



# **Central Rhyl Coastal Defences**

Environmental Statement Volume 2: Technical  
Appendix 10 - Coastal Processes and Flood Risk

September 2021

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Environmental Statement Volume 2: Technical  
Appendix 10 - Coastal Processes and Flood Risk

September 2021

# Issue and Revision Record

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# Appendix 10.1 – Wave Modelling Study



**Balfour Beatty**

A large teal graphic element consisting of a triangle pointing upwards at the top, a horizontal line, and a vertical line on the left side, forming a shape that resembles a stylized 'L' or a corner of a building.

# Central Rhyl Coastal Defences

Wave Modelling Study

September 2021

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# Central Rhyl Coastal Defences

## Wave Modelling Study

September 2021

# Issue and Revision Record

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# Executive summary

As a part of the detailed design work for the Central Rhyl coastal defences, Belfour Beaty (BB) commissioned Mott MacDonald to undertake a numerical wave modelling study to provide the data required for overtopping, wave force and rock sizing design calculations along the frontage. The study considers the year 2020 (baseline) and accounts for climate change impacts.

A two-dimensional wave model was built using MIKE21 flexible mesh (FM) spectral wave (SW) software. Offshore waves from the Met Office WWIII wave and wind hindcast model point 1516 were transformed to the nearshore region, and the model was calibrated/validated against measured data from the Rhyl Flats wave buoy.

J Heffernan Consulting undertook a multivariate analysis of offshore wave, wind and water level data to provide a 10,000-year synthetic record of offshore water level, wave and wind conditions for the baseline (2020) and two epoch years (2070 and 2120). The MIKE21 FMSW model and an emulator approach were used to generate a 10,000-year nearshore data set of wave and water level conditions at a nearshore location. The 1D wave transformation model LITDRIFT was then applied at three cross-shore locations to determine extreme wave and water level conditions at the toe of the coastal defences for use subsequently in overtopping calculations and sea defence design calculations.

# 1 Introduction

## 1.1 Background

Belfour Beaty (BB) commissioned Mott MacDonald to undertake a detailed design of the coastal defences along the Central Rhyl frontage. This work is supported by efficient and accurate numerical wave modelling to replicate the wave conditions at the toe of the defences for subsequent scheme design calculations. This report describes the modelling methodology and presents results for three scenarios for the baseline year (2020) and the years 2070 and 2120, where adjustment to mean water level and wave conditions are made to account for projected climate change impacts.

## 1.2 Report structure

The report comprises five sections summarising the key modelling activities, data and results. The report is structured as follows:

- Section 1 describes the background to this study;
- Section 2 describes the numerical wave modelling approach;
- Section 3 describes the data used to build the models, including bathymetry and topography, water levels, waves and wind;
- Section 4 presents offshore wave relationships;
- Section 5 presents and discussed wave transformation results using two and one-dimensional models; and
- Section 6 summarises the modelling study.

## 2 Methodology

The wave transformation modelling approach used in this study is shown in the schematic in Figure 2-1 and comprised:

- Analysis of the offshore wave conditions;
  - An extreme value analysis (EVA) of offshore wave data from Met Office Wave Watch III wave hindcast model at point 1516 (WWIII 1516) <sup>1</sup> [1] was used to estimate the significant wave height (H<sub>s</sub>) for a range of return periods;
  - Extreme water levels were obtained from the Coastal Flood Boundary conditions dataset (CFB) <sup>2</sup> [2]. The estimated CFB water levels were corrected to the year 2020 (baseline) using UKCP18 RCP 8.5 at the 70th percentile<sup>3</sup> [3];
  - Sea level rise data were extracted from UKCP18 guidelines for RCP8.5 at the 70<sup>th</sup> percentile [3];
  - A multivariate analysis<sup>4</sup> [4] (MVA) was undertaken using the Met Office WWIII data point 1516 and water level data. This analysis gave an offshore 10,000-year synthetic record of water level, H<sub>s</sub>, peak wave period (T<sub>p</sub>), mean wave direction (MWD), wind speed and wind direction; and
  - Relationships between offshore wave parameters were determined.
- The development of a MIKE21 FMSW wave model (Section 5.1) to transform offshore waves to the study frontage. The model was calibrated and validated against the measured data from the Rhyl Flats buoy;
- Simulation of the offshore 10,000-year dataset with the MIKE21 FMSW wave model requires extensive computational time. Thus, the 10,000-year synthetic dataset was reduced by binning it into a specific range of bin sizes<sup>5</sup> [5]. These binned datasets were then applied as boundary conditions and were transformed to a nearshore location using the MIKE21 FMSW model;
- The ratio of offshore to inshore wave conditions (wave heights, wave periods and the angle change in wave direction) were calculated based on the binned simulation results against the offshore 10,000-year dataset;
- The 10,000-year wave/water level inshore dataset was transformed to the toe of the coastal defence structures using the one-dimensional wave transformation model LITDRIFT; and
- The resulting wave and water level characteristics at the toe of the defences were provided for subsequent overtopping and scheme design calculations<sup>6</sup> [6].

### 2.1 MIKE21 FMSW

MIKE21 FMSW is a third-generation state-of-the-art spectral wind-wave model based on the finite volume method on an unstructured mesh that enables full-time domain simulations. The model simulates the growth, decay and transformation of wind-generated waves and swells in

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<sup>1</sup> <https://www.metoffice.gov.uk/research/weather/ocean-forecasting/ocean-waves>

<sup>2</sup> UK Environment Agency, "Coastal flood boundary conditions for the UK: 2018 update," Bristol, UK, 2018.

<sup>3</sup> M. D. Palmer et al., "UKCP18 Marine Report," United Kingdom, 2018.

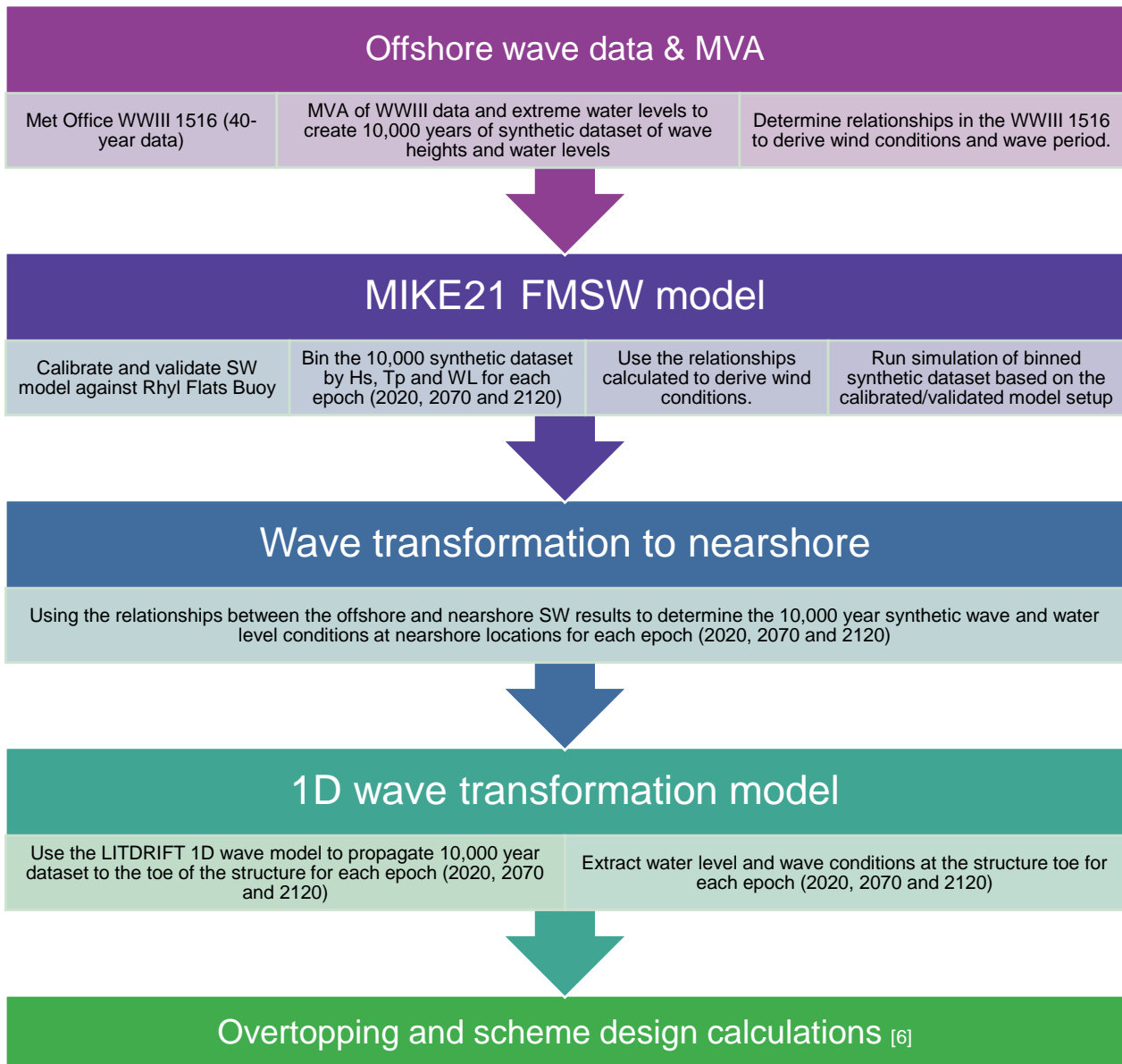
<sup>4</sup> This analysis was performed by J Heffernan Consulting Limited (JHC). JHC, "Central Rhyl Coastal Defences: Extreme Value Analysis, 2020 baseline year analysis, WWIII location 1516", 2021.

<sup>5</sup> "Data binning is the process of grouping individual data values into specific bins or groups according to defined criteria", Mott MacDonald, 2021.

<sup>6</sup> Mott MacDonald, "CR-MMD-00-00-RP-SC-7011 Central Rhyl Coastal Defences – Overtopping Report", 2021.

offshore and coastal areas. The model includes wave growth by the action of wind, nonlinear wave-wave interaction, dissipation due to white-capping dissipation due to bottom friction, dissipation due to depth-induced wave breaking, refraction and shoaling due to depth variations, wave-current interaction, and the effect of time-varying water depth. Therefore, the model is deemed suitable to propagate waves from offshore to the nearshore area at Central Rhyl.

**Figure 2-1 The wave modelling approach**



Source: Mott MacDonald, 2021

## 3 Data

The data inputs for the modelling study included open-source data and site-specific measurement. It is noted that the model outputs from this study rely heavily on the accuracy of the dataset used, and thus they were quality assured before use using bespoke in-house Mott MacDonald tools.

### 3.1 Systems and projections

Geographical data used in this study used the British National Grid (BNG) with OSGB36 horizontal datum. The vertical datum was referenced to Ordnance Datum Newlyn (ODN) at Central Rhyl, estimated to be -4.45m to the Chart Datum (CD).

### 3.2 Bathymetry

The bathymetric data used to build the MIKE21 FM/SW<sup>7</sup> [7] wave model (Section 1.1.1) included:

- Bathymetry data<sup>8</sup> [8] extracted directly from the JBA SWAN mesh;
- Freely available bathymetry data services (UKHO)<sup>9</sup> [9];
- Open source EMODnet (2018 data <sup>10</sup> [10];
- Composite and historical NRW LiDAR data from 2004, 2009, 2011 and 2015;
- Coastal Channel Observatory (CCO)<sup>11</sup> [11] beach monitoring profiles for March 2019 and June 2020; and
- Historical bi-annual monitoring beach surveys from 2002 to 2014 from Denbighshire County Council.

The offshore model bathymetry was built mainly using the UKHO and EMODnet data, prioritising the UKHO data. The nearshore model bathymetry was based on the CCO survey of June 2020. These survey data were selected due to their cross-shore and temporal coverage. For further information regarding historical analysis of the beach levels of Central Rhyl, please refer to the *Sediment Transport Report* (Mott MacDonald, 2021)<sup>12</sup> [12].

### 3.3 Waves

Measured and modelled wave data, quantifying significant wave height (Hs), peak wave period (Tp) and mean wave direction (MWD), were available close to the project site comprising:

- Model hindcast 3-hourly and hourly wave data from 1980 to 2019 (40-year) from the UK Met Office Wave Watch III (WWIII) wave model at point 1516<sup>13</sup> [13] at 53° 32.49' N, 3° 55.8' W in a water depth of approximately 45 mOD; and
- Offshore wave data from May 2007 to December 2019 measured at the Rhyl Flats wave buoy<sup>14</sup> [14] at 53° 22.92'N, 3° 36.18'W in a water depth of approximately 14 mOD for the period.

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<sup>7</sup> DHI, "MIKE 21 – Spectral Wave Module – Scientific Document," Hørsholm, Denmark, 2020.

<sup>8</sup> JBA Consulting, "Point of Ayr to Pensarn Tidal Flood Risk Analysis: Final Model Development Report", 2017.

<sup>9</sup> [Bathymetry data Service \(admiralty.co.uk\)](https://www.admiralty.co.uk/bathymetry) accessed on 2 November 2020.

<sup>10</sup> [EMODnet Bathymetry \(emodnet-bathymetry.eu\)](https://www.emodnet-bathymetry.eu) accessed on 3 November 2020.

<sup>11</sup> [Channel Coastal Observatory Filter](https://www.cco.gov.uk/channel-coastal-observatory-filter) accessed on 17 November 2020.

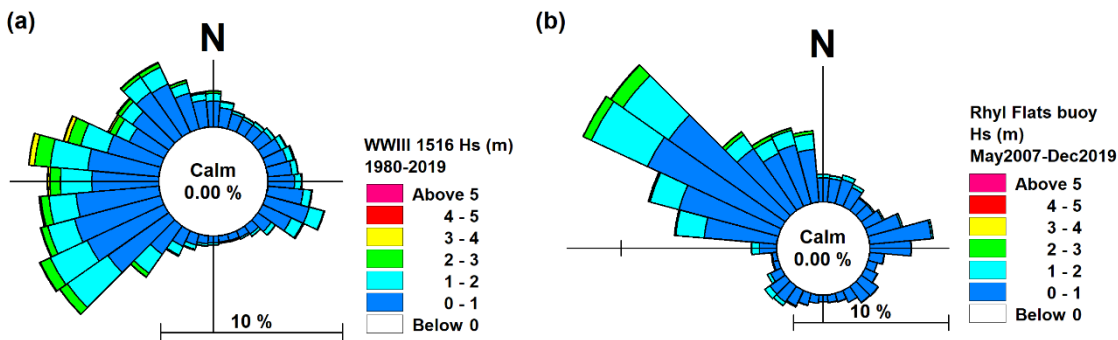
<sup>12</sup> Mott MacDonald, "CR-MMD-00-ZZ-RP-SC-7007: Central Rhyl Coastal Defences - Sediment Transport modelling", 2021

<sup>13</sup> <http://wavenet.cefas.co.uk/hindcast> accessed on 20 October 2020.

<sup>14</sup> [Channel Coastal Observatory Rhyl Flats](https://www.cco.gov.uk/channel-coastal-observatory-rhyl-flats) accessed on 22 October 2020.

Figure 3-19(a,b) show the wave rose plots of Hs against MWD at Met Office WWIII 1516 and Rhyl Flats wave buoy locations, respectively. Figure 3-19(b) shows that the dominant wave direction at Rhyl Flats wave buoy is from the northwest, while Figure 3-19(a) shows that the dominant wave direction at Met Office WWIII 1516 is from the south-west, west and northwest. It is noted that the Rhyl Flats wave buoy is located approximately 10km from the shoreline and approximately 28km inshore from WWIII 1516 (Figure 5-1).

**Figure 3-1 Wave roses derived from: (a) WWIII 1516 wave hindcast data (1980 to 2019); and agent (b) measured wave data from the Rhyl Flats wave buoy from May 2007 to December 2019.**

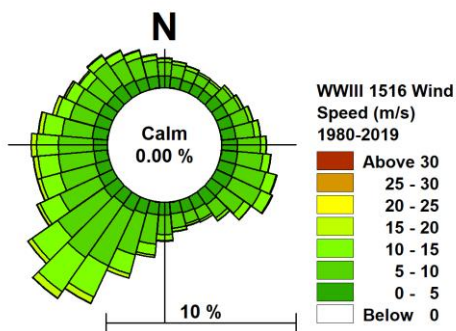


Source Mott MacDonald, 2021

### 3.4 Wind

As the wind is the primary forcing agent contributing to wave formation, using the best available wind data in wave modelling studies is essential. In recognition of this, the present study used wind data from Met Office WWIII 1516 [13]. The wind rose at WWIII 1516 shown in Figure 3-2 shows that the dominant wind direction is from the south-west, west and northwest, and is similar to the wave rose of Hs in Figure 3-1(a).

**Figure 3-2 Wind roses derived from WWIII 1516 wave hindcast data (1980 to 2019).**



Source Mott MacDonald, 2021

### 3.5 Water level

Two sets of water level data were used in this study:

- i. For the MIKE FMSW model calibration/validation (Section 1.1.1), a distance weighted estimate of water levels was interpolated for Central Rhyl between the predicted tides from Liverpool and Llandudno tidal stations. Tides prediction was performed based on the harmonic constituents extracted from Liverpool and Llandudno tidal stations using the MIKE 21 Tidal Prediction Toolbox <sup>15</sup> [15]. The predicted tides were used instead of the measured tides due to the complexity of combining the measured dataset. Furthermore, the calibration station (Rhyl Flats wave buoy) is located in deep water and therefore, the influence of water level is not significant; and
- ii. The measured water level data at Rhyl Harbour, available from 15/10/2008 to 17/02/2017, was combined with the Llandudno predicted and measured tide data in the MVA. Rhyl Harbour data was only available for a portion of the tide curve, above 2mODN. The lower portion of the tide curve was replaced with the Llandudno tide gauge, when available, or the predicted tide at Llandudno.

#### 3.5.1 Extreme water levels

Extreme water levels were obtained from the Coastal flood boundary (CFB) [2] condition data, at point 1136 as shown in Figure 3-3. Extreme water levels and the corresponding Annual Exceedance Probability (AEP) at CFB 1136 are presented in Table 3-1. The estimated CFB water levels were corrected to the year 2020 (baseline) using UKCP18 RCP 8.5 at the 70th percentile [3].

**Figure 3-3 CFB extraction point 1136.**



Source CFB 2017

**Table 3-1 CFB water level at point 1136**

AEP (%)	CFB water level (mOD)
100	4.91
50	4.99
20	5.11
10	5.20
5	5.28

<sup>15</sup> DHI, "MIKE 21 Tidal Analysis and Prediction Module Scientific Documentation," Horsholm, Denmark, 2017.

AEP (%)	CFB water level (mOD)
4	5.31
2	5.39
1.3	5.44
1	5.47
0.7	5.52
0.5	5.56
0.4	5.59
0.3	5.61
0.2	5.68
0.1	5.77
0.01	6.10

Source CFB 2017 and Mott MacDonald, 2021

## 3.6 Climate change

### 3.6.1 UKCP18: Sea-level rise

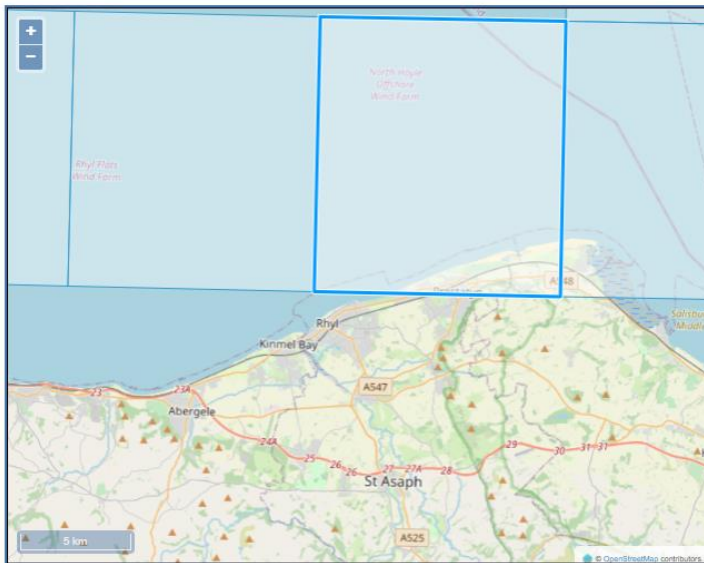
The sea-level rise (SLR) projection used in the study follows UKCP18 guidelines for RCP8.5 at the 70<sup>th</sup> percentile [3]. The SLR values obtained for 2070 and 2120 are shown in Table 3-2. An overview of the extracted grid from the UKCP18 database is shown in Figure 3-4.

**Table 3-2 Sea level rise based on UKCP18 RCP8.5 at the 70<sup>th</sup> percentile.**

Year	Estimated SLR (m)
2020	0
2070	0.36
2120	0.87

Source Mott MacDonald, 2021

**Figure 3-4 Sea level rise extracted grid from UKCP18 database.**



Source UKCP18

### 3.6.2 Wind and wave adjustments to the future epochs

Wind and wave adjustments for 2070 and 2120 were based on Welsh guidelines <sup>16</sup> [16] and involved up-scaling wind speed and wave height by +10%, with a corresponding increase in wave period defined by wave steepness.

## 3.7 Multivariate analysis (MVA)

An extreme value analysis (EVA) of  $H_s$  was undertaken on WWIII 1516 (40-year) using the DHI EVA tool <sup>17</sup> [17] and supplied to JHC (2021) for the MVA. In calculating extreme  $H_s$  values, various probability distribution and estimation methods were considered. The most statistically significant fit for  $H_s$  was obtained using an exponential distribution. This fit is shown in Figure 3-5, together with lines showing the 95% confident intervals.

Similarly, the MVA was undertaken using WWIII 1516 (40-year) and water level data from October 2008 to February 2017 (Section 3.5) to generate a 10,000-year synthetic record of water level,  $H_s$ ,  $T_p$ , MWD, wind speed and wind direction. The climate change allowances defined in the previous section was also incorporated in the MVA.

Table 3-3 shows the estimated extreme  $H_s$  values provided to JHC(2021) and estimated  $H_s$  values from the MVA (JHC, 2021). This table shows that the estimated extreme  $H_s$  from the MVA by JHC (2021) match the supplied  $H_s$ , particularly for the value of estimated  $H_s$  at AEP 2% and 1% for the epochs year this study. A comprehensive MVA methodology and analysis detailed in "JHC Extreme Value Analysis report, 2021" [4].

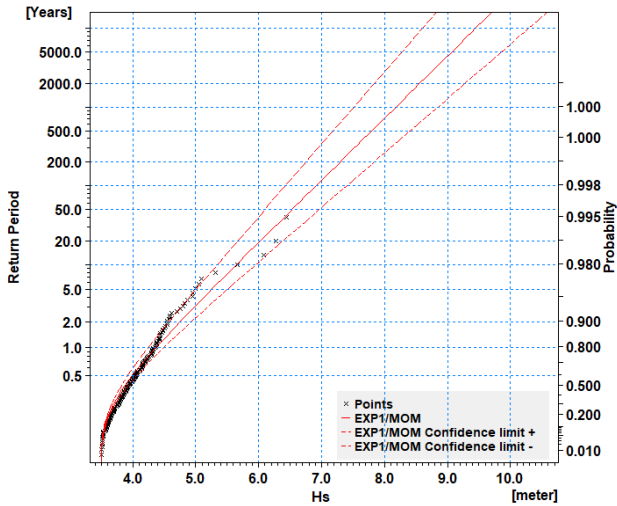
Figure 3-6 shows an example of a scatter plot of the offshore 10,000-year synthetic record of  $H_s$  and water level extracted from the MVA results for the baseline year 2020 and two epochs year, 2070 and 2120. It is observed that, as expected,  $H_s$  and WL values for the baseline increased

<sup>16</sup> Welsh Government, "Adapting to Climate Change: Guidance for Flood and Coastal Erosion Risk Management Authorities in Wales", December 2017.

<sup>17</sup> DHI, "Extreme Value Analysis User Guide", Hørsholm, Denmark, 2017.

for each of the epoch years. Further details of the MVA are given in "JHC Extreme Value Analysis report, 2021" [4].

**Figure 3-5 Exponential probability distribution best fit to Hs vs return period showing the 95% confidence intervals.**



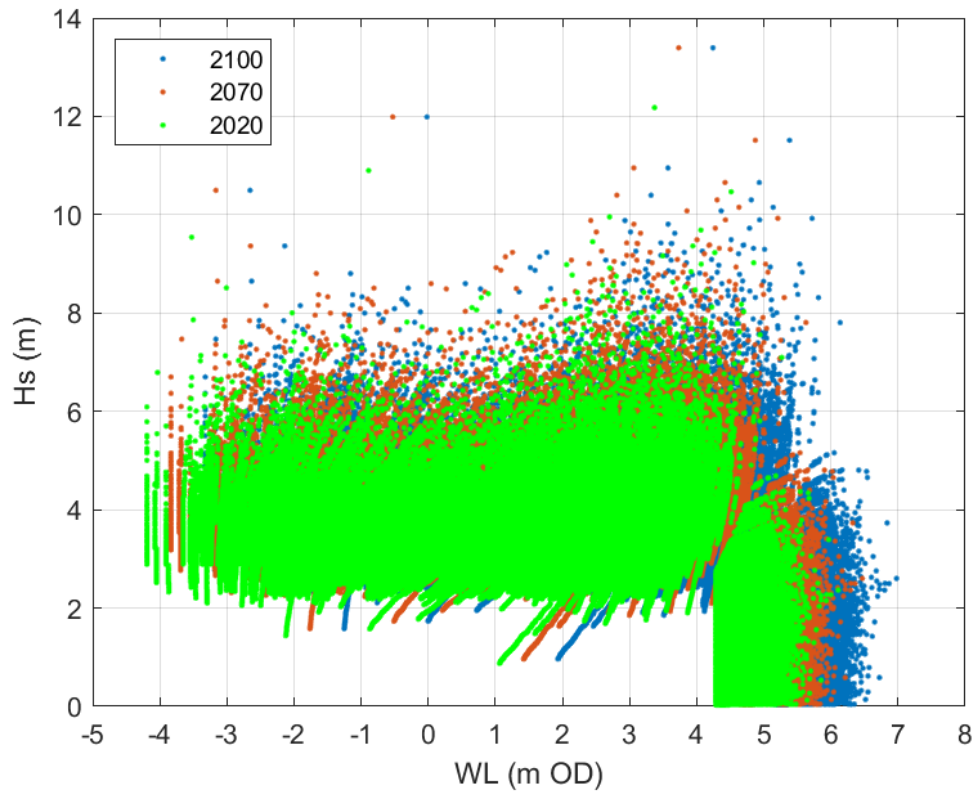
Source Mott MacDonald, 2021

**Table 3-3 Estimated extreme Hs values.**

AEP (%)	Hs (m)	Hs MVA (m)
100	4.37	4.36
20	5.20	5.23
10	5.61	5.61
4	6.14	6.13
2	6.52	6.52
1	6.91	6.93
0.5	7.29	7.41
0.2	7.80	8.06
0.1	8.18	8.47
0.02	9.07	9.06
0.01	9.45	9.20

Source Mott MacDonald and J Heffernan Consulting, 2021

**Figure 3-6** Scatter plot of the 10,000-year synthetic record of Hs against water level (WL) for 2020, 2070 and 2100. Note the increase in both water level and Hs with projected climate change impacts applied.



Source J Heffernan Consulting, 2021

## 4 Offshore wave relationships

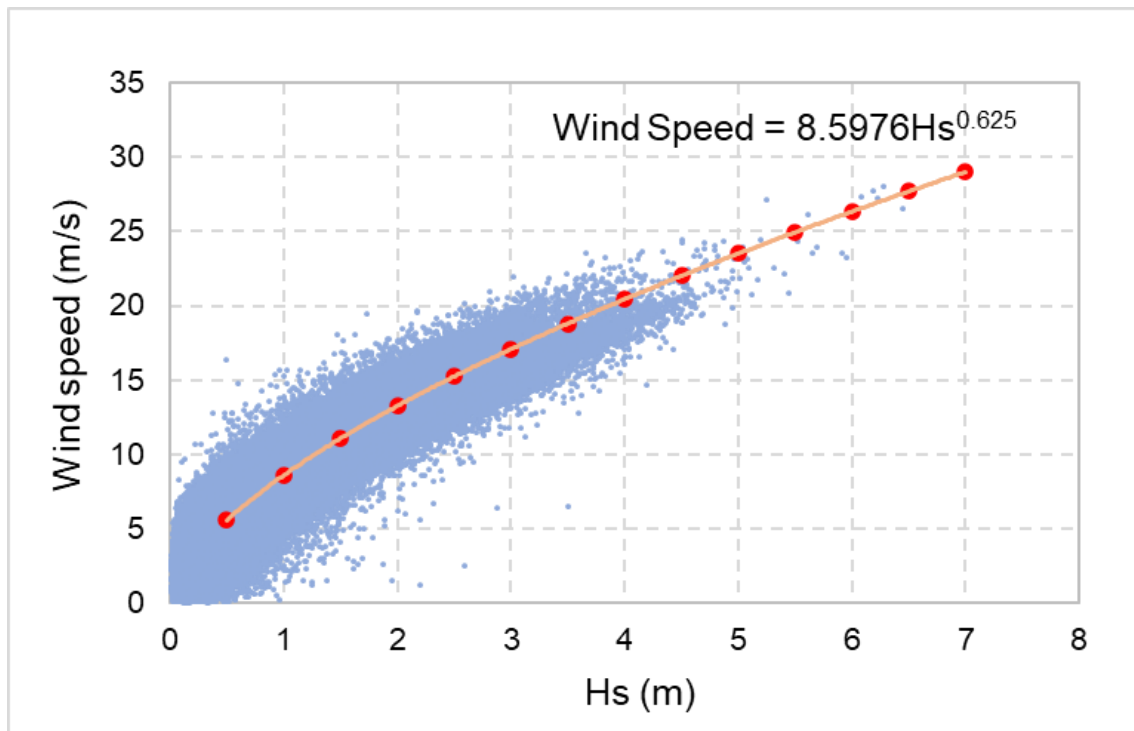
### 4.1 Introduction

The relationships between significant wave height and wind speed and between MWD and wind speed were determined to derive the two-dimensional wave transformation model's boundary conditions based on an emulator approach (Section 5.2). A relationship between  $T_p$  and the mean wave period ( $T_m$ ) was obtained to facilitate the  $T_p$  results' conversion from the 1D wave transformation model (Section 5.3). The resulting  $T_m$  values were used subsequently in the overtopping and design calculations. The WWII 1516 data was used to derive the offshore wave relationships in this section.

### 4.2 Significant wave height and wind speed

The relationship between  $H_s$  and wind speed used in the two-dimensional wave transformation model in Section 5.2 is shown in Figure 4-1.

**Figure 4-1 Relationship between wind speed and significant wave height,  $H_s$ .**

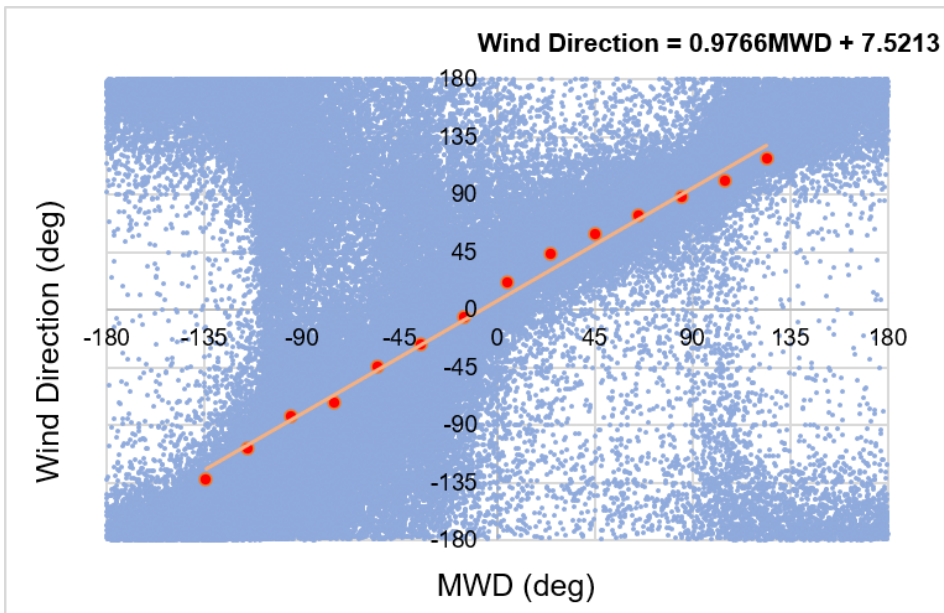


Source Mott MacDonald, 2021

### 4.3 Mean wave direction (MWD) and wind direction

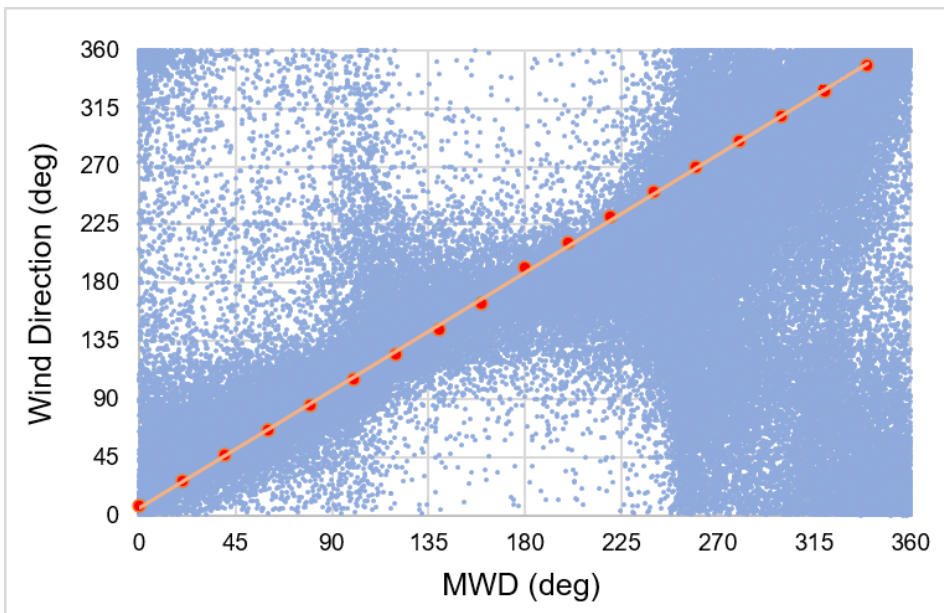
Omnidirectional wind data was used to determine a relationship between MWD and wind direction. All wind direction and MWD data above 180 deg. N was adjusted to establish directional sectors between 0 to 180 deg. N and 0 to -180 deg. N (Figure 4-2). Based on the relationship shown in Figure 4 2, the wind direction and MWD were converted back to 0 to 360 degree. N (Figure 4-3).

**Figure 4-2 Relationship between wind direction and MWD.**



Source Mott MacDonald, 2021

**Figure 4-3 Relationship between wind direction and MWD (0 to 360 degree. N).**

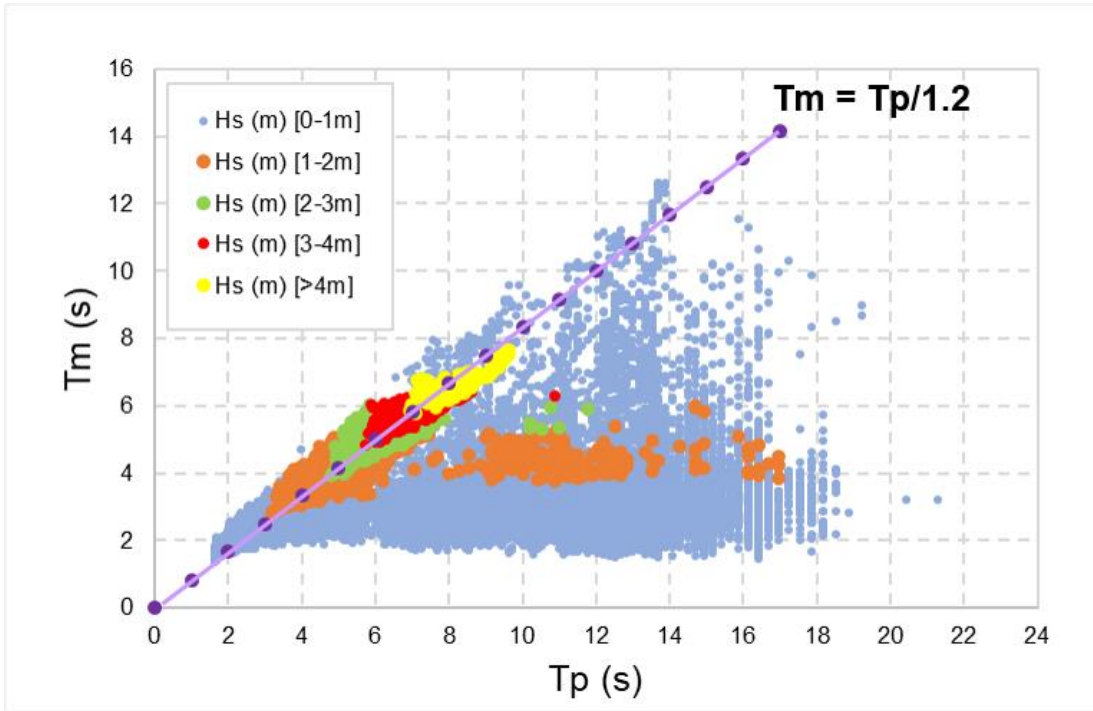


Source Mott MacDonald, 2021

#### 4.4 Peak wave period (Tp) and mean wave period (Tm)

The relationship between Tm and Tp values was derived for the range of Hs values shown in Figure 4-4. Taking a conservative view, the relationship between Tm and Tp used in this study considered only Hs above 4m.

**Figure 4-4 Relationship between Tm and Tp.**



Source Mott MacDonald, 2021

# 5 Wave transformations

## 5.1 Introduction

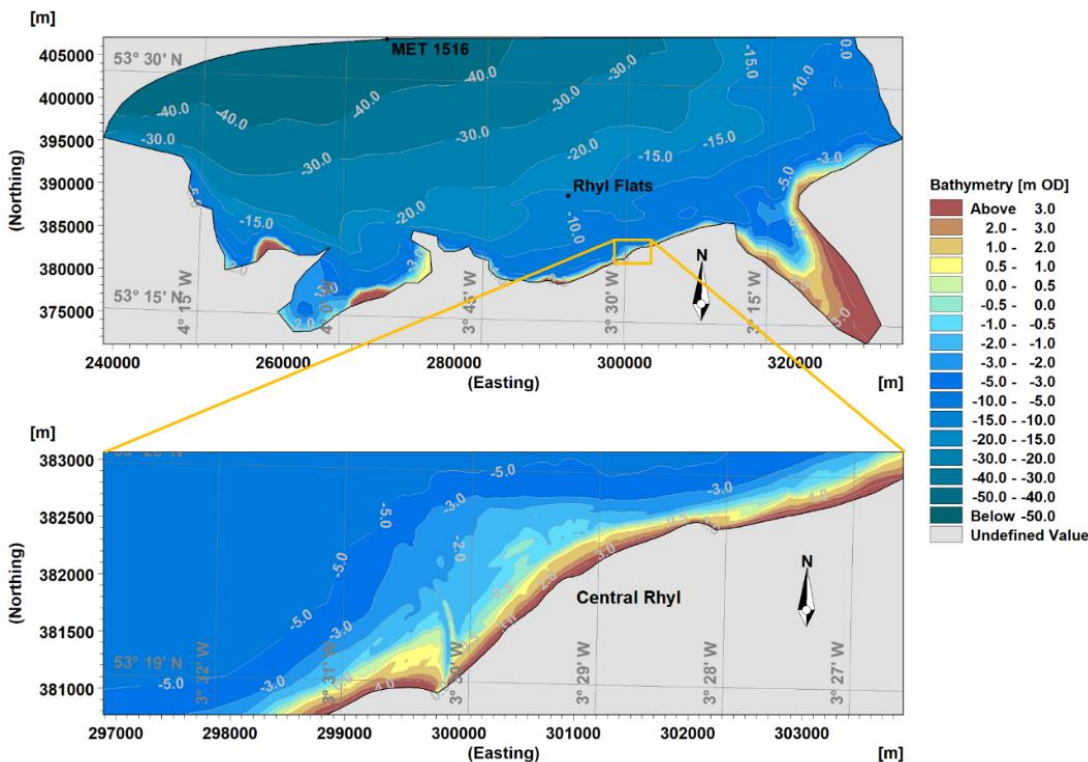
Wave transformations were required to provide extreme waves and water levels for the overtopping and design calculations at the defences. The two-dimensional MIKE21 FMSW [5] model was used to transform offshore waves to the nearshore, and a higher spatial resolution one-dimensional wave transformation model (LITDRIFT) was used to transform waves to the toe of the coastal defence structures.

It was recognised early in the study that using the MIKE21 FMSW alone would require extensive computational time to transform the 10,000-year synthetic record [4] (for each epoch) to the nearshore locations. An emulator approach was undertaken for project efficiency whereby the synthetic records (Hs/Tp/MWD/water level) were binned [5] to reduce the dataset and then used as boundary conditions in the MIKE21 FMSW model. The ratio between the offshore inputs and nearshore binned SW results were calculated as transformation coefficients and then subsequently used as a look-up table to transform the full set of 10,000-year synthetic record into a whole set of 10,000-year data at nearshore locations. More information on this process is provided in Section 5.2.

### 5.1.1 Model Domain (Flexible Mesh)

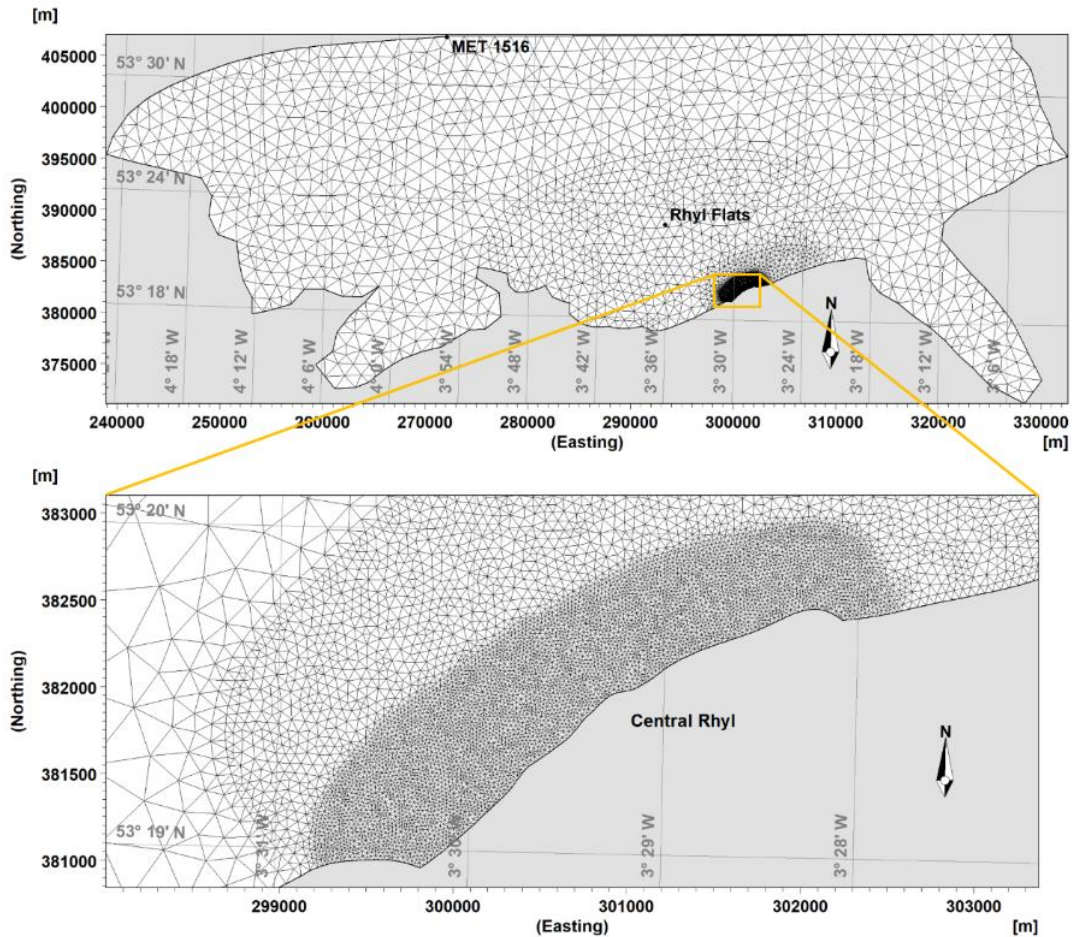
The MIKE21 FMSW model was set up to cover Central Rhyl and all relevant coastal and offshore areas that influence the wave conditions (Figure 5-1). This figure also shows the location of the Rhyl Flats buoy [14] and Met Office WWIII 1516. The mesh resolution for the model domain and the open boundary defined in the model are shown in Figure 5-2.

**Figure 5-1 MIKE21 FMSW model domain and bathymetry. An enlarged view of the Central Rhyl frontage is shown in the lower panel.**



Source Mott MacDonald, 2021

**Figure 5-2 MIKE21 FMSW model flexible mesh. An enlarged view of the Central Rhyl frontage is shown in the lower panel.**



Source Mott MacDonald, 2021

### 5.1.2 Model forcing and water level

The main wind forcing, taken from the WWII 1516 40-year hindcast dataset, was applied as surface boundary conditions. The water level forcing used in the model is described in Section 3.5.

### 5.1.3 Model Setup

**Table 5-1** summarises the MIKE21 FMSW model setup established in the model calibration process (see next section).

**Table 5-1 Summary of the calibrated MIKE21 FMSW model setup**

Parameters		Description
		Local Model
Equation		Directional-Decoupled, Quasi-stationary
Frequency Discretisation	No. of frequency	25
	Min Frequency	0.055 Hz
	Frequency factor	1.1
	No. of direction	360 degree rose (36 directions)
Solution		Quasi-stationary
Quasi-stationary formulation	Geographical space discretisation	Low order, fast algorithm
	Method	New-Raphson Iteration
	Max. number of iterations	100
	Tolerance (RMS-norm of residual)	0.001
	Tolerance (Max-norm of change in sig. wave height)	0.01
	Relaxation Factor	0.05
Water Level Conditions		Interpolated predicted water level (Section 3.5)
Current Conditions		No current included
Wind Forcing	Wind data	Met Office WWIII 1516
	Soft start	0
	Wind Generation Formula	SPM84
Ice Coverage		No ice coverage
Diffraction		No diffraction
Wave Breaking	Model	Wave breaking
	Type of gamma	Specified gamma
	Gamma data	0.78
	Alpha	1
	Gamma (wave steepness)	5
Bottom Friction		No bottom friction
Structures		No structures
Initial Conditions		Spectra from empirical formula from JONSWAP fetch growth expression
Boundary Conditions		Met Office WWIII 1516
Source Mott MacDonald, 2021		

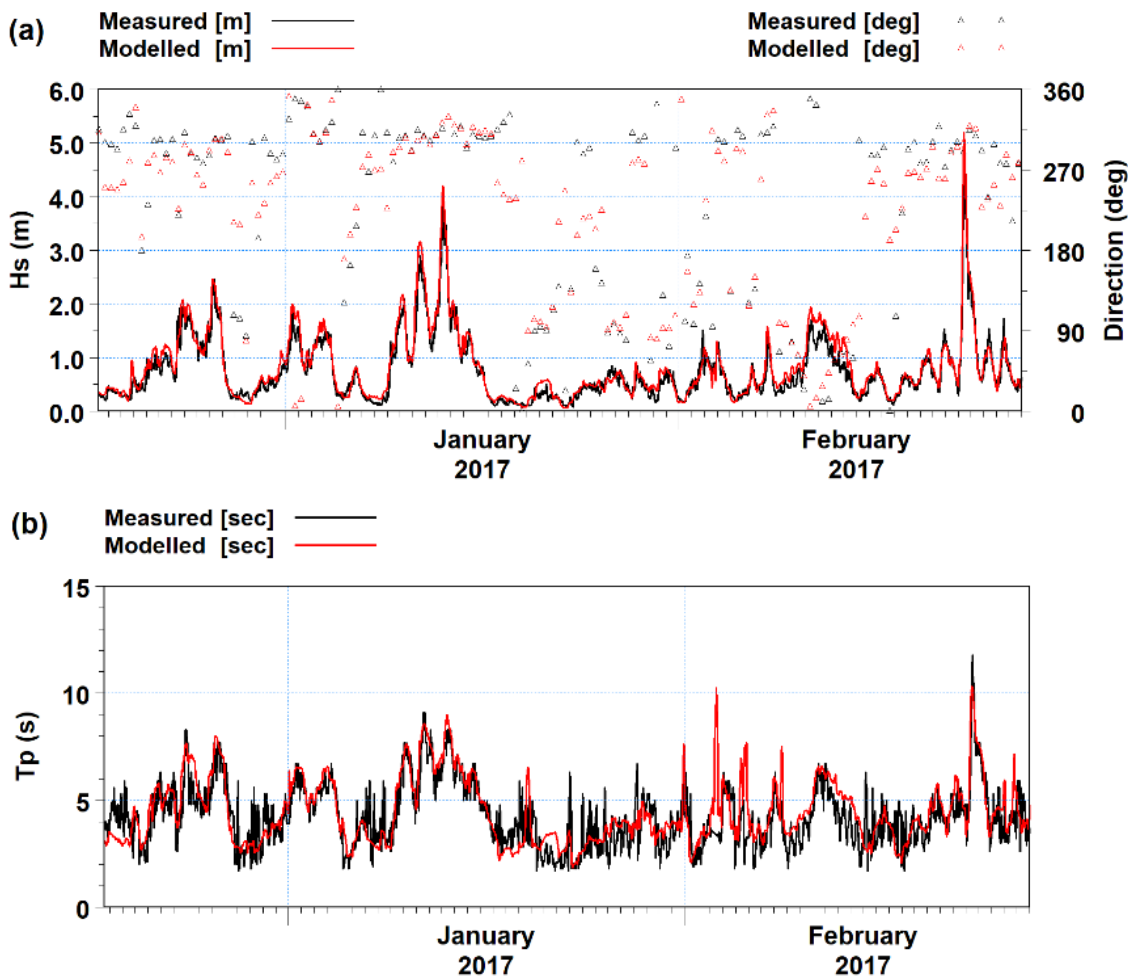
#### 5.1.4 Model Calibration

The MIKE21 FMSW wave model was calibrated against Rhyl Flats wave buoy data from 17 December 2016 to 28 February 2017. This period contains the highest storm waves recorded by the Rhyl Flats wave buoy. Figure 5-3 shows a comparison between measured and modelled data at Rhyl Flats wave buoy for  $H_s$ ,  $T_p$  and MWD. The figure demonstrates that the model predictions compare well with the measured data. Model performance statistics are tabulated in Table 5-2, including root-mean-square error (RMSE), the correlation coefficient of determination ( $R^2$ ), standard deviation (STD) and bias.

Table 5-2 shows that the model can reproduce the measured data with a high accuracy for  $H_s$  (RMSE = 4%,  $R^2 = 0.95$  and bias = - 0.09 m). Similarly,  $T_p$  and MWD also show good agreement against measured data. An overview of the model results during the storm period of 23 February 2017 at 12:00 is also presented in Figure 5-3 with the dominant wave direction from the northwest.

Based on NRW Guidelines (2017)<sup>18</sup> [18] and ABPmer(2013)<sup>19</sup> [19], the modelled Hs should be within +/- 10% of observed Hs, and the modelled mean wave period should be within 20% of the observed wave period. Further, the modelled mean wave direction should be +/- 30% of observed wave directions and scatter index values for Hs, Tp and MWD should be below 10, 20 and 25%, respectively, to conform with the model performance criteria established in the peer-reviewed publication by Williams & Esteves (2017)<sup>20</sup> [20]. The model calibration statistics in Table 5-2 demonstrate that the present MIKE21 FMSW model performance meets the NRW Guidelines [16] and those defined by Williams & Esteves (2017) [20].

**Figure 5-3 Comparisons between measured and modelled data at Rhyl Flats wave buoy between December and February 2017 showing: (a) significant wave height, Hs with the mean wave direction, MWD; and (b) peak wave period, Tp.**



Source Mott MacDonald, 2021

<sup>18</sup> Pye, K., Blott, S. and Brown, J., "Advice to Inform Development of Guidance on Marine, Coastal and Estuarine Physical Processes Numerical Modelling Assessments", NRW Report No: 208, 139 pp, Natural Resources Wales, Cardiff. 2017.

<sup>19</sup> ABPmer, "Numerical Model Calibration and Validation Guidance. ABP Marine Environmental Research Ltd., File Note R/1400/112". ABPmer, Southampton (unpublished), 2013.

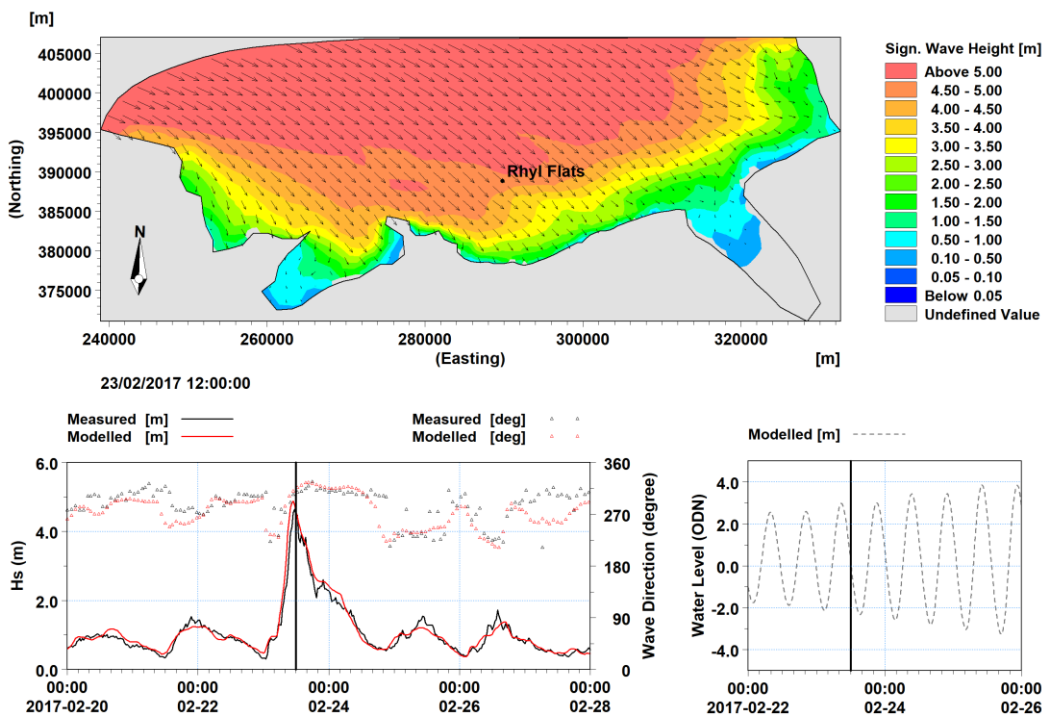
<sup>20</sup> Williams, J.J. and Esteves, L.S., "Guidance on Setup, Calibration, and Validation of Hydrodynamic, Wave and Sediment Models for Shelf Seas and Estuaries". Advances in Civil Engineering, 5251902, 2017.

**Table 5-2 Summary of the calibrated MIKE21 FMSW performance metrics at the Rhyl Flats wave buoy.**

Parameter	Hs	Tp	MWD
Bias	-0.09m	-0.19s	21.5 deg.
Root Mean Square Error (%)	4.0	17.5	22.8
Coefficient of Determination (R <sup>2</sup> )	0.95	0.66	0.42
Scatter Index (%)	5.2	4.1	0.1

Source: Mott MacDonald, 2021

**Figure 5-4 Significant wave height predicted by the MIKE21 FMSW model during the storm period's peak on 23 February 2017 at 12:00UTM.**



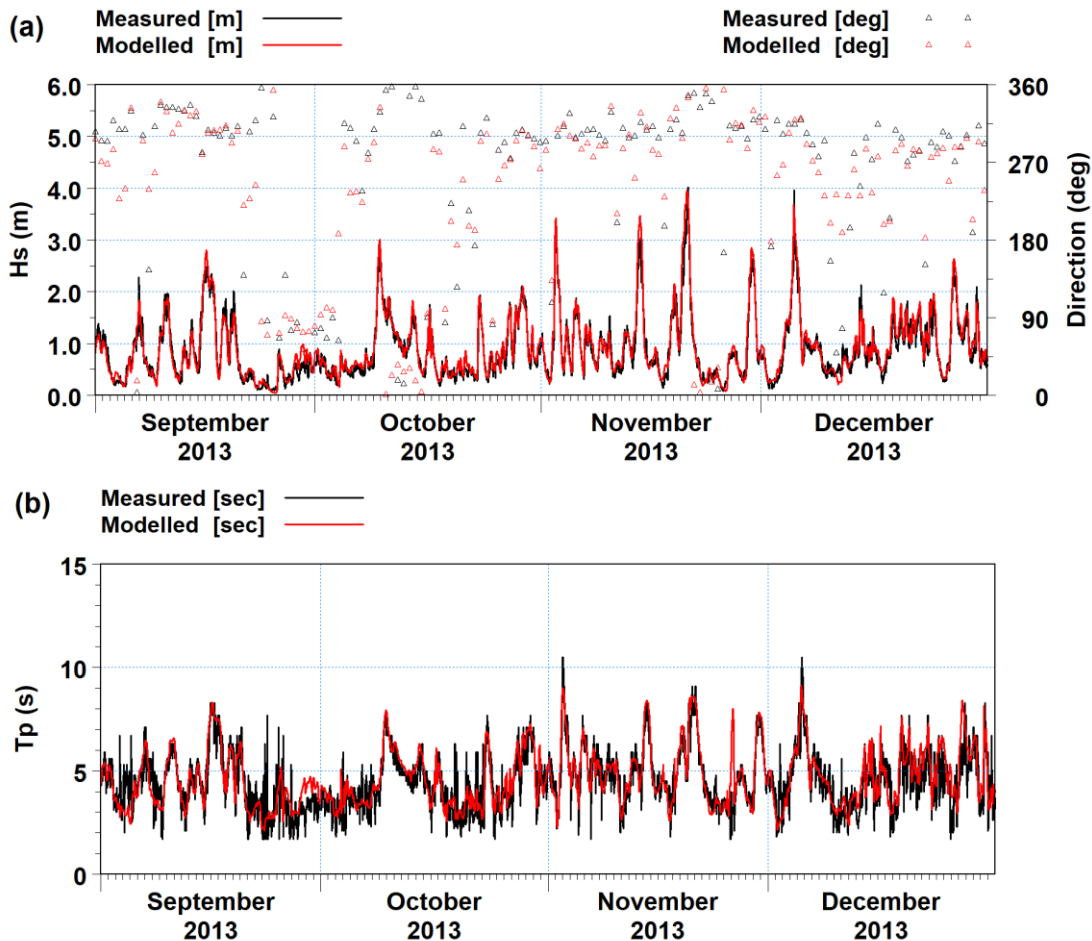
Source: Mott MacDonald, 2021

### 5.1.5 Model Validation

Validation of the wave model was undertaken by running the model for a different observational period. Figure 5-5 shows a comparison plot between measured and modelled data at Rhyl Flats wave buoy for Hs, Tp and MWD between September and December 2013. The model is shown visually to replicate the observed wave conditions well.

Statistics to quantify the model performance are summarised in Table 5-3. Based on the evidence provided in Figure 5-5 and Table 5-3, it is considered that the wave model again reproduces accurately wave conditions at the site and meet the NRW Guidelines [18] and the model performance criteria by Williams & Esteves [20]. Therefore, the model is judged to be suitable to provide reliable estimates of wave conditions in nearshore locations.

Figure 5-5 Comparison plots between measured and modelled data at Rhyl Flats wave buoy between September and December 2013 showing: (a) significant wave height, Hs with the mean wave direction, MWD; and (b) peak wave period, Tp.



Source Mott MacDonald, 2021

Table 5-3 Summary of the validated MIKE21 FMSW performance metrics at the Rhyl Flats wave buoy.

Parameter	Hs	Tp	MWD
Bias	-0.05m	-0.13s	20.6 deg.
Root Mean Square Error (%)	4.1	15.7	22.4
Coefficient of Determination (R <sup>2</sup> )	0.94	0.69	0.44
Scatter Index (%)	4.6	3.4	0.1

Source Mott MacDonald, 2021

## 5.2 2D wave transformation model

Before being transformed with a 1D model to the toe of the defences, extreme wave conditions from the 10,000-year synthetic record (Section 3.7) required transformation to the nearshore using the calibrated/validated MIKE21 FMSW model and an efficient emulator approach.

In the emulator approach, a subset of wave conditions and water levels sufficient to encompass the full range of offshore wave data was run using the MIKE21 FMSW model. The results from these simulations were used as a look-up table to derive transformation coefficients, which enabled the offshore wave dataset to be transformed inshore with accuracy and computational efficiency.

The wave conditions applied to the boundary of the SW wave model (for use with the emulator) were taken from binning the 10,000 synthetic dataset. Binning is the process of reducing the whole dataset into a smaller number of the dataset with a specified range of bin sizes. For example, if the Hs ranges from 0.5 to 12m, then a bin size might be defined as 0.5m so that each bin might represent Hs between 0.5 - 1m, 1 – 1.5m, 1.5 – 2.0m, ..... and 11.5 - 12m. The water level boundaries include the full range of climate change scenarios.

Table 5-4 shows the bins used to define the wave model's boundary conditions and subsequently used by the emulator to transform the offshore 10,000-year dataset to the inshore. The model forcing conditions such as wind speed and wind direction were derived using the offshore wave relationships detailed in Section 4, while the wave directional standard deviation (DSD) was applied as 33.47 deg. N based on the mean DSD extracted from WWIII 1516 dataset.

**Table 5-4: Defined offshore bins used for wave simulation**

Parameter	Min. centre bin condition	Max. centre bin condition	Interval
Hs (m)	0.01	13.4	0.5
Tp (s)	1.6	21.3	1
Water Level (m OD)	-4.5	7	0.5
MWD (degrees)*	0	360	10

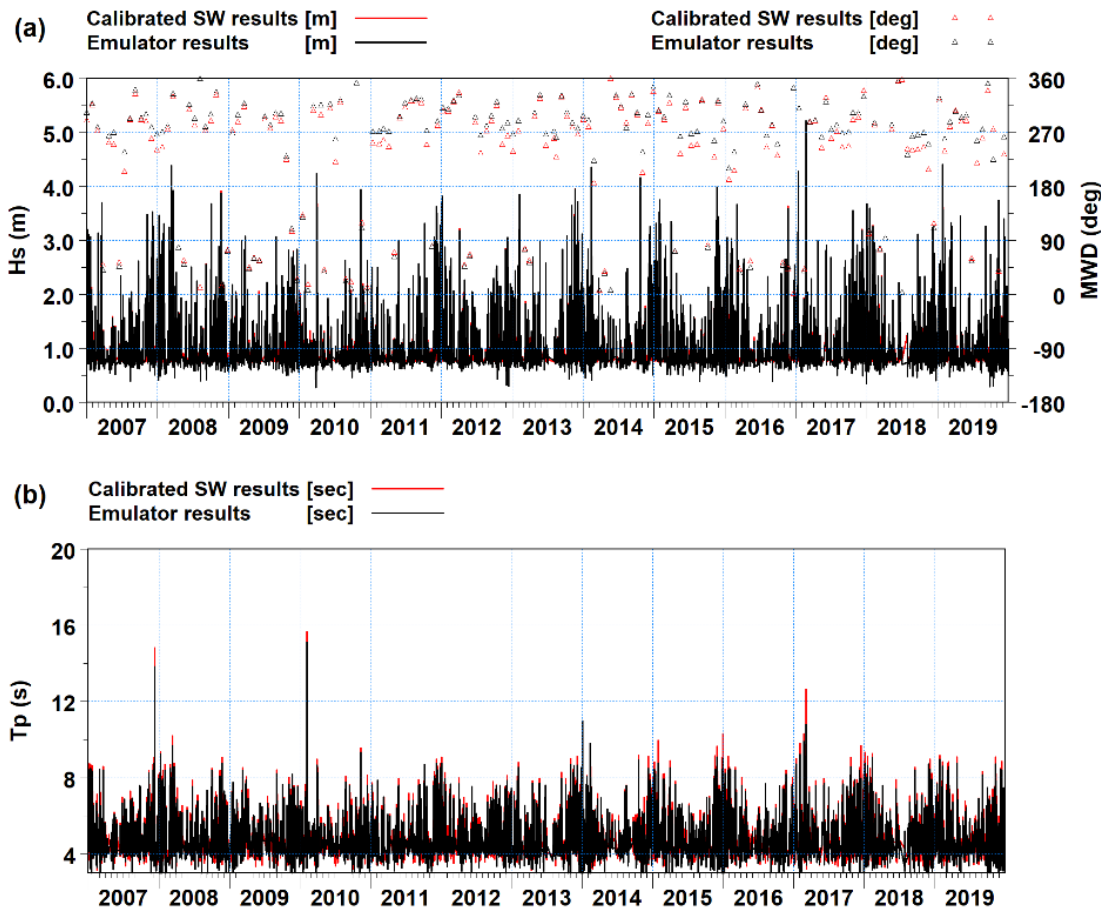
Source Mott MacDonald, 2021

The calibrated/validated MIKE21 FMSW model setup presented in **Table 5-1** was run in directionally-decoupled mode (parameterisation of the frequency domain) to simulate the binned events defined in Table 5-4. The model results were processed to provide a look-up table within the emulator.

The emulator works by considering which bin a particular offshore wave/water level value sits in before calculating the wave components' transformation coefficients. For example, for a given bin, Hs' transformation coefficient is defined as the inshore Hs result at the location of interest, divided by the offshore boundary (or binned) Hs. The offshore Hs wave condition is then multiplied by the wave transformation coefficient to get the nearshore wave condition. These calculations were undertaken for each of the wave parameters. However, the transformation coefficient was based on the change in direction rather than a ratio for wave direction.

The emulator assumes that the transformation coefficients are constant within each binned event. This is a valid assumption with the present bin resolution. As a check, Figure 5-6 shows a comparison plot between the emulator and calibrated MIKE21 FMSW model results at Rhyl Flats location for Hs, Tp and MWD. This figure shows that the emulator approach can reproduce wave conditions from offshore to inshore satisfactorily. However, the data indicate some bias for Tp and MWD. As this occurs only when Hs offshore is less than 1m, the error has no impact on the subsequent wave overtopping calculations.

**Figure 5-6 Comparison plot between the calibrated MIKE21 FMSW model results against emulator results at Rhyl Flat buoy for: (a) significant wave height,  $H_s$  with mean wave direction, MWD; and (b) peak wave period  $T_p$ .**



Source Mott MacDonald, 2021

### 5.3 1D Wave Transformation (using LITDRIFT)

While the MIKE21 FMSW wave modelling characterised well the nearshore wave conditions at the Central Rhyl frontage, this model's resolution was too coarse for defining the waves accurately at the toe of the sea defences for the overtopping calculations. To address this, the 1D wave transformation model LITDRIFT contained in the LITPACK suite of models<sup>21</sup> [21] was undertaken to propagate the nearshore waves from the MIKE21 FMSW wave model to the toe of the sea defences at 1m spatial resolution. LITDRIFT model consists a hydrodynamic model that calculates wave propagation towards the coast using a detailed hydrodynamics process across the profile such as shoaling, refraction, diffraction and wave breaking [21].

As model inputs, the LITDRIFT model used the nearshore wave/water level conditions output from the emulator (derived from the MIKE21 FMSW wave model) and the bed levels along with defined beach profiles. To run, LITDRIFT requires root-mean-square wave height ( $H_{rms}$ ), peak wave period ( $T_p$ ), water level (WL) and mean wave direction (MWD).  $H_{rms}$  was calculated

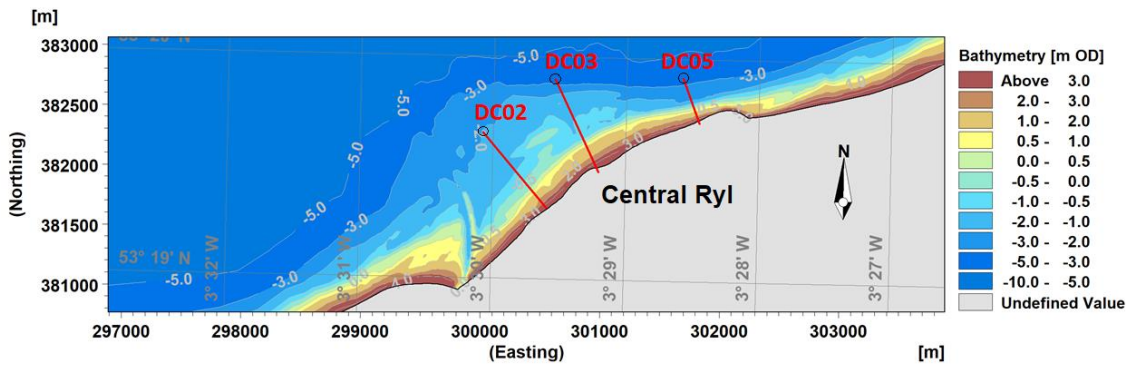
<sup>21</sup> DHI, "Littoral Processes FM - All Modules: Scientific Documentation," Hørsholm, Denmark, 2020.

based on the relationship of  $H_s = 1.416H_{rms}$ <sup>22</sup> [22], and the relationship between  $T_p$  and  $T_m$  (Section 4.4) was used to transform  $T_p$  to  $T_m$  for overtopping calculations.

A sensitivity test to identify the dominant MWD for the LITDRIFT model considered directional sectors of 0 to 360 degree at 10 deg. intervals. The test results show that 0 to 45 deg. N and between 225 to 360 deg. N are the dominant MWD.

Figure 5-7 shows the location of beach profiles DC02, DC03 and DC05. Each of these profiles is shown in Figure 5-8 for the baseline year (2020) and two epochs year (2070 and 2120). A detailed description of the selected baseline and future profiles is described in the Design Philosophy Report<sup>23</sup> [23]. For each of the nearshore wave conditions (baseline and two epochs year), the one-dimensional model was simulated, and a dataset at the toe of each of the profiles was extracted and subsequently used for overtopping calculations [6]. Figure 5-9 shows the results of  $H_s$  and WL extracted at the toe of each of the profiles for the baseline (2020) and two epochs year (2070 and 2120). It is indicated from the figure that  $H_s$  and WL increased from 2020 to 2070 and 2120, respectively, as expected. In addition, DC05 shows higher  $H_s$  compared to DC02 and DC03 due to a deep water influence (Figure 5-7 and Figure 5-8).

**Figure 5-7 Location of the profiles used in 1D wave transformation model**

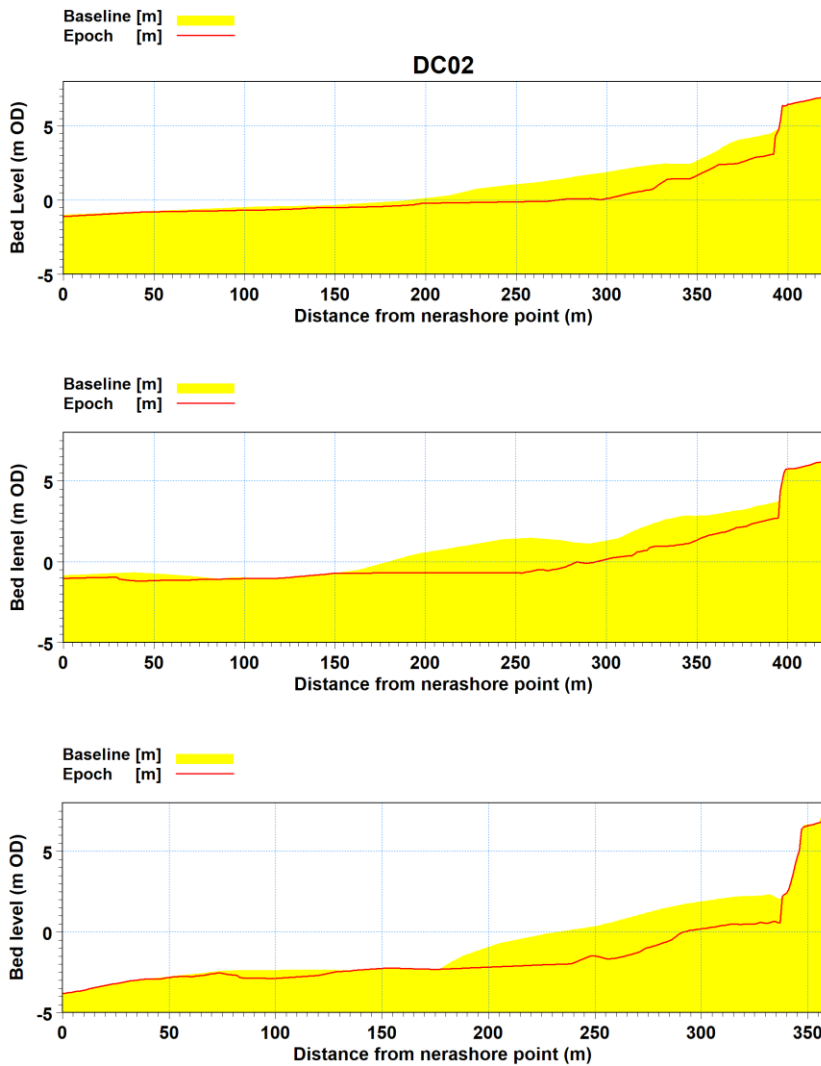


Source Mott MacDonald, 2021

<sup>22</sup> Z. Demirbilek and C. L. Vincent, *Coastal Engineering Manual, Part II, Water Wave Mechanics, Chapter 1, Engineer Manual 1110-2-1100, U.S Army Corps of Engineers, Washington, DC*, 2nd ed., vol. 1100, no. August. Vicksburg, Mississippi: Coastal and Hydraulics Laboratory (CHL), Engineer and Research Center, 2008.

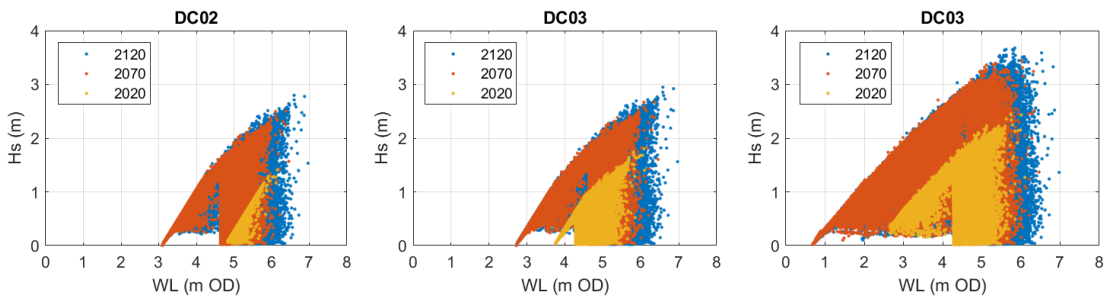
<sup>23</sup> Mott MadConal, "CR-MMD-00-00-RP-Z-0009 Central Rhyl Coastal Defences – Design Input Statement", 2021.

**Figure 5-8 Profiles used in 1D wave transformation model**



Source Mott MacDonald, 2021

**Figure 5-9 Hs and WL at the toe of each of the profiles for each epoch.**



Source Mott MacDonald, 2021

## 6 Summary

A two-dimensional (2D) spectral wave model (MIKE21 FMSW) has been developed to transform efficiently and accurately Met Office WWIII hindcast model wave data from the offshore point 1516 to the nearshore. The model was calibrated/validated against measured data from the Rhyl Flats wave buoy. Model performance metrics were demonstrated to meet NRW requirements and other established standards.

J Heffernan Consulting undertook a multivariate analysis of wave, wind and water level data to provide a 10,000-year synthetic record of offshore water level, Hs, Tp, MWD, wind speed and wind direction for the baseline (2020) and two epochs year (2070 and 2120) [4]. These data were binned and transformed to a nearshore location using the MIKE21 FMSW model and an emulator approach.

The resulting 10,000-year nearshore data set of wave and water level conditions were applied at the offshore boundary of 1D wave transformation LITDRIFT models at three cross-shore locations (DC02, DC03 and DC05) to provide extreme waves and water level conditions at the toe of the coastal defences for subsequent use in wave and still water overtopping calculations and scheme design calculations. [6].

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# Appendix 10.2 – Sediment Modelling Report



**Balfour Beatty**

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# **Central Rhyl Coastal Defences**

Sediment transport modelling

August 2021



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# Central Rhyl Coastal Defences

Sediment transport modelling

August 2021

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C01	25/08/2021	S.Costa N.Del Estal Dominguez	D.Price	J.Williams	Published version following no comments from Balfour Beatty

**Document reference:** 420823 | CR-MMD-00-ZZ-RP-SC-7007 | C01

**Information class:** Standard

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# Executive summary

Mott MacDonald Ltd has been appointed by Balfour Beatty Ltd (BB) on behalf of Denbighshire County Council (DCC) to undertake sediment and wave modelling as part of the Central Rhyl Coastal Defence Scheme. The study area comprises the coastline in Rhyl from Clwyd River's mouth to Splash Point, where East Rhyl Coastal Defence Scheme starts.

The Scheme, located in the central area of Rhyl waterfront, Denbighshire, North Wales, comprises coastal defence works including new rock armour, stepped concrete revetment, promenade raising, sea wall works, access and promenade improvements, along with associated tie-in work.

The comprehensive sediment study undertaken in this report included:

- An extensive literature and data review;
- The historic analysis of the beach frontage;
- Profile evolution modelling to support the design and understand the potential beach drawdown; and
- Coastline evolution modelling to understand the potential impacts of the Scheme on the wider North Wales coastline.

The study uses data from CCO surveys to describe the beach topography, and wave and water levels data from the Mott MacDonald spectral wave model. Sediment properties are defined by data from the geotechnical investigations undertaken for previous studies and the current Scheme. Sea level rise is based on UKCP18 and is used to estimate future conditions for the next 100 years.

The analysis of beach topography showed that the beach levels along the frontage are currently close to their historical minimum, and there is a general erosion trend in the eastern section of the frontage, close to profile DCC05 and Splash Point.

The beach profile response to storm events, with and without the Scheme, have been simulated using two independent 1D numerical models: (a) the profile evolution tool in the littoral drift module (LITPACK) from MIKE by DHI; and (b) infragravity (long-wave) and non-hydrostatic (wave-resolving) versions of the XBeach beach profile evolution model. Comparisons between predictions from these independent models provide a guide to model accuracy and reliability.

The results showed that a simulated 1% AEP storm results in beach drawdown of around 40cm. When climate change effects are included, beach drawdown increases by a further 10cm to 50cm. This value is less than the measured seasonal and inter-annual variability in beach levels and is attributable to greater wave runup and enhanced sediment mobility.

The results of the cross-shore modelling, when the proposed Scheme is included, are similar to the baseline, except for profile DCC02, where beach drawdown for present and future conditions reaches around 1m.

A Littoral Processes FM model (LPFM) was used to simulate the Scheme's impact along the adjacent downdrift shoreline over short to decadal time-scales. The model is based on several modules: (a) LITDRIFT to predict the longshore sediment transport of sediment for individual profiles along the frontage; and (b) a coastal evolution model to predict the long-term changes in coastal position.

Both modelling approaches showed that the Scheme's effects on the adjacent downdrift coastline are small and localised. Cross-shore impacts are confined to the upper beach and

only present when the water level is at or above the toe of the proposed structure. Changes to longshore transport and coastline position are undetectable at more than 400m from Splash Point. There are no effects observed at Prestatyn or the Point of Ayr.

# 1 Introduction

Mott MacDonald Ltd has been appointed by Balfour Beatty Ltd (BB) on behalf of Denbighshire County Council (DCC) to advise about sediment and wave modelling as part of the Central Rhyl Coastal Defence Scheme. The study area comprises the coastline in Rhyl from Clwyd River's mouth to Splash Point, where East Rhyl Coastal Defence Scheme starts.

The Scheme, located in the central area of Rhyl waterfront, Denbighshire, North Wales (Figure 1.1), comprises coastal defence works including new rock armour, stepped concrete revetment, promenade raising, sea wall works, access and promenade improvements, along with associated tie-in work.

**Figure 1.1: Location of Scheme**



Source: DCC, 2019.

The Scheme falls under sub-cell 11a of the North West & North Wales Shoreline Management Plan (SMP2). This subcell 11a extends from the Great Orme's Head (North Wales) to Southport (Figure 1.2).

**Figure 1.2: Overview of SMP2 Cell 11 area. Sub-cell 11a corresponds to the frontage where Rhyll is located.**



Source: Halcrow, 2011

## 1.1 Report structure

The report comprises the following Sections:

**Section 2:** Describes the different literature and data available for the study;

**Section 3:** Provides a detailed analysis of the existing and historic topography data for the frontage to identify trends in foreshore behaviour; and

**Section 4:** Describes numerical cross-shore sediment modelling used to investigate beach responses to extreme events in the present and future.

**Section 5:** Describes numerical coastline evolution modelling used to investigate the potential impacts of the proposed Scheme to the downdrift coastline, up to Point of Ayr.

**Section 6:** Provides a summary and conclusion of all the work undertaken in this report.

## 2 Data review

### 2.1 Summary of previous studies

While numerous Rhyl frontage studies have been undertaken in the past, we summarise in chronological order seven studies judged to be most relevant to the present sediment study.

#### 2.1.1 Coastal process Study – Rhyl to Prestatyn (HR Wallingford, 2008)

To assess the beach development and potential for overtopping and damage to the coastal defences, the HR Wallingford study evaluated coastal processes along the Denbighshire coastline. The study includes an overview of the coastline geology and geomorphology and provides a review of the nearshore waves conditions for normal and extreme events. These wave data are then used to model the alongshore sediment transport and demonstrate a net west to east transport along the North Wales coast.

A very detailed analysis of the beach profiles is also undertaken for the period 2002 to 2007. This study is especially relevant for the current Central Rhyl project, especially for the River Clwyd and Splash Point portion. The analyses indicate an accreting trend in the western portion of this Section of the frontage, mainly in the upper beach. In contrast, the beach close to Splash Point is generally eroding. We draw further on this report's contents in Section 3, dealing with the analysis of historical data.

#### 2.1.2 Cell 11 Regional Monitoring Strategy (CERMS) (Halcrow Group, 2010)

Based on coastal monitoring undertaken in 2010, the Halcrow report provides an update to the baseline summary for the Cell Eleven Regional Monitoring Strategy (CERMS). The Cell 11 coastal area extends from Great Orme's Head in North Wales to the Scottish border in the Solway Firth and is geographically split into five sub-cells, 11a to 11e. Central Rhyl is located in Cell 11a, which extend between Great Orme's Head on the North Wales coast and Southport. The report provides information regarding the offshore and nearshore waves and tidal conditions, and yearly potential sediment transport.

Specific to the Central Rhyl area, the report defines the Rhyl shoreline as orientated away from the prevailing wind and wave direction, increasing net eastwards sediment transport. In common with the HR Wallingford report, it also concludes that the foreshore immediately east of the Afon Clwyd is relatively stable, but beach levels reduce in an easterly direction, and the foreshore at Splash Point and across the Rhyl Golf Links frontage is eroding.

#### 2.1.3 Joint Probability Study (JPS) - SMP2 Supporting Studies (Halcrow Group, 2011a)

The Halcrow Group JPS report documents the input data sets, methods, and results from analysis at selected open-coast locations along the Cell 11 coastline of the joint probability (JP) for extreme nearshore waves and high water levels. It is provided as a supporting study to the North West England and North Wales Shoreline Management Plan 2 and was developed in parallel with coastal modelling studies within the Cell Eleven Tide and Sediment Studies (CETaSS), summarised below.

The Halcrow Group JPS report provides the detailed correlations between extreme wave heights and extreme water levels and the resulting estimates of the joint probabilities between wave heights and water levels for a series of annual exceedance probabilities (return periods). At each of the data extraction points along the frontage, the JPA results are provided as a set of charts and tables that help end-users who require estimates of nearshore wave and water level conditions for planning and coastal engineering applications.

#### **2.1.4 Cell Eleven Tide and Sediment Study (CETaSS) - SMP2 Supporting Studies (Halcrow Group, 2011b)**

Halcrow Group undertook a regional study of tides, waves, and sediment transport for coastal Cell 11 to support the Second round Shoreline Management Plan (SMP) development. It confirmed that the estuaries in Cell 11 had positive sediment budgets with sediment being supplied from offshore. Some modelling studies indicate that this supply of sediment from offshore will continue in the future and increase with sea-level rise. This study also corroborated the net easterly littoral sediment drift.

#### **2.1.5 Central Rhyl and Central Prestatyn Coastal Processes Report (JBA, 2018a)**

The JBA report was undertaken to support the Outline Business Cases (OBCs) for two coastal defence schemes: Central Rhyl; and Central Prestatyn. The report describes the coastal processes influencing these sites on a broad regional and local scale and provides a baseline for the impact assessment of the potential design options. There are several conclusions drawn by JBA which have relevance for the present study including:

- The prevailing wind and wave directions are cross-shore, from the west and south-west, and from the north-west, respectively;
- The tidal range is large and typically around 7m;
- The beaches are predominantly sandy, with some shingle on the upper beach;
- The primary direction of sediment transport is from west to east. In addition to the prevailing eastward longshore sediment transport, ridges and runnels along the Rhyl to Gronant coast, are evidence of complex cross-shore sediment transport processes; and
- Profile analysis confirms the general trend for beach lowering along the Central Rhyl foreshore identified in previous studies.

Other information from this report is considered in Section 3.

#### **2.1.6 East Rhyl Coastal Defence Scheme - Coastal processes assessment and modelling (JBA, 2018b)**

While the main focus of the JBA study was the East Rhyl coastal defence scheme, it also provides a general relevant overview of the sediment dynamics of the North Wales coastline to help the selection of a single preferred option and to the Welsh Government queries regarding the analysis of observational data and numerical modelling (waves, hydrodynamics and sediments).

As part of this coastal assessment and modelling exercise, and in common with previous studies, JBA identified the Rhyl coastline's key attributes, including onshore ridge-runnel dynamics and net eastward sediment transport along the beach face. JBA estimated an onshore delivery of sediment of approximately 20,000 m<sup>3</sup>/yr. This delivery has been attributed to the shoreward migration of sand ridges and sediment losses alongshore.

#### **2.1.7 Central Rhyl Coastal Defence Scheme – Final Report (APEM, 2021)**

APEM completed an environmental survey of the biology of the intertidal sediments of the Central part of Rhyl Beach in November 2020. During the survey, 36 core samples were acquired and analysed for species composition, and a further 12 samples were analysed to determine sediment composition.

Although the particle size analysis (PSA) data showed some variation in sediments across the survey area, the data showed the site is dominated by sand. Mean particle size ranged from 188µm to 2,230µm with an average particle diameter of 493µm across the entire survey area.

At most sample locations, sediments were well sorted or moderately well sorted and the sand component (63 to 2000µm) comprised the majority of the sediment samples.

Additional studies of secondary interest to the present study include:

- North West Estuaries Processes Reports – Dee Estuary (Halcrow Group, 2013a);
- North West Estuaries Processes Reports – Mersey Estuary (Halcrow Group, 2013b);
- Cell Eleven Wave, Tide and Sediment Study (Price *et al.*, 2010);
- Shoreline Management Plan SMP2 (Halcrow Group, 2012);
- Appendix C – Baseline Processes - SMP2 (Halcrow Group, 2011c);
- Regional modelling for improved shoreline management (Pontee *et al.*, 2011); and
- Rhyl – Prestatyn Coastal Defence Strategy Study Report (Martin Wright Associates, 2013).

While these studies provide further context for understanding the area's regional processes, they are not always specific to Central Rhyl frontage.

## 2.2 Hydrodynamic Water levels

The region experiences a macro-tidal climate, with an astronomical (mean spring) tidal range of approximately 7.5m at Rhyl frontage. The tidal range increases to the east of the site. Table 2.1 shows predicted tide levels for central Rhyl, Llandudno and Liverpool. Please note that the Rhyl tides levels have been estimated using distance weighing between the other two stations (Liverpool and Llandudno).

**Table 2.1: Astronomical tide level at Rhyl and nearby tidal stations**

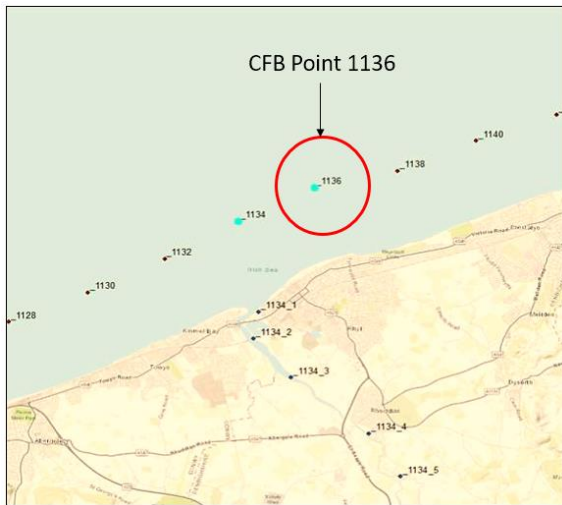
Tide Level	Tide level (mODN)		
	Llandudno	Rhyl*	Liverpool
HAT	4.75	5.03	5.37
MHWS	3.85	3.97	4.47
MHWN	2.05	2.17	2.57
MSL	0.21	0.22	0.37
MLWN	-1.55	-1.70	-1.73
MLWS	-3.35	-3.50	-3.83
LAT	-4.25	-4.56	-4.93

Source: Total Tide, 2021

\*Estimated by JBA, 2018 – Checked by Mott MacDonald, 2021

Measured water level data were available at Rhyl Harbour for the period from October 2008 to February 2017 and from the Llandudno (1971 to 2020) and Liverpool (1991 to 2020) tide gauges. Due to site constraints, the tide gauge does not capture the lower tidal stages. Extreme water levels are available from the Defra coastal flood boundary (CFB-2018) data set. In this study, CFB data were extracted at Location 1136 (Figure 2.1) and were corrected to account for sea-level rise to the year 2020 (baseline) using Met Office UKCP18 RCP 8.5 70<sup>th</sup> percentile (Table 2.2). For additional details regarding the climate change scenarios, please refer to section 2.5.

**Figure 2.1: Coastal Flood Boundary (CFB) extraction point 1136**



Source: Environment Agency, 2017

**Table 2.2: CFB water level at point 1136**

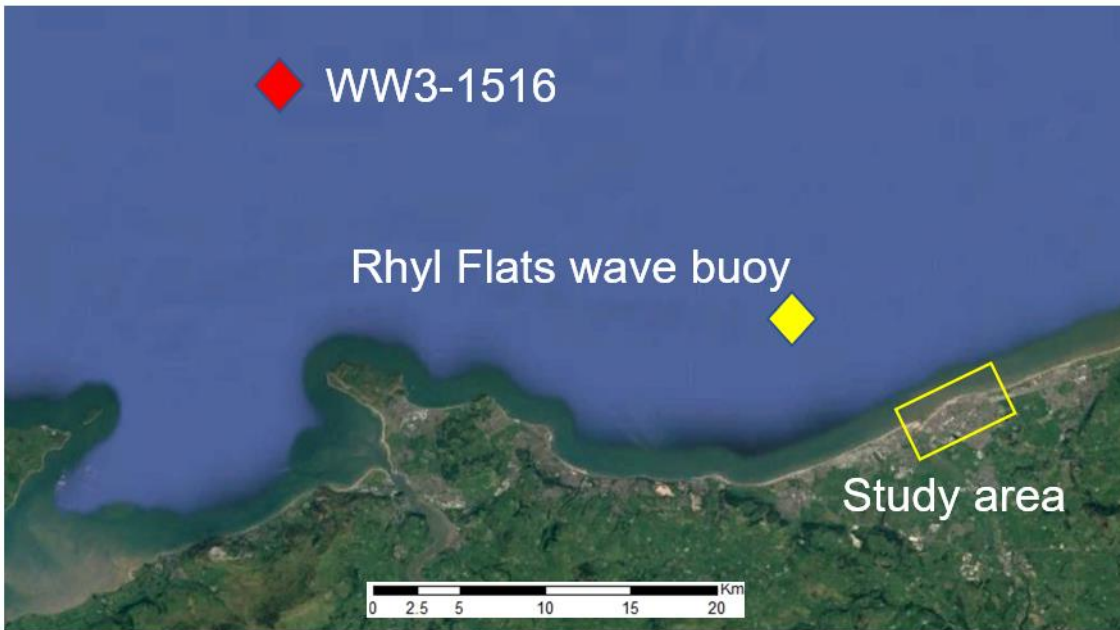
Return period	Annual exceedance probability (AEP) (%)	CFB water level (m ODN)
1	100	4.91
2	50	4.99
5	20	5.11
10	10	5.20
20	5	5.28
25	4	5.31
50	2	5.39
75	1.3	5.44
100	1	5.47
150	0.66	5.52
200	0.5	5.56
250	0.4	5.59
300	0.33	5.61
500	0.2	5.68
1000	0.1	5.77
10000	0.01	6.10

Source: Environment Agency, 2017 and Mott MacDonald 2021

### 2.2.1 Waves

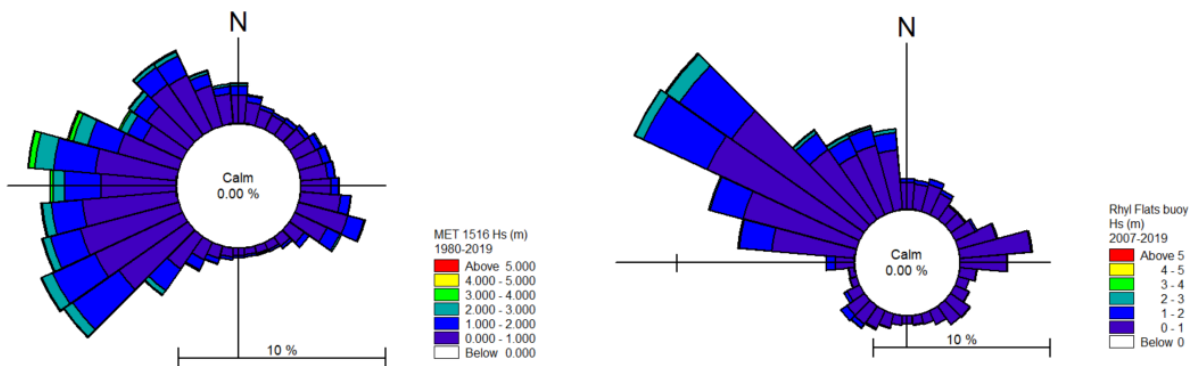
Offshore hindcast waves from the Met Office WaveWatch III (WWIII) model at point 1516 (January 1980 to December 2019) were made available for this study. The Channel Coast Observatory (CCO) wave buoy data at Rhyl Flats was obtained from January 2007 to December 2019. Figure 2.2 shows the location of the above dataset, and Figure 2.3 shows the corresponding wave roses.

**Figure 2.2: Location of the wave data available for this study. The yellow box highlights central Rhyl.**



Source: Mott MacDonald, 2021

**Figure 2.3: Wave roses. Left – Met Office Wave Watch III point 1516. Right – Rhyl Flats wave buoy.**



Source: Mott MacDonald, 2021. Includes Met Office and CCO data, 2021.

At Met Office Wave Watch III point 1516, Figure 2.3 shows that the dominant offshore waves are from the west-south-west direction. In contrast, due to the buoy location closer to shore, the dominant wave direction is from the north-west at the Rhyl Flat wave buoy. Extreme offshore waves calculated at the Met Office location are shown in Table 2.3.

**Table 2.3: Extreme offshore waves at the Met Office 1516 point.**

Return Period	Annual exceedance probability (%)	Significant wave height, Hs (m)
1	100	4.37
5	20	5.20
10	10	5.61
25	4	6.14

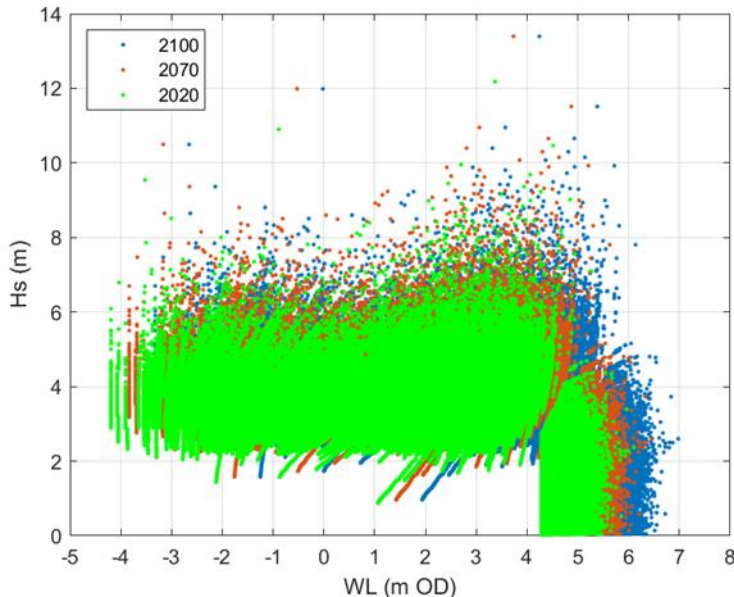
Return Period	Annual exceedance probability (%)	Significant wave height, Hs (m)
50	2	6.52
100	1	6.91
200	0.5	7.29
500	0.2	7.80
1000	0.1	8.18
5000	0.02	9.07
10000	0.01	9.45

Source: Mott MacDonald, 2021.

A multivariate analysis (MVA) was undertaken using WWIII 1516 (40-year) and water level data at Rhyl Harbour da to provide a 10,000-year synthetic record of water level, Hs, Tp, MWD, wind speed and wind direction. This analysis was undertaken by JHC (2021) and included the allowances for climate change.

Figure 2.4 shows an example of a scatter plot of the offshore 10,000-year synthetic record of Hs and water level extracted from the MVA results for the baseline year 2020 and two epochs year, 2070 and 2120. It is observed that as expected Hs and WL values for the baseline increased for each of the epoch years. Further details of the MVA are given in "JHC Extreme Value Analysis report, 2021" and Central Rhyl Wave Model report (in Progress, Mott MacDonald).

**Figure 2.4: Scatter plot of the 10,000-year synthetic record of Hs against water level (WL) for 2020, 2070 and 2120**

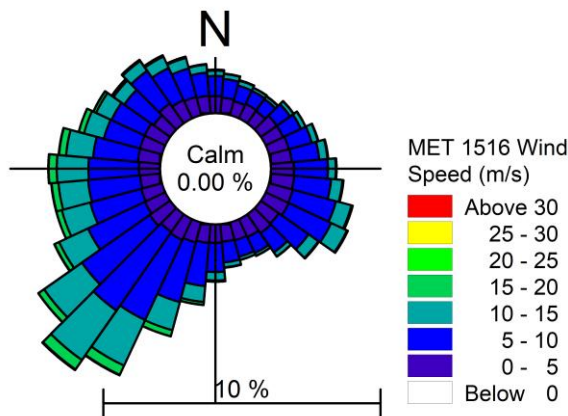


Source: J Heffernan Consulting, 2021

### 2.2.2 Wind

Offshore wind conditions are defined using the hindcast Met Office WWIII point 1516 dataset (January 1980 to December 2019). The wind rose of these data in Figure 2.5 shows that the dominant offshore wind direction is from the south-west.

**Figure 2.5: Wind rose for the Met Office WWill point 1516.**

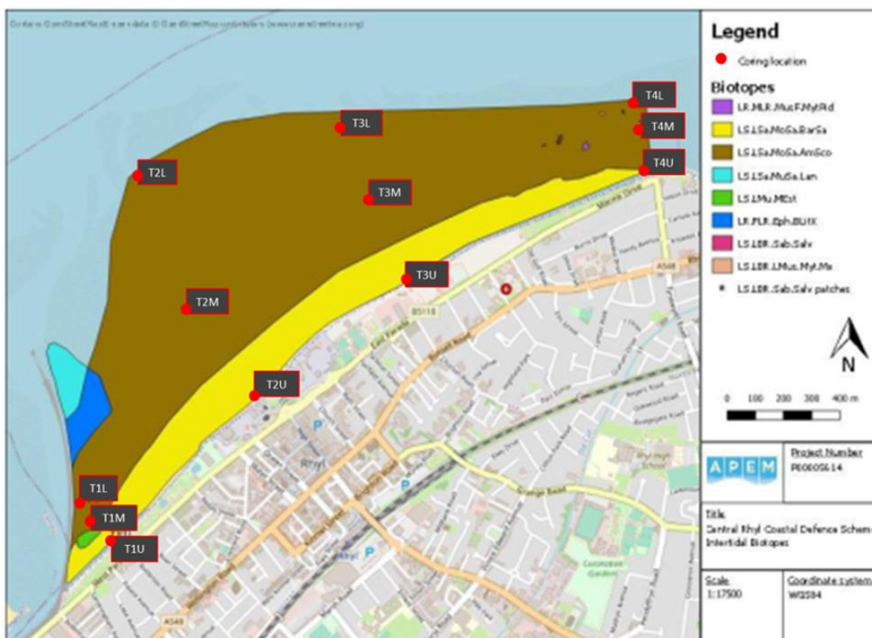


Source: Mott MacDonald, 2021

### 2.3 Sediment data

As part of the Central Rhyl Coastal Defence Scheme, Mott MacDonald appointed APEM Ltd to carry out an environmental and sediment survey. Their scientific report (Ref. P00005614), completed in November 2020, presents the study's findings. Here we consider the physical properties of twelve beach sediment samples obtained at the upper, middle, and lower beach profile sections (Figure 2.6).

**Figure 2.6: Sediment sampling locations**



Source: APEM, 2021

A Particle Size Analysis (PSA) of the twelve beach sediment samples is summarised in Table 2.4. This defines the sediment and quantifies the mean grain size and the fractions *d50* (median), *d10* (10% of the sample smaller than) and *d90* (90% of the sample smaller than).

**Table 2.4: Particle Size Analysis for the sediment samples**

	Mean (µm)	Description	<i>d50</i> (µm)	<i>d10</i> (µm)	<i>d90</i> (µm)
T1 U	2230	Very fine gravel	1679	178	36364
T1 M	221	Fine Sand	219	139	343
T1 L	188	Fine Sand	231	24	343
T2 U	1202	Very Coarse Sand	337	194	2663
T2 M	273	Medium Sand	270	155	471
T2 L	250	Medium Sand	248	154	433
T3 U	268	Medium Sand	268	182	428
T3 M	260	Medium Sand	265	179	397
T3 L	252	Medium Sand	251	166	401
T4 U	308	Medium Sand	296	190	5544
T4 M	209	Fine Sand	201	63	1466
T4 L	251	Medium Sand	246	141	530

Source: APEM, 2021

Table 2.4 and Table 2.5 shows the sampled sediments are dominated by medium sand with an average particle diameter across the study area of 249µm, obtained only by averaging the medium and fine sand samples of Table 2.4. However, at some localised sites, there is a relatively broad range of mean particle sizes across the sampling area that ranges from approximately 188 µm (fine sand) at sample point T1 L to 2230.6 µm (very fine gravel) at sample point T1 U.

**Table 2.5: Summary particle size data at each sampling location.**

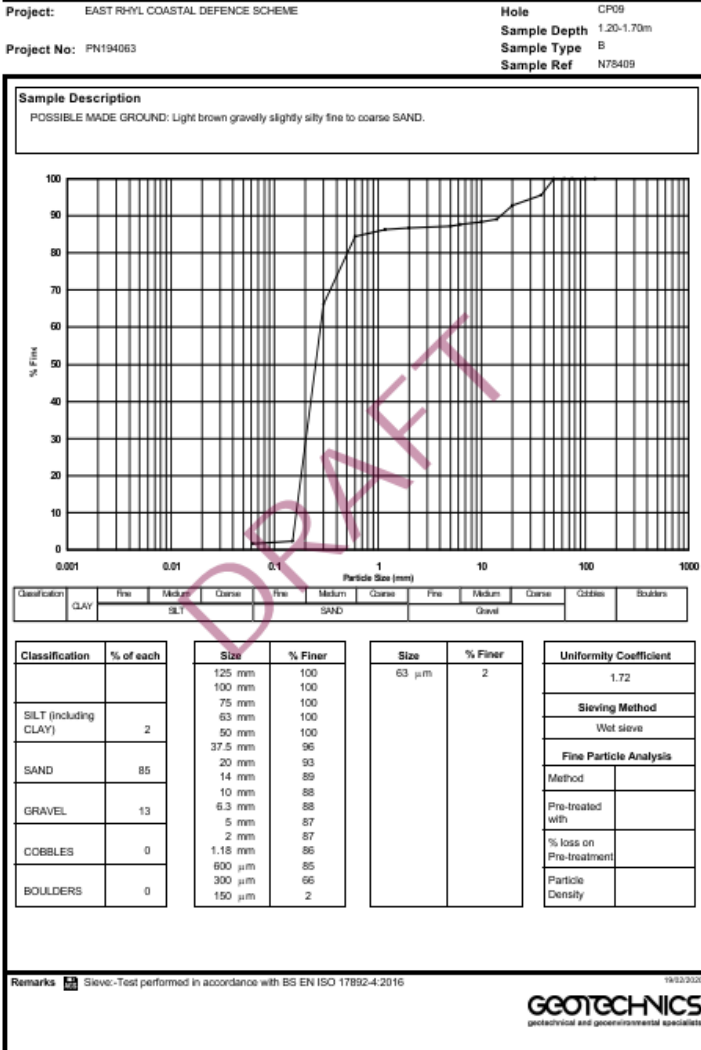
Sample ID	Gravel (>2mm) (%)	Sand (63 to 2000 µm) (%)	Mud (<63 µm) (%)
T1 U	48.7	46.4	4.9
T1 M	1.8	95.5	2.7
T1 L	0.7	85.1	14.2
T2 U	28.2	71.8	0.0
T2 M	1.5	97.1	1.5
T2 L	2.3	96.4	1.3
T3 U	0.4	99.6	0.0
T3 M	0.1	98.7	1.3
T3 L	0.2	99.8	0.0
T4 U	11.1	88.9	0.0
T4 M	8.4	81.9	9.7
T4 L	3.8	94.8	1.4

Source: APEM, 2021.

JBA Consulting acting on behalf of their client Balfour Beatty Civil Engineering Limited and Denbighshire County Council, commissioned Geotechnics Limited to undertake geotechnical and geo-environmental investigations during 2017, 2018 and 2019. Factual reports on the ground and groundwater conditions were issued with the identifications PN173706, PN183790 and PN194063. These investigations also concluded that the project site is dominated by medium sand (Figure 2.7).

**Figure 2.7: Particle size distribution of Borehole CP09 at 1.20 to 1.70m depth. Borehole CP09 is situated to the right of profile DCC05.**

**LABORATORY RESULTS - Particle Size Distribution**

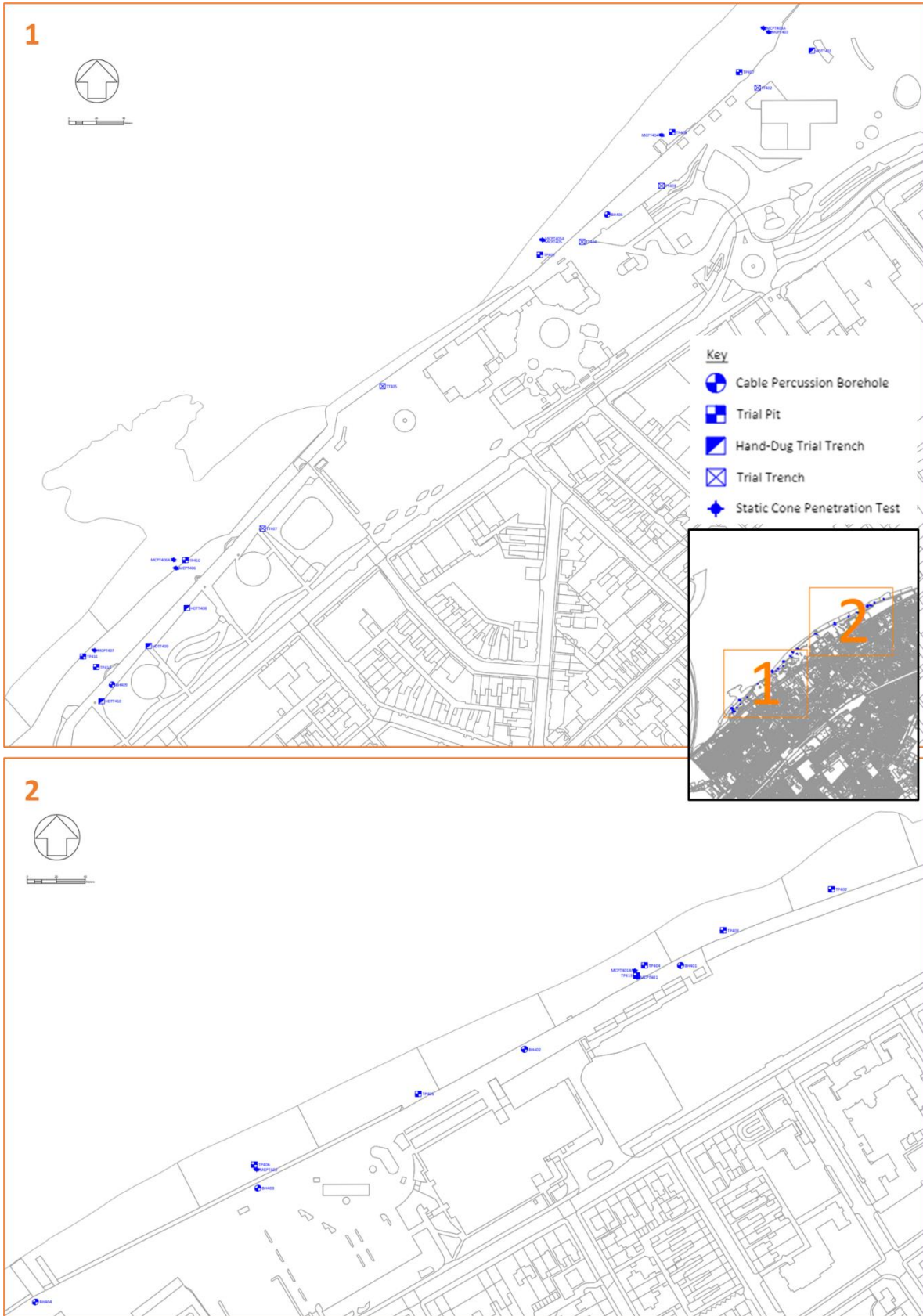


Source: Geotechnics, 2019.

More recently, in March 2021, a geotechnical investigation was commissioned to Geotechnics Limited by Mott MacDonald as part of the detail design of the Central Rhyl Defence Scheme. The object of the investigation was to obtain information on the ground and ground water conditions relating to the design of the proposed works.

As part of the GI, boreholes and trial pits were taken at different depths and were analysed to determine PSD. Both boreholes and trial pits locations were limited to the upper part of the beach, toe of the defences and promenade (Figure 2.8). A summary of the results from the trial pits is provided in Table 2.6.

Figure 2.8: Location of the Geotechnics Limited surveys.



Source: Modified from Geotechnics Limited, 2021

**Table 2.6: Summary of surface sediment grain size from the trial pits obtained by Geotechnics Limited survey**

Trial Pit	D <sub>50</sub> (mm)	Description of sediment
TP402	0.27	First 1m: Brown slightly gravelly slightly silty fine to medium SAND
TP403	0.27	First 2 metres: brown slightly gravelly slightly silty fine to medium SAND
TP404	0.28	First 0.5m: Brown gravelly slightly silty fine to medium SAND.
TP405	0.27	First 0.5m: Light brown slightly gravelly slightly silty fine to medium SAND.
TP406	0.32	First 0.5m: Brown very gravelly slightly silty fine to coarse SAND.
TP407	0.28	From 1m deep to 1.5: Brown gravelly slightly silty fine to coarse SAND.
TP408	0.25	From 2m deep to 3m: Brown mottled dark brown slightly gravelly slightly silty fine to coarse SAND.
TP409	4.22	From 0.5m deep to 1m: Brown very sandy slightly silty fine to coarse GRAVEL.
TP411	5.71	First 0.7m: Light brown very sandy slightly silty fine to coarse GRAVEL. And then from 1m deep to 2m: Brown gravelly to very gravelly fine to medium SAND.
TP412	0.59	First 1m deep: Brown very gravelly slightly silty medium to coarse SAND.

Source: Mott MacDonald, 2021. Contains Geotechnics Limited, 2021

## 2.4 Topographic data

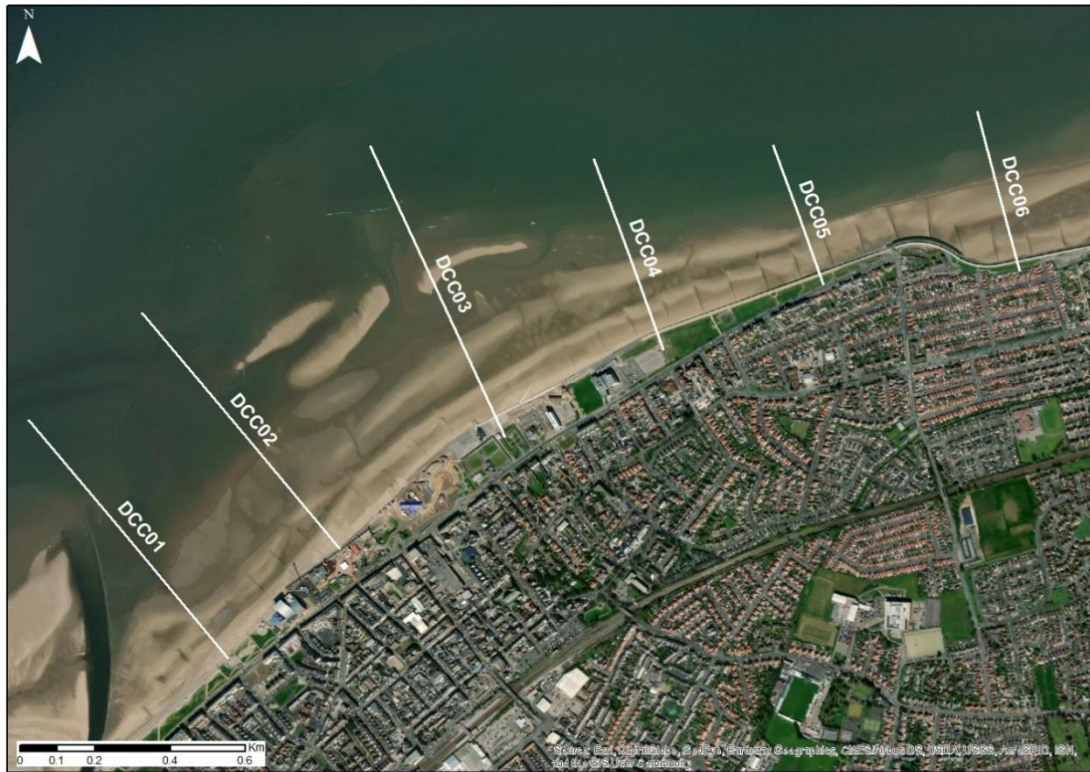
### 2.4.1 Beach profiles

Since the establishment of a coastal monitoring programme in 2002, Denbighshire County Council has undertaken bi-annual beach profiles at 26 location spanning the coast and including west, central and east Rhyl. For the present study, we examine Denbighshire County Council beach profiles obtained between 2002 and 2014 and two additional surveys in March 2019 and June 2020 from the Coastal Channel Observatory website<sup>1</sup>. All profiles are in British National Grid and vertically references to mODN. Figure 2.9 show the location of the selected profiles for this study.

Denbighshire County Council also provided an additional post-storm survey from December 2013. The survey was undertaken after a storm believed to have occurred on 5th December 2013 (but not confirmed). The survey comprises a series of topography points from a small portion of the Central Rhyl frontage and therefore does not have the same profiles locations as the bi-annual surveys (Figure 2.8). This post-storm survey is not used in the historical analysis of Section 3, but it is included as part of the model validation of Section 4.

<sup>1</sup> [Channel Coastal Observatory Filter](#)

**Figure 2.9: Location of the Denbighshire County Council beach profiles analysed in this study.**



Source: Mott MacDonald, 2021

**Figure 2.10: Location of topographic data points in a post-storm beach survey in December 2013.**



Source: Mott MacDonald, 2021 with data from Denbighshire County Council, 2013

## 2.4.2 LiDAR

Composite and historic LiDAR data have been obtained from NRW online archives<sup>2</sup>. Composite LiDAR is derived from a combination of all available data were merged and re-sampled to give the best possible coverage. Table 2.7 summarises these data and defines the spatial resolution by year.

**Table 2.7: Summary of LiDAR data available for the Central Rhyl frontage.**

LiDAR year	Resolution (m)	Additional information
Composite LiDAR	1	Identical to the 2015 dataset
2015	0.5	
2011	1	
2009	1	
2004	2	

Source: Natural Resources Wales, 2021

## 2.5 Climate change

Sea level rise (SLR) used in this study follows the UK Climate Projections 2018 (UKCP18) from the Met Office. The UKCP18 mean sea level projections include three Representative Concentration Pathway (RCP) climate change scenarios, RCP2.6, RCP4.5 and RCP8.5. For this study, Mott MacDonald has followed RCP8.5 at the 70<sup>th</sup> percentile (refer to Figure 2.11 to locate where the data has been extracted). This scenario assumes a higher emission scenario in which there is a linear adjustment of emissions from 2150 and CO<sub>2</sub> stabilisation only after 2250.

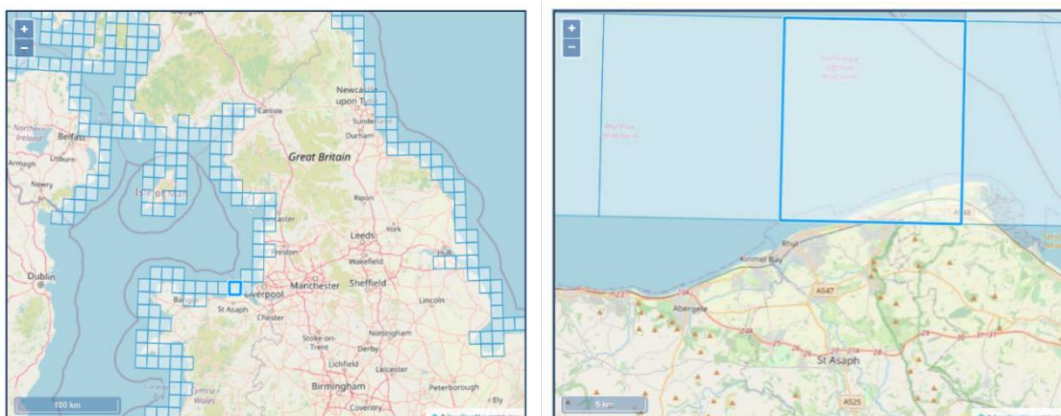
The values to be used for the two-future epoch (2070 and 2120) are shown in Table 2.8.

**Table 2.8: Sea level rise based on UKCP18 RCP8.5 70<sup>th</sup> percentile**

Year	Estimated SLR (m)
2020	0
2070	0.36
2120	0.87

Source: Mott MacDonald, 2021 based on data from Met Office UKCP18.

**Figure 2.11: Sea level rise extracted from UKCP18 database grid location**



Source: Met Office UKCP18, 2018.

<sup>2</sup> [NRW Historic LiDAR Archive \(arcgis.com\)](https://arcgis.com)

## 3 Historical analysis of beach topography

### 3.1 Beach profile analysis

Beach profile data described in Section 2.4.1 were examined graphically to establish the envelope of variability between 2002 to 2020. Figure 3.1 shows plots for profiles DCC01, DCC03 and DCC05 (Figure 2.9). The complete set of plots for all beach profiles is shown in Figure A.1, Appendix A. The key features of these profiles can be summarised as:

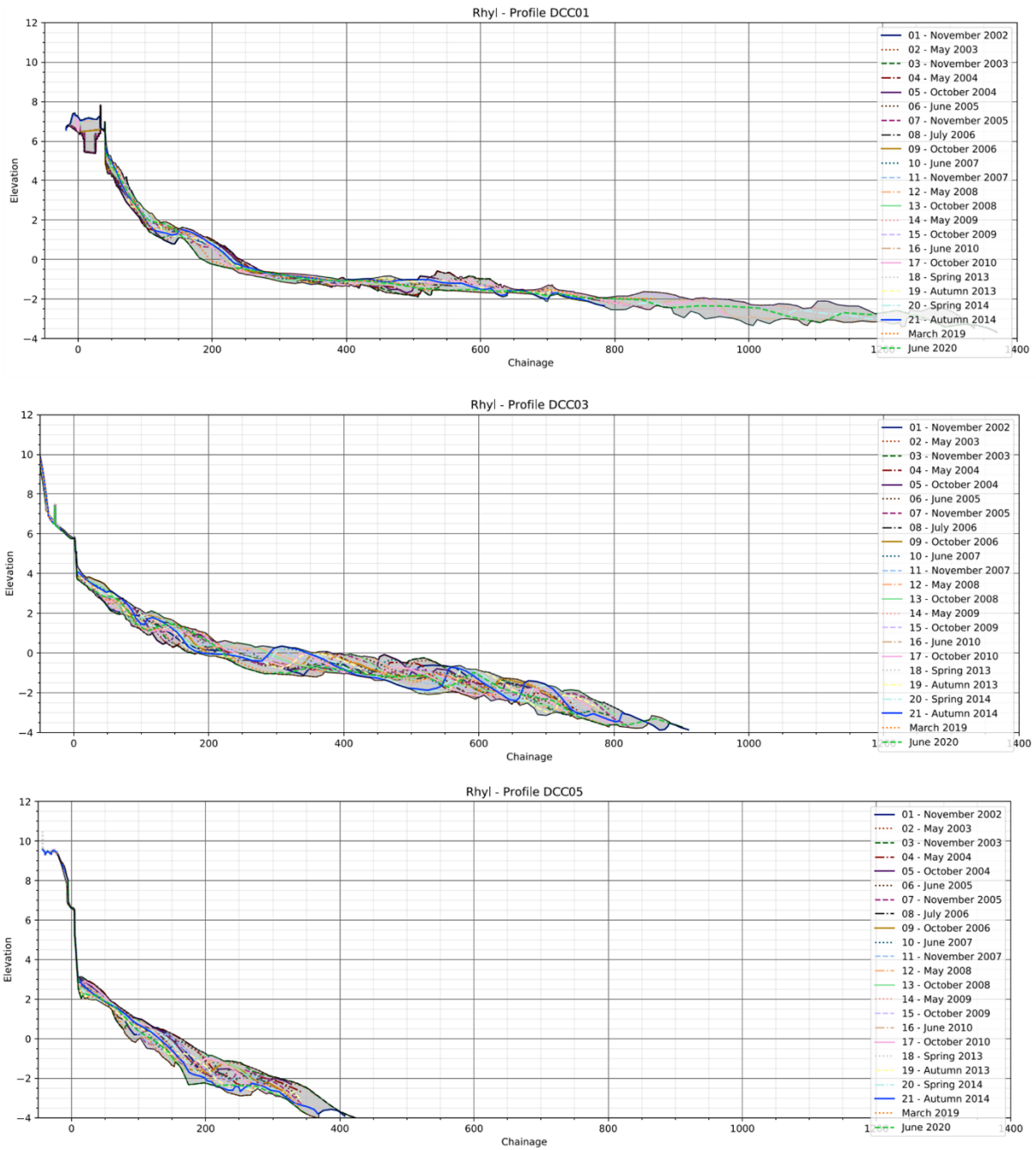
- The cross-shore extent of the beach profiles shown in Figure 3.2 varies considerably and decrease in length towards the east so that profiles DCC05 is only ~400m long, while profile DCC02 has a maximum cross-shore extent of 1400m;
- The envelope of change of profiles DCC01 and DCC02 is very similar. Both profiles show a considerable change in the first 200m from the coastline, with beach levels varying by +/-1m;
- Beach level changes between the 200m and the 500m chainages in profiles DCC01 and DCC02 are small. The data show that this beach area is very stable over the surveyed period (2002 to 2020). This area is adjacent to the river mouth and the river training wall, which may stabilise and protect this part of the foreshore;
- Sandbar/ridge features become more evident further eastwards along the frontage and are observed clearly in profiles DCC03 to DCC05 over the majority of the surveyed topography;
- Temporal changes in eastern beach profiles occur more rapidly as demonstrated by the larger envelope of change (+/-2m) and the on/offshore migration of sandbars over time; and
- The beach slope becomes steeper in the eastward direction.

All profiles show beach level changes up to +/-1m at the defences' toe (DCC02, DCC05). Although these changes are smaller in magnitude than those in the sandbar areas, they are nevertheless significant when considering coastal defence design.

To place the current beach topography within the broader historical context, the most recent beach survey in June 2020 has been plotted with the beach change envelope for profiles DCC01, DCC03 and DCC05 (Figure 3.2). The complete set of plots for all profiles is included in Figure A.1, Appendix A. For several of the profiles, Figure 3.2 shows that beach levels at the time of the June 2020 survey are close to the lowest recorded historical levels at the toe of the defences and over the sandbars.

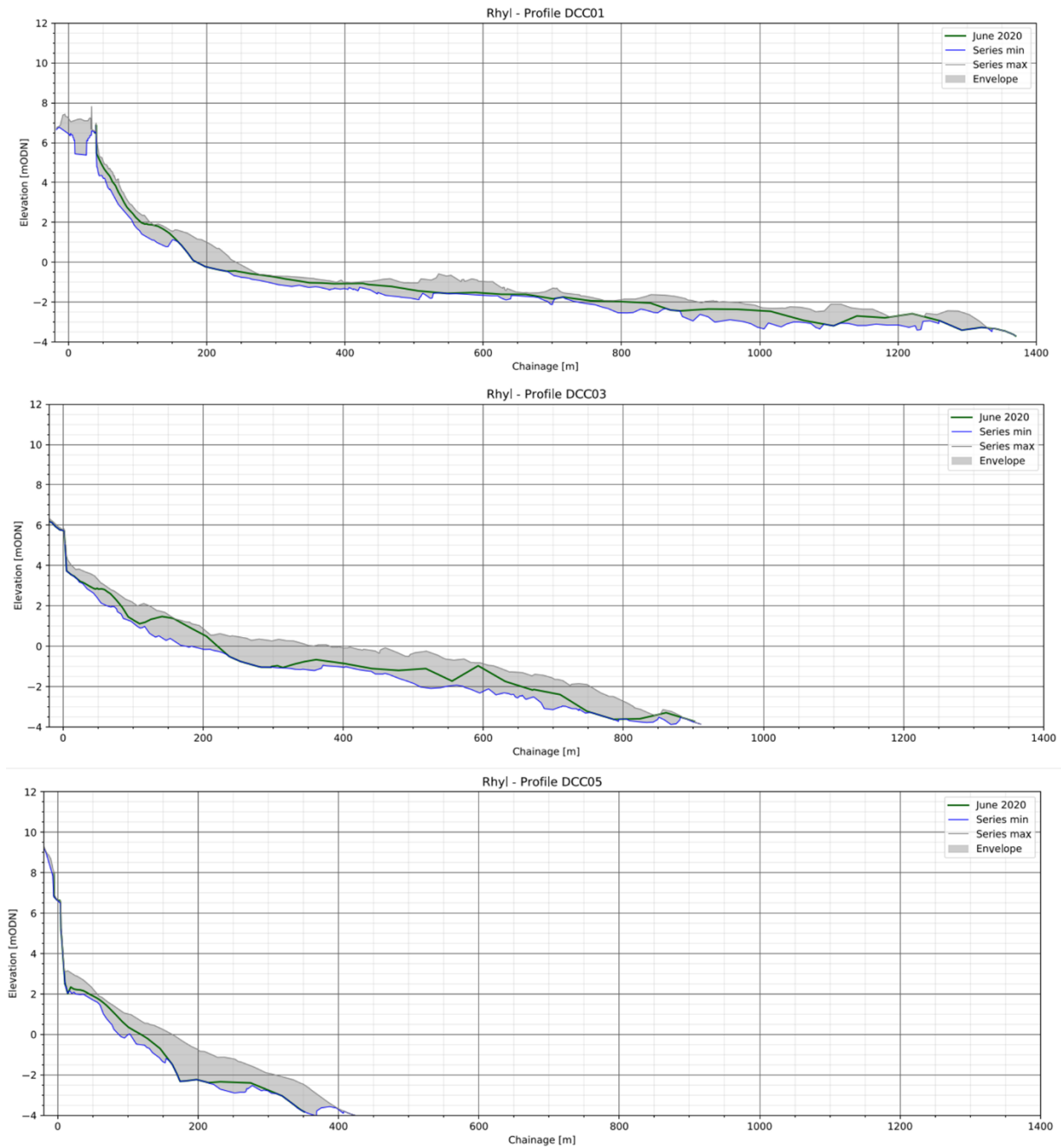
When considering the overall beach profile behaviour along the frontage, the data demonstrate two opposing trends. On the one hand, the eastern part of the frontage shows an erosional trend of decreasing beach levels which reach **the minimum historical value** in June 2020. On the other hand, at profile DCC01, the beach levels at the toe of the defences are accretionary and are currently close to **the upper historical limit**. As previously mentioned, profile DCC01 is located next to the river mouth, close to the training wall, which acts as a hydraulic groyne.

Figure 3.1: All beach surveys (2002 to 2020) for profiles DCC01, DCC03 and DCC05.



Source: Mott MacDonald, 2021. Includes Denbighshire County Council and CCO data.

**Figure 3.2: June 2020 survey, plotted with the data envelope for profiles DCC01, DCC03 and DCC05.**



Source: Mott MacDonald, 2021. Contains Denbighshire County Council and CCO data.

To further understand the historical beach profile behaviour shown in Figure 3.1 and Figure 3.2 and to identify beach profiles trends and possible correlations with the wave climate, beach level values were extracted from summer/autumn and winter/spring profiles at selected distances from the coastal defences.

Figure 3.3 to Figure 3.5 shows for profiles DCC01, DCC02, DCC03 and DCC05 the change in bed levels at the selected chainage. For simplicity, a linear regression line through the data is used to illustrate the profile's general behavioural trend by year and season. The complete set of plots for all profiles is provided in Appendix B.

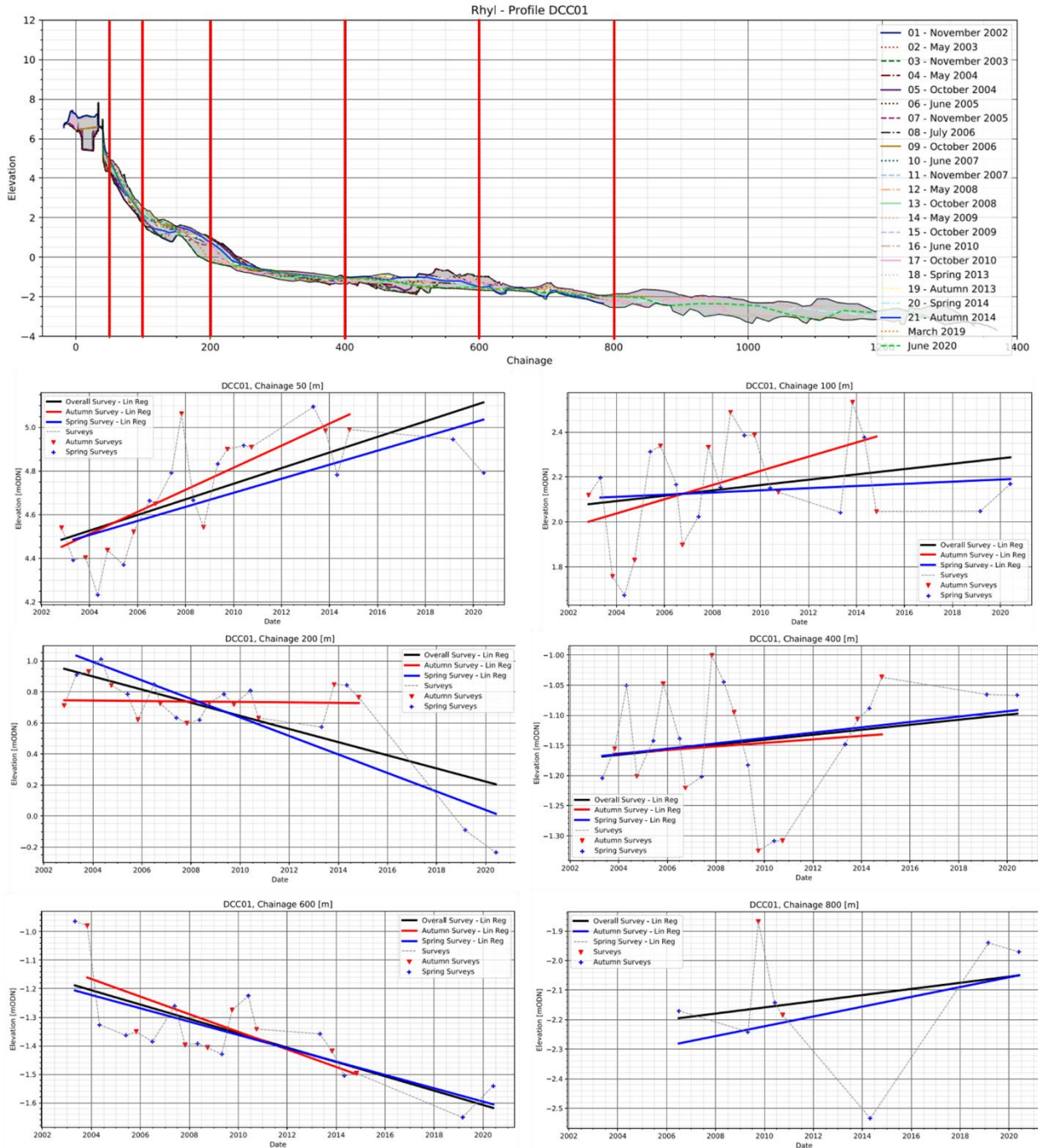
Figure 3.6 summarises the temporal changes in the average regression line gradients (m/year) at various cross-shore distances for all profiles. It is noted that data at some of the offshore locations are too sparse to establish statistically significant gradients and should not be considered further. The complete set of plots, including the season analysis, is included in Figure B.9, Appendix B.

The product-moment correlation coefficient ( $R^2$ ) for the linear regression trends derived from the overall survey data are shown in Figure 3.7. Similar to the slope analysis, it should be noted that some of the "offshore" chainages analysis (>600m) only had a few surveyed points, and therefore there are not statistically significant. The goodness-of-fit ( $R^2$ ) for the season analysis is in Figure B.10, Appendix B.

The p-values for the linear regression derived from the overall survey data are shown in Figure 3.8. Similar to the slope analysis, it should be noted that some of the "offshore" chainages analysis (>600m) only had a few surveyed points, and therefore there are not statistically significant. The p-values for the season analysis is in Figure B.11, Appendix B.

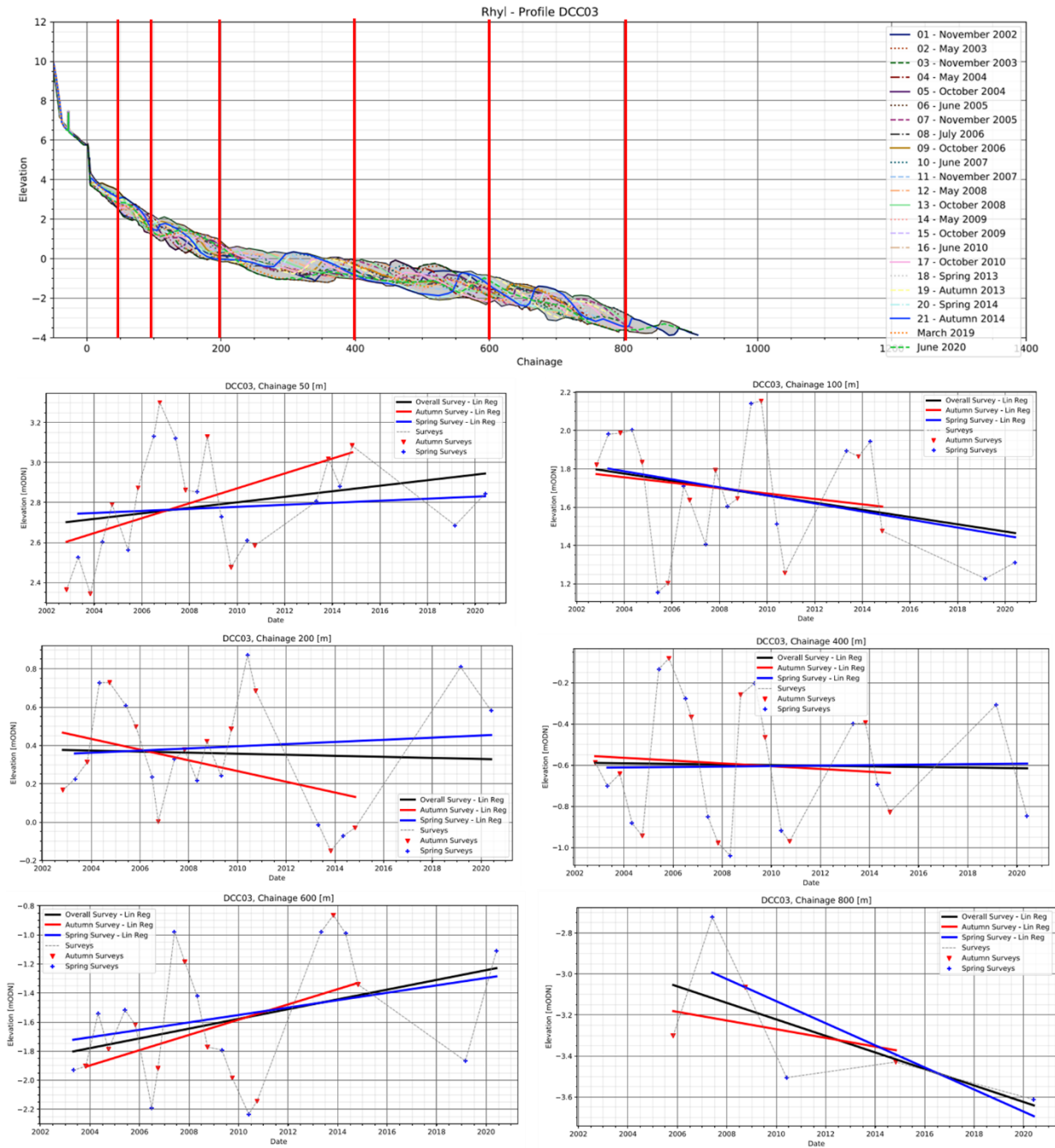
The p-values measure the statistical significance in a hypothesis test, in this case, that the slope is zero. The smaller the p-value, the stronger the evidence that the null hypothesis is rejected. A p-value less than 0.05 (typically  $\leq 0.05$ ) is statistically significant. For example, a low p-value means that the estimated linear regression trend/slope is statistically significant.

**Figure 3.3: Profile DCC01 chainage analysis.** The red lines show the location of the cross-shore sections considered. The plots include all years and distinguish between the season (summer/autumn in red and winter/spring in blue). Linear regression has been fitted through all the data by season.



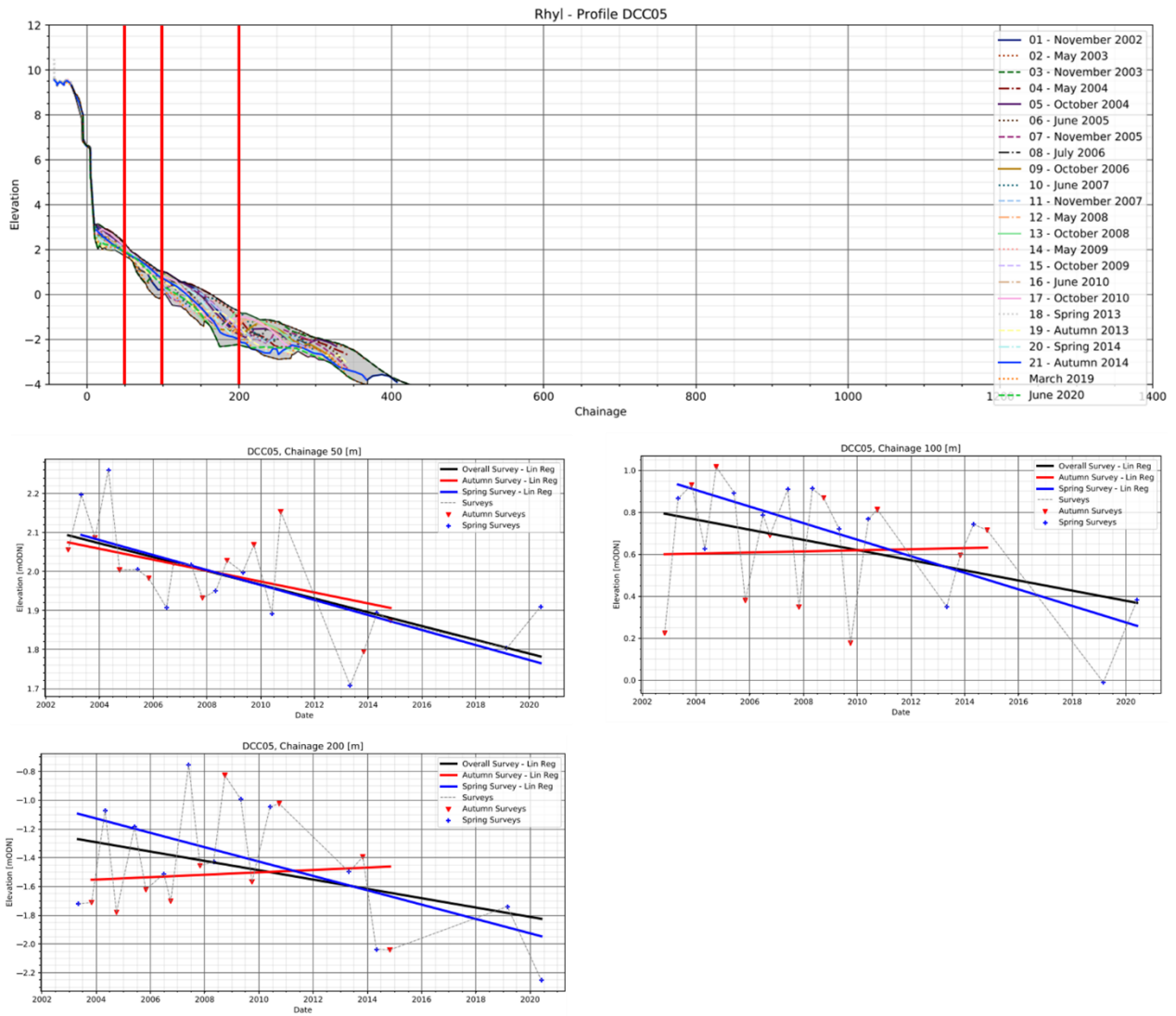
Source: Mott MacDonald, 2021. Contains Denbighshire County Council and CCO data.

**Figure 3.4: Profile DCC03 chainage analysis.** The red lines show the location of the cross-shore sections considered. The plots include all years and distinguish between the season (summer/autumn in red and winter/spring in blue). A linear regression has been fitted through all the data by season.



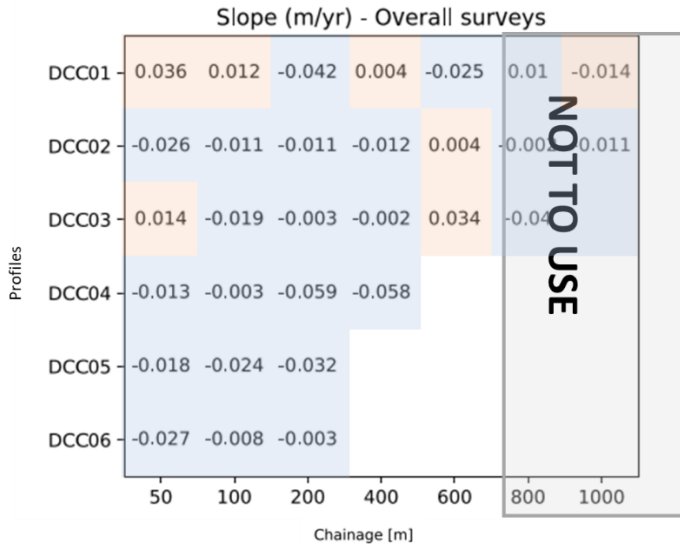
Source: Mott MacDonald, 2021. Contains Denbighshire County Council and CCO data.

**Figure 3.5: Profile DCC05 chainage analysis.** The red lines show the location of the cross-shore sections considered. The plots include all years and distinguish between the season (summer/autumn in red and winter/spring in blue). A linear regression has been fitted through all the data by season.



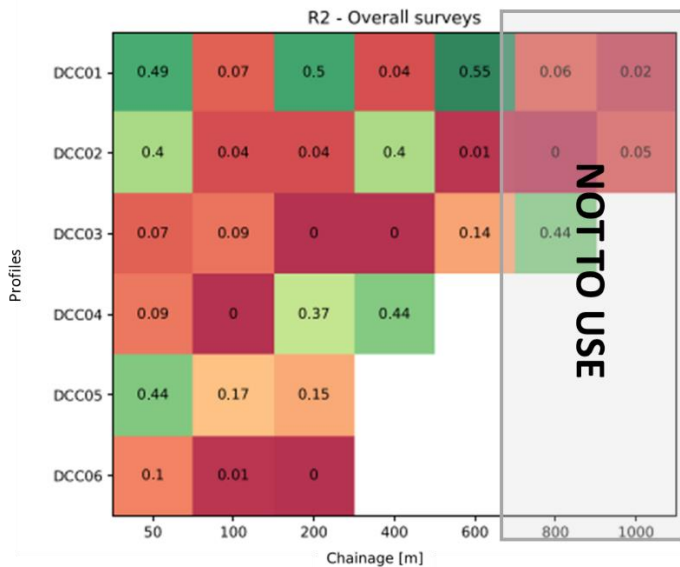
Source: Mott MacDonald, 2021. Contains Denbighshire County Council and CCO data.

**Figure 3.6: Linear regression gradients at cross-shore locations on beach profiles DC001 to DC006. A negative value indicates erosion, while a positive value shows accretion. Please note that the values derived chainage greater than 600m should not be used.**



Source: Source: Mott MacDonald, 2021

**Figure 3.7: Product moment correlation coefficient ( $R^2$ ) for the linear regression values in Figure 3.6. Please note that the values derived chainage greater than 600m should not be used.**



Source: Mott MacDonald, 2021

**Figure 3.8: p-value for the linear regression data in Figure 3.6. Please note that the values derived chainage greater than 600m should not be used. p-value less than 0.05 are highlighted in green.**

P value – Overall surveys

Profiles	50	100	200	400	600	800	1000
DCC01	<0.001	0.239	<0.001	0.343	<0.001	0.559	0.825
DCC02	0.001	0.342	0.345	0.001	0.386	0.878	0.379
DCC03	0.236	0.17	0.84	0.914	0.088	0.148	<b>NOT TO USE</b>
DCC04	0.159	0.837	0.002	0.001			
DCC05	0.001	0.048	0.079				
DCC06	0.133	0.695	0.838				

Source: Mott MacDonald, 2021

The analyses of the beach profiles by cross-shore distance and season shows that:

- Higher beach levels are often observed in summer/autumn;
- Winter/spring surveys show the greatest erosion;
- Except for profiles DCC03 and DCC01, beach levels in the nearshore at chainage 50m decrease towards the frontage's eastern section. This decrease is observed over both seasons and in the overall data;
- Profiles DCC02 and DCC05 are showing a potential erosional trend, which has been demonstrated by the statistical significance of the product-moment correlation coefficient ( $R^2$ ) and p-values (Figure 3.7 and Figure 3.8);
- Except for profile DCC01, the beach levels recorded during the June 2020 survey were close to the lowest recorded in the period 2002 to 2020;
- The behaviour of profile DCC03 at the 50m chainage is oscillatory with no trend apparent. Historically, the beach levels at this location were the lowest at the beginning of the monitoring programme (2002) and have since recovered and have been maintained; and
- Further from the coastline, the profile data show the on/offshore migration of multiple sand bars, with a decreasing trend in the beach levels only identified at the 200m chainage for profiles DCC01, DCC04 and DCC05.

The beach profile behaviour at each location shows that at:

- Profile DCC01 there is an accretion trend in the nearshore (+0.036m/year), and lowering beach levels at chainage 200m (-0.042m/year) and 600m (-0.025m/year), reflecting low beach level in the last couple of surveys (2019 and 2020);
- Profile DCC02 there is an erosion trend of the nearshore area (-0.026m/year) as indicated by the results at chainage 50m, followed by a very stable section of the beach, with minimal

level changes between chainage 100m and 200m. A decrease in the beach levels (-0.012m/year) is observed at the 400m chainage;

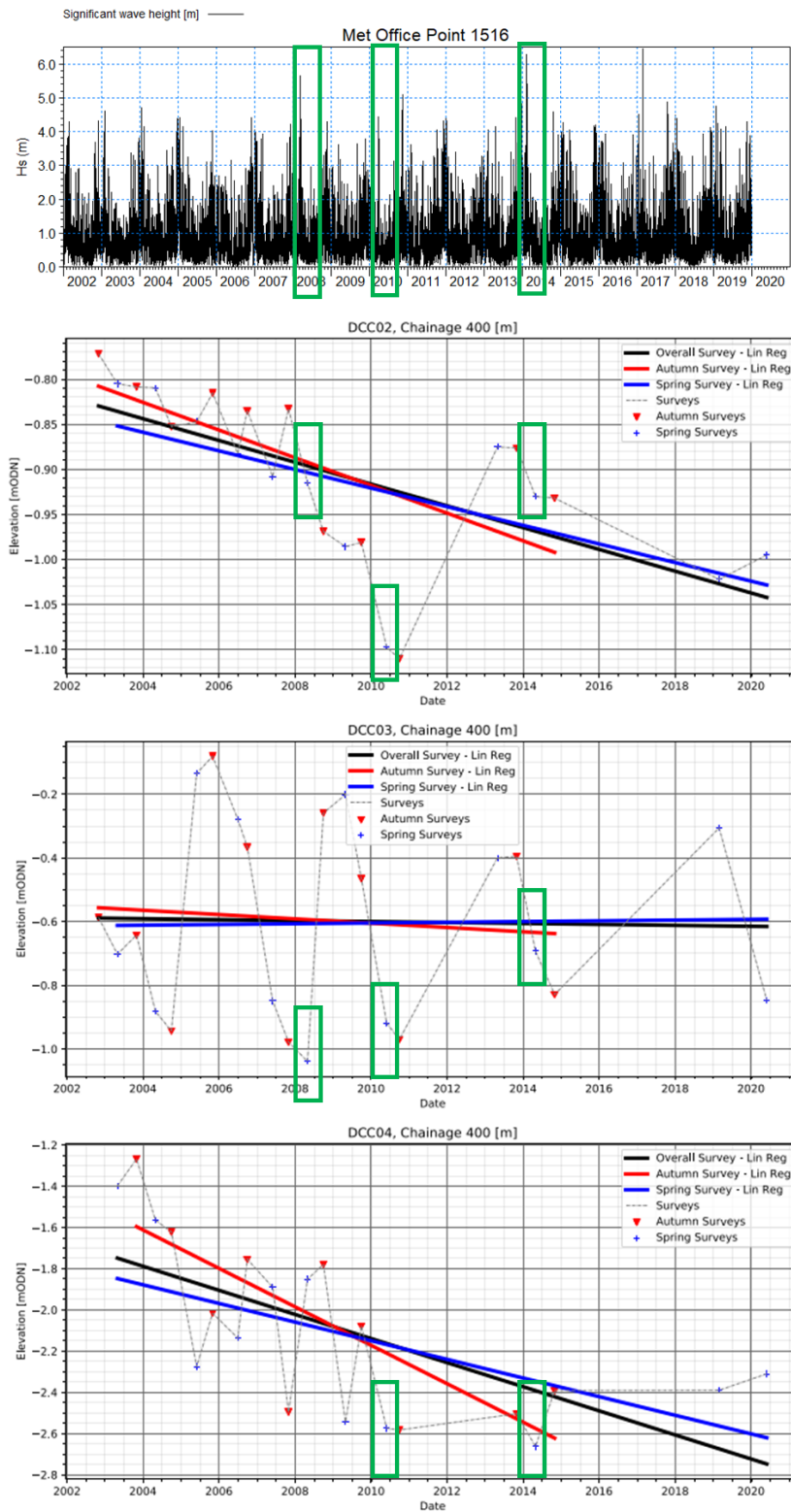
- Profile DCC03 shows general stability with not a clear trend in the data. The profile changes seem to be more related to the natural movement of the sandbars/ridges;
- Profile DCC04 shows a minor erosion trend (-0.013m/year) in the nearshore area at chainage 50m. However, the lower beach profile shows a clear erosional trend at chainages 200m and 400m (-0.058m/year);
- Profile DCC05 shows a general decrease in beach levels over the length of the profile over time, with erosion rates of -0.032m/year in the lower portion of the beach. The erosion rate increases to -0.05m/year if only winter surveys are considered (Figure B.9 in Appendix B). It is also noted that while, during the summer surveys, the lower portion of the profile is accretionary (Figure B.9 in Appendix B), the top of the beach, at chainage 50m, remains erosional; and
- Profile DCC06 also shows a consistent erosional trend (-0.027m/year) with higher rates in the nearshore.

As expected, the analysis has established some evidence of a relationship between the wave climate and the historical beach levels:

The winter/spring survey tends to have lower beach levels than those recorded after the summer/autumn. These differences are attributed to beach erosion by the winter storms that tend to move sediment offshore and results in beach profile lowering;

The WWIII offshore dataset indicates that large storms result in significant beach changes. These changes are shown in Figure 3.9, which shows for profiles DCC02, DCC03 and DCC04, beach profile changes occurring at chainage 400m and the corresponding wave climate at that time. Some large winter storms highlighted in the green boxes in Figure 3.9 are visually related to erosion of the beach. The only exception is on profile DCC04, where a winter storm in 2008 results in localised accretion. This is interpreted as resulting from the on/offshore migration of a bar feature and longshore sediment transport, rather than accretion attributable to the storm *per se*, highlighting the difficulty of interpreting beach behaviour with relatively sparse data.

**Figure 3.9: Bed level change at chainage 400m for profiles DCC02, DCC03 and DCC04. The green boxes identify the period of storms and the reduction in the beach levels from the surveys undertaken after the highlighted storms**



Source: Mott MacDonald, 2021. Contains Denbighshire County Council, CCO and Met Office data

## 3.2 LiDAR analysis

LiDAR data were used to calculate temporal and spatial changes in beach levels, estimate annual erosion/accretion rates, and determine the changes in volume between the surveys. This analysis excluded historical data from 2009 and 2011 since the surveys only covered part of the frontage.

### 3.2.1 2004 and 2015 survey analysis

LiDAR data from 2004 were compared with data from 2015 (Figure 3.11). The results show that the beach, especially for the eastern section for the frontage between profiles DCC04 and DCC05, is generally lower in 2015 than the baseline survey in 2004. Overall, beach levels have decreased by between 0.5m and 1.5m over the 11 years between the survey, an erosion rate of -0.045 to -0.136m/year.

Accretion is observed in the nearshore area of the western end of the frontage at profile DCC01 and probably reflects the change to the coastal defences between 2004 and 2015. A further potential consequence of the West Rhyl Scheme works is evidenced by beach lowering in the first 100m offshore around profile DCC02. Beach levels have here decreased by up to -1.7m between 2004 and 2015, corresponding to an approximate erosion rate up to -0.15 m/year.

In the central portion of the frontage, the beach level changes observed between 2004 and 2015 appear related to the movement of the sandbars, with some accretion (up to 0.8m over the years – +0.07m/year) in the nearshore between profiles DCC03 and DCC04.

### 3.2.2 2015 and 2020 survey analysis

The changes in beach elevation along the frontage between 2015 and 2020 are shown in Figure 3.12. The data indicate some recovery of the beach levels in areas that were previously eroding. Results also indicate that over this period, a large movement of sediment occurred, with erosion/accretion rates generally larger than in the previous LiDAR comparison. The data also show that the areas around profiles DC002 have accreted with beach levels increased by 1m (+0.2m/year).

The area between profile DC004 and DC005 experienced less change in elevation between 2015 and 2020 than previously. However, the overall trend remains erosional for the areas close to the toe of the defences, where beach levels decrease by -0.5m over the period surveyed (-0.1m/year). Between profile DC003 and DC004, limited changes in beach level are recorded, with bed levels changes typically being +/-0.1m over the 5-year period (+/-0.02m/year). In common with the previous analysis, beach elevation changes in the central portion of the frontage, especially from the 200m chainage, seem to be related to the sandbar movement.

### 3.2.3 2004 and 2020 survey analysis

A comparison between the baseline 2004 survey and the most recent one in 2020 is shown in Figure 3.13. The features are very similar to those observed in the 2004-2015 comparison. Areas of erosion are observed around profiles DCC05, both close to the shore and around the 200m chainage. This erosion extends towards profile DCC04 and varies from -0.5m (-0.03 m/year) near the toe of the defences to -1.8m (-0.11m/year) for the lower portion of the beach over the period surveyed. Similar, the erosion previously observed around profile DCC02 in 2004-2015 analysis (-1.7m, -0.154 m/year) (Figure 3.11) is also observed here, with a decrease in the beach levels of -0.8m over the 2004-2020 period (-0.05m/year) (Figure 3.13).

It is important to note that some areas of overall erosion evident in this comparison were not observed between 2015 and 2020. These areas demonstrate that the beach levels increased during the last five years of surveyed data, however, not to the levels of the survey of 2004.

Therefore, when comparing the baseline to the most recent survey (2020), a decrease in the beach levels is observed, such as in the beach around profile DCC02.

In general, the LiDAR analysis shows results very similar to the beach profile analysis of Section 3.1. The beach levels for the eastern section of the frontage have been decreasing in the last 16 years. The western part of the frontage is more stable, with accretion around DCC01, probably due to the implementation of West Rhyl Scheme.

The LiDAR erosion/accretion rates are generally larger than the one from the beach profile analysis of Section 3.1. Since the LiDAR analysis covers the entire frontage, it is able to resolve erosion/accretion "hot spot" that by not necessarily located over the monitoring beach profiles, which can introduce aliasing in the data and lead to misinterpretation of wider area changes.

The rate of beach level change has not been constant over the monitoring period, as shown both by the LiDAR analysis and the beach profile analysis of Section 3.1. The changes depend on the hydrodynamic conditions and on the engineering works in the frontage.

### 3.2.4 Volume analysis

The volume change between the LiDAR surveys have been calculated to further support the finding of the previous sections.

The volume change was only calculated for the overlapping areas between surveys (Figure 3.10). Please note that this area does not include the lower portion of the sandbars located in the central of the frontage, between profiles DCC02 and DCC04.

**Figure 3.10: Area (red box) used for the volume change calculation between LiDAR surveys.**



Source: Mott MacDonald, 2021

Table 3.1 shows the volume change between the several LiDAR surveys over the area of Figure 3.10. The results are indicating that there is a general loss of sediment over the area of analysis. The average rate of change (m/year) is also calculated, and it is similar to the trend observed in the previous sections (Figure 3.6).

**Table 3.1: Volume change between LiDAR surveys. A negative balance shows a loss of sediment in the analysed area.**

LiDAR	Erosion volume (m <sup>3</sup> )	Accretion volume (m <sup>3</sup> )	Balance (m <sup>3</sup> )	Average rate (m/year)
2004 vs 2015	207,000	153,000	-54,000	-0.01
2015 vs 2020	174,000	141,000	-33,000	-0.003
2004 vs 2020	219,000	132,000	-87,000	-0.01

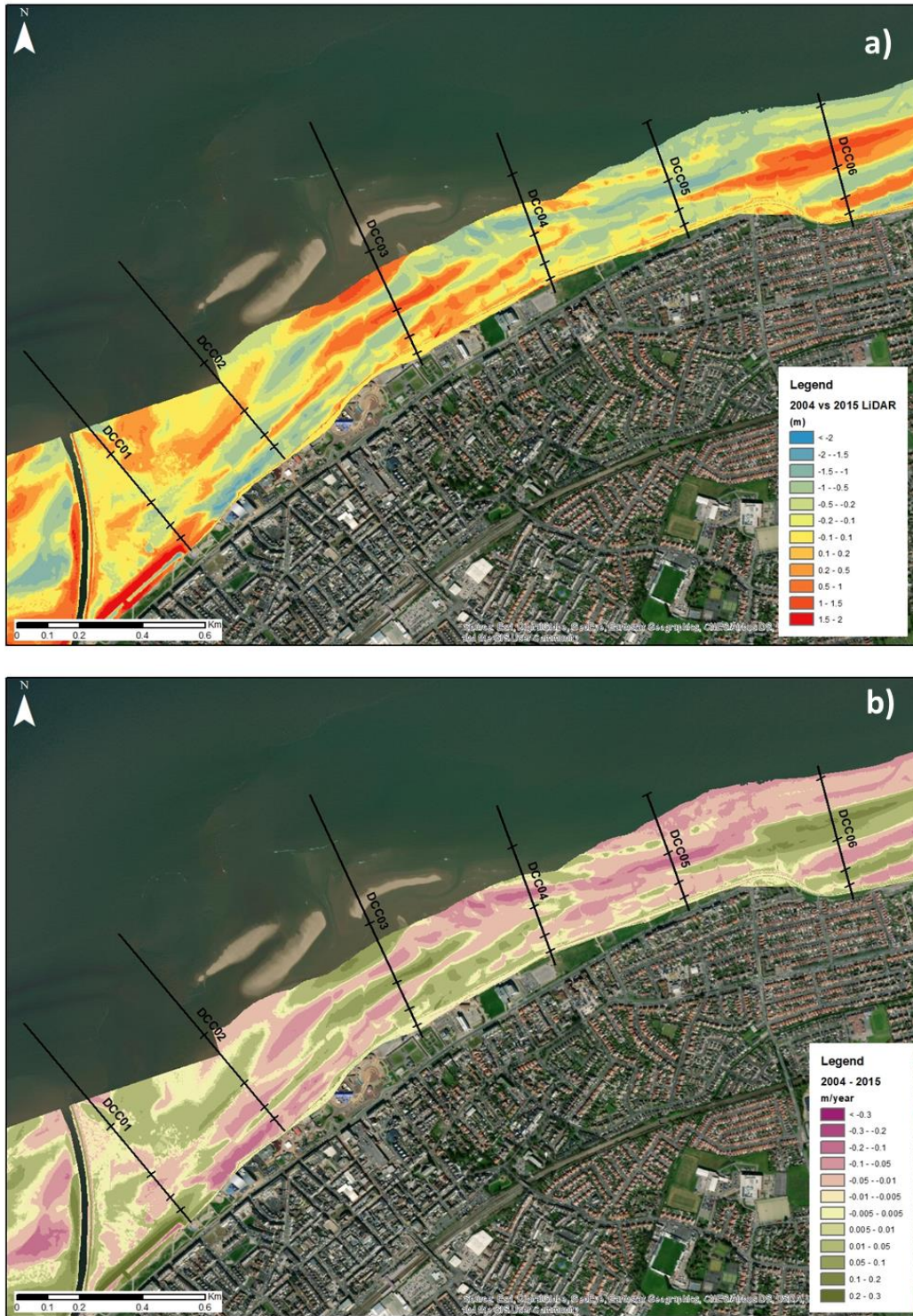
Source: Mott Macdonald, 2021

It is important to note that the analysis shows a general loss of sediment in the area. However, the analysis does not cover the entire foreshore and excludes the lower portion of the beach between profiles DCC02 and DCC04.

The excluded area is a very active part of the foreshore, with a large sediment movement, as shown in the profile analysis. This area could act as a sink for sediments, and therefore, the development and constant movement of the ridges and bars.

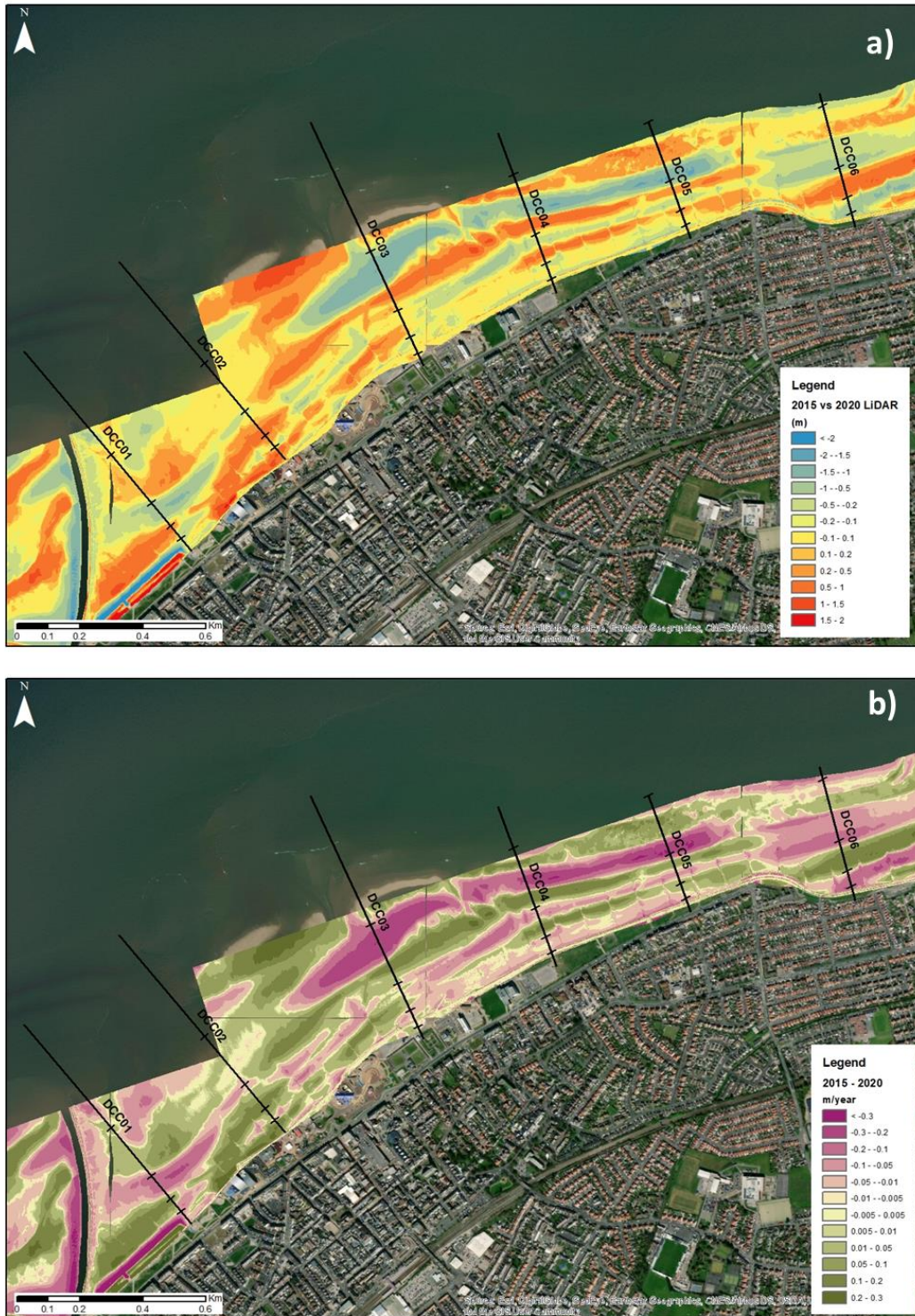
Unfortunately, no LiDAR is available to extend this volume analysis further offshore, and therefore, the volume results here presented should be used with care.

**Figure 3.11: Bed level change between 2004 and 2015 LiDAR survey. Positive values are indicating accretion, while negative values are showing erosion. a) Overall change in metres. b) Yearly bed level change in metres/years.** Please note that the tick marks on the profiles are indicating the 20m, 100m, 200m and 400m chainage as per the profile analysis of Section 3.2



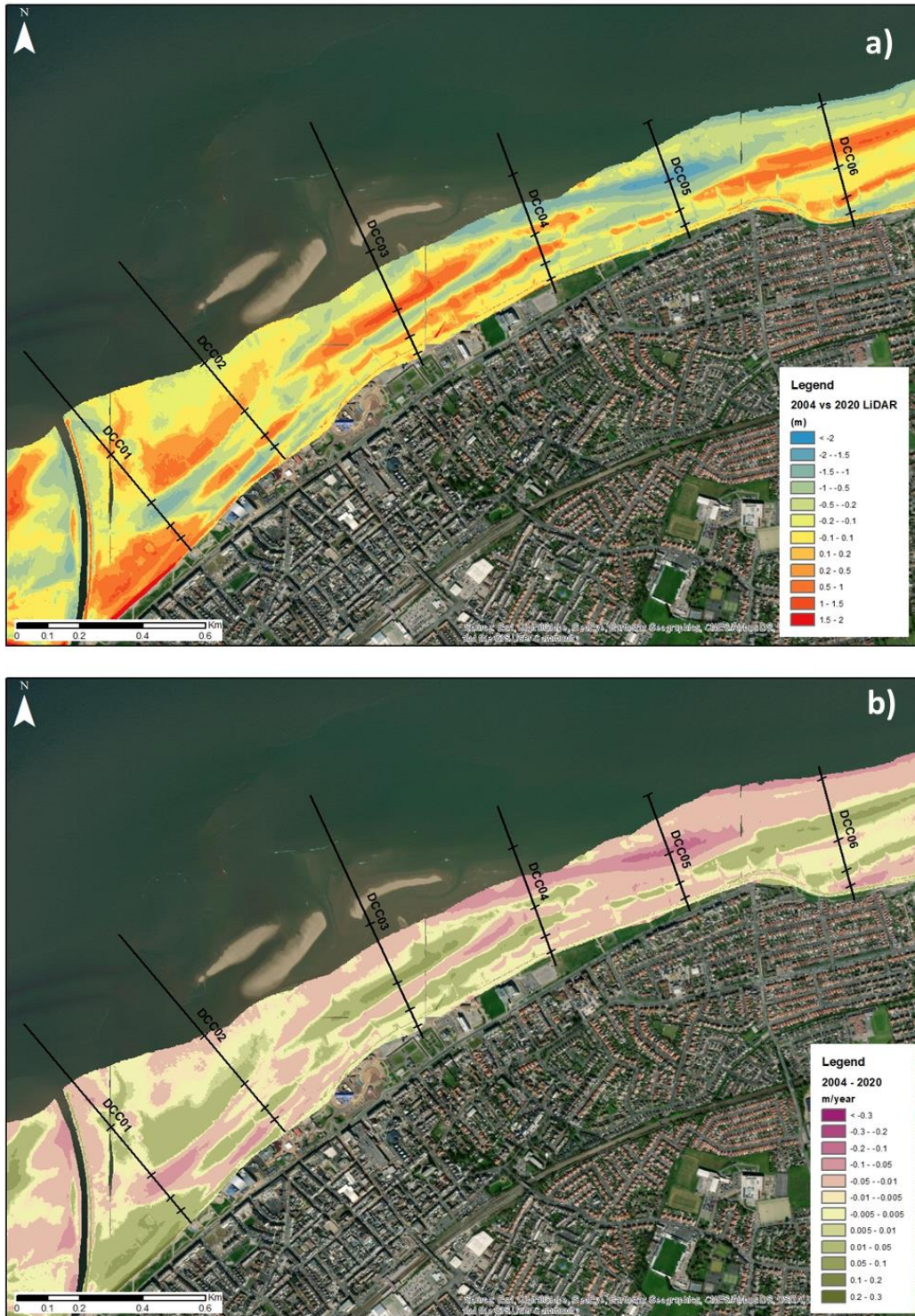
Source: Mott MacDonald, 2021. Contains NRW data.

**Figure 3.12: Bed level change between 2015 and 2020 LiDAR survey. Positive values are indicating accretion, while negative values are showing erosion. a) Overall change in metres. b) Yearly bed level change in metres/years.** Please note that the tick marks on the profiles are indicating the 20m, 100m, 200m and 400m chainage as per the profile analysis of Section 3.2



Source: Mott MacDonald, 2021. Contains NRW and CCO data.

**Figure 3.13: Bed level change between 2004 and 2020 LiDAR survey. Positive values are indicating accretion, while negative values are showing erosion. a) Overall change in metres. b) Yearly bed level change in metres/years.** Please note that the tick marks on the profiles are indicating the 20m, 100m, 200m and 400m chainage as per the profile analysis of Section 3.2



Source: Mott MacDonald, 2021. Contains NRW and CCO data.

### 3.3 Comparison of beach levels with previous studies

While the observations from the beach profile and LiDAR analyses presented in the previous sections align broadly with HR Wallingford results in 2008, the present study has also included a significant amount of new data.

According to HR Wallingford (2008), the western end of the frontage is generally accreting, and profile DCC05 has been eroding up to 10m<sup>3</sup> per metre run per year. The report suggests that of the beach lowering at the seawall in the vicinity of Splash Point.

Estimated erosion trends at the toe of the structure for profile DCC01, DCC03 and DCC05 by HR Wallingford (2008) differ from the estimated in this study (Table 3.2). These differences are attributed to:

- Only data from 2002 to 2007 are considered;
- The values related only to conditions at the toe of the structure and not at the 50m chainage as considered here; and
- Differences in the analysis methodology. HR Wallingford used their tool Beach Profile Analysis System (BDAS) to obtain mean beach levels and beach trends. MM analysis, at specific chainages, calculated the trend in the data and estimated from it an erosion/accretion rate per year.

**Table 3.2: Comparison of beach levels change between HR Wallingford study and MM analysis at the toe of the structure in metres/year**

Profile	HR Wallingford <sup>1</sup>	MM profile analysis – 50m chainage	Difference (m/year)
DCC01	0.12	0.04	0.08
DCC02		-0.03	--
DCC03	-0.02	0.01	-0.03
DCC04		-0.01	--
DCC05	-0.09	-0.02	-0.07

Source: Mott MacDonald, 2021. Contains HR Wallingford, 2008 data.

<sup>1</sup> Estimated based on 2002 to 2007 data

While several reasons can be attributed to the differences in the results between the two studies (Table 3.2), it is believed that the more significant one is the location/chainage of the analysis. As previously shown, the profiles trend significantly changes from one chainage to the other, and therefore, the comparison should be undertaken at the same point in the profile.

Also, the MM analysis has been updated with more than ten years of extra data, and as expected, the results differ. However, the general trend for the frontage is the same as that reported in the HR Wallingford study.

Erosion/accretion rates obtained from the LiDAR data analysis are generally larger than the values reported by HR Wallingford and MM profile analysis. Since the LiDAR analysis covers the entire frontage, it can resolve erosion/accretion "hot spot" that is not necessarily located over the monitoring beach profiles, which can introduce aliasing in the data and lead to misinterpretation of wider area changes.

This interpretation is further supported by the JBA (2018) study of the beach profile data and aerial images undertaken as part of the East Rhyl Coastal Defence Scheme. The report confirmed that the Denbighshire coastline's dynamics are primarily associated with the movement of ridges and runnels along the Rhyl-Gronant coast and that these large-scale

sediment movement may not be fully characterised by single cross-shore beach profiles (Table 3.3).

The CERMS (Halcrow Group, 2010) study reaches a similar conclusion regarding the beach levels trends in the central Rhyl frontage to those arrived at in the present study. According to the CERMS, the foreshore immediately east of the Afon Clwyd is relatively stable, but beach levels reduce in an easterly direction, and the foreshore is eroding by Splash Point and across the Rhyl Golf Links frontage.

**Table 3.3: Onshore sediment delivery derived from ridge dynamics**

Location	Profile	Ridge Area (Cross-Section)	Delivery	Annual Delivery	Length Scale	Transport Rate
Rhyl	DCC05	150 m <sup>2</sup>	4 years	38 m <sup>3</sup> /m	675 m	25,000 m <sup>3</sup> /yr
East Rhyl	DCC06	180 m <sup>2</sup>	3 years	60 m <sup>3</sup> /m	370 m	22,000 m <sup>3</sup> /yr
East Rhyl	DCC07	200 m <sup>2</sup>	7 years	29 m <sup>3</sup> /m	550 m	16,000 m <sup>3</sup> /yr
East Rhyl	DCC08	190 m <sup>2</sup>	6 years	32 m <sup>3</sup> /m	525 m	17,000 m <sup>3</sup> /yr
Prestatyn	DCC15	N/A	N/A	0	3,300 m	0
Gronant	DCC24	400 m <sup>2</sup>	15	27 m <sup>3</sup> /m	900 m	24,000 m <sup>3</sup> /yr
Gronant Beach	DCC26	40 m <sup>2</sup>	8	5 m <sup>3</sup> /m	500 m	2,500 m <sup>3</sup> /yr

Source: JBA, 2018

### 3.4 Summary and Conclusions

- The central Rhyl coastline is characterised by a wide and shallow intertidal foreshore, with extended sandbar features that form characteristic ridge/runnel features.
- The foreshore topography has been monitored since 2002, and the beach elevation data have been analysed several times in different studies. The present study has updated this analysis with the most recent surveys and added further to understand the beach evolution trends along the frontage.
- The results show that the area in the eastern section of the frontage, from profile DCC05 towards Splash Point and, potentially, further east, based on the available data exhibit an erosion trend, with beach levels lowering over the years.
- The LiDAR data show that the eastern section of the frontage's erosion rate varies between -0.1m/year and -0.2m/year for the lower beach and up to -0.03m/year near to the toe of the defences.
- While the erosion rates are modest, it is recommended that it requires further monitoring. Expressly, it is noted that the most recent survey in June 2020 showed that the frontage was close to its historical minimum beach levels.
- It is recommended to include more recent survey data in the present analyses when available to determine if the beach levels have recovered after summer 2020.
- The results of this study and the previous study show that the western part of the frontage has an accretional trend.
- The movement of the sandbars is responsible for the changes observed in the profiles. Together with the longshore transport generated by tidal and wave-induced currents, the sandbars migration is the main supply of sediment to the frontage.
- It is important to note that the foreshore is a continuously changing system, and both the LIDAR and the profile surveys are only providing a snapshot in time and are not capturing the beach's complete behaviour. Periods with lower beach levels could have occurred between surveys, and therefore, not been recorded. Beach levels can change due to a storm

and recover in the next couple of months, and the regular monitoring surveys may not capture these changes.

- It is essential to understand that the lowest beach levels reported here could have missed the lowest beach drawdown level, which has particular importance at the toe of the structure and for design considerations. For that reason, the potential for beach drawdown is examined in Chapter 4.
- Detailed studies of cross-shore and longshore sediment transport are presented in Chapter 4 and Chapter 5, respectively.

## 4 Beach profile modelling

### 4.1 Introduction

It is necessary to understand beach behaviour to assess the new coastal revetment design's function, efficacy and potential impacts on the beach. To do this requires a study of wave interactions with the beach and the new structure that provides predictions of beach drawdown during extreme events (e.g. 1% AEP), for the baseline and the Scheme, both in the present day and in the future (up to 100 years).

It was concluded that the Scheme impacts on alongshore sediment transport will be several orders of magnitude less than the impacts on cross-shore sediment transport. Therefore the beach responses to extreme events were simulated using two independent 1D numerical models: (a) the profile evolution tool in the littoral drift module (LITPACK) from MIKE by DHI; and (b) infragravity (long-wave) and non-hydrostatic (wave-resolving) versions of the XBeach beach profile evolution model. Comparisons between predictions from these independent models provide a guide to model accuracy and reliability.

The profile evolution models simulate tidal changes in the mean water level and non-linear interactions between wave-induced bed shear stresses and the beach sediments. If shear stresses exceed a threshold condition, sediments are mobilised, and the model updates the morphology. Over time, in extreme wave conditions, local regions of erosion and accretion result in changes in the beach profile shape. In this study, sensitivity tests were undertaken with available data to establish the model parameter values giving the most accurate prediction of known beach profile changes.

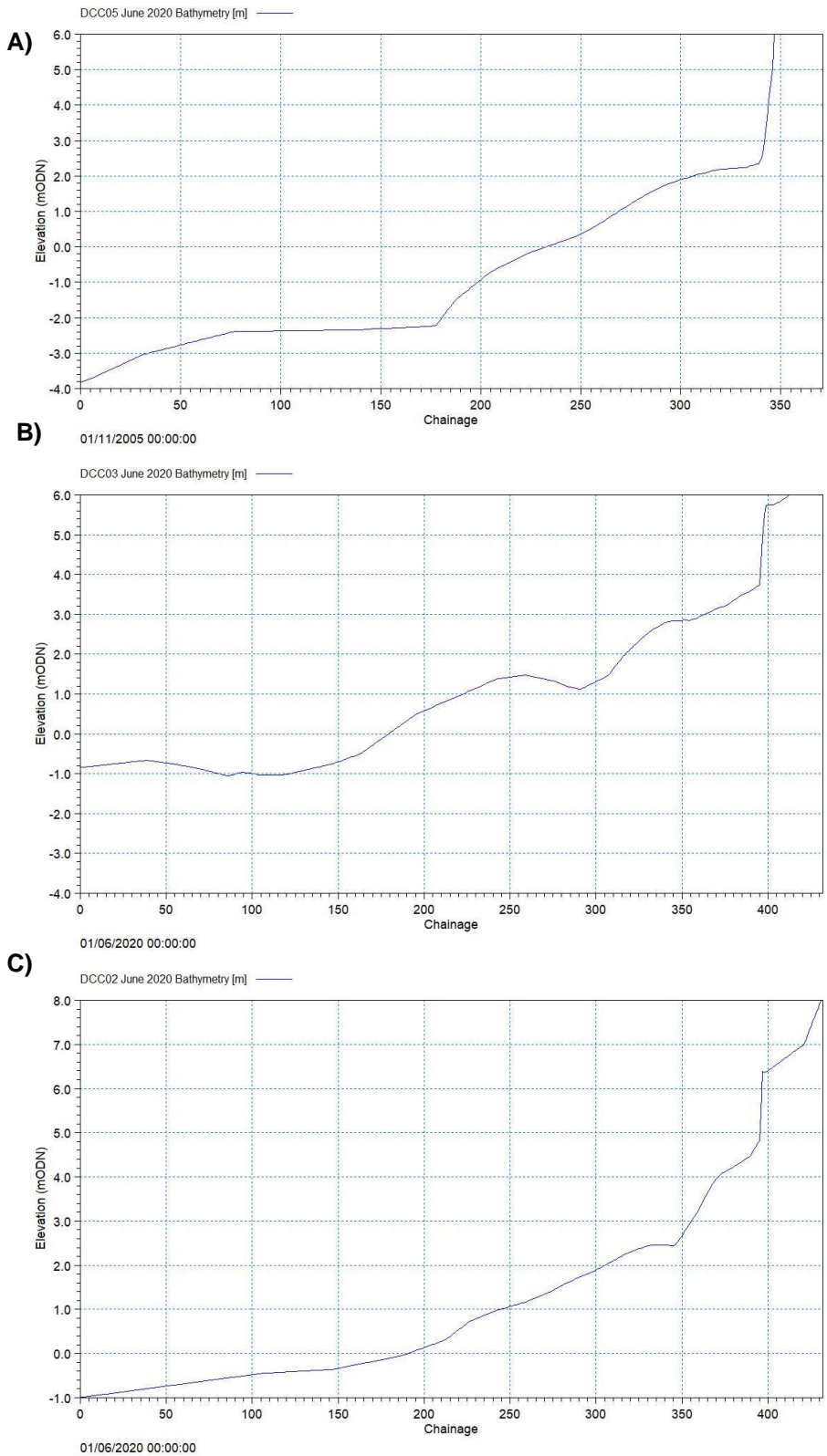
### 4.2 Model setup

#### 4.2.1 Beach profiles

From all the available beach profile data (refer to section 2.4.1 for more details), the contrasting profiles DCC05, DCC03 and DCC02 were selected (Figure 2.9). Profile DCC05, located near Splash Point, was chosen due to its more exposed location, relatively narrow foreshore and likelihood to reach minimum beach drawdown levels. In contrast, profiles DCC03 and DCC02 represent the frontage location with a large foreshore and extensive sandbars likely to reduced beach drawdown potential. The response of these three profiles to an extreme event is considered to define the envelope of potential beach profiles change and provide the best guidance for designing the proposed structures.

Profiles DCC02, DCC03 and DCC05 are shown in Figure 4.1. Profile DCC05 characterises the eastern beach's morphology and has a maximum cross-shore extend of ~400m. Profiles DCC02 and DCC03 have a maximum cross-shore extent of 1400m and are characterised by sandbar/ridge features. Profile DCC05 is also generally steeper than DCC02 and DCC03. Figure 4.1 shows profiles DCC05, DCC03 and DCC02 in June 2020. This date was selected as it represents beach levels close to the minimum values recorded at the defences' toe.

**Figure 4.1: Beach profiles in June 2020: (A) DCC05; (B) DCC03; and (C) DCC02** - Please refer to Figure 2.9 for the location of these profiles. Please note that the profiles extent horizontal extent has been limited to 400m.

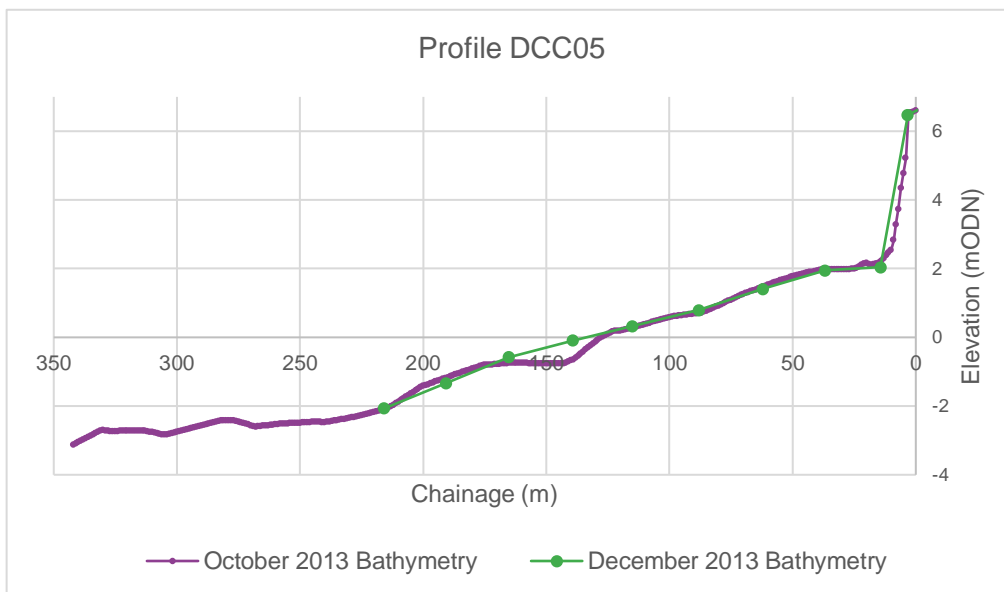


Source: Mott MacDonald, 2021 (includes CCO data).

When using models to simulated beach profiles responses to storms, it is desirable to calibrate and validate the model predictions using measured pre- and post-storm beach profiles. However, owing to survey programmes being established many months or years ahead, the data acquired rarely captures the pre- and post-storm data immediately before and after a storm event. Recourse must be made, therefore, to surveys that best represent these conditions. Specifically, a pre-storm survey should be as close to the actual storm event as possible and post-storm surveys as close to the end of the storm event as possible. Available beach profile data were examined with this in mind, and beach profiles measured in October and December 2013 were identified as being the most suitable for this study (Figure 4.2).

Figure 4.2 show relatively small changes in the beach profile. Sediment in the upper profile has been moved cross-shore, and accretion areas are observed in the middle and at the seaward end of the profile. There is also some minor erosion near the seawall. Due to the reduced spatial resolution of the data from the December 2013 survey, the sea defences' shape does not match between profiles. However, this is not significant from the modelling perspective as the sea defences are defined as hard structures that remain unchanged by wave action during the simulations.

**Figure 4.2: Cross-shore beach profile DCC05 pre- and post-storm (October and December 2013, respectively)**



Source: Mott MacDonald, 2021. Includes CCO data and Denbighshire County Council data.

#### 4.2.2 Sediment properties

The sediment characteristics were taken from the Central Rhyl Coastal Defence Scheme (Final Report. APEM Ref P00005614 Jan. 2021) and were checked against PN194063 East Rhyl Coastal Defence Scheme - Factual Report" from February 2019 and the most recent geotechnical investigation (Geotechnics Limited, March 2021). In both reports, PSA data showed the beach was dominated by medium sand with a  $d_{50}$  of 249 $\mu$ m (Table 2.4 and Table 2.5). There is only a slight variation in sediment properties over the study area. The sediment properties used in the profile evolution models are shown in Table 4.1.

**Table 4.1: Sediment characteristics used in the LITPACK profile evolution model.**

Sediment Description	Value
Relative sediment density	2.65
Mean grain diameter ( $d_{50}$ )	0.25 (mm)
D90	1.03 (mm)
D84	0.46 (mm)
D16	0.22 (mm)
D10	0.14 (mm)
Sediment sorting ( $\sqrt{\frac{d_{84}}{d_{16}}}$ )	1.53
Number of fractions	4

Source: Mott MacDonald, 2021 (Includes Geotechnics, 2019 and APEM, 2021 data)

### 4.2.3 Structure

Non-erodible structure such as the coastal defence needs to be defined in the model. The starting point of the structure is considered to be the starting grid position from where the bed is assumed to be non-erodible to the end of the profile. For this chapter, the baseline structure is considered the existing coastal defence and the Scheme structure is the proposed works.

#### 4.2.3.1 Baseline structure

The existing revetment was represented as being non-erodible in the models from a defined elevation:

- Profile DCC05: revetment assumed to starts at approximately elevation 4.1mODN;

Profile DCC03: revetment assumed to starts at approximately elevation , therefore allowing for some additional beach drawdown.

#### 4.2.3.2 Scheme structure

The proposed works (also known as the Scheme) suggests a stepped revetment with an advance of the defence line up to 25 metres from the existing position of the defences for profile DCC02. And a rock revetment with an advance of the defence line of 8 metres from the existing position of the defences for profiles DCC03 and DCC05. It has been assumed that the beach is maintained as per June 2020 topography survey. For further information regarding the Scheme detail design, please refer to “Design input statement” report (Mott MacDonald, 2021).

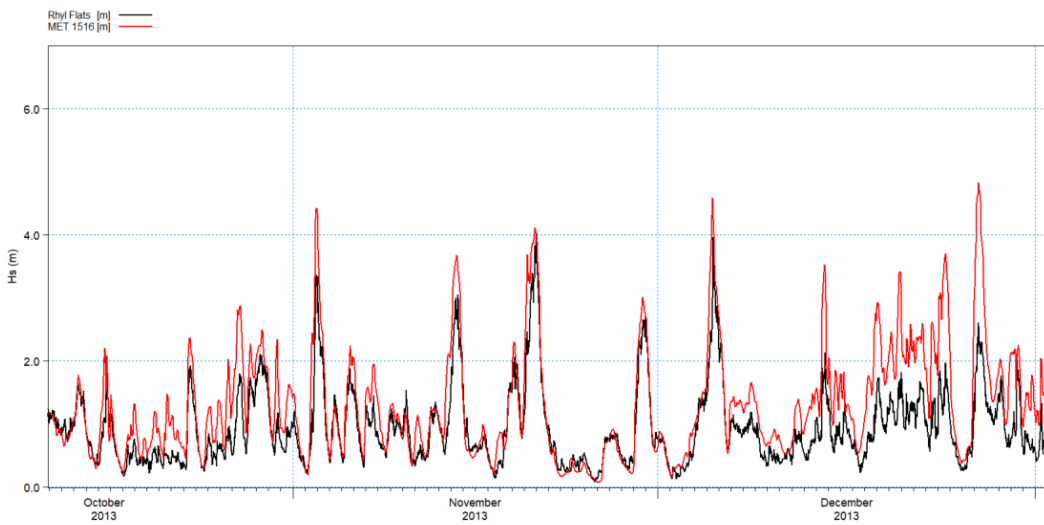
The proposed structure is defined at the following elevations:

- Profile DCC05: revetment assumed to starts at approximately elevation 2.3mODN;
- Profile DCC03: revetment assumed to starts at approximately elevation 3.5mODN; and
- Profile DCC02 revetment assumed to starts at approximately elevation 3.9mODN.

## 4.3 LITPACK model validation

The validation of the LITPACK model used wave boundary conditions obtained by running the MIKE Spectral Wave (SW) model. This model transformed waves from the offshore Met Office WW3 data point to the nearshore between October and December 2013, Figure 4.3 During the three months between beach surveys in October and December, four storms were recorded by both the Met Office data and the Rhyll Flat wave buoy where  $H_s$  exceeded 4m. It is believed, but not confirmed, that the December 2013 post-storm survey was undertaken after the first storm in December.

**Figure 4.3: Offshore wave height for both Met Office WW3 point 1516 and Rhyl flats wave buoy**



Source: Mott MacDonald, 2021. Includes Met Office and Channel Coast Observatory (CCO) data.

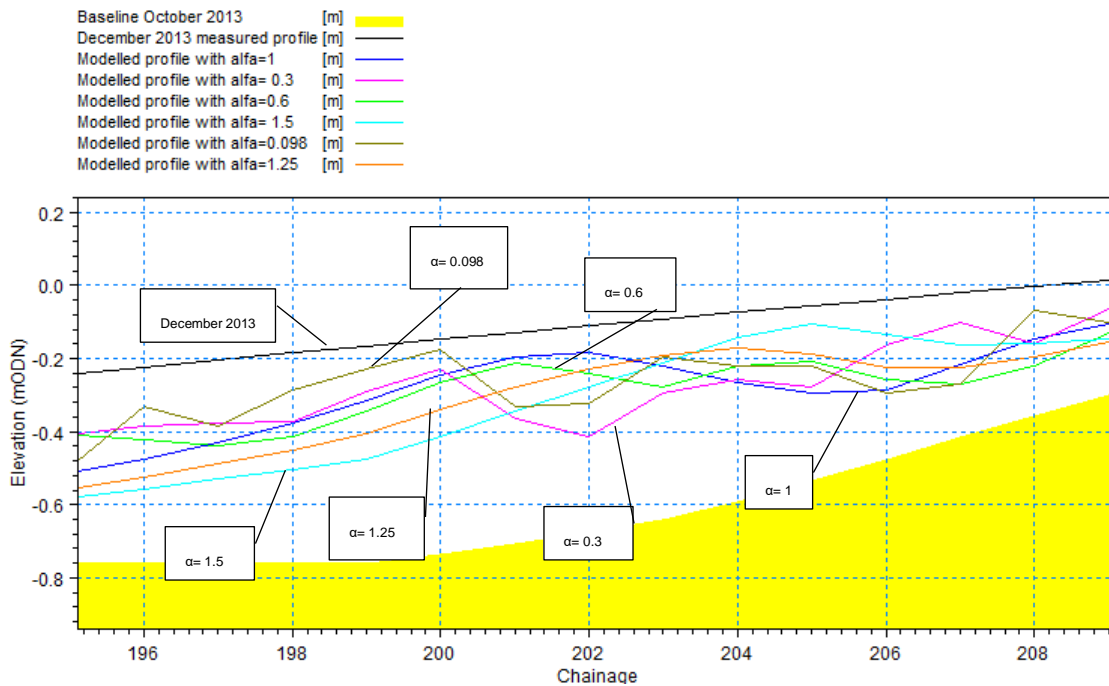
### 4.3.1 LITPACK model sensitivity tests

Sensitivity tests involved varying several model parameters to optimise model performance.

#### 4.3.1.1 Scale parameter

The scale reflects the cross-shore exchange of momentum and is proportional to a characteristic length scale over which the sediment transport is smoothed. It affects the shape of the developing bars so that the higher the value, the longer bars. Following DHI recommendations (Tomasicchio et al., 2020) scale values of 0.15, 0.098, 1.5, 1.0, 0.6 and 0.3 were tested. The sensitivity tests (Figure 4.4) determined that a scale parameter of 1.0 was appropriate.

**Figure 4.4: Results of the scale parameter sensitivity tests. Please note that only a portion of the profile is shown to illustrate the different predicted profile shapes for each scale parameter setting.**



Source: Mott MacDonald, 2021. Includes CCO data.

#### 4.3.1.2 Spectral description of irregular waves

The wave breaking parameter is influenced by the wind and depends on the bottom slope and the incident wave's steepness. In the LITPACK profile evolution model, the breaking parameters  $\Upsilon_1$  and  $\Upsilon_2$  refer to wave steepness and the wave height/depth ratio, respectively. In the present study, the following values were tested:

- $\Upsilon_1 = 0.88$  and  $\Upsilon_2 = 0.8$ ;
- $\Upsilon_1 = 0.88$  and  $\Upsilon_2 = 0.55$ ;
- $\Upsilon_1 = 5$  and  $\Upsilon_2 = 0.8$ ; and
- $\Upsilon_1 = 2$  and  $\Upsilon_2 = 0.8$ .

The best agreement between the measured and the predicted profile as obtained with the settings  $\Upsilon_1 = 5$  and  $\Upsilon_2 = 0.8$ . Please note that these values are also in line with the breaking parameters selected as part of the 1D wave propagation undertaken in the Central Rhyll Wave Modelling study (Mott MacDonald, 2021).

#### 4.3.1.3 Roller coefficient:

This calibration factor expresses the relationship between the energy dissipation rate and the surface roller area. Values of 0.005 and 0.15 were tested, and results showed that profile evolution was best simulated using the DHI recommended value of 0.15.

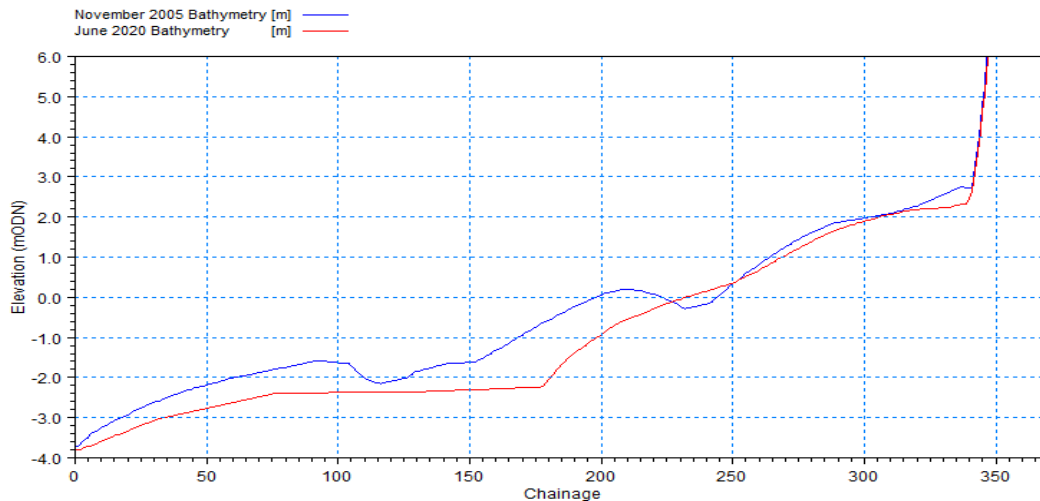
#### 4.3.1.4 Beach elevation

The effect of the initial beach elevation on the profile evolution was also tested during the model validation. Figure 4.5 shows the November 2005 and June 2020 DCC05 profiles representing approximately historical maximum and minimum beach levels. Understanding the initial beach level's effect on the profile's evolution was established by running the model with the lowest and

the highest initial profiles. This test demonstrated that the November 2005 and June 2020 profiles had similar erosion/accretion patterns after simulation of the storm event.

The June 2020 survey was therefore selected as the baseline for the study (section 4.4.1), and it will allow understanding how, from the historical minimum, the beach may lower due to a storm event.

**Figure 4.5: Cross-shore beach profile DCC05 in November 2005 (blue line) and June 2020 (red line), representing approximately the historical maximum and minimum level of the beach.**

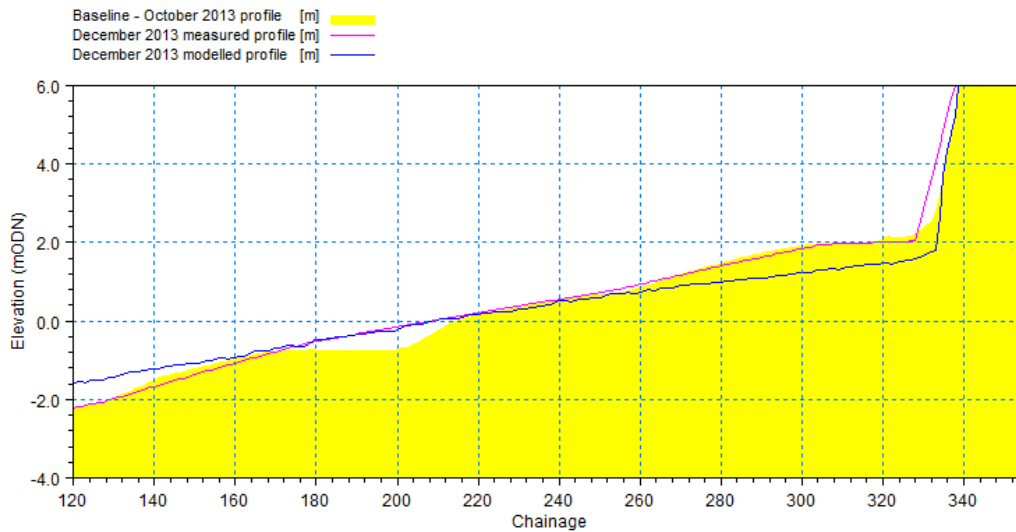


Source: Mott MacDonald, 2021. Includes CCO data.

### 4.3.2 Validation results

Validation of the LITPACK model is shown in Figure 4.6. This shows that the model tends to overpredict the erosion at the toe of the structure and the accretion in the lower beach profile. While differing in the magnitude of change, the results show that the general cross-shore distribution of erosion/accretion predicted by the model is similar to the measured data. The most significant discrepancy between the model results and the observations is found near the defences' toe. The difference arises from a weakness in the model, which cannot simulate the process of post-storm beach recovery. In the context of the present study, this is not important as we are concerned only with beach drawdown. In that regard, the model performance is considered to be conservative.

**Figure 4.6: Representation of the resulted selected validation ("validation 43version2")**



Source: Mott MacDonald, 2021 (includes CCO data)

#### 4.4 LITPACK model simulations

LITPACK profile evolution modelling included three model runs representing a combination of wave height and water levels for a 1% AEP storm:

- in the present day (2020), with two additional wave direction sensitivity tests; and
- in the future (2120), with two additional wave direction sensitivity tests.

In the 2120 model runs, wave height was increased by 10% and mean water level by 0.87m based on the UKCP18-RCP 8.5 with a 70% percentile.

##### 4.4.1 Extreme storm conditions

Table 4.2 shows results from an extreme value analysis (EVA) of significant wave height (Hs) using data from the Met Office WWIII Point 1516 (Figure 2.1). The 1:100 years (1% AEP) significant wave height is highlighted.

**Table 4.2: Predicted extreme wave condition from an EVA analysis using data from the offshore Met office WWIII point 1516. Highlighted in blue is 1% AEP significant wave height.**

Annual exceedance probability (AEP) (%)	Hs (m)
100	4.37
50	4.79
20	5.20
10	5.61
5	5.90
4	6.14
2	6.52
1	6.91
0.5	7.29

Annual exceedance probability (AEP) (%)	Hs (m)
0.2	7.80
0.1	8.18
0.02	9.07
0.01	9.45

Source: Mott MacDonald, 2021.

Using extreme water level data, corrected to the year 2020 using UKCP18 RCP8.5 70<sup>th</sup> percentile (Table 2.2) taken from the coastal flood boundary (CFB) data set, a joint probability analysis (JPA) between waves and water levels was undertaken using the simple JPA calculator from Defra (2005). This analysis provided three combinations of significant wave height (Hs) and water level (WL) representing the present-day 1% AEP storm. The significant wave height was then increased by 10% and water level by 0.87m (UKCP18-RCP 8.5 with 70% percentile) to define the climate change impacts for the year 2120.

Table 4.3 presents the modelling scenarios used in this study based on the results from JPA. Scenarios 1 to 3 corresponds to the 1% AEP storm in the present day (2020), and scenarios 4 to 6 include climate change by 2120. Peak wave period and wind speed and direction were derived by analysing the relationship between wave height and wave direction. Wave direction was defined as perpendicular to the shore, and sensitivity tests (+/-20 deg) were also undertaken. In all cases, wave spreading was defined as 33.4 deg as per the SW model (please refer to Mott MacDonald 2021 wave modelling report for further information).

**Table 4.3: Offshore conditions corresponding to 1% AEP storm for the present day (scenario 1 to 3) and the future (scenario 4 to 6). Mean waves directions for profiles DCC03 and DCC05 is 340degrees and for profile DCC02 is 320degrees.**

Scenario	Water level (mODN)	Significant wave height (Hs) (mODN)	Peak wave period (Tp) (s)	Wind speed (m/s)
1	4.62	6.06	9.4	25.0
1 (+20d)	4.62	6.06	9.4	25.0
1 (-20d)	4.62	6.06	9.4	25.0
2	4.99	4.52	8.4	21.0
3	5.47	2.13	6.2	13.4
4	5.49	6.67	9.8	26.5
4 (+20d)	5.49	6.67	9.8	26.5
4 (-20d)	5.49	6.67	9.8	26.5
5	5.86	4.97	8.7	22.3
6	6.34	2.34	6.4	14.2

Source: Mott MacDonald, 2021. (Includes DEFRA data).

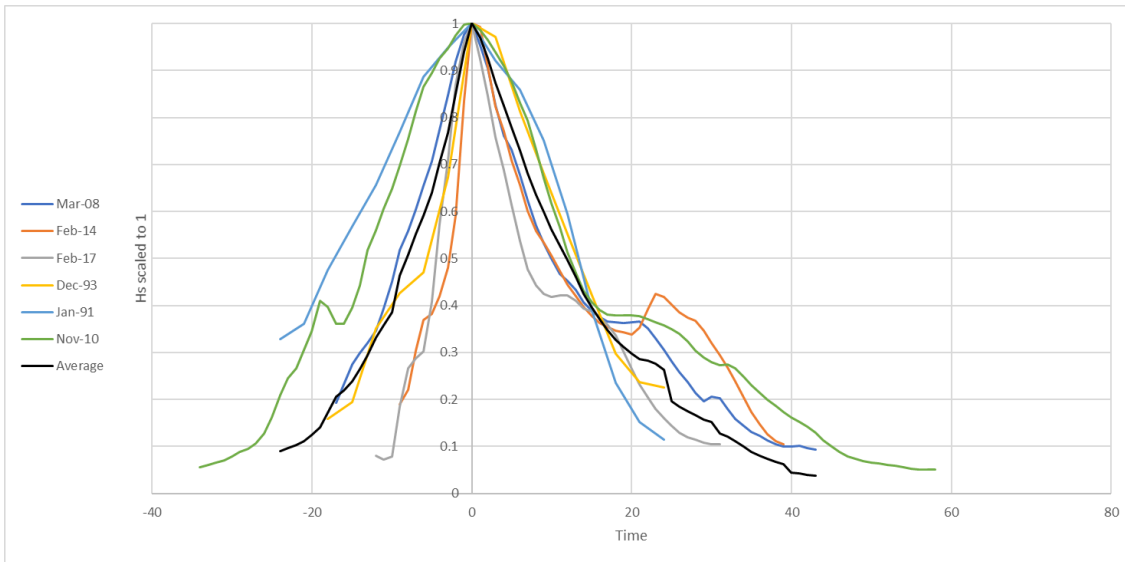
A relationship was derived between wind and wave direction and therefore applied to the SW model simulations as follow::

- MWD = 300 degrees → Wind direction = 300.50 degree
- MWD = 320 degrees → Wind direction = 320.03 degrees
- MWD = 340 degrees → Wind direction = 339.57 degrees
- MWD = 360 degrees → Wind direction = 359.10 degrees

An examination of the temporal changes in significant wave height during seven historical storm events was undertaken to define a typical storm's characteristics. The peak of each storm was

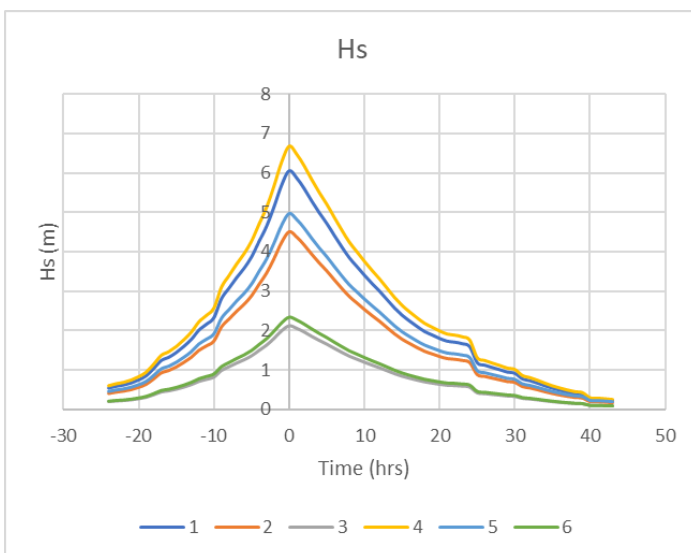
assigned as time = 0hrs, the wave height was normalised, and the average temporal characteristic of the storms was obtained (Figure 4.7). This average storm 'shape' was then rescaled to obtain the temporal storm wave characteristics for each of the scenarios tested (Figure 4.8). Wave period and wind speed data were treated in the same manner.

**Figure 4.7: Normalised significant wave height for historical storms between 2008 and 2017. The black line represents the average storm 'shape'.**



Source: Mott MacDonald, 2021

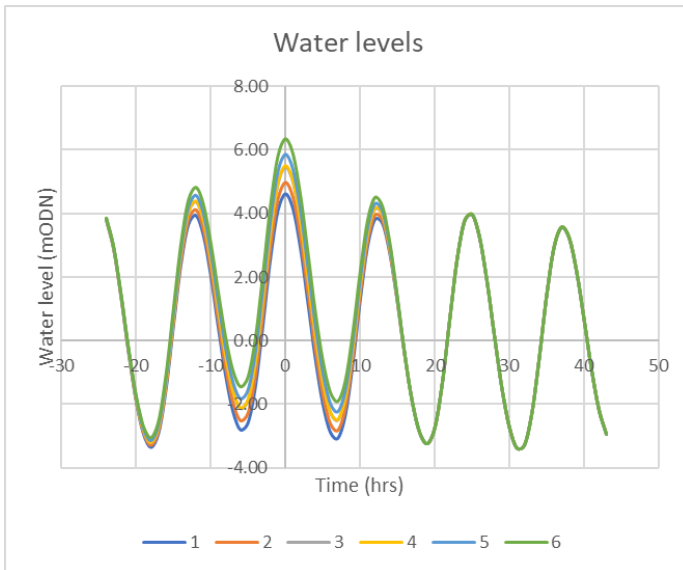
**Figure 4.8: Temporal changes in Hs for each of the six 1% AEP scenarios tested (Table 4.3).**



Source: Mott MacDonald, 2021

The Llandudno CFB surge shape was scaled and added to the extreme water levels to account for the storm surge and define the storm's temporal changes in water level. It is noted that the surge was coincident with high water, and the maximum water levels were coincident with the maximum wave height (Figure 4.9).

**Figure 4.9: Temporal water level changes for each of the six 1% AEP scenarios tested (Table 4.3).**

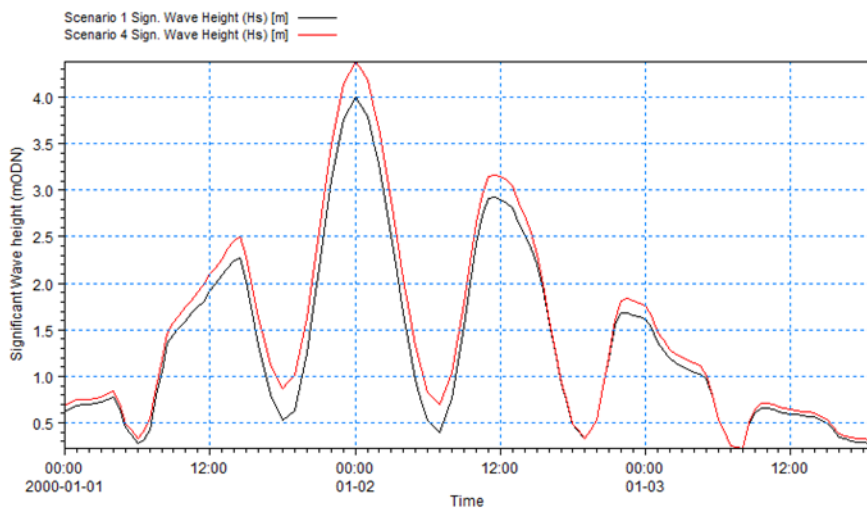


Source: Mott MacDonald, 2021

The existing MIKE21 FM SW model was used to transform the offshore storm wave conditions to the nearshore. Wave and water levels conditions were then extracted from the model at the offshore end of each beach profile and used as boundary conditions for the LITPACK XBeach profile evolution models.

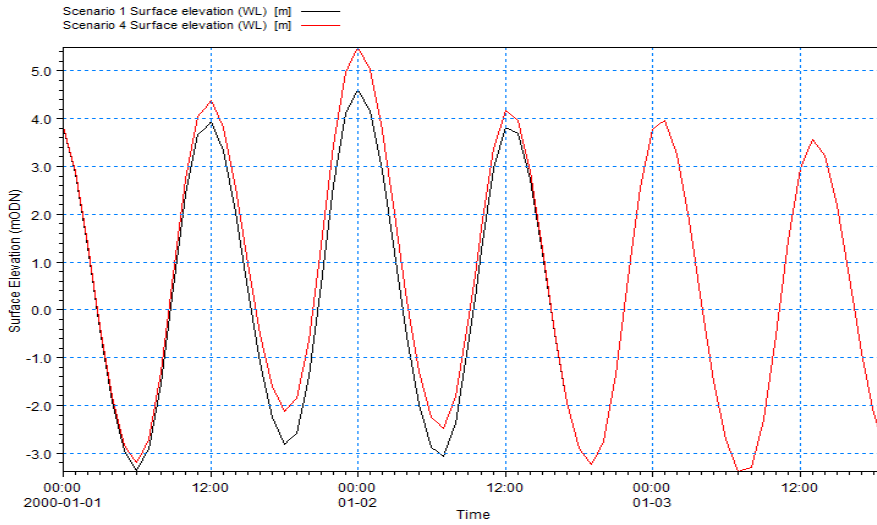
An example of the transformed nearshore wave time-series for profile DCC05 for two scenarios is shown in Figure 4.10 (significant wave height) and Figure 4.11 (water level).

**Figure 4.10: Profile DCC05: Significant wave height (Hs) time-series for Scenario 1 and 4.**



Source: Mott MacDonald, 2021

**Figure 4.11: Profile DCC05: Water elevation time-series for Scenario 1 and 4.**

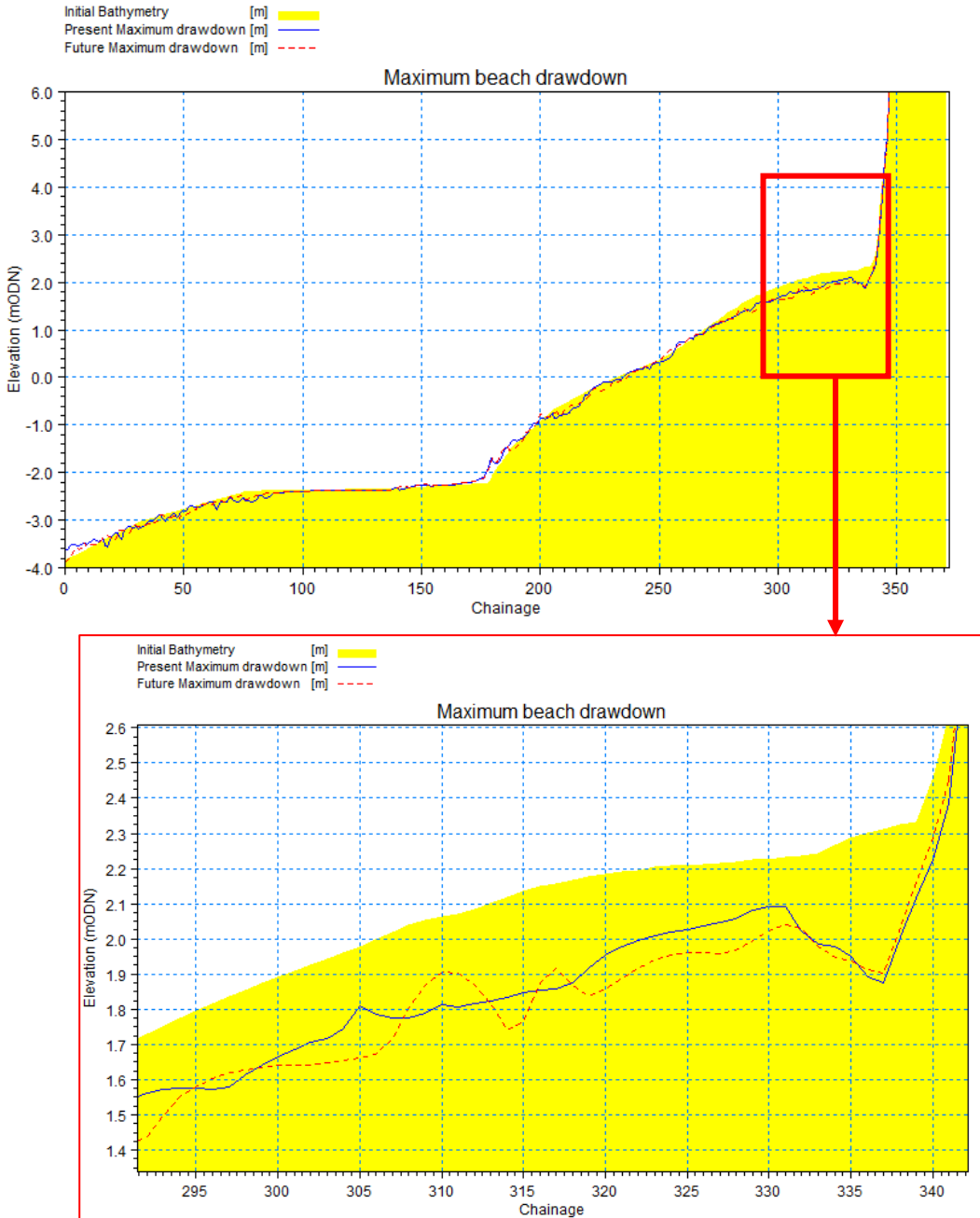


Source: Mott MacDonald, 2021

#### 4.4.2 LITPACK profile evolution - Baseline results

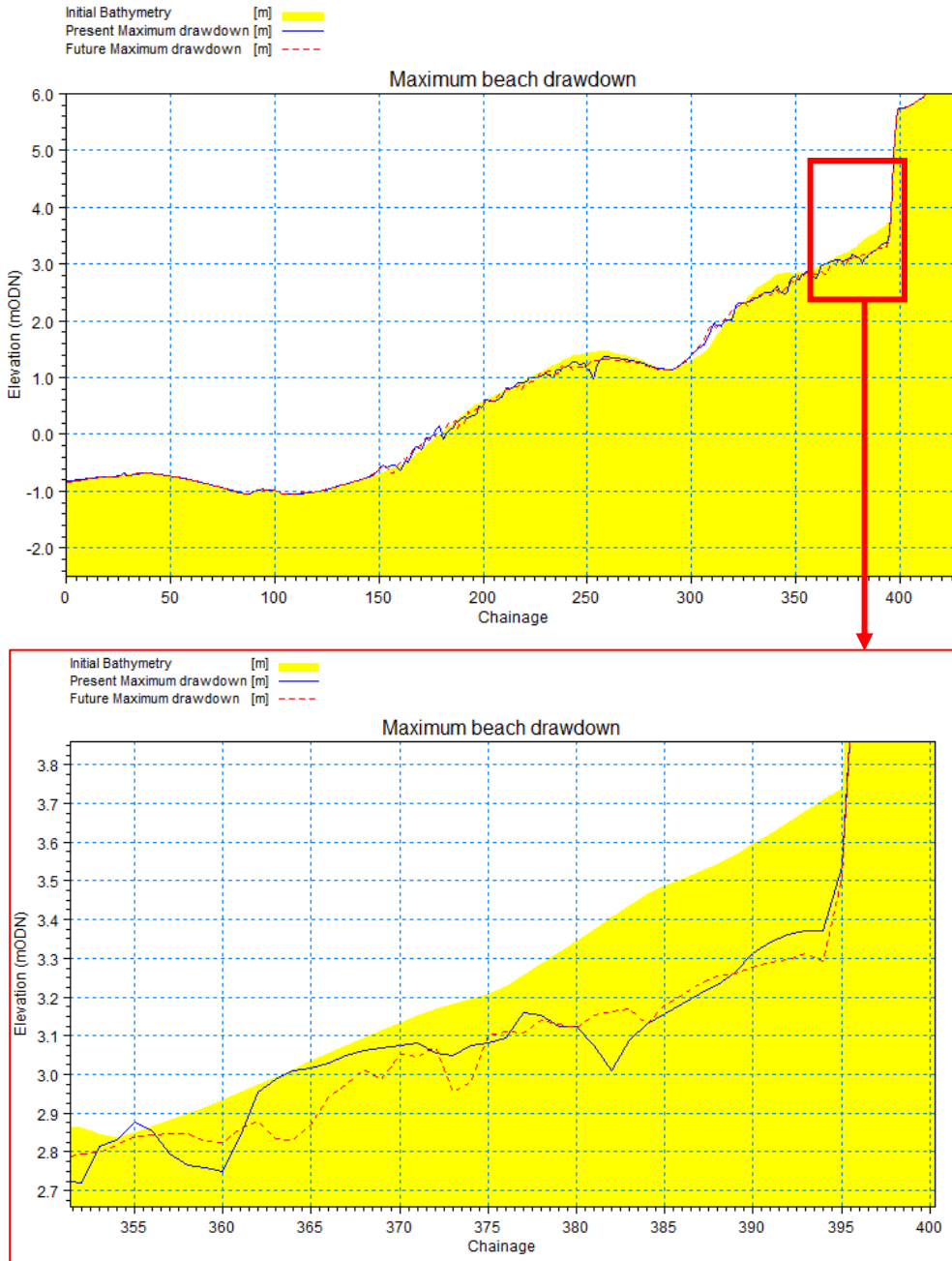
Figure 4.12 shows the initial beach profile DCC05 at the start of a LITPACK profile evolution tool in yellow. Figure 4.12 also shows the maximum beach drawdown for all scenarios considered (Table 4.3). The same information is shown in Figure 4.13 for beach profile DCC03 and Figure 4.14 for beach profile DCC02.

**Figure 4.12: Predicted maximum beach drawdown from the LITPACK profile evolution tool for all scenarios on profile DCC05. The lower panel shows an enlarged view of the upper part of the beach. Please note that the vertical scale of the two images is NOT the same.**



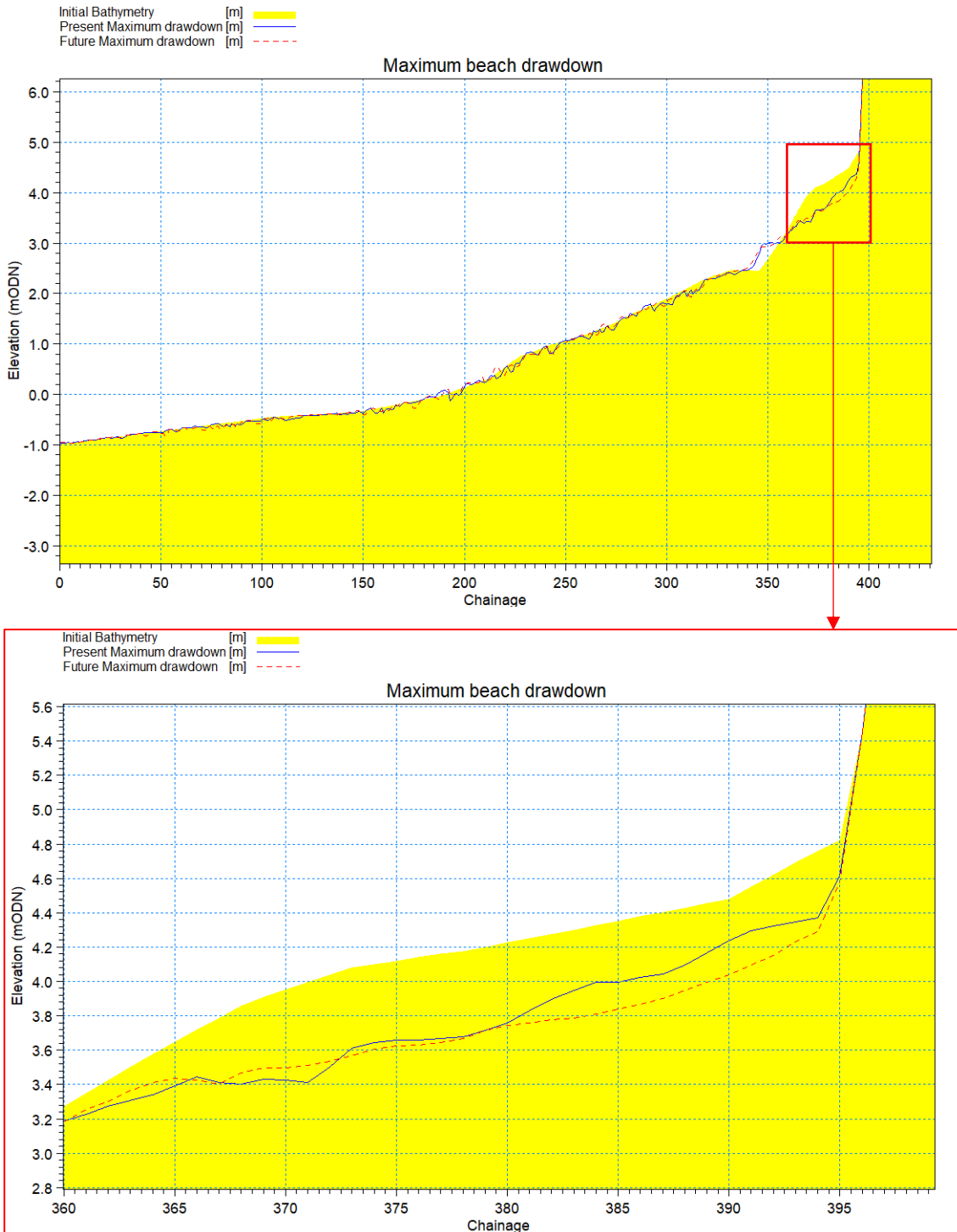
Source: Mott MacDonald, 2021. Includes CCO data.

**Figure 4.13: Predicted maximum beach drawdown from the LITPACK profile evolution tool for all scenarios on profile DCC03. The lower panel shows an enlarged view of the upper part of the beach. Please note that the vertical scale of the two images is NOT the same.**



Source: Mott MacDonald, 2021. Includes CCO data.

**Figure 4.14: Predicted maximum beach drawdown from the LITPACK profile evolution tool for all scenarios on profile DCC02. The lower panel shows an enlarged view of the upper part of the beach. Please note that the vertical scale of the two images is NOT the same.**



Source: Mott MacDonald, 2021. Includes CCO data.

The predicted post-storm beach levels show some erosion near the revetment attributable to enhanced sediment resuspension and cross-shore transport due to wave reflections. The maximum beach drawdown on profiles DCC05, DCC03 and DCC02 for present-day and future scenarios is around 40cm.

An analysis of historical beach profiles from locations DCC05, DCC03 and DCC02 indicate that the beach levels would recover by natural processes in quiescent periods over a few months, and an approximately equilibrium profile would be re-established.

The relatively small beach drawdown during an extreme storm event is attributable to the very dissipative nature of the foreshore, which is completely dry and not influenced by waves for at least half of the tidal cycle. Even during high tidal states, the beach's shallow slope and the sandbars act to dissipate wave energy, and in turn, limit erosion of the upper beach.

The model results reproduce the expected morphological behaviour of the Central Rhyl beach. More significant spatial and temporal changes in the beach profiles are associated with the natural migration of the sandbars in the intertidal region than with extreme storm event.

### 4.4.3 Independent model validation with XBeach

#### 4.4.3.1 Introduction

It is recommended to simulate coastal processes using two or more independent numerical models to reduce uncertainty when data are scarce. In the present study, we have undertaken an independent assessment of LITPACK model results with the XBeach<sup>3</sup> model. The XBeach model has two variants: (a) an infragravity (long-wave) version for wave propagation, long waves and mean flow, sediment transport and morphological changes of the nearshore area and beaches during storms; and (b) a non-hydrostatic (wave-resolving) version for the detailed simulation of storm impacts on beaches. In this study, we use both versions of XBeach and compare the predicted beach profile responses with each other and with the results from the LITPACK profile evolution model presented in Section 4.4.

Both XBeach models were set up as 1D cross-shore models using beach profile DCC05, a steering file, and the model parameter settings recommended by Smit et al. (2010) and McCall et al. (2014). The steering file includes the information on the model grid and bathymetry, boundary conditions, definition of physical processes to be included, description of numerical solvers and other parameters. In common with the LITPACK model, the revetment was defined in the XBeach models as a non-erodible hard-structure, and the depth of available beach sediments was assumed to be unconstrained. The beach sediment properties are defined in Table 4.1. The XBeach models were driven by the same wave and water level conditions used for the LITPACK profile evolution model (Section 4.4.1).

#### 4.4.3.2 Results

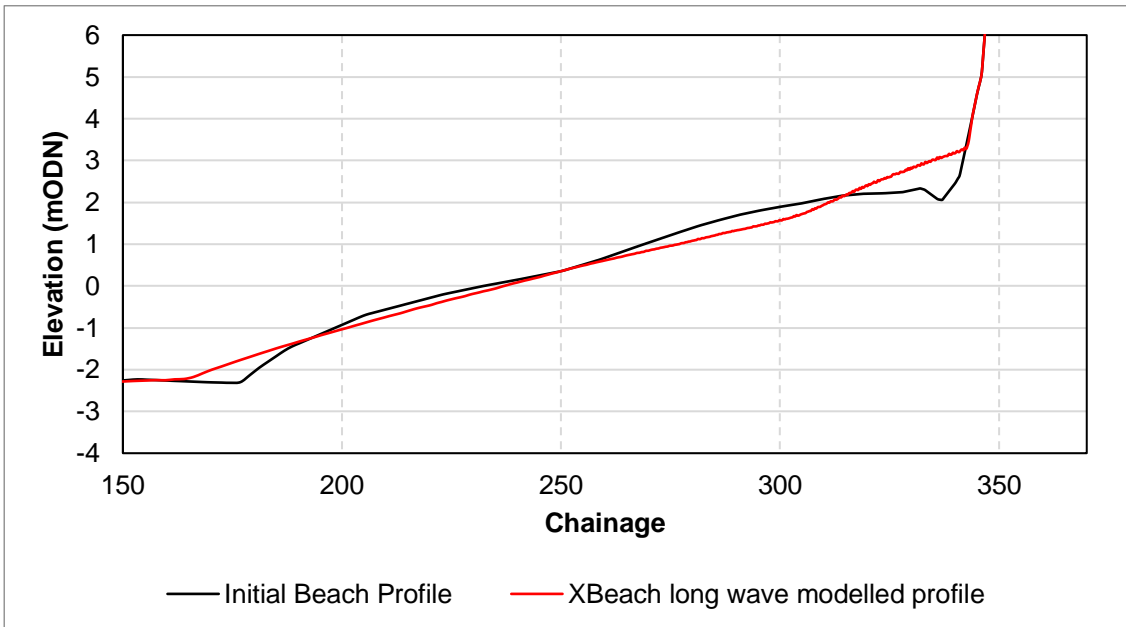
The long-wave and non-hydrostatic XBeach simulation results are shown in Figure 4.15 and Figure 4.16, respectively. Both results show: (a) sediment accretion at the toe of the structure between chainage 320 and 340; (b) limited erosion of the upper beach between chainage 200 and 320; and (c) offshore accretion beyond chainage 200. Differences in the amount of erosion and accretion predicted by the XBeach models are shown in Figure 4.17. The most significant difference is observed near the toe of the revetment, where the XBeach long-wave model predicts approximately 1m more accretion than the XBeach non-hydrostatic model. The localised scour hole is infilled with sediment, and the local beach elevation is raised at this location. Offshore from this accretion zone, the XBeach long-wave model predicts up to 40cm more accretion than the XBeach non-hydrostatic model in the region between chainage 260 and

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<sup>3</sup> <https://oss.deltares.nl/web/xbeach/>

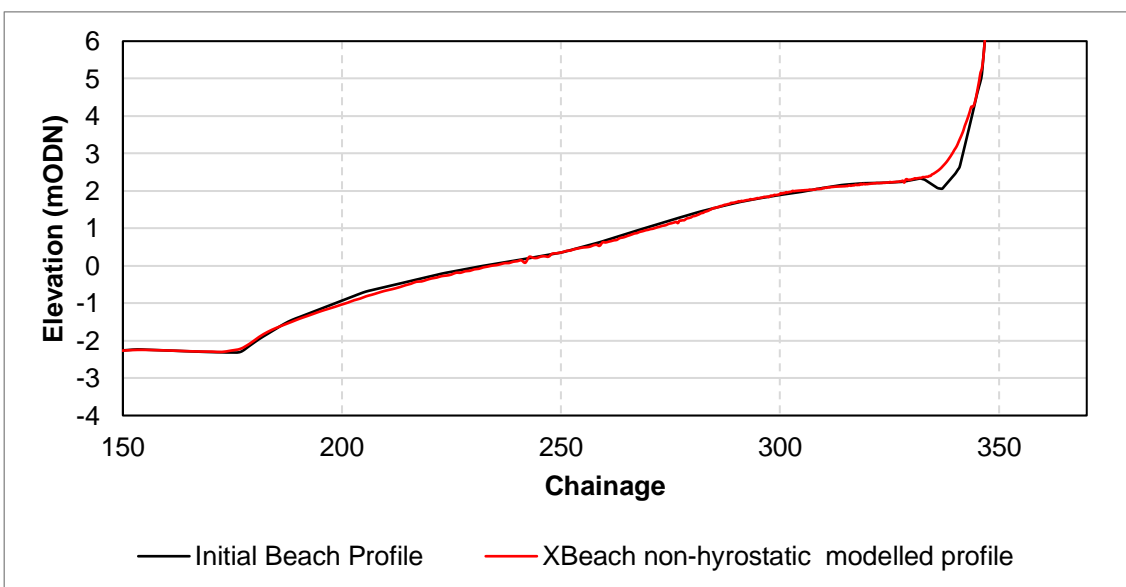
320. Conversely, the XBeach long-wave model predicted more erosion than the XBeach non-hydrostatic model in the region between chainage 170 and 200. In both XBeach models, erosion and accretion volumes are equal, and there is no loss of sediment from the profile.

**Figure 4.15: Results from the XBeach long-wave model simulation showing the measured pre-storm profile DCC05 profile and the predicted post-storm profile for Scenario 1.**



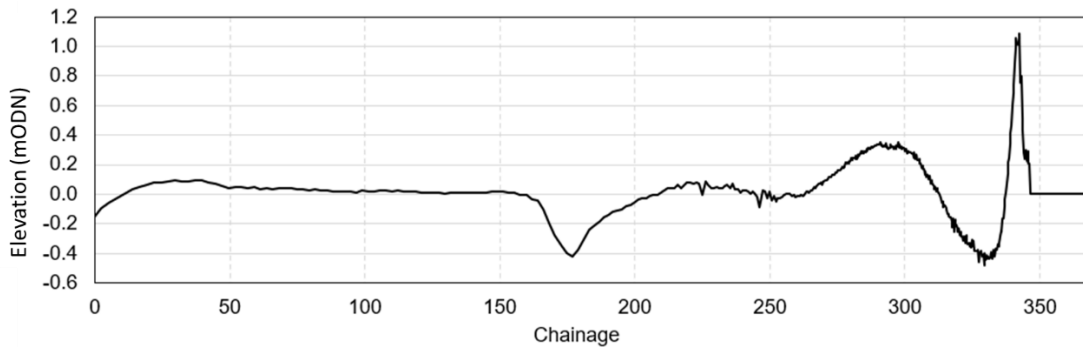
Source: Mott MacDonald, 2021

**Figure 4.16: Results from the XBeach non-hydrostatic model simulation showing the measured pre-storm profile DCC05 profile and the predicted post-storm profile for Scenario 1.**



Source: Mott MacDonald, 2021

**Figure 4.17: Difference between XBeach long-wave and non-hydrostatic model prediction of beach profiles changes during the simulated storm of Scenario 1.**



Source: Mott MacDonald, 2021

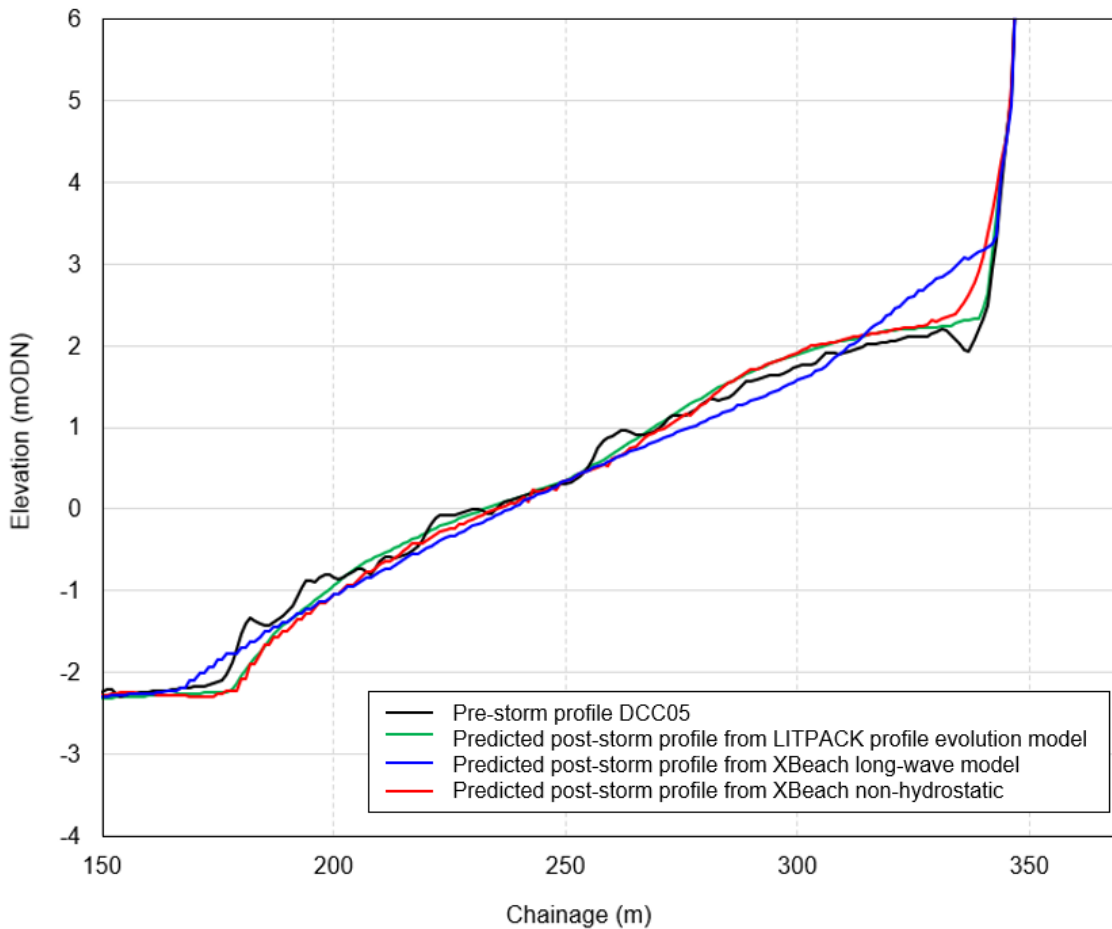
Differences in predicted profile response to the simulated storm between the long-wave and non-hydrostatic versions of XBeach reflect the model parameterisation. Specifically, several parameters determine the sediments' response to applied shear stresses. These are defined differently in each of the model's default mode. Typically these parameters would be adjusted to calibrate the model against measure profile responses. However, in this case, there is uncertainty about how the measured pre-and post-storm DCC05 beach profile relates to the forcing wave conditions, and thus calibration is restricted. Instead, the XBeach model results provide an envelope of potential beach profiles changes to compare with the LITPACK profile response model.

Figure 4.18 shows the pre-storm profile DCC05 (black line), the post-storm profile predicted by the LITPACK profile response model (green line) and the post-storm profiles predicted by the XBeach long-wave (blue line) and non-hydrostatic (red line) models for Scenario 1. The data plotted in this way demonstrate a generally good agreement between the model predictions. The following observations are made:

- The XBeach profiles tend to be slightly smoother and tend to over-predict accretion in front of the revetment;
- All the models predict accretion adjacent to the revetment;
- The XBeach long-wave model predicts erosion on the upper beach, while the LITPACK profile response model, and the XBeach non-hydrostatic models, predict accretion;
- At more than 100m from the revetment, all model predictions agree well; and
- The models tend to smooth out the ridges on the measured profile.

The most important modelling result relates to the magnitude of predicted beach profile changes to the simulated storm compared with the magnitude of the natural temporal changes in the measured beach profiles over weeks, months and years. The predicted beach elevation changes  $O(\pm 0.5\text{m})$  during the simulated storm are around three times less than the measured variability in beach elevation between 2002 and 2020 (Figure 3.1). This result gives confidence that the models are neither greatly over nor under predicting beach responses to the storm and that the results provide a credible guide to likely cross-shore beach responses to extreme events. However, without suitable calibration/validation data, the model prediction precision cannot be quantified with certainty.

**Figure 4.18: Comparison between LITPACK profile evolution and XBeach model prediction of profile DCC05 response to storm conditions for Scenario 1.**



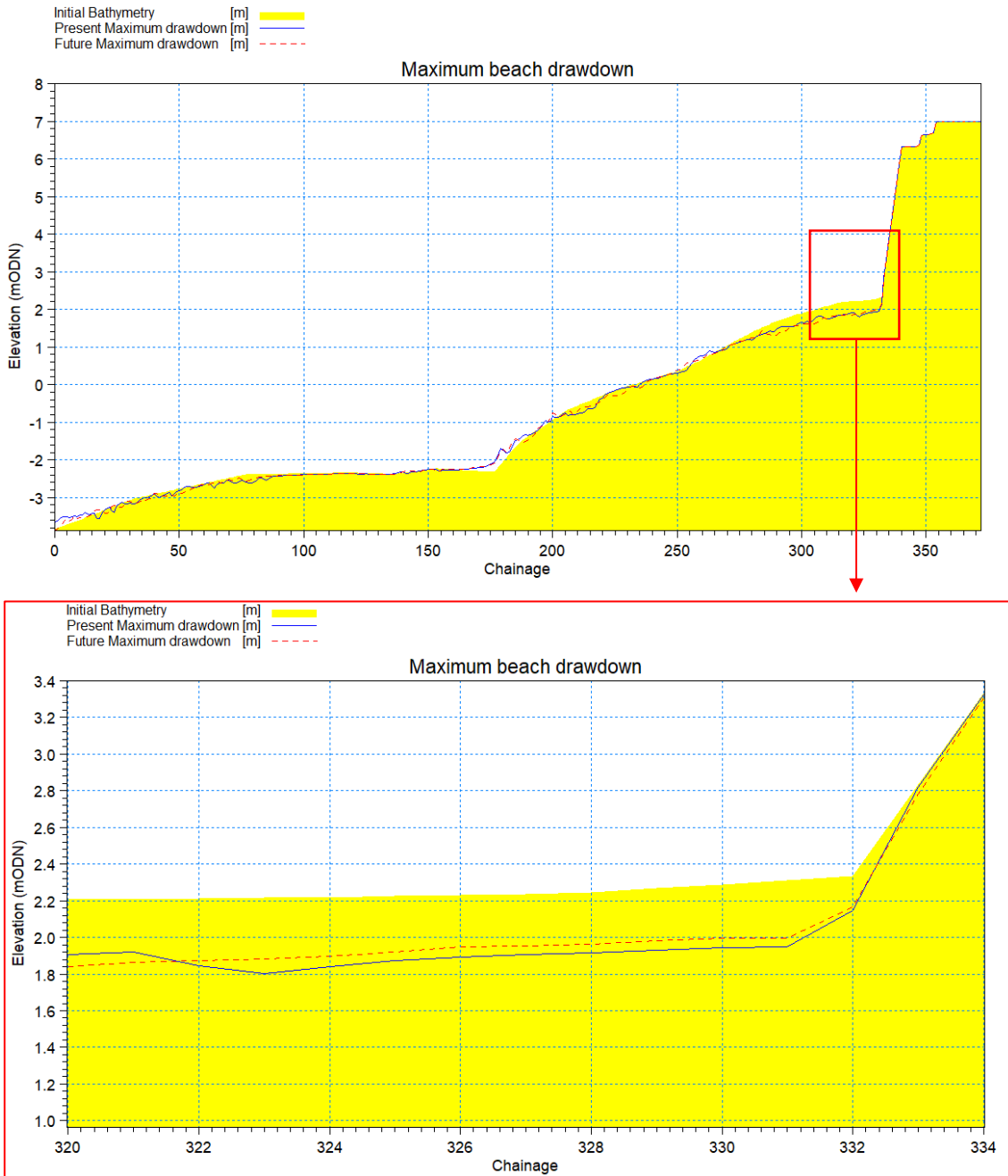
Source: Mott MacDonald, 2021. Includes CCO data.

#### 4.5 LITPACK model simulation - Scheme results

In order to assess the efficacy and potential impacts of the Scheme, the new coastal defence structure was added to the baseline model, and the morphological response to the same extreme events described in Section 4.4.1 were examined using LITPACK.

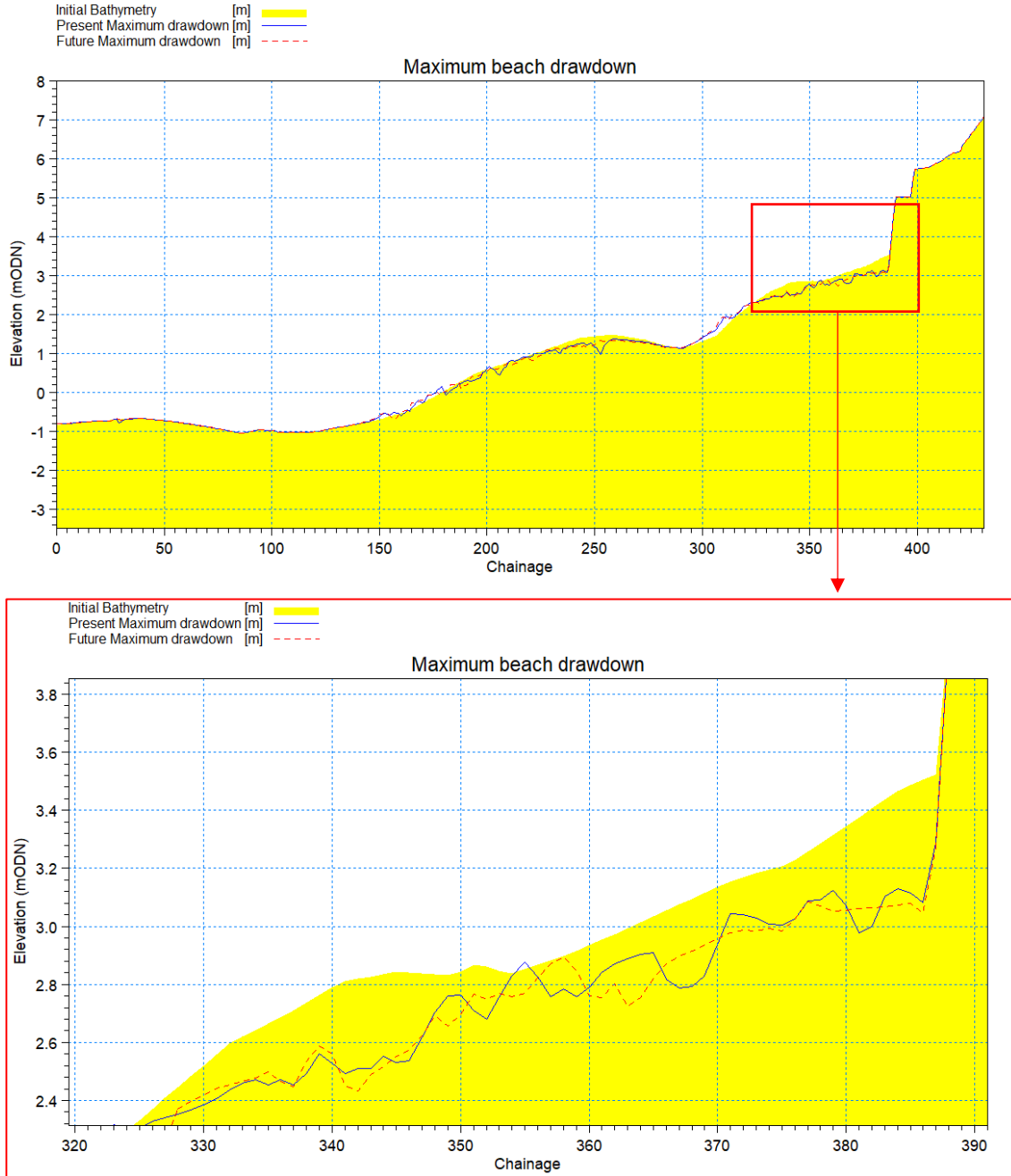
Figure 4.19 shows the initial beach profile DCC05 with the proposed Scheme at the start of a LITPACK profile evolution tool in yellow. Figure 4.19 also shows the maximum beach drawdown for all scenarios considered (Table 4.3). The same information is shown in Figure 4.20 for beach profile DCC03 and in Figure 4.21 for beach profile DCC02.

**Figure 4.19: Predicted maximum beach drawdown from the LITPACK profile evolution tool for all scenarios on profile DCC05 with the Scheme. The lower panel shows an enlarged view of the upper part of the beach. Please note that the vertical scale of the two images is NOT the same.**



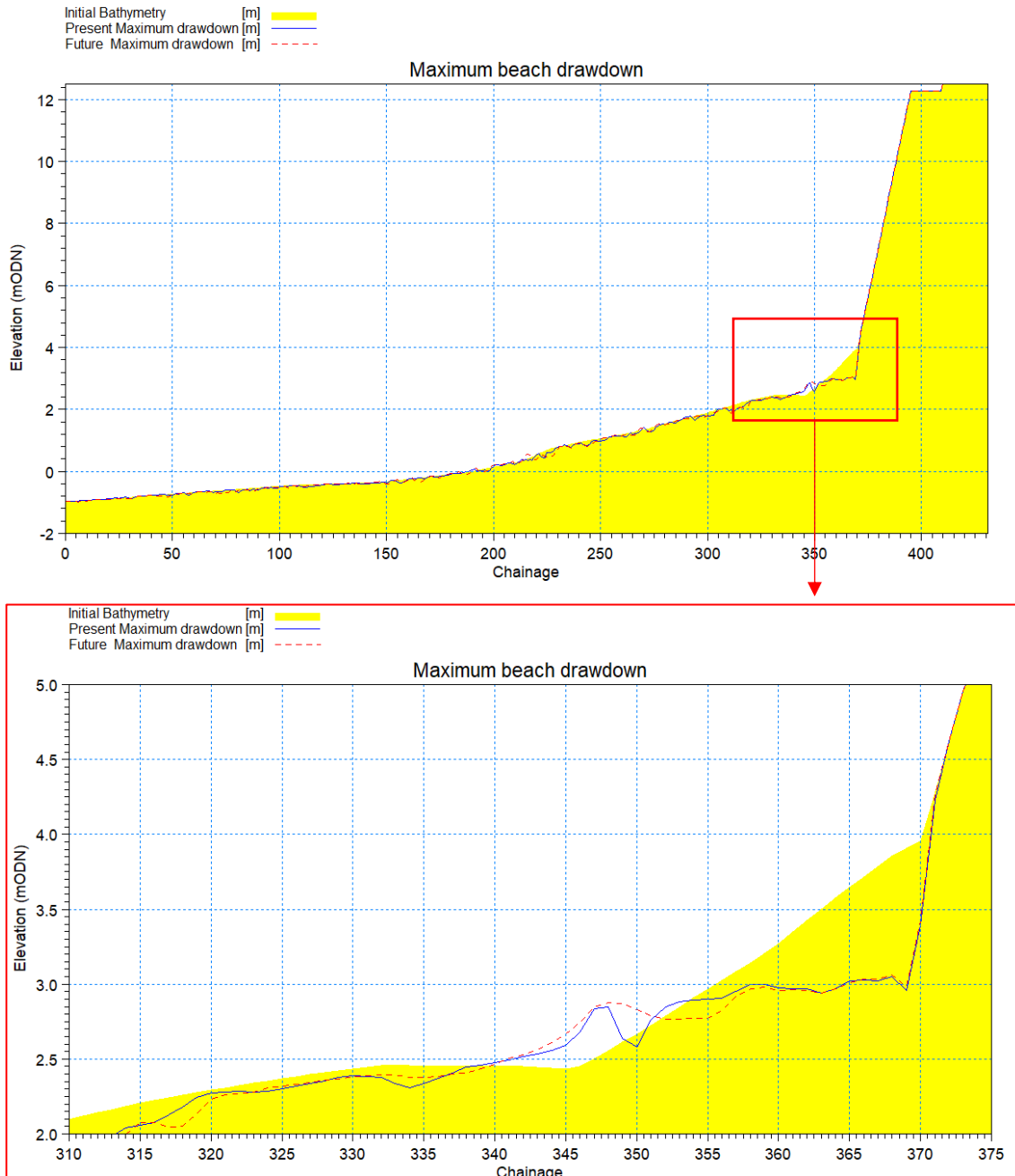
Source: Mott MacDonald, 2021. Includes CCO data.

**Figure 4.20: Predicted maximum beach drawdown from the LITPACK profile evolution tool for all scenarios on profile DCC03 with the Scheme. The lower panel shows an enlarged view of the upper part of the beach. Please note that the vertical scale of the two images is NOT the same.**



Source: Mott MacDonald, 2021. Includes CCO data.

**Figure 4.21: Predicted maximum beach drawdown from the LITPACK profile evolution tool for all scenarios on profile DCC02 with the Scheme. The lower panel shows an enlarged view of the upper part of the beach. Please note that the vertical scale of the two images is NOT the same.**



Source: Mott MacDonald, 2021. Includes CCO data.

The predicted post-storm beach levels with the Scheme in place are generally similar to the baseline results shown in Section 4.4. Erosion near the revetment is attributable to enhanced sediment resuspension and cross-shore transport due to wave reflections. The maximum beach drawdown on profiles DCC05 and DCC03 for present-day and future scenarios is around 40cm. With the Scheme, Profile DCC02 shows a more significant beach drawdown of almost 1m.

As per the baseline conditions and the historical analysis, it is believed that beach levels would recover by natural processes in quiescent periods over a few weeks and months, and a quasi-equilibrium profile would re-establish.

In common with the baseline conditions, the relatively small beach drawdown during an extreme storm event is attributable to the very dissipative nature of the foreshore, which is completely dry and not influenced by waves for at least half of the tidal cycle. Even during high tidal states, the beach's shallow slope and the sandbars act to dissipate wave energy and limit erosion on the upper beach. The modelling results show changes to the beach profiles are confined to the upper beach near the structure, which has a small, local effect.

#### 4.6 Summary

- Cross-shore 1D modelling has demonstrated that a simulated 1% AEP storm results in morphological changes in the beach profile that are less than the typical measured seasonal and inter-annual variability;
- The results indicate that the natural variability in beach morphology in front of the revetment results from the seasonal and inter-annual cross-shore migration of the sandbars, together with the longshore transport generated by tidal and wave-induced currents rather than a cross-shore response to storm events *per se*;
- When modelling a future scenario (2120), including sea-level rise (0.87m) and an increase in wave height (10%), the LITPACK profile response model results show a modest increase in beach drawdown of around 10cm. This is attributable to greater wave runup and enhanced wave-generated bed shear stress;
- The maximum beach drawdown with the Scheme in place is similar to the baseline conditions, except for profile DCC02, where the predicted beach drawdown for present and future conditions is around 1m; and
- The results for the Scheme indicated that the effect of the structure is limited and localised to the upper part of the beach close to the Scheme. There are no significant changes to the intertidal and lower beach profile.

It is again noted that the accuracy of the present models is constrained by data availability for calibration/validation. In order to add additional confidence to the study, modelling was undertaken using independent modelling software. While the results obtained are similar, they should be used with caution in detailed design work.

## 5 Coastline evolution

The MIKE Powered by DHI Littoral Processes FM model (LPFM) was used to simulate the impact of the proposed Central Rhyl sea defence scheme (the Scheme) on the surrounding shoreline over short to decadal time-scales. The Littoral Processes FM model is an integrated modelling system that simulates non-cohesive transport at a point and along quasi-stationary coastlines using an n-line approach. The total sediment transport rate  $q_t$  is calculated using

$$q_t = q_b + q_s \quad (1)$$

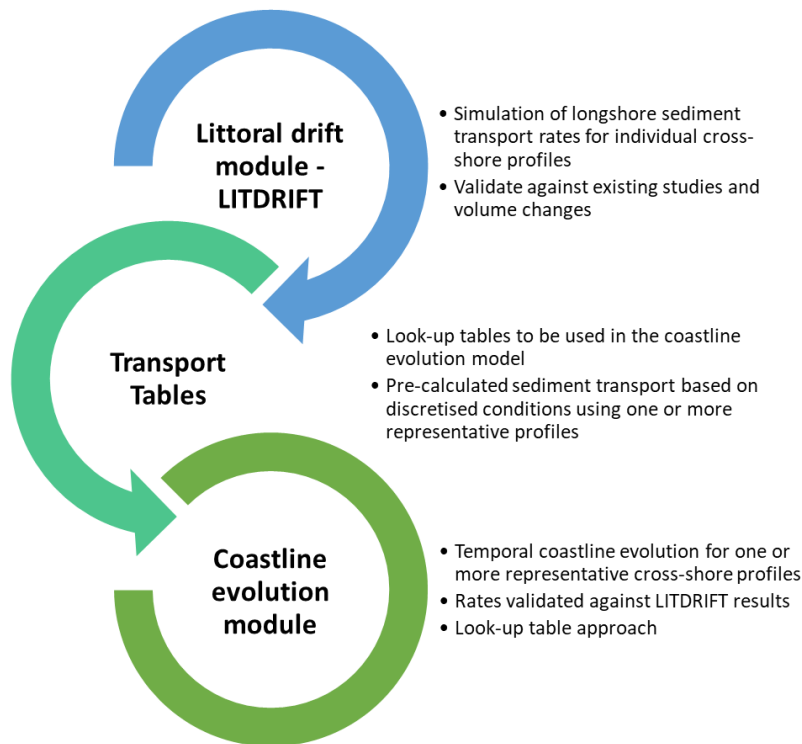
where  $q_b$  is the bed load transport rate and  $q_s$  is the suspended sediment transport rate. Bed load transport is calculated using the Engelund & Fredsøe (1979) formula, and the suspended sediment transport calculated as the product of the instantaneous flow velocities and instantaneous suspended sediment concentrations defined using the vertical diffusion equation (Fredsøe, Anderson, & Silberg, 1985) to calculate the vertical variation of suspended sediment concentration.

The LPFM model (Figure 5.1) comprises:

- **A littoral drift module (LITDRIFT)** that calculates the cross-shore distribution of wave height and direction, the related wave-driven currents and littoral drift for one or several individual cross-shore profiles based on wave and current conditions at a single point;
- **A Transport Tables module** to generate and use predicted sediment transport rates in calculations of the coastline evolution module; and
- **A coastline evolution module** to calculate the temporal coastline evolution for one or more representative cross-shore profiles.

The model requires as input, data defining the nearshore wave conditions. In the present study, the wave conditions from the 40-year Met Office WWII hindcast model at Point 1516 have been transformed to locations between the River Clwyd and Point of Ayr using the MIKE21 SW wave model. The resulting nearshore wave data have been used to predict the littoral drift and the beach evolution for both the baseline (present-day) case and the Scheme.

**Figure 5.1: Littoral Processes FM modelling approach**



Source: Mott MacDonald, 2021

## 5.1 Littoral Processes FM setup data

### 5.1.1 Cross-shore profiles

With coordinates reference to mODN and the British National Grid (OSGB36), a total of 26 cross-shore profiles between the River Clwyd and Point of Ayr were identified as being representative in this sediment transport study of the section of the North Wales coastline (Figure 5.2). The profiles were based on the CCO<sup>4</sup> topography survey of June 2020. This survey covered the intertidal and upper beach in detail and was interpolated to 1m resolution. The lower portion of the profile was extracted from the MIKE21 SW wave model bathymetry. The profiles were extended to the depth of closure estimated using Hallermeier (1981) formula to be -8mODN. The CCO topography survey defined the location of the existing defences along the coastline. These were included as “hard structures” in the profiles to ensure no sediment transport was included in the model for those locations.

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<sup>4</sup> [Channel Coastal Observatory Filter](#)

**Figure 5.2: Cross-shore profiles selected to represent the North Wales coastline, between the River Clwyd and the Point of Ayr.**



Source: Mott MacDonald, 2021. Contains CC0 data, 2020.

### 5.1.2 Coastline profile

The initial location of the coastline was defined using the most recent Google Earth image and OS Master Maps and was assumed to represent the MHW tide level. In the LPFM model, the coastal position was then defined at 50m intervals alongshore as the perpendicular distance from a straight, approximately shore-parallel reference baseline.

### 5.1.3 Wave refraction

The refraction of waves as they approach the shore is accounted for in the coastline evolution model by defining the location of the offshore contour. In the model simulations, the offshore contour was defined as the -8m isobath and was taken from the bathymetric data and defined as perpendicular distance from a straight, approximately shore-parallel reference baseline

### 5.1.4 Beach and nearshore sediment characteristics

Previously several sources defining the sediment distribution along this coastline have been reviewed ( Section 2.3) and have been used here to define the sediment properties along each cross-shore profile. These data comprised:

- Particle Size Analysis (PSA) of twelve beach sediment samples obtained by APEM Ltd in November 2020 at upper, middle, and lower beach locations, (Table 2.4, Figure 2.6);
- PSA data of samples from boreholes and trial pits taken as part of a geotechnical by Geotechnics Limited at different depths at the upper part of the beach, the toe of the defences and promenade (Table 2.6, Figure 2.8); and

- Sediment samples obtained at upper, middle, and lower beach locations by Kenneth Pye Associates Ltd in 2010 as part of the Cell Eleven Tide and Sediment Study (CETaSS), Figure 5.3.

**Figure 5.3: APEM (2020) and Kenneth Pye Associates Ltd (2010) samples locations along the north Wales coastline, between Central Rhyl and Point of Ayr.**



Source: Mott MacDonald, 2021. Contains APEM (2020) and Kenneth Pye Associates Ltd (2010) data.

Table 5.1 shows the selected grain size ( $d_{50}$ ) and grading ( $\sqrt{d_{84}/d_{16}}$ ) for the 26 cross-shore beach profiles. A single characteristic sediment size was defined for each cross-shore profile and was varied alongshore according to the PSD data available. Being representative of a wider portion of the cross-shore profile, the PSD data from the lower beach sample was used. At this location,  $d_{50}$  is smaller than the upper beach locations and thus more mobile. This approach is, therefore, conservative as the sediment transport predicted by the model will be larger than values simulated using the coarser sediments on the upper beach.

**Table 5.1: Summary of sediment size ( $d_{50}$ ) and sediment grading ( $\sqrt{d_{84}/d_{16}}$ ) selected to represent the 26 cross-shore profile from Central Rhyl to Point of Ayr.**

Profile	$d_{50}$ (mm)	Grading
DCC01	0.231	2.1
DCC02	0.248	1.7
DCC03	0.251	1.6
DCC04	0.251	1.6
DCC05	0.246	1.7
DCC06	0.246	1.7
DCC07	0.227	1.2
DCC08	0.227	1.2

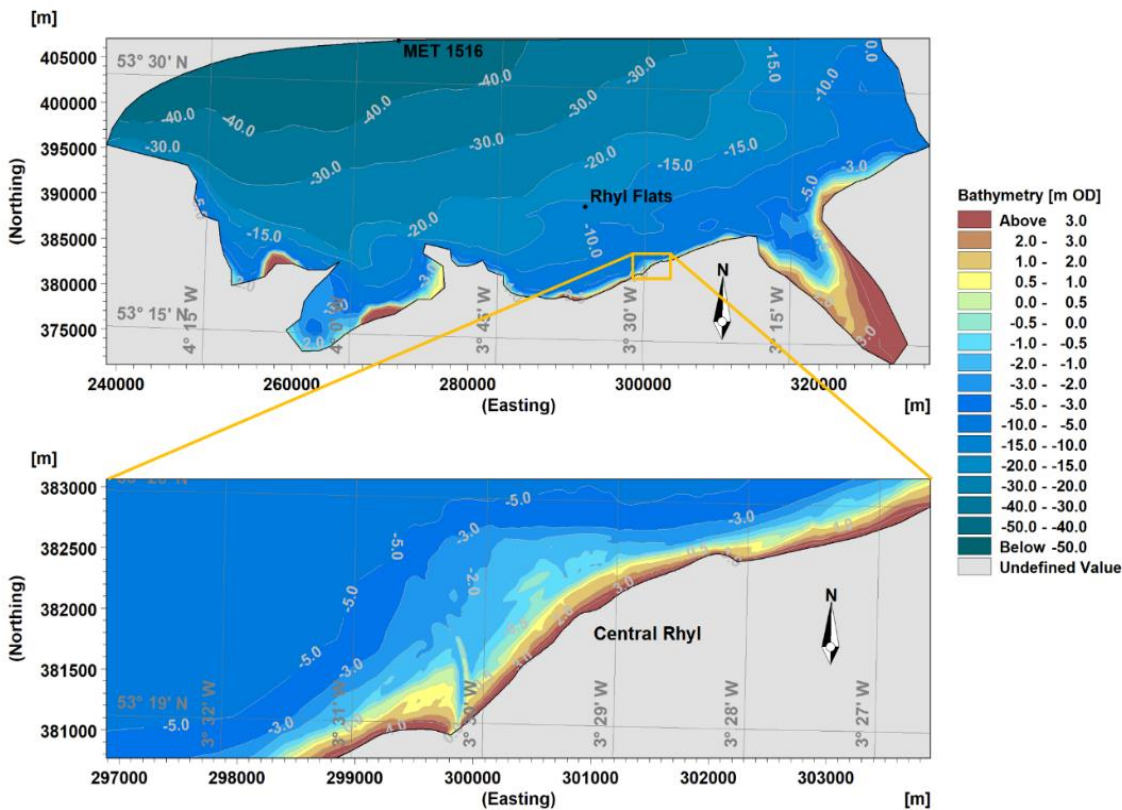
Profile	<i>d</i> 50 (mm)	Grading
DCC09	0.227	1.2
DCC10	0.227	1.2
DCC11	0.227	1.2
DCC12	0.227	1.2
DCC13	0.227	1.2
DCC14	0.247	1.3
DCC15	0.247	1.3
DCC16	0.247	1.3
DCC17	0.247	1.3
DCC18	0.247	1.3
DCC19	0.247	1.3
DCC20	0.247	1.3
DCC21	0.247	1.3
DCC22	0.247	1.3
DCC23	0.246	1.3
DCC24	0.246	1.3
DCC25	0.239	1.2
DCC26	0.239	1.2

Source: Mott MacDonald, 2020

### 5.1.5 Wave data

The calibrated MIKE21 FMSW model was run using hindcast wave and wind data and the astronomical tide at Met Office WWIII Point 1516 as boundary conditions for a period of 40 years (1979-2020). Water level and wave data, including significant wave height (*H*<sub>s</sub>), peak period (*T*<sub>p</sub>), and mean wave direction (MWD), were extracted from the model at locations defined by the depth of closure (Figure 5.4).

**Figure 5.4: Central Rhyll MIKE21 FMSW wave model used to provide waves conditions for each cross-shore profile at locations defined by the depth of closure (-8mODN).**



Source: Mott MacDonald, 2020

## 5.2 Littoral drift model

The littoral drift model (LITDRIFT) was run for ten years (2010-2019). To better understand and validate the transport rates, sensitivity tests described below were undertaken. The main aims of the LITDRIFT modelling were:

- To describe the magnitude, direction and variation of longshore sediment transport from Central Rhyll to the Point of Ayr for the existing conditions (baseline);
- To validate the longshore sediment rates derived from the *Transport tables* used in the Coastline Evolution module, described in Section 5.3 and Section 5.4;
- To validate the baseline results against existing studies and previous historical analysis; and
- To introduce the Central Rhyll Scheme to identify any significant changes in the longshore transport attributable to the Scheme.

When interpreting the results obtained from LITDRIFT the following model limitations must be considered:

- The module calculates the longshore transport for one or several cross-shore profiles based on the wave climate, water levels and sediment properties across the profile;
- Each cross-shore profile is assumed to be shore-normal on a coastline with shore-parallel contours;
- Each profile is independent of those adjacent to it and there is no feedback between profiles;

- The model does not simulate the morphological evolution of the profile over the simulation period. Therefore, it does not adjust the cross-shore normal angle of the profile during the simulation; and
- No cross-shore structure can be included in this model. Only the revetments are included in the profile setup and are defined as “*no sediment transport*” regions of the profiles.

Due to these model limitations, results should be interpreted with care. Nevertheless, the model provides a general understanding of the longshore transport distribution over the profiles and importantly allows an assessment of the potential effect of the Scheme on individual beach profiles.

### 5.2.1 LITDRIFT sensitivity tests

Tests of the LITDRIFT model sensitivity included:

- **Reduction Factor** - The reduction factor is a measure of the amount of directional wave spreading. This process is important for simulating longshore sediment transport as it affects the radiation stress normal to the shore and hence the amount of wave setup. The reduction factor was tested with a value of 1 and 0.5. The longshore transport was considerably reduced using a reduction factor of 0.5 (representing directional spreading associated with wind waves), and the results obtained with this value were more in line with the available rates in the literature.
- **Sediment size** – Model results obtained using a constant grain size along the coastline of 0.249mm with a grading of 1.5 were very similar to longshore transport rates obtained when sediment properties were varied along the coastline. However, since grain size data were available, their use was considered to represent a more realistic scenario.
- **Shoreline orientation** – The shoreline orientation to the wave climate is a critical parameter determining the longshore sediment transport rate. Numerous mobile sandbars characterise this portion of the North Wales coastline, and these features often have an orientation to the incident waves that are different from the coastline. In tests, the orientation of profiles DCC06 to DCC08 and DCC23 to DCC26 was changed to being perpendicular to the sandbars. While this resulted in a decrease in longshore transport rate, the LITDRIFT simulations were undertaken with the profiles orientated with the coastline. This approach is slightly more conservative and allows for changes in sandbar position and orientation through time.

### 5.2.2 LITDRIFT baseline results

To account for seasonal variability, storms, extreme events, and normal conditions, Table 5.2 shows the net annual sediment transport rate ( $Q_{LST}$ ) from the ten-year LITDRIFT simulation at each of the 26 cross-shore profiles. Positive  $Q_{LST}$  values correspond to net longshore transport from west to east.  $Q_{LST}$  values are also used to determine a sediment budget for the coastline, defined as the difference between  $Q_{LST}$  values at adjacent profiles. A negative budget indicates erosion between the two profiles. Please note that the sediment transport at profile DCC01 was manually modified. The original sediment transport (in brackets in Table 5.2) was reduced to reflect the influence of the training wall by allowing sediment transport to occur only over the last 25% of the structure length.

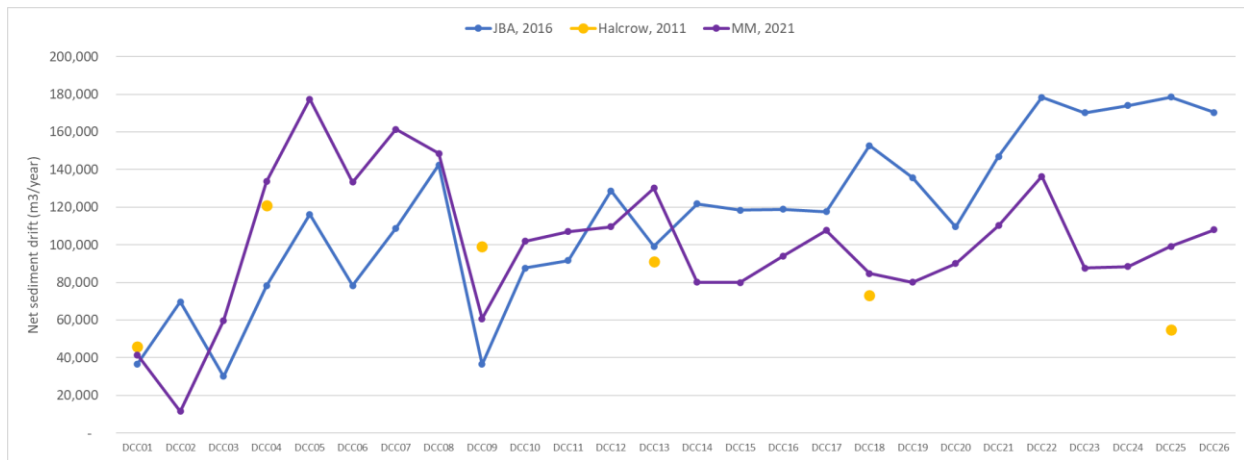
**Table 5.2: Yearly net longshore sediment transport rates ( $Q_{LST}$ ) for each cross-shore profile. A positive transport rate indicates transport west to east. The third column shows the estimated sediment budget for the coastline. A negative budget indicates erosion between the two consecutive profiles.**

Profile	Net sediment transport rate, $Q_{LST}$ (m <sup>3</sup> /year)	Sediment budget (m <sup>3</sup> /year)
DCC01	25,986 (41,496)	
DCC02	11,467	14,519
DCC03	59,809	- 48,342
DCC04	133,675	- 73,867
DCC05	177,330	- 43,654
DCC06	133,284	44,045
DCC07	161,411	- 28,127
DCC08	148,560	12,851
DCC09	60,799	87,761
DCC10	101,936	- 41,137
DCC11	107,091	- 5,155
DCC12	109,604	- 2,513
DCC13	130,192	- 20,589
DCC14	80,073	50,119
DCC15	79,904	170
DCC16	94,003	- 4,099
DCC17	107,695	- 13,693
DCC18	84,809	22,887
DCC19	80,067	4,742
DCC20	90,065	- 9,998
DCC21	110,225	- 20,160
DCC22	136,358	- 26,133
DCC23	87,636	48,722
DCC24	88,401	- 765
DCC25	99,149	- 10,748
DCC26	108,010	- 8,861

Source: Mott MacDonald, 2020

The longshore sediment transport rates from LITDRIFT have been compared to previous studies, including the JBA *East Rhyll coastal defence scheme - coastal processes assessment and modelling*, 2016 and the Halcrow *Cell Eleven Tide and Sediment Study (CETaSS)*, 2011. The comparison between predicted  $Q_{LST}$  values is shown in Figure 5.5. Although  $Q_{LST}$  values in each study are obtained using different data, model software/approach or simulation time, the results show a net west to east transport of sediment with the same order of magnitude.

**Figure 5.5: Validation of annual longshore sediment transport rates ( $Q_{LST}$ ) obtained using LITDRIFT (purple line) compared to existing studies (yellow points and blue line) at each of the 26 cross-shore profiles from Central Rhyl to the Point of Ayr.**



Source: Mott MacDonald, 2021. Contains JBA (2016) and Halcrow (2011) data

Estimates of  $Q_{LST}$ , together with the sediment budget for the entire coastline, are shown in Figure 5.6. The budget results show an erosion trend at Central Rhyl and alternating erosion/accretion areas along the rest of the coastline to the Point of Ayr. The changes in the budget, from accretion to erosion, tend to be related to change in the coastline orientation (e.g. between profile DCC05 and DCC06 and between profiles DCC08 and DCC09).

A tendency to lose sediments, probably into the deeper offshore channel, is predicted in the eastern section of the study area, around the Point of Ayr. However, the physical processes acting at the Point of Ayr are complex, and the longshore sediment transport here is influenced by the complicated hydrodynamics, the offshore banks, the deep channels and the tidal flow into and out of the Dee estuary. The LITDRIFT module only simulates the potential transport according to each profile and, therefore, the potential budgets for the coastline between cross-shore sections only. The relative simplicity of the model does not account for the complex coastal processes at the eastern end of the present study area.

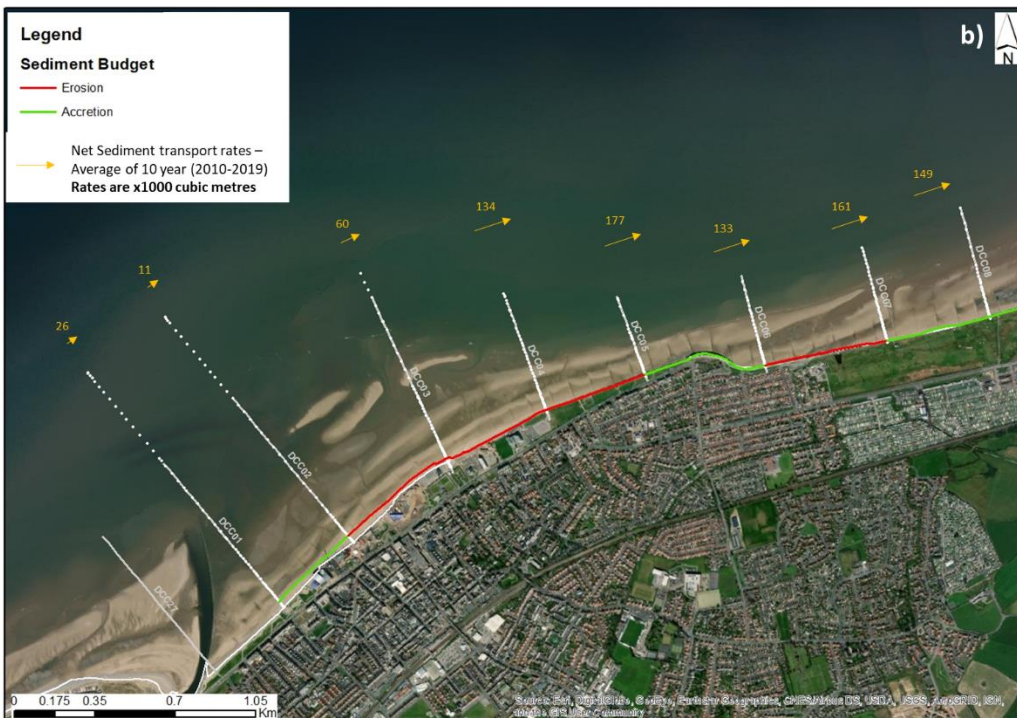
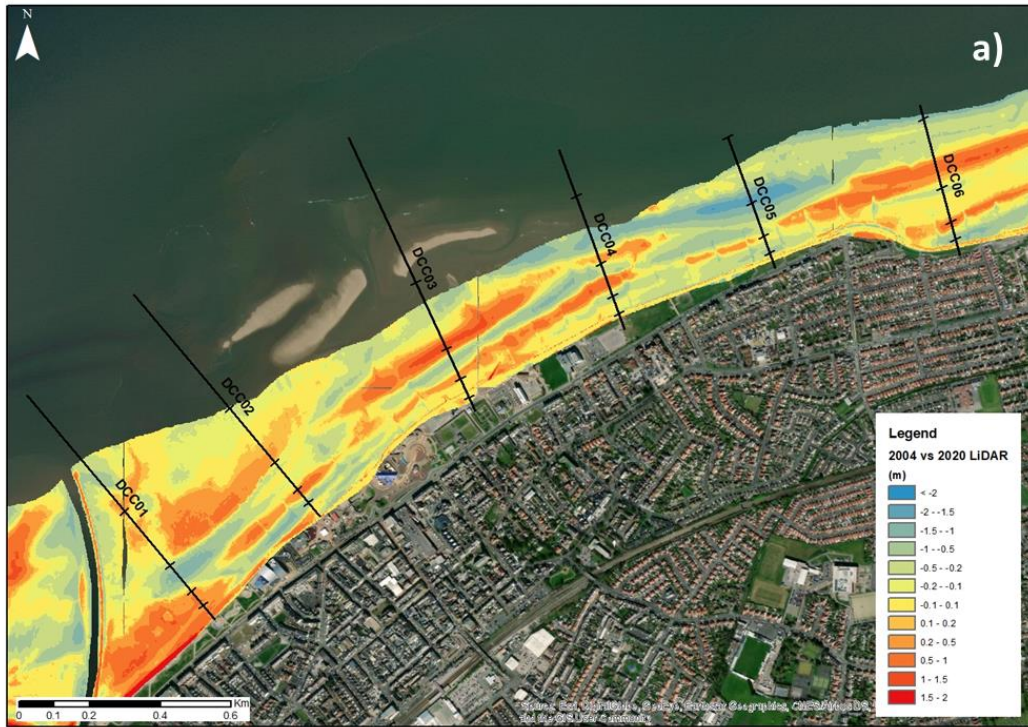
**Figure 5.6: Simulated yearly net longshore sediment transport rates ( $Q_{LST}$ ) for each cross-shore profile. A positive transport rate indicates transport from west to east. The green and red lines show the estimated sediment budget. A negative budget indicates erosion between the two profiles.**



Source: Mott MacDonald, 2021

As additional validation of the LITDRIFT model, the predicted areas of erosion/accretion of the central portion of Rhyll have been compared to the historical analysis undertaken in Chapter 3. Figure 5.7 shows the bed level change between 2004 and 2020 based on LiDAR surveys. Positive values indicate accretion (red), while negative values indicate erosion (blue). These areas were compared with the budget calculated for Rhyll using LITDRIFT. Figure 5.7 shows that in agreement with the sediment budget results, the area protected by the training wall tends to accrete. The central portion of Rhyll frontage is characterised by a combination of accreting and eroding areas, with more clear erosion in the eastern end. Between profile DCC05 and DCC06, there is a considerable change in the shoreline orientation. The area is characterised by the presence of sandbars, which move towards the coast. The sediment budget here from LITDRIFT indicates accretion, while the analysis of LiDAR data indicates an area of alternating accretion and erosion events due to the sandbar movement.

**Figure 5.7: a) Overall bed level change (m) between 2004 and 2020 LiDAR survey. Positive values are indicating accretion, while negative values are showing erosion. Tick marks on the profiles show the 20m, 100m, 200m and 400m chainage; and b) Estimated sediment budget for the Central Rhyl coastline using LITDRIFT. Please note that the scale of the images is not the same.**



Source: Mott MacDonald, 2021. Contains NRW and CCO data.

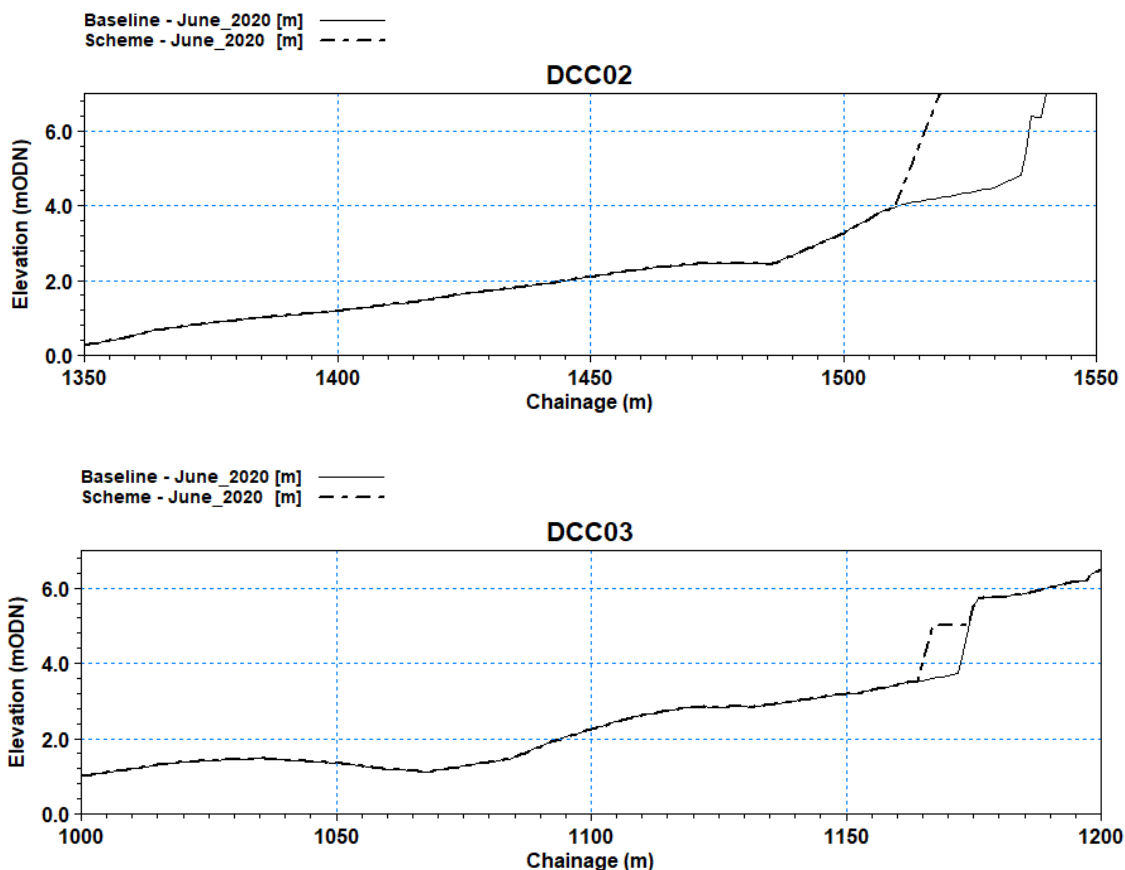
### 5.2.3 LITDRIFT Scheme results

The proposed Central Rhyl Scheme comprising a combination of a stepped and a rock revetment will modify profiles DCC01 to DCC05. For details of the Scheme design, please refer to “Design input statement” report (Mott MacDonald, 2021). Profiles DCC01 to DCC05 were modified to account for both sections of the proposed Scheme so that:

- Profiles DCC01 and DCC02 are located in the area of the stepped revetment. At these locations, the revetment is anticipated to advance the line 23m and 25m, respectively. A slope of 1:3 has been assumed for the structure, and the beach levels have been maintained as per CCO topography survey of June 2020; and
- Profiles DCC03 to DCC05 are located in the proposed rock revetment area. The defence line will be advanced 8m with a slope of 1:2. This situation represents a worst-case scenario in which none of the revetment will be cover by sediment, and the entire rock revetment will be exposed. Consequently, there will be no sediment transport from this area. The beach levels have been maintained as per CCO topography survey of June 2020.

Figure 5.8 shows the modified profiles DCC02 and DCC03 used in the LITDRIFT module. The same changes have been applied to the rest of the profiles.

**Figure 5.8: Schematisation of the Scheme profiles compared to the baseline for profile DCC02 and DCC03 to illustrates the advance of the new Scheme.**



Source: Mott MacDonald, 2021

The LITDRIFT simulations for the Scheme were run for the same 10-year period. The changes to the annual net longshore sediment transport rates are shown in Table 5.3. These results are used to determine a sediment budget for the coastline, calculated as the difference (between adjacent profiles) of the modelled net longshore transport results rates. A negative budget (red)

indicates erosion between the two profiles. Since the proposed Scheme is limited to the central portion of Rhyl, profiles DCC06 to DCC26 have not been modified. However, the sediment budget between DCC05 and DCC06 has changed due to the changes to profile DCC05.

**Table 5.3: Annual net longshore sediment transport rates ( $Q_{LST}$ ) for the cross-shore profile with the proposed Scheme (DCC01 to DCC05). A positive transport rate indicates transport from west to east. The third column shows the estimated sediment budget for the coastline, where a negative budget indicates erosion between the two profiles.**

Profile	Net sediment transport rate $Q_{LST}$ (m <sup>3</sup> /year)	Sediment budget (m <sup>3</sup> /year)
DCC01	25,989 (41,117)	
DCC02	11,208	14,782
DCC03	59,561	- 48,354
DCC04	133,061	- 73,500
DCC05	163,311	- 30,250
DCC06	Same as baseline - 133,284	30,027

Source: Mott MacDonald, 2021

As per the baseline case, the sediment transport at profile DCC01 was manually modified. The original sediment transport, in brackets in Table 5.3, was reduced in order to consider the influence of the training wall, allowing sediment transport to occur only over the last 25% of the structure length.

Table 5.4 compares the baseline and the Scheme  $Q_{LST}$  values. The last column indicates the percentage of change due to the Scheme implementation. Negative values indicate a decrease in the sediment transport rate due to the Scheme.

**Table 5.4: Baseline and Scheme annual net longshore sediment transport rates ( $Q_{LST}$ ) for the Central Rhyl portion of the study frontage. The last column indicates the percentage of change. Negative values represent a decrease in the sediment transport rate due to the Scheme.**

Profile	Baseline - Net sediment transport rate, $Q_{LST}$ (m <sup>3</sup> /year)	Scheme - Net sediment transport rate, $Q_{LST}$ (m <sup>3</sup> /year)	Change in sediment transport rate (%)
DCC01	25,986 (41,496)	25,989 (41,117)	0.0%
DCC02	11,467	11,208	-2.3%
DCC03	59,809	59,561	-0.4%
DCC04	133,675	133,061	-0.5%
DCC05*	177,462	163,311	-8.0%

Source: Mott MacDonald, 2021

The estimated  $Q_{LST}$  values, including the proposed Scheme, are shown in Figure 5.6, together with the sediment budget for the entire coastline and the changes in  $Q_{LST}$  values compared to the baseline for profiles DCC01 to DCC05. The budget results are similar to those presented in the baseline case, showing an erosion trend at Central Rhyl. The results show that the changes in the transport rates due to the Scheme have not altered the sediment budget of the area.

In general, the changes to the  $Q_{LST}$  values due to the Scheme are very small (<5%) and of the order of magnitude of the model accuracy. The largest change in  $Q_{LST}$  is observed at profile DCC05, with a decrease in the sediment transport rate of 8%. This is attributable to the rock revetment and the steeper and shorter nature of the profile, and the protection provided by the large sandbars located along the frontage. Nevertheless, it is important to note that any of the

changes in  $Q_{LST}$  attributable to the new Scheme do not affect the estimated budget for this part of the coastline (Figure 5.6).

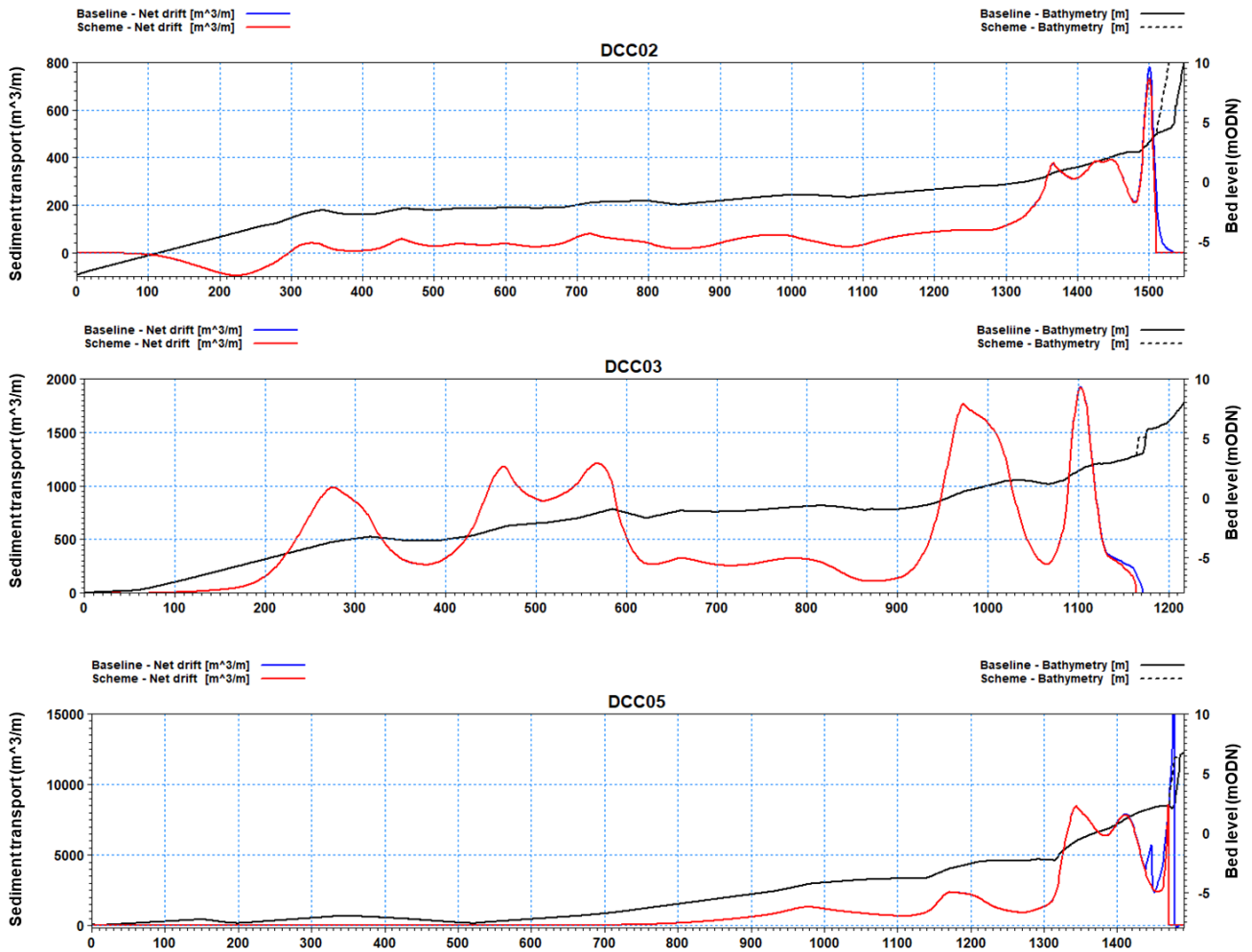
**Figure 5.9: Simulated yearly net longshore sediment transport rates ( $Q_{LST}$ ) values for each cross-shore profile, including the proposed Scheme between profile DCC01 and DCC05. A positive transport rate indicates transport from west to east. The green and red coastline lines show the estimated sediment budget. A negative budget indicates erosion between the two profiles. Compared to the baseline case, the change in the transport rates is shown as percentages in red.**



Source: Mott MacDonald, 2021

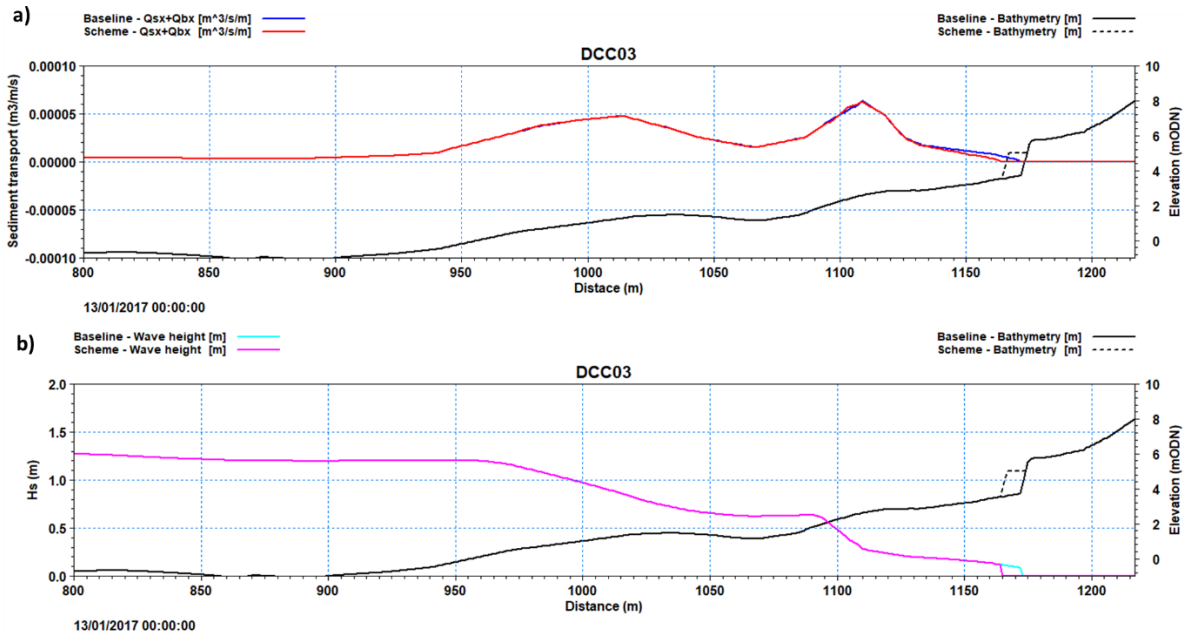
The location along each of the Scheme profiles of changes to longshore transport has been investigated further. Changes in  $Q_{LST}$  values are highly localised and limited to the upper part of the beach, close to the new proposed structure. This region is only affected during the time approaching and at high tide, and for most of the tidal cycle,  $Q_{LST}$  is zero. Figure 5.10 shows the  $Q_{LST}$  distribution along profiles DCC02, DCC03 and DCC05 for both the baseline case (blue line) and the Scheme (red line). The original bathymetry of the profile is also shown in the figure with the proposed Scheme (dashed black line). Changes in  $Q_{LST}$  attributable to the Scheme are confined to the area close to the new structure, where a decrease in the transport rates is observed due to the advance of the defence line. The transport along the rest of the profile is unchanged.

**Figure 5.10: Distribution of yearly net longshore transport ( $Q_{LST}$ ) along profiles DCC02, DCC03 and DCC05. The blue line shows the baseline sediment transport, while the red line indicates the transport with the Scheme in place. The dashed black line shows the proposed new structure. A positive net rate indicates transport west to east.**



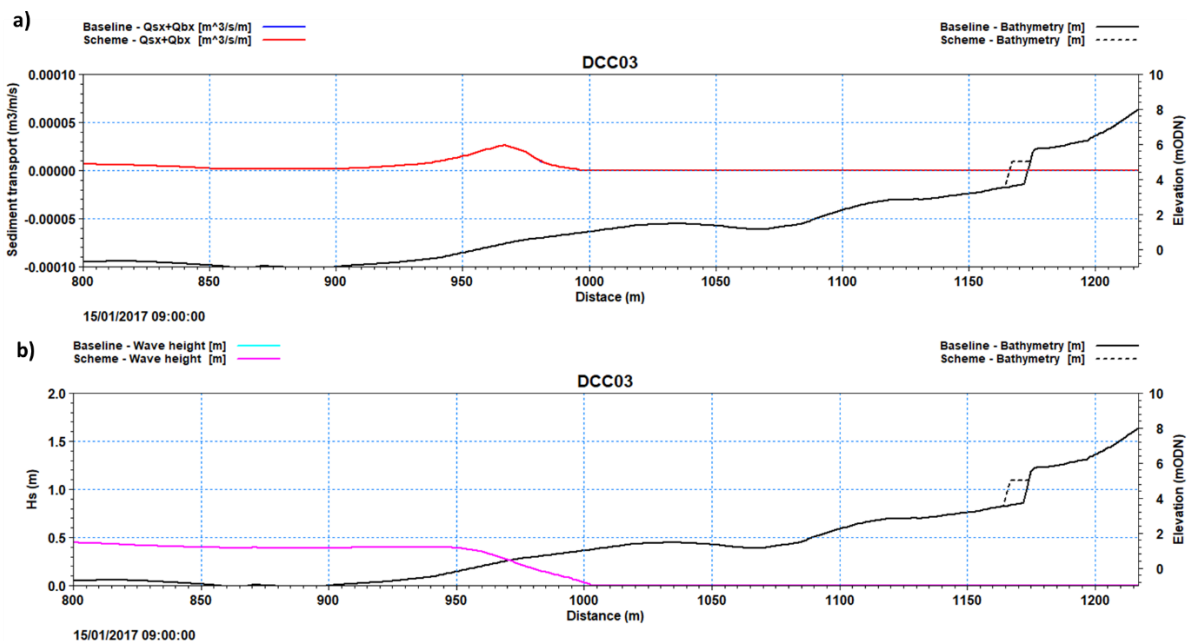
Source: Mott MacDonald, 2021

**Figure 5.11: a) Instantaneous longshore sediment transport ( $m^3/m/s$ ) along profile DCC03 for the baseline (blue line) and the Scheme (red line) case during high water. Please note that the effect of the structure (black dashed line) on the transport is only observed in the upper beach and only extends 30 metres seaward from the defence line; and b) Wave height conditions responsible for the sediment transport for the baseline (light blue) and Scheme (pink) conditions.**



Source: Mott MacDonald, 2021

**Figure 5.12: a) Instantaneous longshore sediment transport ( $m^3/m/s$ ) along profile DCC03 for the baseline (blue line) and the Scheme (red line) case during low water. Please note that there is no effect of the structure (black dashed line) on the transport; and b) Wave height conditions responsible for the sediment transport for the baseline (light blue) and Scheme (pink) conditions.**



Source: Mott MacDonald, 2021

The percentage of time that the water level reaches the Scheme structure has been calculated to be less than 10% of the time for all the profiles with the exception of DCC05. Profile DCC05 is located at the eastern end of the frontage, where the bathymetry is steeper, and the sandbar features that dominate the intertidal area become less evident. Therefore, it is expected that the proposed Scheme will have a more significant impact. However, it should be noted that this change is still limited to the upper part of the beach and only affects the transport rates when the tide is at or above the structure (< 25% of the time).

**Table 5.5: Proposed defence height for profile DCC01 to DCC05 compared to the percentage of the time that the water level is at or above the level where the structure meets the beach. The calculation is based on the 10-year simulation results.**

Profile	The level where the structure meets the beach (mODN)	% of the time that the water is at or above the structure (based on the 10-year simulation results)
DCC01	4.03	3%
DCC02	3.95	3%
DCC03	3.52	8%
DCC04	3.58	7%
DCC05*	2.33	25%

Source: Mott MacDonald, 2021

### 5.3 Transport tables

Using the Littoral drift table generation module, alongshore sediment *Transport Tables* were generated for use in the coastline evolution modelling. Rather than calculate the sand transport at every point on the profile and then integrate it to obtain the net longshore transport for each time interval, this is pre-calculated in the *Transport Tables* and used efficiently as look-up tables. Table 5.6 details the range and intervals of input parameters chosen to derive the transport tables used in this study.

Please note the coastline evolution model, and associated *Transport Tables*, can only use a maximum of 8 profiles. These profiles have been selected to represent the different orientations of the coastline: DCC01, DCC03, DCC05, DCC08, DCC15, DCC21, DCC24 and DCC26 (Figure 5.2).

**Table 5.6: Input parameters range and intervals for *Transport Tables* calculation**

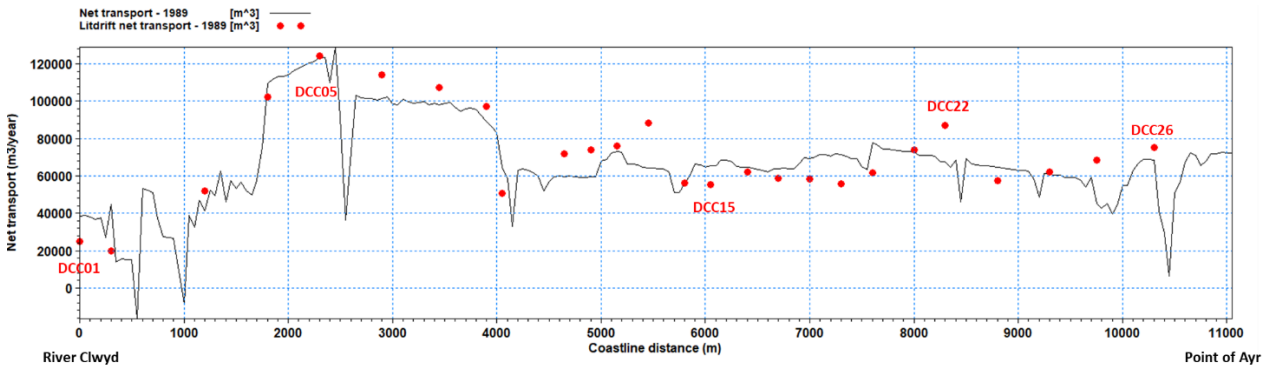
Condition	Minimum	Maximum	No. of values
RMS Wave height (m)	0	3.8	10
In-coming wave angle (relative to profile)	-89	89	18
Peak wave period (s)	1	18	18
Mean water level (MSL)	-4.5	5.0	19

Source: Mott MacDonald, 2021

As a check on the *Transport Table*, the coastline evolution model was run for a year, without any morphological coastline update. The net sediment transport rates obtained from this simulation along the entire coastline were compared to the results obtained using the 26 cross-shore profiles in LITDRIFT (Figure 5.13). The figure shows the comparison between the LITDRIFT net transport rates at each profile (red dots) and the coastal evolution simulation (black line). It demonstrates that the rates calculated using the transport table are similar to the results obtained using the detailed calculation undertaken with the 26 profiles in LITDRIFT, with a difference of +/-20% for most of the profile locations. Importantly, a similar west to east trend in longshore transport is observed.

Figure 5.13, therefore, demonstrates good agreement between the two different modelling approaches and shows that the *Transport Tables* approach, with the discretisation applied, is suitable for the determination of sand transport in the coastline evolution simulations.

**Figure 5.13: Comparison between net sediment transport rate obtained from LITDRIFT (red dots) and Coastal Evolution Module (black line) for a one-year simulation. Some of the LITDRIFT profiles are labelled to provide a location reference.**



Source: Mott MacDonald, 2021

## 5.4 Coastline evolution module – existing conditions

The coastline evolution module simulates the temporal coastline evolution for the coastline between the River Clwyd and the Point of Ayr based on eight representative profiles (section 5.3). The model calculates the movements of the coastline position relative to a straight baseline. The model is, with minor modifications, based on a one-line theory, in which the cross-shore profile is assumed to remain unchanged during erosion/accretion. Thus, the coastal morphology is solely described by the coastline position (cross-shore direction). The sediment transport information is derived from information in pre-generated littoral drift transport tables (Section 5.3).

The study frontage is highly engineered, with revetments, groynes and seawalls located all the way to Prestatyn. These structures are included in the coastline evolution module as follows:

- Groynes are defined as non-permeable structures of various lengths oriented normal to the baseline. Sand transport is blocked partially or fully by the groyne, depending on its length. The sheltering effect from wave action behind a groyne is represented by a reduction in the “undisturbed” transport rates. Please note that the model is limited to a maximum of 40 groynes per simulation; and
- A revetment is a structure introduced into the model to define the limit of coastal retreat. Beach drawdown can continue during a simulation, and the water depth increases in front of the revetment until a minimum is reached, determined by the still water elevation and the nature of the incident waves.

From west to east the structures included in the coastline evolution model to simulate the existing conditions of the study frontage include:

- **River Clwyd training wall** - The river mouth is stabilised by a training wall (Figure 5.14). The structure partially shelters the adjacent coastline from the wave action and, therefore, blocking and modifying the longshore sediment transport. The training wall is represented in the coastline evolution model as a groyne; and
- **Central Rhyl sea defences** – The frontage includes a series of concrete stepped revetments, seawalls and wooden groynes. The defence line has been extracted from an OS

Master Map, and it is included in the coastline evolution model as a revetment structure that prevents any coastal retreat at this location.

Although most of the wooden groynes have been included in the coastline evolution model, in reality, these groynes generally comprise open wooden panels that allowing passage of most of the longshore sediment transport. Therefore, it was considered more conservative to include these structures in the baseline simulation with a large bypass length of 80%, thus allowing more sediment transport in this area. The bypass length is defined in the model as the distance from the tip of the structure where sediment bypassing may occur despite the presence of the structure. The bypass length was chosen by estimating the groynes potential efficiency from available images. Figure 5.15 shows some images of the wooden groynes located along Central Rhyl.

- **Splash Point rock revetment** – The rock revetment at Splash Point has been included in the model as a revetment. Its location has been defined using an OS Master Map.

**East Rhyl sea defences** – Similar to Central Rhyl, East Rhyl sea defences include a combination of seawalls, concrete revetments and wooden groynes. The hard defence line is included as a revetment along the entire East Rhyl frontage, based on an OS Master Map. At this location, most of the wooden groynes (Figure 5.16) are included in the coastline evolution model with a large bypass length (80%), thereby ensuring a more conservative transport along this section of the frontage. In addition to the wooden groynes, a concrete structure extending over the width of the intertidal area (Figure 5.17) is located in between profiles DCC07 and DCC08. The structure blocks part of the sediment transport (Figure 5.17), and therefore, it has been included in the model as a groyne. In order to account for its sheltering effect on the wave action in the transport rates, a bypass length of 25% was defined for this structure.

- **Prestatyn frontage** – This area of the frontage contains a combination of concrete revetments/seawalls and fishtail rock groynes (12 in total). The revetments have been included in the model using an OS Master Map to define the location. The fishtail rock groynes, located between profiles DCC10 and DCC22, have been included in the coastline evolution model as simple groynes, with their length based on the maximum extent of the fish tail portion of the groyne. A bypass length of 25% was defined for these structures. Figure 5.18 shows the rock groyne field at Prestatyn and the clear effect that these structures have on blocking part of the sediment transport of the area.

Due to model limitations, not all the wooden groynes in the study area could be included in the model. The selection of structures to include in the simulation was carefully considered and involved removing small groynes and the groyne located in the shadow area of the training wall. This assumes that the effect of the wall is significantly more than the wooden groynes. Figure 5.19 shows all the groynes structures (wooden, concrete and fishtail) for the frontage. The ones in red are the ones excluded from the baseline simulation. The location of all the structures (groynes and revetments) described above, and included in the coastline evolution model, is shown in Figure 5.20, as per the model setup.

**Figure 5.14: River Clwyd training wall shown below is included in the coastline evolution model as a groyne, to ensure its sheltering effect is considered in the longshore transport rates.**



Source: Google Earth, 2021

**Figure 5.15: Example of the wooden groynes located in Central Rhyl frontage.**



Source: Google Earth, 2021

**Figure 5.16: East Rhyll wooden groynes field. Please note the structures condition and how the sediment is entirely retained by the structures. The image also shows the concrete stepped revetment maintaining the coastline position.**



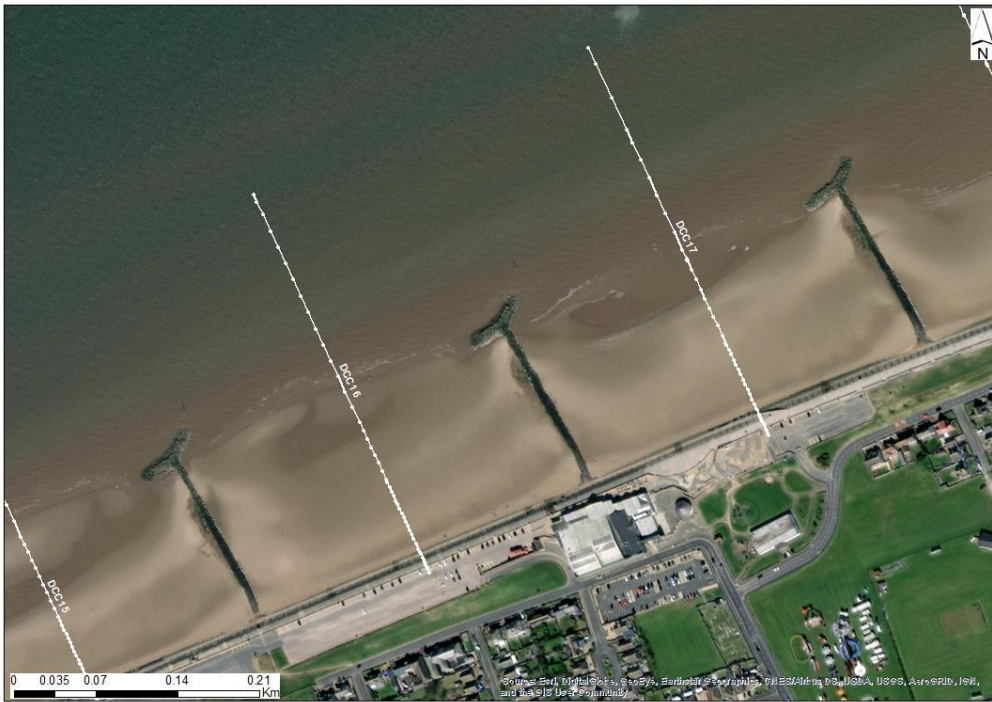
Source: Google Earth, 2021

**Figure 5.17: East Rhyll concrete rock groyne blocking part of the sediment transport. The structure is included in the coastline evolution model.**



Source: Mott MacDonald and Google Earth, 2021

**Figure 5.18: Prestatyn frontage fish tail rock groynes. Due to the nature of these structures and their ability to block part of the sediment transport, they are included in the coastline evolution model.**



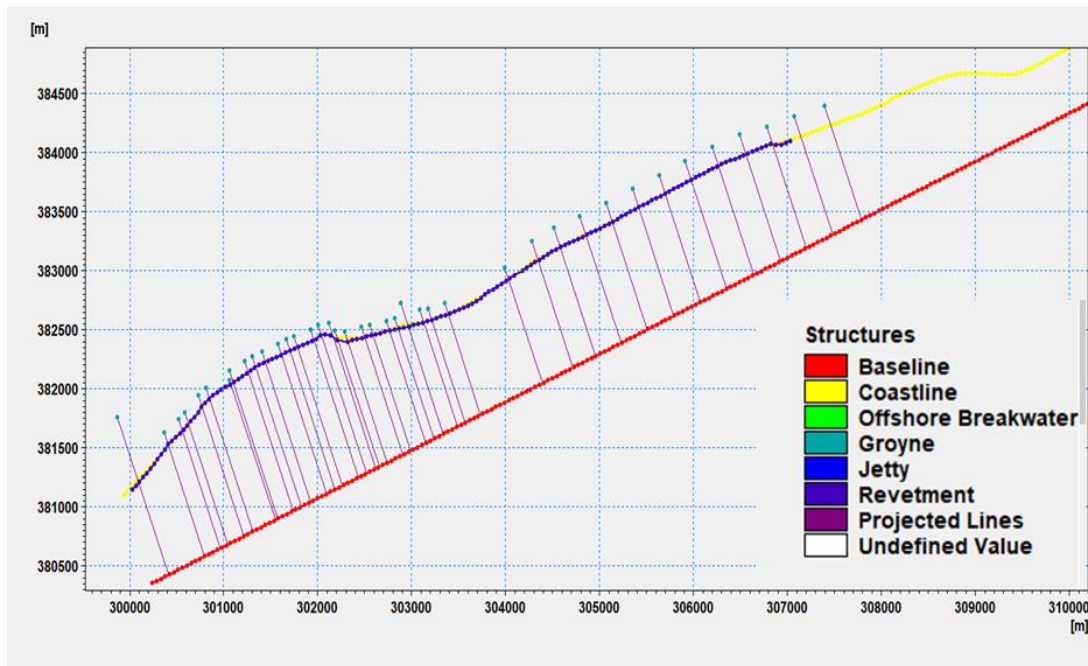
Source: Mott MacDonald, 2021

**Figure 5.19: Location of all groyne structures, including training wall, wooden, concrete and fish tail groynes, on the coastline between Rhyl and the Point of Ayr. The groynes shown in red are excluded from the baseline model simulation.**



Source: Mott MacDonald, 2021

**Figure 5.20: Location of all the structures included in the coastline evolution model for the existing condition simulation. The image also shows the coastline and the baseline of the model.**



Source: Mott MacDonald, 2021

### 5.4.1 Baseline results

For the baseline scenario, and following a two-year model *warm-up* period, the coastline evolution model was run for ten years to understand the potential long-term evolution of the coastline in the study area. The *warm-up* period ensured that the starting point of the ten-year simulation was a smooth coastline, exhibiting the features currently observed in the area (e.g. the accretion against the existing fishtail groynes or in front of some of the revetments).

The ten-year coastline evolution simulation results are shown in Figure 5.21, for the whole frontage, and Figure 5.22, as an example of some specific areas. Both figures detail the position of the current coastline and the simulated coastline position in ten years. The results show that the position of most of the coastline is controlled by the existing hard structures (revetments), and therefore, no coastline retreat is predicted for Central and East Rhyll and Prestatyn frontages.

Some general coastline retreat is observed in the eastern portion of the study area, towards the undefended Point of Ayr frontage. Here the model results indicate that the coastline is retreating to establish a position and orientation commensurate with the rest of the frontage. This portion of the coastline is highly complex due to its interaction with the mouth of the Dee estuary, and the coastline evolution model cannot simulate all the complex sediment and hydrodynamic processes at this location. However, the model results provide a baseline with which to compare simulations of the Scheme and thereby allow assessment of change in longshore transport attributable to the Scheme.

At Central Rhyll, during the ten years of the simulation (Figure 5.22-a), minor changes to the coastline position are observed due to the existing revetments. A clear accretion of the coastline is predicted in the western portion due to the protection provided by the training wall.

This accretion is observed over several years (see Chapter 3), and it is expected to continue as shown from the baseline model results.

Accretion of the coast is also observed along the East Rhyl frontage due to the sheltering effect provided by the revetment at Splash Point (Figure 5.22-a) and the shore-normal structures located here. Additional accretion is also observed along the Prestatyn frontage, especially in the western section, where the fishtail groyne field starts (Figure 5.22-b). This accretion is related to the coastal orientation to the incident wave climate and the corresponding net west to east alongshore sediment transport.

It is important to note that the position of the accreted coastline over the simulation period has the same orientation that the existing accretion behind the structures in the area, indicating that the model is simulating the coastal process of the frontage correctly. This is shown in Figure 5.23, where the baseline coastline in the area of the fishtail groynes is plotted after a period of 10 years.

**Figure 5.21: Position of the coastline at the start (yellow line) and at the end (red line) of the ten year coastline evolution simulation for the entire study area, from Rhyl to the Point of Ayr.**



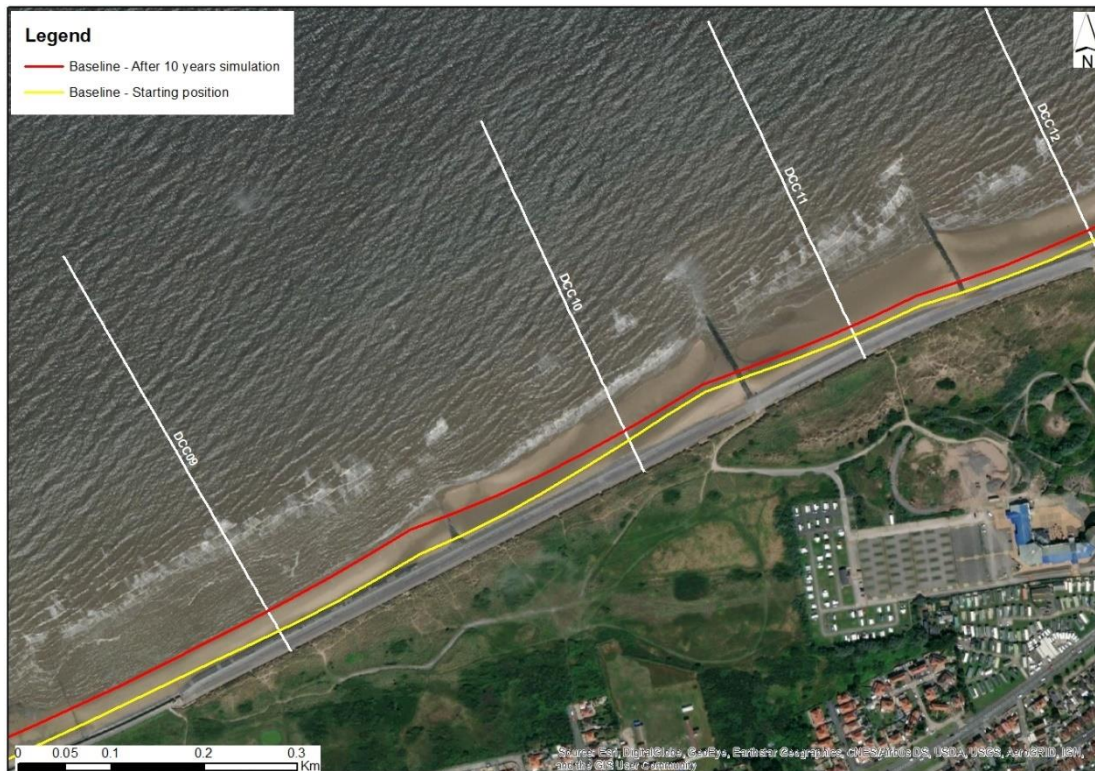
Source: Mott MacDonald, 2021

**Figure 5.22: Position of the coastline at the start (yellow line) and at the end (red line) of the ten year coastline evolution simulation for two specific areas. a) Central Rhyll and a portion of East Rhyll; b) frontage between East Rhyll and Prestatyn.**



Source: Mott MacDonald, 2021

**Figure 5.23: Position of the coastline at the start (yellow line) and at the end (red line) of the ten-year coastline evolution simulation at the fishtail groynes of Prestatyn. The coastline orientation after ten years is very similar to the orientation observed in the aerial image.** Please note that the small shift between the groynes and the coastline position is due to the model coastline grid points resolution (50m) and the way the model assigns the position of the groynes relative to the coastline grid points. The shifts is not affecting the model results.



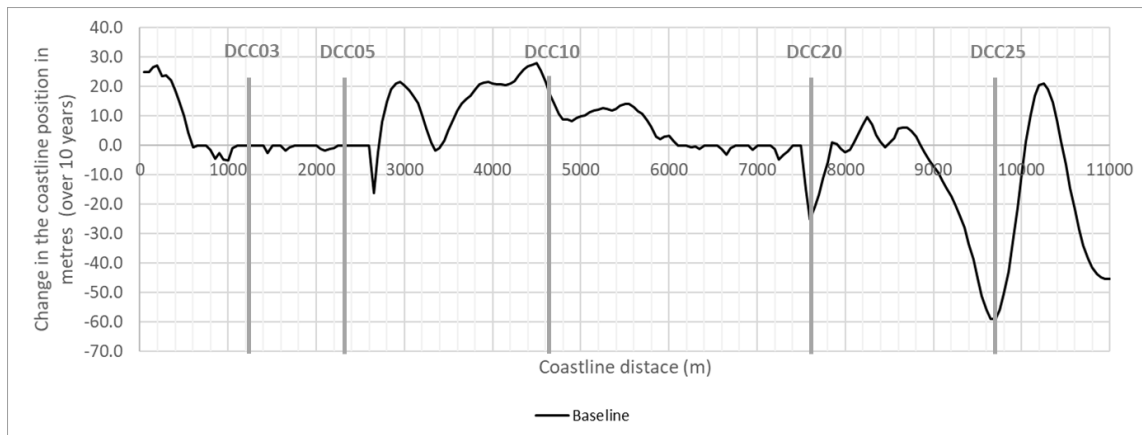
Source: Mott MacDonald, 2021

Figure 5.24 shows the accretion and retreat of the coastline (in metres) over the ten-year simulation, compared to its initial starting point. This figure shows that the shoreline position in the western portion of Central Rhyl, located in the shadow area of the training wall, advanced seaward by up to 27m in ten years (an average of 2.7m per year). No significant changes are observed to Central Rhyl and Splash Point, and no accretion is predicted between the wooden groynes, probably due to their poor condition. A relatively minor retreat of the coastal position between groynes DCC03 and DCC05 is observed in Figure 5.24.

The sheltering effect of Splash Point and the groynes located at East Rhyl, cause the coastline to advance up to 20 m in ten years (an average of 2m per year). This accretion is observed along the frontage up to the middle section of Prestatyn, around cross-shore profile DCC15 (coastline chainage 6000m, Figure 5.24). From this point eastwards, the frontage tends to remain stable, with some localised areas of beach retreat up to the revetment.

For the eastern section of the frontage, where no hard structures of any type are present, the coastline evolution module tends to evolve a straighter coastline, causing a large retreat and advance of the line (chainage 9000m to the end, Figure 5.24).

**Figure 5.24: Coastline accretion (positive) and retreat (negative) over ten years. The grey lines show the position of some of the cross-shore profiles relative to the coastline chainage.**



Source: Mott MacDonald, 2021

It is important to mention that the baseline simulation assumes that all the existing defence included in the model are maintained over the ten-year model simulation period, and there is no additional input of sediment into the system. Previous studies have reported the delivery of sediments to the frontage through the onshore migration of the ridge-runnel sandbar system. JBA (2018b) estimate average onshore sediment delivery between Rhyl and East Rhyl of around 20,000 m<sup>3</sup>/year. Since this sediment source is considered constant and coming from offshore, the proposed Central Rhyl Scheme will not modify it, and thus it has not been included in the present model.

## 5.5 Coastline evolution module – proposed Scheme

The Central Rhyl proposed Scheme will modify the coastal position to profile DCC05 and merge into the existing rock revetment at Splash Point. Therefore, the coastline of the model was modified to represent the advance of the new structures (see Section 5.2.3). Please note that, as per the existing conditions scenario, it is assumed that the MHWs tide level can be used to characterise the coastline position in the Scheme model. The Scheme was also included in the representative profiles used by the coastline evolution model. These were modified as per the LITDRIFT simulations of Section 5.2.3. All the other structures, groynes and the revetments from Splash Point to Prestatyn, were not modified, and the model assumed that these structures would be maintained over the ten-year simulation period.

### 5.5.1 Scheme results

The coastline evolution model for the Scheme scenario was run for ten years (using the same starting coastline as the Baseline simulation) to understand the potential long-term evolution of the study areas' coastline after the Scheme implementation. The simulation was undertaken to compare the baseline and the Scheme, thereby enabling assessment of the Scheme's potential impact along the frontage.

The ten-year simulation results are shown for the whole frontage in Figure 5.25-a, and for Central Rhyl in Figure 5.25-b. Both figures detail the position of the baseline and the Scheme coastlines after ten years. The results show that both predicted coastline positions are very similar. At the scale of the map of Figure 5.25-a, both baseline and Scheme coastline appear to have the same position after the ten-year simulation. However, when the results are analysed in further detail, small changes to the accretion areas can be noted.

**Figure 5.25: Position of the baseline coastline (red line) and the Scheme coastline (light blue line) after the ten-year coastline evolution simulation for: a) the entire study area, from Rhyl to the Point of Ayr; and b) Central Rhyl to Splash Point.**



Source: Mott MacDonald, 2021

The model results show that the Scheme is causing the western section of Central Rhyl, protected by the training wall, to accrete more than in the baseline case (Figure 5.25-a and Figure 5.26-b). In the ten year simulation, the maximum accretion attributable to the Scheme is 34m (3.4m/year, Figure 5.27). This around 5m more than the baseline case (i.e. 27m or 2.7m/year).

The central portion of the Rhyl frontage, where the new defence is proposed, shows no changes after the ten-year simulation. The new coastline does not accrete during the simulation, and its apparent advance, compared to the baseline (Figure 5.25-b), is only related to the advance of the new revetments. Once again, the coastline position is entirely controlled by the hard structure, as per the baseline case.

Minor changes to coastline accretion after Splash Point can be attributed to the Scheme (Figure 5.26-a, Figure 5.27 and Figure 5.28). The advance in the coastline position is small, and of the order of +1m compared to the baseline after the ten-year simulation.

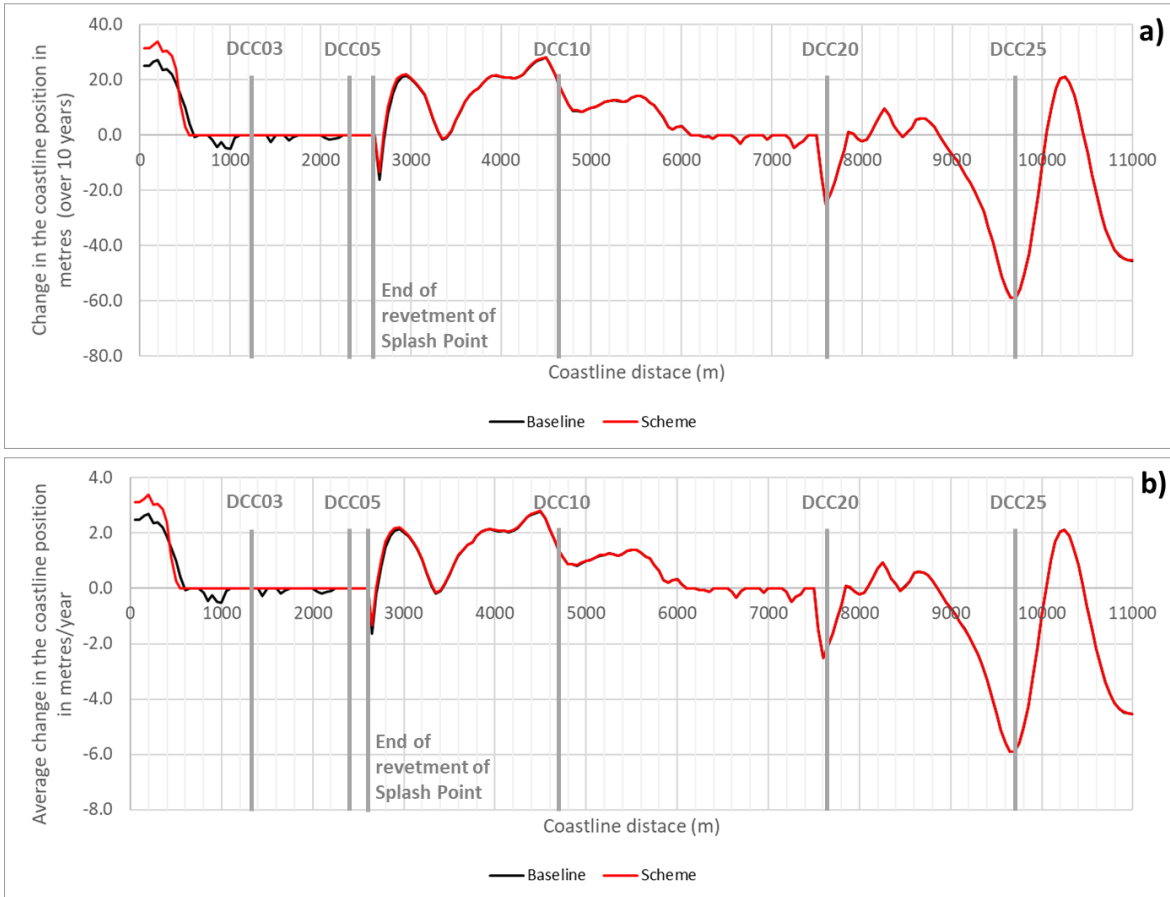
Figure 5.27 and Figure 5.28 also show how the coastline position with and without the Scheme are very similar and that after Splash Point, at East Rhyl (Figure 5.27 and Figure 5.28 – chainage 3000m), the Scheme does not have any detectable effect on the coastline position. The modelling, therefore, demonstrates that the impact of the Scheme is limited to the central portion of Rhyl. The results in Figure 5.28 also show that the effect on the coastline evolution at Central Rhyl is minor and mainly limited to a small increase in the sediment retention behind the training wall.

**Figure 5.26: Enlarged view of the position of the baseline coastline (red line) and the Scheme coastline (light blue line) after the 10-year coastline evolution simulation for: a) beach section after Splash Point; and b) the beach section behind the training wall at Central Rhyl.**



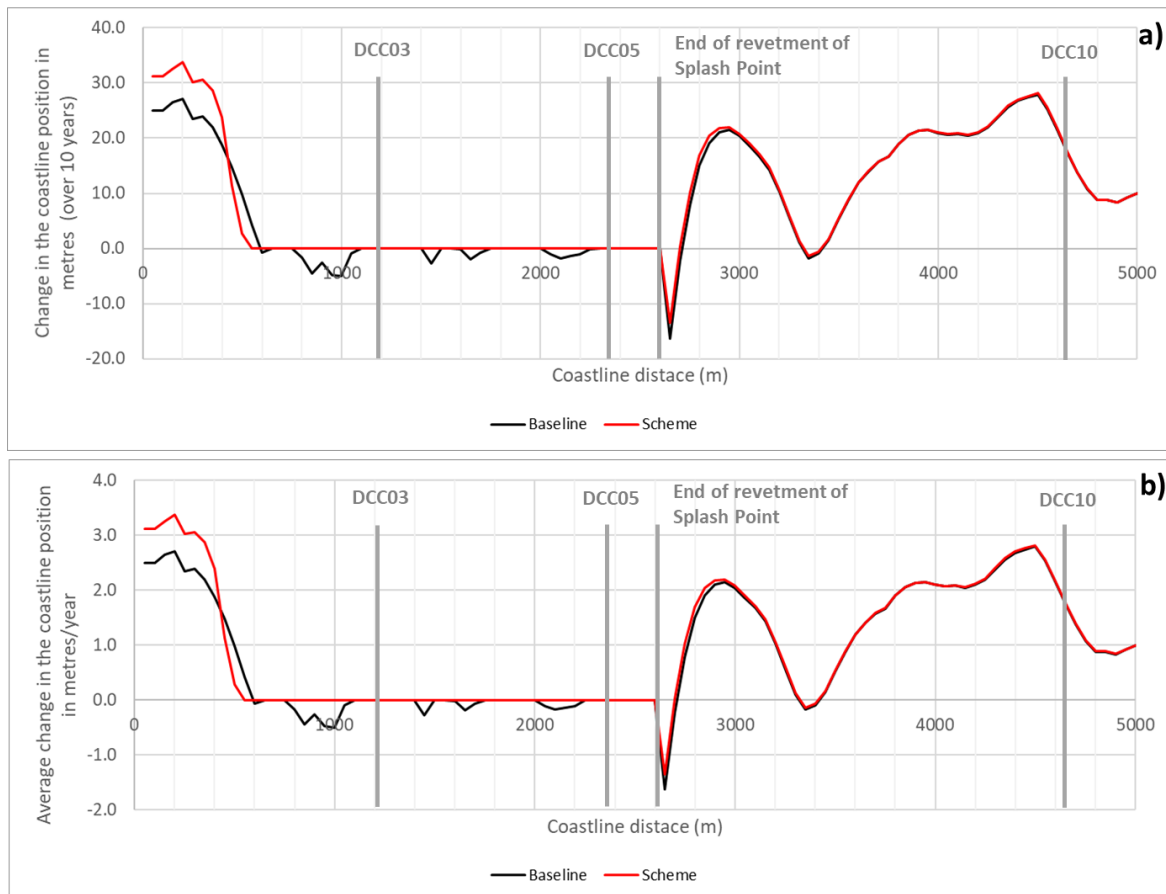
Source: Mott MacDonald, 2021

**Figure 5.27: a) Predicted coastline accretion (positive) and retreat (negative) over ten years; and b) annual average for the baseline conditions (black line) and with the Central Rhyll Scheme in place (red line). The grey lines show the position of some of the cross-shore profiles relative to the coastline chainage.**



Source: Mott MacDonald, 2021

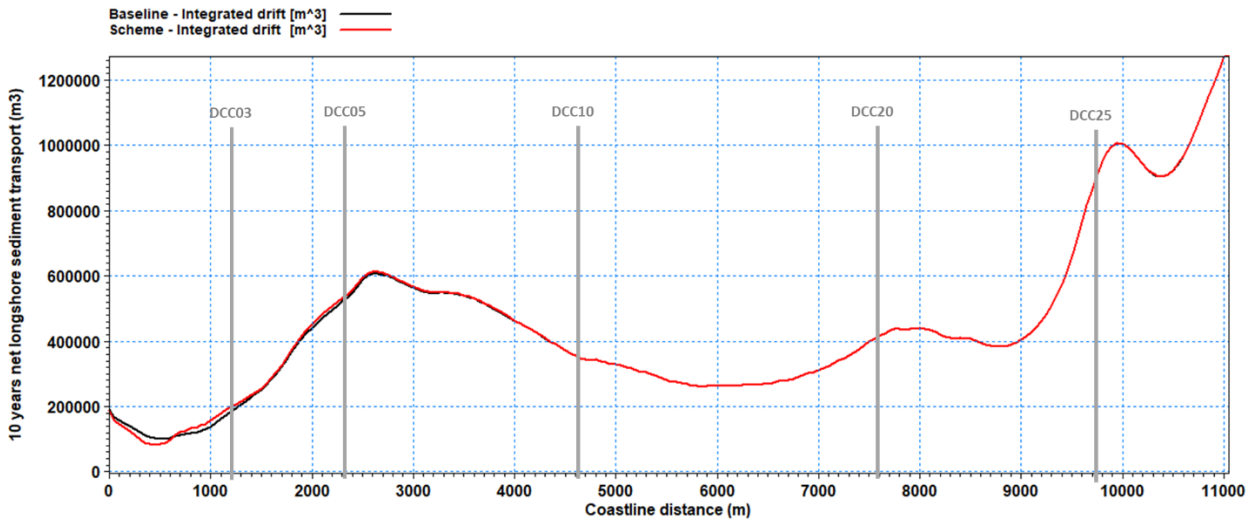
**Figure 5.28: a) Enlarged view of the predicted coastline accretion (positive) and retreat (negative) for Central Rhyll over ten years; and b) and the annual average for the baseline conditions (black line) and with the Central Rhyll Scheme in place (red line). The grey lines are indicating the position of some of the cross-shore profiles relative to the coastline chainage.**



Source: Mott MacDonald, 2021

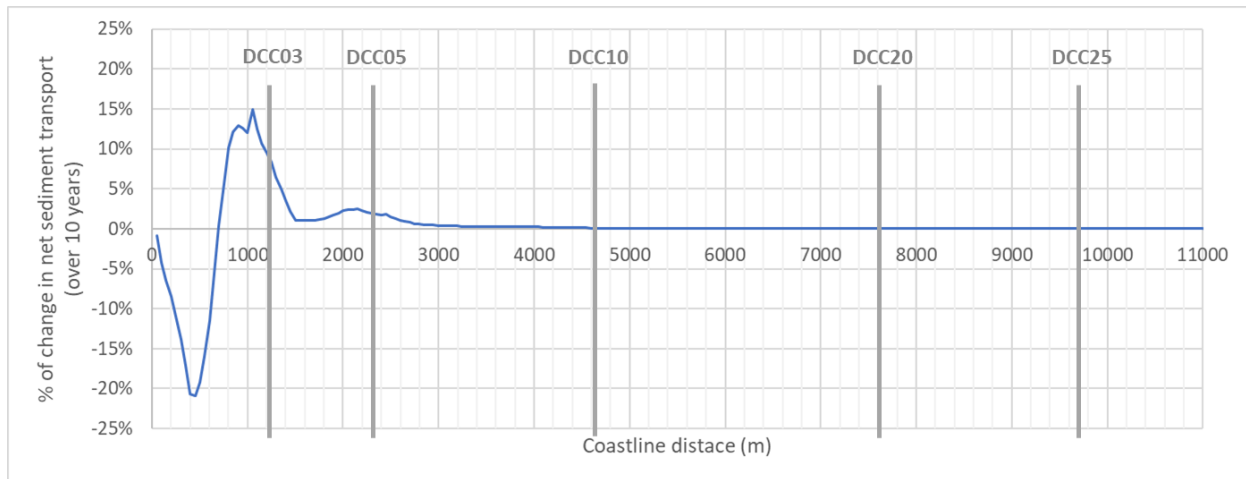
Differences in the net longshore transport over the ten-year simulation for the baseline and with the Central Rhyll Scheme are shown in Figure 5.29. In Figure 5.30, the changes to the net transport rates for the baseline and the Scheme are plotted as percentages. Positive and negative values indicate an increase and a decrease in the net transport rate due to the presence of the Scheme, respectively. Figure 5.30 shows a reduction of the transport rate up to 20% for the first section of the coastline, corresponding to the increased accretion previously described for this section of the frontage. The advance of the coastline due to the new Scheme has provided some additional protection to this area, decreasing the sediment transport rates and increasing the accretion of the coastline.

**Figure 5.29: Net longshore sediment transport ( $m^3$ ) in place (red line). The grey lines show the position of some of the cross-shore profiles relative to the coastline chainage. A positive transport rate indicates west to east sediment transport.**



Source: Mott MacDonald, 2021

**Figure 5.30: Percentage of change in the net sediment transport rate over ten years between the baseline and the proposed Scheme. A positive change indicates an increase in the net longshore sediment transport rates due to the presence of the Scheme. The grey lines show the position of some of the cross-shore profiles relative to the coastline chainage.**



Source: Mott MacDonald, 2021

For the central portion of the Scheme, Figure 5.30 shows an increase in the transport rates up to 15% around profile DCC03. These increases in the transport can be observed, but to a smaller extent (<3%), up to and after profile DCC05. The increase in the sediment longshore transport in the new revetment area is related to the model ability to move/shift the profile back during the simulation period. However, the coastline is fixed by the revetment structure and the combination of the fixed coastline and the rollback of the profiles are causing that the beach in front of the revetment to draw down during the simulation. As a consequence, the incident wave climate is able to mobilise and transport a larger volume of beach sediments.

After Splash Point at East Rhyl frontage (Figure 5.30 chainage >3000m), no changes to the net sediment transport rates are observed, and therefore the impacts caused by the Central Rhyl proposed Scheme are shown to be extremely limited to the development and the adjacent coastline only.

## 5.6 Summary

- Average annual longshore transport rates,  $Q_{LST}$ , computed using LITDRIFT for 26 representative beach profiles between Central Rhyl-Splash Point and the eastern portion of the frontage, close to the Point of Ayr in a ten-year simulation have been validated against other studies. The results show a net sediment transport direction from west to east, with values along the frontage varying in the range of 50,000 to 180,000 m<sup>3</sup>/year.
- As part of the model validation, the sediment budget for the frontage has been estimated from the predicted transport rates and compared with previous historical analyses. The budget indicates a general erosion trend for the central portion of Rhyl, accretion behind the training wall and erosion/accretion areas all the way to the Point of Ayr, mainly controlled by the orientation of the coastline.
- LITDRIFT was also run with the Scheme in place for profiles DCC01 to DCC05. The results showed that the Scheme has localised impacts that are confined to the upper part of the beach profile, close to the new defences. Further, this region of the beach only contributes to the alongshore sediment flux when the water level is at or above the toe of the new structure. This generally occurs for less than 10% of the time, and thus its effects are very limited.
- Over a simulation period of ten years, the coastline evolution has been modelled with and without the Scheme in place. Both simulations have very similar results, with changes to the position of the predicted coastline only occurring close to the proposed Scheme.
- The coastline evolution model shows that the Scheme impacts on coastline position are only barely noticeable up to the area to the east of Splash Point. There are no effects observed at Prestatyn or the Point of Ayr.
- The coastline evolution model was also used to predict the changes in  $Q_{LST}$ . The results obtained show an increase in  $Q_{LST}$  attributable to the Scheme. A maximum increase in  $Q_{LST}$  of 15% was observed around profile DCC03. This reduces to around 3% after profile DCC05, and it is imperceptible from profile DCC06.

## 6 Summary and Recommendations

A numerical modelling study has been undertaken to support the development of the Central Rhyl defence Scheme and provide the evidence required to assess the Scheme impacts on the wider coastal environment objectively. The study presented here includes an extensive literature and data review, historical analysis of the beach frontage, and cross-shore and coastline evolution modelling.

The modelling results demonstrate that the Scheme's effects on the cross-shore and alongshore transport of sediment along the coastline are confined to the Scheme location, where cross-shore impacts are limited to the upper beach and only present when the water level is at or above the toe of the proposed structure. Impacts to the longshore transport of sediment are imperceptible after 400m east of Splash Point. On the basis of this evidence, the impacts of the Scheme at the eastern end of the coastal cell at the Point of Ayr are therefore judged to be indiscernible.

The historical analysis undertaken in Chapter 3 concluded that:

- On the basis of the available data, the area in the eastern section of the frontage, from profile DCC05 towards Splash Point and potentially further east, exhibits an erosional trend.
- The LiDAR data shows that the eastern section of the frontage's erosion rate varies between -0.1m/year and -0.2m/year for the lower beach and up to -0.03m/year near to the toe of the defences.
- The western part of the frontage is accretional.
- The movement of the sandbars of the ridge-runnel beach system is responsible for the changes observed in the profiles. The onshore migration of the sandbars provided the main supply of sediment from offshore to the frontage.
- It is important to note that the foreshore is a continuously changing system, and both the LiDAR and the profile surveys are only providing a snapshot in time and are not capturing the beach's complete behaviour. Periods with lower beach levels could have occurred between surveys, and therefore, not have been recorded. Beach levels can change due to a storm and recover in the next couple of months, and the regular monitoring surveys may not capture these changes.

The beach profile modelling of Chapter 4 concluded that:

- Cross-shore 1D modelling has demonstrated that a simulated 1% AEP storm results in morphological changes in the beach profile that is less than typical measured seasonal and inter-annual variability;
- When modelling a future scenario (2120), the LITPACK profile response model results show a modest increase in beach drawdown of around 10cm;
- The maximum beach drawdown with the Scheme in place is similar to the baseline conditions (~40cm), with exception of profile DCC02, which resulted in a larger beach drawdown for present and future conditions of almost 1m;
- The results indicate that the natural variability in beach morphology in front of the revetment results from the seasonal and inter-annual cross-shore migration of the sandbars, together with the longshore transport generated by tidal and wave-induced currents rather than a cross-shore response to storm events *per se*.
- While results were validated with independent modelling software and results obtained are similar, they should be used with caution in any structural design exercise, and confidence levels should be added to the beach drawdown values here reported.

The beach profile modelling of Chapter 5 concluded that:

- The results show net sediment transport from west to east, with values varying from 50,000 to 180,000 m<sup>3</sup>/year with larger transport rates around Central Rhyl-Splash Point and the eastern portion of the frontage, close to the Point of Ayr. The values agree with other sediment transport studies.
- The sediment budget obtained from the longshore rates indicates erosion for the central portion of Rhyl, accretion behind the training wall and erosion/accretion mainly controlled by the orientation of the coastline to the Point of Ayr.
- The coastline evolution model results for a simulation period of ten years are similar with and without the Scheme included in the model. Changes to the position of the predicted coastline are only noted close to the proposed Scheme. No effect of the Scheme is detected at Prestatyn and Point of Ayr.
- The results show no beach accretion in front of the new Scheme. However, an increase in the accretion rates is noted in the area behind the training wall and, to a lesser extent, just after the revetment of Splash Point.
- The coastline evolution model was also used to predict the changes in the net longshore transport rates. The results obtained show an increase in  $Q_{LST}$  due to the Scheme. The maximum increase of 15% is noted around profile DCC03 and is reduced after this point to <3% around profile DCC05. Changes east of profile DCC06 are imperceptible.
- The areas of increased alongshore transport may provide the source material for increased accretion at other locations and are probably related to the minor beach drawdown in the area of the new revetment.
- Results from LITDRIFT and the shoreline evolution model concur and demonstrate that the impacts of the proposed Scheme at Central Rhyl on coastal sediment transport process and shoreline position are slight and are limited to the vicinity of the Scheme both in the longshore and cross-shore direction.

From the results of the study, it is recommended to:

- Undertake further beach monitoring is required. This is now considered to be urgent since the June 2020 survey showed that the frontage was close to its historical minimum beach levels.
- Include more recent survey data in the present historical analyses to determine if the beach levels have recovered after the summer of 2020.
- Treat with caution the beach drawdown estimates obtained during the modelling of the profile evolution if the data are being used in the structure's design. Allowances for uncertainties must be included.
- Continue beach monitoring after construction to ensure the maximum beach drawdown is not exceeded and to provide an early warning should beach levels fall more than expected. The monitoring will also allow an independent assessment of any as-built Scheme impacts not detected by the modelling work.

The models presented in this report are based on the best available information for the site at this stage. The modelling results and analyses are therefore dependent not only on the model limitations and assumptions but also on the quality and accuracy of this data. When using results from this study, it is essential to interpret them carefully and make allowances for uncertainties in the model predictions.



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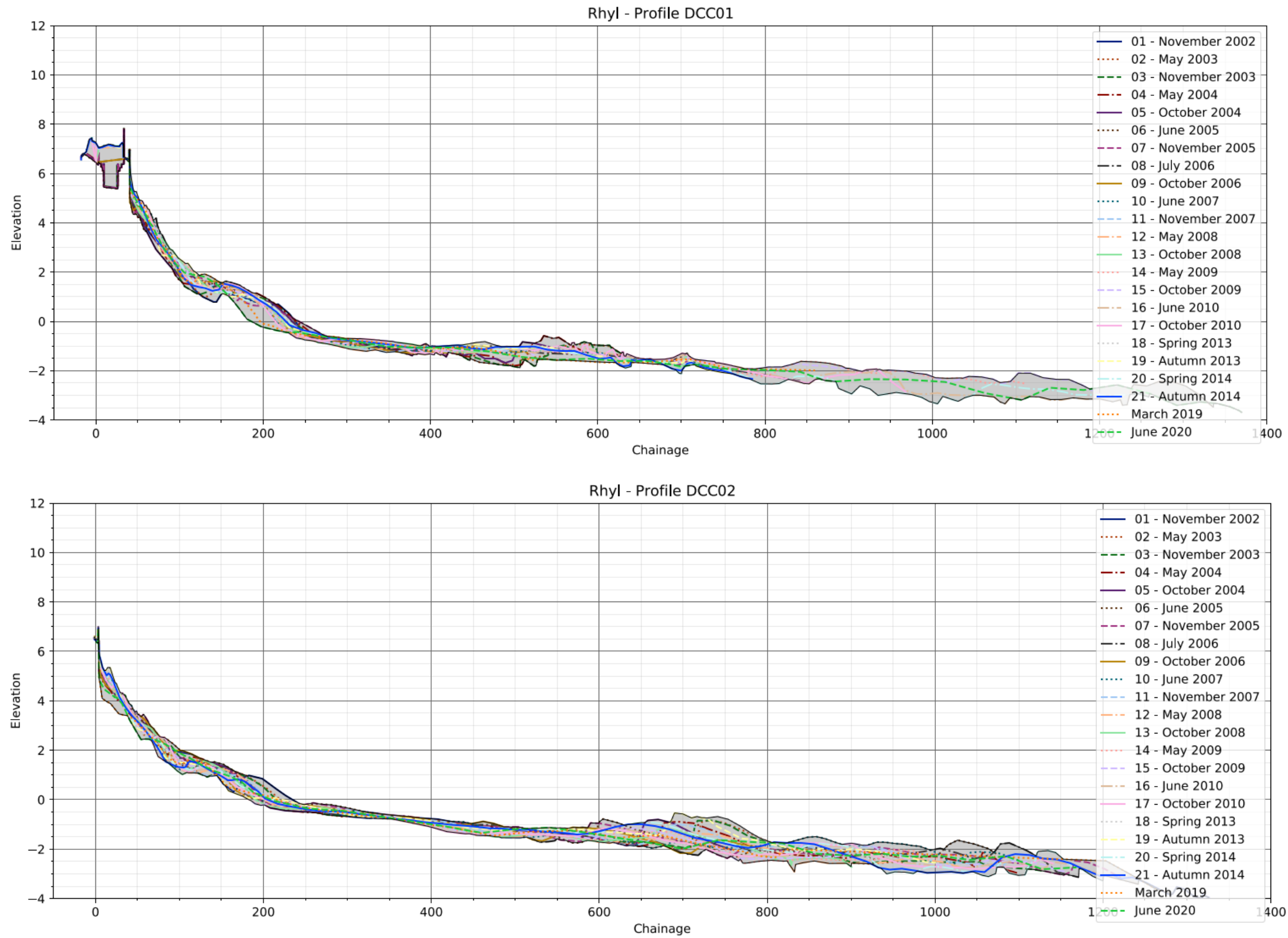
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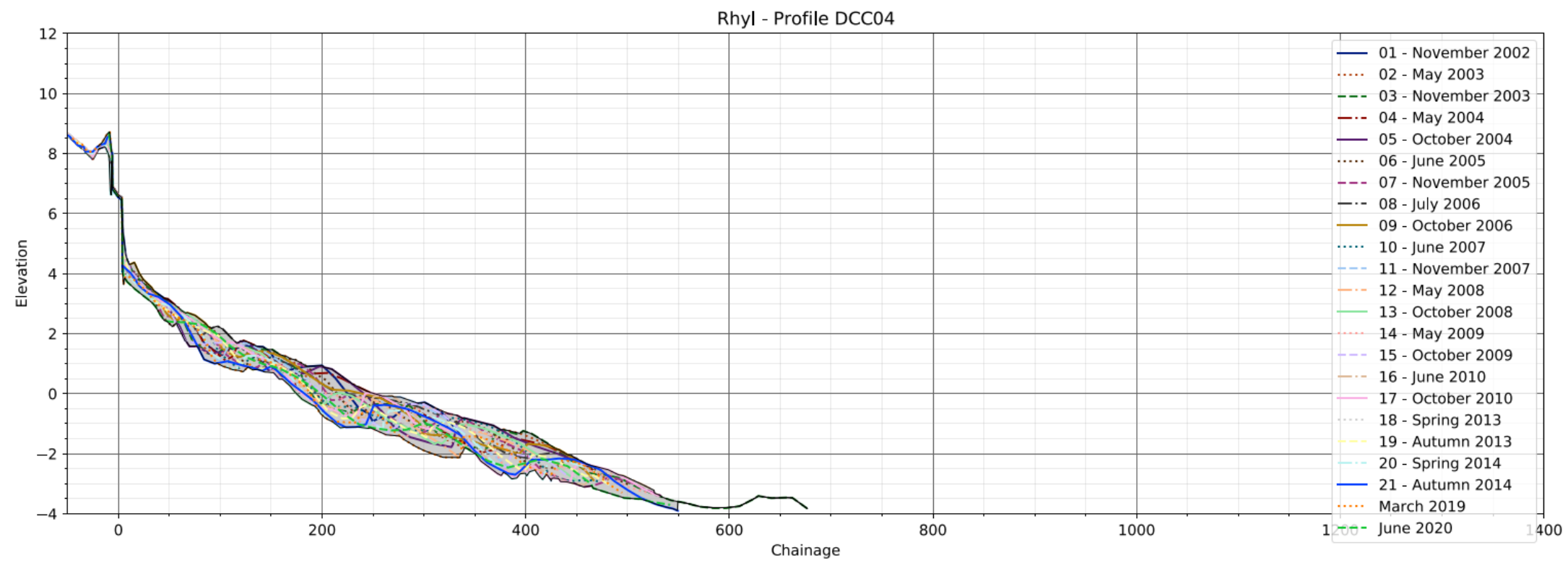
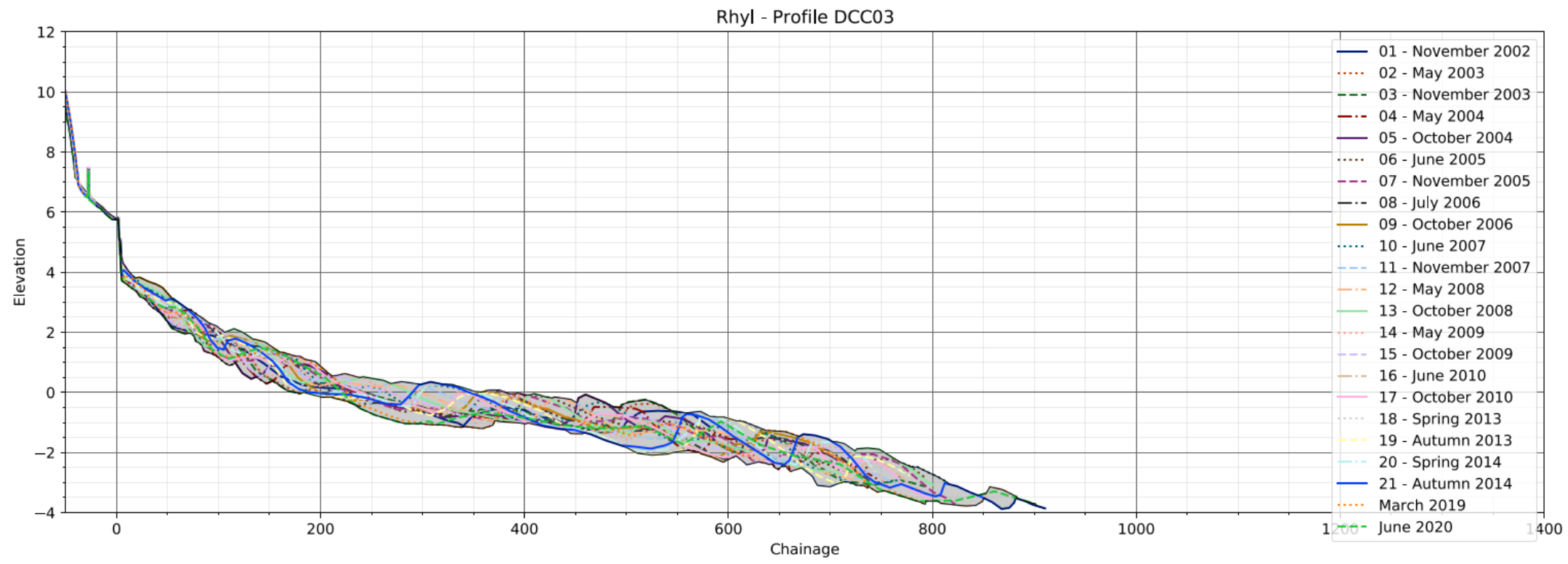
# Appendices

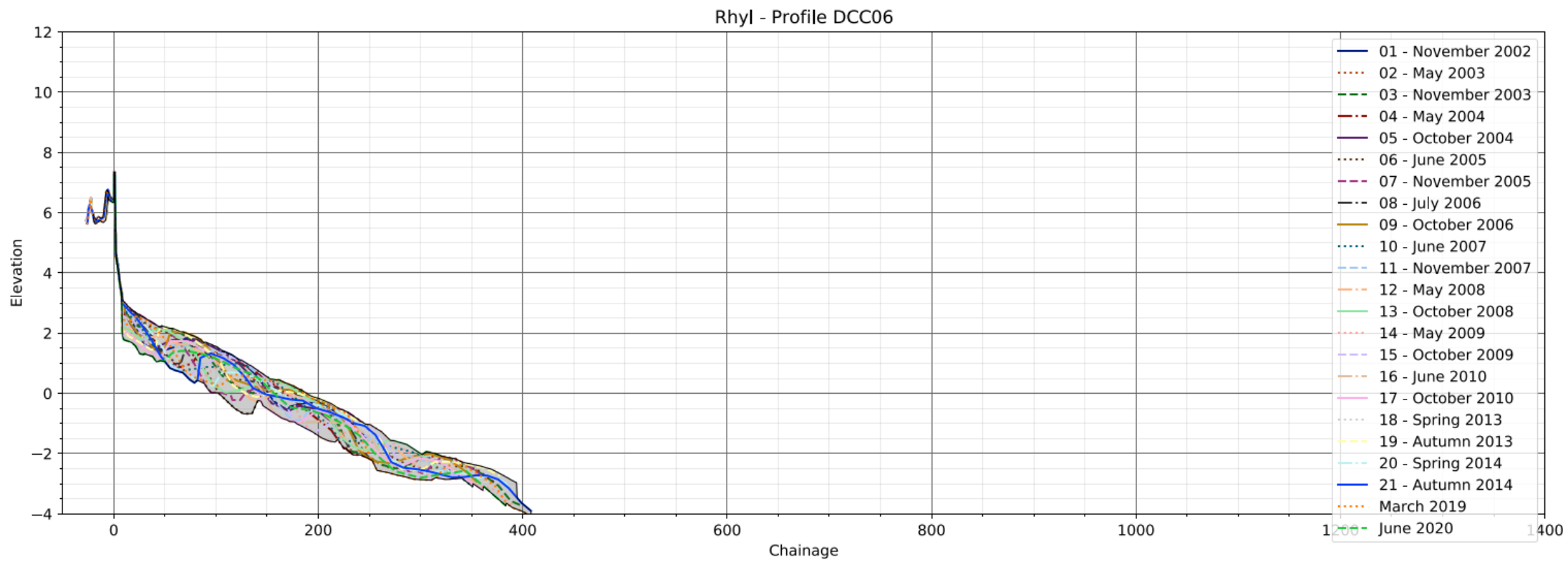
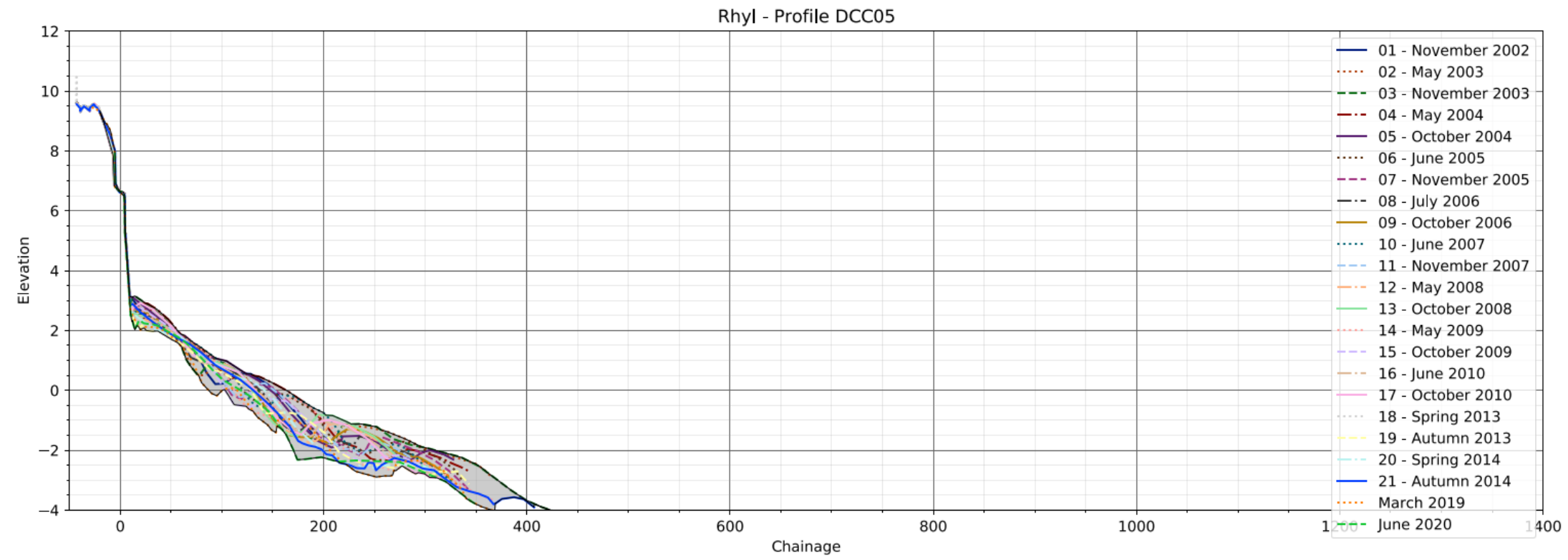
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## A. Beach profile change envelopes

Figure A.1: All beach surveys (2002 to 2020) for profiles DCC01 to DCC06.

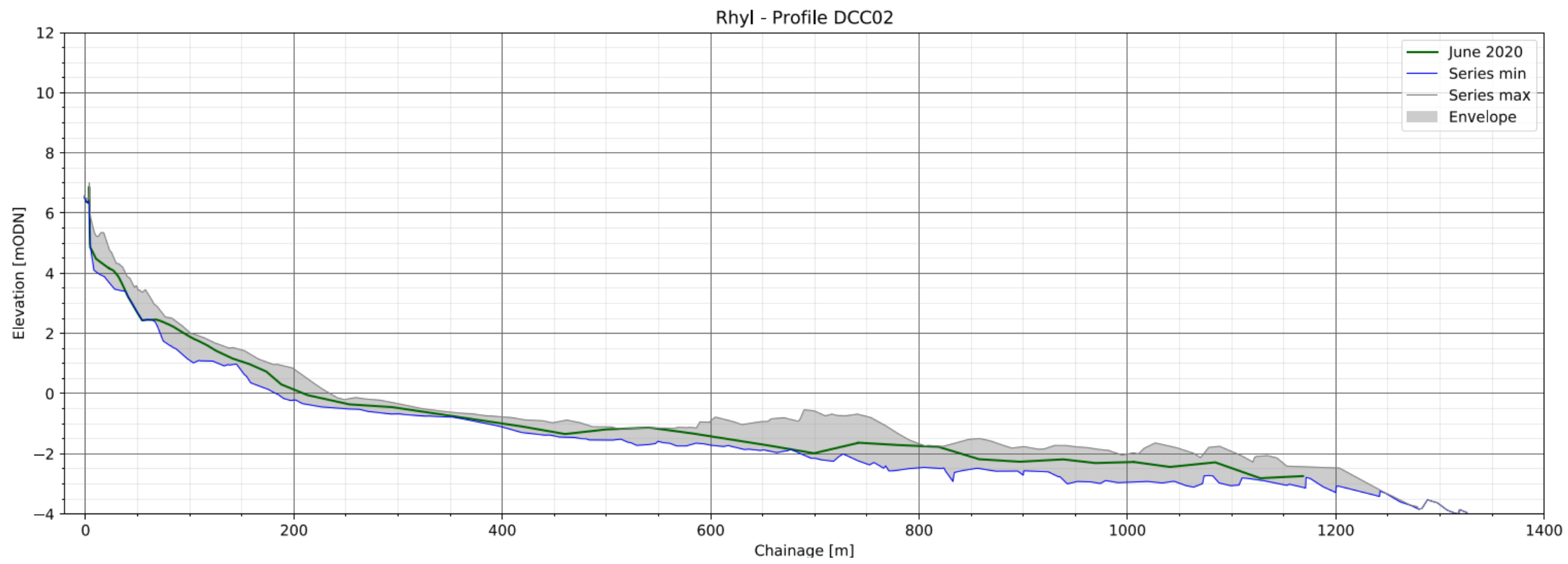
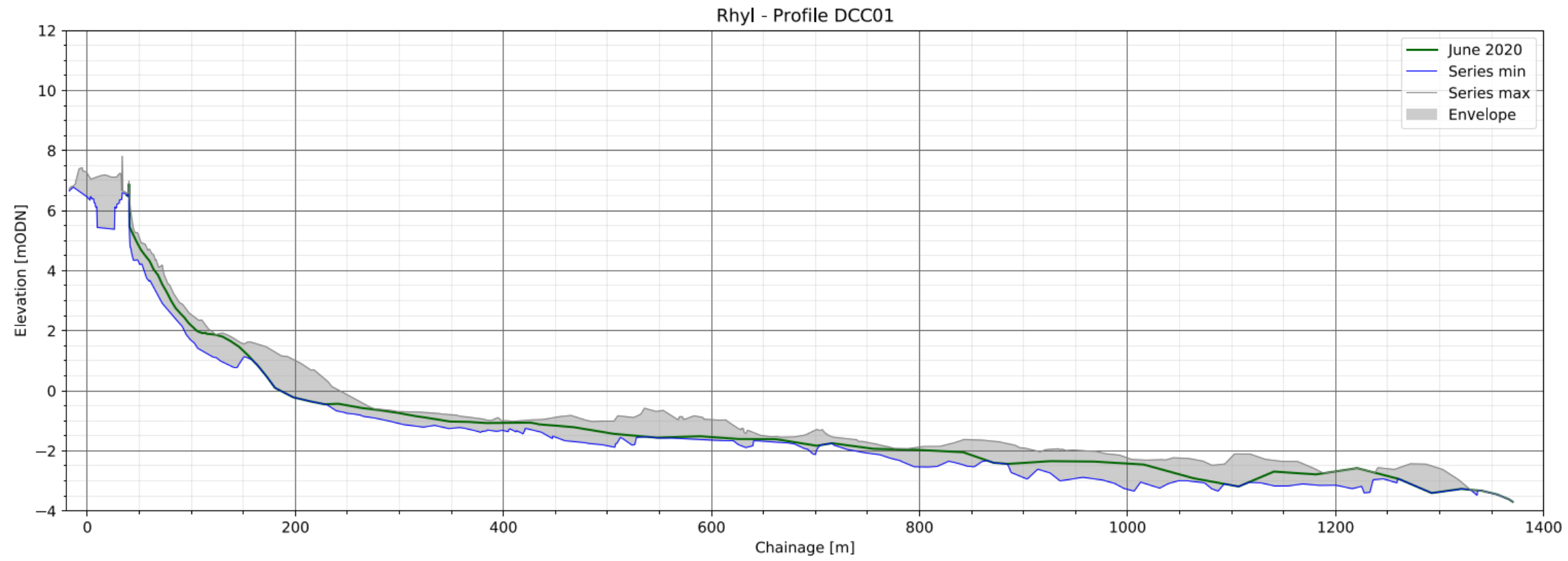


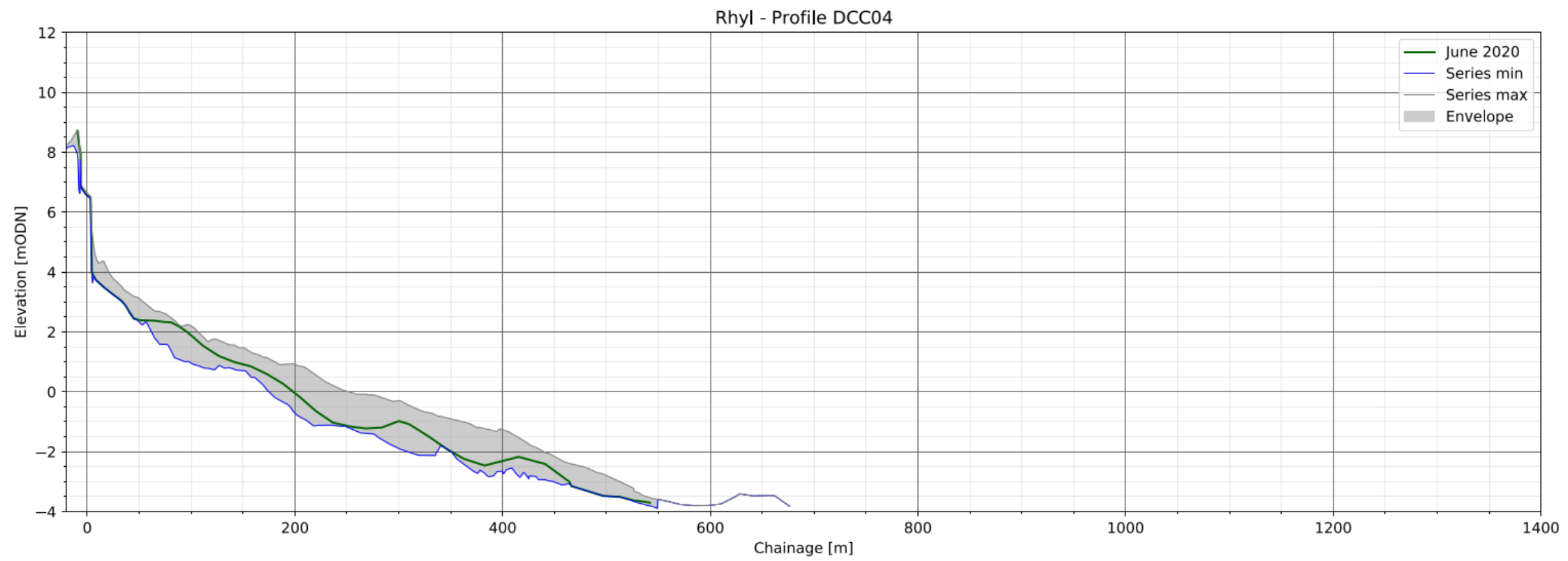
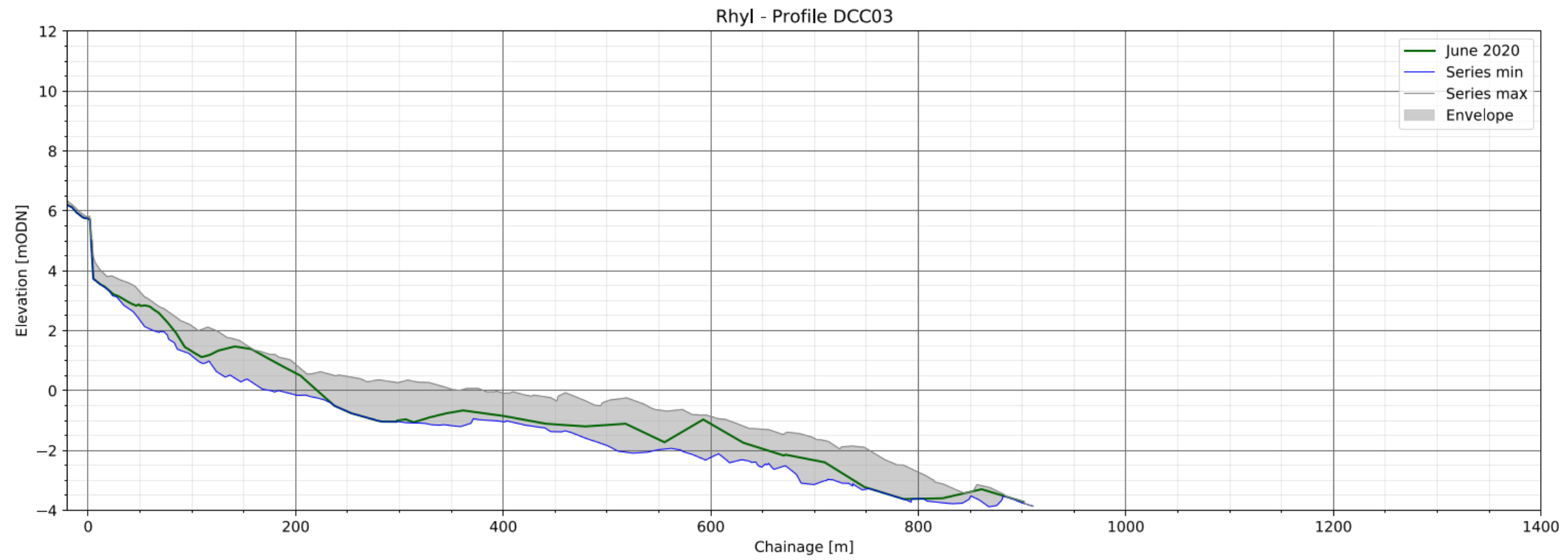


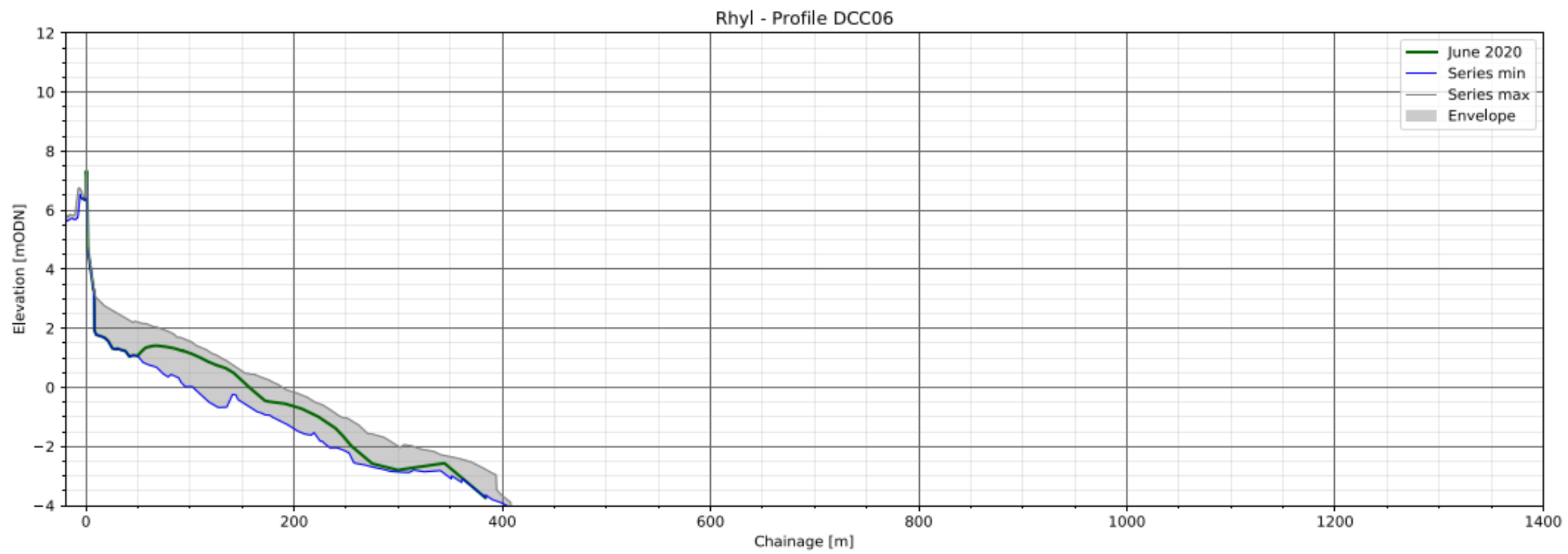
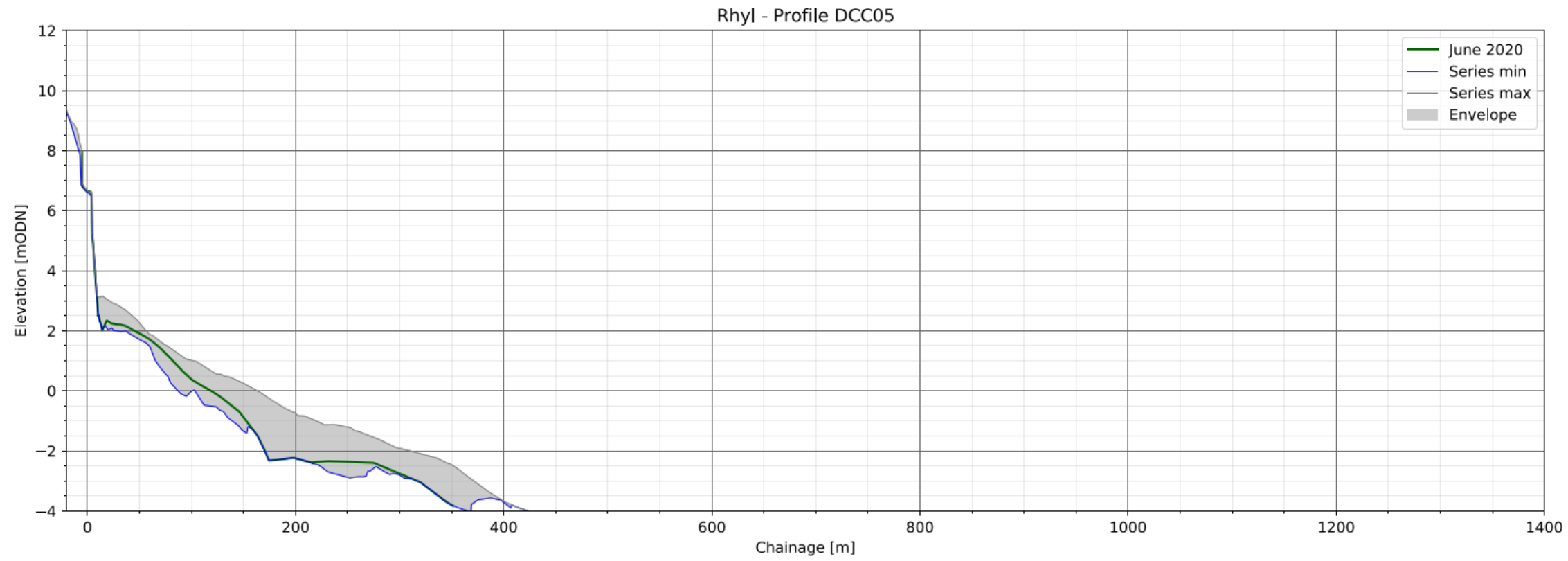


Source: Mott MacDonald, 2021. Includes Denbighshire County Council and CCO data.

Figure A.2: June 2020 survey, plotted with the surveyed envelope, profiles DCC01 to DCC05.



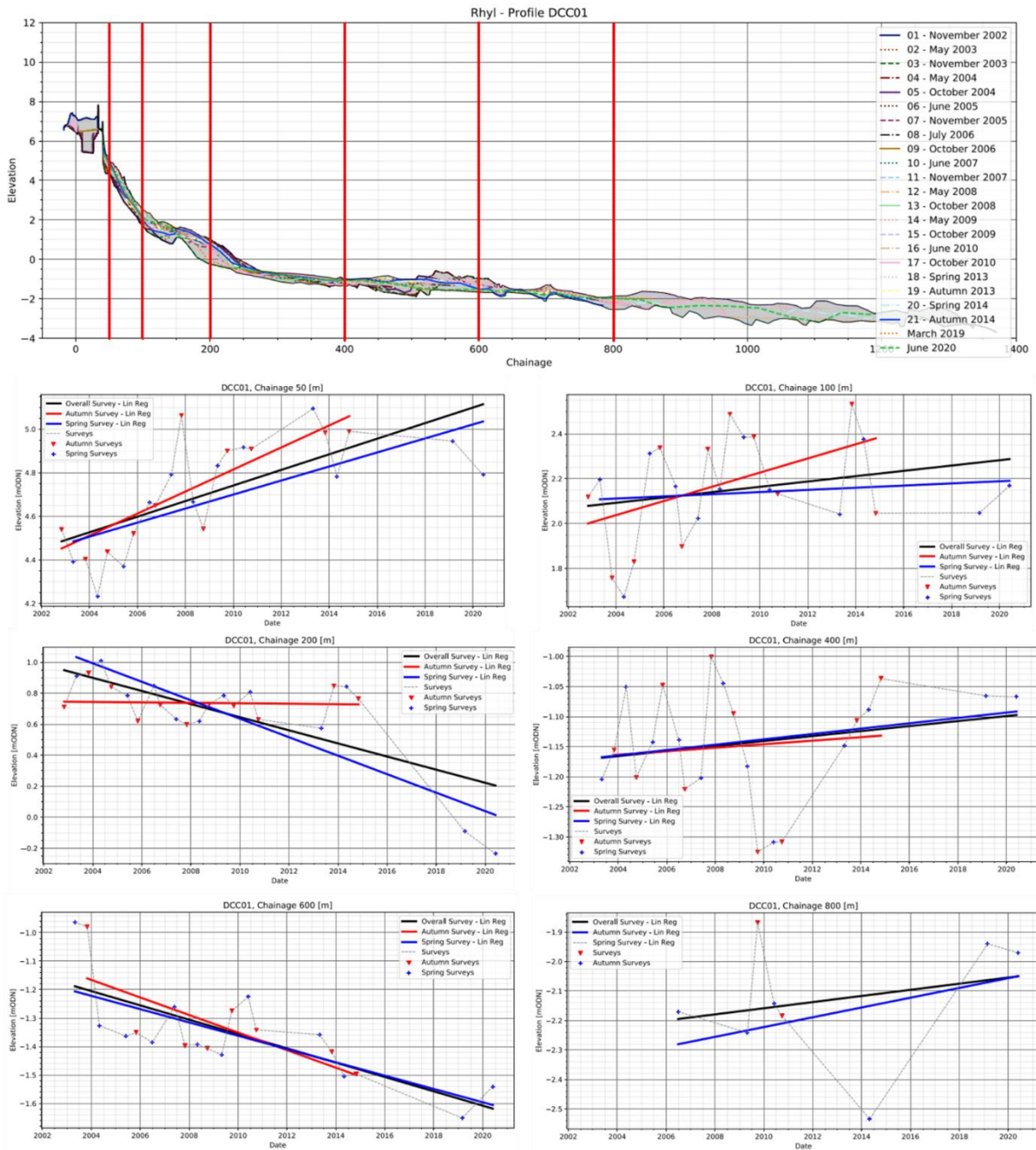




Source: Mott MacDonald, 2021. Includes Denbighshire County Council and CCO data.

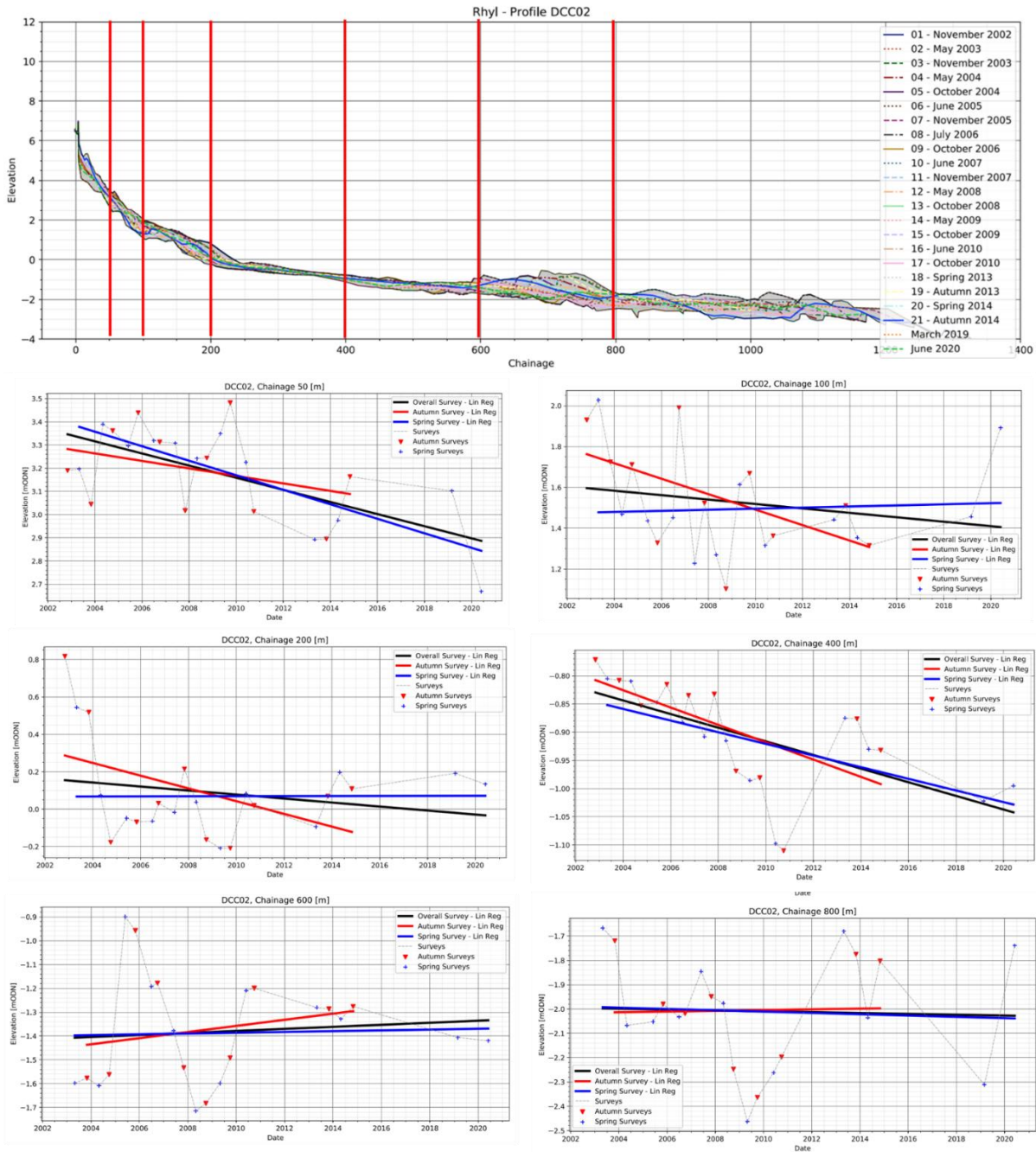
## B. Beach profiles: chainage analysis

**Figure B.3: Profile DCC01 chainage analysis.** The surveys are plotted according to the season (summer/autumn in red and winter/spring in blue). A regression line has been fitted through all the data according to the season. The location on the chainages and the profile are indicated by the red lines in the profile plot.



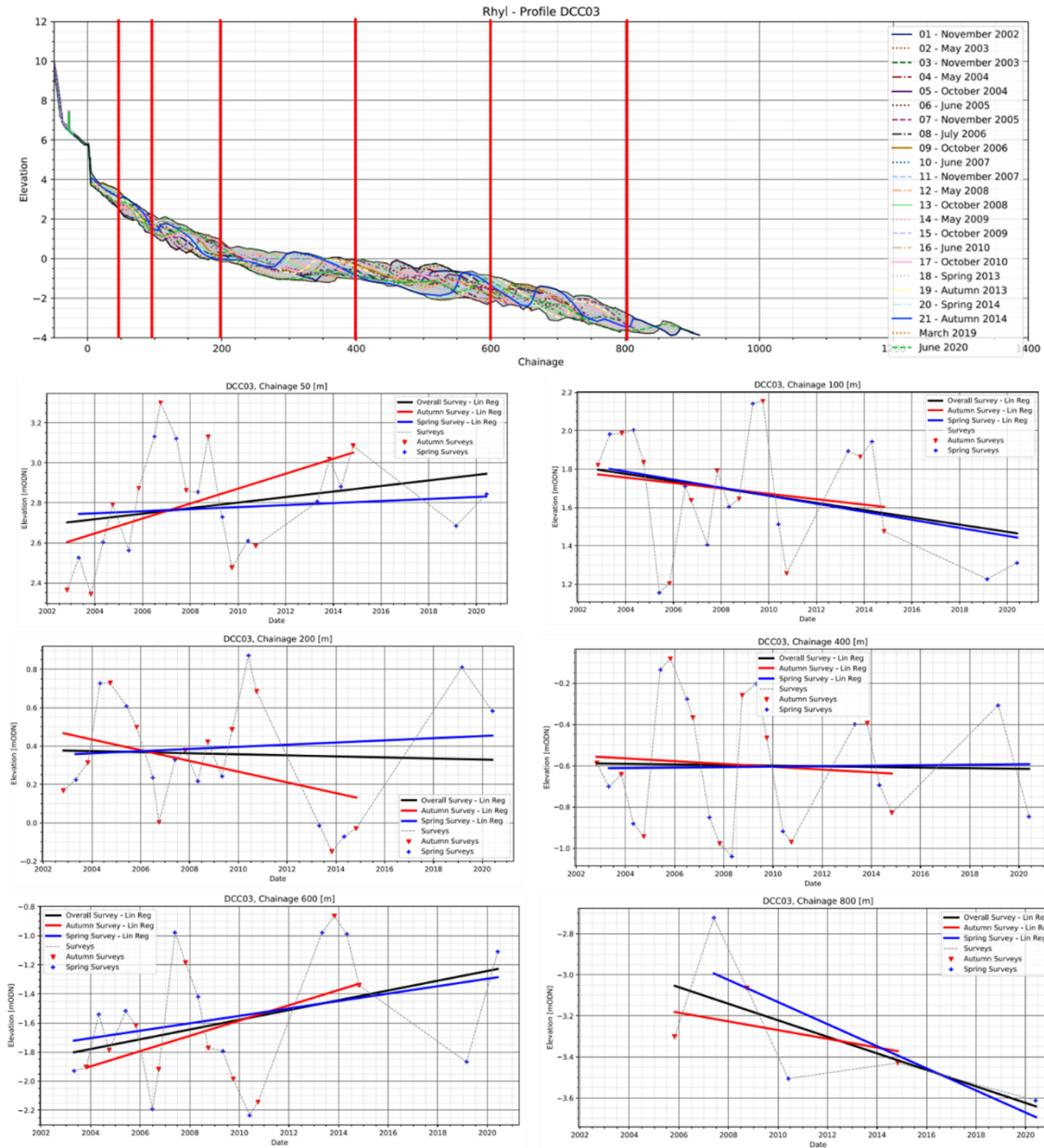
Source: Mott MacDonald, 2021. Includes Denbighshire County Council and CCO data.

**Figure B.4: Profile DCC02 chainage analysis. The surveys are plotted according to the season (summer/autumn in red and winter/spring in blue). A regression line has been fitted through all the data according to the season. The location on the chainages and the profile are indicated by the red lines in the profile plot.**



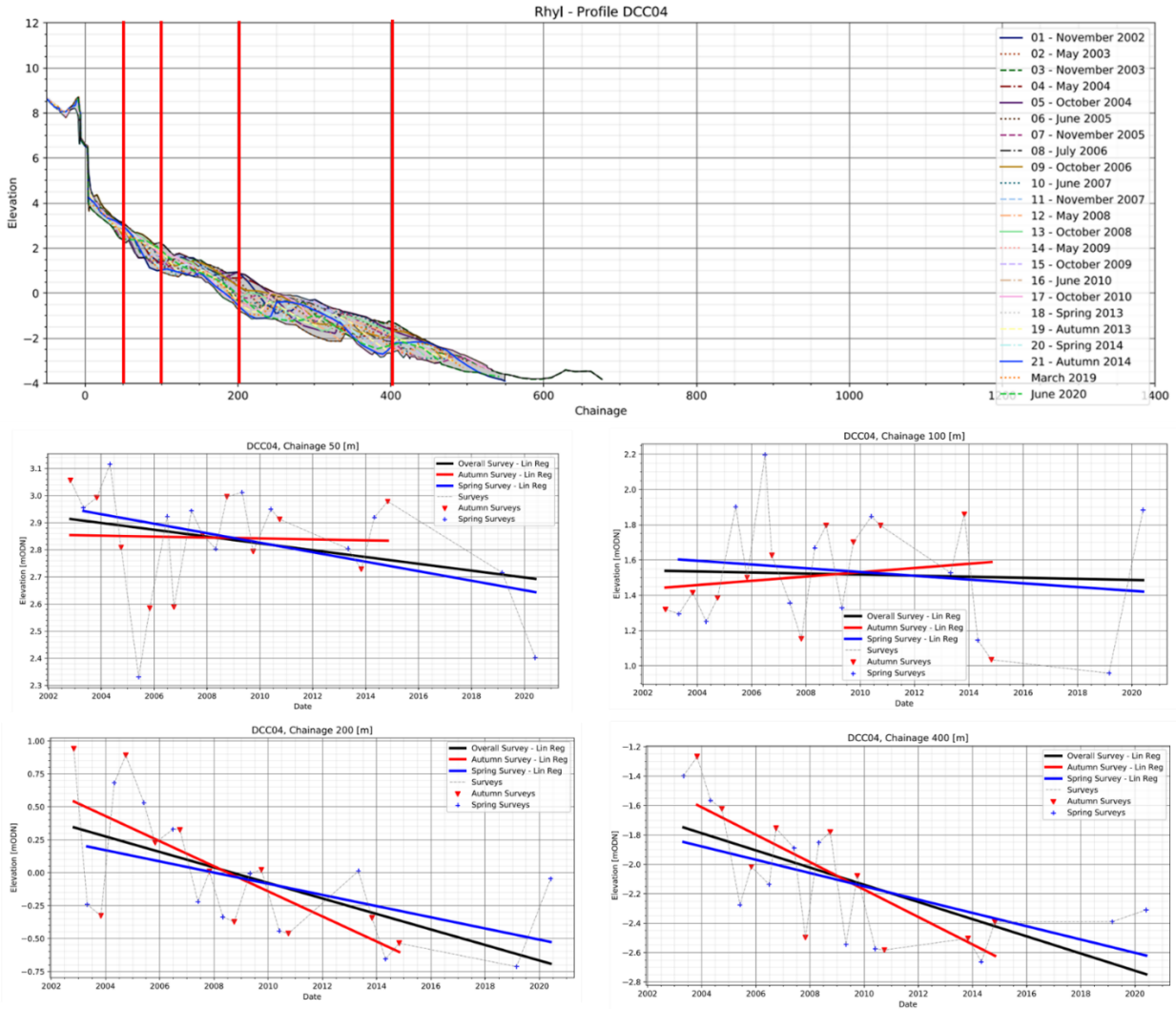
Source: Mott MacDonald, 2021. Includes Denbighshire County Council and CCO data.

**Figure B.5: Profile DCC03 chainage analysis. The surveys are plotted according to the season (summer/autumn in red and winter/spring in blue). A regression line has been fitted through all the data according to the season. The location on the chainages and the profile are indicated by the red lines in the profile plot.**



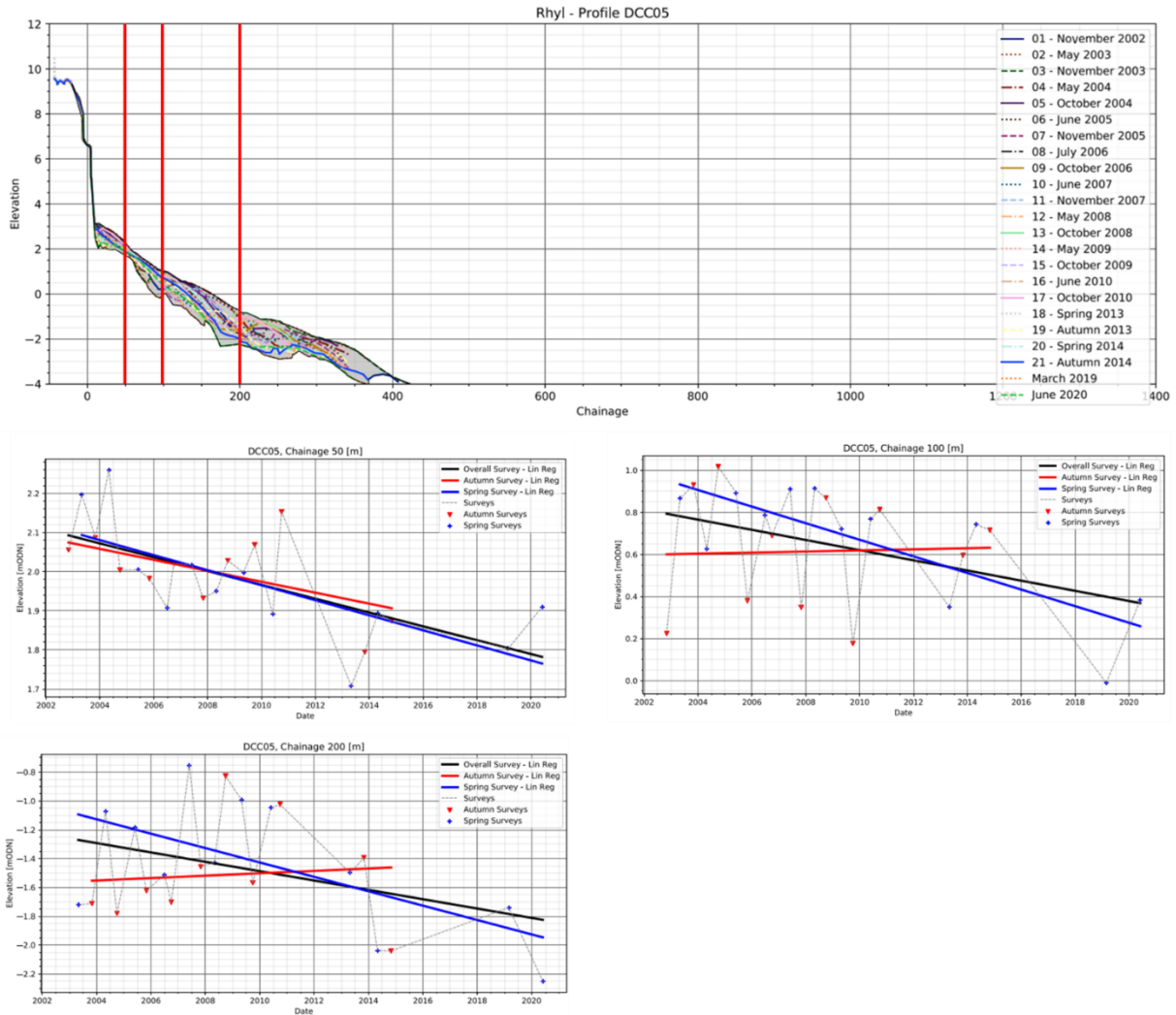
Source: Mott MacDonald, 2021. Includes Denbighshire County Council and CCO data.

**Figure B.6: Profile DCC04 chainage analysis. The surveys are plotted according to the season (summer/autumn in red and winter/spring in blue). A regression line has been fitted through all the data according to the season. The location on the chainages and the profile are indicated by the red lines in the profile plot.**



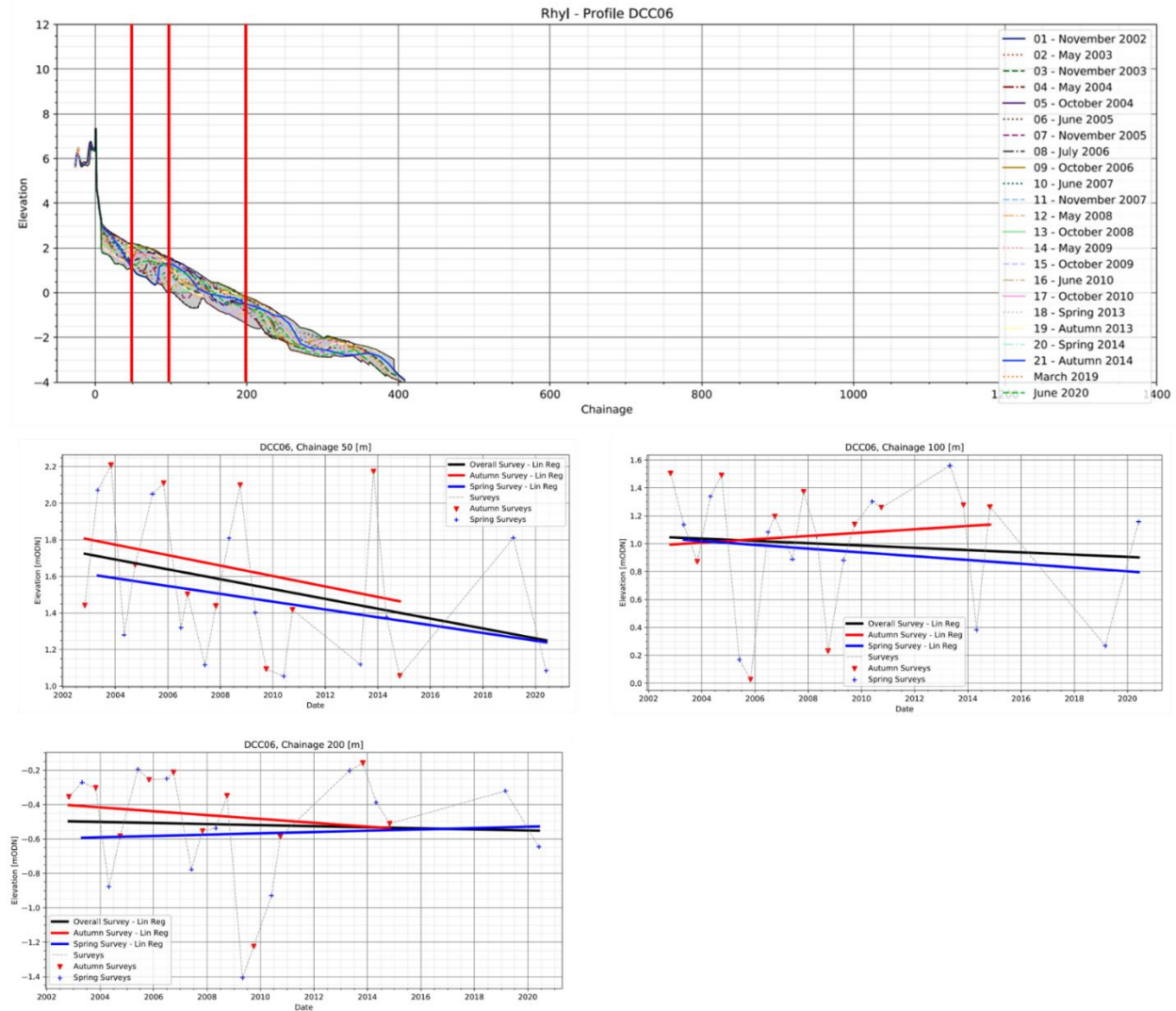
Source: Mott MacDonald, 2021. Includes Denbighshire County Council and CCO data.

**Figure B.7: Profile DCC05 chainage analysis. The surveys are plotted according to the season (summer/autumn in red and winter/spring in blue). A regression line has been fitted through all the data according to the season. The location on the chainages and the profile are indicated by the red lines in the profile plot.**



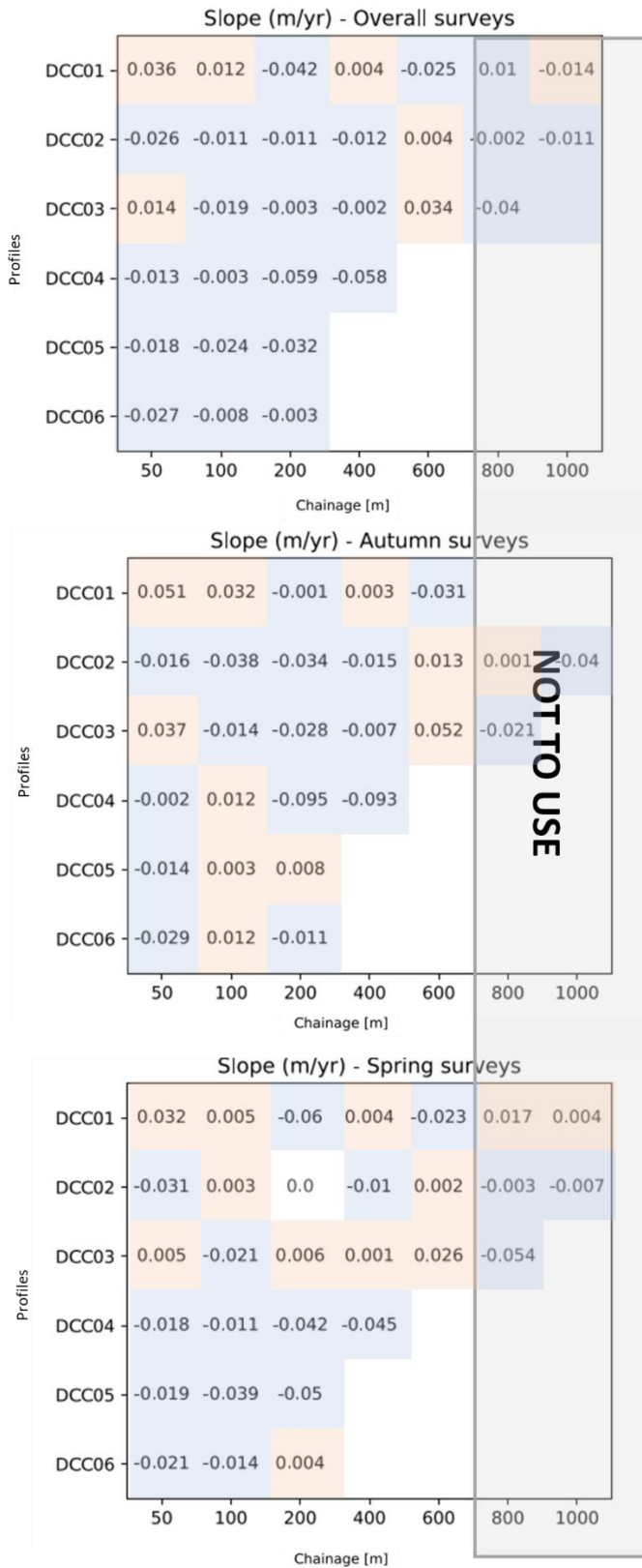
Source: Mott MacDonald, 2021. Includes Denbighshire County Council and CCO data.

**Figure B.8: Profile DCC06 chainage analysis. The surveys are plotted according to the season (summer/autumn in red and winter/spring in blue). A regression line has been fitted through all the data according to the season. The location on the chainages and the profile are indicated by the red lines in the profile plot.**



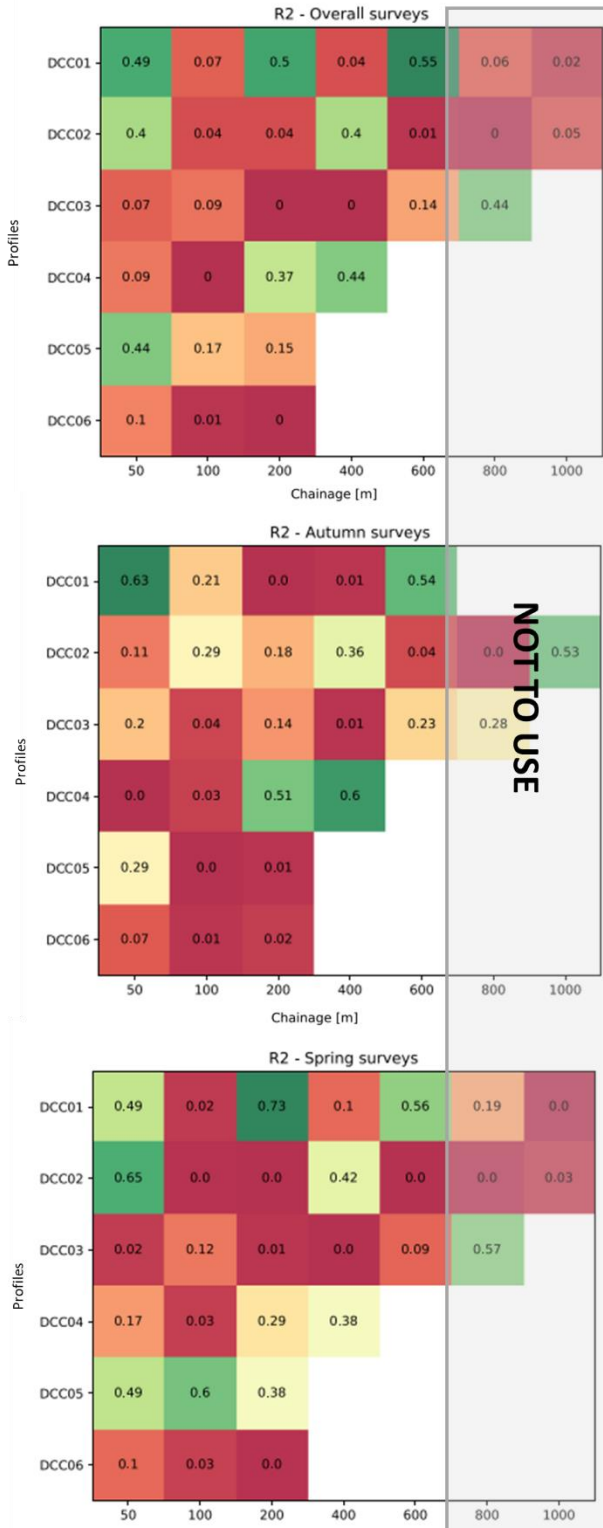
Source: Mott MacDonald, 2021. Includes Denbighshire County Council and CCO data.

**Figure B.9: Linear regression slope values in metres per year from the chainage analysis (overall, summer/autumn and winter/spring trend). Please note that the values derived above chainage 600m should not be used.**



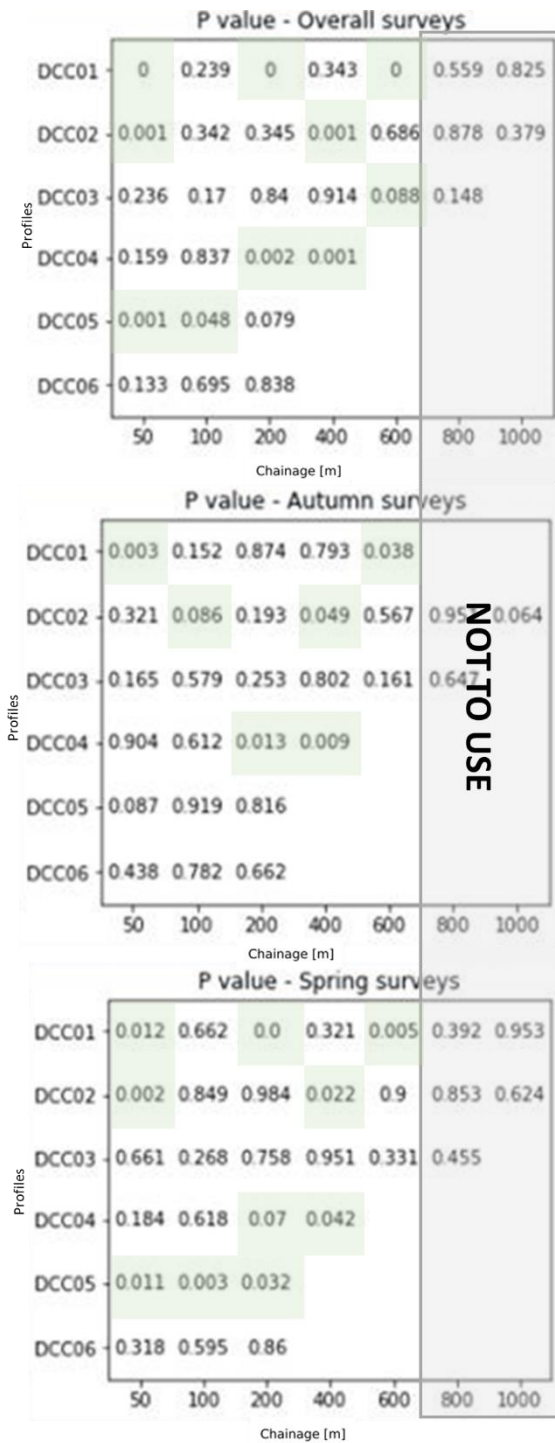
Source: Mott MacDonald, 2021

**Figure B.10: Product moment correlation coefficient ( $R^2$ ) value for the stated chainage location and beach profile. Please note that the values derived chainage greater than 600m should not be used.**



Source: Mott MacDonald, 2021

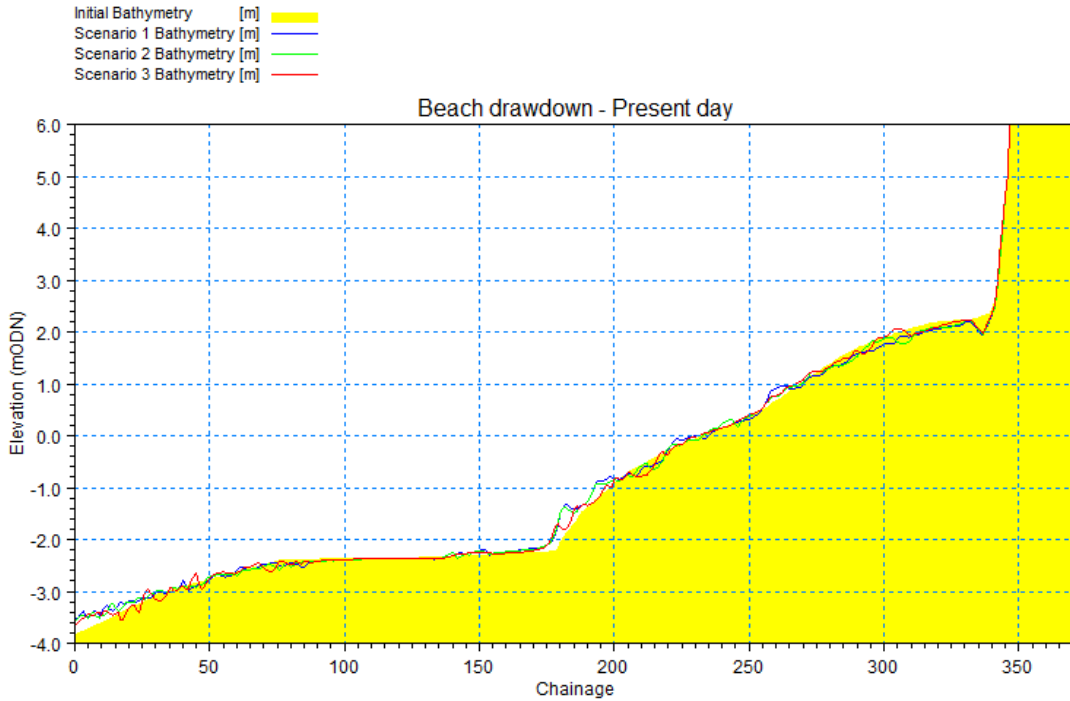
**Figure B.11: P-values for the stated chainage location and beach profile. Please note that the values derived chainage greater than 600m should not be used**



Source: Mott MacDonald, 2021

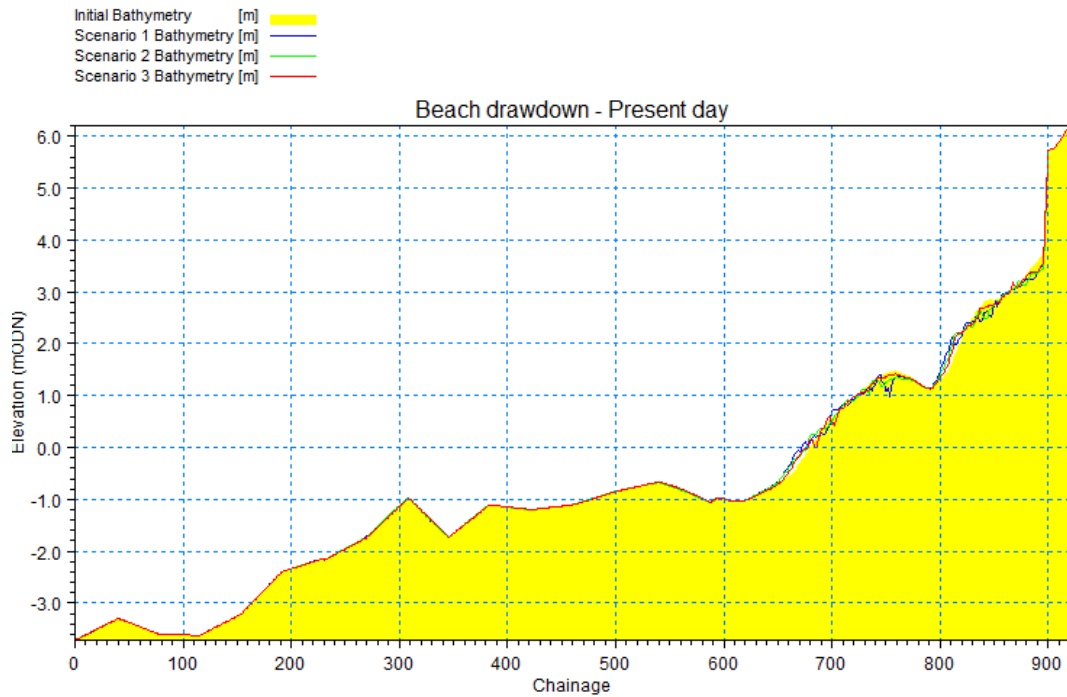
## C. Profile Evolution Scenarios

**Figure C.12: Pre- and post-storm DCC05 beach profiles predicted by the LITPACK profile evolution model for the present-day scenarios (Table 4.3).**



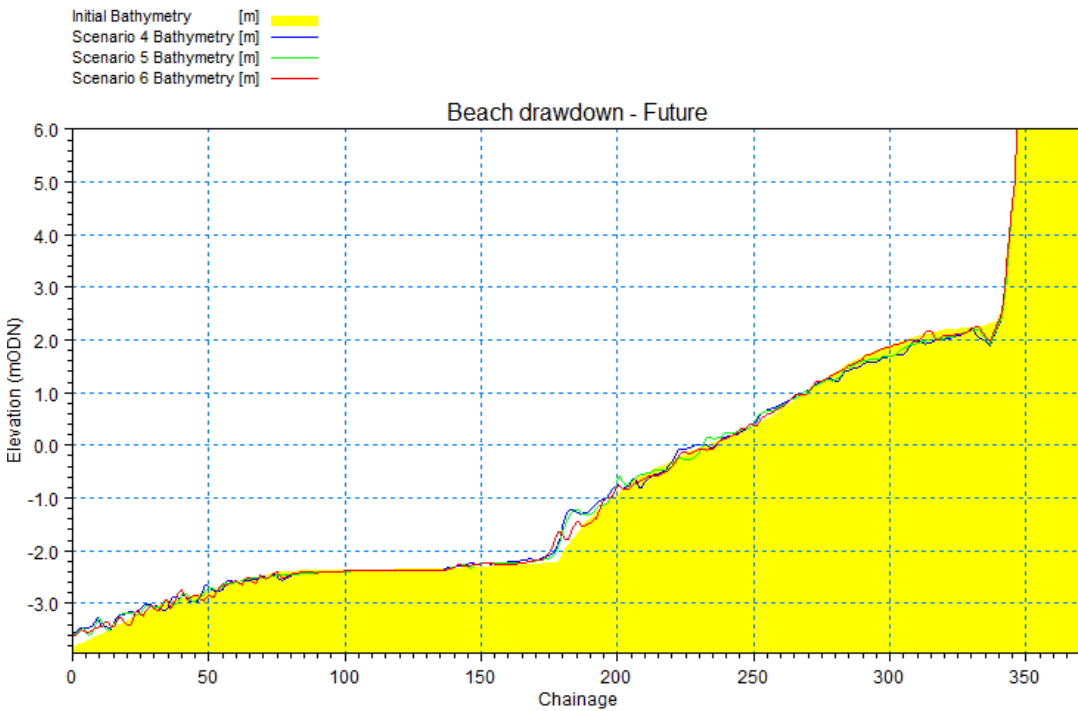
Source: Mott MacDonald, 2021. Includes CCO data

**Figure C.13: Pre- and post-storm DCC03 beach profiles predicted by the LITPACK profile evolution model for the present-day scenarios (Table 4.3).**



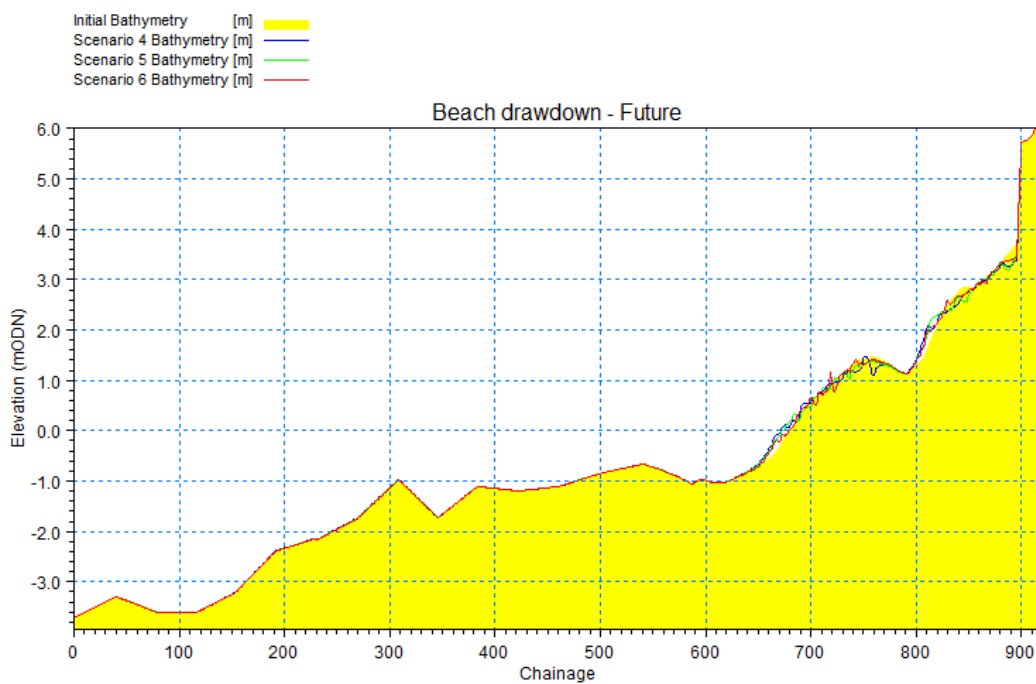
Source: Mott MacDonald, 2021. Includes CCO data

**Figure C.14: Pre- and post-storm DCC05 beach profiles predicted by the LITPACK profile evolution model for the climate change scenarios (Table 4.3).**



Source: Mott MacDonald, 2021. Includes CCO data

**Figure C.15: Pre- and post-storm DCC03 beach profiles predicted by the LITPACK profile evolution model for the climate change scenarios (Table 4.3).**



Source: Mott MacDonald, 2021. Includes CCO data





# Appendix 10.3 – Wave Overtopping and Inundation Modelling



# Central Rhyl Coastal Defences

Wave Overtopping and Inundation Modelling

June 2021



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# Central Rhyl Coastal Defences

Wave Overtopping and Inundation Modelling

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# Issue and Revision Record

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# Executive summary

Mott MacDonald reviewed the information from the OBC design previously undertaken. Following the review, updated overtopping and inundation modelling was undertaken to confirm the validity of the proposed OBC scheme. Additionally, the assessment defines the proposed defence geometry within Section E-H of the proposed scheme to achieve the aspiration of no flooding under a 0.5%AEP event in 2120. The results of this assessment and inundation model are used to confirm and refine the basis for the concept design geometry.

This report documents the issues identified in the previous wave overtopping assessment and inundation modelling and summarises the methodology and criteria used by Mott MacDonald for the wave overtopping assessment. It subsequently presents the corresponding overtopping results and related inundation modelling. Finally, it concludes with outcomes for the concept design which recommends the revetment geometry in Section E-H as:

- Stepped Revetment with a slope of 1:2.
- Revetment Crest at +7.85mOD.
- Promenade Width of 10.1m (minimum).
- Rear wall crest at +9.25mOD, 1.4m above revetment crest.
- The rear wall will require inclusion of a compliant bullnose geometry.

# 1 Introduction

Mott MacDonald Ltd (Mott MacDonald) has been appointed by Balfour Beatty Civil Engineering Limited (BB) on behalf of Denbighshire County Council (DCC) to undertake the design of the Central Rhyll Coastal Defences Scheme (herein referred to as “the Scheme”). The Scheme comprises the coastline in Rhyll from the mouth of the River Clwyd to Splash Point, where East Rhyll Coastal Defences Scheme starts.

The Scheme, located in the central area of Rhyll waterfront, Denbighshire, North Wales (Figure 1.1), comprises coastal defence works including new rock armour, stepped concrete revetment, promenade raising, sea wall works, access and promenade improvements, along with associated tie-in work.

**Figure 1.1: Location of Scheme**



Source: DCC, 2019.

The overall ambition of the Central Rhyll Coastal Defences Scheme is to provide a standard of protection to properties from the 1 in 200 year Return Period (RP) (also known as the 0.5% Annual Exceedance Probability (AEP)) coastal flood event for 100 years.

As part of the works, modelling of the overtopping performance of the frontage is required to confirm the level of inundation risk based on the Outline Business Case (OBC) preferred option as well as to inform the geometry of the design revetment section.

## 1.1 Purpose of this report

This report documents the work undertaken to model the overtopping performance of the preferred option identified in the OBC as:

- Option S1 (rock toe at existing defences) in Sections A – D; and,
- Option S2 (new defence) in Sections E – H.
- The OBC concluded that no work was required in Section I as a recent scheme has been implemented in this Section. Therefore, the overtopping performance of this section is not discussed herein.

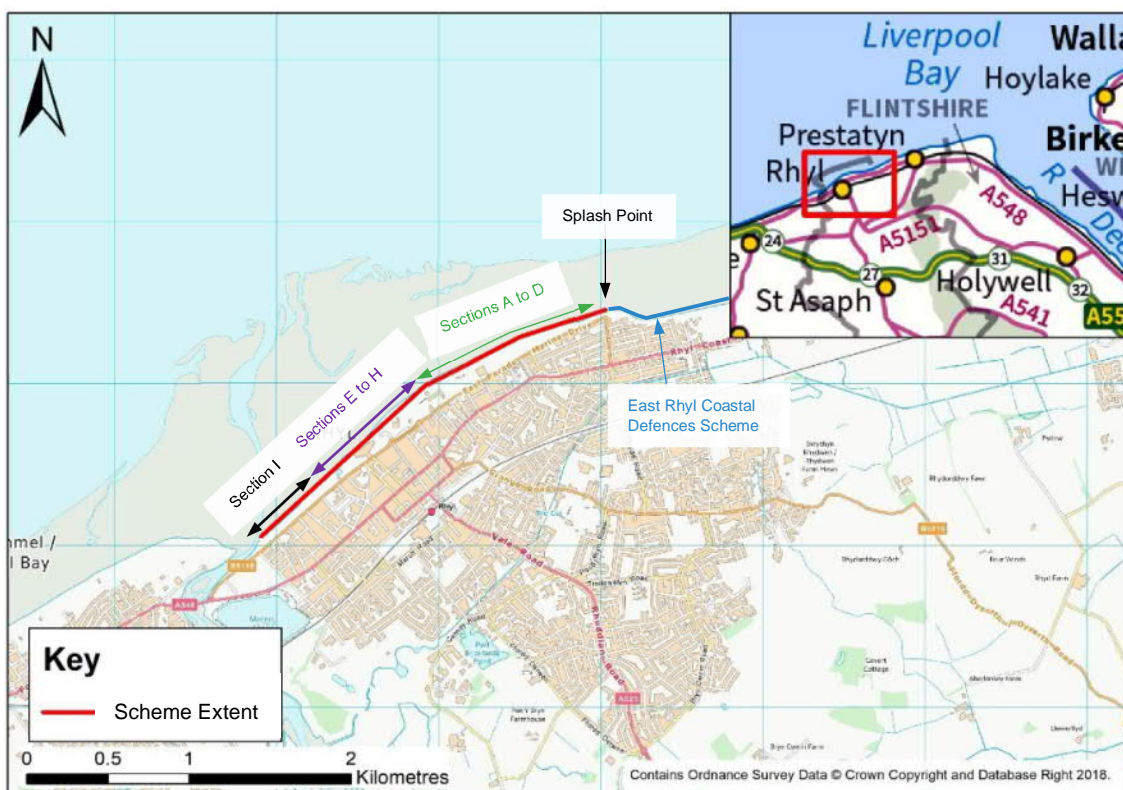
The locations on the frontage of Sections A – D and E – H as well as Section I are denoted on Figure 1. below.

This report presents a review of previous overtopping and inundation modelling which was part of the OBC development, and then presents the updated overtopping and inundation modelling including information on the input data used, methodology and the results and outcomes from that modelling.

## 1.2 Frontage sections

The Rhyl frontage was split within the OBC into 9 sections from A to I. Section A starts at Splash Point in the east with the sections running west to Section I adjacent to the mouth of the River Clwyd.

**Figure 1.2: Frontage Sections**



Source: DCC, 2019.

## 1.3 Aims and objectives

The main aim of the overtopping and inundation modelling is to provide:

- Overtopping rates for each section for use as inputs to inundation modelling to “validate the performance of the concept designs”;
- Confirmation of the geometry of the design cross section for Section E-H;
- Inputs to inundation modelling for the Flood Consequence Assessment.

Consequently, the requirements of the modelling are to determine:

- Peak overtopping discharge for design conditions.
- Overtopping discharge hydrographs over a tidal cycle for the inundation modelling
- Compliance with design requirements for Section E-H by assessment of both:
  - $q$  (mean overtopping discharge per metre structure width) and
  - $V_{\max}$  (maximum individual overtopping wave volume per unit crest width).
- Inundation modelling results.

#### 1.4 Overtopping and inundation uncertainty

Modelling and assessment of processes that are linked to random natural processes and prediction of future events (such as storms driving overtopping and inundation) are subject to high levels of uncertainty. Furthermore, modelling should be considered only as a fair representation of the processes, underlying data, and key influencing factors, which are limited by the granularity and resolution of models.

Mott MacDonald has followed accepted procedures in providing the services but given the residual risk associated with any prediction and the variability which can be experienced in flood conditions, Mott MacDonald takes no liability for and gives no warranty against actual flooding of any property (clients or third party) or the consequences of flooding in relation to the performance of the service. This report has been prepared for the purposes of presenting the results of the wave overtopping and inundation modelling and assessment only.

## 2 Review of OBC overtopping assessment

As required by the scope, modelling should be developed based on previous work undertaken as part of the OBC and endeavour to utilise that previous work. This would allow for efficiency and compatibility with previous work. Prior to utilising the OBC modelling of overtopping a review of previous work has been undertaken to ensure the methodology is suitable, appropriate and meets best practice.

### 2.1 OBC overtopping modelling

#### 2.1.1 Assessment summary

The OBC methodology modelled overtopping using the Artificial Neural Network (ANN) associated with EurOtop 2018 (Van der Meer, et al., 2018) to model overtopping along the Central Rhyl frontage for four options – the final preferred option was a combination of two of these options.

The Outline Business Case (OBC) assessed the following options:

- Baseline assessment, comprising of two scenarios:
  - Walkaway (WA) – no maintenance, decline in performance, breaches
  - Business As Usual (BAU) – continue present maintenance approach, decline in performance, breaches.
- S1 (Rock toe at existing defences) – no failure of defences over the 100 year appraisal period, no breaches
- S2 (New defence) – new defence, reduced overtopping, no erosion/breaches
- S3 (combination of S1 and S2) – new defence between Section E and Section H, S1 measures in A-D.

Inundation resulting from the modelled overtopping rates was subsequently assessed using an ESTRY-TuFlow 2D inundation model.

#### 2.1.2 Defence geometries assessed

##### 2.1.2.1 Baseline assessment (WA and BAU)

The baseline assessment was based on topographical/survey data of unknown provenance. However, it has been demonstrated to broadly (though not exactly) correspond to the shapes found in the DTM (LiDAR) data transmitted by Balfour Beatty that was used in the Inundation modelling.

##### 2.1.2.2 S1 – Rock toe at existing defences

This was modelled based on section geometry identical to the present day (2018) BAU geometry, to which water level/wave combinations have been applied.

##### 2.1.2.3 S2 – New defence

This was modelled based on a defined section geometry which was designed based on incremental assessment of many different geometries in the ANN. The final assessed design section that was selected was:

**Table 2.1: JBA final assessed design section geometry parameters**

Stepped revetment crest level (m AOD)	Wall crest level (m AOD)	Slope (1 in X)	Crest width (m)
+7.25	+8.45	3	7

Source: Concept Design Technical Note - S1, S2 and S3 (JBA Consulting, 2019).

Overtopping values generated in the ANN for each return period were then applied to the overtopping boundary along with a tidal boundary in the ESTRY model.

#### 2.1.2.4 S3 – Combination of S1 and S2

S3 was not modelled explicitly. Instead it was assumed that the benefits associated with S2 would simply be achieved by S3.

S3 requires to be modelled as results could indicate significant amount of overtopping for portions of the frontage. This is a key outcome of the overtopping assessment in this part of the project.

#### 2.1.3 Identified risks and issues with OBC approach

The full risk and issue review outcomes relating to the overtopping methodology are presented in Appendix A. The key items are summarised below:

Waves, and wave and water level combinations, used as inputs for the OBC assessment were not accurate. These issues were previously flagged by Mott MacDonald and subsequently waves were remodelled, and a revised Multi-Variate Analysis undertaken to determine the wave and water level combinations. The overtopping modelling utilises these revised combinations and therefore direct comparison between results presented herein with the previous OBC is invalid.

The key outcome of the review was that the OBC methodology utilised the ANN approach, however as the geometry and wave conditions used in the assessment were often outside of the neural networks training data the results were flagged. This flagging was not reported in the OBC methodology.

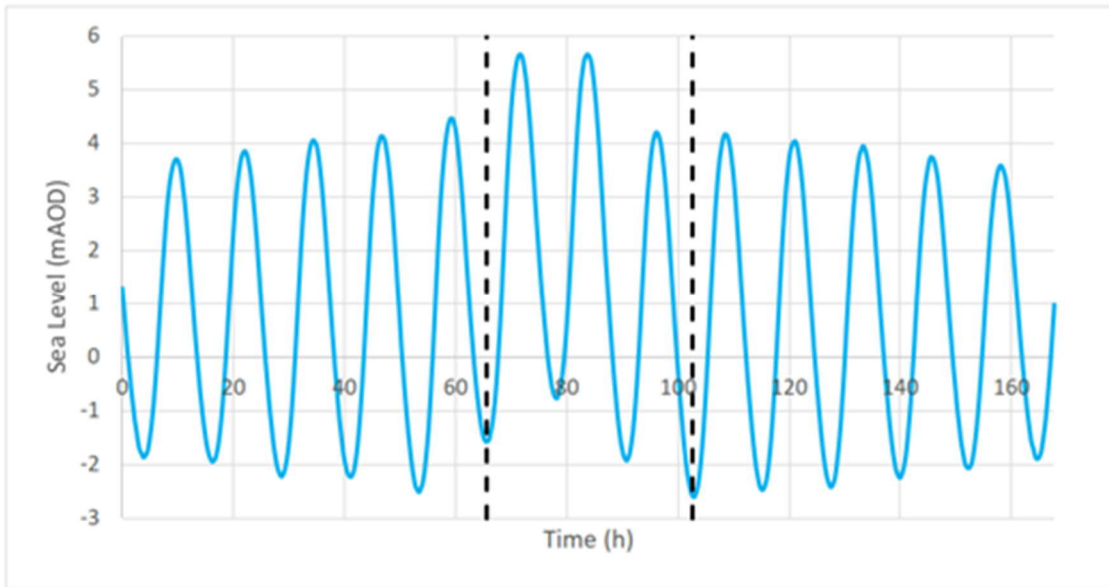
Mott MacDonald's approach will be to utilise the empirical equations within EurOtop; this approach is based on the same guidance as the ANN approach, however the use of the equations allows for better interpretation and assessment of the results and allows a wider range of geometry functions to be considered.

## 2.2 Inundation modelling

The OBC inundation model uses an offshore tidal boundary and several overtopping boundaries to model inundation.

The OBC inundation modelling was carried out using the tidal boundary shown below with overtopping volumes applied during period from 81.5 h to 86.25 h, the second peak in the tidal phase around the peak of the surge.

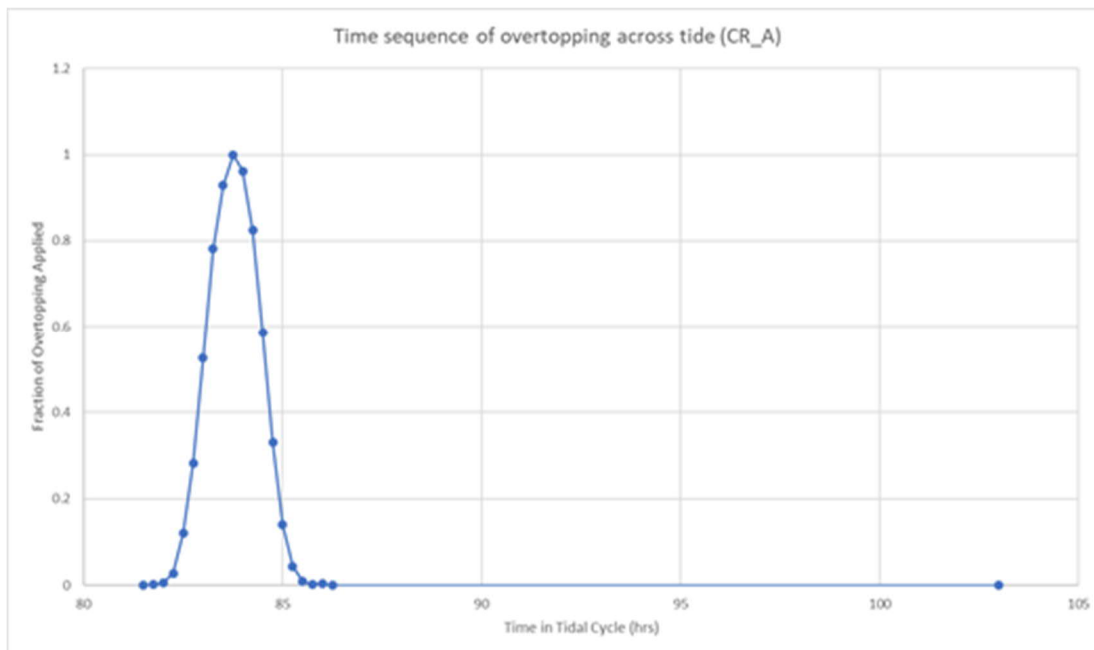
**Figure 2.1: 2018 0.5% AEP tidal boundary condition input time series and simulation window**



Source: Central Rhyl and Central Prestatyn Inundation Modelling Report (JBA Consulting, 2019).

Overtopping was factored as shown in the following distribution. The distribution is constant across all sections and return periods and reaches a peak of 1 coincident with the second peak at the peak of the surge.

**Figure 2.2: Overtopping factor distribution over tidal cycle**

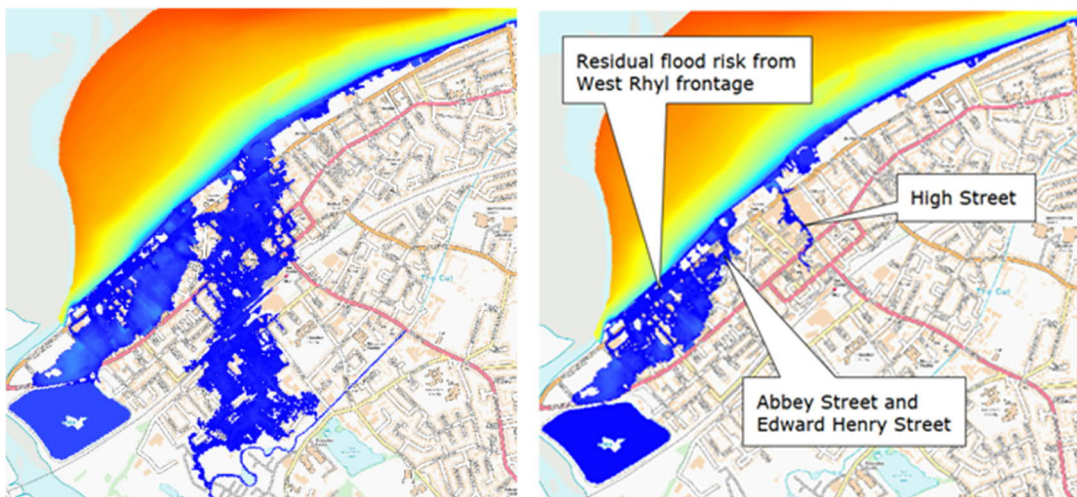


Source: Mott MacDonald based on data from JBA.

### 2.2.1 Identified risks and issues with inundation modelling

1. The link between the tide applied at the tidal boundary and the water levels used in the overtopping calculation were not clear. Given the variation of sections and the multifarious wave-water level combinations for overtopping, the same uncertainty applies for how a tidal cycle has been chosen to match all of them.
2. The factoring used does not clearly produce a reasonable approximation of how overtopping changes over the tidal cycle.
3. The inundation model has only been run for S1 and S2. It has not been run for S3 (the combination of S1 and S2 that is to be implemented). The OBC appendix on economics assumes S2 and S3 perform identically. This seems to be wrong when identical events for S1 and S2 are compared (See Figure 2.3). S3 should be modelled as there are some portions of the frontage where there may be a significant amount of overtopping. This is a key outcome of the overtopping assessment in this part of the project.

**Figure 2.3: Predicted maximum depth grids – JBA outputs Options S1 and S2, 0.5% AEP 2118**



Source: Central Rhyl and Central Prestatyn Inundation Modelling Report (JBA Consulting, 2019).

#### 4. Issue with large flow from West Rhyl

The Central Rhyl and Central Prestatyn Inundation Modelling Report (JBA Consulting, 2019), depicts large volumes of water coming from the West Rhyl area (Section I). In the report it is mentioned that in that scheme a lower Standard of Protection (SoP) was used. However, according to Balfour Beatty (BB) design drawings, the West Rhyl Scheme appears as a solid embankment with a wide crest at +8.40 m AOD. That contradicts the JBA information and the reasoning for large amounts of water going through the defence at the rates indicated.

From the overtopping schematisations of JBA, it was observed that their calculations were done based on an approximation of the “as built” defence, including the rear embankment crest at +8.29m AOD. Even though the topography has included both walls, the schematisation has omitted both walls (conservative). The lower wall was likely omitted because the promenade in this configuration effectively represented a berm for the purposes of the ANN calculation.

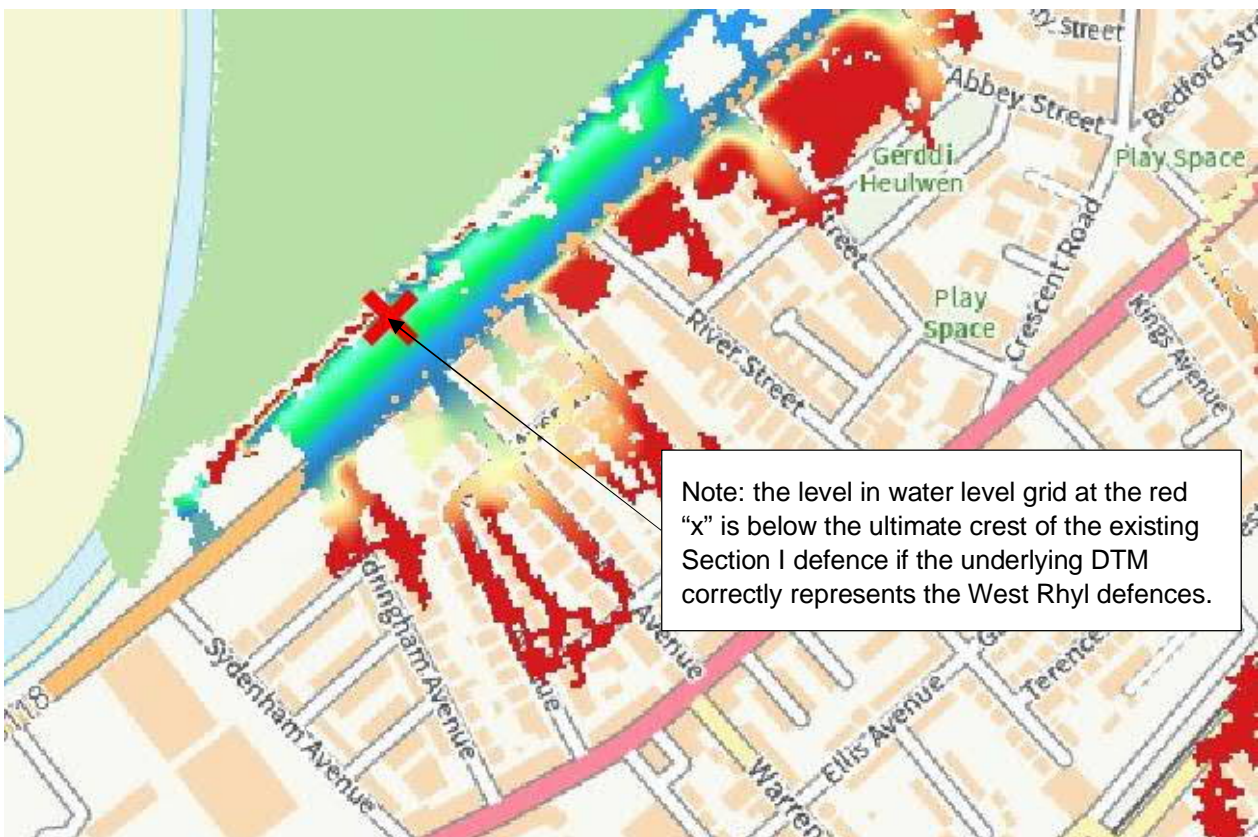
The levels used in the overtopping schematisation for Section I however were not as indicated in the West Rhyl drawings. The upper promenade should have a level of +8.40 m AOD at the seaward edge, but in the schematisation it is at +8.29 m AOD. Also, the landward wall (which does not appear in the wave overtopping assessment) is modelled with

a crest at +8.968 m AOD instead of at +9.10m AOD shown in the design drawings. This is a significant discrepancy as in the OBC modelling results for the BAU 2018 20% AEP event (2018s1313\_026\_T5\_2018\_BAU\_2018\_h\_Max), a water level of at least +8.8046 m AOD is identified. That results in a flow route that may not exist if the upper wall was included in the assessment.

There is uncertainty over the inclusion of defence lines in the TuFlow model. Some lines could have been included in the inundation mapping, but from initial observations, these are not in the region of CR\_I (the West Rhyl part of the model). This would lead to underestimation of the performance of the West Rhyl scheme.

- 5. No breach scenario was found to have been modelled for Section I. The modelling report indicates that a breach could be predicted at Year 50, but it was deemed that the high ground elevations would prevent any issues, so the breach was ignored.

**Figure 2.4: Water level at Section I in BAU 2018 20% AEP event**



Source: Mott MacDonald

### 2.3 Review Findings

Our review of the OBC overtopping modelling, concluded that the previous methodology has significant limitations which are likely to affect the robustness of results produced and thus impact the effectiveness of any detailed design.

Additionally, the updated wave and water levels combinations for the revised modelling and multi-variate analysis undertaken by Mott MacDonald will form the basis of the overtopping modelling input. This dataset includes larger waves that were truncated in previous studies.

An alternative approach to overtopping calculations will be adopted by Mott MacDonald that is based on similar design standards and guidance but provides a more investigative and robust approach to modelling the overtopping performance of the frontage structures.

Some changes to the TuFlow element of the numerical model are to be implemented to ensure the model is representative of the structures within Section D, refer to Section 5.1.

## 3 Wave overtopping modelling

This section details the background methodology and underlying data utilised in the overtopping modelling. For outcomes, interpretation and discussion on the overtopping modelling results refer to Section 4.

### 3.1 Wave overtopping criteria for design sections

The criteria set out in EurOtop Manual (Van der Meer, et al., 2018) and in the older version of the EurOtop Manual (Pullen, et al., 2007) for  $q$  and  $V_{max}$ , have been reviewed to assess the level of risk associated with wave overtopping for different scenarios.

The OBC set an overtopping discharge ( $q$ ) limit of 5 l/s/m in the 0.5% AEP plus climate change (2118) for any designed cross sections.

Communication from Denbighshire County Council has confirmed that the promenade immediately along the frontage may be managed/closed during storms and that prevention of flooding is the key criterion rather than ensuring pedestrian safety on the promenade (as closures and signs can accommodate pedestrian safety). The 5 l/s/m limit is quite onerous for an AEP such as 0.5% AEP however it was demonstrated that it prevented flooding within the OBC modelling.

For the design section Mott MacDonald considered a number of criteria to ensure that the overtopping limit for designed sections was appropriate to prevent significant damage to assets behind the new defence and to prevent inundation:

1. Limits for structural damage to the areas behind the defence are defined in the EurOtop Manual. However, for areas of promenade and carriageway landward of the crest no limits are given in the latest version of the guidance (2018) and the limits from the previous version of the guidance (2007) have been adopted, as shown in Table 3.1. Those values are defined to ensure that the promenade is not damaged during extreme events behind the crest of the revetment/seawall during 0.5% AEP events in 2120 (which is the design Standard of Protection (SoP)).

**Table 3.1: Recommended discharge limits for assets**

Surface Type	$q$ (l/m/s)	Return period (years)
Paved	200	100
Grass	50	100

Source: Mott MacDonald adopted from EurOtop Manual (Pullen, et al., 2007)

2. To ensure that the limit on overtopping rate to prevent inundation was not too onerous and allows design of an efficient structure Mott MacDonald undertook several inundation runs. The overtopping rate from the initial proposed geometry was tested, however for the initial proposed geometry (circa 70 l/s/m) this rate caused significant inundation and subsequently as the overtopping analysis progressed through a range of geometries, a range of limits on overtopping rate along Section E-H was tested to determine the overtopping limit that would prevent significant inundation. Inundation modelling of limits of 15, 10 and 5 l/s/m were undertaken (refer to Section 5.3.1). This showed that the limit on overtopping rate in a 0.5% AEP storm criteria to prevent inundation of assets within the town of Rhyll could be as high as 10 l/s/m. This limit is significantly more onerous than the criteria in point 1 above and became the determining criteria for the structure geometry.

- For pedestrians immediately behind the crest of the defence structure, it is recommended that a safety check is undertaken for to ensure that overtopping limits in more frequent events (when people may not be so aware of the dangers) are suitably low so as not to be dangerous, overtopping rates will be assessed for the 1 in 5-year wave and water level conditions in 2120 against the criteria below.

**Table 3.2: Recommended discharge limits for users**

User	$V_{max}$ (l/m)	Return period (years)
Pedestrian	600	1 - 10

Source: Mott MacDonald adopted from EurOtop Manual (Van der Meer, et al., 2018)

Overtopping measurements vary widely even within a single series of tests in physical modelling. It would not be unreasonable to expect the results of a physical model to be between a half and twice the predicted value for the structure. Strict adherence to the specified overtopping limits, particularly as they are upper bound predictions, is not appropriate as in many instances this may result in impractical structures. Professional judgement and risk assessment maybe used to determine if marginal exceedances of these criteria are acceptable.

### 3.2 Wave overtopping methodology

Overtopping rates are calculated by modelling the volume of water that passes over the crest of the defence. This overtopping assessment has been undertaken according to the guidelines of the EurOtop Manual (Van der Meer, et al., 2018). The upper bound values (design approach) of the EurOtop Manual (Van der Meer, et al., 2018) empirical equations were used.

Traditional design approaches make use of Joint Probability Return Periods, these define a limited number of wave and water level events corresponding to a return period. Typically, this approach then ignores variation in wave period and direction which are assumed to be worst case, or at best may be limited to a couple of typical directions/periods. This approach therefore assumes that the limited range of inputs accurately describe the performance of the structure, and that the resulting overtopping rates have the same probability as the inputs.

Recent advances in design approach makes use of a more complex Multi-Variate Analysis (MVA), and this approach was used for this assessment. The MVA approach simulates a pseudo 10,000 year dataset of storms (in total over 300,000 events) which is then applied to structure geometries via the empirical formula provided in EurOtop. Each event in the timeseries varies water level, wave height, wave period, and wave direction. This produces a much larger envelope of input parameters than the traditional methods ensuring that results are more robust and not limited by the selection of a small number of events. Each geometry change is remodelled with the MVA dataset to ensure that changes in overtopping physics are captured as each storm combination of wave, water level, direction, and period will perform differently with each geometry. The advantage of this method is that it tests a larger range of conditions and therefore models the probability of overtopping more robustly, this provides a significantly greater level of certainty that the design structure will perform as expected over traditional design methods.

Structure geometry parameters along with wave and water level combinations are run through script which automatically apply the rules and empirical equations defined in EurOtop. The script is efficient and allowing a large number of calculations to be complete in relatively short time. The script output defines all calculated parameters and records the decision tree to enable in-depth analysis of the results. Alongside the calculated parameters the overtopping results for all calculations are also output:

- Mean overtopping discharge ( $q$ ), defined as the average overtopping volume from all waves, it is estimated in litres per second per metre length of defence (l/s/m).

- Individual maximum overtopping volume ( $V_{max}$ ), is defined as the maximum discharge from a single wave in litres per metre length of defence (l/m).

Once overtopping rates are output for the wave and water level combinations for each structure geometry the results can be sorted or ranked to provide information on the performance of the structure for various return periods. These results ( $q$  and  $V_{max}$ ) can then be checked against the overtopping criteria to establish the performance of differing geometric structure configurations. For definition of those criteria for this project refer to Section 3.1. The main assumptions for the calculation can be found in Section 3.3.

Assessment of each section informs only on the performance of each section individually and not as a frontage under a single storm event. This is because the storm generating the highest overtopping rate for a given return period will differ for each section owing to the structure geometries and variation in exposure along the frontage. However, within the dataset the storms are all indexed and therefore if required can be linked so that overtopping rates for all sections for a given storm may be determined.

The overtopping assessment is further complicated along the Rhyl frontage as overtopping rate does not directly translate into inundation of assets. For some sections (notably Sections A and B) whilst high overtopping discharges maybe calculated at the frontage, much of the overtopping water would not generate flooding within Rhyl owing to the topography of the land behind the frontage. Overtopping and inundation modelling has therefore focused on assessing sections individually to determine the contribution to the overall flooding which cannot be easily determined when modelling the frontage as a combined event.

### 3.2.1 Structure geometries assessed

Numerous existing defence geometries and geometry variations for the proposed design were tested to validate the OBC option and to robustly test the design geometry. All existing geometries were tested for the present day. Sections A to D were also assessed for 2120 future scenarios as these structures are to be retained. The design geometries were tested at 2120 and then subsequently tested for 2070 and present day to confirm their performance and look for efficiencies in the design. A summary of the key assessed geometries is shown in Table 3.3 below), this list is not an exhaustive list of variations tested but relates to key decisions on the proposed geometry documented herein.

**Table 3.3: Key defence geometry variations assessed**

ID	DCC Profile Used	Wall Height (m)	Revetment Crest level (mOD)	Crest Width (m)	Slope	Surface Type
<b>Existing Structure Geometry</b>						
SECTION_A – Existing Geometry	DCC05	1.56	6.4	9.5	1V:1.5H	Stepped
SECTION_B – Existing Geometry	DCC04	2	6.5	9.35	Vertical	Smooth
SECTION_C – Existing Geometry	DCC04	1.72	6.48	14.7	Vertical	Smooth
SECTION_D – Existing Geometry	DCC03	N/A	5.8	16.7	1V:2H	Stepped
SECTION_E – Existing Geometry	DCC02	3.45	N/A	16.4	Vertical	Smooth
SECTION_F – Existing Geometry	DCC02	2.1	N/A	13.3	Vertical	Smooth
SECTION_G – Existing Geometry	DCC02	1.45	5.0	5.3	1V:2H	Stepped
SECTION_H – Existing Geometry	DCC01	0.7	6.2	7.5	Vertical	Smooth
<b>Section E-H Proposed Geometry</b>						
Initial Concept Option Geometry	DCC02	1.15	7.85	5.3	1V:3H	Stepped
Geometry Variation 1	DCC02	1.15	7.85	10.1	1V:3H	Stepped

ID	DCC Profile Used	Wall Height (m)	Revetment Crest level (mOD)	Crest Width (m)	Slope	Surface Type
<b>Existing Structure Geometry</b>						
Geometry Variation 2	DCC02	1.4 (BN)	7.85	10.1	1V:3H	Stepped
Geometry Variation 3	DCC02	1.4 (BN)	7.85	10.1	1V:2H	Stepped

BN – Indicates addition of bullnose on rear crest wall.

Source: Mott MacDonald

### 3.3 Wave overtopping assessment decisions and assumptions

#### 3.3.1 Input assumptions

The inputs to the calculations comprise the water levels, wave heights and wave periods for each section and return period, the structure geometry, and the beach profile for each of the sections assessed (including beach drawdown).

##### 3.3.1.1 Wave and water level combinations (storm conditions)

Wave-water level combinations were produced by Mott MacDonald for epochs of 2020, 2070 and 2120 with waves transformed and extracted at the defence toe levels. These include many combinations for each section (more than 300,000). Each combination was comprised of:

- deep water wave height
- spectral wave period at deep water
- extreme water level
- wave height at the toe of the defence
- wave direction at the toe of the defence

These were filtered to remove events with zero water depth or wave height at the toe of the structure being tested.

##### 3.3.1.2 Overtopping storm duration

The overtopping storm duration is used to calculate the number of waves, which is used to estimate  $V_{max}$  according to EurOtop (Van der Meer, et al., 2018). A typical storm duration of six hours was assumed for storm events where the depth at the toe was larger than 1m and three hours for smaller depths, this reflects the magnitude and severity of the likely storms.

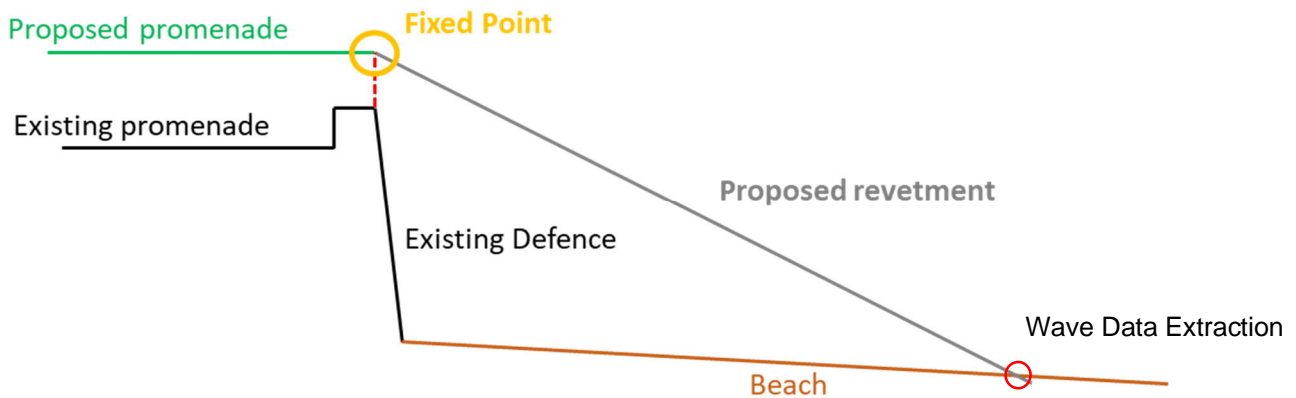
##### 3.3.1.3 Beach profiles and sections

DCC sections DCC02, DCC03 and DCC05 from June 2020 were used to transform waves to the nearshore and extract the wave data at the toe of the defence. These sections are sufficient to represent the whole frontage as the beach profile does not show any significant variations between DCC sections, the defences are fairly regular along the frontage and using the captured existing toe and crest levels from these sections cross-referenced to the latest topographical information (PM SURVEYS UK Ltd, 2021) is adequate. Also, the detailed information available on the frontage (i.e. actual survey information going seaward of the defence) is limited to these DCC sections, and this represents the best available information on toe elevation.

For the future scenarios (50 and 100 years, in 2070, and 2120 respectively), new profiles were produced, where the foreshore bathymetry was adjusted to account for future beach drawdown that may occur owing to sediment losses driven by climate change impacts.

To estimate the extraction point of the wave data at the toe of the defence, the geometry of the proposed structure was considered to determine offsets from the existing seawall as shown in the sketch of Figure 3.1.

**Figure 3.1: Sketch of fixed point of the proposed defence at the front face of existing defence**



Source: Mott MacDonald

#### 3.3.1.4 Defence geometries

The defence geometry parameters used for the overtopping modelling were:

- Slope of the structure (if not a vertical wall)
- Slope crest level (if not a vertical wall)
- Crest (promenade) width
- Toe level
- Foreshore slope
- Wall height (if applicable, refer to Figure 3.2)
- Defence angle to north
- Defence Roughness factor
- Bullnose height (if applicable, refer to Figure 3.2)
- Bullnose angle (if applicable, refer to Figure 3.2)

#### Existing defences

The existing defences were assessed for the Business As Usual (BAU) scenario (which is assumed identical to the Walkaway scenario in terms of overtopping performance). Information from DCC bathymetric sections DCC01 to DCC05 from June 2020, the latest topographic survey (PM SURVEYS UK Ltd, 2021), site photos and site visit were used to define the existing structure geometries (inputs listed above). Some assumptions were to assess the defences:

1. Some of the existing defences of the frontage are comprised of two vertical walls with the promenade between them. There is a lack in the guidelines for such cases. Therefore, it has been assumed that:
  - a. If the water level is below the promenade level, the defence is schematized as a vertical wall with crest level equal to the crest level of the landwards (higher) wall. A reduction for the width of the promenade has been added in post-processing.
  - b. If the water level is above the promenade level, the defence was schematized with the seawards vertical wall as a very steep slope, followed by a wide promenade and a crest wall.

2. In cases where the existing defence is comprised of a revetment leading up to a vertical wall, followed by the promenade and another vertical wall, it has been assumed that the seawards vertical wall is part of the revetment and the revetment slope has been adjusted (steeper) to account for this assumption.
3. To determine the geometrical parameters (toe level, crest level, crest width, wall height), a representative profile cross-section was selected from the latest topographic survey (PM SURVEYS UK Ltd, 2021) for each section of the frontage.

### Proposed defences

For the proposed design, DCC02 bathymetric profile, site photos and the latest topographic survey (PM SURVEYS UK Ltd, 2021) were used to assess different design option geometries. The new defence will defend frontage sections E to H. These are all best represented by DCC02 except for section H which relates to DCC01.

A comparison of DCC01 and DCC02, indicated that the gentler slope and lower seabed nearshore at DCC02 provides more adverse conditions. Although further offshore DCC02 becomes shallower than DCC01, considering the wave heights and extreme water levels at the location, the waves are depth limited. Therefore, the seabed level at the toe has the greatest impact. Thus, DCC02 represents a more adverse scenario that still presents a good fit with DCC01 and was used for the assessment.

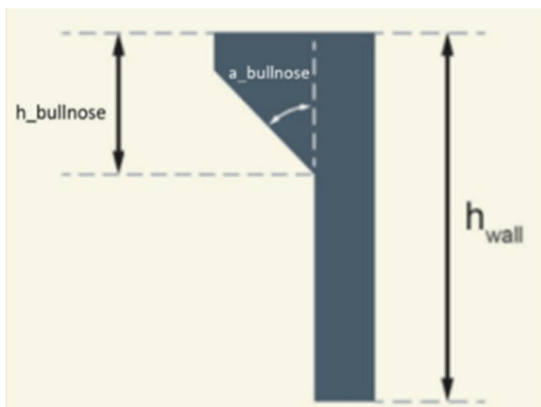
### Defence roughness

The assumed roughness factor was  $\gamma f = 0.8$  for the stepped revetment and  $\gamma f = 1$  for a smooth revetment or vertical walls.

### Bullnose (return wall)

Bullnose reductions were included in the assessment of geometric variations of the proposed section. This is comprised of the bullnose height and bullnose angle as shown in Figure 3.2.

**Figure 3.2: Identification of bullnose parameters**



Source: Mott MacDonald - modified image from (Van der Meer, et al., 2018).

## 3.3.2 Calculation assumptions

### 3.3.2.1 Selection of solving methodology depending on water level

The empirical formulae of EurOtop (Van der Meer, et al., 2018) both for the estimation of  $q$  (used in the Empirical Equation Approach) and for the estimation of  $V_{max}$  (used in both approaches), are different for slopes (revetment) and vertical walls.

The approach in using the formulae is the following:

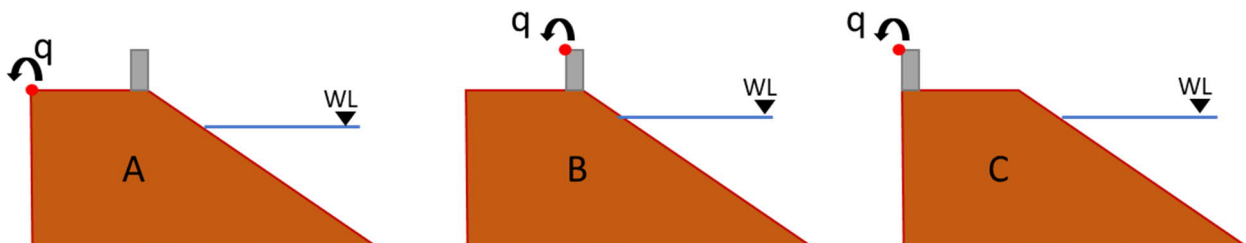
- For revetments, if the water level is below the crest of the revetment, the smooth slope formulae (according to EurOtop Manual chapter 5) for the estimation of  $q$  and  $V_{max}$  were used (including reductions for wide promenade, crest wall and bullnose as applicable).
- For the case of a vertical wall without a revetment, or if the water level is above the crest of the revetment and a wall is present at the crest of the revetment, the vertical wall formulae (according to EurOtop Manual chapter 7) were used for the estimation of  $q$  and  $V_{max}$  (including reductions for a mound at the foot of the wall to account for the revetment (if present), and for the effect of bullnoses, parapets and battered walls as applicable).
- If there is negative freeboard ( $R_c$ ), then  $q$  is calculated for zero freeboard and  $q_{overflow}$  is calculated and added to that value, except for large negative freeboards where overflow is the governing process.

The above (overflow) has also been applied when solving for vertical walls, even though not mentioned in the EurOtop Manual (Van der Meer, et al., 2018) for such cases.

### 3.3.2.2 Crest wall on smooth slope location limitation

There is a limitation of the EurOtop (Van der Meer, et al., 2018) when solving with the smooth slope solving methodology (see 3.3.2.1 above), in cases where there is a crest wall at the seaward edge of a smooth slope defence and there is a wide promenade landwards of that wall (see Figure 3.3 – A). The overtopping rate,  $q$ , can be either estimated directly landwards of the wall without considering the effect of the wide promenade (see Figure 3.3– B) or including the effect of the wall and a wide promenade, where the latter is seawards of the defence (see Figure 3.3– C).

**Figure 3.3: Limitation in calculating  $q$  depending on wall and promenade location**



Note: A cannot be estimated in EurOtop (Van der Meer, et al., 2018), while B and C can be calculated.  
Source: Mott MacDonald

In cases as A of Figure 3.3, an assumption in accordance to Section 7.5.3 of (Van der Meer, et al., 2018) is used to add a reduction to overtopping for the wide promenade (crest) using the wavelength.

### 3.3.2.3 Vertical walls on wide crests (promenades)

In the case when solving using the methodology for vertical walls (see Section 3.3.2.1 of this report) where there is a promenade or wide crest in front of the wall (EurOtop vertical wall case with mound at the toe – composite structure), the width of the promenade/crest (in this case the mound) is not taken into account. EurOtop (Van der Meer, et al., 2018) assumes that the mound width is approximately equal to the wave height at the defence toe (or smaller). In reality the mound could be much wider, and thus overtopping could be overestimated.

### 3.3.2.4 Mean wave period $T_m$

The mean wave period  $T_m$  is used in estimating  $V_{max}$ . It has been assumed that  $T_m$  is equal to  $T_{m-1,0}$  in deep water.

### 3.3.2.5 Effect of currents

It is assumed that no currents are acting on the defence (i.e. no overtopping reduction owing to the effect of currents has been included in the calculation).

### 3.3.2.6 Effect of oblique waves

The effect on overtopping due to wave obliquity has been included for sloping structures, under the assumption of short-crested storm waves. In the case where a solving methodology for vertical walls is followed (see Section 3.3.2.1 of this report), it is assumed that the waves are acting perpendicular to the structure.

### 3.3.2.7 Wave breaking criterion

According to EurOtop (Van der Meer, et al., 2018) wave breaking occurs when the breaking parameter  $\xi_{m-1,0}$  is approximately 2. Due to the physical processes involved in wave breaking, the limit on breaking parameter below which wave breaking occurs is not precise and is dependent on many factors. The EurOtop manual chooses to use the value of 1.8. However, specifically for the estimation of the  $\gamma^*$  and  $\gamma_v$  reduction factors (for smooth slopes), to provide continuity with the range of application of the equations, it is assumed that waves break when  $\xi_{m-1,0} > 2.2$ ; this is consistent with EurOtop Section 5.4.7.

### 3.3.2.8 Assumption beyond formulae validity range

Reduction factors  $\gamma^*$  and  $\gamma_v$  are applicable in the assessment of smooth slopes. In cases where the calculated value of either parameter was outside upper range of the formulae's validity (e.g.  $h_{wall}/R_c$  too large), the upper boundary of the validity range was used instead. This is because using a factor of 1 (no reduction) was considered unrealistically over-conservative.

### 3.3.2.9 Wide promenade reduction

When the reduction for a wide promenade  $\gamma_{prom}$  (smooth slopes) is applicable in the assessment, it can be applied in cases c, d and e of EurOtop (Van der Meer, et al., 2018) Section 5.4.7. However, the upper range of application for  $G_c/L_{m-1,0}$  is given as 0.5 and 0.4 depending on the case. To simplify, the upper range of 0.4 (most conservative between the two) has been applied.

The wide promenade reduction factor of EurOtop (Van der Meer, et al., 2018) equation 5.49 has been applied as a sequential pattern, as wider promenades that give  $G_c/L_{m-1,0}$  over the applicability range will still reduce overtopping. When  $G_c/L_{m-1,0}$  is larger than the maximum applicability range,  $\gamma_{prom}$  was estimated by linear interpolation between 0.812 and 1.

### 3.3.2.10 Effect of bullnose (return-wall) for vertical wall methodology

Due to errors identified in the interpolation between values in EurOtop (Van der Meer, et al., 2018) Fig.7.23 (vertical walls solving methodology), the method used to calculate the reduction factor  $k_{bn}$  was updated to amend the error.

### 3.3.2.11 Effect of battered walls for vertical wall methodology

When applicable, an increase to  $q$  to account for the effect of battered walls has been applied (vertical wall solving methodology). EurOtop (Van der Meer, et al., 2018) equation 7.11 provides only two values for specific wall inclines (5:1 and 10:1). An increase has been applied for wall inclines between 45 and 90 degrees by interpolating between the values given in the manual.

### 3.3.2.12 Weibull shape factor b for vertical wall methodology

The EurOtop (Van der Meer, et al., 2018) equation 7.28 provides only two values for the Weibull dimensionless shape factor b used to estimate  $V_{max}$  (vertical wall solving methodology). When this was applied in the calculation, an interpolation between the two values was undertaken for higher accuracy.

## 4 Wave overtopping results

This section presents the results of the overtopping assessment undertaken by Mott MacDonald in validating the OBC option and determining the design structure geometry for Section E-H.

The results presented herein are based on the revised multi-variate analysis (MVA) of wave and water level extremes for the Rhyl frontage undertaken by Mott MacDonald. The revised multi-criteria analysis contains larger wave combinations than previous documented at OBC, and as a result the overtopping performance / standard of protection of structures presented is not directly comparable to the values presented in the OBC.

Wave overtopping results are shown to three significant figures to illustrate the variability of overtopping rates. Where decimal places are included this does not imply a greater accuracy of calculation. All calculations are based on empirical equations and have an underlying variability, for further information refer to EurOtop (Van der Meer, et al., 2018).

### 4.1 Existing defences

The existing defences are modelled to understand the levels of performance of the current structures against overtopping. This can demonstrate the current standard of protection and how that protection deteriorates in the future (by applying climate change allowances).

The existing defences along the Rhyl frontage have been split into discrete sections in accordance with the lengths outlined in the OBC. Each section is broadly similar in its defence type and sections change only where there is a significant difference in defence type or geometry.

#### 4.1.1 Present day (2020)

Table 4.1 presents the wave overtopping rates determined for the individual section of the Rhyl frontage in Sections A to D for the present day (2020). These overtopping rates represent the current overtopping performance. These sections are under the S1 option of the OBC and are to be retained with no works undertaken to substantially improve their overtopping performance.

It should be noted that Section D has a significantly lower crest than other defence sections, additionally as the promenade is extremely wide and has some areas of open space through which water can flow no rear wall was calculated, instead the walls of the Rhyl Events Area were represented in the TuFlow model so that flow around these walls which may cause flooding would be represented. As outlined in Section 3.2, the overtopping rates presented here may not produce inundation within Rhyl but are simply volumes that pass the overtopping assessment line which is typically the rear wall of the promenade (with the exception of Section D).

**Table 4.1: Wave overtopping of existing defences: Sections A to D – Present day (2020)**

	Overtopping Discharge q (l/s/m)			
	Section A	Section B	Section C	Section D
<b>1 in 1000 yr. RP (0.1% AEP)</b>	34.4	15.3	7.42	702.
<b>1 in 200 yr. RP (0.5% AEP)</b>	15.4	7.17	4.03	426
<b>1 in 75 yr. RP (1.3% AEP)</b>	8.88	4.77	2.70	329
<b>1 in 30 yr. RP (3.3% AEP)</b>	3.40	3.20	1.70	223
<b>1 in 5 yr. RP (20% AEP)</b>	0.23	1.35	0.30	89.6
<b>1 in 2 yr. RP (50% AEP)</b>	0.07	0.78	0.16	46.3

Return periods relate to the storm impacting the individual sections.

Source: Mott MacDonald, 2021

Table 4.2 presents the wave overtopping modelling results for the individual sections in Sections E to H for the existing defences in present day (2020). These sections are to be improved with a single defence structure under the S2 option of the OBC which determined that these sections were a major source of inundation as climate change impacts are realised.

**Table 4.2: Wave overtopping of existing defences: Sections E to H - Present day (2020)**

	Overtopping Discharge q (l/s/m)				
	Section E	Section F	Section G (stepped)*	Section G (vertical)*	Section H
<b>1 in 1000 yr. RP (0.1% AEP)</b>	0.57	16.8	113	90.3	51.9
<b>1 in 200 yr. RP (0.5% AEP)</b>	0.19	3.97	45.2	34.5	17.3
<b>1 in 75 yr. RP (1.3% AEP)</b>	0.09	1.64	25.5	19.5	8.18
<b>1 in 30 yr. RP (3.3% AEP)</b>	0.04	0.56	14.0	8.11	0.57
<b>1 in 5 yr. RP (20% AEP)</b>	0.01	0.06	3.09	1.16	0.00
<b>1 in 2 yr. RP (50% AEP)</b>	0.00	0.01	0.73	0.36	0.00

\*Note: Section G is split into “stepped” and “vertical” as for the present-day case part of the stepped revetment is covered by the beach resulting and therefore this section performs differently in terms of overtopping performs differently.

Return periods relate to the storm impacting the individual sections.

Source: Mott MacDonald, 2021

#### 4.1.2 Present day single storm illustration

When a storm hits a coastal frontage such as Rhyl a single water and wave combination occurs. Encounter conditions along the frontage will still vary, but this variation is due to local conditions at each section e.g. exposure and angle of frontage section to incoming waves and the geometry type. This means that a single storm will cause significant differences in overtopping rates within a single storm along the frontage. This variation is illustrated in Table 4.3 and Table 4.4 below which show the variation in overtopping rate and individual section return period across the frontage for the same storm index.

**Table 4.3: Example variation of overtopping rate and return period for a single storm across the frontage: Sections A to D – Present day (2020)**

	Section A	Section B	Section C	Section D
<b>Overtopping Rate q (l/s/m)</b>	4.20	5.41	0.50	435
<b>Overtopping Return Period for individual sections (yrs.)</b>	37	102	8	217

Source: Mott MacDonald, 2021

**Table 4.4: Example variation of overtopping rate and return period for a single storm across the frontage: Sections E to H - Present day (2020)**

	Section E	Section F	Section G (stepped)*	Section G (vertical)*	Section H
<b>Overtopping Rate q (l/s/m)</b>	0.11	0.95	34.6	30.9	17.6
<b>Overtopping Return Period for individual sections (yrs.)</b>	105	45	116	167	204

\*Note: Section G is split into “stepped” and “vertical” as for the present-day case part of the stepped revetment is covered by the beach resulting and therefore this section performs differently in terms of overtopping performs differently.

Source: Mott MacDonald, 2021

The variation in structure types along the Rhyl frontage and especially the dominance of overtopping rates in Section D (owing to the much lower crest level) and the lack of direct linkage between overtopping rates and water causing inundation of assets within Rhyl makes the modelling of flooding more complex. The assessment of a single storm across the frontage could in this instance result in an under estimation of the proposed structure performance as overtopping rates are dominated by sections that are not within the area of the proposed defence upgrade and therefore any geometrical changes to the proposed design will not reduce overtopping rates in that section. Owing to this the decision was made to continue work assessing each section individually so that the influence of the higher overtopping rates in the sections not proposed for overtopping improvement works did not unduly influence the assessment of the proposed design structure.

#### 4.1.3 Future case (2120)

Modelling of the existing structures progressed to the future case in 2120 to illustrate the deterioration in wave overtopping performance that would be expected to occur owing to climate change and future beach drawdown. The results of this assessment can be compared with those in Section 4.1.1 to illustrate the increases in expected overtopping rates.

**Table 4.5: Wave overtopping of existing defences: Sections A to D – Future 100 year (2120)**

	Overtopping Discharge q (l/s/m)			
	Section A	Section B	Section C	Section D
<b>1 in 1000 yr. RP (0.1% AEP)</b>	276	397	150	2,205.
<b>1 in 200 yr. RP (0.5% AEP)</b>	163	316.	106	1,850
<b>1 in 75 yr. RP (1.3% AEP)</b>	100	270	84.4	1,580
<b>1 in 30 yr. RP (3.3% AEP)</b>	67.6	220	66.3	1,370
<b>1 in 5 yr. RP (20% AEP)</b>	33.1	137	34.5	1,080
<b>1 in 2 yr. RP (50% AEP)</b>	20.1	94.7	21.2	932

Source: Mott MacDonald

**Table 4.6: Wave overtopping of existing defences: Sections E to H - Future 100 year (2120)**

	q section E [l/s/m]	q section F [l/s/m]	q section G (stepped)* [l/s/m]	q section G (vertical)* [l/s/m]	q section H [l/s/m]
<b>1 in 1000 yr. RP (0.1% AEP)</b>	142	472	772	N/A	68.3
<b>1 in 200 yr. RP (0.5% AEP)</b>	102	350	590	N/A	39.3
<b>1 in 75 yr. RP (1.3% AEP)</b>	74.7	277	483	N/A	32.4
<b>1 in 30 yr. RP (3.3% AEP)</b>	56.0	221	396	N/A	26.6
<b>1 in 5 yr. RP (20% AEP)</b>	27.4	133	244	N/A	18.0
<b>1 in 2 yr. RP (50% AEP)</b>	16.8	94.8	168	N/A	14.0

\*Note: Section G was split into “stepped” and “vertical” for the present-day case as part of the stepped revetment is covered by beach, with future beach loss the structure stepped revetment becomes exposed over the full length. Return periods in this table relate to the storm impacting individual frontages.

Source: Mott MacDonald, 2021

## 4.2 Initial Concept Design

The OBC preferred option for Sections E to H is to replace the existing defences with a single defence structure along their length. The initial proposed concept defence geometry for Section E-H is to provide a stepped revetment fronting a promenade with rear crest wall.

The geometric parameters of the Initial OBC concept are outlined below:

- 1 in 3 stepped revetment slope.
- Crest level at 7.85mOD.
- A variable width promenade (initially modelled as 5.3 considered to be representative of the minimum promenade width).
- Rear crest wall level at +9.00mOD (minor 50mm reduction to make it consistent with a typical handrail height).
- Although the concept option showed a bullnose on the rear wall this was excluded from initial configurations as this was consistent with the OBC analysis which used the ANN method and is therefore unable to assess the effect of a bullnose geometry. Additionally, removal of this element if possible, would provide a saving on construction cost.

The OBC reported that this geometry would provide a reduction in overtopping rates to achieve 5l/s/m. However, this was based on the OBC’s MVA which was been found to have truncated wave inputs and therefore is likely to under predict the actual rate of overtopping. Overtopping results using the updated MVA are presented in Table 4.7 for 3 temporal scenarios:

1. Present Day 2020: This scenario allows for current beach variation and exclude allowances for future sea level rise. It defines how the structure would perform initially after construction.
2. Overtopping in 2070: This scenario includes the full future beach drawdown allowance but only 50 years sea level rise. It is undertaken to illustrate potential adaptability approaches for dealing with sea level rise which may reduce initial capital costs.
3. Overtopping in 2120: The 2120 scenario includes beach draw down and 100 years sea level rise. It illustrates the likely structure performance at the end of its design life.

**Table 4.7: Overtopping results for the initial proposed concept design for Section E–H**  
**Overtopping discharge  $q$  (l/s/m)**

	2020	2070	2120
<b>1 in 1000 yr. RP (0.1% AEP)</b>	0.86	44.6	122
<b>1 in 200 yr. RP (0.5% AEP)</b>	0.08	25.6	72.3
<b>1 in 75 yr. RP (1.3% AEP)</b>	0.01	17.7	55.4
<b>1 in 30 yr. RP (3.3% AEP)</b>	0.00	11.7	40.5
<b>1 in 5 yr. RP (20% AEP)</b>	0.00	4.48	19.6
<b>1 in 2 yr. RP (50% AEP)</b>	0.00	2.12	11.1

\*Note: Return periods relate to the storm impacting the entire frontage (Sections A to H), return periods are not specific to individual sections.

Source: Mott MacDonald, 2021

The results in Table 4.7 show that for the Present Day (2020) the proposed concept structure suitably reduces the overtopping rates below the OBC recommended threshold (i.e. 5 l/s/m) to reduce flooding in Rhyll. However, considering future climate change the proposed concept would not achieve the targeted overtopping limit (increased to 10l/s/m based on the inundation tests undertaken as part of this study) in either the 100 (2120) or the intermediate 50 year (2070) period. Thus, it would be expected that once significant climate change allowances are realised (in less than 50 years) the performance of the concept design option would not provide the anticipated level of protection against the 0.5%AEP event.

### 4.3 Variations of the concept design

As the initial concept option was demonstrated to underperform against the targeted criteria further assessment of geometrical changes were investigated. This section summarises the key geometrical variances (but doesn't report all testing that was undertaken) that were tested with the overtopping rates for the 0.5% AEP event presented in Table 4.8.

Changes to the revetment crest level were avoided as further increases of the revetment crest/promenade level would exacerbate transitions where the new structure interfaced with existing levels. Level increases in the promenade would also be less cost effective as it would increase fill volumes across the entire structure footprint, as well as pushing the revetment toe further seaward.

#### 4.3.1 Geometric variation 1

Owing to concept design development and consideration of buildability of the structures the promenade width was increased to a minimum of 10.1m. This was the minimum promenade width required to allow for a suitable crane operational width.

The increased promenade width helped to reduce overtopping rates but remained outside of the required 10l/s/m limit.

#### 4.3.2 Geometric variation 2

To increase the protection a further modification was to increase the height of the rear crest wall and provide the addition of a bullnose (as depicted in Figure 3.2). Multiple rear crest level heights were tested however a practical limitation of 1.4m was applied as this relates to the recommended barrier height for cyclists. Wall heights above this would prevent views for most. The tested options found that impractical wall heights would be required to limit overtopping.

### 4.3.3 Geometric variation 3

Review of the overtopping rates and geometric variations found that overtopping rates were not decreasing as expected when geometric changes were applied. Detailed assessment of the overtopping inputs and calculated parameters found that certain water level and wave combinations were driving high overtopping. The combinations driving high overtopping were because of overly steep waves breaking on the revetment slope. The breaking wave criterion within EurOtop is critical as it defines which reduction parameters can and cannot be applied. However, the breaking wave criterion is a limitation of the extent of physical modelling tests contained in the dataset that defines the empirical equations, rather than a physically induced criterion owing to the wave breaking processes.

To overcome this, geometric variations of revetment slope were tested. Changing the slope of the revetment causes the wave breaking to occur differently. Slope gradients were increased to 1:2.5 and 1:2 and overtopping tests were carried out. The results of these test showed that a steeper slope of 1:2 was more effective in reducing the calculated wave overtopping for those conditions.

Using a steeper revetment slope to reduce overtopping is contrary to typical overtopping performance where a shallower slope is most likely to reduce overtopping rates. However, for cases where the breaking wave criteria is encountered this significantly limits the application of promenade and rear crest wall reductions. However as stated this is a limitation of the underlying data. Derogation from using the wave breaking criterion and the implementation of a shallower slope may be possible and could result in increased wave overtopping performance, however this would require physical modelling of a cross section in a wave flume which is not being undertaken.

Taking into account the results of the overtopping assessment and inundation modelling and the criteria outlined in Section 3.1, the concept design that complies with the aspiration of providing protection from flooding in the 0.5% AEP event is defined below, this is the recommended geometry for the design of Section E-H:

- A stepped revetment with 1:2 slope,
- Revetment crest level at +7.85mOD
- 10.1m promenade width (minimum) between the revetment crest and the rear upstand wall. The promenade should have a suitable seaward crossfall applied to promote free draining.
- Rear crest wall level of +9.25mOD, a height of 1.4m above the revetment crest. Note that the visible wall height relative to the promenade will be <1.4m when the promenade crossfall is applied.
- The rear crest wall should be provided with a bullnose. The bullnose should have a height of 0.4m and be angled at 45 degrees.

### 4.3.4 Overtopping results of geometric variations to the concept design

The overtopping results for the geometric variations to the concept design for the 0.5%AEP for the 2120 scenario are presented in Table 4.8.

**Table 4.8: Overtopping results for the 0.5% AEP for geometric variations of concept design for sections E – H**

Geometry	Overtopping discharge q (l/s/m)
	2120
Geometric Variation 1	61.4
Geometric Variation 2	43.0

## Geometry

## Overtopping discharge q (l/s/m)

	2120
<b>Geometric Variation 3</b>	9.00

\*Note: Return periods relate to the storm impacting the entire frontage (Sections A to I), return periods are not specific to individual sections.

Source: Mott MacDonald, 2021

### 4.3.5 Overtopping results of recommended geometric variation 3

Table 4.9 presents a full range of overtopping rates for the recommended geometric variation 3 (refer to Section 4.3.3). This shows the present day performance of the structure in terms of overtopping as well as the performance allowing for 50 and 100 years climate change for a range of return periods. The table shows that for 2120 the overtopping discharge for the 0.5% AEP event remains below 10 l/s/m. For the 20% AEP in 2120 the Vmax value remains below 600 which is the criteria for pedestrian safety and therefore it would be expected that on the current estimate of climate change that immediately behind the defence would remain safe in events up to the 20% AEP. It should be noted though that as the area behind would not be expected to be in use in larger storms and there would be a low risk of danger to pedestrians as there is no risk of being washed to sea because of the rear wall. The promenade in front of the wall would be expected to be closed to the public during storms.

For the present day case overtopping is very low and would be expected to remain safe during storms up to the 1.3% AEP.

**Table 4.9: Overtopping results for the proposed concept design for sections E – H**

	Overtopping discharge q (l/s/m)			Individual Wave Discharge (Vmax)		
	2020	2070	2120	2020	2070	2120
<b>1 in 1000 yr. RP (0.1% AEP)</b>	0.55	5.49	24.4	446	2,284	5,902
<b>1 in 200 yr. RP (0.5% AEP)</b>	0.11	2.85	9.00	164	1,775	1,285
<b>1 in 75 yr. RP (1.3% AEP)</b>	0.02	1.04	5.49	53	919	2,167
<b>1 in 30 yr. RP (3.3% AEP)</b>	0.00	0.33	2.93	20	81	1,759
<b>1 in 5 yr. RP (20% AEP)</b>	0.00	0.04	0.72	0	16	153
<b>1 in 2 yr. RP (50% AEP)</b>	0.00	0.01	0.28	0	5	75

\*Note: The Vmax value for the 0.5% AEP 2120 has been checked and the lower Vmax is as a result of the wave and water level combination producing a flatter wave distribution.

Source: Mott MacDonald, 2021

Overtopping results in 2070 were reviewed to potentially identify where an adaptive approach may offer capital cost efficiencies in delivering the scheme. Changes to the front revetment and promenade were excluded from consideration as these would be particularly expensive and potentially impractical to adapt in the future. Reductions to the rear wall were considered however the overtopping rates for 2070 are only marginally reduced from those in 2120 and would therefore only allow minor changes to the wall crest height. Given the rear wall requires a bull nose to provide the required protection in 2120 any minor reduction in wall height would be impractical to achieve and therefore not likely to provide any significant cost efficiency.

### 4.3.6 Inundation rates in Section D

Inundation rates in Section D are shown to be very high, as discussed in Section 4.1.1, this is because of the lower crest level and the representation of the overtopping within the inundation model. Note that because of the difference in the frontage levels and topography the overtopping criteria (10 l/s/m) defined for Section E-H would not apply to this frontage. The significantly lower promenade crest and in places the absence of a rear wall means that significant flooding can be caused by the 0.5% AEP overtopping event in the future, refer to Section 5.3.5. Additional overtopping modelling was undertaken for Section D to illustrate the variance of the overtopping rates with respect to return period and epoch.

Table 4.10 shows that for the inundation modelling in Section 5.3.5 a rate of 1850 l/s/m is achieved which drives the significant flooding. Overtopping rates less than half of this are likely to be required to minimise flooding within Rhyl which means that towards the end of the 100 year forecast it is likely that flooding would be an almost annual event. Even in 50 years' time climate change would be having a significant impact and some flooding extent within Rhyl is likely to occur in events as low as the 1 in 5 year.

**Table 4.10: Overtopping results for the proposed concept design for Section D**

	Overtopping discharge q (l/s/m)		
	2020	2070	2120
<b>1 in 1000 yr. RP (0.1% AEP)</b>	702	1,380	2,205.08
<b>1 in 200 yr. RP (0.5% AEP)</b>	426	1,130	1,849.87
<b>1 in 75 yr. RP (1.3% AEP)</b>	329	992	1,577.60
<b>1 in 30 yr. RP (3.3% AEP)</b>	223	866	1,365.37
<b>1 in 5 yr. RP (20% AEP)</b>	89.6	611	1,080
<b>1 in 2 yr. RP (50% AEP)</b>	46.3	480	932

\*Note: Return periods relate to the storm impacting the entire frontage (Sections A to H), return periods are not specific to individual sections.

Source: Mott MacDonald, 2021

## 5 Inundation modelling

The TuFlow model from the OBC was provided to form the basis of the inundation modelling for assessment in this study. This section details the updates made to the modelling following Mott MacDonald's review and presents the results of the models run.

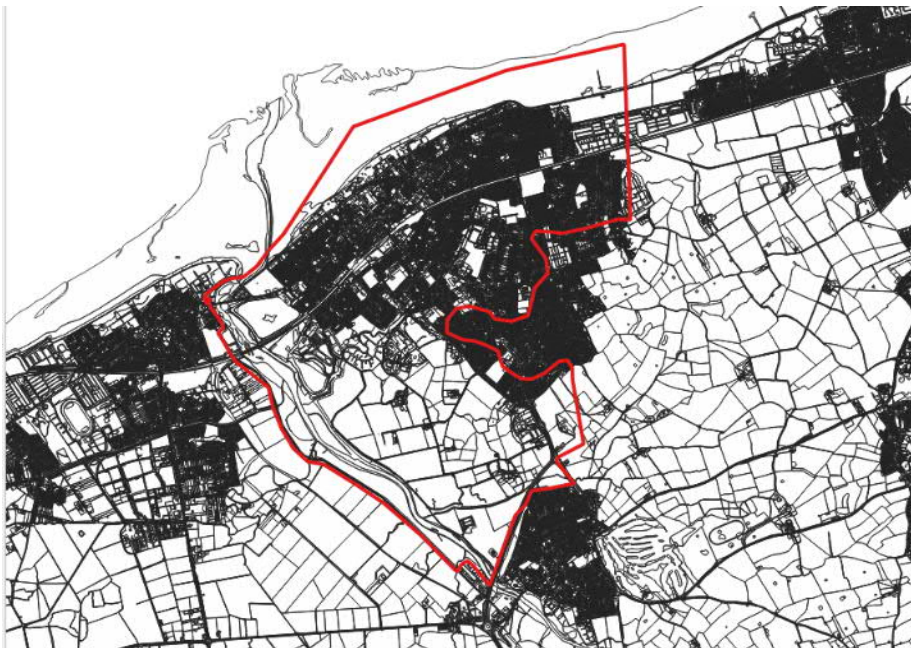
### 5.1 Updates to the TuFlow model

The hydraulic model as received (2018 Baseline) did not run successfully and was required to be updated to be used for the Scheme. The baseline model, or the adjustments to the topographic data used for the modelling, was not adjusted.

The following updates were made:

- The model extent was reduced to model Rhyl only, reducing the coastline modelled from 22km to 8km. The final extent of the model is shown in Figure 5.1
- The River Clwyd channel was originally modelled in 1D; however, this was causing issues to run the model. Therefore, the River Clwyd 1D model was removed and replaced with a 2D only component using the cross-sections from the 1D model as the basis of the channel representation.
- The banks along the River Clwyd were updated to match the LiDAR locations.
- A single flood tide event was modelled, with the start times and end times adjusted, plus the initial water level adjusted to suit.
- The tidal boundary and model extent were brought closer to the shoreline to minimise water surface modelled and reduce instabilities.
- Added 'walls' in Section D to represent the high walls of the Rhyl Events Arena which are used to block portions of the overtopping water flow.

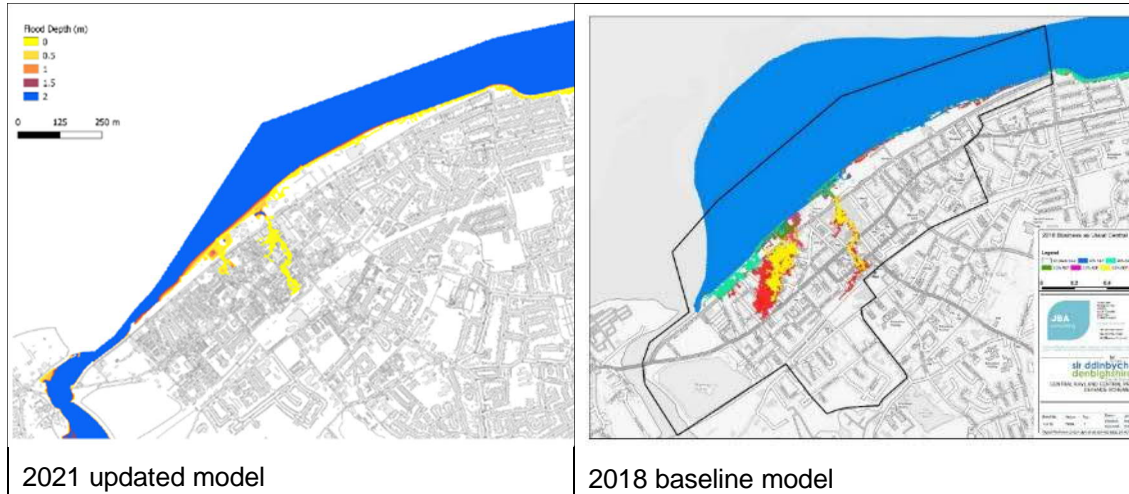
**Figure 5.1: Updated Model Extent (Red Line)**



Source: Mott MacDonald, 2021

To confirm that the hydraulic model was providing the same results for the Rhyl area as the original baseline model, the baseline input values were used to simulate the 0.5% AEP 2018 Business As Usual JBA model results as shown in Figure 2. The comparison of the yellow marked results indicate that the model largely predicted the same flood extents. It is noted that at the golf course to the east of Rhyl water levels were increased by 0.3m, probably due to the reduced hinterland extent. This increase in water level does not affect the water levels at Rhyl itself caused by overtopping.

**Figure 5.2: Flood extent comparison (yellow extents)**



Source: Mott MacDonald, 2021 and JBA

Following the initial comparison tests, further adjustments to wave overtopping and bank wall locations were changed to match the updated do something options. To ensure that accurate overtopping volumes were applied some secondary overtopping lines (where the profiles differed significantly from the typical section) were introduced in Sections C and D. The wave overtopping input locations are shown in Figure 5.3.

**Figure 5.3: Wave overtopping input locations**



Source: Mott MacDonald, 2021

## 5.2 Inundation model inputs

A single wave and water level combination is required for the inundation modelling with the wave overtopping calculated as a time variable discharge owing to tidal influences. Both the tidal boundary and the calculated overtopping hydrograph associated with the same storm are applied as boundary conditions in the TuFlow model.

The approach to produce the inundation wave overtopping input was as follows:

1. Selection of the storm that produces the desired return period; this would be the wave-water level combination to be used for the inundation modelling.
2. For each wave condition and section, calculate the overtopping rate at 15 minute intervals with a tidally varying water level, applying depth limitation on waves as necessary due to the changing water level.

For Section D where overflowing of the promenade occurs owing to the low crest level, overflow rates were calculated using the methods contained in EurOtop (2018). These were applied in combination with the tidal level to represent the momentum that wave action would have in driving water over the promenade surface. Exclusion of the overflow calculation and reliance on only the tidal boundary to drive overflow (which would ignore wave momentum) would be likely to underestimate the flow of water over the promenade.

## 5.3 Inundation model results

### 5.3.1 Overtopping rate tests

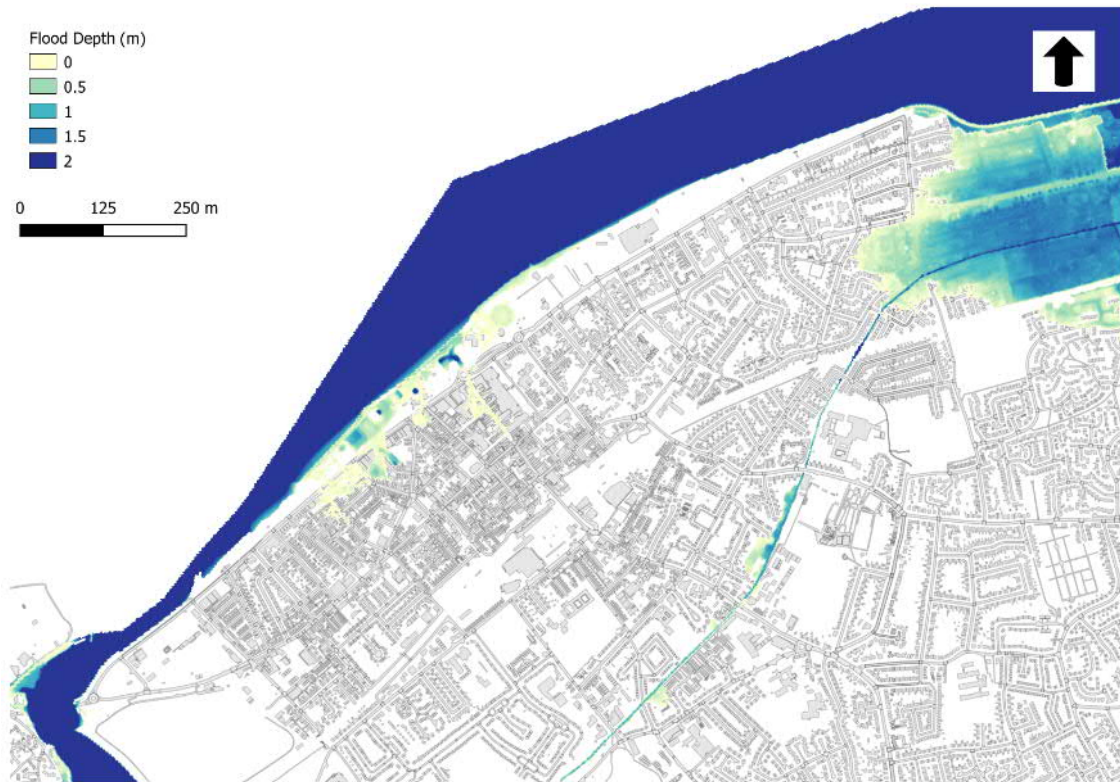
To understand the impact of overtopping rates within the proposed Section E–H and to set the limit of overtopping a range of test runs were conducted, refer to Section 3.1. Starting at 15 l/s/m, which was informed by rates used in initial test runs, overtopping rates of 15 l/s/m, 10 l/s/m, and 5 l/s/m were conservatively tested. These runs helped to determine that the risk of onset of substantial flooding within Rhyl from overtopping contributions from Section E–H occurred when the overtopping rate within that section exceeds approximately 10 l/s/m which was subsequently adopted as the limiting design criteria.

**Figure 5.4: Section E-H Only – 15l/s/m Overtopping Rate**



Source: Mott MacDonald, 2021

**Figure 5.5: Section E-H Only – 10l/s/m Overtopping Rate**



Source: Mott MacDonald, 2021

**Figure 5.6: Section E-H Only – 5l/s/m Overtopping Rate**



Source: Mott MacDonald, 2021

### 5.3.2 Section A – Existing Defences

This shows the inundation modelling for Section A in 2120. For overtopping rate refer to the 0.5%AEP event in Table 4.5. No flooding is shown originating from this section under the 0.5%AEP event in 2120. Note that flooding to the east of this section is caused by overtopping rates assessed in the original OBC water applied in the original TuFlow model.

The overtopping values (and therefore inundation) in Section A would not be substantially reduced through the implementation of the OBC scheme. But as illustrated this would not result in inundation within Rhyl.

**Figure 5.7: Section A Only – 0.5%AEP in 2120**



Source: Mott MacDonald, 2021

### 5.3.3 Section B – Existing Defences

This shows the inundation modelling for Section B in 2120. For overtopping rates refer to the 0.5%AEP event in Table 4.5. No flooding is shown originating from this section under the 0.5%AEP event in 2120. Note that flooding to the east of this section is caused by overtopping rates assessed in the original OBC water applied in the original TuFlow model.

The overtopping values (and therefore inundation) in Section B would not be substantially reduced through the implementation of the OBC scheme. But as illustrated this would not result in inundation within Rhyl.

**Figure 5.8: Section B Only – 0.5%AEP in 2120**



Source: Mott MacDonald, 2021

### 5.3.4 Section C – Existing Defences

This shows the inundation modelling for Section C in 2120. For overtopping rates refer to the 0.5%AEP event in Table 4.5. No flooding is shown originating from this section under the 0.5%AEP event in 2120 within Rhyl, though it is noted that some assets immediately on the promenade may be at localised risk. Note that flooding to the east of this section is caused by overtopping rates assessed in the original OBC water applied in the original TuFlow model.

The overtopping values (and therefore inundation) in Section C would not be substantially reduced through the implementation of the OBC scheme. But as illustrated this would not result in inundation within Rhyl.

**Figure 5.9: Section C Only – 0.5%AEP in 2120**



Source: Mott MacDonald, 2021

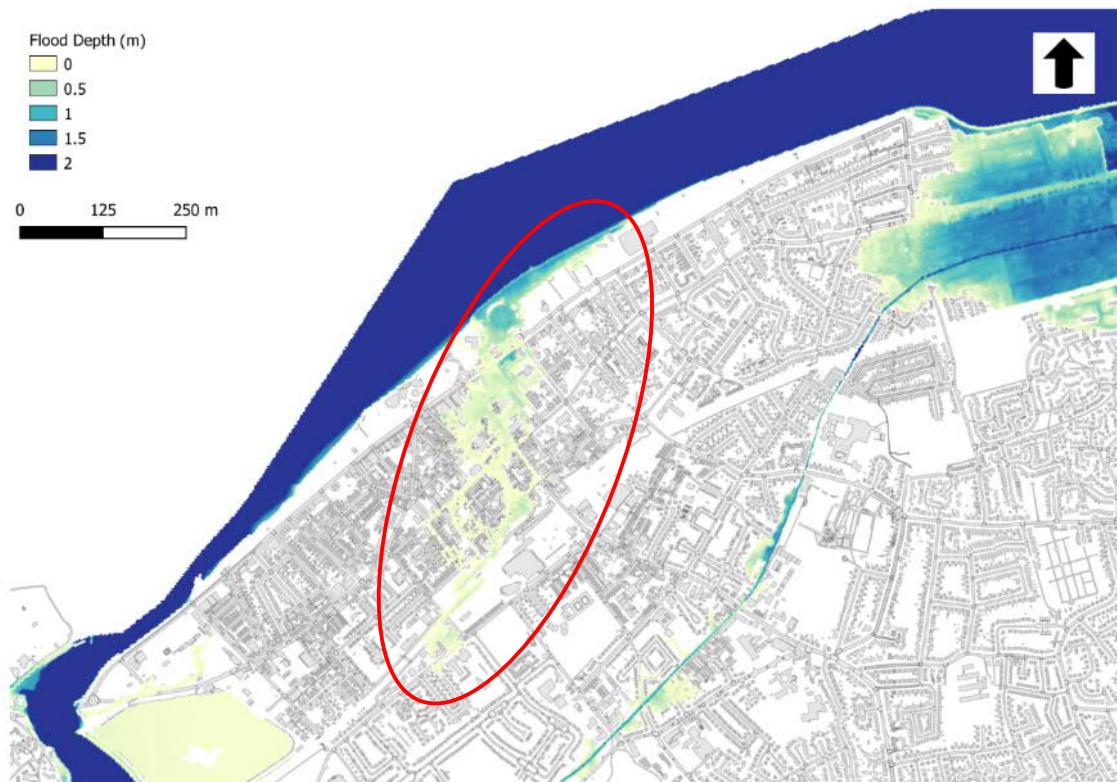
### 5.3.5 Section D – Existing Defences

This shows the inundation modelling for Section D in 2120. For overtopping rates refer to the 0.5%AEP event in Table 4.5.

The inundation modelling shows that flooding is caused by overtopping water passing over and around the RhyI Event Arena area and onwards into RhyI town under the 0.5%AEP event in 2120.

Implementation of the current OBC scheme would not produce a significant reduction in overtopping rates and thus inundation extents from overtopping along Section D. The risk of inundation will increase over time as climate change occurs. Further modelling of overtopping rates in 2070 has been undertaken to help inform on the discussion on if and when overtopping intervention is required, refer to Section 4.3.6.

**Figure 5.10: Section D Only – 0.5%AEP in 2120**



Source: Mott MacDonald, 2021

### 5.3.6 Section E-H – Proposed Defences

This shows the inundation modelling for Section E-H in 2120 for the recommended geometry variation (3, refer to Section 4.3.3). For overtopping rates refer to the 0.5%AEP event in Table 4.9. The inundation modelling shows that for this geometry significant flooding of assets within Rhyl is avoided under the 0.5%AEP event in 2120. Some flooding is noted in Section D; however, this is because of high tidal levels.

Implementation of the updated geometry to the OBC scheme would produce significant reductions in overtopping rates and thus inundation extents from overtopping along Section E-H. Inundation modelling shows no significant flooding in the future and therefore present day rates which are significantly lower would be insufficient to cause observable flooding beyond the immediate promenade.

**Figure 5.11: Section E-H (Geometry Variation 3) Only – 0.5%AEP in 2120**



Source: Mott MacDonald, 2021

## 6 Outcomes

This section presents concise outcomes against the aims of the overtopping and inundation modelling defined in Sections 3 and 4.

### 6.1 Validation of the performance of the concept designs

For sections A to D where no works are proposed to be implemented to significantly improve overtopping performance, the overtopping results presented herein confirm that for Section A to C the contribution from overtopping to inundation within Rhyl is minimal and therefore substantial works are not required at present. With future climate change significant storms do increase overtopping rates considerably. Management and maintenance of assets immediately on the promenade and just behind may be required should significant storms occur.

The overtopping analysis for Section D has shown that when the overtopping is modelled there is a significant risk that flood water may pass through the section and cause inundation of assets within Rhyl. This outcome should be considered further, and the risk assessed to determine if the proposed scheme should consider increasing flood protection measures in this area.

Overtopping of the existing defences within sections E to H are demonstrated to be sufficient to cause flooding within Rhyl. The recommendation by the OBC to provide improvements to the overtopping performance to protect assets behind the promenade is valid. Furthermore, testing of a revised defence structure geometry consisting of a stepped revetment and rear wall shows that suitably limited overtopping rates can be practically achieved. Refer further to Section 6.2

### 6.2 Confirmation of the design geometry in Section E-H

The overtopping and inundation assessment has demonstrated that an overtopping rate of less than 10 l/s/m is required to prevent significant inundation of assets within Rhyl. Subsequent testing of structure geometries (considering known buildability issues at the time) confirm the following structure parameters are recommended to meet the 0.5% AEP standard of protection in 2120:

- Stepped Revetment with a slope of 1:2.
- Revetment Crest at +7.85mOD.
- Promenade Width of 10.1m (minimum).
- Rear wall crest at +9.25mOD, 1.4m above revetment crest.
- The rear wall will require inclusion of a compliant bullnose geometry

It should be noted that the selection of a 1:2 slope is driven by wave breaking limitations within the EurOtop guidance. Typically, shallower revetment slopes would be expected to deliver lower overtopping rates. Adoption of a slope shallower than 1:2 given the wave conditions encountered at Rhyl would require a derogation from the EurOtop guidance. 2D physical scale model testing would be required to prove the derogation is valid.

### 6.3 Outputs for the flood consequence Assessment

Flood maps have been created suitable for use in demonstrating that the scheme results in only localised flooding along the frontage in storms up to the 0.5%AEP event in 2120, except for Section D. With the implementation of the scheme the modelling shows that flooding to commercial and residential assets within Rhyl would not be expected.

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## A. OBC Overtopping and Inundation Review

1. Waves and the wave and water level combinations used as inputs for the OBC assessment were not accurate.
  - a. This issue was previously flagged by Mott MacDonald and subsequently waves were remodelled, and a revised Multi-Variate Analysis undertaken to determine the wave and water level combinations. The overtopping modelling utilises these revised combinations and therefore direct comparison between results presented herein with the previous OBC is invalid.
2. Unknown source of topographical/bathymetry survey data used. This affects the geometry of the BAU scenario as well as the toe level for all cases.
  - a. MM have utilised recent topography survey (PM SURVEYS UK Ltd, 2021) undertaken for the project.
3. The OBC methodology applied varying beach drawdown for each section and epoch. There is uncertainty over applied values. This affects the geometry of BAU scenario as well as the toe level for all cases.
  - a. MM have reviewed beach drawdown values applied and used the reassessed values for beach drawdown over the appraisal period.
4. Incorrect use of Peak Wave Period ( $T_p$ ) as Spectral Wave Period ( $T_{m-1,0}$ ) as input to the ANN ( $T_{m-1,0} = 1.1 * T_p$ ).
  - a. MM have utilised correct period parameter required in calculations.
5. Wave periods have been adjusted to prevent wave steepness exceeding 0.06. This affects wave and water level combinations where waves are breaking.
  - a. MM have modelled waves to the toe of the structure to account for changes in period due to wave breaking.
6. The OBC wave and water level combination sorting and selection process was applied before depth-limiting waves which can result in excluding cases with high overtopping.
  - a. MM have modelled waves to the toe of the structure to account for depth limitation of wave heights.
7. In cases where the geometry contains two walls (landward and seaward of the promenade), one wall is not taken into account (required assumption as only one wall can be implemented in EurOtop 2018 (Van der Meer, et al., 2018)). This is a conservative estimate.
  - a. MM have applied a different approach as set out in Section 3.3.1.4.
8. The OBC preferred option drawings illustrate a return wall (bullnose). However, the effect of a bullnose on the wall has not been considered in their overtopping assessment. This is a limitation of the ANN as it is not trained with data for bullnoses. However, provision of a bullnose would reduce overtopping from the results provided by the ANN.
  - a. MM have not utilised the ANN and are therefore able to apply reductions for the effects of a bullnose.
9. The way the waves were truncated by JBA resulted in cases with negative depths / wave heights for shallow depths. This creates issues with the ANN results, if they are not filtered to the ANN input validity range (Mott MacDonald identified that they were not).
  - a. Mott MacDonald has utilised the empirical equations approach to calculate overtopping rather than using the ANN approach used by JBA.

10. Even though JBA stated that the ANN results were “filtered out” that have error values  $E > 0.5$ , or input outside the ANN validity range, Mott MacDonald identified that this results were actually not removed. This makes the JBA overtopping results unreliable.
  - a. Mott MacDonald has utilised the empirical equations approach to calculate overtopping rather than using the ANN approach used by JBA.
11. For many of the results that pass checks on  $E$  and on input parameter validity (i.e. that input parameters are within the validity range prescribed by the ANN manual), the ANN raised a flag on a large number of the results indicating that “they present a configuration that is beyond the range of training of the NN tool. This may affect the predictions of  $q$ ” or indicating “they present a configuration that is far from the range of training of the NN tool. This may lead to wrong or meaningless predictions of  $q$ ”. This is identified in the resulting .log error files downloaded along with the results but is not acknowledged in the assessment of overtopping rates.

A spot check on a few of these confirmed that some of the results showed low (and in some cases zero) values of  $E$  (the error value from the ANN) and several showed no error on input parameter validity. These were considered as “valid” results in the assessment of JBA.

For S2 (preferred option) files that include option assessment, these also showcased many results listed in the .log file with the above issue.

  - a. Mott MacDonald has utilised the empirical equations approach to calculate overtopping rather than using the ANN approach used by JBA.
12. JBA schematized most of the defences geometry by assuming the promenade as a berm and the vertical walls as very steep slopes. There is uncertainty over the effects of this schematization, as this is likely inconsistent with the ANN training data.
  - a. Mott MacDonald has utilised the empirical equations approach to calculate overtopping rather than using the ANN approach used by JBA.
13. JBA stated in their Technical Note - Wave Overtopping (JBA Consulting, 2019) that a depth breaker parameter of 0.55 was used in their wave truncation to depth limit the waves. However, reviewing the Python scripts used in their assessment, a value of 0.6 was identified. There is uncertainty over which value was used. Additionally, breaker parameters can range up to 0.8 particularly in areas of more complex bathymetry.
  - a. MM have transformed waves to the toe using a numerical model to represent wave steepness and breaking more accurately. In the model the upper limit of the breaker parameter was set as 0.8.
14. JBA used a very onerous 5 l/s/m overtopping limit in 0.5% AEP plus Climate Change (2118) and made no checks against  $V_{max}$ .
  - a. The current project has revised this limit, see Section 3.1.



## Appendix 10.4 – WFD Assessment



# **Central Rhyl Coastal Defences**

Water Framework Directive (WFD) Assessment

September 2021



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# Central Rhyl Coastal Defences

Water Framework Directive (WFD) Assessment

September 2021



# Issue and Revision Record

Revision	Date	Originator	Checker	Approver	Description
P01	30/07/21	R. Booley	C. Postlethwaite C. Williams	S H Smith	First Issue for client comment
C01	03/09/21	A. Jones	C. Williams	N M Price	Published version with minor amendment following Balfour Beatty comments

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**Information class:** Standard

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# 1 Introduction

## 1.1 Project background

### 1.1.1 Overview

This Water Framework Directive Regulations (WFD) Assessment has been prepared by Mott MacDonald Ltd (Mott MacDonald) for Balfour Beatty Civil Engineering Ltd (BB) on behalf of Denbighshire County Council (DCC) for the construction of the Central Rhyl Coastal Defence Scheme (hereafter referred to as 'the Scheme').

The Scheme, located in the central area of Rhyl waterfront, Denbighshire, North Wales, is dependent on obtaining sufficient funding and comprises coastal defence works in the form of scour protection, stepped concrete revetment, promenade raising, sea wall works, access and promenade improvements, and associated tie-in works.

Rhyl is a Victorian coastal resort and the largest town in Denbighshire, with a well-established history as a tourist destination. There are ongoing and future plans to regenerate Rhyl to enhance its appeal to tourists and investors; however, the coastal defences for the Central Rhyl frontage are deteriorating and there is the risk that this integral asset for Rhyl's regeneration could be lost without intervention.

Historically, Central Rhyl has been protected from coastal flooding by hard engineered defence structures constructed from at least the 1900s. These comprise a series of stepped revetments, recurve walls and vertical walls in various states of repair. The existing defences no longer meet acceptable performance standards and are known to be deteriorating.

The existing promenade in the area of the Scheme can become inundated during storms, causing flooding of the promenade and surrounding areas. In the future this will be exacerbated by the effects of climate change, which is predicted to increase both the frequency and magnitude of flooding. The existing coastal defences and promenade require improvements to safeguard the area from overtopping with consideration given to the potential effects of climate change.

Subject to funding, this Scheme aims to improve the existing coastal defences to provide increased flood and erosion protection to people and property in Rhyl along with supporting local regeneration, achieving wider community benefits alongside flood risk management.

### 1.1.2 Scheme location

The Scheme is located along the coastal frontage of the town of Rhyl, North Wales. It extends from approximately 400m east of the mouth of the River Clwyd, to just west of Splash Point (and the start of the East Rhyl Coastal Defence Scheme, which was under construction at the time of writing this report). Figure 1.1 presents the approximate extent of the Scheme as outlined by the Central Rhyl Coastal Defences Outline Business Case (OBC) (DCC, 2019)<sup>1</sup>.

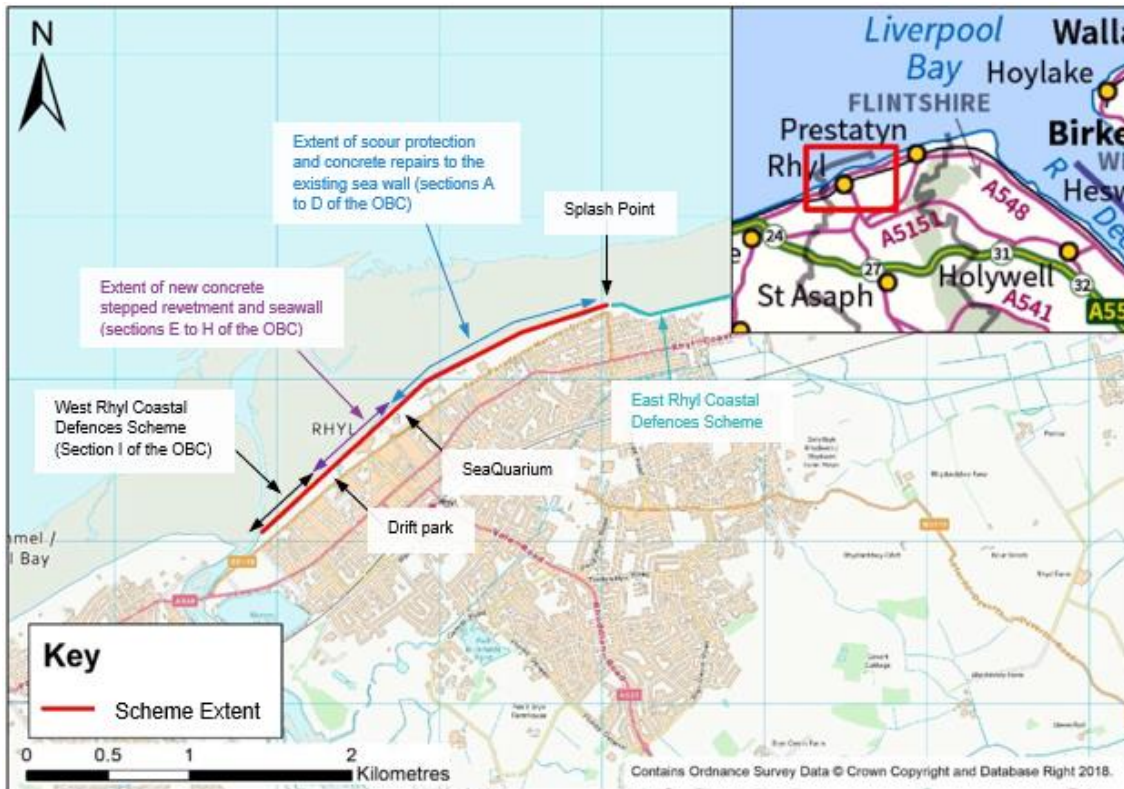
As part of the OBC, the Scheme was split into nine sections along its length: sections A – I, with Section A at the eastern end and Section I at the western end.

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<sup>1</sup> Denbighshire County Council, 2019. Central Rhyl Coastal Defence Scheme Outline Business Case.

The western end of the Scheme (i.e. west of Drift park and OBC Section I) is the location of the West Rhyl Coastal Defences Scheme. The West Rhyl Coastal Defences Scheme was constructed in 2015, and no further works are proposed at this location.

**Figure 1.1: Scheme location and sections as per the Central Rhyl Coastal Defences OBC**



Source: Central Rhyl Coastal Defence Scheme OBC, DCC, October 2019. Additional annotations by Mott MacDonald Ltd 2021

The national grid references for the western and eastern Scheme boundaries are (300223,381323) and (301998,382433) respectively. The Scheme is located along approximately 2.15km of coastline from the western end of the children’s play area near River Street in the west to Tynewydd Road in the east.

The area of permanent construction is located along the beach adjacent to and over the northern-most area of the promenade. In this area, the beach predominantly comprises intertidal soft sediment and sand with some intertidal gravel and cobbles, predominantly at the landward edge. There are a limited number of slipways for vehicular beach access.

The principal highways bounding the southern side of the promenade are West Parade for the western half of the Scheme area (west of the SeaQuarium) and East Parade becoming Marine Drive to the far east.

## 1.2 Existing defence condition

Historically, central Rhyl has been protected from coastal flooding by hard engineered defence structures. These comprise a series of stepped revetments, recurve walls and vertical walls in various states of repair. A sand beach and timber groyne field fronts the hard defences, offering

some protection to the toe of the vertical structures. Much of the frontage also has fixed or demountable secondary defences that are typically located to the rear of the promenade.

Much of the existing defences are in poor to very poor condition, with some sections effectively being life expired. In these areas numerous defects have been observed during surveys including weak/eroded concrete, signs of movement, failing repair work, cracking, delamination, additional loading, lack of reinforcement and absence of protective revetment. These defects have the potential to compromise the overall integrity of the defences in the future, resulting in increased coastal erosion and flood risk to Rhyl.

In addition, beach lowering processes were noted throughout the frontage, leaving the toe of the defence vulnerable to failure through scour and beach lowering in the short term.

### 1.3 Scheme proposals

The Scheme red line boundary can be seen in Appendix A of this report.

#### 1.3.1 Coastal Works

The Scheme is to include the following key coastal works:

- The construction of approximately 1.45km of scour protection and concrete repairs, from Splash Point to the SeaQuarium (sections A to D of the OBC, see Figure 1.1). The scour protection will consist of boulders being placed at the base of the existing coastal defence structures;
- The construction of approximately 750m of new concrete stepped revetment, from the SeaQuarium to opposite Drift Park (sections E to H of the OBC, see Figure 1.1);
- New pedestrian beach accesses through the proposed revetments to replace the existing ones and extension of existing beach accesses through the rock scour protection; and
- The extension of outfalls and protection works to existing outfalls that would be covered by the new revetment.

#### 1.3.2 Promenade works

The Scheme also includes the following works to the promenade:

- Raised promenade from the SeaQuarium to opposite Drift Park (sections E to H of the OBC, see Figure 1.1);
- New seawalls at the back of the promenade from the SeaQuarium to opposite Drift Park (sections E to H of the OBC, see Figure 1.1);
- New flood gates for access points through the new rear seawalls between the SeaQuarium and opposite Drift Park;
- New retaining structures where appropriate at the back of the promenade between the SeaQuarium and opposite Drift Park, to accommodate level differences between the raised promenade and adjacent land; and
- New ramps or steps to provide access to raised promenade between the SeaQuarium and opposite Drift Park.

#### 1.3.3 Management and maintenance

The Scheme includes future management and maintenance actions comprising:

- Inspection, maintenance and repair of coastal defence structures;

- Inspection, maintenance and repair of Promenade and sea wall;
- Inspection, maintenance and repair of accesses, drainage, highways, lighting and other minor assets;
- Management of wind-blown sand;
- Maintenance of landscaping; and
- Beach management.

#### **1.4 Purpose of WFD assessment**

Given the nature of the Scheme, the works require a WFD Assessment to determine the effects of the Scheme on ecological, hydromorphological and chemical quality and identify any potential impacts that could cause deterioration in the current status of the waterbody or could hinder the waterbody from meeting its WFD objectives in the future.

## 2 Legislation and assessment methodology

### 2.1 Legislative context

The WFD is originally European legislation which aims to protect and improve the water environment within member states of the EU. The WFD was transposed into UK law in 2003 by the Water Environment (Water Framework Directive) (England and Wales) Regulations 2003. Following Britain's exit from the EU, these Regulations were revoked and replaced with the Water Environment (Water Framework Directive) (England and Wales) Regulations 2017. Natural Resources Wales (NRW) is the statutory body responsible for implementing the WFD Regulations in Wales.

The WFD requires the identification and setting of objectives for protecting and improving 'waterbodies'. Waterbodies include rivers, streams, lakes, reservoirs, estuaries, coastal waters, canals and groundwaters. The standard objective is to achieve 'good status', or 'good potential' (if the waterbody is artificial or heavily modified). Further to this, the WFD must prevent deterioration in status for all waterbodies. The original target date for all waterbodies to achieve 'good status' was 2015; however, this can be extended to either 2021 or 2027. Good 'status' or 'potential' is designated based on the assessment of ecological and chemical components in surface waters. Ecological status consists of biological quality elements, physico-chemical supporting elements and hydromorphological supporting conditions. For groundwater, status consists of quantitative and qualitative elements. The WFD environmental objectives are outlined in Table 2.1.

**Table 2.1: WFD environmental objectives**

Objectives (from Article 4 of WFD)	Reference and description
4.1(a)(i)	WFD1 - Member States shall implement the necessary measures to prevent deterioration of the status of all bodies of surface water.
4.1(a)(ii)	WFD2 - Member States shall protect, enhance and restore all bodies of surface water, subject to the application of subparagraph (iii) for artificial and heavily modified bodies of water, with the aim of achieving good surface water status by 2015.
4.1(a)(iii)	WFD3 - Member States shall protect and enhance all artificial and heavily modified bodies of water, with the aim of achieving good ecological potential (GEP) and good surface water chemical status by 2015.
4.1(a)(iv)	WFD4 - Progressively reduce pollution from priority substances and cease or phasing out emissions, discharges and losses of priority hazardous substances.
4.1(b)(i)	WFD5 - Prevent Deterioration in Status and prevent or limit input of pollutants to groundwater.

Source: Water Framework Directive 2000/60/EC<sup>2</sup>

River Basin Management Plans (RBMPs) set out how organisations, stakeholders and communities will work together to improve the water environment. RBMPs report the baseline status of waterbodies within each River Basin District (RBD), corresponding to WFD reporting

<sup>2</sup> Directive 2000/60/EC of the European Parliament and the Council of 23 October 2000 establishing a framework for Community action in the field of water policy (2000), Accessed online: [https://eur-lex.europa.eu/resource.html?uri=cellar:5c835afb-2ec6-4577-bdf8-756d3d694eeb.0004.02/DOC\\_1&format=PDF](https://eur-lex.europa.eu/resource.html?uri=cellar:5c835afb-2ec6-4577-bdf8-756d3d694eeb.0004.02/DOC_1&format=PDF)

cycles. The first RBMPs were published in 2009 for RBMP Cycle 1 and were updated for RBMP Cycle 2 in 2015. RBMPs are reviewed and updated every six years so a new RBMP is due to be published in 2021. The Scheme is within the Western Wales RBMP.

Waterbodies can be classified as artificial (waterbodies created in a location where no waterbody existed before), or heavily modified (waterbodies where the natural conditions have been substantially altered for human benefit and have substantially changed in character).

Mitigation measures are identified to achieve 'Good Ecological Potential' for Artificial Waterbodies (AWB) or Heavily Modified Waterbodies (HMWB), in lieu of the ability to achieve 'Good Ecological status' seen in natural waterbodies. Mitigation measures are practicable steps that can be taken to reverse or offset adverse impacts from human activities such as impoundments for water resources, power generation or structures that provide flood defence.

## 2.2 Guidance

The most applicable guidance for WFD assessment in Wales is:

- Guidance for assessing activities and projects for compliance with the Water Framework Directive, *Natural Resources Wales*, 2018.
- NRW OGN 072: Complying with the Water Framework Directive Regulations 2017: how to assess and appraise projects and activities, *Natural Resources Wales*, Jan 2021.

## 2.3 Assessment methodology

This WFD assessment has been informed by a desk-based study based on publicly available data and follows the guidance from government documents outlined in Section 2.2.

### 2.3.1 WFD baseline

The WFD Baseline will identify the WFD waterbodies including:

- River waterbodies;
- Groundwater catchment waterbodies; and,
- Designated sites / Protected Areas.

This baseline has been informed by a desk-based analysis of publicly available information using sources as seen in Table 2.2.

**Table 2.2: Desk-based study sources**

Source	Link	Description
Natural Resources Wales - Watch Water Wales	<a href="http://naturalresourceswales.gov.uk">Water Watch Wales (naturalresourceswales.gov.uk)</a>	Provides information on the WFD waterbodies.
Natural Resources Wales Portal - Lle Map	<a href="http://gov.wales">Lle - Map (gov.wales)</a>	Provides information on the WFD waterbodies, potential environmental constraints, Protected Areas and designated sites.
Natural Resources Wales - River Basin Management Plans (RBMPs)	<a href="#">Natural Resources Wales / River Basin Management Plans 2015 - 2021</a>	Provides information on the designated River Basin Management Plans; the progress reports and the objectives.
National Library of Scotland – Map Images	<a href="http://nls.uk">Side by side georeferenced maps viewer - Map images - National Library of Scotland (nls.uk)</a>	Comparison for historic Ordnance Survey (OS) maps and present-day mapping, allowing long-term changes in river morphology and planform to be identified.
Department of Environment, Food & Rural Affairs and (Defra) – MAGIC Map	<a href="http://defra.gov.uk">MAGIC (defra.gov.uk)</a>	Provides information on the WFD waterbodies, potential environmental constraints, Protected Areas and designated sites.

Due to the nature and implied connectivity of the coastal environment, waterbodies within a 5km radius have been assessed within the WFD screening.

### 2.3.2 Stage 1: WFD screening assessment

The WFD Screening Assessment determines whether the Scheme needs to be screened in or out for further assessment. This is based on the NRW WFD Guidance report<sup>3</sup>, as summarised in Appendix B.

If the proposed works are not listed in Table 5 or Appendix 2 of the 2018 NRW WFD Guidance report, the Scheme has to be assessed against the WFD risk screening thresholds (Section 2.3.2.1).

#### 2.3.2.1 WFD Risk screening thresholds

WFD risk screening thresholds help determine if the activity poses a risk to the delivery of WFD objectives and indicates whether further assessment is needed.

The thresholds look at the type of physical works proposed and the length of waterbody that might be affected. The combination of these two factors provides a red, amber or green ‘traffic light’ system that indicates the level of potential WFD Risk (Table 2.3).

<sup>3</sup> Guidance for assessing activities and projects for compliance with the Water Framework Directive, *Natural Resources Wales*, 2018.

**Table 2.3: WFD risk traffic light system**

Colour	Risk to delivery of WFD objectives	Need for further assessment
Green	None - Low <sup>4</sup>	No further assessment required.
Amber	Medium – High, depending on the sensitivity of the location.	Yes – further assessment required. These activities should be sent for review by area geomorphology, fisheries and biodiversity experts.
Red	High	Yes – further assessment required. These activities should be sent for review by area geomorphology, fisheries and biodiversity experts.

Source: NRW WFD Guidance report

### 2.3.3 Stage 2: WFD scoping assessment

This stage aims to identify elements within waterbodies which may be impacted as a result of the activity. As part of scoping, the focus is on identifying components of the Scheme that may have the potential to cause and impact and the quality elements (Chemical status and Ecological status) potentially impacted.

Waterbodies can be scoped out at this stage if it can be robustly demonstrated that there will be no impacts. The process of the Scoping assessment is summarised in Appendix B.

### 2.3.4 Stage 3: detailed compliance assessment

This stage considers the potential impacts of an activity, identified ways to avoid or minimise impacts, and concludes if the activity may prevent any quality element within any waterbody achieving good status/potential or may cause deterioration.

Mitigation measures and recommendations are proposed in this section to ensure the Scheme is WFD compliant.

<sup>4</sup> Green activities still need to be screened for high status, designations and wider environment, and activities must still be undertaken in line with pollution prevention and invasive species best practise guidance.

## 3 Stage 1: WFD screening

### 3.1 Scope of works at risk of affecting the water environment

The following elements of the Scheme have the potential to impact the water environment and have been screened into the assessment.

- Construction Works:
  - Construction of rock scour protection and concrete repairs;
  - Construction of new concrete revetment and associated beach access points;
  - Extension of outfalls and protection works to existing outfalls; and
  - Construction of raised promenade and associated retaining structures.
- Maintenance / Operational:
  - Inspection, maintenance and repair of coastal defence structures;
  - Inspection, maintenance and repair of accesses, drainage, highways, lighting and other minor assets;
  - Management of wind-blown sand; and
  - Beach management.

### 3.2 WFD screening assessment

By applying the methodology outlined in Appendix B, this section will identify the waterbodies that may be impacted by the screened in activities, as such, these waterbodies will require further assessment in Stage 3. This section will identify the activities that **may cause deterioration** or **prevent a waterbody from meeting its objectives**, as such, these activities will be screened-in and assessed further.

The Scheme's proposed activities are not listed within Appendix A2.2 of the NRW Guidance, indicating a potential risk and therefore it is considered all of the proposed activities require further assessment and should be considered at Stage 3.

## 4 Stage 2: WFD scoping assessment

This stage identifies waterbodies and WFD quality elements that are at risk from the Scheme's proposed activities and require further assessment.

### 4.1 WFD waterbody Identification

The Scheme is located on the coast of North Wales, adjacent to the mouth of Afon Clwyd. The following WFD waterbodies have been identified within close proximity of the Scheme:

- North Wales Coastal Waterbody (GB641011650000); Scheme within waterbody;
- Clwyd Permo-Triassic Sandstone Groundwater Waterbody (GB41001G202100); Scheme within waterbody; and,
- Clwyd Transitional Waterbody (GB541006608000); Scheme adjacent to waterbody.

The Scheme is not located within a river waterbody catchment area. The closest river waterbody catchments within 5km of the Scheme are;

- Gele (GB110066059980); approximately 0.91km west of the Scheme, separated by Afon Clwyd;
- Glanfyddion Cut (GB110066059990); approximately 2.9km south; and,
- Pont Robin Cut (Bodelwyddan)(GB110066059970); approximately 3.1km south, separated by Afon Clwyd.

### 4.2 WFD scoping assessment

By applying the methodology outlined in Appendix B, this section will identify the waterbodies that may be impacted by the screened in activities, as such, these waterbodies will require further assessment in Stage 3.

The Scheme's proposed activities are not listed within Appendix A2.2 of the NRW Guidance, indicating a potential risk and therefore Table 4.1: uses expert judgement (nature of proposed activities and distance to Scheme) to determine which WFD waterbodies have been screened-in or -out of the assessment.

**Table 4.1: Screening outcomes**

Waterbody name	Waterbody ID	Reason	Scoping outcome
<b>Coastal Waterbodies</b>			
North Wales	GB641011650000	The Scheme is within this waterbody. The Scheme will change the morphology of the area to prevent the risk of flooding, as such there is a potential for the Scheme to impact the waterbody's objectives during construction.	<b>Scoped - in</b>
<b>Groundwater Bodies</b>			
Clwyd Permo-Triassic Sandstone	GB41001G202100	The Scheme does not involve groundwater abstraction. Any piling works are anticipated to be along the coastline on the edge of the waterbody catchment and it is not considered for the works to alter flow paths, create pollution pathways or for the waterbody's objectives to be impacted.	Scoped-out
<b>Transitional Waterbodies</b>			
Clwyd	GB541006608000	The waterbody is adjacent to the western border of the Scheme and, as such, has the potential to impact the waterbody's objectives during construction.	<b>Scoped - in</b>
<b>River Waterbodies</b>			
Rhyl Cut	GB110066060000	The Scheme is approximately 1.5km west of this waterbody. The Scheme is not immediately connected to this waterbody therefore, it is not anticipated for the waterbody's objectives to be impacted by the Scheme.	Scoped-out
Gele	GB110066059980	The Scheme is approximately 1.8km west of this waterbody. The Scheme is not immediately connected to this waterbody therefore, it is not anticipated for the waterbody's objectives to be impacted by the Scheme.	Scoped-out
Glanfyddion Cut	GB110066059990	The Scheme is approximately 2.9km west of this waterbody. The Scheme is not immediately connected to this waterbody therefore, it is not anticipated for the waterbody's objectives to be impacted by the Scheme.	Scoped-out

### 4.3 WFD Baseline of scoped-in waterbodies

A visual representation of the identified WFD waterbodies is provided in Figure 4.1, and a summary of the Baseline WFD status of the waterbodies is provided in Table 4.2:.



**Table 4.2: Summary of scoped-in WFD baseline status<sup>5</sup>**

Waterbody parameters	Waterbody name	
	North Wales	Clwyd
Waterbody ID	GB641011650000	GB541006608000
Waterbody Type	Coastal	Transitional
Waterbody Area (km <sup>2</sup> )	146.278	0.644
Distance from Scheme	0m (within)	0m (adjacent)
Hydromorphological Designation	Heavily Modified	Heavily Modified
Overall Status	Moderate	Moderate
Chemical Status	Fail	Good
Dissolved Inorganic Nitrogen	Good	Unknown
Dissolved Oxygen	High	Unknown
Annex 10 chemicals, priority hazardous substances and other pollutants	Fail	Good
Ecological Status	Moderate	Moderate
Invertebrates	Good	Unknown
Phytoplankton	High	Unknown
<b>Waterbody Objectives and Measures (2017)</b>		
Failing WFD elements	Dissolved Inorganic Nitrogen, Mercury	Angiosperm, Mitigation Measures Assessment, Saltmarsh
Overall Status	Good by 2021	Good by 2027
Chemical Status	Good by 2021	Good by 2015
Ecological Status	Good by 2021	Good by 2027

Source: Water Watch Wales Cycle 2, 2015

#### 4.4 Protected areas

Wider environmental legislation must be considered when assessing the compliance of proposals under WFD. Proposals cannot be WFD compliant if they adversely affect the integrity of protected areas; WFD aims to achieve relevant standards or objectives for these sites.

Table 4.3: identifies the Protected Areas within 2km of the Scheme to be considered within the Detailed Assessment, a visual representation of the Protected Areas is provided in Appendix C.

No Sites of Scientific Interest (SSSI), Special Areas of Conservations (SAC), Ramsar sites or local designations were identified within 2km of the Scheme.

Nitrate Vulnerable Zones (NVZ) have been designated within the RBMPs to identify areas where nitrate concentrations in waterbodies are high or increasing, or waterbodies are, or may become, eutrophic due to agricultural nitrate pollution. NVZ 135 has been identified to the south

<sup>5</sup> Unknown: No information was provided in the Water Watch Wales Classification spreadsheet.

of the Scheme. It is not anticipated for the Scheme to create new/or impact groundwater pathways, therefore NVZs have not been considered further.

**Table 4.3: WFD Protected areas identified**

Protected Area type	Name	Distance from Scheme	Associated WFD Catchments
<b>Special Protection Areas (SPA)</b>	Liverpool Bay / Bae Lerpwl	1km north	North Wales (GB64101165000)
<b>Bathing Waters</b>	Rhyl East	0m (within)	North Wales (GB64101165000)
	Marine Lake, Rhyl	0.4km south	Clwyd (GB541006608000)
	Rhyl	1km north	North Wales (GB64101165000)

Source: Water Watch Wales, NRW

#### 4.4.1 Sensitive Habitats

The following 'WFD sensitive habitats' have been identified within 500m of the Scheme:

- Higher Sensitivity Habitats:
  - Mussel Beds (A1.22, A2.72, A5.62, A4.24, A3.361); and,
  - Saltmarsh (A2.5).
- Lower Sensitivity Habitats:
  - Gravel & Cobbles (A2.1, A5.1);
  - Intertidal Soft Sediment (A2.2, A2.3, A2.4);
  - Subtidal Soft Sediment (A5.2, A5.3, A5.4); and,
  - Rocky shore (A1).

*Sabellaria alveolate* Reefs have been identified within 100m of the Scheme, with two patches of the habitat considered large enough to form the reef biotope (a habitat of principle importance), as such, this habitat is to be considered within this WFD assessment.

#### 4.5 WFD Elements Scoping Assessment

During construction, the Scheme may present potential risks to a number of WFD quality elements. These include:

- Hydromorphology;
- Biology – habitats;
- Biology – fish;
- Protected areas;
- Water quality; and,
- Invasive and non-native species (INNS).

Table 4.4: and Table 4.5 details the potential risk from the Scheme to each of these WFD quality elements in the scoped in waterbodies (North Wales Coastal Waterbody and Clwyd

Transitional Waterbody, respectively) and determines which area at risk and require to be progressed to Stage 3.

**Table 4.4: Scoping in/out of WFD quality elements for North Wales coastal waterbody**

WFD Quality Element	Consideration of Scheme activity	Potential risk?	Stage 3 Assessment required?
Hydromorphology	Could impact on the hydromorphology (for example morphology or tidal patterns) of a water body at high status?	No. Overall status is Moderate	X
	Could significantly impact the hydromorphology of any water body (including morphological conditions and tidal patterns)?	No. The Scheme has the potential to alter the morphological conditions along the coastal frontage, however, this is anticipated to be locally. Given the size of the waterbody in comparison to the footprint of the Scheme, it is not anticipated for the change in morphology to be significant in terms of the waterbody as a whole. In addition, there is extensive coastal protection along the frontage of the coastal waterbody, therefore, the Scheme is considered to be relatively minor in relation to the entire waterbody.	X
	Physical footprint is greater than 1% of surface area of water body or greater than 0.5km <sup>2</sup> ?	No. The area within the redline boundary is 0.43km <sup>2</sup> , and the area of permanent construction is anticipated to be approximately 0.24km <sup>2</sup> . These areas are 0.29% and 0.16% of the waterbody, given the size of the waterbody in comparison to the footprint of the Scheme, it is not anticipated for the Scheme size to be significant in terms of the waterbody as a whole.	X
	Is in a water body that is heavily modified for the same use as the activity?	Yes. The waterbody is heavily modified for coastal protection.	✓
Priority Habitats and Species	Physical footprint is greater than 1% of surface area of water body or greater than 0.5km <sup>2</sup>	Yes. The Scheme is located along approximately 2.15km of the coastline. However, given the size of the waterbody in comparison to the footprint of the Scheme, it is not anticipated for the Scheme size to be significant in terms of the waterbody as a whole.	X
	Is within 500m of any higher sensitivity habitat?	Yes. There are multiple clusters of Mussel Beds and <i>Sabellaria alveolate</i> Reef within 500m of the Scheme.	✓
	1% or more of any lower sensitivity habitat?	Yes. The Scheme covers more than 1% of lower sensitivity habitats. However, due to the vast coverage of the habitat across the coastal frontage and within the entire waterbody, it is considered for the Scheme be relatively minor in relation to the entire waterbody.	X
<b>Biology - fish</b>	Will impact on normal fish behaviour like movement, migration, spawning; species composition and abundance; or mechanical injury?	No. No physical barriers, chemical changes, habitat changes, significant changes to water quality or quantity, no changes to depth or flow is anticipated to impact fish behaviour.	X
	Is in a transitional water body and could affect fish?	No. Coastal Waterbody	X
	Is outside of a transitional water body and could impact upon migratory fish?	No. No physical barriers to impact migratory fish.	X

WFD Quality Element	Consideration of Scheme activity	Potential risk?	Stage 3 Assessment required?
<b>Water Quality</b>	Could affect water clarity, temperature, salinity, oxygen levels, nutrients or microbial patterns continuously for longer than a spring neap tidal cycle (about 14 days)	No. The works within the inter tidal zone will be undertaken at low tide, minimising the potential impact on water quality. I	X
	Is in a water body with a phytoplankton status of moderate, poor or bad	Yes. Phytoplankton status is Moderate	✓
	Is in a water body with a history of harmful algae	No.	X
	Puts water quality at risk from your activity through the use, release or disturbance of chemicals	No. Release or disturbance of chemicals is not anticipated.	X
<b>Protected Areas</b>	Within 2km of any WFD protected area	Yes. The Scheme is within a Bathing Water Area, and a SPA is within 2km.	✓
<b>INNS</b>	Introduce or spread INNS (including materials or equipment that have come from, had use in or travelled from other water bodies; or activities that help spread INNS).	No. There is a potential risk for the introduction of INNS through material and plant activity into the adjacent waterbody, however, mitigation measures will be included in the Construction Environment Management Plan. Therefore, the risk of INNS can be scoped out.	X

**Table 4.5: Scoping in/out of WFD quality elements for Clwyd transitional waterbody**

WFD Quality Element	Consideration of Scheme activity	Potential risk?	Stage 3 Assessment required?
<b>Hydromorphology</b>	Could impact on the hydromorphology (for example morphology or tidal patterns) of a water body at high status?	No. Overall status is Moderate	X
	Could significantly impact the hydromorphology of any water body (including morphological conditions and tidal patterns)?	No. The Scheme is not anticipated to impact the hydromorphology of the transitional waterbody.	X
	Physical footprint is greater than 1% of surface area of water body or greater than 0.5km <sup>2</sup> ?	No. The Scheme is located adjacent to this waterbody with no works within the waterbody.	X
	Is in a water body that is heavily modified for the same use as the activity?	No. The Scheme is adjacent to this waterbody and is not anticipated to be directly impacted by the Scheme.	X
<b>Priority Habitats and Species</b>	Physical footprint is greater than 1% of surface area of water body or greater than 0.5km <sup>2</sup>	Yes. The Scheme is located along approximately 2.15km of the coastline. However, given the size of the waterbody in comparison to the footprint of the Scheme, it is not anticipated for the Scheme size to be significant in terms of the waterbody as a whole.	X
	Is within 500m of any higher sensitivity habitat?	Yes. There are patches of Saltmarshes within 500m of the Scheme, however, the Scheme is not anticipated to directly impact these habitats or for the water quality to change to indirectly impact these habitats. Providing standard best practice measures are adhered to, this element is not considered further.	X
	1% or more of any lower sensitivity habitat?	No. The Scheme does not cover any lower sensitivity habitats within this waterbody as the proposed works are adjacent to this waterbody.	X
<b>Biology - fish</b>	Will impact on normal fish behaviour like movement, migration, spawning; species composition and abundance; or mechanical injury?	No. The Scheme is adjacent to this waterbody and is not anticipated to directly impact fish behaviour within the waterbody.	X
	Is in a transitional water body and could affect fish?	No. The Scheme is adjacent to this waterbody and is not anticipated to directly impact fish within the waterbody.	X
	Is outside of a transitional water body and could impact upon migratory fish?	No. The Scheme is adjacent to this waterbody and is not anticipated to directly impact fish migration within the waterbody.	X
<b>Water Quality</b>	Could affect water clarity, temperature, salinity, oxygen levels, nutrients or microbial patterns continuously for longer	No. The majority of the works will be undertaken at low tide in the adjacent waterbody, minimising the potential impact on the water quality. If marine vessels are required for the works, there is a potential for an increase in turbidity levels in the surrounding waterbodies, however,	X

WFD Quality Element	Consideration of Scheme activity	Potential risk?	Stage 3 Assessment required?
	than a spring neap tidal cycle (about 14 days)	this would not be <b>continuous</b> , and any impact would be temporary and limited to <b>the</b> construction phase.	
	Is in a water body with a phytoplankton status of moderate, poor or bad	No. Phytoplankton status is Unknown	X
	Is in a water body with a history of harmful algae	No.	X
	Puts water quality at risk from your activity through the use, release or disturbance of chemicals	No. Release or disturbance of chemicals is not anticipated.	X
<b>Protected Areas</b>	Within 2km of any WFD protected area	Yes. There is a Bathing Waters area within this waterbody which may be impacted by the Scheme.	✓
<b>INNS</b>	Introduce or spread INNS (including materials or equipment that have come from, had use in or travelled from other water bodies; or activities that help spread INNS).	No. There is a potential risk for the introduction of INNS through material and plant activity into the adjacent waterbody, however, mitigation measures will be included in the CEMP. Therefore, the risk of INNS can be scoped out.	X

## 4.6 In-combination impacts

This section identifies the plans and projects proposed within 2km of the Scheme which may result in in-combination impacts on the marine environment. This section only includes plans/projects which have been applied for, as required by the NRW Guidance Document.

The following projects have been identified:

### 4.6.1 Projects within 2km of the Scheme

#### 4.6.1.1 East Rhyl Coastal Defences Scheme

The East Rhyl Coastal Defences Scheme is located in Rhyl, adjacent to the eastern boundary of the Scheme and is led by DCC with support from the Welsh Government. The project will help protect the area from current storms, flooding and sea level rises.

The East Rhyl Coastal Defences Scheme comprise a rock armour revetment designed to absorb wave energy during future storm events. The rock armour is approximately 600m long by 30m wide. The sea wall is also being replaced with a larger structure and the promenade is being raised by less than 1m to maintain views of the sea from the coastal path.

The East Rhyl Coastal Defences Scheme rock revetment works is currently under construction, with the anticipated project completion in early summer 2022, as such there will be an overlap with the Scheme and potential in-combination impacts is to be assessed further.

Brief overview of the project:

- Distance from Scheme: Within RLB;
- Funding: CRMP (Welsh Government);
- Current stage: Construction;
- Length: 600m;
- Outline description: Rock revetment;
- Construction start date (funding dependent): Late spring 2019; and
- Anticipated construction period: Complete early Summer 2022.

#### 4.6.1.2 Llanddulas to Kinmel Bay Coastal Defence Improvements (Conwy County)

This project comprises of a number of sites across 11km of coastline and involves coastal defence improvements occurring across the sites. The nature of coastal defence improvements depends on the existing condition of the defences. The section of particular interest in relation to the Scheme is the work proposed at Kinmel Bay beach due to the close proximity to the Scheme works.

Kinmel Bay beach works will comprise enhancement of the rock revetment at the eastern end (~450m), reprofiling of car park entrance, upgrade/ replacement of flood gate to be at the same level as the wall, raising of seawall crest by ~300mm along the length of the unit in question and reinstatement of sand dunes where necessary along the stretch to the east of the car park at the end of St Asaph Avenue where gaps exist along the landward side of the foot/cycle path.

Brief overview of the project:

- Distance from Scheme: ~340m (to Quay Street car park compound) from eastern extent (opposite side of Clwyd Estuary);
- Funding: CRMP (Welsh Government);

- Current stage: Detailed Design;
- Length: Six sites over an 11km length of coastline (not continuous);
- OBC works description: Various interventions across a number of sites;
- Construction start date (funding dependent): 2022; and
- Anticipated construction period: Approximately 2 years intermittently.

At the time of writing, this project was not proceeding with a planning application, however it is understood the project would be shortly applying for one, with construction commencing at a similar time as the Scheme. Therefore, there is a potential for this project to overlap with the Scheme and potential in-combination impacts is to be assessed further.

#### 4.6.1.3 Central Prestatyn Coastal Defence Improvements

The Central Prestatyn Coastal Defence improvements project involves the creation of a new flood embankment set back from the existing defence. It is being led by DCC with support from the Welsh Government. It would surround the western, southern and eastern boundaries of Rhyl Golf Course, situated along the Rhyl Coast Road.

The project will include construction of an embankment approximately ~1.66km long, sheet piles will run down the centre of the embankment, rite-in rock revetments, patch repairs to the steps of the promenade, and two rock armour scour protection areas (totalling ~30m in length), along the toe of the existing stepped revetment to provide protection to the existing defence against the risk of undermining from discharged flood water through the new culverts.

Brief overview of the project:

- Distance from Scheme: ~400m from eastern extent of RLB;
- Funding: CRMP (Welsh Government);
- Current stage: Detailed Design;
- Length: ~1.66km landward embankment;
- Outline description: Flood embankment construction;
- Construction start date (funding dependent): Spring 2022; and
- Anticipated construction period: Until late 2025.

At the time of writing, this project was not proceeding with a planning application, however it is understood the project would be shortly applying for one. The construction timeline is expected to be Spring 2022, therefore, there is a potential for this project to overlap with the Scheme and potential in-combination impacts is to be assessed further.

#### 4.6.2 Other coastal defence projects along the North Wales Coast

##### 4.6.2.1 Colwyn Bay Waterfront Project (Old Colwyn / Phase 2b)

Colwyn Bay is located around 13km to the west of the Scheme. The Colwyn Bay Waterfront Project was conceived in 2007 as a result of the strategic assessment of options for future flood and coastal erosion risk management. During the strategic assessment process it was identified that maintaining or improving the aging defences alone would not attract people to Colwyn Bay as this approach would not address the low beach levels present and there was strong public support to determine a more robust solution to be used as the catalyst for regeneration within the wider Colwyn Bay area.

There are numerous phases of work with some phases complete and others due to commence construction (funding and permission dependent) within the next year:

- Phase 1 (a, b, c): 150m rock groyne structure, linear rock revetment and sheet piled wall, beach recharge and promenade enhancement works including the construction of Porth Eirias. Completed October 2014;
- Phase 2a: Coastal regeneration, improved coastal defences (sea wall), access and promenade enhancements. Complete September 2017;
- Splashpoint: Emergency works completed, comprising a short section (300m of 30m deep rock revetment. Completion ongoing;
- Old Colwyn Waterfront Coastal Defence and Active Travel Scheme (formerly Phase 3): Construction of a 1.15km long ~32m wide rock revetment against the sea wall between Porth Eirias and Splashpoint. Construction anticipated September 2021; and
- Phase 2b: Beach recharge works (~1 million tonnes import of dredged sand to be imported to the beach via a pipeline from a dredging vessel off-shore), groyne improvements (no revetment import needed – revetment to be recycled) and promenade enhancements. Construction anticipated March 2022.

Brief overview of the project:

- Distance from Scheme: 13km;
- Funding: Resilient Roads (Old Colwyn)/CRMP (Phase 2b);
- Current stage: Construction (Old Colwyn)/detailed design (Phase 2b);
- Length: 1.2km (Old Colwyn)/0.8km (Phase 2b);
- OBC works description: Rock revetment (Old Colwyn)/Beach nourishment (Phase 2b);
- Construction start date (funding dependent): September 2021/March 2022; and
- Anticipated construction period: 2 years/12 months.

The construction works at Old Colwyn are currently underway, with the anticipation for Phase 2b to follow shortly, as such there will be an overlap with the Scheme and potential in-combination impacts is to be assessed further.

#### 4.6.2.2 Penrhyn Bay Coastal Defences

At the time of writing, limited information was available for this project. The available current scope of works includes; the construction of a T-shaped rock armour groyne, beach nourishment adjacent to the groyne, public realm enhancements to the promenade, and rock armour.

Brief overview of the project:

- Distance from Scheme: 17km;
- Funding: CRMP;
- Current stage: Detailed Design;
- Length: 0.8km;
- OBC works description: Rock promontory;
- Construction start date (funding dependent): March 2022; and
- Anticipated construction period: Anticipated to be ~9 months

As the project is along the North Wales coast and has the potential to overlap with the Scheme, there is a potential risk of in-combination impacts which is to be assessed further.

#### 4.6.2.3 Llandudno Coastal Defences

At the time of writing, the scope of works is not known, however there is a potential for the construction period to overlap.

Brief overview of the project:

- Distance from Scheme: 20km;
- Funding: CRMP (Welsh Government);
- Current stage: Detailed Design;
- Length: 1.6km (West Shore)/2.8km (North Shore);
- OBC works description: Increase height of sea wall by 300mm (West Shore)/possible beach nourishment (North Shore);
- Construction start date (funding dependent): March 2022; and
- Anticipated construction period: Unknown.

As the project is along the North Wales coast and has the potential to overlap with the Scheme, there is a potential risk of in-combination impacts which is to be assessed further.

### 4.7 Cumulative impacts

This section identifies potential cumulative impacts on the marine environment as a result of the Scheme and all developments already built and operational.

The North Wales coastal waterbody and Clwyd Transitional waterbody are heavily modified for the purpose of coastal defence. The surrounding developments are residential and commercial buildings. During operational, it is not anticipated for the Scheme and these developments to have cumulative impacts on the surrounding waterbodies. As such, the cumulative impacts will not be assessed further.

### 4.8 Summary

To conclude the Scoping Assessment, there is a potential for the Scheme to impact the ability of either North Wales Coastal Waterbody (GB64101165000) or the Clwyd Transitional Waterbody (GB541006608000) to achieve its objectives.

As the Scheme is within the North Wales Coastal Waterbody and the purpose is to install coastal defences to prevent flooding, there is a potential for the Scheme to impact on the waterbodies objective's, or ability to meet 'good' status. The Clwyd Transitional Waterbody is in close proximity to the Scheme and has the potential for the Scheme to impact the waterbodies objectives. Therefore, both waterbodies need to be further considered in the Stage 3: Detailed Assessment.

Impacts on the Protected Areas within or in close proximity of the Scheme, either during construction or operation, cannot be ruled out of the screening assessment. Therefore, these sites need to be further considered in the Stage 3: Detailed Assessment.

The WFD Elements to be considered further in the Stage 3: Detailed Assessment are summarised in Table 4.6:

**Table 4.6: Summary of Scoped-in WFD Elements**

WFD Quality Element	North Wales Coastal Waterbody	Clwyd Transitional Waterbody
Hydromorphology	ü Development for coastal protection	X Not considered further
Priority Habitats and Species	ü Higher sensitivity habitats within 500m of the Scheme: Mussel beds	X Not considered further
Biology – fish	X Not considered further	X Not considered further
Water Quality	ü Phytoplankton status of Moderate	X Not considered further
Protected Areas	ü Liverpool Bay SPA, Rhyl East Bathing Waters, Marine Lake Bathing Waters, and Rhyl Bathing Waters	ü Liverpool Bay SPA, Rhyl East Bathing Waters, Marine Lake Bathing Waters, and Rhyl Bathing Waters
INNS	X Not considered further	X Not considered further

The following projects have the potential for in-combination impacts as they are along the coastline;

- East Rhyl Coastal Defences Scheme;
- Llanddulas to Kinmel Bay Coastal Defence Improvements (Conwy County);
- Central Prestatyn Coastal Defence Improvements;
- Colwyn Bay Waterfront Project (Old Colwyn / Phase 2b);
- Penrhyn Bay Coastal Defences; and,
- Llandudno Coastal Defences.

## 5 Stage 3: Detailed compliance assessment

This stage considers the potential impacts of an activity, identifies ways to avoid or minimise impacts and concludes if the activity may prevent any WFD quality elements within any WFD waterbody achieving good status/potential or may cause deterioration.

Five receptors were identified as being at risk from the Scheme:

- Hydromorphology (development for coastal protection);
- Priority Habitats and Species (higher sensitivity habitats within 500m of the Scheme: Mussel beds and *Sabelliria* Reef);
- Water Quality (phytoplankton status of Moderate); and,
- Protected Areas (Liverpool Bay SPA, Rhyl East Bathing Waters, Marine Lake Bathing Waters, and Rhyl Bathing Waters).

### 5.1 Mitigation measures

In line with policy and best practice, the Scheme will produce a CEMP prior to construction to reduce potential risks to the environment. The CEMP will include, but not be limited to the following:

- Works on the intertidal area (to include piling activities) would be undertaken on the mid to low tide to reduce potential disturbance to SPA species, propagation of noise and vibration through the water column and suspension of sediment in the water column;
- Storage of fuels and chemicals to be above any flood water level/tidal area, and where possible away from high-risk locations (more than 10m from a waterbody);
- Refuelling of plant and equipment would only be permitted at designated refuelling areas. Fuel would be pumped into machines to minimise the risk of spillage. All refuelling and bulk deliveries would be supervised at all times;
- Pollution spill kits to be made available where required and Toolbox Talks are given to required staff;
- Visual monitoring of identified waterbodies; and,
- Throughout the construction and operational phases, best practice guidance in reference to pollution prevention would be followed; CIRIA (2015)<sup>6</sup>.

### 5.2 Detailed compliance assessment

A WFD Compliance assessment has been completed for each receptor for both WFD waterbodies (North Wales and Clwyd), as shown in Table 5.1: and Table 5.2 respectively

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<sup>6</sup> CIRIA 2015. Charles, P, Edwards, P (eds), Environmental good practice on site guide (fourth edition) (C741)

**Table 5.1: Potential impacts on WFD status of the North Wales Coastal Waterbody**

WFD Quality Element	Potential impacts	Compliance, mitigation and recommendations	Will the Scheme result in deterioration of WFD Quality Element?
<p><b>Hydromorphology:</b> Development for coastal protection</p>	<p>The construction of the revetment (sections E to H) will require excavation of beach material to provide a solid foundation. During the construction phase, machinery will track along the foreshore to excavate existing material and to place new materials. This could result in the loosening of beach material, making it more available for transporting along the frontage and to be suspended into the water column. However, works are to be undertaken at low water, reducing the potential for suspended solids.</p> <p>Material from the excavation will be evenly distributed along the foreshore following the construction of a section of revetment that was excavated. This will allow the material to remain in the system and maintain the sediment balance.</p> <p>The construction phase may lead to short term adverse impacts on sediment transport through the temporary placement of rock used for scour protection construction (in sections A to D) in the intertidal zone prior to re-location to its final location. It is assumed that rock would be stockpiled immediately seaward of its eventual location at the base of the existing revetments and seawalls. This temporary storage of materials may affect current localised sediment transport patterns, leading to settling out of materials in the vicinity of the stockpiled materials and the reduction in sediment transported along the frontage. It is considered that once the stockpiles have been removed the beach would return to its current state.</p> <p>The revetment in front of the existing seawall may result in the alteration of waves incident at the seawall. This will provide a less reflective surface and as such is expected to reduce scour and thus reduce further erosion and lowering of the beach along the frontage. However, with time beach levels are anticipated to reach equilibrium levels.</p>	<p>Standard best practice measures are to be followed throughout the construction phase, with a CEMP produced prior to construction works. On-spot visual monitoring is recommended to assess any visible changes to the waterbody.</p> <p>Stockpile locations would be agreed with the DCC and NRW once a Contractor has been appointed and recorded in the CEMP.</p> <p>During construction, to reduce short term negative impact on hydrodynamics and the sediment transport regime, materials, such as rock used for scour protection construction, will be stored as high up the beach as practicable, close to the working area. Where this is not possible, the size of the stockpile will be assessed to ensure that changes to the current flows are not creating areas of scour. Monitoring of the beaches around the stockpile will be undertaken and the beach levels reinstated should beach lowering be recorded. A Contractor-led beach inspection scheme will be agreed in advance with the DCC and NRW and detailed in the CEMP. Stockpiles will only be temporary and will not be permitted to remain beyond the end of the construction phase.</p> <p>Excavation of waste materials will be kept to the minimum required in order to allow a solid founding for new structures without the removal and disturbance of excess materials. This will minimise displacement of removed sediments onto the foreshore that might be dispersed leading to an increase in suspended sediments.</p>	<p><b>No</b> - There is already extensive coastal protection along the frontage of the coastal waterbody. Any impacts to the waterbody are anticipated to be localised and not result in deterioration at the waterbody scale, as such providing embedded mitigation measures are implemented and standard best practice measures are followed, the Scheme would be WFD compliant.</p>
<p><b>Priority Habitats and Species<sup>7</sup>:</b> Higher sensitivity habitats within 500m of the</p>	<p><b>Mussel Beds:</b> Acoustic underwater noise and vibration from piling works have the potential to propagate through bedrock and sediment and impact shellfish species.</p>	<p>Standard best practice measures are to be followed throughout the construction phase, with a CEMP produced prior to construction works. On-spot visual</p>	<p><b>No</b> - Any impacts to the waterbody are anticipated to be localised and not result in deterioration at the</p>

<sup>7</sup> Potential impacts and associated mitigation measures relating to the Priority Habitats and Species have been extracted from the Central Rhyl Coastal Defences Environment Statement Volume 1:Main Text, Biodiversity Chapter, *Mott MacDonald*, 2021.

WFD Quality Element	Potential impacts	Compliance, mitigation and recommendations	Will the Scheme result in deterioration of WFD Quality Element?
Scheme: Mussel beds and <i>Sabellaria</i> Reef	<p>Water pollution through the risk of oil and fuel spills within the intertidal area could prove toxic to the shellfish species.</p> <p>An increase in turbidity from construction activities has the potential to reduce the levels of dissolved oxygen in the water column and impacting on the respiration rate of the shellfish species.</p> <p><b>Sabellaria Reefs:</b></p> <p>Water pollution through the risk of oil and fuel spills within the intertidal area could prove toxic to the shellfish species.</p> <p>An increase in turbidity from construction activities has the potential to reduce the levels of dissolved oxygen in the water column and impacting on the respiration rate of the shellfish species.</p>	<p>monitoring is recommended to assess any visible changes to the waterbody.</p>	<p>waterbody scale, as such providing embedded mitigation measures are implemented and standard best practice measures are followed, the Scheme would be WFD compliant.</p>
<b>Water Quality:</b> Phytoplankton status of Moderate	<p>The Scheme would not result in an increase in the amount of organic material being deposited into the waterbody and is not anticipated to result in any impact upon phytoplankton.</p> <p>During construction, there is a potential risk of water pollution from oil or fuel spills which could result in a deterioration of water quality of the waterbody.</p>	<p>Standard best practice measures are to be followed throughout the construction phase, with a CEMP produced prior to construction works detailing the practical and necessary measures required during construction to prevent the pollution of the surrounding environment. On-spot visual monitoring is recommended to assess any visible changes to the waterbody.</p>	<p>No - Any impacts to the waterbody are not anticipated to result in deterioration at the waterbody scale, as such providing embedded mitigation measures are implemented and standard best practice measures are followed, the Scheme would be WFD compliant.</p>
<b>Protected Areas:</b> Liverpool Bay SPA, Rhyl East Bathing Waters, Marine Lake Bathing Waters, and Rhyl Bathing Waters	<p><b>Liverpool Bay SPA<sup>8</sup>:</b></p> <p>Designated for the wintering and breeding bird assemblages it supports.</p> <p>Visual disturbance: During construction, the works have the potential to visually disturb birds due to the increase in human activity on the shoreline.</p> <p>Noise and Vibration: During construction, there will be an increase in noise and vibration which has the potential to disturb or displace foraging birds, especially during winter months.</p> <p>During construction, there is a potential risk of water quality deterioration from oil or fuel spills which could result in a</p>	<p>Standard best practice measures are to be followed throughout the construction phase, with a CEMP produced prior to construction works detailing the practical and necessary measures required during construction to prevent the pollution of the surrounding environment. On-spot visual monitoring under the WFD directive is recommended to assess any visible changes to the waterbody.</p> <p>The Scheme to adhere to mitigation measures outlined in the Habitats Regulations Assessment (HRA).</p>	<p>No - Any impacts to the waterbody are anticipated to be localised and not result in deterioration at the waterbody scale, as such providing embedded mitigation measures are implemented and standard best practice measures are followed, the Scheme would be WFD compliant.</p>

<sup>8</sup> Potential impacts and associated mitigation measures relating to the Liverpool Bay SPA have been extracted from the Central Rhyl Coastal Defences Environment Statement Volume 1: Main Text, Biodiversity Chapter, Mott MacDonald, 2021.

WFD Quality Element	Potential impacts	Compliance, mitigation and recommendations	Will the Scheme result in deterioration of WFD Quality Element?
	<p>deterioration of the SPA and adversely impact the designated interest features.</p> <p>During construction, the stockpiles of rock for the scour protection (section A to D) will be stored on the beach prior to placement and will be on the intertidal area over a tidal cycle. This has the potential to increase suspended sediment and reduce visibility which has the potential to adversely impact the food source of the overwintering birds.</p> <p>During construction, it is anticipated for artificial lighting to be required, which has the potential to disturb and displace birds using the area closer to shore.</p> <p>During operations, there is a potential for the maintenance works to result in the same impacts as during construction (visual disturbance, noise and vibrations, water quality, turbidity), however it would depend on the scale of the works required.</p> <p><b>Bathing Waters:</b></p> <p>There is potential for suspended sediments resulting from construction activities to affect the water quality of the Bathing Waters in the immediate vicinity of the Scheme, although the water quality of the overall North Wales waterbody would not be affected.</p>		

**Table 5.2: Potential impacts on WFD status of the Clwyd Transitional Waterbody**

WFD Quality Element	Potential impacts	Compliance, mitigation and recommendations	Will the Scheme result in deterioration of WFD Quality Element?
<p><b>Protected Areas:</b>                      Liverpool Bay SPA, Rhyl East Bathing Waters, Marine Lake Bathing Waters, and Rhyl Bathing Waters</p>	<p><b>Liverpool Bay SPA:</b></p> <ul style="list-style-type: none"> <li>Designated for the wintering and breeding bird assemblages it supports.</li> <li>Visual disturbance: During construction, the works have the potential to visually disturb birds due to the increase in human activity on the shoreline.</li> <li>Noise and Vibration: During construction, there will be an increase in noise and vibration which has the potential to disturb or displace foraging birds, especially during winter months.</li> <li>During construction, there is a potential risk of water quality deterioration from oil or fuel spills which could result in a deterioration of the SPA and adversely impact the designated interest features.</li> <li>During construction, the stockpiles of rock for the scour protection (sections A to D) will be stored on the beach prior to placement and will be on the intertidal area over a tidal cycle. This has the potential to increase suspended sediment and reduce visibility which has the potential to adversely impact the food source of the overwintering birds.</li> <li>During construction, it is anticipated for artificial lighting to be required, which has the potential to disturb and displace birds using the area closer to shore.</li> <li>During operations, there is a potential for the maintenance works to result in the same impacts as during construction (visual disturbance, noise and vibrations, water quality, turbidity), however it would depend on the scale of the works required.</li> </ul> <p><b>Bathing Waters:</b></p> <ul style="list-style-type: none"> <li>There is potential for suspended sediments resulting from construction activities to affect the water quality of the Bathing Waters in the immediate vicinity of the Scheme, although the water quality of the overall North Wales waterbody would not be affected.</li> </ul>	<ul style="list-style-type: none"> <li>Standard best practice measures are to be followed throughout the construction phase, with a CEMP produced prior to construction works detailing the practical and necessary measures required during construction to prevent the pollution of the surrounding environment. On-spot visual monitoring is recommended to assess any visible changes to the waterbody.</li> <li>The Scheme to adhere to mitigation measures outlined in the Habitats Regulations Assessment (HRA).</li> </ul>	<p><b>No</b> - Any impacts to the waterbody are anticipated to be localised and not result in deterioration at the waterbody scale, as such providing embedded mitigation measures are implemented and standard best practice measures are followed, the Scheme would be WFD compliant.</p>

### 5.3 In-combination impact assessment

The following coastal improvement projects were identified as having potential for in-combination effects:

- East Rhyl Coastal Defences Scheme;
- Llanddulas to Kinmel Bay Coastal Defence Improvements (Conwy County);
- Central Prestatyn Coastal Defence Improvements;
- Colwyn Bay Waterfront Project (Old Colwyn / Phase 2b);
- Penrhyn Bay Coastal Defences; and,
- Llandudno Coastal Defences.

It is anticipated for all the projects to overlap, however, the only projects which have progressed to planning application or construction are the East Rhyl Coastal Defences Scheme and the Colwyn Bay Waterfront Project.

All coastal defence projects need to be considered in-combination as the cumulative effects of the projects commencing within the same timescale have the potential to change the hydromorphology of the waterbody and result in a deterioration of the WFD quality of the waterbody.

Providing mitigation measures are adhered to, and water quality does not deteriorate at a local level, it is not anticipated for there to be a cumulative impact on the water quality of the waterbody as a whole.

## 6 Conclusions and Recommendations

### 6.1 Assessment outcomes

#### 6.1.1 Stage 1: WFD Screening

The Scheme's proposed activities are not listed within Appendix A2.2 of the NRW Guidance, indicating a potential risk and therefore it is considered all of the proposed activities have been subject to further assessment at Stage 3.

The following proposed activities **may cause deterioration** or **prevent a waterbody from meeting its objectives**, as such, these activities will be screened-in and assessed further:

- Construction Works:
  - Construction of rock scour protection and concrete repairs;
  - Construction of new concrete revetment and associated beach access points;
  - Extension of outfalls and protection works to existing outfalls; and,
  - Construction of raised promenade and associated retaining structures.
- Maintenance / Operational:
  - Inspection, maintenance and repair of coastal defence structures;
  - Inspection, maintenance and repair of accesses, drainage, highways, lighting and other minor assets;
  - Management of wind-blown sand; and,
  - Beach management.

#### 6.1.2 Stage 2: WFD Scoping

To conclude the Scoping Assessment, there is a potential for the Scheme to impact the ability of either North Wales Coastal Waterbody (GB64101165000) or the Clwyd Transitional Waterbody (GB541006608000) to achieve its objectives.

Table 6.1: summaries the waterbodies identified to be scoped-in and assessed further in the scoping assessment.

**Table 6.1: Screened-in Waterbody Summary**

Waterbody name	Waterbody ID	Reason for scoping-in	Overall status
<b>Coastal Waterbodies</b>			
North Wales	GB641011650000	The Scheme is within this waterbody. The purpose of the Scheme is to change the morphology of the area to prevent the risk of flooding, as such there is a potential for the Scheme to impact the waterbodies objectives during construction.	Moderate
<b>Transitional Waterbodies</b>			
Clwyd	GB541006608000	The Scheme is entirely downstream of this waterbody but is within close proximity to potentially impact the waterbodies objectives during construction.	Moderate

Impacts on the Protected Areas within or in close proximity of the Scheme, either during construction or operation, cannot be ruled out of the screening assessment (with the exception of the NVZ which has been scoped-out.). Therefore, these sites have been considered further in in the Stage 3: Detailed Assessment.

The WFD Elements considered further in the Stage 3: Detailed Assessment included:

- North Wales Coastal Waterbody:
  - Hydromorphology: Development for Coastal Protection;
  - Priority Habitats and Species: Higher sensitivity habitats within 500m of the Scheme: Mussel Beds;
  - Water Quality: Phytoplankton status of Moderate; and,
  - Protected Areas: Liverpool Bay SPA, Rhyll East Bathing Waters, Marine Lake Bathing Waters, and Rhyll Bathing Waters.
- Clwyd Transitional Waterbody
  - Protected Areas: Liverpool Bay SPA, Rhyll East Bathing Waters, Marine Lake Bathing Waters, and Rhyll Bathing Waters; and.

### 6.1.3 Stage 3: Detailed compliance assessment

The Stage 3 assessment considered the potential impacts of the Scheme activities on the WFD quality elements, identified ways to avoid or minimise impacts, and concluded if the activity may prevent any WFD quality element within any WFD water body achieving good status/potential or may cause deterioration.

The assessment found that given:

- The mitigation already incorporated into the Scheme; and,
  - The relatively small scale of works in the context of the WFD waterbodies as a whole;
- any impacts on the WFD waterbody were considered to be localised and temporary, and not considered to cause deterioration of the WFD waterbody status.

## 6.2 Conclusions

This WFD Assessment has found that, when considering the mitigation already proposed, the Scheme is very unlikely to affect the WFD status or cause any deterioration of the WFD quality elements for the North Wales Coastal Waterbody or Clwyd Transitional Waterbody.

## A. Scheme Extent



Key to Symbols

Scheme red line boundary

Notes

- This drawing is intended only to be used to support the Environmental Impact Assessment for the proposed Central Rhyll Coastal Defences Scheme, and it should not be used for any other purpose. It presents the red line boundary for the Central Rhyll Coastal Defences Scheme.
- Contains OS data © Crown Copyright and database right 2020. Contains data from OS Zoomstack, Maxar, Microsoft

Rev	Date	Drawn	Description	Ch'k'd	App'd
C02	03/09/21	SL	Accepted for EIA	CW	NMP
P03	03/08/21	SA	Updated for ES review and comment	CW	NP
C01	02/02/21	SA	Accepted for EIA scoping	CW	NP
P02	18/12/20	SA	EIA Scoping for acceptance	NS	NP
P01	27/11/20	SA	EIA Scoping for review and comment	CL	NMP

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Client

Title  
Central Rhyll Coastal Defences  
Red Line Boundary

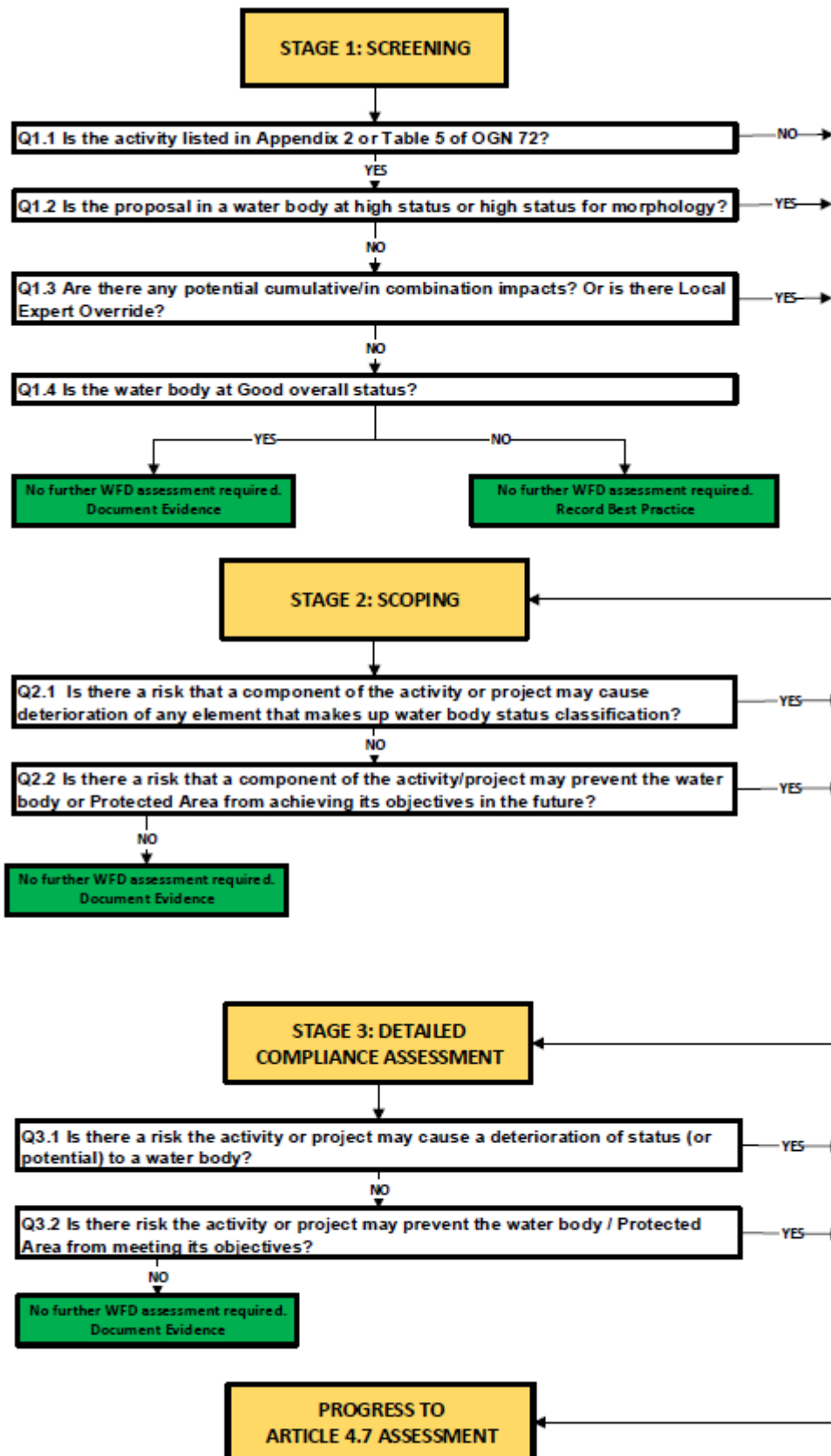
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Drawn	S Li	SL	Coordination	N Spofforth	NS
GIS Check	S Anstice	SA	Approved	N M Price	NMP

Scale at A3	Status	Rev	Security
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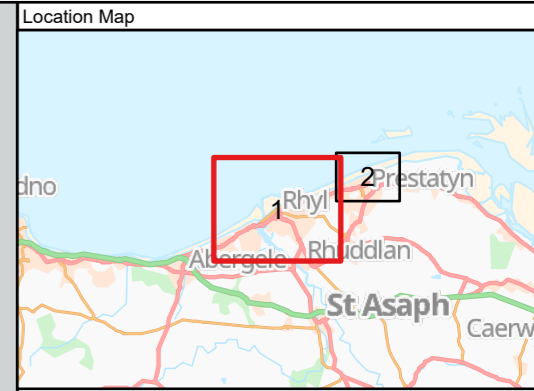
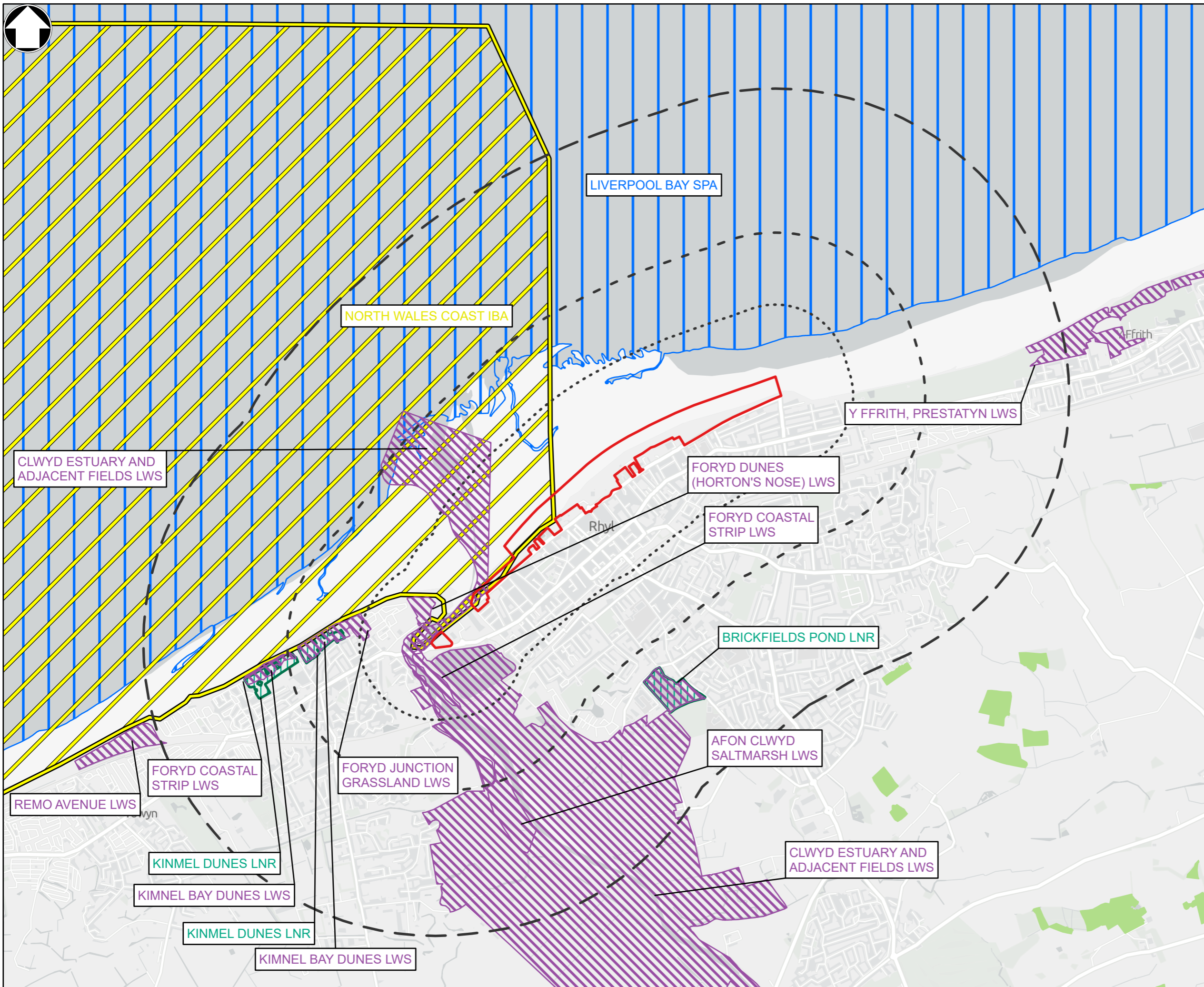


## **B. WFD Compliance Assessment Process Flowchart**

Figure 1: WFD Compliance Assessment Process Flowchart



## C. Protected Area Map



**Key to Symbols**

- Scheme red line boundary
- Scheme red line boundary buffer (500m)
- Scheme red line boundary buffer (1km)
- Scheme red line boundary buffer (2km)
- Local Wildlife Sites (LWS)
- Important Bird Areas (IBA)
- Special Protection Areas (SPA)
- Local Nature Reserves (LNR)
- Ancient Woodland

**Notes**

1. This drawing is intended only to be used to support the Environmental Impact Assessment for the proposed Central Rhyll Coastal Defences Scheme, and it should not be used for any other purpose. It represents the Statutory and Non-Statutory Designated Sites within 2km of the Central Rhyll Coastal Defences Scheme.
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P01	03/08/21	SL	For Review and Comment	CW	NP
Rev	Date	Drawn	Description	Ch'kd	App'd

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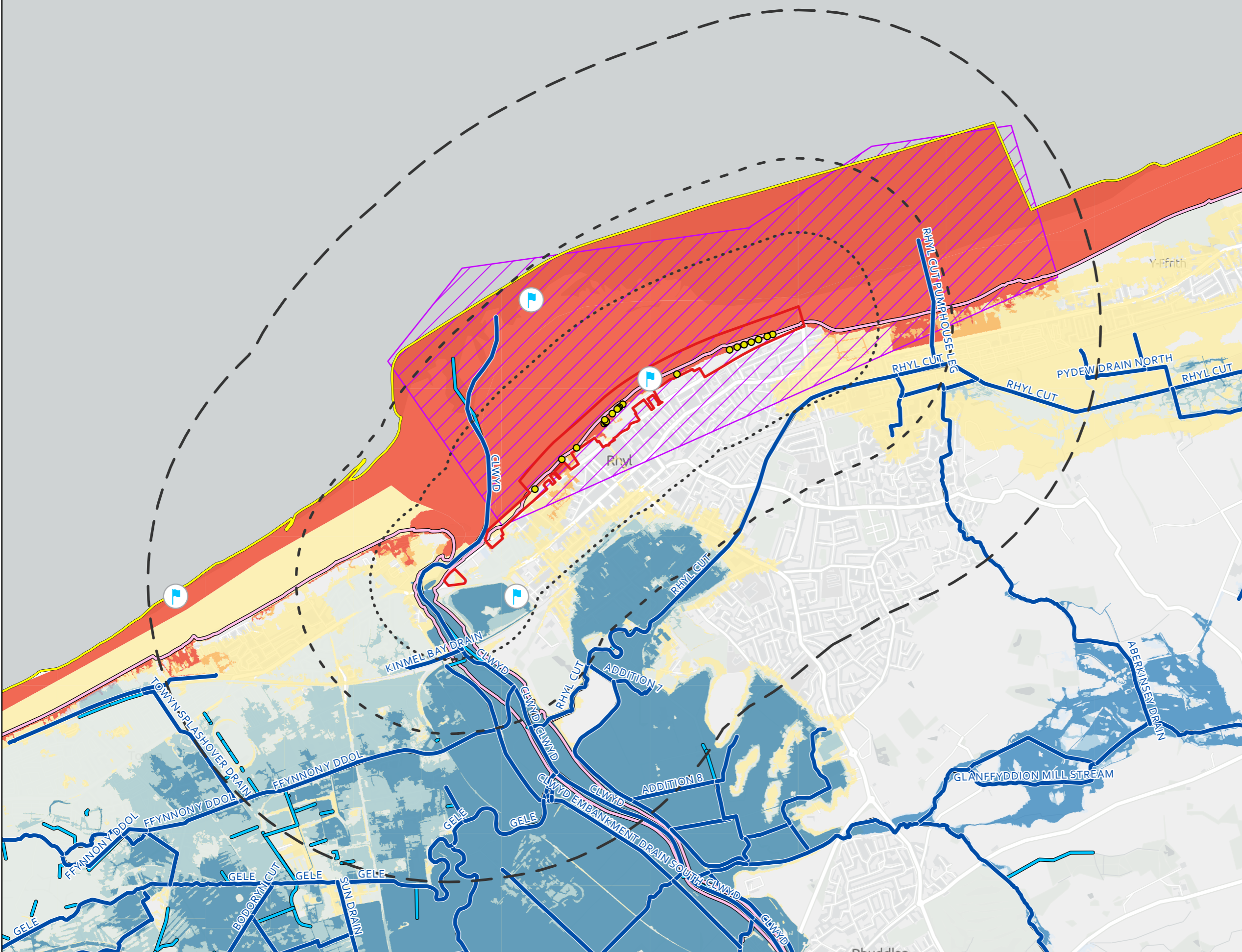
Client

**denbighshire** County Council

**Balfour Beatty**

Title Central Rhyll Coastal Defences  
Statutory and Non-Statutory Designated Sites within 2km  
of the Scheme  
Page 1 of 2

Designed	N Spofforth	NS	Eng Check	C Williams	CW
Drawn	S Li	SL	Coordination	N Spofforth	NS
GIS Check	S Anstice	SA	Approved	N M Price	NMP
Scale at A3	Status	Rev	Security		
1:25,000	A4	C01	STD		



**Key to Symbols**

Scheme red line boundary	Sensitive areas - Bathing Waters
500m buffer of Scheme extent	Mean low water mark
1km buffer of Scheme extent	Mean high water mark
2km buffer of Scheme extent	<b>Flood risk from rivers</b>
Existing outfalls	High
Bathing Water monitoring point	Medium
Main rivers	Low
Other watercourses	<b>Flood risk from sea</b>
	High
	Medium
	Low

**Notes**

- This drawing is intended only to be used to support the Environmental Impact Assessment for the proposed Central Rhyll Coastal Defences Scheme, and it should not be used for any other purpose. It presents the hydrological, coastal and Water Framework Directive (WFD) constraints only.
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- Outfall locations are indicative locations, hand-drawn based on drawing PMS20222-03 on 07/06/2021. Should be confirmed prior to any works.
- The Point of Ayr is 10.5km away from the Scheme red line boundary.

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P01	03/08/21	SL	For Review and Comment	CW	NP
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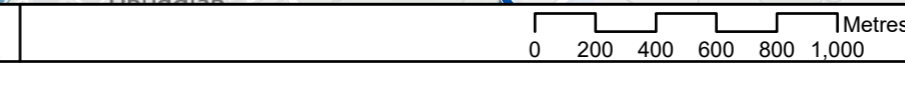
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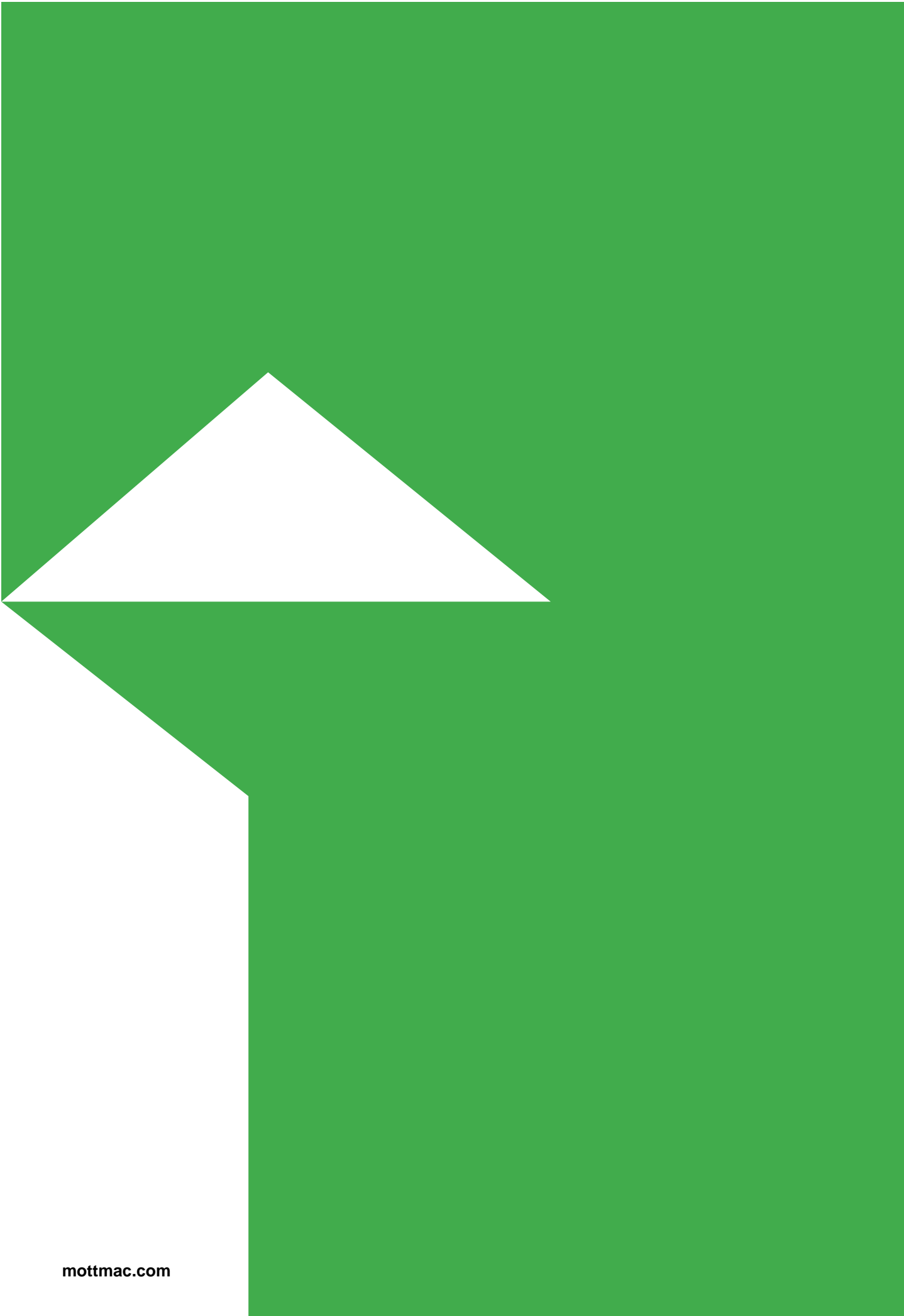
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Title  
Central Rhyll Coastal Defences  
Hydrological, Coastal and WFD Constraints

Designed	V Tonks	VD	Eng Check	C Williams	CW
Drawn	S Li	SL	Coordination	N Spofforth	NS
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## Appendix 10.5 – NRW Meeting Minutes

- Meeting with Natural Resources Wales, Balfour Beatty Civil Engineering Ltd and Mott MacDonald on 19 May 2021 (Sediment modelling update)
- Meeting with Natural Resources Wales, Balfour Beatty Civil Engineering Ltd and Mott MacDonald on 29 June 2021 (Sediment modelling update Nr. 2)

<b>Project title</b>	Central Rhyl Coastal Defences
<b>Subject</b>	Sediment modelling update
<b>Location</b>	Remote Teams Meeting
<b>Date/time of meeting</b>	19/05/21 10:00 to 10:45
<b>Project number</b>	100420823
<b>Attendees</b>	Emmer Litt (Natural Resources Wales) Ian Beckett (Balfour Beatty Civil Engineering Ltd) Nicholas Price (Mott MacDonald) Victoria Deakin (Mott MacDonald) Shirin Costa (Mott MacDonald) Darren Price (Mott MacDonald)
<b>Apologies</b>	Matt Cowle (Mott MacDonald), Wayne Hope (Denbighshire County Council)
<b>Recorded by</b>	Victoria Deakin
<b>Distribution</b>	Attendees plus apologies

Item	Text	Action
1	<p><b>Introduction and Objectives</b></p> <p>Shirin Costa introduced the workshop and outlined the key objective as follows (PowerPoint presentation used);</p> <ul style="list-style-type: none"> <li>• Provide an update on the sediment modelling</li> </ul>	
2	<p>Sediment modelling update</p> <p>Shirin Costa presented the modelling results to date. Emmer Litt indicated that the good thing about the model is it can show long term timescales. A question was raised regarding if the beach would reach equilibrium over long term for the cross shore changes. Shirin Costa and Darren Price advised that the long term evolution model will link profiles together based on the sediment transport model. This will also feedback changes in the cross shore into the model.</p> <p>There is no significant changes to the sediment budgets over the whole system seen in the current modelling results. Emmer Litt asked if it was possible to take the profiles at the Point of Ayr (last 4 profiles in the model) and look at them in detail to show the changes to the transport rates. These results could then be presented in the modelling report and Environmental Statement.</p> <p>Emmer Litt advised that the Point of Ayr is currently eroding which is different to what is showing in model – Darren Price and Shirin Costa advised that the physical processes acting at the Point of Ayr are</p>	

Item	Text	Action
	<p>complex, whilst we are only considering the impact to the longshore littoral transport brought about by the potential development at Rhyl. Many factors could affect the accretion/erosion at the Point of Ayr beyond littoral transport alone. Changes to the tidal prism of the estuary could have an impact, as could interactions with offshore sandbanks and deep channels. The modelling we are undertaking will identify the change to the supply of sediment to the Point of Ayr (from the west),but will not investigate the changes brought about by other factors which could be part of longer term trends.</p>	
	<p>Emmer Litt asked if the offshore movement of sediment affects the Central Rhyl frontage. Shirin Costa and Darren Price advised that the material from the offshore banks can be added into the evolution model based on available data / information. This could be undertaken as a sensitivity test using some of the onshore sediment movements estimated in other studies.</p>	
	<p>Emmer Litt asked what scenarios are we looking at for the modelling, as this will be something that is reviewed in the Environmental Statement. Shirin Costa and Darren Price identified that the cross shore modelling includes a 1 in 100 storm for present and future conditions (2120) both baseline and with the scheme. For the coastline evolution model a period of time (likely to be between 6 months and a year) will be run to smooth out inconsistencies in the model. The baseline and proposed layout will be run for 10 years which will include the storm events within the 10 years. This also allows changes in the summer and winter profiles / storms etc.</p>	
	<p>Emmer Litt advised that the sediment movement in Central Rhyl is west to east offshore but east to west in the nearshore.</p>	
	<p>Shirin Costa and Darren Price presented the plots showing the proportion of the time the tide is affecting the structure is limited. These plots can also be added into the modelling report.</p>	
	<p>Emmer Litt questioned if it was possible to hindcast backwards in the model. Shirin Costa advised that LiDAR has been analysed which shows the changes in erosion and accretion (compared against the results). These changes have been presented within the sediment modelling report as part of the historical analysis chapter, which analysis profiles and LiDAR trends for Central Rhyl.</p>	
	<p>Emmer Litt advised that, at present, based on what has been presented in the meeting, there are no concerns with the modelling approach. It was also highlighted that NRW would not want to see material being locked out of the system in the revetment.</p>	
	<p>A meeting to review the longshore transport modelling results will be proposed to discuss the outputs. Emmer Litt asked that the project team consider the aim of the modelling throughout to determine the impact of the scheme on the coastal processes and the impacts downdrift at the SAC.</p>	<p><b>Victoria Deakin to organise</b></p>

<b>Project title</b>	Central Rhyl Coastal Defences
<b>Subject</b>	Sediment modelling update 2
<b>Location</b>	Remote Teams Meeting
<b>Date/time of meeting</b>	29/06/21 09.30 to 10:15
<b>Project number</b>	100420823
<b>Attendees</b>	Emmer Litt (Natural Resources Wales) Jodie Pullen (Natural Resources Wales) Ian Beckett (Balfour Beatty Civil Engineering Ltd) Nicholas Price (Mott MacDonald) Victoria Deakin (Mott MacDonald) Shirin Costa (Mott MacDonald) Darren Price (Mott MacDonald)
<b>Apologies</b>	Matt Cowle (Mott MacDonald), Wayne Hope (Denbighshire County Council)
<b>Recorded by</b>	Victoria Deakin
<b>Distribution</b>	Attendees plus apologies

Item	Text	Action
1	<p><b>Introduction and Objectives</b></p> <p>Victoria Deakin introduced the workshop and outlined the key objective as follows (PowerPoint presentation used);</p> <ul style="list-style-type: none"> <li>• Provide an update on the sediment modelling</li> </ul>	
2	<p><b>Sediment modelling update</b></p> <p>Shirin Costa presented the modelling results to date and concluded that there are no significant effects seen on the coastline directly to the east of the scheme and no effects on the coastal processes towards the Point of Ayr.</p> <p>Emmer Litt indicated that the presentation was good and useful to get the background on the project before seeing the Environmental Statement. She was also pleased to see the scheme assessed over the longer term.</p> <p>A question was raised regarding the choice of profiles within the model. Shirin Costa advised that this was based on having 3 profiles within the scheme boundary to represent the differing options on the frontage and also the change in coastline. The remaining profiles represent the changes in the coastline from the scheme to the Point of Ayr.</p>	

Item	Text	Action
	<p>Emmer Litt asked about how the transport rates seen in this modelling study differ from those in previous studies. Shirin Costa advised that model has identified results in the same order of magnitude as those undertaken by HR Wallingford, Halcrow for the Cell 11 SMP project and JBA.</p> <p>Emmer Litt advised that from the material that has been presented there does not look like there will be significant effects on the coastal processes, and she has no significant concerns at this stage, however this will be reviewed fully when the Environmental Statement is provided.</p> <p>Emmer Litt questioned if the model would be used to identify where any material that is excavated from the scheme and placed in front of the new defences would go. Shirin Costa and Darren Price confirmed that this would be assessed in the Environmental Statement.</p>	
<b>3</b>	<p><b>Scheme update</b></p> <p>Victoria Deakin and Nick Price highlighted the programme remains the same as previously discussed with the marine licence anticipated to be issued to NRW end of August 2021.</p> <p>Victoria Deakin confirmed that the Environmental Statement would be the same for the marine licence and the planning applications.</p> <p>Ian Beckett confirmed that the revetment slope has now been finalised and will be a 1 in 2 slope. This is based on the overtopping assessment. Nick Price advised that following the overtopping assessment the geometry for section E-H has been finalised, the revetment will have a crest level of 7.85m and a 1.4m high rear wall with a bullnose.</p> <p>Victoria Deakin to set up a meeting with NRW for the last week of July or the second week in August to provide an update on the programme.</p> <p>Victoria Deakin to liaise with internal Mott MacDonald team on if there is Sabellaria in the area. If a meeting is needed with Ben Ray in NRW Victoria will let Emmer Litt know.</p>	<p><b>Victoria Deakin to organise</b></p>

# Appendix 10.6 – Flood Consequence Assessment



**Balfour Beatty**

# Central Rhyl Coastal Defences

Flood Consequence Assessment

September 2021

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Balfour Beatty Civil  
Engineering Limited

# Central Rhyl Coastal Defences

## Flood Consequence Assessment

September 2021

# Issue and Revision Record

Revision	Date	Originator	Checker	Approver	Description
P01	05/08/2021	R. Wilson	V. Deakin	Z. Hutchison	First issue for comment
C01	03/09/2021	V. Deakin	B. Riley	Z. Hutchison	Published version with minor amendment following Balfour Beatty comments
C02	16/09/2021	V. Deakin	B. Riley	Z. Hutchison	Published version with minor amendment following Balfour Beatty comments

**Document reference:** 420823 | CR-MMD-00-00-RP-T-3057 | C02

**Information class:** Standard

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# Executive summary

AEP	Annual Exceedance Probability
BB	Balfour Beatty Civil Engineering Limited
DCC	Denbighshire County Council
FCA	Flood Consequence Assessment
FR&I	Flood Risk and Infrastructure
HM	Harbourmaster
Hs	Significant wave height
NGR	National Grid Reference
NWC	North West & Central – Network Rail
NWR	Natural Resources Wales
PPW	Planning Policy Wales
OD	Ordnance Datum
OS	Open Spaces
RCP	Representative Concentration Pathway
RP	Return period
TAN15	Technical Advice Note 15: Development and Flood Risk

# 1 Introduction

## 1.1 Project background

### 1.1.1 Overview

The Central Rhyl Coastal Defences Scheme (herein referred to as ‘the Scheme’) is a design and build scheme being delivered by Balfour Beatty Civil Engineering Limited (BB) for Denbighshire County Council (DCC). Mott MacDonald Ltd (Mott MacDonald) has been commissioned by BB to support with the development and delivery of the Scheme.

The Scheme, located in the central area of Rhyl waterfront, Denbighshire, North Wales, is dependent on obtaining sufficient funding and comprises coastal defence works in the form of scour protection, stepped concrete revetment, promenade raising, sea wall works, access and promenade improvements, and associated tie-in works.

Rhyl is a Victorian coastal resort and the largest town in Denbighshire, with a well-established history as a tourist destination. There are ongoing and future plans to regenerate Rhyl to enhance its appeal to tourists and investors; however, the coastal defences for the Central Rhyl frontage are deteriorating and there is the risk that this integral asset for Rhyl’s regeneration could be lost without intervention.

Historically, Central Rhyl has been protected from coastal flooding by hard engineered defence structures constructed from at least the 1900s. These comprise a series of stepped revetments, recurve walls and vertical walls in various states of repair. The existing defences no longer meet acceptable performance standards and are known to be deteriorating.

The existing promenade in the area of the Scheme can become inundated during storms, causing flooding of the promenade and surrounding areas. In the future this will be exacerbated by the effects of climate change, which is predicted to increase both the frequency and magnitude of flooding. The existing coastal defences and promenade require improvements to safeguard the area from overtopping with consideration given to the potential effects of climate change.

Subject to funding, this Scheme aims to improve the existing coastal defences to provide increased flood and erosion protection to people and property in Rhyl along with supporting local regeneration, achieving wider community benefits alongside flood risk management.

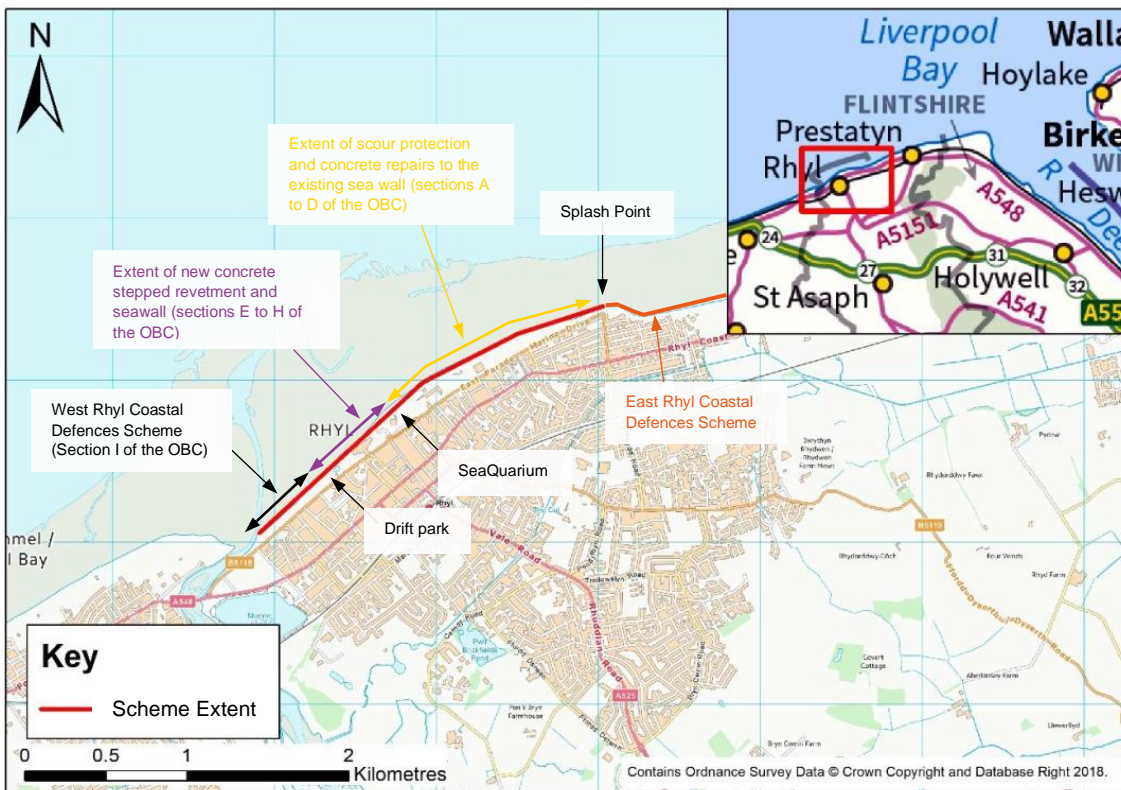
### 1.1.2 Scheme location

The Scheme is located along the coastal frontage of the town of Rhyl, North Wales. It extends from approximately 400m east of the mouth of the River Clwyd, to just west of Splash Point (and the start of the East Rhyl Coastal Defence Scheme, which was under construction at the time of writing this report). Figure 1-1 presents the approximate extent of the Scheme as outlined by the Central Rhyl Coastal Defences Outline Business Case (herein referred to as the “OBC”) (DCC, 2019)<sup>1</sup>. The scheme boundary for the construction are presented in Appendix A.

---

<sup>1</sup> Denbighshire County Council, 2019. Central Rhyl Coastal Defence Scheme Outline Business Case.

**Figure 1-1: Scheme Location and sections at per the Central Rhyl Coastal Defences Outline Business Case**



Source: Central Rhyl Coastal Defence Schemes Outline Business Case, DCC, October 2019. Additional annotations by Mott MacDonald 2021

As shown in Figure 1-1 as part of the OBC, the Scheme was split into nine sections along its length: Sections A – I, with Section A at the eastern end and Section I at the western end.

The western end of the Scheme (i.e. west of Drift park and OBC Section I) is the location of the West Rhyl Coastal Defences Scheme. The West Rhyl Coastal Defences Scheme was constructed in 2015, and no further works are proposed at this location.

### 1.1.3 Scheme proposals

The Scheme is to include the following key coastal works:

- The construction of approximately 1.45km of scour protection and concrete repairs, from Splash Point to the SeaQuarium (sections A to D of the OBC, see Figure 1-1). The scour protection will consist of boulders being placed at the base of the existing coastal defence structures.
- The construction of approximately 750m of new concrete stepped revetment, from the SeaQuarium to opposite Drift Park (sections E to H of the OBC, see Figure 1-1).
- New pedestrian beach accesses through the proposed revetments to replace the existing ones and extension of existing beach accesses through the rock armour scour protection.
- The extension of outfalls and protection works to existing outfalls that would be covered by the new revetment.

The Scheme also includes the following works to the promenade:

- Raised promenade from the SeaQuarium to opposite Drift Park.
- New seawalls at the back of the promenade from the SeaQuarium to opposite Drift Park.

- New flood gates for access points through the new rear seawalls between the SeaQuarium and opposite Drift Park.
- New retaining structures where appropriate at the back of the promenade between the SeaQuarium and opposite Drift Park to accommodate level differences between the raised promenade and adjacent land.
- New ramps or steps to provide access to raised promenade between the SeaQuarium and opposite Drift Park.

## 1.2 Purpose of this report

The purpose of this report is to present the Flood Consequence Assessment (FCA) to accompany the Planning application and the Environmental Statement for the Scheme.

This FCA establishes *‘whether appropriate mitigation measures can be incorporated within the design of the development to ensure that development minimises risk to life, damage to property and disruption to people living and working on the site or elsewhere in the floodplain<sup>2</sup>*.

Mott MacDonald has followed accepted procedure in providing the services but given the residual risk associated with any prediction and the variability which can be experienced in flood conditions, Mott MacDonald takes no liability for and gives no warranty against actual flooding of any property (clients or third party) or the consequences of flooding in relation to the performance of the service. This report has been prepared for the purposes of informing an Environmental Statement and supporting applications for a Marine Licence and Planning consent only.

---

<sup>2</sup> Welsh Assembly Government. (2004). Planning Policy Wales – Technical Advice Note 15: Development and Flood Risk. [online]. Welsh Assembly Government Available at: <https://gov.wales/sites/default/files/publications/2018-09/tan15-development-flood-risk.pdf> [Accessed 13 July 2020].

## 2 Current Flood Risk and Proposed Construction Methods

### 2.1 Current Flood Risk

The Central Rhyl defences no longer meet acceptable performance standards and are currently deteriorating with defects associated with the structures consistent with the age and materials of the structures in question.

The central section is no longer able to prevent wave overtopping for the desired standard of protection under present day conditions. With climate change resulting in increased sea levels and storm frequency/severity, the future effectiveness of the existing defences will continue to decline. Within the OBC it was identified that the structures are likely to deteriorate to such a condition that failure would occur by the year 2068 if the current maintenance regime continues (in a “Business as Usual” scenario). However, if the maintenance is withdrawn (in a “Walkaway” scenario), the structures will deteriorate over a shorter timeframe, and are likely to fail by the year 2038. During a visual condition surveys completed by Mott MacDonald in December 2020 and January 2021 the defences in Section A to D were found to be in a worse condition than those in the OBC. Once the current defence structures are compromised, the coastline will recede leading to loss of the promenade and associated assets including the Wales Coast Path and National Cycle Route 5. Failure and subsequent breach of the defences would lead to an increase in flood risk to people and property in the central Rhyl area which would have a significant impact on the tourism economy of Rhyl and the wider North Wales area.

Therefore, subject to funding, this Scheme aims to improve the existing coastal defences to provide increased flood and erosion protection to people and property in Rhyl along with supporting local regeneration, achieving wider community benefits alongside flood risk management. The Scheme is proposed to manage flood risk to 548 residential and 44 non-residential properties over 100 years and manage erosion risk to key tourism infrastructure. Furthermore, this supports the Hold the Line policy within the Shoreline Management Plan 2.

### 2.2 Proposed Scheme – impacts to current flood risk

Overtopping of the existing defences within sections E to H are demonstrated to be sufficient to cause flooding within Rhyl. The recommendation by the OBC to provide improvements to the overtopping performance within these sections to protect assets behind the promenade is valid. Furthermore, testing of a revised defence structure geometry in these sections consisting of a stepped revetment and rear wall shows that suitably limited overtopping rates can be practically achieved.

The overtopping and inundation assessment has demonstrated that an overtopping rate of less than 10 l/s/m is required to prevent significant inundation of assets within Rhyl. Subsequent testing of structure geometries (considering known buildability issues at the time) confirm the following structure parameters are recommended to meet the 0.5%AEP standard of protection in 2120:

- Stepped Revetment with a slope of 1:2.
- Revetment Crest at +7.85mOD.
- Promenade Width of 10.1m (minimum).
- Rear wall crest at +9.25mOD, 1.4m above revetment crest.
- The rear wall will require inclusion of a compliant bullnose geometry

## 2.3 Construction Works Methodology

The Scheme is currently at detailed design stage. The information below has been collated through discussions with the Contractor for the scheme. The below is a preliminary outline methodology and is subject to amendment, consent and construction stage contractor methodology and risk assessments. Where any uncertainty exists a reasonable worst case has been assumed for all construction activities.

### 2.3.1 Timescales

Construction works would commence in 2022 and an overall construction period of around two and a half years is anticipated.

Depending on the exact start date, the main construction period may continue until around Spring/Summer 2025.

### 2.3.2 Phasing

There would be two distinct sections of the Scheme, with the works required in the western section predominantly comprising installing a new stepped revetment in front of the existing defences and raising the existing promenade; and in the eastern section predominantly comprising installing rock scour protection to the toe of the existing defences:

- The works in the western section (i.e. new revetment construction and promenade raising) would be ongoing throughout the whole construction period; and
- The works in the eastern section (i.e. mainly rock toe placement) would be completed in a shorter duration, anticipated to take approximately one and a half years to complete. It is likely that these works would commence at the start of the main construction period and therefore be completed earlier than the overall end date. The works in this section would be more localised in nature.

It is intended that placement of the rock toe would commence at the easternmost end of the Scheme, and work towards the west. Subject to agreement with DCC, areas would be fully completed and handed back by the contractor in stages (sectional completion approach) to reopen public access to the beach and promenade to these sections

### 2.3.3 Enabling works

Initial/enabling works would be considered likely to include the setup and provision of accesses to construction compounds and work locations, and mobilisation of site welfare facilities. Key enabling works include:

- Access compounds are proposed to be at Quay Street, Marine Drive, John Street and limited welfare units and parking along the Central Access.
- Access to the centre of the scheme would be required in the vicinity of the Rhyll Events Arena.
- It is assumed that the existing access from Marine Drive opposite Grosvenor Road (currently in use to access the East Rhyll project compound) would be retained and reused for access from the road to the promenade. A satellite compound would be re-established in this area (on a smaller scale to the present East Rhyll project compound).
- Sections of groynes would be removed on the upper beach along the full extents of the Scheme as required to be clear of the permanent works, plus an allowance of an additional 15m is anticipated for working room and to permit free plant access along the beach.
- Temporary slipway accesses would be constructed to allow plant to access the beach. Three access points are expected to be required in the vicinity of the access locations. Due to tidal setting and temporary nature, this slipway will be subject to regular maintenance.

### 2.3.4 Sections A-D: Rock toe scour protection works and wall repairs

Concrete repair works would commence as an early activity, and initially require inspection and confirmation of the defects/repair types and their extents to confirm the work carried out accounts for any recent deterioration.

A variety of concrete repairs would be required. In small areas, as part of the repair, break out of the defect area may be required. Hand arm vibration will be minimised, by using remote control breaker, similar to Brokk 70. Where access is limited, handheld breakers would be used for limited durations.

Fill material would be imported to the beach and used to fill any voids beneath the existing revetment or sea wall.

The concrete repair works would take place at many locations along the frontage in this section and progress along the frontage over a period of time. At the same time, work would commence at discrete locations where existing access steps require extending, or new access points would be created

Following formation of the concrete foundation, it is anticipated that the finished profile of the steps would be installed using pre-cast concrete sections lifted into place with a large (40-50t) tracked excavator. The precast units would be delivered onto the beach using a tractor and trailer unit. Once the concrete repair works and access step extensions have been completed along a section of the frontage, it is possible to commence the main activity of importing and placing rock to provide scour protection in front of the existing defences.

Rock would be imported directly onto the beach and would be stocked such that the quantity is at least four weeks ahead of the placement schedule. Rock would be briefly stored along the beach near the works as it progresses.

Tracked excavators would excavate to formation at the designated shift location. Where required, articulated dumper trucks would be used to move the excavation arisings beyond the extents of the dig where it would remain unless used for backfilling of the toe.

Temporary beach storage would not interfere or block any access to the emergency ramp. Temporary works designs would be required to ensure no risk of undermining the existing structures and this may require sheet piling adjacent to them.

### 2.3.5 Sections E-H: Promenade Raising and New Precast Concrete Stepped Revetment Works

To provide improved coastal protection the Promenade crest height needs to be raised with a new stepped revetment in front of the promenade and rear seawall on the landward side of the promenade. The rear seawall would also act to retain the promenade fill material and prevent any unacceptable loading onto adjacent existing structures. The main activities, in approximate sequence, are as follows:

- Construction of temporary access for plant onto/along the beach;
- Installation of sheet piles at the new revetment toe;
- Construction of sheet pile capping beam – tidal concrete operation;
- Widening of promenade to the seaward side to provide an access/working platform;
- Excavation/perforation of the existing promenade surfacing and breaking down top section of existing seawall;
- Installation of precast wall sections to the rear of the promenade (new retaining/seawall) and in-situ concrete footing;

- Raising promenade level and installation of crane platform to suit the new promenade crest level;
- Building up of revetment profile – drainage layer and concrete blinding for pre-cast concrete revetment steps;
- Installation of pre-cast revetment steps – likely to be by 120t crawler crane working from the promenade crane platform;
- In-situ concrete works to complete the new promenade surfacing and construction of beam to tie the revetment into the promenade; and
- Install signage, lines, replacement lighting columns, guardrails/handrails and other street furniture.

#### 2.3.5.1 Beach access for Revetment Construction

A beach access ramp would be constructed. A very early activity would be the demolition of existing kiosk buildings close to the SeaQuarium/Central Car Park to enable construction of the slipway in this area and generate necessary working room.

Beach sand levels will be managed to endure safe accessways and operational platforms for tracked plant.

At the end of each tidal shift, plant operating on the beach would be removed to a safe holding area during the high tide and until the next shift. The nearest temporary slipway to the location of works on the beach would be used and the holding area is likely to be in the vicinity of the John Street access or the Central access, near the slipway.

#### 2.3.5.2 Piling and Capping Beam

Prior to the piling operation commencing it would be necessary to reduce the beach level in the vicinity of the new revetment toe and the excavation arisings would be placed beyond the extents of the working area where it would remain on the beach. Reduction of the beach levels would take place in advance of the piling operation to reduce excavation time where required during the tidal shifts.

Piling would be installed using a leader, typically fitted with a variable vibratory hammer or a vibratory hammer attached to a tracked excavator depending on ground conditions. At regular intervals a pair of piles would be driven deeper to facilitate gabion basket drainage installation as described below.

Following the piling operation, a second gang would excavate around the piles to the formation level of the pile capping beam. At gabion basket locations the deeper excavation would be completed, and stone filled gabion baskets installed above the lowered sheet piles to permit adequate drainage.

Pre-fabricated reinforcement cages would be lifted into the formwork using an excavator and concrete poured. Alternatively, BB is currently investigating the use of fibre reinforced concrete to eliminate the need for steel reinforcement and simplify the works in a tidal environment. This is on the basis that the capping beam level is below the long-term possible beach draw-down level and therefore it would not be subject to abrasion and have any potential to release fibres into the marine environment.

#### 2.3.5.3 Promenade Widening, Rear Sea Wall, and Crane Platform Construction

The promenade would be temporarily widened throughout most of the revetment construction area by excavating the beach material to a suitable formation level and backfilling with a reinforced earth structure with retained face, e.g. using gabion basket or wire mesh that are tied to the earth reinforcement.

It is likely that a section of the top of the existing seawall would need to be demolished (likely munched down with any existing reinforcement cut) and sections of the existing promenade surfacing and fill removed. This would enable the working platform to be constructed at the required level (below the existing promenade level) with sufficient width for following construction activities. Excavator mounted breakers would be used to remove sections of the existing promenade concrete slab and provide drainage routes where required through any sections that do not require removal.

Precast wall units would be delivered to site and lifted into position (likely using a crawler crane). The rear wall foundations would be backfilled and the promenade and temporary widening area filled with additional material, compacted in layers, to bring the ground level up above the current promenade level.

The existing kiosk in the vicinity of SC2 would need to be demolished prior to construction of the rear seawall.

#### 2.3.5.4 Revetment Installation

When the preceding works are sufficiently progressed, the revetment installation operation would commence. This work would be carried out sequentially and is anticipated to work from the east back towards the main access point near John Street with all material deliveries from the John Street access.

The beach material would be excavated behind the previously completed sheet pile capping beam to the suitable profile for the drainage layer. Suitable drainage material would be imported and placed. Then working up towards the promenade level, fill material would be imported and installed above existing beach level, and further drainage material placed to the required profile. The work in progress would be capped with 150mm concrete blinding layer to provide protection from tidal/wave action. It is also likely that transverse walls would be constructed perpendicular to the shoreline at regular intervals, either of mass concrete, gabion basket or steel mesh faced retained earth, at regular intervals sufficient to compartmentalise the work in progress and further protect it or limit the extent of any damage from the sea.

The fill and blinding works would be carried out in small sections to suit the working window within the tidal shifts. This activity would be carried out to suit the rate of progress of the follow-on precast installation activity, so that the completed and blinded profile is no more than 2 or 3 shifts ahead of the precast placement. This would further limit the risk of damage/rework.

Precast revetment units would be delivered by tractor and trailer unit to the promenade where a 120t crawler crane would lift them and place them into position on the prepared slope. The daily work would be planned to confirm that at the end of each shift the revetment units are completed to a sufficiently high level to reduce the risk of damage or movement due to tide/waves. Grouting between the blinding and precast units would be carried out as soon as practicable after placement.

#### 2.3.5.5 Promenade Concrete and Finishing Works

When the revetment installation activity has progressed sufficiently and is clear of the Central Access area (in the vicinity of the SeaQuarium and Central Car Park entrance), the follow-on concreting/surfacing works at promenade level would commence.

Once the heavy civil engineering works are complete, the street furniture, signs, lighting, landscaping, road markings etc would be installed throughout sections E to H.

#### 2.3.6 Construction Compounds

Currently the following compounds are anticipated:

- Quay Street: Sufficient parking for workers, office accommodation (potentially double-storey), welfare units for eating/kitchen, toilets, drying room, stores, materials storage area; storage areas for precast units ideally on trailers, storage for other materials, mains electric and water connection, potentially assembly of reinforcement cages.
- Marine Drive: Sufficient Parking for workers associated with rock scour protection works (qty TBC), limited welfare facilities and potentially small satellite office, very limited material storage (e.g. geotextile for rock works, potentially a small quantity of precast for steps. – assumed rock deliveries would pass straight through and onto beach), potential wheel wash facility, concrete washout.
- John Street: Limited welfare units, unlikely to have any storage or very minimal/short term, potentially holding area for deliveries to promenade if congested, concrete washout, parking for beach plant off-shift. N.B. very little room available, may need to make use of promenade or adjacent areas to the west – included these in red line boundary.
- Central Access: Limited welfare units, concrete washout, parking for beach plant off-shift. Potential for these uses to extend to area to west of RNLI station (including some material storage) and would not rule out use of that area.

### 3 Flood Zone Category and Justification

Planning Policy Wales (PPW) edition 11 was published in 2021 and outlines the land use planning policies and the planning system for the Wales. The PPW is supported by the Technical Advice Notes (TANs), including TAN15: Development and Flood Risk. The TAN15 is guidance for Welsh local planning authorities to reduce flood risk and encourage developments away from high risk areas. The TAN 15 provides a framework within which risks arising from both river and coastal flooding, and from additional run-off from development in any location, can be assessed. The TAN15 has been followed within this FCA to justify the proposed Scheme. An update to TAN 15 is anticipated in September 2021, given the timing of the detailed design process, the current TAN 15 has been used.

Within the TAN15, Wales is divided into three flood zones, with flood zone A representing little to no risk to flood zone C being the areas of higher flood risk. Flood zone C is then further subdivided into flood zones C1 and C2, indicating if the area has defences or not. Table 3.1, taken from the TAN15, outlines these flood zones.

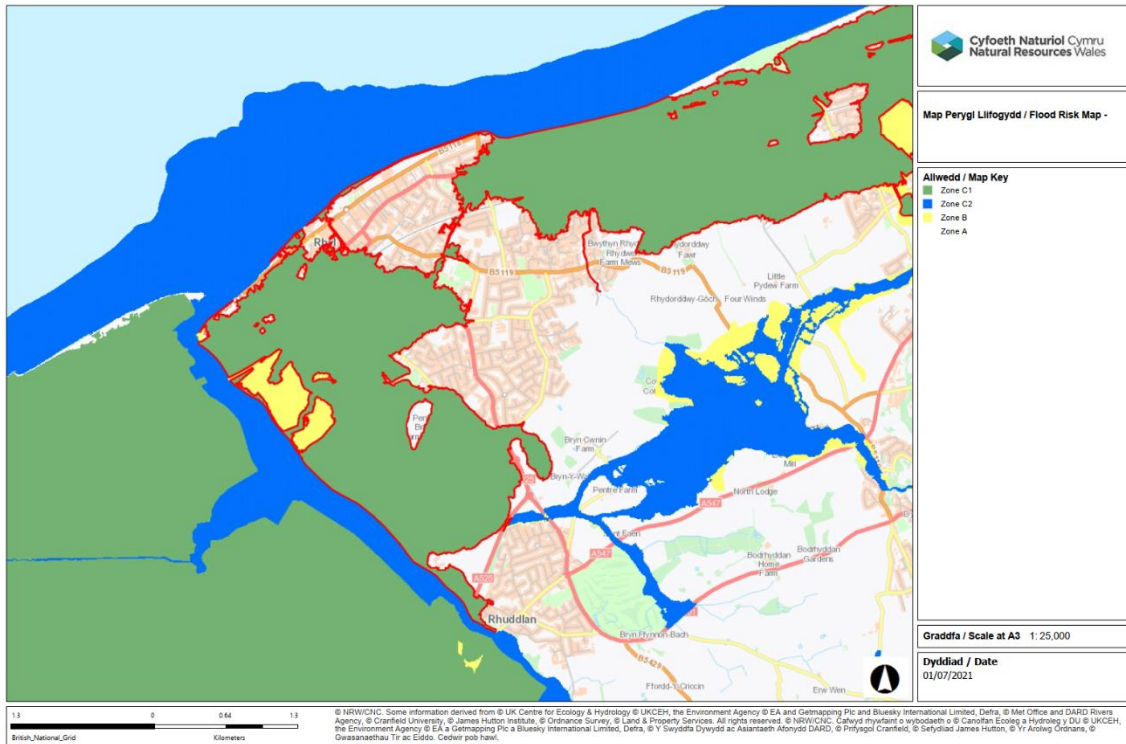
**Table 3.1: Flood Zone Classification**

Description of zone	Zone	Use Within the Precautionary Framework
Considered to be at little or no risk of fluvial or tidal/coastal flooding.	A	Used to indicate that justification test is not applicable and no need to consider flood risk further
Areas known to have been flooded in the past evidenced by sedimentary deposits.	B	Used as part of a precautionary approach to indicate where site levels should be checked against the extreme (0.1%) flood level. If site levels are greater than the flood levels used to define adjacent extreme flood outline there is no need to consider flood risk further.
Based on Environment Agency extreme flood outline, equal to or greater than 0.1% (river, tidal or coastal)	C	Used to indicate that flooding issues should be considered as an integral part of decision making by the application of the justification test including assessment of consequences.
Areas of the floodplain which are developed and served by significant infrastructure, including flood defences.	C1	Used to indicate that development can take place subject to application of justification test, including acceptability of consequences.
Areas of the floodplain without significant flood defence infrastructure.	C2	Used to indicate that only less vulnerable development should be considered subject to application of justification test, including acceptability of consequences. Emergency services and highly vulnerable development should not be considered.

Source: Welsh Assembly Government, 2004.

Indicated in the Welsh Government Development Advice Map (Figure 3.1), the site is located within Flood Zone C1. This zone is based on the extreme flood outline, greater than or equal to a 0.1% (river, tidal or coastal) Annual Exceedance Probability (AEP).

**Figure 3.1: RhyI Flood Risk Map**



Source: Welsh Government, 2017. (Contains Natural Resources Wales information © Natural Resources Wales and database right. All rights reserved. Some features of this information are based on digital spatial data licensed from the UK Centre for Ecology & Hydrology © UKCEH, the Environment Agency © EA and Getmapping Plc and Bluesky International Limited [2015]. Defra, Met Office and DARD Rivers Agency © Crown copyright. © Cranfield University. © James Hutton Institute. Contains OS data © Crown copyright and database right. Land & Property Services © Crown copyright and database right.)

The Welsh Government's Technical Advice Note 15 - Development and Flood Risk (TAN15) outlines the different land uses to flooding. However, coastal defence schemes are not classified within this land use classification, and therefore fall under 'other' for this FCA assessment. In Section 9 'Summary of Policy Requirements' of the TAN15, development types that are classified as 'other' can be considered within Flood Zone C1. The proposed Scheme by virtue of its purpose must be located within the coastal frontage, and as it is a form of flood defence infrastructure, it is not required to be subjected to the justification test of the TAN15.

## 4 TAN15 Acceptability Criteria

Section 9 'Summary of Policy Requirements' of the TAN15, states that developments classified as 'other' within Zone C1 must satisfy the below Acceptability Criteria:

- Acceptable consequences for nature of use;
- Occupiers aware of flood risk;
- Desirable if effective flood warning and evacuation routes/procedure provided depending on nature of proposal; and
- No increase in flooding elsewhere.

The criteria listed above are reviewed in Section 6 to Section 9. However, the potential sources of flooding are considered in Section 5 to assist with this assessment.

## 5 Potential Sources of Flooding and Probability

### 5.1 Historic Flooding

Along the Rhyl frontage in the past, severe overtopping of the sea defences has resulted in flooding of properties:

- In February 1990, a storm event led to 108 residential properties flooding caused by the overtopping of sea defences in Rhyl and Prestatyn; and
- In December 2013, a storm event resulted in significant overtopping of the coastal defences at Rhyl, causing flooding to residential properties, with significant damage to the coastal infrastructure. This event led to the deep flooding of 130 residential properties, and 400 people had to be evacuated from their homes, and others had to be rescued by boat. This flood event predominantly impacted East Rhyl but also affected the Central Rhyl promenade.

### 5.2 Coastal Flooding

As the proposed Scheme is a coastal flood defence infrastructure project the primary source of flood risk within the project area is therefore from coastal flooding.

The extreme wave and water levels for the Scheme area were extracted from Modelling completed by Mott MacDonald in 2021<sup>3</sup> at a location offshore from Central Rhyl. This modelling used offshore wave conditions from the Met Office Wave Watch III wave hindcast model<sup>4</sup> and extreme water levels from the Coastal Flood Boundary conditions dataset<sup>5</sup>. The estimated CFB water levels were corrected to the year 2020 (baseline) using UKCP18 RCP 8.5 at the 70th percentile<sup>6</sup>.

The sea-level rise (SLR) projection used in the study follows UKCP18 guidelines for RCP8.5 at the 70th percentile<sup>6</sup>. The SLR values obtained for 2070 and 2120 are shown in Table 5.1.

**Table 5.1: Sea level rise based on UKCP18 RCP8.5 at the 70th percentile**

Year Estimated SLR (m)	Year Estimated SLR (m)
2020	0
2070	0.36
2120	0.87

Source: Mott MacDonald 2021

A Multivariate analysis (MVA) was undertaken using the wave and water level data to generate a 10,000 year synthetic record of water level, Hs, Tp, MWD, wind speed and wind direction. Wave transformations were required to provide extreme waves and water levels for the overtopping and design calculations at the sea defences. The two-dimensional MIKE21 FMSW [5] model was used to transform offshore waves to the nearshore, and a higher spatial resolution one-dimensional wave transformation model (LITDRIFT) was used to transform waves to the toe of the coastal defence structures.

<sup>3</sup> Mott MacDonald 2021 "CR-MMD-00-00-RP-MO-7010 Central Rhyl Coastal Defence Wave Modelling Report"

<sup>4</sup> <https://www.metoffice.gov.uk/research/weather/ocean-forecasting/ocean-waves>

<sup>5</sup> UK Environment Agency, "Coastal flood boundary conditions for the UK: 2018 update," Bristol, UK, 2018

<sup>6</sup> M. D. Palmer et al., "UKCP18 Marine Report," United Kingdom, 2018.

As shown on the cross-section drawings in Appendix B, the proposed Scheme will have the revetment set at +7.85m OD and a rear crest wall at +9.25m OD.

The wave overtopping of the coastal defences along this frontage has been assessed. Both mean overtopping discharge ( $q$ ), which is the average overtopping from all waves, and individual maximum overtopping volume ( $V_{max}$ ), which is the maximum discharge from a single wave, were assessed in the Mott MacDonald Overtopping Assessment (Appendix C). These values are presented in Table 5.2 and Table 5.3 below which presents the current day overtopping with no scheme and Table 5.4 the overtopping with the scheme in place.

**Table 5.2: Wave overtopping of existing defences: Sections A to D – Future 100 year (2120)**

	Overtopping Discharge $q$ (l/s/m)			
	Section A	Section B	Section C	Section D
1 in 1000 yr. RP (0.1% AEP)	276	397	150	2,205
1 in 200 yr. RP (0.5% AEP)	163	316	106	1,850
1 in 75 yr. RP (1.3% AEP)	100	270	84.4	1,580
1 in 30 yr. RP (3.3% AEP)	67.6	220	66.3	1,370
1 in 5 yr. RP (20% AEP)	33.1	137	34.5	1,080
1 in 2 yr. RP (50% AEP)	20.1	94.7	21.2	932

**Table 5.3: Wave overtopping of existing defences: Sections E to H - Future 100 year (2120)**

	Overtopping Discharge $q$ (l/s/m)			
	Section E	Section F	Section G (stepped revetment)	Section H
1 in 1000 yr. RP (0.1% AEP)	142	472	772	68.3
1 in 200 yr. RP (0.5% AEP)	102	350	590	39.3
1 in 75 yr. RP (1.3% AEP)	74.7	277	483	32.4
1 in 30 yr. RP (3.3% AEP)	56	221	396	26.6
1 in 5 yr. RP (20% AEP)	27.4	133	244	18
1 in 2 yr. RP (50% AEP)	16.8	94.8	168	14

**Table 5.4: Overtopping results for the proposed concept design for sections E – H**

	Overtopping Discharge $q$ (l/s/m)			Individual Wave Discharge ( $V_{max}$ )		
	2020	2070	2120	2020	2070	2120
<b>1 in 1000 yr. RP (0.1% AEP)</b>	0.55	5.49	24.4	446	2,284	5,902
<b>1 in 200 yr. RP (0.5% AEP)</b>	0.11	2.85	9	164	1,775	1,285
<b>1 in 75 yr. RP (1.3% AEP)</b>	0.02	1.04	5.49	53	919	2,167
<b>1 in 30 yr. RP (3.3% AEP)</b>	0	0.33	2.93	20	81	1,759
<b>1 in 5 yr. RP (20% AEP)</b>	0	0.04	0.72	0	16	153
<b>1 in 2 yr. RP (50% AEP)</b>	0	0.01	0.28	0	5	75

Note: The  $V_{max}$  value for the 0.5% AEP 2120 has been checked and the lower  $V_{max}$  is as a result of the wave and water level combination producing a flatter wave distribution.

Following the criteria set out in the EurOtop II Manual<sup>7</sup> and in the EurOtop Manual<sup>8</sup>, the overtopping limits for structural designs and to people and vehicles were reviewed. Table 5.5 presents these overtopping discharge limits.

**Table 5.5: Recommended Discharge Limits for Assets and Users**

Type	Limit	Return Period [yrs]
<b>Surface</b>	<b><math>q</math> [l/m/s]</b>	
Paved	200	100
Grass	50	100
<b>User</b>	<b><math>V_{max}</math> [l/m]</b>	
Pedestrian	600	1 - 10
Vehicle	2,000	1 - 10

Source: EurOtop II Manual and EurOtop Manual

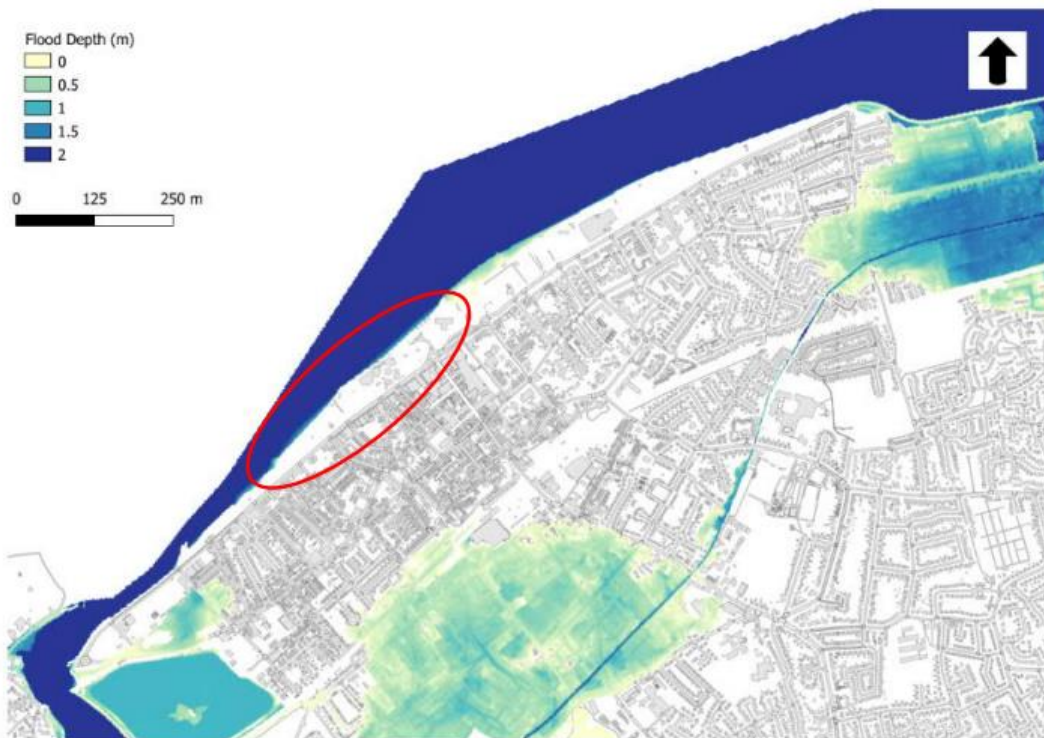
The proposed Scheme will have the revetment set at +7.85m OD and a rear crest wall at +9.25m OD. From the overtopping assessment it can be seen that the paved area of the promenade will not be damaged during the lifetime of the project. However, mitigation measures, such as road / promenade closures during severe storms will be required, along with barriers to prevent vehicle and pedestrian access along the promenade.

The results of the overtopping assessment were modelling in TUFLOW to map the inundation. Figure 5.1 presents the results of the inundation modelling for the overtopping from Sections E – H under the 0.5% AEP in 2120.

<sup>7</sup> Van der Meer, J., Allsop, N., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., . . . Zanuttigh, B. (2018). EurOtop. Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application. [online]. Available at: [www.overtopping-manual.com](http://www.overtopping-manual.com) [Accessed 13 July 2020].

<sup>8</sup> Pullen, T., Allsop, N. W., Bruce, T., Kortenhaus, A., Schüttrumpf, H., & Meer, J. W. (2007). EurOtop. Wave Overtopping of Sea Defences and Related Structures: Assessment Manual. Die Küste.

**Figure 5.1: Section E-H (Geometry Variation 3) Only – 0.5%AEP in 2120**



Source: Mott MacDonald, 2021

### 5.3 Fluvial Flooding

There are no fluvial watercourses within the site boundary. However, there are two in the vicinity of the Scheme:

- The Rhyl Cut, (classified as a Main River by NRW flood mapping<sup>9</sup>) is present to the immediate east of the Scheme area; and
- The Clwyd, (classified as a Main River by NRW flood mapping<sup>9</sup>) is present to the immediate west of the Scheme area

The Clwyd River has been included in the inundation modelling. Assessment of the Rhyl Cut flood risk was undertaken as part of the Central Rhyl and Prestatyn Coastal Defences OBCs<sup>10</sup>, within a combined hydraulic model<sup>11</sup>. This modelling shows that assets within the Central Rhyl study area are not at flood risk from Rhyl Cut or Prestatyn Gutter.

This Scheme focuses on the impact of sea overtopping on the frontage. The diversion of the existing outfalls along the beach within the Scheme will allow the river to flow as is and therefore not result in an increase in fluvial flooding.

<sup>9</sup> Natural Resources Wales (undated) Welsh Government Advice Maps [Online] Available at: [https://maps.cyfoethnaturiolcymru.gov.uk/Html5Viewer/Index.html?configBase=https://maps.cyfoethnaturiolcymru.gov.uk/Geocortex/Essentials/REST/sites/Flood\\_Risk/viewers/Flood\\_Risk/virtualdirectory/Resources/Config/Default&layerTheme=3](https://maps.cyfoethnaturiolcymru.gov.uk/Html5Viewer/Index.html?configBase=https://maps.cyfoethnaturiolcymru.gov.uk/Geocortex/Essentials/REST/sites/Flood_Risk/viewers/Flood_Risk/virtualdirectory/Resources/Config/Default&layerTheme=3) Accessed November 2020

<sup>10</sup>

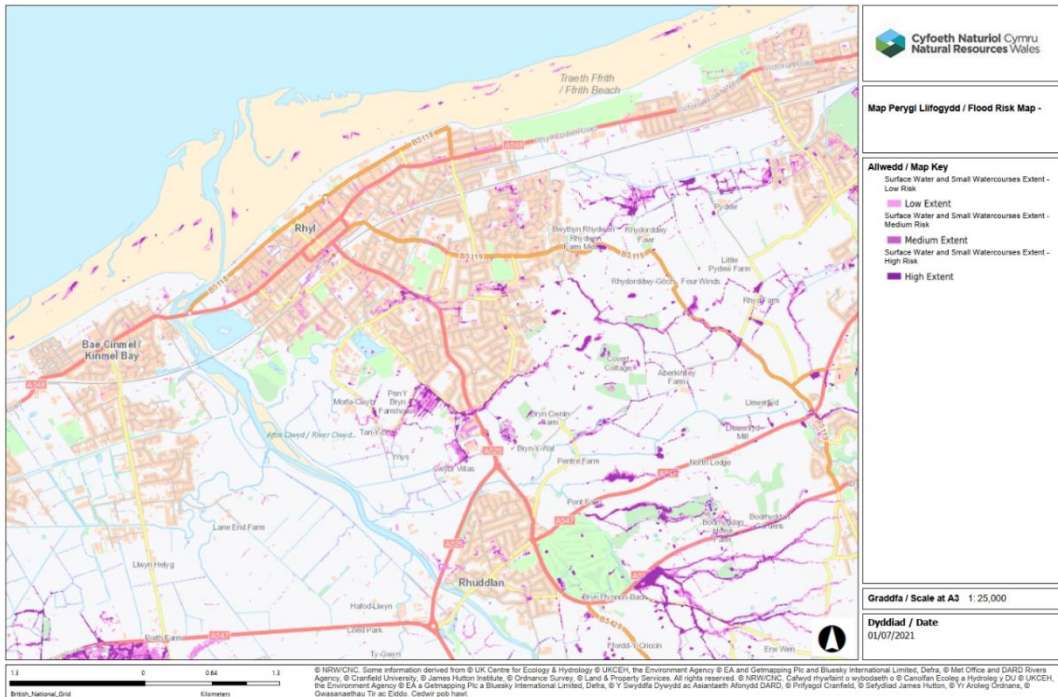
Central Rhyl Coastal Defence Scheme Outline Business Case, JBA, October 2019

<sup>11</sup> Central Rhyl and Central Prestatyn Inundation Modelling Report, JBA, May 2019

## 5.4 Surface Water Flooding

The NRW surface water flood risk maps show that there is some surface water flood risk in the area of interest (Figure 5.2). These areas of surface water appear to be in low-lying areas of the town.

**Figure 5.2: Central Rhyl surface water flood risk**



Source: Welsh Government Development Advice, 2020

## 5.5 Construction Flood Risk Management

The Contractor would produce a Construction Flood Risk Management Plan to set out the methodology to be followed during construction to ensure coastal and flood protection of the site is maintained throughout the construction phase.

This plan would also set out the emergency access and egress procedures and specific site precautions necessary to protect both human life and the environment in the case of storm overtopping events occurring during the construction period.

The placement of stockpiled rock armour immediately seaward of where it is to be used within the revetment would help to dissipate wave energy and provide some limited additional protection during any storm events.

## 5.6 Potential Flooding impacts of Proposed Scheme

The impacts on flood risk during the construction of the Scheme are considered to be mostly negligible. It is also assumed that the Contractor would develop and implement a robust flood management plan during the works. The construction site, equipment and timing of construction activity would need to consider the risk of flooding from tidal and wave conditions as best practice.

**Table 5.6: Potential impacts of proposed Scheme**

Effects	Construction	Operation
Potential Insignificant Effects	<p>There is a risk that during the scheme construction the removal of existing assets could increase the flooding to the properties in the vicinity of the scheme. It is assumed the Contractor would have a flood risk management plan in place, works would not be undertaken in stormy weather and the scheme will be constructed in phases.</p>	<p>Ongoing reviews of the Flood Management Plan required and updated where any changes to guidance or climate change projections occur.</p>
Potential Significant Effects	<p>Construction site and equipment could be at risk of getting inundated from overtopping. There is a risk that during the scheme construction the removal of existing assets could increase the flooding to the highway in the vicinity of the scheme. It is assumed the Contractor would have a flood risk management plan in place, works would not be undertaken in stormy weather and the scheme will be constructed in phases.</p>	<p>The installation of the new coastal defences strongly complies with policy objectives. Overall, the impact of the Scheme is considered to be significantly beneficial as it would provide additional tidal flood protection for Central RhyI.</p>

Source: Mott MacDonald Ltd, 2021

## 6 Acceptable Consequences for Nature of Use

Section A1.14 of the TAN15 outlines that there is 'a frequency threshold of flooding below which flooding of development should not be allowed.'<sup>2</sup> TAN15 outlines an indicative guidance on what the frequency threshold might be for different type of developments, in terms of annual probability of occurrence.

**Table 6.1: Flood Frequency Thresholds from TAN15**

Type of Development	Threshold Frequency (Yrs)	
	Fluvial	Tidal
Residential	1%	0.5%
Commercial/ retail	1%	0.5%
Industrial	1%	0.5%
Emergency services	0.1%	0.5%
General infrastructure	1%	0.5%

Source: Welsh Assembly Government, 2004

As shown in Table 6.1, no flooding should occur during a 0.5% (1 in 200-year RP) for general infrastructure over the lifetime of the proposed Scheme. As presented in Section 5, the proposed Scheme between Section E – H will not flood due to high water levels or extreme wave height. Within Section D, there is a residual flood risk to infrastructure as no increase to the wall heights/revetments are currently planned within this section.

From A1.15 of the TAN15, beyond the threshold frequency outlined above, the proposed development would be expected to flood under extreme conditions. Table 6.2 presents the indicative guidance from the TAN15 on what is considered to be tolerable flood conditions for different types of developments.

**Table 6.2: Tolerable Conditions from TAN15**

Type of Development	Maximum Depth of Flooding (mm)	Maximum Rate of Rise of Floodwaters (m/hr)	Maximum Speed of inundation (of flood risk area (hrs))	Maximum Velocity of Floodwaters (m/s)
Residential (habitable rooms)	600	0.1	4	0.15
	600			0.3
Commercial/ retail	600	0.3	2	0.15
	600			0.3
Industrial	1000	0.3	2	0.3
	1000			0.45
Emergency services	450	0.1	4	0.15
	600			0.3
General infrastructure	600	0.3	2	0.3
	600			0.3

Source: Welsh Assembly Government, 2004

As outlined in Section 5 the results show that residual overtopping will still occur and impact the promenade area. This will exceed the discharge limit for users under a number of return periods for both present day and in 2120. However, overtopping will not exceed the discharge limit for a paved surface, such as the paved promenade. Therefore, the paved promenade is not likely to be damaged during extreme events.

Mitigation measures for the risks to pedestrians and vehicles are discussed and outlined in Section 7.

## 7 Occupiers Awareness of Flood Risk and Flood Warnings Procedures

As this is a coastal defence Scheme, there will be no direct resident or business occupants to inform in the event of a flood warning being issued.

With the Scheme in place there are anticipated to be no changes to the current flood management which includes:

- Event forecasting: A large proportion of Rhyl is located within NRW defined Flood Warning areas ('Prestatyn and Rhyl' and 'Clwyd Left Bank'). A coastal flood forecasting system is currently run by NRW, and if coastal conditions exceed established thresholds an alert or warning is issued; and
- Event response: Upon receipt of an NRW coastal alert/warning, or through anecdotal or observed wave overtopping by emergency responders.

## 8 Flood Resilient Design

The primary flood resilient design aspect of the Scheme is the construction of the concrete stepped revetment, the raising of the promenade (where required) and the construction of the flood wall. The proposed Scheme will have the revetment set at +7.85m OD with a 1:2 slope and a rear wall crest at +9.25m OD, which will include a bullnose geometry. Raising these structures will reduce the current frequency of coastal flooding and overtopping volumes.

The current promenade falls toward the sea wall / steps and the runoff discharges directly on to the beach. It is proposed in the new layout that this principle is maintained, and an approximate 1:40 fall across the proposed promenade is incorporated towards the sea for 'over the edge' drainage.

Along the promenade there are a number of locations where surface water drainage from the Children's Village discharges on to the promenade at ground level and runs across the promenade and through holes in the seawall. This will be accommodated within the proposed design.

Currently seven flood gates are proposed to be incorporated along the rear promenade wall to ensure a consistent flood defence level at the rear of the promenade. The flood gates are required to maintain the existing pedestrian and vehicle access points to / from the promenade.

## 9 No Increase in Flooding Elsewhere

The proposed coastal defences are at risk of overtopping under various scenarios depending on the sections (see section 5 for further information). The proposed defences will improve the level of protection from present day levels for the Central Rhyl frontage.

Existing drainage outfalls through the seawall will be maintained by extending outfalls through the proposed concrete stepped revetment and rock armour toe. The promenade will be profiled to encourage drainage out to sea, and kerbside drains will direct surface water flooding through the existing outfalls.

Additionally, the impacts on flood risk during the construction of the Scheme are considered to be manageable. The construction site, equipment and timing of construction activity would need to consider the risk of flooding from tidal and wave conditions as best practice.

## 10 Summary and Conclusions

The proposed Scheme comprises of coastal defence works in the form of a new rock revetment and concrete stepped revetment, with scour protection in other areas where current overtopping risks are not as high. The coastal defences must be located along the coastal frontage by virtue of their purpose.

The proposed Scheme will have the revetment set at +7.85m OD and a rear wall crest at +9.25m OD, which will include a bullnose geometry. The Scheme will have significant beneficial impacts to tidal flood inundation risk in Rhyll, and will provide a 0.5% AEP standard of protection in 2120 by reducing the overtopping rate to less than 10 l/s/m.

Residual overtopping will still occur and impact the promenade area, however will be an improvement from the current risk at the frontage. This will exceed the discharge limit for users under a number of return periods for both present day and in 2120. However, overtopping will not exceed the discharge limit for a paved surface, such as the paved promenade. Therefore, the paved promenade is not likely to be damaged during extreme events.

The flood risk to pedestrians and vehicles will be mitigated through flood warning systems, barriers to restrict access to the promenade, maintaining existing drainage outfalls through the seawall, and by extending outfalls through the proposed rock revetment. The promenade will be profiled to encourage drainage out to sea, and kerbside drains will direct surface water flooding through outfalls under the promenade out to sea as well.

# 11 Recommendations

The proposed Scheme should maintain the existing drainage outfalls through the seawall and extend the outfalls through the proposed revetment. The promenade will also need to be graded to encourage surface water drainage out to sea and kerb drains to be installed to direct surface water from the promenade highway under the promenade out to sea.

A Flood Management Plan, defining the conditions requiring promenade/road closures during the operational phase and how these are to be ascertained, should be prepared.

A copy of this report should be submitted to the Planning Authority in support of the planning application.

## A. Location Plan and Existing Arrangement

Please find the following drawing: CR-MMD-00-00-DR-EN-3047 - Area of Permanent Construction



**Key to Symbols**

- Scheme red line boundary
- Indicative location of toe of existing seawall or revetment, including buried elements (see note 2)
- Proposed reasonable worst case permanent works footprint for the Scheme

**Notes**

- This drawing is intended only to be used to support the Environmental Impact Assessment for the proposed Central Rhyl Coastal Defences Scheme, and it should not be used for any other purpose. It presents the reasonable worst case footprint for the proposed Central Rhyl Coastal Defences Scheme.
- The indicative location of the toe of the existing coastal defences (which includes elements buried below the beach) shown on this drawing has been derived from site observations, historical photos, historical records and localised trial excavations.
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- Contains OS data © Crown Copyright and database right 2020 Contains data from OS Zoomstack, Maxar, Microsoft

C01	03/09/21	SL	Accepted for EIA	CW	NMP
P01	03/08/21	SL	For Review and Comment	CW	NP
Rev	Date	Drawn	Description	Ch'k'd	App'd

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**Client**

**sir ddinbych denbighshire** County Council

**Balfour Beatty**

**Title**  
Central Rhyl Coastal Defences  
Area of Permanent Construction

Designed	M Cowie	MC	Eng. Check	C Williams	CW
Drawn	S Li	SL	Coordination	N Spofforth	NS
GIS Check	H Wheldon	HW	Approved	N M Price	NMP
Scale at A3	Status	Rev	Security		
1:7,500	A4	C01	STD		

Drawing Number  
**CR-MMD-00-00-DR-EN-3047**



## B. Proposed Scheme Plan and Cross-Sections

Please find included the following drawings:

- CR-MMD-00-ZZ-DR-SC-7001-C02 General Arrangement
- CR-MMD-00-ZZ-DR-SC-7002-C02 General Arrangement
- CR-MMD-00-XX-DR-SC-7101-C02 Coastal Structures Sections
- CR-MMD-00-XX-DR-SC-7102-C02 Coastal Structures Sections
- CR-MMD-00-XX-DR-SC-7103-C02 Coastal Structures Sections
- CR-MMD-00-XX-DR-SP-6001-C02 Prom Structures Preliminary Plan View GA
- CR-MMD-00-XX-DR-SP-6100-C02 Prom Structures Preliminary Plan View GA
- CR-MMD-00-XX-DR-SP-6101-C02 Prom Structures Preliminary Plan View GA

**Safety, Health & Environmental Information**

It is assumed that all works will be carried out by a competent contractor working, where appropriate, to an approved method statement.

**⚠** In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following risks and information which are referenced to the Designers' Hazard Elimination and Management Record referenced CR-MMD-00-00-RP-Z-0007:

**Ref: Hazard/ risk summary**

SC1 Proposed steps become inaccessible or difficult to access from the beach as a result of beach levels dropping below bottom landing levels, resulting in injury as pedestrians try to access or egress.

SC2 Injury through falling due to marine growth on access steps.

SC7 Geometry of existing structures different to existing as-built information or topographical information, or unknown if below ground, which may affect the design and delay works on site.

SC11 Excavation collapse resulting in injury to operators and/or damage to plant.

SC12 UXOs or other buried obstructions are encountered resulting in death and injury.

SC14 Damage to or clashes with existing service infrastructure particularly during excavation and sheet piling works and particularly to drainage assets extending seaward from the existing sea wall.

SC17 Risk of undermining of existing structures, or damage to existing structures.

SC18 Risk of existing seawall instability or collapse due to plant loading on adjacent promenade during construction. This is of particular concern to seawall at Section E.

SC20 Asbestos may be present in structures.

The transitional arrangement between the proposed new revetment and rock scour protection to be confirmed at detailed design. Worst anticipated general arrangement shown

Proposed promenade works not shown. See promenade structures discipline drawings for details.

- Notes**
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  - Unless noted otherwise, all dimensions are in millimetres, all levels in metres above Ordnance Datum (AOD). Do not scale from this drawing, if in doubt ask Mott MacDonald.
  - Further options maybe possible subject to undertaking Detailed Design, though the reasonable worst case arrangement has been shown.
  - Drawings do not show extent and location of services. See utility plans / statutory undertakers plans for service information. Contractor to undertake pre-construction survey to determine full extent of services present in area.
  - Locations of outfalls are shown approximately
  - Mott MacDonald Ltd. has followed accepted procedure in providing the services but given the residual risk associated with any prediction and the variability which can be experienced in flood conditions, Mott MacDonald Ltd. takes no liability for and gives no warranty against actual flooding of any property (clients or third party) or the consequences of flooding in relation to the performance of the service.
  - Existing ground levels are assumed from NRW's Digital Terrain Model (DTM via LiDAR), downloaded in October 2020 (50cm resolution). A topographical survey is required to confirm existing ground levels. Topographical features will be reviewed upon receipt of pending topographical survey, which could affect extent and arrangement of proposed works, for which a variance of 1m has been included.
  - Signage required such as 'do not climb on rock scour protection' locations to be confirmed in detailed design.

Proposals subject to change following, detailed design, consultation and necessary approvals

- Key to symbols**
- Proposed designated points of access to the beach from the promenade
  - Proposed concrete stepped revetment
  - Proposed rock structures (Rock toe scour protection)
  - Toe of proposed concrete stepped revetment
  - Indicative location of toe of existing seawall or revetment

**Reference drawings**

CR-MMD-00-ZZ-DR-SC-7000	Scheme Key Plan
CR-MMD-00-ZZ-DR-SC-7002	Preliminary Design Plan View - General Arrangement Sheet 02 of 02
CR-MMD-00-ZZ-DR-SC-7101	Preliminary Design - Typical Sections Sheet 01 of 03
CR-MMD-00-ZZ-DR-SC-7102	Preliminary Design - Typical Sections Sheet 02 of 03
CR-MMD-00-ZZ-DR-SC-7103	Preliminary Design - Typical Sections Sheet 03 of 03
CR-MMD-00-XX-DR-SP-6001	Prom Structures Preliminary Plan View GA
CR-MMD-00-XX-DR-SP-6100	Prom Structures Preliminary Typical Sections and Details
CR-MMD-00-XX-DR-SP-6101	Prom Structures Preliminary Typical Sections and Details
CR-MMD-00-XX-RP-SC-7005	Visual Condition Survey and Recommendations for Repair Works

Rev	Date	Drawn	Description	Ch'k'd	App'd
C02	02/09/2021	PS	Client Reviewed, minor updates - EIA Accepted	GW	ZH
P03	10/06/2021	PS	Preliminary design for EIA and consents	GW	ZH
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P02	22/01/2021	PS	Acceptance Issue following Clients Comments	FL	ZH
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Reasonable Worst Case Design for EIA Only

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Client

**Balfour Beatty**

**sir ddinbych denbighshire**  
County Council

Title

**Central Rhyll Coastal Defences COASTAL STRUCTURES Preliminary Design Plan View - General Arrangement Sheet 01 of 02**

Designed	P. Kacperek	PK	Eng check	G Wallace	GW
Drawn	P. Steers	PS	Coordination	M. Cowle	MC
Dwg check	G Wallace	GW	Approved	Z. Hutchison	ZH
MMD Project Number	420823	Scale at A1	1:2000	Security	STD
Suitability Description	Stage 3b - Accepted	Suit. Code	A4	Revision	C02
Drawing Number	CR-MMD-00-ZZ-DR-SC-7001				

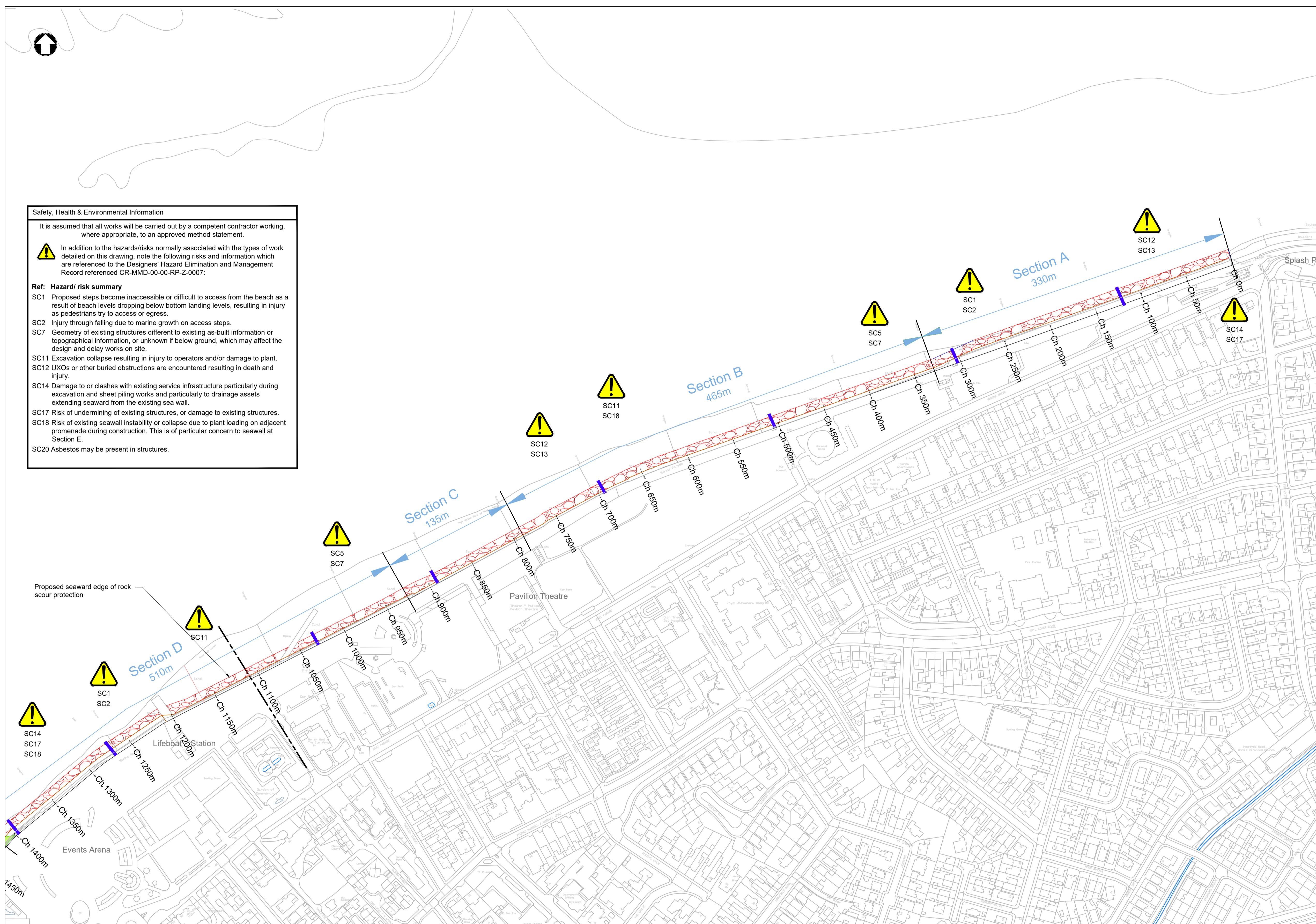
**Safety, Health & Environmental Information**

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- SC7 Geometry of existing structures different to existing as-built information or topographical information, or unknown if below ground, which may affect the design and delay works on site.
- SC11 Excavation collapse resulting in injury to operators and/or damage to plant.
- SC12 UXOs or other buried obstructions are encountered resulting in death and injury.
- SC14 Damage to or clashes with existing service infrastructure particularly during excavation and sheet piling works and particularly to drainage assets extending seaward from the existing sea wall.
- SC17 Risk of undermining of existing structures, or damage to existing structures.
- SC18 Risk of existing seawall instability or collapse due to plant loading on adjacent promenade during construction. This is of particular concern to seawall at Section E.
- SC20 Asbestos may be present in structures.



- Notes**
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  6. Locations of outfalls are shown approximately.
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  9. Signage required such as 'do not climb on rock scour protection' locations to be confirmed in detailed design.

Proposals subject to change following, detailed design, consultation and necessary approvals

- Key to symbols**
- █ Proposed designated points of access to the beach from the promenade
  - █ Proposed rock structures (Rock toe scour protection)
  - █ Indicative location of toe of existing seawall or revetment

- Reference drawings**
- CR-MMD-00-ZZ-DR-SC-7000 Scheme Key Plan
  - CR-MMD-00-ZZ-DR-SC-7001 Preliminary Design Plan View - General Arrangement Sheet 01 of 02
  - CR-MMD-00-ZZ-DR-SC-7101 Preliminary Design - Typical Sections Sheet 01 of 03
  - CR-MMD-00-ZZ-DR-SC-7102 Preliminary Design - Typical Sections Sheet 02 of 03
  - CR-MMD-00-ZZ-DR-SC-7103 Preliminary Design - Typical Sections Sheet 03 of 03
  - CR-MMD-00-XX-DR-SP-6001 Prom Structures Preliminary Plan View GA
  - CR-MMD-00-XX-DR-SP-6100 Prom Structures Preliminary Typical Sections and Details
  - CR-MMD-00-XX-DR-SP-6101 Prom Structures Preliminary Typical Sections and Details
  - CR-MMD-00-XX-RP-SC-7005 Visual Condition Survey and Recommendations for Repair Works

Rev	Date	Drawn	Description	Ch'k'd	App'd
C02	02/09/2021	PS	Client Reviewed, minor updates - EIA Accepted	GW	ZH
P03	10/06/2021	PS	Preliminary design for EIA and Consents	GW	ZH
C01	10/06/2021	PS	Stage 3b accepted	GW	ZH
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**Reasonable Worst Case Design for EIA Only**

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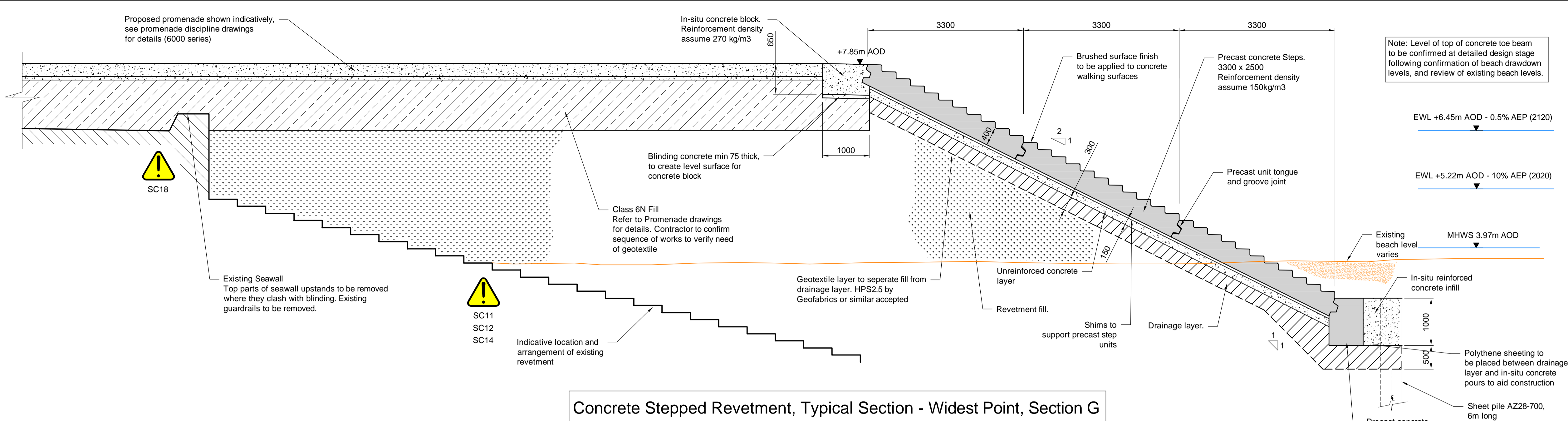
**Balfour Beatty**

**sir ddinbych denbighshire**  
County Council

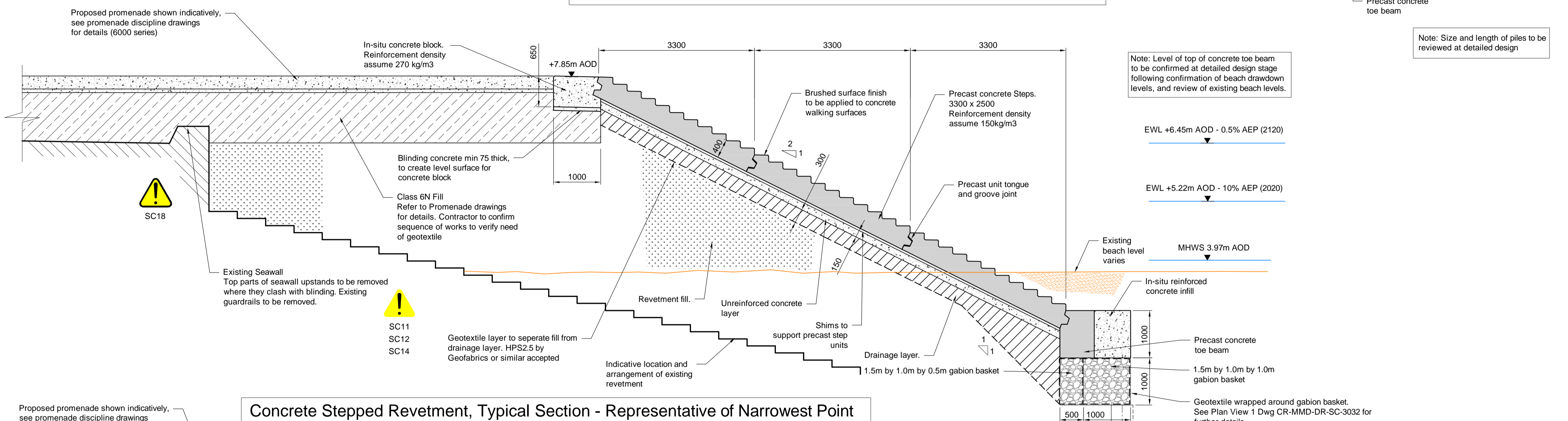
**Title**  
Central Rhyll Coastal Defences  
COASTAL STRUCTURES  
Preliminary Design  
Plan View - General Arrangement  
Sheet 02 of 02

Designed	P. Kacperek	PK	Eng check	G Wallace	GW
Drawn	P. Steers	PS	Coordination	M. Cowle	MC
Dwg check	G Wallace	GW	Approved	Z. Hutchison	ZH
MMD Project Number	420823	Scale at A1	1:2000	Security	STD
Suitability Description	Stage 3b - Accepted			Suit. Code	A4
Drawing Number	CR-MMD-00-ZZ-DR-SC-7002			Revision	C02

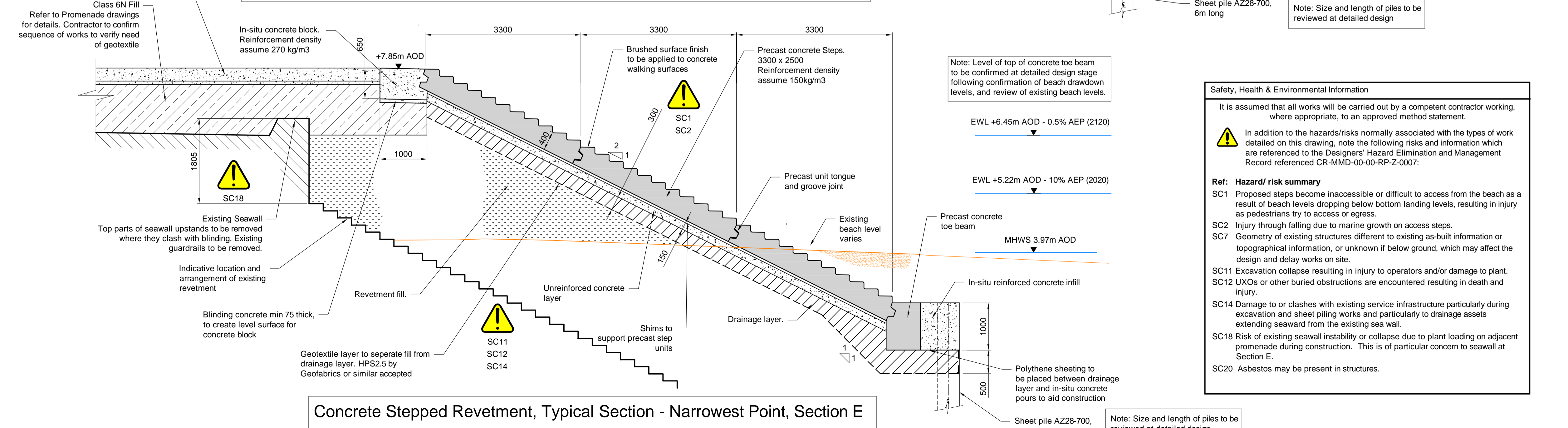
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Concrete Stepped Revetment, Typical Section - Widest Point, Section G



Concrete Stepped Revetment, Typical Section - Representative of Narrowest Point Relative to Existing Structures



Concrete Stepped Revetment, Typical Section - Narrowest Point, Section E

**Notes**

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- Sheet pile toe detail to be confirmed at detailed design stage.
- Mott MacDonald Ltd. has followed accepted procedure in providing the services but given the residual risk associated with any prediction and the variability which can be experienced in flood conditions, Mott MacDonald Ltd. takes no liability for and gives no warranty against actual flooding of any property (clients or third party) or the consequences of flooding in relation to performance of the service. Existing street furniture, guardrails, services and outfalls not shown.
- Tide levels are extracted from Welsh Government: Adopting to Climate Change Guidance, 2017. Extreme water levels are derived from Coastal Flood Boundary Conditions for the UK: 2018 update and UK Climate Projections18 Marine Report, 2018.
- Existing ground levels are assumed from NRW 's Digital Terrain Model (DMT via Lidar) downloaded in October 2020 (50cm resolution). A topographical survey is required to confirm existing ground levels. Topographical features will be reviewed upon receipt of pending topographical survey, which could affect extent and arrangement of proposed works for which a variance of 1m has been included.
- Drawings do not show extent and location of services. See utility plans / statutory undertakers plans for service information. Contractor to undertake pre construction survey to determine full extent of existing services present within working area.

**Proposals subject to change following detailed design, consultation and necessary approvals.**

**Key to symbols**

	Revetment Fill		Class 6N Fill
	Drainage layer		In-situ Concrete
	In-situ reinforced concrete infill		Precast Concrete
	Beach Material		Sheet pile AZ28-700, 6m long

MHSW: Mean High Water Spring  
EWL: Extreme Water Level  
AEP: Annual Exceedance Probability

**Reference drawings**

CR-MMD-00-XX-DR-SC-7000	Scheme Key Plan
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CR-MMD-00-ZZ-DR-SC-7002	Preliminary Design Plan View - General Arrangement Sheet 02 of 02
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CR-MMD-00-XX-DR-SP-6101	Prom Structures Preliminary Typical Sections and Details
CR-MMD-00-XX-RR-SC-7005	Visual Condition Survey and Recommendations for Repair Works

Rev	Date	Drawn	Description	Ch'k'd	App'd
C02	02/09/2021	PS	Client reviewed, minor updates - EIA Accepted	GW	ZH
P04	10/06/2021	PS	Preliminary design for EIA and consents	GW	ZH
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Client

**Balfour Beatty**

**sir ddinbych denbighshire**  
Cynghor County Council

Title

**Central Rhyll Coastal Defences COASTAL STRUCTURES Preliminary Design Typical Sections Sheet 01 of 03**

Designed	P. Kacperek	PK	Eng check	G. Wallace	GW
Drawn	P. Kacperek	PS	Coordination	M. Cowle	MC
Dwg check	G. Wallace	GW	Approved	Z. Hutchison	ZH
MMD Project Number	420823	Scale at 1:50	Security	STD	
Suitability Description	Stage 3b - Accepted	Suit. Code	A4		
Drawing Number	CR-MMD-00-ZZ-DR-SC-7101	Revision	C02		

**Safety, Health & Environmental Information**

It is assumed that all works will be carried out by a competent contractor working, where appropriate, to an approved method statement.

**Hazard/ risk summary**

SC1 Proposed steps become inaccessible or difficult to access from the beach as a result of beach levels dropping below bottom landing levels, resulting in injury as pedestrians try to access or egress.

SC2 Injury through falling due to marine growth on access steps.

SC7 Geometry of existing structures different to existing as-built information or topographical information, or unknown if below ground, which may affect the design and delay works on site.

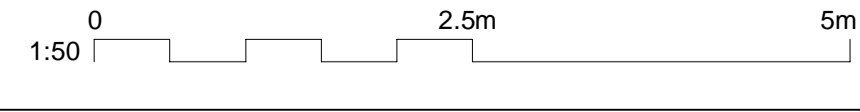
SC11 Excavation collapse resulting in injury to operators and/or damage to plant.

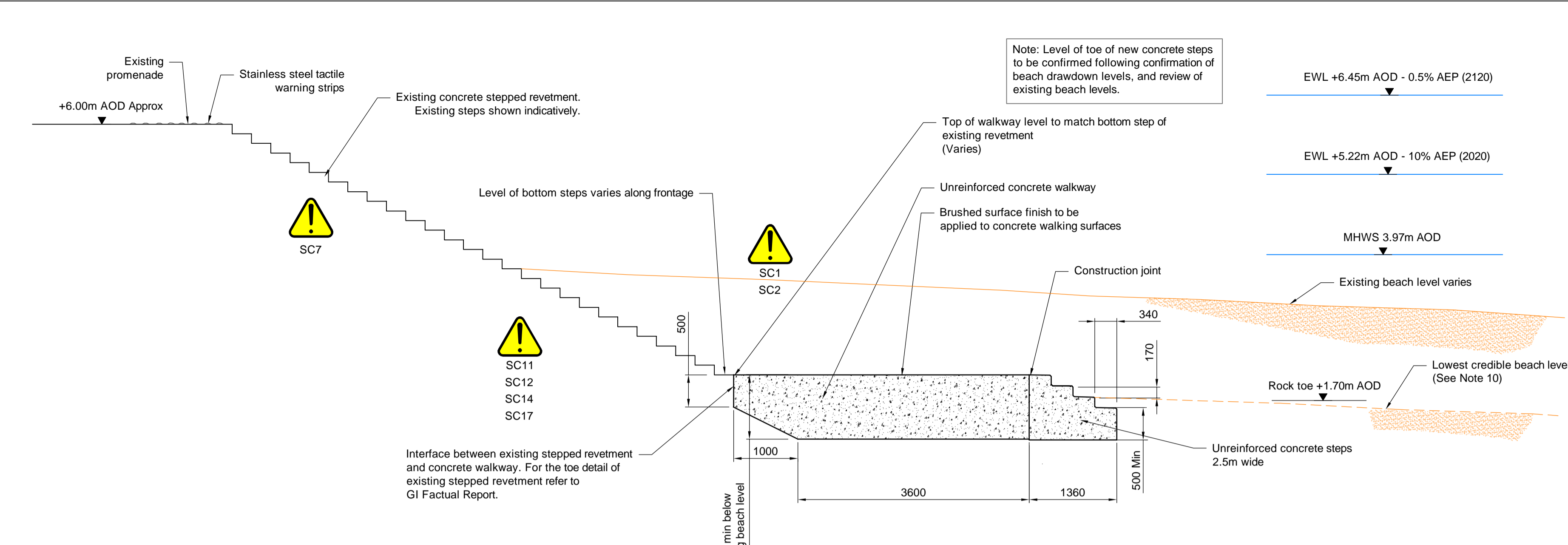
SC12 UXOs or other buried obstructions are encountered resulting in death and injury.

SC14 Damage to or clashes with existing service infrastructure particularly during excavation and sheet piling works and particularly to drainage assets extending seaward from the existing sea wall.

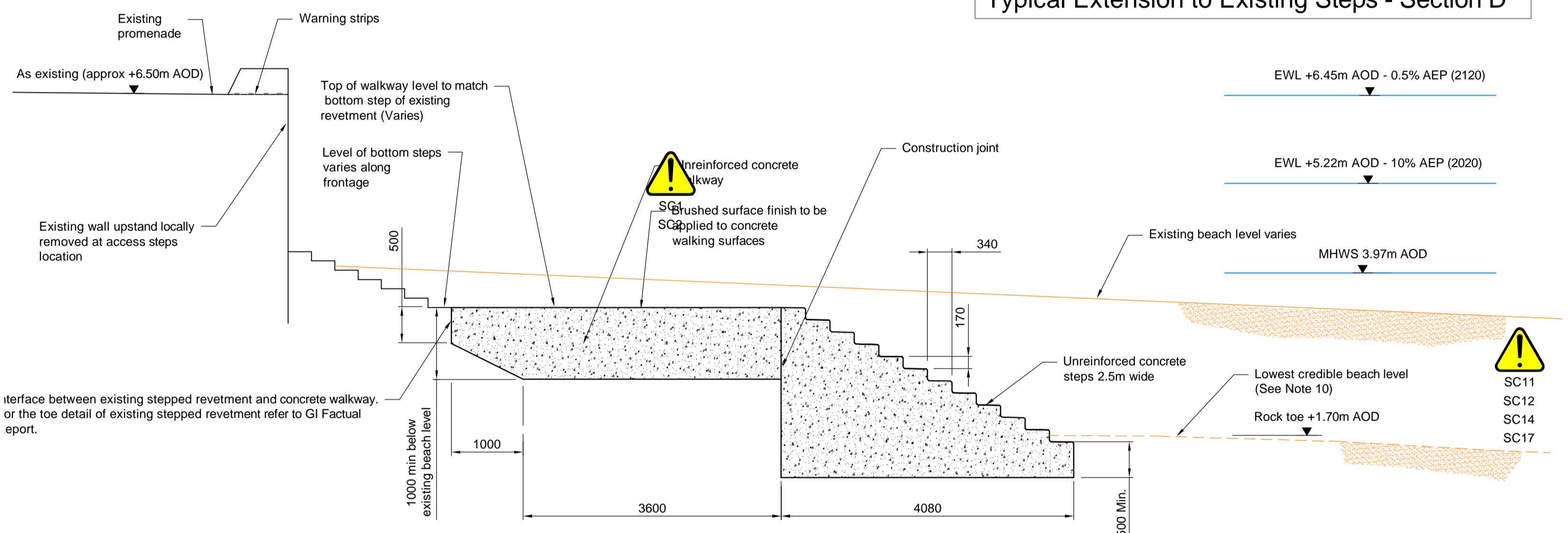
SC18 Risk of existing seawall instability or collapse due to plant loading on adjacent promenade during construction. This is of particular concern to seawall at Section E.

SC20 Asbestos may be present in structures.

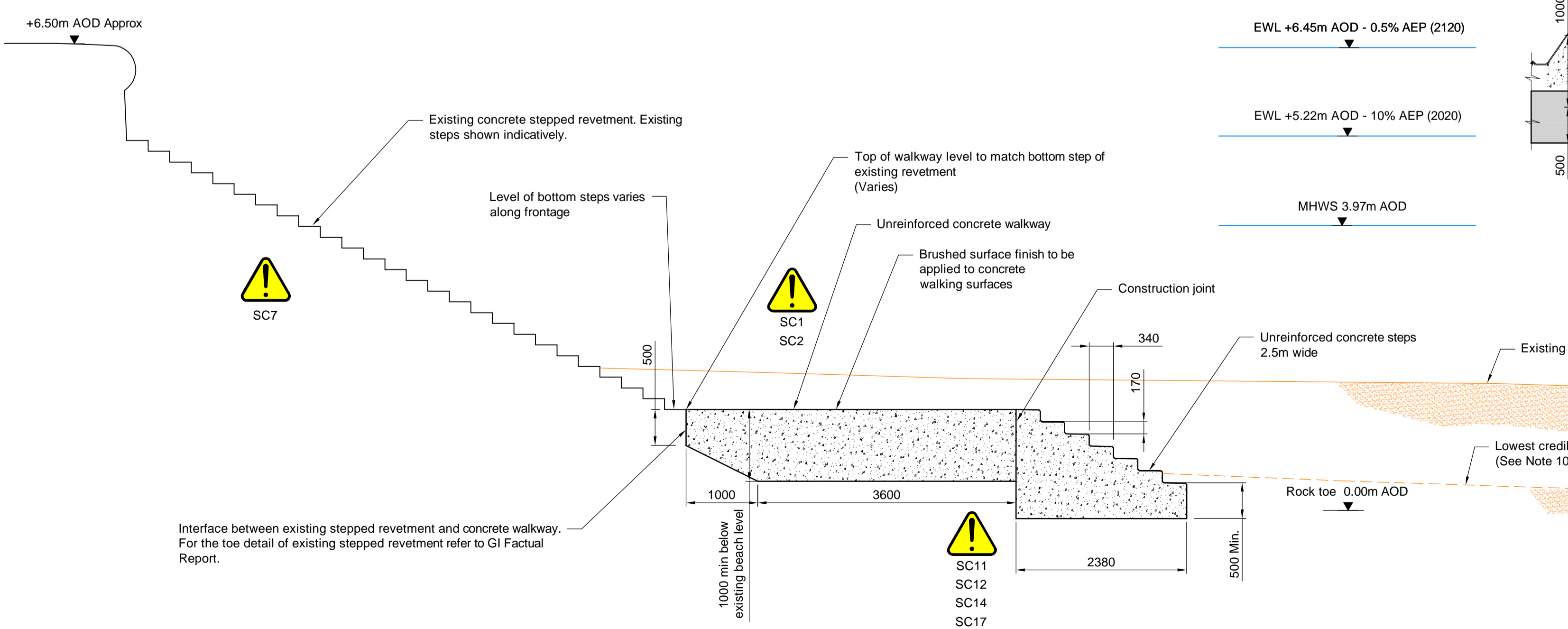




Typical Extension to Existing Steps - Section D



Typical Section of New Steps - Sections B & C



Typical Extension to Existing Steps - Section A

**Safety, Health & Environmental Information**

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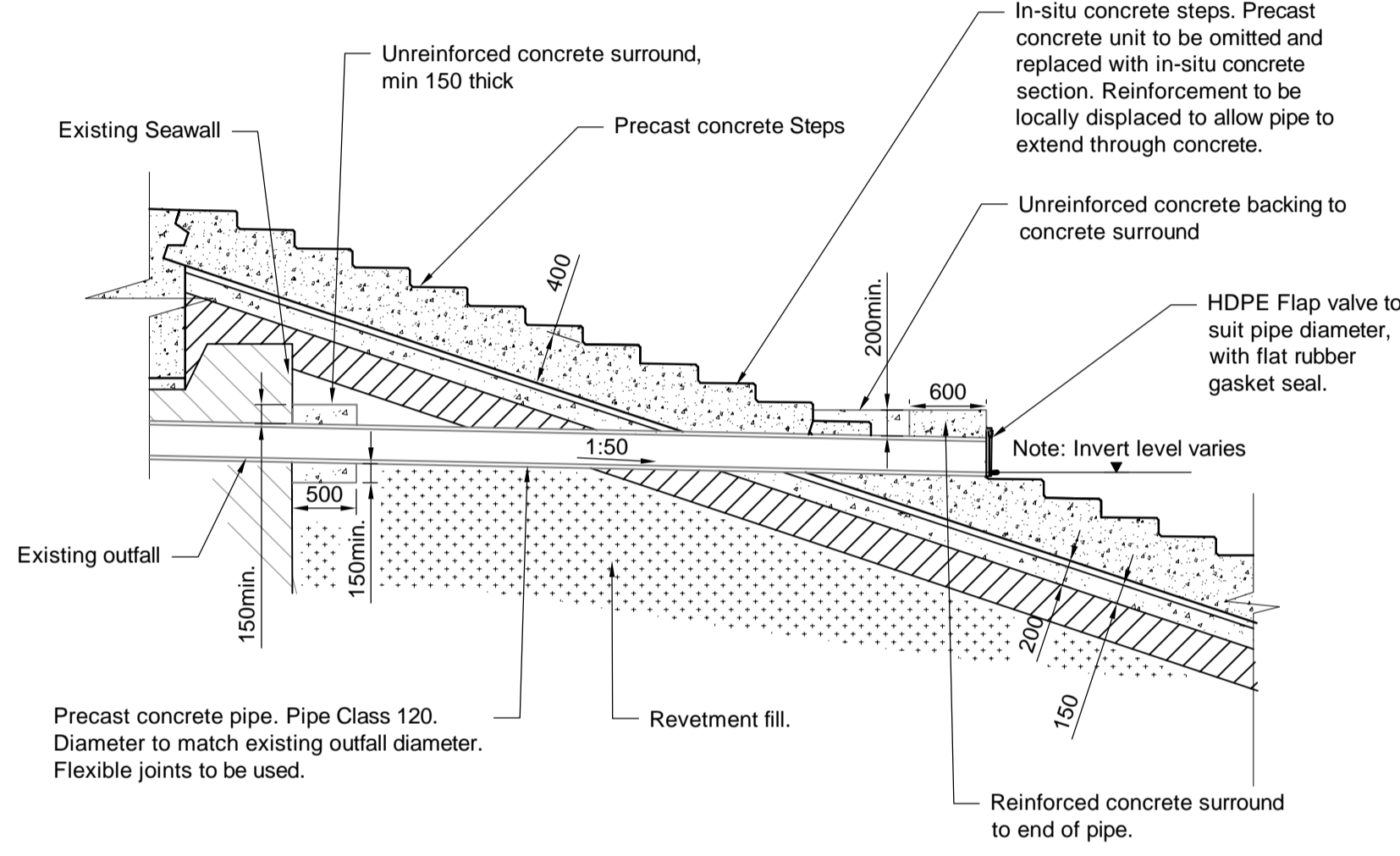
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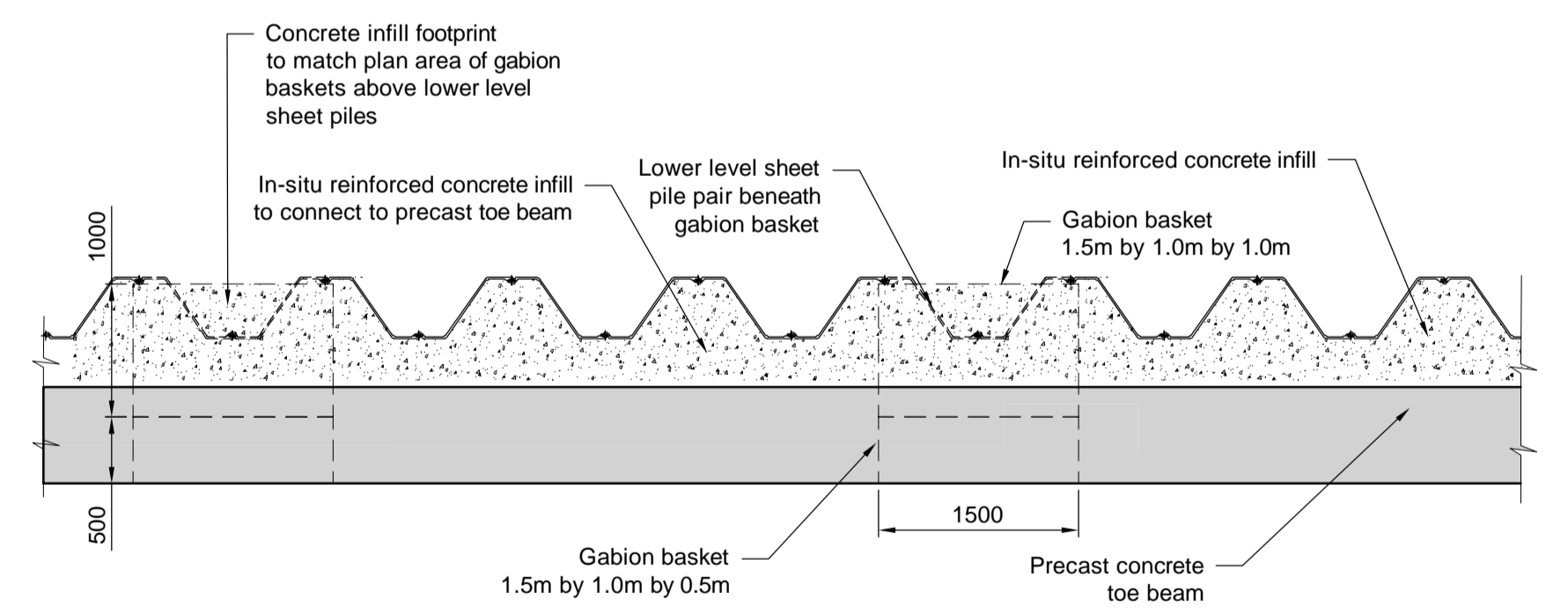
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  - Tactile warning strips to cover a 800mm wide area, starting 400mm back from top step nosing.
  - Existing street furniture, guardrails, services and outfalls not shown.
  - Tide levels are extracted from Welsh Government: Adopting to Climate Change Guidance, 2017. Extreme water levels are derived from Coastal Flood Boundary Conditions for the UK: 2018 update and UK Climate Projections 18 Marine Report, 2018.
  - Existing ground levels are assumed from NRW's Digital Terrain Model (DMT Via Lidar) downloaded in October 2020 (50cm resolution). A topographical survey is required to confirm existing ground levels. Topographical features will be reviewed upon receipt of pending topographical survey, which could affect extent and arrangement of proposed works, for which a variance of 1m has been included.
  - Drawings do not show extent and location of services. See utility plans / statutory undertakers plans for service information. Contractor to undertake pre construction survey to determine full extent of existing services present within working area.
  - Lowest credible beach level added for illustrative purpose. Rock levels not adjusted at Preliminary Design Stage and are subject to further review of the lowest credible beach level at Detailed Design.

Proposals subject to change following detailed design, consultation and necessary approvals.



Outfall Extension Typical Section - Sections E to H



Plan View - Concrete Stepped Revetment Toe Detail  
(Re Typical Sections see Drawing CR-MMD-00-ZZ-DR-SC-3022)

**Key to symbols**

	In-situ Concrete	MHWS: Mean High Water Spring
	Beach Material	EWL: Extreme Water Level
		AEP: Annual Exceedance Probability
		* LCB - Lowest Credible Beach Level

**Reference drawings**

CR-MMD-00-XX-DR-SC-7000	Scheme Key Plan
CR-MMD-00-ZZ-DR-SC-7001	Preliminary Design Plan View - General Arrangement Sheet 01 of 02
CR-MMD-00-ZZ-DR-SC-7002	Preliminary Design Plan View - General Arrangement Sheet 02 of 02
CR-MMD-00-ZZ-DR-SC-7101	Preliminary Design - Typical Sections Sheet 01 of 03
CR-MMD-00-ZZ-DR-SC-7103	Preliminary Design - Typical Sections Sheet 03 of 03
CR-MMD-00-XX-DR-SP-6001	Prom Structures Preliminary Plan View GA
CR-MMD-00-XX-DR-SP-6100	Prom Structures Preliminary Typical Sections and Details
CR-MMD-00-XX-DR-SP-6101	Prom Structures Preliminary Typical Sections and Details
CR-MMD-00-XX-RP-SC-7005	Visual Condition Survey and Recommendations for Repair Works

Rev	Date	Drawn	Description	Ch'k'd	App'd
C02	02/09/2021	PS	Accepted with updated LCB *	GW	ZH
P04	10/06/2021	PS	Preliminary design for EIA and consents	GW	ZH
C01	10/06/2021	PS	Stage 3b accepted	GW	ZH
P03	24/03/2021	PK	Minor amendments made	GW	ZH
P02	22/01/2021	PS	Acceptance Issue following Client Comment	FL	ZH
P01	04/11/2021	PS	Preliminary Issue for Comments	FL	ZH

Status Stamp

Reasonable Worst Case Design for EIA Only

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Client

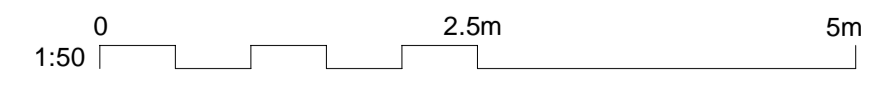
**Balfour Beatty**

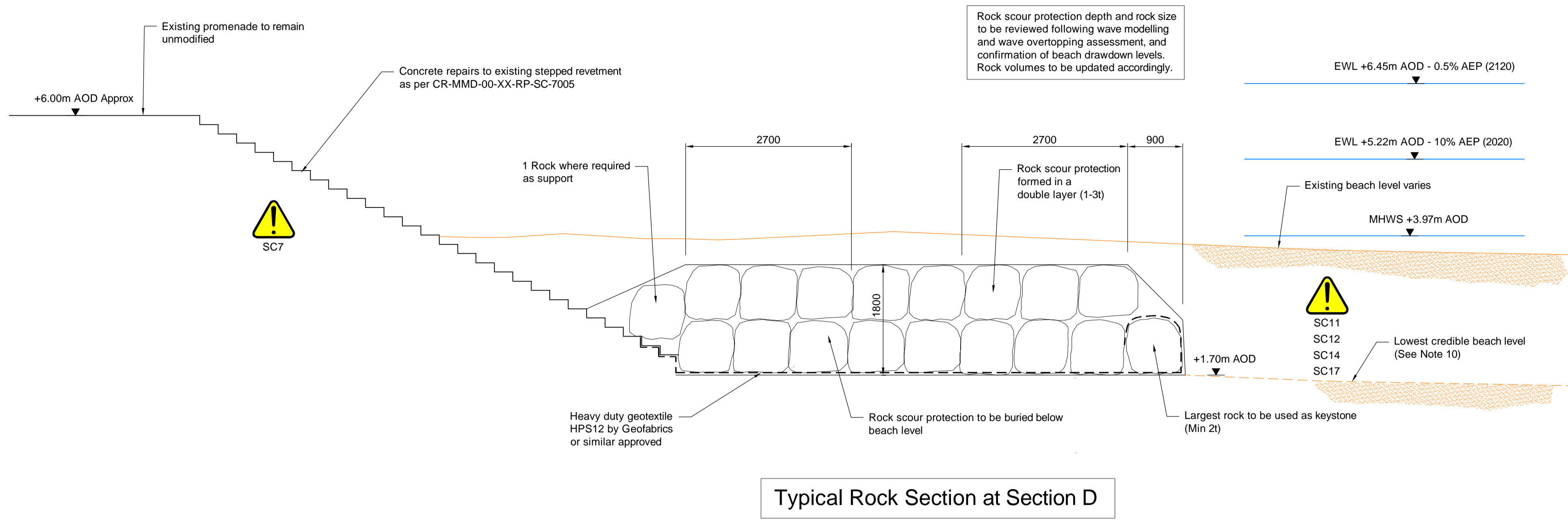
**sir ddinbych denbighshire**  
County Council

Title

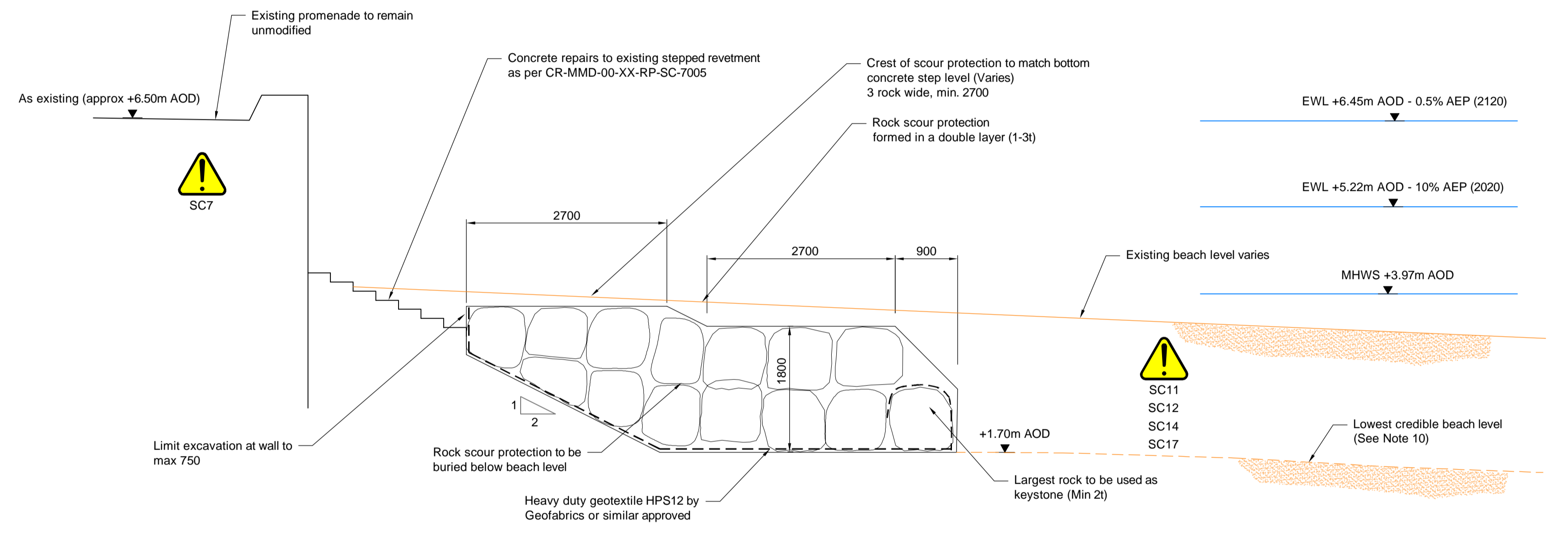
**Central Rhyll Coastal Defences COASTAL STRUCTURES Preliminary Design Typical Sections Sheet 02 of 03**

Designed	P. Kacperek	PK	Eng check	G. Wallace	GW
Drawn	P. Kacperek	PK	Coordination	M. Cowle	MC
Dwg check	G. Wallace	GW	Approved	Z. Hutchison	ZH
MMD Project Number	420823	Scale at A1	1:50	Security	STD
Suitability Description	Stage 3b - Accepted	Suit. Code	A4	Revision	C02
Drawing Number	CR-MMD-00-ZZ-DR-SC-7102				

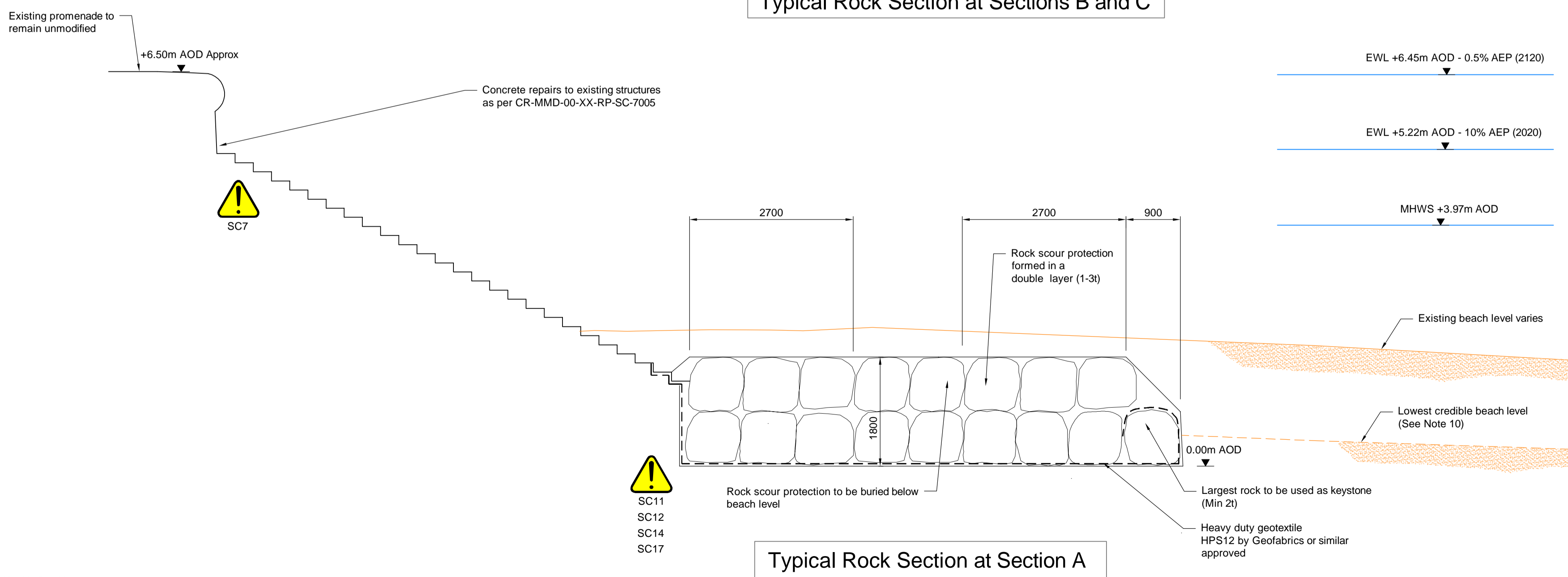




Typical Rock Section at Section D



Typical Rock Section at Sections B and C



Typical Rock Section at Section A

**Safety, Health & Environmental Information**

It is assumed that all works will be carried out by a competent contractor working, where appropriate, to an approved method statement.

**⚠** In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following risks and information which are referred to the Designers' Hazard Elimination and Management Record referenced CR-MMD-00-00-RP-Z-0007:

**Ref: Hazard/ risk summary**

SC7 Geometry of existing structures different to existing as-built information or topographical information, or unknown if below ground, which may affect the design and delay works on site.

SC11 Excavation collapse resulting in injury to operators and/or damage to plant.

SC12 UXOs or other buried obstructions are encountered resulting in death and injury.

SC14 Damage to or clashes with existing service infrastructure particularly during excavation and sheet piling works and particularly to drainage assets extending seaward from the existing sea wall.

SC17 Risk of undermining existing structures, or damage to existing structures.

SC18 Risk of existing seawall instability or collapse due to plant loading on adjacent promenade during construction. This is of particular concern to seawall at Section E.

SC20 Asbestos may be present in structures.

- Notes**
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  - This drawing shall be read in conjunction with all discipline information. Any discrepancies shall be referred to Mott MacDonald before proceeding.
  - Unless noted otherwise, all dimensions are in millimetres, all levels in metres above Ordnance Datum (AOD). Do not scale from this drawing, if in doubt ask Mott MacDonald.
  - Mott MacDonald Ltd. has followed accepted procedure in providing the services but given the residual risk associated with any prediction and the variability which can be experienced in flood conditions, Mott MacDonald Ltd. takes no liability for and gives no warranty against actual flooding of any property (clients or third party) or the consequences of flooding in relation to the performance of the service.
  - Existing street furniture, guardrails, services and outfalls not shown.
  - Tide levels are extracted from Welsh Government: Adopting to Climate Change Guidance, 2017. Extreme water levels are derived from Coastal Flood Boundary Conditions for the UK: 2018 update and UK Climate Projections18 Marine Report, 2018.
  - Existing ground levels are assumed from NRW's Digital Terrain Model (DMT Via Lidar) downloaded in October 2020 (50cm resolution). A topographical survey is required to confirm existing ground levels. Topographical features will be reviewed upon receipt of pending topographical survey, which could affect extent and arrangement of proposed works, for which a variance of 1m has been included.
  - Drawings do not show extent and location of services. See utility plans / statutory undertakers plans for service information. Contractor to undertake pre construction survey to determine full extent of existing services present within working area.
  - Signage required 'do not climb on rock scour protection' locations to be confirmed in detailed design.
  - Lowest credible beach level added for illustrative purpose. Rock levels not adjusted at Preliminary Design Stage and are subject to further review of the lowest credible beach level at Detailed Design.

Proposals subject to change following detailed design, consultation and necessary approvals.

**Key to symbols**

Beach Material

MHWS: Mean High Water Spring  
 EWL: Extreme Water Level  
 AEP: Annual Exceedance Probability  
 \* LCB - Lowest Credible Beach Level

**Reference drawings**

CR-MMD-00-XX-DR-SC-7000	Scheme Key Plan
CR-MMD-00-ZZ-DR-SC-7001	Preliminary Design Plan View - General Arrangement Sheet 01 of 02
CR-MMD-00-ZZ-DR-SC-7002	Preliminary Design Plan View - General Arrangement Sheet 02 of 02
CR-MMD-00-ZZ-DR-SC-7101	Preliminary Design - Typical Sections Sheet 01 of 03
CR-MMD-00-ZZ-DR-SC-7102	Preliminary Design - Typical Sections Sheet 02 of 03
CR-MMD-00-XX-DR-SP-6001	Prom Structures Preliminary Plan View GA
CR-MMD-00-XX-DR-SP-6100	Prom Structures Preliminary Typical Sections and Details
CR-MMD-00-XX-DR-SP-6101	Prom Structures Preliminary Typical Sections and Details
CR-MMD-00-XX-RP-SC-7005	Visual Condition Survey and Recommendations for Repair Works

Rev	Date	Drawn	Description	Ch'k'd	App'd
C02	02/09/2021	PS	Accepted with updated LCB *	GW	ZH
P03	10/06/2021	PS	Preliminary design for EIA and Consents	GW	ZH
C01	10/06/2021	PS	Stage 3b accepted	GW	ZH
P02	27/01/2021	PS	Acceptance issue following Clients Comments	GW	ZH
P01	22/01/2021	PS	Acceptance issue following Clients Comments	FL	ZH

Status Stamp

**Reasonable Worst Case Design for EIA Only**

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Client

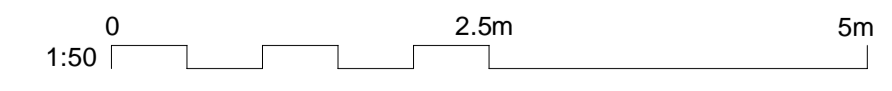
**Balfour Beatty**

**sir ddinbych denbighshire**  
 County Council

Title

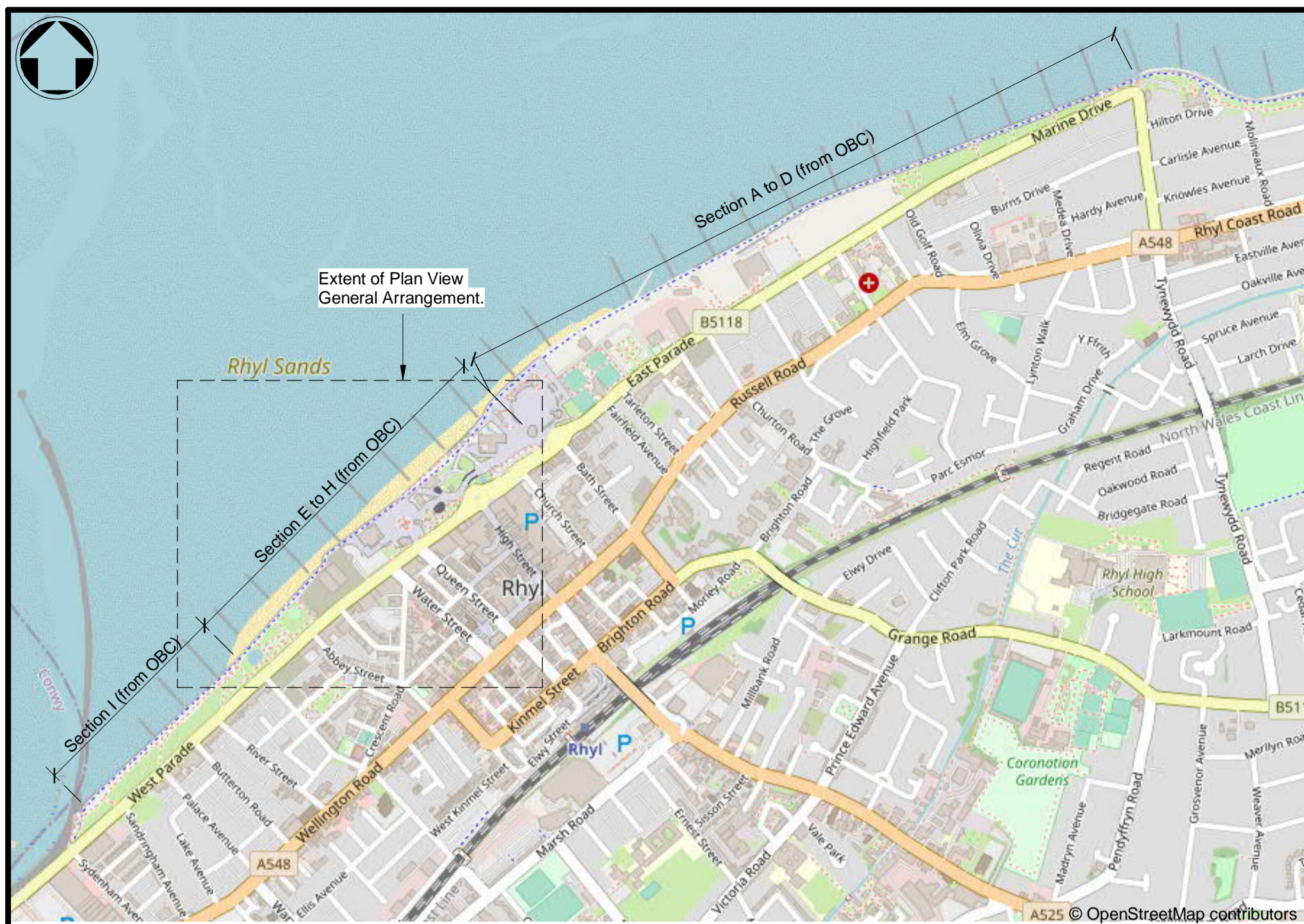
**Central Rhyll Coastal Defences  
 COASTAL STRUCTURES  
 Preliminary Design  
 Typical Sections  
 Sheet 03 of 03**

Designed	P. Kacperk	PK	Eng check	G Wallace	GW
Drawn	P. Steers	PS	Coordination	M. Cowle	MC
Dwg check	G Wallace	GW	Approved	Z. Hutchison	ZH
MMD Project Number	420823		Scale at A1	1:50	
Suitability Description	Stage 3b - Accepted		Security	STD	
Drawing Number	CR-MMD-00-ZZ-DR-SC-7103		Suit. Code	A4	
			Revision	C02	



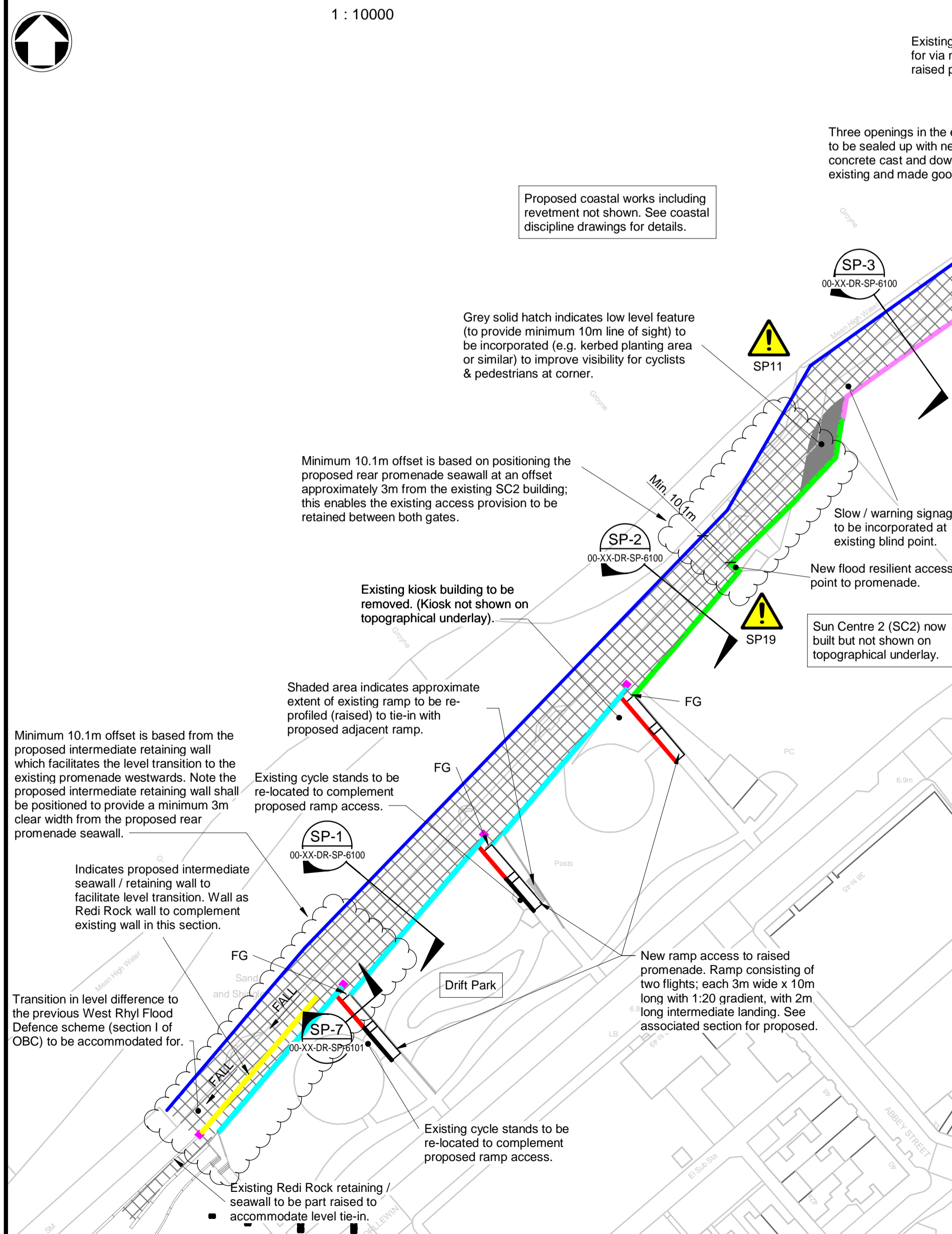
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**SITE LOCATION PLAN**

1 : 10000



**PLAN VIEW GENERAL ARRANGEMENT**

1 : 1000

**SAFETY, HEALTH & ENVIRONMENTAL INFORMATION**

It is assumed that all works will be carried out by a competent contractor working where appropriate to an approved method statement.

In addition to the hazards / risks normally associated with the types of work detailed on the drawings, note the following risks & information which are referenced to the Designers' Hazard Elimination and Management Record referenced CR-MMD-00-00-RP-Z-0007.

Ref:	Hazard / risk summary:	Ref:	Hazard / risk summary:
SP2 & SP3	Damage to or clashes with existing services; including electrical, telecommunication, drainage, water etc. Utility surveys to be undertaken and bridging details to be considered in design.	SP17	Restricted beach access to vehicles unless new ramp access is provided. Client to confirm.
SP5	Damage to existing walls / structures at rear of promenade due to overloading from additional loads from the proposed construction.	SP19	Pedestrians climbing on proposed rear promenade seawalls present risk of injury or unauthorised access to private property. Security recommended along SC2, SeaQuarium and lower level areas of the Children's Village.
SP9	Possible restricted access to existing manholes / inspection chambers due to proposed construction. Inspection chambers and services possibly need to be raised, consultation needed with utility providers.	SP20	Asbestos Register not provided. Asbestos may be present in structures.
SP10	Cyclists falling off the exposed promenade seaward edge. Edge restraint 1.245m high specified along seaward edge, however since this does not provide required 1.4m high protection for cyclists; proposals are subject to keeping cyclists away from the seaward edge by means such as cycle lane markings.	SP32	Unknown / unforeseen ground conditions. Ground conditions could affect extent and arrangement of proposed works.
SP11	Collision between cyclists / pedestrians. Design incorporates improvement to some existing pinch points / line of sight.	SP34	Coastal flooding risk. See drawing note 4.
SP14	Overtopping discharge presenting injury risk to promenade users during storm events. Wave overtopping analysis has been undertaken and promenade is proposed to be raised to reduce overtopping discharge.	SP37	Crane overloading existing foundations at the rear of the promenade.

- Notes**
- This drawing is intended to be used as a basis for the Environmental Impact Assessment for the proposed Central Rhyl Coastal Defences Scheme, and it should not be used for any other purpose. It represents the reasonable worst case arrangement of the Scheme.
  - This drawing shall be read in conjunction with all discipline information. Any discrepancies shall be referred to Mott MacDonald before proceeding.
  - Unless noted otherwise, all dimensions are in millimetres, all levels in metres above ordnance datum (AOD). Do not scale from drawing, if in doubt ask Mott MacDonald.
  - Mott MacDonald has followed accepted procedure in providing the services but given the residual risk associated with any prediction and the variability which can be experienced in flood conditions, Mott MacDonald takes no liability for and gives no warranty against actual flooding of any property (clients or third party) or the consequences of flooding in relation to the performance of the service.
  - Drawings do not show extent and location of services. See utility plans / statutory undertakers plans for service information. Contractor to undertake pre construction survey to determine full extent of existing services present within working area.
  - Street lighting, street furniture, existing services, existing seawalls, adjoining buildings, movement joints etc not shown.
  - These preliminary design promenade structures drawings only include sections E to H of the Central Rhyl Coastal Defence Outline Business Case (OBC) by JBA dated October 2019. Repair work to the existing coastal defences are not shown; for information see the visual condition survey & repair report CR-MMD-00-XX-RP-SC-7005 and associated drawings.
  - Design of the rear promenade walls are based on the following load cases:
    - Load case 1: Imposed (variable) water load of 10kN/m<sup>2</sup> UDL and horizontal wave load of 12kN/m at 335mm above promenade FFL.
    - Load case 2: Imposed (variable) service load of 20kN/m<sup>2</sup> UDL and horizontal load of 1.5kN/m at 1200mm above promenade FFL.
- Proposals subject to change, detailed design, consultation and necessary approvals.

- Key to symbols**
- Indicates approximate extent of promenade raising.
  - Blue line preliminarily indicates unless noted otherwise minimum 10.1m offset from the seaward edge of rear promenade seawall to the possible landward edge of promenade (further offset allowances have been included to consider pinch points, alignment and wall build ups). Note where clearance is already sufficient the alignment of the existing seawall is used. Final alignment of revetment and crest beam to be determined by coastal design team, in conjunction with BB and DCC and subject to stakeholder consultations and approvals.
  - Wall Type SP-1: Indicates approximate extent of proposed rear promenade seawall detail adjacent to Drift Park.
  - Wall Type SP-2: Indicates approximate extent of proposed rear promenade seawall detail adjacent to SC2.
  - Wall Type SP-3: Indicates approximate extent of proposed rear promenade seawall detail adjacent to Sky Tower / Vue Cinema.
  - Wall Type SP-4: Indicates approximate extent of proposed rear promenade seawall detail adjacent to Children's Village / underground car park.
  - Wall Type SP-5: Indicates approximate extent of proposed rear promenade seawall detail between Children's Village and SeaQuarium.
  - Wall Type SP-6: Indicates approximate extent of proposed rear promenade seawall detail adjacent to SeaQuarium.
  - Wall Type SP-7: Indicates proposed retaining wall to accommodate ramps or steps.
  - Wall Type SP-8: Indicates proposed intermediate seawall / retaining wall to facilitate level transition.
  - Indicates proposed flood gate (overall geometry, performance specification and design loadings to be provided by Mott MacDonald Ltd).
  - Indicates existing timber & slate beacons to be retained with slate section partially buried by the promenade raising.

**Reference drawings**

CR-MMD-00-XX-DR-SP-6000	Promenade Structures General Notes
CR-MMD-00-XX-DR-SP-6001	Promenade Structures Plan View GA
CR-MMD-00-XX-DR-SP-6100 & 6101	Prom. Structures Typical Sections & Details
CR-MMD-00-XX-DR-SC-7001 & 7002	Coastal Structures Plan View GA's
CR-MMD-00-XX-DR-SC-7101 to 7103	Coastal Structures Typical Sections

Rev	Date	Drawn	Description	Ch'k'd	App'd
C02	03.09.2021	JH	Accepted for EIA following client comments	GWM	NP
P03	10.06.2021	JH	Preliminary design for EIA and consents.	GWM	SS
C01	04.06.2021	JH	Stage 3b - accepted contract revision.	GWM	SS
P02	18.12.2020	JH	Issued for acceptance following client comments.	GWM	NP
P01	30.10.2020	JH	Preliminary issue for comment	GWM	SS

**Status Stamp**

**Reasonable worst case design for EIA only**

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**Client**

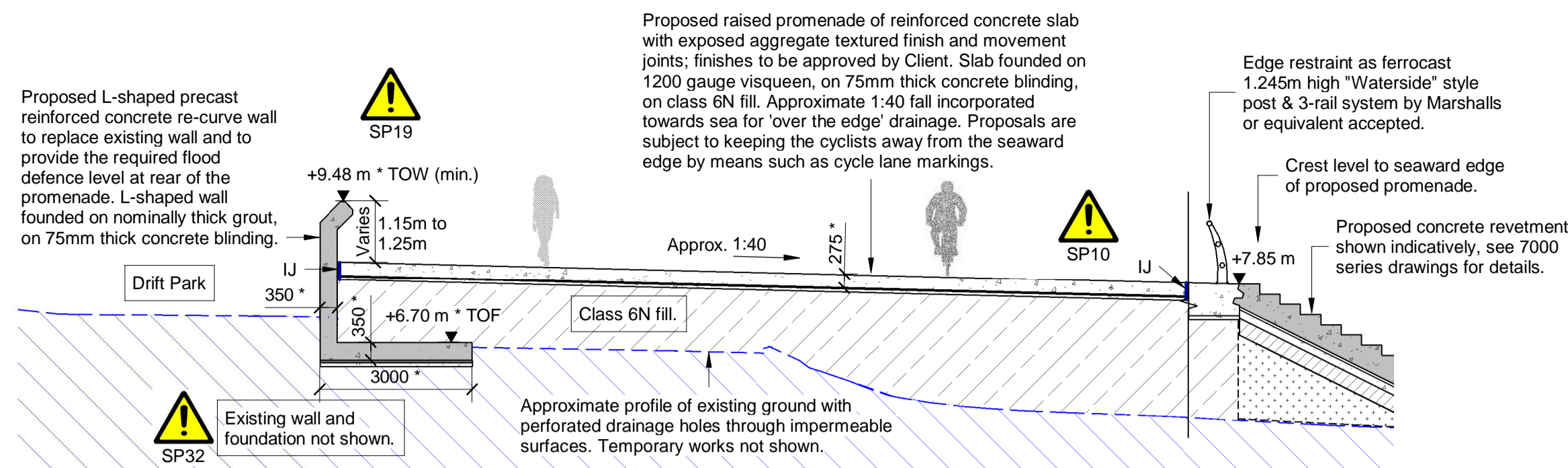
**Balfour Beatty**

**sir ddinbych denbighshire**  
County Council

**Title**

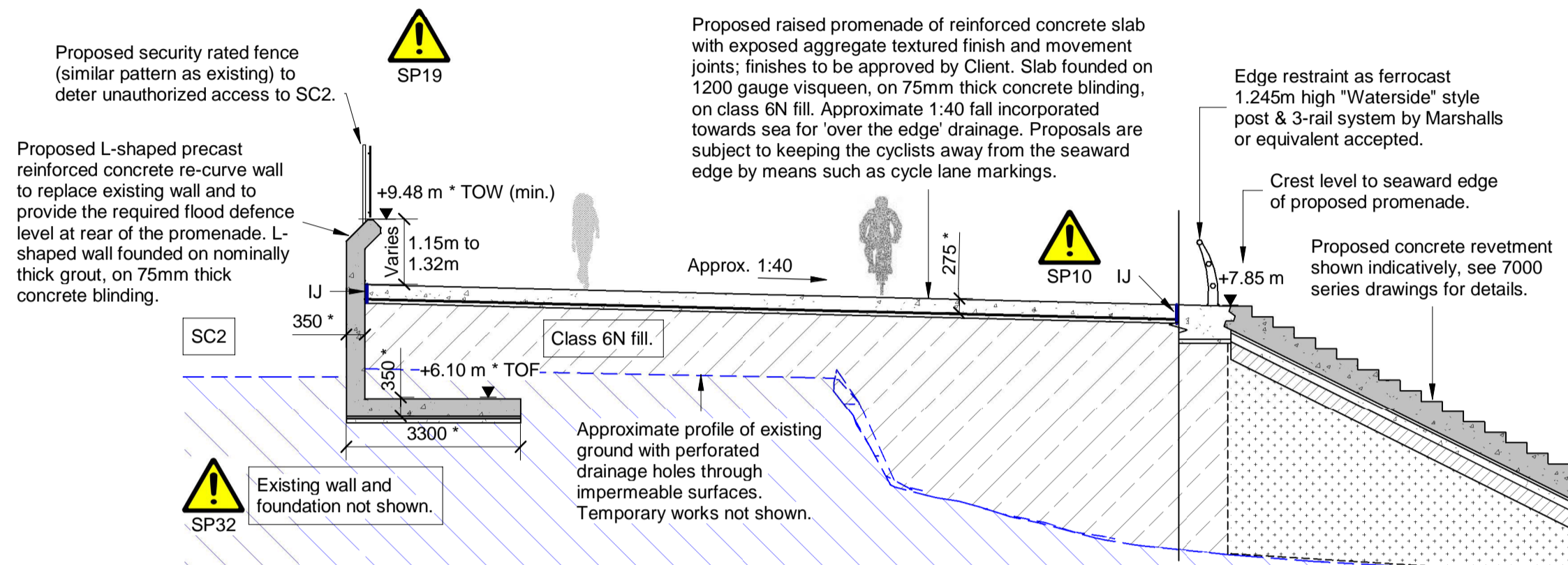
**Central Rhyl Coastal Defences PROMENADE STRUCTURES Preliminary Design Plan View General Arrangement**

Designed	J. Hughes	Eng.concept	G. Mitchell
Drawn	J. Hughes	Coordination	M. Cowle
Dwg.check.	G. Mitchell	Approved	S. Smith
MMD Project Number	420823	Scale at A1	As indicated
Suitability Description	Stage 3b - Accepted	Security	STD
Drawing Number	CR-MMD-00-XX-DR-SP-6001	Suit. Code	A4
		Rev	C02



**SP-1 Wall Type SP-1 Typical Section (Drift Park) - New replacement wall**

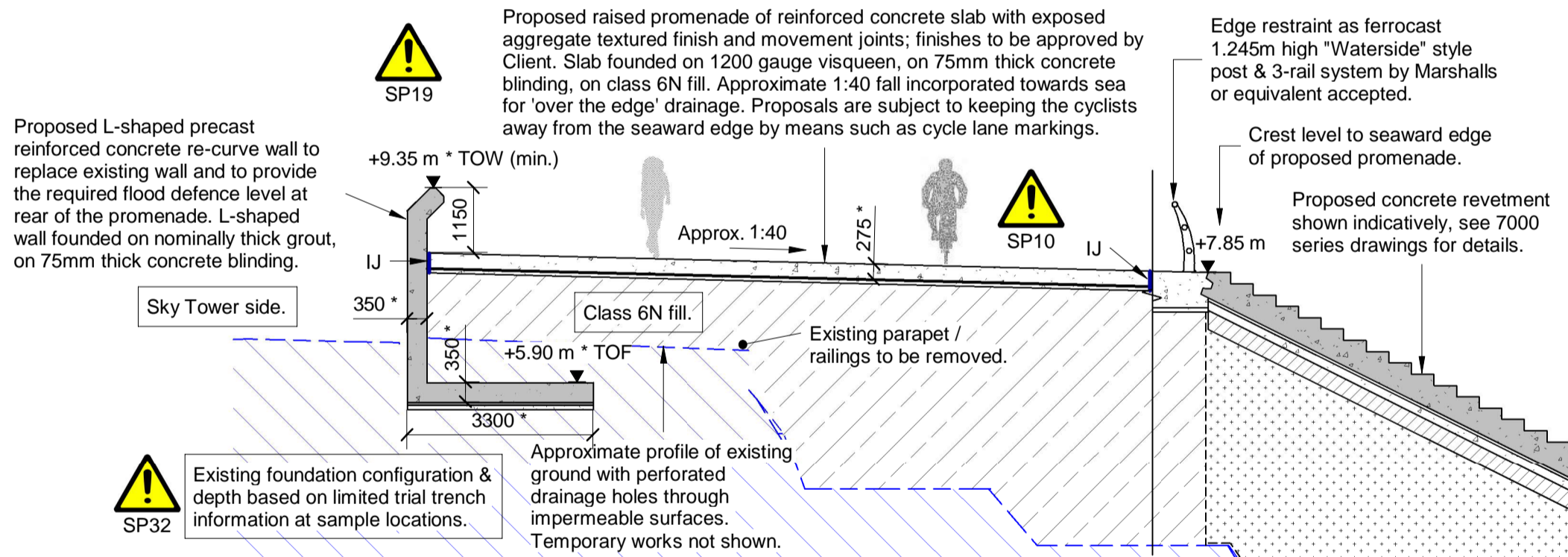
00-XX-DR-SP-6001 1 : 100



**SP-2 Wall Type SP-2 Typical Section (SC2) - New replacement wall**

00-XX-DR-SP-6001 1 : 100

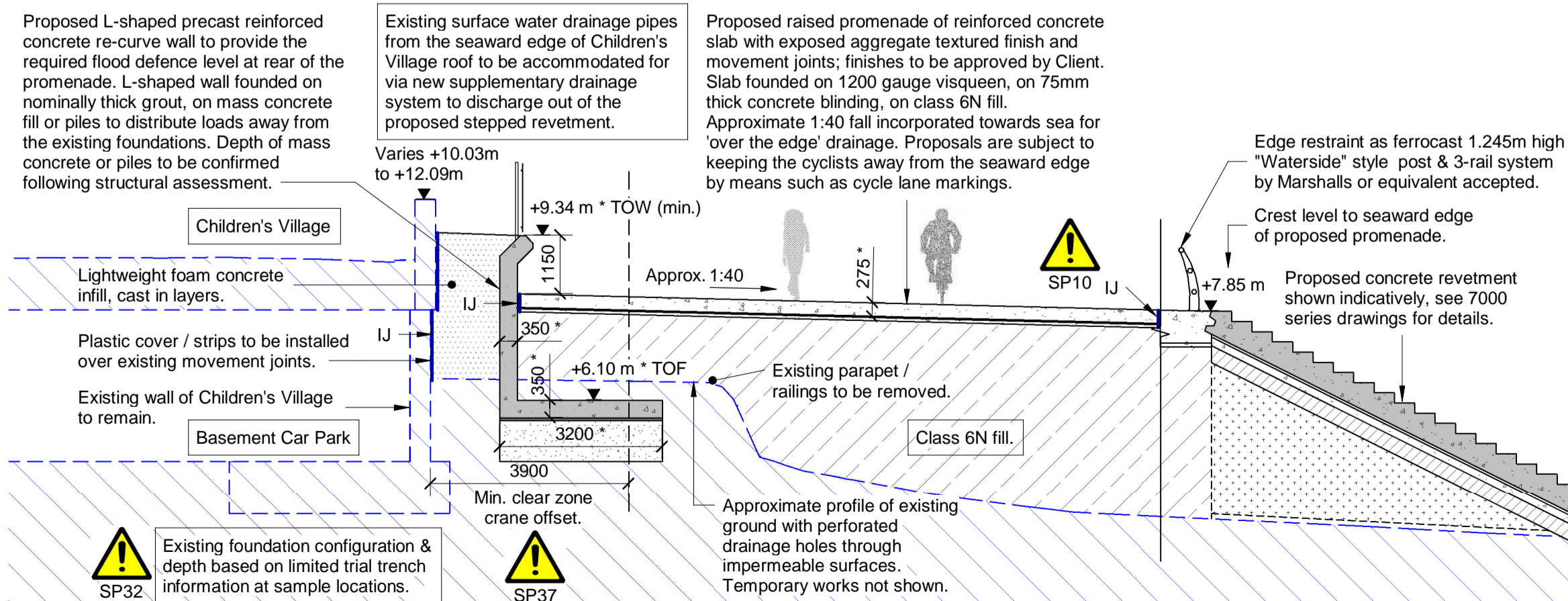
**Note this option is the worse case (out of 3A & 3B) for quantity of new concrete required.**



**SP-3A Wall Type SP-3A Typical Section (Sky Tower) - Option A (New replacement wall)**

00-XX-DR-SP-6001 1 : 100

**Note this option is the worse case (out of 4A & 4B) for encroachment into the coastline given the additional space required for the proposed offset wall.**



**SP-4A Wall Type SP-4A Typical Section (Children's Village) - Option A (New offset wall)**

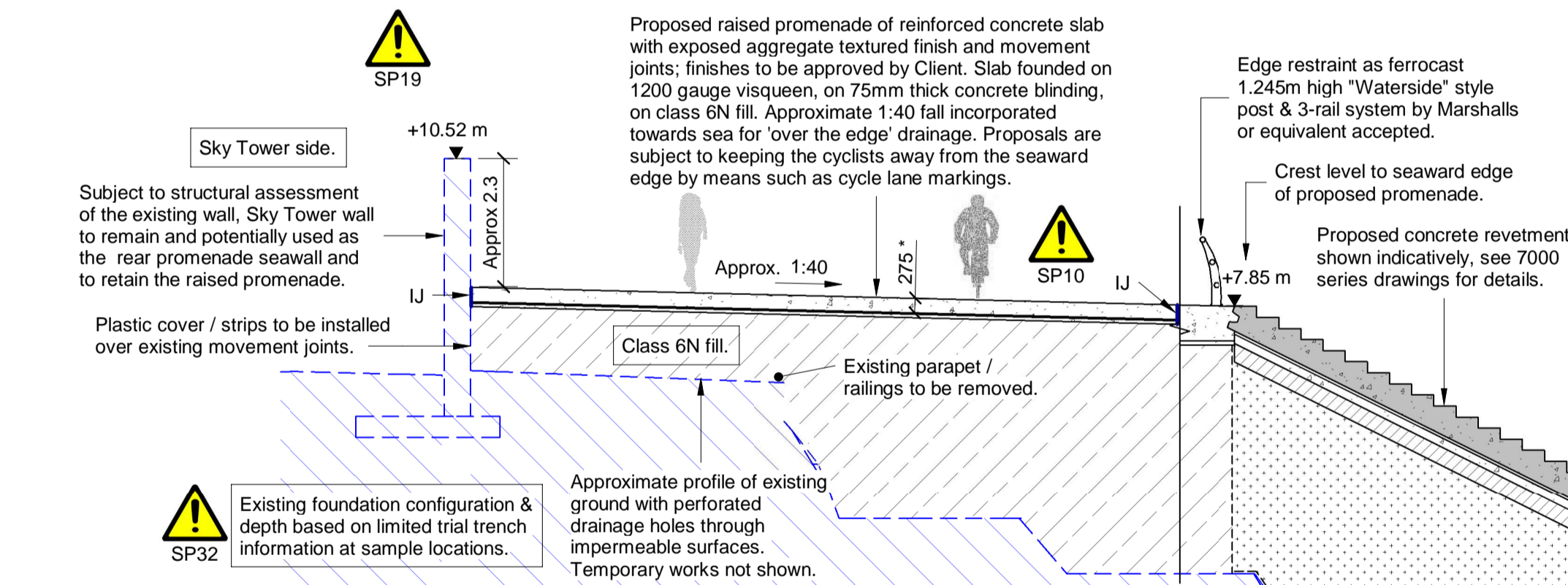
00-XX-DR-SP-6001 1 : 100

**SAFETY, HEALTH & ENVIRONMENTAL INFORMATION**

It is assumed that all works will be carried out by a competent contractor working where appropriate to an approved method statement.

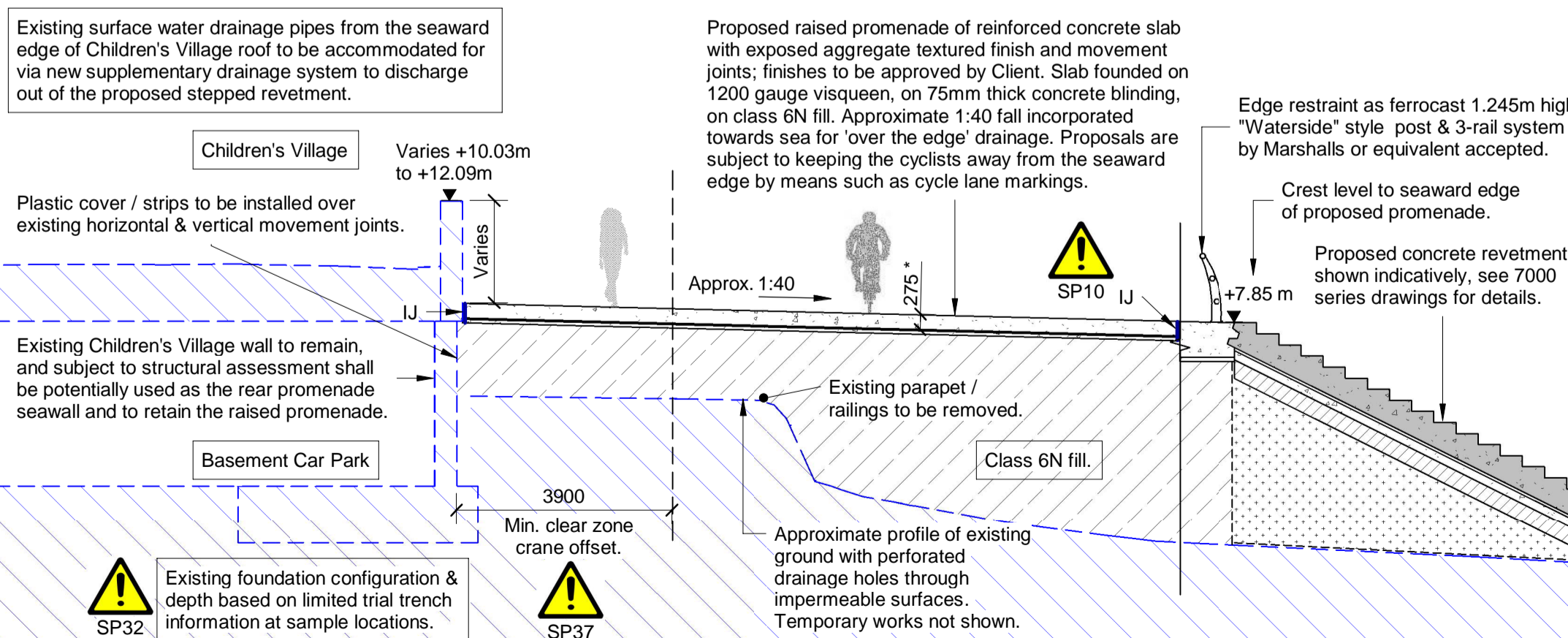
In addition to the hazards / risks normally associated with the types of work detailed on the drawings, note the following risks & information which are referenced to the Designers' Hazard Elimination and Management Record referenced CR-MMD-00-00-RP-Z-0007.

Ref:	Hazard / risk summary:	Ref:	Hazard / risk summary:
SP2 & SP3	Damage to or clashes with existing services; including electrical, telecommunications, drainage, water etc. Utility surveys to be undertaken and bridging details to be considered in design.	SP17	Restricted beach access to vehicles unless new ramp access is provided. Client to confirm.
SP5	Damage to existing walls / structures at rear of promenade due to overloading from additional loads from the proposed construction.	SP19	Pedestrians climbing on proposed rear promenade seawalls present risk of injury or unauthorized access to private property. Security recommended along SC2, SeaQuarum and lower level areas of the Children's Village.
SP9	Possible restricted access to existing manholes / inspection chambers due to proposed construction. Inspection chambers and services possibly need to be raised; consultation needed with utility providers.	SP20	Asbestos Register not provided. Asbestos may be present in structures.
SP10	Cyclists falling off the exposed promenade seaward edge. Edge restraint 1.245m high specified along seaward edge, however since this does not provide required 1.4m high protection for cyclists; proposals are subject to keeping cyclists away from the seaward edge by means such as cycle lane markings.	SP32	Unknown / unforeseen ground conditions. Ground conditions could affect extent and arrangement of proposed works.
SP11	Collision between cyclists / pedestrians. Design incorporates improvement to some existing pinch points / line of sight.	SP34	Coastal flooding risk. See drawing note 4.
SP14	Overtopping discharge presenting injury risk to promenade users during storm events. Wave overtopping analysis has been undertaken and promenade is proposed to be raised to reduce overtopping discharge.	SP37	Crane overloading existing foundations at the rear of the promenade.



**SP-3B Wall Type SP-3B Typical Section (Sky Tower) - Option B (Use existing)**

00-XX-DR-SP-6001 1 : 100



**SP-4B Wall Type SP-4B Typical Section (Children's Village) - Option B (Use existing)**

00-XX-DR-SP-6001 1 : 100

- Notes**
- This drawing is intended to be used as a basis for the Environmental Impact Assessment for the proposed Central Rhyl Coastal Defences Scheme, and it should not be used for any other purpose. It represents the reasonable worst case arrangement of the Scheme.
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  - Drawings do not show extent and location of services. See utility plans / statutory undertakers plans for service information. Contractor to undertake pre construction survey to determine full extent of existing services present within working area.
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  - These preliminary design promenade structures drawings only include sections E to H of the Central Rhyl Coastal Defence Outline Business Case (OBC) by JBA dated October 2019. Repair work to the existing coastal defences are not shown; for information see the visual condition survey & repair report CR-MMD-00-XX-RP-SC-7005 and associated drawings.
  - Design of the rear promenade walls are based on the following load cases:
    - Load case 1: Imposed (variable) wall load of 10kN/m<sup>2</sup> UDL and horizontal wave load of 12kN/m at 335mm above promenade FFL.
    - Load case 2: Imposed (variable) service load of 20kN/m<sup>2</sup> UDL and horizontal load of 1.5kN/m at 1200mm above promenade FFL.

Proposals subject to change, detailed design, consultation and necessary approvals.

**Key to symbols**

	Existing terrain / structures.
	Proposed in-situ reinforced concrete.
	Proposed precast reinforced concrete.
	Proposed in-situ mass concrete.
	Proposed class 6N compacted granular in accordance with Series 600 Volume 1 Specification for Highway Works.

\* = Preliminary dimensions and levels subject to change and detailed design.

**Abbreviations:**  
 TOF = Top of foundation  
 TOW = Top of wall  
 FFL = Finish floor level  
 Min. = Minimum  
 IJ = Isolation joint

**Reference drawings**

CR-MMD-00-XX-DR-SP-6000: Promenade Structures General Notes	
CR-MMD-00-XX-DR-SP-6100 & 6101: Prom. Structures Typical Sections & Details	
CR-MMD-00-XX-DR-SC-7001 & 7002: Coastal Structures Plan View GA's	
CR-MMD-00-XX-DR-SC-7101 to 7103: Coastal Structures Typical Sections	

Rev	Date	Drawn	Description	Ch'k'd	App'd
C02	03.09.2021	JH	Accepted for EIA following client comments	GWM	NP
P03	10.06.2021	JH	Preliminary design for EIA and consents.	GWM	SS
C01	04.06.2021	JH	Stage 3b - accepted contract revision.	GWM	SS
P02	18.12.2020	JH	Issued for acceptance following client comments.	GWM	NP
P01	30.10.2020	JH	Preliminary issue for comment	GWM	SS

Status Stamp

Reasonable worst case design for EIA only

**MOTT MACDONALD**

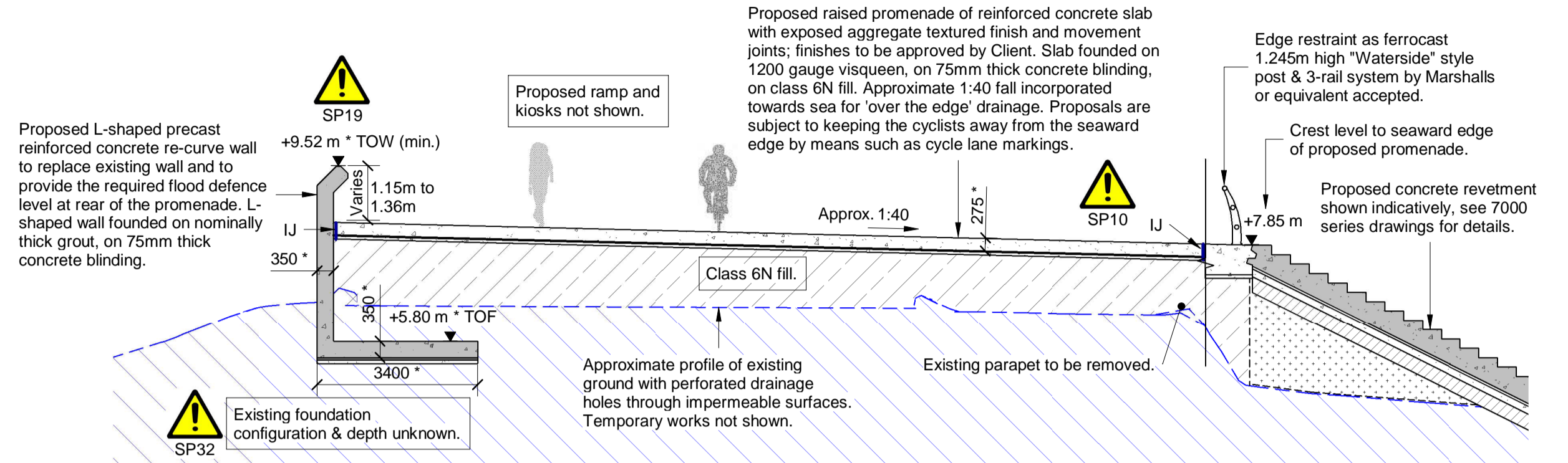
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**Balfour Beatty**

**sir ddinbych denbighshire**  
 County Council

**Central Rhyl Coastal Defences PROMENADE STRUCTURES Preliminary Design Typical Sections & Details**

Designed	J. Hughes	Eng. concept	G. Mitchell
Drawn	J. Hughes	Coordination	M. Cowle
Dwg check.	G. Mitchell	Approved	S. Smith
MMD Project Number	420823	Scale at A1	As indicated
Suitability Description	Stage 3b - Accepted	Security	STD
Drawing Number	CR-MMD-00-XX-DR-SP-6100	Suit. Code	A4
		Rev	C02



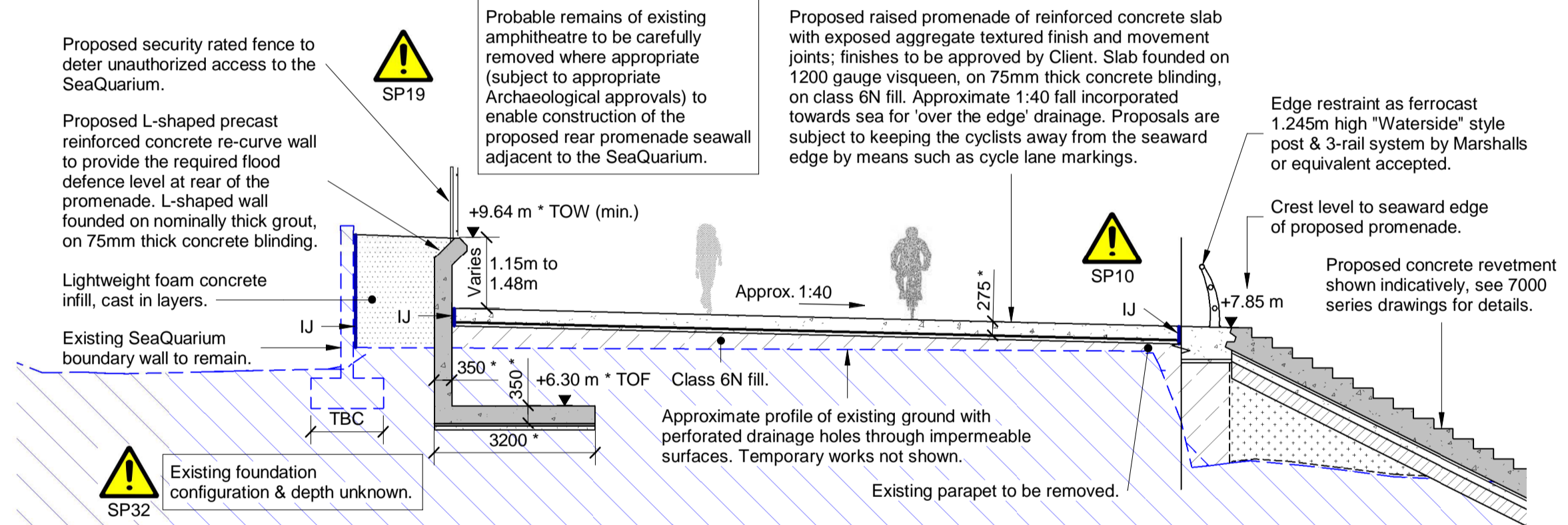
**Wall Type SP-5 Typical Section (between Children's Village & SeaQuarium) - New replacement wall**

00-XX-DR-SP-6001 1 : 100

**SAFETY, HEALTH & ENVIRONMENTAL INFORMATION**

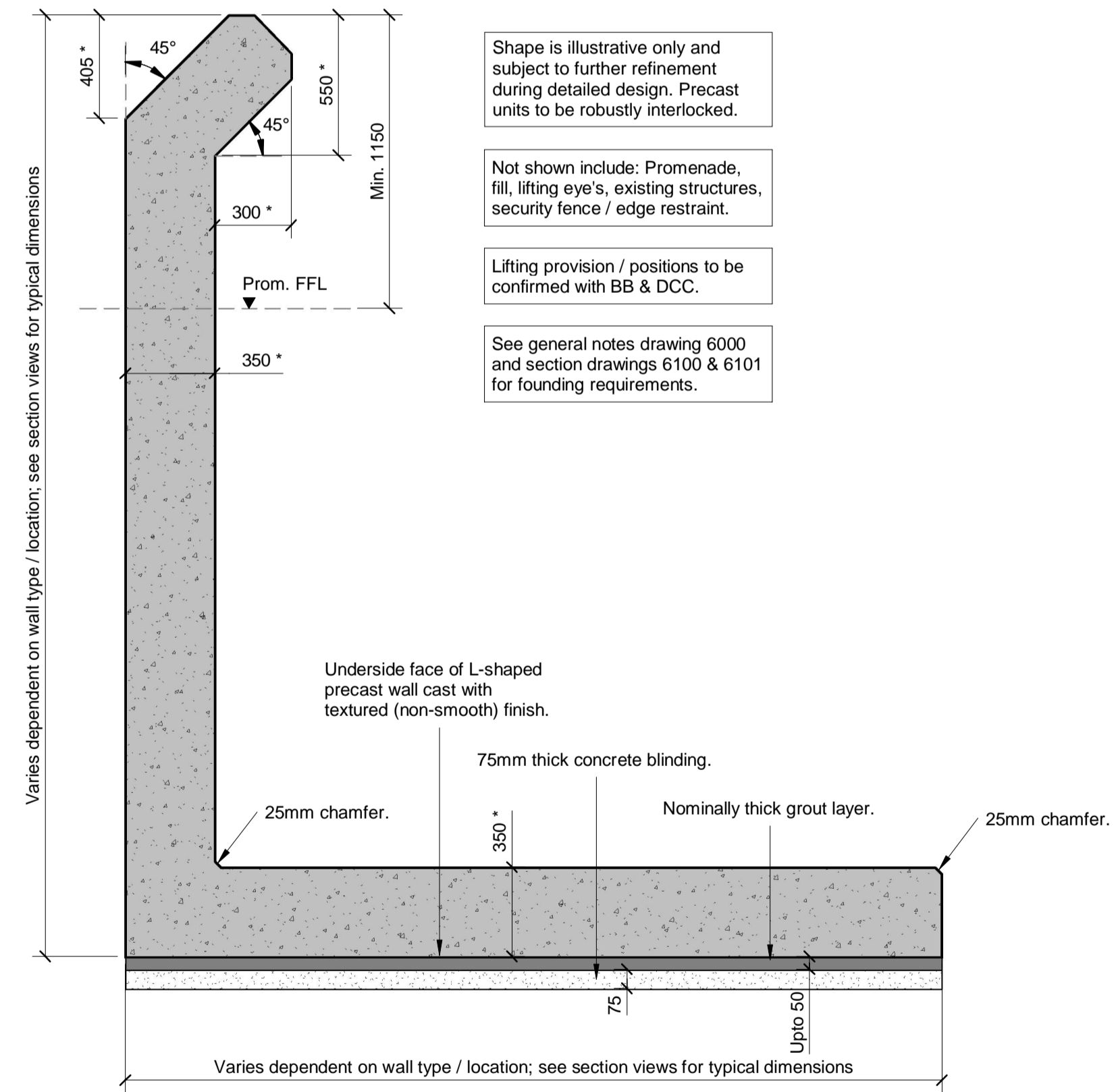
It is assumed that all works will be carried out by a competent contractor working where appropriate to an approved method statement. In addition to the hazards / risks normally associated with the types of work detailed on the drawings, note the following risks & information which are referenced to the Designers' Hazard Elimination and Management Record referenced CR-MMD-00-00-RP-Z-0007.

- Ref: Hazard / risk summary:**
- SP2 Damage to or clashes with existing services; including electrical, telecommunication, drainage, water etc. Utility surveys to be undertaken and bridging details to be considered in design.
  - SP3
  - SP5 Damage to existing walls / structures at rear of promenade due to overloading from additional loads from the proposed construction.
  - SP9 Possible restricted access to existing manholes / inspection chambers due to proposed construction. Inspection chambers and services possibly need to be raised; consultation needed with utility providers.
  - SP10 Cyclists falling off the exposed promenade seaward edge. Edge restraint 1.245m high specified along seaward edge, however since this does not provide required 1.4m high protection for cyclists; proposals are subject to keeping cyclists away from the seaward edge by means such as cycle lane markings.
  - SP11 Collision between cyclists / pedestrians. Design incorporates improvement to some existing pinch points / line of sight.
  - SP14 Overtopping discharge presenting injury risk to promenade users during storm events. Wave overtopping analysis has been undertaken and promenade is proposed to be raised to reduce overtopping discharge.
- Ref: Hazard / risk summary:**
- SP17 Restricted beach access to vehicles unless new ramp access is provided. Client to confirm.
  - SP19 Pedestrians climbing on proposed rear promenade seawalls present risk of injury or unauthorized access to private property. Security recommended along SC2, SeaQuarium and lower level areas of the Children's Village.
  - SP20 Asbestos Register not provided. Asbestos may be present in structures.
  - SP32 Unknown / unforeseen ground conditions. Ground conditions could affect extent and arrangement of proposed works.
  - SP34 Coastal flooding risk. See drawing note 4.
  - SP37 Crane overloading existing foundations at the rear of the promenade.



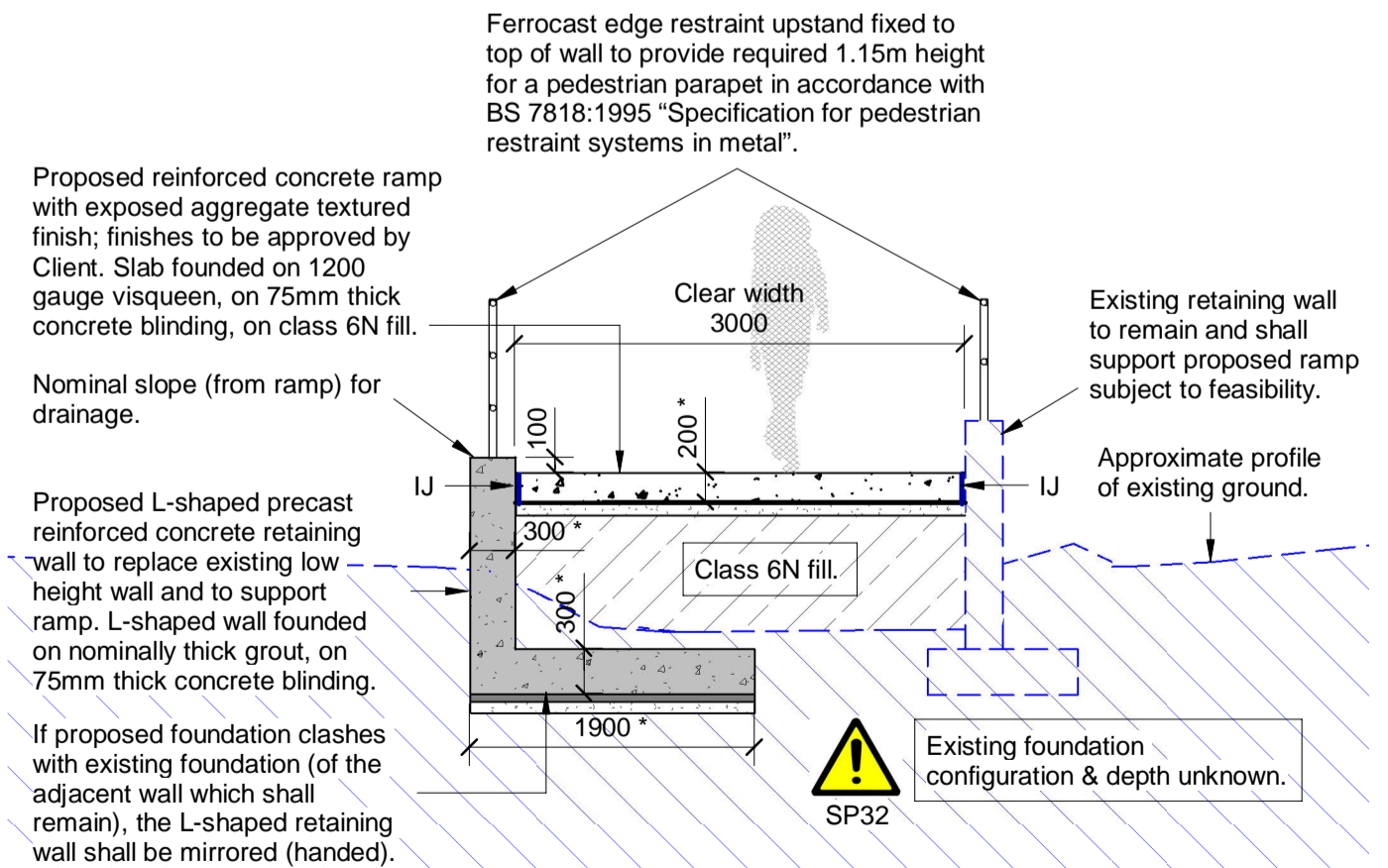
**Wall Type SP-6 Typical Section (SeaQuarium) - New offset wall**

00-XX-DR-SP-6001 1 : 100



**Generic L-shaped precast reinforced concrete retaining wall detail**

1 : 20



**Wall Type SP-7 Typical Section (Drift Park Ramps)**

00-XX-DR-SP-6001 1 : 50

- Notes**
1. This drawing is intended to be used as a basis for the Environmental Impact Assessment for the proposed Central Rhyll Coastal Defences Scheme, and it should not be used for any other purpose. It represents the reasonable worst case arrangement of the Scheme.
  2. This drawing shall be read in conjunction with all discipline information. Any discrepancies shall be referred to Mott MacDonald before proceeding.
  3. Unless noted otherwise, all dimensions are in millimetres, all levels in metres above ordnance datum (AOD). Do not scale from drawing, if in doubt ask Mott MacDonald.
  4. Mott MacDonald has followed accepted procedure in providing the services but given the residual risk associated with any prediction and the variability which can be experienced in flood conditions, Mott MacDonald takes no liability for and gives no warranty against actual flooding of any property (clients or third party) or the consequences of flooding in relation to the performance of the service.
  5. Drawings do not show extent and location of services. See utility plans / statutory undertakers plans for service information. Contractor to undertake pre construction survey to determine full extent of existing services present within working area.
  6. Street lighting, street furniture, existing services, existing seawalls, adjoining buildings, movement joints etc not shown.
  7. These preliminary design promenade structures drawings only include sections E to H of the Central Rhyll Coastal Defence Outline Business Case (OBC) by JBA dated October 2019. Repair work to the existing coastal defences are not shown; for information see the visual condition survey & repair report CR-MMD-00-XX-RP-SC-7005 and associated drawings. Design of the rear promenade walls are based on the following load cases:
    - Load case 1: Imposed (variable) water load of 10kN/m<sup>2</sup> UDL and horizontal wave load of 12kN/m at 335mm above promenade FFL.
    - Load case 2: Imposed (variable) service load of 20kN/m<sup>2</sup> UDL and horizontal load of 1.5kN/m at 1200mm above promenade FFL.
- Proposals subject to change, detailed design, consultation and necessary approvals.**

- Key to symbols**
- Existing terrain / structures.
  - Proposed in-situ reinforced concrete.
  - Proposed precast reinforced concrete.
  - Proposed in-situ mass concrete.
  - Proposed class 6N compacted granular in accordance with Series 600 Volume 1 Specification for Highway Works.
- \* = Preliminary dimensions and levels subject to change and detailed design.
- Abbreviations:**  
 TOF = Top of foundation  
 TOW = Top of wall  
 FFL = Finish floor level  
 Min. = Minimum  
 IJ = Isolation joint

**Reference drawings**

CR-MMD-00-XX-DR-SP-6000: Promenade Structures General Notes
CR-MMD-00-XX-DR-SP-6001: Promenade Structures Plan View GA
CR-MMD-00-XX-DR-SP-6100 & 6101: Prom. Structures Typical Sections & Details
CR-MMD-00-XX-DR-SC-7001 & 7002: Coastal Structures Plan View GA's
CR-MMD-00-XX-DR-SC-7101 to 7103: Coastal Structures Typical Sections

Rev	Date	Drawn	Description	Ch'k'd	App'd
C02	03.09.2021	JH	Accepted for EIA following client comments	GWM	NP
P03	10.06.2021	JH	Preliminary design for EIA and consents.	GWM	SS
C01	04.06.2021	JH	Stage 3b - accepted contract revision.	GWM	SS
P02	18.12.2020	JH	Issued for acceptance following client comments.	GWM	NP
P01	30.10.2020	JH	Preliminary issue for comment	GWM	SS

**Reasonable worst case design for EIA only**

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**sir ddinbych denbighshire**  
 County Council

**Title**  
 Central Rhyll Coastal Defences  
 PROMENADE STRUCTURES  
 Preliminary Design  
 Typical Sections & Details

Designed	J. Hughes	Eng.concept	G. Mitchell
Drawn	J. Hughes	Coordination	M. Cowle
Dwg.check.	G. Mitchell	Approved	S. Smith
MMD Project Number	420823	Scale at A1	As indicated
Suitability Description	Stage 3b - Accepted	Security	STD
Drawing Number	CR-MMD-00-XX-DR-SP-6101	Suit. Code	A4
		Rev	C02

## C. Wave Overtopping Report



**Balfour Beatty**

A large teal graphic element consisting of a triangle pointing upwards at the top, a horizontal line, and a vertical line on the left side, forming a partial frame for the title.

# **Central Rhyl Coastal Defences**

Wave Modelling Study

September 2021

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Engineering Limited

# Central Rhyl Coastal Defences

## Wave Modelling Study

September 2021

# Issue and Revision Record

Revision	Date	Originator	Checker	Approver	Description
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C01	08/09/21	RA/SC	JW	ZH	Published version following no comments from Balfour Beatty

**Document reference:** 420823 | CR-MMD-00-00-RP-MO-7010 | C01

**Information class:** Standard

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# Executive summary

As a part of the detailed design work for the Central Rhyl coastal defences, Belfour Beaty (BB) commissioned Mott MacDonald to undertake a numerical wave modelling study to provide the data required for overtopping, wave force and rock sizing design calculations along the frontage. The study considers the year 2020 (baseline) and accounts for climate change impacts.

A two-dimensional wave model was built using MIKE21 flexible mesh (FM) spectral wave (SW) software. Offshore waves from the Met Office WWIII wave and wind hindcast model point 1516 were transformed to the nearshore region, and the model was calibrated/validated against measured data from the Rhyl Flats wave buoy.

J Heffernan Consulting undertook a multivariate analysis of offshore wave, wind and water level data to provide a 10,000-year synthetic record of offshore water level, wave and wind conditions for the baseline (2020) and two epoch years (2070 and 2120). The MIKE21 FMSW model and an emulator approach were used to generate a 10,000-year nearshore data set of wave and water level conditions at a nearshore location. The 1D wave transformation model LITDRIFT was then applied at three cross-shore locations to determine extreme wave and water level conditions at the toe of the coastal defences for use subsequently in overtopping calculations and sea defence design calculations.

# 1 Introduction

## 1.1 Background

Belfour Beaty (BB) commissioned Mott MacDonald to undertake a detailed design of the coastal defences along the Central Rhyl frontage. This work is supported by efficient and accurate numerical wave modelling to replicate the wave conditions at the toe of the defences for subsequent scheme design calculations. This report describes the modelling methodology and presents results for three scenarios for the baseline year (2020) and the years 2070 and 2120, where adjustment to mean water level and wave conditions are made to account for projected climate change impacts.

## 1.2 Report structure

The report comprises five sections summarising the key modelling activities, data and results. The report is structured as follows:

- Section 1 describes the background to this study;
- Section 2 describes the numerical wave modelling approach;
- Section 3 describes the data used to build the models, including bathymetry and topography, water levels, waves and wind;
- Section 4 presents offshore wave relationships;
- Section 5 presents and discussed wave transformation results using two and one-dimensional models; and
- Section 6 summarises the modelling study.

## 2 Methodology

The wave transformation modelling approach used in this study is shown in the schematic in Figure 2-1 and comprised:

- Analysis of the offshore wave conditions;
  - An extreme value analysis (EVA) of offshore wave data from Met Office Wave Watch III wave hindcast model at point 1516 (WWIII 1516) <sup>1</sup> [1] was used to estimate the significant wave height (H<sub>s</sub>) for a range of return periods;
  - Extreme water levels were obtained from the Coastal Flood Boundary conditions dataset (CFB) <sup>2</sup> [2]. The estimated CFB water levels were corrected to the year 2020 (baseline) using UKCP18 RCP 8.5 at the 70th percentile<sup>3</sup> [3];
  - Sea level rise data were extracted from UKCP18 guidelines for RCP8.5 at the 70<sup>th</sup> percentile [3];
  - A multivariate analysis<sup>4</sup> [4] (MVA) was undertaken using the Met Office WWIII data point 1516 and water level data. This analysis gave an offshore 10,000-year synthetic record of water level, H<sub>s</sub>, peak wave period (T<sub>p</sub>), mean wave direction (MWD), wind speed and wind direction; and
  - Relationships between offshore wave parameters were determined.
- The development of a MIKE21 FMSW wave model (Section 5.1) to transform offshore waves to the study frontage. The model was calibrated and validated against the measured data from the Rhyl Flats buoy;
- Simulation of the offshore 10,000-year dataset with the MIKE21 FMSW wave model requires extensive computational time. Thus, the 10,000-year synthetic dataset was reduced by binning it into a specific range of bin sizes<sup>5</sup> [5]. These binned datasets were then applied as boundary conditions and were transformed to a nearshore location using the MIKE21 FMSW model;
- The ratio of offshore to inshore wave conditions (wave heights, wave periods and the angle change in wave direction) were calculated based on the binned simulation results against the offshore 10,000-year dataset;
- The 10,000-year wave/water level inshore dataset was transformed to the toe of the coastal defence structures using the one-dimensional wave transformation model LITDRIFT; and
- The resulting wave and water level characteristics at the toe of the defences were provided for subsequent overtopping and scheme design calculations<sup>6</sup> [6].

### 2.1 MIKE21 FMSW

MIKE21 FMSW is a third-generation state-of-the-art spectral wind-wave model based on the finite volume method on an unstructured mesh that enables full-time domain simulations. The model simulates the growth, decay and transformation of wind-generated waves and swells in

---

<sup>1</sup> <https://www.metoffice.gov.uk/research/weather/ocean-forecasting/ocean-waves>

<sup>2</sup> UK Environment Agency, "Coastal flood boundary conditions for the UK: 2018 update," Bristol, UK, 2018.

<sup>3</sup> M. D. Palmer et al., "UKCP18 Marine Report," United Kingdom, 2018.

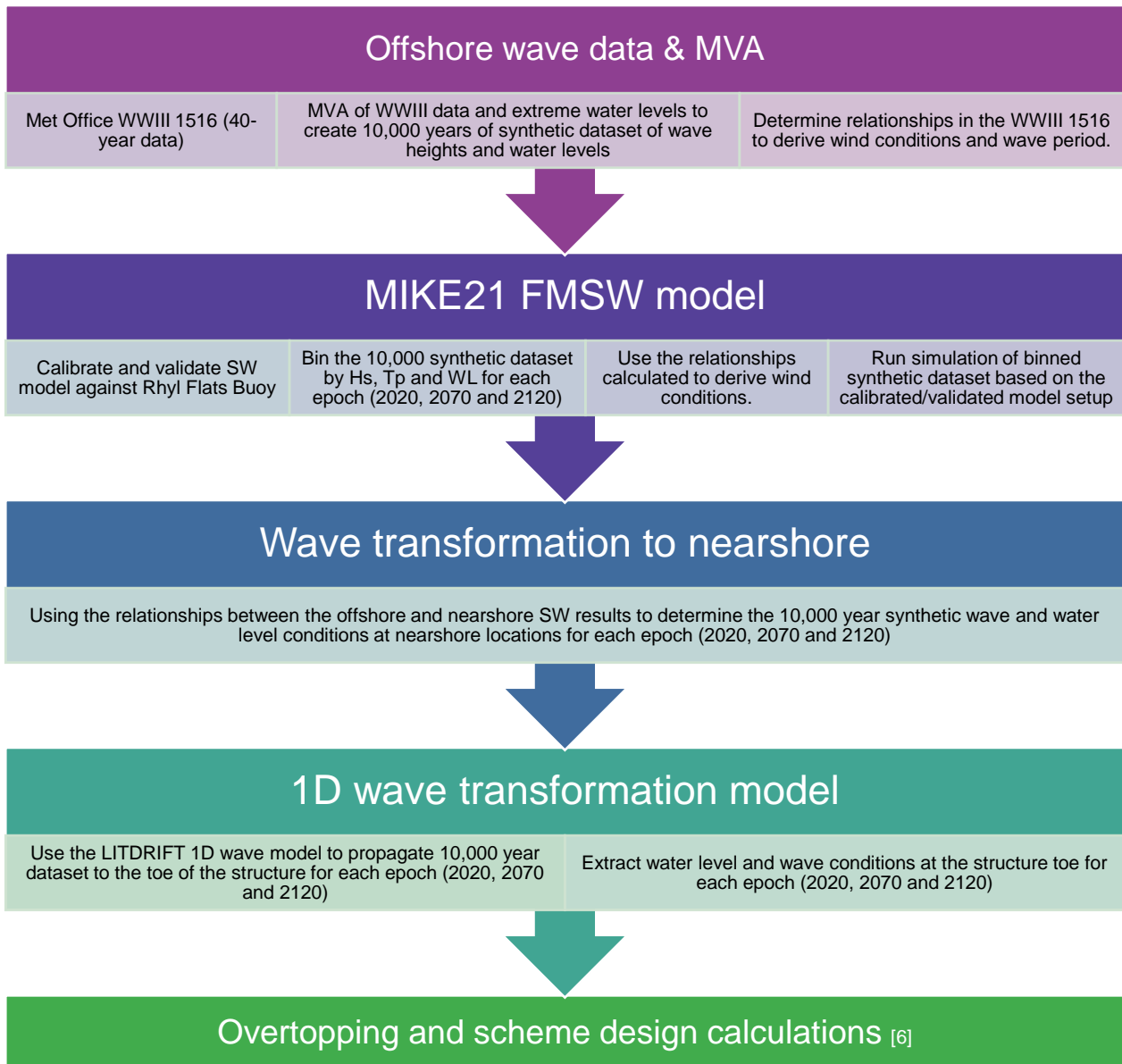
<sup>4</sup> This analysis was performed by J Heffernan Consulting Limited (JHC). JHC, "Central Rhyl Coastal Defences: Extreme Value Analysis, 2020 baseline year analysis, WWIII location 1516", 2021.

<sup>5</sup> "Data binning is the process of grouping individual data values into specific bins or groups according to defined criteria", Mott MacDonald, 2021.

<sup>6</sup> Mott MacDonald, "CR-MMD-00-00-RP-SC-7011 Central Rhyl Coastal Defences – Overtopping Report", 2021.

offshore and coastal areas. The model includes wave growth by the action of wind, nonlinear wave-wave interaction, dissipation due to white-capping dissipation due to bottom friction, dissipation due to depth-induced wave breaking, refraction and shoaling due to depth variations, wave-current interaction, and the effect of time-varying water depth. Therefore, the model is deemed suitable to propagate waves from offshore to the nearshore area at Central Rhyl.

**Figure 2-1 The wave modelling approach**



Source: Mott MacDonald, 2021

## 3 Data

The data inputs for the modelling study included open-source data and site-specific measurement. It is noted that the model outputs from this study rely heavily on the accuracy of the dataset used, and thus they were quality assured before use using bespoke in-house Mott MacDonald tools.

### 3.1 Systems and projections

Geographical data used in this study used the British National Grid (BNG) with OSGB36 horizontal datum. The vertical datum was referenced to Ordnance Datum Newlyn (ODN) at Central Rhyl, estimated to be -4.45m to the Chart Datum (CD).

### 3.2 Bathymetry

The bathymetric data used to build the MIKE21 FM/SW<sup>7</sup> [7] wave model (Section 1.1.1) included:

- Bathymetry data<sup>8</sup> [8] extracted directly from the JBA SWAN mesh;
- Freely available bathymetry data services (UKHO)<sup>9</sup> [9];
- Open source EMODnet (2018 data <sup>10</sup> [10];
- Composite and historical NRW LiDAR data from 2004, 2009, 2011 and 2015;
- Coastal Channel Observatory (CCO)<sup>11</sup> [11] beach monitoring profiles for March 2019 and June 2020; and
- Historical bi-annual monitoring beach surveys from 2002 to 2014 from Denbighshire County Council.

The offshore model bathymetry was built mainly using the UKHO and EMODnet data, prioritising the UKHO data. The nearshore model bathymetry was based on the CCO survey of June 2020. These survey data were selected due to their cross-shore and temporal coverage. For further information regarding historical analysis of the beach levels of Central Rhyl, please refer to the *Sediment Transport Report* (Mott MacDonald, 2021)<sup>12</sup> [12].

### 3.3 Waves

Measured and modelled wave data, quantifying significant wave height (Hs), peak wave period (Tp) and mean wave direction (MWD), were available close to the project site comprising:

- Model hindcast 3-hourly and hourly wave data from 1980 to 2019 (40-year) from the UK Met Office Wave Watch III (WWIII) wave model at point 1516<sup>13</sup> [13] at 53° 32.49' N, 3° 55.8' W in a water depth of approximately 45 mOD; and
- Offshore wave data from May 2007 to December 2019 measured at the Rhyl Flats wave buoy<sup>14</sup> [14] at 53° 22.92'N, 3° 36.18'W in a water depth of approximately 14 mOD for the period.

---

<sup>7</sup> DHI, "MIKE 21 – Spectral Wave Module – Scientific Document," Hørsholm, Denmark, 2020.

<sup>8</sup> JBA Consulting, "Point of Ayr to Pensarn Tidal Flood Risk Analysis: Final Model Development Report", 2017.

<sup>9</sup> [Bathymetry data Service \(admiralty.co.uk\)](https://www.admiralty.co.uk/bathymetry) accessed on 2 November 2020.

<sup>10</sup> [EMODnet Bathymetry \(emodnet-bathymetry.eu\)](https://www.emodnet-bathymetry.eu) accessed on 3 November 2020.

<sup>11</sup> [Channel Coastal Observatory Filter](https://www.cco.gov.uk/channel-coastal-observatory-filter) accessed on 17 November 2020.

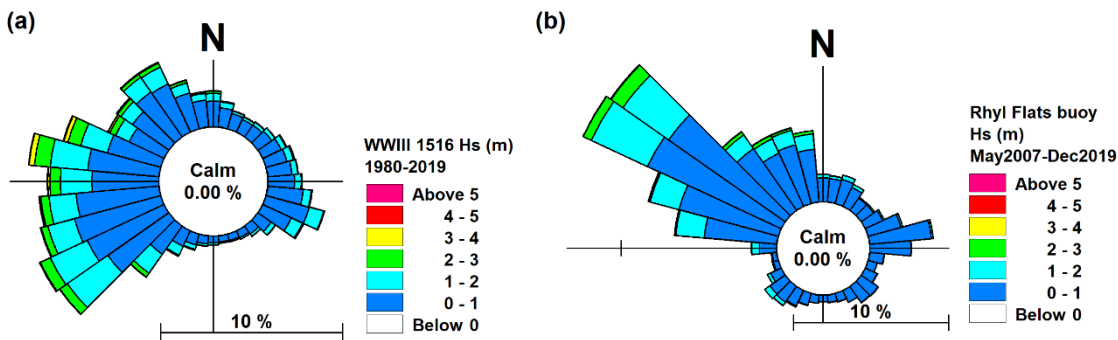
<sup>12</sup> Mott MacDonald, "CR-MMD-00-ZZ-RP-SC-7007: Central Rhyl Coastal Defences - Sediment Transport modelling", 2021

<sup>13</sup> <http://wavenet.cefas.co.uk/hindcast> accessed on 20 October 2020.

<sup>14</sup> [Channel Coastal Observatory Rhyl Flats](https://www.cco.gov.uk/channel-coastal-observatory-rhyl-flats) accessed on 22 October 2020.

Figure 3-19(a,b) show the wave rose plots of Hs against MWD at Met Office WWIII 1516 and Rhyl Flats wave buoy locations, respectively. Figure 3-19(b) shows that the dominant wave direction at Rhyl Flats wave buoy is from the north-west, while Figure 3-19(a) shows that the dominant wave direction at Met Office WWIII 1516 is from the south-west, west and north-west. It is noted that the Rhyl Flats wave buoy is located approximately 10km from the shoreline and approximately 28km inshore from WWIII 1516 (Figure 5-1).

**Figure 3-1 Wave roses derived from: (a) WWIII 1516 wave hindcast data (1980 to 2019); and agent (b) measured wave data from the Rhyl Flats wave buoy from May 2007 to December 2019.**

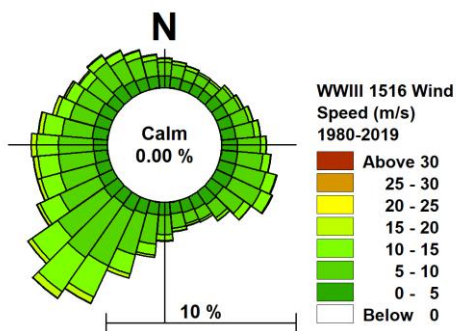


Source Mott MacDonald, 2021

### 3.4 Wind

As the wind is the primary forcing agent contributing to wave formation, using the best available wind data in wave modelling studies is essential. In recognition of this, the present study used wind data from Met Office WWIII 1516 [13]. The wind rose at WWIII 1516 shown in Figure 3-2 shows that the dominant wind direction is from the south-west, west and north-west, and is similar to the wave rose of Hs in Figure 3-1(a).

**Figure 3-2 Wind roses derived from WWIII 1516 wave hindcast data (1980 to 2019).**



Source Mott MacDonald, 2021

### 3.5 Water level

Two sets of water level data were used in this study:

- i. For the MIKE FMSW model calibration/validation (Section 1.1.1), a distance weighted estimate of water levels was interpolated for Central Rhyl between the predicted tides from Liverpool and Llandudno tidal stations. Tides prediction was performed based on the harmonic constituents extracted from Liverpool and Llandudno tidal stations using the MIKE 21 Tidal Prediction Toolbox <sup>15</sup> [15]. The predicted tides were used instead of the measured tides due to the complexity of combining the measured dataset. Furthermore, the calibration station (Rhyl Flats wave buoy) is located in deep water and therefore, the influence of water level is not significant; and
- ii. The measured water level data at Rhyl Harbour, available from 15/10/2008 to 17/02/2017, was combined with the Llandudno predicted and measured tide data in the MVA. Rhyl Harbour data was only available for a portion of the tide curve, above 2mODN. The lower portion of the tide curve was replaced with the Llandudno tide gauge, when available, or the predicted tide at Llandudno.

#### 3.5.1 Extreme water levels

Extreme water levels were obtained from the Coastal flood boundary (CFB) [2] condition data, at point 1136 as shown in Figure 3-3. Extreme water levels and the corresponding Annual Exceedance Probability (AEP) at CFB 1136 are presented in Table 3-1. The estimated CFB water levels were corrected to the year 2020 (baseline) using UKCP18 RCP 8.5 at the 70th percentile [3].

**Figure 3-3 CFB extraction point 1136.**



Source CFB 2017

**Table 3-1 CFB water level at point 1136**

AEP (%)	CFB water level (mOD)
100	4.91
50	4.99
20	5.11
10	5.20
5	5.28

<sup>15</sup> DHI, "MIKE 21 Tidal Analysis and Prediction Module Scientific Documentation," Horsholm, Denmark, 2017.

AEP (%)	CFB water level (mOD)
4	5.31
2	5.39
1.3	5.44
1	5.47
0.7	5.52
0.5	5.56
0.4	5.59
0.3	5.61
0.2	5.68
0.1	5.77
0.01	6.10

Source CFB 2017 and Mott MacDonald, 2021

## 3.6 Climate change

### 3.6.1 UKCP18: Sea-level rise

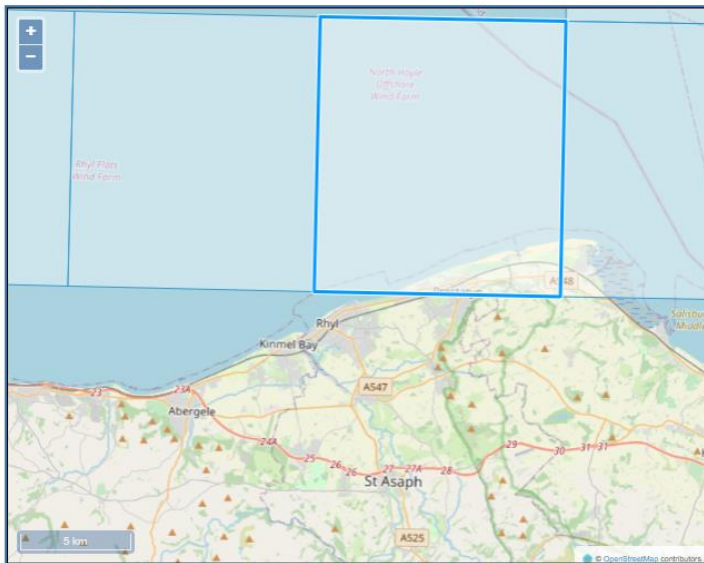
The sea-level rise (SLR) projection used in the study follows UKCP18 guidelines for RCP8.5 at the 70<sup>th</sup> percentile [3]. The SLR values obtained for 2070 and 2120 are shown in Table 3-2. An overview of the extracted grid from the UKCP18 database is shown in Figure 3-4.

**Table 3-2 Sea level rise based on UKCP18 RCP8.5 at the 70<sup>th</sup> percentile.**

Year	Estimated SLR (m)
2020	0
2070	0.36
2120	0.87

Source Mott MacDonald, 2021

**Figure 3-4 Sea level rise extracted grid from UKCP18 database.**



Source UKCP18

### 3.6.2 Wind and wave adjustments to the future epochs

Wind and wave adjustments for 2070 and 2120 were based on Welsh guidelines <sup>16</sup> [16] and involved up-scaling wind speed and wave height by +10%, with a corresponding increase in wave period defined by wave steepness.

## 3.7 Multivariate analysis (MVA)

An extreme value analysis (EVA) of  $H_s$  was undertaken on WWIII 1516 (40-year) using the DHI EVA tool <sup>17</sup> [17] and supplied to JHC (2021) for the MVA. In calculating extreme  $H_s$  values, various probability distribution and estimation methods were considered. The most statistically significant fit for  $H_s$  was obtained using an exponential distribution. This fit is shown in Figure 3-5, together with lines showing the 95% confident intervals.

Similarly, the MVA was undertaken using WWIII 1516 (40-year) and water level data from October 2008 to February 2017 (Section 3.5) to generate a 10,000-year synthetic record of water level,  $H_s$ ,  $T_p$ , MWD, wind speed and wind direction. The climate change allowances defined in the previous section was also incorporated in the MVA.

Table 3-3 shows the estimated extreme  $H_s$  values provided to JHC(2021) and estimated  $H_s$  values from the MVA (JHC, 2021). This table shows that the estimated extreme  $H_s$  from the MVA by JHC (2021) match the supplied  $H_s$ , particularly for the value of estimated  $H_s$  at AEP 2% and 1% for the epochs year this study. A comprehensive MVA methodology and analysis detailed in "JHC Extreme Value Analysis report, 2021" [4].

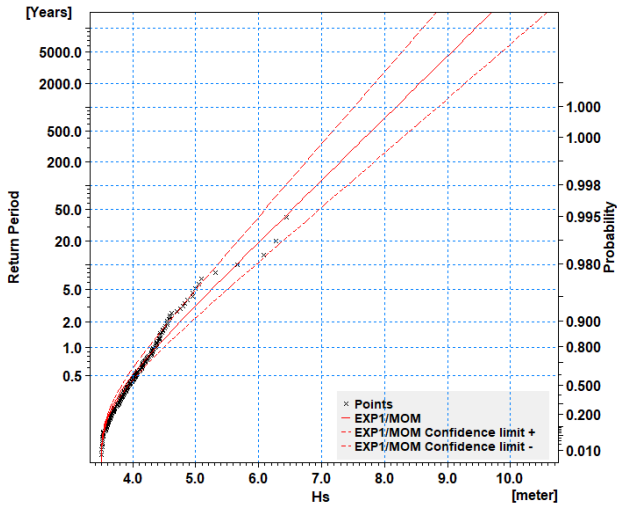
Figure 3-6 shows an example of a scatter plot of the offshore 10,000-year synthetic record of  $H_s$  and water level extracted from the MVA results for the baseline year 2020 and two epochs year, 2070 and 2120. It is observed that, as expected,  $H_s$  and WL values for the baseline increased

<sup>16</sup> Welsh Government, "Adapting to Climate Change: Guidance for Flood and Coastal Erosion Risk Management Authorities in Wales", December 2017.

<sup>17</sup> DHI, "Extreme Value Analysis User Guide", Hørsholm, Denmark, 2017.

for each of the epoch years. Further details of the MVA are given in "JHC Extreme Value Analysis report, 2021" [4].

**Figure 3-5 Exponential probability distribution best fit to Hs vs return period showing the 95% confidence intervals.**



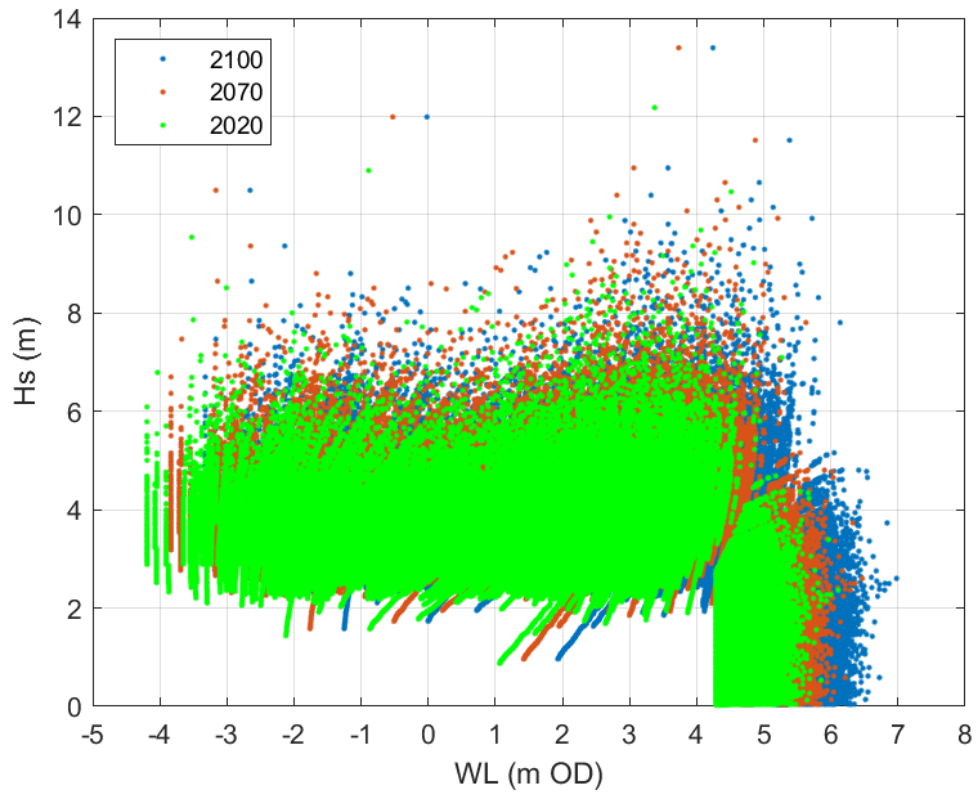
Source Mott MacDonald, 2021

**Table 3-3 Estimated extreme Hs values.**

AEP (%)	Hs (m)	Hs MVA (m)
100	4.37	4.36
20	5.20	5.23
10	5.61	5.61
4	6.14	6.13
2	6.52	6.52
1	6.91	6.93
0.5	7.29	7.41
0.2	7.80	8.06
0.1	8.18	8.47
0.02	9.07	9.06
0.01	9.45	9.20

Source Mott MacDonald and J Heffernan Consulting, 2021

**Figure 3-6** Scatter plot of the 10,000-year synthetic record of Hs against water level (WL) for 2020, 2070 and 2100. Note the increase in both water level and Hs with projected climate change impacts applied.



Source J Heffernan Consulting, 2021

## 4 Offshore wave relationships

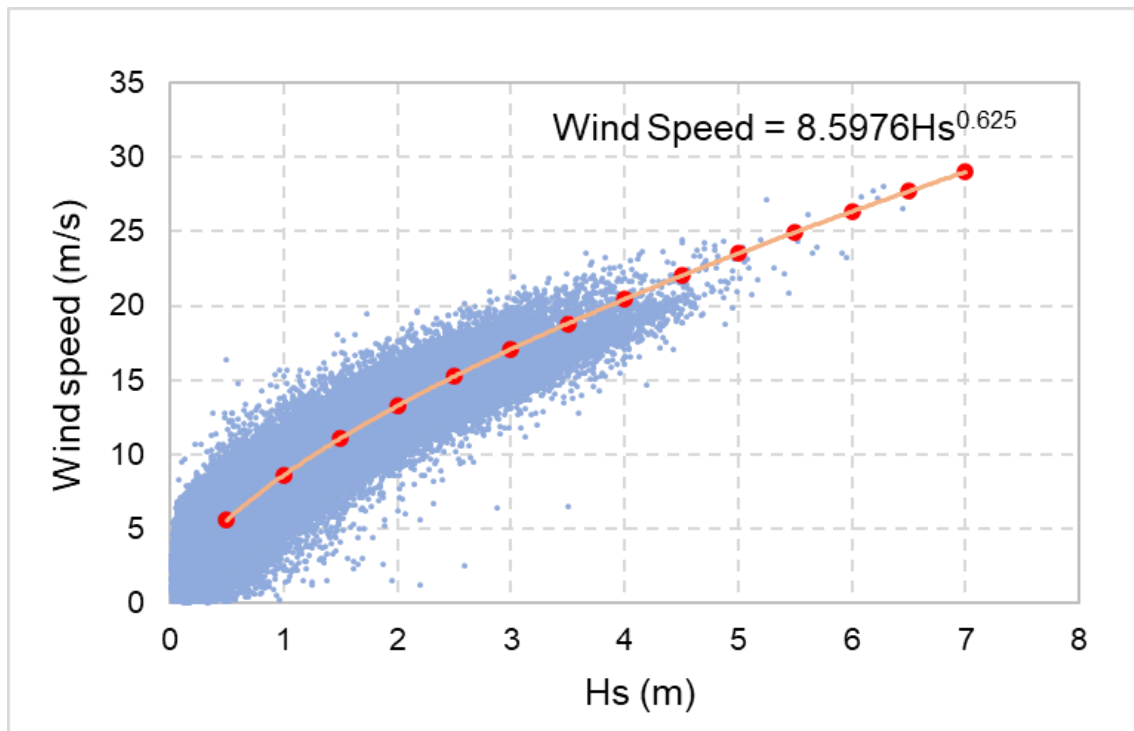
### 4.1 Introduction

The relationships between significant wave height and wind speed and between MWD and wind speed were determined to derive the two-dimensional wave transformation model's boundary conditions based on an emulator approach (Section 5.2). A relationship between  $T_p$  and the mean wave period ( $T_m$ ) was obtained to facilitate the  $T_p$  results' conversion from the 1D wave transformation model (Section 5.3). The resulting  $T_m$  values were used subsequently in the overtopping and design calculations. The WWII 1516 data was used to derive the offshore wave relationships in this section.

### 4.2 Significant wave height and wind speed

The relationship between  $H_s$  and wind speed used in the two-dimensional wave transformation model in Section 5.2 is shown in Figure 4-1.

**Figure 4-1 Relationship between wind speed and significant wave height,  $H_s$ .**

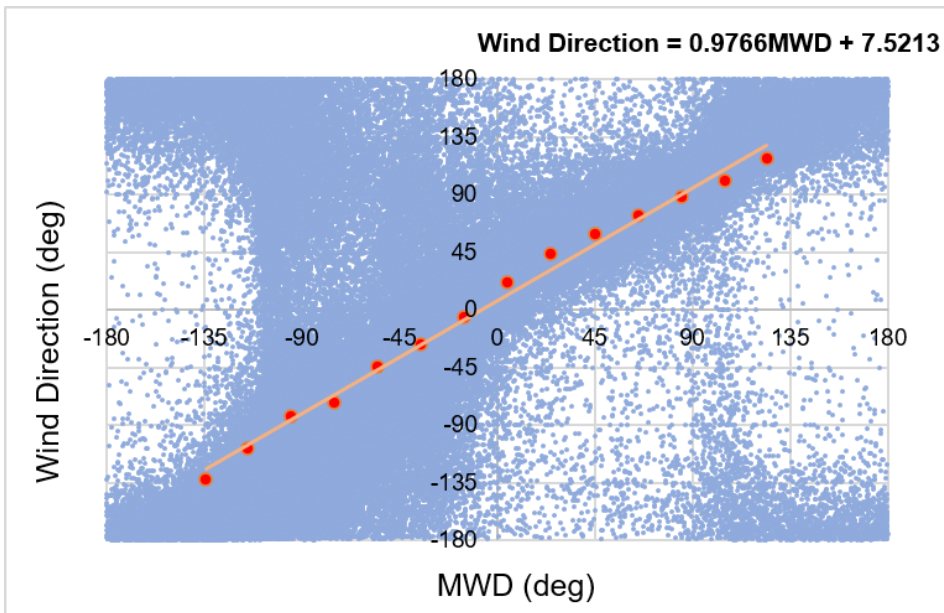


Source Mott MacDonald, 2021

### 4.3 Mean wave direction (MWD) and wind direction

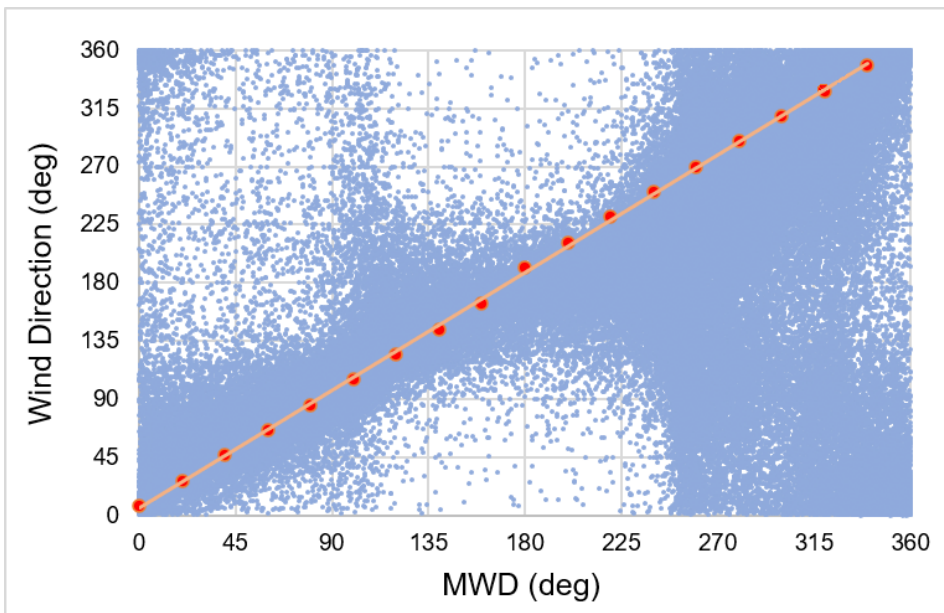
Omnidirectional wind data was used to determine a relationship between MWD and wind direction. All wind direction and MWD data above 180 deg. N was adjusted to establish directional sectors between 0 to 180 deg. N and 0 to -180 deg. N (Figure 4-2). Based on the relationship shown in Figure 4 2, the wind direction and MWD were converted back to 0 to 360 degree. N (Figure 4-3).

**Figure 4-2 Relationship between wind direction and MWD.**



Source Mott MacDonald, 2021

**Figure 4-3 Relationship between wind direction and MWD (0 to 360 degree. N).**

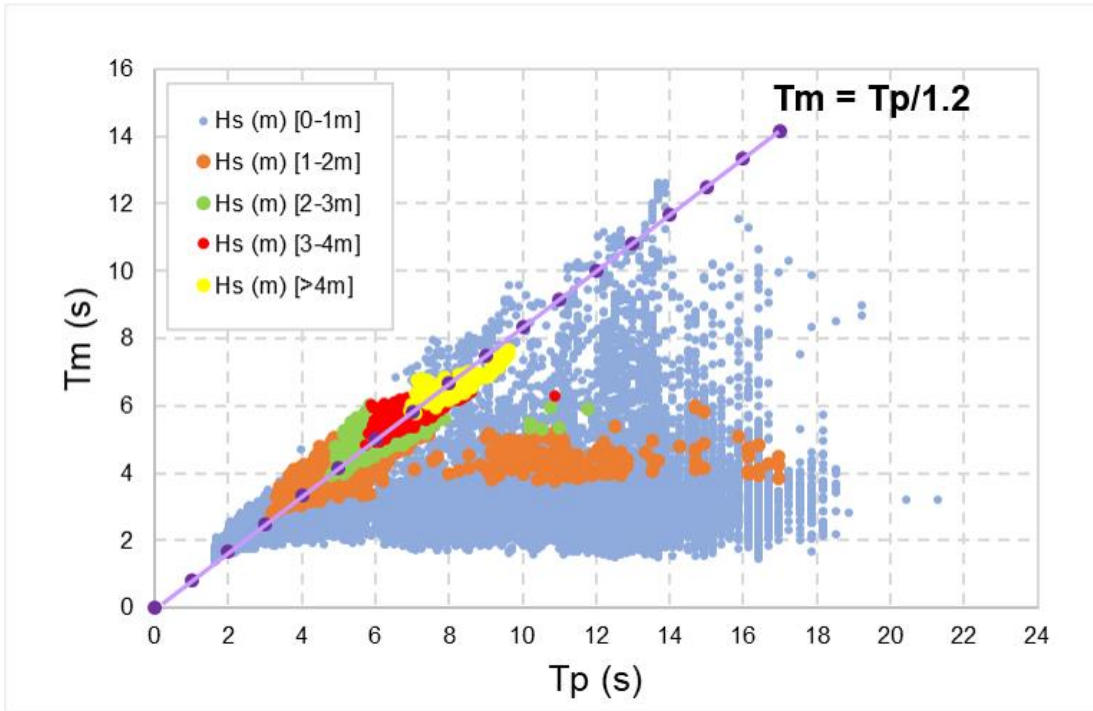


Source Mott MacDonald, 2021

#### 4.4 Peak wave period (Tp) and mean wave period (Tm)

The relationship between Tm and Tp values was derived for the range of Hs values shown in Figure 4-4. Taking a conservative view, the relationship between Tm and Tp used in this study considered only Hs above 4m.

**Figure 4-4 Relationship between Tm and Tp.**



Source Mott MacDonald, 2021

# 5 Wave transformations

## 5.1 Introduction

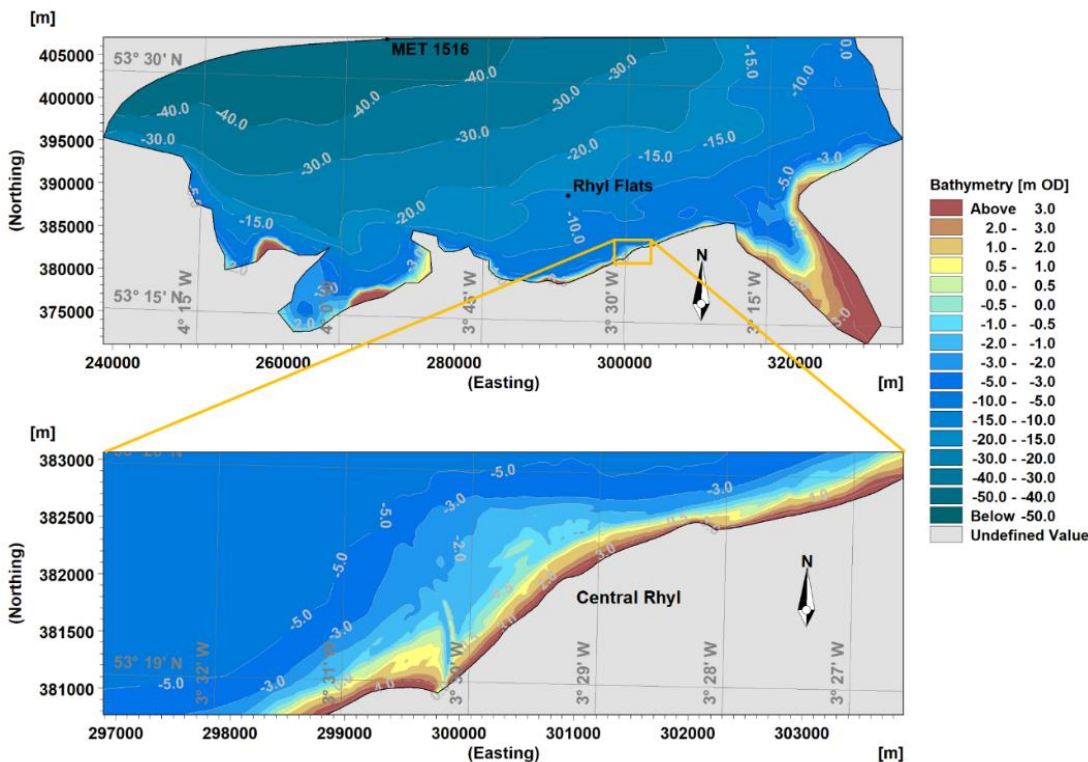
Wave transformations were required to provide extreme waves and water levels for the overtopping and design calculations at the defences. The two-dimensional MIKE21 FMSW [5] model was used to transform offshore waves to the nearshore, and a higher spatial resolution one-dimensional wave transformation model (LITDRIFT) was used to transform waves to the toe of the coastal defence structures.

It was recognised early in the study that using the MIKE21 FMSW alone would require extensive computational time to transform the 10,000-year synthetic record [4] (for each epoch) to the nearshore locations. An emulator approach was undertaken for project efficiency whereby the synthetic records (Hs/Tp/MWD/water level) were binned [5] to reduce the dataset and then used as boundary conditions in the MIKE21 FMSW model. The ratio between the offshore inputs and nearshore binned SW results were calculated as transformation coefficients and then subsequently used as a look-up table to transform the full set of 10,000-year synthetic record into a whole set of 10,000-year data at nearshore locations. More information on this process is provided in Section 5.2.

### 5.1.1 Model Domain (Flexible Mesh)

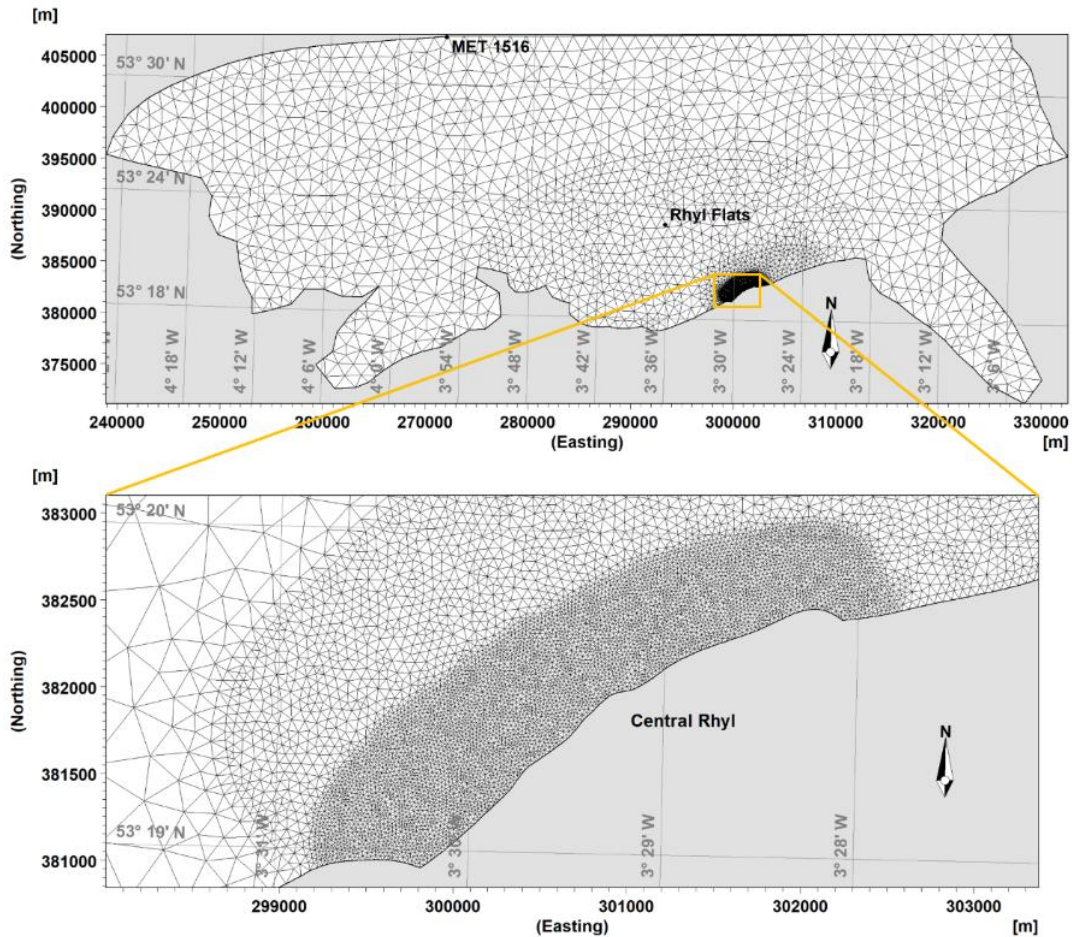
The MIKE21 FMSW model was set up to cover Central Rhyl and all relevant coastal and offshore areas that influence the wave conditions (Figure 5-1). This figure also shows the location of the Rhyl Flats buoy [14] and Met Office WWIII 1516. The mesh resolution for the model domain and the open boundary defined in the model are shown in Figure 5-2.

**Figure 5-1 MIKE21 FMSW model domain and bathymetry. An enlarged view of the Central Rhyl frontage is shown in the lower panel.**



Source Mott MacDonald, 2021

**Figure 5-2 MIKE21 FMSW model flexible mesh. An enlarged view of the Central Rhyl frontage is shown in the lower panel.**



Source Mott MacDonald, 2021

### 5.1.2 Model forcing and water level

The main wind forcing, taken from the WWII 1516 40-year hindcast dataset, was applied as surface boundary conditions. The water level forcing used in the model is described in Section 3.5.

### 5.1.3 Model Setup

**Table 5-1** summarises the MIKE21 FMSW model setup established in the model calibration process (see next section).

**Table 5-1 Summary of the calibrated MIKE21 FMSW model setup**

Parameters		Description
		Local Model
Equation		Directional-Decoupled, Quasi-stationary
Frequency Discretisation	No. of frequency	25
	Min Frequency	0.055 Hz
	Frequency factor	1.1
	No. of direction	360 degree rose (36 directions)
Solution		Quasi-stationary
Quasi-stationary formulation	Geographical space discretisation	Low order, fast algorithm
	Method	New-Raphson Iteration
	Max. number of iterations	100
	Tolerance (RMS-norm of residual)	0.001
	Tolerance (Max-norm of change in sig. wave height)	0.01
	Relaxation Factor	0.05
Water Level Conditions		Interpolated predicted water level (Section 3.5)
Current Conditions		No current included
Wind Forcing	Wind data	Met Office WWIII 1516
	Soft start	0
	Wind Generation Formula	SPM84
Ice Coverage		No ice coverage
Diffraction		No diffraction
Wave Breaking	Model	Wave breaking
	Type of gamma	Specified gamma
	Gamma data	0.78
	Alpha	1
	Gamma (wave steepness)	5
Bottom Friction	Model	No bottom friction
Structures		No structures
Initial Conditions		Spectra from empirical formula from JONSWAP fetch growth expression
Boundary Conditions		Met Office WWIII 1516
Source Mott MacDonald, 2021		

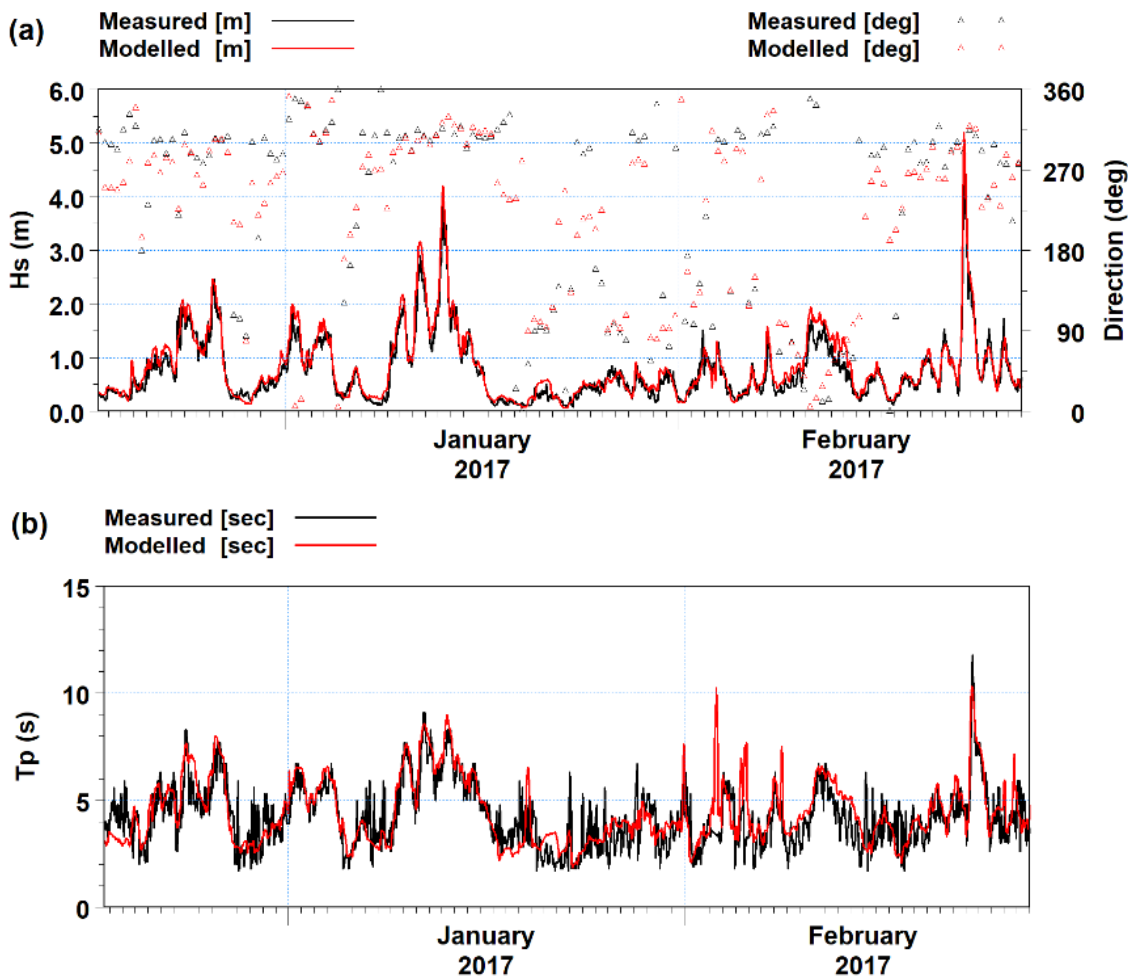
#### 5.1.4 Model Calibration

The MIKE21 FMSW wave model was calibrated against Rhyl Flats wave buoy data from 17 December 2016 to 28 February 2017. This period contains the highest storm waves recorded by the Rhyl Flats wave buoy. Figure 5-3 shows a comparison between measured and modelled data at Rhyl Flats wave buoy for  $H_s$ ,  $T_p$  and MWD. The figure demonstrates that the model predictions compare well with the measured data. Model performance statistics are tabulated in Table 5-2, including root-mean-square error (RMSE), the correlation coefficient of determination ( $R^2$ ), standard deviation (STD) and bias.

Table 5-2 shows that the model can reproduce the measured data with a high accuracy for  $H_s$  (RMSE = 4%,  $R^2$  = 0.95 and bias = - 0.09 m). Similarly,  $T_p$  and MWD also show good agreement against measured data. An overview of the model results during the storm period of 23 February 2017 at 12:00 is also presented in Figure 5-3 with the dominant wave direction from the northwest.

Based on NRW Guidelines (2017)<sup>18</sup> [18] and ABPmer(2013)<sup>19</sup> [19], the modelled Hs should be within +/- 10% of observed Hs, and the modelled mean wave period should be within 20% of the observed wave period. Further, the modelled mean wave direction should be +/- 30% of observed wave directions and scatter index values for Hs, Tp and MWD should be below 10, 20 and 25%, respectively, to conform with the model performance criteria established in the peer-reviewed publication by Williams & Esteves (2017)<sup>20</sup> [20]. The model calibration statistics in Table 5-2 demonstrate that the present MIKE21 FMSW model performance meets the NRW Guidelines [16] and those defined by Williams & Esteves (2017) [20].

**Figure 5-3 Comparisons between measured and modelled data at Rhyl Flats wave buoy between December and February 2017 showing: (a) significant wave height, Hs with the mean wave direction, MWD; and (b) peak wave period, Tp.**



Source Mott MacDonald, 2021

<sup>18</sup> Pye, K., Blott, S. and Brown, J., "Advice to Inform Development of Guidance on Marine, Coastal and Estuarine Physical Processes Numerical Modelling Assessments", NRW Report No: 208, 139 pp, Natural Resources Wales, Cardiff. 2017.

<sup>19</sup> ABPmer, "Numerical Model Calibration and Validation Guidance. ABP Marine Environmental Research Ltd., File Note R/1400/112". ABPmer, Southampton (unpublished), 2013.

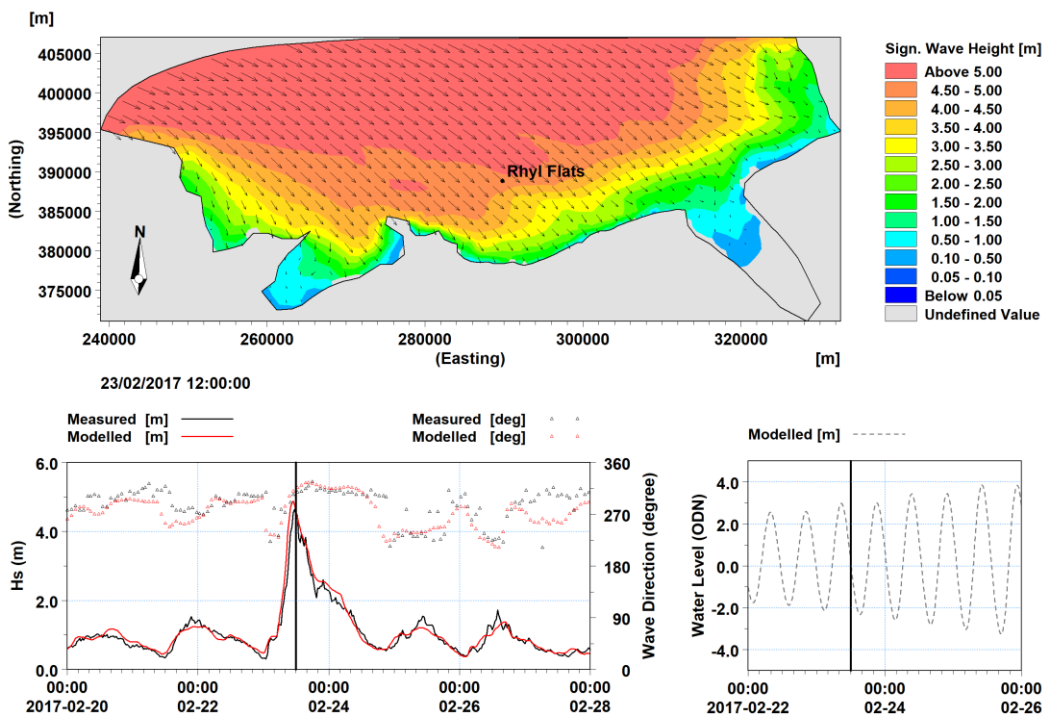
<sup>20</sup> Williams, J.J. and Esteves, L.S., "Guidance on Setup, Calibration, and Validation of Hydrodynamic, Wave and Sediment Models for Shelf Seas and Estuaries". Advances in Civil Engineering, 5251902, 2017.

**Table 5-2 Summary of the calibrated MIKE21 FMSW performance metrics at the Rhyl Flats wave buoy.**

Parameter	Hs	Tp	MWD
Bias	-0.09m	-0.19s	21.5 deg.
Root Mean Square Error (%)	4.0	17.5	22.8
Coefficient of Determination (R <sup>2</sup> )	0.95	0.66	0.42
Scatter Index (%)	5.2	4.1	0.1

Source: Mott MacDonald, 2021

**Figure 5-4 Significant wave height predicted by the MIKE21 FMSW model during the storm period's peak on 23 February 2017 at 12:00UTM.**



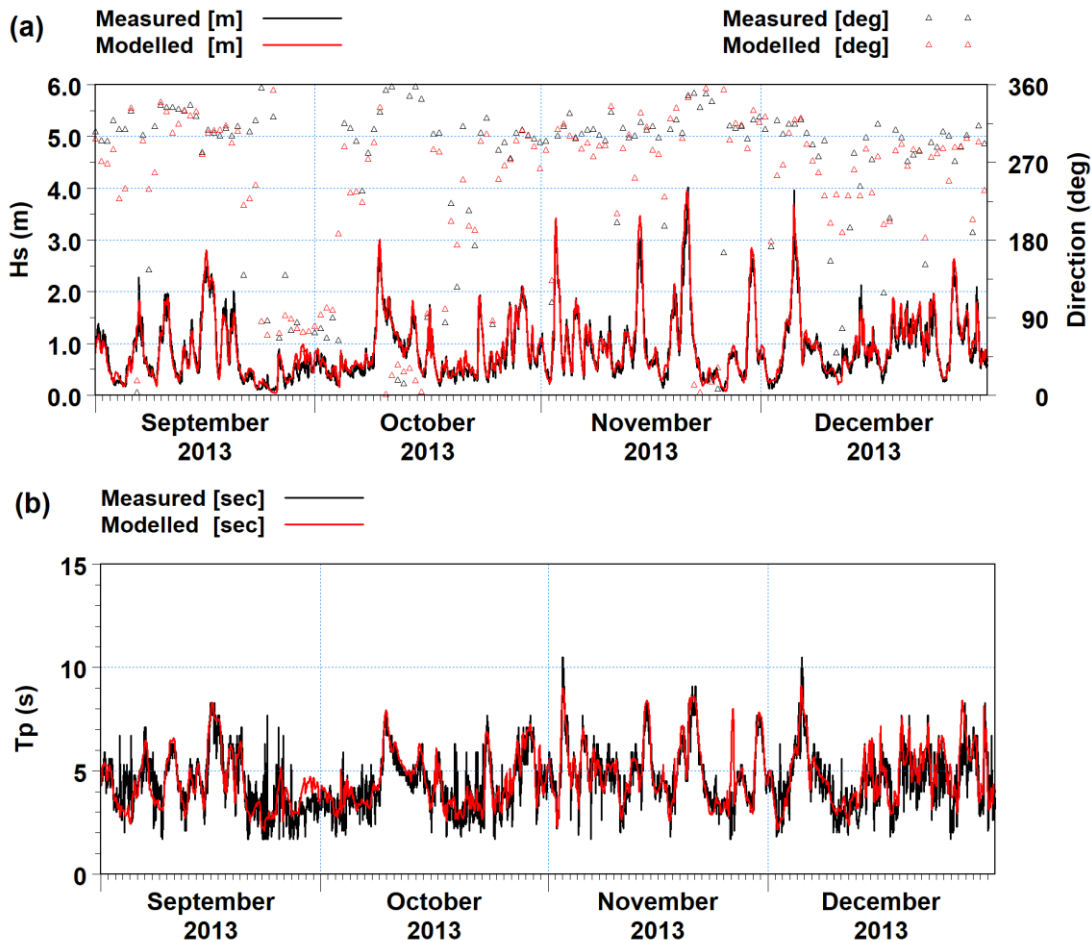
Source: Mott MacDonald, 2021

### 5.1.5 Model Validation

Validation of the wave model was undertaken by running the model for a different observational period. Figure 5-5 shows a comparison plot between measured and modelled data at Rhyl Flats wave buoy for Hs, Tp and MWD between September and December 2013. The model is shown visually to replicate the observed wave conditions well.

Statistics to quantify the model performance are summarised in Table 5-3. Based on the evidence provided in Figure 5-5 and Table 5-3, it is considered that the wave model again reproduces accurately wave conditions at the site and meet the NRW Guidelines [18] and the model performance criteria by Williams & Esteves [20]. Therefore, the model is judged to be suitable to provide reliable estimates of wave conditions in nearshore locations.

Figure 5-5 Comparison plots between measured and modelled data at Rhyl Flats wave buoy between September and December 2013 showing: (a) significant wave height, Hs with the mean wave direction, MWD; and (b) peak wave period, Tp.



Source Mott MacDonald, 2021

Table 5-3 Summary of the validated MIKE21 FMSW performance metrics at the Rhyl Flats wave buoy.

Parameter	Hs	Tp	MWD
Bias	-0.05m	-0.13s	20.6 deg.
Root Mean Square Error (%)	4.1	15.7	22.4
Coefficient of Determination (R <sup>2</sup> )	0.94	0.69	0.44
Scatter Index (%)	4.6	3.4	0.1

Source Mott MacDonald, 2021

## 5.2 2D wave transformation model

Before being transformed with a 1D model to the toe of the defences, extreme wave conditions from the 10,000-year synthetic record (Section 3.7) required transformation to the nearshore using the calibrated/validated MIKE21 FMSW model and an efficient emulator approach.

In the emulator approach, a subset of wave conditions and water levels sufficient to encompass the full range of offshore wave data was run using the MIKE21 FMSW model. The results from these simulations were used as a look-up table to derive transformation coefficients, which enabled the offshore wave dataset to be transformed inshore with accuracy and computational efficiency.

The wave conditions applied to the boundary of the SW wave model (for use with the emulator) were taken from binning the 10,000 synthetic dataset. Binning is the process of reducing the whole dataset into a smaller number of the dataset with a specified range of bin sizes. For example, if the Hs ranges from 0.5 to 12m, then a bin size might be defined as 0.5m so that each bin might represent Hs between 0.5 - 1m, 1 – 1.5m, 1.5 – 2.0m, ..... and 11.5 - 12m. The water level boundaries include the full range of climate change scenarios.

Table 5-4 shows the bins used to define the wave model's boundary conditions and subsequently used by the emulator to transform the offshore 10,000-year dataset to the inshore. The model forcing conditions such as wind speed and wind direction were derived using the offshore wave relationships detailed in Section 4, while the wave directional standard deviation (DSD) was applied as 33.47 deg. N based on the mean DSD extracted from WWIII 1516 dataset.

**Table 5-4: Defined offshore bins used for wave simulation**

Parameter	Min. centre bin condition	Max. centre bin condition	Interval
Hs (m)	0.01	13.4	0.5
Tp (s)	1.6	21.3	1
Water Level (m OD)	-4.5	7	0.5
MWD (degrees)*	0	360	10

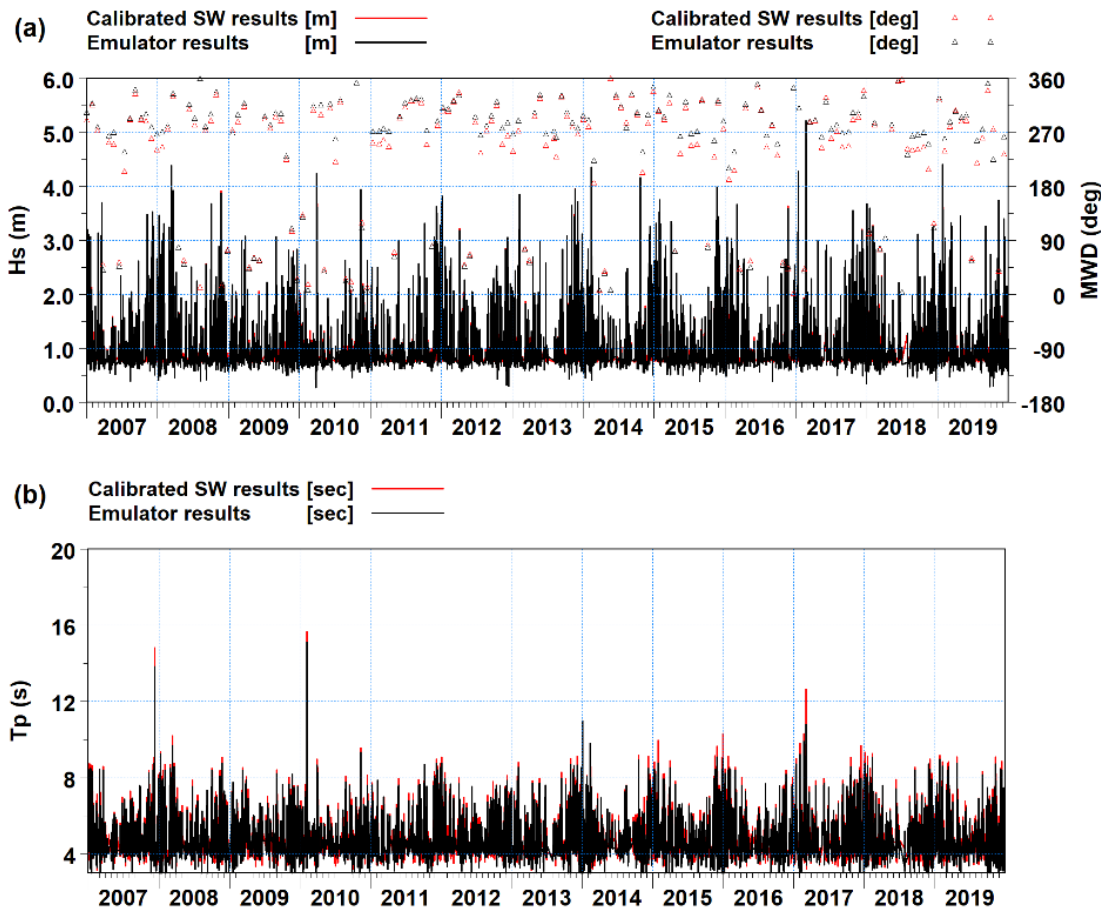
Source Mott MacDonald, 2021

The calibrated/validated MIKE21 FMSW model setup presented in **Table 5-1** was run in directionally-decoupled mode (parameterisation of the frequency domain) to simulate the binned events defined in Table 5-4. The model results were processed to provide a look-up table within the emulator.

The emulator works by considering which bin a particular offshore wave/water level value sits in before calculating the wave components' transformation coefficients. For example, for a given bin, Hs' transformation coefficient is defined as the inshore Hs result at the location of interest, divided by the offshore boundary (or binned) Hs. The offshore Hs wave condition is then multiplied by the wave transformation coefficient to get the nearshore wave condition. These calculations were undertaken for each of the wave parameters. However, the transformation coefficient was based on the change in direction rather than a ratio for wave direction.

The emulator assumes that the transformation coefficients are constant within each binned event. This is a valid assumption with the present bin resolution. As a check, Figure 5-6 shows a comparison plot between the emulator and calibrated MIKE21 FMSW model results at Rhyl Flats location for Hs, Tp and MWD. This figure shows that the emulator approach can reproduce wave conditions from offshore to inshore satisfactorily. However, the data indicate some bias for Tp and MWD. As this occurs only when Hs offshore is less than 1m, the error has no impact on the subsequent wave overtopping calculations.

**Figure 5-6 Comparison plot between the calibrated MIKE21 FMSW model results against emulator results at Rhyl Flat buoy for: (a) significant wave height,  $H_s$  with mean wave direction, MWD; and (b) peak wave period  $T_p$ .**



Source Mott MacDonald, 2021

### 5.3 1D Wave Transformation (using LITDRIFT)

While the MIKE21 FMSW wave modelling characterised well the nearshore wave conditions at the Central Rhyl frontage, this model's resolution was too coarse for defining the waves accurately at the toe of the sea defences for the overtopping calculations. To address this, the 1D wave transformation model LITDRIFT contained in the LITPACK suite of models<sup>21</sup> [21] was undertaken to propagate the nearshore waves from the MIKE21 FMSW wave model to the toe of the sea defences at 1m spatial resolution. LITDRIFT model consists a hydrodynamic model that calculates wave propagation towards the coast using a detailed hydrodynamics process across the profile such as shoaling, refraction, diffraction and wave breaking [21].

As model inputs, the LITDRIFT model used the nearshore wave/water level conditions output from the emulator (derived from the MIKE21 FMSW wave model) and the bed levels along with defined beach profiles. To run, LITDRIFT requires root-mean-square wave height ( $H_{rms}$ ), peak wave period ( $T_p$ ), water level (WL) and mean wave direction (MWD).  $H_{rms}$  was calculated

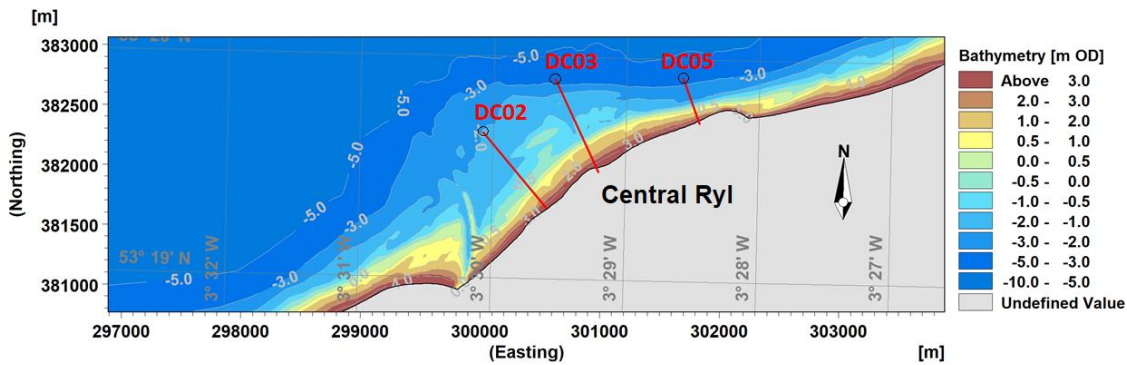
<sup>21</sup> DHI, "Littoral Processes FM - All Modules: Scientific Documentation," Hørsholm, Denmark, 2020.

based on the relationship of  $H_s = 1.416H_{rms}$ <sup>22</sup> [22], and the relationship between  $T_p$  and  $T_m$  (Section 4.4) was used to transform  $T_p$  to  $T_m$  for overtopping calculations.

A sensitivity test to identify the dominant MWD for the LITDRIFT model considered directional sectors of 0 to 360 degree at 10 deg. intervals. The test results show that 0 to 45 deg. N and between 225 to 360 deg. N are the dominant MWD.

Figure 5-7 shows the location of beach profiles DC02, DC03 and DC05. Each of these profiles is shown in Figure 5-8 for the baseline year (2020) and two epochs year (2070 and 2120). A detailed description of the selected baseline and future profiles is described in the Design Philosophy Report<sup>23</sup> [23]. For each of the nearshore wave conditions (baseline and two epochs year), the one-dimensional model was simulated, and a dataset at the toe of each of the profiles was extracted and subsequently used for overtopping calculations [6]. Figure 5-9 shows the results of  $H_s$  and WL extracted at the toe of each of the profiles for the baseline (2020) and two epochs year (2070 and 2120). It is indicated from the figure that  $H_s$  and WL increased from 2020 to 2070 and 2120, respectively, as expected. In addition, DC05 shows higher  $H_s$  compared to DC02 and DC03 due to a deep water influence (Figure 5-7 and Figure 5-8).

**Figure 5-7 Location of the profiles used in 1D wave transformation model**

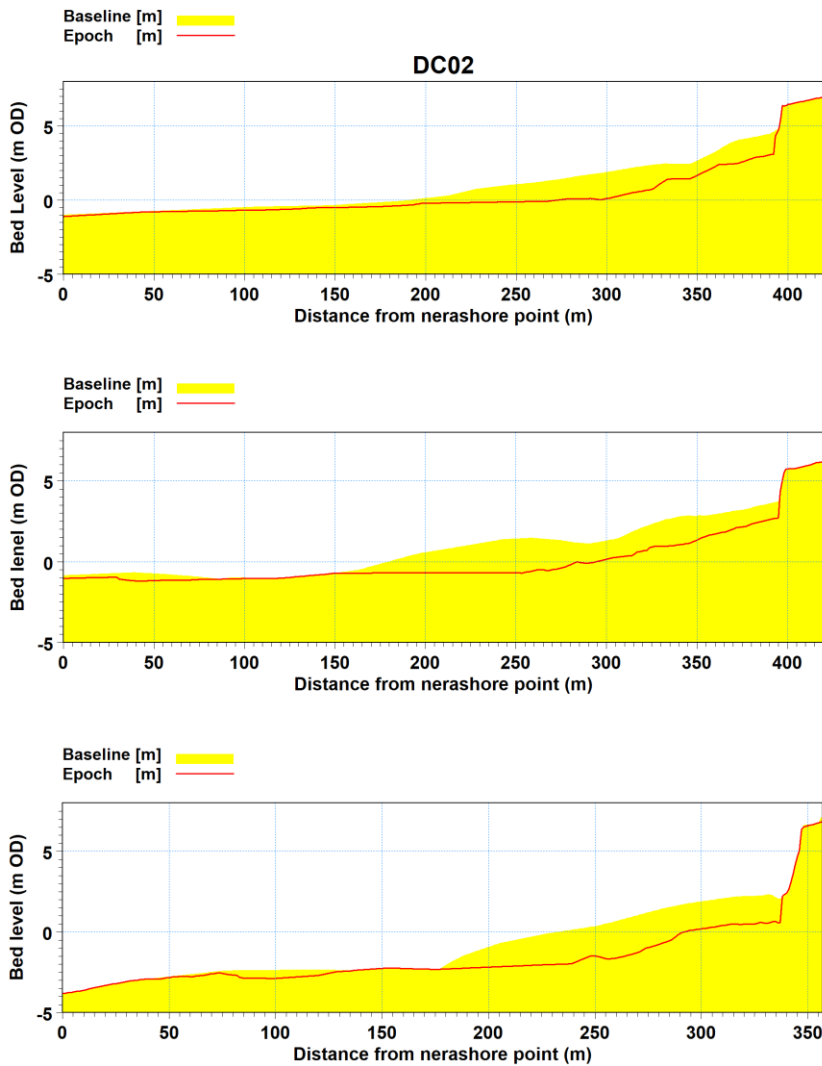


Source Mott MacDonald, 2021

<sup>22</sup> Z. Demirbilek and C. L. Vincent, *Coastal Engineering Manual, Part II, Water Wave Mechanics, Chapter 1, Engineer Manual 1110-2-1100, U.S Army Corps of Engineers, Washington, DC*, 2nd ed., vol. 1100, no. August. Vicksburg, Mississippi: Coastal and Hydraulics Laboratory (CHL), Engineer and Research Center, 2008.

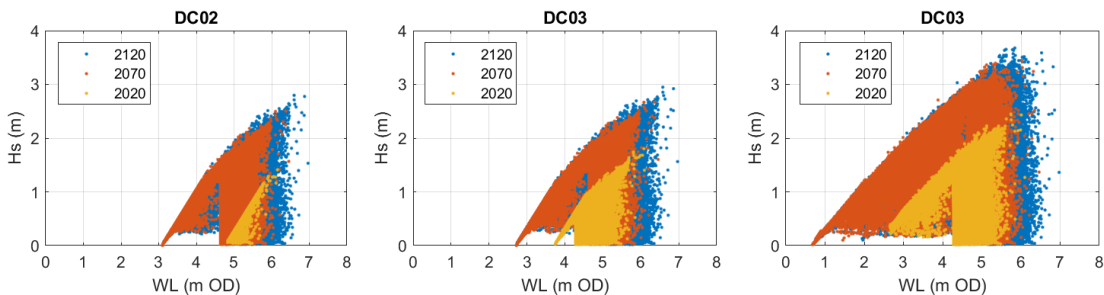
<sup>23</sup> Mott MadConal, "CR-MMD-00-00-RP-Z-0009 Central Rhyl Coastal Defences – Design Input Statement", 2021.

**Figure 5-8 Profiles used in 1D wave transformation model**



Source Mott MacDonald, 2021

**Figure 5-9 Hs and WL at the toe of each of the profiles for each epoch.**



Source Mott MacDonald, 2021

## 6 Summary

A two-dimensional (2D) spectral wave model (MIKE21 FMSW) has been developed to transform efficiently and accurately Met Office WWIII hindcast model wave data from the offshore point 1516 to the nearshore. The model was calibrated/validated against measured data from the Rhyl Flats wave buoy. Model performance metrics were demonstrated to meet NRW requirements and other established standards.

J Heffernan Consulting undertook a multivariate analysis of wave, wind and water level data to provide a 10,000-year synthetic record of offshore water level, Hs, Tp, MWD, wind speed and wind direction for the baseline (2020) and two epochs year (2070 and 2120) [4]. These data were binned and transformed to a nearshore location using the MIKE21 FMSW model and an emulator approach.

The resulting 10,000-year nearshore data set of wave and water level conditions were applied at the offshore boundary of 1D wave transformation LITDRIFT models at three cross-shore locations (DC02, DC03 and DC05) to provide extreme waves and water level conditions at the toe of the coastal defences for subsequent use in wave and still water overtopping calculations and scheme design calculations. [6].

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