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Aberaeron Coastal Defence Scheme

Options Assessment – Numerical Modelling Report

Ceredigion County Council

23 March 2020

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This document has 37 pages including the cover.

Document history

Revision	Purpose description	Originated	Checked	Reviewed	Authorised	Date
Rev 1.0	For Information	TM/NA	RGB	AW	RAM	23/03/20

Client signoff

Client	Ceredigion County Council
Project	Aberaeron Coastal Defence Scheme
Job number	5182114
Client signature / date	

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1. Introduction

Ceredigion County Council (CCC) has commissioned Atkins to complete the detailed design of the proposed coastal defence scheme as outlined in MWA (2017) [1] which includes raising the northern harbour wall (along Quay Parade and the Pwll Cam Inner Harbour) and rebuilding the degraded section of the South Pier.

The previous PAR study undertaken by Hyder [3] found that by raising the secondary harbour wall along Quay Parade to a proposed level of +5.2m Ordnance Datum (OD), the return period at which flooding started to occur to any property (year 2011) was at the 1.33% AEP (1 in 75 year) event and only 2 properties were forecast to flood at the 0.5% AEP (1 in 200) event. By 2111, due to increases in sea level, the return period at which flooding occurs reduces to the 100% AEP (1 in 1 year) event. A total of 22 properties were forecast to flood in the 0.5% AEP (1 in 200) event in 2111. The resulting flood damages were used to form the basis of the economic assessment of the proposed scheme.

However, Atkins' review of the Hyder modelling showed wave heights were underestimated near to the harbour entrance and close to the Harbourmaster Hotel in particular. This led to concerns that not only were the flood damages higher than anticipated, but that the resulting overtopping volumes in storm events were a significant risk to public safety. It was accepted that in order to reduce the wave heights and the resulting overtopping to tolerable levels that marine structures would be required at the harbour entrance.

The design approach taken is to re-establish the level of performance of the Hyder PAR study of no flooding in the 1 in 75 year event at the opening year (now 2021) event and no flooding in the 1 in 1 year event in 2121.

This report summarises the numerical modelling that has been undertaken to analyse various proposed layouts of engineering structures that will act to reduce wave heights within Aberaeron harbour. This will identify the most effective option in reducing wave heights to achieve the required overtopping SoP. This document outlines the methodology and results of the modelling.

Figure 1-1 – Aberaeron Harbour Plan

1.1. Site and Scheme Description

Aberaeron is situated on the west coast of Wales in Cardigan Bay, approximately 30km south of Aberystwyth (see Figure 1-2). Much of Aberaeron is centred around the harbour, with the coastal frontage split into North and South Beaches either side of the harbour entrance.

Figure 1-2 – Aberaeron Location

Water depths in the harbour are generally shallow (see Figure 1-3); bed levels are around -0.1mOD within the harbour to roughly -1.5mOD at the harbour entrance. Water depths in the harbour vary between 5.1m to 6.5m at Mean High Water Springs.

Figure 1-3 – Aberaeron Harbour Entrance

1.2. Modelling Aim

The aim of the modelling work is to simulate waves approaching Aberaeron harbour during extreme events and to analyse various layouts of engineering structures that will act to reduce wave heights within Aberaeron harbour.

2. Model Data and Options

2.1. Model Data

Data from several sources has been used to conduct the modelling as listed in Table 2-1.

Table 2-1 – Model Data List

Data provider (year)	Data type	Data Use
Shoreline Surveys (2019)	Multibeam Survey	Definition of Model Bathymetry
Lle Geo-Portal (2014-2016)	LiDAR	Definition of Land Boundary and Model Bathymetry

2.1.1. Bathymetry

Bathymetry used for the model is from a multibeam survey undertaken by Shoreline Surveys in August 2019. Levels were provided in Chart and Ordnance Datum. The modelling work was undertaken in metres above Ordnance Datum. The bathymetry for the model was augmented with LiDAR data to provide a complete digital terrain model (DTM) of the model area.

2.1.2. Offshore Wave Data

A total of 39 years (1980 to 2019) of hindcast wave data at three hours intervals (including wave height, period and direction) was purchased from the UK Met Office at an offshore location (52.334°N, 4.415°W) as illustrated in Figure 2-1. The offshore wave data point is approximately 14km north-west of Aberaeron in approximately 16m of water (from mean sea level).

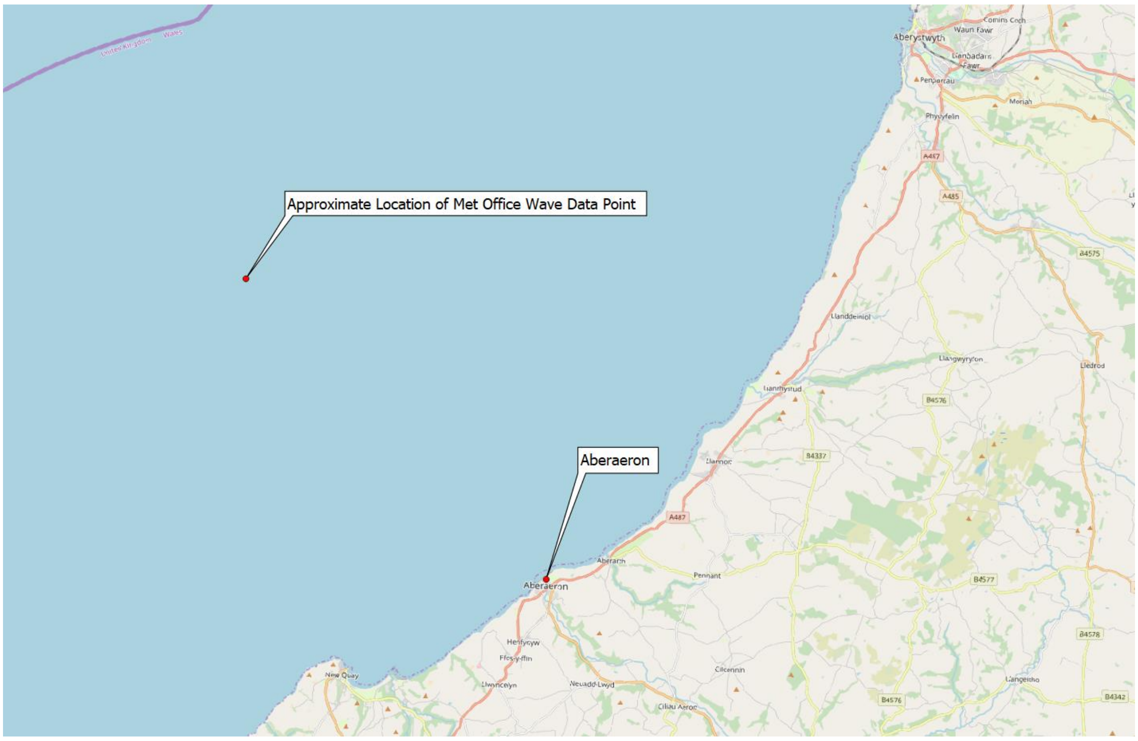


Figure 2-1 – Location of Met Office Wave Data Point

2.1.3. Modelled Options

The existing harbour layout (Do Nothing) plus four different options were modelled to ascertain which was the most effective for reducing wave heights at the north wall of Aberaeron harbour. The structures modelled for each option were:

- Option 0 – (Do Nothing). No additional structures.
- Option 1 – A 170m long and 55m wide breakwater at bed level, with a minimum crest level of +4mOD, approximately 200m from the harbour entrance.
- Option 2 – A 170m long and 55m wide breakwater at bed level, with a minimum crest level of +4mOD, approximately 280m from the harbour entrance.
- Option 3 – A 220m long and 30m wide level breakwater at bed, with a minimum crest level of +4mOD, extending from the existing north pier.
- Option 4 – A 220m long extension to the existing north pier (modelled as land boundary in model with no wave transmission).

The layout of the four structure options within the model is shown in Figure 2-2 and Figure 2-3.

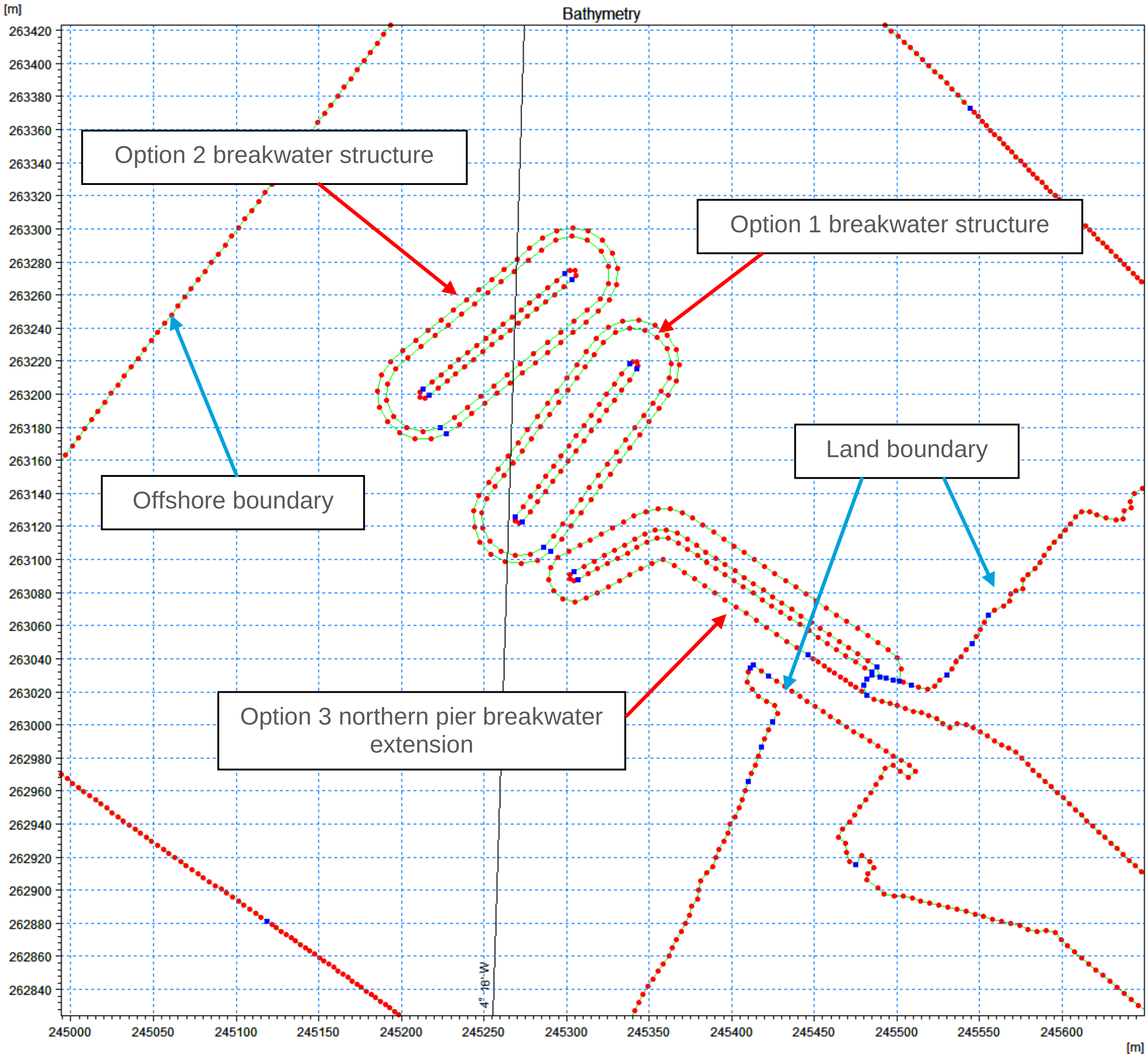


Figure 2-2 – Layout of Options 1, 2 and 3 within the Model

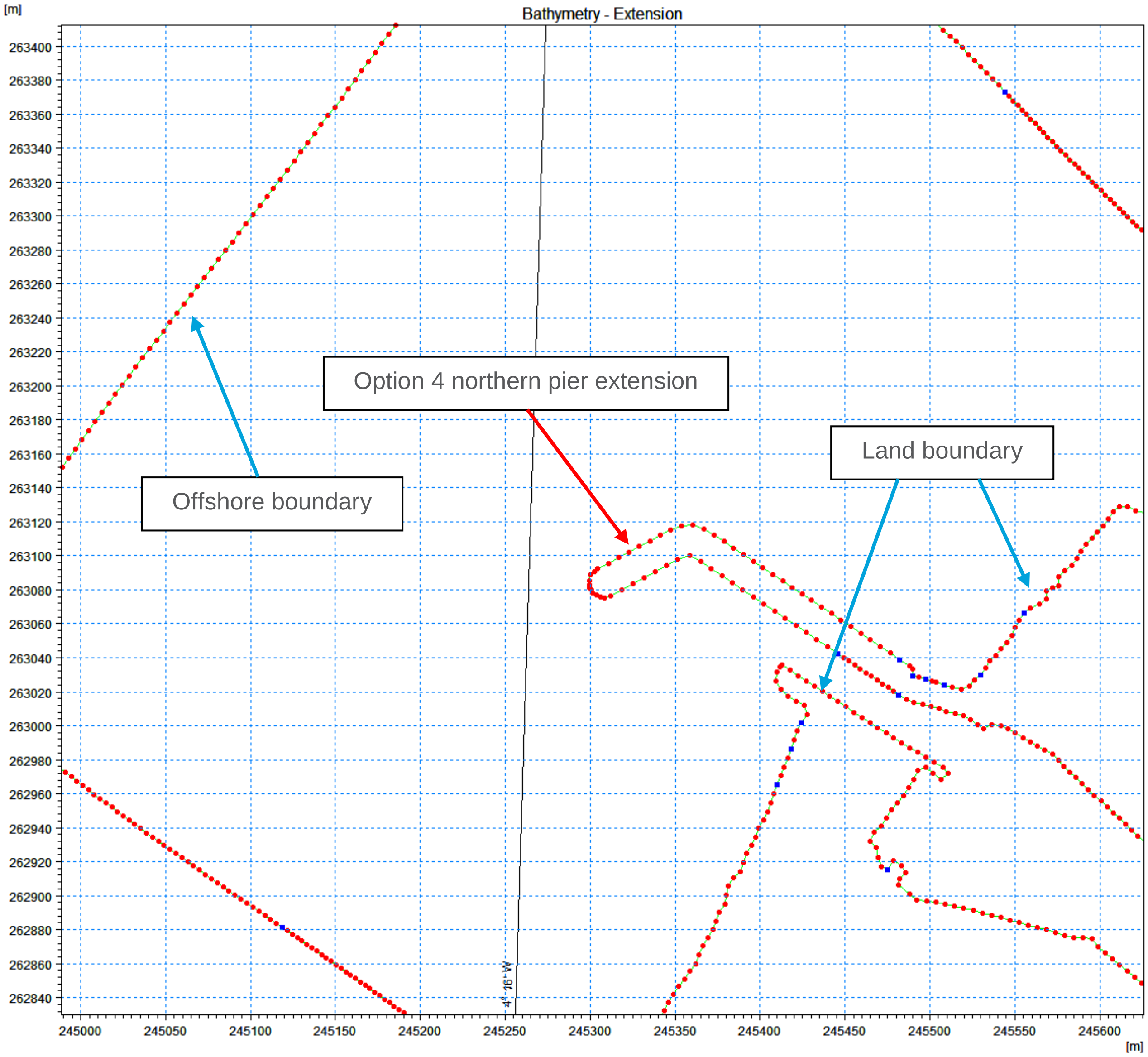


Figure 2-3 – Layout of Option 4 within the Model

3. Wave Model

3.1. Methodology

The following section describes the model set up and the input conditions used for the numerical wave modelling.

3.1.1. Software

The modelling was conducted using MIKE 3 Wave FM. MIKE 3 Wave FM provides simulation tools to model 3D free surface flows. The software is suitable for the work undertaken as it is capable of accurately modelling wave transmission through porous structures in three dimensions and accounts for wave diffraction, refraction and reflection.

The model simulates the growth, decay and transformation of waves generated in the north-west of the domain (see Figure 3-1 below).

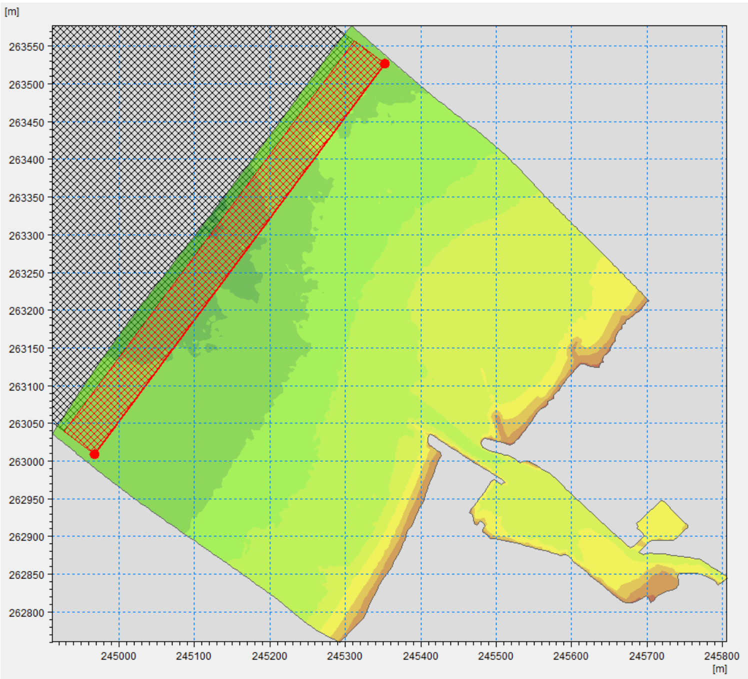


Figure 3-1 – Wave Generation Zone

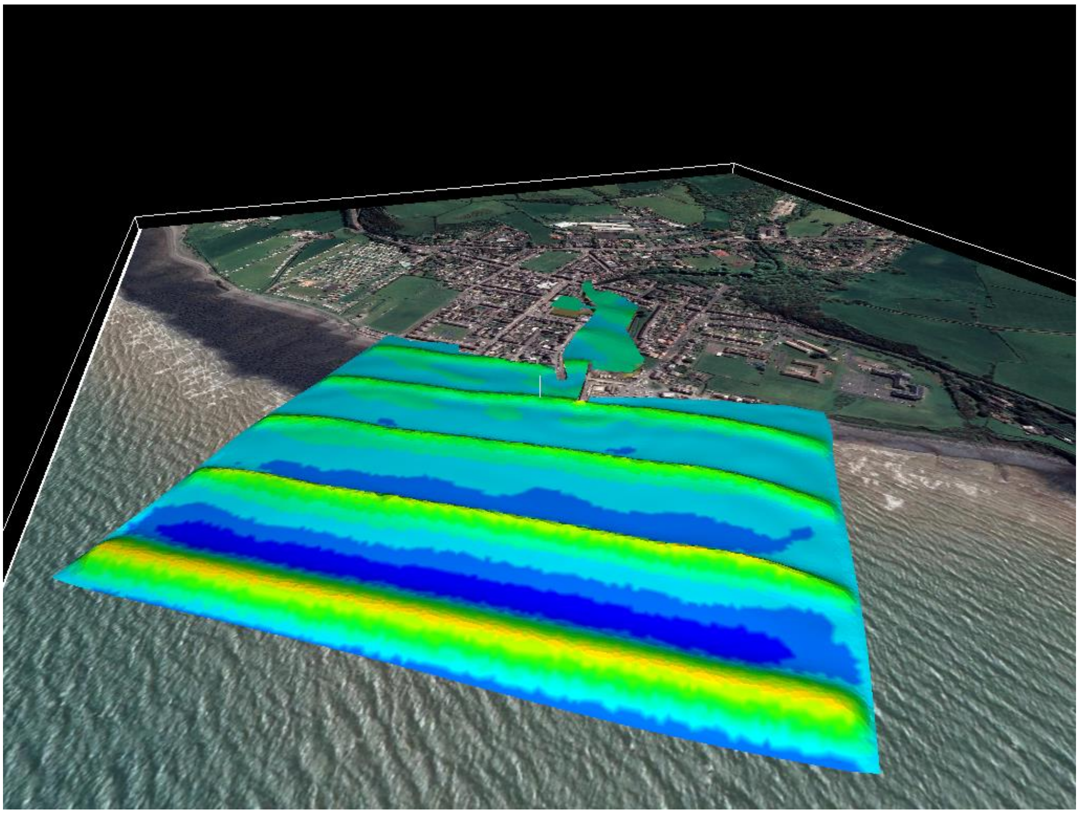


Figure 3-2 – Aberaeron Model as viewed in MIKE Animator Plus

3.1.2. Model Mesh

The model has been constructed using a high-resolution flexible mesh, using a mixture of quadrangular and triangular mesh elements. A flexible mesh has the advantage of specifying areas of interest and increasing resolution in these areas accordingly.

The mesh comprises 25,500 elements, with 13,400 nodes. The resolution of the mesh varies from 75m² in non-significant areas of the harbour to 15m² at the entrance of the harbour. The land boundaries for the beaches and harbour were generated using a shapefile imported from an AutoCAD model of Aberaeron. The boundary was then validated using LiDAR data values.

Bathymetric scatter data was interpolated using the mesh created to generate the model bathymetry. The model was assigned 6 vertical (sigma) layers to create a 3D domain. A total of 6 layers were chosen so that an adequate outline of the breakwater structures could be generated whilst ensuring model runtimes were reasonable. Figure 3-3 below shows the model domain and mesh used for the wave modelling.

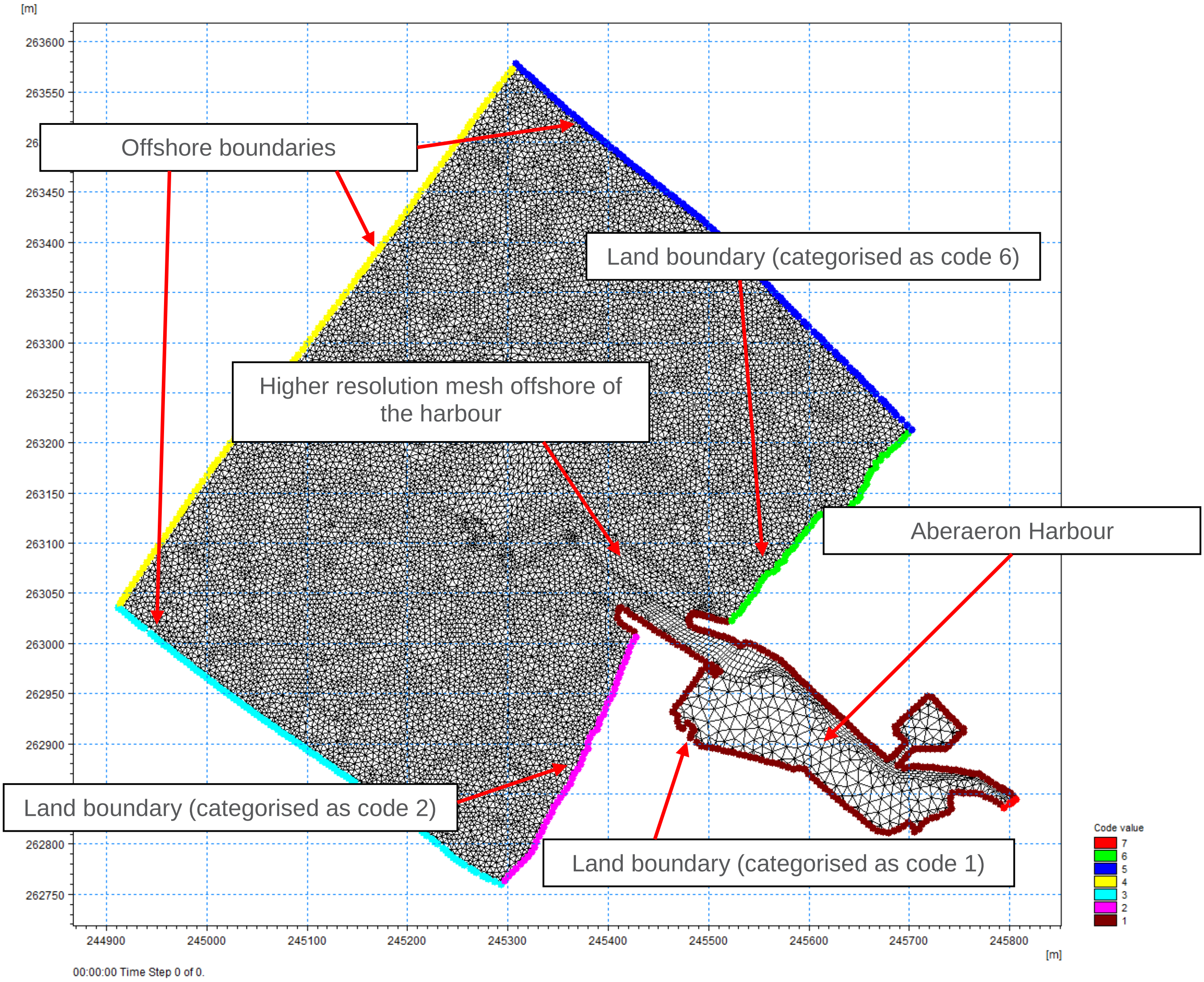


Figure 3-3 – Model Domain and Mesh Used for the Model

The resultant bathymetry over the whole domain is presented in Figure 3-4. The bathymetry near the harbour entrance is presented in Figure 3-5.

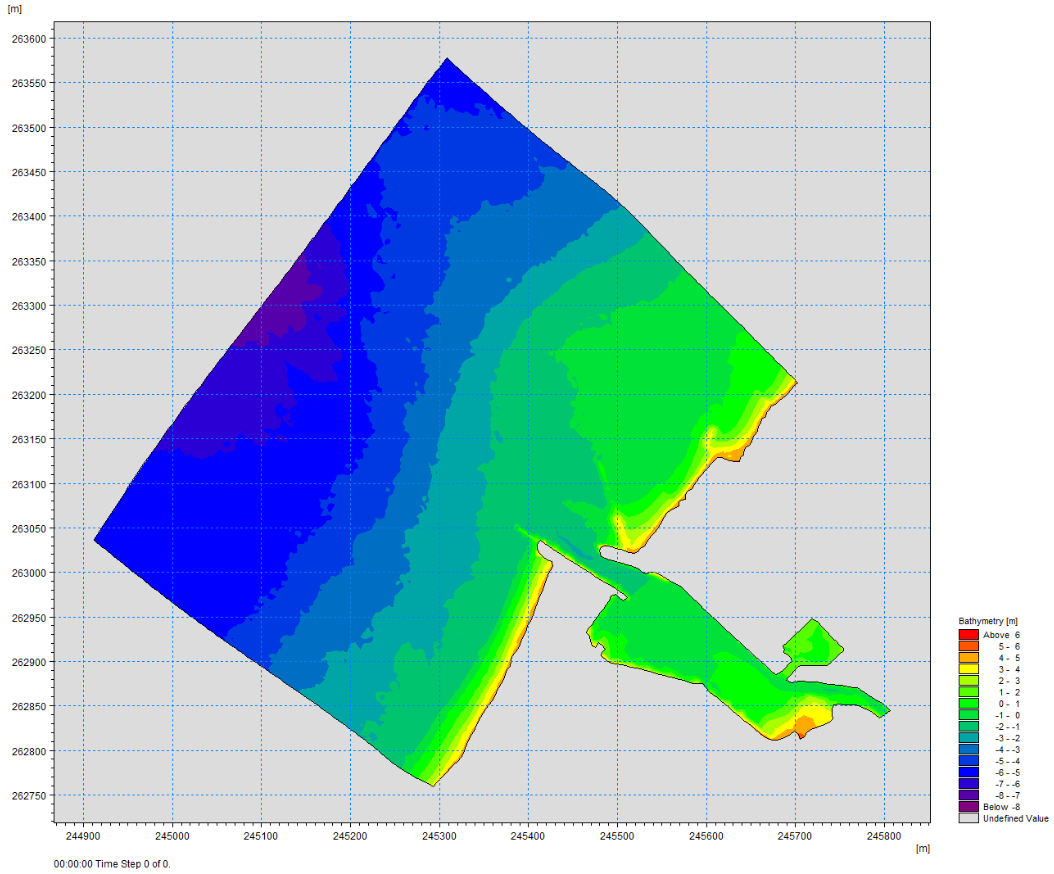


Figure 3-4 – Bathymetry Extent Used for the Model Domain

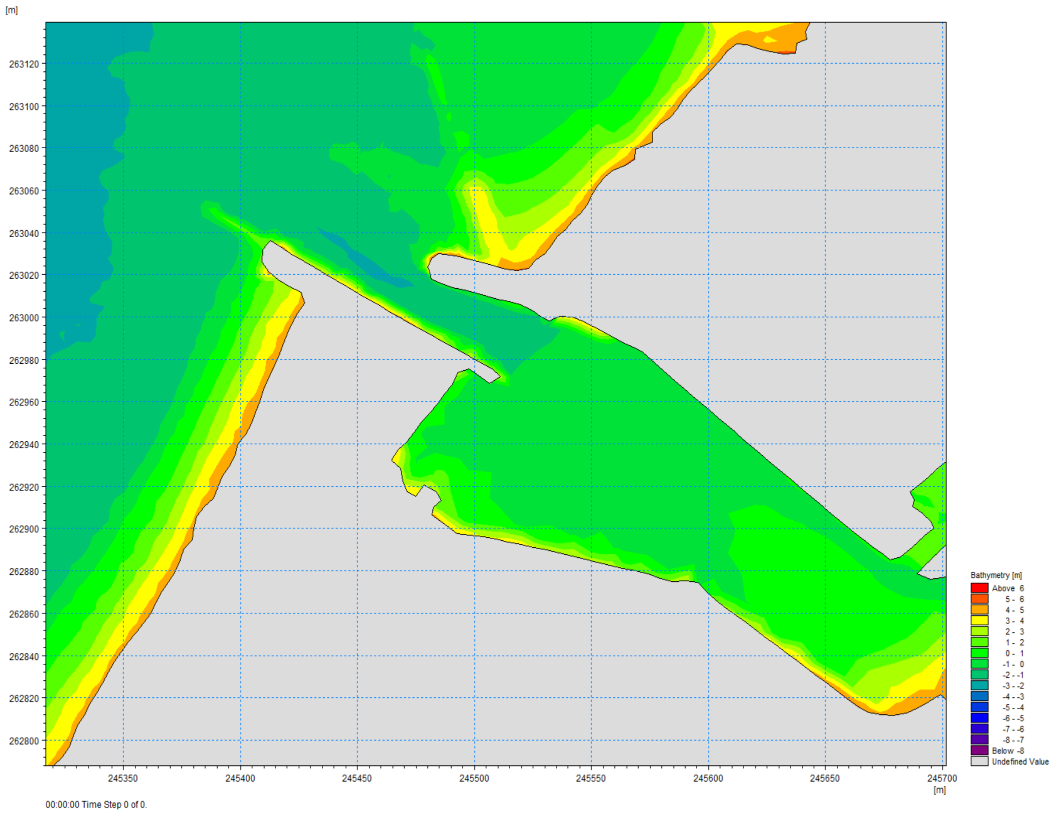


Figure 3-5 – Bathymetry Near the Harbour Entrance

3.1.3. Structures

The effect of the structure options (as outlined in Section 2.1.3) have been included by altering the porosity of the domain. The structures were built up in six sigma layers of equal depth from the seabed to the water level, thereby creating a 3D porosity map of the domain. The porosity of the structures was set to 0.8 as this was deemed suitable for representing the effect of a breakwater built using material of differing stone size. Model option 4 (extended north pier) was created by extending the land boundary (see Figure 2-3) further out to sea along the proposed alignment.

The same mesh was used for modelling options 1-3. However, for option 4 the mesh required some minor alterations to accommodate the pier extension. The 3D porosity map also altered between options to represent the location and shape of the structure being modelled.

3.1.4. Sponge Map

A 2D sponge map (see Figure 3-6) has been created to prevent wave reflections from the model boundaries. The wave generation zone itself (see Figure 3-1) acts as a further sponge layer preventing any reflected waves interfering with the waves generated at the offshore boundary.

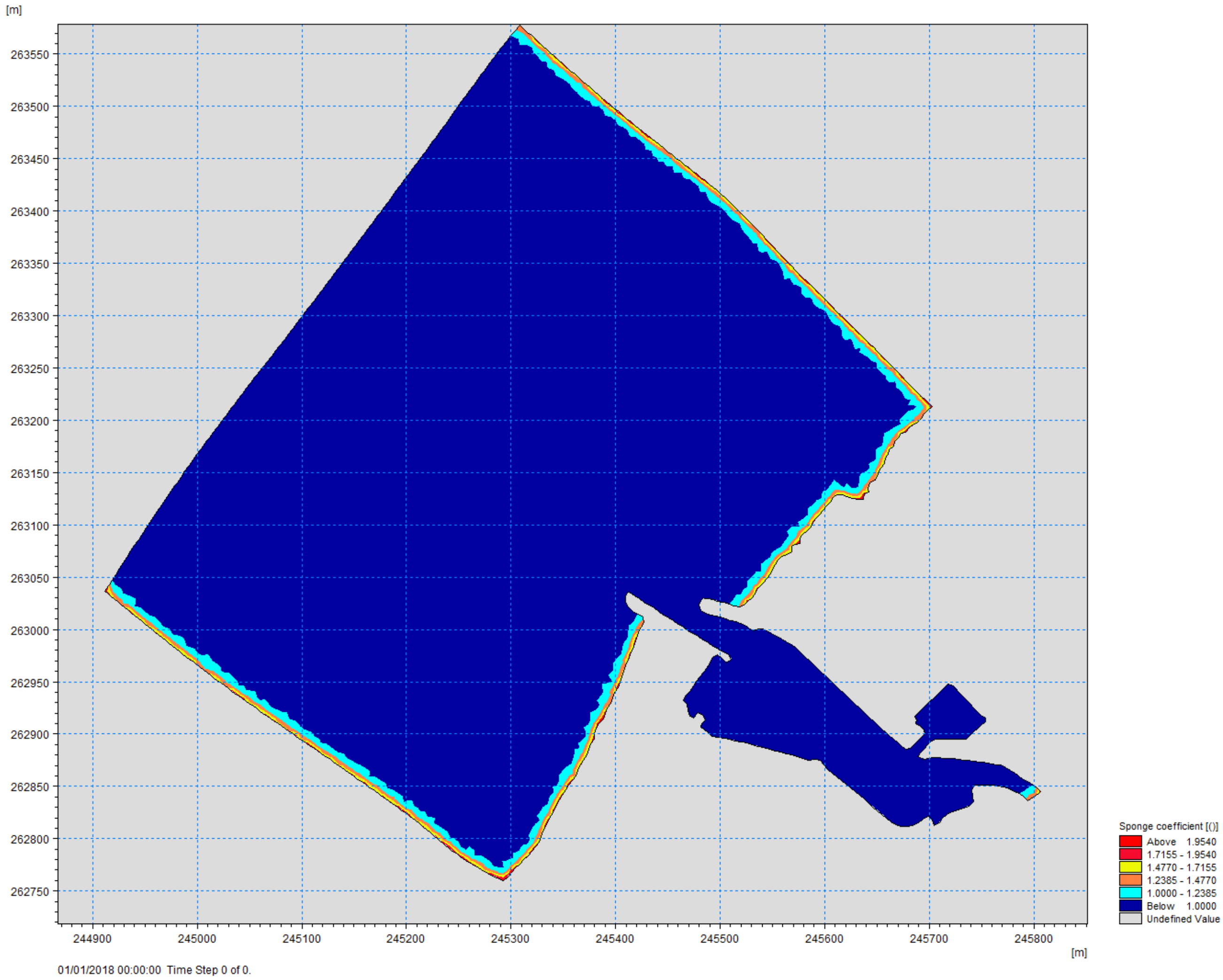


Figure 3-6 – Sponge Layer Map

3.2. Extreme Value Analysis

3.2.1. Aim

To derive design conditions at the offshore data point (see Section 2.1.2), an Extreme Value Analysis (EVA) was conducted. The outputs of the EVA analysis were used in joint probability calculations to determine the boundary conditions for the wave model.

3.2.2. Methodology

EVA was conducted on the offshore wave timeseries data using the MIKE Zero EVA tool. The Partial Duration Series (PDS) method was used to extract extreme values from the data to produce an extreme values series. An average number of events of 1 per year was selected to provide the 39 largest wave heights. Selecting the top 39 events will statistically only include events greater than a 1 in 1 year return period in the extreme value series.

The alternative method Annual Maximum Series (AMS) provides the maximum value in each analysis year. The PDS setting as stated above will give the largest 39 values from the entire data set. Each analysis year might contain more than one event that is within the largest 39 events. Thus, in order to ensure that largest extremes are selected for EVA the PDS method was used.

The 'goodness of fit' of 32 Probability Distribution Estimation (PDE) methods against the five available Plotting Position Methods (PPM) available in MIKE EVA were compared. It was found the Weibull PPM provided the best goodness of fit statistics and was therefore used for the next stage of the analysis (visual inspection of the PDEs against the extreme value series in terms of goodness of fit).

Following this, all 32 PDE using the Weibull PPM only were plotted and based on visual inspection of the graphs were reduced to the best three in terms of goodness of fit in comparison to the extreme value series for final comparison and choice of which PDE shall be used for the EVA output.

3.2.3. Results

Figure 3-7 shows the distribution of extreme significant wave heights. Visually a log normal distribution (lower bound quantile method II), using method of moments, PDE (LN3/MOM/LBQII) was found to be the best fit to the extreme value series (a goodness of fit statistic of 0.99707) and, of the three best fitting distributions, provides the most conservative extremes. Therefore, this was chosen to provide the extremes.

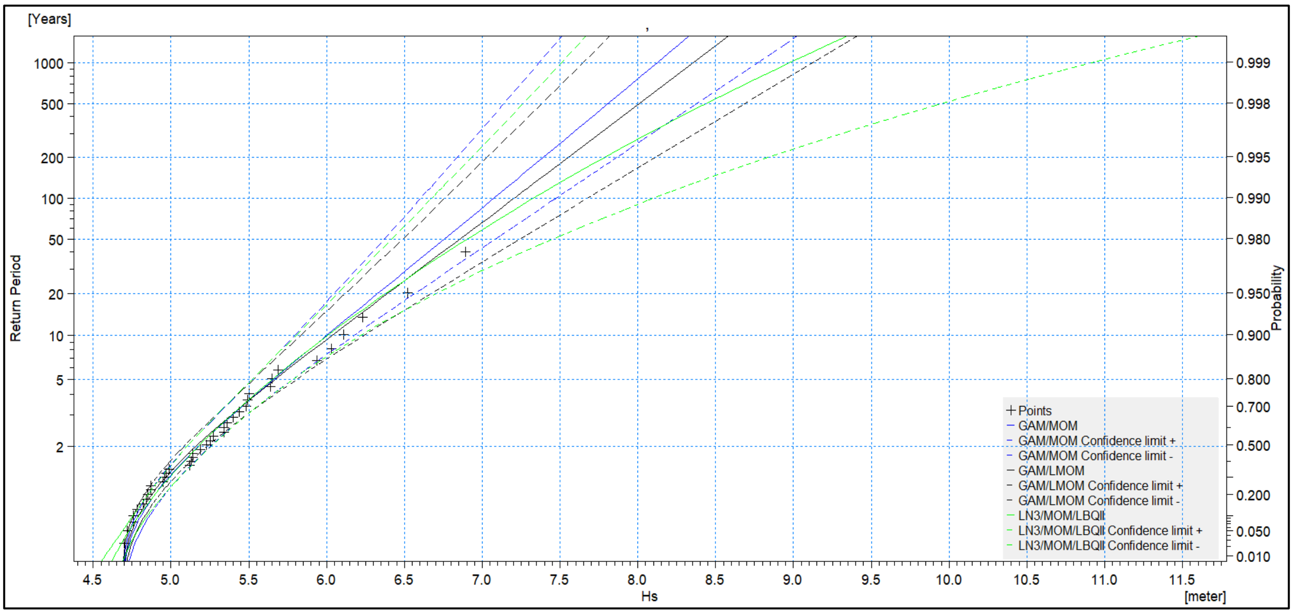


Figure 3-7 – Significant Wave Height (Hs) Distribution (green is LN3/MOM/LBQII)

Table 3-1 below summarises the results of the EVA.

Table 3-1 – Summary of EVA Results (Scheme opening - 2021)

Return Period	Significant Wave Height, Hs (m)
1	4.56
2	5.18
5	5.65
10	6.01
20	6.38
25	6.50
50	6.90
75	7.15
100	7.33
200	7.79
500	8.44
1000	8.97

3.3. Offshore Boundary Conditions

Table 3-2**Error! Reference source not found.** presents the extreme offshore wave and water level conditions derived at the model boundary through EVA and joint probability calculations (using the Desk Study Approach as per Defra/EA 2005 [2]). The water levels used for the joint probability analysis are from the Coastal Flood Boundary Conditions UK Mainland and Islands, Environment Agency (2011) dataset.

Table 3-2 – Extreme Offshore Boundary Conditions

Return Period (Year)	Water Level (mOD)	Hs (m)	Tp (s)
1 in 75 (2021)	3.28	7.15	11.40
1 in 75 (2021)	3.54	6.36	10.59
1 in 75 (2021)	3.7	5.4	9.55
1 in 75 (2021)	3.74	5.14	9.26
1 in 1 (2021)*	3.28	3.39	7.12
1 in 1 (2121)*	4.4	3.74	7.58

*Joint probability calculations only provided one useable joint probability pair for 1 in 1 year events. The Desk Study Approach spreadsheet provides JP pairs outside of the input data range using extrapolation. It is stated in the guidance that if they are clearly wrong they can be disregarded. Which they have been for this case.

The offshore boundary of the model is inshore of the Met Office data point (see Figure 3-8). This was deemed suitable as wave conditions will not significantly transform between the data point and model boundary as water depth varies from approximately 16 to 12m from MSL. This also allowed the model domain to be reduced, increasing the mesh resolution around the breakwater structures and harbour whilst maintaining an acceptable model runtime.

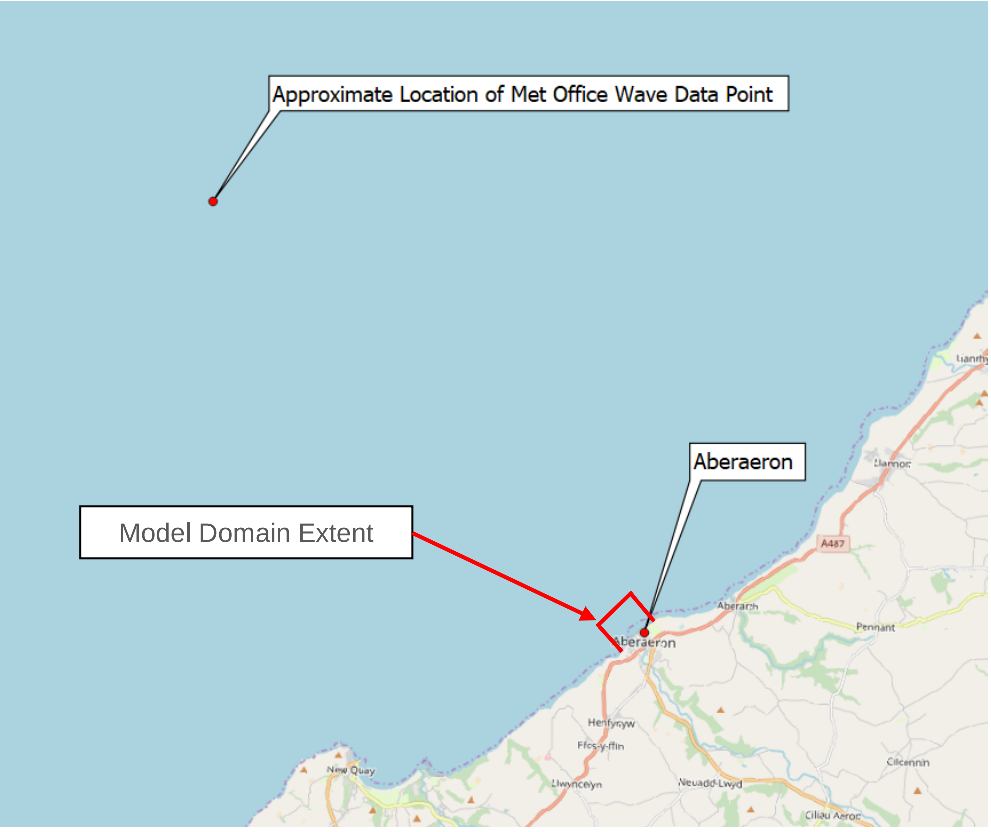


Figure 3-8 – Location of Met Office Wave Data Point in Relation to Model Boundary

Water levels were applied as a constant surface elevation across the model domain.

Using the wave data received (see Section 2.1.2), the number of waves against direction for the period between 1980 and May 2019 were plotted (see Figure 3-9).

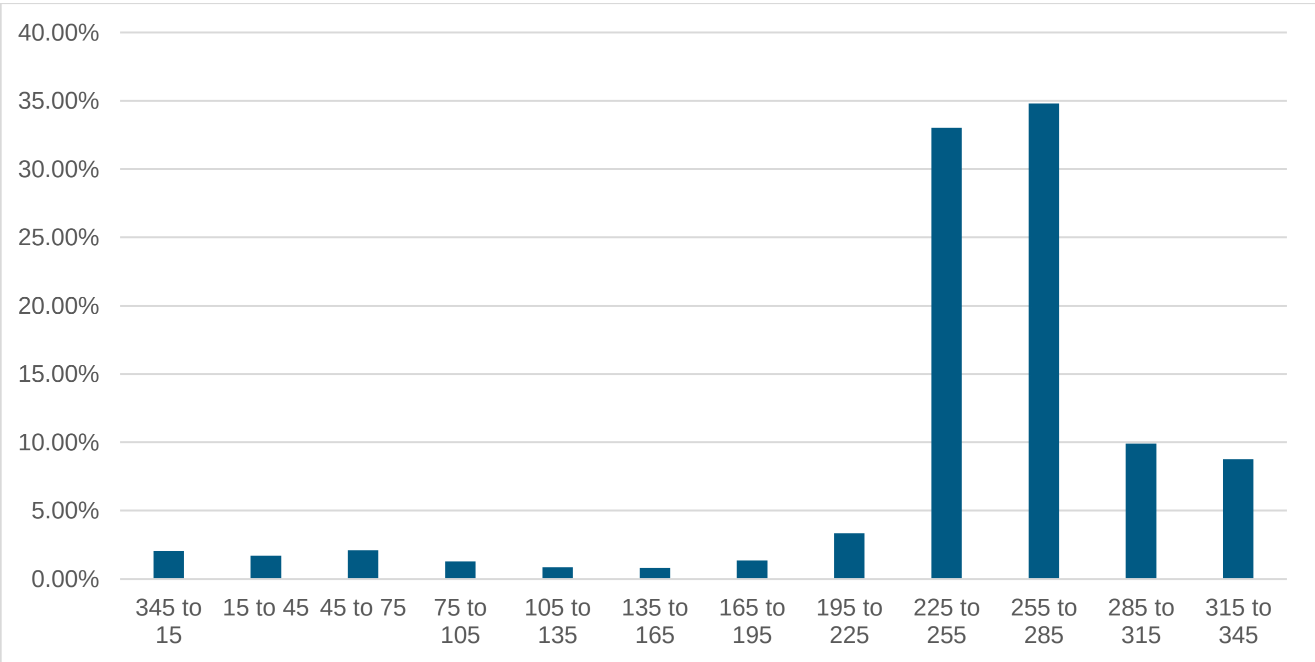


Figure 3-9 – Wave Height by Wave Direction (1980 to 2019)

As can be seen the majority of waves fall within 225°N to 285°N. The harbour entrance is protected from waves approaching from west of 285°N by the South Pier. Waves approaching between 285°N and 315°N would go straight into the harbour. Waves approaching from between 315°N and 360°N could affect overtopping at the harbour entrance through reflection from the south pier. In order to assess the range of effectiveness of the options, three wave directions have been tested:

1. 304°N
2. 334°N
3. 274°N

Direction 304°N is approximately parallel to the alignment of the south pier, with the other directions being +/- 30° either side for sensitivity testing. 274°N is within the most common wave direction vector. Modelling of this will provide insight in terms of the options' performance from the most common storm direction.

3.4. Modelled Scenarios

For the 1 in 75-year (2021) scenario, the joint probability pair with the largest wave height was selected from 4 no. joint-probability pairs, this is most likely to result in the largest wave heights within the harbour.

Table 3-3 – Modelled Wave Events and Respective Properties

Run	Layout	Return Period	Year	Wave Direction (degrees north)	Water Level (mOD)	Hs (m)	Tp (s)
1	0	1 in 75	2021	304	3.28	7.15	11.4
2	1	1 in 75	2021	304	3.28	7.15	11.4
3	2	1 in 75	2021	304	3.28	7.15	11.4
4	3	1 in 75	2021	304	3.28	7.15	11.4
5	4	1 in 75	2021	304	3.28	7.15	11.4
6	0	1 in 75	2021	334	3.28	7.15	11.4
7	1	1 in 75	2021	334	3.28	7.15	11.4
8	2	1 in 75	2021	334	3.28	7.15	11.4
9	3	1 in 75	2021	334	3.28	7.15	11.4
10	4	1 in 75	2021	334	3.28	7.15	11.4
11	0	1 in 75	2021	274	3.28	7.15	11.4
12	1	1 in 75	2021	274	3.28	7.15	11.4
13	2	1 in 75	2021	274	3.28	7.15	11.4
14	3	1 in 75	2021	274	3.28	7.15	11.4
15	4	1 in 75	2021	274	3.28	7.15	11.4
16	0	1 in 1	2121	304	4.40	3.74	7.58
17	1	1 in 1	2121	304	4.40	3.74	7.58
18	2	1 in 1	2121	304	4.40	3.74	7.58
19	3	1 in 1	2121	304	4.40	3.74	7.58
20	4	1 in 1	2121	304	4.40	3.74	7.58

4. Model Results

4.1. Wave Disturbance Coefficient Maps

For the purposes of this report, discussion on model results is focused on the wave height recorded at the north wall of Aberaeron harbour, along Quay Parade (see Figure 4-2).

Models have been run to produce a 2D map of wave disturbance coefficient values in the domain. Wave disturbance coefficient is a measure of the wave height at any point in the domain relative to the specified wave height at the offshore boundary. The key for these plots is shown below in Figure 4-1. All values above 0.7 are represented by the same colour due to all these regions being unsuitable for the degree of wave reduction required.

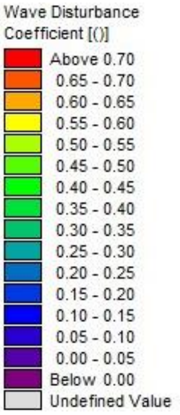


Figure 4-1 – Key Used to Show Wave Disturbance Coefficients

4.1.1. Comparison Points

Five points have been designated along the northern wall harbour wall of Aberaeron at which wave heights are to be ascertained. The five points chosen are presented below in Figure 4-2. By multiplying the wave disturbance coefficient at each location, by the input wave height, the wave height at each point was obtained.

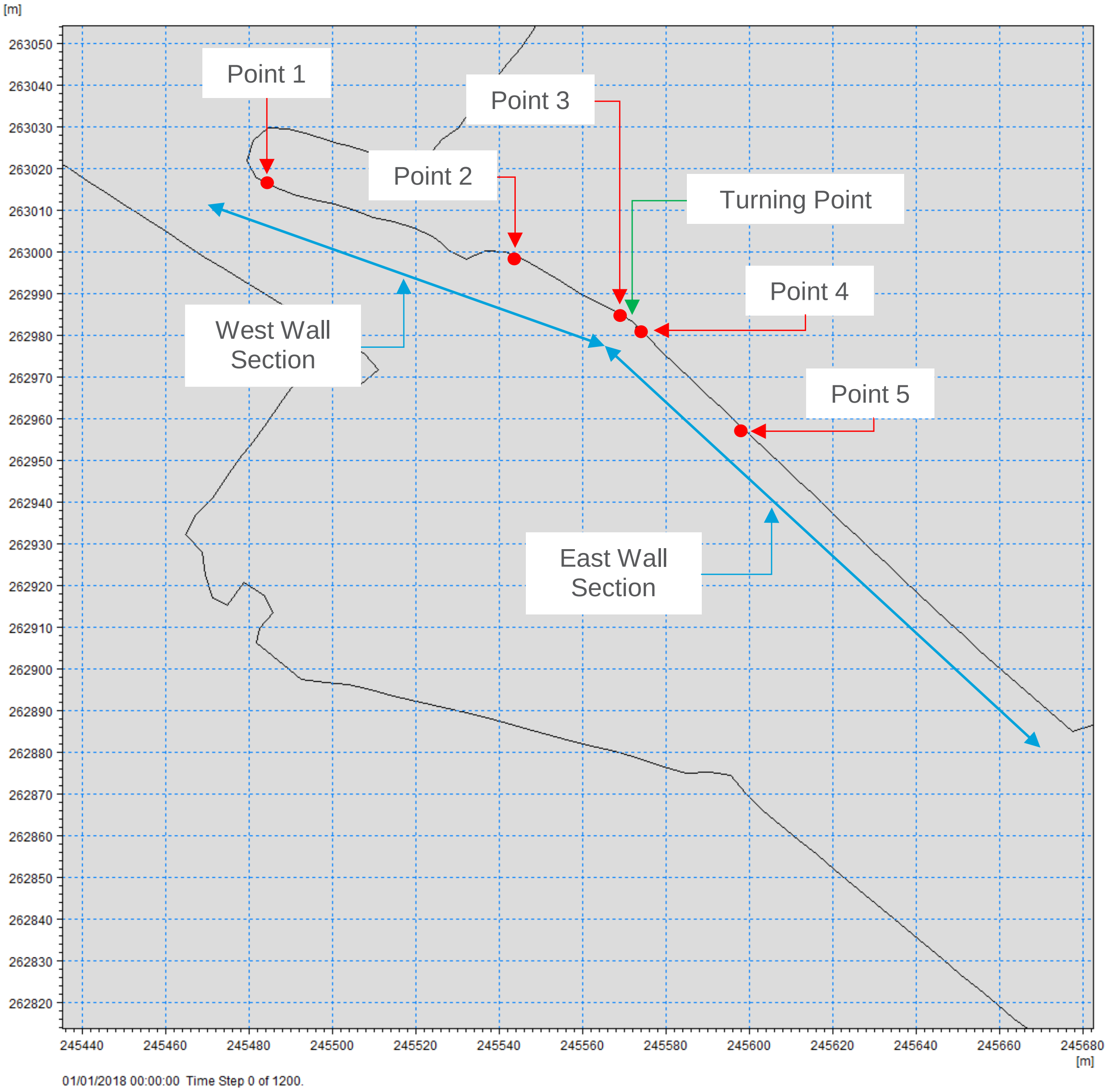


Figure 4-2 – Position of Points Used to Compare Structure Options

4.1.2. Critical Wave Heights

Videos available of previous storm events indicate that waves entering the harbour run along the west section of the wall to the 'turning point' and impact the east section of the wall at an oblique angle. Previous studies indicate this at about 80^0 to the wall based on a wave approach from outside of the harbour of around 304^0 [1] [3].

Figure 4-2 indicates discrete sections of the north harbour wall. These sections were assigned different 'critical wave heights'. The critical wave heights along the western and eastern sections have been used as thresholds for assessing the effectiveness of each layout option. These are the wave heights:

i) For the west section of the north harbour wall, where waves entering the harbour do not impact but instead run along the wall, the critical wave height has been determined based on: a minimum freeboard allowance of 300mm; an allowance of 0.02m for fluvial water level uplift for the water levels in the harbour based on modelling that has previously been performed by Atkins (5182114-ATK-MAR-GEN-CA-C-1031 Rev. 1.0). The critical wave height formula west of the turning point is shown below:

Critical wave height = [(5.2mOD - 0.3m freeboard allowance) – (extreme water level + fluvial uplift)] x2

Above the critical wave height the wave crest will exceed the top of the proposed secondary set back wall causing flooding.

ii) For the east section of the north harbour wall, empirical calculations of wave overtopping discharges based on EurOtop 2 [4] have been performed. These discharges have been compared to thresholds for pedestrian safety to identify what the critical wave heights would be, above which safe pedestrian limits would be exceeded at the secondary set back wall of 5.2mOD. The results of these calculations are presented in **Error! Reference source not found.** below.

There is limited guidance and physical testing for wave obliquity greater than 60 degrees. Therefore, it is considered that the empirical calculations are likely to overestimate overtopping along this section.

Table 4-1 - Critical Wave Heights at the North Wall of Aberaeron Harbour

	Critical Wave Height (m)	
Event	West Section	East Section
1 in 75 (2021)	3.20	1.42
1 in 1 (2121)	0.96	0.93

4.2. Results

In the following sections, wave disturbance plots and the significant wave height at the five points along the north harbour wall are presented for each model run.

The colours in the following tables represent the following:

- Green - below the wave height threshold.
- Red - above the wave height threshold.

4.2.1. 1 in 75 year Event from 304 degrees in 2021

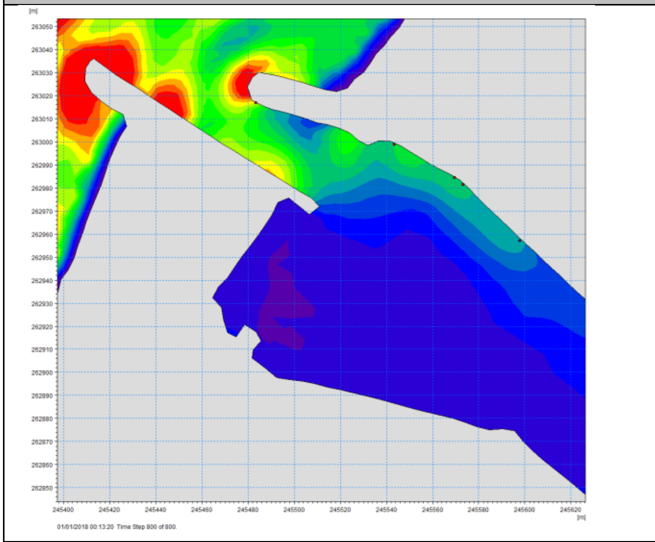
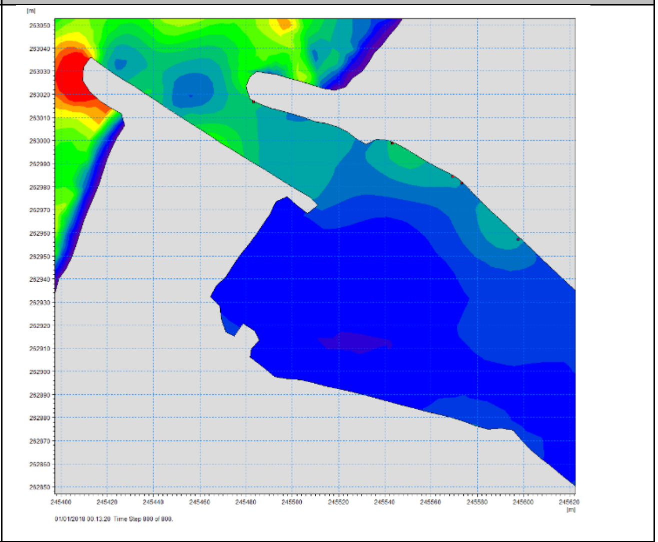
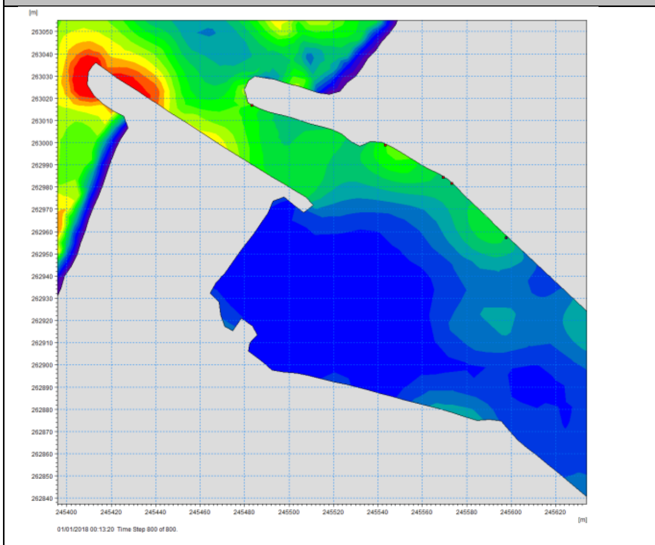
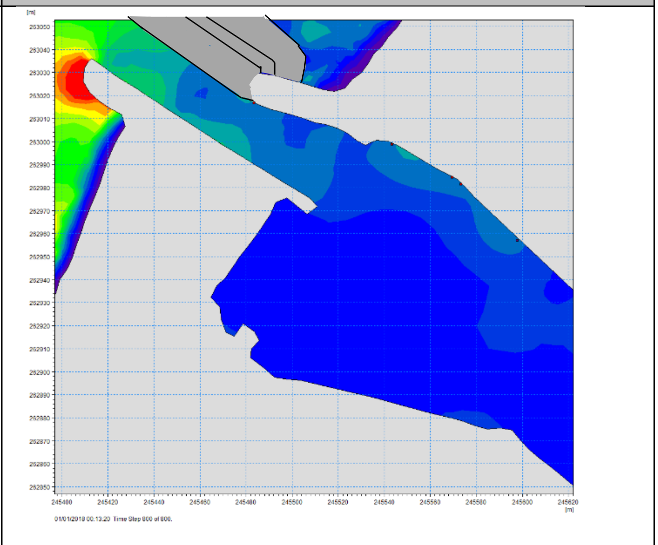
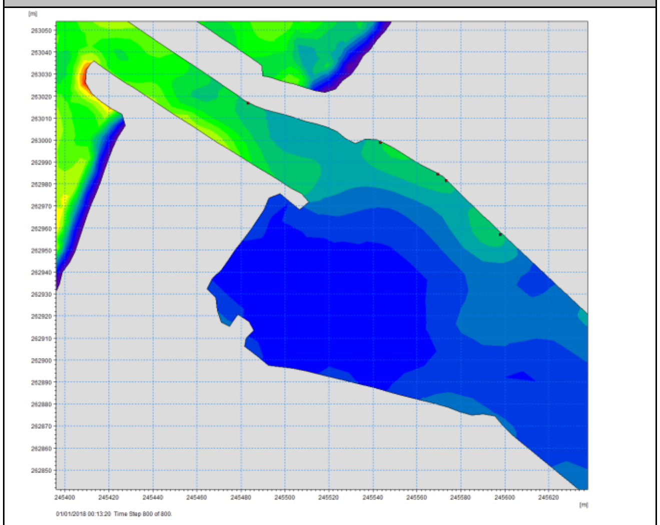
Option 0	Option 1
	
Option 2	**Option 3**
	
Option 4	
	

Figure 4-3 – Wave Disturbance for 1 in 75 year Event from 304 Degrees in 2021

Table 4-2 – 1 in 75 year Event from 304 Degrees in 2021

Model Structure Option	Significant Wave Height, H_s (m)				
	West Wall			East Wall	
	Point 1	Point 2	Point 3	Point 4	Point 5
Option 0	1.62	1.87	1.31	1.34	1.37
Option 1	1.02	1.05	0.84	0.82	0.85
Option 2	1.36	1.50	1.12	1.11	1.14
Option 3	0.78	0.81	0.70	0.69	0.70
Option 4	1.10	1.15	1.06	1.04	1.04

4.2.2. 1 in 75 year Event from 334 Degrees in 2021

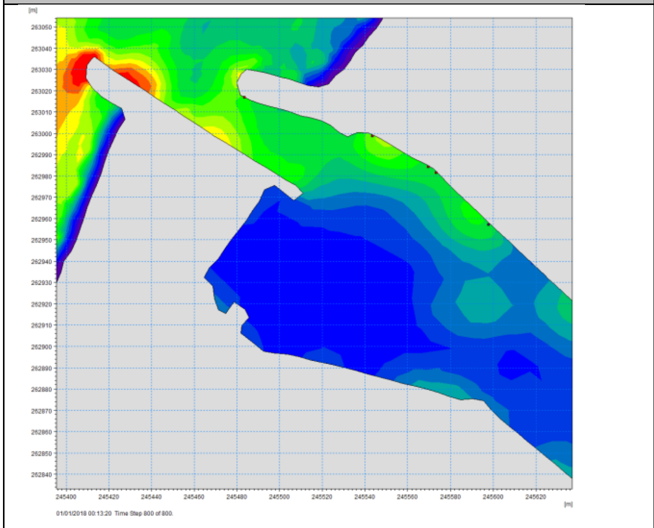
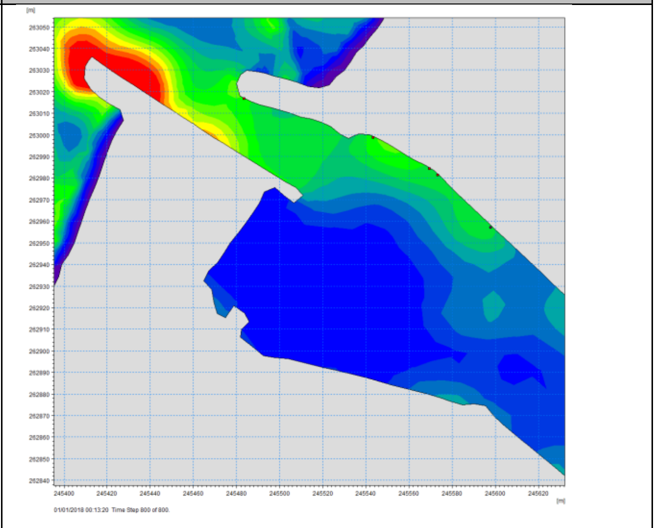
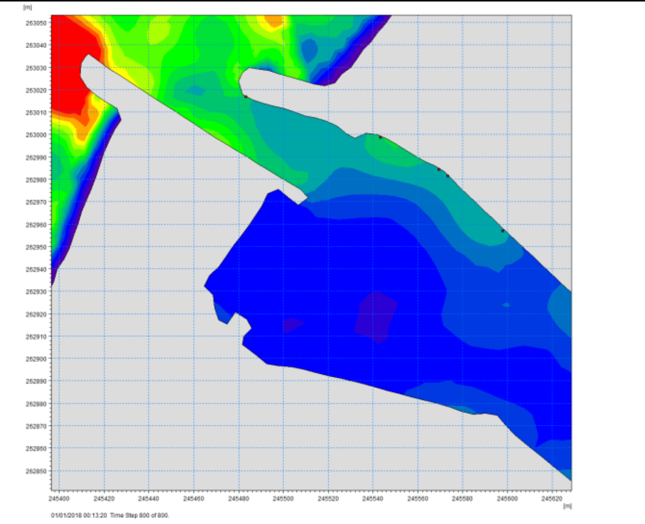
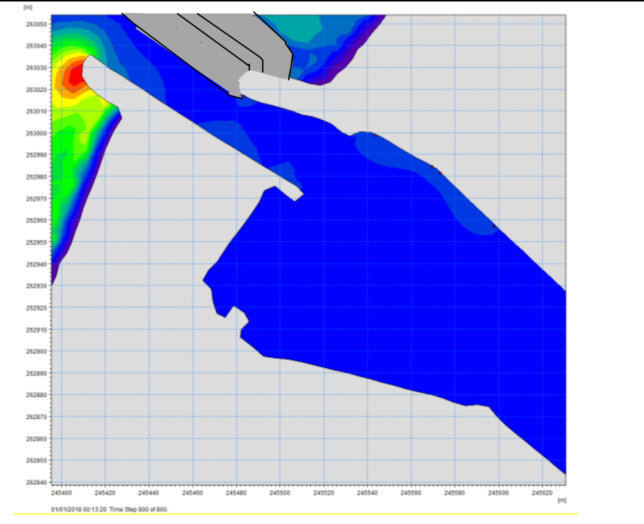
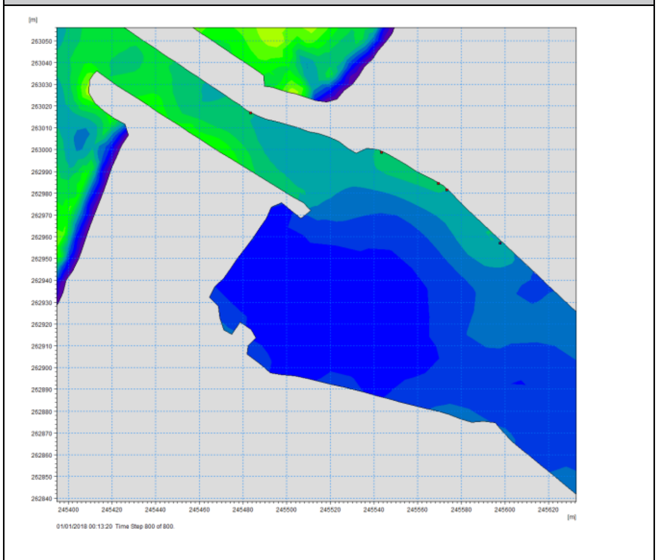
Option 0	Option 1
	
Option 2	Option 3
	
Option 4	
	

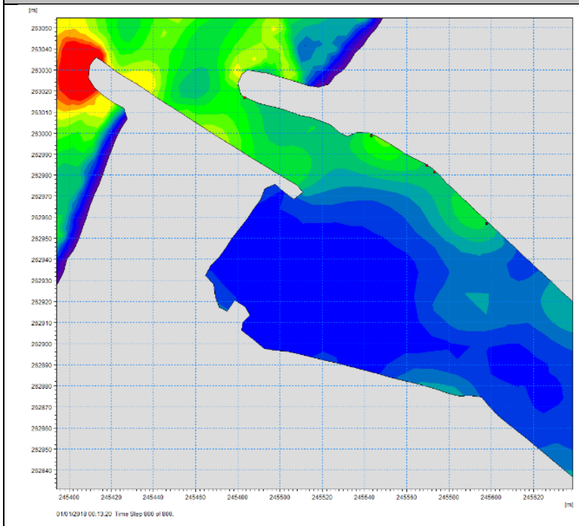
Figure 4-4 – Wave Disturbance for 1 in 75 year Event from 334 Degrees in 2021

Table 4-3 – 1 in 75 year Event from 334 Degrees in 2021

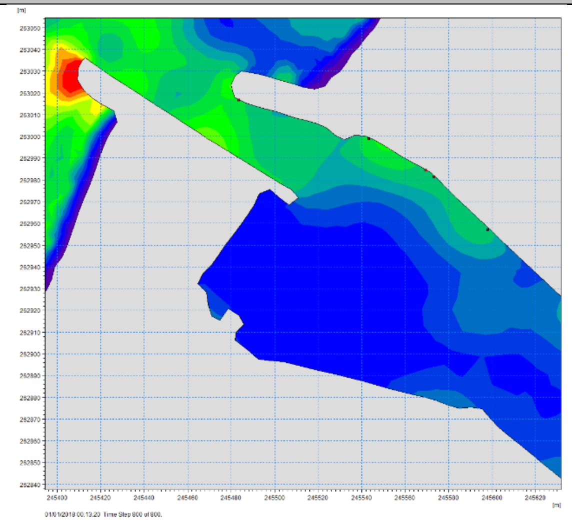
Model Structure Option	Significant Wave Height, H_s (m)				
	West Wall			East Wall	
	Point 1	Point 2	Point 3	Point 4	Point 5
Option 0	1.33	1.64	1.26	1.24	1.27
Option 1	1.35	1.45	1.29	1.26	1.22
Option 2	1.00	1.14	0.95	0.95	0.88
Option 3	0.52	0.53	0.50	0.50	0.50
Option 4	0.98	1.07	1.00	0.99	0.96

4.2.3. 1 in 75 year Event from 274 Degrees in 2021

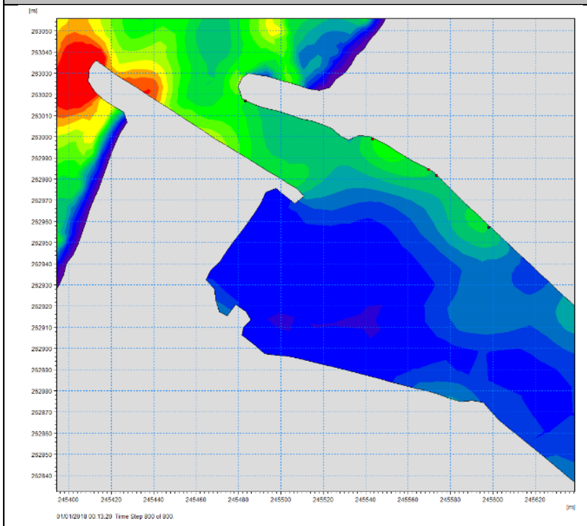
Option 0



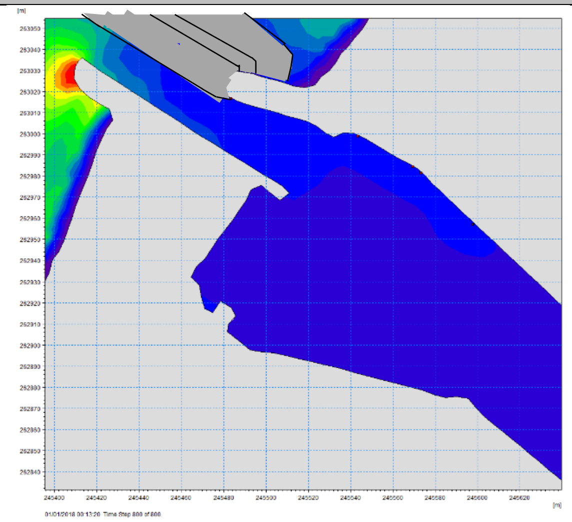
Option 1



Option 2



Option 3



Option 4

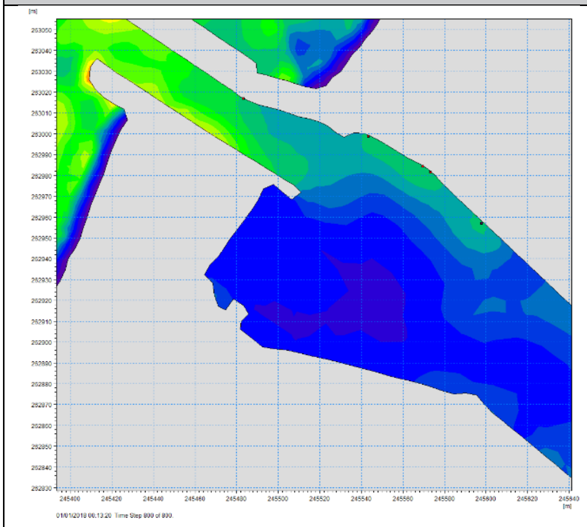


Figure 4-5 – Wave Disturbance for 1 in 75 year Event from 274 Degrees in 2021

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Table 4-4 – 1 in 75 year Event from 274 Degrees in 2021

Model Structure Option	Significant Wave Height, H_s (m)				
	West Wall			East Wall	
	Point 1	Point 2	Point 3	Point 4	Point 5
Option 0	1.18	1.48	1.13	1.11	1.15
Option 1	1.17	1.28	1.11	1.09	1.07
Option 2	1.24	1.34	1.16	1.13	1.16
Option 3	0.42	0.37	0.40	0.40	0.37
Option 4	1.10	1.06	1.14	1.13	1.01

4.2.4. 1 in 1 year Event from 304 Degrees in 2121

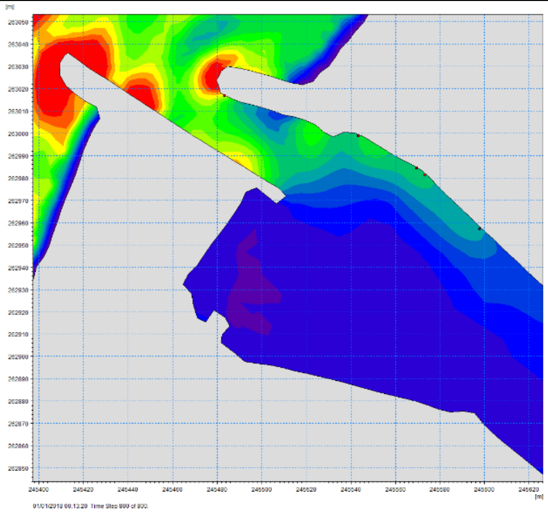
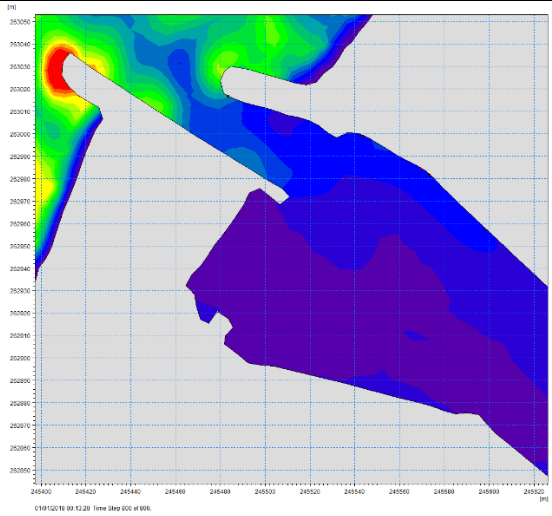
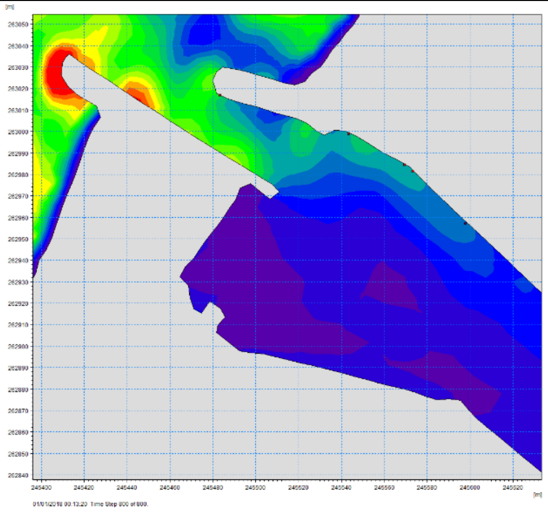
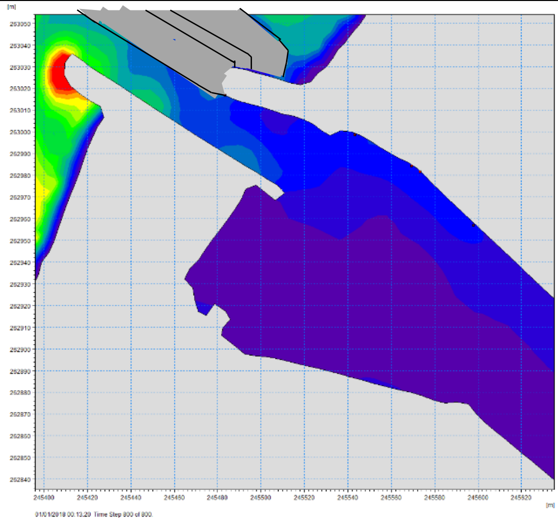
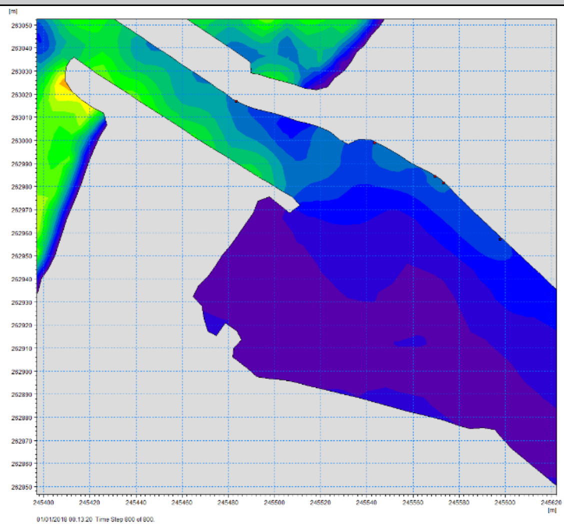
Option 0	Option 1
	
Option 2	Option 3
	
Option 4	
	

Figure 4-6 – Wave Disturbance for 1 in 1 year Event from 304 Degrees in 2121

Table 4-5 – 1 in 1 year Event from 304 Degrees in 2121

Model Structure Option	Significant Wave Height, H_s (m)				
	West Wall			East Wall	
	Point 1	Point 2	Point 3	Point 4	Point 5
Option 0	1.98	1.54	1.43	1.45	1.17
Option 1	1.01	0.59	0.61	0.61	0.54
Option 2	1.39	1.28	1.25	1.26	1.04
Option 3	0.78	0.58	0.59	0.60	0.51
Option 4	0.96	1.20	0.92	0.92	0.78

4.2.5. Performance Evaluation of the Modelled Options

4.2.5.1. 1 in 75 year event from 304 degrees in 2021

For this scenario, all options recorded significant wave heights below critical values. Option 3 consisting of the north pier breakwater extension was found to be the most effective structure, with the lowest wave heights recorded at all five points. Options 1 and 4 are less effective at decreasing wave heights with option 2 being the least effective in comparison to all other options.

4.2.5.2. 1 in 75 year event from 334 degrees in 2021

The results from 334 degrees showed that again all wave heights were below critical values, and again option 3 was again the most effective, decreasing wave heights significantly along all comparison points. After option 3, options 2 and 4 are the best performing. Option 1 is the least effective option.

4.2.5.3. 1 in 75 year event from 274 degrees in 2021

All wave heights recorded were below critical values in this scenario. Option 3 again was found to be the most effective with a significant wave reduction in comparison to Option 0. Option 4 causes some of the waves at the turning point to be slightly greater than the existing case which is likely to be caused by waves reflecting off the non-porous structure. Options 1 and 2 both provide some reduction in wave height.

4.2.5.4. 1 in 1 year event from 304 degrees in 2121

The results show that not all wave heights are below the critical values. Option 3 is again the most effective at reducing wave heights in the harbour and was the only option in which all results were below the critical wave heights. Options 1 and 4 both have one location above the critical wave height. Option 2 recorded no locations below the critical wave height.

4.2.6. Comparison of Options

Table 4-6 below summarises the number of point locations at which each option was below the critical wave height in all events listed below:

Table 4-6 – Comparison of Option Results

Event	Option 0	Option 1	Option 2	Option 3	Option 4
1 in 75 year event from 304 degrees in 2021	5	5	5	5	5
Range of wave heights	*1.31m-1.62m*	*0.82m-1.05m*	*1.11m-1.36m*	*0.70m-0.81m*	*1.04m-1.15m*
1 in 75 year event from 334 degrees in 2021	5	5	5	5	5
Range of wave heights	*1.24m-1.33m*	*1.22m-1.35m*	*0.88m-1.00m*	*0.50m-0.53m*	*0.96m-1.0m7*
1 in 75 year event from 274 degrees in 2021	5	5	5	5	5
Range of wave heights	*1.11m-1.18m*	*1.07m-1.17m*	*1.13m-1.34m*	*0.37m-0.42m*	*1.01m-1.14m*
1 in 1 year event from 304 degrees in 2121	0	4	0	5	4
Range of wave heights	*1.17m-1.98m*	*0.54m-1.01m*	*1.04m-1.39m*	*0.51m-0.78m*	*0.78m-1.20m*

As can be seen Option 3 is the most effective in comparison to the other options.

4.3. Further Model Runs

Following the comparison of the options (see Section 4.2.6), further model runs of the 1 in 75 (2021) joint probability pair with the largest water level (3.74mOD) was undertaken. This pair was chosen to ascertain if the lower than expected wave heights for the 1 in 75 event recorded in Section 4.2 were due to waves being depth limited in the harbour. The input parameters used in these model runs are listed in Table 4-7 below.

Table 4-7 – Modelled Wave Events

Run	Layout	Return Period	Year	Wave Direction (degrees north)	Water Level (mOD)	Hs (m)	Tp (s)
1	0	1 in 75	2021	304	3.74	5.14	9.26
2	1	1 in 75	2021	304	3.74	5.14	9.26
3	2	1 in 75	2021	304	3.74	5.14	9.26
4	3	1 in 75	2021	304	3.74	5.14	9.26
5	4	1 in 75	2021	304	3.74	5.14	9.26

The results from these model runs are summarised in Figure 4-7 and Table 4-8.

4.3.1. 1 in 75 year Event from 304 Degrees in 2021

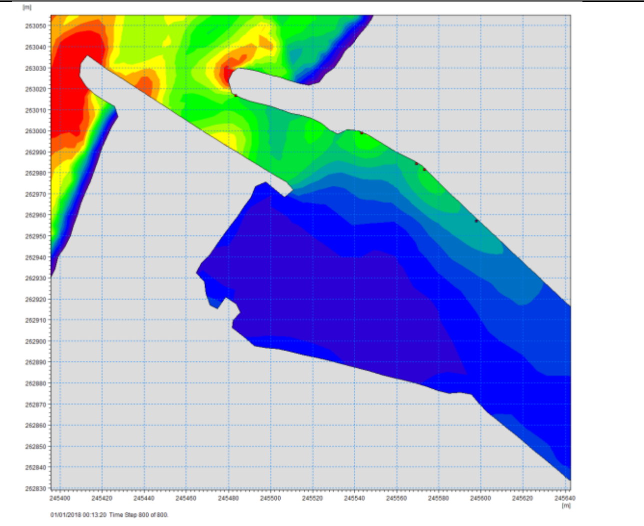
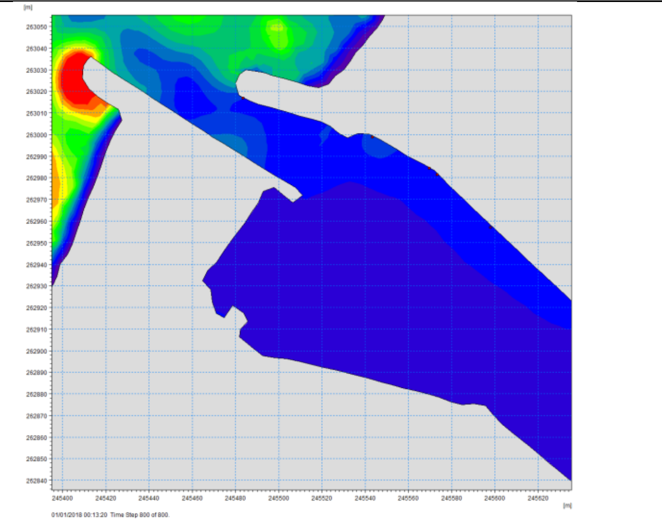
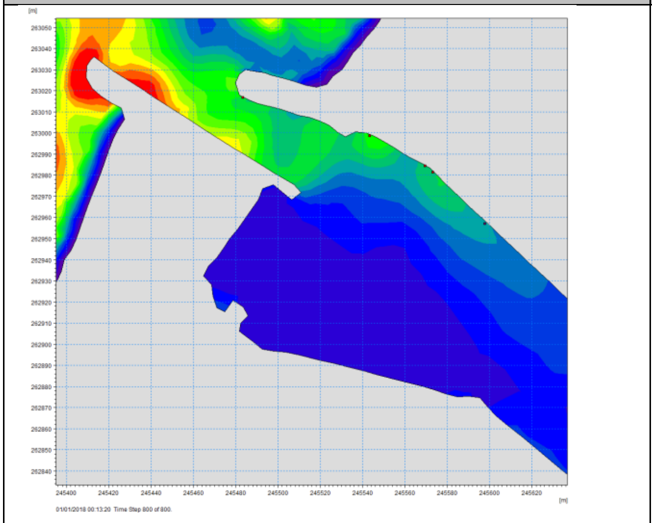
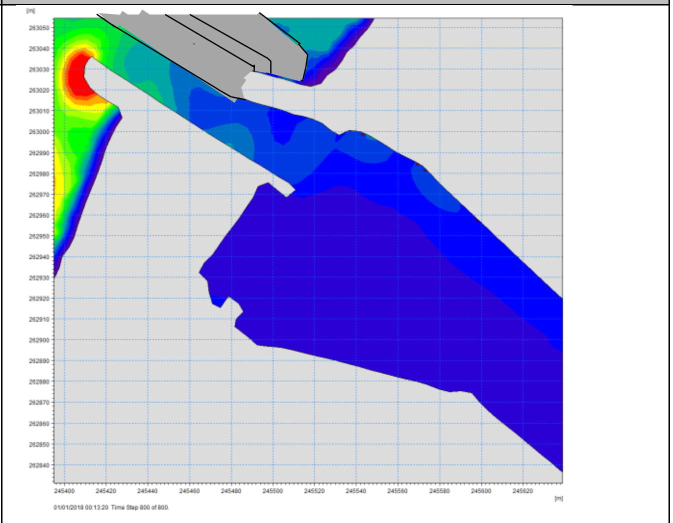
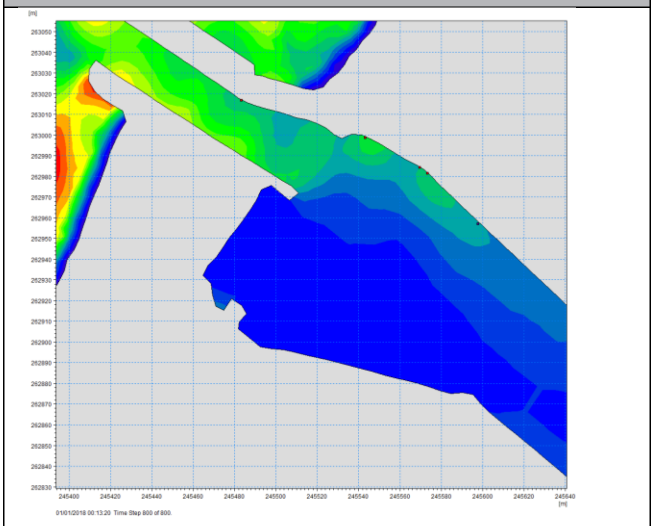
Option 0	Option 1
	
Option 2	Option 3
	
Option 4	
	

Figure 4-7 – Wave Disturbance for 1 in 75 year Event from 304 Degrees in 2021

Table 4-8 – 1 in 75 year Event from 304 Degrees in 2021

Model Structure Option	Significant Wave Height, H_s (m)				
	West Wall			East Wall	
	Point 1	Point 2	Point 3	Point 4	Point 5
Option 0	1.46	1.68	1.26	1.38	1.00
Option 1	0.48	0.65	0.50	0.53	0.45
Option 2	1.26	1.63	1.18	1.30	0.95
Option 3	0.61	0.75	0.57	0.62	0.50
Option 4	1.00	1.26	0.93	1.02	0.79

4.3.2. Performance Evaluation of the Modelled Options

As per section 4.2, all wave heights recorded are below the critical wave heights. However, for this scenario, with a greater water depth, option 1 performs the best followed by option 3, with options 2 and 4 providing some decrease to the wave heights at the northern harbour.

When comparing these results to those in Table 4-5, the significant wave heights within the harbour are mostly smaller for this scenario (larger water depth), suggesting the worst case joint probability scenarios are those with greater offshore wave height rather than water level.

It is noted that Option 0 (Do Nothing) exceeds the critical wave heights in the 1 in 1 year event in 2121 but does not exceed the critical wave heights for any of the 1 in 75 year events in 2021. This indicates the increased susceptibility to wave overtopping over time due to the effect of not mitigating the effects of larger waves occurring in combination with sea level rise.

5. Conclusions and Recommendations

From the analysis undertaken it is clear that the most effective option for decreasing wave heights along the north harbour wall is option 3 (220m long north pier breakwater extension).

Option 3 reduced wave heights below the critical levels for all modelled wave directions. As described in Section 4.1.2, the critical wave heights east of the turning point are considered to be conservative. This is due to the limits of guidance and physical testing of waves at the obliquity experienced at this location within the harbour.

It is recommended that option 3 is taken forward to the next stage of modelling for option development and refinement. It is also recommended that the following is investigated at the next stage of modelling:

- A single structure width has been modelled for all options. Increasing the structure width along the entire section or just at the outer arm could further reduce wave heights within the harbour.

- The 1 in 75-year case has been analysed using the joint probability pairs with the highest wave height. One further set of these pairs using the highest water level were run as described in Section 4.3, however it is recommended at the next stage of option refinement that all the joint probability pairs are tested. This would include identification of the standard of protection provided by Option 3 following refinement (if required).

- A possible option to further reduce wave heights in the harbour would be to incorporate a concrete wall into the outer arm of Option 3. This would reduce wave transmission through the structure.

- To more accurately represent the breakwater structures in the model, the 3D porosity map should be updated. Two or more layers of material should be simulated in the breakwater by using different porosity values. To better simulate the material layers, further vertical (sigma) layers may need to be added, enhancing the resolution of the domain.

- The average mesh resolution in the modelling work detailed was roughly 20m^2 to allow for a reduced runtime, necessary for the number of model runs undertaken. In future work, the need for further refinement should be considered.

- Consideration of the coastal processes likely to effect sediment movements (erosion and deposition) needs to be assessed for each of the options, particular in respect of the adjacent Sabelleria reef. This may be a deciding factor between options 1 and 3.

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6. References

[1] M. W. A. (MWA), “Project Appraisal Report Review- Aberaeron Harbour,” 2014.

[2] E. A. -. DEFRA, “Joint Probability: Dependence Mapping and Best Practice- Technical Report on Dependence Mapping, R&D Technical Report (FD2308/TR1),” 2005.

[3] Hyder, “Aberaeron Flooding PAR, Overtopping and Flooding, Numerical Modelling Report: 5007-UA002520-UU41R-01,” 2012.

[4] “EurOtop 2- Manual on Wave Overtoping of Sea Defences and Related Structures,” EurOtop, 2018.

[5] Opus, “Aberaeron Coastal Defence Scheme Design Strategy,” 2017.

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