refine the design for each of the three options; the wave and water level conditions associated with the ten worst overtopping (from baseline) were tested with numerous configurations of each defence option. This was undertaken for the 2122 epoch, (as discussed in Section 3.5). The configuration for each option that resulted in the lowest overtopping was then carried through to the full overtopping analysis.

The full overtopping analysis was then done for the 2022 and the 2122 epoch, using the same methodology described in Section 3.6. Each design option is discussed in the following sections together with the assessment of extreme overtopping.

3.8.1 Option 1: Rock armour berm over upper slope

Option 1 design is a wide flat berm structure. To improve overtopping performance rock armour would be placed over the entire structure. Rock armour will be placed at the toe of defence with a slope of 1:2 up to a berm. The berm will then extend horizontally back by 12.5 m to the base of a concrete wave return wall. Table 14 presents the overtopping from the ten 2122 events against the numerous configurations for design Option 1.

Of the design configurations initially provided the best performing, in respect to minimising overtopping had a berm height of 5.44 mODN and a wave return wall with a crest level of 5.84 mODN, as schematised in Figure 17. The extreme overtopping rates for this configuration are presented in Table 15 and Table 16 for the 2022 and 2122 epoch respectively.

Table 14. Option 1 design configurations and the mean overtopping rate for the 10 events

Crest level (mODN)	Berm level (mODN)	Mean Overtopping Rate (l/s/m)
4.84	4.44	99.55
5.34	4.44	73.28
5.34	4.94	63.5
5.84	4.44	49.62
5.84	4.94	45.17
5.84	5.44	38.91

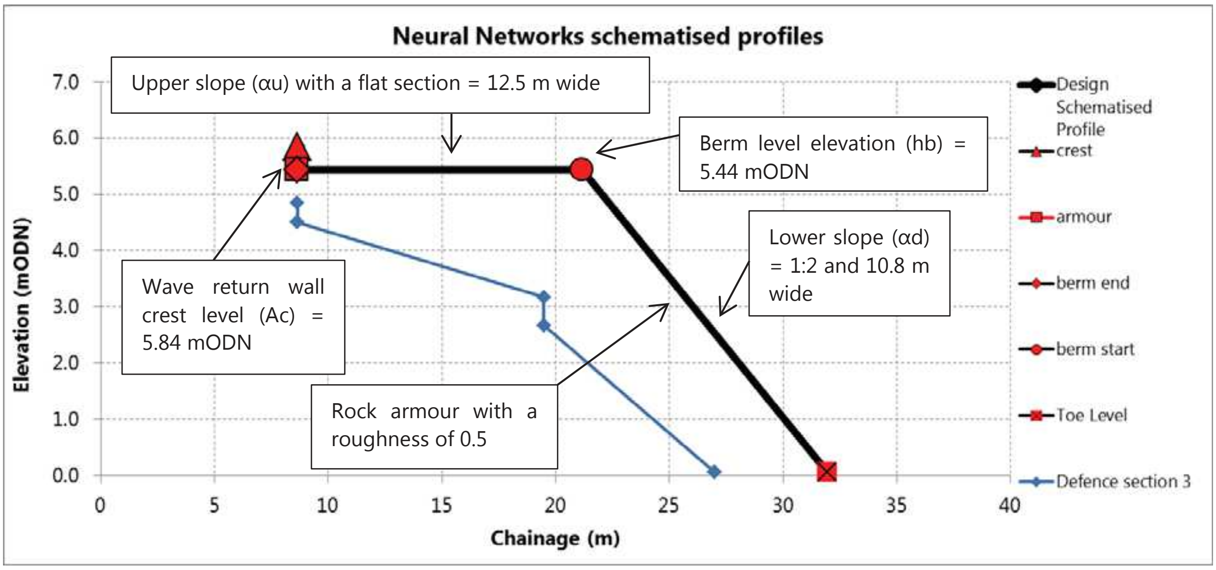


Figure 17. Neural Network schematised profiles of Option 1 design (Defence Section 3 overlaid)

Table 15. Calculated mean overtopping rates for Option 1 design per return period for base year 2022

Return Period	Mean Overtopping Rate (l/s/m) Option 1 design
10	4.96
50	10.77
100	14.30
200	18.65
1,000	33.13

Table 16. Calculated mean overtopping rates for Option 1 design per return period for base year 2122

Return Period	Mean Overtopping Rate (l/s/m) Option 1 design
10 cc	24.38
50 cc	45.85
100 cc	56.23
200 cc	67.35
1,000 cc	96.40

3.8.2 Option 2: High rock armour berm over existing lower seawall

Option 2 design places rock armour over the lower wall raising the crest level to cause wave breaking away from the promenade. The toe of the defence will be located to allow a slope of 1:2 up to a berm.

The overtopping to the promenade is calculated by assessing the overtopping at the crest, and applying a set-back equation to derive the overtopping at the location of the raised parapet wall. This set-back calculation does not take into account the effects of the raised parapet wall on overtopping. The set-back from the structure to the raised parapet is 10.5 m. Table 17 presents the overtopping at the promenade from the ten 2122 events against the numerous configurations for design Option 2.

Of the design configurations initially provided the best performing, in respect to minimising overtopping had a crest level is 6.34 mODN with a crest width of 4 m (Figure 18). Table 18 and Table 19 display the predicted extreme overtopping rates, for this configuration, for 2022 and 2122 epochs respectively.

Table 17. Option 2 design configurations and the mean overtopping rate for the 10 events

Crest level (mODN)	Mean Overtopping Rate (l/s/m)
5.34	6.27
5.84	3.03
6.34	1.42

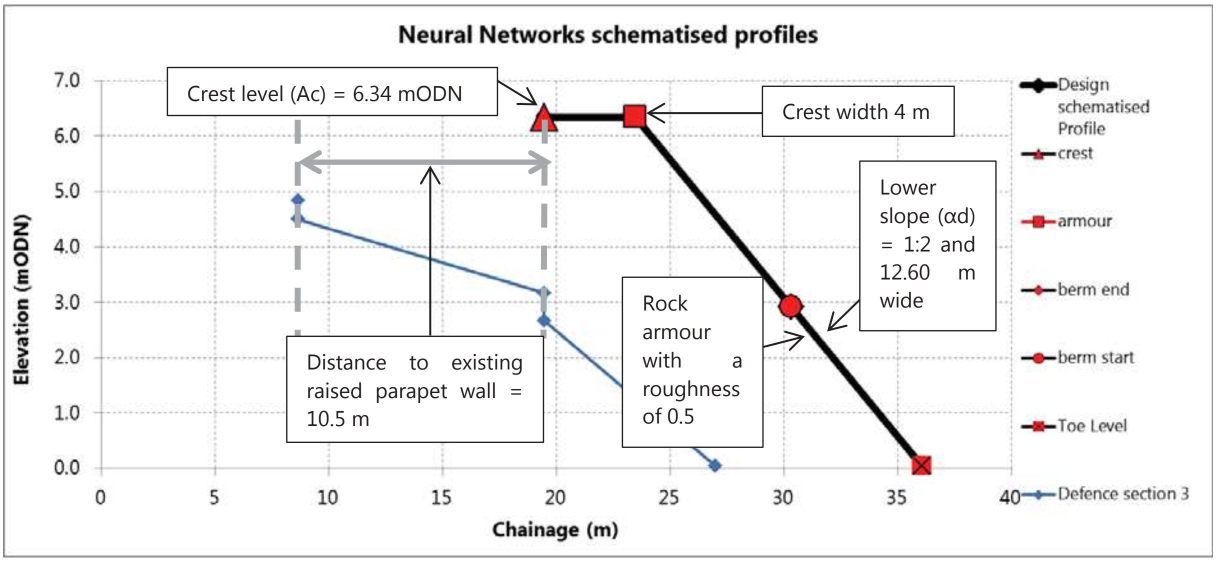


Figure 18. Neural Network schematised profiles of Option 2 design and Defence Section 3 overlaid

Table 18. Calculated mean overtopping rates for Option 2 design per return period for base year 2022

Return Period	Mean Overtopping Rate (l/s/m)
	Option 2 design
10	0.13
50	0.24
100	0.29
200	0.34
1,000	0.49

Table 19. Calculated mean overtopping rates Option 2 design per return period for base year 2122

Return Period	Mean Overtopping Rate (l/s/m)
	Option 2 design
10 cc	0.69
50 cc	1.32
100 cc	1.70
200 cc	2.16
1,000 cc	3.65

3.8.3 Option 3: Rock armour berm extended seaward

Option 3 design is a wide flat berm structure. Rock armour is again placed over the entire structure and the toe advanced seaward. The toe of defence will be created with a slope of 1:2 up to a berm. The upper slope extends horizontally back by 21.5 m to the base of the concrete wave return wall. Table 20 presents the overtopping at the promenade from the ten 2122 events against the numerous configurations for design Option 2.

Of the design configurations initially provided the best performing, in respect to minimising overtopping had a berm height of 4.94 mODN and a wave return wall with a crest level of 5.34 mODN (Figure 19). Table 21 and Table 22 display the predicted extreme overtopping per return period for this configuration, for the 2022 and 2122 epochs respectively.

Table 20. Option 3 design configurations and the mean overtopping rate for the 10 events

Crest level (mODN)	Berm level (mODN)	Mean Overtopping Rate (l/s/m)
4.84	3.94	40.71
4.84	4.44	34.41
5.34	3.94	30.80
5.34	4.44	26.50
5.34	4.94	22.33

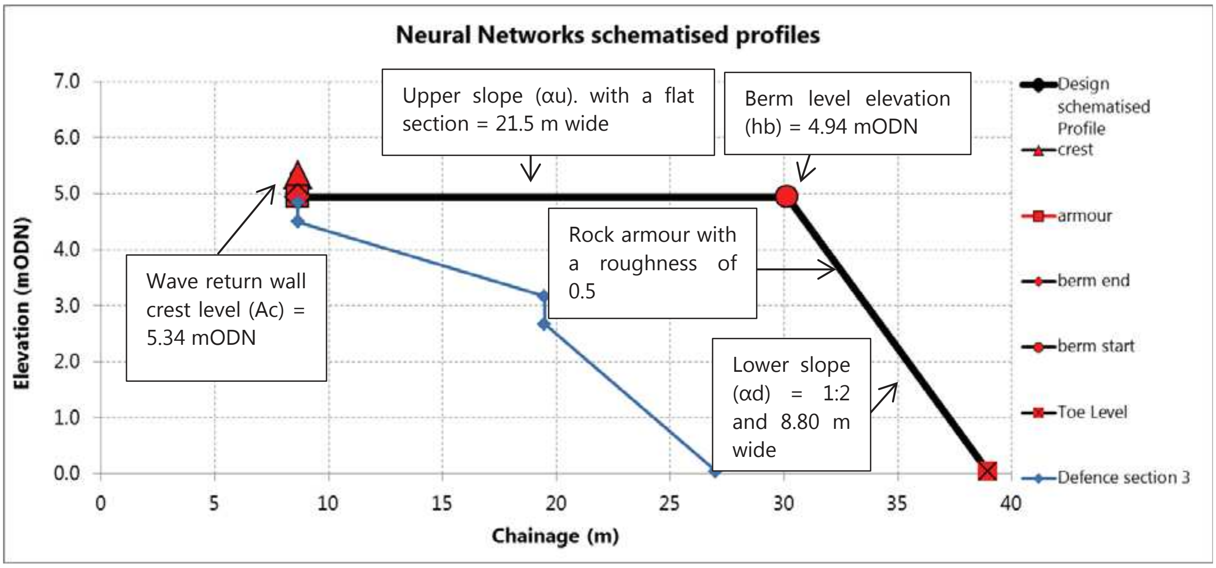


Figure 19. Neural Network schematised profiles of Option 3 design and Defence Section 3 overlaid.

Table 21. Calculated mean overtopping rates for Option 3 design per return period for base year 2022

Return Period	Mean Overtopping Rate (l/s/m)
	Option 3 design
10	3.33
50	6.61
100	8.45
200	10.59
1,000	17.09

Table 22. Calculated mean overtopping rates for Option 3 design per return period for base year 2122

Return Period	Mean Overtopping Rate (l/s/m)
	Option 3 design
10 cc	11.65
50 cc	20.08
100 cc	23.72
200 cc	27.35
1,000 cc	35.79

4 Summary and Conclusions

Portgordon frequently suffers inundation due to wave overtopping as a result of poorly performing sea defences. Wave modelling and overtopping analysis was conducted to support the development of a high level appraisal of different defence options.

A 37 year hindcast of waves and winds was run through a local MIKE21 SW wave model. The wave modelling calculated that the most significant waves to affect the Portgordon frontage come from a northerly direction and have a significant wave height of *circa* 3 m at the toe of the defences. Wave conditions derived at the toe of the defences were then used in Neural Networks overtopping model to assess overtopping rates.

The overtopping modelling was initially validated against two events, those occurring on the 05/12/2013 and 09/10/2014. This validation exercise indicated that the western side of Portgordon (Defence Section 3) was the most vulnerable to overtopping. The eastern side of the Portgordon showed lower overtopping and this was attributed to two factors;

- Wave shadowing from Portgordon harbour;
- The defence heights along the eastern side of the frontage are higher than the western side.

The overtopping for Defence Section 3 (the section resulting in the worst case overtopping) was subject to extremes analysis, for two separate epochs, namely, 2022 and 2122. This indicated that for all return periods over the 1 in 10 year event, the extremes overtopping was calculated to be greater than 10 l/s/m for both epochs. Overtopping at 10 l/s/m is the EurOtop (2007) limit for well-prepared pedestrians and where driving a low speeds for vehicles commences. This highlights the poor performance of the defences at Portgordon.

To improve the defence's performance three different defence designs were considered:

- Option 1: Rock armour berm over upper slope;
- Option 2: High rock armour berm over existing lower seawall;
- Option 3 Rock armour berm extended seaward.

For each of the three defence options a number of different configurations were considered, 14 different defence configurations in total. Each configuration of the defences was performance tested with the wave and water level conditions associated with the ten worst overtopping from the baseline undertaken for the 2122 epoch. The best performing configuration from the Option 1 design (rock armour berm over upper slope) had a berm height of 5.44 mODN with a width of 12.5 m to a wave return wall with a crest level of 5.84 mODN.

The best performing configuration from the Option 2 design (high rock armour berm over existing lower seawall) had the lower wall raised to 6.34 mODN with a crest width of 4 m. The best performing Option 3 design had a berm height of 4.94 mODN and 21.5 m wide to wave return wall with a crest level of 5.34 mODN.

Comparative analysis between the different design options showed that the best performing defence option was Option 2 design (lower wall raising) with a crest level of 6.34 mODN. The best performing Option 2 design configuration had extremes analysis conducted for both epochs. The extremes analysis showed significant improvements to overtopping performance for Portgordon reducing overtopping to significantly below 10 l/s/m for all return periods considered for both the 2022 and 2122 epochs. Thus this defence combination should reduce overtopping frequency and inundation volumes at Portgordon considerably.

5 References

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National Centers for Environmental Prediction (NCEP) Climate Forecast System v2 (CFSv2): http://rda.ucar.edu/datasets/ds094.1/

UKHO INSPIRE Portal & Bathymetry DAC: http://aws2.caris.com/ukho/mapViewer/map.action

6 Abbreviations/Acronyms

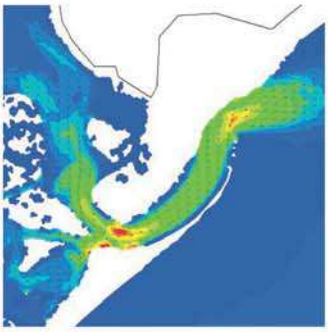
CD	Chart Datum
CFSR	Climate Forecast System Reanalysis
CLASH	Crest Level Assessment of coastal Structures by full scale monitoring, Neural Network prediction and Hazard analysis on permissible wave overtopping
DHI	Danish Hydraulic Institute
EurOtop	European Overtopping Manual
GEBCO	General Bathymetric Chart of the Ocean
GPD	Generalised Pareto Distribution
Hs	Significant wave height
LiDAR	Light Detection And Ranging
MDir	Mean Direction
MHWS	Mean High Water Spring
mODN	Meters above Ordnance Datum Newlyn
MSL	Mean Sea Level
NCAR	National Center for Atmospheric Research
NCEP	National Centers for Environmental Prediction
NNT	Neural Network Tool
NOAA	National [American] Oceanic and Atmospheric Administration
ODN	Ordnance Datum Newlyn
OSGB	Ordnance Survey Great Britain
OSGB36	Ordnance Survey Great Britain 1936
OT	Overtopping
PAR	Project Appraisal Report
SEASTATES	ABPmer forecast website
SW	Spectral Wave
Tp	Peak wave period
TUFLOW	Two-dimensional Unsteady FLOW
UK	United Kingdom
UKCP09	UK Climate Projections 09
UKCS	UK Continental Shelf
UKHO	United Kingdom Hydrographic Office
UTM30	Universal Transverse Mercator coordinate system
VORF	Vertical Offshore Referencing Frame
WGS84	World Geodetic System 1984

Cardinal points/directions are used unless otherwise stated.

SI units are used unless otherwise stated.

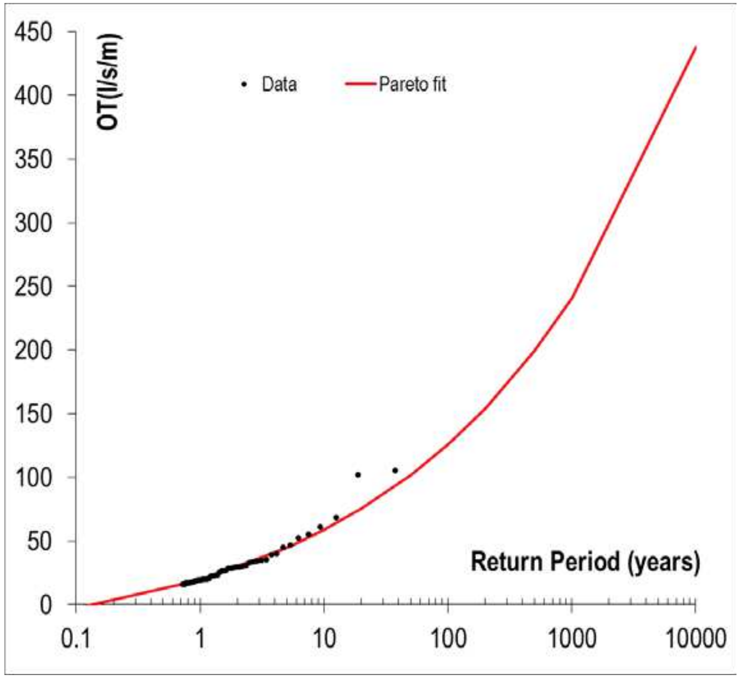
Appendices



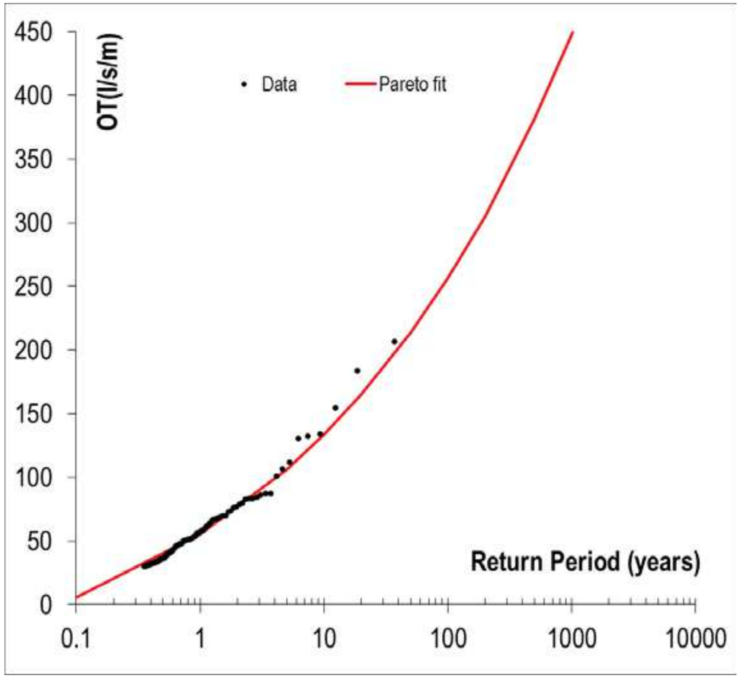




Innovative Thinking - Sustainable Solutions

A “Maintained Existing Defence” Overtopping GPD Figures



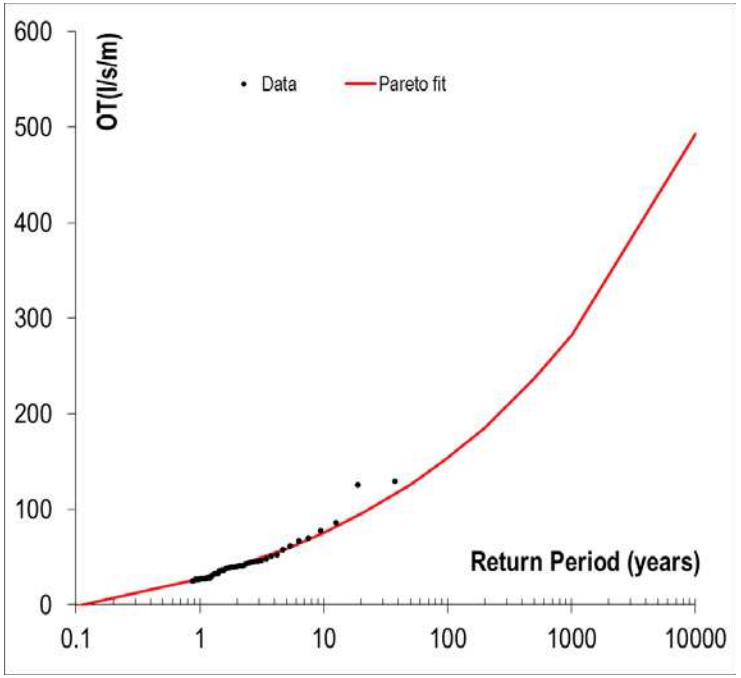
“Maintained existing defence” 2022



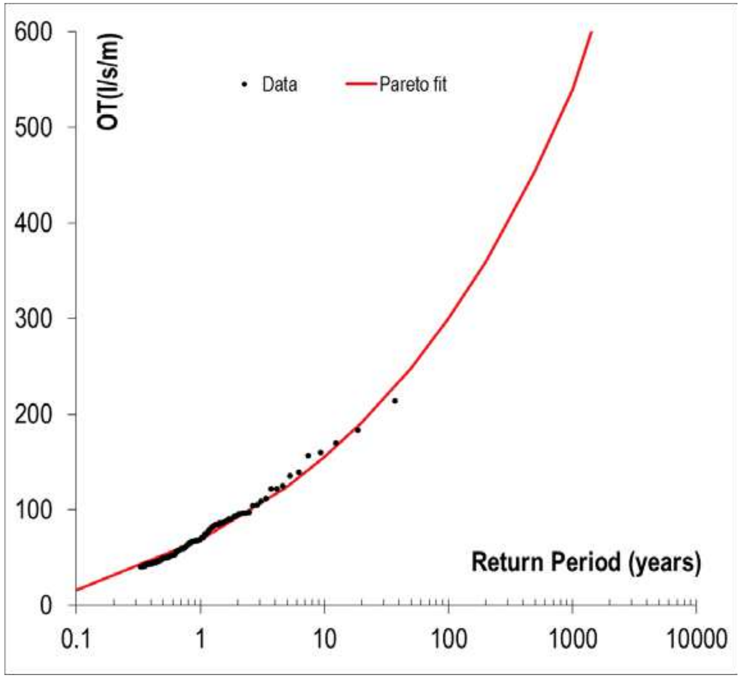
“Maintained existing defence” 2122

Figure A1. “Maintained existing defence” overtopping GPD figures

B “Failed Existing Defence” Overtopping GPD Figures



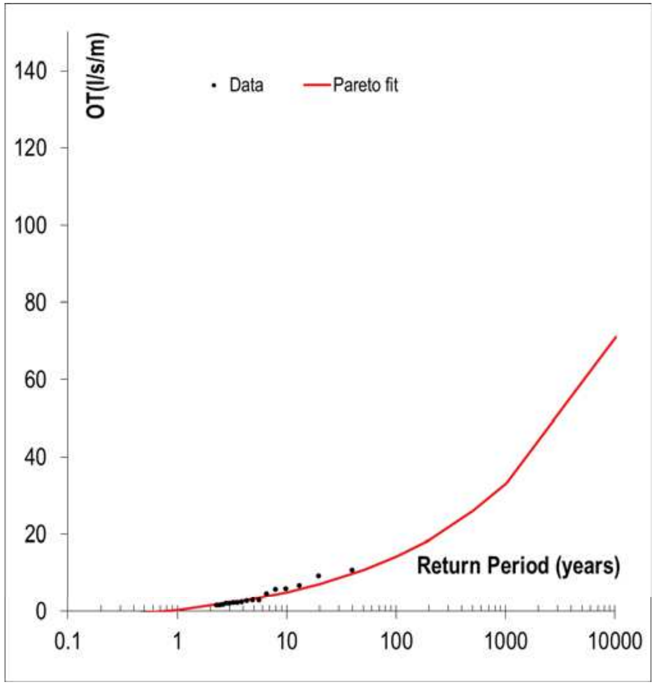
“Failed existing defence” 2022



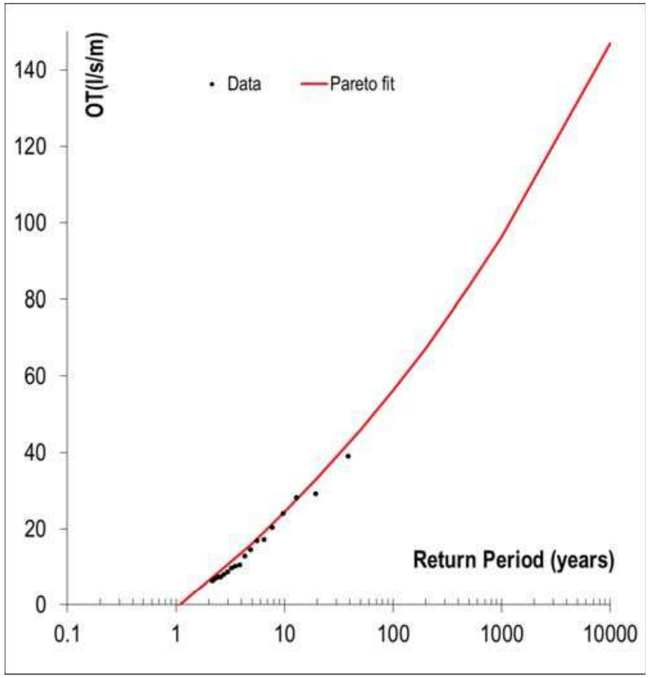
“Failed existing defence” 2122

Figure B1. **“Failed existing defence” overtopping GPD figures**

C "Design" Overtopping GPD Figures

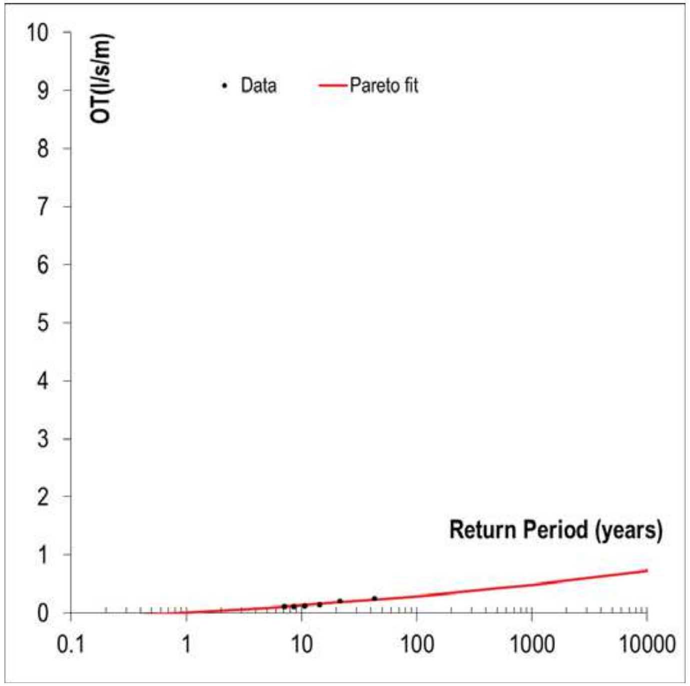


"Option 1 design" 2022

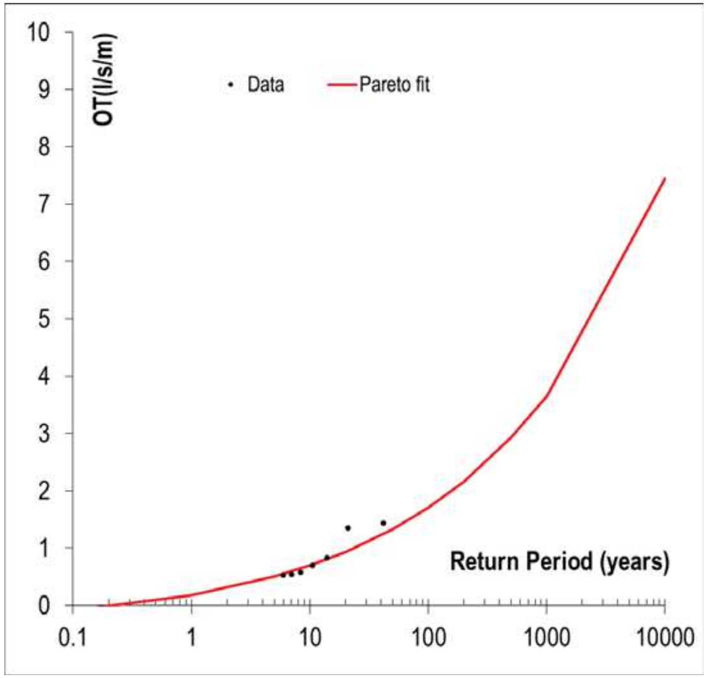


"Option 1 design" 2122

Figure C1. "Option 1 design" overtopping GPD figures

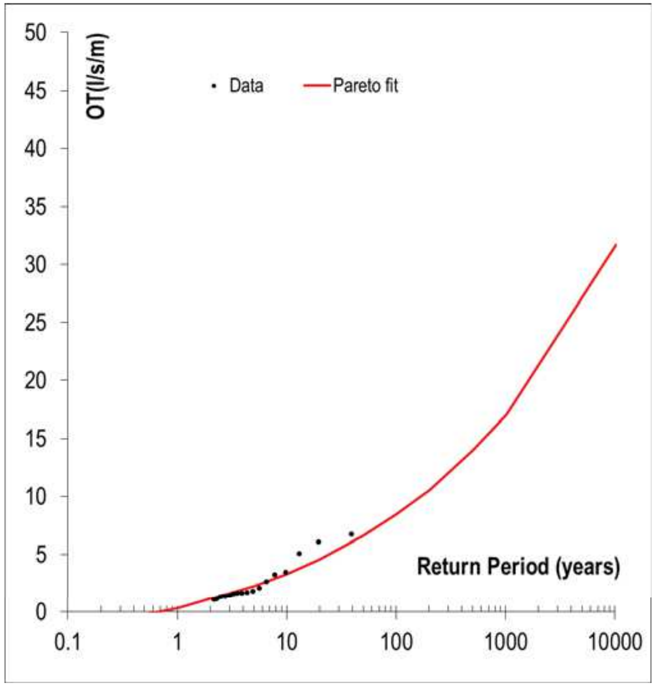


"Option 2 design" 2022

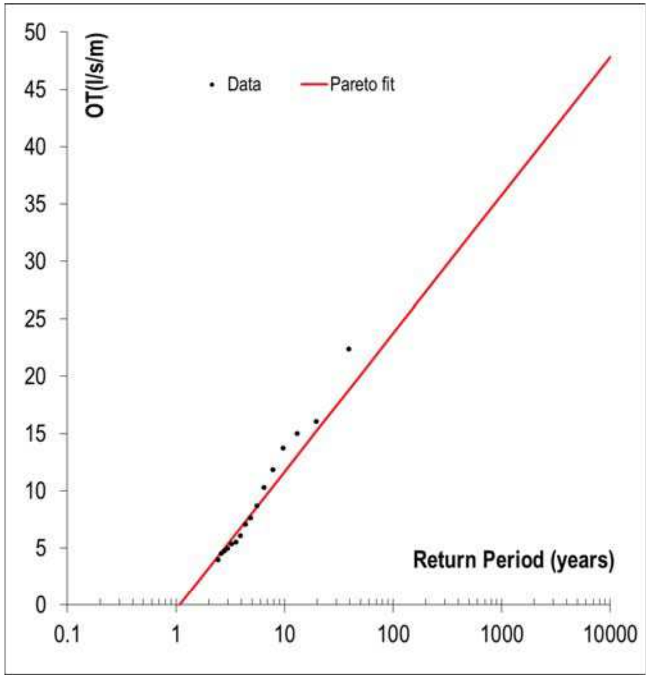


"Option 2 design" 2122

Figure C2. "Option 2 design" overtopping GPD figures



"Option 3 design" 2022



"Option 3 design" 2122

Figure C3. "Option 3 design" overtopping GPD figures

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Appendix B. ABPmer Portgordon Sea Defence Options Overtopping Assessment

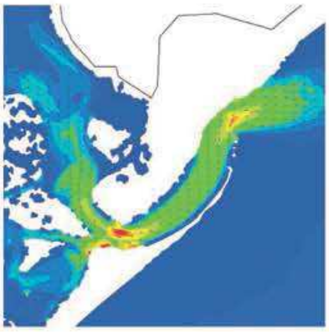
Jacobs

Portgordon Sea Defence Options

Overtopping assessment

December 2017




Innovative Thinking - Sustainable Solutions

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Portgordon Sea Defence Options

Overtopping assessment

December 2017

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Contents

Appendices

Tables

Figures

1 Introduction

Jacobs is undertaking a study to review options for the sea defences at Portgordon (Figure 1), on behalf of Moray Council. The revetment at Portgordon often suffers from wave overtopping which results in the flooding of the road and properties behind the sea defence. To assist Jacobs, ABPmer has undertaken wave modelling and overtopping analysis to support the development and appraisal of different defence options (ABPmer, 2017). This report details the overtopping analysis of two different defence Options provided by Jacobs for the 2122 epoch.

Source: Map data ©2016 Google. Image © 2016 Terrametrics. Data © SIO, NOAA, U.S. Navy, NGA, GEBCO

Figure 1. Aerial imagery of Portgordon

1.1 Existing defences

Portgordon is defended from coastal flooding by linear defences, which extend around 700 m in length from the western harbour arm. These defences are backed by the main coastal road through Portgordon. The existing defences differ only very slightly along the frontage, primarily in crest height and defence alignment. The study area has been characterised into three defence lengths (Table 1 and Figure 2) based on the difference in crest height, defence facing angle relative to the coastline and variation in wave conditions. The full overtopping extreme analysis has been completed for all three defence lengths.

Table 1. Defence lengths

Defence Section	Length (m)
Defence 1 (Def_1)	194.02
Defence 2 (Def_2)	215.58
Defence 3 (Def_3)	303.61

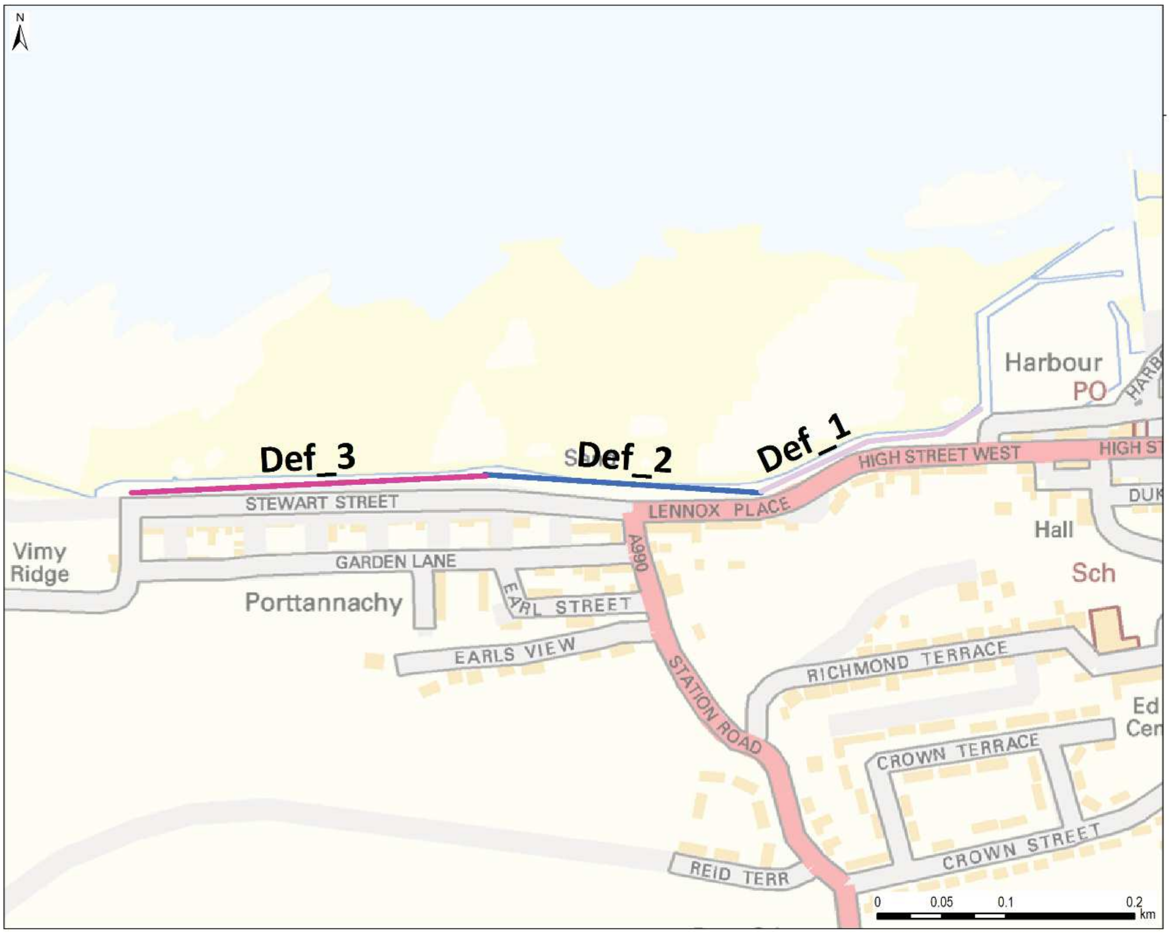


Figure 2. Indicative location of the three defence sections

2 Methodology

The methodology presented below is the same that was used previously in ABPmer (2017) and is presented for continuity.

2.1 Overtopping

Overtopping has been calculated using the Neural Network Tool (NNT) developed under EurOtop (2007) as the most suitable tool for calculating overtopping. However, the NNT does have some limitations and requires engineering judgement when applying the tool and interpreting the overtopping results.

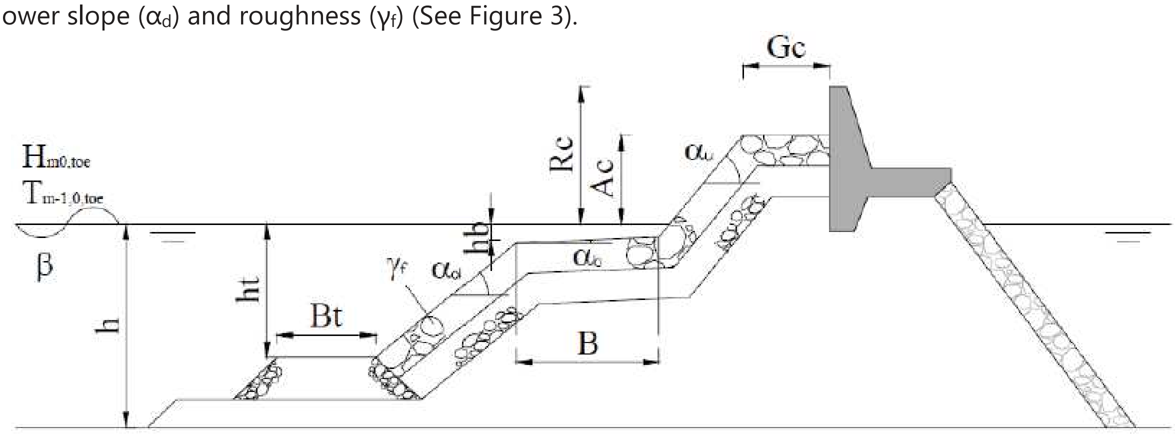
Wave height, periods and directions were extracted from a 37-year record of waves which were transformed to the toe of the structure by the numerical modelling exercise described in ABPmer (2017). These waves were also paired with the coincident water levels, and data derived for the site for the 2122 epoch.

Coincident waves and water levels for the 2122 epoch data sets were then run through the NNT to obtain a continuous time-series record of overtopping rate for the defence Options. The results from these predictions were then subsequently analysed as part of the extremes overtopping assessment (Section 3.3) to derive overtopping discharges for a range of return periods and specifically the 200-year return period in the 2122 epoch.

The overtopping rates acquired were then used to extrapolate extreme overtopping discharges for a range of return periods. The significant advantage of the proposed method is that overtopping was calculated from over 300,000 historical combinations of waves and water levels, rather than relying on extreme water levels defined offshore (as found in the Environment Agency extreme coastal flood boundary data (2011)). The unique event combinations that resulted in the largest overtopping rates for each defence section are then used to construct a design overtopping hydrograph as per the Environment Agency (2011) methodology.

2.2 Schematisation of defences

Within the NNT defences are schematised using 15 geometric parameters which include; crest height (Rc), armour height (Ac), armour width (Gc), berm elevation (hb), berm width (B), upper slope (α_u), lower slope (α_d) and roughness (γ_f) (See Figure 3).



Source: Coeveld *et al*, 2005: CLASH Database

Figure 3. Schematisation descriptors for a defence profile using Neural Networks overtopping tool

Calculation of the overtopping discharge at a feature (e.g. footpath) setback from the defence crest used EurOtop (2007) manual guidance. Initially tests were conducted on the defences prior to full overtopping analysis on the final three versions of the defence were undertaken to determine the relative sensitivity of different possible elements of a future design. The results of the sensitivity testing are presented in Appendix A.

3 Defence Options

The defence options tested for this report are a continuation from those analysed in ABPmer (2017). The three previous defence designs comprised:

- Option 1: Rock armour berm over an upper slope;
- Option 2: High rock armour berm over existing lower seawall; and
- Option 3 Rock armour berm extended seaward.

For this further analysis two Options were provided by Jacobs. These were:

- Option 4: Stepped revetment; and
- Option 5: Vertical wall with wave return.

For each of these Options three variations have been tested to determine the overtopping discharges hence the relative performance of the designs. The results of the testing are presented in the following sections.

3.1 Option 4 - Stepped revetment

Option 4 is a stepped revetment structure of the type shown in Figure A1. To improve overtopping performance based on the sensitivity testing, rock armour would be placed over the toe of the structure to the berm of 3.17 mODN, from the berm to the base of the vertical wave return wall will be constructed of concrete steps. The entire slope of the structure will be 1:2.5 (vertical : horizontal) up to the base of the wave return wall at 5.34 mODN. The recurve at the crest was tested with variable crest heights. The overtopping rate is calculated at the footpath 7 m from the crest using the EurOtop (2007) guidance. The three versions of the defence tested were:

- Version 1 = A 1 m return wall (6.34 mODN) and rock armour;
- Version 2 = A 1.5 m return wall (6.84 mODN) and rock armour; and
- Version 3 = A 1 m return wall (6.34 mODN) with no rock armour.

The defence elements levels and model combinations for the NNT are shown in Table 2 and Figure 4.

Table 2. Schematised defence Option 4

Defence	Defence Descriptor	Version 1 Levels (mODN)	Version 2 Levels (mODN)	Version 3 Levels (mODN)
Option 4	Toe (at Def_1)	0.50		
	Toe (at Def_2)	0.00		
	Toe (at Def_3)	0.05		
	Berm level	3.17		
	Defence slope	1:2.5		
	Crest level	6.34	6.84	6.34
	Defence material (roughness coefficient)	Rock armour (0.55) toe/ stepped revetment (0.8) to concrete vertical wall (1)	Rock armour toe (0.55)/ stepped revetment (0.8) to concrete vertical wall (1)	Stepped revetment (0.8) from toe to concrete vertical wall (1)

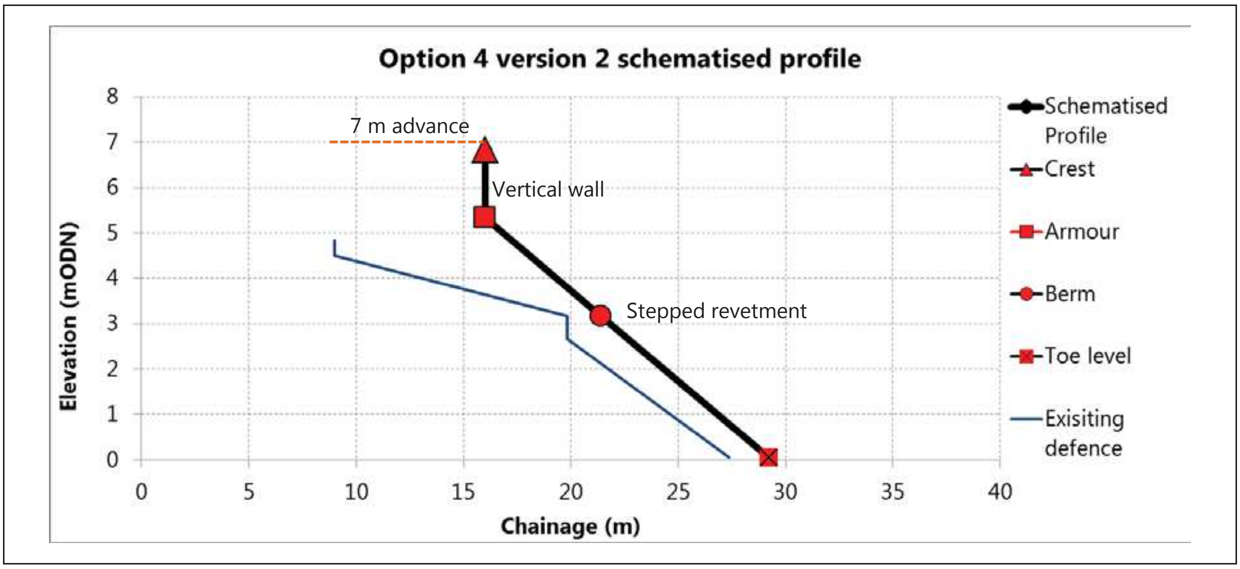


Figure 4. Example NNT schematisation (Version 2)

3.2 Option 5 - Vertical wall with wave return

The Option 5 design is a vertical wall with wave return. Overtopping is calculated at the footpath 13.5 m set back the crest using EurOtop (2007) guidance. The three versions of the defence to be tested are:

- Version 1 = Crest at 5.84 mODN;
- Version 2 = Crest at 6.34 mODN; and
- Version 3 = Crest at 7.34 mODN.

The defence element levels and model combinations for the NNT are shown in Table 3 and Figure 5.

Table 3. Schematised defence Option 5

Defence	Defence Descriptor	Version 1 Levels (mODN)	Version 2 Levels (mODN)	Version 3 levels (mODN)
Defence Option 5	Toe (at Def_1)		0.50	
	Toe (at Def_2)		0.00	
	Toe (at Def_3)		0.05	
	Defence slope	Vertical	Vertical	Vertical
	Crest level	5.84	6.34	7.34
	Defence material (roughness coefficient)	Concrete vertical wall (1)	Concrete vertical wall (1)	Concrete vertical wall (1)

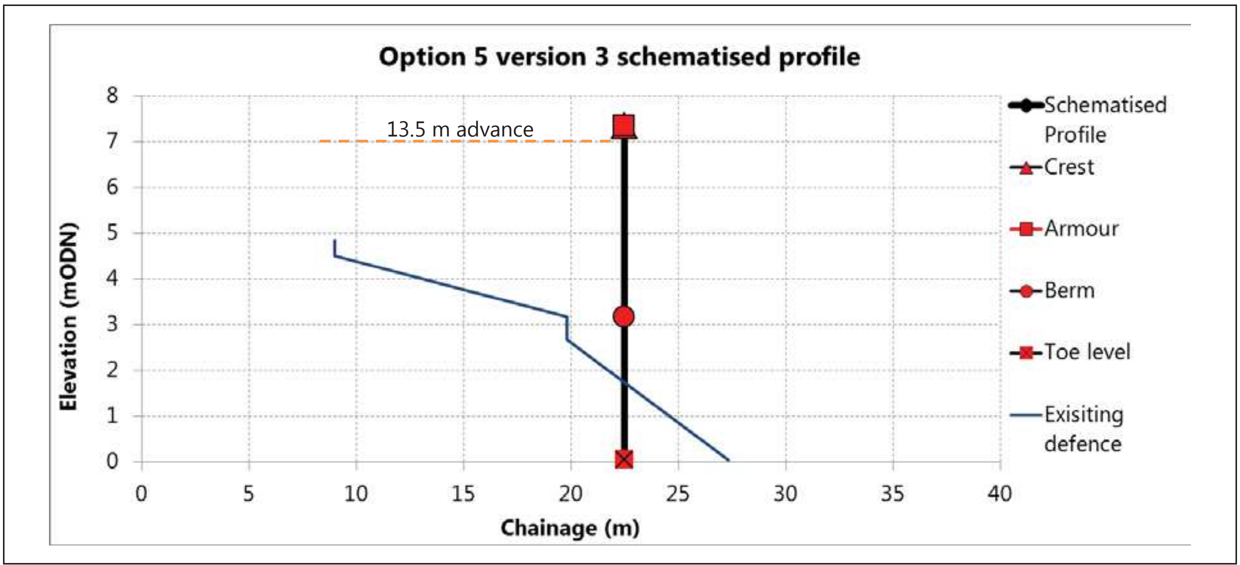


Figure 5. Example NNT schematisation (Version 3)

3.3 Performance testing

This section provides the performance testing results for each of the variations for defence Options 4 and 5. The best (lowest overtopping rate) defence option version has then been taken forward for the full overtopping analysis (Section 4).

To establish the best performing defence version (lowest overtopping rate) the 37-year time series of incident wave conditions at the toe of the structure were taken from the nearshore wave model results (ABPmer, 2017) at the most western extraction point (Def_3) (Figure 2). This was the section that suffered the highest overtopping rates for the existing defence. This time series was then passed through the NNT to develop a 37-year time series of mean overtopping rates for each defence Option version. The results from the best performing defence version for each option were then subsequently analysed as part of the extremes overtopping assessment to derive overtopping rates for the 200-year return period for that section of the Portgordon defence. These best performing defence Option versions were then used for the whole frontage and time series of wave and water levels extracted from the model at the toe of defence sections 1 and 2. This method therefore takes account of the variations in wave conditions caused by the bathymetry and orientation of the coast. The highest hindcast 2122 overtopping rate for each design versions are presented in Table 4.

Table 4. The hindcast 2122 epoch highest mean overtopping rate for each defence option version at the footpath

Defence Option Version	Option 4: Defence Section (l/s/m)			Option 5: Defence Section (l/s/m)		
	1	3	3	1	2	2
1	5.61	6.13	6.95	7.78	9.90	10.10
2	3.18	3.33	3.85	4.77	6.02	6.51
3	8.27	11.20	12.58	1.35	1.43	1.81

These overtopping results from this method of analysis for the 2122 epoch showed that; in respect to minimising overtopping:

- Overtopping would still be greatest at defence Section 3;
- The best performing Option 4 was version 2 (6.84 mODN crest); and
- The best performing Option 5 was version 3 (7.34 mODN crest).

These two defence option configurations were carried forward to the extremes overtopping analysis for each defence section as described in Section 4.

4 Extremes Overtopping Analysis

To derive overtopping rates for different return periods, the 37-year record of overtopping for 2122 epoch were run through a Generalised Pareto Distribution (GPD) extremes package. The overtopping events from the hindcast time series above a threshold are selected and are plotted against return period. The Pareto distribution of the overtopping and return periods were then fitted using the software package in2extremes (see Gilleland & Katz, 2016) to the overtopping events. In this process, the shape and scale parameters of the fitted data are determined. The Pareto fit to the data is visually assessed, and if necessary the threshold is reselected and the extrapolation refitted to the data to improve the fit quality. This is a subjective process guided by the behaviour of the scale and shape parameters at various thresholds, and by the experience of the practitioner. Further details on threshold selection can be found in Coles (2001). The final shape and scale parameters are used to extrapolate the theoretical fit to the data to determine extreme conditions for various return periods.

The overtopping resultant fits are provided in Appendix B for each Option for each crest height. Tables Table 5 and Table 6 provide the 200-year return period overtopping rate for the best preforming Option 4 version 2 and Option 5 version 3.

Table 5. Calculated 200-year return period in the 2122 epoch for the defence Option 4 version 2 for each defence section

Defence Option 4 Version 2	200-Year Return Period Mean Overtopping Rate l/s/m
Defence Section 1	4.54
Defence Section 2	5.62
Defence Section 3	7.57

Table 6. Calculated 200-year return period in the 2122 epoch for the defence Option 5 version 3 for each defence section

Defence Option 5 Version 3	200-year Return Period Mean Overtopping Rate l/s/m
Defence Section 1	2.23
Defence Section 2	3.72
Defence Section 3	4.42

The resultant overtopping rates for each defence section are related to the guidance to overtopping thresholds given in the EurOtop (2007) manual.

Overtopping rates are above 1 l/s/m and below 10 l/s/m. In accordance with EurOtop (2007) guidance, the overtopping has a 'Hazard type and reason' - '*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*'.

5 Extreme Overtopping Hydrographs

5.1 Design tide hydrographs

To estimate overtopping volumes during a storm event, idealised design tide hydrographs have been generated for each return period event, for each defence section for the 2122 epoch. To generate these design tide hydrographs the Environment Agency preferred method (Environment Agency, 2011) was adopted. This method was developed with the support of the Scottish Environment Protection Agency.

To achieve this, a design tide hydrograph was constructed using a distance weighted mean high-water spring (MHWS) base astronomical tide extracted from Total Tide software for Buckie and Lossiemouth. This base astronomical tide was combined with a scaled Moray Firth design surge shape profile (Environment Agency, 2011) such that the water level at the peak of the overtopping event from the hindcast period is equivalent to the required water level for the overtopping event (Figure 6).

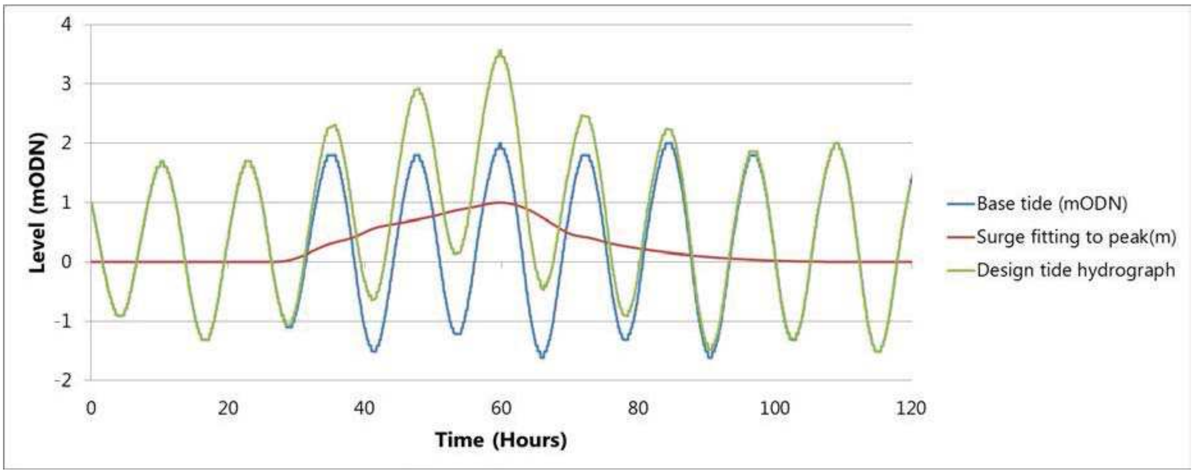


Figure 6. Indicative design tide hydrographs

5.2 Design overtopping hydrographs

To create design overtopping hydrographs the design tidal hydrograph (described in Section 5.1) was run through the NNT as described below. This provided a unique overtopping profile shape for each defence section.

In this assessment, a uniform wave condition (wave height, period and direction) was used over the design tide hydrograph. This wave event equated to the worst overtopping condition obtained at the defence section in the hindcast 37-year overtopping record. The wave conditions for this event are then run through the NNT over the design tidal hydrograph (Section 5.1), providing a 25-hour overtopping hydrograph for each defence section.

In this assessment, the overtopping hydrograph is then scaled to the calculated overtopping extreme return periods (see Section 4) providing a wave overtopping design hydrograph for the 200-year return period. This is a derivation of the method set out in Environment Agency (2011), but we believe is in line with the forthcoming Environment Agency 'State of the Nation' approach.

The method therefore focuses upon defining the *result* of the extreme event (i.e. the actual overtopping of a defence) rather than defining the event itself. We believe that this approach more closely reflects latest industry advances in flood risk assessment. The overtopping hydrographs for each return period were provided in digital format to be used in inundation model inputs.

6 Conclusion

The revetment at Portgordon often suffers from wave overtopping which results in the flooding of the road and properties behind the sea defence. Two defence Options were assessed with three crest height version of each, for the 2122 epoch to assist Jacobs in an economic analysis; namely:

- A stepped revetment (option 4); and
- Vertical wall (option 5).

The best performing defence for Option 4 (stepped revetment) was version 2 (6.84 mODN crest with rock toe). Options 5 (vertical wall) version 3 was the best performing with a crest of 7.34 mODN. Both these defence options reduced overtopping likelihood and mean volumes substantially, by *circa* 98% and 97% from the existing conditions.

7 References

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8 Abbreviations/Acronyms

GPD	Generalised Pareto Distribution
MHWS	Mean High-Water Spring
mODN	metre Ordnance Datum Newlyn
NNT	Neural Network Tool
OT	Overtopping

Cardinal points/directions are used unless otherwise stated.

SI units are used unless otherwise stated.

Appendices

Innovative Thinking - Sustainable Solutions

A Initial Sensitivity Testing

The overtopping sensitivity of the different defence components was tested using Neural Networks Overtopping Tool (NNT) and the 10 worst overtopping events from the 2122 hindcast epoch for the existing "Do Minimum" defence configuration (ABPmer, 2017 Report 2801). Testing using the 10 worst conditions allows the sensitivity tests to include variations in wave and water level characteristics to be considered rather than against a single hydrodynamic condition. Both Option 4 (with 5 m set back) and 5 (with a 15 m set back) reduce overtopping from 206 l/s/m in the existing "Do Minimum" defence in the 2122 epoch at the footpath by *circa* 95% and 44% respectively. It should be noted from the detailed extremes analysis the "Do Minimum" overtopping rate for the 200-year return period for the 2122 epoch was calculated to be 305.5 l/s/m, the extremes analysis will be conducted on the chosen design.

A.1 Option 4 - Stepped revetment initial sensitivity testing

Based on Figure A1 the Neural Network overtopping tool was used to test the initial defence schematisation and the sensitivity of several defence features.

- Crest level variations (±0.5 m) (i.e. 5.84 mODN to 7.78 mODN);
- Slope variations (±0.5) (1:2 to 1:3); and
- Crest wall styles (Recurve or vertical).

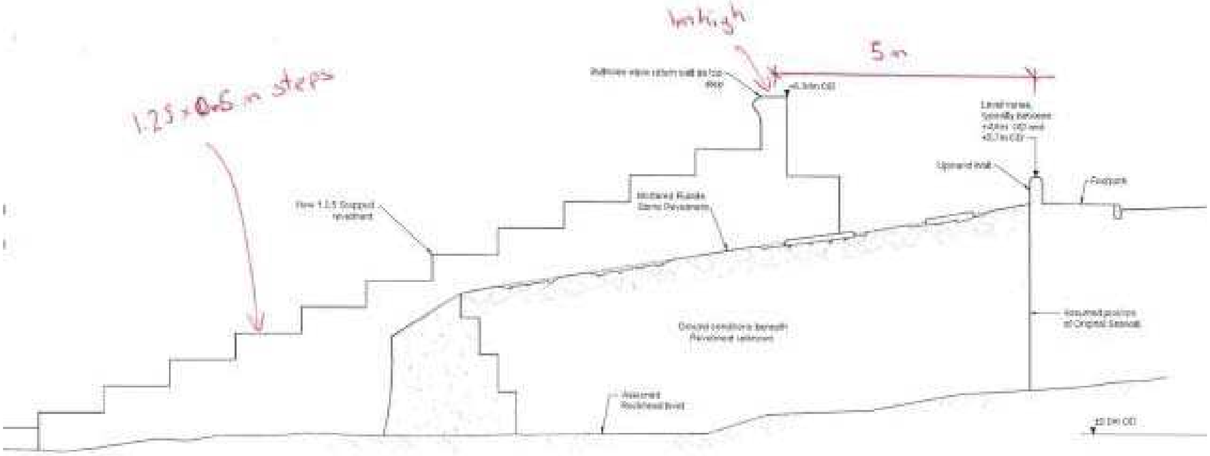

Figure not scale, supplied by: Jacobs

Figure A1. Option 4 - Stepped revetment

Table A1. Option 4 - Sensitivity test

Defence Sensitivity Iterations	OT (l/s/m) at Primary Defence	OT (l/s/m) 5 m Back at Footpath	OT (l/s/m) 7 m Back at Footpath	OT (l/s/m) 10 m Back at Footpath
"Do Minimum Defence 3"	206 (at footpath)	-	-	-
Option 4 base design (Crest 6.34 mODN, slope 1:2.5, roughness 0.8)	49.0	9.8	7.0	4.9
Recurve crest increase 0.5 m (6.84 mODN)	25.2	5.0	3.6	2.5

Defence Sensitivity Iterations	OT (l/s/m) at Primary Defence	OT (l/s/m) 5 m Back at Footpath	OT (l/s/m) 7 m Back at Footpath	OT (l/s/m) 10 m Back at Footpath
Recurve crest decrease 0.5 m (5.84 mODN)	92.9	18.6	13.2	9.3
Crest wall to 7.34 mODN	17.7	3.5	2.5	1.8
Crest wall to 7.84 mODN	9.1	1.8	1.3	0.9
Slope +0.5 (Flatter) (slope of 1:3)	39.1	7.8	5.6	3.9
Slope -0.5 (Steeper) (slope of 1:2)	88.4	17.7	12.6	8.8
No recurve at the crest (Vertical wall) (crest of 6.34 mODN)	67.1	13.4	9.6	6.7
Rock armour slope (1:1.5 and roughness 0.55)	99.6	19.9	14.2	10.0

A.2 Option 4 - Complex rock armour toe and stepped revetment initial sensitivity testing

This variation is based on Figure A1. The Neural Network overtopping tool was used to test the initial defence schematisation and the sensitivity of several defence features. The results are presented in Table A2. The variations of Option 4v2 are;

- Slope consistent at 1:2.5;
- Rock revetment from bed level to +3.17 mODN (0.55 roughness factor), then concrete stepped revetment (0.8 roughness factor) to +5.34 mODN; and
- Wave return wall at crest with crest level variations (+ 0. 5 m) (i.e. 6.34 mODN to 6.84 mODN).

Table A2. Option 4 complex rock armour toe and stepped revetment initial sensitivity testing

Defence Sensitivity Iterations	OT (l/s/m) at Primary Defence	OT (l/s/m) 5 m Back at Footpath	OT (l/s/m) 7 m Back at Footpath	OT (l/s/m) 10 m Back at Footpath
Base design 4 with complex rock armour toe and stepped mid-section. Crest at 6.34 mODN	34.2	6.8	4.9	3.4
Complex design 4 with crest increase to 6.84 mODN	18.6	3.7	2.7	1.9

A.3 Option 5 - Wave return wall initial sensitivity testing

Based on Figure A2 the Neural Network overtopping tool was used to test the initial defence schematisation and the sensitivity of several defence features. The base design includes rock armouring at the toe of the structure up to 2.92 mODN (1:1.5 rock armour slope).

- Crest level variations (±0.5 m) i.e. 5.84 mODN to 6.34 mODN;
- Slope variations of the rock armoured toe (±0.5) (i.e. 1:1 to 1:2) including the removal of rock armour abutting the base of structure (slope changed to vertical and roughness changed); and
- Crest wall styles (Recurve or vertical).

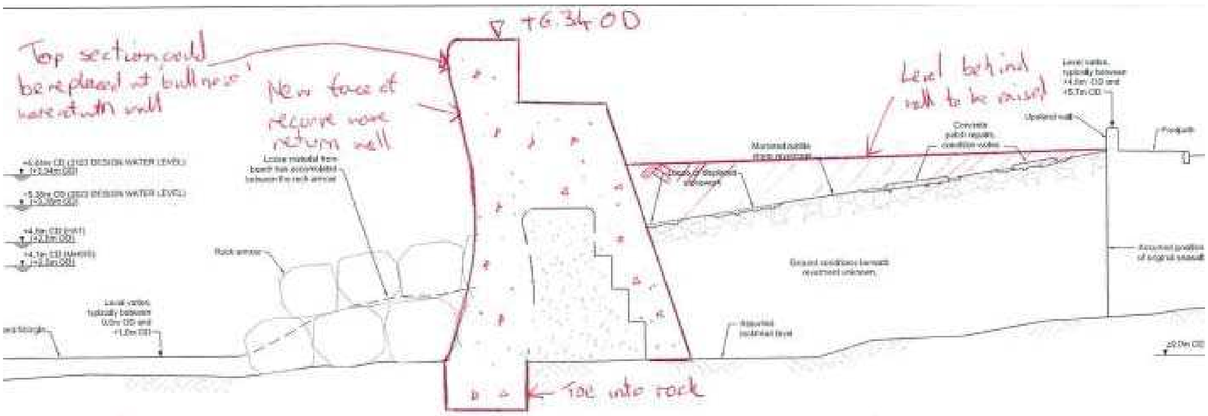

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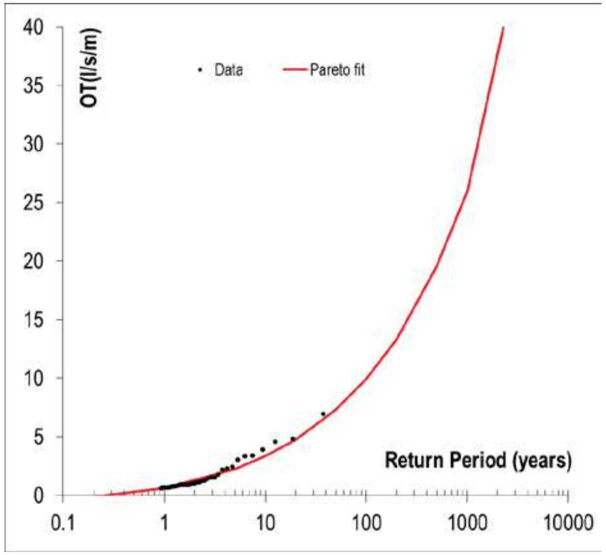
Figure A2. **Option 5 wave return wall**

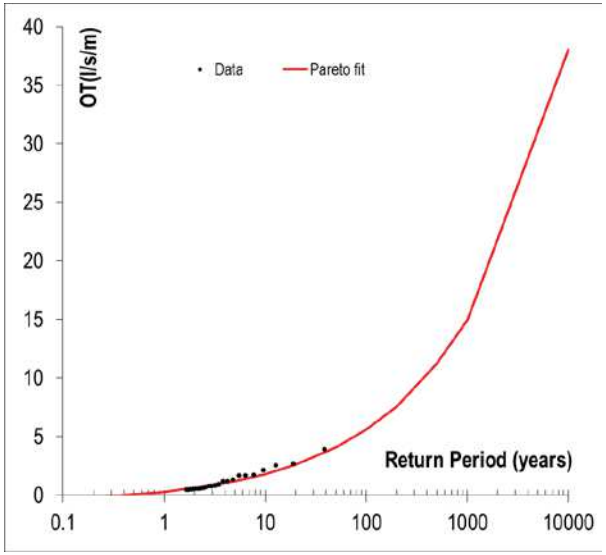
Table A3. **Option 5 sensitivity test**

Defence Sensitivity Iterations	OT (l/s/m) at Primary Defence	OT (l/s/m) 15 m Back at Footpath
"Do Minimum Defence 3"	-	206 (at footpath)
Option 5 Base Design (With small Rock Armour toe) (6.34 mODN)	115.9	7.7
Recurve crest increase 0.5 m (6.84 mODN)	60.9	4.1
Recurve crest decrease 0.5 m (5.84 mODN)	214.3	14.3
Recurve crest increase 7.34 mODN	32.2	2.1
Recurve crest increase 7.84 mODN	17.5	1.2
Recurve crest increase 8.34 mODN	9.9	0.7
Slope +0.5 (Flatter) (slope of 1:2)	105.1	7.0
Slope -0.5 (Steeper) (slope of 1:1)	116.3	7.8
No recurve (crest at 6.34 mODN)	124.3	8.3
Vertical concrete wall (with recurve return wall) (Crest at 6.34 mODN, vertical wall, roughness changed to 1 for whole structure	87.9	5.9
Vertical concrete wall (with recurve return wall) (Crest at 6.84 mODN, vertical wall, roughness changed to 1 for whole structure	47.3	3.2
Vertical concrete wall (with recurve return wall) (Crest at 7.34 mODN, vertical wall, roughness changed to 1 for whole structure	24.5	1.6
Vertical concrete wall (with recurve return wall) (Crest at 7.84 mODN, vertical wall, roughness changed to 1 for whole structure	12.5	0.8
Vertical concrete wall (with recurve return wall) (Crest at 8.34 mODN, vertical wall, roughness changed to. 1 for whole structure	6.5	0.4

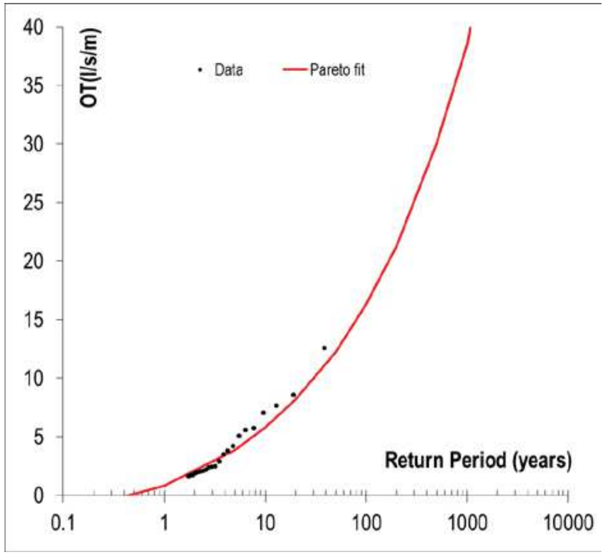
B Overtopping Extremes



Option 4 version 1

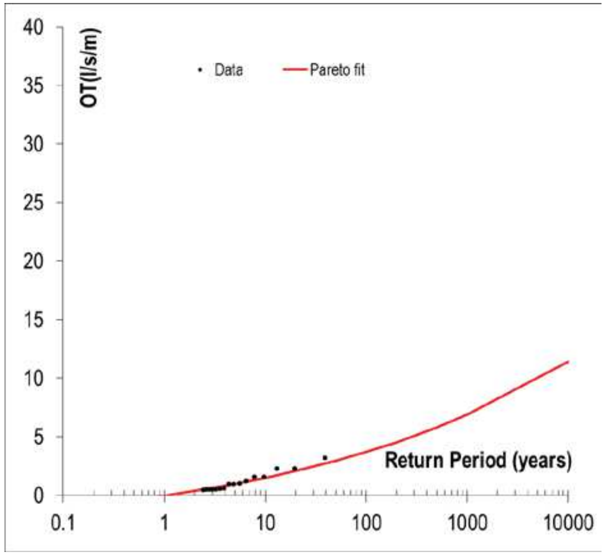


Option 4 version 2

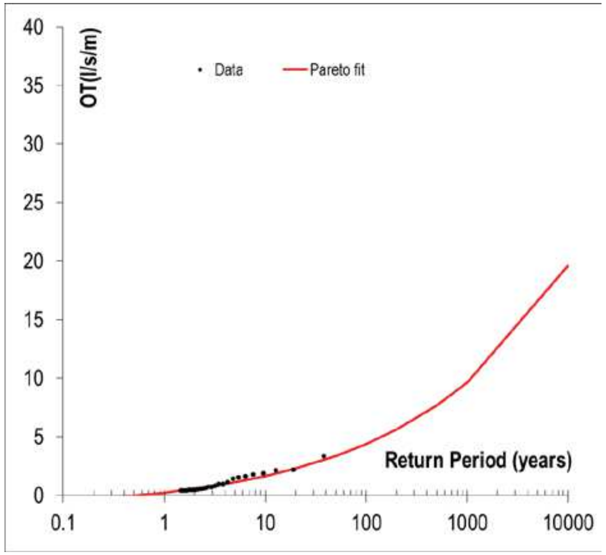


Option 4 version 3

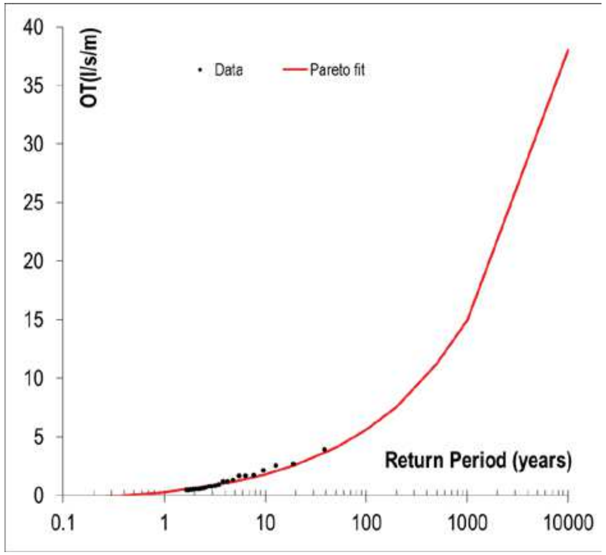
Figure B1. Defence Option 4 design overtopping extreme figures for 2122 Epoch Defence 3 conditions



Option 4 version 2 Defence 1 conditions

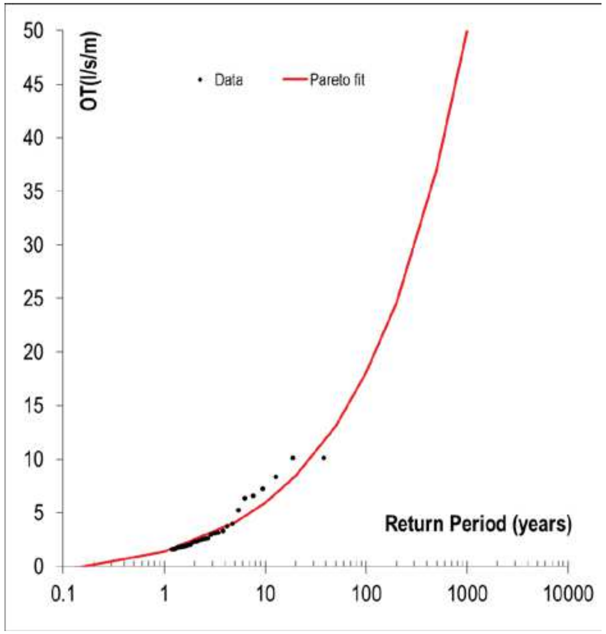


Option 4 version 2 Defence 2 conditions

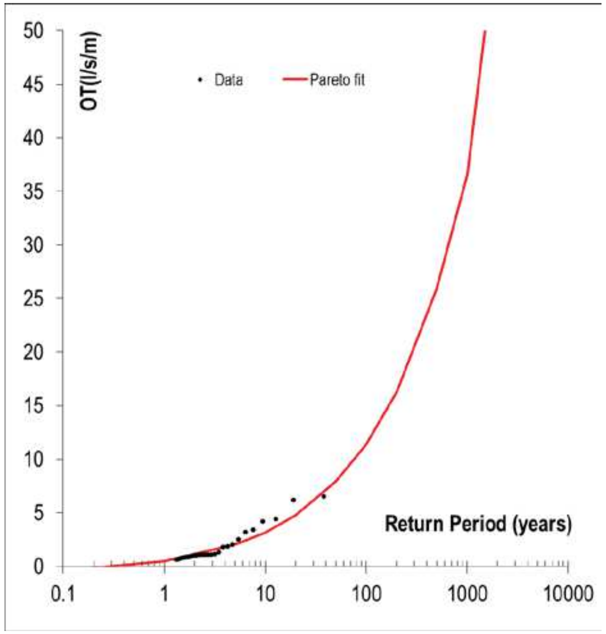


Option 4 version 2 Defence 3 conditions

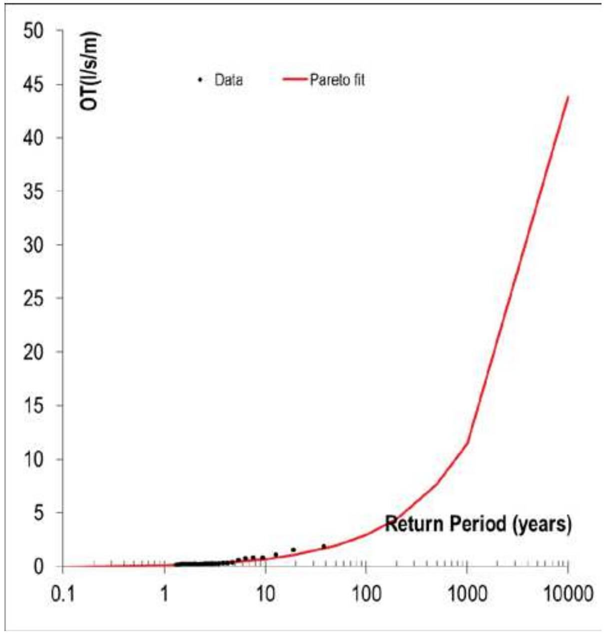
Figure B2. Defence Option 4 Version 2 design overtopping extreme figures for 2122 Epoch Defence 1, 2 and 3 conditions



Option 5 version 1

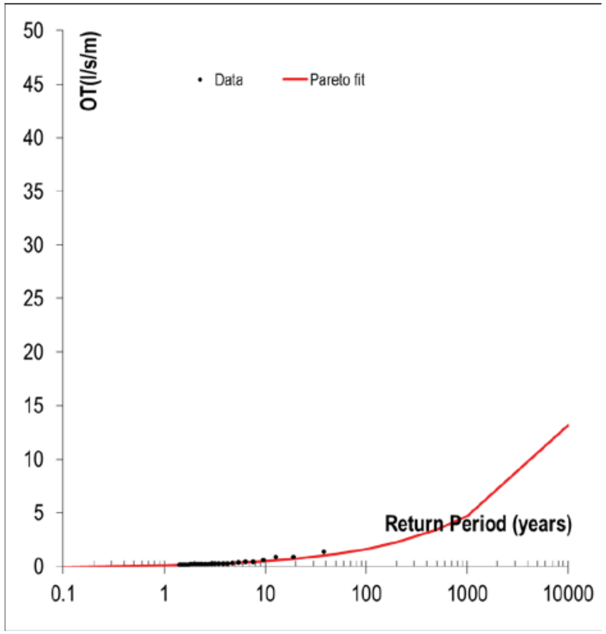


Option 5 version 2

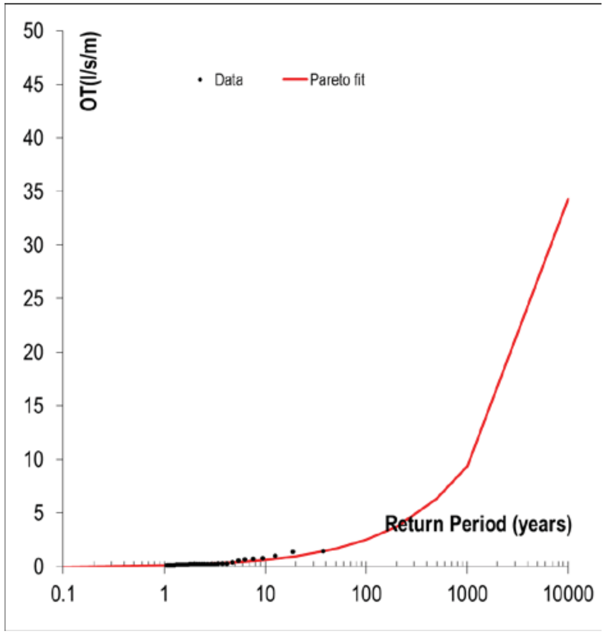


Option 5 version 3

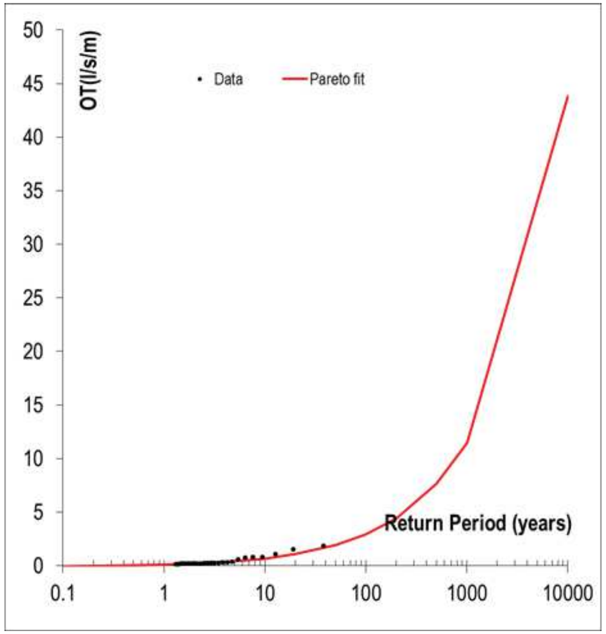
Figure B3. Defence Option 5 design overtopping extreme figures for 2122 Epoch Defence 3 conditions



Option 5 version 3 Defence 1 conditions



Option 5 version 3 Defence 2 conditions



Option 5 version 3 Defence 3 conditions

Figure B4. Defence Option 5 Version 3 design overtopping extreme figures for 2122 Epoch Defence 1, 2 and 3 conditions

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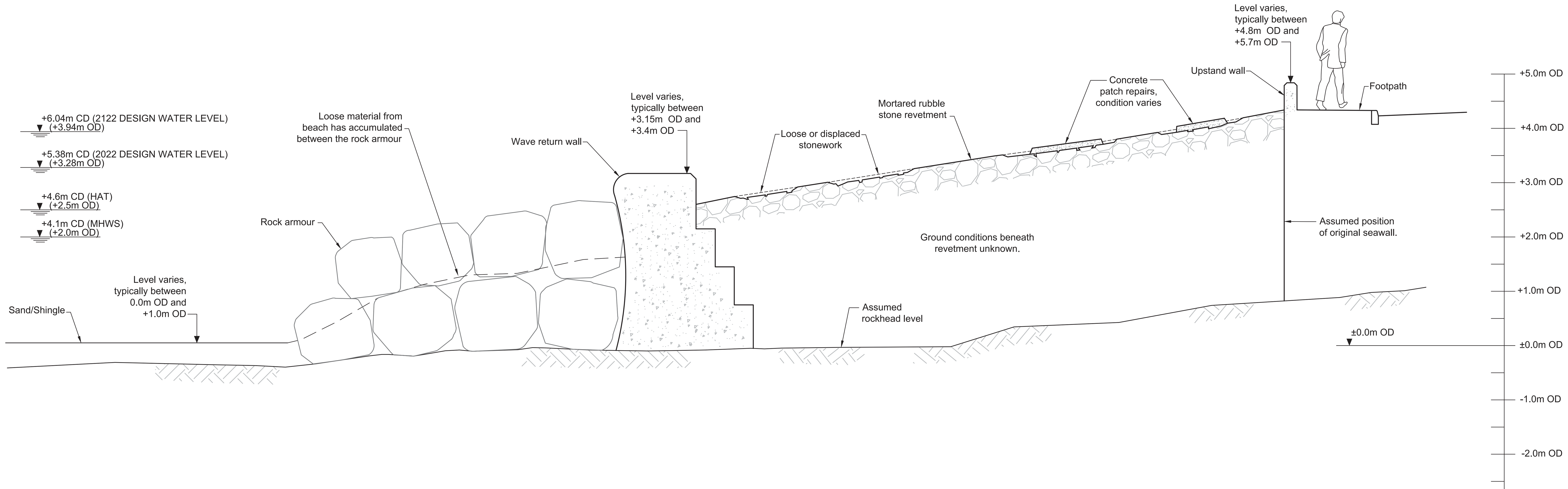
Appendix C. Drawings

Drawings included:

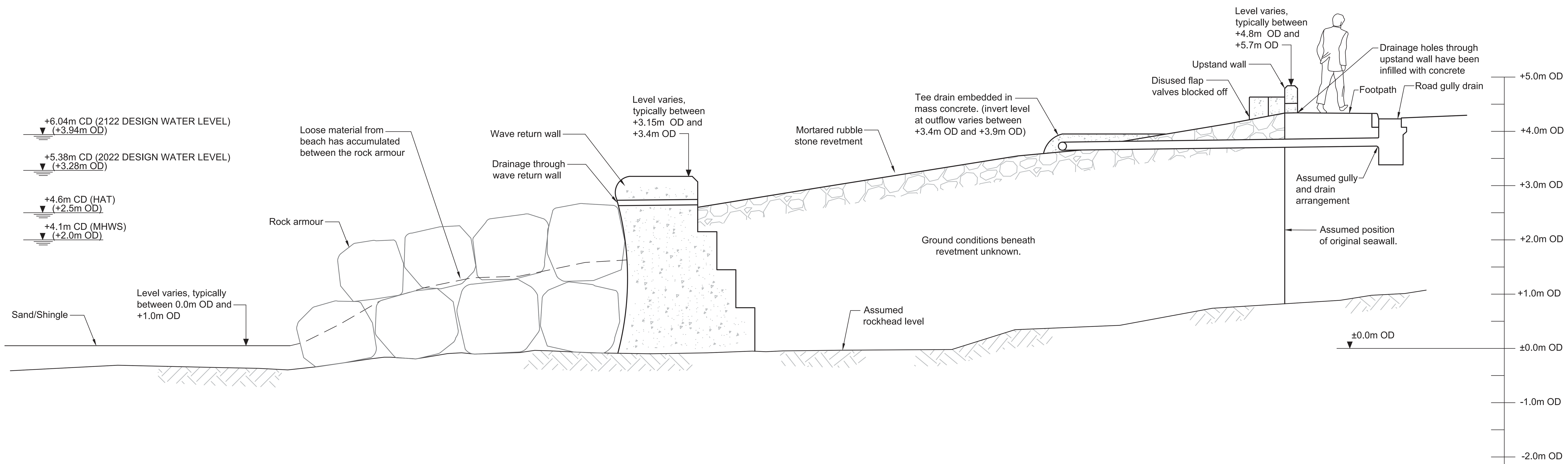
- ND800401_001 Rev P02 TYPICAL EXISTING ARRANGEMENT OF SEA DEFENCES
- ND800401_002 Rev P02 OPTION 1
- ND800401_003 Rev P02 OPTION 2
- ND800401_004 Rev P02 OPTION 3
- ND800401_005 Rev P01 OPTION 4
- ND800401_006 Rev P01 OPTION 5

Notes :

1. 0.0m Ordnance Datum is equal to +2.1m Chart Datum
2. Arrangements shown are indicative of typical observations made on site. Layout details have been assumed where site conditions are unknown.
3. Results of a topographical survey carried out on 8/03/13 by Property & Land Surveys (Highland) Ltd. have been used to develop this typical cross section. Reference should be made to drawing 853101 for details of topography of the site.
4. The design water levels are based on 37 years hindcast data and corrected for climate change, as per ABP Mer report "Portgordon Wave and Overtopping Modelling" June 2017.



Cross Section
Showing Typical Arrangement of Existing Sea Defences
(Scale - 1:50)



Cross Section
Showing Typical Arrangement of Existing Drainage
(Scale - 1:50)

Rev	Rev. Date	Purpose of revision	Drawn	Checkd	Rev'd	Apprv'd
P02	FEB. 2017	FOR INFORMATION	KMA	AAG	CEP	JAS
0	MAY 2017	FOR INFORMATION	AE	KS		

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Client
THE MORAY COUNCIL

Project
PORTGORDON SEA DEFENCES

Drawing title
TYPICAL EXISTING ARRANGEMENT OF SEA DEFENCES

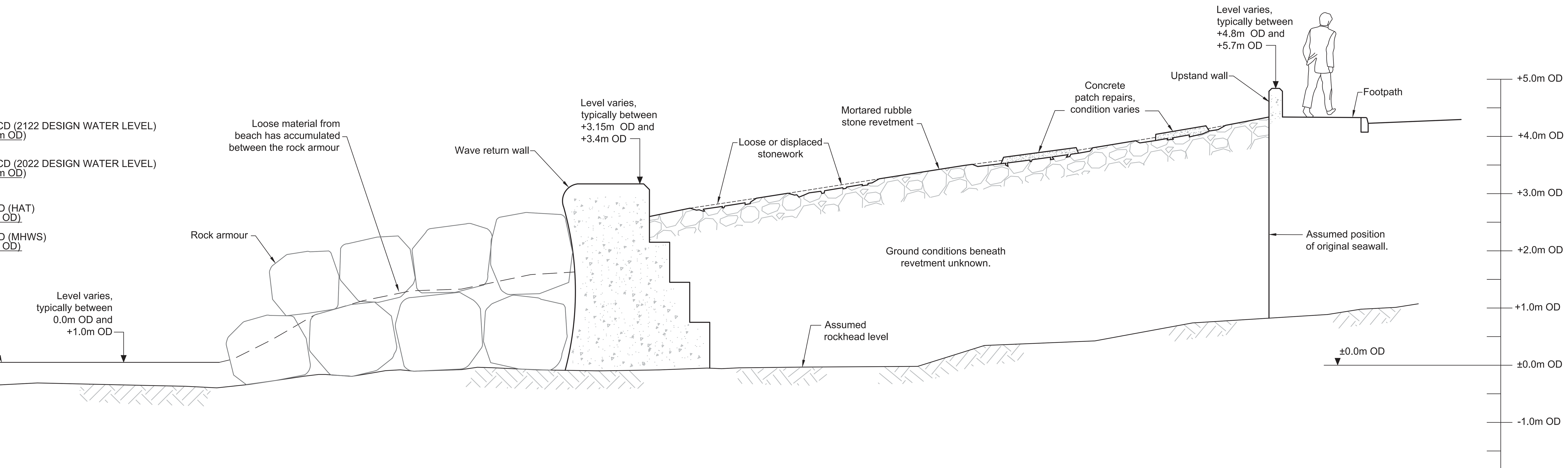
Drawing status
FOR INFORMATION

Scale	1:50 @ A1	DO NOT SCALE
Jacobs No.	ND800401	
Client no.		

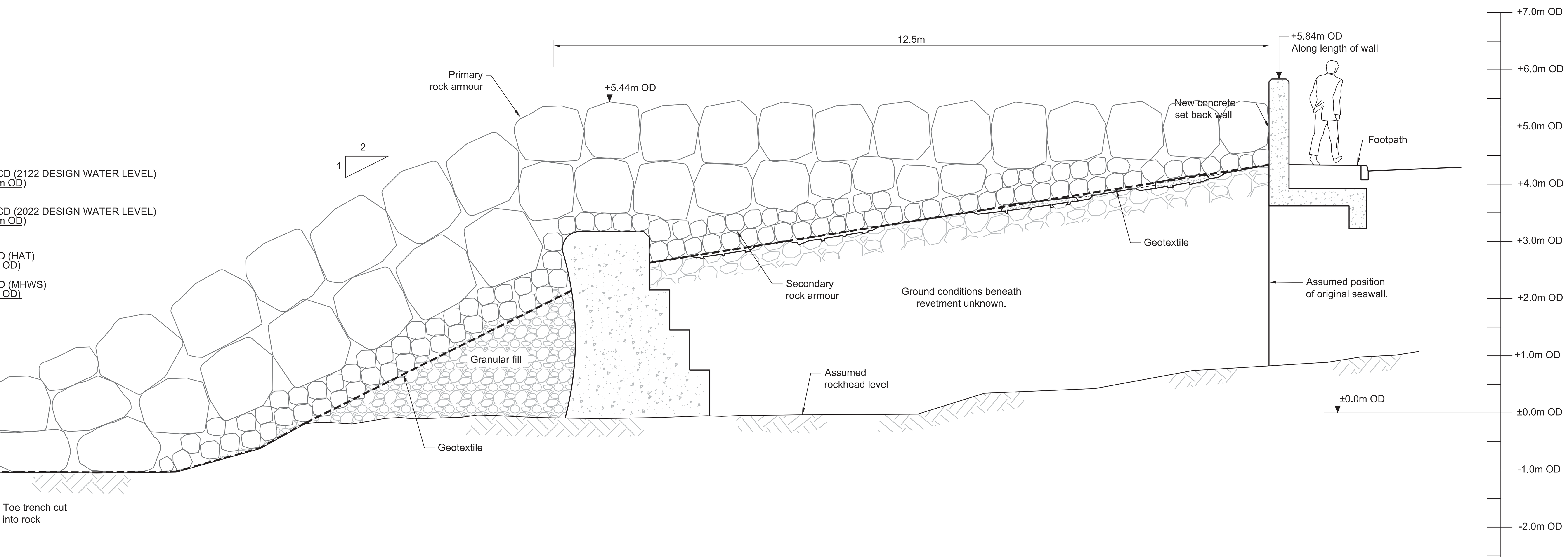
Drawing number: ND800401_001
Rev: P02

Notes :

1. 0.0m Ordnance Datum is equal to +2.1m Chart Datum
2. Arrangements shown are indicative of typical observations made on site. Layout details have been assumed where site conditions are unknown.
3. Results of a topographical survey carried out on 8/03/13 by Property & Land Surveys (Highland) Ltd. have been used to develop this typical cross section. Reference should be made to drawing 853101 for details of topography of the site.
4. The design water levels are based on 37 years hindcast data and corrected for climate change, as per ABP Mer report "Portgordon Wave and Overtopping Modelling" June 2017.



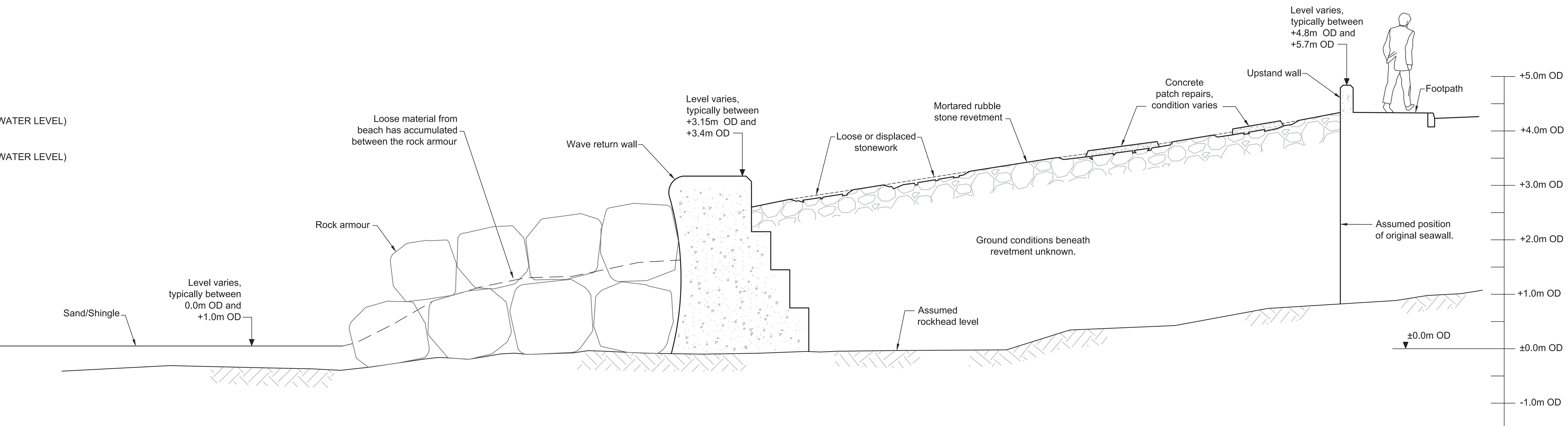
Cross Section
Showing Typical Arrangement of Existing Sea Defences
(Scale - 1:50)



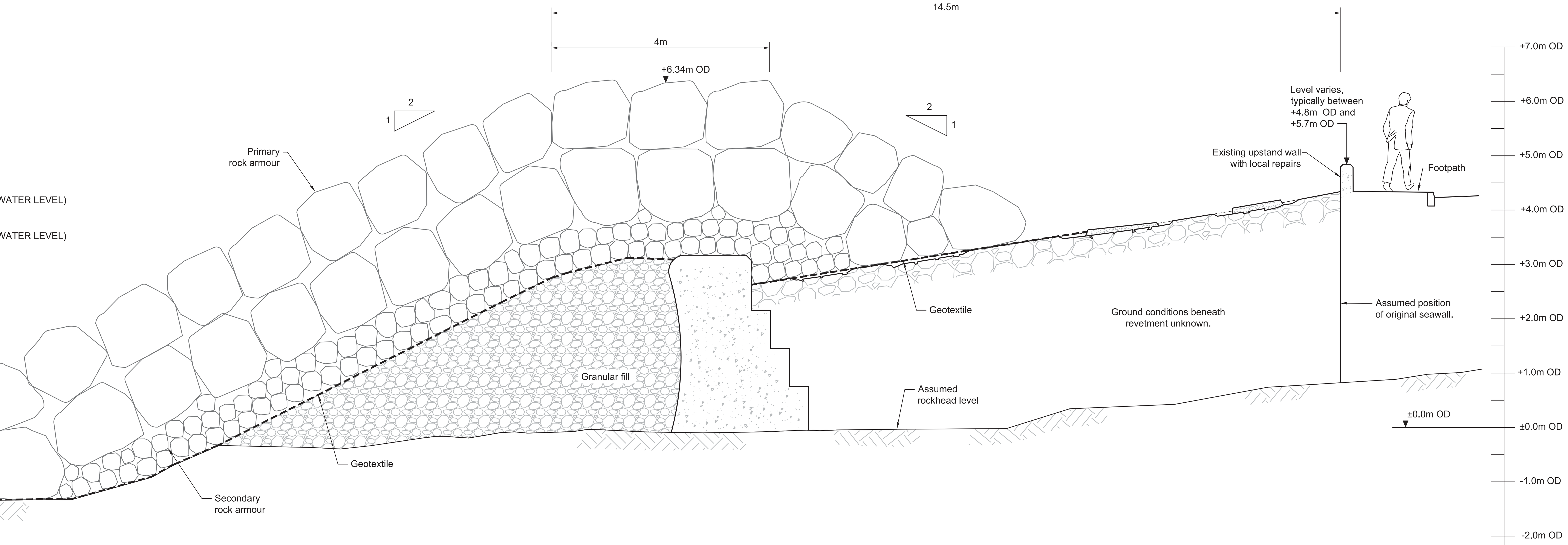
Cross Section
Showing Typical Arrangement of Proposals for Option 1
(Scale - 1:50)

Notes :

1. 0.0m Ordnance Datum is equal to +2.1m Chart Datum
2. Arrangements shown are indicative of typical observations made on site. Layout details have been assumed where site conditions are unknown.
3. Results of a topographical survey carried out on 8/03/13 by Property & Land Surveys (Highland) Ltd. have been used to develop this typical cross section. Reference should be made to drawing 853101 for details of topography of the site.
4. The design water levels are based on 37 years hindcast data and corrected for climate change, as per ABP Mer report "Portgordon Wave and Overtopping Modelling" June 2017.



Cross Section
Showing Typical Arrangement of Existing Sea Defences
(Scale - 1:50)



Cross Section
Showing Typical Arrangement of Proposals for Option 2
(Scale - 1:50)

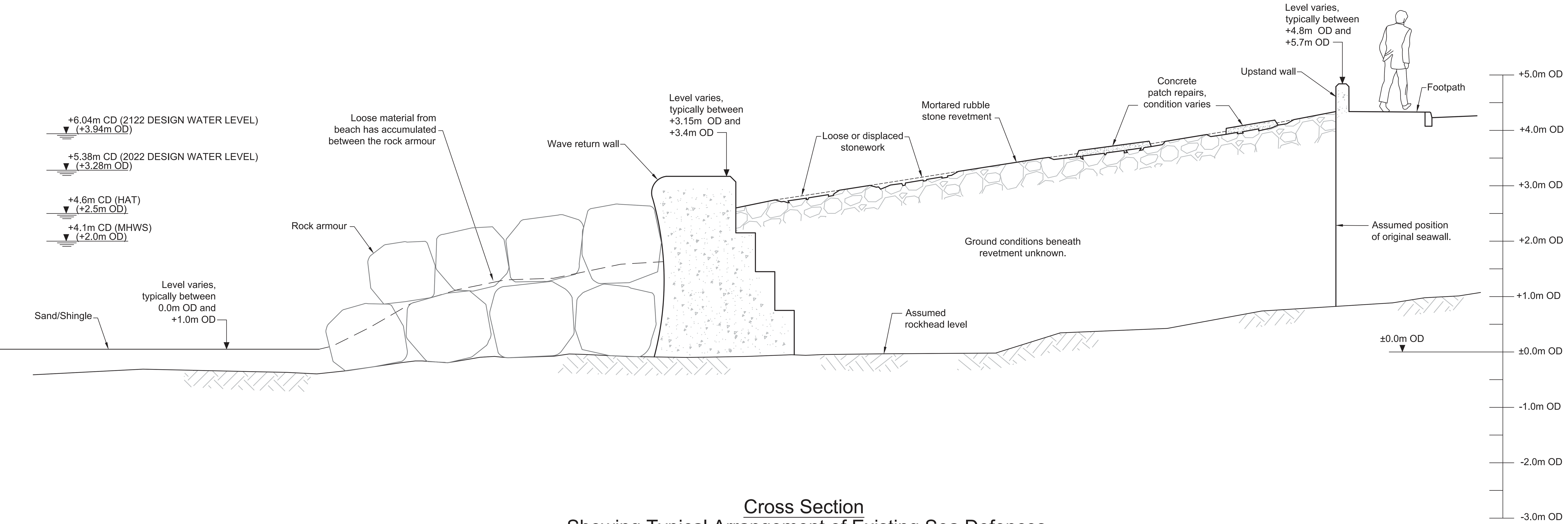
Notes :

1. 0.0m Ordnance Datum is equal to +2.1m Chart Datum

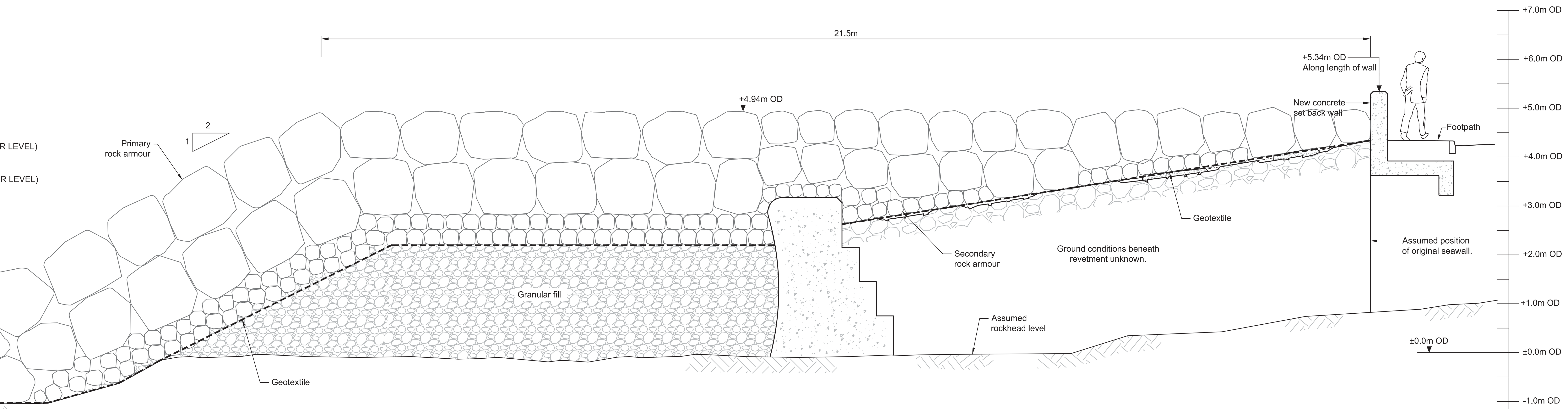
2. Arrangements shown are indicative of typical observations made on site. Layout details have been assumed where site conditions are unknown.

3. Results of a topographical survey carried out on 8/03/13 by Property & Land Surveys (Highland) Ltd. have been used to develop this typical cross section. Reference should be made to drawing 853101 for details of topography of the site.

4. The design water levels are based on 37 years hindcast data and corrected for climate change, as per ABP Mer report "Portgordon Wave and Overtopping Modelling" June 2017.



Cross Section
Showing Typical Arrangement of Existing Sea Defences
(Scale - 1:50)



Cross Section
Showing Typical Arrangement of Proposals for Option 3
(Scale - 1:50)

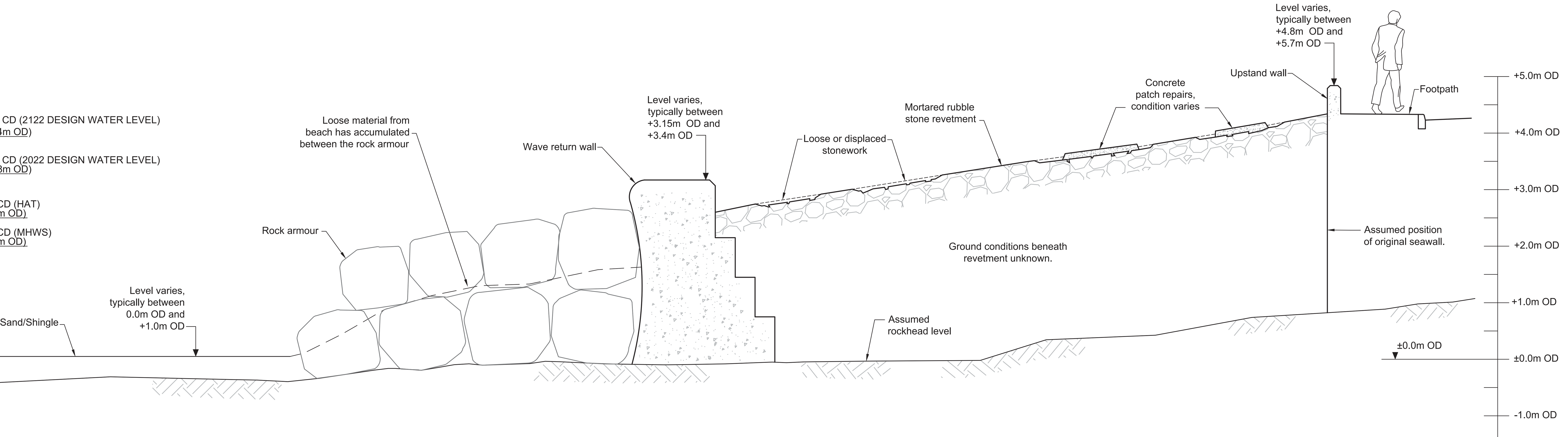
Notes :

1. 0.0m Ordnance Datum is equal to +2.1m Chart Datum

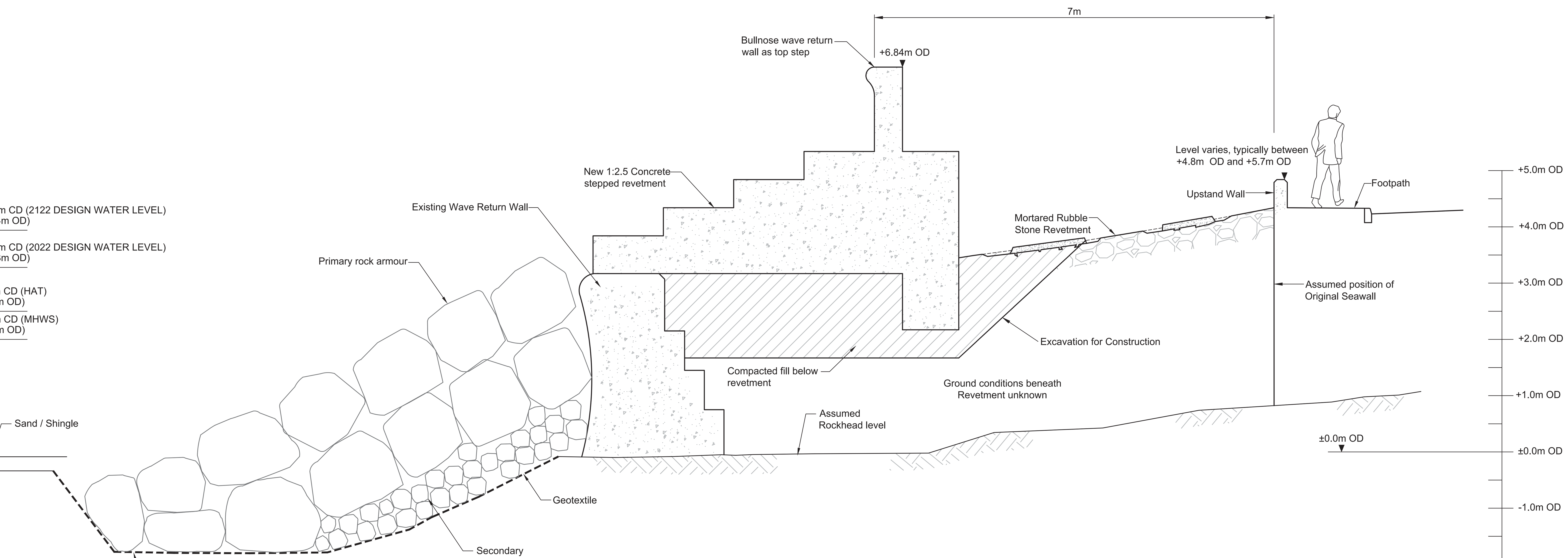
2. Arrangements shown are indicative of typical observations made on site. Layout details have been assumed where site conditions are unknown.

3. Results of a topographical survey carried out on 8/03/13 by Property & Land Surveys (Highland) Ltd. have been used to develop this typical cross section. Reference should be made to drawing 853101 for details of topography of the site.

4. The design water levels are based on 37 years hindcast data and corrected for climate change, as per ABP Mer report "Portgordon Wave and Overtopping Modelling" June 2017.



Cross Section
Showing Typical Arrangement of Existing Sea Defences
(Scale - 1:50)



Cross Section
Showing Typical Arrangement of Proposals for Option 4

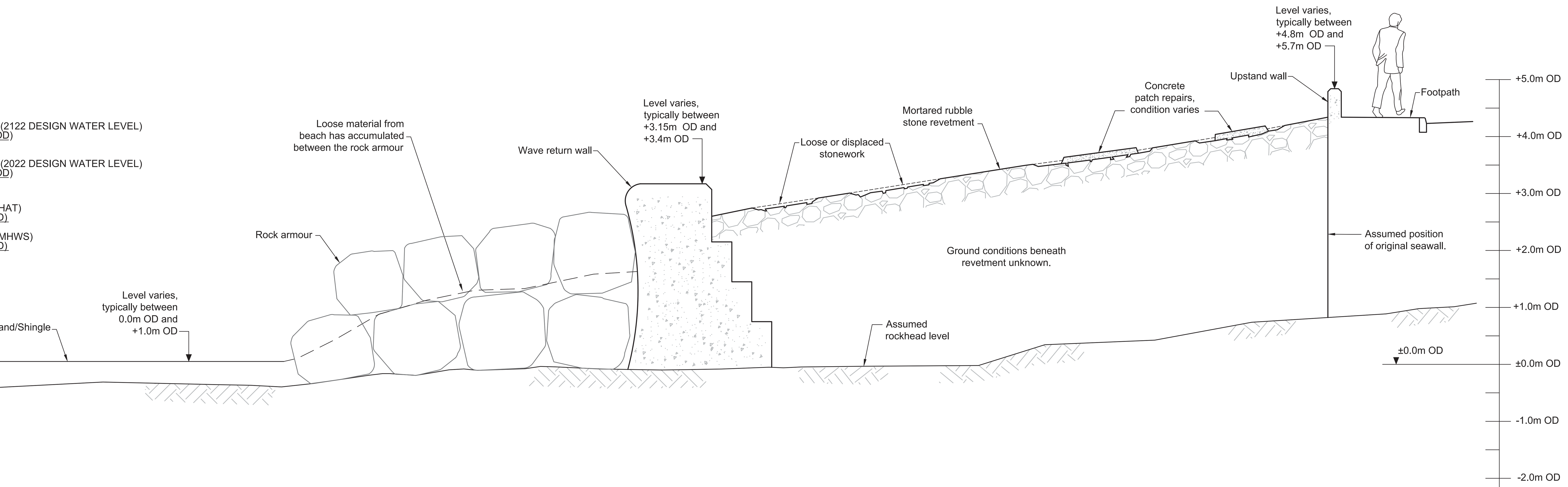
Notes :

1. 0.0m Ordnance Datum is equal to +2.1m Chart Datum

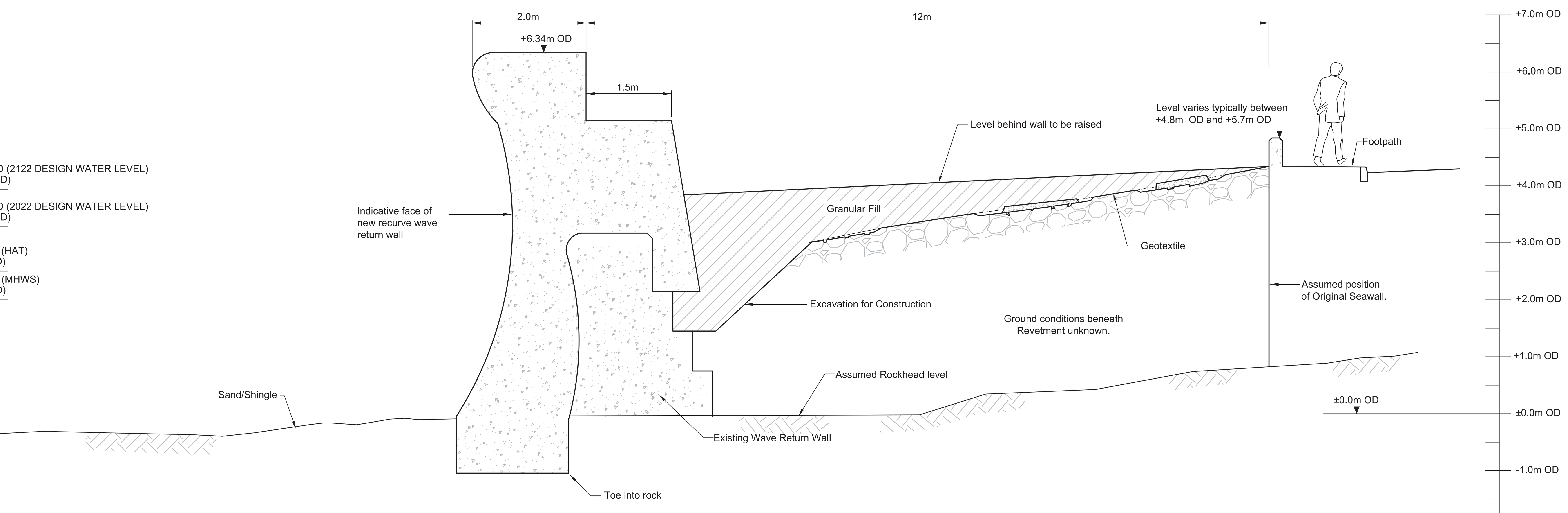
2. Arrangements shown are indicative of typical observations made on site. Layout details have been assumed where site conditions are unknown.

3. Results of a topographical survey carried out on 8/03/13 by Property & Land Surveys (Highland) Ltd. have been used to develop this typical cross section. Reference should be made to drawing 853101 for details of topography of the site.

4. The design water levels are based on 37 years hindcast data and corrected for climate change, as per ABP Mer report "Portgordon Wave and Overtopping Modelling" June 2017.



Cross Section
Showing Typical Arrangement of Existing Sea Defences
(Scale - 1:50)



Cross Section
Showing Typical Arrangement of Proposals for Option 5
(Scale - 1:50)

Appendix D. Drainage Model Verification

This section discusses the verification process carried out on the hydraulic drainage model used to produce predict the extent and depth of flood water, refer to Section 5 for further details.

D.1 Model Performance

Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model convergence was achieved. Convergence refers to the ability of the modelling software to arrive at a solution for which the variation of the found solution between successive iterations is either zero or negligibly small and lies within a pre-specified tolerance limits.

The mass balance error outputs from the Tuflow 2D model have also been checked. The acceptable mass balance error range is +/- 1% of the total flood volume. The mass error for all the scenarios modelled is considered good with the peak cumulative mass errors all less than 1%. The change in volume (dVol) throughout the model simulation has also been checked and has been found to vary in line with the inflow boundaries, which is an indicator of good convergence of the 2D model.

Figure D-1 shows that for a 200-year event in year 2122 modelled with the Do Minimum scenario, the cumulative mass error is all less than 1% in absolute value which is the tolerance. This mass error diagnostic is typical for all the events modelled.

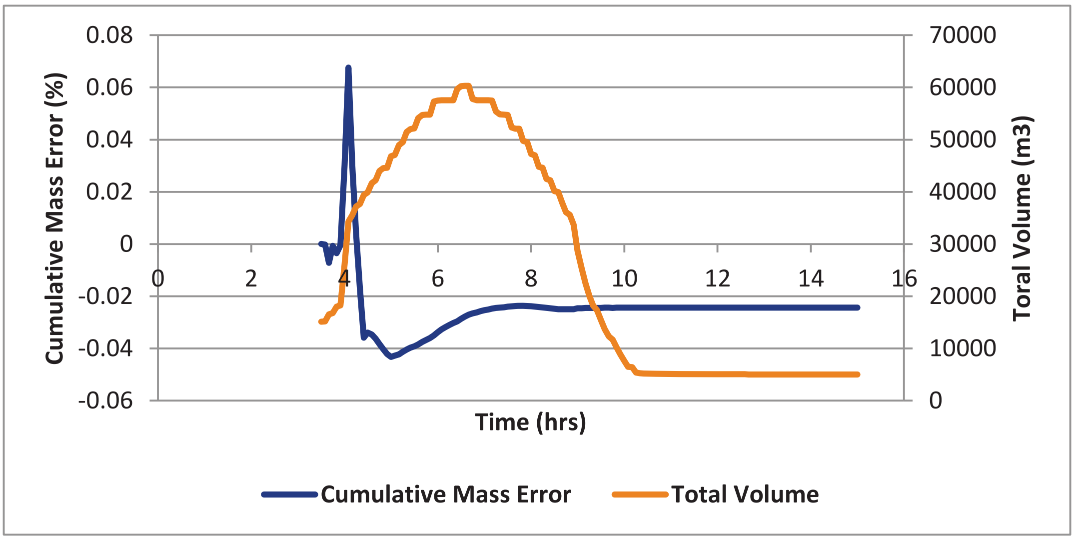


Figure D-1: 2D cumulative mass error and change in volume – Do Minimum, 2122 epoch.

D.2 Model Calibration

The model has not been quantitatively calibrated as no associated data is available to carry this out. Model performance has been checked as well as the consistency of model results between different scenarios simulated.

Appendix E. BCA Calculation

Costs and benefits of options							
	Costs and benefits £k						
Option number	**Do Nothing**	**Do minimum**	**Rock Upper Slope**	**High Rock Armour**	**Rock Armour Bern Extended Seaward**	**Stepped Revetment**	**Wave Return Wall**
Option name	Do-nothing	Do minimum	Rock Armour Berm Upper Slope	High Rock Armour	Rock Armour Bern Extended Seaward	Stepped Revetment	Wave Return Wall
AEP or SoP (where relevant)	Overtopping 1:10	Overtopping 1:10	Designed 1:100	Designed 1:100	Designed 1:200	Designed 1:200	Designed 1:200
COSTS:							
PV capital costs	0	0	11,133	10,600	16,131	11,889	16,407
PV operation and maintenance costs	0	307	344	295	454	391	291
PV other	0	0	0	0	0	0	0
Optimism bias adjustment	0	184	6,886	6,537	9,951	7,368	10,019
PV negative costs (e.g. sales)	0	0	0	0	0	0	0
PV contributions							
Total PV Costs £k excluding contributions	0	492	18,363	17,432	26,536	19,647	26,716
Total PV Costs £k taking contributions into account	0	492	18,363	17,432	26,536	19,647	26,716
BENEFITS:							
PV monetised flood damages	10,186	9,030	752	122	570	200	247
PV EcMap Damage	7,207	6,931	681	0	338		
PV Total Damge	17,393	15,961	1,433	122	908	200	247
PV monetised flood damages avoided		1,431	15,959	17,271	16,485	17,192	17,146
PV monetised erosion damages	0	0	0	0	0	0	0
PV monetised erosion damages avoided (protected)		0	0	0	0	0	0
Total monetised PV damages £k	17,393	15,961	1,433	122	908	200	247
Total monetised PV benefits £k		1,431	15,959	17,271	16,485	17,192	17,146
PV damages (from scoring and weighting)							
PV damages avoided/benefits (from scoring and weighting)							
PV benefits from ecosystem services							
Total PV damages £k	17,393	15,961	1,433	122	908	200	247
Total PV benefits £k		1,431	15,959	17,271	16,485	17,192	17,146

Figure E-1 BCA Calculation

Appendix F. Risk to Life Calculation

Risk to Life Calculation

	Scenario													
	example	**Do minimum 1/1**	1/50	1/100	1/200	**Do nothing 1/10**	1/50	1/100	1/200	Option 1	Option 2	Option 3	Option 4	Option 5
Depth (m)	3	0.170	0.22	0.24	0.25	0.190	0.23	0.25	0.27	0.22	0	0.08	0	0
Velocity (m/sec)	2	1.575	1.575	1.575	1.575	1.575	1.575	1.575	1.575	1.575	1.575	1.575	1.575	1.575
debris factor (DF)	1	1.000	1	1	1	1	1	1	1	1	1	1	1	1
Flood Hazard Rating (HR)	**8.5**	**1.35275**	**1.4565**	**1.498**	**1.51875**	**1.39425**	**1.47725**	**1.51875**	**1.56025**	**1.4565**	**1**	**1.166**	**1**	**1**
Flood warning	2.15	2.15	2.15	2.15	2.15	2.15	2.15	2.15	2.15	2.15	2.15	2.15	2.15	2.15
speed of onset	3	2	2	2	2	2	2	2	2	2	2	2	2	2
nature of area	2	2	2	2	2	2	2	2	2	2	2	2	2	2
Area Vulnerability (AV)	**7.15**	**6.15**	**6.15**	**6.15**	**6.15**	**6.15**	**6.15**	**6.15**	**6.15**	**6.15**	**6.15**	**6.15**	**6.15**	**6.15**
% residents suffering from long-term illne	15%	5.03%	5.03%	5.03%	5.03%	5.03%	5.03%	5.03%	5.03%	5.03%	5.03%	5.03%	5.03%	5.03%
%residents aged 75 or over	10%	9.12%	9.12%	9.12%	9.12%	9.12%	9.12%	9.12%	9.12%	9.12%	9.12%	9.12%	9.12%	9.12%
People Vulnerability (Y)	**25%**	**14%**	**14%**	**14%**	**14%**	**14%**	**14%**	**14%**	**14%**	**14%**	**14%**	**14%**	**14%**	**14%**
N(Z) is the population within the flooding zone **	10	56	61	66	68	59	64	68	69	63	-	39	-	-
Flood Hazard Rating	8.5	1.35275	1.4565	1.498	1.51875	1.39425	1.47725	1.51875	1.56025	1.4565	1	1.166	1	1
Area Vulnerability	7.15	6.15	6.15	6.15	6.15	6.15	6.15	6.15	6.15	6.15	6.15	6.15	6.15	6.15
Value of X Population exposed to risk	0.60775	0.083194125	0.08957475	0.092127	0.093403125	0.085746375	0.090850875	0.093403125	0.095955375	0.08957475	0.0615	0.071709	0.0615	0.0615
People Vulnerability	0.25	0.141469762	0.141469762	0.141469762	0.141469762	0.141469762	0.141469762	0.141469762	0.141469762	0.141469762	0.141469762	0.141469762	0.141469762	0.141469762
N(ZE)	6	5	5	6	6	5	6	6	6.6	5.7	0.0	2.8	0.0	0.0
N# of Injuries	**3.04**	**1.33**	**1.54**	**1.71**	**1.79**	**1.42**	**1.66**	**1.79**	**1.87**	**1.60**	**0.00**	**0.79**	**0.00**	**0.00**
N# of Deaths	**1.033175**	**0.126834**	**0.159038**	**0.180926**	**0.192499**	**0.140234**	**0.172862**	**0.192499**	**0.206606**	**0.165039**	**0.000000**	**0.065385**	**0.000000**	**0.000000**
Total cost of fatality (uflited to 2022)		£ 222,133.12	£ 278,534.51	£ 316,869.65	£ 337,137.26	£ 245,602.98	£ 302,745.84	£ 337,137.26	£ 361,844.34	£ 289,045.24	£ -	£ 114,513.89	£ -	£ -
Total cost of injury (uplifted to 2022)		£ 184,710.11	£ 215,111.39	£ 237,937.95	£ 249,698.17	£ 198,147.17	£ 230,525.56	£ 249,698.17	£ 260,869.01	£ 223,228.80	£ -	£ 110,472.59	£ -	£ -
Total		**£ 406,843.24**	**£ 493,645.90**	**£ 554,807.60**	**£ 586,835.42**	**£ 443,750.16**	**£ 533,271.39**	**£ 586,835.42**	**£ 622,713.36**	**£ 512,274.04**	**£ -**	**£ 224,986.48**	**£ -**	**£ -**
Cost per fatality (2022) prices	1,751,375.23													
Cost per injury (2022) prices	139,254.90													

Figure F-1 Risk to Life Calculation

Appendix G. Road Diversion Calculation (Social)

METHOD 2: THE DIVERSION-VALUE METHOD

The simplest way of applying the above equation is to assume that cars will be diverted on to neighbouring roads and therefore the distance that they travel will increase; however their speed will be unaffected. For example, suppose that 15,000 vehicles travel through the local network each hour and will have to travel on average 2 kilometres further but their average speed (40 kph) will not be reduced. In this scenario, the cost of that flood event will be equal to 15,000 * 0.40[1] * 2 for each hour of the disruption due to flooding. If the flood lasts six hours, the costs of traffic disruption amounts to £72,000. In this instance, the figure is small and therefore it is disproportionate to refine this value further using more sophisticated modelling.

Flood hours 8

	Do nothing	Do minimum	Option 1	Option 2	Option 3	Option 4	Option 5
Vehicles per hour (C Road - not built up)	150	150	150	150	150	150	150
Speed travelled (km/h)	40	40	40	40	40	40	40
Resource cost	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Extra distance travelled	8.7	8.7	8.7	1.1	8.7	1.1	1.1
Cost per hour flood event	522	522	522	66	522	66	66
8 hour flood	£4,176.00	£4,176.00	£4,176.00	£528.00	£4,176.00	£528.00	£528.00
Uplifed to 2022 values	£ 4,610.64	£ 4,610.64	£ 4,610.64	£ 582.95	£ 4,610.64	£ 582.95	£ 582.95

	Do Nothing				Do miminum			
Vehicles per hour (C Road - not built up)	150	150	150	150	150	150	150	150
Speed travelled (km/h)	40	40	40	40	40	40	40	40
Resource cost	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Extra distance travelled	8.7	8.7	8.7	8.7	8.7	8.7	8.7	8.7
Cost per hour flood event	522.00	522.00	522.00	522.00	522.00	522.00	522.00	522.00
8 hour flood	£4,176.00	£4,176.00	£4,176.00	£4,176.00	£4,176.00	£4,176.00	£4,176.00	£4,176.00
Uplifed to 2022 values	£ 4,610.64	£ 4,610.64	£ 4,610.64	£ 4,610.64	£ 4,610.64	£ 4,610.64	£ 4,610.64	£ 4,610.64

Table 6.11 Total resource costs of travel as a function of speed (pence/km) (updated to 2017 values)

Total resource costs (pence per km)								
Speed (km/hr)	5	10	20	40	50	80	100	120
Car average p/km	268	136	71	40	34	22	20	18
LGV average p/km	314	163	87	49	42	32	30	28
OGV1 p/km	345	183	100	57	49	38	-	-
OGV2 p/km	441	239	135	82	70	56	-	-
PSV p/km	1985	1017	533	288	239	-	-	-

Data supplied by the Department for Transport (2012)

This is Table 6.15 in the MCM 2013

Department for Transport (DfT) (2012) UNIT 3.5.6: Values of time and vehicle operating costs. In Transport Analysis Guidance (TAG). Department for Transport: London. Accessible at http://www.dft.gov.uk/webtag/. Accessed 26 March 2013.

Table 10.1: Normal Traffic Conditions - Urban areas	
Road Type	Number of Vehicles per Hour
A Road (built-up)	1,600
A Road (not built-up)	1,100
B Road (built-up)	500
B Road (not built-up)	400
C Road (built-up)	300
C Road (not built-up)	200
Unclassified (built-up)	150
Unclassified (not built-up)	50

Source: Table 5.2, page 5-24, part 5.

Figure G-1 Road Diversion Calculation (Social)